Beaufort County 2018 Stormwater Management Implementation Guide:

An Update to the 2006 Stormwater Management Plan

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Introduction

This update to the 2006 stormwater master plan (SWMP) for Beaufort County, South Carolina presents the results of a limited update to certain watersheds and datasets used in the development of the original SWMP. The report summarizes the work performed, findings, and recommendations developed by Applied Technology & Management, Inc. (ATM) as part of this update.

This updated Executive Summary is immediately followed by the original Executive Summary from the 2006 SWMP. While portions of the SWMP were updated in this revision, some of the original information in areas outside of the revised sections remains the same as published in 2006. For clarity of previous assumptions and methodology, the original sections of the 2006 SWMP are reproduced herein to provide one location for all current information in the SWMP.

2018 Updated Background and Purpose

In 2015, Beaufort County and its partnering municipalities engaged Applied Technology & Management, Inc. (ATM) to update portions of this report and to revise certain portions of the models to reflect changes since the implementation of the 2006 SWMP. This implementation guide provides actions for watersheds throughout the County.

Until 2006, stormwater management was flood prevention management and focused primarily on moving stormwater away from roads and developments as rapidly as possible, with minimal concerns for the impacts the rapid movement of stormwater had on the unique and sensitive estuarine environment that exists throughout Beaufort County.

Since the implementation of this SWMP in 2006, considerable additional advances have been occurring in the understanding of stormwater management. Additional monitoring data and locations are now available, and the County and partnering municipalities have adopted a new rate structure to continue the implementation and operation of the stormwater utility. This update was undertaken to identify the seven watersheds that have changed the most since the previous data was gathered and to update the models and information to deliver a dynamic document that will provide updated information for implementation of improvements based on more current data.

Since completion of the 2006 SWMP, the County has accomplished the following:

- Established the level of service (LOS) and extent of service (EOS) for the County Stormwater Utility
- Developed a Capital Improvements Plan (CIP) and updated it in 2015

- Created an in-depth and detailed stormwater best management practice (BMP) manual and revised and updated the manual in 2015
  - Completed some key stormwater retrofit projects and begun new projects to implement the CIP
  - Implemented ordinances with the County Zoning and Development Standards Ordinance (ZDSO) that require stormwater treatment and discharge systems to meet certain requirements
  - Implemented a new stormwater ordinance in 2015
  - Continued to build its inventory of existing stormwater conveyance systems and update the County's GIS database
  - Implemented an updated stormwater utility rate structure in corporation with the municipalities in 2016

In addition, the municipalities have implemented many of their own stormwater conveyance systems and water quality BMPs.

- Town of Bluffton accomplishments include:
  - 2007 Adoption of a Stormwater Ordinance and BMP Manual.
  - 2009 Established USCB Water Quality Laboratory.
  - 2010 Revision of Stormwater Ordinance & BMP Manual to include stormwater volume control for water quality.
  - 2011 Adoption of the May River Watershed Action Plan with policies, programs and projects aimed at reducing fecal coliform in the May River.
  - 2013 Completion of New Riverside Pond for water quality improvement.
  - 2016 Completion of the Pine Ridge Irrigation Re-use project for stormwater volume reduction.
  - Continuing to build its stormwater infrastructure GIS database.
- The Town of Hilton Head has implemented new stormwater control systems with associated BMPs and is in the first phases of dredging and cleaning the many aged stormwater ponds within the community.
- The City of Beaufort has developed its stormwater ordinance and incorporated stormwater quality BMPs into its planning documents. The City is in the process of identifying aged stormwater infrastructure for capital planning purposes.
- The Town of Port Royal has constructed the first regional stormwater management system and continues to expand the scope of the stormwater management system service areas. The Town is in the process of inventorying its piped drainage systems and continues its street sweeping program.

Since the 2006 SWMP was implemented, the County has experienced continued growth in critical areas of the estuary and continued closure of Shellfish Harvesting Areas. To address these issues, as well as new federally mandated regulations, the County has:

- Voluntarily developed and implemented new strict stormwater volume control regulations

- Been designated by South Carolina Department of Health and Environmental Control (SCDHEC) as a Phase II small municipal separate storm sewer system (MS4) community
- Had a total maximum daily load (TMDL) adopted for the Okatie River, Chechessee River, and Beaufort River

All these major changes, as well as new and changing growth patterns related to development, have resulted in the need to update the 2006 SWMP.

A summary of the actions accomplished as part of this 2018 Implementation Guide Update is as follows:

- Performed an in-depth review of the 2006 SWMP to identify areas needing updating.
- Updated growth area mapping throughout the County and municipalities to determine growth and infill areas since 2006 utilizing a new 2016 high-resolution aerial photo and 2013 light detection and ranging (LiDAR) data.
- Reviewed hydraulic and water quality modeling performed in 2006 and updated models in the following seven priority watersheds chosen by the County and municipalities, focusing on watersheds with significant development and/or growth since 2006:
    - Beaufort River
    - Calibogue Sound
    - Colleton River
    - Coosaw River
    - May River
    - Morgan River
    - New River
- Investigated documented customer complaints to identify areas of concern through a series of public meetings held in the summer of 2016.
- Compared current findings against 2006 SWMP findings.
- Developed a revised CIP list based on updated models
- Developed a recommended inventory list

Figure ES-1 has not been updated because the overall watershed boundaries remain the same. This figure is a location map showing Beaufort County boundaries, major water bodies, tidal wetlands, upland areas, roads, and watershed boundaries.

Figure ES-2U is an update to Figure ES-2 and shows the areas of Beaufort County that the Stormwater Implementation Committee (SWIC), which is comprised of staff from each jurisdiction, selected for updated hydrologic and hydraulic modeling.

Figure ES-3U is an update to Figure ES-3 and shows the areas the SWIC selected for updated water quality modeling. Average annual pollution loads from the highlighted areas were calculated based on the updated land use information for the watersheds. In addition, bacteria concentrations were recalculated in many of the major tidal rivers and creeks, based on bacteria loadings from the load model, and calibrated tidal mixing and bacteria loss rate coefficients.

2018 Updated Hydrologic and Hydraulic Analysis Results

Locations of road overtopping problems identified by the 2006 SWMP were reviewed for the updated watersheds. Where changes occurred, locations were removed or added, as indicated by the updated hydrologic and hydraulic models. As in the previous version of this report, solutions for these problem areas in updated watersheds focused on upgrading culverts at the flooding road crossings or raising roadway elevations above flood levels.

As in the 2006 SWMP, the updated watershed analyses focused on the primary stormwater management system (PSMS) and does not address the potential for flooding of the secondary drainage system.

Locations of road overtopping problems identified by the hydrologic and hydraulic analysis are presented in Figure ES-4U.

2018 Updated Water Quality Analysis Results

Table ES-4U summarizes the classification of the water quality segments in Beaufort County water bodies based on the evaluation of the bacteria data from 1999 through 2016 for the selected watersheds. This analysis included data from additional stations that came into service post-2000 that had not been previously included. For each watershed, the tables show the number of water segments receiving “A”, “B”, “C” and “D” classifications, plus the number of segments of unknown quality (because there are no sampling stations). The table indicates that 71 percent of the water quality segments that are monitored have an “A” or “B” LOS, which means that bacteria standards are expected to be met in the long term. The remaining 29 percent of monitored water quality segments are at a “C” or “D” level, which means that bacteria standards are not expected to be met in the long term.

The table also indicates that many of the water quality segments are still not monitored by SCDHEC. Forty-seven percent of the modeled water quality segments were not monitored for the entire 17-year period. Some segments are in small tidal creeks and the headwaters of tidal rivers that perhaps would not be expected to meet the standards even under undeveloped conditions because the discharges of watershed runoff flows and loads are not subject to sufficient tidal mixing. Conversely, some segments may not be monitored because they are not affected by urban development.

Results for existing land use conditions are presented in Table ES-5U. Table ES-5U shows that 73 percent of the modeled water quality segments have an “A” or “B” LOS, and the remaining 27 percent have a “C” or “D” LOS.

The results of the analysis were used to make recommendations for water quality controls and water quality monitoring.

2018 Updated Master Plan Components

2018 Update to PSMS Enhancements

The hydrologic and hydraulic analysis identified additional locations in the updated watersheds on the PSMS that are not expected to meet the County-defined LOS for road overtopping, in addition to removing some previously modeled as a problem. Problem solutions were identified by evaluating culvert upgrades to increase the flow conveyance capacity of the PSMS and detention storage to reduce peak flows. It is recommended that these areas be reviewed in conjunction with overall water quality BMPs recommended as part of the 2018 CIP to determine if flow controls can be incorporated into the regional BMPs to help address PSMS overtopping.

Table ES-6U is an update to Table ES-6 and summarizes the costs of updated PSMS projects in the seven watersheds.

2018 Updated Water Quality Controls for Existing Development

The water quality analysis identified a number of water quality segments that are not currently meeting the fecal coliform bacteria water quality standard (based on monitoring data) and/or are not predicted to currently meet the bacteria water quality standard (based on model results for unmonitored segments). Some of these segments that are not in compliance with the bacteria standards would not achieve compliance even with treatment of all urban runoff by BMPs because tidal mixing and water body bacteria loss rates are insufficient relative to stormwater runoff bacteria loads from urban and non-urban areas.

The results of the analysis led to an assessment of potential water quality BMPs that could potentially improve water quality conditions. The analysis identified eight water quality segments that could potentially show an improvement in water quality LOS. An evaluation of potential regional BMP sites identified eight sites (Figure ES-5U) that had high potential as BMP locations as they had relatively limited potential for wetland impacts and relatively low costs of land acquisition and construction relative to the pollution load reductions that the BMP is expected to provide. Table ES-7U summarizes the costs of the recommended regional water quality projects in the seven watersheds. These projects will be added to the current CIP project list.

2018 Updated Water Quality Monitoring

An updated water quality monitoring program is recommended for Beaufort County only. The goals of the program include the following:

- Characterize baseline water quality via ambient (grab) sampling
- Identify seasonal trends and overall trends over time using long-term ambient sampling data

- Evaluate dry weather (ambient) and wet weather (automatic sampling) water quality in selected areas for comparison to pollutant concentration values used in the watershed water quality modeling effort
- Evaluate sources of bacteria (human, bird, pets, wildlife) in locations where measured bacteria levels are substantially higher than expected, based on the watershed and receiving water quality modeling

It is recommended that Beaufort County staff be responsible for monitoring on the tributaries to the major open water tidal river segments and BMP monitoring. Where coordination with other municipalities is occurring, this should be continued. This monitoring will be done in conjunction with SCDHEC's existing monitoring programs.

Water quality data from Beaufort County, the Town of Bluffton and Hilton Head Island were collected and analyzed for standard statistical parameters and for trends. The identification of appropriate sampling sites for grab sampling and automatic storm event sampling was based on the water quality statistical analysis, the current LOS for water quality segments, and the existing land use distribution. In all, four sites were selected for automatic sampling, and 52 sites were selected for grab sampling. These sites are provided on Figure ES-6U.

Sampling would be conducted on a monthly basis. Sampling events will note weather conditions, flow conditions, and tidal condition (ebb and flood). Field parameters monitored during each sampling event include temperature, dissolved oxygen (DO), conductivity/salinity, pH and turbidity. Samples will be collected and analyzed for the following parameter list:

- *Enterococci* (saltwater)
- *Escherichia coli* (*E. coli*) (freshwater)
- Fecal coliform bacteria
- Total suspended solids (TSS)
- Biochemical oxygen demand (BOD)
- Ammonia nitrogen
- Nitrite and nitrate nitrogen
- Total Kjeldahl nitrogen (TKN)
- Total phosphorus
- Chlorophyll-a
- Total organic carbon (TOC) quarterly
- Metals (cadmium, chromium, copper, iron, lead, manganese, mercury, nickel and zinc) quarterly
- Hardness, quarterly

Samples collected will be characterized as either "dry" or "wet" samples, based on the amount of precipitation received over the 72 hours preceding sample collection. If less than 0.1 inch of rain fell in the 72 hours before the time of sampling, the samples will be classified as dry weather samples. If 0.1 inch of rain or more fell during the previous 72-hour period, the sample will be categorized as a wet weather sample. By identifying the weather conditions preceding each sampling event, it is hoped that contaminant concentrations can be linked to base- or low-flow conditions, or high-flow associated with stormwater runoff, thus providing valuable diagnostic information regarding potential source(s) of pollution.

Results from the laboratory analysis and field-collected parameters will be compared to the applicable water quality standards and criteria contained in SCDHEC Rule R.61-68, Water Classifications and Standards. Modifications to the plan, including stations to be sampled and observed concentrations, will occur based on the results obtained. Recommended statistical evaluations include standard descriptive statistics including data distribution, trend analysis (Kendall-Tau) and inter-station comparison (Mann Whitney, Wilcoxon).

Four stations would also include automatic sampling stations, so that sampling will be activated during storm events and stormwater runoff sampling can be reliably conducted. The four sites will be selected to represent runoff quality from different urban land use types (e.g., industrial, residential/golf course) and observed receiving water quality. In general, the same parameters will be sampled. Measurements of rainfall, stage, velocity and flow rate will also be made at the automatic sampling stations. The purpose of this sampling is to provide additional information to better define relationships be runoff event mean concentrations (EMCs) and receiving water quality. Preliminary pollutant loading modeling has revealed locations where resultant fecal coliform loads from the model were not excessive as compared to other areas but associated receiving waters were known “hot spots” based on evaluation of water quality data (i.e., tidal creek areas of May River and Okatie River). Other factors such as salinity regime changes, flushing, etc., also have an effect on observed fecal coliform levels in receiving waters. In addition to providing local EMC data to support future modeling efforts, this also provides insights to the importance of the various factors that affect receiving quality. It is anticipated that 12 or more storm event samples will need to be collected at each location to estimate EMCs with a reasonable confidence (95%). The actual number will depend on the variability of the data record at each location.

SCDHEC stations, classified as “shellfish” stations, will be evaluated concurrently for bacteria and salinity data. The objective is to use the collected data for comparison to the water quality model results and to determine if the model parameters provided a reasonable simulation of bacteria conditions or whether the model should be refined with adjusted mixing and first-order loss parameter values.

In general, there was good agreement between the measured values and the model results. However, some of the reaches did not have good agreement. This is likely due to how the hydrodynamics of the systems are being modeled. The approach that has been used to date is based on the net flow advection of the various reaches and is a quasi-steady-state approach. This is an acceptable approach in most cases. However, given the tide range that exists in the county’s receiving waters and the dynamic salinity regimes present, a detailed 3-dimensional hydrodynamic model, such as the Environmental Fluid Dynamics Code (EFDC), is required to adequately simulate the tidal fluctuations and salinity-density gradients that exist in the receiving waters.

Development of a 3-D hydrodynamic model would be a significant effort but would provide the proper hydrodynamic foundation for improved water quality predictions.

2006 Executive Summary

Introduction

This report presents and recommends a stormwater master plan (SWMP) for Beaufort County, South Carolina, based on a study conducted by Thomas & Hutton Engineering Co. (T&H) and Camp Dresser & McKee Inc. (CDM) for the Beaufort County Stormwater Management Utility. The report summarizes the work performed, findings, and recommendations for managing the quantity and quality of stormwater in the County.

Figure ES-1 presents a location map showing Beaufort County boundaries, major water bodies, tidal wetlands, upland areas, and roads. The figure also shows watershed boundaries. In all, 12 watersheds were defined.

Background and Study Purpose

Stormwater management methods have evolved significantly since the 1970s. Before then, stormwater management focused primarily on moving stormwater away from a developed area as rapidly as possible, with little or no consideration of receiving water impacts. Then, stormwater management methods began to require the detention of stormwater to reduce the peak flows from developments for purposes of flood control and streambank erosion control. Most recently, the retention and detention of stormwater has been designed to reduce stormwater pollution loads as well as reducing flooding and erosion impacts.

Focus on the protection of Beaufort County's water bodies was advanced in the mid-1990s with the formation of the Clean Water Task Force. This task force, a volunteer citizens group, worked with local and state scientists and public officials to identify potential pollution sources and to develop a set of recommendations for action. General categories of pollution sources included stormwater, central wastewater treatment, onsite disposal systems (septic tanks), boating impacts, and monitoring and enforcement.

Beaufort County acted in accordance with one of the Task Force's recommendations by enacting a stormwater utility in 2001. The stormwater utility assesses a stormwater fee to residential, commercial and industrial property owners, and the fees collected are dedicated to stormwater-related activities. These may include operation and maintenance of stormwater systems, implementation of improvements to reduce stormwater-related problems such as flooding and stormwater runoff pollution, and related studies.

This SWMP and report were funded through the fees collected by the stormwater utility. The study was designed to identify problem areas related to stormwater, and to recommend a plan to solve problems and better control the impacts of stormwater on receiving waters in Beaufort County.

A parallel study evaluated the rate structure that is used to determine the stormwater utility fees. Together, the two studies provide the County with the information necessary to implement an updated fee structure designed to finance the recommended activities of the plan.

Study Elements

The elements of the master plan study included the following:

- Approach development. This included the establishment of Level of Service (LOS) for both water quantity (e.g., flood protection) and water quality (e.g., compliance with water quality standards), selection of computer modeling tools for the evaluation of watershed conditions and solutions for problem areas, and identification of potential management measures that would be evaluated in the study.
- Watershed data collection. This included the acquisition and review of water quality data, acquisition of pertinent physical data (e.g., land use, soil types), acquisition and review of local rainfall data, identification of areas with features such as septic tanks and existing stormwater controls, and mapping of known flooding areas based on discussion with County and municipal staffs.
- Stormwater management system inventory. This included the definition of the PSMS, which is essentially the primary system of storage, channels and culverts that carry flows from the land to the receiving water bodies; characterization of the existing system (e.g., culvert size and shape, condition, degree of siltation); and entry of appropriate PSMS data into a database for use in stormwater modeling.
- Hydrologic and hydraulic model development and application. This included the development of computer simulation models to represent watershed physical characteristics (e.g., channel cross-sections, culvert size, roadway elevations); calculation of stormwater runoff hydrographs (time series of runoff flows) for selected design storm events; routing of the runoff flows through the PSMS; identification of problem areas such as locations with road overtopping; and evaluations of alternatives to reduce or mitigate the identified problems.
- Water quality modeling. This included the development of computer simulation models to calculate the pollution loads from the watersheds to the County receiving waters, plus computer simulation models to evaluate bacteria concentrations in many of the receiving waters; comparison of receiving water bacteria concentrations to water quality standards; and evaluation of how management measures such as best management practices (BMPs) are expected to influence the compliance with water quality standards.
- Stormwater master plan development. This included the preparation of this report; a recommendation of appropriate management measures based on the

evaluations from previous study elements; estimation of costs associated with the recommended measures; and discussion of the implementation of plan elements relative to anticipated revenues from the stormwater utility.

County Watershed Characteristics

Figure ES-2 presents the areas of Beaufort County that were analyzed for detailed hydrologic and hydraulic modeling. The PSMS in Beaufort County (including the Town of Hilton Head Island) includes 164 square miles of land area. Design storm runoff flows from the PSMS area were routed through the PSMS hydraulic network, which included 168 miles of open channels and more than 300 stream crossings.

The LOS established for the design storms, developed in conjunction with County staff, is as follows:

- Evacuation routes: Road is passable for the 100-year design storm.
- Other roads: Road is passable for the 25-year design storm.
- Buildings: Flood stages will be managed below finished first-floor elevations. Modeled 100-year design storm flood elevations were compared with geographic information system (GIS) coverages of buildings, Federal Emergency Management Agency (FEMA) 100-year base flood elevations (BFEs), and light detection and ranging (LiDAR) ground elevations near those buildings to identify potential building flooding. Unfortunately, the County GIS and database do not have complete records of structure locations and finished first-floor elevations, so the study could not conclude whether or not structures in inundated areas were actually subject to flood damages. However, the analysis did indicate that the modeled 100-year peak water elevations were consistently lower than the BFEs identified by FEMA, which means that structures built in accordance with the FEMA BFEs should not be flooded because the stormwater system is inadequate. (The FEMA BFEs reflect storm surge conditions.).

The 25-year design storm and 100-year design storm include total rainfall depths of 8 inches and 10 inches, respectively, over a 24-hour period, with roughly 89 percent of the total rainfall occurring in the middle 2 hours of the event [using the Soil Conservation Service (SCS) Type III distribution].

The design storm evaluations also considered the water surface elevation at the downstream end of the PSMS, because downstream (tailwater) water elevations can affect the flow capacity of the PSMS. For the Town of Hilton Head Island, the mean high tide was used, for consistency with previous studies. For the rest of the County, a more conservative value (the mean annual high tide) was used. These water elevations were applied as a constant value over the course of the design storm so that the modeling reflected the maximum impact of downstream water elevations.

Figure ES-3 presents the areas of Beaufort County that were analyzed for water quality modeling. The total analyzed area is 725 square miles. Average annual pollution loads from the highlighted areas were calculated. In addition, bacteria concentrations were calculated in many of the major tidal rivers and creeks, based on bacteria loadings from the load model, and calibrated tidal mixing and bacteria loss rate coefficients.

The LOS for water quality focused on the concentrations of bacteria in County water bodies. Using historical fecal coliform bacteria data collected in the 1990s, long-term geometric mean bacteria concentrations at various sampling locations were calculated and then evaluated with respect to the short-term and long-term compliance with the bacteria standards at those locations.

Table ES-1 summarizes the various LOS categories that were established, indicating the relationship between each level and the short-term and long-term compliance with bacteria water quality standards. At the “A” level, both standards are expected to be achieved during any short-term (36-sample) period. At the “B” level, it is expected that the 90th percentile standard may not be achieved in all short-term periods but will be met in the long term. At the “C” level, the 90th percentile standard is not expected to be met in the long term. At the “D” level, neither standard is expected to be met in all short-term periods, and it is possible that both standards will not be met in the long term.

For this study, a “non-degradation” LOS was used as the basis for evaluating the impacts of new development and benefits of management measures. In other words, the focus was to determine whether the receiving waters are expected to maintain their current classification (A, B, C or D) in the future. The study also investigated the potential for improving the LOS of segments with an existing “C” or “D” LOS.

Table ES-2 summarizes the extent of development that was used in the analysis of existing and future land use conditions. Existing land use reflects existing County land use maps, aerial photographs and local knowledge. Future land use is based on a “buildout” condition developed by Beaufort County staff.

For each watershed, Table ES-2 lists the overall percent of urban imperviousness, as well as the range in urban impervious cover in basins within the watershed, and the basin(s) with the greatest impervious cover. Overall, the percent urban imperviousness increases from 7 percent (existing) to 9 percent (future). Watersheds having the greatest impervious cover now include Calibogue Sound (including the Town of Hilton Head Island), Colleton River, and Beaufort River. Watersheds that will see the greatest increases due to future development include May River, Colleton River, New River and Beaufort River.

Hydrologic and Hydraulic Analysis Results

Locations of road overtopping problems identified by the hydrologic and hydraulic analysis are presented in Figure ES-4. A total of 119 locations were identified as having road overtopping for the appropriate LOS design storm (100-year for evacuation routes, 25-year for other roads). In general, solutions for these problem areas focused on upgrading culverts at the flooding road crossings. Detention to reduce flooding was evaluated along the primary stormwater system but was found to be unsuitable. Most of the best regional storage locations had substantial existing wetlands, so the detention facilities would need to be "off-line" facilities constructed on higher ground adjacent to the existing wetlands. The expense associated with the significant excavation that would be required and land acquisition costs were very high relative to cost savings that would be achieved by reducing or eliminating the required downstream culvert upgrades.

Table ES-3 summarizes the number of problem areas by watershed and provides the anticipated costs associated with the solution of the problems. These planning level costs were developed for each project based on an estimated construction cost, plus a percentage to account for contingencies and engineering costs. The conceptual probable capital cost of the improvements is $22.9 million (based on December 2004 dollars).

The identified problem areas were classified as either "public" or "private" projects. Public projects are those that are located on public lands. In contrast, private projects are located in private subdivisions, military facilities, and other non-public areas. Of the $22.9 million in improvements, $15.3 million are considered public projects. It is anticipated that the utility will focus on the public projects.

The Town of Hilton Head Island, which is relatively fully developed, was studied previously in 1995, when a detailed storm drainage study was conducted. The purpose of the drainage study was to prepare an island-wide drainage inventory, identify flood prone areas, and present corrective actions to eliminate the flooding for a 25-year storm. Since 1995, the Town of Hilton Head Island and many of the plantations have embarked on a massive capital improvement program to upgrade their storm drainage system to accommodate the 25-year storm. The Town of Hilton Head Island's CIP budget for the improvements was $17 million. Approximately $12 million has already been spent, $3 million additional is under contract, and an estimated $1.5 million will be bid in the year 2005. In addition to the Town's $17 million drainage capital improvement program, both Sea Pines Plantation and Hilton Head Plantation have each constructed more than $1.9 million of drainage improvements in the past 10 years. Through these improvements, Hilton Head Island has eliminated the majority of the flooding problems for the 25-year, 24-hour storm.

The differences between the 1995 study and this study are itemized in the report. However, in summary, the 2004 study assumes all areas will be fully developed according to the zoning map and some of the watersheds have changed due to the much more accurate LIDAR topography. Through these refinements, other improvements have been identified and are recommended in this report. The conceptual probable capital cost for the recommended improvements for Hilton Head Island is $1.8 million (based on December 2004 dollars). Of that total, $1.2 million is allocated to public projects.

This analysis focused on the PSMS and does not address the potential for flooding of the secondary drainage system. The secondary drainage system may include tributary area and conveyance systems leading to evacuation routes. In general, these secondary systems can be evaluated using less sophisticated engineering analysis than was conducted for the PSMS. County staff should review the secondary drainage system, particularly as it applies to the evacuation routes identified in the study.

Water Quality Analysis Results

Table ES-4 summarizes the classification of the water quality segments in Beaufort County water bodies based on the evaluation of the 1990s bacteria data. For each watershed, the tables show the number of water segments receiving “A’, “B”, “C” and “D” classifications, plus the number of segments of unknown quality (because there are no sampling stations). The table indicates that 78 percent of the water quality segments that are monitored have an “A” or “B” LOS, which means that bacteria standards are expected to be met in the long term. The remaining 22 percent of monitored water quality segments are at a “C” or “D” level, which means that bacteria standards are not expected to be met in the long term.

Table ES-4 also indicates that the South Carolina Department of Health and Environmental Control (SCDHEC) did not monitor many of the water quality segments during the 1990s. More than half of the modeled water quality segments were not monitored for the entire 10-year period. In some cases, stations were added toward the end of the 1990s, and did not provide a complete long-term data set. Other segments are in small tidal creeks and the headwaters of tidal rivers that perhaps would not be expected to meet the standards even under undeveloped conditions, because the discharges of watershed runoff flows and loads are not subject to sufficient tidal mixing. Conversely, some segments may not be monitored because they are not affected by urban development.

Results for existing and future land use conditions are presented in Table ES-5. In general, the table shows that the existing LOS is maintained under future conditions, which were evaluated based on the implementation of wet detention pond BMPs for new development. This assumption was made because new development is required to have BMPs, and wet detention ponds are the dominant BMP type applied in Beaufort County. In addition, Table ES-5 shows that 71 percent of the modeled water quality segments have an “A” or “B” LOS, and the remaining 29 percent have a “C” or “D” LOS.

Additional analysis was conducted to evaluate “best case” and “worst case” scenarios. The “best case” scenario was conducted for existing land use with 100 percent treatment of urban runoff with wet detention pond BMPs. Although this is not possible because existing development limits the land available and suitable for BMPs, the results show which water quality segments would benefit from BMP implementation, as opposed to segments that are affected primarily by natural bacterial loads and limited tidal mixing and/or limited bacterial loss rate in the water. The “worst case” scenario was conducted for future buildout land use with no BMPs (i.e., all BMPs fail to provide any benefit). The results show which water quality segments will be most sensitive to the effectiveness of the existing BMPs and BMPs on future development. The results of the analysis were used to make recommendations for water quality controls and water quality monitoring.

Master Plan Components

Stormwater Control Regulations

Based on the findings of this study, existing stormwater controls Beaufort County that are currently applies appear to be appropriate for water quantity and water quality control, although there are some potential refinements (e.g., peak flow control for 100-year design storm).

For water quantity, new development is required to reduce the post-development peak runoff rate to pre-development peak runoff rate for design storms with return periods of 25 years or less. This requirement is more restrictive than the State standards, which require matching the peak runoff flow rate for design storm return periods of 10 years or less.

For water quality, new development is required to provide BMPs that control runoff pollution loads to an “anti-degradation” level. When future conditions were evaluated with BMPs on all new development, the results indicated that virtually all of the water quality segments maintained the same bacteria LOS that they had for existing conditions.

PSMS Enhancements

The hydrologic and hydraulic analysis identified 130 locations on the PSMS that are not expected to meet the County LOS for road overtopping. Problem solutions were identified by evaluating culvert upgrades to increase the flow conveyance capacity of the PSMS and detention storage to reduce peak flows. The evaluation of regional sites, which are typically in areas of existing wetlands, would be expensive to construct relative to cost savings achieved by reducing the magnitude of downstream improvements. Thus, the recommended solutions focus on increasing the conveyance capacity of the PSMS.

The recommended projects were assigned priority levels. The following five priority levels were established.

- Priority 1 – Road overtopping of 0.1 foot or more on evacuation routes (100-year design storm).
- Priority 2 – Road overtopping of 0.1 foot or more on non-evacuation routes (25-year storm) for major roads with no convenient alternative route.
- Priority 3 - Road overtopping of 0.1 foot or more on non-evacuation routes (25-year storm) for major roads with a convenient alternative route or a major neighborhood road with no alternative route.

- Priority 4 - Road overtopping of 0.1 foot or more on non-evacuation routes (25-year storm) for neighborhood roads with a convenient alternative route or minor neighborhood roads, with 100-year flooding greater than 0.5 foot OR 100-year road overflow velocity greater than 1 foot per second.
- Priority 5 - Road overtopping of 0.1 foot or more on non-evacuation routes (25-year storm) for neighborhood roads with a convenient alternative route or minor neighborhood roads (same as Priority 4), with 100-year flooding less than 0.5 foot AND 100-year road overflow velocity less than 1 foot per second.

In addition, each project was assigned a flood depth category. These are as follows:

- Flood level A: Greater than 9 inches of flood depth
- Flood level B: Flood depth of 6 to 9 inches
- Flood level C: Flood depth of 3 to 6 inches
- Flood level D: Flood depth of less than 3 inches

Table ES-6 summarizes the total cost of PSMS projects by priority and flood level.

Water Quality Controls for Existing Development

The water quality analysis identified a number of water quality segments that are not currently meeting the fecal coliform bacteria water quality standard (based on monitoring data) and/or are not predicted to currently meet the bacteria water quality standard (based on model results for unmonitored segments). Sensitivity analysis indicated that many of these segments that are not in compliance with the bacteria standards would not achieve compliance even with treatment of all urban runoff by BMPs, because tidal mixing and water body bacteria loss rates are insufficient relative to stormwater runoff bacteria loads from urban and non-urban areas.

The analysis did, however, identify 12 water quality segments that could potentially show an improvement in LOS from a “C” or “D” level to an “A” or “B” level. For segments with known problems achieving the standards, areas recommended for potential BMP implementation to treat stormwater from existing development. These areas are shaded in Figure ES-5.

An evaluation of potential regional BMP sites identified eight sites (Figure ES-5). These selected areas had relatively limited potential for wetland impacts, and relatively low costs of land acquisition and construction relative to the pollution load reductions that the BMP is expected to provide.

Water Quality Monitoring

A water quality monitoring program is recommended for Beaufort County. The goals of the program would include the following:

- Establish baseline water quality via ambient (grab) sampling
- Identify seasonal trends and overall trends over time using long-term ambient sampling data
- Evaluate dry weather (ambient) and wet weather (automatic sampling) water quality in selected areas for comparison to pollutant concentration values used in the watershed water quality modeling effort
- Evaluate quality of inflow to and outflow from selected BMPs (automatic sampling) for comparison to efficiency values used in this study and in the BMP Manual
- Evaluate sources of bacteria (human, bird, pets, wildlife) in locations where measured bacteria levels are substantially higher than expected based on the watershed and receiving water quality modeling

It is recommended that Beaufort County staff be responsible for monitoring on the tributaries to the major open water tidal river segments and BMP monitoring. For open water segments that are of interest, it is recommended that SCDHEC conduct the monitoring, as an extension of its existing monitoring programs.

The identification of appropriate sampling sites for grab sampling and automatic storm event sampling was based on the water quality sensitivity analysis, the current LOS for water quality segments, and the existing and future land use distribution. In all, four sites were selected for automatic sampling, and 14 sites were selected for grab sampling. These sites are provided on Figure ES-5.

For automatic sampling, four sites were selected that, in general, have the following characteristics: tributary to water quality segments that are not meeting water quality standards, dominated by a single land use type (e.g., industrial, residential), essentially fully developed, and located in a water quality basin designated for exploration of BMP retrofit opportunities. Data collected from these stations should be compared to the concentrations assigned in the watershed water quality model.

For grab sampling, 14 sites were selected that, in general, have the following characteristics: tributary to water quality segments that are expected to drop in LOS if BMPs are not effective, and a tributary area that will undergo extensive urban development in the future. The data from these stations will provide a basis for evaluating whether the water quality in the tributary is degrading as a result of new development.

The recommendations also include the evaluation of several wet detention pond BMPs, which are the dominant BMP type in Beaufort County. In particular, the efficiency of bacteria removal in wet ponds is critical in the evaluation of the protection that BMPs will provide to County receiving waters. No specific locations are recommended. However, the pond(s) should have well-defined inflow and outflow locations for sampling.

The study recommends coordination with SCDHEC to determine if SCDHEC would consider adding additional shellfish program stations (bacteria sampling) and ambient sampling (nutrients, metals) in 12 open water sites. These open water segments include locations that are considered sensitive based on the water quality modeling, plus some segments where the model predicts standards will not be met, but there are no data to validate the model. These sites are shown in Figure ES-5.

An independent peer review concluded that Beaufort County may wish to conduct additional sampling beyond the base recommended program to assess impacts on habitat in the tidal tributaries. Additional study is recommended to clearly define the objectives of this monitoring and develop program details (e.g., station selection and prioritization, frequency and duration of sampling, sample parameters).

Operations and Maintenance

For this study, the consideration of operation and maintenance has focused on the PSMS. Specific activities would include the maintenance of the bridge and culvert locations along the PSMS and the maintenance of the open channels in the PSMS. Routine maintenance of the stream crossings would include clearing of the headwater structures of obstruction and removal of silt from culverts. Maintenance of the open channels would primarily include clearing of obstructions.

Maintenance costs for the secondary stormwater management system were evaluated by the County staff and Town of Hilton Head Island staff, based on previous years' experience.

Inventory of Secondary Stormwater Management System

The master plan study developed an inventory of the PSMS, so future inventory efforts should focus on data collection for the secondary stormwater management system. Particularly in the City of Beaufort and the Town of Port Royal, maps showing the system often have outdated, incomplete or incorrect information. A complete inventory would be useful in assessing the capacity of the system and evaluating the extent of required maintenance in those areas.

Additional and On-going Study and Analysis

One recommendation is the development of flood inundation mapping and a current structure database that includes finished first-floor elevation, to evaluate potential for structural flood damage. This would help the jurisdictions identify structural flooding areas and give flood control projects in those areas a higher priority.

It should be noted that study analysis indicated that, in almost every case, the 100-year water elevations predicted by the model were lower than the 100-year BFE on maps FEMA developed. Consequently, homes built after the implementation of the FEMA flood mapping should not have finished first-floor elevations that would result in structural flood damage.

Other potential on-going activities would include periodic updates of the water quality models as land use, PSMS conduit sizes, and other physical data change.

An independent peer review suggested additional water quality model applications to (1) evaluate the model performance against a second set of independent data, and (2) conduct sensitivity analysis and uncertainty analysis to show how changes in model input values affect the results of the modeling. Further study has been recommended in the plan in accordance with the peer review findings.

Public Information

Public information should be included in any stormwater master plan. Advantages of an effective public information program include the following:

- Improve public awareness of how individual activities can affect water quality, and encourage activities (e.g., recycling) that control pollution sources
- Increase public awareness of success stories (i.e., show benefits of specific projects or activities funded by the utility)
- Enhance public involvement in protection of water quality on a watershed or basin basis (e.g., septic tank maintenance, fertilizer application)

Numerous methods can be implemented, such as creating/distributing water quality literature and media campaigns.

No specific methods are recommended for Beaufort County, although an annual budget is recommended based on experience with other jurisdictions and costs of other plan elements.

Planning Level Costs for Plan Components

Table ES-7 summarizes the costs of the various elements of the recommended stormwater master plan. In some cases, these are annual costs (e.g., maintenance), while others are one-time costs for specific projects (e.g., PSMS improvement design and construction).

The total cost for annual (ongoing) activities is $5.4 million, and the total cost of specific projects and studies is $33.2 million, based on December 2004 dollars (Table ES-7). These cost estimates are based on previous experience, utilizing unit costs such as cost of culverts in terms of dollars per foot of pipe or inventory costs in terms of dollars per acre of study area.

Implementation of the Plan Components

The implementation of the master plan will depend upon the costs required to implement the recommendations, as compared to the revenue being generated by the stormwater utility. Based on the proposed new rate structure for the utility and a base annual cost of $40 per year per billing unit, the utility is expected to generate $4.8 million per year in revenue (April 2005 estimate). By comparison, the annual costs listed in Table ES-7 already exceed the expected annual revenue, even before specific projects are considered.

This report provides several examples of potential expenditures for a 10-year planning horizon. Ultimately, the stakeholders (e.g., jurisdiction staff, citizens, regulatory agencies) will determine the appropriate level of revenue and expenditure for an effective program.

Local jurisdictions have approved increases above the $40 base rate and, therefore, the annual revenue will likely be greater than that shown in Section 16 of the report.

TABLE ES-1

LEVEL OF SERVICE CATEGORIES FOR WATER QUALITY

LEVEL OF SERVICE CLASSIFICATION	LONG-TERM FECAL COLIFORM GEOMEAN CONCENTRATION (#/100 ML)	ANTICIPATED EXCEEDANCE OF BACTERIA WATER QUALITY STANDARDS	
		GEOMEAN OF 14/100 ML	NO MORE THAN 10% OF SAMPLES EXCEEDING 43/100 ML
A	less than or equal to 7	No 36-sample period	No 36-sample period
B	greater than 7 and less than or equal to 8.7	No 36-sample period	Some 36-sample periods but not long-term
C	greater than 8.7 and less than or equal to 10	No 36-sample period	Long-term
D	greater than 10	Some 36-sample periods, perhaps long-term	Long-term

TABLE ES-2

WATER QUALITY BASIN URBAN IMPERVIOUSNESS

WATERSHED	URBAN IMPERVIOUSNESS (%) RANGE BY BASIN EXISTING	URBAN IMPERVIOUSNESS (%) RANGE BY BASIN FUTURE	URBAN IMPERVIOUSNESS (%) TOTAL WATERSHED EXISTING	URBAN IMPERVIOUSNESS (%) TOTAL WATERSHED FUTURE	BASIN WITH GREATEST IMPERVIOUSNESS
Calibogue Sound	0 - 31	0 - 32	11	12	Broad Creek 4
May River	0 - 10	0 - 18	5	11	May River 3, May River 4
Colleton River	4 - 26	4 - 30	10	14	Sawmill Creek 2
Chechessee River	0 - 8	0 - 15	2	3	Skull Creek North 1, Ballenger Neck
New River	0 - 14	4 - 21	5	10	New River 1
Beaufort River	1 - 47	2 - 53	15	19	Battery Creek 4
Coosaw River	0 - 21	0 - 25	5	7	Brickyard Creek, McCalleys Creek 1
Whale Branch West	1 - 12	3 - 17	6	8	Middle Creek 2
Morgan River	0 - 15	0 - 21	5	7	Rock Springs Creek 1, Rock Springs Creek 2
Broad River	3 - 10	3 - 11	8	10	Broad River 3, Broad River 4
Combahee River	1 - 4	1 - 4	3	3	Combahee River 1
Coastal	2	3	2	3	---

TABLE ES-3

PLANNING LEVEL COSTS FOR

PRIMARY STORMWATER MANAGEMENT SYSTEM IMPROVEMENTS

	NUMBER OF	COST (MILLION DOLLARS)		
WATERSHED	PROBLEMS	TOTAL	PUBLIC	PRIVATE
Calibogue Sound *	6	1.2	0.6	0.6
May River	5	0.9	0.9	0.0
Colleton River	26	3.3	2.1	1.2
Chechessee River	2	0.1	0.0	0.1
New River	6	0.4	0.4	0.0
Beaufort River	17	2.7	2.7	0.0
Coosaw River	17	6.8	2.0	4.8
Whale Branch West	8	1.2	1.2	0.0
Morgan River	5	0.7	0.6	0.1
Broad River	17	3.3	3.1	0.2
Combahee River	2	0.2	0.2	0.0
Coastal	3	0.3	0.3	0.0
Hilton Head Island	5	1.8	1.2	0.6
TOTAL	119	22.9	15.3	7.6

* excludes Town of Hilton Head Island

Note: Cost estimates based on December 2004 dollars.

TABLE ES-4

WATER QUALITY LEVEL OF SERVICE BASED ON MONITORING DATA

WATERSHED	Number of Segments Having Level of Service				
	A	B	C	D	UNKNOWN
Calibogue Sound	8	0	3	1	15
May River	3	0	0	0	5
Colleton River	3	1	0	2	5
Chechessee River	6	0	0	1	8
New River	---	---	---	---	---
Beaufort River	5	5	0	0	11
Coosaw River	3	4	0	0	12
Whale Branch West	1	0	0	1	7
Morgan River	5	2	0	5	17
Broad River	---	---	---	---	---
Combahee River	---	---	---	---	---
Coastal	---	---	---	---	---
TOTAL	34	12	3	10	80
% OF TOTAL	24%	9%	2%	7%	58%
% OF MEASURED	58%	20%	5%	17%	---

TABLE ES-5

WATER QUALITY LEVEL OF SERVICE BASED ON MODEL RESULTS

	Number of Segments Having Level of Service							
	Model - Existing Land Use				Model - Future Land Use			
WATERSHED	A	B	C	D	A	B	C	D
Calibogue Sound	21	2	1	3	21	2	0	4
May River	7	0	0	1	7	0	0	1
Chechessee River	12	0	1	2	12	0	1	2
Colleton River	3	3	0	5	3	2	0	6
New River	---	---	---	---	---	---	---	---
Beaufort River	10	2	3	6	10	2	3	6
Coosaw River	11	4	0	4	10	5	0	4
Whale Branch West	4	2	0	3	4	1	1	3
Morgan River	11	6	4	8	10	5	3	11
Broad River	---	---	---	---	---	---	---	---
Combahee River	---	---	---	---	---	---	---	---
Coastal	---	---	---	---	---	---	---	---
TOTAL	79	19	9	32	77	17	8	37
% OF TOTAL	57%	14%	6%	23%	55%	12%	6%	27%

TABLE ES-6U

PLANNING LEVEL COST ESTIMATES FOR PSMS IMPROVEMENTS

BY WATERSHED

PRIVATE & PUBLIC PROJECTS

WATERSHED	PRIVATE PROJECTS	PUBLIC PROJECTS	ESTIMATED TOTAL COSTS (PUBLIC AND PRIVATE)
Calibogue Sound	$1,086,000	$872,000	$1,958,000
May River	N/A	$1,521,000	$1,521,000
Colleton River	$1,132,000	$2,413,000	$3,545,000
New River	N/A	$646,000	$646,000
Beaufort River	N/A	$3,932,000	$3,932,000
Coosaw River	$6,898,000	$2,931,000	$9,829,000
Morgan River	$117,000	$604,000	$721,000
Total	$9,233,000	$12,919,000	$22,152,000

Cost estimates based on January 2018 dollars

TABLE ES-6

PLANNING LEVEL COST ESTIMATES FOR PSMS IMPROVEMENTS
BY PRIORITY AND FLOODING CATEGORY -
PUBLIC PROJECTS ONLY

PRIORITY	FLOODING CATEGORY				
	A	B	C	D	TOTAL
1	$1,751,000	$1,879,000	$1,258,000	$1,080,000	$5,968,000
2	$772,000	$942,000	$843,000	$153,000	$2,710,000
3	$2,202,000	$317,000	$467,000	$183,000	$3,169,000
4	$1,042,000	$1,301,000	$576,000	$402,000	$3,321,000
5	$0	$0	$0	$185,000	$185,000
TOTAL	$5,767,000	$4,439,000	$3,144,000	$2,003,000	$15,353,000

Note: Cost estimates based on December 2004 dollars.

TABLE ES-7U

PLANNING LEVEL COST ESTIMATES

REGIONAL BMP WATER QUALITY PROJECTS

WATERSHED	WATER QUALITY BASIN NAME	BMP PROJECT IDENTIFIER	PLANNING LEVEL COST ESTIMATE
Calibogue Sound	Broad Creek 4	Broad Creek 4	$992,000
	Jarvis Creek 2	Jarvis Creek 2	$2,444,000
Colleton River	Sawmill Branch 1	Sawmill Branch 1	$2,064,000
	Sawmill Branch 2	Sawmill Branch 2	$1,071,000
Beaufort River	Battery Creek 2	Battery Creek N1	$1,370,000
		Battery Creek N2	$619,000
	Albergotti Creek 2	Albergotti Creek 2	$602,000
Coosaw River	Lucy Point Creek North 2	Lucy Pt. Creek	$438,000
Morgan River	Rock Springs Creek 1	Rock Springs Creek 1	$431,000
TOTAL			$10,030,000

Cost estimates based on January 2018 dollars

TABLE ES-7

PLANNING LEVEL COST ESTIMATES FOR PLAN ELEMENTS

PLAN ELEMENT	ANNUAL COST (DOLLARS PER YEAR)	PROJECT COST (DOLLARS)
Stormwater Control Regulations	$100,000	$0
PSMS Enhancements	$0	$15,353,000
Water quality controls (existing development)	$0	$14,300,000
Water quality monitoring	$300,000	$100,000
Annual maintenance	$3,200,000	$0
Inventory of secondary stormwater management system	$0	$3,000,000
Additional and on-going study and analysis	$50,000	$430,000
Public information	$100,000	$0
Bonded debt service (Town of Hilton Head Island)	$1,200,000	$0
Utility administration	$400,000	$0
TOTAL	$5,350,000	$33,183,000

NOTES:

1. Annual costs account for ongoing activities (BMP inspections, water quality sampling and analysis, maintenance of the primary and secondary stormwater management system, model updates, and public information)
2. Project costs include primary stormwater management system enhancements (e.g., culvert upgrades), land purchase and construction associated with regional BMPs to control existing development, collection of inventory data for secondary stormwater management systems, and specific recommended additional studies.
3. Cost estimates based on December 2004 dollars.

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Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
Watersheds	CDM / T&H	2004
PSMS	CDM / T&H	2004

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Figure ES-1

CDM Camp Dresser & McKee Inc.

Beaufort County
Areas of Detailed Hydrologic and Hydraulic Modeling

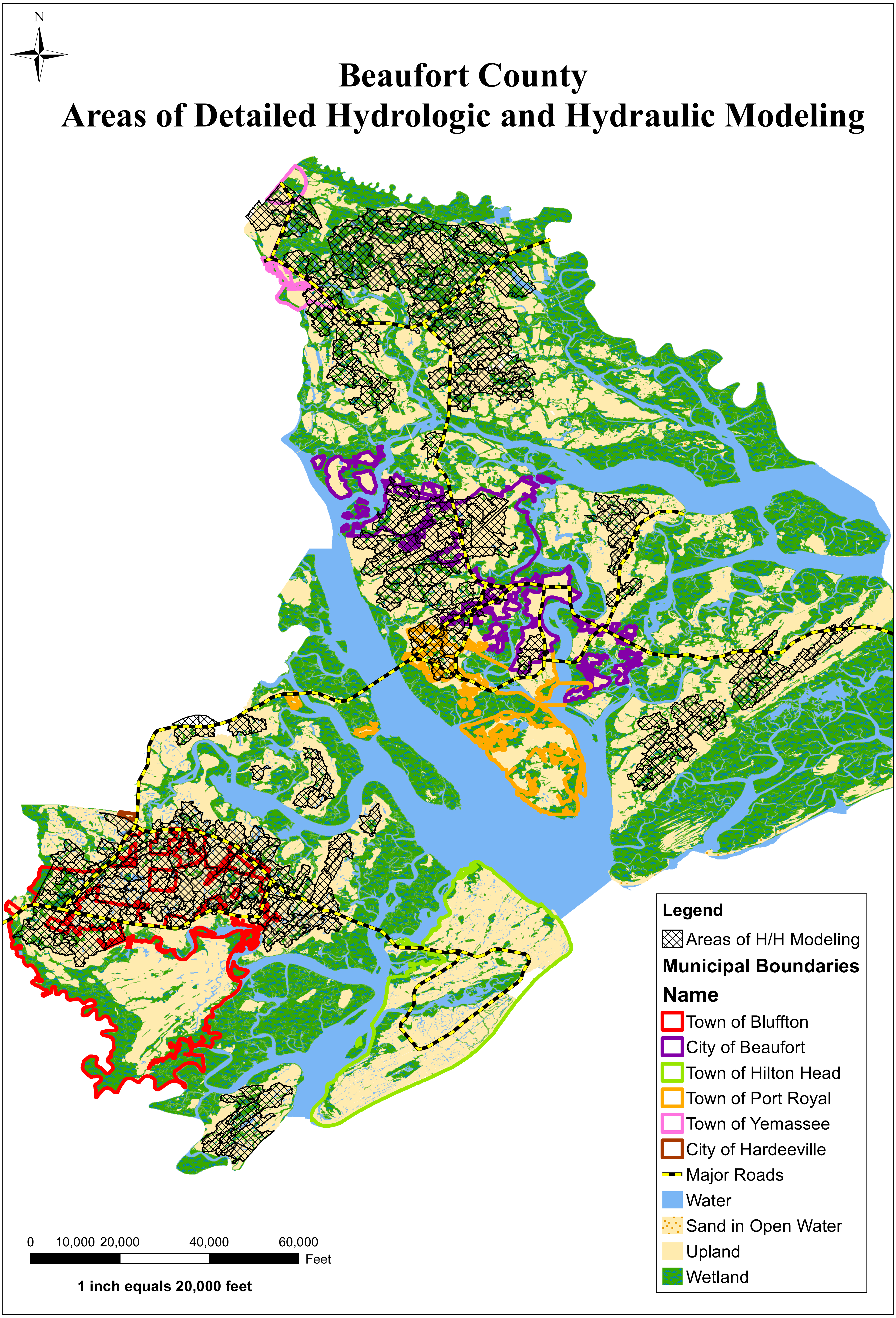

Figure ES-2U

Beaufort County
Areas of Detailed Hydrologic and Hydraulic Modeling

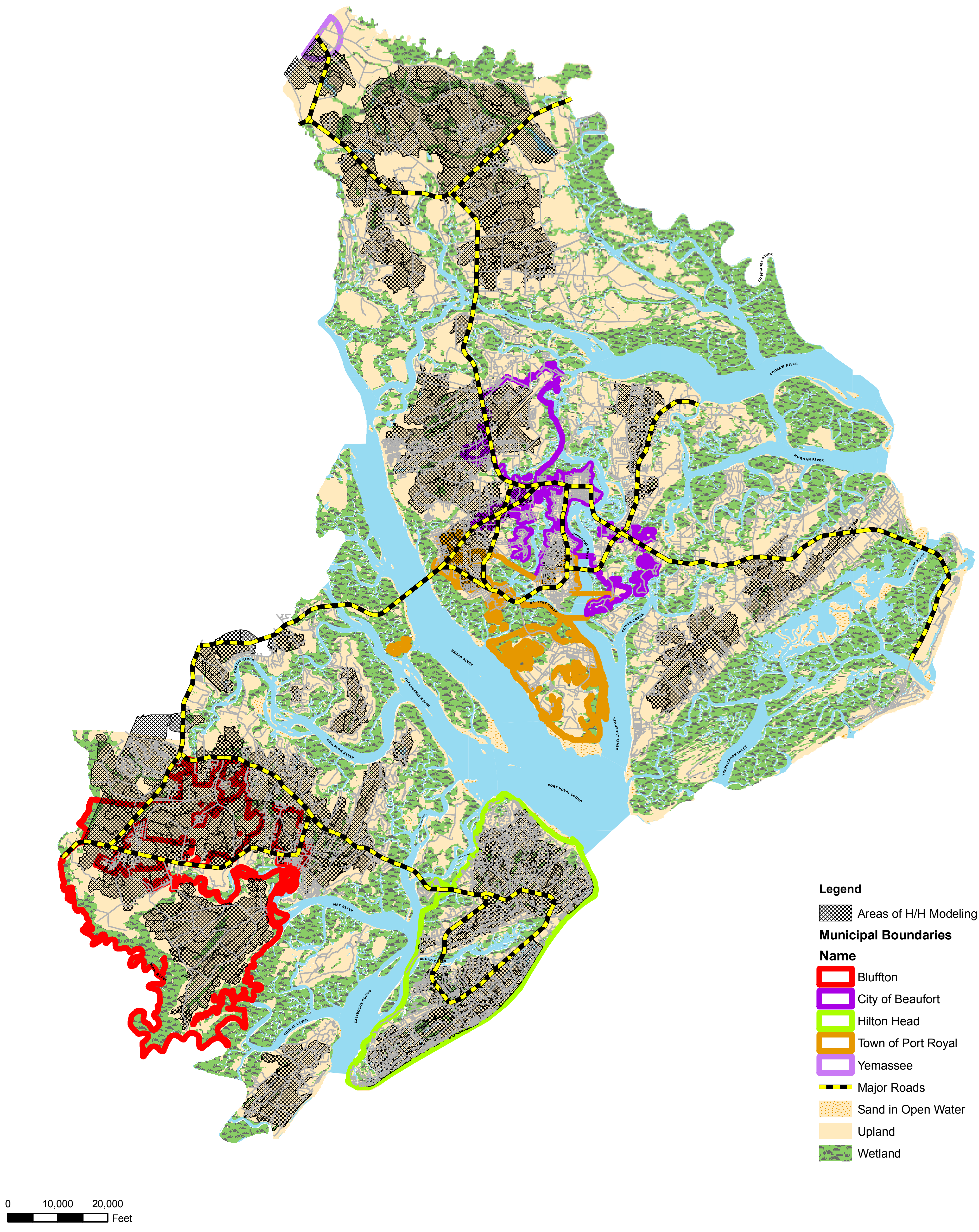

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Figure ES-2

Beaufort County
Areas of Water Quality Modeling

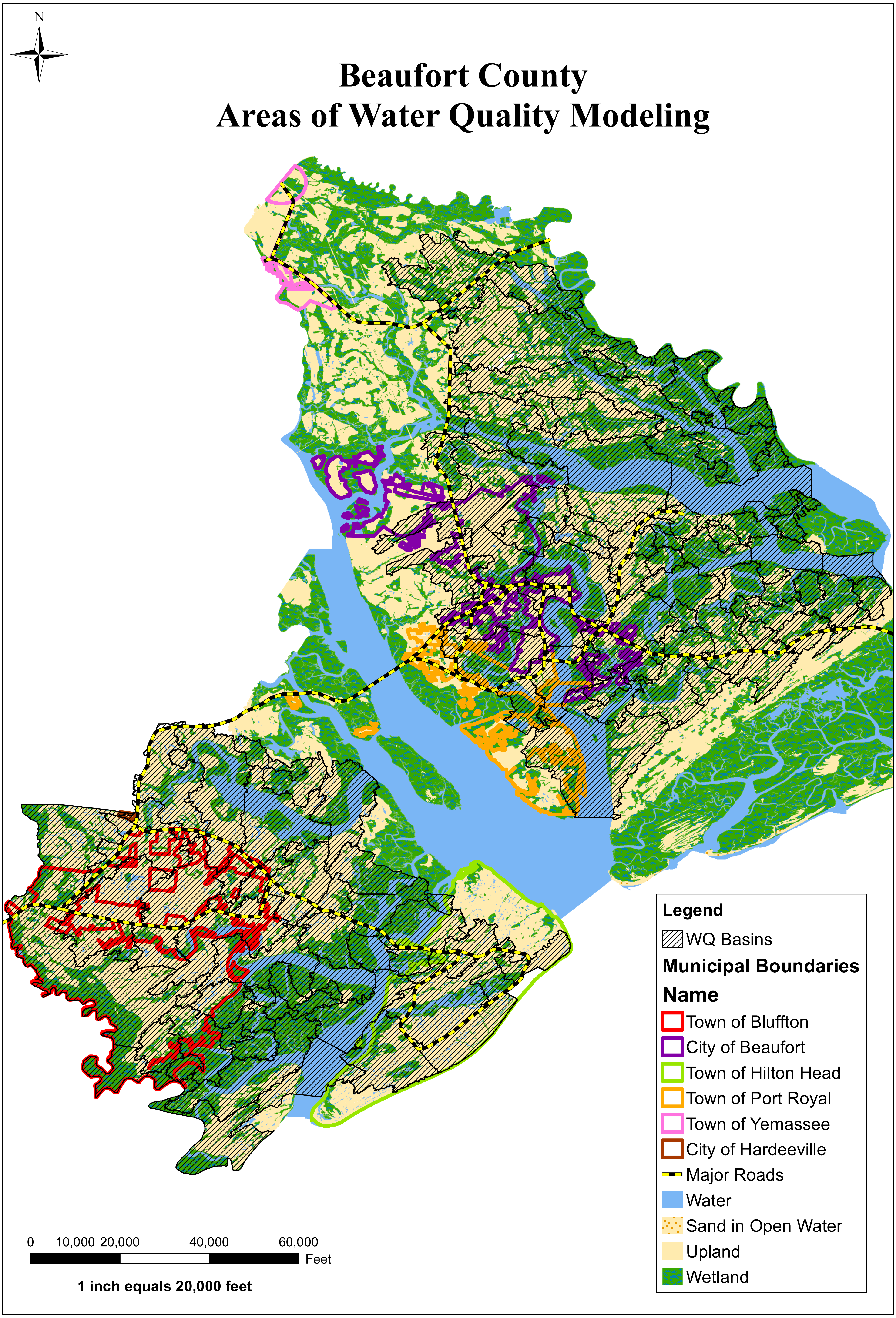

Figure ES-3U

ATM DESIGN ENGINEERING CONSULTING

Beaufort County
Areas of Water Quality Modeling

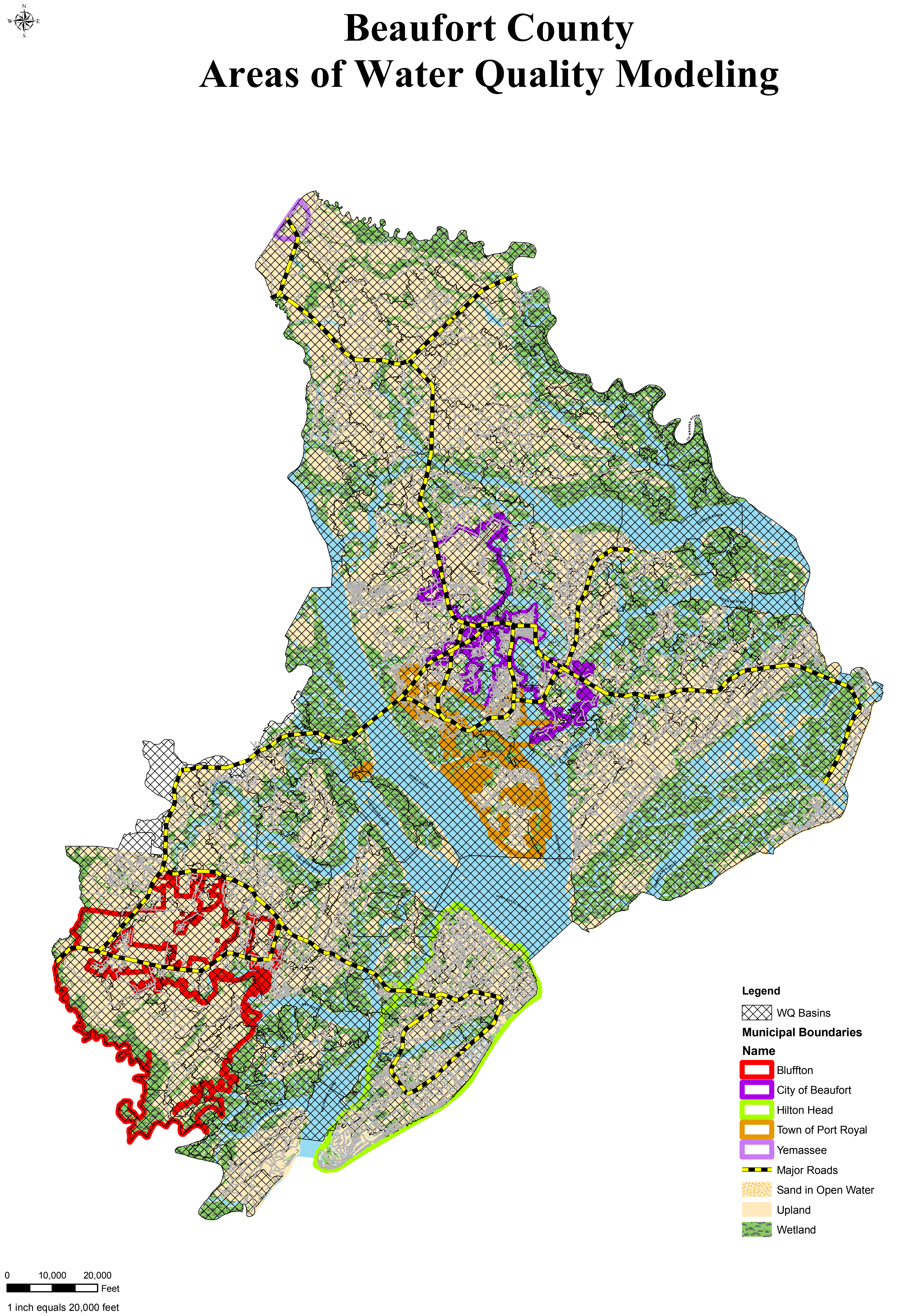

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Figure ES-3

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Land Use / Land Cover	USGS	
Municipal Boundaries	Beaufort County	2002

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DATA	SOURCE	DATE
WQ Basins	CDM / T&H	2004
PSMS	CDM / T&H	2004

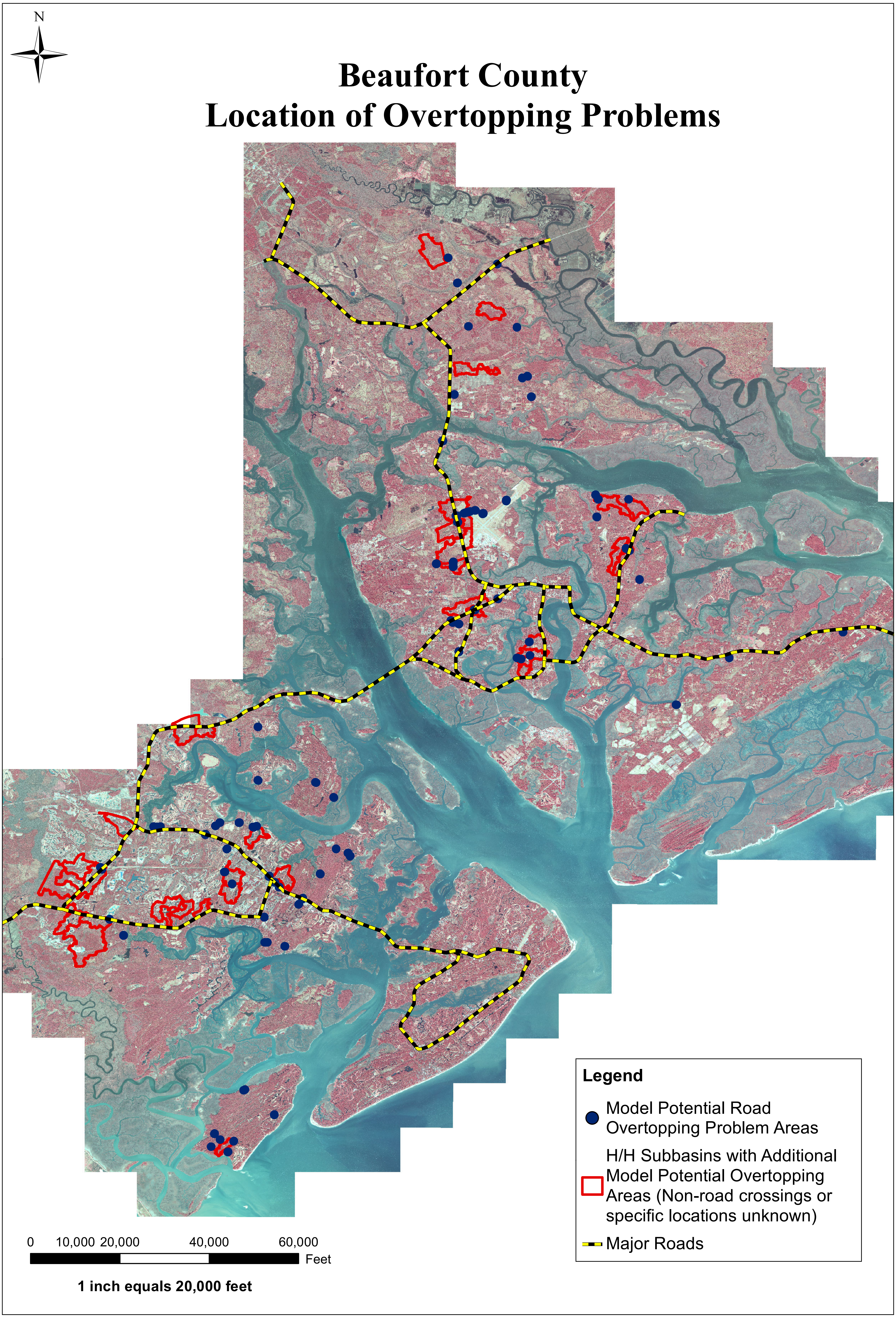

Figure ES-4U

ATM DESIGN ENGINEERING CONSULTING

Beaufort County Location of Road Overtopping Problems

Legend

● Model - Potential Problem Areas

0 10,000 20,000 Feet

1 inch equals 20,000 feet

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Figure ES-4

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DATA	SOURCE	DATE
Modeled Problem Areas	CDM / T&H	2004

Beaufort County
Potential Location for Water Quality Improvements and Monitoring

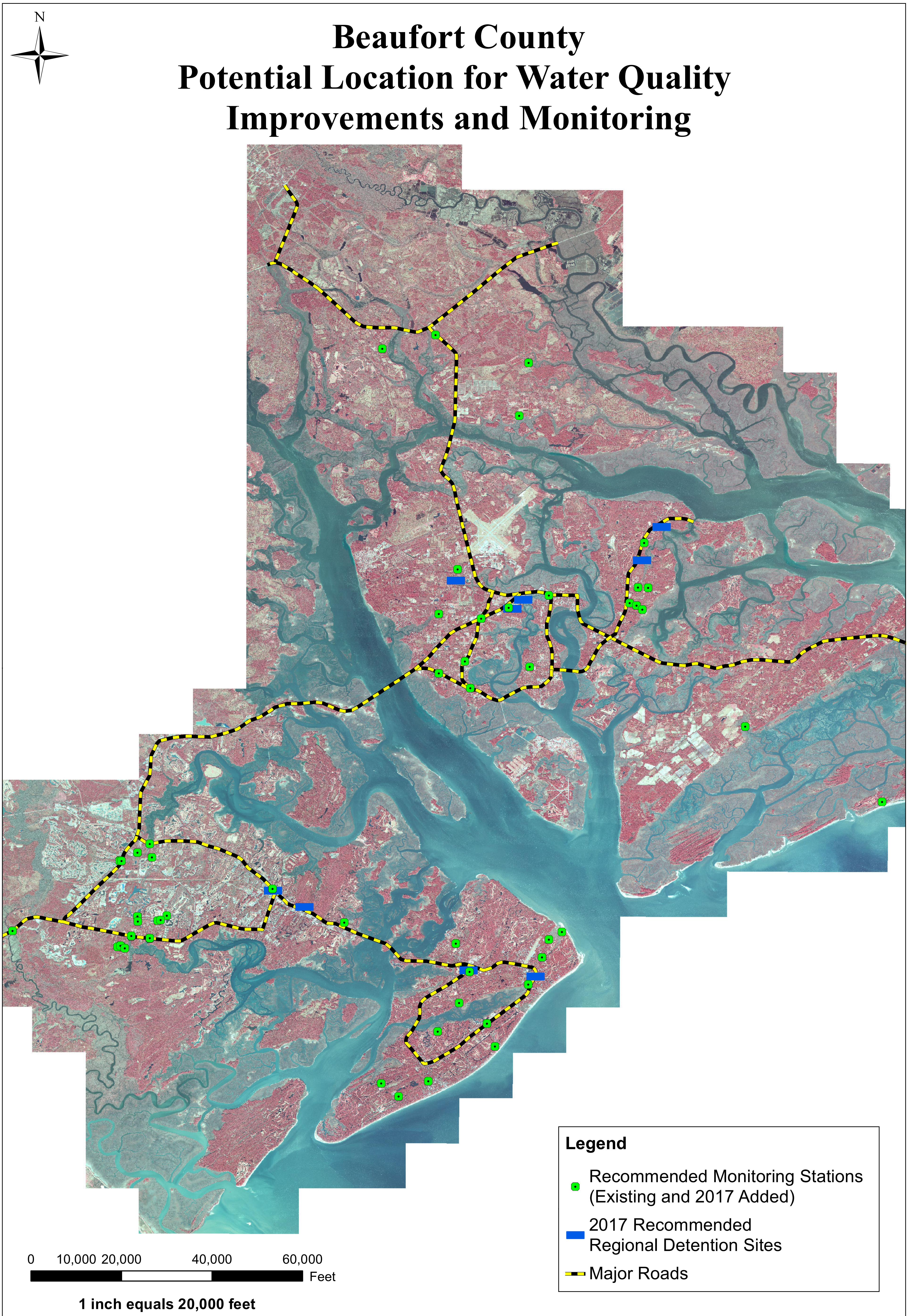

Figure ES-5U

ATM DESIGN ENGINEERING CONSULTING

Beaufort County
Potential Locations for Water Quality Improvements and Monitoring

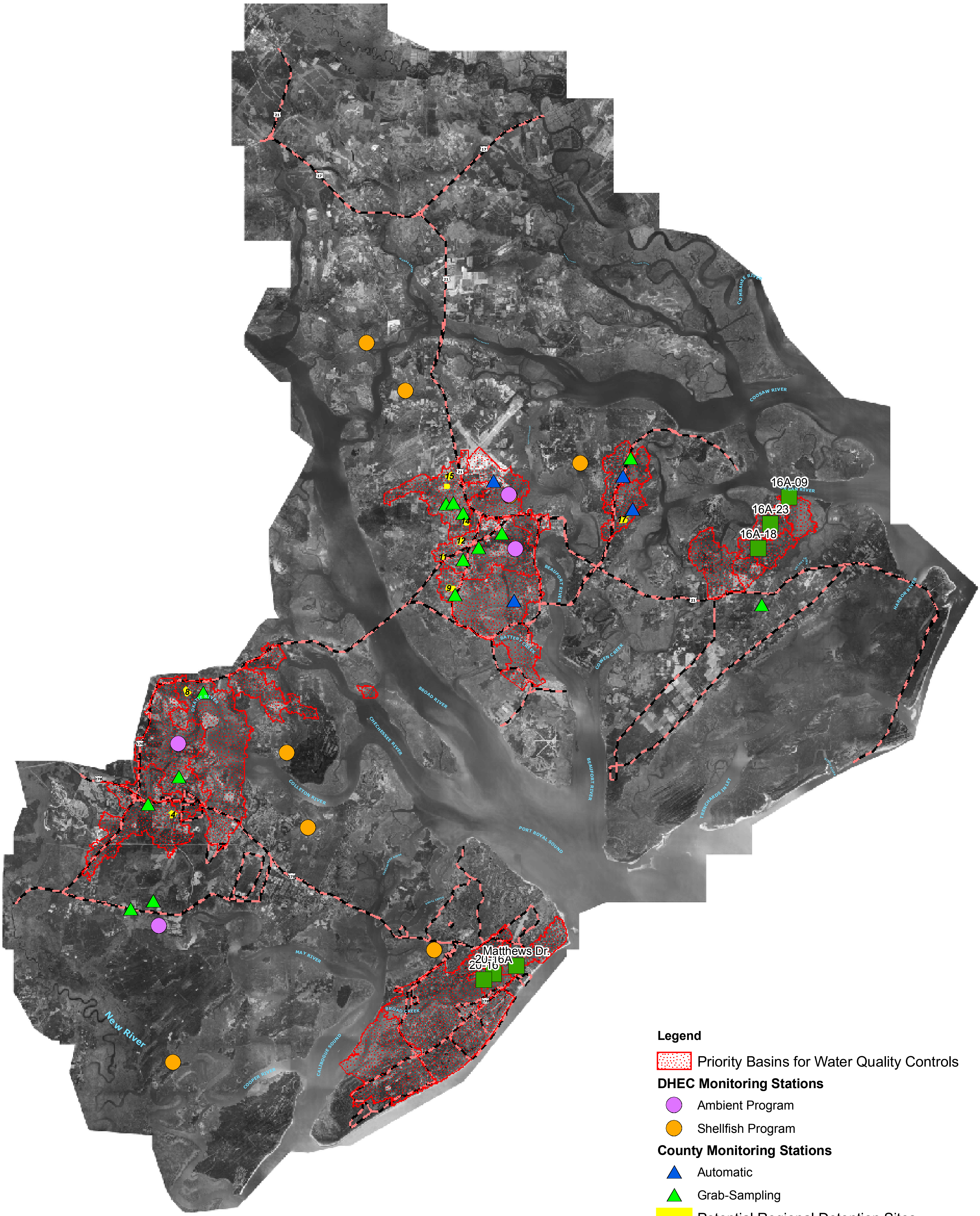

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Figure ES-5

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DATA	SOURCE	DATE
WQ Monitoring Plan	CDM / T&H	2004

Section 1
Introduction

This report presents and recommends a stormwater master plan (SWMP) for Beaufort County, South Carolina, based on a study conducted by Thomas & Hutton Engineering Co. (T&H) and Camp Dresser & McKee Inc. (CDM) and updated by Applied Technology & Management, Inc. (ATM) for the Beaufort County Stormwater Management Utility. The report summarizes the work performed, findings, and recommendations for managing the quantity and quality of stormwater in the County.

1.1 Description of the Study Area

Figure 1-1 presents a location map showing Beaufort County boundaries, major water bodies, tidal wetlands, upland areas, and roads. The figure also shows watershed boundaries. In all, 12 watersheds were defined.

Nine of the twelve watersheds have boundaries that are completely or almost completely within Beaufort County boundaries. These include the Calibogue Sound, May River, Chechessee River and Colleton River watersheds south of the Broad River, and the Beaufort River, Coosaw River, Whale Branch West, Morgan River, and Coastal watersheds north of the Broad River. For the remaining three watersheds (Broad River, New River, Combahee River), the tributary area within the Beaufort County boundaries is small relative to the tributary areas from other counties.

Many of the water bodies in the County are classified as either Outstanding Resource Waters (ORW) or Shellfish Harvesting Waters (SFH), which require a high level of water quality. Constituents such as fecal coliform bacteria are strictly limited due to potential human health impacts of shellfish consumption.

1.2 Study Elements

The elements of the master plan study included the following:

- Approach development. This included the establishment of level of service (LOS) for both water quantity (e.g., flood protection) and water quality (e.g., compliance with water quality standards), selection of computer modeling tools for the evaluation of watershed conditions and solutions for problem areas, and identification of potential management measures that would be evaluated in the study.
- Watershed data collection. This included the acquisition and review of water quality data, acquisition of pertinent physical data (e.g., land use, soil types), acquisition and review of local rainfall data, identification of areas with features such as septic tanks and existing stormwater controls, and mapping of known flooding areas based on discussion with County staff and evaluation of current floodplain maps.

- Stormwater system inventory. This included the definition of the primary stormwater management system (PSMS), which is essentially the primary conveyance system of channels and culverts that carry flows from the land to the receiving water bodies, characterization of the existing system (e.g., culvert size and shape, condition, degree of siltation), and entry of appropriate PSMS data into a database.
- Hydrologic and hydraulic model development and application. This included the development of computer simulation models to represent watershed physical characteristics (e.g., channel cross-sections, culvert size, roadway elevations), calculation of stormwater runoff hydrographs (time series of runoff flows) for selected design storm events, routing of the runoff flows through the PSMS, identification of problem areas such as locations with road overtopping, and evaluations of alternatives to eliminate the identified problems.
- Water quality modeling. This included the development of computer simulation models to calculate the pollution loads from the watersheds to the County receiving waters, plus computer simulation models to evaluate bacteria concentrations in many of the receiving waters; comparison of receiving water bacteria concentrations to water quality standards; and evaluation of how management measures such as best management practices (BMPs) are expected to influence the achievement of the water quality standards.
- Stormwater management master plan development. This included the preparation of this report, a recommendation of appropriate management measures based on the evaluations from previous study elements, estimation of costs associated with the recommended measures, and prioritization/phasing of the recommended measures.

1.3 Scope of Report

This report summarizes the results of the work performed under this study and presents recommendations for managing stormwater and water quality in Beaufort County. Recommendations include culvert/bridge upgrades, maintenance of bridges/culverts and open channels, application of existing County runoff control requirements for new development, investigation of potential water quality control measures in selected County areas, water quality monitoring, and public information.

The report is divided into 17 sections, including this introduction (Section 1). Section 2 presents the overall methodology for conducting the study and defines watershed characteristics. Sections 3 through 14 each provide the documentation of the analysis for one of the 12 watersheds shown in Figure 1-1. The documentation of the hydrologic and hydraulic modeling for Hilton Head Island is presented in Section 15, as well as water quality information for Hilton Head Island developed in the evaluation of the Calibogue Sound, Chechessee River and Broad River watersheds.

The recommended plan is presented is Section 16, and references are presented in Section 17.

Details for the hydrologic and hydraulic analyses, itemized planning-level cost estimates for stormwater management system improvements, and 100-year inundation mapping are included in separate appendices, one appendix per watershed, plus one for Hilton Head Island. In addition, a separate appendix is dedicated to documentation of the geographic information system (GIS) files developed as part of this study.

1.4 2018 Updates to the Report

The update to the report consists of revisions to the seven watershed sections included as part of the 2015 contract with ATM. The Stormwater Implementation Committee (SWIC), comprising staff from each jurisdiction, reviewed information pertaining to changes in land cover, land use, and pollution loading for all watersheds and prioritized the watersheds based on change in impervious cover, pollution loading, and location within the County. Upon review, the following watersheds were chosen for updating.

- Beaufort River
- Calibogue Sound
- Colleton River
- Coosaw River
- May River
- Morgan River
- New River

The updated sections of this report include the following sections corresponding to the watersheds chosen:

- Section 3 Calibogue Sound
- Section 4 May River
- Section 6 Colleton River
- Section 7 New River
- Section 8 Beaufort River
- Section 9 Coosaw River
- Section 11 Morgan River

A summary of the items updated are below along with some descriptions and limitations to the data and results.

- Reviewed Land Cover from 2006 report and compare to 2016 Land Cover and Land Use to compare 2006 conditions to current.
- Compared Current (2016) conditions to future. In some cases, future conditions were exceeded, however it was determined that future conditions outlined in the original report were still valid since most areas had not met or exceeded the

previously identified future conditions and where they did, they attained full buildout for all practical purposes

- Water Quality Updates –Updated watershed pollutant loading estimates using updated 2016 Land Cover and Land Use, BMP coverage, and updated septic tank coverage. Updated receiving water quality models to include new pollutant loading and long-term flow estimates and determine new modeling water quality level of service (LOS). Performed statistical analyses of available water quality data for an assessment of central tendency and trends and determined updated LOS based on data. Reviewed the ongoing water quality monitoring program and made recommendations based on the water quality data analysis. A technical memorandum summarizing the water quality data analyses, modeling and recommended changes to the monitoring program was prepared for the County and is included in the appendix to this update.
- Recommend updates to CIP – Recommended CIP projects based on results of the Water Quality updates in this report. These projects dealt with regional solutions that would produce results beneficial to the watershed, not specific jurisdictions.
- Recommended areas for additional inventory. The information used to prepare the original report was in ICPR 3.0. Complete ICPR files were not available, and the entire model for Section 15 was not available. Additionally, some inconsistencies between GIS data provided and computer models (Water Quality and hydrologic and hydraulic) required that some areas be investigated further in the future to reconcile the discrepancies. A map and list of these areas was provided to the County and is included as a part of the Appendix to this update. ATM made an exhaustive effort to locate all problem areas, including those that were identified in the 2006 ICPR model, but are no longer problem areas according to the updated ICPR modeling.
- It was assumed that the existing geometry and network of piping was not changed. The update did not allow for gathering of additional information on the pipe networks beyond that which was provided as part of the data gathering task. This means that there are additional areas (specifically where development has occurred after approximately 2013) where there should be a concentrated effort to identify any additional PSMS structures or features installed.
- Due to minimal field time as per the scope to perform this update, it was assumed that times of concentration utilized by the hydrologic and hydraulic model were unchanged from the 2006 report. Overall this assumption is sound, however it may be necessary to adjust some watersheds in the future.
- Shaded subbasins shown to identify those with additional unlocated overtopping areas and/or those with additional inventory needs / recommendations (previously provided to the County).
- While the county updated their rainfall information and storm requirements from the 25-year to the 100-year storm in the 2017 BMP Manual, this work was completed after the start of this analysis and was not utilized in this study. Rainfall was used in the study update was not changed from original models.

Computer models and data received by ATM from the county:

1. GIS Model –
    a. Pipe Inventory in 9/2016 and 3/2017
    b. 2013 Land Use
    c. 2016 Land use was provided when it was determined that 2013 was not suitable for the update.
    d. Water quality watersheds and subwatersheds
    e. Hydrologic and hydraulic watersheds (except HHI information in Section 15)
2. ICPR 3.0 Model
3. WMM Model
4. WASP Model
5. SWMM4 Model
6. 2006 SWMP
    a. Adobe pdf of entire 2006 SWMP
    b. MS Word document of select sections
        i. Table of Contents
        ii. Executive Summary
        iii. Sections 1 through 14
    c. Excel File with Tables in Executive Summary and Sections 1-14

Beaufort County Watersheds

THOMAS & HUTTON ENGINEERING CO.
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(912) 234-5300

CDM Camp Dresser & McKee Inc.

Figure 1-1

Job Number: 15178.00	Scale: 1" = 20,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\County_Watersheds_figure1-1.mxd			Vertical Datum: NAVD88

Disclaimer

Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Watersheds	CDM / T&H	2004
PSMS	CDM / T&H	2004

Section 2
Data and Methodology

This section presents a discussion of the various hydrologic, hydraulic, and water quality data and computer simulation models used or developed for the stormwater master plan (SWMP) along with a presentation of methodology including the preparation, calibration and validation of the models.

2.1 Stormwater Master Plan Modeling

An important aspect of the Beaufort County SWMP is the proper evaluation of water quantity (flooding) and water quality (nonpoint source pollutants). A good understanding of water quantity helps determine the most effective methods of controlling flooding and protecting public safety. A proper understanding of water quality and its control is essential to achieve the high quality of environmental protection desired by the County and is required to assist in permitting of selected alternatives. A series of computer models and tools were applied to simulate existing conditions and to quantify changes in flows, flood stages, velocities and nonpoint source pollutant loads in the study area due to future development.

This section documents the methods that were used to perform the water quantity and water quality modeling evaluations, including identification of the serious problems to be addressed, the structure of the model software, and the basis for the data and guidelines used in the modeling to represent the study areas within the County.

2.1.1 Stormwater Model Framework

The following paragraphs briefly highlight the water quantity and water quality model framework.

2.1.1.1 Water Quantity

CDM and T&H used the Interconnected Pond Routing Model (ICPR) Version 3 to simulate water quantity. ICPR offers many desirable features, which include the following:

- County staff are familiar with the model and comfortable with the calculation methods used in the model
- The Federal Emergency Management Agency (FEMA) approved the model for use in floodplain analysis. Therefore, the models developed in this study can be used to support changes in existing FEMA floodplain mapping in the County, although this is not included in the scope of the master plan study.
- Version 3 includes a graphical user interface (GUI) that is useful for developing stormwater system network schematics, entering and verifying model input, and viewing and presenting model results.

- ICPR can account for tidal influence, backwater effects, detention/retention pond routing and other features that are necessary for modeling in Beaufort County.

ICPR offers options for calculating runoff volumes and routing runoff generated by rainfall events. The model is used to develop runoff hydrographs from defined subbasins within a watershed. These hydrographs are then used as input at appropriate points in the hydraulic network. The ICPR hydrologic model was used to develop hydrographs for the design storms that were routed through the hydraulic network to assess the capacity of the existing hydraulic system.

ICPR provides dynamic flood routing for the channels, lakes, and stormwater infrastructure in the County's PSMS. Stages and flows from ICPR formed the basis for developing flood summary tables. Stages estimated by ICPR can be the basis for potential FEMA floodplain/elevation revisions, which is beyond the scope of this analysis. ICPR also reports conduit peak velocities for use in problem area identification. ICPR was used to route the design storms throughout the County's PSMS.

The ICPR model was used to evaluate the 2-year, 10-year, 25-year and 100-year design storms, with duration of 24 hours and a U.S. Soil Conservation Service (SCS) Type III distribution. This is discussed further in Section 2.2.6 of this report.

2.1.1.2 Water Quality

To assess annual average pollution loads in defined watersheds, ATM and CDM used the Watershed Management Model (WMM) (CDM, 1998). WMM is a Windows-based program CDM developed originally with funding from the Florida Department of Environmental Protection (FDEP) (Gao, 2003) to estimate relative changes in annual/seasonal nonpoint pollutant loads from land use, land use changes, and implementation of BMPs. WMM is still used by FDEP in some applications for the determination of total maximum daily loads (TMDLs) for impaired water bodies. WMM estimates loads based on local hydrology and non-point loading factors [event mean concentrations (EMCs)] that relate land use patterns and percent imperviousness in a watershed to per-acre pollutant loadings. Options are also available for calculating point source loads and septic tank impacts

For selected tidal rivers, ATM and CDM applied a combination of two U.S. Environmental Protection Agency (EPA) models: the Stormwater Management Model (SWMM) and the Water Quality Analysis Simulation Program (WASP). SWMM (Huber and Dickinson, 1992) was used to conceptually evaluate the simplified 1-dimensional hydrodynamics of the tidal river systems, based on information such as river cross-section geometry and bathymetry, and tidal range. ATM converted the original SWMM model to SWMM5 for use in the water quality modeling. WASP (Wool et al., 2000) uses input such as hydrodynamic data (from SWMM), average annual pollution load data (from WMM) and instream water quality process parameters to evaluate river concentrations of selected pollutants.

WMM provides annual point and nonpoint source pollutant load estimates for each watershed. For this study, pollutant loads were estimated for seven water quality constituents. Six of the constituents are among those that are monitored as part of the National Pollutant Discharge Elimination System (NPDES) stormwater permitting process. These include: five-day biochemical oxygen demand (BOD5), total suspended solids (TSS), total nitrogen (TN), total phosphorus (TP), lead (Pb), and zinc (Zn). Fecal coliform bacteria were the seventh water quality constituent because instream water quality standards for bacteria are very stringent in the SHW and ORW in the County. The WMM results are best used for relative comparisons of land use and BMP changes. Therefore, the model results were used to identify trends in nonpoint source pollutant loads, compare point versus nonpoint source loads, and identify effectiveness of BMP control options.

SWMM contains an unsteady hydraulic flow routing model for open channel and/or closed conduit systems. It uses a link-node (conduit-junction) representation of the stormwater management system in an explicit finite difference solution of the equations of gradually varied, unsteady flow. The program will simulate time-varying tidal elevations and tidal inflow/outflow. For this study, average annual flows from WMM were combined with time-varying downstream tidal conditions (based on average tidal range) and river cross-sectional geometry to calculate flows and volumes of water in the tidal rivers. These values were used to develop hydraulic data such as average net advective flow between river segments that are used by the WASP river water quality model.

The WASP model (Wool et al., 2000) is an EPA model that uses the 1-dimensional advective flow data (from SWMM), plus estimated average annual pollution loads (from WMM) and instream pollutant decay process coefficients, based on literature values and comparison of measured and modeled concentrations, to calculate salinity and fecal coliform bacteria concentrations in the tidal rivers. The tidal river model was calibrated so that modeled instream concentrations based on existing land use conditions were consistent with measured concentrations from the 1990s. The same parameter values were used in conjunction with flows and loads for the 2016 condition.

2.1.2 ICPR Hydrologic Model

This section presents further information on the ICPR hydrologic model.

As discussed, the hydrologic model used for this study is ICPR Version 3. For the Beaufort County analysis, ATM, CDM and T&H applied the curve number (CN) approach originally developed by the U.S. Department of Agriculture (USDA) SCS (1986). Under this approach, the volume of runoff generated by a model subbasin for a particular storm event is calculated as a function of the area's CN, which, in turn, depends upon the soil characteristics, vegetative cover and impervious cover of the area.

The program simulates the time series of runoff flow rates based on a unit hydrograph approach. The shape of the hydrograph is dependent upon the subbasin time of concentration, which is a representation of how long it takes for runoff to go from the most distant point in the subbasin to the subbasin outlet. The time of concentration will be affected by factors such as the subbasin size and shape, land slope, and flow length. Program results can be saved for input to the hydraulic model to perform dynamic hydraulic routing in downstream reaches.

2.1.3 ICPR Hydraulic Model

ICPR Version 3 is a hydraulic flow routing model for open channel and/or closed conduit systems. It uses a link-node (conduit-junction) representation of the stormwater management system. The hydraulic model receives hydrograph input at specific junctions by file transfer from the ICPR hydrologic model, and/or by manual input. The model performs hydraulic routing of stormwater flows through the PSMS to the points of discharge or outfalls. It simultaneously considers both the storage and conveyance aspects of stormwater management facilities. The program will simulate branched or looped networks; backwater due to tidal or nontidal conditions; free surface flow; pressure flow or surcharge; flow reversals; flow transfer by weirs, orifices, and pumping facilities; and storage at online or offline facilities.

2.1.4 WMM – Water Quality Loading Model

ATM and CDM used the WMM to estimate relative nonpoint source loads from the study area. WMM calculates annual or seasonal nonpoint source loads from direct runoff based upon the EMCs and runoff volumes associated with different land use types. Data required for WMM application includes land use distribution, runoff pollutant concentrations for each land use type, average annual precipitation, and runoff coefficients for pervious and impervious area. Additional information that can be provided includes annual baseflow rates and pollutant concentrations.

Some of the features of the WMM include:

- Estimates annual runoff pollution loads and concentrations for nutrients (TN, TP), heavy metals (lead, zinc), oxygen demand and sediment (BOD5, TSS), and fecal coliform bacteria based upon EMCs, land use, percent impervious, and annual rainfall
- Estimates runoff pollution load reduction due to partial or full-scale implementation of up to five different types of structural BMPs
- Applies a delivery ratio to account for reduction in runoff pollution load due to uptake or removal in stream courses
- Estimates annual pollution loads from stream baseflow
- Estimates point source loads for comparison with relative magnitude of nonpoint pollution loads

- Estimates pollution loads from failing septic tanks.

Stormwater pollution control strategies that may be identified and evaluated using the WMM include:

- Non-structural controls (e.g., land use controls, buffer zones, etc.)
- Structural controls (e.g., onsite and regional detention basins, wet detention ponds, dry detention ponds, etc.)

The model provides a basis for planning-level evaluations of the relative changes in long-term (annual or seasonal) nonpoint pollution loads and the relative benefits of nonpoint pollution management strategies to reduce these loads. WMM evaluates alternative management strategies (combinations of non-structural and structural controls) to develop the stormwater management plan.

2.1.5 SWMM and WASP Tidal River Water Quality Model

For the tidal river hydraulics and water quality modeling, ATM and CDM used the EPA SWMM and WASP. SWMM was used to completely calculate flows and volumes in the tidal rivers based on defined tidal boundary conditions, calculated land-based inflows, and defined river bathymetry. The calculated flows and volumes were then used as input to the WASP model, which calculated instream concentrations of selected constituents based on the land-based constituent loads, downstream boundary constituent concentration, and parameters that define instream water quality processes (e.g., tidal dispersion, die-off of bacteria).

The focus of the river modeling was on concentrations of fecal coliform bacteria. Bacteria concentrations have been monitored extensively in the County tidal rivers. In some cases, the concentrations have exceeded State water quality standards. The modeling framework can also be used to evaluate other water quality constituents (e.g., nutrients) with existing water quality computations already in the WASP model.

2.1.6 Water Quantity Model Calibration

Calibration and verification are desirable to establish a reality check of predicted stages, flows, and velocities. For calibration or verification, data must be available in the form of rainfall, stage, flow, and/or highwater marks for specific storm events, land use, and hydraulic conditions. Beaufort County has a limited number of rainfall gaging stations and no long-term stations measuring upland streamflows, so the hydrology and hydraulic models were not calibrated. Instead, the results developed by the model (e.g., road overtopping and/or structural flooding for particular design storms) were compared to known high water marks or historical flooding to validate the results generated by the model. In addition, problem areas were reviewed with County staff to evaluate whether the results calculated by the models were reasonable.

2.1.7 Water Quality Model Calibration

The water quality model calibration focused primarily on the comparison of measured and modeled concentrations of fecal coliform bacteria in the tidal receiving waters of Beaufort County. Receiving water concentrations were modeled based on average pollutant loads and tidal conditions. Resulting average receiving water concentrations were compared to available measured concentrations to demonstrate the validity of the water quality model.

The initial analysis focused on salinity concentrations in the receiving waters. Measured and modeled concentrations were compared to verify the model's ability to accurately represent the mixing of freshwater and tidal inflows to the receiving water. Key factors in the calculation of receiving water salinity include the net tidal flow (advection) between tidal river segments, tidal dispersion between tidal river segments, downstream boundary salinity concentration, and average freshwater inflow from the receiving water's tributary area.

After the salinity modeling, the calibration focused on comparison of measured and modeled concentrations of fecal coliform bacteria. Key factors in the calculation of receiving water bacteria concentrations include the net tidal flow (advection) between tidal river segments, tidal dispersion between tidal river segments, downstream boundary bacteria concentration, and the average freshwater inflow and associated bacteria concentration from the receiving water's tributary area. Another key factor is the bacteria loss rates in the receiving water. The net loss rate provides an overall representation of the processes occurring in the receiving water such as base mortality, light mortality, settling, and regrowth.

Preliminary estimates of the net first-order bacteria loss rates for each receiving water segment were developed based on the methodology developed by T&H (2001) for evaluating bacteria removal in wet detention ponds. The methodology defines the overall bacteria loss as a function of three factors, which include a base die-off rate, loss due to light, and loss due to settling. Of the three factors, the base die-off rate and loss due to light tend to dominate the overall loss rate, and loss due to settling is minimal.

If necessary, the preliminary loss rate estimates were adjusted within a typical range of literature values (Thomann and Mueller, 1987) to provide the best comparison between measured and modeled bacteria concentrations in the receiving water.

2.2 ICPR Hydrologic Parameters

Hydrologic model parameters used for the model simulations are described in this section.

2.2.1 Topographic Data

Topographic data were used to define hydrologic boundaries, overland flow slopes, critical flood elevations, channel and overbank geometries, and stage-area-storage relationships. Beaufort County light detection and ranging (LiDAR) data were the major source of topographic data for the project. The LiDAR data were used to develop a digital elevation model (DEM) that was hydroenforced to account for flow patterns that are affected by hydraulic structures such as culverts. Figure 2-1 shows an example coverage along with hydrologic subbasins and the PSMS. The vertical accuracy of the LiDAR data is ±1 foot. The vertical datum is North American Vertical Datum 1988 (NAVD88), and the horizontal datum is North American Datum (NAD) 1983. Other sources that were considered include the following:

- U.S. Geological Survey (USGS) 7.5-minute series quadrangles (1 inch = 2,000 feet 5-foot contour interval);
- Available subdivision and stormwater improvement plans obtained from the County/Town of Hilton Head Island;
- Available drainage studies obtained from the County/Town of Hilton Head Island; and
- Survey data provided by the County/Town of Hilton Head Island.

2.2.2 Basin and Subbasin Areas

Hydrologic subbasins were generally defined by natural physical features or constructed stormwater management systems that control and direct stormwater runoff to a common outfall. The following general criteria were used to determine subbasin boundaries.

- Large-scale physical features such as railroad grades and major roads were used to establish hydrologic divides.
- Subbasin boundaries were delineated where structures or topographic features could appreciably impound water for the 100-year event.
- The present condition subbasin delineations were considered to be approximately the same as the future case since the County will regulate future development to maintain present peak discharges and overall flow schemes.
- Existing construction plans, reports/studies and limited field reconnaissance were used to determine ambiguous boundaries.
- The level of detail used in the delineations was consistent with the problem area analysis. The Town of Hilton Head Island portion of the PSMS was detailed to a greater level than the remainder of the county due to the substantial amount of existing development. The majority of Hilton Head Island development is major residential/golf course plantations, with many lagoons constructed for aesthetic and storm water management purposes. Many of these lagoons serve as storage

and flow attenuation for stormwater runoff and water quality BMPs during a rain event.

Based on previous experience, the typical subbasin size is in the range of 200 to 300 acres for the majority of Beaufort County. Smaller subbasins (50 to 100 acres) were delineated in highly developed areas (i.e., Town of Hilton Head Island) and areas with known flooding problems on the PSMS. Larger subbasins (up to 600 acres or more) were delineated in some cases for rural areas where minimal development has or is expected to occur.

Hydrologic basins were generated using GIS tools in conjunction with the DEM developed from the County LiDAR data. Subbasin outlet points were defined at selected locations (e.g., major tributaries, stream crossings, regional detention pond locations), and the GIS tools delineated the area that is tributary to the outlet points. Since the Town of Hilton Head Island has extensive underground stormwater piping that is not detected by GIS (LiDAR), the DEM required hydro-enforcement to obtain accurate subbasin delineations. The resulting digitized subbasin polygons were analyzed to provide required hydrologic information such as tributary area and average land slope. Figure 2-2 shows the hydrologic basins and PSMS analyzed in this study.

2.2.3 Land Use, Impervious Area and Curve Numbers

Land use data were used to estimate the extent of impervious areas for individual subbasins for use in runoff volume calculations. An existing land use map for the County was developed from the February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps (to define extent of water and wetlands), plus local knowledge of development completed between February 2002 and June 2003 (Figure 2-3). The future land use map was developed by filling in the existing land use map, replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and the Town of Hilton Head Island future land use maps, as well as zoning maps for Beaufort County, Town of Hilton Head Island and Town of Port Royal.

Table 2-1 presents the land use categories and associated hydrologic characteristics, including percent imperviousness and CNs for various soil types. In the hydrology model, the CN is one parameter used to determine how much rainfall is converted to surface runoff, with higher CN values producing more runoff. Major factors that affect the CN value for a particular land area include the soil type, impervious cover, and antecedent moisture condition (AMC).

The CN approach was used to determine the volume of surface water runoff for the evaluated design storms. The CN approach empirically accounts for the amount of rainfall that will be lost through depression storage on the land surface and infiltration into the soil on pervious land areas. For a given design storm, the volume of runoff from pervious land areas will depend on the AMC (i.e., the amount of rainfall that has occurred for several days prior to the event). The model is capable of using several AMCs. AMC I depicts soils that are extremely dry, simulating drought conditions. AMC II depicts soils that are moderately wet with storage potential, simulating normal everyday rain patterns. AMC III depicts soils that are fully saturated, with minimal storage potential, simulating an extreme rainy weather pattern. For this study, an average AMC II was used for all design storm analyses.

For a particular model subbasin, the composite CN is calculated based on the distribution of land use and soil type in the subbasin. The GIS represents the subbasin as a series of small grid areas and assigns each grid area a specific land use type and soil type, and a corresponding CN. The CN values for each small grid area are then area-weighted to develop the overall CN for the subbasin.

2.2.4 Soil Types and Characteristics

Soils data are a key input in evaluating stormwater runoff volumes from pervious land area. Information on soil types was obtained from the SCS Soil Survey of Beaufort County, South Carolina (SCS, 1980). Each soil type is assigned to a soil association, a soils series, and to one of the four Hydrologic Soil Groups (A, B, C, or D) established by the SCS (Figure 2-4). Hydrologic Soil Group A comprises soils with a very high infiltration potential and a low runoff potential. Hydrologic Soil Group D comprises of soils with very low infiltration potential and a high runoff potential. The other two categories fall between A and D soil groups. Dual class soils (e.g., A/D) mean that a hard pan or impermeable layer limits vertical infiltration, but the surficial soils are highly permeable and could infiltrate as a Class A soil if the confining layer was cut with a ditch or swale.

For this study, dual hydrologic group soils were evaluated based on degree of drainage and were represented as one soil group (A, B, C, or D). Generally, dual group soils were treated as hydrologic group D unless a confirmed lowering of the adjacent water table had occurred as a result of development. For more information on specific soils or soil groups, consult the USDA-SCS National Engineering Handbook, Section 4, Hydrology (USDA, SCS, 1972).

2.2.5 Subbasin Time of Concentration

The SCS unit hydrograph method was used to develop hydrographs (i.e., time series of surface runoff flow rates) for the model subbasins. The calculated surface runoff volume (a function of the land use and soils discussed in Sections 2.2.3 and 2.2.4) is distributed based on model input parameters, which include the time of concentration (T_C) and the peak runoff factor.

The time of concentration is generally described as the time it takes for runoff to travel from the most distant hydraulic point in the subbasin to the subbasin outlet and can be estimated using several methods. In this study, the following equation (USDA, SCS, 1972) was used to estimate the time of concentration for a hydrologic subbasin:

$$Tc = 1.67 * L^{0.8} * (S' + 1)^{0.7} / (1900 * S^{0.5})$$

Where:
Tc = time of concentration (hours)
L = flow length (ft)
S = mean subbasin slope (percent)
S' = potential water storage = (1000/CN) – 10
CN = curve number

Like the CN, the values for flow length and mean subbasin slope were generated via the LiDAR data.

2.2.6 Rainfall Intensities and Quantities

Rainfall data are used by the hydrologic model in the determination of runoff volumes for the design storms. Data are generally characterized by amount (inches), intensity (inches per hour), frequency/return period (years) duration (hours), spatial distribution (locational variance), and temporal distribution (time variance). Daily rainfall data are available for a rain gage at Beaufort beginning in 1930, and hourly data are available for airport gages in Savannah, GA, and Charleston, SC, beginning in 1948.

For the Beaufort County stormwater master plan study, the analyzed design storms included 24-hour duration storms, with return periods of 2 years, 10 years, 25 years, and 100 years. State regulations require new development to limit post-development peak flows to pre-development levels for the 2-year and 10-year design storms. County regulations are more stringent, additionally requiring peak flow control for the 25-year design storm. The 100-year storm is typically evaluated to estimate extreme flood impacts and evacuation route planning.

Table 2-2 presents design rainfall amounts for the 2-, 10-, 25-, and 100-year frequency, 24-hour-duration storms from several sources. These include the Weather Bureau's Technical Paper No. 40 (USDA, SCS, 1961), as well as values calculated from the available rainfall data at the Beaufort, Savannah and Charleston gages. Rainfall periods at these gages ranged from 71 years (Beaufort) to 46 years (Charleston).

Technical Paper No. 40 (TP40) presents maps showing lines of equal rainfall depth, similar to the way that topographic maps show lines of equal land elevation. Maps are presented for various storm durations and return periods, including the duration (24 hours) and return periods (2, 10, 25, and 100 years) considered in the Beaufort County study.

Values for the Savannah and Charleston stations were calculated using the methodology presented in TP40. The hourly rainfall data were analyzed to develop a 24-hour maximum rainfall for each year of record, and this annual series was fit to a Gumbel extreme distribution to develop the rainfall depth for each return period. Recognizing that some years may have more than one extreme storm event, conversion factors were applied to account for the difference between results generated for annual series (highest value for each year only) and for partial series (all high values, regardless of year in which they occur). Based on TP40, appropriate conversion factors for the 2-year and 10-year return periods are 1.01 and 1.14, respectively. No conversion factors are recommended for greater return periods.

Values for the Beaufort station were calculated using the same methodology as for the Savannah and Charleston stations. However, an additional correction factor was applied to the results because the statistics were based on daily, rather than hourly, data. TP40 suggests that a factor of 1.13 is appropriate, based on comparison of statistics calculated using hourly and daily data. The rationale is that measuring a specific 24-hour period and recording that as the daily rainfall is not likely to actually measure the maximum 24-hour rainfall, which is likely to overlap two 24-hour periods.

As shown in the table, the rainfall depths for all sources are similar, with the range of depths at any return period limited to 0.6 inch of rain or less. The values from TP40 tend to be less than or equal to the values generated using hourly or daily records from the nearest rain gage locations, with the Beaufort gage results typically having the highest values for return period of 25 years or more, and the Charleston gage results having the highest values for return periods of 10 years or less.

Rainfall intensities were then generated for each design storm using the SCS Type III rainfall distribution (USDA, SCS, 1986). The Type III distribution was developed by the SCS to represent Gulf of Mexico and Atlantic coastal areas, where tropical storms bring large 24-hour rainfall amounts. As shown in Figure 2-5, about half of the rainfall occurs during the middle 2 hours of the design storm event (Hours 11 to 13). About 19 percent of the storm rainfall occurs during the most intense 15-minute period in the storm event.

In summary, rainfall quantities for the four design storms used for this study are as follows:

- 100-Year/24-Hour – 10.0 inches of rainfall, with 7.6 inches/hour 15-minute peak intensity
- 25-Year/24-Hour – 8.0 inches of rainfall, with 6.1 inches/hour 15 minute. peak intensity
- 10-Year/24-Hour – 7.0 inches of rainfall, with 5.3 inches/hour 15-minute peak intensity
- 2-Year/24-Hour – 4.5 inches of rainfall, with 3.4 inches/hour 15-minute peak intensity

These values have been used in numerous stormwater control infrastructure designs in Beaufort County and are very close to the values calculated from long-term records at the Beaufort, Savannah and Charleston gages.

2.3 Hydraulic Parameters

The County's PSMS consists of stream, canals, culverts, detention ponds, and storm sewer systems that discharge into tidal rivers (Figure 2-2). The first step in the hydraulic model development is the creation of a simplified representation of the actual system. This is done by developing a model schematic, which can also be used for checking model input data and interpreting model results. Typically, the schematic will show the subbasin load points for inflow, conveyance channels, and structures, as well as the storage and linking junctions. Identification numbers for various system elements are also shown on the schematic. The schematic provides a quick reference between the actual physical situation and the model system. The following paragraphs describe the information used to develop the ICPR hydraulic models.

2.3.1 Primary Stormwater Management System Inventory

A detailed inventory of the PSMS is one component of this study. To date, two major studies of the PSMS have been performed for Beaufort County. The Beaufort Engineering Services, Inc. (BES) study (1994) analyzed the majority of the Beaufort County PSMS. This study was general in nature. The Island Wide Drainage Study (IWDS) (1995) analyzed the stormwater system of Hilton Head Island. This study was extremely detailed and considered the secondary drainage system as well as the primary drainage system. Both studies have been utilized to extract supplemental inventory data used in this study.

For the majority of Beaufort County, a preliminary PSMS was mapped on USGS quadrangle maps based on the previous drainage study by BES (1995). In general, this preliminary PSMS included stormwater conveyances systems with a tributary area of 320 acres or greater and, in some cases, tributary areas of less than 320 acres were considered in urban areas. Survey crews collected field data to define stream crossings (e.g., culvert size and shape, distance from culvert invert to top-of-road) based on the initial PSMS. The crews also noted drainage features that were not identified in the BES study and collected field data for these features. County staff reviewed the initial PSMS maps and added known drainage features that were considered part of the PSMS. Figure 2-6 shows an example inventory system.

For the Town of Hilton Head Island, an initial PSMS was mapped on USGS quadrangle maps based on the IWDS prepared by T&H (1995) and engineering experience in the area. In general, the IWDS methodology included conveyance systems with a tributary area of 5 acres or greater. The small tributary areas are due to the extensive existing development on the island. Survey crews collected field data to define stream crossings (e.g., culvert size and shape, distance from culvert invert to top-of-road) for the IWDS.

Although rare, the crews also noted drainage features that were not identified in the IWDS and collected field data for these features. County and Town of Hilton Head Island staff reviewed the initial PSMS maps and added known drainage features that were considered part of the PSMS. Figure 2-6 shows an example inventory system.

Open channel cross-section dimensions were obtained and input for the hydraulic modeling using a combination of LiDAR and survey data. Initially, the cross-section geometry was determined using a DEM developed from the LiDAR data. In some cases, the LiDAR data did not detect the incised cross-section of the channel. In those cases, surveyed cross-sections of the incised channel were used to define the channel portion of the cross-section, while the LiDAR data defined the overbanks of the cross-section. The data were spliced together to represent the unused channel plus floodplain overbank. Since the Beaufort County LiDAR uses NAVD88, the survey data often was converted from North Geodetic Vertical Datum 1929 (NGVD29) to NAVD88. The datum conversion factors vary by geographic location. For Beaufort County/Hilton Head Island, approximately 0.9 foot must be subtracted from NGVD29 to obtain elevations in NAVD88.

PSMS inventory information has been stored in a database developed as part of this master plan project. The types of information recorded for the inventoried facilities include locations, lengths, pipe diameters, pipe construction material, and pipe invert elevations. This information formed the foundation for the model representation of the PSMS.

2.3.2 Floodplains and Floodways

Along coastal areas, two classifications of floodplains. tidal and stormwater, generally exist. Tidal floodplains are the result of tide and wind-generated flood stages whereas stormwater (sometimes called fluvial) floodplains are associated with rainfall. It is common practice for FEMA floodplain studies to consider tidal and stormwater flood events independently and then superimpose the independent results to produce comprehensive tidal/stormwater floodplain maps.

For Beaufort County, the FEMA Flood Insurance Rate Maps (FIRMs) identify much of the County as floodprone, with 100-year base flood elevations (BFEs) of 12 to 22 ft NGVD, which is approximately equivalent to 11 to 21 ft NAVD. The Flood Insurance Study (FIS) focused exclusively on tidal storm surge analysis, which depends upon the local storm characteristics and bathymetric characteristics. These elevations include the base stillwater elevation plus additional water height due to tidal waves. The highest BFEs are located at or near the shoreline, where the wave heights are the greatest, and are lower inland where wave heights will have attenuated. Figure 2-7 shows the FEMA floodplain.

Clearly, the storm water master plan for Beaufort County did not consider the control of these extreme storm surge events. Instead, the analysis of the PSMS focused on providing sufficient flow carrying capacity, subject to a less-extreme tidal boundary condition such as the 1-year stillwater. This is discussed in Section 2.3.4.

2.3.3 Stage Area Relationships

Stage-area information was developed in GIS using the LiDAR elevation data for major depressional areas that could not be uniformly incorporated into channel/wetland cross sections. This process was used to more accurately reflect floodplain storage. The same procedure was applied to existing detention ponds on the PSMS that were modeled explicitly. Stage-area relationships for existing facilities were obtained from topographic data shown on record plans or estimated from the new topographic mapping generated using the LiDAR data.

2.3.4 Boundary Conditions

Hydraulic boundary conditions are needed to simulate the tailwater effects of the tidal rivers and sounds on peak water elevations in the PSMS evaluated with the ICPR model. The mean annual high tide value was used for the majority of Beaufort County. For the portion of the study pertaining to Hilton Head Island, the average of the mean high water (MHW) elevation and mean higher high water (MHHW) elevation was used as a tidal boundary.

For the majority of Beaufort County, available tidal data were reviewed to determine appropriate mean annual high tide values for Beaufort County. The main source of data was the Center for Operational Oceanographic Products and Services (CO-OPS). The center is part of the National Ocean Service (NOS), National Oceanographic and Atmospheric Administration (NOAA). CO-OPS collects, analyzes and distributes historical and predicted water levels.

Table 2-3 summarizes tidal information developed from CO-OPS site data. Each of the stations listed in the table has an associated benchmark sheet, which identifies key tidal elevations such as MHW, mean low water (MLW), and NAVD88, which is the elevation basis for the DEM. The annual maximum elevation was developed by averaging the maximum water elevation for the period of record at each station, which was often only a single year. Because of the limited measured data, the values for several stations were averaged to develop an overall annual maximum elevation for use as the downstream boundary condition for the hydraulic model.

A review of the data suggested that different downstream boundary conditions may be appropriate for different receiving waters in the County. As shown in Table 2-3, stations that can be associated with Calibogue Sound and Port Royal Sound as the source of incoming tidal water tend to have a higher annual maximum high tide than the stations that can be associated with St. Helena Sound. For receiving waters associated with Calibogue and Port Royal Sound, the average value of 5.6 ft NAVD was used as the downstream boundary condition for design storm hydraulic modeling. For receiving waters associated with St. Helena Sound, the average value of 4.7 ft NAVD was used as the downstream boundary condition.

For the New River, there is only a single station with benchmark information, and extreme high tide data were not available to calculate an annual maximum elevation. Based on the relationship between annual maximum elevation and MHW elevation at other stations, an annual maximum elevation of 4.5 ft NAVD was estimated for the New River at the Highway 170 bridge. This value was used as the boundary condition for the New River at any location upstream of the Highway 170 bridge. Review of tidal range information at other New River stations without benchmark sheets suggests that the Calibogue/Port Royal Sound annual high water value of 5.6 ft NAVD is appropriate for the New River at and downstream of Doughboy Island. Between the Highway 170 bridge and Doughboy Island, the downstream boundary elevation was estimated by interpolating between the Highway 170 and Doughboy Island values.

For the Town of Hilton Head Island, development started in the 1950s. Many of the roads, parking lots and existing developments are at elevations well below the mean annual high tide. Also, Hilton Head Island is extensively developed, and the majority of the island's lagoon water levels are at elevations 4 NGVD29 or lower. In contrast to the remainder of Beaufort County, the bulk of the drainage outfalls for the Town of Hilton Head Island drain directly to tidal creeks and marshes. The majority of the remainder of Beaufort County is higher in elevation than Hilton Head Island and drains through a series of long wetlands that eventually empty into tidal outfalls. Direct connections with tidal areas, as opposed to draining through a series of wetlands, are much more effective and efficient in preventing flooding. To retrofit the Town of Hilton Head Island's existing drainage systems to comply with the mean annual high tide (6.5 NGVD29; 5.6 NAVD88) tailwater boundary condition, substantial drainage system upgrades and dikes would be required within some area.

The 1995 IWDS utilized the 25-year storm with a tailwater elevation of 3.9 NGVD29 (3.0 NAVD88). The 3.9 NGVD29 tailwater condition is an average of the MHHW and the MHW and was determined appropriate and practical by Town staff and T&H. For the 1995 IWDS, this tailwater elevation was determined to be "reasonable" due to the island's low elevations, direct discharge from outfalls to the marsh, and its stage of development. Since a tailwater of 3.9 NGVD29 has been justifiably implemented in past studies and designs for the Town, and no historical flooding of designs implementing this tailwater have been documented, a tailwater elevation of 3.9 NGVD29 (3.0 NAVD88) is implemented for the Hilton Head Island portion of this study. As history indicates, construction of drainage systems originally designed with tidal tailwater elevations of 3.9 NGVD29 has yielded a safe, economical and practical engineering solution to discharging stormwater on Hilton Head Island.

2.4 Watershed Water Quality Parameters

The quality of stormwater runoff is directly related to the land use, imperviousness, and the extent of structural and non-structural BMPs associated with that land use. In this study, numeric estimates of the annual stormwater loadings were developed to assess the source and magnitude of pollutant loads along with effectiveness of existing and future stormwater in Beaufort County. WMM was used to develop estimates from land use, rainfall, and streamflow. The capabilities of the public domain version are documented in a Compendium of Watershed-Scale Management Models for TMDL Development (Shoemaker et al., 2001).

The calculations of the model will be based on the observation that the flow-weighted concentration of pollutants in stormwater runoff is characteristic for each type of land use. That is, the runoff from medium-density single-family residential parcels, for example, flow-weighted contains similar concentrations of bacteria, nutrients and other pollutants. In contrast, commercial areas are characterized by different flow-weighted concentrations in the runoff. Land-use-based flow-weighted concentrations was originally derived from EPA's Nationwide Urban Runoff Program (NURP) conducted during the early 1980s (EPA, 1983). This program collected the runoff from more than 2,000 storms from individual and mixed land use watersheds across the country and analyzed it for a wide spectrum of pollutants. Recently, the results of EPA's municipal NPDES stormwater permit program have been used to supplement and refine the earlier NURP data.

2.4.1 Rainfall

Daily rainfall data were available for a rainfall gage designated as Beaufort Seven SW in Beaufort County. Data from this gage, presented in the Beaufort County Stormwater Management Drainage Plan (BES, 1995) were previously used to determine the average annual rainfall for the Beaufort County Manual for Stormwater Best Management Practices (CDM, 1998; CDM, 2003). The recent data collection updates the original database by including data through the year 2000.

The daily rainfall data were analyzed to re-evaluate the average annual rainfall for purposes of estimating average annual runoff totals for existing and future land use conditions and to determine the frequency associated with various daily rainfall totals.

Table 2-4 summarizes the average monthly and annual rainfall data over the period of 1930 through 2000. As shown, the average annual rainfall at the gage is 48.4 inches per year, which is the same value as was used in the BMP Manual.

2.4.2 Stormwater Runoff Quantity

The watershed characteristic that most affects the amount of runoff (and, therefore, the pollutant loading) is the land use distribution and the percentage of impervious land cover associated with each land use type. Structures such as parking lots, roadways, roofs and other structures that cover the land and prohibit rain from infiltrating the soil are known as impervious areas, and most of the rainfall onto impervious surface is converted to runoff. Conversely, pervious areas such as forests and lawns typically allow infiltration of most of the rainfall, and only a small fraction of rainfall is converted to runoff.

For purposes of estimating runoff from impervious areas, it will be estimated that 90 percent of the rainfall on impervious areas becomes runoff. The other 10 percent is lost to evaporation of water captured in depression storage on the impervious surface. Percent impervious values used for the water quality evaluations are shown in Table 2-5. With an average annual rainfall of 48.4 inches per year and a runoff coefficient of 0.90, the average annual runoff from impervious land area is 43.6 inches per year.

Water and wetlands land use require special consideration. In this study, open water and tidal marshland associated with the tidal river are treated differently than water and wetlands located in the upland areas. In the upland areas, the water and wetlands land uses were assigned an imperviousness of 25 percent, which results in 30 percent of rainfall converted to flow into the tidal rivers. This value is consistent with studies from the southeastern United States. (CDM, 2000). All flow from these areas was attributed to the surface runoff with no baseflow. For the open water and tidal marshlands, a runoff coefficient of 1.0 was assigned (i.e., 100 percent conversion of rainfall to runoff).

Based on a previous analysis for the May River watershed (CDM and T&H, 2002), the estimated runoff coefficient for pervious land area is 0.10 (i.e., 10 percent of rainfall is converted to runoff). With an average annual rainfall of 48.4 inches and a runoff coefficient of 0.10, the average annual runoff from pervious land area is 4.8 inches per year.

2.4.3 Stormwater Runoff Quality

During a storm event, the concentration of pollutants in the runoff varies considerably over time. For example, the concentration of oily substances from roadways is highest during the first part of the storm, and then decline quickly after the bulk of the material has been washed off. This is known as the first-flush phenomenon. However, the concentration in the first-flush runoff is not representative of the entire storm. To estimate the loading from a storm, the flow-weighted average concentration is needed. Known as the EMC, the flow-weighted concentration is derived as the average of total loading divided by total runoff for a series of storm events. In practice, the runoff quality is sampled periodically throughout the storm event. For each sampling interval, the concentration and the quantity of runoff are combined to get a loading for the interval. At the end of the storm, the results are summed to develop the EMC (total mass divided by total runoff), which describes the average concentration for the storm. These results are combined with the results from many storms (e.g., 20 or more) and statistically evaluated to arrive at a representative EMC for each land use.

While some deviations exist, generally the results are transferable throughout a region (e.g., South Carolina), especially for relative comparisons. This is possible because the characteristics of the land use tend to be similar. For example, the amount of roadway and amount of residential area maintained as lawns is similar for residential parcels of similar densities (homes per acre).

The EMCs chosen for use in the County are provided in Table 2-6. Many of these values were presented in the Beaufort County BMP Manual and are based on extensive sampling of storm events at stations throughout the southeastern United States (CDM, 1998; CDM, 2003).

2.4.4 Baseflow Quantity

In addition to estimating stormwater runoff loads, the WMM calculates loadings associated with base flow as a separate routine. Based on a previous analysis for the May River watershed (CDM and T&H, 2002), the assumed average annual baseflow for pervious land area is 7 inches per year.

Therefore, the resulting total flow from pervious land area is about 12 inches per year (5 inches of runoff and 7 inches of baseflow), which is consistent with long-term USGS flow records for gages that are close to the study area and thought to be representative of the study area.

2.4.5 Baseflow Quality

The values presented in Table 2-7 were used to calculate the annual loads due to baseflow (groundwater flow) to the watershed receiving waters. These values were developed from local monitoring data collected by T&H at the Eagle’s Pointe and Buckwalter sites (T&H, 2002).

2.4.6 Wastewater Discharges

There are several direct point source discharges in Beaufort County. These include the following:

- Parris Island wastewater treatment facility (WWTF) (Beaufort River)
- Southside WWTF (Beaufort River)
- Shell Point WWTF (Beaufort River)
- U.S. Marine Corps Air Station (Albergotti Creek)
- U.S. Marine Corps Air Station (Broad River)

Discharge monitoring reports (DMRs) and information from the EPA Permit Compliance System (PCS) for each of these discharges were obtained and evaluated. These sources provide monthly records of measured flows and pollutant concentrations that were used to establish the flows and concentrations for existing conditions. In some cases, the water quality constituents of interest were not measured as part of the DMR process. In these cases, typical discharge concentrations from the literature were assigned.

Table 2-8 lists the assigned flows and concentrations. For the point sources, the values for flow, BOD, TSS and fecal coliform bacteria are based on average DMR or PCS values. The DMRs did not include TN (only ammonia nitrogen is sampled), TP, lead and zinc, so the values in Table 2-8 are based on typical literature values.

Table 2-8 lists the values assigned for sprayfield application. Flows and loads calculated for sprayfield applications are based on the following assumptions, established in previous studies (CDM, 1993).

- Flow to watershed receiving waters from a sprayfield is 25 percent of the total spray application rate. This is based on the WMM results for pervious areas, which in this study, assume that 48 inches of rainfall produces 12 inches of receiving water flow.
- Sprayfield practices are assumed to remove 95 percent of the constituent mass applied to the sprayfield (i.e., 5 percent of constituent load onto sprayfield reaches the receiving water).
- To get the 5 percent delivery of constituent, assuming that 25 percent of the flow gets to the receiving water, the assigned concentrations are 20 percent of the actual concentration of the applied effluent. Therefore, the concentration values in Table 2-8 for sprayfields are 20 percent of the average values for the three direct point source discharges.

For example, assume that 1 million gallons per day (mgd) of effluent is applied to a sprayfield with a constituent concentration of 1 milligram per liter (mg/L). The load of constituent to the land surface is 8.3 pounds per day (1 mgd * 1 mg/L * 8.34 (conversion factor to get in units of pounds per day)). The expected discharge to the receiving water is 0.25 mgd (25 percent flow delivery), with a load of 0.4 pounds per day (95 percent load reduction). The corresponding concentration of the delivered flow is 0.20 mg/L (0.4 mgd / 0.4 pounds per day / 8.34 conversion factor), which is 20 percent of the applied concentration.

Table 2-9 summarizes the direct discharge and indirect discharge (i.e., sprayfield) flows by watershed in Beaufort County. For existing conditions, the direct discharges are based on the values in Table 2-8, and the indirect discharge values are based on data provided by the Public Service Districts (PSDs) on the Town of Hilton Head Island, and the Beaufort Jasper Water and Sewer Authority. For future conditions, the flows were estimated based on the increase in residential land in the watersheds, and corresponding estimate of population change in the watershed.

Calculations indicate that the loads generated by direct and indirect wastewater discharges typically are a very small fraction of the total load in the Beaufort County watersheds.

2.4.7 Failing Septic Tanks

Some of the existing development in Beaufort County is serviced by septic tanks, and it is likely that some of these tanks are failing to provide proper treatment. Reasons for septic tank failure include high water table, structural failure, unsuitable soils, direct connection between septic tank and receiving waters, and failure to provide maintenance of the septic tank. Failing septic tanks are expected to discharge high concentrations of nutrients and bacteria.

Nutrient and bacteria concentrations for failing septic tanks were developed from a review of septic tank leachate monitoring studies. Typical concentrations established based on the literature values are as follows:

- TN: 30 mg/L
- TP: 2 mg/L
- Fecal coliform bacteria: 750,000/100 milliliters (mL)

These values reflect pollutant removal within the soil of roughly 50 percent for TN, and 90 percent for TP and bacteria, based on average effluent concentration cited in the literature (CDM, 1993; EPA, 2001).

Nutrient and bacteria loadings for specific land uses were calculated by multiplying the concentrations by a flow rate. The flow rate for a particular land use depends upon the number of residents per acre, and the per capita flow rate.

Table 2-10 shows the septic tank flow rates developed for various land uses. For residential land uses, a per capita flow rate of 75 gallons per day was established. This value is at the high end of the range of flow rates documented in the literature. This value was applied along with the typical residential density (units per acre) and population (number of persons per household) to establish the total residential flow rate. For non-residential urban land uses, the flow values were set equal to flows for high density residential land use.

The table also lists the loading factor used in WMM to reflect the impact of failing septic tanks. In WMM, the surface runoff load is multiplied by this factor to assess the combined load from surface runoff and failing septic tanks. For example, if the TN surface runoff load is 10 pounds per acre per year (lb/acre/yr), and the failing septic tank factor value is 2.0, the model calculates that the combined load from surface runoff and failing septic tanks is 20 lb/acre/yr.

A final consideration in the loading analysis for failing septic tanks is the failure rate (i.e., what percentage of the septic tanks are failing). Previous studies (CDM, 1993) have estimated failure rate ranging from 8 to 20 percent. For the Sarasota Bay National Estuary Program Study (CDM, 1993), permitting data from the County Health Department indicated that an average of 1.6 percent of septic tanks in the County were being repaired annually. Recognizing that a septic system may fail for years before being repaired, the value of 1.6 was multiplied by a factor of 5 (assuming average period of failure before repair is 5 years), to establish an 8 percent failure rate. This value is consistent with a septic tank survey conducted in Jacksonville, Florida, by the Department of Health and Rehabilitative Services. In the study, an inspection of more than 800 sites revealed about 90 violations, or a failure rate of 12 percent.

In the absence of any detailed surveys such as those conducted in Jacksonville, a typical failure rate of 10 percent will be used for Beaufort County. Discussion of the failure rate with Health Department staff suggests that this value is reasonable.

2.4.8 Structural Best Management Practices

The State of South Carolina and Beaufort County both have regulations that require treatment of stormwater runoff. Stormwater treatment is commonly provided in the form of structural facilities, such as wet detention ponds, extended dry detention ponds, infiltration facilities and vegetated swales. Known as a form of BMP, these structures provide different pollutant removal efficiencies. The effectiveness of a given BMP depends on the type and size of facility and type of pollutant. For example, if a particular pollutant exists mostly in the dissolved form, then a BMP which relies on settling of solid particles to achieve pollutant reduction will be less effective.

Beaufort County has a manual for stormwater BMPs that is the basis for the evaluation of BMP plans for proposed new urban development. The manual provides information regarding the selection of appropriate BMPs based on the development size, intensity of development and site characteristics (e.g., soil type). For the most common structural BMP types, the manual offers guidance on the proper design of the facility to enhance pollutant removal capability and discusses routine and non-routine maintenance requirements.

One of the most common BMP types in Beaufort County is the wet detention pond, which has a permanent pool of water. Wet ponds are one of the most effective BMPs in removing pollutants and offer an aesthetic benefit and potential for other uses (e.g., recreation) depending on the pond size. In general, wet ponds are designed to achieve a 2-week residence time. The Beaufort County manual provides permanent pool sizing criteria based on an average 2-week residence time for the wettest month of the year (August), so the annual mean residence time is in excess of 2 weeks.

For other BMPs such as extended dry detention ponds and infiltration BMPs, the manual has design criteria based on a water quality volume (i.e., amount of runoff that can be captured) and a drawdown time (i.e., how long does it take the facility to empty after the storm ends). For Beaufort County, the manual provides sizing criteria that are based on the capture and treatment of 90 percent of the stormwater runoff. The drawdown time is 24 hours, for consistency with State regulations.

Swales provide areas for settling of particulate matter (and attached pollutants), and thus are more efficient at removing pollutants which tend to be associated with solids. Swales are not designed to capture a significant portion of the runoff, but simply to slow the movement of stormwater to enhance the settling.

Table 2-11 lists the types of stormwater BMPs that are addressed in the County BMP manual, along with the removal efficiencies used in WMM. The removal efficiencies were updated using data from CWP (2007), the International Stormwater BMP Database (bmpdatabase.org) (Geosyntec and Wright Water Engineers, 2012) and the FDEP Basin Management Action Plan (BMAP) program. The BMP coverage (including type) within the County for existing land use conditions is presented in the chapters that document the water quality analyses.

2.4.9 Model Calculations

The estimation of watershed pollutant loading is accomplished by determining the flow rate and associated pollutant concentration with each load source (e.g., surface runoff, baseflow, wastewater discharges) and using those data to calculate the watershed load. The model calculates load by source so relative contributions can be compared. Loads are also calculated with and without BMPs to show the load reduction benefits provided by the BMPs.

2.5 Tidal River Segment Water Quality Parameters

The evaluation of tidal river water quality began with an analysis of existing monitoring data. Monitoring stations on Beaufort County and 303(d) locations (where the State has determined that water quality standards are not being met) are presented in Figure 2-8.

Selected tidal rivers were analyzed to evaluate water quality concentrations for fecal coliform bacteria. River concentrations were calculated and compared to applicable water quality standards and/or criteria to assess whether the standards and criteria are achieved under existing and future land use conditions, with various management strategies. Figure 2-9 shows the conceptually modeled tidal rivers.

2.5.1 Selected Tidal Rivers

The tidal river analysis focused on rivers for which the tributary area is entirely or primarily inside the Beaufort County boundaries. These include the following:

- Calibogue Sound (includes Mackay Creek, Old House Creek, Jarvis Creek, Broad Creek, Skull Creek, and Cooper River)
- Okatie/Colleton River (includes Callawassie Creek, Sawmill Creek)

- Chechessee River (includes Mackay Creek, Skull Creek, Chechessee Creek
- Morgan River (includes Parrot Creek, Bass Creek, Coffin Creek, Village Creek, Eddings Point Creek, Jenkins Creek, Lucy Point Creek, Rock Springs Creek)
- Coosaw River (includes McCalleys Creek, Lucy Point Creek, Brickyard Creek, Bull River/Wimbee Creek, and Williman Creek)
- Whale Branch (includes Huspa Creek, Haulover Creek, and Middle Creek)
- Beaufort River (includes Cowen Creek, Capers Creek, Distant Island Creek, Broomfield Creek, Albergotti Creek, Brickyard Creek, and Battery Creek)

2.5.2 Tidal River Segment Volumes

For the purposes of tidal river water quality modeling, available tidal data were reviewed to determine appropriate tidal ranges for Beaufort County. The main source of data was CO-OPS. The center is part of NOS NOAA. CO-OPS collects, analyzes and distributes historical and predicted water levels.

Table 2-3 summarizes tidal information developed from CO-OPS site data. Each of the stations listed in Table 2-3 has an associated benchmark sheet, which identifies key tidal elevations such as MHW, MLW, and NAVD88 which is the elevation basis for the DEM. The MHW and MLW values were used to develop a mean tidal range that was used in determining typical low tide and high tide volumes for the tidal rivers.

Transects were drawn across each tidal river from the downstream boundary to the headwaters. At each transect, the cross-sectional area at MLW was determined based on USGS quadrangle maps and NOAA NOS nautical charts. The MLW volume between transects was calculated as the average of the MLW cross-sectional area at the transects, multiplied by the distance between transects. The intertidal volume (i.e., the difference between MLW and MHW volume) was calculated by averaging the open water MLW surface area and the combined open water/tidal marsh surface area at MHW, and multiplying the average surface area by the mean tidal range between MHW and MLW.

The calculated MLW and MHW values were used to subdivide each tidal river into segments. The tidal prism approach used in the May River study (CDM and T&H, 2002) was used for the river segmentation. Through this methodology, river segments were established so that the MLW volume in a downstream segment was less than or equal to the MHW volume in the immediate upstream segment. This is necessary to be consistent with the theory behind the WASP receiving water model, which assumes each river segment is completely mixed.

2.5.3 Movement of Flows and Bacteria between Tidal River Segments

The WASP model used a tidally averaged approach, with annual average flows and loads taken to model salinity and bacteria concentrations in selected tidal river segments. Tidally averaged models account for advective flow and transport of salinity and bacteria based on net flow between segments over the tidal cycle. Tidal mixing between segments is taken into account by establishing appropriate dispersion coefficients in the model.

In the tidal rivers, SWMM5 was used to calculate the one-dimensional advective flows between tidal river segments. Each river segment was defined as a storage node in SWMM, with surface area values based on the low tide and high tide area defined in the GIS for open water and tidal marsh. The storage nodes were connected by short open channel segments in SWMM, with USGS topographic maps and/or bathymetric charts used to characterize the cross-section geometry at the boundaries between the river segments. Downstream boundary tidal conditions were applied in SWMM (based on values in Table 2-3) so that SWMM could define the time-varying downstream stage during a typical tidal cycle.

SWMM used the time-varying tidal boundary conditions and the estimated average flows from the river segment tributary areas to determine time-varying flows between model segments. During periods when the tide is coming in, flow is generally directed from the downstream segment to the upstream segment. Then, the flow goes from upstream to downstream segments when the tide is going out. SWMM calculated the time-varying flow over the tidal cycle, and summarizes the net flow over the simulation, which is used to determine the net flow from one river segment to another.

The net flow determined by SWMM was used in the WASP water quality model to simulate the advective movement of salinity and fecal coliform bacteria between river segments. In small tidal tributaries (e.g., Albergotti Creek, Battery Creek), the net flow is essentially equivalent to the freshwater flow from the segment tributary area. In contrast, some of the tidal rivers are influenced by tidal inflows at multiple locations.

For example, Brickyard Creek connects the Beaufort River and the Coosaw River, with tidal boundaries of Port Royal Sound and St. Helena Sound, respectively. As shown in Table 2-3, the average tidal boundary at Port Royal Sound has a greater tidal range and higher high tide value than at St. Helena Sound. As a result, SWMM calculates a net advective flow up the Beaufort River and Brickyard Creek into the Coosaw River.

The tidal mixing between river segments is evaluated using dispersion coefficients in the model. These dispersion coefficients were established based on comparison between modeled salinity values for existing land use conditions and average salinity values calculated from 1990s monitoring data.

2.5.4 Existing Tidal River Segment Salinity and Bacteria Concentrations

Monitoring data collected by the South Carolina Department of Health and Environmental Control (SCDHEC) during the 1990s were analyzed to determine baseline existing concentrations for salinity and fecal coliform bacteria. SCDHEC collects random monthly samples at tidal river stations, including many stations in the tidal rivers that were evaluated in this study. The 1990s data represent a good long-term record of concentrations that reflect monitoring during a period that includes years of average, above average and below average rainfall. Data beyond 1999 were obtained after the initial data analysis had been conducted and, in general, the bacteria concentrations in this period were low because it was a period of below-average rainfall. Consequently, it was concluded that the 1990s data provided a better overall representation of bacteria levels, and the newer data were not added to the analysis.

2.5.5 Downstream Boundary Salinity and Constituent Concentrations

Because of the substantial impact of tidal mixing and flushing in the tidal rivers, river segment concentrations of salinity and bacteria are significantly affected by the downstream boundary concentrations, particularly for the most downstream tidal segments. The boundary concentrations for existing conditions was set based on measurements at sampling stations (if available) or set based on the concentrations in the most downstream tidal river segments.

2.5.6 Fecal Coliform Bacteria Net Loss Rates

In the tidal river segments, the fecal coliform bacteria net loss rate was modeled as a first-order loss rate. Initially, a value of 1.0/day was estimated, which is equivalent to a 50 percent loss of bacteria per day. This value was adjusted through the model calibration process to provide better agreement between the measured and modeled geometric mean bacteria concentrations.

2.6 Level of Service for Water Quantity and Quality

The LOS for the Beaufort County PSMS refers to the desired level of protection against water quantity and water quality impacts. The LOS selected for this study are discussed below.

2.6.1 Water Quantity

For water quantity, the LOS considers problems such as road overtopping and structure flooding, specifying to what extent these features will be protected. Based on discussion with County staff, the LOS specifies that evacuation routes should be passable for the 100-year design storm, and any other roads should be passable for the 25-year design storm. Evacuation routes will be considered passable if there are two lanes (24 feet width) of road that are above water at all times during the 100-year design storm, based on the PSMS hydraulics model. Other roads will be considered passable if there is one lane (12 feet width) of road that is above water at all times during the 25-year design storm event. For building flooding, buildings should be protected from the 100-year design storm. Specifically, the first-floor finished elevation of any structure should be higher than the peak water elevation calculated by the PSMS hydraulics model for the 100-year design storm.

Unfortunately, the local jurisdictions do not have a database of finished first-floor elevations, so the results of the design storm analyses could not be used to identify structures that would suffer flood damage. However, the 100-year design storm flood stages were compared to the FEMA 100-year BFEs, and in virtually all cases, the FEMA flood elevations were higher than the modeled flood elevations. Thus, any structures built after the FEMA BFEs were established should have finished first-floor elevations that are higher than the modeled peak flood stages. In addition, maps showing land inundation were prepared at all locations where the evacuation routes crossing the PSMS were overtopped by the 100-year design storm.

2.6.2 Water Quality

In exploring existing water quality, a number of data sources were reviewed. The review occurred early in the project (2002).

These sources included the following:

- **South Carolina Department of Health and Environmental Control (SCDHEC) Ambient Water Quality Monitoring Program**. Data from a total of 22 stations were obtained and evaluated for parameters such as dissolved oxygen (DO), BOD, water temperature, salinity, phosphorus, nitrogen, and fecal coliform bacteria. Data were typically collected on a monthly basis. Many of the stations had very long periods of record (20 years or more). These stations provide data for only 12 of the 139 water quality segments that were modeled with the WASP receiving water model, and many were at the mouth of a major river (e.g., May River, Colleton River) where adverse water quality impacts are less likely than in the headwater areas of those rivers.
- **South Carolina Department of Health and Environmental Control (SCDHEC) Shellfish Monitoring Program**. Data from more than 80 stations were obtained and evaluated for parameters such as water temperature, salinity, and fecal coliform bacteria. Data were typically collected on a monthly basis. These stations provide data for 59 of the 139 water quality segments that were modeled with the WASP receiving water model.
- **South Carolina Estuarine and Coastal Assessment Program (SCECAP).** One grab sample is collected during the summer months at randomly-selected estuarine stations throughout coastal South Carolina. At the time of the analysis, data were available for the years 1999 and 2000. One or two data points were available for 40 of the 139 water quality segments that were modeled with the WASP receiving water model. Parameters that were sampled include nitrogen, phosphorus, and chlorophyll-a.
- **South Atlantic Bight Land Use – Coastal Ecosystem Study (LU-CES).** Annual progress reports for 2001-2002 and 2002-2003 were reviewed for pertinent information. These reports tended to be geared more toward research rather than straight data collection, and thus did not provide many data. Much of

the research was focused in the Colleton River watershed (Okatie River and tributaries) and Calibogue Sound watershed (Hilton Head Island). One interesting observation is that fecal coliform bacteria concentrations were sampled in a number of ponds, and the geomean of the data collected at all ponds was lower than would be expected based on the fecal coliform runoff concentrations and wet pond BMP removal efficiency (80 percent) used in this master plan study.

- **An Environmental Study of Broad Creek and the Okatee River.** Water, sediment and biological samples were collected in the Okatee (Okatie) River and Broad Creek (Hilton Head Island) to determine baseline conditions. Overall, many of the environmental and biological measures were consistent with other non-degraded estuarine sites in South Carolina, with greater evidence of stress in some of the tidal creeks and flats (SCDHEC, 2000). The authors found that contaminant levels and biological stress in Broad Creek was less than expected given the highly-developed nature of Hilton Head Island and hypothesized that nonpoint source controls may be the reason. Differences in measured concentrations in samples at the two sites was complicated by the fact that the Okatie River samples were taken during a dry period, whereas the samples were taken in Broad Creek the day after a 1.3-inch rain event.
- **Baseline Assessment of Environmental and Biological Conditions in the May River, Beaufort County, South Carolina.** This study was conducted and completed concurrently with the master plan study. SCDNR, USGS, and NOAA collaborated on the study. Water quality, sediment quality, and biological quality were measured in headwater creeks, large tidal creeks, and open tidal waters. The study concluded that most of the estuarine habitats are in good condition, and several areas showing some stress are likely affect by natural phenomena rather than anthropomorphic affects (SCDNR, 2004).
- **SCDHEC 303(d) List**. Every two years, SCDHEC prepares a priority list of water bodies that do not currently meet State water quality standards. The list (known as the 303(d) list) is developed by comparing the State standards to monitoring data collected by the State. In Beaufort County, most of the waters are classified as either Shellfish Harvesting (SFH) or ORW. A number of Beaufort County waters are listed on the year 2002 303(d) list, almost exclusively due to measured concentrations of DO and fecal coliform bacteria.

For this study, the water quality LOS will include the attainment of the fecal coliform bacteria standards in the Shellfish Harvesting and ORW, to the maximum extent practicable. Reasons for selecting bacteria as a focus for the LOS include the following:

- Non-attainment of bacteria water quality standards can result in temporary or permanent closing of shellfish harvesting areas, which would have social and economic impacts on the County
- The State has an extensive network of bacteria sampling stations, which provide substantial data for the calibration of the models that calculate bacteria loads to

the rivers and calculate the processes (e.g., bacteria die-off, tidal flushing) that affect river bacteria concentrations

- Literature findings support the premise that stormwater runoff from urban development tends to increase watershed bacteria loads (relative to undeveloped land) and water body bacteria concentrations

The relationship between stormwater management and waterbody DO levels is more uncertain. There are a number of factors that make the evaluation of low waterbody DO concentrations complex:

- In some cases, tidally influenced areas and wetlands may have naturally low DO levels, which would not be raised through stormwater management controls
- Waterbody DO concentrations are also affected by physical characteristics such as water temperature and reaeration (transfer of oxygen to the water from overlying air), which again would not be affected by stormwater management controls
- Water body DO concentrations are often lowest during dry weather, low-flow conditions
- Stormwater runoff generally has a relatively high concentration of DO and moderate concentrations of oxygen-demanding substances, except in situations where sanitary sewer overflows (SSOs), combined sewer overflows (CSOs) or illicit connections are discharging to the water body.
- State water quality monitoring data may not be sufficient to develop a model that can accurately represent the complex interactions between DO concentrations and the many processes that affect the water body concentrations.

Because the reasons for low DO concentrations are very complex and may not be directly related to stormwater pollution loads, achievement of DO standards will not be part of the LOS, though stormwater management measures to limit the discharge of stormwater loads of oxygen-demanding material will be evaluated.

Another potential water quality LOS is the control of algae growth in tidal waters. The State does not currently have nutrient-related water quality standards or criteria for estuarine systems at this time, though numeric criteria for TN, TP, and chlorophyll-a (a measure of algal biomass) have been developed for South Carolina lakes. Due to lack of river monitoring data for nutrients and particularly for algae, river concentrations of nutrients or algae will not be part of the LOS, though stormwater management measures to limit the discharge of stormwater loads of nutrients will be evaluated.

Selected tidal rivers were analyzed to evaluate water quality concentrations for fecal coliform bacteria. River concentrations were calculated and compared to applicable water quality standards and/or criteria to assess whether the standards and criteria are achieved under existing and future land use conditions, with various management strategies

The mean and distribution of salinity and bacteria data were evaluated by tidal river model segment. This means that if more than one monitoring station was located within a river segment, the data were pooled to establish the mean and distribution of concentrations within the river segment. For salinity, the average (arithmetic mean) and 90 percent confidence interval for the average were calculated. The “confidence interval” concept accounts for the fact that the “true” average concentration may be somewhat higher or lower than the average that is calculated using a number of random grab samples. For bacteria, the geometric mean and the 90 percent confidence interval of geometric mean was calculated for each river segment. The geometric mean was calculated for bacteria because the tidal river water quality standards are in part based on the geometric mean.

There are two fecal coliform bacteria standards that apply in the Beaufort County tidal rivers. These are:

- The geometric mean of bacteria concentrations shall not exceed 14/100 mL.
- No more than 10 percent of the bacteria concentrations shall exceed a concentration of 43/100 mL.

SCDHEC compares monitoring results with these standards by evaluating three years of monitoring data (i.e., 36 monthly random grab samples) to determine whether the standards have been met for that period.

Consequently, additional analysis was done for the 1990s fecal coliform bacteria. As noted above, the geometric mean for the 1990s (and 90 percent confidence interval for the geometric mean) was calculated. The 10-year record was also analyzed to determine the maximum geometric mean based on 36 consecutive samples (i.e., worst-case condition from the 1990s to determine compliance with the geometric mean standard). Analysis was also done to determine the 90th percentile bacteria concentration for the entire period, as well as the highest 90th percentile value for 36 consecutive samples (again, worst-case condition from the 1990s to determine compliance with the standard allowing only 10 percent of samples to exceed 43/100 mL).

Figure 2-10 shows 1990s geometric means plotted against the 36-sample maximum 90th percentile bacteria concentration value. Each point on the plot represents the long-term mean and the 36-sample maximum 90th percentile value for a single sampling station (a total of 80 stations). The horizontal line represents the bacteria water quality standard (that no more than 10 percent of the samples shall exceed 43/100 mL). The vertical line (at a geomean concentration of 7/100 mL) represents the geomean value at or below which the 43/100 mL standard is expected to be met at all times. It also represents the value above which the 43/100 mL standard is expected to be exceeded during some 36-sample periods.

As shown in the figure, there are a few stations at which the geomean is less than 7/100 mL but the 90th percentile value is greater than 43/100 mL. However, there are also several stations at which the geomean is greater than 7/100 mL and the 90th percentile value is less than 43/100 mL. The value of 7/100 mL was chosen such that the number of stations that do not follow the general rule for achieving or not achieving the 43/100 mL standard would be minimized, and that the chance of falsely predicting standard exceedance was equal to the chance of falsely predicting standard attainment. In this case, the graph show, 7 of 80 stations (less than 10 percent) that do not follow the general rule, almost evenly split between falsely predicting attainment (3 stations in the upper left quadrant of the graph) and falsely predicting exceedance (4 stations in lower right quadrant of graph).

Figure 2-11 shows 1990s geometric means plotted against the 1990s 90th percentile bacteria concentration value. Each point on the plot represents the long-term mean and long-term 90th percentile values for a single sampling station (a total of 80 stations). The horizontal line represents the bacteria water quality standard (that no more than 10 percent of the samples shall exceed 43/100 mL). The vertical line (at a geomean concentration of 8.7/100 mL) represents the geomean value at or below which the 43/100 mL standard is expected to be met in the long term. It also represents the value above which the 43/100 mL standard is expected to be exceeded in the long term.

As shown in the figure, there are a few stations at which the geomean is less than 8.7/100 mL but the 90th percentile value is greater than 43/100 mL. However, there are also several stations at which the geomean is greater than 8.7/100 mL and the 90th percentile value is less than 43/100 mL. The value of 8.7/100 mL was chosen such that the number of stations that do not follow the general rule for achieving or not achieving the 43/100 mL standard would be minimized, and that the chance of falsely predicting standard exceedance was equal to the chance of falsely predicting standard attainment. In this case, the graph show, 5 of 80 stations (less than 10 percent) that do not follow the general rule, almost evenly split between falsely predicting attainment (3 stations in the upper left quadrant of the graph) and falsely predicting exceedance (2 stations in lower right quadrant of graph).

Figure 2-12 shows 1990s geometric means plotted against the 36-sample maximum geomean concentration value. Each point on the plot represents the long-term mean and 36-sample maximum geomean values for a single sampling station (a total of 80 stations). The horizontal line represents the geomean bacteria water quality standard (14/100 mL). The vertical line (at a geomean concentration of 10/100 mL) represents the geomean value at or below which the 36-sample geomean standard is expected to be met at all times. It also represents the value above which the 36-sample geomean standard is expected to be exceeded during some 36-sample periods.

As shown in the figure, there are a few stations at which the long-term geomean is less than 10/100 mL but the 36-sample maximum geomean is greater than 14/100 mL.

However, there are also several stations at which the long-term geomean is greater than 10/100 mL and the 36-sample maximum geomean value is less than 14/100 mL. The value of 10/100 mL was chosen such that the number of stations that do not follow the general rule for achieving or not achieving the long-term geomean standard of 14/100 mL would be minimized, and that the chance of falsely predicting standard exceedance was equal to the chance of falsely predicting standard attainment. In this case, the graph shows 3 of 80 stations (less than 5 percent) that do not follow the general rule, almost evenly split between falsely predicting attainment (1 station in the upper left quadrant of the graph) and falsely predicting exceedance (2 stations in lower right quadrant of graph).

Based on these results, the following LOS for bacteria water quality, based on long-term geomean fecal coliform bacteria concentrations, offer various levels of bacteria standard achievement.

- Level A river segments (long-term geomean less than or equal to 7/100 mL) are expected to meet the geomean standard (14/100 mL) and the 90th percentile standard (43/100 mL) for any 36-sample period.
- Level B river segments (long-term geomean greater than 7/100 mL and less than or equal to 8.7/100 mL) are expected to meet the geomean standard (14/100 mL) for any 36-sample period, and are expected to meet the 90th percentile standard in the long term, but the 90th percentile standard is expected to be exceeded during some 36-sample periods.
- Level C river segments (long-term geomean greater than 8.7/100 mL and less than or equal to 10/100 mL) are expected to meet the geomean standard (14/100 mL) for any 36-sample period but are not expected to meet the 90th percentile standard in the long term.
- Level D river segments (long-term geomean greater than 10/100 mL) are expected to exceed the geomean standard (14/100 mL) for some 36-sample periods and are expected to exceed the 90th percentile standard in the long-term and during some 36-sample periods.

These levels are listed in order from most desirable (Level A) to least desirable (Level D).

These levels will be used in conjunction with an "anti-degradation" approach to evaluate the water quality impacts in the tidal rivers. The statistics developed using existing bacteria monitoring data will be used to classify each of the tidal river segments under one of the four levels. Under the "anti-degradation" approach, the goal of the stormwater master plan will be to achieve the same level of water quality as is currently achieved under existing conditions. For example, if a river segment has an existing long-term geometric mean concentration of 8/100 mL, it would be classified as a Level B segment. The water quality models would then be used to project the long-term bacteria geometric mean in that segment for future conditions (e.g., with anticipated future development and BMPs in accordance with the County BMP Manual) to see if the river segment maintains its Level B status (less than 8.7/100 mL, as discussed above). If not, additional management measures will be evaluated to see what measures would be needed to maintain that level.

2.7 Alternative Management Measures for Water Quantity and Quality

The modeling studies considered a number of alternative management measures for control of water quantity and water quality. Those measures are discussed in the following sections.

2.7.1 Water Quantity

For water quantity, problems occur when the PSMS does not have sufficient capacity to carry the peak flows associated with the defined LOS. There are several methods that can be applied to solve these capacity problems:

- Increase the conveyance capacity of the PSMS
- Reduce peak flows with detention storage
- Combination of the above

The appropriate measure will depend upon considerations such as maintaining the LOS, system-wide cost of implementation, and site constraints.

System capacity can be increased in several ways. The most common would be replacing undersized culverts or adding additional culverts to pass more flow at a road crossing that is overtopped under the current culvert configuration. However, such culvert enhancements must be evaluated to make sure that passing the peak flow more efficiently at the current problem area does not result in new problems downstream of the current problem area. Another example of increasing capacity is raising the roadway at the stream crossing. In some cases, road overtopping may occur because the road is at a low elevation relative to the downstream tidal boundary or because there is little freeboard between the top of the culvert and the roadway.

Peak flows can be reduced by providing detention storage upstream of the problem area. Temporarily storing water upstream of the problem area serves to reduce the peak flows that the PSMS needs to pass downstream of the detention. The suitability of detention storage is primarily based on physical characteristics such as the availability of undeveloped land that can be used as the location of the detention storage, and the natural topography at the potential detention site.

Of course, it may be appropriate to both increase existing PSMS capacity and reduce peak flows with detention at a particular problem area. There may be situations in which the area available for detention is not quite sufficient to fully solve the flooding problem but would substantially reduce the additional required culvert capacity.

2.7.2 Water Quality

Various BMPs can be considered for use in the County's Stormwater Master Plan for retrofit treatment of existing development and treatment of future development. The BMPs are grouped as structural (constructed facilities) and non-structural (regulation or ordinances).

The following is a list of structural BMPs that is included in the County's BMP manual:

- Wet detention ponds
- Extended dry detention ponds
- Modified extended dry detention basin
- Infiltration facility
- Grass swale with check dams
- Biofiltration swale
- Bioretention facility
- Innovative technology (commercially constructed units, e.g., Stormceptor or Stormtreat)

These structural BMPs are designed to capture and treat stormwater runoff from urban development. In this study, it was assumed that all future development would be served by BMPs in accordance with the County BMP Manual. Wet detention (the typical BMP applied in the County) was assumed as the BMP for future development.

In contrast to structural BMPs, nonstructural BMPs generally reduce stormwater pollution loads by reducing the amount of pollution generated by stormwater runoff, rather than treating the runoff. Examples of nonstructural BMPs include the following:

- Land use planning and management can be used to integrate County goals into the development and redevelopment process. Management measures may include modification or restrictions of certain land use activities. Greater restrictions may be warranted where development can affect impaired, threatened, or significant water bodies. Because increased pollutant loadings and flooding correspond to increase in impervious cover, land use planning can become an effective control measure.
- Public information programs would provide the County with a strategy for informing its employees, the public, and businesses about the importance of protecting stormwater from improperly used, stored, and disposed pollutants. Residents should be aware that a variety of hazardous products are used in the

home and that their improper use and disposal can pollute stormwater. Likewise, improper disposal of oils, antifreeze, paints, and solvents can end up in streams and lakes, poisoning fish and wildlife.

- Fertilizer application controls could be implemented through a public information program by making the public and professional fertilizer users aware of the problems associated with overuse of fertilizers. Overuse of fertilizers will cause excessive runoff of nutrients to surface waters thereby wasting money for the homeowner/professional user and potentially degrading the receiving water body.
- Pesticide and herbicide use controls could be implemented in a manner similar to fertilizer application controls.
- Public information program on proper maintenance of septic tank systems
- Solid waste management can include public information regarding the adverse impacts of littering and poor solid waste management (e.g., obstructing open channels, culverts, and storm sewers). This can also include pet droppings and illegal dumping into storm drains, wooded areas, and ditches. Pet droppings can be a source of coliform bacteria and pathogens.
- Street sweeping can be an effective method of improving street aesthetics in developed areas and, depending on the type of equipment used, can be an effective pretreatment method of water quality control.
- Impervious area minimization would limit the amount of directly connected impervious area (DCIA) on a site and promote the use of green buffer zones around paved areas for infiltration. For example, roof runoff from structures can be directed to green buffer zones or shallow swales around houses. In addition, parking lots and driveways can be graded to landscaped/grassed areas or swales, reducing direct runoff to the storm drainage system.
- Erosion and sediment control on construction sites provides for the protection of receiving waters from sediment loads. Proper control during construction can be accomplished with gravel filter weirs, sediment fences, and temporary berms or swales. Currently, the County has an ordinance requiring erosion and sediment control on construction sites.
- Operation and maintenance can be one of the most effective non-structural BMPs. For publicly owned treatment facilities, routine maintenance and inspection should be performed. For privately owned facilities, maintenance is not typically performed by a municipality. There are several options that can be pursued by a municipality to help ensure that proper maintenance is being conducted. These options include a certification program initiated by a municipality that requires all approved subdivision ponds (private) to be recertified by the owner on a predetermined time interval. The recertification may be done by a state certified/trained inspector or engineer. Enforcement of maintenance of privately owned facilities is one of the most difficult problems for privately owned facilities. Potential enforcement measures may include

County intervention (after sufficient notification) where critical maintenance is done by the County and the cost of the maintenance is billed to the owner or by other means as deemed necessary by the municipality. Another option would be to consider the assessment of fines.

2.7.3 Regional vs. Onsite Structural Controls

Where practicable, regional facilities were considered for water quantity and quality control within the County. The following discussion is provided for detention pond applications, which tend to be cost-effective when sited regionally.

In the case of future urban development or retrofit of existing development, the onsite approach (also known as piecemeal approach to stormwater control) involves the delegation of responsibilities for BMP deployment to local land developers or the use by the County of BMPs serving small areas due to site constraints. Each developer is responsible for constructing a structural BMP at the development site to control nonpoint pollution loadings from the site. Detention pond BMPs provided onsite typically have contributing areas of 20 to 50 acres. The local government is responsible for reviewing each structural BMP design to ensure conformance with specified design criteria, for inspecting the constructed facility to ensure conformance with the design, and for ensuring that a maintenance plan is implemented for the facility.

The regional approach to stormwater control involves strategically siting regional structural BMPs to control nonpoint pollution loadings from multiple development projects. For ponds serving new development, the front-end costs for constructing the structural BMP are assumed by the developer and/or the local government that administers the regional BMP plan. BMP capital costs can then be recovered from upstream developers on a "pro rata" basis as development occurs. Individual regional BMPs are phased in as development occurs rather than constructing all regional facilities at one time. Maintenance responsibility for regional structural BMPs can be assumed by the developer (or designee with certified maintenance bonds) or by the local government. For retrofit of existing development, regional BMPs may also be used to cost-effectively treat areas that are near the areas that are retrofit for water quantity controls but that cannot be cost-effectively treated. The regional approach addresses concurrence for the entire watershed while the onsite approach does not address this issue.

A regional BMP system offers benefits that are equal to or greater than onsite BMP benefits at a lower cost. Most of the advantages of the regional approach over the onsite approach can be attributed to the need for fewer structural facilities that are strategically located within the watershed. The specific advantages of the regional approach are summarized in the following listing.

- Reduction in capital costs for structural BMPs: The use of a single stormwater detention facility to control runoff from approximately 5 to 15 development sites within approximately a 100- to 600-acre area permits the local government to take advantage of economies-of scale in designing and constructing the regional facility. In other words, the total capital cost (e.g., construction, land acquisition, engineering design) of several small onsite detention BMPs is greater than the cost of a single regional detention pond BMP which provides the same total storage volume in a strategic place.

- Reduction in maintenance costs: Since there are fewer stormwater detention facilities to maintain, the annual cost of maintenance programs is significantly lower. Moreover, since regional detention facilities can be designed to facilitate maintenance activities, annual maintenance costs are further reduced in comparison with onsite facilities. Examples of design features that are typically only feasible at regional BMP facilities to reduce maintenance costs include: access roads that facilitate the movement of equipment and work crews onto the site (by comparison, detention facilities implemented under the onsite approach are often located in residential backyards); additional sediment storage capacity (e.g., sediment forebay) to permit an increase in the time interval between facility clean out operations; and onsite disposal areas for sediment and debris removed during clean out.
- Greater reliability: A regional BMP system will be more reliable than an onsite BMP system because it will more likely be maintained. With fewer facilities to maintain and design features that reduce maintenance costs, the regional BMP approach is much more likely to result in an effective long-term maintenance program. Due to the greater number of facilities, the onsite BMP approach tends to result in a large number of facilities that do not get adequately maintained and, therefore, soon cease to function as designed. Many municipalities who start off with the onsite approach eventually switch to the regional approach to address the lack of maintenance of the onsite systems and to increase the overall effectiveness of the stormwater management program. Regional facilities however, cannot be so large that incremental water quality protection is lost. For instance, if a regional detention facility is at the bottom of a 10-square-mile basin, no water quality protection would be provided to the upstream rivers and streams as urbanization occurs. Another problem with an excessively large regional facility is the impact of the facility on existing wetlands. In rural areas, an excessively large pond would inundate large wetland areas, which would make permitting of the structures extremely difficult. Experience shows that a regional pond should be limited to approximately a 100- to 600-acre tributary area.
- Opportunities to manage existing nonpoint pollution loadings: Nonpoint pollution loadings from existing developed areas can be affordably controlled at the same regional facilities that are sited to control future urban development. This is because the provision of additional storage capacity to control runoff from existing development in the facility's contributing area is reasonable in cost due to economies of scale. Alternatively, existing development can be retrofit in lieu of treating other existing development that is being retrofit for water quality

control. By comparison, the costs of retrofitting existing development sites with onsite detention BMPs to control existing nonpoint pollution loadings may be prohibitively expensive or extremely difficult due to site constraints/conditions.

- Fair to land developers: Land developers recognize that economies of scale available at a single regional BMP facility should produce lower capital costs in comparison with several onsite detention facilities. They also tend to prefer the regional BMP approach because it eliminates the need to set aside acreage for an onsite facility other than pretreatment and conveyance to the regional pond. This could permit an increase in the number of dwelling units within the development site while still providing sufficient stormwater management. The additional cost of a pond sized for future development can be passed on to the developer. Developers can "buy" into the regional system and eliminate on-site BMP requirements, thus minimizing cost to the public. Regional facilities also offer the ability to maximize mining of fill material.
- Multipurpose uses: Regional facilities can often be landscaped to offer recreational and aesthetic benefits. Jogging and walking trails, picnic areas, ballfields, and canoeing or boating are some of the typical uses. For example, portions of the facility used for flood control can be kept dry, except during floods, and can be used for exercise areas, soccer fields, or football fields. Wildlife benefits can also be provided in the form of islands or preservation zones, which allow a view of nature within the park schemes. Gradual swales can also be worked into the park concept to provide pretreatment around paved areas, such as parking lots or access roads.

TABLE 2-1
LAND USE CATEGORIES AND ASSOCIATED CHARACTERISTICS
FOR ICPR DESIGN STORM MODELING

Urban Systems

		CN for Hydrologic Soil Group			
Land Use	**% Impervious**	**A**	**B**	**C**	**D**
Low-Density Residential	10%	45	65	78	82
Medium-Density Residential	25%	54	70	80	85
High-Density Residential	50%	69	80	86	89
Institutional	38%	61	75	83	87
Industrial / Transportation	72%	81	88	91	93
Commercial / Business	85%	89	92	94	95
Golf Courses	1%	39	61	74	80
Impervious	100%	98	98	98	98
Open Space*	1%	39	61	74	80

*e.g., parks, cemeteries

Agricultural Systems

		CN for Hydrologic Soil Group			
Land Use	**% Impervious**	**A**	**B**	**C**	**D**
Row Crop	1%	64	75	82	85
Silvaculture	1%	32	58	72	79

Natural Systems

		CN for Hydrologic Soil Group			
Land Use	**% Impervious**	**A**	**B**	**C**	**D**
Open Water	100%	100	100	100	100
Forested Wetland	100%	98	98	98	98
Non-Forested Wetland	100%	98	98	98	98
Sandy Area	100%	98	98	98	98
Forestland	1%	25	55	70	77
Grassland	1%	30	58	71	78

Source: USDA, SCS, 1986.

TABLE 2-2
24-HOUR RAINFALL DEPTHS FOR DESIGN STORMS

Data Source	24-hour Design Rainfall (inches) for Various Return Periods			
	2-year	10-year	25-year	100-year
TP-40 (USDA, SCS, 1961)	4.5	6.9	7.9	10.0
Beaufort 7 SW gage (daily rainfall)	4.7	7.1	8.4	10.5
Savannah Airport (hourly rainfall)	4.7	6.8	8.1	10.1
Charleston Airport (hourly rainfall)	5.0	7.1	8.3	10.3

TABLE 2-3
TIDAL INFORMATION FOR BEAUFORT COUNTY

Gage Location	Observation Dates Start	Observation Dates End	Annual Max Elev (ft NAVD)	Mean High Water (ft NAVD)	Mean Low Water (ft NAVD)	Average Tidal Range (ft)
Calibogue Sound/Port Royal Sound						
Huspa Creek	Jul-79	Jun-80		3.6	-4.5	8.1
Whale Branch	Mar-78	Feb-79	5.5	3.3	-4.4	7.7
Beaufort	Jan-78	Dec-84	5.6	3.2	-4.2	7.4
Battery Creek	Mar-78	Feb-79	5.4	3.4	-4.3	7.6
Okatee River	Mar-78	Feb-79	5.9	3.5	-4.6	8.1
Distant Island Creek	Mar-80	Feb-81	5.5	3.3	-3.6	6.9
Station Creek	Mar-78	Feb-79	5.4	3.0	-3.8	6.8
Skull Creek South	May-78	Apr-79		3.1	-4.2	7.3
Broad Creek	Jul-78	Feb-79		3.3	-4.2	7.5
Average			5.6	3.3	-4.2	7.5
St. Helena Sound						
Wimbee Creek	Dec-77	Nov-78		2.8	-3.6	6.4
Eddings Point Creek	Mar-78	Feb-79	4.8	2.7	-3.7	6.4
Harbor River	Feb-75	Jan-76	4.6	2.6	-3.5	6.1
Johnson's Creek	Mar-75	Feb-76	4.6	2.5	-3.4	5.9
Fripp Inlet	Mar-78	Feb-79	4.9	2.5	-3.6	6.1
Jenkins Creek	Mar-81	May-81		2.9	-3.9	6.8
Average			4.7	2.7	-3.6	6.3
New River						
New River at 170	Aug-79	Feb-80		2.4	-1.0	3.3

NOTES:

1. Annual maximum elevation is based on annual series developed from monthly extremes obtained from CO-OPS website *co_ops.nos.noaa.gov/data_res.html*
2. Mean high water and mean low water values were developed from data on benchnmark sheets obtained at CO-OPS website *co_ops.nos.noaa.gov/benchmarks.*
3. Average tidal range is difference between mean high water and mean low water.

TABLE 2-4
MONTHLY AND ANNUAL RAINFALL TOTALS
BEAUFORT 7 SW RAIN GAGE

Month	Average Rainfall (inches)
January	3.4
February	3.1
March	3.9
April	2.8
May	3.5
June	5.4
July	6.3
August	6.9
September	5.3
October	2.7
November	2.1
December	2.9
TOTAL	48.4

TABLE 2-5
LAND USE CATEGORIES AND ASSOCIATED RUNOFF COEFFICIENTS FOR ANNUAL LOAD CALCULATIONS

Urban Systems

Land Use	% Impervious	Impervious Runoff Coefficient	Pervious Runoff Coefficient	Average Annual Runoff (inches/year)
Low-Density Residential	10%	0.90	0.10	8.7
Medium-Density Residential	25%	0.90	0.10	14.5
High-Density Residential	50%	0.90	0.10	24.2
Institutional	38%	0.90	0.10	19.6
Industrial / Transportation	72%	0.90	0.10	32.7
Commercial / Business	85%	0.90	0.10	37.8
Golf Courses	1%	0.90	0.10	5.2
Impervious	100%	0.90	0.10	43.6
Open Space*	1%	0.90	0.10	5.2

*e.g., parks, cemeteries

Agricultural Systems

Land Use	% Impervious	Impervious Runoff Coefficient	Pervious Runoff Coefficient	Average Annual Runoff (inches/year)
Row Crop	1%	0.90	0.10	5.2
Silvaculture	1%	0.90	0.10	5.2

Natural Systems

Land Use	% Impervious	Impervious Runoff Coefficient	Pervious Runoff Coefficient	Average Annual Runoff (inches/year)
Open Water	100%	1.00	0.10	48.4
Forested Wetland	100%	0.25	0.10	12.1
Non-Forested Wetland	100%	1.00	0.10	48.4
Sandy Area	100%	1.00	0.10	48.4
Forestland	1%	0.90	0.10	5.2
Grassland	1%	0.90	0.10	5.2

TABLE 2-6
RUNOFF EVENT MEAN CONCENTRATIONS (EMCs) FOR ANNUAL LOAD CALCULATIONS

Urban Systems

Land Use	BOD (mg/l)	TSS (mg/l)	Total-P (mg/l)	Total-N (mg/l)	Lead (mg/l)	Zinc (mg/l)	Fecal Coliform (#/100 ml)
Low-Density Residential	11	117	0.40	1.9	0.020	0.078	32,200
Medium-Density Residential	11	117	0.40	1.9	0.020	0.078	32,200
High-Density Residential	10	116	0.29	1.9	0.016	0.119	21,750
Institutional	10	117	0.23	1.9	0.016	0.119	32,200
Industrial / Transportation	10	116	0.23	1.9	0.016	0.119	11,100
Commercial / Business	10	116	0.23	1.9	0.016	0.119	11,300
Golf Courses	2	26	1.30	2.6	0.009	0.041	6,400
Impervious	10	116	0.23	1.9	0.016	0.119	11,300
Open Space*	2	26	0.10	1.3	0.001	0.006	6,400

*e.g., parks, cemeteries

Agricultural Systems

Land Use	BOD (mg/l)	TSS (mg/l)	Total-P (mg/l)	Total-N (mg/l)	Lead (mg/l)	Zinc (mg/l)	Fecal Coliform (#/100 ml)
Row Crop	4	55	1.30	2.6	0.009	0.041	6,400
Silvaculture	4	55	0.14	2.1	0.009	0.041	6,400

Natural Systems

Land Use	BOD (mg/l)	TSS (mg/l)	Total-P (mg/l)	Total-N (mg/l)	Lead (mg/l)	Zinc (mg/l)	Fecal Coliform (#/100 ml)
Open Water	3	6	0.16	1.3	0.006	0.146	6,400
Forested Wetland	2	26	0.10	1.3	0.001	0.006	6,400
Non-Forested Wetland	3	6	0.16	1.3	0.006	0.146	6,400
Sandy Area	3	6	0.16	1.3	0.006	0.146	6,400
Forestland	2	26	0.10	1.3	0.001	0.006	6,400
Grassland	2	26	0.10	1.3	0.001	0.006	6,400

Source: CDM, 2003

TABLE 2-7
BASEFLOW EVENT MEAN CONCENTRATIONS (EMCs) FOR ANNUAL LOAD CALCULATIONS

BOD (mg/l)	**TSS (mg/l)**	**Total-P (mg/l)**	**Total-N (mg/l)**	**Lead (mg/l)**	**Zinc (mg/l)**	**Fecal Coliform (#/100 ml)**
3	18	0.16	1.0	0.001	0.001	200

Source: T&H sampling - Eagle's Pointe and Buckwalter

TABLE 2-8
POINT SOURCE FLOWS AND CONCENTRATIONS FOR ANNUAL LOAD CALCULATIONS

Direct Discharges

Discharge/ Receiving Water	**Flow (mgd)**	**BOD (mg/l)**	**TSS (mg/l)**	**Total-P (mg/l)**	**Total-N (mg/l)**	**Lead (ug/l)**	**Zinc (ug/l)**	**Fecal Coliform (#/100 ml)**
Shell Point (Beaufort River)	0.30	6.0	4.7	***4.0***	***20.0***	***25***	***100***	7
Southside (Beaufort River)	1.49	5.8	4.0	***4.0***	***20.0***	***25***	***100***	5
Parris Island (Beaufort River)	1.14	10.2	20.9	***4.0***	***20.0***	***25***	***100***	4
USMC Air Station (Albergotti Creek)	0.18	16.5	18.5	***4.0***	***20.0***	***25***	***100***	13
USMC Air Station (Broad River)	0.45	8.5	7.0	***4.0***	***20.0***	***25***	***100***	3
Cherry Point WWTP (New River)	2.50	*10.0*	*10.0*	***4.0***	***20.0***	***25***	***100***	*3*

Sprayfields

Discharge/ Receiving Water	**Flow (mgd)**	**BOD (mg/l)**	**TSS (mg/l)**	**Total-P (mg/l)**	**Total-N (mg/l)**	**Lead (ug/l)**	**Zinc (ug/l)**	**Fecal Coliform (#/100 ml)**
Various Locations	25% of applied water	1.5	2.0	0.8	4.0	5	20	1

1. For direct discharges, flows and concentrations are from Discharge Monitoring Reports for parameters that are monitored.
2. For direct discharges, values in italics are not monitored, and were set based on typical wastewater characteristics (CDM, 1993)
3. Sprayfield concentrations are based on 80-90% reduction in concentration in the soil.

TABLE 2-9

ESTIMATED WASTEWATER FLOWS

BEAUFORT COUNTY WATERSHEDS

WATERSHED	INDIRECT DISCHARGES (MGD)		DIRECT DISCHARGES (MGD)	
	EXISTING	FUTURE	EXISTING	FUTURE
Calibogue Sound	4.0	4.5	0.0	0.0
May River	0.3	0.8	0.0	0.0
Chechessee River	0.1	0.1	0.0	0.0
Colleton River	0.6	0.8	0.0	0.0
New River	0	0	2.5	7.5
Beaufort River	0	0	3.1	3.1
Coosaw River	0	0	0.0	0.0
Whale Branch West	0	0	0.0	0.0
Morgan River	0.2	0.6	0.0	0.0
Broad River	0.9	0.9	0.5	0.5
Combahee River	0	0	0.0	0.0
Coastal	0	0	0.0	0.0
TOTAL	6.1	7.7	6.1	11.1

NOTES:

1. Existing direct discharge values based on Discharge Monitoring Reports and EPA permit Compliance System Reports
2. Existing indirect discharge data based on data provided by BJW&SA, and PSDs for Town of Hilton Head Island
3. Future indirect discharge data based on comparison of existing and future land uses in sewer service areas.

TABLE 2-10
FAILING SEPTIC TANK LOADS

Land Use	Septic Flow (gal/ac/day)	Failing Septic Total-P (lb/ac/yr)	Runoff Total-P (lb/ac/yr)	Total-P Load Ratio	Failing Septic Total-N (lb/ac/yr)	Runoff Total-N (lb/ac/yr)	Total-N Load Ratio	Failing Septic Fecal Col. (#/ac/yr)	Runoff Fecal Col. (#/ac/yr)	Fecal Col. Load Ratio
Low Density Residential	188	1.1	0.8	2.4	17.1	3.7	5.6	1.9E+12	2.9E+11	7.7
Medium Density Residential	750	4.6	1.3	4.5	68.5	6.2	12.1	7.7E+12	4.8E+11	17.0
High Density Residential	1875	11.4	1.6	8.2	171.2	10.4	17.5	1.9E+13	5.4E+11	36.5
Institutional	1875	11.4	1.0	12.0	171.2	8.4	21.4	1.9E+13	6.4E+11	30.8
Industrial/Transportation	1875	11.4	1.7	7.6	171.2	14.1	13.2	1.9E+13	3.7E+11	52.7
Commercial/Business	1875	11.4	2.0	6.7	171.2	16.2	11.6	1.9E+13	4.4E+11	45.1

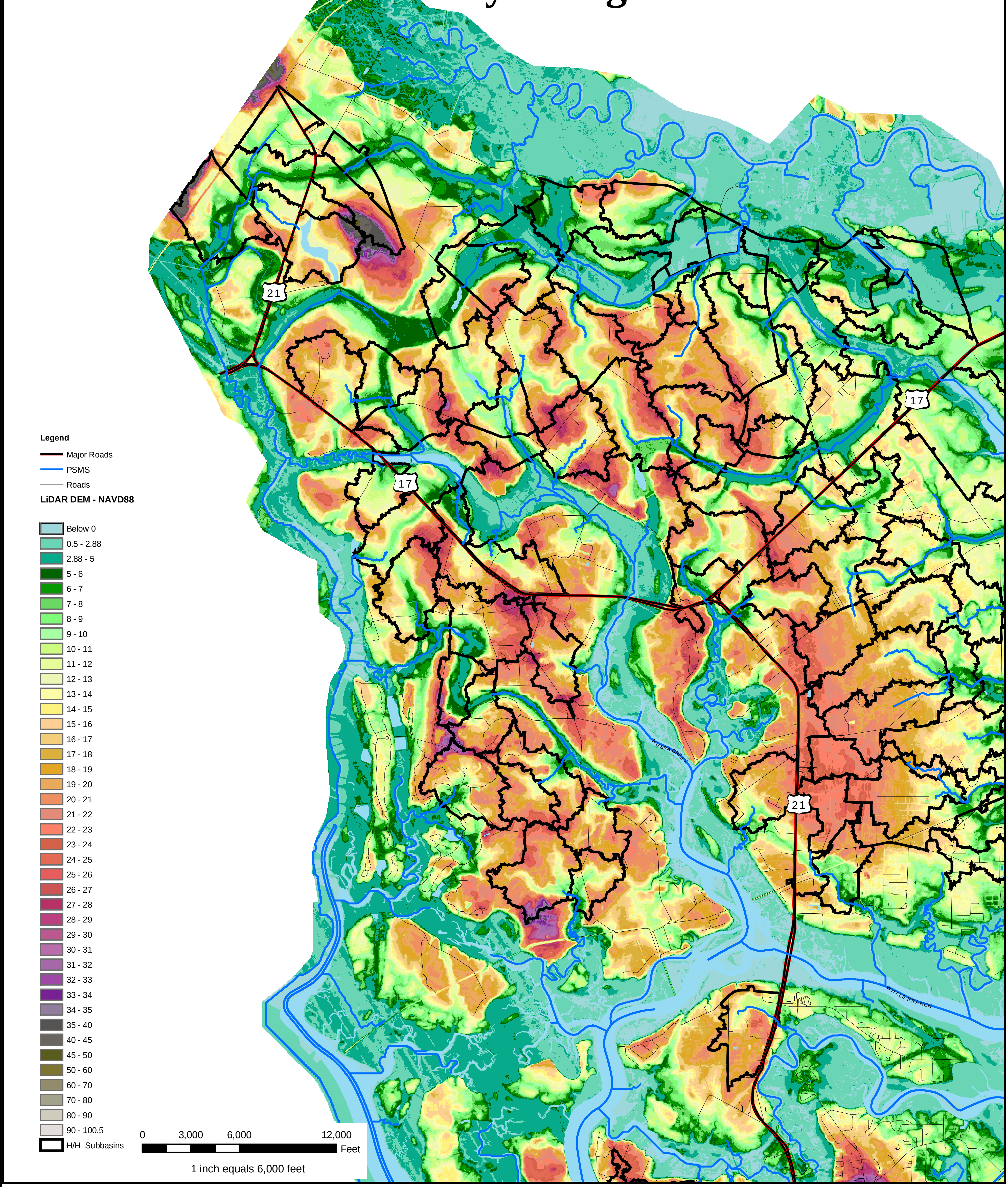
1. Flows in gallons per day for residential areas are based on the following:
   a. Unit flow rate of 75 gallons per capita per day
   b. 2.5 people per dwelling unit
   c. Dwelling unit density ranging from 1 per acre (low density) to 10 per acre (high density).
2. Flow rate for commercial, industrial and institutional is presumed to be similar to high density residential.
3. Assumed concentrations for failing septic tank discharges are:
   a. 2 mg/l for total P (CDM, 1993)
   b. 30 mg/l for total N (CDM, 1993)
   c. 750,000 per 100 ml for fecal coliform bacteria (USEPA, 2001)
4. Runoff loads are calculated based on runoff (Table 2-5) and EMC (Table 2-6) data.

TABLE 2-11
BMPs AND ASSOCIATED REMOVAL EFFICIENCIES FOR ANNUAL LOAD CALCULATIONS

BMP Type	BOD	TSS	Total-P	Total-N	Lead	Zinc	Fecal Coliform
Wet Detention Basin	40%	80%	60%	40%	80%	70%	80%
Extended Dry Detention Basin	30%	80%	30%	15%	80%	50%	35%
Modified Extended Dry Detention Basin	35%	80%	45%	25%	80%	60%	50%
Infiltration	75%	90%	55%	45%	75%	75%	90%
Grass Swale with Check Dams	20%	70%	25%	20%	60%	40%	30%
Biofiltration Swale	10%	30%	15%	10%	30%	25%	10%
Bioretention	50%	80%	55%	30%	80%	60%	70%
Innovative Technology							
- Swirl Concentrator	30%	80%	30%	15%	80%	50%	10%
- Settling/Filtration	30%	80%	30%	15%	80%	50%	35%
- Settling/Wetland	40%	80%	60%	40%	80%	70%	70%

Source: CDM, 2003.

Northern Beaufort County
Example of LiDAR DEM
Primary Stormwater Management System
and
Hydrologic Model Subbasins



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SAVANNAH, GEORGIA 31405
(912) 234-5300

CDM Camp Dresser & McKee Inc.

Figure 2-1

Job Number: 15178.00	Scale: 1" = 6,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\County-LiDARexample_figure2-1.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Roads	Beaufort County	2002

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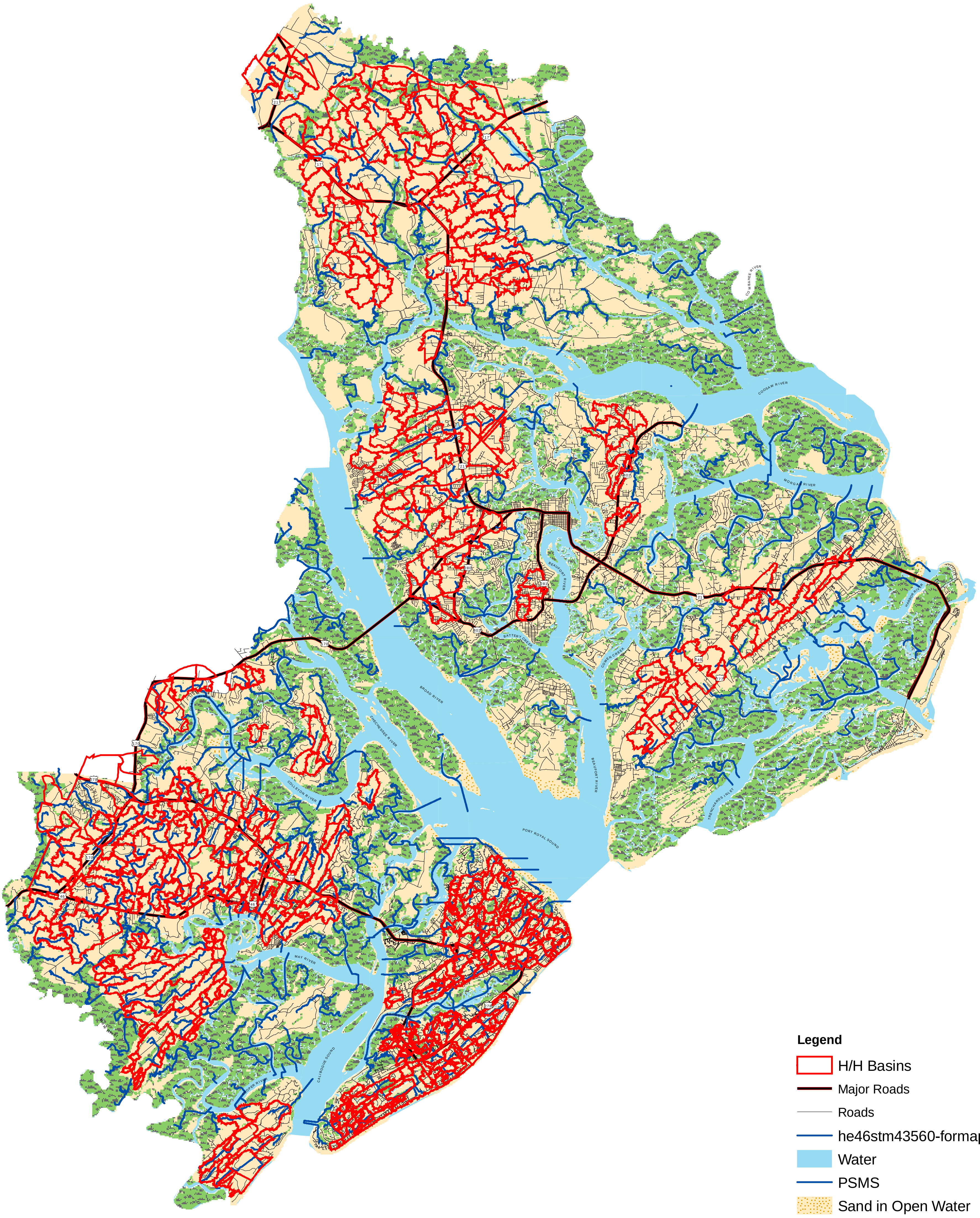
DATA	SOURCE	DATE
LiDAR DEM	T&H / Beaufort County	2003
PSMS	CDM / T&H	2004
H/H Subbasins	CDM / T&H	2004

Beaufort County
Hydrologic Units
and
Primary Stormwater Management System



Legend

- H/H Basins
- Major Roads
- Roads
- he46stm43560-formap
- Water
- PSMS
- Sand in Open Water
- Upland
- Wetland

0 10,000 20,000 Feet

1 inch equals 20,000 feet

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CDM Camp Dresser & McKee Inc.

Figure 2-2

Disclaimer

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Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

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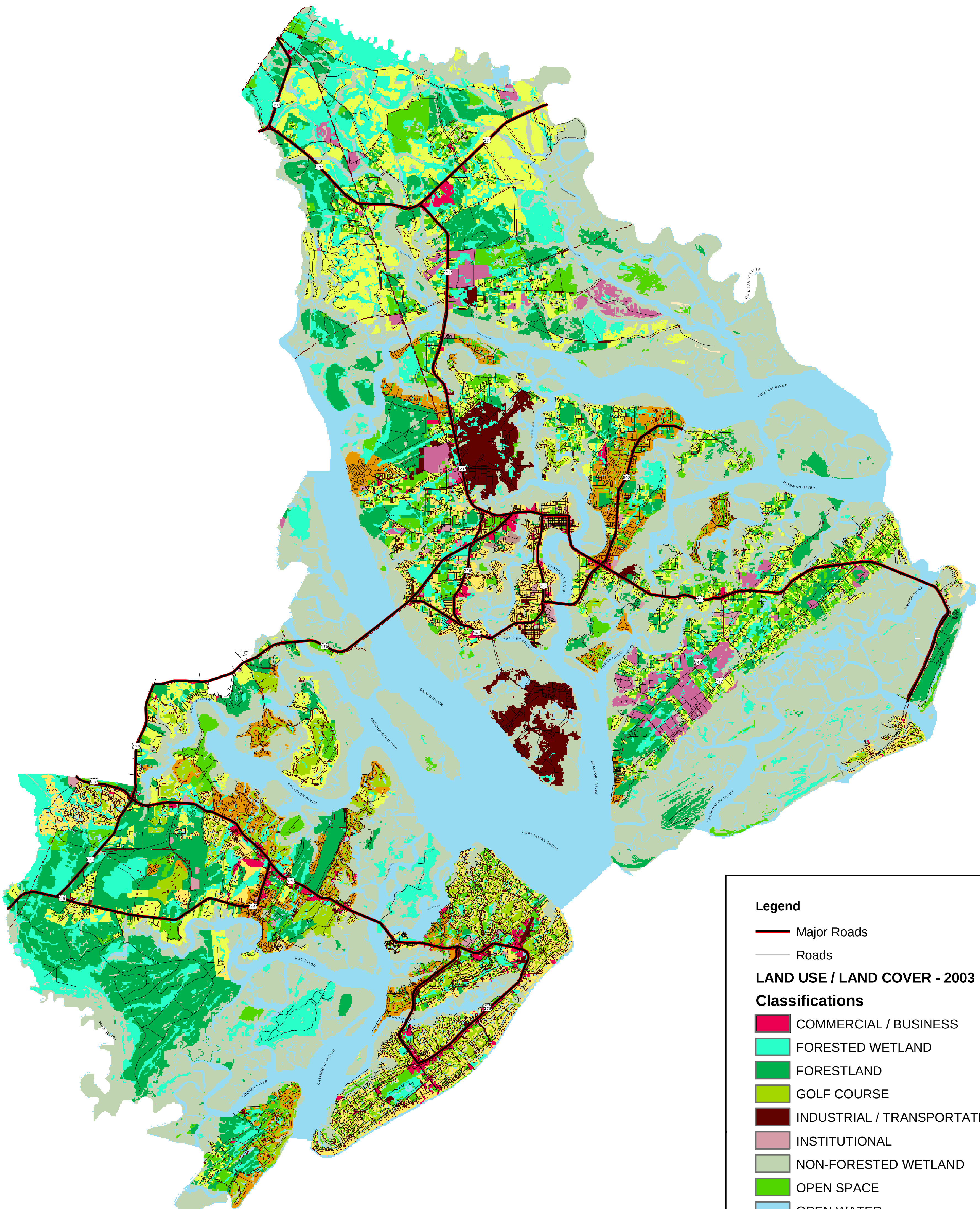
DATA	SOURCE	DATE
H/H Basins	CDM / T&H	2004
PSMS	CDM / T&H	2004

Beaufort County
Existing Land Use / Land Cover



Legend

- Major Roads
- Roads

LAND USE / LAND COVER - 2003

Classifications

- COMMERCIAL / BUSINESS
- FORESTED WETLAND
- FORESTLAND
- GOLF COURSE
- INDUSTRIAL / TRANSPORTATION
- INSTITUTIONAL
- NON-FORESTED WETLAND
- OPEN SPACE
- OPEN WATER
- RESIDENTIAL - HIGH DENSITY
- RESIDENTIAL - LOW DENSITY
- RESIDENTIAL - MEDIUM DENSITY
- ROW CROP
- SANDY AREA
- SILVACULTURE

0 10,000 20,000
Feet

1 inch equals 20,000 feet

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Figure 2-3

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Roads	Beaufort County	2002

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DATA	SOURCE	DATE
Land Use / Land Cover	Beaufort County / T&H / CDM	2003

Beaufort County
NRCS Soils
Hydrologic Group

0 10,000 20,000
Feet

1 inch equals 20,000 feet

Legend

Major Roads

Roads

NRCS Soils - CN Hydrologic Group

Hydrologic Soil Group

A

B

C

D

Water

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Figure 2-4

CDM Camp Dresser & McKee Inc.

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Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Soils	SC DNR	

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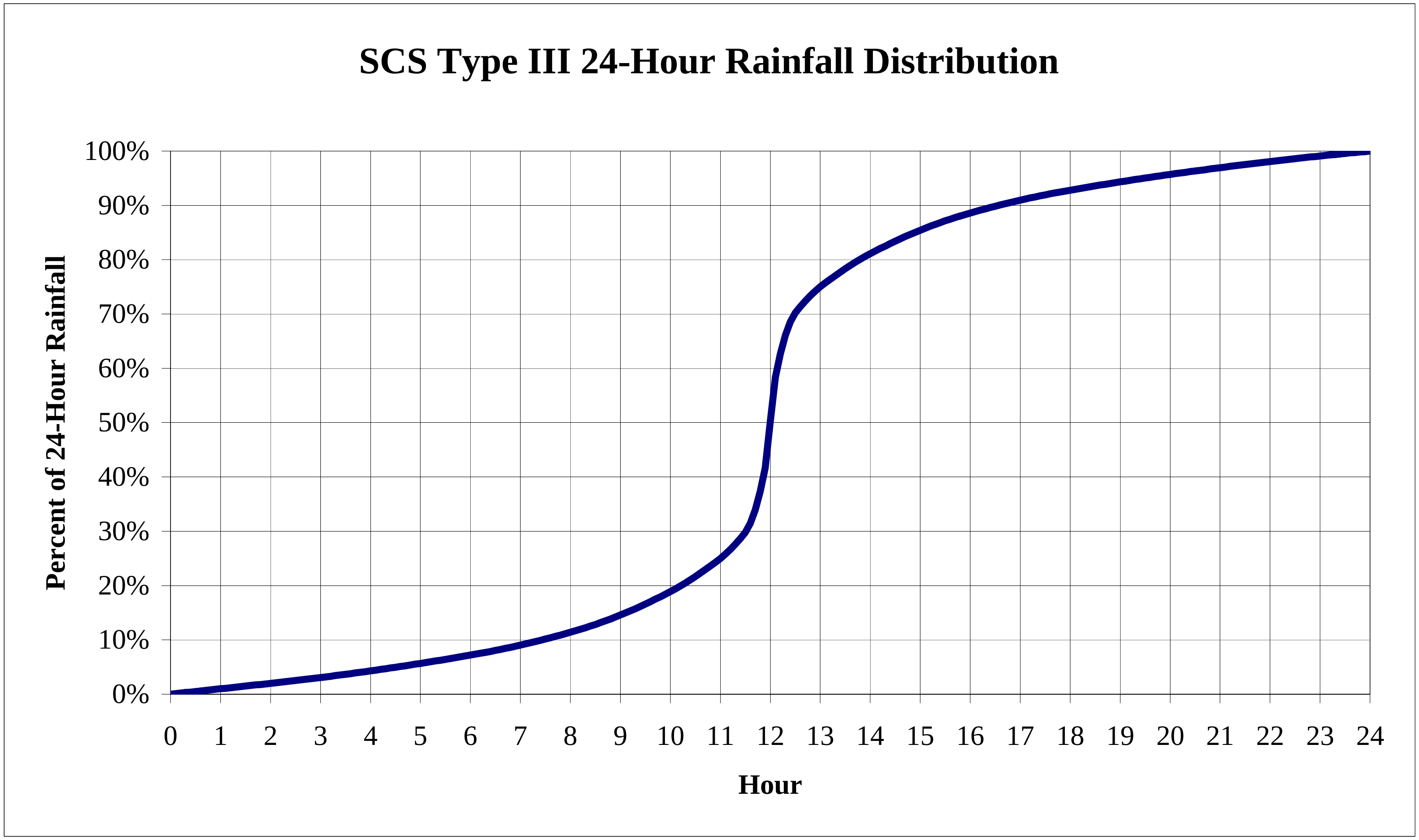

Figure 2-5 SCS Type III 24-Hour Rainfall Distribution

Southern Beaufort County
Example of PSMS Inventory
Buck Island Basin

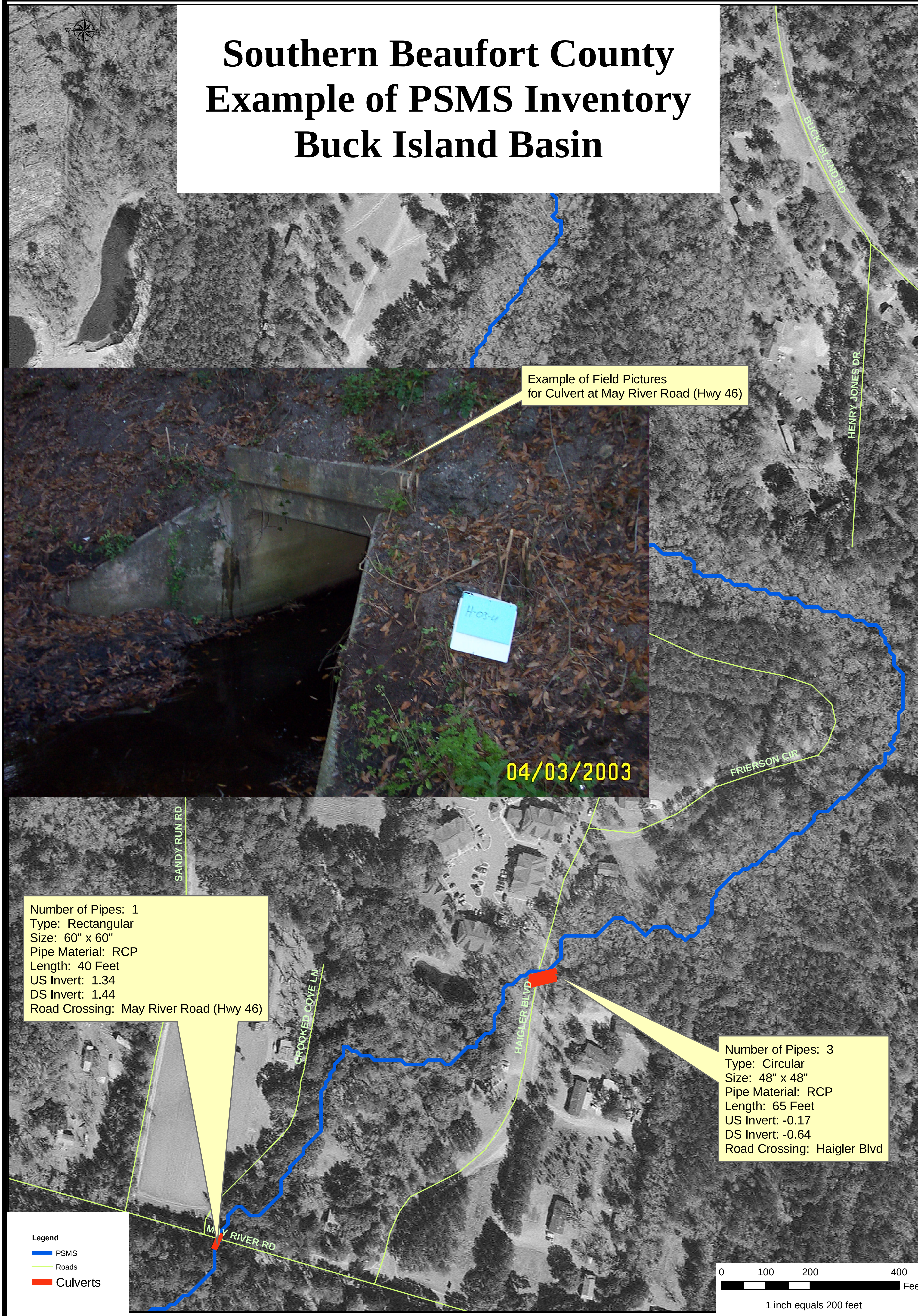

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Figure 2-6

CDM Camp Dresser & McKee Inc.

Job Number: 15178.00	Scale: 1" = 200'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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Roads	Beaufort County	2002

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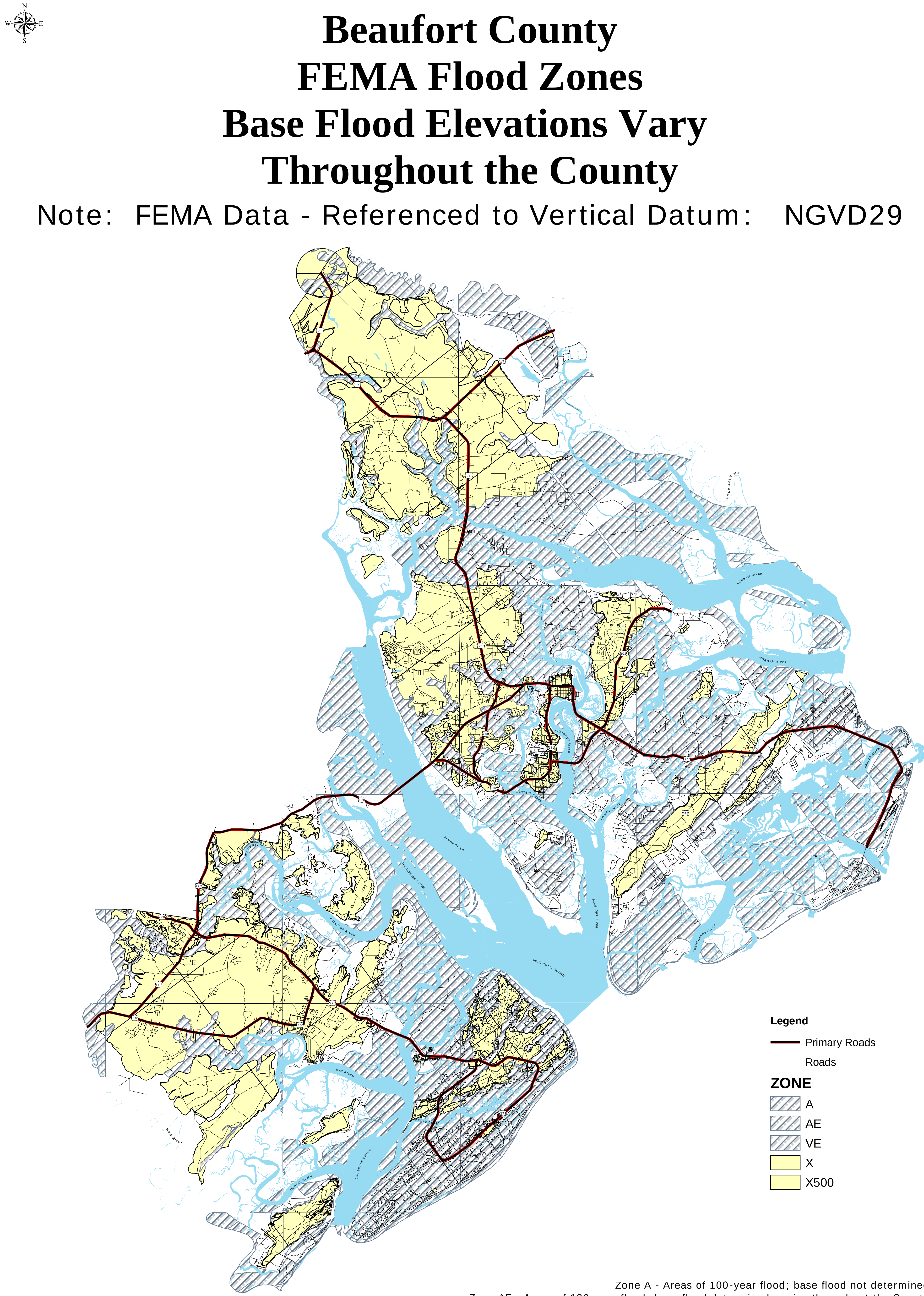
DATA	SOURCE	DATE
Aerial	T&H / Beaufort County	2003
PSMS	CDM / T&H	2004
Culverts	CDM / T&H	2004

Beaufort County
FEMA Flood Zones
Base Flood Elevations Vary Throughout the County

Note: FEMA Data - Referenced to Vertical Datum: NGVD29



0 10,000 20,000 Feet

1 inch equals 20,000 feet

Zone A - Areas of 100-year flood; base flood not determined
Zone AE - Areas of 100-year flood; base flood determined, varies throughout the County
Zone VE - Areas of 100-year coastal flood with velocity (wave action)
Zone X and Zone X500 - Areas outside the 1% chance of flooding during 100-year flood

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Figure 2-7

CDM Camp Dresser & McKee Inc.

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DATA	SOURCE	DATE
Roads	Beaufort County	2002
FEMA Flood Zones	FEMA	1998

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Beaufort County
Water Quality Stations
and
SC DHEC Impaired Point Locations

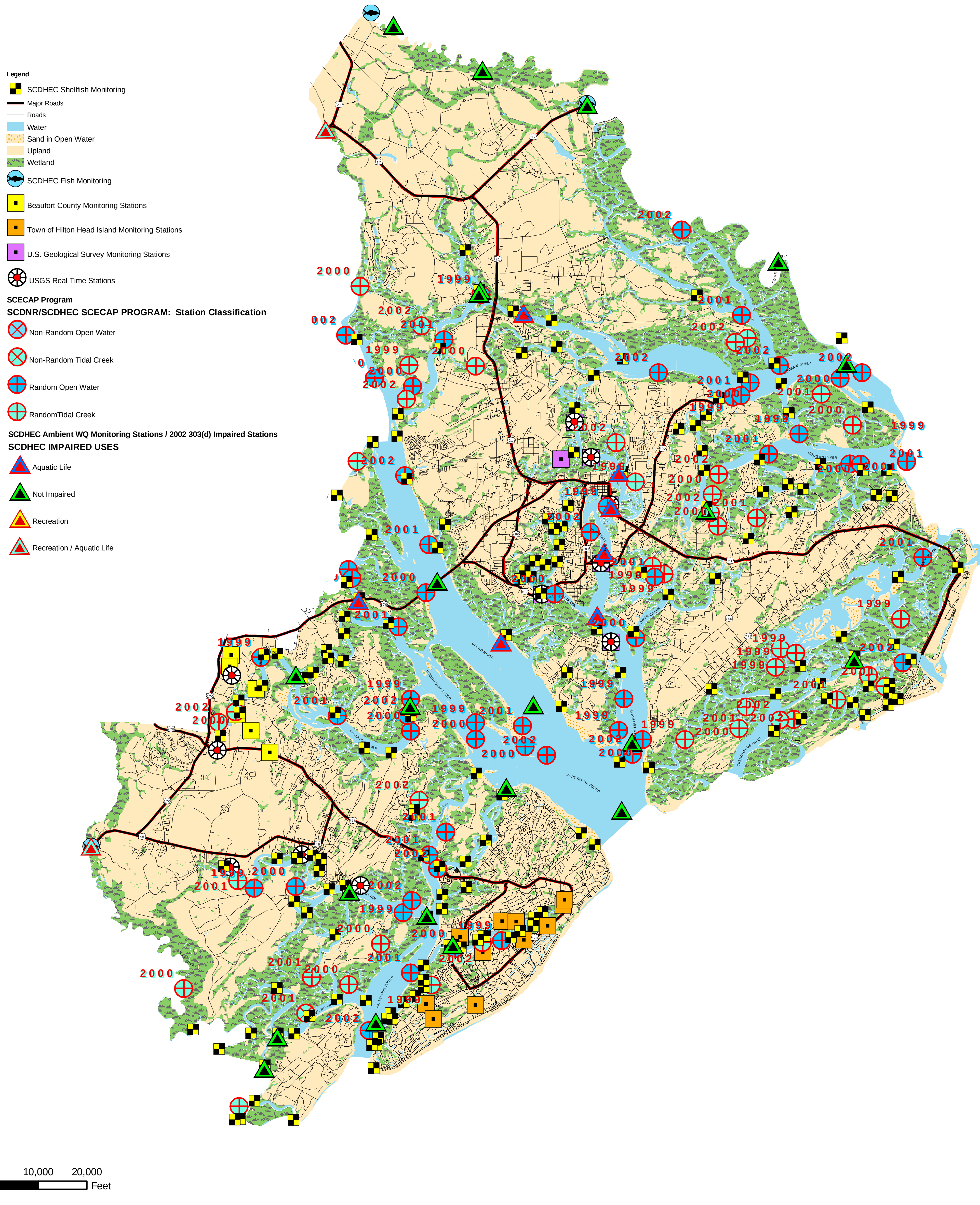

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Figure 2-8

CDM Camp Dresser & McKee Inc.

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Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
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DATA	SOURCE	DATE
Roads	Beaufort County	2003
Shellfish	SC DHEC	1999 - 2003
Fish Monitoring	SC DHEC	1999 - 2003
SCE CAP	SC DNR	1999 - 2003
USGS Real Time	USGS	1999 - 2003
Beaufort County Monitoring	Beaufort County	1999 - 2003
Town of Hilton Head	Town of Hilton Head Island	1999 - 2003
USGS	USGS	1999 - 2003

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Beaufort County
Water Quality Basins
and
Water Quality Modeling - Water Bodies

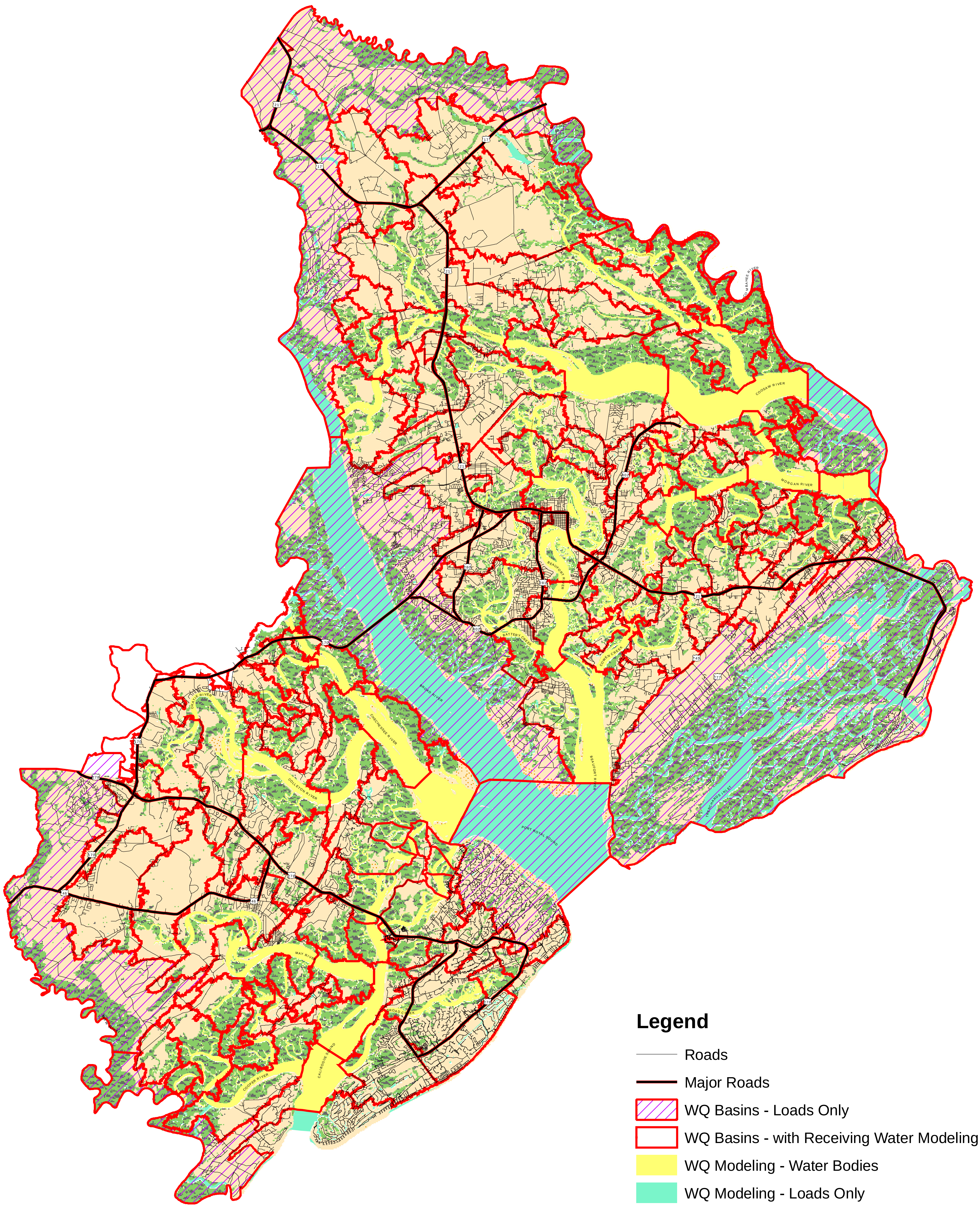

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Figure 2-9

CDM Camp Dresser & McKee Inc.

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Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
WQ Basins	CDM / T&H	2004
WQ Modeling - Water Bodies	CDM / T&H	2004

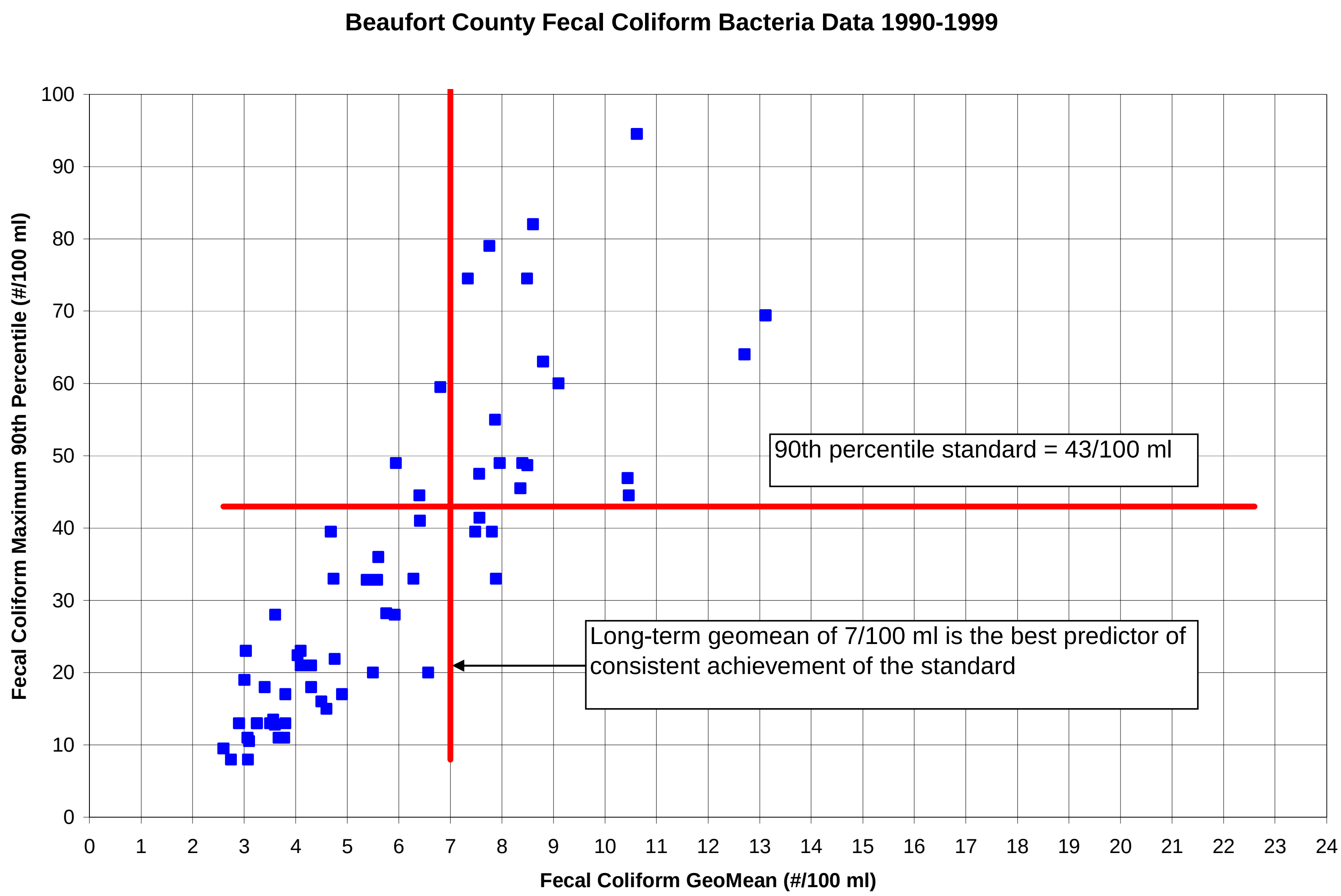

Figure 2-10 Relationship between Long-Term GeoMean and 36-Sample Maximum 90th Percentile Fecal Coliform Concentrations at Sampling Stations in Beaufort County.

Note: Each point represents data for one sampling station in Beaufort County.

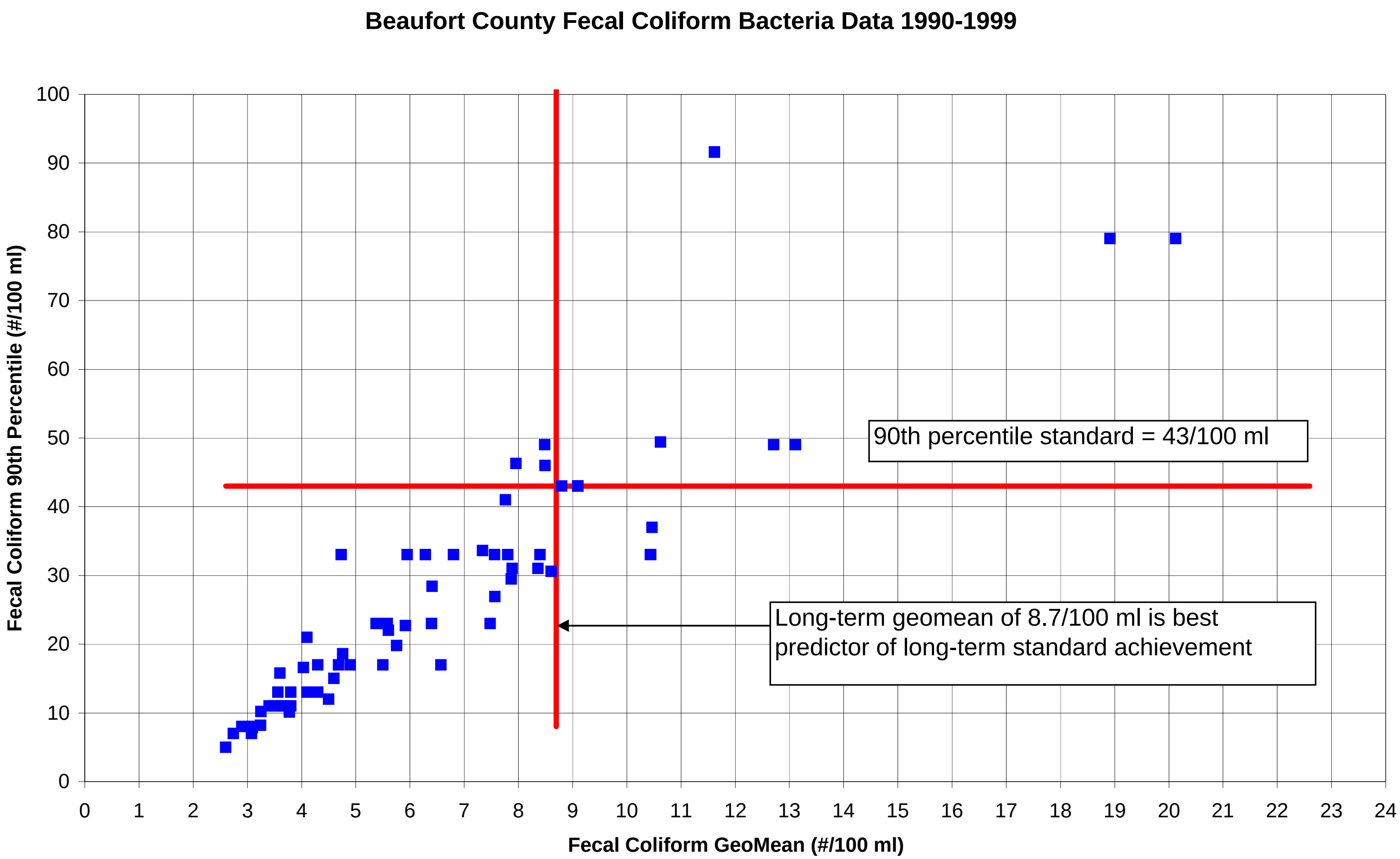

Figure 2-11 Relationship between Long-Term GeoMean and Long-Term 90th Percentile Fecal Coliform Concentrations at Sampling Stations in Beaufort County.

Note: Each point represents data for one sampling station in Beaufort County.

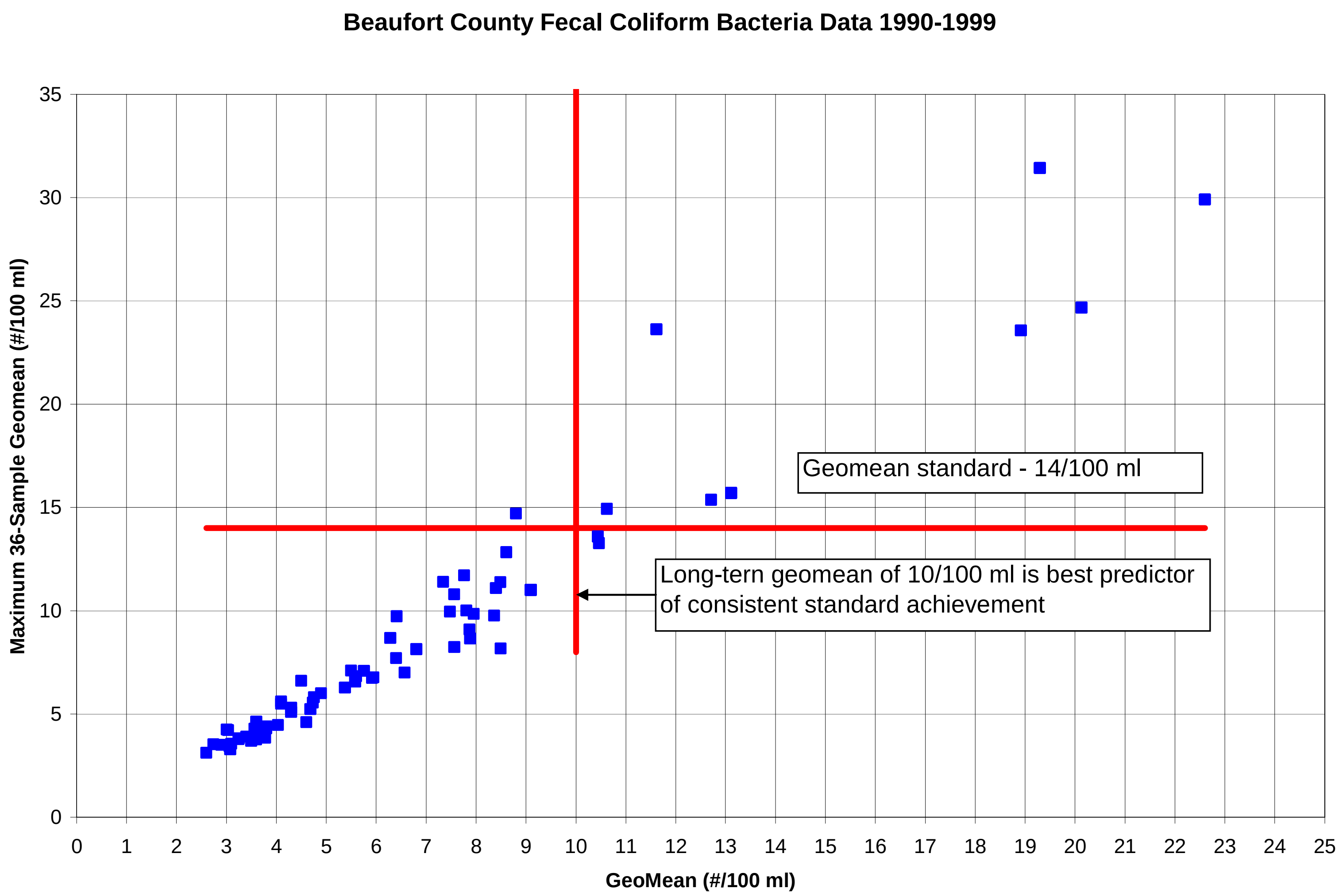

Figure 2-12 Relationship between Long-Term GeoMean and 36-Sample Maximum Geomean Fecal Coliform Concentrations at Sampling Stations in Beaufort County.

Note: Each point represents data for one sampling station in Beaufort County.

Section 3
Calibogue Sound Watershed Analysis

This section describes the physical features of the Calibogue Sound watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

3.1 Overview

The Calibogue Sound watershed is located south of the Broad River (see Figure 3-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in Bluffton Township, Town of Hilton Head Island, and Daufuskie Island that is tributary to the Calibogue Sound. Major Calibogue Sound tributaries included in the analysis are Broad Creek, Cooper River, Bull Creek, Old House Creek, Jarvis Creek, Skull Creek, Bryan Creek and Savage Creek.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several "hydrologic" basins. These are listed in Table 3-1 and presented in Figure 3-2. Table 3-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were updated to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

It should be noted that the hydrologic and hydraulic analysis presented in this section does not include the Town of Hilton Head Island. The analysis of the Town of Hilton Head Island is presented in Section 15 of the report and was not updated.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into basins, and the tidal receiving waters were subdivided into receiving water segments. These are listed in Table 3-2 and presented in Figure 3-3. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were completed to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

3.2 Hydrologic and Hydraulic Analysis

The ICPR Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the Calibogue Sound watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for Current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP. It was determined that the future analysis previously assumed has not yet been reached for most watersheds.

3.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Calibogue Sound basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include area, curve number, and time of concentration.

Table 3-3 lists the hydrologic parameter values for the Calibogue Sound PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the Calibogue Sound PSMS basins is presented in Table 3-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 3-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

3.2.2 Model Results

Tables in Appendix A list the summary of the results of the updated study including Updated Areas and CNs for the Calibogue Sound subbasins.

For existing land use, aerial maps generated in summer 2016 and local information were used to estimate the percentage of existing urban development. Appendix A also includes tables that list the peak water elevation values for model node locations along the Calibogue Sound PSMS.

Specific problem areas identified by the modeling are listed in Table 3-6 and presented in Figure 3-4. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs and found that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded.

Table 3-6 indicates the road crossings that are being overtopped by the design storm events. The Town of Hilton Head Island is considered separately in Section 15 of this report.

Evaluation of solutions to prevent these problems is discussed in the next section of this report.

3.2.3 Management Strategy Alternatives

The problems areas listed in Table 3-6 were evaluated by reviewing the previous report results and reviewing the culverts in the ICPR hydraulic model. In the original 2006 study, the ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 3-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

3.3 Water Quality Analysis

ATM used the WMM and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the Calibogue Sound watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions.

3.3.1 Land Use and BMP Coverage

Table 3-8 presents the existing land use estimates for the Calibogue Sound water quality basins. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 35 percent of the Calibogue Sound watershed area consists of urban systems (e.g., residential, commercial, golf course) and 65 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 13 percent of the watershed.

Estimates of BMP coverage for existing land use is presented in Table 3-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County and the Town of Hilton Head Island. Under existing land use conditions, 51 percent of the urban systems in the watershed are served by BMPs (primarily on the Town of Hilton Head Island).

3.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in Table 3-10. The existing land use values reflect areas that are not designated as “sewered” areas by the Beaufort-Jasper Water & Sewer Authority or the Public Service Districts (PSDs) on the Town of Hilton Head Island.

For existing land use conditions, 19 percent of the urban systems in the watershed are served by septic tanks.

Wastewater discharges are roughly 3 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be slightly higher (between 3 and 4 mgd). There are no direct discharges to receiving waters in the watershed.

3.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Calibogue Sound water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing land use conditions. The results are presented in Table 3-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

Direct and indirect wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing discharge of wastewater is limited to roughly 4 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be slightly higher (between 4 and 5 mgd). Using the values in Table 2-9, the wastewater load accounts for 3 to 4 percent of the total watershed load for nutrients (TN and TP) and less than 1 percent of the load for other constituents.

3.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Calibogue Sound watershed. The model actually includes Calibogue Sound, May River, Colleton River, and Chechessee River watersheds because they are interconnected at several points. Only the Calibogue Sound will be discussed in this section. A schematic of the model is presented as Figure 3-5.

Existing conditions for bacteria concentrations in the Calibogue Sound watershed are presented in Table 3-12. For each water quality basin river reach, the table lists the SCDHEC stations for which the 1990s bacteria data were analyzed, the concentrations calculated in the analysis, water quality concentration trends and the LOS associated with these concentrations (as discussed in Section 2.6.2). As shown in the table, SCDHEC data were only available in fourteen of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations in thirteen of the fourteen segments meet the water quality standards, and so these segments have an “A” LOS. Segments that do not meet the “A” LOS include one segment in Broad Creek (Broad Creek 3).

For informational purposes, Figure 3-6 presents a map of the LOS based on the monitoring data analysis, compared to SCDHEC “shellfish classification” (based on the 2016 SCDHEC reports for shellfish areas 16A, 19 and 20). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the LOS, so there may not be a direct relationship between LOS and shellfish classification presented in the map. In general, however, segments with an “A” LOS are expected to have the lowest probability of receiving a “restricted” classification, and segments with a “D” LOS are expected to have the highest probability of receiving a “restricted” classification.

Physical characteristics assigned to the updated model reaches are presented in Table 3-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the “characteristic length” (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the SCDHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Table 3-14 presents the updated values used in the existing condition model. Much of the flow to the tidal river segments comes from direct rainfall on the open water and tidal wetlands, as opposed to stormwater runoff and baseflow, and some of the basins have very little change in land use from existing to future conditions. Concentration remain relatively constant because of the substantial amount of open water/tidal wetland area and the relatively limited development in some basins, as well as the BMPs for new development, which are assumed to have a high level of treatment efficiency.

Table 3-15 shows the updated net advective flows between segments. The hydrodynamic model (SWMM) indicates that there is a substantial net flow from the Chechessee River to Calibogue Sound via Mackays Creek and Skull Creek. Bull Creek also carries flow from the May River south to Cooper River, which discharges to Calibogue Sound.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. The calibrated loss-rate coefficients from the 2006 study were used in the updated simulations.

Figure 3-7 is a graph showing a comparison between measured and modeled salinity data along the Calibogue Sound main stem. The figure shows that the salinity data calculated by the model is very close to the average measured value and is in all cases well within the 90 percent confidence interval of the mean of the salinity data. Measured salinity values do not vary much along the main stem.

Figures 3-8 and 3-10 are graphs showing a comparison between measured and modeled salinity data for Broad Creek and for Old House Creek/Jarvis Creek, respectively. These are tributaries whose contributing area is entirely within the Town of Hilton Head Island. The figures show that the salinity data calculated by the model is very close to the average measured value and is in all cases well within the 90 percent confidence interval of the mean of the salinity data. Measured and modeled salinity values drop noticeably at the upstream segments of Broad Creek, whereas the measured and modeled salinity values do not vary much in Old House Creek/Jarvis Creek.

Figure 3-9 is a graph showing a comparison between measured and modeled salinity data along the Cooper River. Unlike the other figures, the Cooper River figure does not show a good agreement between the measured and modeled salinity values. The modeled values are too high at the most downstream segment, and too low in the upper segments. Adjusting dispersion parameters further may improve the salinity results but provides a worse match between measured and modeled bacteria, which will be presented later. It is possible that further discretization of the model (i.e., more reaches) would provide better results.

Figure 3-11 is a graph showing a comparison between measured and modeled salinity data along Skull Creek and Mackays Creek. The figure shows that the salinity data calculated by the model is very close to the average measured value and is in all cases well within the 90 percent confidence interval of the mean of the salinity data. Measured salinity values do not vary much along the main stem.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration for Calibogue Sound watershed are presented in Figures 3-15 through 3-20. The graphs generally show the same type of results as the salinity plots. Results for Calibogue Sound (Figure 3-15), Broad Creek (Figure 3-16, Old House Creek/Jarvis Creek (Figure 3-18) and Skull Creek/Mackays Creek (Figure 3-16) show very good agreement between the measured values and the model results. The Cooper River (Figure 3-14) shows some discrepancies between measured and modeled bacteria values. As it was for salinity, the modeled value at the downstream segment is too high, and it is too low at the next upstream station.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 3-16. The loss rates ranged from 0.5/day to 2.0/day. The lowest values are typically applied at the downstream end of the main stem and major. This makes sense if it is presumed that bacteria loss is in part due to light mortality, because the water depths are much greater at the downstream end of the main stem and major tributaries, and light would penetrate less of the total depth in those areas.

Based on water quality sampling data and model results, the following conclusions are:

- Problem basins include Broad Creek 3 and 4, Jarvis Creek 2
- Request that SCDHEC add bacteria sampling stations in the water quality basins Cooper River Trib and Jarvis Creek 2, to validate model results
- Evaluate opportunities for retrofit BMPs or modification of existing ponds in the Broad Creek water quality basins to the maximum extent practicable.
- Two regional water quality BMPs are proposed in Broad Creek 4 and Jarvis Creek 2

Discussion of water quality related recommendations for monitoring and regional BMPs (below) in the Calibogue Sound watershed are presented as part of the overall recommended monitoring program for Beaufort County contained in the Appendix of this report.

3.3.5 Management Strategy Alternatives

In analyzing the watershed, two feasible regional detention sites were identified. The area tributary to the Jarvis Creek 2 Regional BMP site includes approximately 923 acres of commercial, golf course and single-family development built prior to volume control stormwater regulations. There are stormwater best management practices, such as detention facilities, in the area. The project would be to construct modifications to the existing regional wet detention pond in vicinity of William Hilton Parkway and Sol Blatt Jr. Parkway. Proposed modifications include permanent pool expansion, littoral shelf creation and structure modification to provide stormwater runoff water quality treatment and volume reduction. Due to the presence of some wetlands in the area, project design would involve delineation and avoidance of the wetlands. Jarvis Creek is impaired by bacteria pollution.

A new WMM scenario was developed for the Jarvis Creek 2 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Jarvis Creek 2 water quality basin of approximately 13%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Calibogue Sound:

Parameter	lb/yr removed
Total Nitrogen	646
Total Phosphorus	148
TSS	74,000

The area tributary to the Broad Creek 4 Regional BMP site includes approximately 750 acres of golf course and single-family development built prior to volume control stormwater regulations. There are stormwater best management practices, such as detention facilities, in the area. The project would be to create additional storage via modified structure from golf course and to construct a regional wet detention pond adjacent to William Hilton Parkway. The project will provide stormwater runoff water quality treatment and volume reduction. Due to the presence of some wetlands in the area, project design would involve delineation and avoidance of the wetlands. Broad Creek is impaired by bacteria pollution.

A new WMM scenario was developed for the Broad Creek 4 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Broad Creek 4 water quality basin of approximately 17%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Calibogue River:

Parameter	lbs/yr removed
Total Nitrogen	527
Total Phosphorus	130
TSS	59,910

The results of the water quality analysis suggest that several areas (e.g., Broad Creek, Cooper River) do not meet the bacteria water quality standards under existing conditions. It is interesting to note that the Cooper River area has very little development in the existing condition, suggesting that there are natural sources that are causing the high bacteria levels. It is not expected that controls on development would result in the achievement of the standards if they are being exceeded by natural sources. In contrast, other areas such as Broad Creek appear to be affected by urban development, and it is appropriate to evaluate measures that could be taken to meet the water quality standards, or perhaps more realistically, to improve the existing LOS. As discussed above, these activities would include retrofit of existing development that does not have ponds, and modification of existing ponds that may not have been designed for water quality control.

For informational purposes, the areas with "A" and "B" type soils are presented in Figure 3-18. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

3.4 Planning Level Cost Estimates for Management Alternatives

Table 3-18 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Calibogue Sound watershed (excluding the Town of Hilton Head Island, which is discussed in Section 15 of this report). As shown in the table, the projects are estimated to have a total cost of $1.958 million based on January 2018 dollars. Details of the cost estimate for each project are shown in Appendix A.

Two regional CIP projects were identified in the Calibogue Sound watershed. These two projects are estimated to have a total cost of $3.45 million and are detailed in the CIP in Appendix O.

TABLE 3-1
HYDROLOGIC BASINS
CALIBOGUE SOUND WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Haig Point	552	1	552
Melrose	274	1	274
Moss Creek East	176	2	88
Moss Creek West	262	2	131
Ramshorn Creek	221	1	221
Webb Tract	229	1	229
Wildlife Preserve	306	1	306
TOTAL	2,020	9	224

TABLE 3-2
WATER QUALITY BASINS
CALIBOGUE SOUND WATERSHED

Basin Name	Tributary Area (acres)
Calibogue Sound 1	2,956
Calibogue Sound 2	3,377
Calibogue Sound 3	1,238
Calibogue Sound 4	2,182
Calibogue Sound 5	2,376
Broad Creek 1	4,219
Broad Creek 2	7,846
Broad Creek 3	750
Broad Creek 4	1,417
Cooper River 1	5,256
Cooper River 1	2,969
Cooper River 1	582
Cooper River Trib	1,561
Bull Creek/Cooper 1	1,058
Bull Creek/Cooper 2	516
Bull Creek/Cooper 3	461
Hoophole Creek	646
Old House Creek	288
Jarvis Creek 1	927
Jarvis Creek 2	1,924
Skull Creek South 1	2,986
Skull Creek South 2	381
Mackays Creek South	986
Bryan Creek 1	550
Bryan Creek 2	204
Savage Creek 1	374
Savage Creek 2	82
TOTAL	48,110

TABLE 3-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
CALIBOGUE SOUND WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Haig Point Basin					
HP_M1	552	78	148	79	142
Melrose Basin					
MS_M1	274	78	196	82	174
Moss Creek East Basin					
MCE_M1	134	78	78	79	78
MCE_T1	41	83	61	82	45
Moss Creek West Basin					
MCW_M1	167	77	76	79	73
MCW_M2	94	89	47	86	46
Ramshorn Creek Basin					
RC_M1	221	74	173	83	133
Webb Tract Basin					
WT_M1	229	81	110	84	101
Wildlife Preserve Basin					
WP_M1	306	73	177	74	177
Average	224	79	140	80	130

TABLE 3-4
HYDRAULIC DATA SUMMARY
CALIBOGUE SOUND WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Haig Point	0	0	0	0	0	1	2	0
Melrose	0	0	1	3	0	1	1	0
Moss Creek East	2	1,262	3	4	0	4	1	1
Moss Creek West	4	2,848	2	4	0	4	0	1
Ramshorn Creek	5	5,319	0	0	0	0	0	0
Webb Tract	3	2,194	2	2	0	0	2	0
Wildlife Preserve	3	3,035	2	6	0	5	3	1
TOTAL	17	14,658	10	19	0	15	9	3

TABLE 3-5
CULVERT DATA FOR HYDROLOGIC BASINS
CALIBOGUE SOUND WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Haig Point Basin						
No road crossings in this basin						
Melrose Basin						
Masters Drive	MS_M-1A	18"x18"	50	5.3	6.9	25
	1B	18"x18"	50	5.4		
	1C	18"x18"	50	5.2		
Moss Creek East Basin						
Moss Creek Drive	MCE_M-1	36"x36"	80	1.4	7.6	25
Wax Myrtle Lane	MCE_M-3	48"x48"	58	2.3	12.0	25
Fording Island Road	MCE_T1-3A	24"x24"	177	5.8	11.0	100
	3B	36"x36"	177	5.6		
Moss Creek West Basin						
Moss Creek Drive	MCW_M-1	42"x42"	70	1.9	8.5	25
Fording Island Road	MCW_M-7A	36"x36"	200	6.0	11.7	100
	7B	36"x36"	200	5.0		
	7C	36"x36"	200	5.4		
Ramshorn Creek Basin						
No road crossings in this basin						
Webb Tract Basin						
Cooper River Landing Road	WT_M-2	30"x30"	30	1.7	5.2	25
Freeport Road	WT_M-4	18"x18"	30	4.3	6.2	25
Wildlife Preserve Basin						
Bayley Road	WP_M-2A	24"x24"	50	-1.0	6.3	25
	2B	24"x24"	50	-1.0		
	2C	24"x24"	50	-1.0		
Colleton River Drive	WP_M-3A	18"x18"	60	2.5	4.7	25
	3B	18"x18"	60	2.4		
	3C	18"x18"	60	2.5		

TABLE 3-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
CALIBOGUE SOUND WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Haig Point Basin					
No Overtopping Identified					
Melrose Basin					
Masters Drive	MS_M-1	6.9	6.9	2	7.2
				10	7.2
				25	7.2
				100	7.2
Moss Creek East Basin					
No Overtopping Identified					
Moss Creek West Basin					
No Overtopping Identified					
Ramshorn Creek Basin					
No Overtopping Identified					
Webb Tract Basin					
Cooper River Landing Rd.	WT_M-11	5.2	5.2	2	5.8
				10	6.0
				25	6.1
				100	6.2
Freeport Road	WT_M-14	6.2	6.2	2	6.9
				10	7.1
				25	7.2
				100	7.5
Wildlife Preserve Basin					
Bayley Road	WP_M-8	6.3	6.5	10	6.6
				25	6.7
				100	6.8
Colleton River Drive	WP_M-16	4.7	4.7	2	5.4
				10	6.6
				25	6.7
				100	6.8

TABLE 3-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
CALIBOGUE SOUND WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Melrose Basin			
Masters Drive	MS_M-1A 1B 1C	18"x18" 18"x18" 18"x18"	Replace culverts with ten 36" pipes; set culvert inverts at 3.6 ft NAVD
Moss Creek East Basin			
*Moss Creek Drive	MCE_M-1	36"x36"	Add one 24" pipe to existing culverts
Moss Creek West Basin			
No improvements required			
Ramshorn Creek Basin			
No improvements required			
Webb Tract Basin			
Cooper River Landing Road	WT_M-2	30"x30"	Replace culvert with four 8 ft by 5 ft box culverts, Raise road from elevation 5.2 ft to elevation 7.6 ft NAVD (length of 670 ft)
Freeport Road	WT_M-4	18"x18"	Replace culvert with twelve 36" pipes, Raise road from elevation 6.2 ft to elevation 7.6 ft NAVD (length of 640 ft)
Wildlife Preserve Basin			
Bayley Road	WP_M-2A 2B 2C	24"x24" 24"x24" 24"x24"	Replace culverts with three 4 ft by 4 ft box culverts
Colleton River Drive	WP_M-3A 3B 3C	18"x18" 18"x18" 18"x18"	Replace culverts with one 7 ft by 4 ft box culvert, Raise road from elevation 4.7 ft to elevation 7.6 ft NAVD (length of 660 ft)

* Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR.

TABLE 3-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
CALIBOGUE SOUND WATERSHED

Land Use Type	Broad Creek 1 (acres)	Broad Creek 2 (acres)	Broad Creek 3 (acres)	Broad Creek 4 (acres)	Bryan Creek 1 (acres)	Bryan Creek 2 (acres)	Bull Creek / Cooper 1 (acres)	Bull Creek / Cooper 2 (acres)	Bull Creek / Cooper 3 (acres)	Calibogue Sound 1 (acres)	Calibogue Sound 2 (acres)	Calibogue Sound 3 (acres)	Calibogue Sound 4 (acres)	Calibogue Sound 5 (acres)	Calibogue Sound 6 (acres)
Agricultural/Pasture	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Commercial	213	502	55	94	0	0	0	0	0	28	0	3	23	8	16
Forest/Rural Open	0	34	2	4	7	16	9	0	0	40	60	0	3	263	0
Golf Course	210	1248	1	248	0	0	0	0	0	129	52	0	6	158	248
High Density Residential	1004	2333	92	534	0	0	0	0	0	98	0	18	0	0	826
Industrial	362	890	53	169	0	0	0	0	0	76	37	5	49	93	207
Institutional	44	28	5	8	0	0	0	0	0	0	0	0	0	0	0
Low Density Residential	7	0	0	0	0	0	0	0	0	0	2	1	13	1	0
Medium Density Residential	14	176	0	0	0	0	0	0	0	231	230	0	79	201	0
Open Water/Tidal	1388	1872	478	159	163	95	985	255	199	2005	2866	1181	1740	1471	703
Silviculture	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Urban Open	617	505	61	93	0	0	1	54	0	296	13	2	8	9	209
Wetland/Water	360	259	2	107	380	93	63	206	262	52	118	28	261	172	1
TOTAL	4219	7845	750	1417	550	204	1058	516	461	2956	3377	1238	2182	2376	2211
Urban Imperviousness (%)	23%	29%	18%	34%	0%	0%	0%	0%	0%	6%	3%	1%	3%	5%	26%

TABLE 3-8 (CONTINUED)
WATER QUALITY BASIN LAND USE DISTRIBUTION
CALIBOGUE SOUND WATERSHED

Land Use Type	Calibogue Sound 7 (acres)	Cooper River 1 (acres)	Cooper River 2 (acres)	Cooper River 3 (acres)	Cooper River Trib (acres)	Hoophole Creek (acres)	Jarvis Creek 1 (acres)	Jarvis Creek 2 (acres)	Mackays Creek South (acres)	Old House Creek (acres)	Savage Creek 1 (acres)	Savage Creek 2 (acres)	Skull Creek South 1 (acres)	Skull Creek South 2 (acres)	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Commercial	0	4	17	0	0	0	21	126	85	23	0	0	70	9	1297
Forest/Rural Open	54	506	667	0	873	23	3	33	17	0	0	0	86	0	2698
Golf Course	297	23	0	0	0	0	0	194	243	0	0	0	18	5	3079
High Density Residential	0	5	0	0	0	0	63	388	21	1	0	0	242	33	5657
Industrial	61	64	0	0	0	0	22	257	117	35	0	0	140	10	2649
Institutional	0	0	0	0	0	0	0	135	0	0	0	0	4	0	225
Low Density Residential	1	55	0	0	0	0	0	2	14	0	0	0	23	0	120
Medium Density Residential	232	151	0	0	0	0	72	175	204	107	0	0	208	0	2080
Open Water/Tidal	425	3666	1966	550	521	486	707	260	272	113	340	70	1494	254	26684
Silviculture	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Urban Open	92	233	1	0	0	50	27	285	6	8	25	6	140	22	2761
Wetland/Water	27	549	117	32	190	88	12	69	7	1	9	6	562	48	4082
TOTAL	1189	5255	2769	582	1584	646	927	1924	986	288	374	82	2986	382	51332
Urban Imperviousness (%)	9%	2%	1%	0%	1%	0%	9%	30%	23%	25%	0%	0%	11%	8%	13%

TABLE 3-9
WATER QUALITY BASIN BMP COVERAGE
CALIBOGUE SOUND WATERSHED

Land Use Type	Broad Creek 1	Broad Creek 2	Broad Creek 3	Broad Creek 4	Bryan Creek 1	Bryan Creek 2	Bull Creek/ Cooper 1	Bull Creek/ Cooper 2
Commercial	18.0%	27.7%	21.6%	23.0%	0.0%	0.0%	0.0%	0.0%
Golf Course	99.8%	93.6%	86.4%	100.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	67.2%	84.9%	86.2%	79.8%	0.0%	0.0%	0.0%	0.0%
Industrial	10.4%	34.4%	38.3%	57.2%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	0.0%	60.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
TOTAL	52.8%	71.8%	53.3%	77.1%	0.0%	0.0%	0.0%	0.0%

TABLE 3-9 (CONTINUE)
WATER QUALITY BASIN BMP COVERAGE
CALIBOGUE SOUND WATERSHED

Land Use Type	Bull Creek/ Cooper 3	Calibogue Sound 1	Calibogue Sound 2	Calibogue Sound 3	Calibogue Sound 4	Calibogue Sound 5	Calibogue Sound 6	Calibogue Sound 7	Cooper River 1	Cooper River 2	Cooper River 3
Commercial	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Golf Course	0.0%	0.0%	92.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	88.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	24.6%	9.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	0.0%	0.0%	29.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
TOTAL	0.0%	0.0%	39.8%	66.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

TABLE 3-9 (CONTINUE)
WATER QUALITY BASIN BMP COVERAGE
CALIBOGUE SOUND WATERSHED

Land Use Type	Cooper River Trib	Hoophole Creek	Jarvis Creek 1	Jarvis Creek 2	Mackays Creek South	Old House Creek	Savage Creek 1	Savage Creek 2	Skull Creek South 1	Skull Creek South 2	TOTAL
Commercial	0.0%	0.0%	15.0%	15.0%	7.6%	0.0%	0.0%	0.0%	1.0%	100.0%	19.2%
Golf Course	0.0%	0.0%	0.0%	99.8%	0.0%	0.0%	0.0%	0.0%	100.0%	100.0%	74.6%
High Density Residential	0.0%	0.0%	96.8%	97.3%	0.0%	0.0%	0.0%	0.0%	53.4%	100.0%	78.3%
Industrial	0.0%	0.0%	12.1%	37.2%	0.0%	0.0%	0.0%	0.0%	24.5%	100.0%	25.6%
Institutional	0.0%	0.0%	0.0%	0.2%	0.0%	0.0%	0.0%	0.0%	77.1%	0.0%	1.2%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	9.4%
TOTAL	0.0%	0.0%	42.6%	51.1%	0.7%	0.0%	0.0%	0.0%	23.0%	100.0%	51.0%

TABLE 3-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
CALIBOGUE SOUND WATERSHED

Land Use Type	Broad Creek 1	Broad Creek 2	Broad Creek 3	Broad Creek 4	Bryan Creek 1	Bryan Creek 2	Bull Creek/ Cooper 1	Bull Creek/ Cooper 2	Bull Creek/ Cooper 3
Commercial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.1%	0.0%	1.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	1.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	5.0%	0.9%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
TOTAL	0.1%	0.0%	0.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

TABLE 3-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
CALIBOGUE SOUND WATERSHED

Land Use Type	Calibogue Sound 1	Calibogue Sound 2	Calibogue Sound 3	Calibogue Sound 4	Calibogue Sound 5	Calibogue Sound 6	Calibogue Sound 7	Cooper River 1	Cooper River 2	Cooper River 3
Commercial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	1.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%	1.1%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	21.1%	0.0%	0.0%	0.0%	0.0%	3.3%	0.0%	0.0%
Medium Density Residential	0.3%	1.6%	0.0%	0.0%	0.0%	0.0%	0.0%	1.9%	0.0%	0.0%
TOTAL	0.1%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%	0.0%

TABLE 3-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
CALIBOGUE SOUND WATERSHED

Land Use Type	Cooper River Trib	Hoophole Creek	Jarvis Creek 1	Jarvis Creek 2	Mackays Creek South	Old House Creek	Savage Creek 1	Savage Creek 2	Skull Creek South 1	Skull Creek South 2	TOTAL
Commercial	0.0%	0.0%	0.0%	0.0%	15.8%	0.0%	0.0%	0.0%	0.0%	0.0%	1.0%
High Density Residential	0.0%	0.0%	0.0%	0.1%	0.0%	0.0%	0.5%	0.0%	0.0%	0.0%	0.1%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%	0.0%	0.0%	0.0%	0.0%	0.2%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.7%	0.0%	0.0%	0.0%	1.8%
Medium Density Residential	0.0%	0.0%	0.0%	1.2%	0.0%	3.8%	1.4%	0.0%	0.0%	0.0%	0.9%
TOTAL	0.0%	0.0%	0.3%	0.0%	1.4%	1.4%	0.0%	0.0%	0.1%	0.0%	0.1%

TABLE 3-11
AVERAGE ANNUAL LOADS FOR CALIBOGUE SOUND WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
Calibogue Sound 1	2,956	8,320	82,645	124	72,145	8.03E+14	157	30,896	4,069	383,000	3,119
Calibogue Sound 2	3,377	10,980	95,771	131	94,373	9.33E+14	185	39,261	4,915	277,000	4,213
Calibogue Sound 3	1,238	4,382	36,329	49	37,513	3.46E+14	71	15,500	1,899	80,349	1,715
Calibogue Sound 4	2,182	6,949	61,985	90	59,820	5.94E+14	119	25,080	3,087	224,000	2,602
Calibogue Sound 5	2,376	6,259	60,613	95	54,186	5.88E+14	115	23,222	3,095	281,000	2,273
Broad Creek 1	4,219	9,610	131,000	232	82,617	9.58E+14	194	37,033	4,497	927,000	2,981
Broad Creek 2	7,845	16,852	246,000	436	142,000	1.48E+15	321	63,645	7,987	1,640,000	4,759
Broad Creek 3	750	2,277	25,573	41	19,485	1.95E+14	43	8,112	1,012	134,000	817
Broad Creek 4	1,417	2,704	43,253	75	22,462	2.39E+14	49	10,187	1,250	305,000	642
Cooper River 1	5,255	14,724	126,000	179	127,000	1.25E+15	240	52,804	6,463	421,000	5,408
Cooper River 2	2,769	7,625	62,076	85	65,346	6.05E+14	119	27,045	3,257	164,000	2,854
Cooper River 3	582	2,036	16,506	22	17,442	1.61E+14	33	7,198	880	35,329	794
Cooper River Trib	1,584	2,502	18,754	27	21,432	1.97E+14	31	8,845	989	73,990	755
Bull Creek (Cooper) 1	1,058	3,657	29,617	40	31,326	2.88E+14	59	12,928	1,578	64,009	1,421
Bull Creek (Cooper) 2	516	1,199	9,039	13	10,272	9.46E+13	16	4,239	477	34,443	372
Bull Creek (Cooper) 3	461	1,040	7,626	11	8,912	8.21E+13	13	3,678	401	34,217	292
Hoophole Creek	646	1,901	15,131	21	16,282	1.50E+14	29	6,720	804	38,527	702
Old House Creek	288	715	11,869	21	6,447	1.11E+14	21	3,045	429	102,000	245
Jarvis Creek 1	927	2,934	29,041	41	25,239	2.62E+14	53	10,634	1,332	116,000	1,096
Jarvis Creek 2	1,924	3,564	60,941	111	30,675	4.31E+14	82	14,223	1,786	491,000	933
Skull Creek South 1	2,986	7,574	82,428	130	65,571	7.49E+14	140	28,300	3,481	484,000	2,519
Skull Creek South 2	382	1,115	9,917	14	9,430	8.32E+13	16	3,889	458	27,689	387
Mackays Creek South	986	1,997	33,280	64	17,932	3.07E+14	59	9,058	1,494	299,000	658
Bryan Creek 1	550	1,055	7,347	11	9,034	8.32E+13	11	3,728	383	42,346	243
Bryan Creek 2	204	466	3,471	5	3,987	3.67E+13	6	1,646	183	14,117	139
Savage Creek 1	374	1,255	10,179	14	10,751	9.90E+13	20	4,437	542	21,677	490
Calibogue Sound 6	2,211	5,036	83,960	159	45,326	7.48E+14	144	21,905	3,216	775,000	1,766
Calibogue Sound 7	1,189	2,214	26,708	32	19,549	1.60E+14	31	9,011	1,382	34,297	695
Savage Creek 2	82	262	2,112	3	2,246	2.01E+13	4	927	110	4,116	100
TOTAL	51,332	131,204	1,429,171	2,276	1,128,800	1.21E+16	2,381	487,196	61,456	7,528,106	44,990

TABLE 3-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS

CALIBOGUE SOUND WATERSHED

Water Quality Basin ID	DHEC Station(s)	Years of Record	No. of Samples	Fecal Coliform Concentrations: Long-Term Average Geomean (#/100 ml)	Long-Term Average 90th Percentile (#/100 ml)	Most Recent 3 Year Values Geomean (#/100 ml)	Most Recent 3 Year Values 90th Percentile (#/100 ml)	Trend	Level of Service
Calibogue Sound 1	20-02, 20-03, 20-19A	1999-2016	618	3	8	3.29	11.37	Increasing	A
Calibogue Sound 2	20-22, 20-05	1999-2016	416	2.92	8	2.83	7.8	Decreasing	A
Calibogue Sound 3	20-06	1999-2016	208	3.19	13	3.03	7.49	Decreasing	A
Calibogue Sound 4	20-07	1999-2016	208	2.68	7	2.26	4.5	Decreasing	A
Calibogue Sound 5	20-20 A	1999-2016	208	3.67	13	3.73	13.69	No Trend	A
Calibogue Sound 6	20-26, 20-14A	1999-2016	205	2.95	11	2.97	12.57	Increasing	A
Calibogue Sound 7	NA	1999-2016	NA	NA	NA	NA	NA	NA	NA
Broad Creek 1	20-29, 20-15, 20-28, 20-03, 20-15A	1999-2016	771	4.44	17	3.72	14	Decreasing	A
Broad Creek 2	20-24, 20-18, 20-17B, 20-04A	1999-2016	826	6.52	23	6.02	22	Decreasing	A
Broad Creek 3	20-16, 20-25	1999-2016	410	8.16	33	7.84	40.21	Decreasing	B
Broad Creek 4	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cooper River 1	19-03, 19-09, 19-17A	1999-2016	625	3.26	11	3.17	8.52	No Trend	A
Cooper River 2	19-02	1999-2016	209	5.03	17	6.26	23	No Trend	A
Cooper River 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cooper River Trib	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bull Creek/Cooper 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bull Creek/Cooper 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bull Creek/Cooper 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Hoophole Creek	NA	NA	NA	NA	NA	NA	NA	NA	NA
Old House Creek	NA	NA	NA	NA	NA	NA	NA	NA	NA
Jarvis Creek 1	20-23	1999-2016	208	4.99	28.14	7.22	49	Increasing	A
Jarvis Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Skull Creek South 1	20-10, 20-11, 20-12	1999-2016	622	2.77	7	2.44	7.8	Decreasing	A
Skull Creek South 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Mackays Creek South	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bryan Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bryan Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Savage Creek 1	19-11	1999-2016	208	3.19	11	3.7	12.78	No Trend	A
Savage Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA

TABLE 3-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

CALIBOGUE SOUND WATERSHED

Water Quality Basin ID	South WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Calibogue Sound 1	1	5.15E+07	Ocean	10,463	3,586	450
Calibogue Sound 2	2	4.88E+07	Calibogue Sound 1	13,400	5,053	225
			May River 1	5,185	3,356	300
Calibogue Sound 3	3	1.04E+07	Calibogue Sound 2	4,789	4,313	225
Calibogue Sound 4	4	8.91E+06	Calibogue Sound 3	2,564	2,784	225
Calibogue Sound 5	5	4.35E+06	Calibogue Sound 4	1,545	4,393	450
			Mackays Creek North 2	561	4,393	150
Broad Creek 1	6	7.02E+06	Calibogue Sound 1	1,606	4,408	180
Broad Creek 2	7	7.03E+06	Broad Creek 1	834	5,262	300
Broad Creek 3	8	1.33E+06	Broad Creek 2	700	4,023	20
Broad Creek 4	9	1.27E+05	Broad Creek 3	346	1,143	20
Cooper River 1	10	1.68E+07	Calibogue Sound 1	3,318	7,106	100
Cooper River 2	11	7.97E+06	Cooper River 1	1,082	7,129	10
Cooper River 3	12	1.60E+06	Cooper River 2	704	5,053	50
Cooper River Trib	13	8.64E+05	Cooper River 2	284	2,237	50
Bull Creek/Cooper 1	14	2.74E+06	Cooper River 1	894	2,763	300
Bull Creek/Cooper 2	15	1.37E+06	Bull Creek/Cooper 1	609	2,253	300
Bull Creek/Cooper 3	16	5.55E+05	Bull Creek/Cooper 2	440	1,770	300
Hoophole Creek	17	7.79E+05	Bull Creek/Cooper 1	352	1,416	300
Old House Creek	18	1.61E+05	Calibogue Sound 2	314	1,184	150
Jarvis Creek 1	19	1.34E+06	Calibogue Sound 3	649	3,454	450
Jarvis Creek 2	20	2.26E+05	Jarvis Creek 1	293	1,851	150
Skull Creek South 1	21	6.99E+06	Calibogue Sound 3	1,126	4,342	150
Skull Creek South 2	22	2.60E+06	Skull Creek South 1	1,960	2,945	150
			Skull Creek North 2	3,343	2,945	150
Mackays Creek South	23	3.43E+05	Calibogue Sound 4	215	1,119	150
Bryan Creek 1	24	4.35E+05	Calibogue Sound 2	452	1,283	150
Bryan Creek 2	25	1.63E+05	Bryan Creek 1	272	949	150
Savage Creek 1	34	1.07E+06	Bull Creek/Cooper 3	341	2,012	150
			Bull Creek/May River	648	2,012	225
Savage Creek 2	35	3.60E+05	Savage Creek 1	436	1,041	225

TABLE 3-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMM

FOR CALIBOGUE SOUND WATER QUALITY BASINS

Water Quality Basin ID	South WASP Segment	EXISTING LAND USE Flow (cfs)	EXISTING LAND USE Fecal Coliform (#/100 ml)
Calibogue Sound 1	1	13.9	1,088
Calibogue Sound 2	2	17.9	1,093
Calibogue Sound 3	3	7.0	1,068
Calibogue Sound 4	4	11.3	1,076
Calibogue Sound 5	5	10.6	1,056
Broad Creek 1	6	16.7	969
Broad Creek 2	7	29.6	827
Broad Creek 3	8	3.7	1,026
Broad Creek 4	9	4.9	757
Cooper River 1	10	24.6	1,043
Cooper River 2	11	12.8	1,007
Cooper River 3	12	3.3	1,068
Cooper River Trib	13	4.7	834
Bull Creek/Cooper 1	14	5.9	1,066
Bull Creek/Cooper 2	15	2.1	955
Bull Creek/Cooper 3	16	1.8	943
Hoophole Creek	17	3.1	1,022
Old House Creek	18	1.2	1,327
Jarvis Creek 1	19	4.8	1,072
Jarvis Creek 2	20	6.5	909
Skull Creek South 1	21	12.9	1,044
Skull Creek South 2	22	1.8	944
Mackays Creek South	23	3.6	1,198
Bryan Creek 1	24	1.9	896
Bryan Creek 2	25	0.8	945
Savage Creek 1	34	2.0	1,058
Savage Creek 2	35	0.4	1,042

TABLE 3-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES

CALIBOGUE SOUND WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs)
		Existing
Calibogue Sound 1	Ocean	1,722
Calibogue Sound 2	Calibogue Sound 1	1,513
May River 1	Calibogue Sound 2	17
Calibogue Sound 3	Calibogue Sound 2	1,474
Calibogue Sound 4	Calibogue Sound 3	745
Calibogue Sound 5	Calibogue Sound 4	731
Mackays Creek North 2	Calibogue Sound 5	720
Broad Creek 1	Calibogue Sound 1	55
Broad Creek 2	Broad Creek 1	38
Broad Creek 3	Broad Creek 2	8.6
Broad Creek 4	Broad Creek 3	4.9
Cooper River 1	Calibogue Sound 1	140
Cooper River 2	Cooper River 1	21
Cooper River 3	Cooper River 2	3.3
Cooper River Trib	Cooper River 2	4.7
Bull Creek/Cooper 1	Cooper River 1	95
Bull Creek/Cooper 2	Bull Creek/Cooper 1	86
Bull Creek/Cooper 3	Bull Creek/Cooper 2	84
Hoophole Creek	Bull Creek/Cooper 1	3.1
Old House Creek	Calibogue Sound 2	1.2
Jarvis Creek 1	Calibogue Sound 3	11.0
Jarvis Creek 2	Jarvis Creek 1	6.5
Skull Creek South 1	Calibogue Sound 3	710
Skull Creek North 2	Skull Creek South 2	695
Skull Creek South 2	Skull Creek South 1	697
Mackays Creek South	Calibogue Sound 4	3.6
Bryan Creek 1	Calibogue Sound 2	2.7
Bryan Creek 2	Bryan Creek 1	0.8
Savage Creek 1	Bull Creek/Cooper 3	82
Bull Creek/May River	Savage Creek 1	80
Savage Creek 2	Savage Creek 1	0.4

TABLE 3-16

FECAL COLIFORM MODELING RESULTS

CALIBOGUE SOUND WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)	Modeled Level of Service
		Existing	Existing
Calibogue Sound 1	0.5	3.6	A
Calibogue Sound 2	0.5	3.4	A
Calibogue Sound 3	0.5	3.9	A
Calibogue Sound 4	1.0	4.2	A
Calibogue Sound 5	1.0	4.5	A
Broad Creek 1	0.7	6.2	A
Broad Creek 2	1.0	7.7	B
Broad Creek 3	1.0	10.8	D
Broad Creek 4	1.0	22.2	D
Cooper River 1	0.7	5.0	A
Cooper River 2	0.7	5.8	A
Cooper River 3	1.0	5.5	A
Cooper River Trib	1.0	9.0	C
Bull Creek/Cooper 1	1.0	5.2	A
Bull Creek/Cooper 2	1.0	4.8	A
Bull Creek/Cooper 3	1.0	4.7	A
Hoophole Creek	1.0	5.7	A
Old House Creek	1.0	4.3	A
Jarvis Creek 1	2.0	4.8	A
Jarvis Creek 2	2.0	9.6	C
Skull Creek South 1	1.0	3.9	A
Skull Creek South 2	1.0	3.3	A
Mackays Creek South	1.0	7.4	B
Bryan Creek 1	1.0	4.1	A
Bryan Creek 2	1.0	4.4	A
Savage Creek 1	2.0	3.7	A
Savage Creek 2	2.0	3.5	A

NOTE: Water quality basins with lower LOS are highlighted.

Table 3-17 not applicable in the update.

TABLE 3-18 (Updated 2017)

PLANNING LEVEL COST ESTIMATES FOR

CALIBOGUE SOUND WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
MS_M-1 *	Road overtopping at Masters Drive Replace existing 3 - 18" RCP with 10 - 36" RCP	$129,000
WP_M-2 *	Road overtopping at Bayley Road Replace existing 3 - 24" RCP with 3 - 4'x4' box culverts	$152,000
WP_M-3 *	Road overtopping at Colleton River Drive Replace existing 3 - 18" RCP with 1 - 7'x4' box culverts Raise road 2.9 ft (length of 660 ft)	$805,000
WT_M-2	Road overtopping at Cooper River Landing Road Replace existing 1 - 30" RCP with 4 - 8'x5' box culverts Raise road 2.4 ft (length of 670 ft)	$520,000
WT_M-4	Road overtopping at Freeport Road Replace existing 1 - 18" CMP with 20 - 36" RCP Raise road 1.4 ft (length of 640 ft)	$352,000
	TOTAL	$1,958,000

* Conduits marked by asterisk are on private land

Costs are in January 2018 dollars.

See Appendix for basis of cost estimates.

Figure 3-1

Calibogue Sound Watershed

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Figure 3-2. Hydrologic Model Subbasins

Calibogue Sound Watershed

ATM DESIGN ENGINEERING CONSULTING

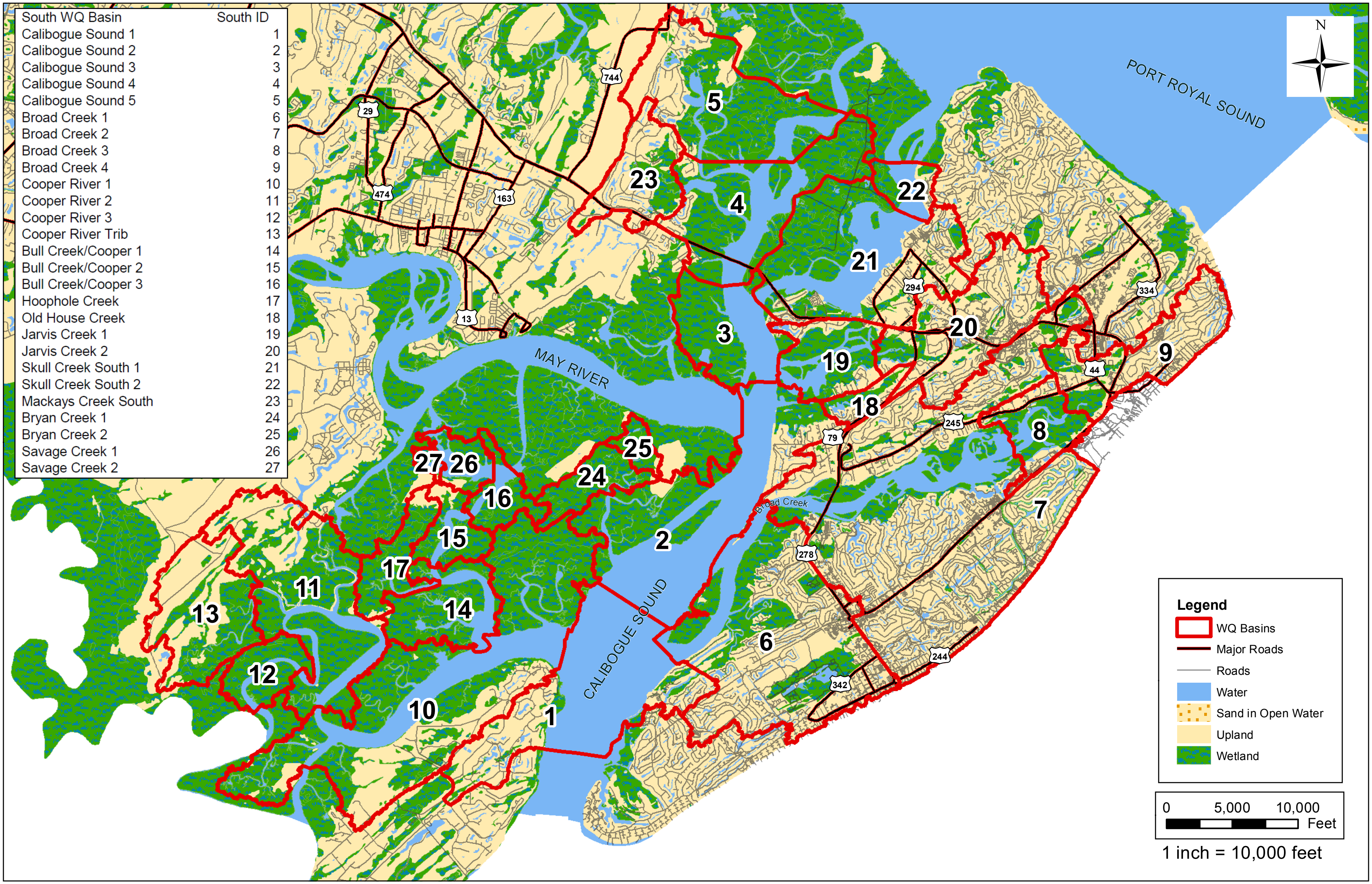
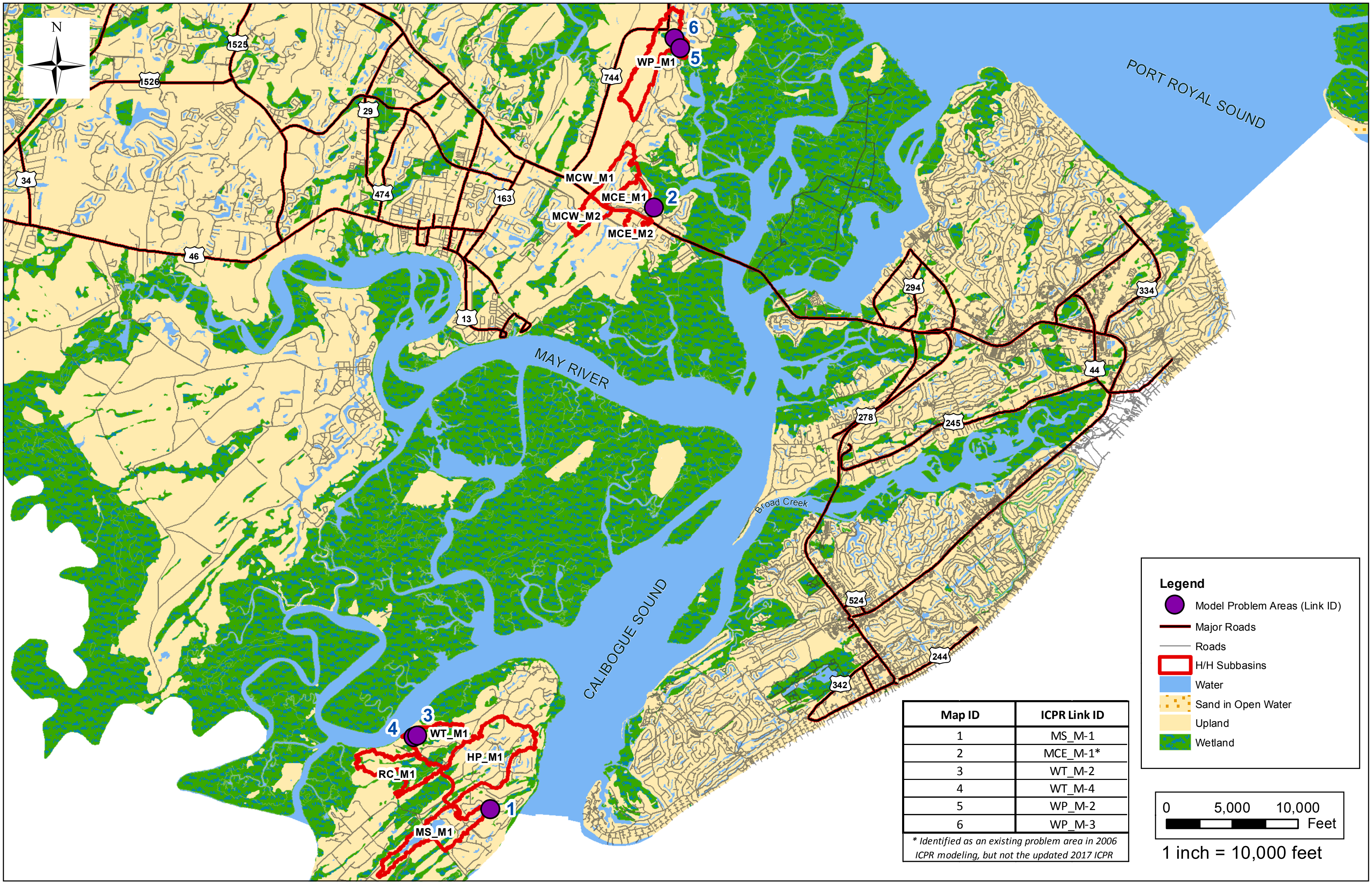


Figure 3-3

Calibogue Sound Watershed
Water Quality Basins

ATM DESIGN ENGINEERING CONSULTING



Map ID	ICPR Link ID
1	MS_M-1
2	MCE_M-1*
3	WT_M-2
4	WT_M-4
5	WP_M-2
6	WP_M-3

** Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR*

Figure 3-4. ICPR Identified PSMS Overtopping Problem Areas

Calibogue Sound Watershed

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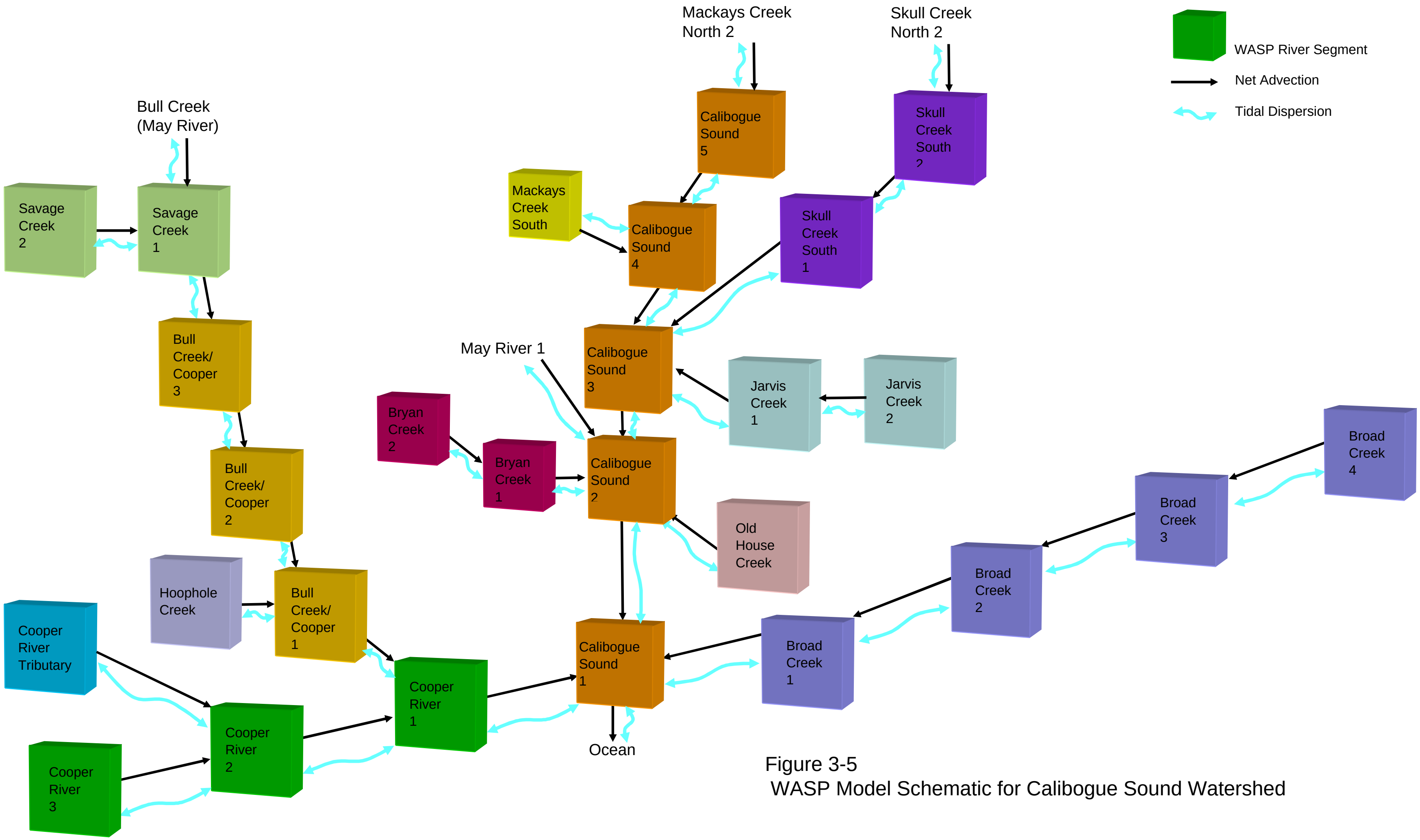

Figure 3-5
WASP Model Schematic for Calibogue Sound Watershed

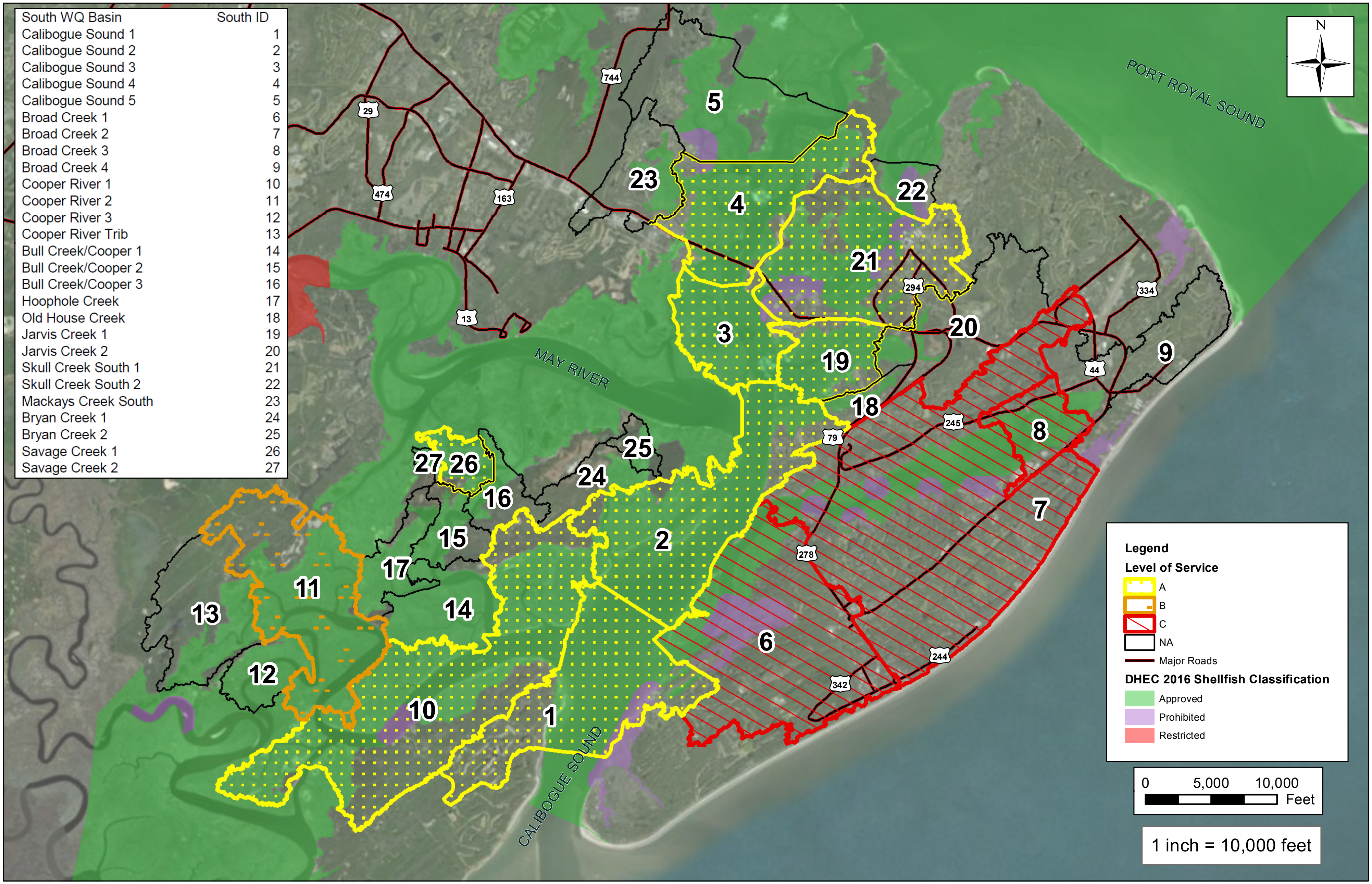

Figure 3-6

Calibogue Sound Watershed
Shellfish Classification and Existing Level of Service

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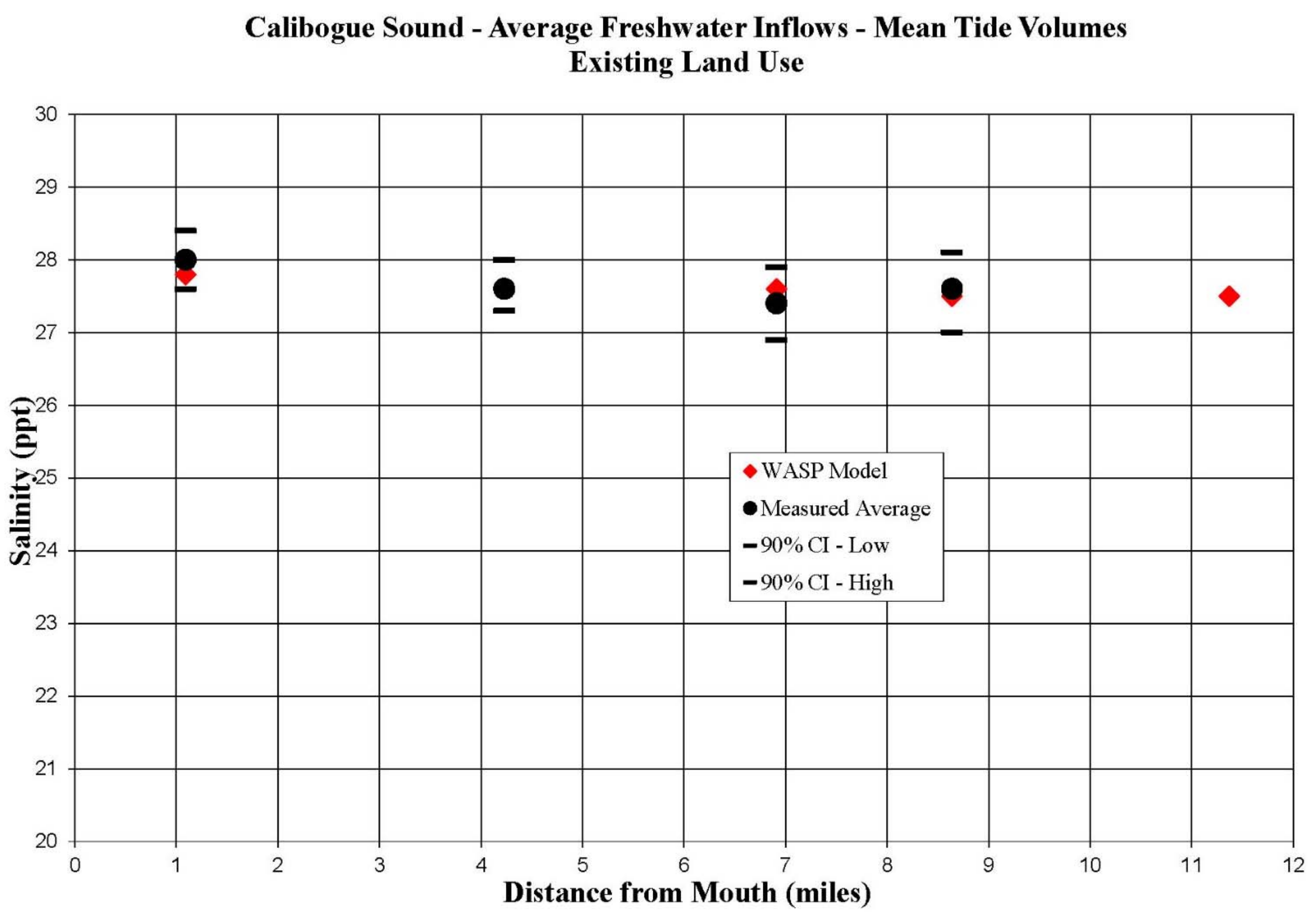

Figure 3-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Calibogue Sound - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

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ENGINEERING
CONSULTING

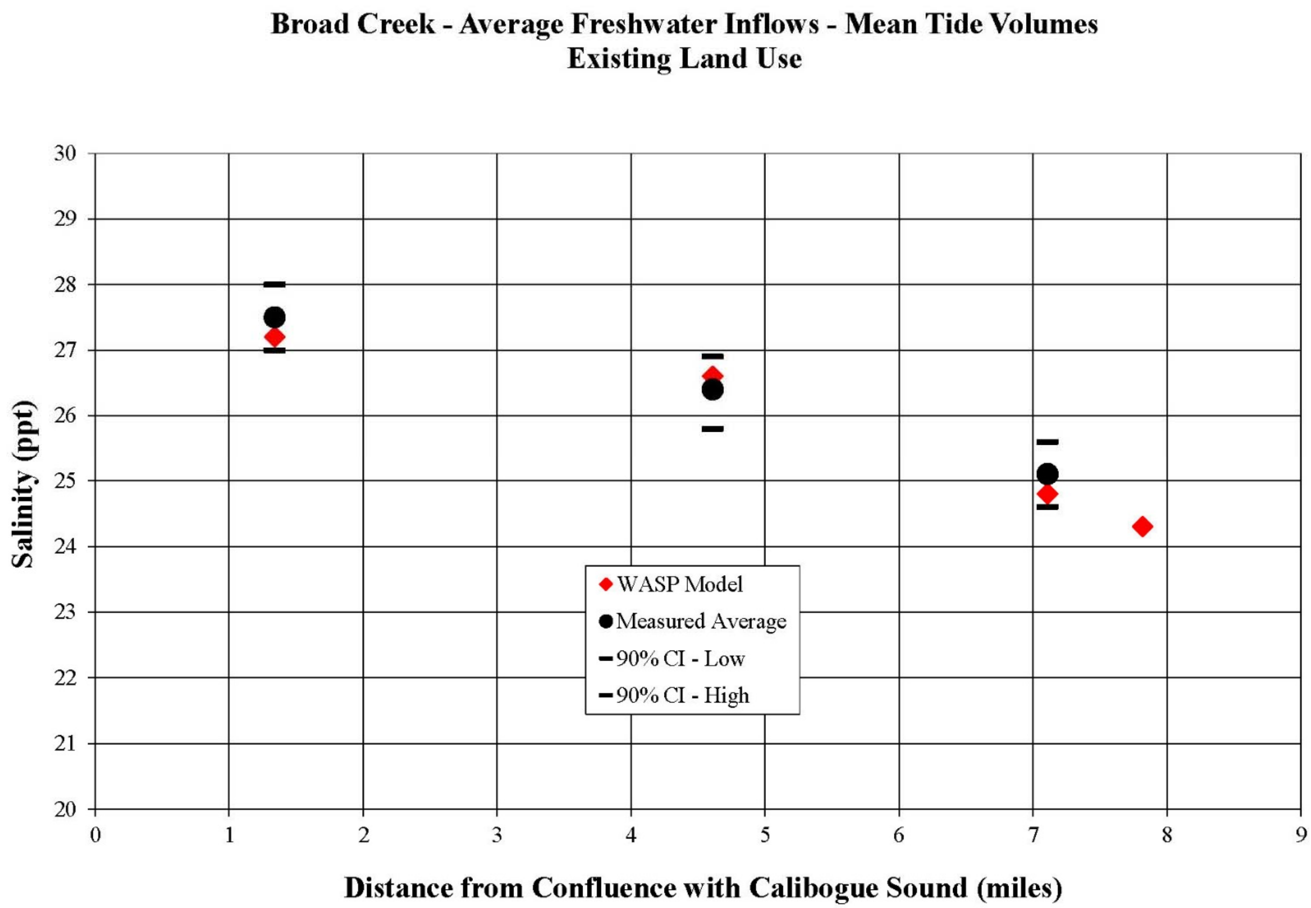


Figure 3-8. Comparison of WASP Model Results with Long-Term Monitoring Data in Broad Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

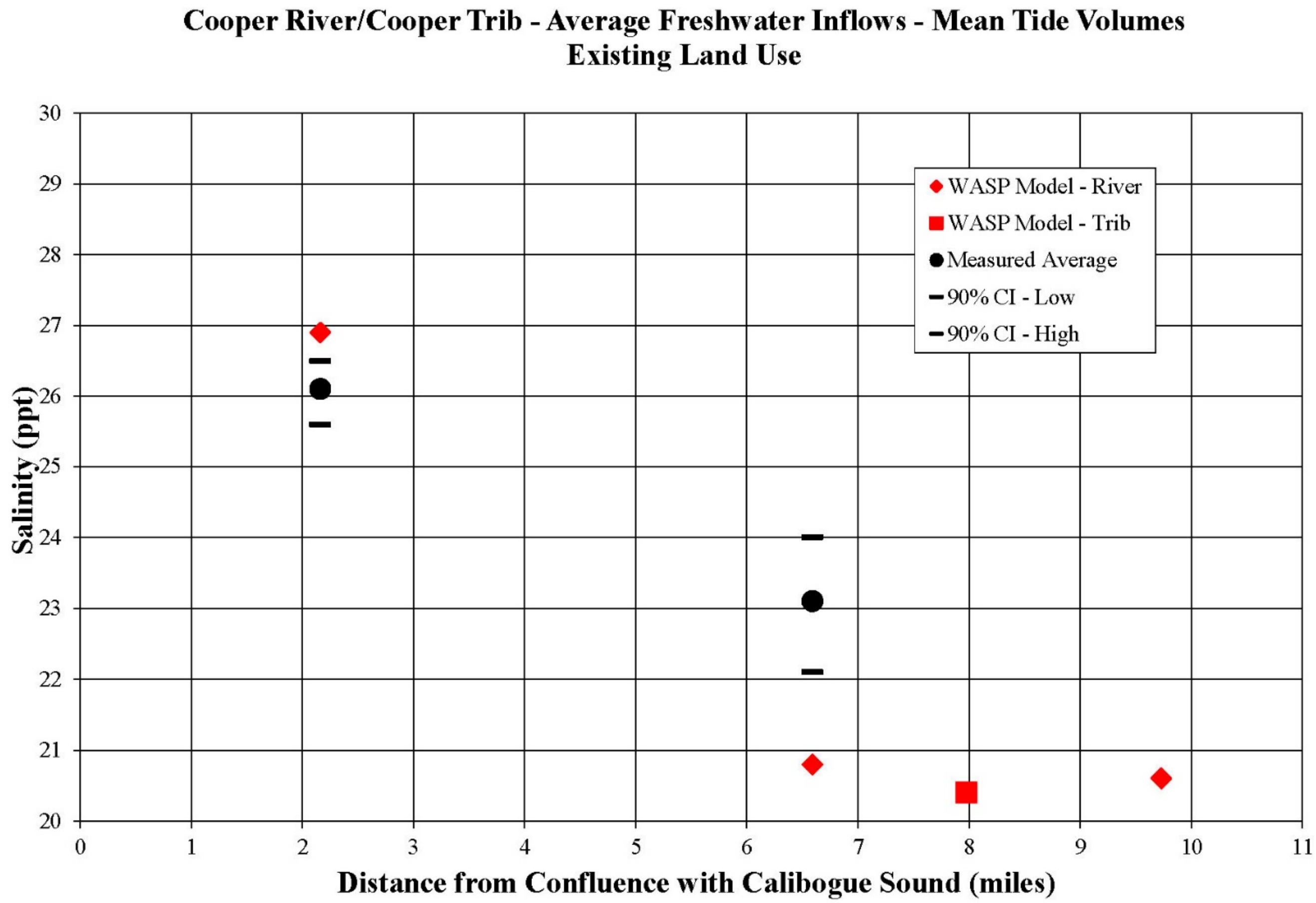


Figure 3-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Cooper River - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

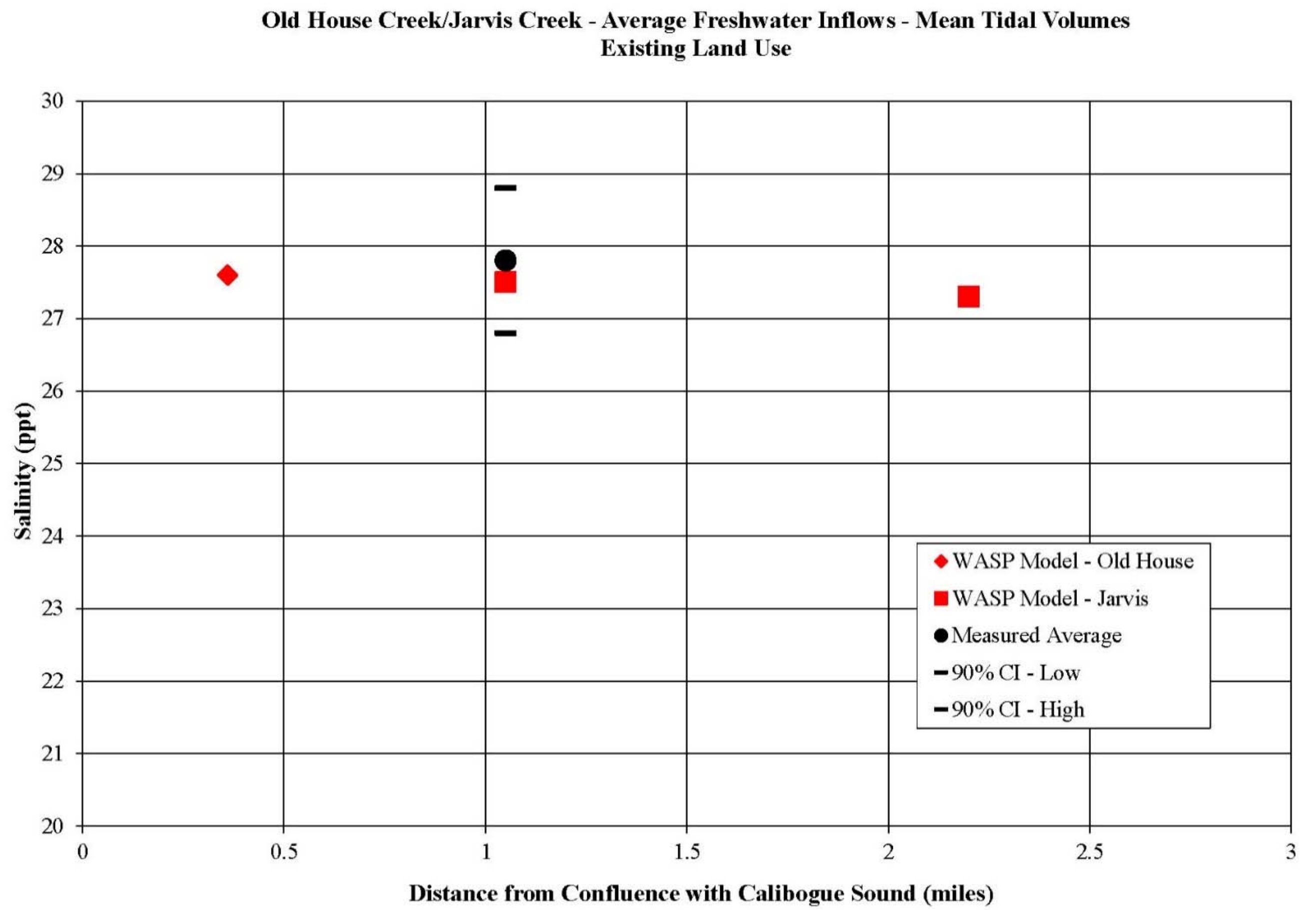


Figure 3-10. Comparison of WASP Model Results with Long-Term Monitoring Data in Old House and Jarvis Creeks - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

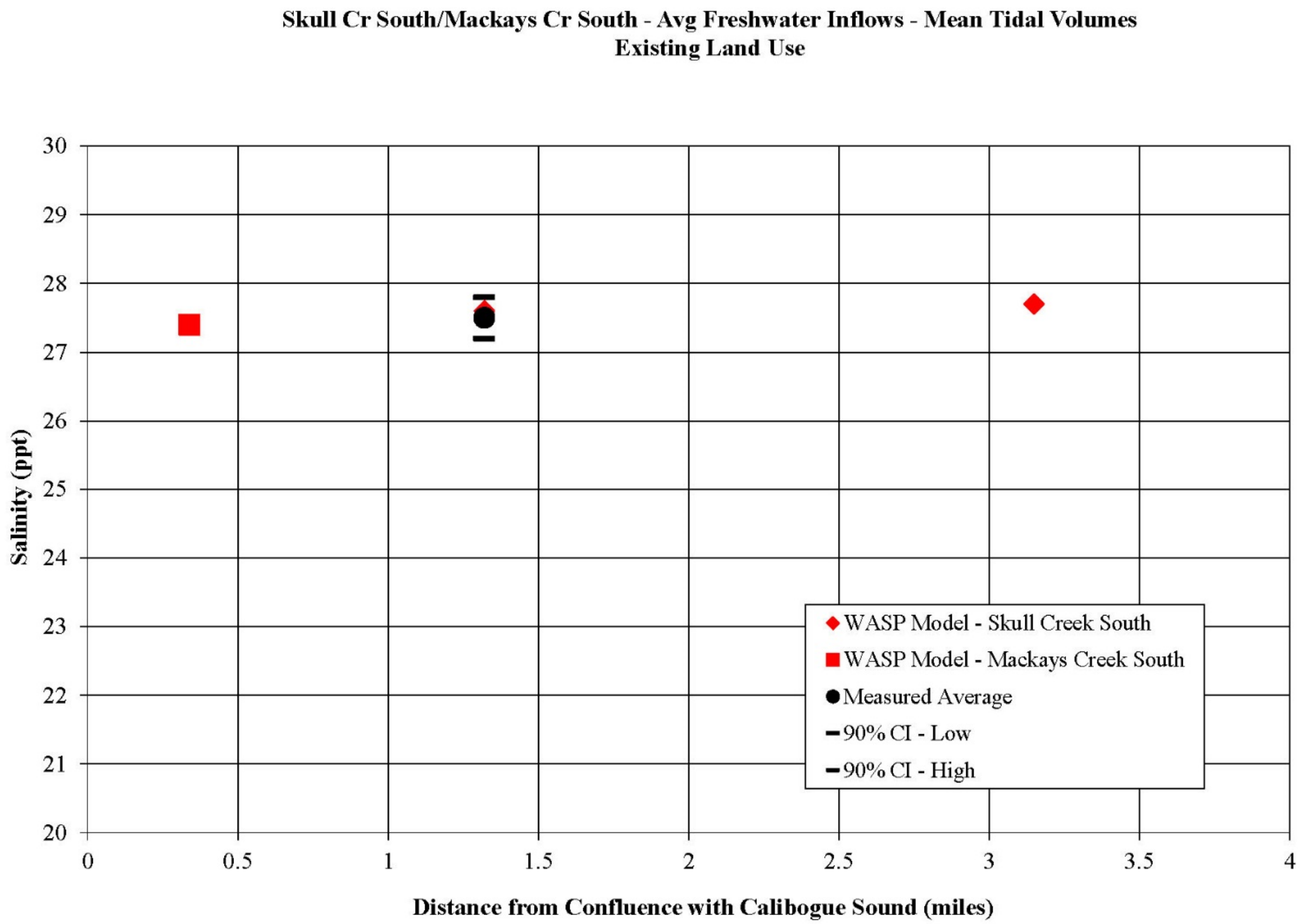


Figure 3-11. Comparison of WASP Model Results with Long-Term Monitoring Data in Skull Creek South - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

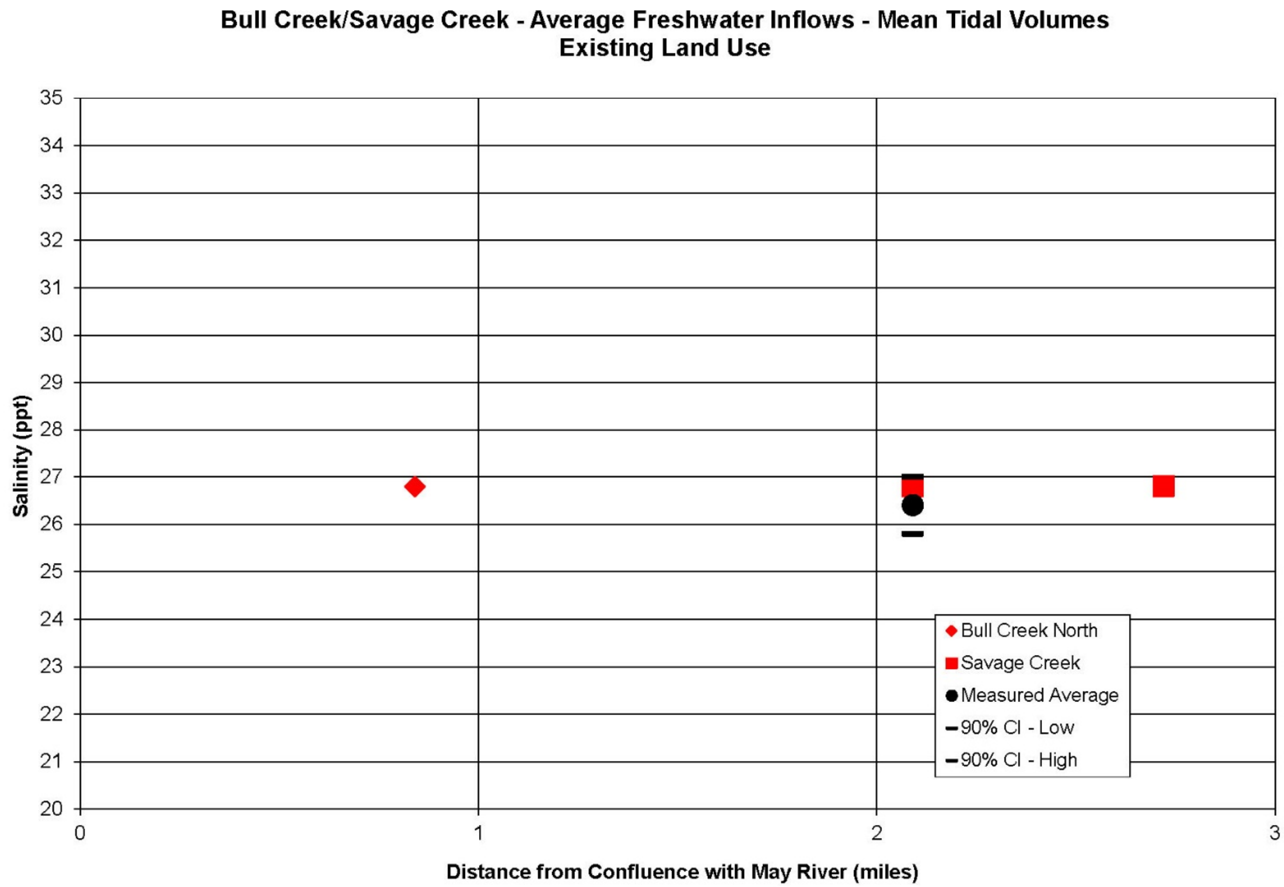


Figure 3-12. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull Creek and Savage Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

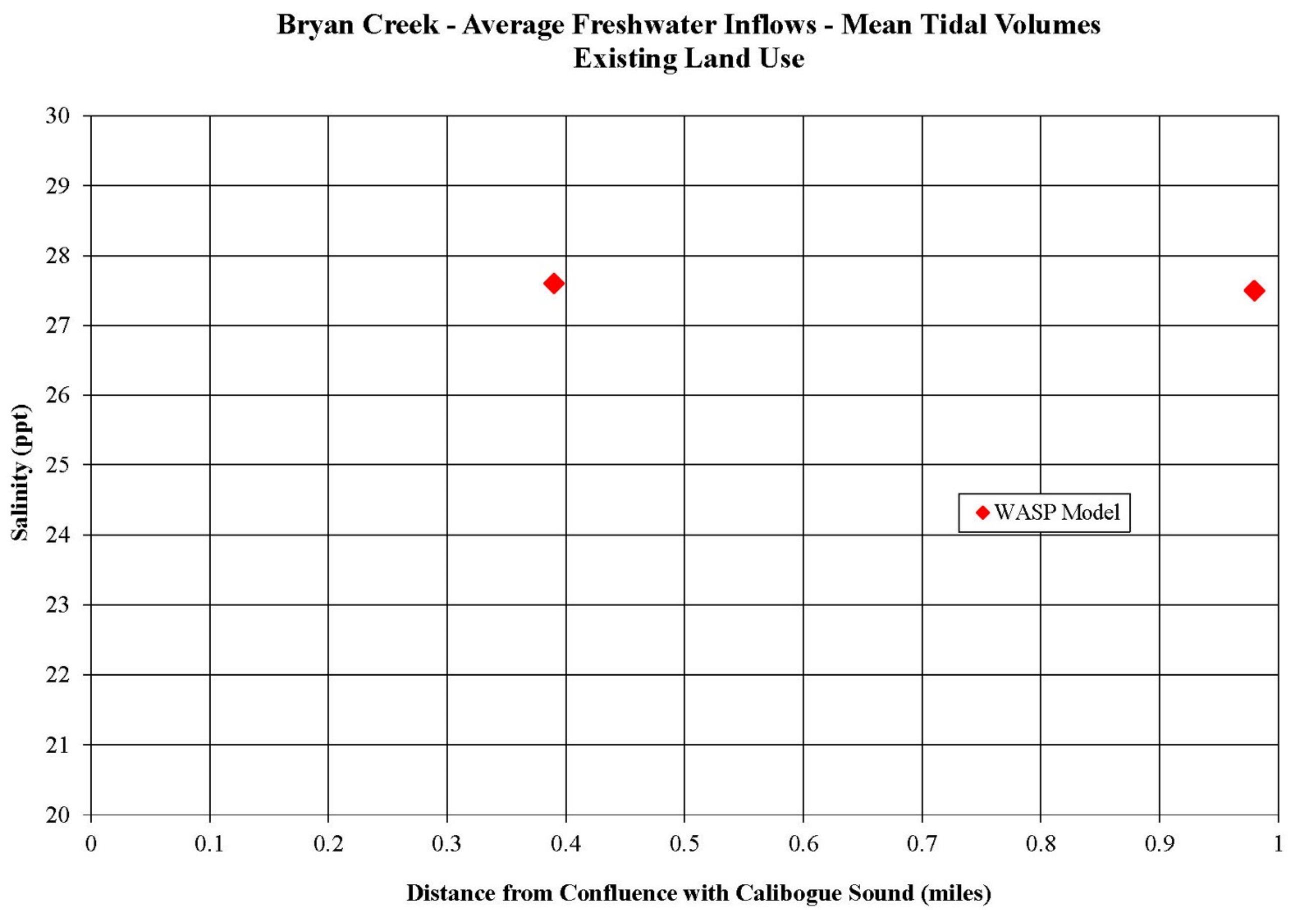


Figure 3-13. Comparison of WASP Model Results with Long-Term Monitoring Data in Bryan Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

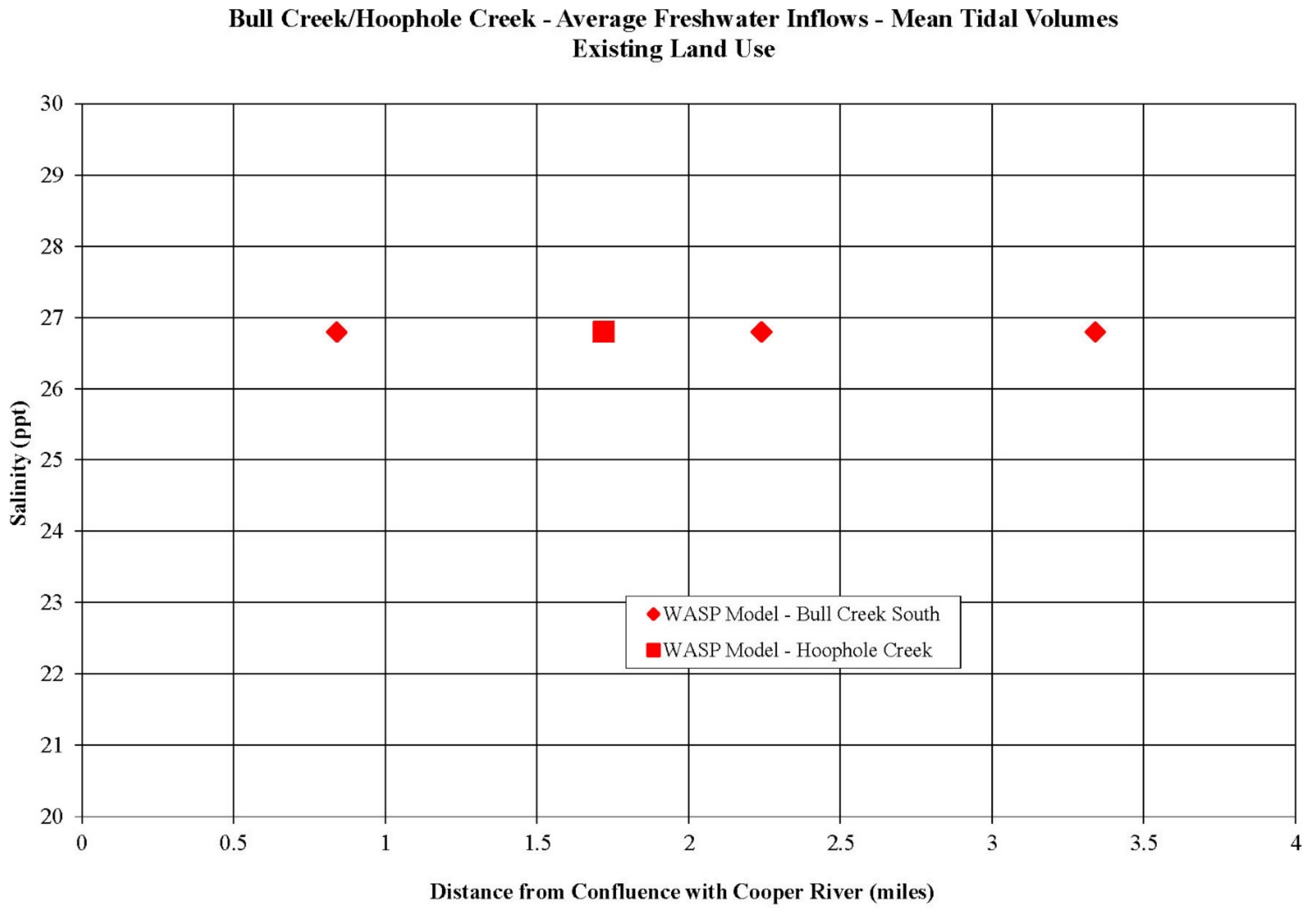


Figure 3-14. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull Creek and Hoophole Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

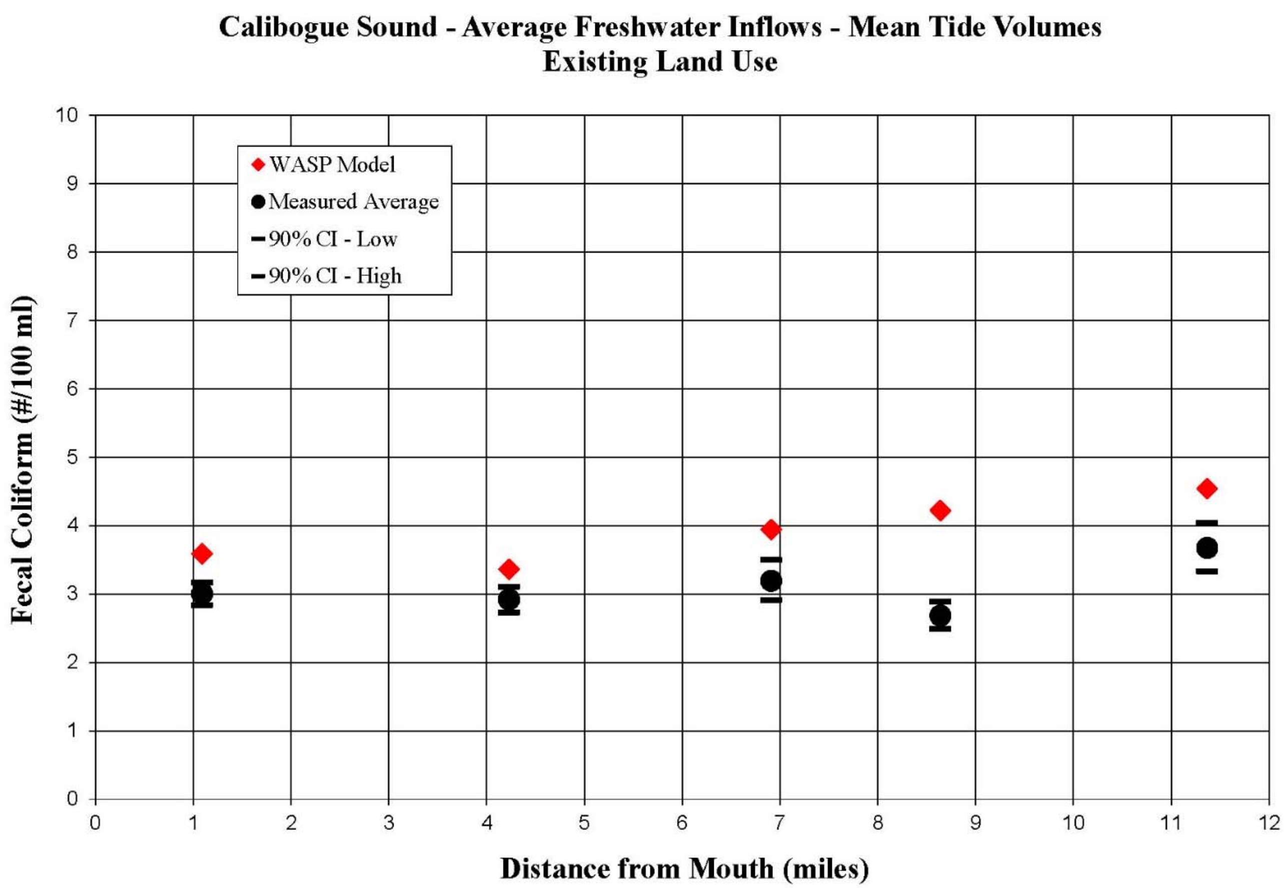


Figure 3-15. Comparison of WASP Model Results with Long-Term Monitoring Data in Calibogue Sound - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

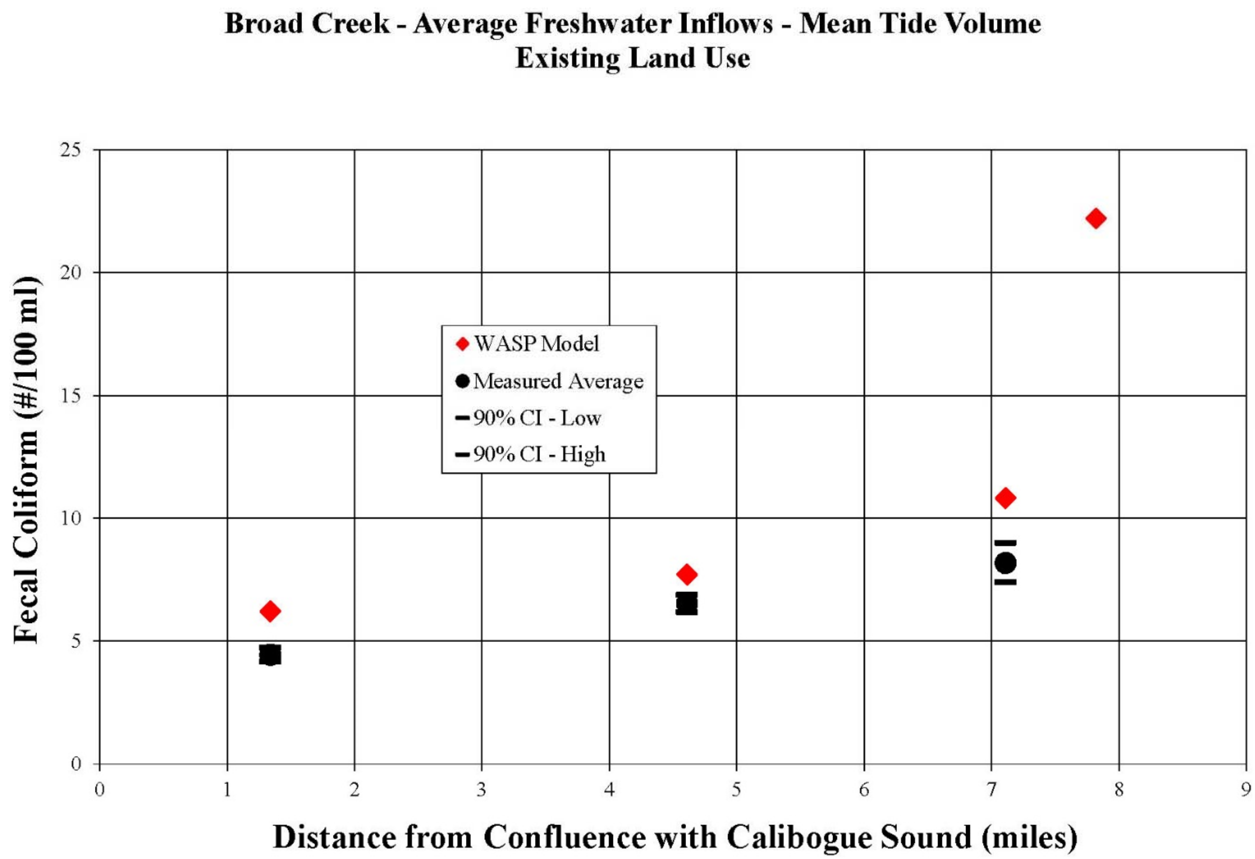


Figure 3-16. Comparison of WASP Model Results with Long-Term Monitoring Data in Broad Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

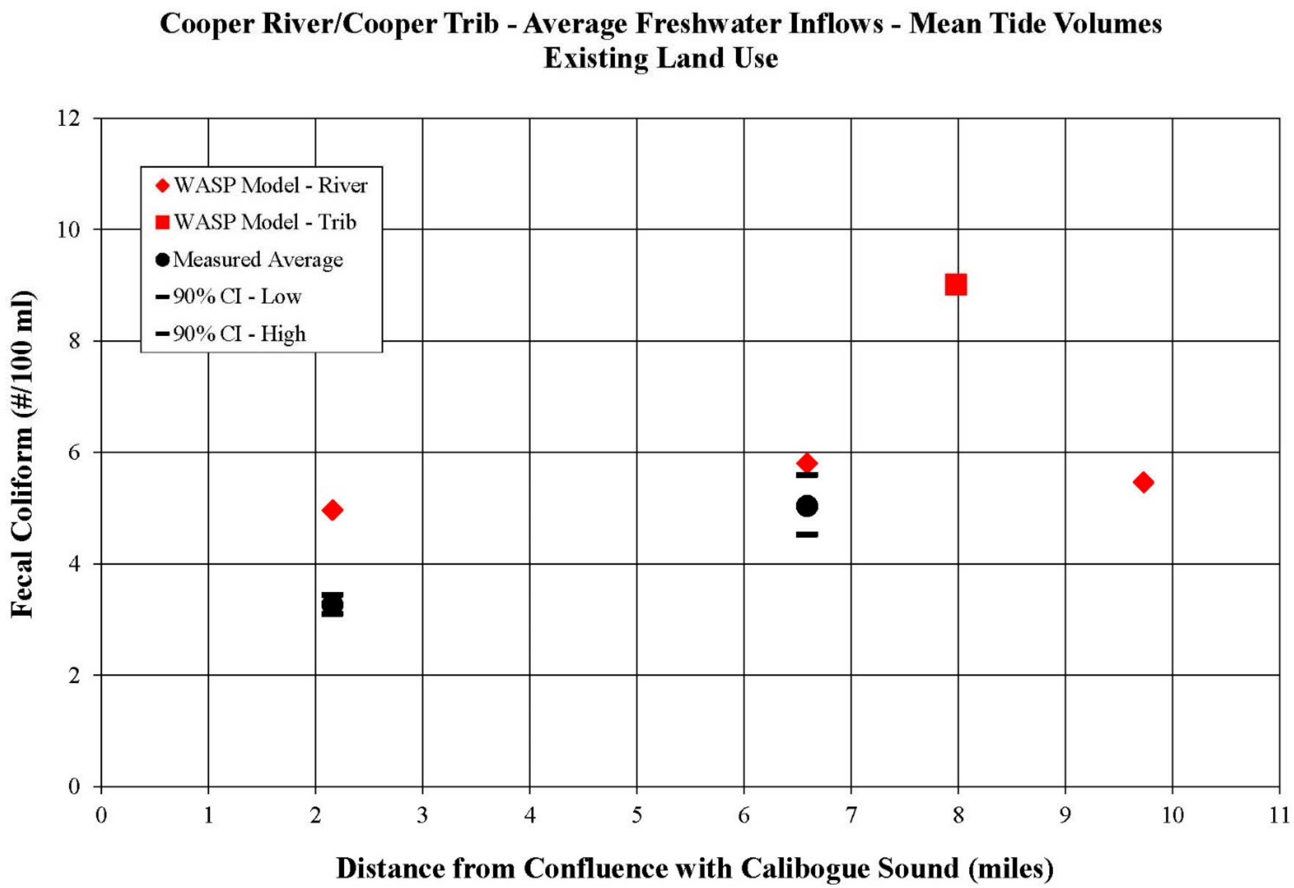


Figure 3-17. Comparison of WASP Model Results with Long-Term Monitoring Data in Cooper River - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

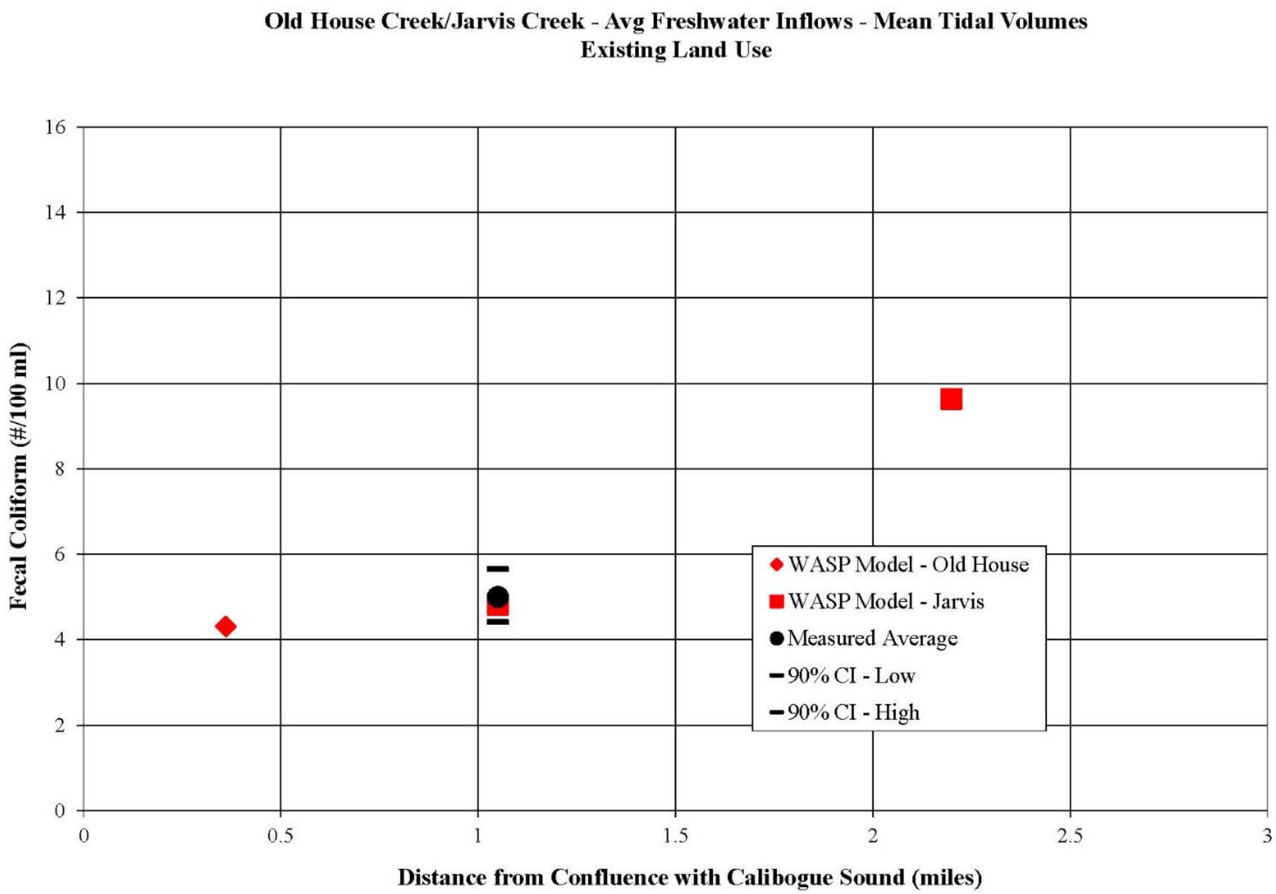


Figure 3-18. Comparison of WASP Model Results with Long-Term Monitoring Data in Old House and Jarvis Creeks - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

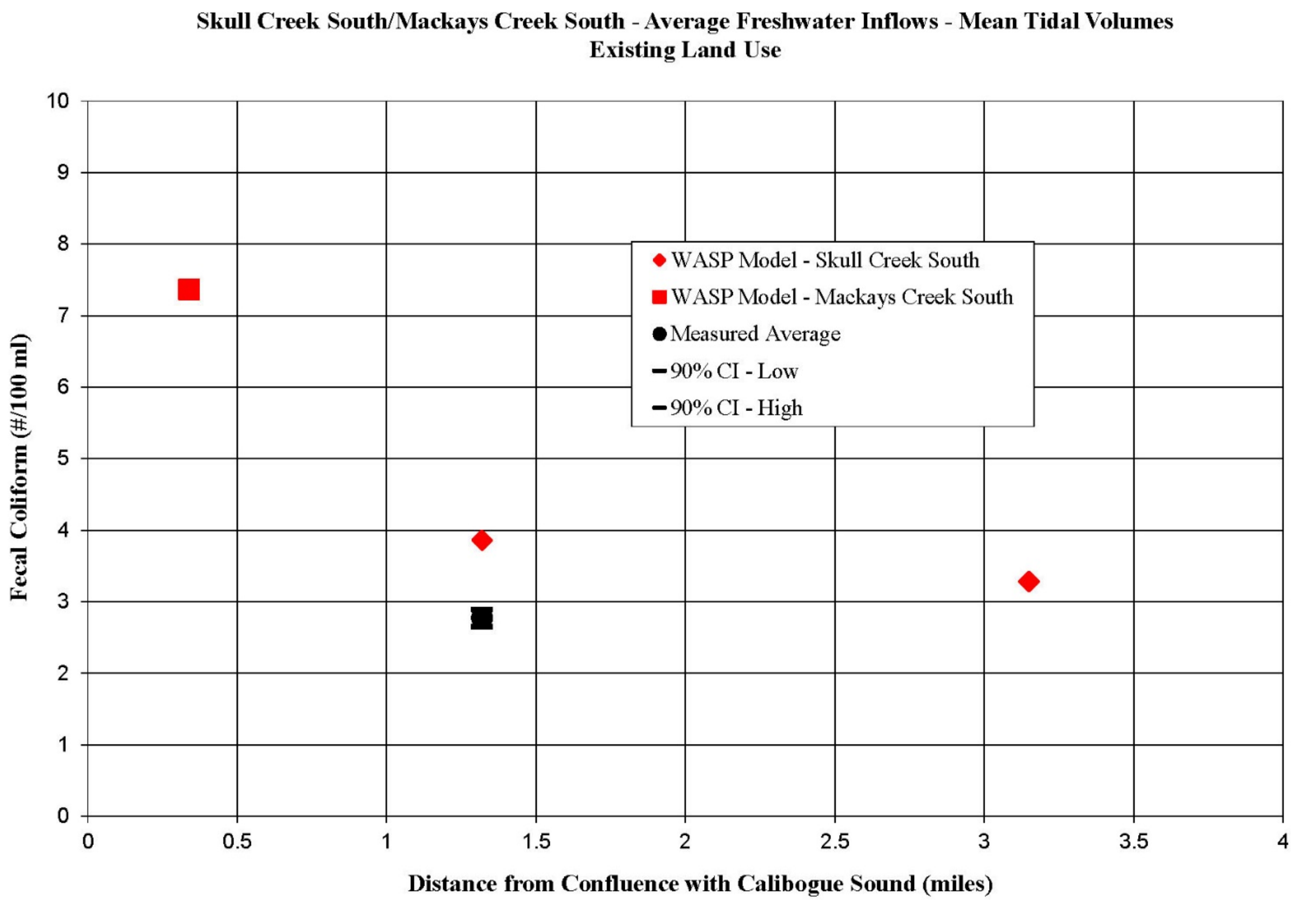


Figure 3-19. Comparison of WASP Model Results with Long-Term Monitoring Data in Skull Creek South - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

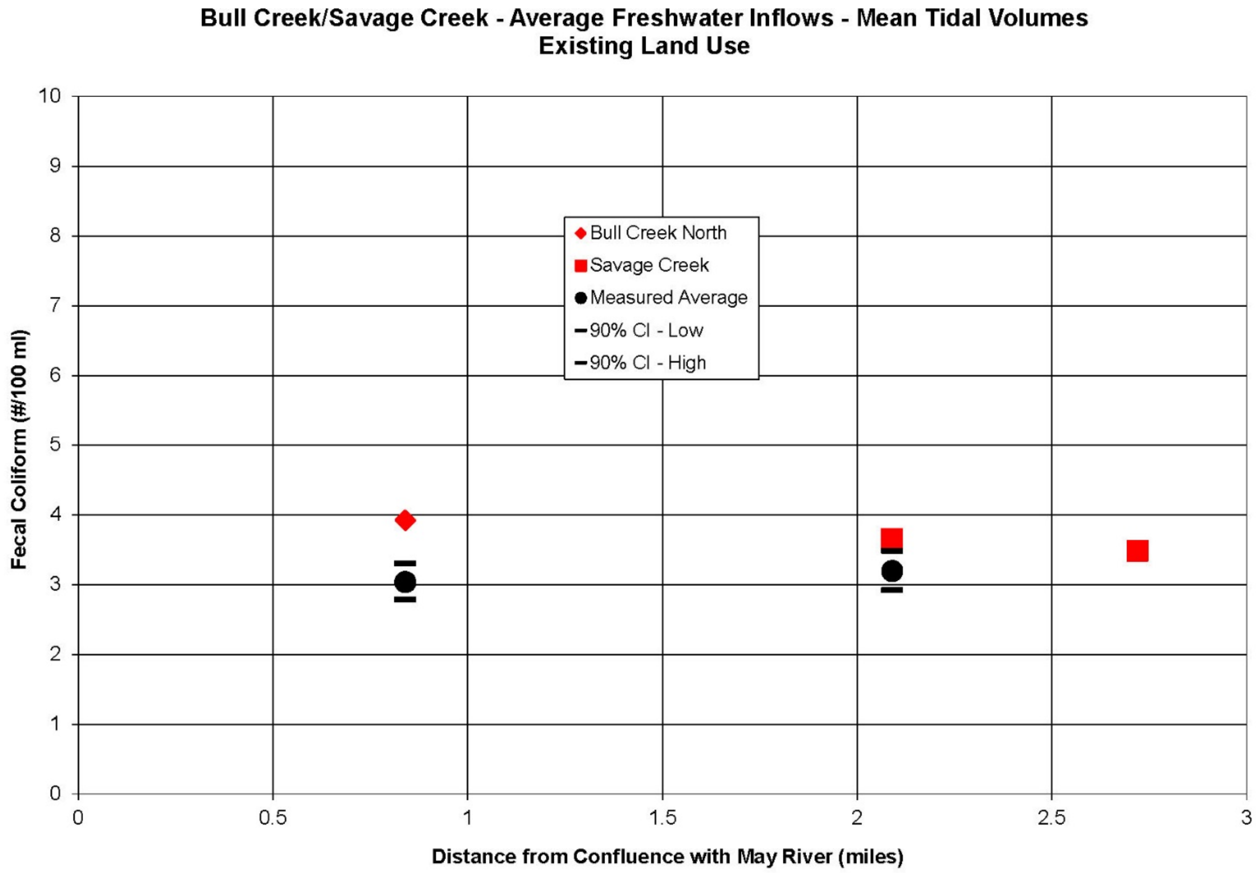


Figure 3-20. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull Creek and Savage Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

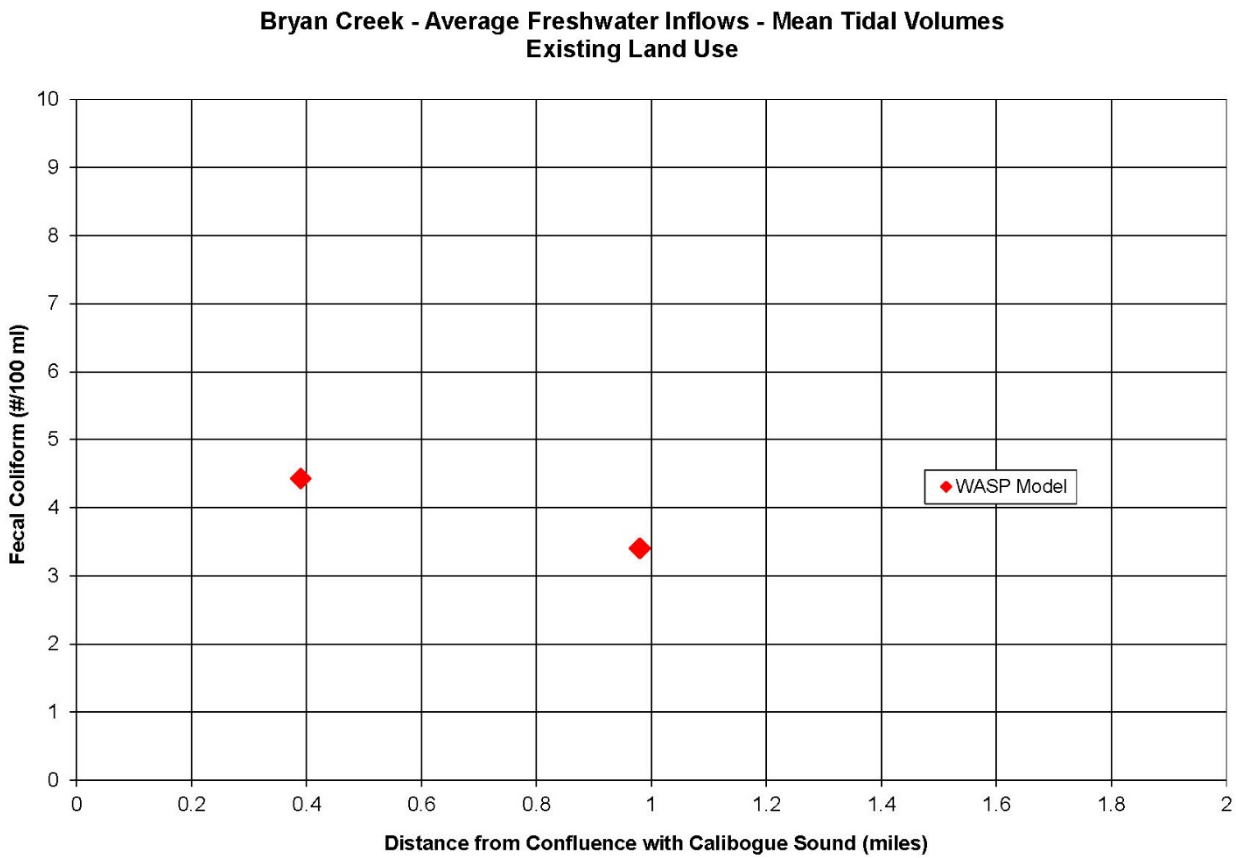


Figure 3-21. Comparison of WASP Model Results with Long-Term Monitoring Data in Bryan Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

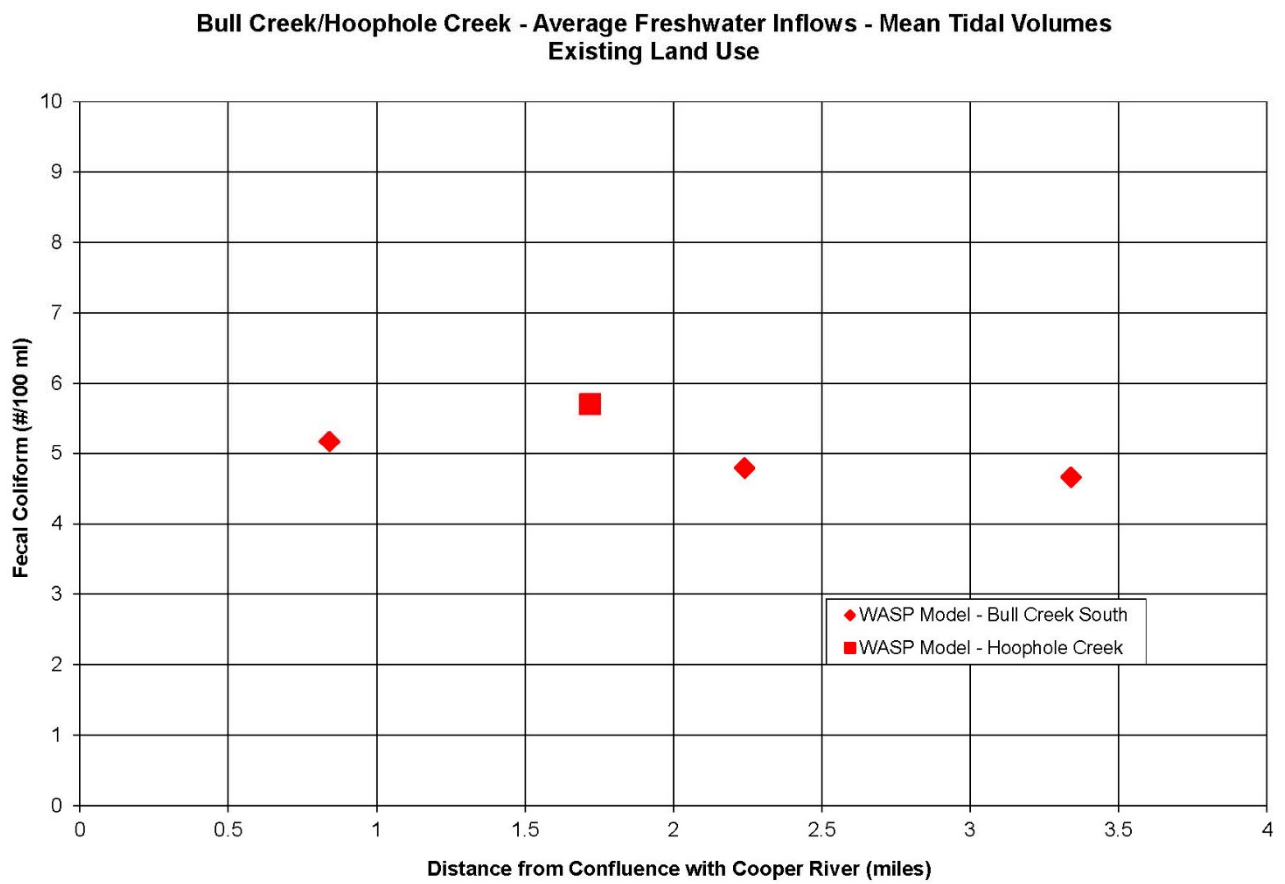
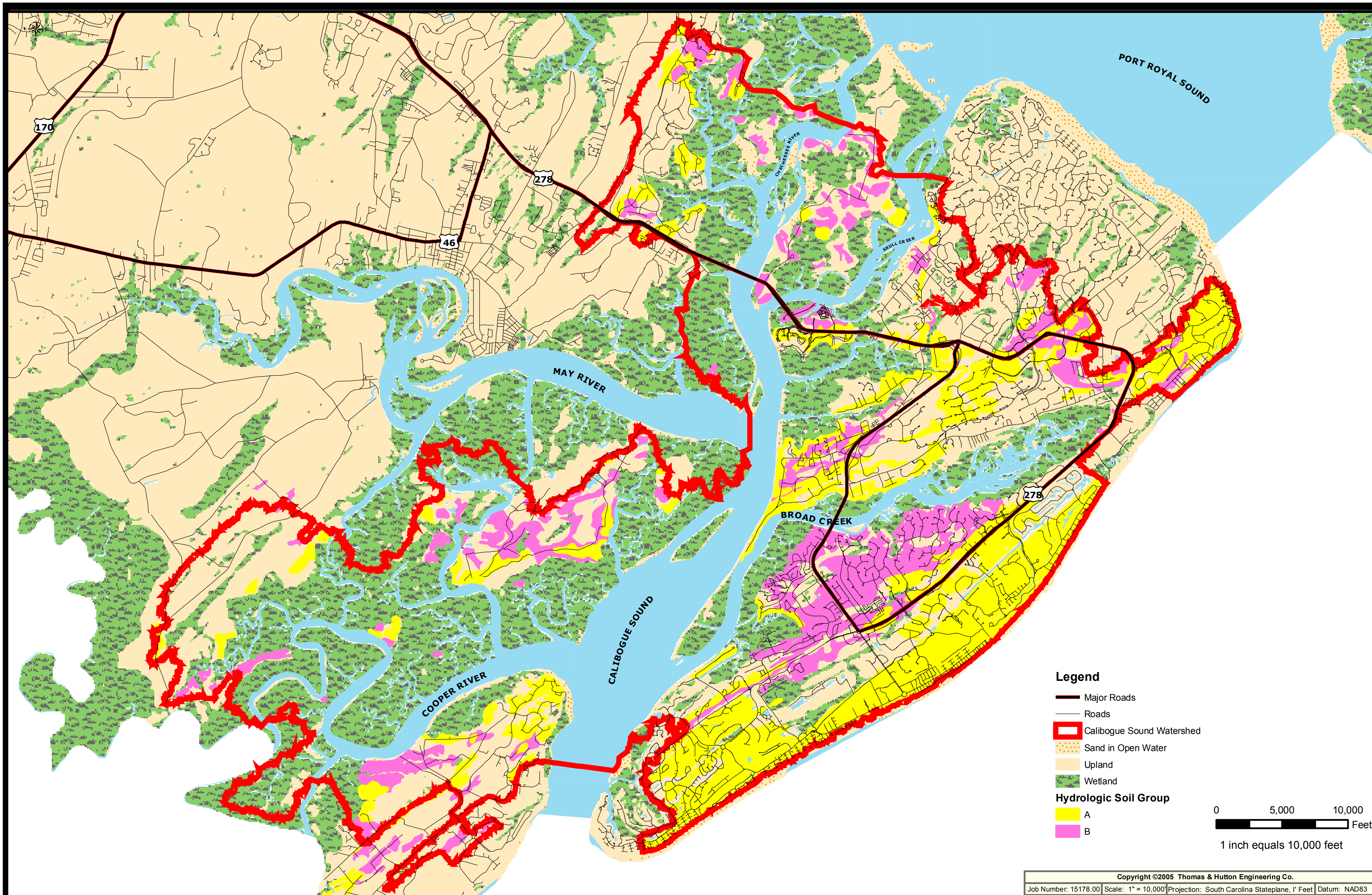


Figure 3-22. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull Creek and Hoophole Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Figure 3-23 is not applicable in the update.



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File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\CalibogueSoundA-BSoils_figure3-18.mxd			Vertical Datum: NAVD88

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Calibogue Sound Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

CDM Camp Dresser & McKee Inc.

Figure 3-24

Disclaimer
Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Section 4
May River Watershed Analysis

This section describes the physical features of the May River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

4.1 Overview

The May River watershed is located south of the Broad River (see Figure 4-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in Bluffton Township and the Town of Bluffton that is tributary to the May River. Major May River tributaries included in the analysis are Bull Creek and Bass Creek.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several "hydrologic" basins. These are listed in Table 4-1 and presented in Figure 4-2. Table 4-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were updated to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into "water quality" basins, and the tidal receiving waters were subdivided into receiving water "segments". These are listed in Table 4-2 and presented in Figure 4-3. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were completed to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

4.2 Hydrologic and Hydraulic Analysis

The ICPR, Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the May River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP. It was determined that the future analysis previously assumed has not yet been reached for most watersheds.

4.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each May River basin consisted of one or more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 4-3 lists the hydrologic parameter values for the May River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the May River PSMS basins is presented in Table 4-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 4-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

4.2.2 Model Results

Tables in Appendix B list the summary of the results of the updated study including Updated Areas and CNs for the May River subbasins.

For existing land use, aerial maps generated in the summer of 2016 and local information were used to estimate the percentage of existing urban
Appendix B also includes tables that list the peak water elevation values for model node locations along the May River PSMS.

Specific problem areas identified by the modeling are listed in Table 4-6 and presented in Figure 4-4. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs, and results showed that the FEMA elevations (based on storm

surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded.

Table 4-6 indicates the road crossings that are being overtopped by the design storm events. Evaluation of solutions to prevent these problems is discussed in the next section of this report.

4.2.3 Management Strategy Alternatives

The problems areas listed in Table 4-6 were evaluated by reviewing the previous report results and reviewing the culverts in the ICPR hydraulic model. In the original 2006 study, the ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 4-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

4.3 Water Quality Analysis

ATM used the WMM and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the May River watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions.

4.3.1 Land Use and BMP Coverage

Table 4-8 presents the existing land use estimates for the May River water quality basins. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 37 percent of the May River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 63 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 12 per cent of the watershed.

Estimates of BMP coverage for existing land use in presented in Table 4-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County and the Town of Bluffton in accordance with respective ordinances and BMP manuals. Values are presented for developed urban land uses. The “total” value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall “total” BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 27 percent of the urban systems in the watershed served by BMPs.

4.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing land use in presented in Table 4-10. The existing land use values reflect areas that are not designated as “sewered” areas by the Beaufort-Jasper Water & Sewer Authority.

Values are presented for developed urban land uses. The “total” value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall “total” septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 53 percent of the urban systems in the watershed (e.g., residential, commercial) are served by septic.

Based on available data, the estimated wastewater discharge under existing conditions is 0.3 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be 0.8 mgd based on increase in residential land between existing and future conditions. There are no direct discharges to receiving waters in the watershed.

4.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the May River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing land use conditions. The results are presented in Table 4-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

For individual water quality basins, the greatest changes in flows and loads occur in the May River 4 and May River 5 basins. This is because these two basins had the greatest amount of development, and because these basins have the smallest fraction of open water and tidal wetland land use.

Wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing discharge of wastewater is limited to roughly 0.3 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be higher (0.8 mgd). Using the values in Table 2-9, the wastewater load for existing conditions accounts for 0.3 to 0.5 percent of the total watershed load for nutrients (TN and TP) and 0.0 to 0.1 percent of the load for other constituents. In the future condition, the wastewater load for existing conditions accounts for 1 to 2 percent of the total watershed load for nutrients (TN and TP) and 0.0 to 0.3 percent of the load for other constituents.

4.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the May River watershed. The model actually includes Calibogue Sound, May River, Colleton River, and Chechessee River watersheds because they are interconnected at several points. Only the May River will be discussed in this section. A schematic of the model is presented as Figure 4-5.

Existing conditions for bacteria concentrations in the May River are presented in Table 4-12. For each water quality basin river reach, the table lists the SCDHEC stations for which the bacteria data were analyzed, the concentrations calculated in the analysis, water quality concentration trends and the LOS associated with these concentrations (as discussed in Section 2.6.2). As shown in the table, SCDHEC data were only available in four of the river model segments. It is noted that May River 4 currently has an LOS of “D” with an increasing trend in bacteria concentration

For informational purposes, Figure 4-6 presents a map of the LOS based on the monitoring data analysis, compared to SCDHEC “shellfish classification” (based on the 2016 SCDHEC reports for shellfish area 19). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the LOS, so there may not be a direct relationship between LOS and shellfish classification presented in the map. In general, however, segments with an “A” LOS are expected to have the lowest probability of receiving a “restricted” classification, and segments with a “D” LOS are expected to have the highest probability of receiving a “restricted” classification.

Physical characteristics assigned to the model reaches are presented in Table 4-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the “characteristic length” (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the SCDHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Table 4-14 presents the values used in the existing condition model.

Flow in the tidal creek headwaters comes primarily from stormwater runoff and baseflow. Moving from the headwaters areas into the downstream tidal segments of the May River, flow to these tidal river segments comes from direct rainfall on the open water and tidal wetlands and the tidal prism, with the tidal prism fluxes being dominant.,. Concentrations remain relatively constant because of the substantial amount of open water/tidal wetland area and the relatively limited development in some basins, as well as the BMPs for new development, which are assumed to have a high level of treatment efficiency.

Table 4-15 shows the net advective flows between segments. The hydrodynamic model (SWMM) indicates that there is a substantial net flow from the May River (May River 2) to Bull Creek. The May River Baseline Study also found this flow pattern from the May River to Bull Creek.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. The calibrated loss-rate coefficients from the 2006 study were used in the updated simulations.

Figure 4-7 is a graph showing a comparison between measured and modeled salinity data along the May River main stem (the only watershed river reaches with monitoring data). The figure shows that the salinity data calculated by the model is very close to the average measured value and is in all cases well within the 90 percent confidence interval of the mean of the salinity data.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration is presented in Figure 4-8. The graph shows very good agreement between the measured values and the model results.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 4-16. The loss rates ranged from 0.5/day to 2.8/day. The lowest values are applied at the downstream end of the May River, and the highest values are applied at the upstream end of the May River. This makes sense if it is presumed that bacteria loss is in part due to light mortality, because the water depths are much greater at the downstream end of the May River, and therefore light would be less of a factor relative to the shallower reaches at the upstream end of May River.

Based on water quality sampling data and model results, the following conclusions are:

- Problem basins include May River 4 and 5. It should be noted that the May River 4 water quality basin still has an LOS “A” designation based modeling. Evaluation of the most recent water quality data however indicates an LOS “D” with an increasing trend in bacteria concentrations.
- The water quality management plan as contained in the May River Watershed Action Plan proposes projects to address these two basins.
- The results of the modeling (WMM and WASP) validates where the focus areas of the May River Watershed Action Plan are. However, the resolution of the models does not allow for a more detailed assessment than what is already occurring as part of the watershed action plan implementation. A number of BMPs, particularly wet detention, have been implemented, with monitoring indicating bacteria concentration reductions at the outfall with increasing bacteria concentrations moving farther downstream from the discharge point. This issue is receiving further attention. It is possible that the conditions of the receiving wetlands and streams where they have been hydrologically altered are serving as a bacteria source. These systems, when healthy possess mechanisms that can reduce bacteria concentrations.

Discussion of water quality related recommendations for monitoring in the May River watershed are presented as part of the overall recommended monitoring program for Beaufort County contained in the Appendix of this report.

4.3.5 Management Strategy Alternatives

The results of the water quality analysis suggest that although a significant number of BMPs have been implemented in the basin for new development and some existing development, the effectiveness of required BMPs in reducing bacteria loads may not be sufficient to maintain the existing high LOS (A) in most of the reaches. In the extreme headwater reach of the May River (May River 5), the LOS is “D” under existing land use conditions. At low tide, this reach is essentially all freshwater, and therefore is not capable of supporting shellfish or other saltwater species. Continued monitoring of the May River 4 tributary inflows and open water is recommended to validate the performance of BMPs for existing and new development in that sensitive segment. This segment currently has an LOS of “A” based on the modeling but statistical analysis of the available data shows an increasing trend in bacteria concentrations which warrants continuing attention.

The graphs show very good agreement between the measured values and the model results for most of the reaches and poor agreement in May River Basin 4. In water quality modeling, most performance metrics indicate a model that predicts a value 45-60% of the observed value is considered fair or satisfactory (Moriasi et. al, 2007, Donigian, 2002). The poor prediction is likely due to how the hydrodynamics of the systems are being modeled. The approach that has been used to date is based on the net flow advection of the various reaches and is a quasi-steady-state approach. This is an acceptable approach in most cases and has utility in this case as it allows for the comparison of water quality management and their effectiveness. However, given the tide range that exists in the county’s receiving waters and the dynamic salinity regimes present, a detailed 3-dimensional hydrodynamic model, such as the Environmental Fluid Dynamics Code (EFDC), is required to adequately simulate the tidal fluctuations and salinity-density gradients that exist in the receiving waters. Development of a 3-D hydrodynamic model would be a significant effort but would provide the proper hydrodynamic foundation for improved water quality predictions. The Town of Bluffton is currently developing detailed stormwater models which could be extended to provide time series of pollutant loads as inputs to the more detailed hydrodynamic and water quality models described above.

For informational purposes, the areas with “A” and “B” type soils are presented in Figure 4-10. In general, these soils are more suitable for infiltration BMPs than areas with “C” and “D” type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

4.4 Planning Level Cost Estimates for Management Alternatives

Table 4-18 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the May River watershed. As shown in the table, the projects are estimated to have a total cost of $1.521 million based on January 2018 dollars. Details of the cost estimate for each project are shown in Appendix B.

TABLE 4-1 (Updated 2017)
HYDROLOGIC BASINS
MAY RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Alljoy Landing	307	1	307
Bluffton East*	524	2	262
Buckingham	539	2	270
Buck Island	326	3	109
Bluffton West	190	3	63
May River	400	1	400
Rose Dhu Creek	3,755	16	235
Stoney Creek	4,935	14	352
Ulmer	506	2	253
TOTAL	11,483	44	261

* ATM Updated Areas (based on updated and improved watershed delineations)

TABLE 4-2
WATER QUALITY BASINS
MAY RIVER WATERSHED

Basin Name	Tributary Area (acres)
May River 1	1,688
May River 2	4,163
May River 3	5,165
May River 4	5,703
May River 5	6,187
Bass Creek	2,186
May River Trib	1,739
Bull Creek	824
TOTAL	27,654

TABLE 4-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
MAY RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use Curve Number	Existing Land Use Time of Concentration (minutes)	Future Land Use Curve Number	Future Land Use Time of Concentration (minutes)
Alljoy Landing Basin					
AL_M1	307	72	168	79	134
Bluffton East Basin					
BE_M1*	292	86	94	87	85
BE_M2	232	90	79	89	75
Buckingham Basin					
BH_M1	241	77	82	78	82
BH_M2	298	82	91	83	89
Buck Island Basin					
BI_M1	47	64	80	71	68
BI_M2	73	80	51	79	51
BI_M3	205	80	137	82	126
Bluffton West Basin					
BW_M1*	277	86	39	74	38
BW_M2	42	87	43	87	43
BW_T1	96	86	77	86	76
May River Basin					
MR_M1	400	73	137	78	115
Rose Dhu Creek Basin					
RDC_M1	329	81	196	71	185
RDC_M2	141	84	130	76	113
RDC_M3A	85	82	52	89	48
RDC_M3B	87	82	52	89	49
RDC_M4	376	78	164	80	145
RDC_M5	270	83	626	83	491
RDC_M6	302	79	151	85	123
RDC_M7	182	79	132	86	113
RDC_M8	32	88	52	87	52
RDC_T1A	232	82	118	80	107
RDC_T1B	54	75	52	84	40
RDC_T2	458	82	176	77	153
RDC_T3A	260	75	138	83	107
RDC_T3B	122	82	116	78	106
RDC_T4	628	82	125	81	99
RDC_T5	198	77	118	83	97
Stoney Creek Basin					
SC_M1	150	71	99	75	82
SC_M2	209	73	146	76	124
SC_M3	245	82	84	88	77
SC_M4	432	82	139	89	122
SC_M5	285	83	141	85	111
SC_T1A	483	78	162	86	143
SC_T1B	273	79	138	82	132
SC_T1C	1,065	79	267	81	230
SC_T1D	349	72	216	71	192
SC_T2	516	81	177	82	160
SC_T3	241	81	109	89	100
SC_T4A	276	81	131	82	105
SC_T4B	111	77	91	80	78
SC_T5	299	83	139	87	120
Ulmer Basin					
U_M1	265	77	98	80	87
U_M2	241	84	90	86	75
Average					

* ATM Updated Areas (based on updated and improved watershed delineations)

TABLE 4-4
HYDRAULIC DATA SUMMARY
MAY RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Alljoy Landing	5	5,641	1	2	0	0	1	0
Bluffton East	4	3,480	2	2	1	1	1	0
Buckingham	8	7,689	2	2	0	2	1	2
Buck Island	5	5,909	2	4	0	0	1	0
Bluffton West	7	3,002	6	6	1	3	1	0
May River	1	508	2	6	0	1	2	0
Rose Dhu Creek	58	55,903	24	65	1	13	42	3
Stoney Creek	59	61,666	2	2	0	3	0	0
Ulmer	3	2,653	3	5	0	1	2	0
TOTAL	150	146,451	44	94	3	24	51	5

TABLE 4-5
CULVERT DATA FOR HYDROLOGIC BASINS
MAY RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Alljoy Landing Basin						
Ulmer Road	AL_M-1A	36"x36"	37	1.7	5.8	25
	1B	30"x30"	37	1.9		
Bluffton East Basin						
Bridge Street	BE_M-1	Bridge	44	0.8	19.4	25
Bruin Road	BE_M-4A	36"x36"	58	13.1	19.0	25
	4B	36"x36"	58	13.2		
Buckingham Basin						
Buckingham Plantation Drive	BH_M-3	48"x48"	230	0.5	8.3	25
Buckingham Plantation Drive	BH_M-5	20"x20"	65	4.7	7.5	25
Buck Island Basin						
May River Road (State Hwy 46)	BI_M-2	60"x60"	40	1.3	13.3	100
Haigler Boulevard	BI_M-4A	48"x48"	65	-0.2	11.6	25
	4B	48"x48"	65	-0.2		
	4C	24"x24"	65	-0.1		
Bluffton West Basin						
Bridge Street	BW_M-1	Bridge	30	0.2	15.0	25
Lawrence Street	BW_M-4	48"x48"	100	2.9	17.6	25
May River Road (State Hwy 46)	BW_M-6	42"x42"	78	13.2	21.2	100
Lawrence Street	BW_T1-3	2 - 18"x18"	60	15.2	20.5	25
Wharf Street	BW_T1-6	30"x30"	54	16.5	21.6	25
May River Road (State Hwy 46)	BW_T1-8	24"x24"	70	18.3	24.3	100
May River Basin						
Palmetto Bluff Road	MR_M-1A	48"x48"	50	-0.8	6.8	25
	1B	48"x48"	50	-0.7		
	1C	36"x36"	50	1.3		
New Palmetto Bluff Road	MR_M-3A	36"x36"	60	3.3	11.5	25
	3B	60"x60"	80	1.7		
	3C	60"x60"	80	1.7		

TABLE 4-5
CULVERT DATA FOR HYDROLOGIC BASINS
MAY RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Rose Dhu Creek Basin						
Windmill Road	RDC_M-2A	144"x90"	35	2.0	11.3	25
	2B	144"x90"	35	2.0		
Sedgewick Avenue	RDC_M-5	2 - 42"x42"	1058	5.0	14.0	25
Farnsleigh Avenue	RDC_M-8A	48"x48"	190	5.0	15.0	25
	8B	48"x48"	203	5.0		
Farm Lake Drive	RDC_M-10	48"x48"	767	7.6	16.6	25
Cattle Run Way	RDC_M-11A	24"x24"	181	7.5	16.1	25
	11B	24"x24"	235	9.0		
Farm Lake Drive	RDC_M-11.1A	48"x48"	522	7.5	16.2	25
	11.1B	30"x30"	392	9.3		
Cattle Run Way	RDC_M-12	36"x36"	331	11.0	16.2	25
Old Bridge Drive	RDC_M-15	2 - 24"x24"	64	13.0	18.0	25
Old Bridge Drive	RDC_M-17	2 - 36"x36"	100	13.2	20.3	25
Hampton Hall Boulevard	RDC_M-23A	42"x42"	70	14.7	21.2	25
	23B	36"x36"	72	17.0		
Buckwalter Parkway	RDC_M-25A	36"x36"	200	17.2	23.3	25
	23B	36"x36"	200	17.2		
	23C	36"x36"	200	17.2		
	23D	36"x36"	200	17.2		
	23E	36"x36"	200	17.2		
	23F	36"x36"	200	17.2		
Hampton Hall Boulevard	RDC_T1-1.1	Bridge	45	5.4	15.8	25
Buckwalter Parkway	RDC_T1-23A	36"x36"	120	16.9	22.2	25
	23B	36"x36"	120	16.9		
	23C	36"x36"	120	16.9		
	23D	36"x36"	120	16.9		
	23E	36"x36"	120	16.9		
	23F	36"x36"	120	16.9		
Farm Lake Drive	RDC_T3-1	48"x48"	375	7.8	17.5	25
Farm Lake Drive	RDC_T3-3	48"x48"	100	10.0	21.8	25
Unknown (The Farm)	RDC_T3-4	48"x48"	350	16.0	23.0	25

TABLE 4-5
CULVERT DATA FOR HYDROLOGIC BASINS
MAY RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Farm Lake Drive	RDC_T3-6	48"x48"	116	18.1	24.1	25
Buckwalter Parkway	RDC_T3-8	60"x60"	160	18.0	24.6	25
Unknown (Pine Ridge)	RDC_T3-11	42"x42"	450	15.5	24.0	25
Unknown (Pine Ridge)	RDC_T3-14	42"x42"	530	19.5	23.0	25
Hampton Hall Boulevard	RDC_T6-2	36"x36"	46	14.0	19.6	25
Farnsleigh Avenue	RDC_T6-4	36"x36"	44	15.0	19.5	25
Buckwalter Parkway	RDC_T7-1	24"x24"	750	21.0	28.0	25
Buckwalter Parkway	RDC_T9-3	36"x36"	350	20.5	24.0	25
Stoney Creek Basin						
May River Road (State Hwy 46)	SC_T1-4	72"x48"	30	-0.8	18.1	100
Old Miller Road	SC_T6-2	42"x42"	70	7.3	15.0	25
Ulmer Basin						
Alljoy Road	U_M-1	48"x48"	140	5.3	15.3	25
Confederate Avenue	U_M-3A	36"x36"	40	10.2	15.5	25
	3B	36"x36"	40	10.2		
Ulmer Road	U_M-6A	36"x36"	40	12.6	16.8	25
	6B	36"x36"	40	12.7		

TABLE 4-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
MAY RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Alljoy Landing Basin					
Ulmer Road	AL_M-1	5.8	5.8	2	6.8
				10	6.8
				25	6.8
				100	6.8
Bluffton East Basin					
Bruin Road	BE_M-21	19.0	19.0	2	19.5
				10	19.6
				25	19.6
				100	19.7
Buckingham Basin					
No Overtopping Identified					
Buck Island Basin					
No Overtopping Identified					
Bluffton West Basin					
No Road Crossing	BW_T1-18	N/A	21.2 (Top of Channel Bank)	2	21.5
				10	21.6
				25	21.6
				100	21.7
May River Basin					
Palmetto Bluff Road	MR_M-1	6.8	6.8	10	7.0
				25	7.1
				100	7.3
Rose Dhu Creek Basin					
No Road Crossing	RDC_M-46	N/A	14.0 (Top of Lagoon Bank)	10	14.2
				25	14.6
				100	15.3
Farnsleigh Avenue	RDC_M-60	15.0	15.0 (Top of Lagoon Bank)	100	15.3
Cattle Run Way	RDC_M-95	16.2	16.2	100	16.5
Old Bridge Drive	RDC_M-111	18.0	17.5 (Top of Channel Bank)	100	17.5
Location Unknown	RDC_T3-59	23	23.0	100	23.1
No Road Crossing	RDC_T3-60	N/A	23.0 (Top of Lagoon Bank)	100	23.1
Stoney Creek Basin					
May River Road (State Hwy 46)	SC_T1-34	18.1	18.1	100	18.2
No Road Crossing	SC_T1-44	N/A	18.1 (Top of Channel)	25	18.2
				100	18.2
No Road Crossing	SC_T1-68	N/A	18.1 (Top of Channel)	25	18.2
				100	18.2
No Road Crossing	SC_T1-212	N/A	25.1 (Top of Channel)	10	25.3
				25	25.4
				100	25.9
Ulmer Basin					
Alljoy Road	U_M-2	15.3	15.3	10	15.5
				25	15.8
				100	16.0
Confederate Avenue	U_M-13	15.5	15.5	10	16.0
				25	16.1
				100	16.4

TABLE 4-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
MAY RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Alljoy Landing Basin			
Ulmer Road	AL_M-1A 1B	36"x36" 30"x30"	Raise road from elevation 5.8 to elevation 7.6 NAVD (length of 1,200 ft), Replace culverts with one 8 ft by 4 ft box culvert
Bluffton East Basin			
Bruin Road	BE_M-4A 4B	36"x36" 36"x36"	Replace culverts with two 5 ft by 5 ft box culverts and set box culvert inverts to match U/S & D/S channel inverts
Buckingham Basin			
No improvements required			
Buck Island Basin			
No improvements required			
Bluffton West Basin			
No improvements required			
May River Basin			
Palmetto Bluff Road	MR_M-1A 1B 1C	48"x48" 48"x48" 36"x36"	Add two 48-inch RCP culverts to existing culverts
Rose Dhu Creek Basin			
No improvements required			
Stoney Creek Basin			
May River Road (State Hwy 46)	SC_T1-4	72"x48"	Raise road from elevation 18.1 ft to elevation 18.3 ft NAVD
Ulmer Basin			
Alljoy Road	U_M-1	48"x48"	Replace culvert with one 5 ft by 5 ft box culvert
Confederate Avenue	U_M-3A 3B	36"x36" 36"x36"	Replace culverts with two 8 ft by 4 ft box culverts

TABLE 4-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
MAY RIVER WATERSHED

Land Use Type	Bass Creek (Acres)	Bull Creek (May) (Acres)	May River 1 (Acres)	May River 2 (Acres)	May River 3 (Acres)	May River 4 (Acres)	May River 5 (Acres)	May River Trib (Acres)	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	0	0	8	0	8
Commercial	61	0	0	0	207	55	130	17	471
Forest/Rural Open	294	0	108	539	833	1545	1950	301	5569
Golf Course	436	0	0	43	0	61	0	0	540
High Density Residential	57	0	0	163	366	1377	543	36	2541
Industrial	40	0	0	57	306	213	229	0	845
Institutional	1	0	0	0	49	118	1	9	178
Low Density Residential	4	0	0	332	971	891	652	0	2849
Medium Density Residential	33	0	0	338	561	200	769	274	2174
Open Water/Tidal	1215	645	1469	2634	1567	734	494	1006	9764
Silviculture	0	0	0	0	37	146	578	0	761
Urban Open	1	41	32	45	190	282	180	17	787
Wetland/Water	45	138	91	202	186	237	492	78	1470
TOTAL	2186	824	1701	4352	5273	5858	6025	1738	27958
Urban Imperviousness (%)	6%	0%	0%	6%	16%	19%	14%	6%	12%

TABLE 4-9
WATER QUALITY BASIN BMP COVERAGE
MAY RIVER WATERSHED

Land Use Type	Bass Creek	Bull Creek (May)	May Rive 1	May River 2	May River 3	May River 4	May River 5	May River Trib	TOTAL
Commercial	3.4%	0.0%	0.0%	0.0%	11.4%	3.1%	85.3%	46.8%	31.1%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	38.6%	0.0%	0.0%	17.6%
High Density Residential	0.0%	0.0%	0.0%	73.6%	14.6%	52.1%	100.0%	100.0%	44.5%
Industrial	0.0%	0.0%	0.0%	0.0%	1.5%	35.9%	11.6%	0.0%	14.7%
Institutional	0.0%	0.0%	0.0%	0.0%	5.3%	78.4%	0.0%	0.0%	55.1%
Low Density Residential	0.0%	0.0%	0.0%	100.0%	63.9%	62.8%	6.7%	0.0%	57.6%
Medium Density Residential	0.0%	0.0%	0.0%	23.7%	11.4%	100.0%	43.8%	88.4%	46.1%
TOTAL	0.1%	0.0%	0.0%	44.4%	16.0%	48.0%	16.5%	16.4%	27.0%

TABLE 4-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
MAY RIVER WATERSHED

Land Use Type	Bass Creek	Bull Creek (May)	May River 1	May River 2	May River 3	May River 4	May River 5	May River Trib	TOTAL
Commercial	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%	0.0%	0.0%	0.1%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%	0.0%	0.1%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	2.3%	2.4%	5.3%	4.9%	0.0%	3.8%
Medium Density Residential	3.1%	0.0%	0.0%	1.1%	1.8%	1.1%	2.9%	0.0%	1.8%
TOTAL	0.0%	0.0%	0.0%	0.3%	0.6%	0.9%	0.9%	0.0%	0.5%

TABLE 4-11

AVERAGE ANNUAL LOADS FOR MAY RIVER WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
May River 1	1,701	5,505	44,441	60	47,151	4.34E+14	87	19,459	2,367	99,122	2,119
May River 2	4,352	11,212	107,000	146	96,560	1.01E+15	190	40,385	5,092	405,000	3,999
May River 3	5,273	10,064	146,000	234	89,117	1.30E+15	240	40,017	5,315	1,130,000	3,189
May River 4	5,858	8,447	129,000	195	73,246	9.98E+14	166	32,599	4,058	966,000	2,090
May River 5	6,025	7,105	104,000	157	61,630	8.54E+14	134	27,648	3,364	808,000	1,361
Bass Creek	2,186	5,242	50,529	81	45,271	4.68E+14	97	19,945	2,963	241,000	1,915
May River Trib	1,738	4,357	40,238	52	37,312	3.60E+14	68	15,367	1,878	127,000	1,503
Bull Creek (May)	824	2,527	20,113	27	21,648	1.99E+14	39	8,934	1,069	51,305	933
TOTAL	27,958	54,459	641,321	952	471,935	5.62E+15	1,021	204,354	26,106	3,827,427	17,109

TABLE 4-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS

MAY RIVER WATERSHED

Water Quality Basin ID	DHEC Station(s)	Years of Record	No. of samples	Fecal Coliform Concentrations				Trend	Level of Service
				Long-Term Average		Most Recent 3 Year Values			
				Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)		
May River 1	20-05	NA	208	2.39	5	2.03	3.53	Decreasing	A
May River 2	19-01, 19-25	1999-2016	415	3.17	8	3.7	13	No Trend	A
May River 3	19-16, 19-18, 19-26, 19-24, 19-19C	1999-2016	863	5.47	17	7.96	33	Increasing	A
May River 4	19-19B, 1919-A, 19-19,	1999-2016	385	14.67	79	25.98	79	Increasing	D
May River 5	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bass Creek	NA	NA	NA	NA	NA	NA	NA	NA	NA
May River Trib	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bull Creek (May)	19-12	1999-2016	208	3.03	8.29	2.75	7.69	Decreasing	A

TABLE 4-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

MAY RIVER WATERSHED

Water Quality Basin ID	South WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Tidal Dispersion Values Length (m)	Tidal Dispersion Values Coefficient (m^2/s)
May River 1	26	1.82E+07	Calibogue Sound 2	5,185	3,356	300
May River 2	27	2.20E+07	May River 1	3,695	5,504	150
May River 3	28	7.53E+06	May River 2	2,617	8,513	150
May River 4	29	1.67E+06	May River 3	497	6,373	450
May River 5	30	1.22E+05	May River 4	110	3,154	75
Bass Creek	31	2.97E+06	May River 1	1,077	4,408	225
May River Trib	32	2.20E+06	May River 2	808	3,356	300
Bull Creek (May)	33	1.88E+06	May River 2	473	2,763	300
			Savage Creek 1	648	2,012	225

TABLE 4-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATION FROM WMM

FOR MAY RIVER WATER QUALITY BASINS

Water Quality Basin ID	South WASP Segment	EXISTING LAND USE	
		Flow (cfs)	Fecal Coliform (#/100 ml)
May River 1	26	9.0	1,048
May River 2	27	19.0	989
May River 3	28	18.1	1,053
May River 4	29	16.4	857
May River 5	30	14.7	786
Bass Creek	31	9.0	1,011
May River Trib	32	7.4	975
Bull Creek (May)	33	4.2	1,034

TABLE 4-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES

MAY RIVER WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs) Existing
May River 1	Calibogue Sound 2	17
May River 2	May River 1	0.7
May River 3	May River 2	49
May River 4	May River 3	31
May River 5	May River 4	15
Bass Creek	May River 1	9.0
May River Trib	May River 2	7.4
May River 2	Bull Creek (May)	76
Bull Creek (May)	Savage Creek 1	80

TABLE 4-16

FECAL COLIFORM MODELING RESULTS

MAY RIVER WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)	Modeled Level of Service
		Existing	Existing
May River 1	0.5	3.4	A
May River 2	1.0	3.6	A
May River 3	2.0	4.6	A
May River 4	2.8	6.9	A
May River 5	2.8	45.2	D
Bass Creek	1.0	5.0	A
May River Trib	1.0	4.8	A
Bull Creek (May)	1.0	3.9	A

Table 4-17 not applicable in the update.

TABLE 4-18 (Updated 2017)

PLANNING LEVEL COST ESTIMATES FOR

MAY RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
AL_M-1	Road overtopping at Ulmer Road Replace existing 1 - 36" RCP and 1 - 30" RCP with 1 - 8'x4' box culvert Raise road 1.8 ft (length of 1,200 ft)	$756,000
BE_M-4	Road overtopping at SC 46 Replace existing 2 - 36" CMP with 2 - 5'x5' box culverts	$156,000
MR_M-1	Road overtopping at Palmetto Bluff Road Add 2 48-in RCP culverts to existing 2 - 48" and 1 - 36" RCP	$67,000
U_M-1	Road overtopping at Alljoy Road Replace existing 1 - 48" CMP with 1 - 5'x5' box culvert	$212,000
SC_T1-4	Road Overtopping at May River Road (State HWY 46) Raise road from elevation 18.1 ft to elevation 18.3 ft NAVD	$157,000
U_M-3	Road overtopping at Confederate Avenue Replace existing 2 - 36" RCP with 2 - 8'x4' box culverts	$173,000
	TOTAL	$1,521,000

Costs are in January 2018 dollars.
See Appendix for basis of cost estimates.

Figure 4-1

May River Watershed

ATM DESIGN ENGINEERING CONSULTING

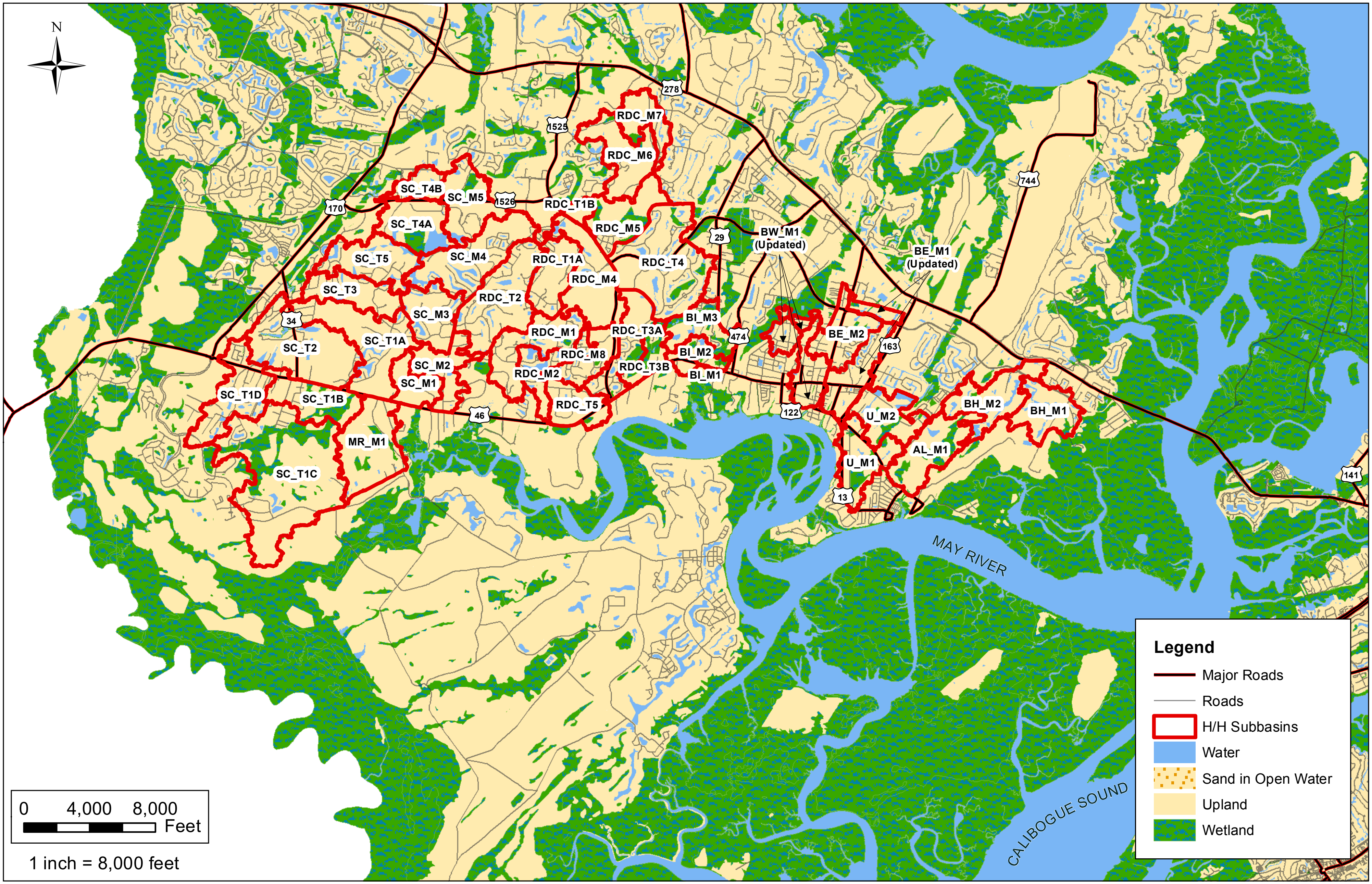


Figure 4-2. Hydrologic Model Subbasins

May River Watershed

ATM DESIGN ENGINEERING CONSULTING

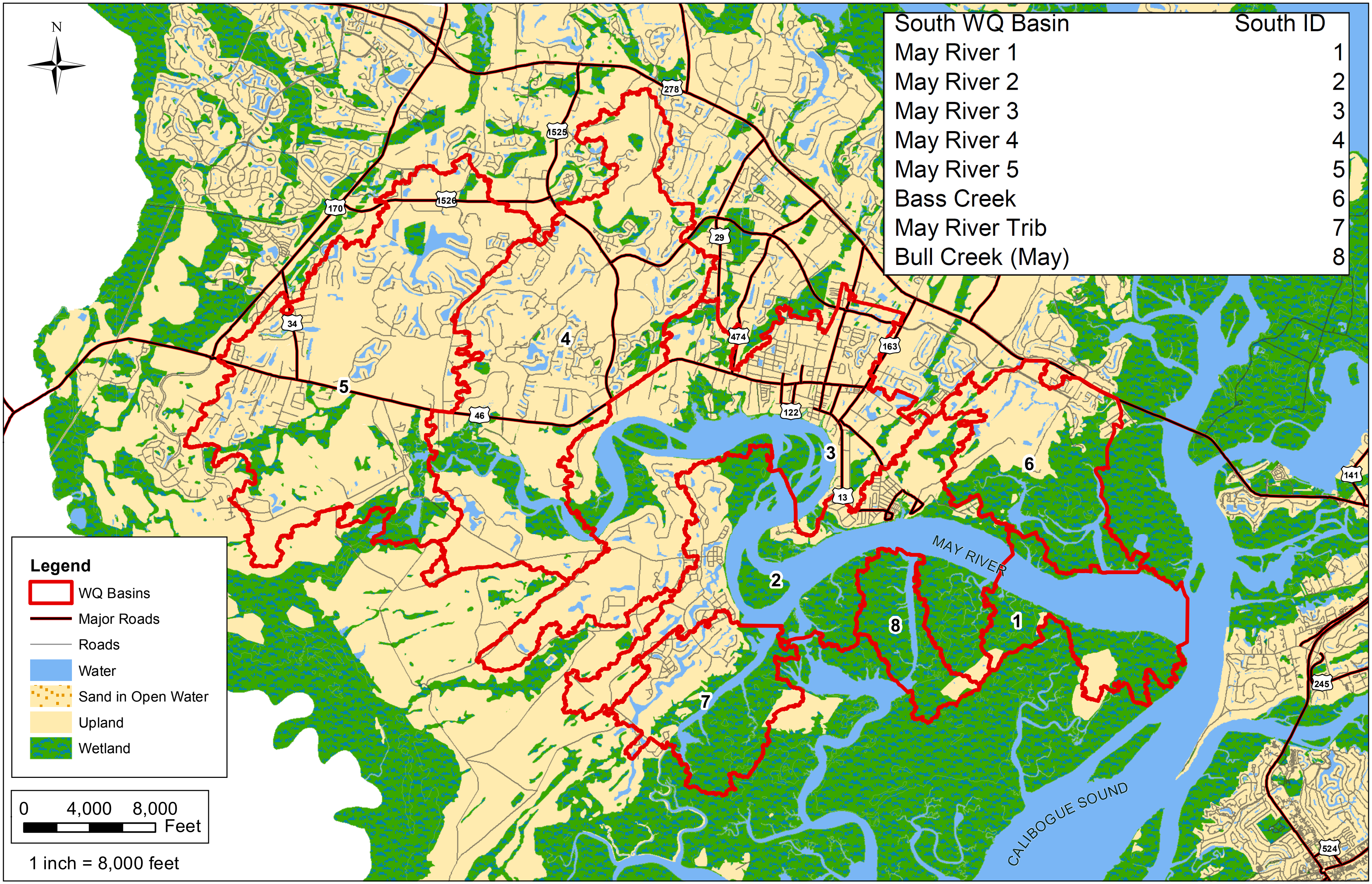
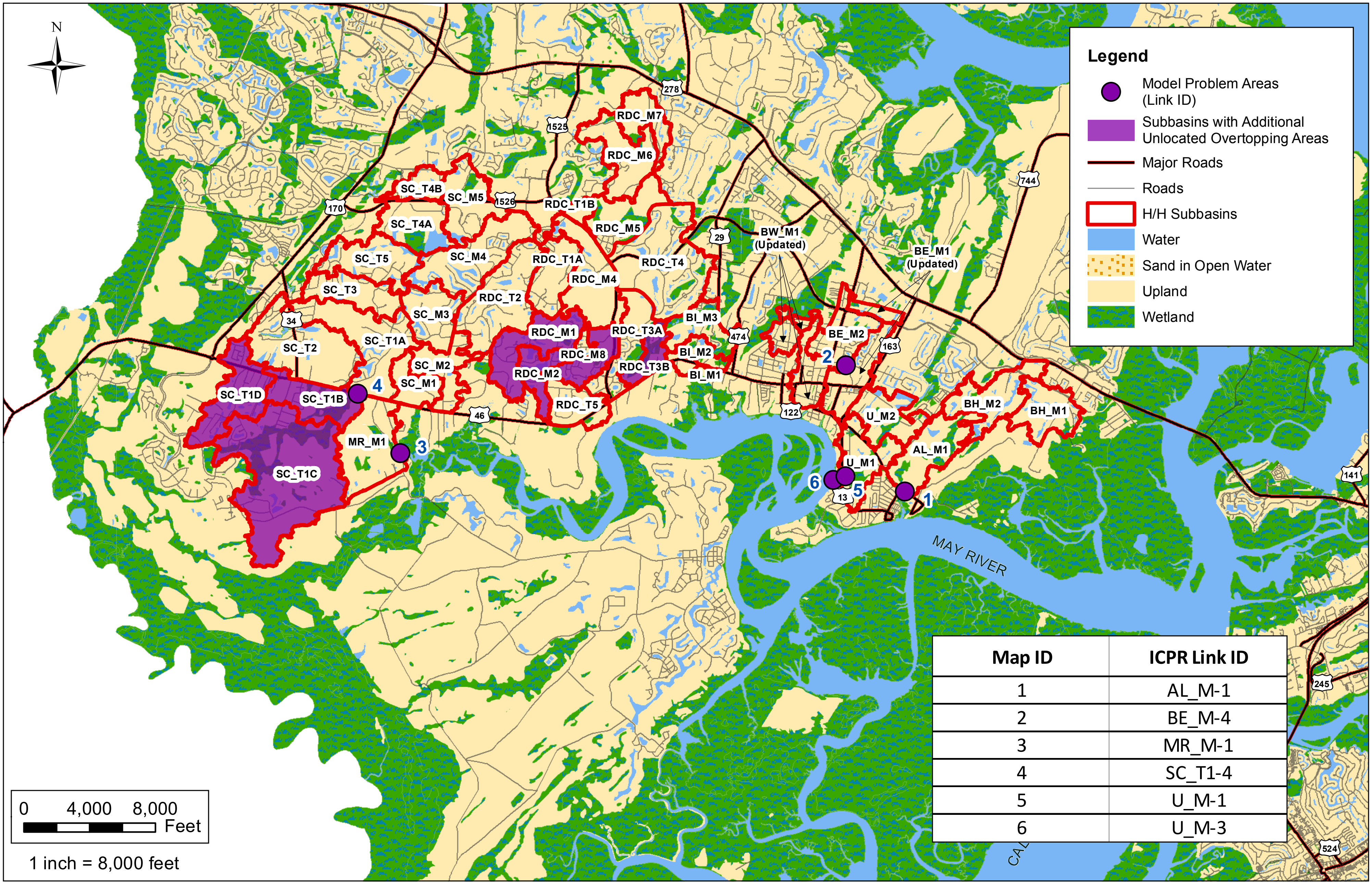


Figure 4-3

May River Watershed
Water Quality Basins

ATM DESIGN ENGINEERING CONSULTING



Map ID	ICPR Link ID
1	AL_M-1
2	BE_M-4
3	MR_M-1
4	SC_T1-4
5	U_M-1
6	U_M-3

Figure 4-4. ICPR Identified PSMS Overtopping Problem Areas

May River Watershed

ATM DESIGN ENGINEERING CONSULTING

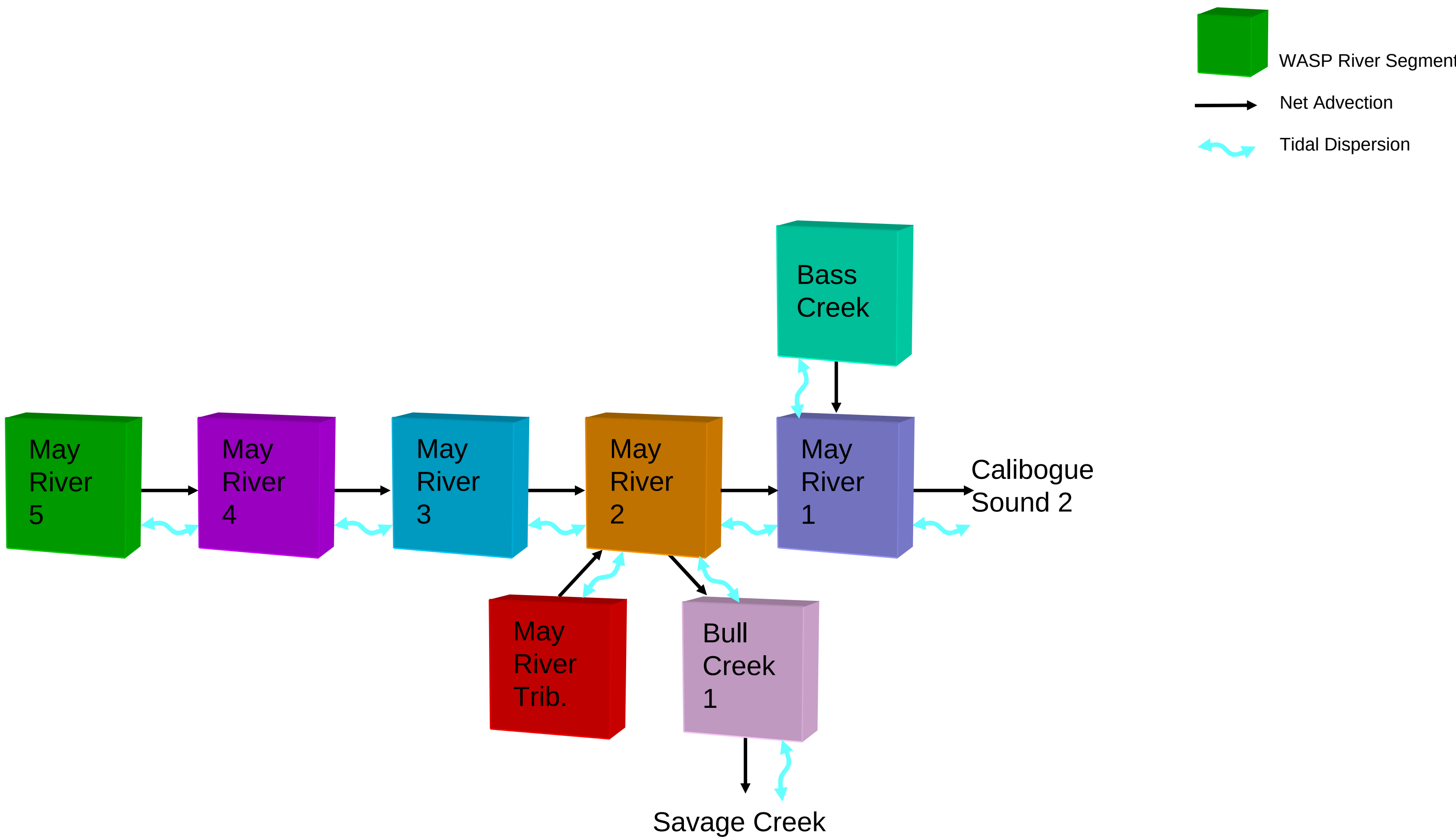


Figure 4-5
WASP Model Schematic for May River Watershed

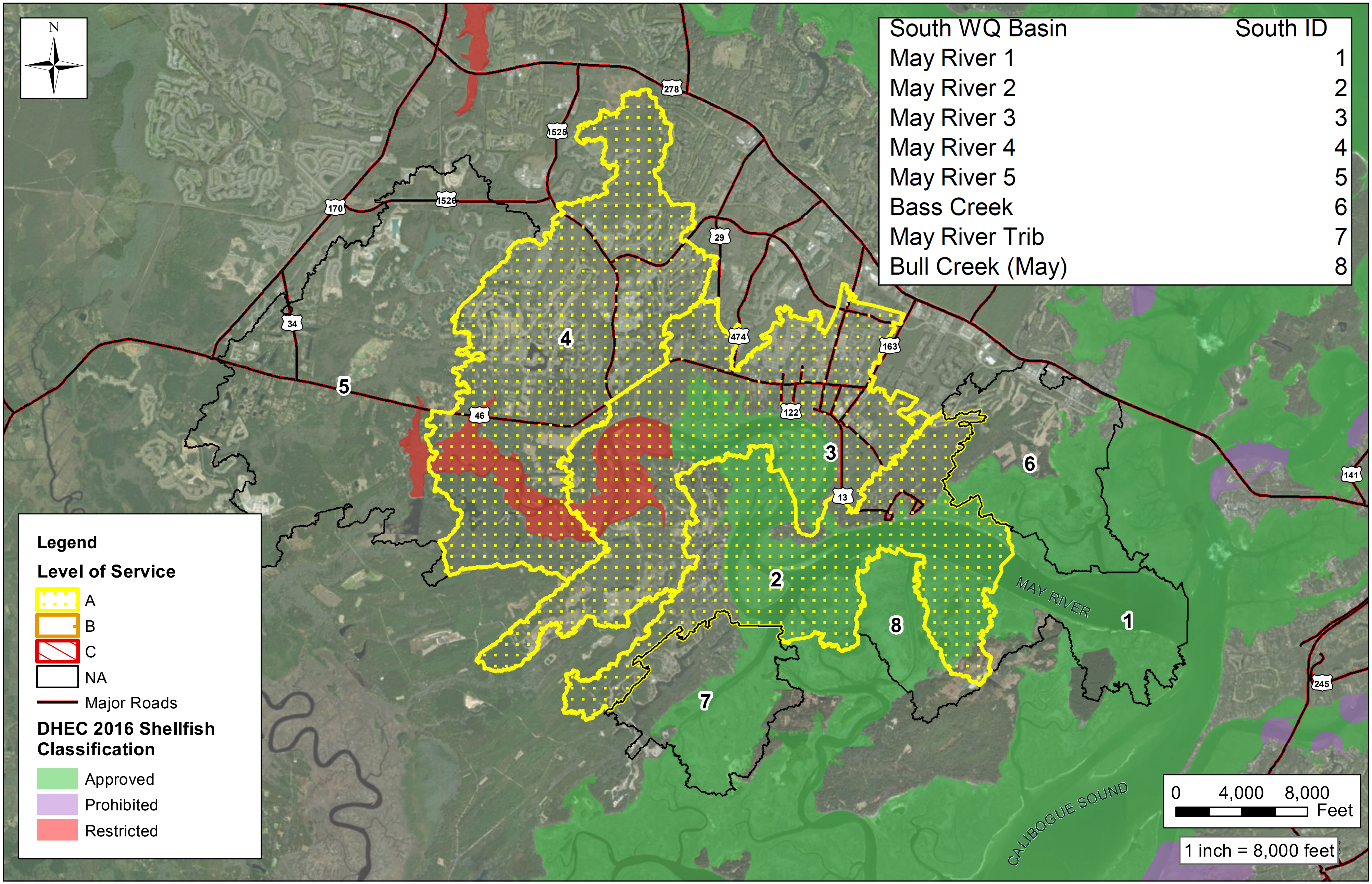


Figure 4-6

May River Watershed
Shellfish Classification and Existing Level of Service

ATM DESIGN ENGINEERING CONSULTING

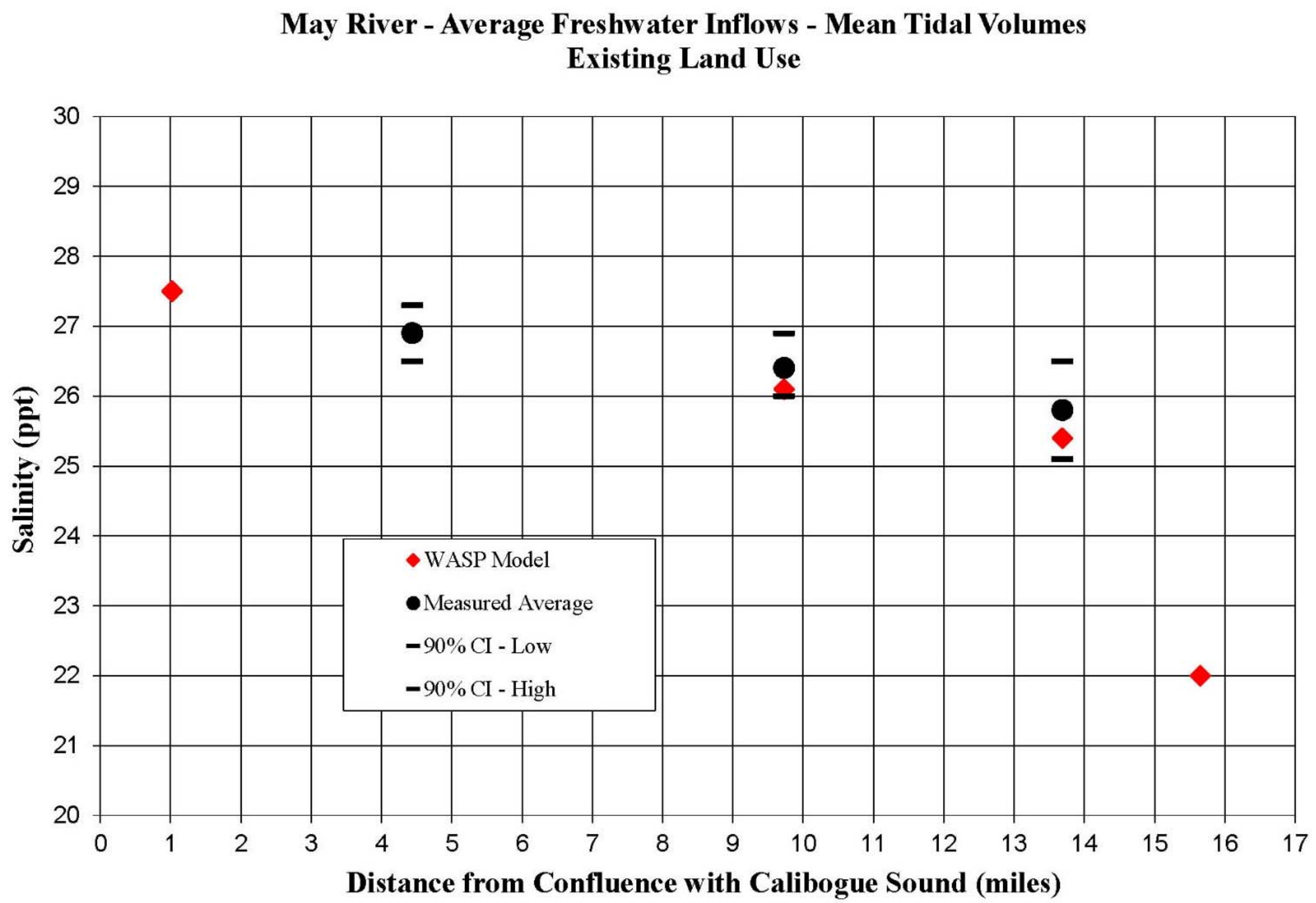


Figure 4-7. Comparison of WASP Model Results with Long-Term Monitoring Data in May River– Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

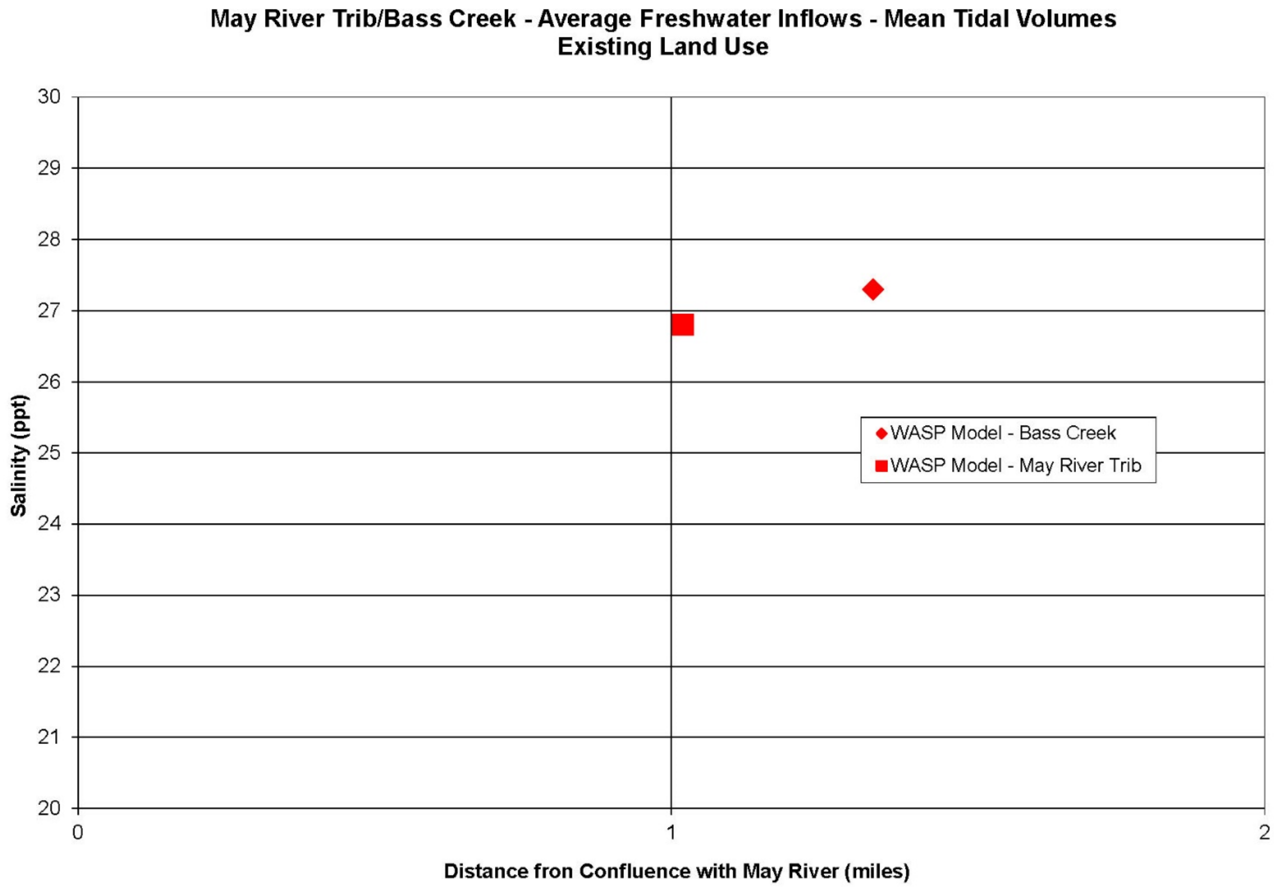


Figure 4-8. Comparison of WASP Model Results with Long-Term Monitoring Data in May River Tributary and Bass Creek– Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

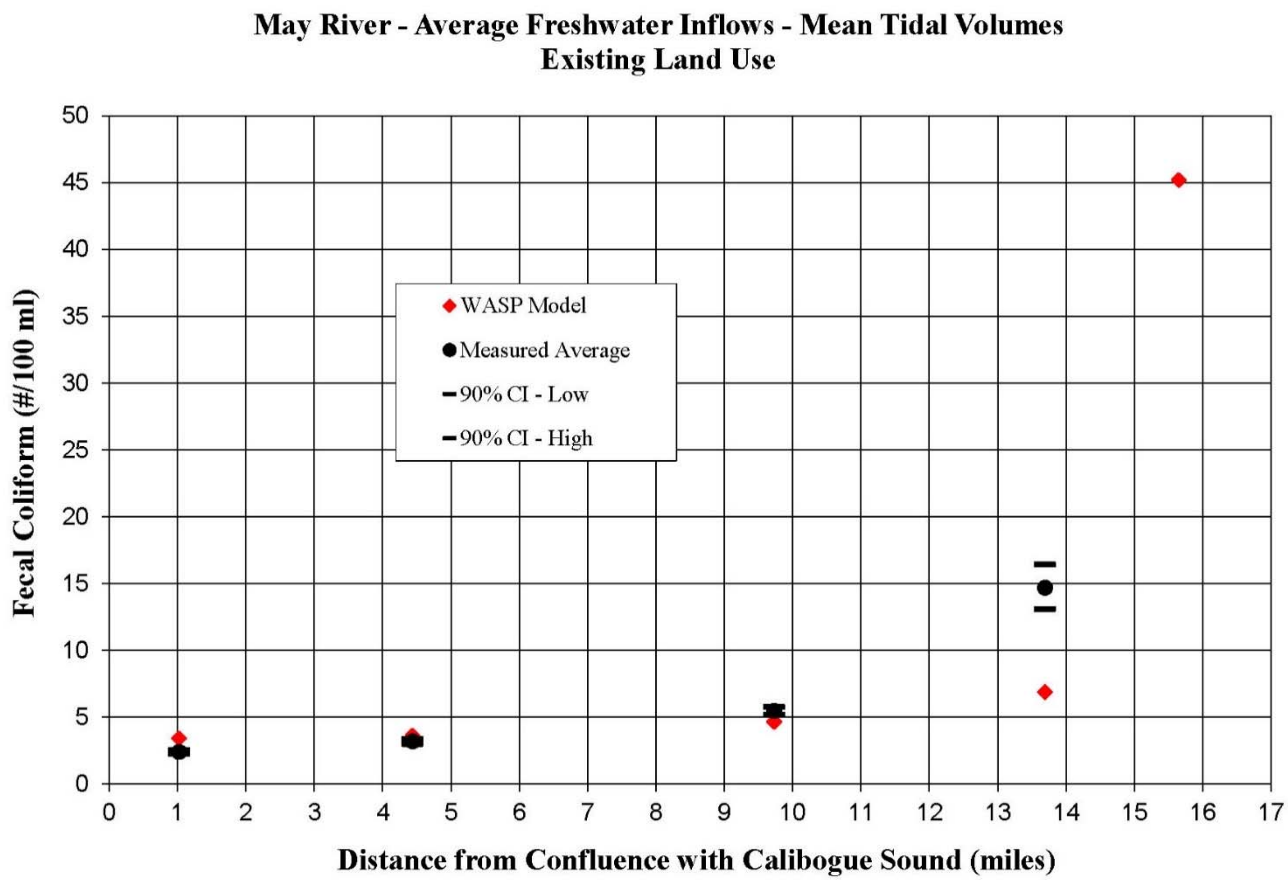


Figure 4-9. Comparison of WASP Model Results with Long-Term Monitoring Data in May River - Fecal Coliform Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

ATM DESIGN ENGINEERING CONSULTING

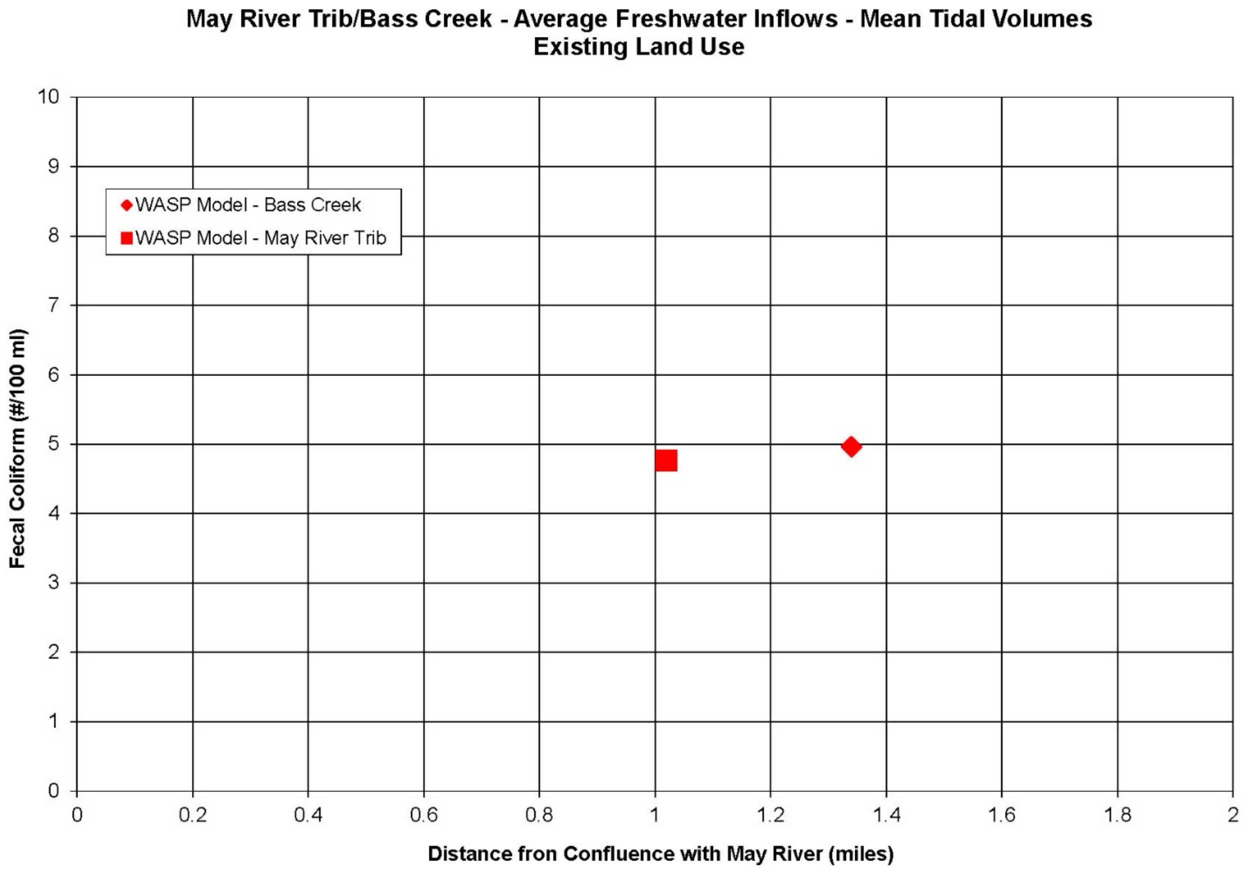
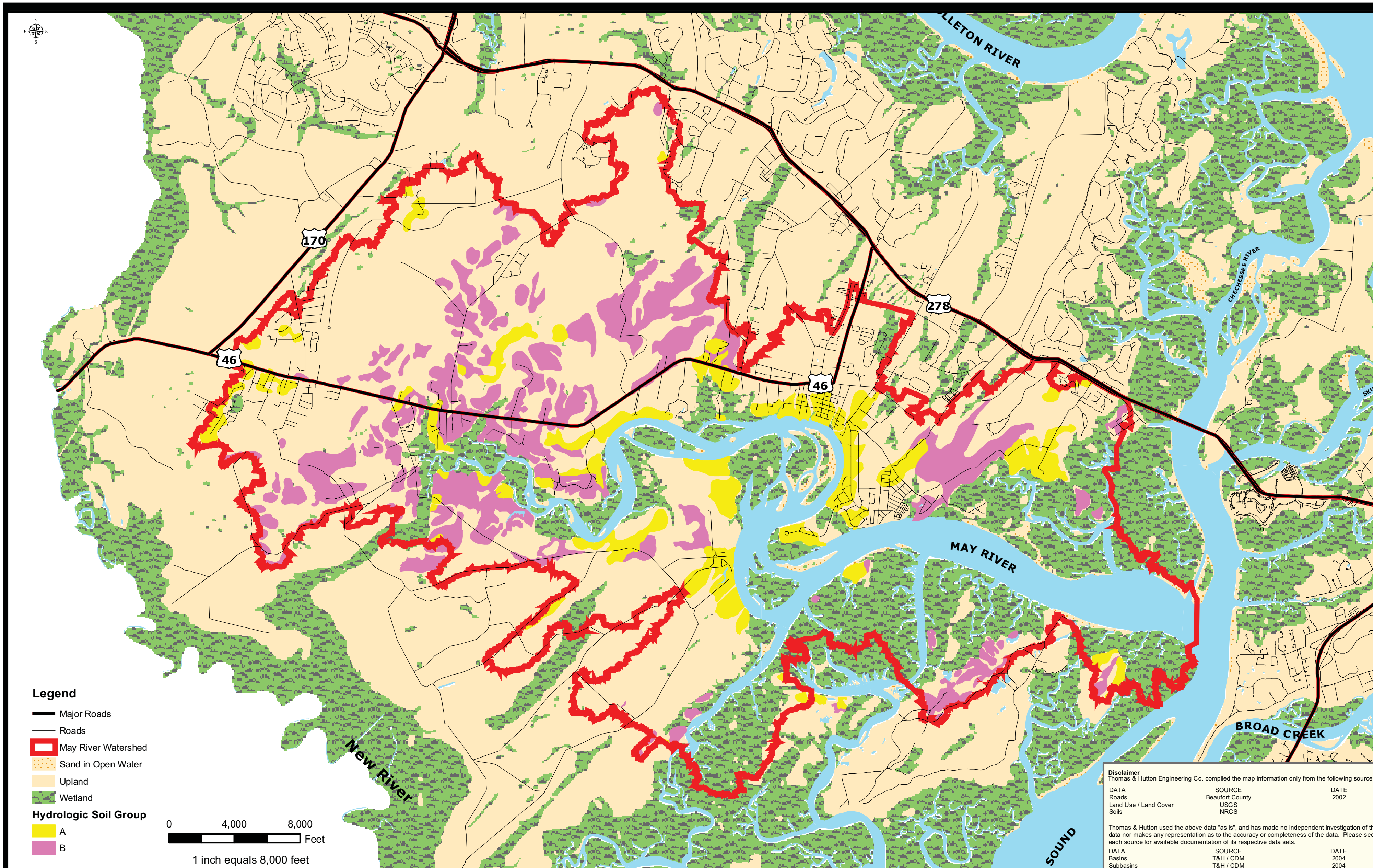


Figure 4-10. Comparison of WASP Model Results with Long-Term Monitoring Data in May River Tributary and Bass Creek - Fecal Coliform Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Figure 4-11 is not applicable in the update.



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Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\MayRiverSoils_figure4-10.mxd			Vertical Datum: NAVD88

May River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

CDM Camp Dresser & McKee Inc.

Figure 4-12

Section 5
Chechessee River Watershed Analysis

This section describes the physical features of the Chechessee River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

5.1 Overview

The Chechessee River watershed is located south of the Broad River (see **Figure 5-1**). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in Bluffton Township that is tributary to the Chechessee River. Major Chechessee River tributaries included in the analysis are Skull Creek, Mackays Creek and Chechessee Creek.

For the hydrologic and hydraulic analysis of the Primary Stormwater Management System (PSMS), the watershed includes several hydrologic basins. These are listed in **Table 5-1**, and presented in **Figure 5-2**. Table 5-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were done to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into “water quality” basins, and the tidal receiving waters were subdivided into receiving water segments. These are listed in **Table 5-2**, and presented in **Figure 5-3**. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were done to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

5.2 Hydrologic and Hydraulic Analysis

CDM and T&H used the Interconnected Pond Routing Model (ICPR), Version 3 for the hydrologic and hydraulic analyses of the PSMS in the Chechessee River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were conducted for existing and future land use conditions, with and without alternative management strategies.

The ICPR model is a “link-node” model, representing the PSMS as a series of nodes (stream locations) connected by links (open channels, pipes, culverts). Figures in Appendix C show model schematics of the Chechessee River PSMS basins, with a separate schematic for each basin.

5.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Chechessee River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 5-3 lists the hydrologic parameter values for the Chechessee River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development.

Hydraulic summary information for the Chechessee River PSMS basins is presented in **Table 5-4**. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in **Table 5-5**. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate level of service.

Details regarding specific open channel segments, storage areas, weirs and tide gates are presented in Appendix C.

5.2.2 Model Results

Tables in Appendix C also list the peak flow values for the Chechessee River subbasins. Each table lists peak flows for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak flows are listed by subbasin for various land cover and stormwater management controls, which include the following:

- Undeveloped land
- Existing land use without peak shaving controls
- Existing land use with existing peak shaving controls
- Future land use without peak shaving controls

- Future land use with existing and future peak shaving controls

It should be noted that the tables include values for “uncontrolled” and “controlled” peak flows for the 2-year, 10-year and 25-year design storms. The “uncontrolled” peak flow assumes no peak shaving facilities in the subbasin. In contrast, the “controlled” value accounts for peak shaving facilities in the subbasin.

For existing land use, aerial maps and local information were used to estimate the percentage of existing urban development that is served by peak shaving facilities. The “controlled” peak flow value was then calculated by considering the difference in peak flow between totally undeveloped conditions and existing conditions with no controls. For example, suppose that a subbasin of 100 acres has an undeveloped 2-year peak flow of 20 cfs, and an uncontrolled existing peak flow of 50 cfs, and further suppose that 60 percent of the urban development is controlled by peak shaving facilities. In this case, it is assumed that the existing peak flow is reduced by 60 percent of the difference between undeveloped and developed peak flow (50 – 20 = 30 cfs; 60 percent of 30 cfs = 18 cfs reduction due to peak shaving), and therefore the maximum controlled peak flow will be 32 cfs (50 – 18).

For future land use, the “controlled” peak flow is set equal to the “controlled” peak flow for existing land use, because new development is subject to State and County peak flow regulations. Note, however, that the future condition will still generate more stormwater runoff volume, even though the peak flow is the same. The result is that the peak flow rate will be sustained for a longer period of time under future conditions.

Tables in Appendix C list the peak water elevation values for model node locations along the Chechessee River PSMS. Each table lists peak stages for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak stages are listed for existing and future land use conditions, with the existing hydraulic system.

Specific problem areas identified by the modeling are listed in **Table 5-6** and presented in **Figure 5-4**. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

Structural flooding was also considered for the 100-year design storm. In locations where the PSMS evacuation route crossings are overtopped by the 100-year design storm, figures were developed showing the area of inundation upstream of the overtopped road. These figures are presented in Appendix C. In addition, the peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) base flood elevations, and results showed that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA base flood elevations should not be flooded.

Table 5-6 indicates that two road crossing in the Chechessee River watershed PSMS are being overtopped by the design storm events. One of the hydrologic and hydraulic basins has no problems, and the rest have one problem area.

Evaluation of solutions to prevent these problems is discussed in the next section of this report.

5.2.3 Management Strategy Alternatives

The problems areas listed in Table 5-6 were evaluated by modifying the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in **Table 5-7**. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, circular culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

5.3 Water Quality Analysis

CDM and T&H used the Watershed Management Model (WMM) and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the May River watershed. WMM was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, total nitrogen (total N), total phosphorus (total P), BOD, lead, zinc and total suspended solids (TSS). WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions. The same parameter values were used for evaluation of future conditions, which reflect higher flows and loads from the watershed.

5.3.1 Land Use and BMP Coverage

Table 5-8 presents the existing land use and future land use estimates for the Chechessee River water quality basins. The existing land use data were gathered from a number of sources, including February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps, plus local knowledge of development completed between February 2002 and June 2003. The future land use map was developed by "filling in" the existing land use map and by replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and Town of Hilton Head Island future land use maps and zoning maps.

Under existing land use conditions, 14 percent of the Chechessee River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 86 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 2 per cent of the watershed.

Under future land use conditions, 17 percent of Chechessee River watershed area consists of urban systems, and 83 percent consists of natural systems. The major change in land use distribution is the conversion of forest/rural land to urban land uses. As a result of projected future development, urban imperviousness increases to about 3 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in **Table 5-9**. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County in accordance with the County BMP Manual. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 22 percent of the urban systems in the watershed are served by BMPs. Under future land use conditions, 41 percent of the urban systems are served by BMPs. This increase from existing to future reflects both the increase in urban land use and the 100 percent coverage of the new development with BMPs in accordance with the County BMP Manual.

5.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in **Table 5-10**. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water and Sewer Authority. For future development, areas that are zoned "rural" or "conservation" were assumed to be served by septic tanks, and other areas were assumed to be served by sewer.

Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 33 percent of the urban systems in the watershed are served by septic tanks. Under future land use conditions, 49 percent of the urban systems are served by septic tanks. This increase in watershed septic tanks coverage reflects that the relatively small amount of development anticipated for future conditions within the Chechessee River watershed will be served by septic tanks.

Based on available data, the estimated wastewater discharge under existing conditions is 0.1 million gallons per day (mgd) of land application (e.g., golf course irrigation), and the future discharge is also expected to be 0.1 mgd based on limited increase in residential land between existing and future conditions. There are no direct discharges to receiving waters in the watershed.

5.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Chechessee River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in **Table 5-11** for existing and future land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

An overall comparison of the WMM modeling results (Table 5-11) indicates that future flows and constituent loads generally increase or decrease a small amount (less than 1 percent) over their existing counterparts; however, in the case of TSS loads, a decrease of 4 percent is experienced. The TSS load reduction reflects the fact that BMPs are typically very efficient in removing sediment suspended in stormwater runoff. It should also be noted that the relatively flat difference in loads for several constituents (e.g., total N, zinc) is because direct rainfall on the open water/tidal wetland area provides a significant fraction of the total load to the Chechessee River.

In addition, all of the basins have little or no change in land use from existing to future conditions.

Wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing and future discharge of wastewater is limited to roughly 0.1 mgd of land application (e.g., golf course irrigation). Using the values in Table 2-9, the wastewater load for existing conditions accounts for 0.2 to 0.3 percent of the total watershed load for nutrients (total nitrogen and total phosphorus) and 0.0 to 0.1 percent of the load for other constituents.

5.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Chechessee River watershed. The model actually includes Calibogue Sound, May River, Colleton River, and Chechessee River watersheds because they are interconnected at several points. Only the Chechessee River will be discussed in this section. A schematic of the model is presented as **Figure 5-5**.

Existing conditions for bacteria concentrations in the Chechessee River are presented in **Table 5-12**. For each water quality basin river reach, the table lists the DHEC stations for which the 1990s bacteria data were analyzed, the concentrations calculated in the analysis, and the "level of service" associated with these concentrations (as discussed in Section 2.6.2). As shown in the table, DHEC data were only available in seven of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations meet the water quality standards at all segments except Chechessee Creek 2 , and so the segments other than Chechessee Creek 2 have an "A" level of service. Chechessee Creek has a "D" level of service based on the methodology discussed in section 2.6.2, though only the 90th percentile standard was exceeded by the measured 1990s data.

For informational purposes, **Figure 5-6** presents a map of the level of service based on the monitoring data analysis, compared to the Department of Health and Environmental Control (DHEC) "shellfish classification" (based on the 2002 DHEC reports for shellfish areas 17, 18 and 20). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the level of service, so there may not be a direct relationship between level of service and shellfish classification presented in the map. In general, however, segments with an "A" level of service are expected to have the lowest probability of receiving a "restricted" classification, and segments with a "D" level of service are expected to have the highest probability of receiving a "restricted" classification.

Physical characteristics assigned to the model reaches are presented in **Table 5-13**. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the "characteristic length" (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the DHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. **Tables 5-14** and **5-15** show the values used in the existing and future condition models.

A review of Table 5-14 shows that there is typically little change in flow or concentration between existing and future land use. For flow, this is because much of the flow to the tidal river segments comes from direct rainfall on the open water and tidal wetlands, as opposed to stormwater runoff and baseflow, and some of the basins have very little change in land use from existing to future conditions. Concentration remain relatively constant because of the substantial amount of open water/tidal wetland area and the relatively limited development in some basins, as well as the BMPs for new development, which are assumed to have a high level of treatment efficiency.

Table 5-15 shows the net advective flows between segments, which also do not change substantially from existing to future land use. In both cases, the hydrodynamic model (SWMM) indicates that there is a substantial net flow from the Chechessee River to Skull Creek and Mackays Creek.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. In general, a loss rate of 1.0/day was assumed initially, and values were then adjusted to achieve a better match between modeled and measured data. The final calibration values will be discussed below.

Figure 5-7 is a graph showing a comparison between measured and modeled salinity data along the Chechessee River main stem. The figure shows that the salinity data calculated by the model is very close to the average measured value, and is in all cases well within the 90 percent confidence interval of the mean of the salinity data.

Figures 5-8 and **5-9** are graphs showing a comparison between measured and modeled salinity data along Skull Creek/Mackays Creek (Figure 5-8) and Chechessee Creek (Figure 5-9). Again, the model does a good job of matching the measured data, as the salinity data calculated by the model is very close to the average measured value, and is in all cases well within the 90 percent confidence interval of the mean of the salinity data.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration is presented in **Figures 5-10** through **5-12**. The graphs show very good agreement between the measured values and the model results on the Chechessee River main stem. The agreement is not as good on the tributaries, especially in Mackays Creek, where the model calculates a concentration that is higher than the upper bound of the 90 percent confidence interval of the mean measured data. However, since the measured and modeled mean value is much lower than the threshold between the "A" and "B" level of service, this is not considered to be important.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in **Table 5-16**. Most of the values were in the range of 0.5/day to 1.4/day. The Chechessee River 5 segment required a relatively high value (4/day) to calibrate the bacteria concentration in that segment, which suggests that the model may be overestimating the load to the segment, or is not accounting for other processes that are occurring to reduce the river bacteria concentrations.

After the model was applied for existing conditions, it was then applied for future conditions. The physical characteristics and first-order loss rate from the existing land use model were kept the same in the future land use model. The only changes were the net advective flows and the bacteria loads.

The bacteria concentrations calculated under future land use conditions are presented in Table 5-16 as well. A comparison of concentrations under existing and future land use conditions shows little difference. According to the model, all river reaches will have the same level of service in the future as they do under existing conditions.

In order to estimate the degree to which stormwater management measures are expected to affect instream bacteria concentrations, two sensitivity runs were conducted. The first was run for the existing land use condition, and represents a "best-case" scenario in which all existing development is controlled by BMPs. The second was run for the future land use condition, and represents a "worst-case" condition in which no development is served by BMPs. Analyzing the results of these scenarios indicate the benefits of retrofitting existing development with BMPs, and the potential degradation of river segments if BMPs fail.

The results of the analysis are presented in **Table 5-17**. This table is similar to Table 5-16, in this case showing water quality basin segment fecal coliform concentrations for the "best case" and "worst case" analyses. Segments that show change (e.g., better LOS for the "best case" or degraded LOS for the "worst case") are highlighted.

A review of the "best-case" scenario indicates that three model segments show improvement in the existing level of service. These include Chechessee Creek 2, Chechessee Trib – Ballenger Neck, and Chechessee Trib – Spring Island. The Chechessee Creek 2 river segment goes from a "C" to a "B" level of service, and the Chechessee Trib – Ballenger Neck and Chechessee Trib – Spring Island segments go from a "D" to a "C" level of service. Note that the improvement in Chechessee Creek

2 assumes 100 percent BMP coverage in that water quality basin as well as upstream and downstream water quality basins such as Chechessee Creek 1, Chechessee Trib – Ballenger Neck and Chechessee Trib – Spring Island.

A review of the "worst-case" scenario indicates that one model segment shows degradation in the future level of service when no BMPs are assumed. This is Chechessee Creek 2, which drops from a "C" to a "D" level, though the "worst case" concentration (10.2/100 ml) is just above the 10/100 threshold for the "D" rating.

Based on water quality sampling data and model results, the following recommendations are made:

- Evaluate opportunities for retrofitting existing development in the Chechessee Creek 2 and Chechessee Trib – Ballenger Neck water quality basins to the maximum extent practicable.

5.3.5 Management Strategy Alternatives

The results of the water quality analysis suggest that the limited amount of future development in the watershed, combined with the effectiveness of required BMPs in reducing bacteria loads from new development, will maintain the existing level of service (typically level A) in all watershed reaches, and only the Chechessee Creek 2 segment and its tributaries (Ballenger Neck and Spring Island) do not have an "A" level of service. For these segments, additional controls should be considered to improve the level of service. As discussed above, these activities would include retrofit of existing development that does not have BMPs, and modification of existing ponds that may not have been designed for water quality control.

Elements of the water quality management plan for the Chechessee River watershed are presented in **Figure 5-13**. Sampling stations shown in the figure include existing DHEC sites. Also identified are "priority" water quality basins. Sensitivity analysis results suggest that load changes in these basins are most likely to result in an improved or degraded LOS in the receiving waters.

For informational purposes, the areas with "A" and "B" type soils are presented in **Figure 5-14**. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

5.4 Planning Level Cost Estimates for Management Alternatives

Table 5-18 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the May River watershed. As shown in the table, the two projects are estimated to have a total cost of $0.1 million based on December 2004 dollars. Details of the cost estimate for each project are shown in Appendix C.

The prioritization of these projects, and projects identified for other watersheds, is discussed in Section 16 of this report.

TABLE 5-1
HYDROLOGIC BASINS
CHECHESSEE RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Callawassee Road West	526	2	263
Foot Point	347	1	347
Spring Island 2	105	1	105
TOTAL	978	4	244

TABLE 5-2
WATER QUALITY BASINS
CHECHESSEE RIVER WATERSHED

Basin Name	Tributary Area (acres)
Chechessee River 1	5,434
Chechessee River 2	4,434
Chechessee River 3	2,138
Chechessee River 4	523
Chechessee River 5	1,156
Skull Creek North 1	516
Skull Creek North 2	552
Mackays Creek North 1	428
Mackays Creek North 2	158
Mackays Creek North - Corn Island	560
Broad/Chechessee Trib	143
Chechessee Creek 1	1,452
Chechessee Creek 2	1,582
Chechessee Trib - Ballenger Neck	493
Chechessee Trib - Spring Island	212
TOTAL	19,780

TABLE 5-3
HYDROLOGIC SUBBASIN CHARACTERISTICS
CHECHESSEE RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Callawassee Road West Basin					
CRW_M1	221	68	147	70	138
CRW_M2	305	71	155	74	142
Foot Point					
FP_M1	347	62	191	62	191
Spring Island 2 Basin					
SI2_M1	105	79	68	79	68
Average	244	70	140	71	135

TABLE 5-4
HYDRAULIC DATA SUMMARY
CHECHESSEE CREEK WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Callawassee Road West	6	4,716	2	1	1	1	1	0
Foot Point	1	1,085	1	1	0	1	1	0
Spring Island 2	0	0	1	1	0	1	2	1
TOTAL	7	5,801	4	3	1	3	4	1

TABLE 5-5
CULVERT DATA FOR HYDROLOGIC BASINS
CHECHESSEE RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Callawassee Road West Basin						
Heyward Pointe	CRW_T1-3	Bridge	25	3.60	9.00	25
Callawassee Drive	CRW_T1-5	18"x18"	45	5.7	11.5	25
Foot Point Basin						
Unknown	FP_M-3	15"x15"	36	4.7	8.3	25
Spring Island 2 Basin						
Shrimp Pond Road	SI2_M-2	15"x15"	42	3.1	7.6	25

TABLE 5-6
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
CHECHESSEE RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Callawassee Road West Basin					
Callawassee Drive	CRW_T1-18	11.5	25	12.0	12.0
Foot Point Basin					
No Overtopping					
Spring Island 2 Basin					
Shrimp Pond Road	SI2_M-2	7.6	25	8.1	8.1

TABLE 5-7
RECOMMENDED CULVERT IMPROVEMENTS
CHECHESSEE RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Callawassee Road West Basin			
Callawassee Drive	CRW_T1-5	18"x18"	Replace culvert with one 48" pipe
Foot Point Basin			
No improvements required			
Spring Island 2 Basin			
Shrimp Pond Road	SI2_M-2	15"x15"	Replace culvert with four 36" pipes, Replace both riser and bubbler structures with 24 in by 72 in rectangular horizontal weirs

TABLE 5-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
CHECHESSEE RIVER WATERSHED

Existing Land Use Type	Chechessee River 1 (acres)	Chechessee River 2 (acres)	Chechessee River 3 (acres)	Chechessee River 4 (acres)	Chechessee River 5 (acres)	Skull Creek North 1 (acres)	Skull Creek North 2 (acres)	Mackays Creek North 1 (acres)	Mackays Creek North 2 (acres)	Mackays Creek North - Corn Island (acres)	Broad/Chechessee Trib (acres)
Agricultural/Pasture	0	0	0	0	7	0	0	0	0	0	0
Commercial	0	0	0	2	0	0	0	0	0	0	0
Forest/Rural Open	18	5	137	31	37	0	0	0	0	29	0
Golf Course	273	80	32	0	0	8	48	0	0	0	0
High Density Residential	0	0	0	0	0	69	40	0	0	0	0
Industrial	34	8	36	27	20	12	13	0	0	0	0
Institutional	0	0	0	0	10	0	0	0	0	0	0
Low Density Residential	169	1	195	16	32	0	0	0	0	0	0
Medium Density Residential	99	48	0	0	0	0	0	0	0	0	0
Open Water/Tidal	4,798	4,244	1,588	367	1,014	351	356	302	153	488	143
Silviculture	0	0	0	0	0	0	0	0	0	0	0
Urban Open	10	42	122	77	37	10	17	0	0	0	0
Wetland/Water	33	6	28	3	0	66	78	126	5	42	0
TOTAL	5,434	4,434	2,138	523	1,156	516	552	428	158	560	143
Urban Imperviousness (%)	1%	0%	2%	4%	2%	8%	5%	0%	0%	0%	0%

Future Land Use Type	Chechessee River 1 (acres)	Chechessee River 2 (acres)	Chechessee River 3 (acres)	Chechessee River 4 (acres)	Chechessee River 5 (acres)	Skull Creek North 1 (acres)	Skull Creek North 2 (acres)	Mackays Creek North 1 (acres)	Mackays Creek North 2 (acres)	Mackays Creek North - Corn Island (acres)	Broad/Chechessee Trib (acres)
Agricultural/Pasture	0	0	0	0	7	0	0	0	0	0	0
Commercial	0	0	0	6	0	0	0	0	0	0	0
Forest/Rural Open	1	0	133	2	28	0	0	0	0	0	0
Golf Course	272	80	71	0	0	11	57	0	0	0	0
High Density Residential	0	0	0	0	0	69	40	0	0	0	0
Industrial	34	8	37	26	21	12	13	0	0	0	0
Institutional	0	0	0	0	10	0	0	0	0	0	0
Low Density Residential	186	1	195	48	64	0	0	0	0	29	0
Medium Density Residential	99	53	22	11	1	0	0	0	0	0	0
Open Water/Tidal	4,798	4,244	1,588	367	1,013	350	357	302	152	488	143
Silviculture	0	0	0	0	0	0	0	0	0	0	0
Urban Open	10	42	64	61	12	6	8	0	0	0	0
Wetland/Water	33	6	28	3	0	67	78	126	6	43	0
TOTAL	5,434	4,434	2,138	523	1,156	516	552	428	158	560	143
Urban Imperviousness (%)	1%	0%	2%	6%	2%	8%	5%	0%	0%	1%	0%

TABLE 5-8 (CONTINUED)
WATER QUALITY BASIN LAND USE DISTRIBUTION
CHECHESSEE RIVER WATERSHED

Existing Land Use Type	Chechessee Creek 1 (acres)	Chechessee Creek 2 (acres)	Chechessee Trib - Ballenger Neck (acres)	Chechessee Trib - Spring Island (acres)	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	7
Commercial	0	1	1	0	4
Forest/Rural Open	163	294	167	19	900
Golf Course	106	194	0	12	754
High Density Residential	0	0	0	0	108
Industrial	29	72	26	7	283
Institutional	0	4	0	0	13
Low Density Residential	125	287	56	1	882
Medium Density Residential	36	131	28	34	376
Open Water/Tidal	982	488	200	136	15,608
Silviculture	0	0	0	0	0
Urban Open	0	0	16	0	331
Wetland/Water	11	112	0	2	514
TOTAL	1,452	1,582	493	212	19,780
Urban Imperviousness (%)	3%	7%	7%	7%	2%

Future Land Use Type	Chechessee Creek 1 (acres)	Chechessee Creek 2 (acres)	Chechessee Trib - Ballenger Neck (acres)	Chechessee Trib - Spring Island (acres)	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	7
Commercial	0	1	5	0	12
Forest/Rural Open	49	53	1	19	286
Golf Course	107	194	0	12	806
High Density Residential	0	0	0	0	109
Industrial	28	72	26	7	284
Institutional	0	3	0	0	13
Low Density Residential	235	526	108	1	1,394
Medium Density Residential	39	132	154	34	545
Open Water/Tidal	982	488	200	135	15,606
Silviculture	0	0	0	0	0
Urban Open	0	0	0	0	203
Wetland/Water	11	112	0	2	516
TOTAL	1,452	1,582	493	212	19,781
Urban Imperviousness (%)	4%	9%	15%	7%	3%

TABLE 5-9
WATER QUALITY BASIN BMP COVERAGE
CHECHESSEE RIVER WATERSHED

Existing Land Use Type	Chechessee River 1 (%)	Chechessee River 2 (%)	Chechessee River 3 (%)	Chechessee River 4 (%)	Chechessee River 5 (%)	Skull Creek North 1 (%)
Commercial	0%	0%	0%	0%	0%	0%
Golf Course	0%	0%	0%	0%	0%	100%
High Density Residential	0%	0%	0%	0%	0%	100%
Industrial	0%	0%	0%	0%	0%	97%
Institutional	0%	0%	0%	0%	0%	0%
Low Density Residential	0%	0%	0%	0%	0%	0%
Medium Density Residential	0%	0%	0%	0%	0%	0%
TOTAL	0%	0%	0%	0%	0%	99%

Future Land Use Type	Chechessee River 1 (%)	Chechessee River 2 (%)	Chechessee River 3 (%)	Chechessee River 4 (%)	Chechessee River 5 (%)	Skull Creek North 1 (%)
Commercial	0%	0%	0%	60%	0%	0%
Golf Course	0%	0%	55%	0%	0%	100%
High Density Residential	0%	0%	0%	0%	0%	100%
Industrial	0%	0%	4%	1%	0%	97%
Institutional	0%	0%	0%	0%	0%	0%
Low Density Residential	9%	0%	0%	66%	51%	0%
Medium Density Residential	0%	9%	100%	100%	100%	99%
TOTAL	3%	4%	19%	51%	35%	100%

TABLE 5-9 (CONTINUED)
WATER QUALITY BASIN BMP COVERAGE
CHECHESSEE RIVER WATERSHED

Existing Land Use Type	Skull Creek North 2 (%)	Mackays Creek North 1 (%)	Mackays Creek North 2 (%)	Mackays Creek North - Corn Island (%)	Broad/Chechessee Trib (%)
Commercial	0%	0%	0%	0%	0%
Golf Course	100%	0%	0%	0%	0%
High Density Residential	100%	0%	0%	0%	0%
Industrial	100%	0%	0%	0%	0%
Institutional	0%	0%	0%	0%	0%
Low Density Residential	0%	0%	0%	0%	0%
Medium Density Residential	0%	0%	0%	0%	0%
TOTAL	100%	0%	0%	0%	0%

Future Land Use Type	Skull Creek North 2 (%)	Mackays Creek North 1 (%)	Mackays Creek North 2 (%)	Mackays Creek North - Corn Island (%)	Broad/Chechessee Trib (%)
Commercial	0%	0%	0%	0%	0%
Golf Course	100%	0%	0%	0%	0%
High Density Residential	100%	0%	0%	0%	0%
Industrial	100%	0%	0%	0%	0%
Institutional	0%	0%	0%	0%	0%
Low Density Residential	0%	0%	0%	100%	0%
Medium Density Residential	95%	0%	0%	0%	0%
TOTAL	100%	0%	0%	100%	0%

TABLE 5-9 (CONTINUED)
WATER QUALITY BASIN BMP COVERAGE
CHECHESSEE RIVER WATERSHED

Existing Land Use Type	Chechessee Creek 1 (%)	Chechessee Creek 2 (%)	Chechessee Trib - Ballenger Neck (%)	Chechessee Trib - Spring Island (%)	TOTAL (%)
Commercial	0%	98%	0%	0%	21%
Golf Course	100%	85%	100%	0%	43%
High Density Residential	0%	0%	0%	0%	100%
Industrial	9%	24%	0%	0%	16%
Institutional	0%	9%	0%	0%	2%
Low Density Residential	0%	15%	0%	0%	5%
Medium Density Residential	0%	15%	0%	0%	5%
TOTAL	37%	36%	0%	0%	22%

Future Land Use Type	Chechessee Creek 1 (%)	Chechessee Creek 2 (%)	Chechessee Trib - Ballenger Neck (%)	Chechessee Trib - Spring Island (%)	TOTAL (%)
Commercial	0%	95%	80%	0%	72%
Golf Course	100%	84%	100%	0%	47%
High Density Residential	0%	0%	0%	0%	100%
Industrial	10%	22%	1%	0%	16%
Institutional	0%	8%	0%	0%	2%
Low Density Residential	47%	54%	48%	0%	40%
Medium Density Residential	7%	13%	82%	0%	34%
TOTAL	54%	52%	62%	0%	41%

TABLE 5-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
CHECHESSEE CREEK WATERSHED

Existing Land Use Type	Chechessee River 1 (%)	Chechessee River 2 (%)	Chechessee River 3 (%)	Chechessee River 4 (%)	Chechessee River 5 (%)	Skull Creek North 1 (%)	Skull Creek North 2 (%)
Commercial	0%	0%	0%	100%	0%	0%	0%
High Density Residential	0%	0%	0%	0%	0%	0%	0%
Industrial	0%	0%	22%	100%	100%	0%	0%
Institutional	0%	0%	0%	0%	100%	0%	0%
Low Density Residential	0%	100%	8%	100%	100%	0%	0%
Medium Density Residential	0%	11%	0%	0%	0%	0%	0%
TOTAL	0%	11%	10%	100%	100%	0%	0%

Future Land Use Type	Chechessee River 1 (%)	Chechessee River 2 (%)	Chechessee River 3 (%)	Chechessee River 4 (%)	Chechessee River 5 (%)	Skull Creek North 1 (%)	Skull Creek North 2 (%)
Commercial	0%	0%	0%	99%	0%	0%	0%
High Density Residential	0%	0%	0%	0%	0%	0%	0%
Industrial	0%	0%	20%	100%	99%	0%	0%
Institutional	0%	0%	0%	0%	100%	0%	0%
Low Density Residential	9%	100%	8%	100%	100%	0%	0%
Medium Density Residential	0%	10%	0%	97%	99%	0%	0%
TOTAL	6%	10%	9%	100%	100%	0%	0%

TABLE 5-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
CHECHESSEE RIVER WATERSHED

Existing Land Use Type	Mackays Creek North 1 (%)	Mackays Creek North 2 (%)	Mackays Creek North - Corn Island (%)	Broad/Chechessee Trib (%)	Chechessee Creek 1 (%)
Commercial	0%	0%	0%	0%	0%
High Density Residential	0%	0%	0%	0%	0%
Industrial	0%	0%	0%	0%	42%
Institutional	0%	0%	0%	0%	0%
Low Density Residential	0%	0%	0%	0%	5%
Medium Density Residential	0%	0%	0%	0%	100%
TOTAL	0%	0%	0%	0%	

Future Land Use Type	Mackays Creek North 1 (%)	Mackays Creek North 2 (%)	Mackays Creek North - Corn Island (%)	Broad/Chechessee Trib (%)	Chechessee Creek 1 (%)
Commercial	0%	0%	0%	0%	0%
High Density Residential	0%	0%	0%	0%	0%
Industrial	0%	0%	0%	0%	41%
Institutional	0%	0%	0%	0%	0%
Low Density Residential	0%	0%	99%	0%	49%
Medium Density Residential	0%	0%	0%	0%	100%
TOTAL	0%	0%	99%	0%	55%

TABLE 5-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
CHECHESSEE CREEK WATERSHED

Existing Land Use Type	Chechessee Creek 2 (%)	Chechessee Trib - Ballenger Neck (%)	Chechessee Trib - Spring Island (%)	TOTAL (%)
Commercial	11%	1%	0%	55%
High Density Residential	0%	0%	0%	0%
Industrial	29%	94%	0%	40%
Institutional	4%	0%	0%	74%
Low Density Residential	59%	100%	0%	33%
Medium Density Residential	42%	100%	2%	33%
TOTAL	50%		0%	33%

Future Land Use Type	Chechessee Creek 2 (%)	Chechessee Trib - Ballenger Neck (%)	Chechessee Trib - Spring Island (%)	TOTAL (%)
Commercial	11%	3%	0%	48%
High Density Residential	0%	0%	0%	0%
Industrial	29%	95%	0%	39%
Institutional	4%	0%	0%	75%
Low Density Residential	68%	100%	0%	54%
Medium Density Residential	42%	100%	2%	49%
TOTAL	59%	98%	2%	49%

TABLE 5-11

AVERAGE ANNUAL LOADS FOR CHECHESSEE RIVER WATERSHED WATER QUALITY BASINS

EXISTING LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Chechessee River 1	5,436	18,252	155,000	419,000	8,478	65,225	305	7,011	1.50E+15
Chechessee River 2	4,433	15,646	129,000	286,000	6,955	55,519	257	6,140	1.25E+15
Chechessee River 3	2,138	6,452	57,121	195,000	2,947	23,168	107	2,353	5.55E+14
Chechessee River 4	523	1,571	15,496	71,057	769	6,352	29	568	2.10E+14
Chechessee River 5	1,156	3,887	34,109	103,000	1,784	14,529	66	1,494	3.96E+14
Skull Creek North 1	516	1,594	14,167	40,867	662	5,514	23	535	1.16E+14
Skull Creek North 2	552	1,617	13,750	40,050	691	5,612	23	535	1.16E+14
Mackays Creek North 1	428	1,303	10,211	31,356	542	4,560	18	437	9.85E+13
Mackays Creek North 2	158	562	4,570	9,621	244	1,986	9	220	4.42E+13
Mackays Creek North - Corn Island	559	1,878	15,631	40,838	822	6,645	30	709	1.52E+14
Broad/Chechessee Trib	143	518	4,229	8,459	226	1,833	8	206	4.09E+13
Chechessee Creek 1	1,452	4,182	39,917	165,000	2,045	15,574	74	1,489	4.35E+14
Chechessee Creek 2	1,582	3,301	40,253	272,000	1,915	13,457	64	877	4.91E+14
Chechessee Trib - Ballenger Neck	493	1,105	11,846	72,138	578	4,743	20	328	1.84E+14
Chechessee Trib - Spring Island	212	606	6,198	30,699	310	2,237	11	211	6.02E+13
TOTAL	19,782	62,474	551,498	1,785,085	28,968	226,954	1,044	23,113	5.65E+15

TABLE 5-11 (CONTINUED)

AVERAGE ANNUAL LOADS FOR CHECHESSEE RIVER WATERSHED WATER QUALITY BASINS

FUTURE LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Chechessee River 1	5,436	18,258	156,000	419,000	8,482	65,270	305	7,014	1.50E+15
Chechessee River 2	4,433	15,648	129,000	272,000	6,907	55,380	254	6,133	1.23E+15
Chechessee River 3	2,138	6,468	57,827	202,000	2,991	23,270	109	2,360	5.62E+14
Chechessee River 4	523	1,571	15,100	62,405	750	6,312	27	564	2.00E+14
Chechessee River 5	1,156	3,887	33,825	97,431	1,767	14,464	65	1,491	3.85E+14
Skull Creek North 1	516	1,594	14,169	40,921	662	5,514	23	535	1.16E+14
Skull Creek North 2	552	1,617	13,759	40,062	697	5,618	23	536	1.16E+14
Mackays Creek North 1	428	1,303	10,211	31,356	542	4,560	18	437	9.85E+13
Mackays Creek North 2	158	562	4,570	9,621	244	1,986	9	220	4.42E+13
Mackays Creek North - Corn Island	559	1,878	15,631	40,838	822	6,645	30	709	1.52E+14
Broad/Chechessee Trib	143	518	4,229	8,459	226	1,833	8	206	4.09E+13
Chechessee Creek 1	1,452	4,182	38,918	144,000	1,998	15,528	70	1,480	4.18E+14
Chechessee Creek 2	1,582	3,301	38,558	236,000	1,819	13,157	58	862	4.34E+14
Chechessee Trib - Ballenger Neck	494	1,206	14,835	78,053	657	5,600	22	350	2.13E+14
Chechessee Trib - Spring Island	212	606	6,198	30,698	310	2,237	11	211	6.02E+13
TOTAL	19,782	62,599	552,830	1,712,844	28,874	227,374	1,032	23,108	5.57E+15
Percent increase over existing		0%	0%	-4%	0%	0%	-1%	0%	-1%

TABLE 5-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS
CHECHESSEE RIVER WATERSHED

Water Quality Basin ID	DHEC Station(s)	Fecal Coliform Concentrations				Level of Service
		Long-Term Average		Maximum 36-Sample Values		
		Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)	
Chechessee River 1	None	NA	NA	NA	NA	NA
Chechessee River 2	None	NA	NA	NA	NA	NA
Chechessee River 3	17-07	3.1	8	3.6	11	A
Chechessee River 4	17-08	3.1	8	3.4	11	A
Chechessee River 5	17-17	3.6	13	4.3	14	A
Skull Creek North 1	20-13	3.4	11	3.9	18	A
Skull Creek North 2	None	NA	NA	NA	NA	NA
Mackays Creek North 1	20-09	2.9	8	3.5	13	A
Mackays Creek North 2	None	NA	NA	NA	NA	NA
Mackays Creek North - Corn Island	None	NA	NA	NA	NA	NA
Broad/Chechessee Trib	None	NA	NA	NA	NA	NA
Chechessee Creek 1	18-12, 18-13	5.4	23	6.3	33	A
Chechessee Creek 2	18-10, 18-11, 18-14	10.4	33	13.6	47	D
Chechessee Trib - Ballenger Neck	None	NA	NA	NA	NA	NA
Chechessee Trib - Spring Island	None	NA	NA	NA	NA	NA

TABLE 5-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

CHECHESSEE RIVER WATERSHED

Water Quality Basin ID	South WASP Segment	Exchange with Water Quality Basin ID	Tidal Dispersion Values		
			Area (m^2)	Length (m)	Coefficient (m^2/s)
Chechessee River 1	36	Broad River	16,660	3,059	150
Chechessee River 2	37	Chechessee River 1	8,871	5,021	150
		Colleton River 1	5,688	5,724	180
Chechessee River 3	38	Chechessee River 2	2,830	4,924	100
Chechessee River 4	39	Chechessee River 3	1,556	2,494	20
Chechessee River 5	40	Chechessee River 4	1,266	1,366	20
Skull Creek North 1	41	Chechessee River 1	2,366	954	75
Skull Creek North 2	42	Skull Creek North 1	2,068	1,191	75
Mackays Creek North 1	43	Chechessee River 1	1,102	1,447	450
Mackays Creek North 2	44	Mackays Creek North 1	1,010	949	450
Mackays Creek North - Corn Island	45	Mackays Creek North 2	627	467	450
Broad/Chechessee Trib	46	Broad River	1,285	954	150
		Chechessee River 2	1,338	954	150
Chechessee Creek 1	47	Chechessee River 3	1,641	4,342	50
Chechessee Creek 2	48	Chechessee Creek 1	418	3,460	50
Chechessee Trib - Ballenger Neck	49	Chechessee Creek 1	221	1,086	20
Chechessee Trib - Spring Island	50	Chechessee Creek 2	473	921	20

TABLE 5-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMM
FOR CHECHESSEE RIVER WATER QUALITY BASINS

Water Quality Basin ID	South WASP Segment	EXISTING LAND USE Flow (cfs)	EXISTING LAND USE Fecal Coliform (#/100 ml)	FUTURE LAND USE Flow (cfs)	FUTURE LAND USE Fecal Coliform (#/100 ml)
Chechessee River 1	36	25.2	1,391	25.2	1,387
Chechessee River 2	37	21.6	1,411	21.6	1,410
Chechessee River 3	38	8.9	1,349	8.9	1,353
Chechessee River 4	39	2.2	1,426	2.2	1,379
Chechessee River 5	40	5.4	1,421	5.4	1,406
Skull Creek North 1	41	2.2	1,211	2.2	1,211
Skull Creek North 2	42	2.2	1,188	2.2	1,187
Mackays Creek North 1	43	1.8	1,297	1.8	1,297
Mackays Creek North 2	44	0.8	1,409	0.8	1,409
Mackays Creek North - Corn Island	45	2.6	1,387	2.6	1,387
Broad/Chechessee Trib	46	0.7	1,422	0.7	1,422
Chechessee Creek 1	47	5.8	1,362	5.8	1,321
Chechessee Creek 2	48	4.6	1,268	4.6	1,182
Chechessee Trib - Ballenger Neck	49	1.5	1,286	1.7	1,343
Chechessee Trib - Spring Island	50	0.8	1,413	0.8	1,413

TABLE 5-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES
CHECHESSEE RIVER WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs)	
		Existing	Future
Chechessee River 1	Broad River	788	794
	Skull Creek North 1	688	690
	Mackays Creek North 1	711	711
Chechessee River 2	Chechessee River 1	2,160	2,169
Colleton River 1	Chechessee River 2	104	110
Chechessee River 3	Chechessee River 2	29	29.3
Chechessee River 4	Chechessee River 3	7.5	7.5
Chechessee River 5	Chechessee River 4	5.4	5.4
Skull Creek North 1	Skull Creek North 2	690	692
Skull Creek North 2	Skull Creek South 2	692	694
Mackays Creek North 1	Mackays Creek North 2	713	713
Mackays Creek North 2	Calibogue Sound 5	717	717
Mackays Creek North - Corn Island	Mackays Creek North 2	2.6	2.6
Broad River	Broad/Chechessee Trib	2,006	2,007
Broad/Chechessee Trib	Chechessee Creek 2	2,006	2,007
Chechessee Creek 1	Chechessee River 3	13	12.8
Chechessee Creek 2	Chechessee Creek 1	5.4	5.4
Chechessee Trib - Ballenger Neck	Chechessee Creek 1	1.5	1.7
Chechessee Trib - Spring Island	Chechessee Creek 2	0.8	0.8

TABLE 5-16

FECAL COLIFORM MODELING RESULTS
CHECHESSEE RIVER WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)		Modeled Level of Service	
		Existing	Future	Existing	Future
Chechessee River 1	0.5	3.3	3.3	A	A
Chechessee River 2	0.5	3.1	3.1	A	A
Chechessee River 3	0.8	3.5	3.5	A	A
Chechessee River 4	1.0	3.0	2.9	A	A
Chechessee River 5	4.0	3.8	3.7	A	A
Skull Creek North 1	1.0	3.3	3.3	A	A
Skull Creek North 2	1.0	3.3	3.4	A	A
Mackays Creek North 1	1.0	3.5	3.5	A	A
Mackays Creek North 2	1.0	3.7	3.7	A	A
Mackays Creek North - Corn Island	1.0	3.8	3.8	A	A
Broad/Chechessee Trib	1.0	3.0	3.0	A	A
Chechessee Creek 1	0.7	6.2	6.0	A	A
Chechessee Creek 2	1.0	9.5	8.9	C	C
Chechessee Trib - Ballenger Neck	1.4	11.6	10.5	D	D
Chechessee Trib - Spring Island	1.4	12.7	12.2	D	D

TABLE 5-17

SENSITIVITY ANALYSIS RESULTS
CHECHESSEE RIVER WATERSHED

Water Quality	Bacteria	Modeled Geomean Conc (#/100 ml)		Modeled Level of Service	
Basin ID	Loss Rate (1/day)	Best Case	Worst Case	Best Case	Worst Case
Chechessee River 1	0.5	3.2	3.3	A	A
Chechessee River 2	0.5	2.9	3.1	A	A
Chechessee River 3	0.8	3.3	3.6	A	A
Chechessee River 4	1.0	2.7	3.0	A	A
Chechessee River 5	4.0	3.6	3.8	A	A
Skull Creek North 1	1.0	3.2	3.4	A	A
Skull Creek North 2	1.0	3.2	3.6	A	A
Mackays Creek North 1	1.0	3.4	3.6	A	A
Mackays Creek North 2	1.0	3.6	3.7	A	A
Mackays Creek North - Corn Island	1.0	3.7	3.8	A	A
Broad/Chechessee Trib	1.0	3.0	3.0	A	A
Chechessee Creek 1	0.7	5.4	6.5	A	A
Chechessee Creek 2	1.0	7.3	10.2	B	D
Chechessee Trib - Ballenger Neck	1.4	9.6	14.0	C	D
Chechessee Trib - Spring Island	1.4	10.0	13.4	C	D

NOTES:

1. Best case represents existing land use with wet detention BMPs serving all existing development.
2. Worst case represents future land use with no BMPs.
3. Water quality segments that show change from base model results (e.g., improved LOS for best case or degraded LOS for worst case) are highlighted.

TABLE 5-18

PLANNING LEVEL COST ESTIMATES FOR
CHECHESSEE RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
CRW_T1-5	Road overtopping at Callawassee Drive	$29,000
	Replace existing 1 - 18" RCP with 1 - 48" RCP	
SI2_M-2 *	Road overtopping at Shrimp Pond Road	$48,000
	Replace existing 1 - 15" RCP with 4 - 36" RCP	
	Replace existing riser structure with rectangular riser with 1 - 24"x72" horizontal weir	
	Replace existing bubbler with rectangular bubbler with 1 - 24"x72" horizontal weir	
	TOTAL	$77,000

* Conduits marked by asterisk are on private land

Costs are in December 2004 dollars.

See Appendix C for basis of cost estimates.

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Chechessee River Watershed

Figure 5-1

CDM Camp Dresser & McKee Inc.

Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ChechesseeRiverWatershed_figure5-1.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Chechessee River Watershed Hydrologic Model Subbasins

Figure 5-2

Job Number: 15178.00	Scale: 1" = 6,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ChechesseeRiverHH_figure5-2.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Figure 5-3

Chechessee River Watershed Water Quality Basins

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Chechessee River Watershed PSMS Problem Areas

Figure 5-4

Job Number: 15178.00	Scale: 1" = 6,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ChechesseeRiverHH-problems_figure5-4.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Subbasins	T&H / CDM	2004
Problem Locations	T&H / CDM	2004

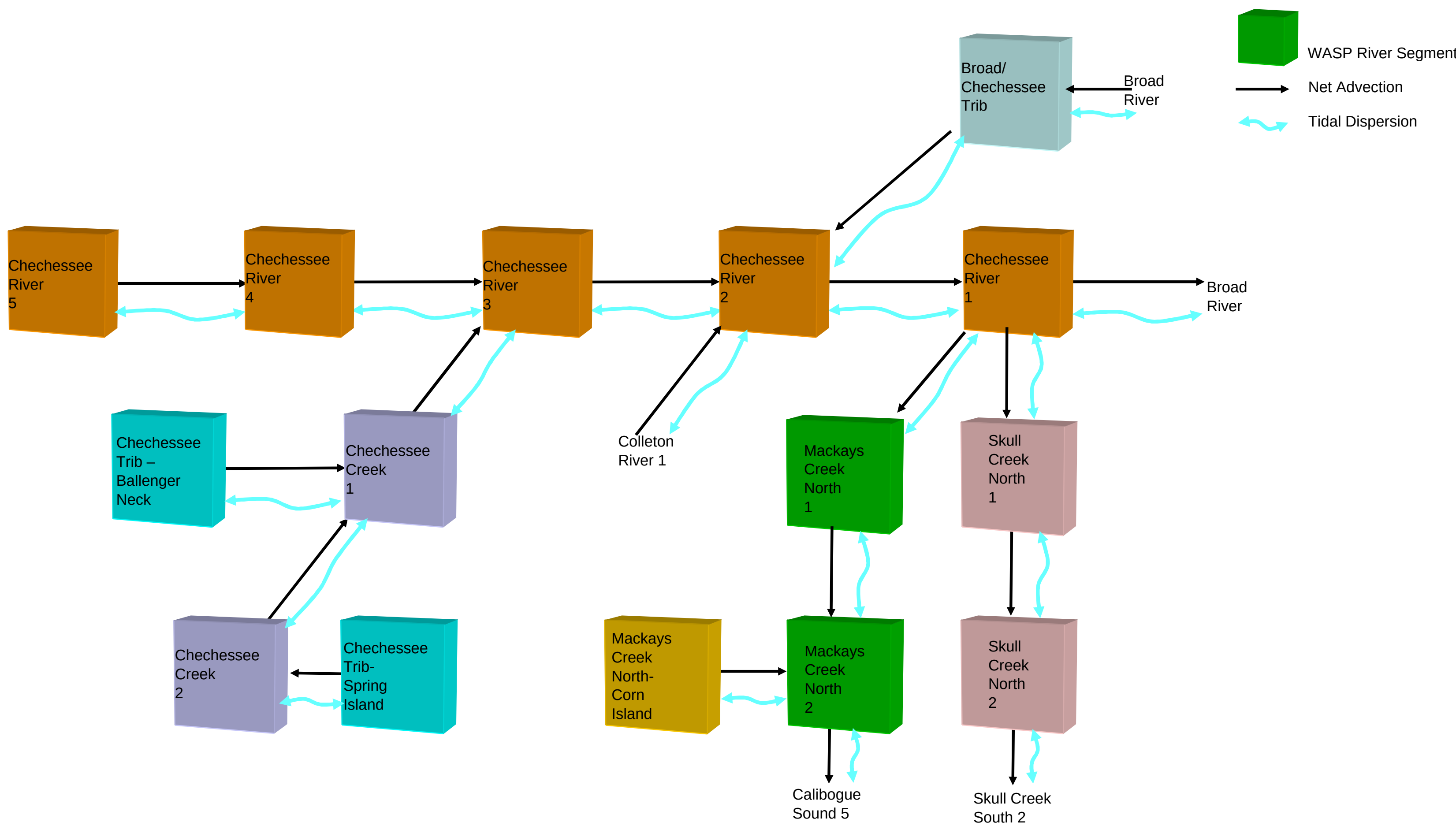

Figure 5-5
WASP Model Schematic for Chechessee River Watershed

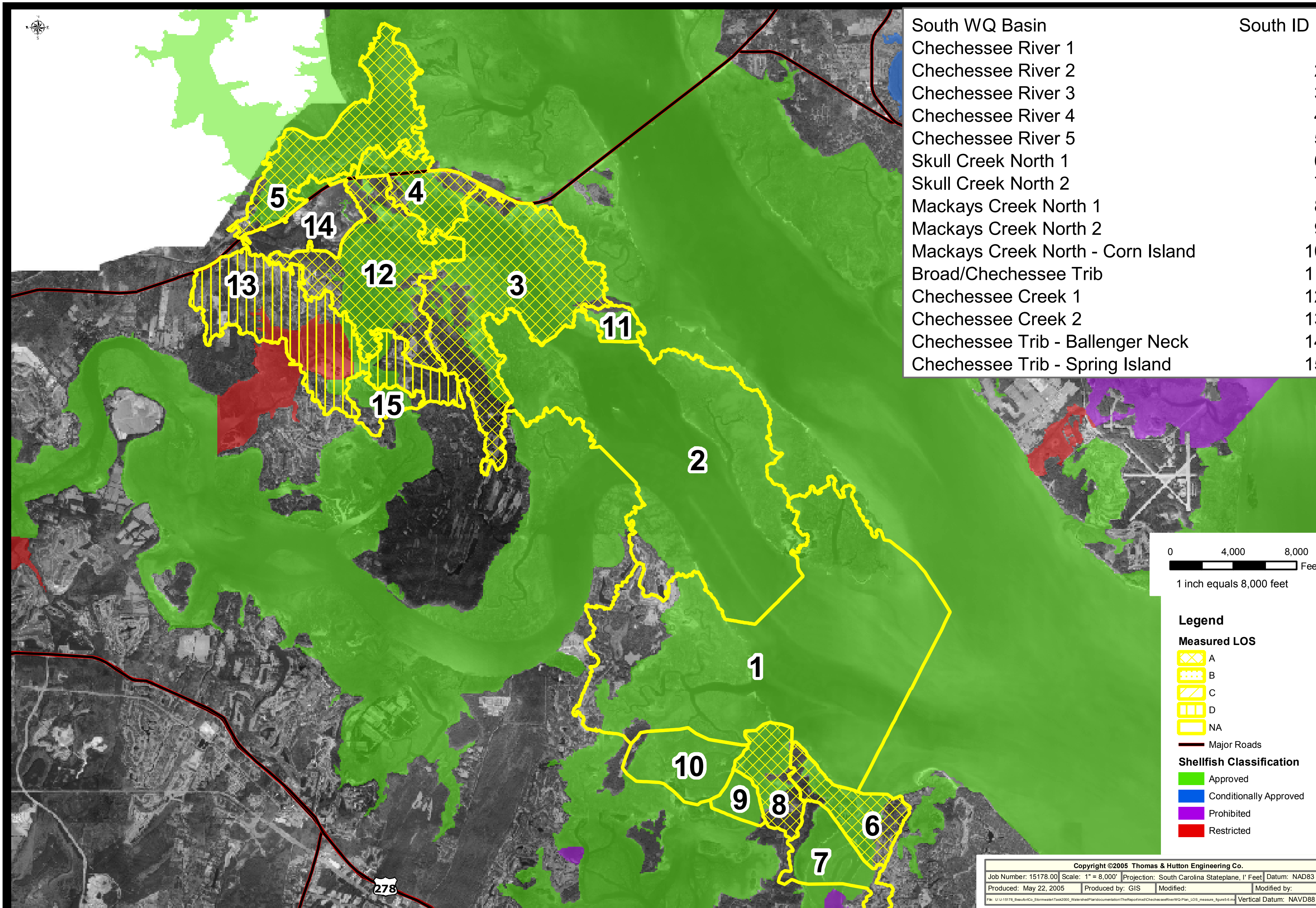

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Figure 5-6

Chechessee River Watershed
Shellfish Classification and Existing Level of Service

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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Aerial Photography	Beaufort County	2002
Shellfish Classifications	SCDHEC	2004

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

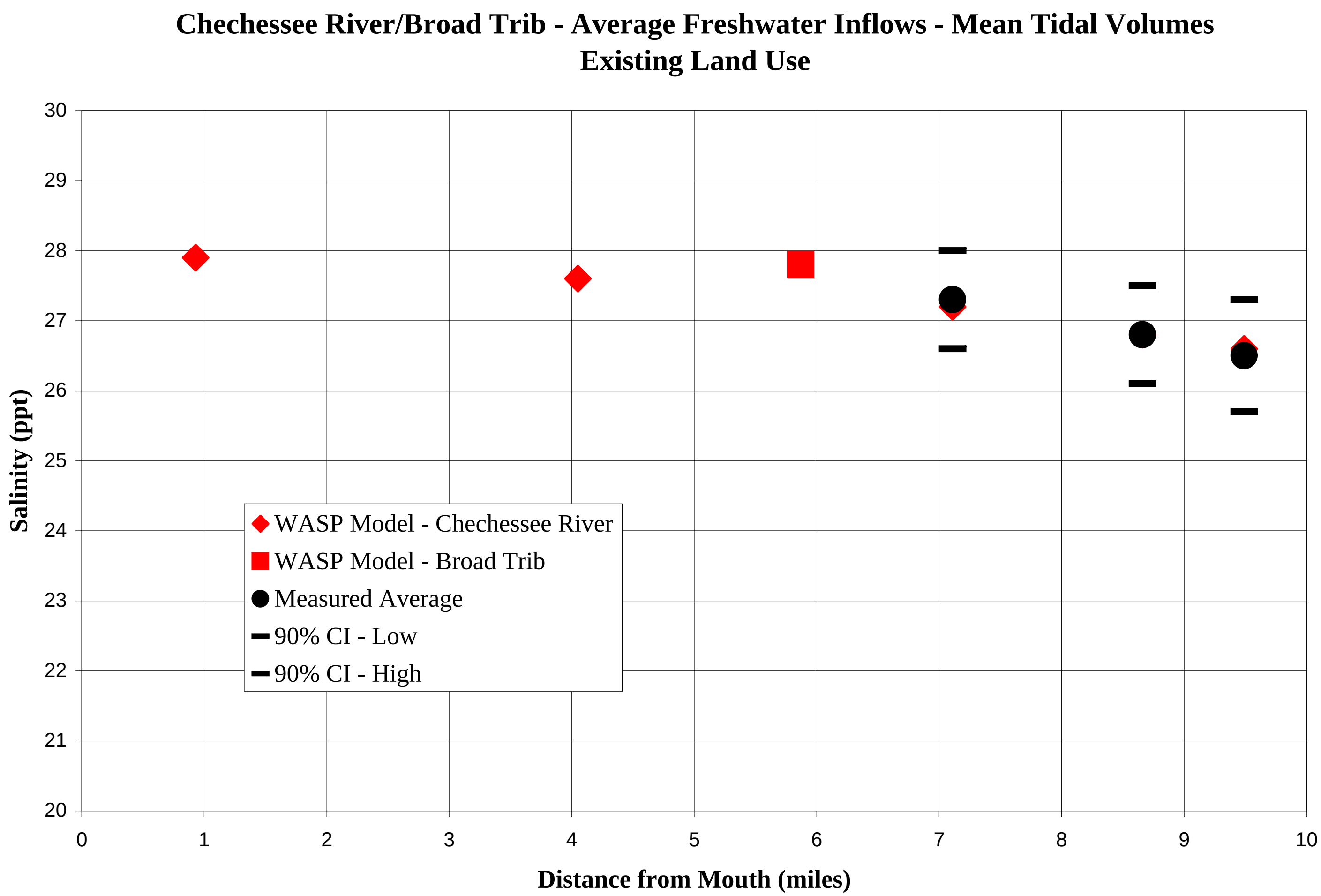

Figure 5-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Chechessee River - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

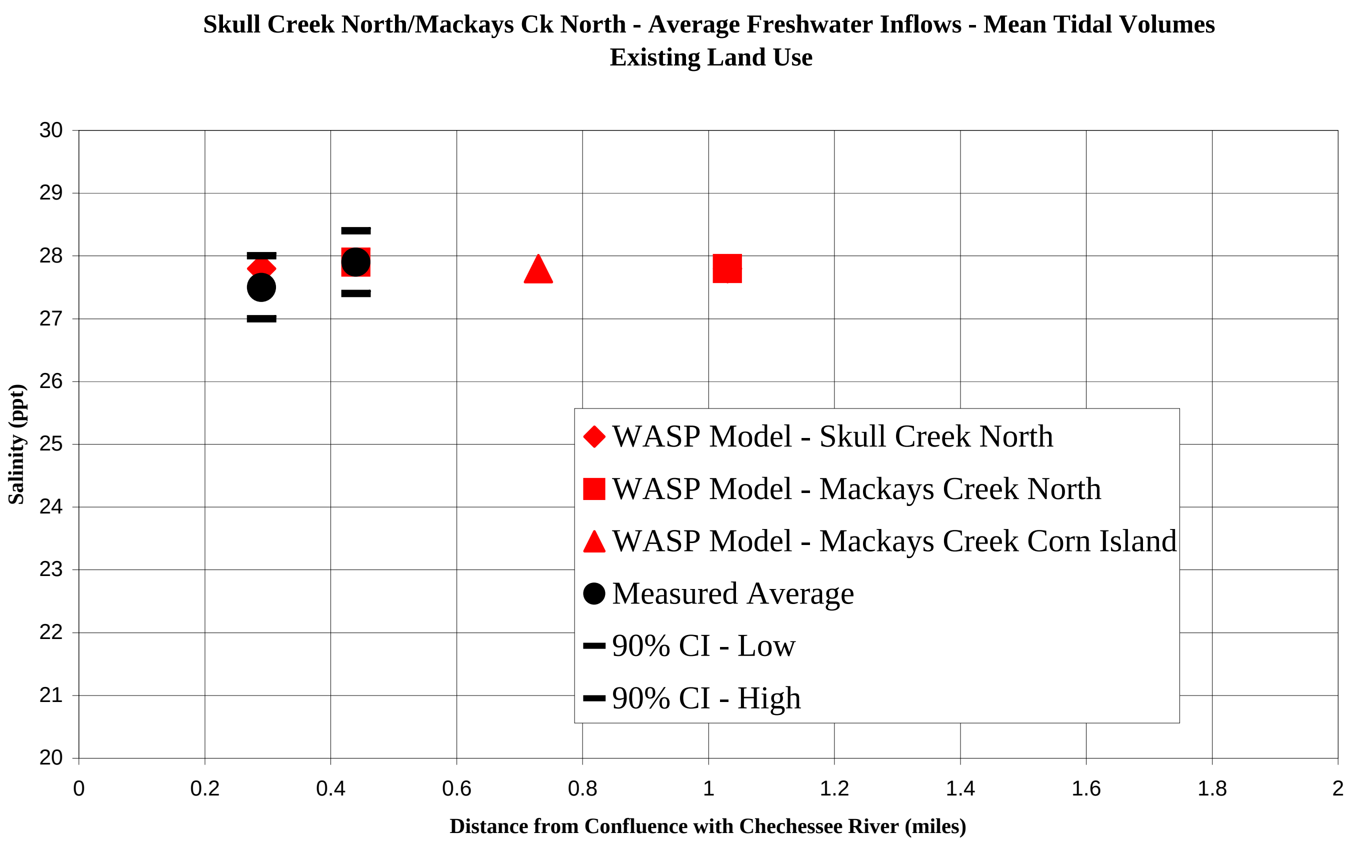

Figure 5-8. Comparison of WASP Model Results with Long-Term Monitoring Data
in Skull Creek North/Mackays Creek North - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

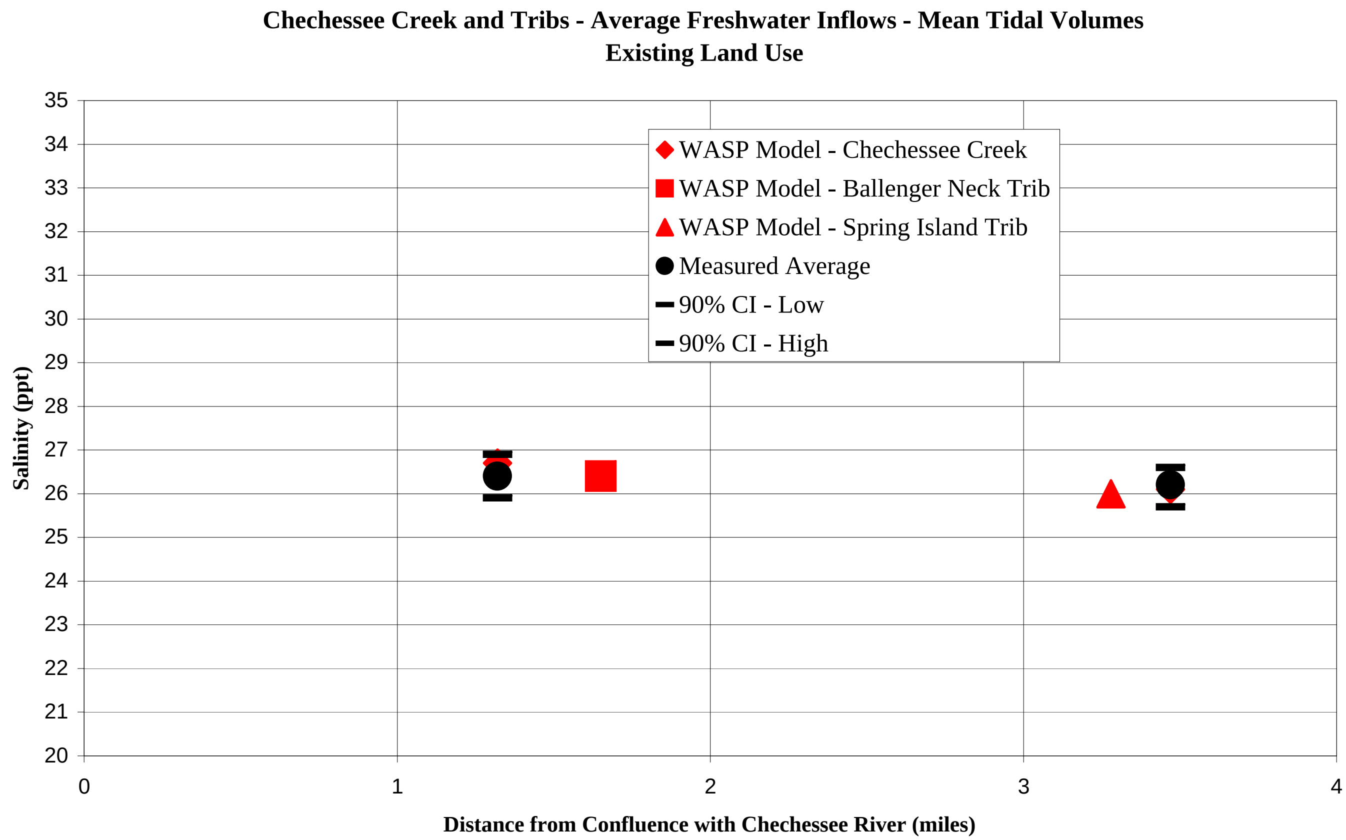

Figure 5-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Chechessee Creek and Tributaries - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

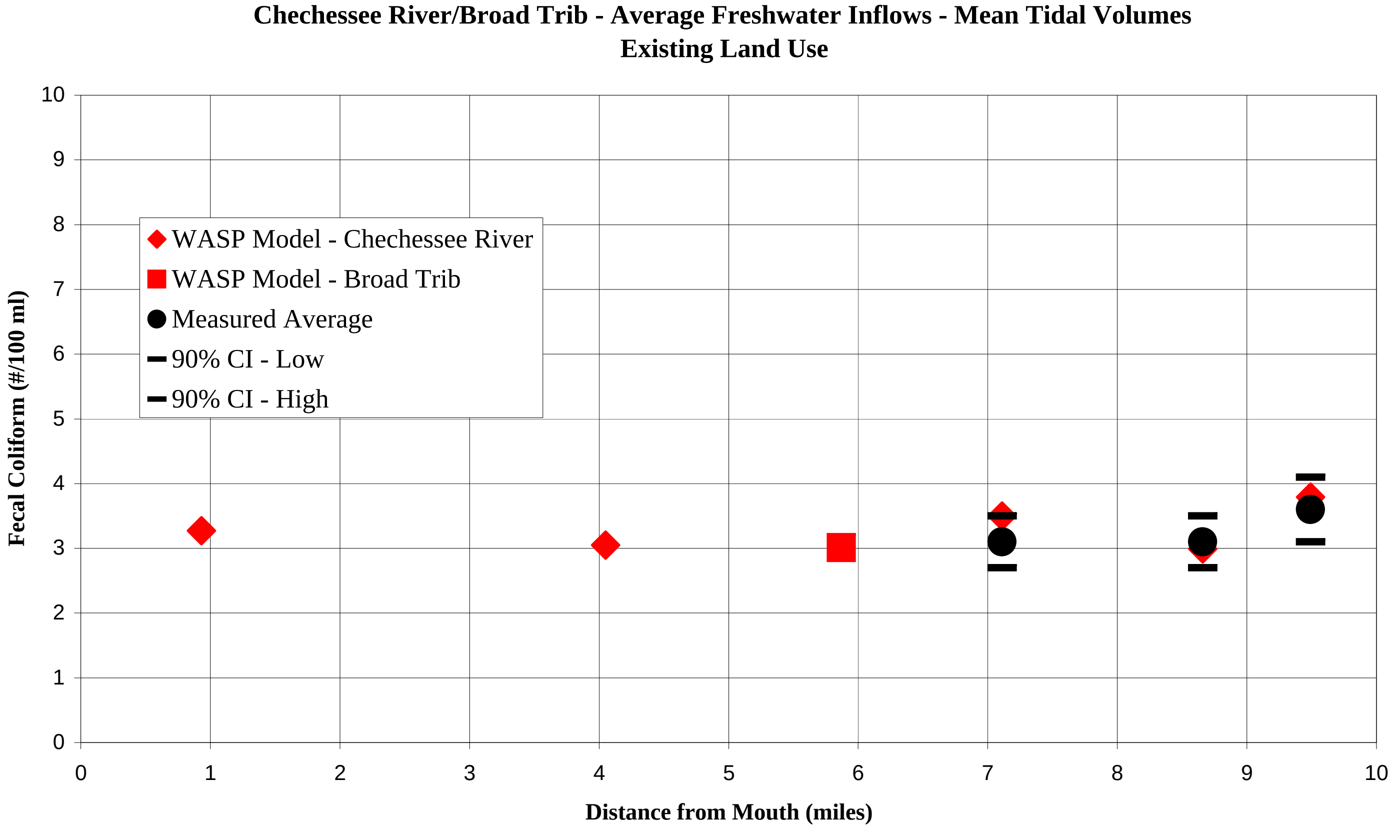

Figure 5-10. Comparison of WASP Model Results with Long-Term Monitoring Data in Chechessee River - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

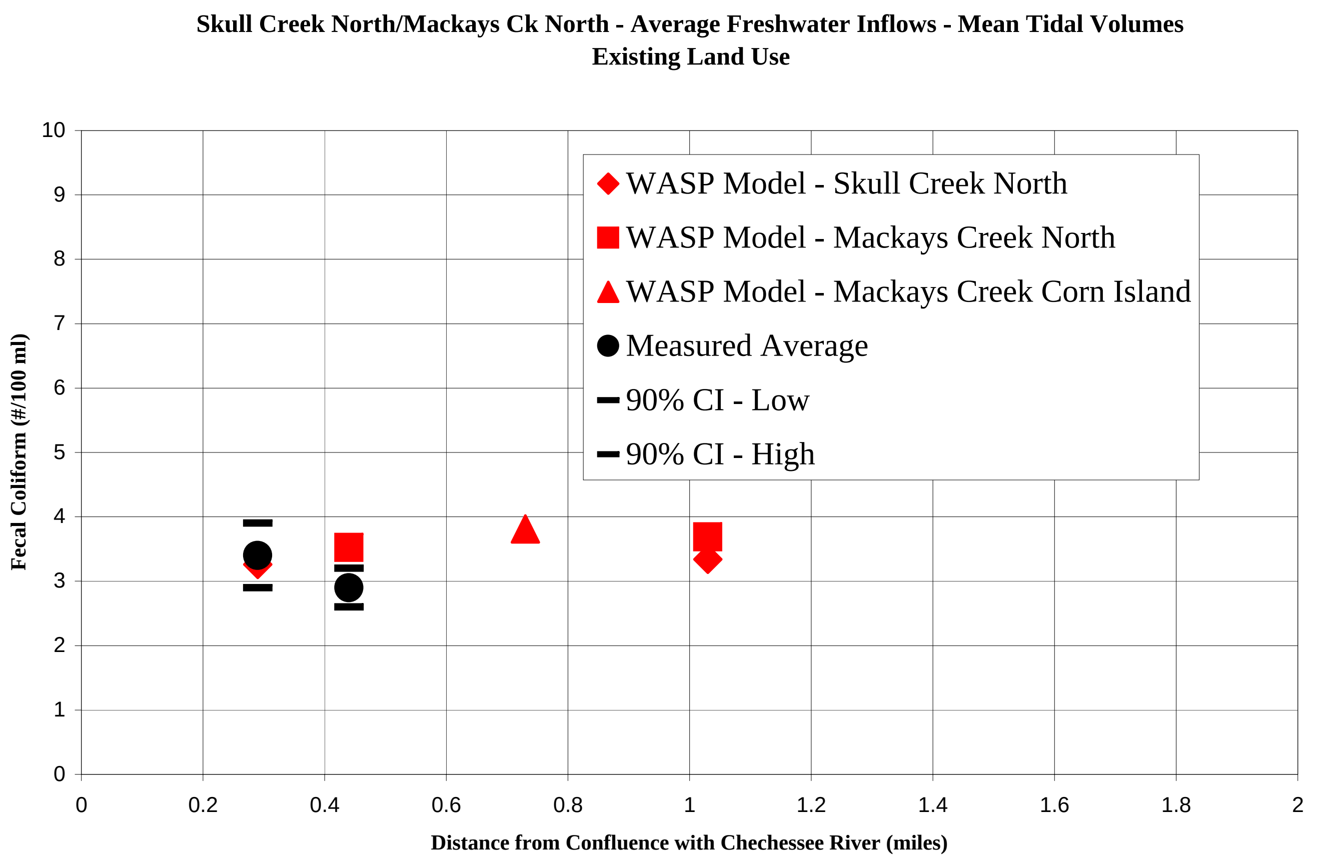

Figure 5-11. Comparison of WASP Model Results with Long-Term Monitoring Data in Skull Creek North/Mackays Creek North - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Chechessee Creek and Tribs - Average Freshwater Inflows - Mean Tidal Volumes
Existing Land Use

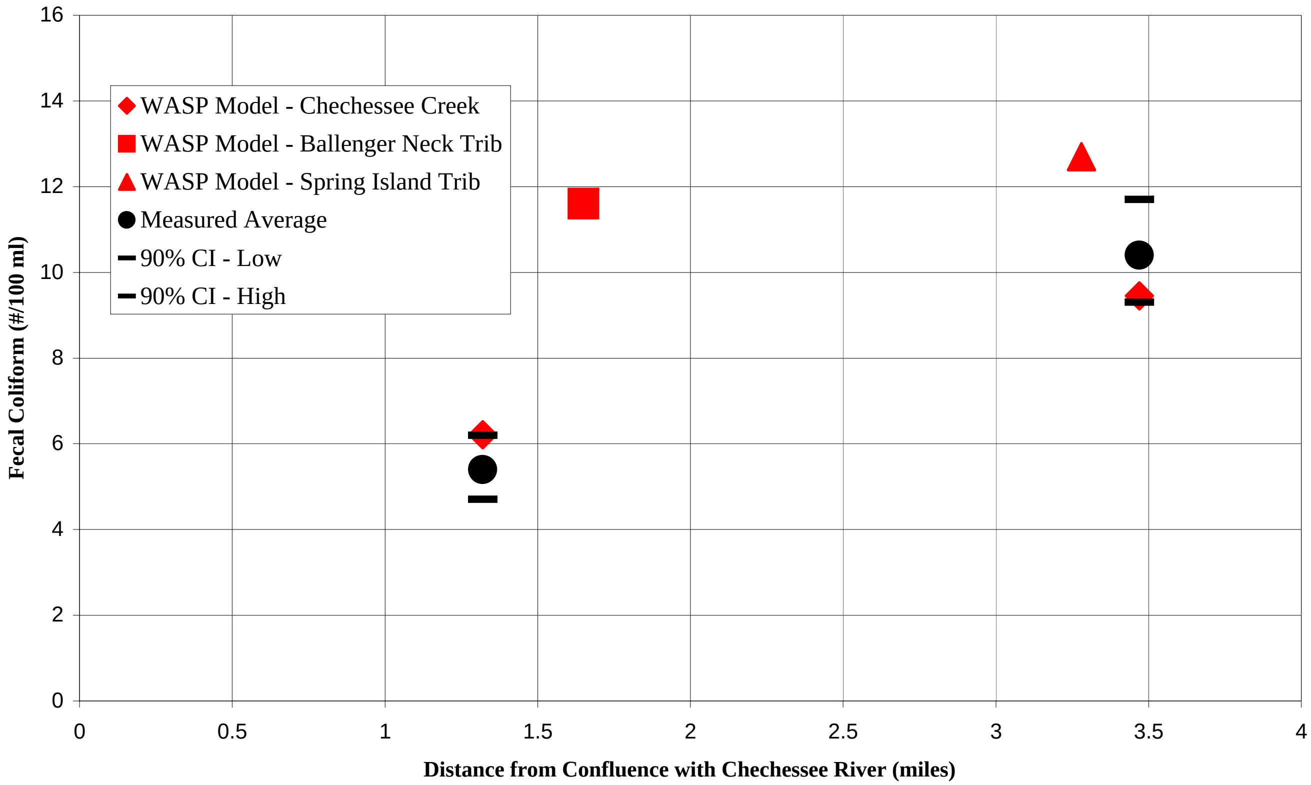

Figure 5-12. Comparison of WASP Model Results with Long-Term Monitoring Data in Chechessee Creek and Tributaries - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ChechesseeRiverWQ-Plan_figure5-13.mxd			Vertical Datum: NAVD88

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Figure 5-13

Chechessee River Watershed Water Quality Plan Elements

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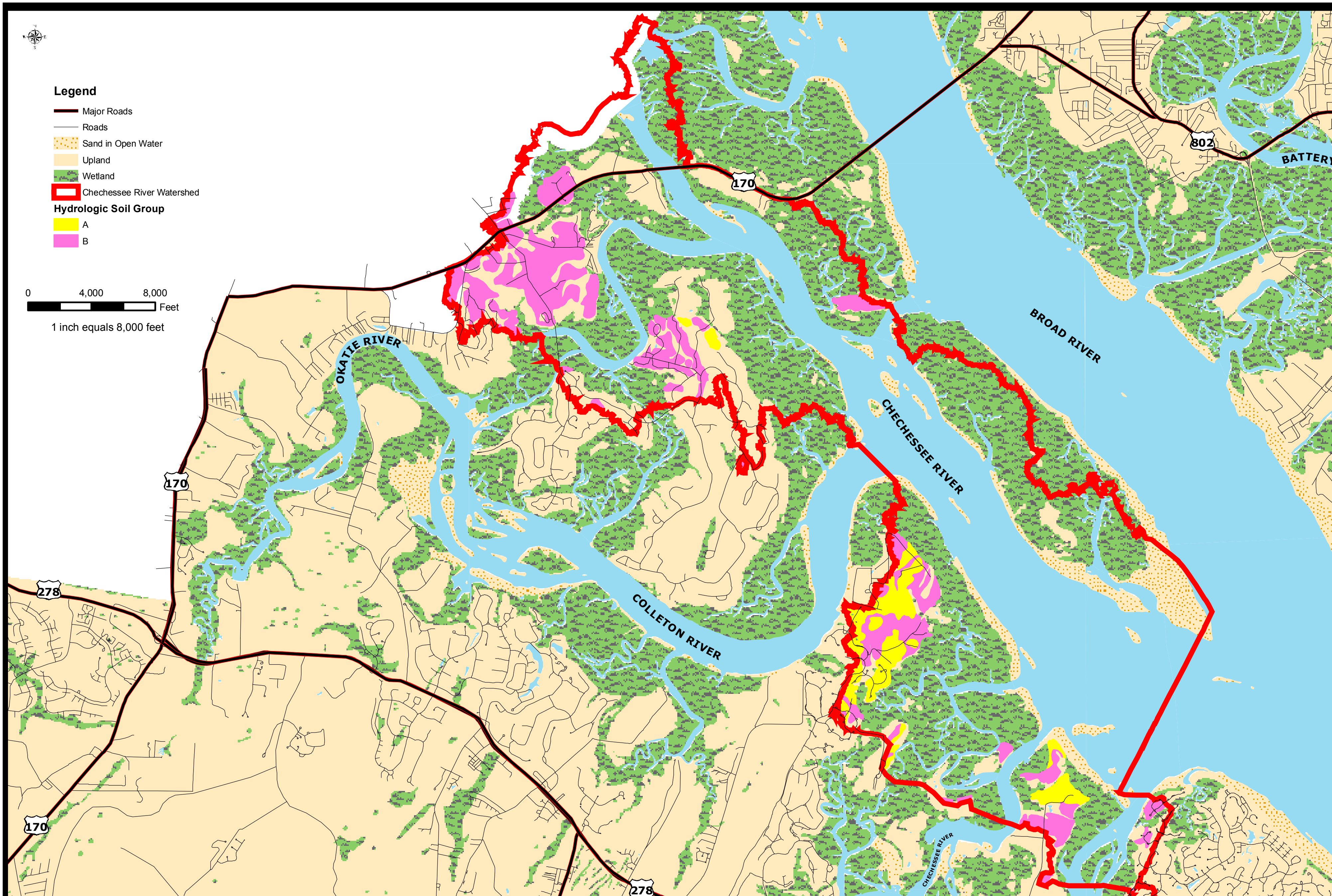
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Existing WQ Information	SC DHEC	2005

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
WQ Recommendations	T&H / CDM	2004



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Chechessee River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 5-14

CDM Camp Dresser & McKee Inc.

Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ChechesseeRiverSoils_figure5-14.mxd			Vertical Datum: NAVD88

Disclaimer
Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Section 6
Colleton River Watershed Analysis

This section describes the physical features of the Colleton River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

6.1 Overview

The Colleton River watershed is located south of the Broad River (see Figure 6-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area primarily in Bluffton Township that is tributary to the Colleton River. Major Colleton River tributaries included in the analysis are the Okatie River (headwater end of Colleton River), Sawmill Creek and Callawassee Creek.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several hydrologic basins. These are listed in Table 6-1 and presented in Figure 6-2. Table 6-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were updated to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into “water quality” basins, and the tidal receiving waters were subdivided into receiving “water segments”. These are listed in Table 6-2 and presented in Figure 6-3. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were done to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

6.2 Hydrologic and Hydraulic Analysis

The ICPR, Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the Colleton River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP.

6.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Colleton River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 6-3 lists the hydrologic parameter values for the Colleton River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. . In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the Colleton River PSMS basins is presented in Table 6-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 6-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

6.2.2 Model Results

Tables in Appendix D list summary of the results of the updated study including Updated Areas and CNs values for the Colleton River subbasins.

For existing land use, aerial maps generated in the summer of 2016 and local information were used to estimate the percentage of existing urban development.

Tables in Appendix D also includes tables that list the peak water elevation values for model node locations along the Colleton River PSMS. Specific problem areas identified by the modeling are listed in Table 6-6 and presented in Figure 6-4. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs, and results showed that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded.

Table 6-6 indicates the road crossings that are being overtopped by the design storm events.

Evaluation of solutions to prevent these problems is discussed in the next section of this report.

6.2.3 Management Strategy Alternatives

The problems areas listed in Table 6-6 were evaluated by reviewing the previous report results and reviewing the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 6-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were often used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

6.3 Water Quality Analysis

ATM used the WMM and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the Colleton River watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions.

It should be noted that the analysis includes about 2,900 acres of tributary area that is located in Jasper County.

6.3.1 Land Use and BMP Coverage

Table 6-8 presents the existing land use estimates for the Colleton River water quality basins. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 41 percent of the Colleton River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 59 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 13 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in Table 6-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County in accordance with the County BMP Manual. Values are presented for developed urban land uses. The “total” value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall “total” BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 14 percent of the urban land area in the watershed is served by BMPs.

6.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in Table 6-10. The existing land use values reflect areas that are not designated as “sewered” areas by the Beaufort-Jasper Water & Sewer Authority.

Values are presented for developed urban land uses. The “total” value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall “total” septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 27 percent of the urban systems in the watershed are served by septic tanks.

Based on available data, the estimated wastewater discharge under existing conditions is 0.6 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be 0.8 mgd based on increase in residential land between existing and future conditions. There are no direct discharges to receiving waters in the watershed.

6.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Colleton River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing land use conditions.

The results are presented in Table 6-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr). The total loads are presented for all basins (including small basins in Jasper County that are tributary to the watershed) and for Beaufort County watershed area only.

Wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing discharge of wastewater is limited to roughly 0.6 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be higher (0.8 mgd). Using the values in Table 2-9, the wastewater load for existing conditions accounts for 0.6 to 0.9 percent of the total watershed load for nutrients (TN and TP) and 0.0 to 0.2 percent of the load for other constituents. In the future condition, the wastewater load accounts for 0.8 to 1.2 percent of the total watershed load for nutrients (TN and TP) and 0.0 to 0.2 percent of the load for other constituents.

6.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Colleton River watershed. The model actually includes Calibogue Sound, May River, Colleton River, and Chechessee River watersheds because they are interconnected at several points. Only the Colleton River will be discussed in this section. A schematic of the model is presented as Figure 6-5.

Existing conditions for bacteria concentrations in the Colleton River are presented in Table 6-12. For each water quality basin river reach, the table lists the SCDHEC stations for which the bacteria data were analyzed, the concentrations calculated in the analysis, water quality concentration trends and the LOS associated with these concentrations (as discussed in Section 2.6.2). As shown in the table, SCDHEC data were only available in six of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations in the Colleton River river segments meet the water quality standards, and so these segments have an “A” LOS. In contrast, the Okatie River 2 segment exceeds both the geomean and 90th percentile standards and has a “D” LOS. The Okatie River 1 segment is also considered a “D” segment based on the methodology discussed in Section 2.6.2, even though the measured data did not show an exceedance of either the geomean or

90th percentile standards during the 1990s. The Colleton River – Tidal Flats segment also has a "C" LOS with exceedance of both bacteria standards.

For informational purposes, Figure 6-6 presents a map of the LOS based on the monitoring data analysis, compared to SCDHEC "shellfish classification" (based on the 2016 SCDHEC reports for shellfish areas 18). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the LOS, so there may not be a direct relationship between LOS and shellfish classification presented in the map. In general, however, segments with an "A" LOS are expected to have the lowest probability of receiving a "restricted" classification, and segments with a "D" LOS are expected to have the highest probability of receiving a "restricted" classification.

Physical characteristics assigned to the model reaches are presented in Table 6-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the "characteristic length" (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the SCDHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Table 6-14 presents the values used in the existing condition model. The flow to the tidal river segments comes primarily from direct rainfall on the open water and tidal wetlands, as opposed to stormwater runoff and baseflow.

Table 6-15 shows the net advective flows between segments.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. The calibrated loss-rate coefficients from the 2006 study were used in the updated simulations.

Figures 6-7 and 6-8 are graphs showing a comparison between measured and modeled salinity data along the Colleton River main stem and the Colleton River Tidal Flats, respectively. The figures show that the salinity data calculated by the model is very close to the average measured value and is in all cases well within the 90 percent confidence interval of the mean of the salinity data.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration is presented in Figures 6-9 and 6-10. The graphs show good agreement between the measured values and the model results with the model underestimating concentrations in the upper portions of the river (Okatie River 1 and 2).

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 6-16. The loss rates ranged from 0.5/day to 1.0/day. The lowest values are applied at the downstream end of the Colleton River, and the highest values are applied at the upstream end of the river. This makes sense if it is presumed that bacteria loss is in part due to light mortality, because the water depths are much greater at the downstream end of the Colleton River, and therefore light would be less of a factor relative to the shallower reaches at the upstream end.

Based on water quality sampling data and model results, the following conclusions are:

- Problem basins include Okatie River 1, 2 and 3, Sawmill Branch 1 and 2 and Colleton River - Tidal Flats
- Two new regional water quality BMPs are proposed in Sawmill Branch 1 and 2

Discussion of water quality related recommendations for monitoring and regional BMPs in the Colleton River watershed are presented as part of the overall recommended monitoring and CIP program for Beaufort County contained in the Appendix of this report.

6.3.5 Management Strategy Alternatives

In analyzing the watershed, two feasible regional detention sites were identified. The area tributary to the Sawmill Branch 1 Regional BMP site includes approximately 310 acres of commercial and single-family development built prior to volume control stormwater regulations. There are stormwater best management practices, such as detention facilities, in the area. The project would be to construct a regional detention facility to provide stormwater runoff water quality treatment and volume reduction. Due to the presence of multiple wetlands in the area, project design would involve delineation and avoidance of the wetlands. It would be implemented so that wetlands were not disturbed, by digging the wet detention pond outside of the delineated wetlands and maintaining the normal pool level of the wet detention pond at the approximate wetland elevation based on the LiDAR data.

A new WMM scenario was developed for the Sawmill Branch 1 Regional BMP and its contributing basin using the updated WMM database.   Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Sawmill Branch 1 water quality basin of approximately 12%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Colleton River:

Parameter	lbs/yr removed
Total Nitrogen	809
Total Phosphorus	190
TSS	95,912

The area tributary to the Sawmill Branch 2 Regional BMP site includes approximately 270 acres of commercial and single-family development built prior to volume control stormwater regulations as well as some undeveloped area. There are stormwater best management practices, such as detention facilities, in the area. The project would be to construct a regional detention facility to provide stormwater runoff water quality treatment and volume reduction. Due to the presence of multiple wetlands in the area, project design would involve delineation and avoidance of the wetlands. It would be implemented so that wetlands were not disturbed, by digging the wet detention pond outside of the delineated wetlands and maintaining the normal pool level of the wet detention pond at the approximate wetland elevation based on the LiDAR data.

A new WMM scenario was developed for the Sawmill Branch 2 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Sawmill Branch 2 water quality basin of approximately 11%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Colleton River:

Parameter	lb/yr removed
Total Nitrogen	247
Total Phosphorus	72
TSS	28,495

For the water quality basins identified above, additional controls should be considered. This could include retrofit of existing development that does not have BMPs, and modification of existing ponds that may not have been designed for water quality control.

For informational purposes, the areas with “A” and “B” type soils are presented in Figure 6-12. In general, these soils are more suitable for infiltration BMPs than areas with “C” and “D” type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

6.4 Planning Level Cost Estimates for Management Alternatives

Table 6-20 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Colleton River watershed. As shown in the table, the projects are estimated to have a total cost of $3.545 million based on January 2018 dollars. Details of the cost estimate for each project are shown in Appendix D.

Two regional CIP projects were identified in the Colleton River watershed. These two projects are estimated to have a total cost of $3.13 million and are detailed in the CIP in Appendix O.

TABLE 6-1 (Updated 2017)
HYDROLOGIC BASINS
COLLETON RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Belfair East	277	2	138
Berkeley Creek	876	5	175
Burnt Church	606	4	151
Callawassee Island	144	1	144
Camp St. Mary's	823	3	274
Kitty's Crossing*	958	4	240
Okatie Center	345	1	345
Okatie West	3,042	10	304
Pepper Hall	208	2	104
Pinkney Colony South	417	3	139
Rose Hill East	958	3	319
Sawmill Creek	1,062	4	266
Sawmill Creek East	358	1	358
Sawmill Creek West	189	1	189
Simmonsville/Hidden Lakes Canal	1,675	7	239
Spring Island 1	339	1	339
Spring Island 3	205	1	205
Spring Island 4	218	1	218
Spring Island 5	211	1	211
Wadell	368	2	184
TOTAL	13,278	57	233

* ATM Updated Areas (based on updated and improved watershed delineations)

TABLE 6-2
WATER QUALITY BASINS
COLLETON RIVER WATERSHED

Basin Name	Tributary Area (acres)
Colleton River 1	3,740
Colleton River 2	5,856
Colleton River 3	6,291
Okatie River 1	4,348
Okatie River 2	930
Okatie River 3	3,452
Sawmill Creek 1	3,319
Sawmill Creek 2	1,186
Callawassee Creek 1	1,548
Callawassee Creek 2	455
Colleton Tidal Flats	656
Jasper County 1	618
Jasper County 2	1,890
Jasper County 3	412
TOTAL	34,701

TABLE 6-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
COLLETON RIVER WATERSHED

		Existing Land Use		Future Land Use	
ICPR Subbasin ID	Tributary Area (acres)	Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Belfair East Subwatershed					
BRE_M1	80	64	83	61	83
BRE_M2	197	81	80	78	80
Berkeley Creek Subwatershed					
BC_M1	98	84	61	85	61
BC_M2	179	86	62	84	61
BC_M3	326	85	112	89	104
BC_T1A	119	85	68	89	68
BC_T1B	155	83	65	91	55
Burnt Church Subwatershed					
BTC_M1	277	82	86	83	85
BTC_M2	146	92	62	94	57
BTC_M3*	128	88	81	91	68
BTC_T1	65	92	45	94	41
Callawassee Island Subwatershed					
CI_M1	144	74	81	74	81
Camp St. Mary's Subwatershed					
CSM_M1	155	71	94	75	87
CSM_M2	504	81	160	82	155
CSM_T1	164	63	163	63	163
Kitty's Crossing Subwatershed					
KC_M1	207	79	95	72	95
KC_M2	272	89	88	91	77
KC_M3*	330	88	90	87	86
KC_M4*	150	88	92	86	79
Okatie Center Subwatershed					
OC_M1	345	89	88	91	86
Okatie West Subwatershed					
OW_M1	324	80	152	86	114
OW_M2	535	77	237	79	190
OW_M3	212	88	93	86	85
OW_M4	403	86	132	86	111
OW_T1A	311	83	117	79	101
OW_T1B	442	81	196	84	142
OW_T1C	329	86	94	86	75
OW_T2	187	88	53	87	48
OW_T3A	232	84	91	87	75
OW_T3B	67	79	76	82	69
Pepper Hall Subwatershed					
PH_M1	78	81	57	81	54
PH_M2	131	84	71	88	61
Pinkney Colony South Subwatershed					
PCS_M1	159	77	85	81	85
PCS_M2	109	86	88	90	77
PCS_M3	148	83	72	85	69
Rose Hill East Subwatershed					
RHE_M1	372	74	118	77	118
RHE_M2	128	90	39	90	39
RHE_M3	458	83	116	84	116
Sawmill Creek Subwatershed					
SMC_M1	368	83	115	83	113
SMC_M2	311	92	79	91	74
SMC_M3	276	85	101	87	97
SMC_T1	107	90	57	89	54
Sawmill Creek East Subwatershed					
SMCE_M1	358	74	197	75	193
Sawmill Creek West Subwatershed					
SMCW_M1	189	66	164	72	138
Simmonsville/ Hidden Lakes Canal Subwatershed					
SHLC_M1	212	76	87	75	87
SHLC_M2	245	82	90	80	90
SHLC_M3	393	88	91	89	89
SHLC_M4	252	86	56	90	54
SHLC_M5	274	75	108	78	99
SHLC_T1	218	75	116	72	116
SHLC_T2	81	81	58	84	53
Spring Island 1 Subwatershed					
SI1_M1	339	76	142	77	142
Spring Island 3 Subwatershed					
SI3_M1	205	79	102	80	102
Spring Island 4 Subwatershed					
SI4_M1	218	77	92	77	92
Spring Island 5 Subwatershed					
SI5_M1	211	58	139	62	139
Wadell Subwatershed					
W_M1	201	69	133	68	133
W_M2	167	73	130	73	130
Average	233	81	100	82	93

* ATM Updated Areas (based on updated and improved watershed delineations)

TABLE 6-4
HYDRAULIC DATA SUMMARY
COLLETON RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Belfair East	3	3,114	1	0	1	1	0	0
Berkeley Creek	11	10,022	3	3	1	1	2	0
Burnt Church	12	8,393	5	10	1	5	8	0
Callawassee Island	1	523	1	2	0	1	1	0
Camp St. Mary's	6	4,405	3	5	0	2	1	0
Kitty's Crossing	12	11,822	2	2	1	1	1	0
Okatie Center	3	3,037	1	2	0	1	0	0
Okatie West	40	41,626	6	22	0	2	4	0
Pepper Hall	3	1,629	1	1	0	2	3	3
Pinkney Colony South	5	4,584	3	7	0	1	3	0
Rose Hill East	7	8,888	5	16	0	1	5	10
Sawmill Creek	8	9,319	3	7	0	3	3	0
Sawmill Creek East	0	0	1	2	0	0	1	0
Sawmill Creek West	2	2,331	0	0	0	0	0	0
Simmonsville/Hidden Lakes Canal	21	18,079	9	17	0	5	11	7
Spring Island 1	0	0	1	2	0	1	1	0
Spring Island 3	0	0	2	2	0	2	1	2
Spring Island 4	0	0	1	1	0	1	0	0
Spring Island 5	0	0	1	1	0	1	1	0
Wadell	4	3,420	2	2	0	2	2	0
TOTAL	138	131,192	51	104	4	33	48	22

TABLE 6-5
CULVERT DATA FOR HYDROLOGIC BASINS
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Belfair East Subwatershed						
Cumberland Drive	BRE_M-4	Bridge	50	7.3	14.1	25
Berkeley Creek Subwatershed						
Unknown	BC_M-3A	120"x72"	60	4.2	15.1	25
	3B	120"x72"	60	4.2		
Berkeley Hall Boulevard	BC_T1-1	Bridge	25	-1.1	9.0	25
Fording Island Road (US Hwy 278)	BC_T1-6	24"x24"	130	16.9	21.0	100
Burnt Church Subwatershed						
Meridian Point Drive	BTC_M-5	Bridge	34	7.2	11.6	25
Fording Island Road (US Hwy 278)	BTC_M-8	24"x24"	130	14.8	19.1	100
Fording Island Road (US Hwy 278)	BTC_T1-2	30"x30"	175	16.7	21.7	100
Meridian Point Drive	BTC_T2-3	6 - 15"X15"	48	10.3	11.6	25
Meridian Point Drive	BTC_T3-2	2 - 15"X15"	48	10.5	11.6	25
Callawassee Island Subwatershed						
Winding Oak Drive	CI_M-1A	15"x15"	33	3.7	7.3	25
	1B	15"x15"	33	3.6	7.3	25
Camp St. Mary's Subwatershed						
Camp St. Mary	CSM_M-4A	30"x30"	40	13.7	18.2	25
	4B	24"x24"	40	13.4		
Old Bailey's Road	CSM_T1-3A	30"x30"	36	9.1	17.7	25
	3B	30"x30"	36	8.9		
Okatie Highway (State Hwy 46)	CSM_T1-5	48"x48"	118	14.3	22.0	100
Kitty's Crossing Subwatershed						
Waterford Drive	KC_M-2	Bridge	35	2.9	13.3	25
Fording Island Road (US Hwy 278)	KC_M-6A	42"x42"	200	9.7	19.3	100
	6B	42"x42"	200	9.9		
Okatie Center Subwatershed						
Okatie Highway (State Hwy 46)	OC_M-3A	18"x18"	264	9.1	17.3	100
	3B	42"x42"	182	8.8		

TABLE 6-5
CULVERT DATA FOR HYDROLOGIC BASINS
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Okatie West Subwatershed						
Okatie Highway (State Hwy 46)	OW_M-9	2 - 72"x72"	32	5.0	13.4	100
Bull Hill Road	OW_M-17A	36"x36"	45	30.4	33.5	25
	17B	36"x36"	45	30.0		
Okatie Highway (State Hwy 46)	OW_M-19	24"x24"	50	30.5	34.8	100
Buckwalter Parkway	OW_T1-14A	36"x36"	120	15.5	21.9	25
	14B	36"x36"	120	15.4		
	14C	36"x36"	120	15.4		
	14D	36"x36"	120	15.4		
	14E	36"x36"	120	15.4		
	14F	36"x36"	120	15.4		
Blythe Island Drive	OW_T3-2	8 - 36"x36"	60	9.1	13.4	25
Buckwalter Parkway	OW_T3-7A	30"x30"	80	19.2	24.4	25
	7B	30"x30"	80	18.7		
	7C	30"x30"	80	18.9		
Pepper Hall Subwatershed						
Graves Road	PH_M-7	18"x18"	30	11.6	13.3	25
Pinkney Colony South Subwatershed						
Spartine Cresent	PCS_M-1A	48"x48"	40	-0.2	7.3	25
	1B	48"x48"	40	-0.1		
	1C	48"x48"	40	-0.3		
Pinkney Colony Road	PCS_M-4A	24"x24"	30	9.0	12.5	25
	4B	24"x24"	30	8.9		
Fording Island Road (US Hwy 278)	PCS_M-8A	30"x30"	220	15.6	20.7	100
	8B	30"x30"	220	15.4		
Rose Hill East Subwatershed						
Rose Hill Way	RHE_M-1	4 - 48"x48"	40	0.9	9.1	25
Rose Hill Way	RHE_M-3	3 - 48"x48"	100	4.0	13.2	25
Martingale East	RHE_M-7	3 - 48"x48"	83	9.1	17.8	25
Fording Island Road (US Hwy 278)	RHE_M-8	2 - 48"x48"	260	11.0	21.1	100

TABLE 6-5
CULVERT DATA FOR HYDROLOGIC BASINS
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Clubhouse Drive	RHE_M-10A	36"x36"	60	12.7	19.8	25
	10B	36"x36"	60	12.7		
	10C	36"x36"	60	12.8		
	10D	36"x36"	60	12.7		
Sawmill Creek Subwatershed						
Fording Island Road (US Hwy 278)	SMC_M-6A	30"x30"	220	6.8	13.6	100
	6B	30"x30"	220	7.8		
	6C	30"x30"	220	7.7		
Mulrain Way	SMC_M-8	36"x36"	100	9.3	13.6	25
Heritage Lakes Drive	SMC_M-11A	36"x36"	50	11.3	15.1	25
	11B	36"x36"	50	11.4		
	11C	36"x36"	50	11.2		
Sawmill Creek East Subwatershed						
Sawmill Creek Road	SMCE_M-1A	30"x30"	50	-0.8	5.78	25
	SMCE_M-1B	30"x30"	50	-1.2		
Sawmill Creek West Subwatershed						
No road crossings in this basin						
Simmonsville/ Hidden Lakes Canal Subwatershed						
Cross Tide Park	SHLC_M-1	42"x42"	150	0.6	11.1	25
		42"x42"	150	0.7	11.1	25
		42"x42"	150	0.7	11.1	25
Belfair Oaks Boulevard	SHLC_M-2	3 - 42"x 42"	35	3.5	10.4	25
Belfair Oaks Boulevard	SHLC_M-6A	60"x60"	36	2.9	12.5	25
	6B	60"x60"	36	3.1		
	6C	60"x60"	36	2.9		
Fording Island Road (US Hwy 278)	SHLC_M-10A	48"x48"	400	7.5	21.0	100
	10B	2 - 48"x48"	400	7.5		
Kensington Blvd	SHLC_M-15	156"x102"	50	8.1	15.9	25
Regent Avenue	SHLC_M-17	156"x96"	63	8.4	19.7	25
Tower Road	SHLC_M-23	48"x48"	40	11.8	20.5	25
Hyon Road	SHLC_M-27	48"x48"	27	14.2	19.9	25
Buck Island Road	SHLC_T2-3	24"x24"	30	22.1	25.6	25
Spring Island 1 Subwatershed						
Spring Island Drive	SI1_M-1A	36"x36"	50	4.2	12.0	25
	1B	36"x36"	50	4.3		

TABLE 6-5
CULVERT DATA FOR HYDROLOGIC BASINS
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Spring Island 3 Subwatershed						
Spring Island Drive	SI3_M-1	64"x18"	30	0.2	8.0	25
Spring Island Drive	SI3_M-2	42"x42"	120	4.3	19.8	25
Spring Island 4 Subwatershed						
Spring Island Drive	SI4_M-1	42	45	9.4	15.5	25
Spring Island 5 Subwatershed						
Spring Island Drive	SI5_M-1	42	60	4.3	11.2	25
Wadell Subwatershed						
Sawmill Creek Road	W_M-2	42"x42"	70	3.1	13.2	25
Sawmill Creek Road	W_M-7	18"x18"	50	13.4	17.0	25

TABLE 6-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Belfair East Subwatershed					
No Overtopping Identified					
Berkeley Creek Subwatershed					
No Overtopping Identified					
Burnt Church Subwatershed					
Meridian Point Drive	BTC_M-36	11.6	11.6	10	11.9
				25	11.9
				100	11.9
Fording Island Road (US Hwy 278)	BTC_M-72	19.1	19.1	2	19.2
				10	19.5
				25	19.5
				100	19.5
Fording Island Road (US Hwy 278)	BTC_T1-6	21.7	21.7	10	21.6
				25	21.7
				100	21.8
Callawassee Island Subwatershed					
Winding Oak Drive	CI_M-1	7.3	7.3	10	7.6
				25	7.6
				100	7.7
Camp St. Mary's Subwatershed					
Camp St. Mary	CSM_M-27	18.2	18.2	10	18.5
				25	18.6
				100	18.8
Kitty's Crossing Subwatershed					
Fording Island Road (US Hwy 278)	KC_M-43	19.3	19.3	10	19.4
				25	19.8
				100	20.0
Okatie Center Subwatershed					
No Road Crossing	OC_M-32	N/A	15.6 (Top of Lagoon Berm)	100	15.7
Okatie West Subwatershed					
Bull Hill Road	OW_M-175	33.5	33.5	10	33.8
				25	33.8
				100	33.8
Okatie Highway (State Hwy 170)	OW_M-184	34.8	34.5	2	35.0
				10	35.5
				25	35.5
				100	35.5

TABLE 6-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Pepper Hall Subwatershed					
Private Road / Driveway	PH_M-8	6.1	6.1	2	7.4
				10	7.7
				25	7.7
				100	7.7
Private Road / Driveway	PH_M-15	7.2	7.2	2	8.7
				10	9.1
				25	9.1
				100	9.1
Graves Road	PH_M-23	13.3	14.0	2	14.2
				10	14.3
				25	14.3
				100	14.3
Pinkney Colony South Subwatershed					
Pinkney Colony Road	PCS_M-14	12.5	12.5	2	12.8
				10	12.8
				25	12.8
				100	12.8
Fording Island Road (US Hwy 278)	PCS_M-51	20.7	20.7	100	20.8
Rose Hill East Subwatershed					
Private Road / Driveway	RHE_M-1	9.5	9.5	100	9.6
Clubhouse Drive	RHE_M-69	19.8	19.6	10	19.6
				25	19.8
				100	20.0
Sawmill Creek Subwatershed					
Fording Island Road (US Hwy 278)	SMC_M-63	13.6	13.6	100	14.1
Mulrain Way	SMC_M-77	13.6	13.8	10	14.1
				25	14.1
				100	14.2
Sawmill Creek East Subwatershed					
No Overtopping Identified					
Sawmill Creek West Subwatershed					
No Overtopping Identified					
Simmonsville/ Hidden Lakes Canal Subwatershed					
Private Road / Driveway	SHLC_M-1	Unknown	5.5	2	6.6
				10	6.7
				25	6.7
				100	6.7
Belfair Oaks Boulevard	SHLC_M-18	10.4	10.4	10	10.3
				25	10.5
				100	10.5
Tower Road	SHLC_M-159	20.5	20.5	100	20.5
Location Unknown	SHLC_M-163	Unknown	18.0	2	18.2
				10	18.7
				25	19.5
				100	20.6
Hyon Road	SHLC_M-166	19.9	19.9	25	20.3
				100	20.6
Location Unknown	SHLC_M-169	19.8	19.8	10	19.9
				25	20.3
				100	20.6
Location Unknown	SHLC_M-172	20.8	20.8	10	20.8
				25	21.0
				100	21.0
Buck Island Road	SHLC_T2-23	25.6	25.6	25	25.5
				100	26.0

TABLE 6-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Spring Island 1 Subwatershed					
No Overtopping Identified					
Spring Island 3 Subwatershed					
Spring Island Drive	SI3_M-1	8.0	8.0	10	8.2
				25	8.4
				100	8.5
Spring Island 4 Subwatershed					
No Overtopping Identified					
Spring Island 5 Subwatershed					
No Overtopping Identified					
Wadell Subwatershed					
Sawmill Creek Road	W_M-47	17.0	17.0	2	16.9
				10	17.4
				25	17.5
				100	17.7

TABLE 6-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Belfair East Basin			
No improvements required			
Berkeley Creek Basin			
No improvements required			
Burnt Church Basin			
Meridian Point Drive	BTC_M-5	Bridge	Excavate channel section under bridge
Fording Island Road (US Hwy 278)	BTC_M-8	24"x24"	Replace culvert with two 6 ft by 4 ft box culverts
Fording Island Road (US Hwy 278)	BTC_T1-2	30"x30"	Add one 48" pipe to existing culvert
Callawassee Island Basin			
Winding Oak Drive	CI_M-1A 1B	15"x15" 15"x15"	Add one 36" pipe to existing culverts
Callawassee Road West Basin			
Callawassee Drive	CRW_T1-5	18"x18"	Replace culvert with one 6 ft by 4 ft box culvert
Camp St. Mary's Basin			
Camp St. Mary	CSM_M-4A 4B	30"x30" 24"x24"	Replace culverts with two 5 ft by 4 ft box culverts
Kitty's Crossing Basin			
Fording Island Road (US Hwy 278)	KC_M-6A 6B	42"x42" 42"x42"	Replace culverts with two 7 ft by 4 ft box culverts
Okatie Center Basin			
No improvements required			
Okatie West Basin			
Bull Hill Road	OW_M-17A 17B	36"x36" 36"x36"	Add nine 36" pipes to existing culverts
Okatie Highway (State Hwy 46)	OW_M-19	24"x24"	Replace culvert with four 6 ft by 4 ft box culverts
*Blythe Island Drive	OW_T3-2	8 - 36"x36"	Add three 6 ft by 4 ft box culverts to existing culverts
Pepper Hall Basin			
Graves Road	PH_M-7	18"x18"	Replace culvert with four 36" pipes; Set upstream culvert inverts to 9.5 ft NAVD
Pinkney Colony South Basin			
*Spartine Cresent	PCS_M-1A 1B 1C	48"x48" 48"x48" 48"x48"	Add one 8 ft by 4 ft box culvert to existing culverts
Pinkney Colony Road	PCS_M-4A 4B	24"x24" 24"x24"	Replace culverts with eight 36" pipes
Fording Island Road (US Hwy 278)	PCS_M-8A 8B	30"x30" 30"x30"	Add one 4 ft by 4 ft box culvert to existing culverts

TABLE 6-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
COLLETON RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Rose Hill East Basin			
*Rose Hill Way	RHE_M-3	3 - 48"x48"	Replace culverts with three 6 ft by 6 ft box culverts, Lower weir invert to 9.7 ft NAVD and increase weir height to 38 in, Add one more weir for a total of four
*Martingale East	RHE-M-7	3 - 48"x48"	Add one 48" pipe to existing culverts, Increase height of "short" weirs to 31 in, Drop invert of "short" weirs to 14.9 ft NAVD, Add one more riser for a total of four, with one "tall" weir and one "short" weir on each riser
*Fording Island Road (US Hwy 278)	RHE-M-8	2 - 48"x48"	Replace culverts with two 7 ft by 5 ft box culverts; This improvement necessary to eliminate backwater flooding Clubhouse Drive
Clubhouse Drive	RHE_M-10A	36"x36"	Replace culverts with four 48" pipes
	10B	36"x36"	
	10C	36"x36"	
	10D	36"x36"	
Sawmill Creek Basin			
Fording Island Road (US Hwy 278)	SMC_M-6	3 - 30"x30"	Replace culverts with two 9 ft by 5 ft box culverts
Mulrain Way	SMC_M-8	36"x36"	Replace culvert with three 7 ft by 4 ft box culverts
Sawmill Creek East Basin			
*Sawmill Creek Road	SMCE_M-1A	30"x30"	Replace culverts with one 8 ft by 5 ft box culvert
	SMCE_M-1B	30"x30"	
Sawmill Creek West Basin			
No improvements required			
Simmonsville/ Hidden Lakes Canal Basin			
Belfair Oaks Boulevard	SHLC_M-2	3 - 42"x 42"	Replace culverts with three 8 ft by 4 ft box culverts; Add one 60 in by 90 in weir for a total of three
Tower Road	SHLC_M-23	48"x48"	Replace culvert with one 8 ft by 5 ft box culvert
Hyon Road	SHLC_M-27	48"x48"	Replace culvert with one 8 ft by 5 ft box culvert
Spring Island 1 Basin			
*Spring Island Drive	SI1_M-1A	36"x36"	Add one 36" pipe to existing culverts
	1B	36"x36"	
Spring Island 2 Basin			
*Shrimp Pond Road	SI2_M-2	15"x15"	Replace culvert with four 36" pipes; Increase dimensions of both riser and bubbler structures to 24 in by 72 in
Spring Island 3 Basin			
Spring Island Drive	SI3_M-1	64"x18"	Add one 36" pipe and a second drop structure same as existing
Spring Island 4 Basin			
No improvements required			
Spring Island 5 Basin			
No improvements required			
Wadell Basin			
Sawmill Creek Road	W_M-7	18"x18"	Replace culvert with three 36" pipes

* Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR.

TABLE 6-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
COLLETON RIVER WATERSHED

Land Use Type	Callawassee 1 (acres)	Callawassee 2 (acres)	Colleton River 1 (acres)	Colleton River 2 (acres)	Colleton River 3 (acres)	Colleton Tidal Flats (acres)	Okatie River 1 (acres)	Okatie River 2 (acres)	Okatie River 3 (acres)	Sawmill Branch 1 (acres)	Sawmill Branch 2 (acres)	TOTAL (acres)
Agricultural/Pasture	18	0	22	55	284	0	15	52	5	39	9	499
Commercial	4	4	13	162	60	5	77	57	213	241	240	1077
Forest/Rural Open	281	3	424	240	679	161	975	177	1082	1357	117	5497
Golf Course	37	46	101	367	400	18	367	8	163	69	78	1654
High Density Residential	0	0	0	454	340	0	146	25	790	284	173	2212
Industrial	74	26	111	295	253	13	201	70	264	233	99	1639
Institutional	0	0	0	2	11	0	22	0	14	0	8	57
Low Density Residential	392	46	340	517	1033	0	213	95	99	52	12	2798
Medium Density Residential	81	57	197	430	278	96	671	43	19	200	60	2132
Open Water/Tidal	608	250	2424	2957	2615	339	1120	304	179	496	134	11425
Silviculture	0	0	0	0	0	0	0	0	0	0	0	0
Urban Open	5	0	92	49	90	0	117	57	108	41	18	577
Wetland/Water	48	24	15	327	196	25	256	6	516	306	238	1957
TOTAL	1548	455	3740	5855	6239	656	4179	893	3452	3319	1186	31522
Urban Imperviousness (%)	8%	9%	5%	13%	10%	6%	12%	15%	23%	18%	32%	13%

TABLE 6-9
WATER QUALITY BASIN BMP COVERAGE
COLLETON RIVER WATERSHED

Land Use Type	Callawassee 1	Callawassee 2	Colleton River 1	Colleton River 2	Colleton River 3	Colleton Tidal Flats	Okatie River 1	Okatie River 2	Okatie River 3	Sawmill Branch 1	Sawmill Branch 2	TOTAL
Commercial	0.0%	0.0%	0.0%	15.9%	0.0%	0.0%	40.9%	32.2%	63.6%	10.1%	0.0%	21.9%
Golf Course	0.0%	0.0%	0.0%	0.0%	41.1%	0.0%	61.1%	0.1%	0.0%	0.0%	0.0%	17.3%
High Density Residential	0.0%	0.0%	0.0%	0.0%	7.6%	0.0%	82.0%	0.0%	55.3%	0.0%	0.0%	29.4%
Industrial	0.0%	0.0%	0.0%	0.0%	14.0%	0.0%	54.3%	3.8%	12.6%	0.2%	0.0%	10.7%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%	0.0%	0.0%	0.0%	0.0%	0.1%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	18.2%	0.0%	40.2%	15.6%	0.0%	0.0%	0.0%	15.8%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	5.1%	0.0%	62.9%	0.0%	0.7%	0.0%	0.0%	6.6%
TOTAL	0.0%	0.0%	0.0%	0.4%	16.4%	0.0%	49.1%	11.1%	17.8%	0.7%	0.0%	13.6%

TABLE 6-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
COLLETON RIVER WATERSHED

Land Use Type	Callawassee 1	Callawassee 2	Colleton River 1	Colleton River 2	Colleton River 3	Colleton Tidal Flats	Okatie River 1	Okatie River 2	Okatie River 3	Sawmill Branch 1	Sawmill Branch 2	TOTAL
Commercial	0.0%	0.0%	0.0%	0.0%	0.3%	0.0%	0.0%	0.0%	0.6%	1.0%	0.0%	0.4%
High Density Residential	0.0%	0.0%	0.0%	0.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%
Industrial	4.1%	3.0%	8.2%	2.0%	1.0%	0.0%	0.9%	0.0%	0.0%	0.9%	0.0%	1.6%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	8.8%	9.8%	8.4%	7.3%	6.9%	0.0%	0.9%	0.0%	3.7%	0.0%	4.5%	6.5%
Medium Density Residential	0.0%	0.0%	0.0%	0.5%	0.2%	0.0%	0.7%	0.0%	0.0%	1.2%	5.7%	0.6%
TOTAL	2.4%	1.2%	1.0%	0.8%	1.2%	0.0%	0.2%	0.0%	0.2%	0.2%	0.3%	0.7%

TABLE 6-11
AVERAGE ANNUAL LOADS FOR COLLETON RIVER WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
Colleton River 1	3,740	9,927	97,335	142	86,093	9.80E+14	186	36,841	4,841	428,000	3,714
Colleton River 2	5,855	14,567	178,000	286	128,000	1.70E+15	323	57,310	7,966	1,200,000	5,180
Colleton River 3	6,239	13,030	155,000	238	114,000	1.50E+15	275	50,806	7,320	990,000	4,491
Okatie River 1	4,179	7,104	83,737	127	60,988	7.02E+14	123	26,378	3,530	483,000	2,028
Okatie River 2	893	1,780	24,517	43	15,655	2.05E+14	42	7,197	989	192,000	595
Okatie River 3	3,452	4,967	81,613	152	42,835	5.75E+14	102	20,069	2,498	729,000	964
Sawmill Creek 1	3,319	5,075	81,410	154	45,272	6.86E+14	129	21,703	2,821	793,000	1,404
Sawmill Creek 2	1,186	2,339	46,244	87	21,161	3.46E+14	74	10,823	1,454	499,000	668
Callawassee Creek 1	1,548	3,008	36,374	54	26,572	3.84E+14	67	11,799	1,651	240,000	1,028
Callawassee Creek 2	455	1,142	12,985	21	10,012	1.30E+14	25	4,432	638	77,021	411
Colleton Tidal Flats	656	1,505	15,473	22	13,125	1.59E+14	29	5,618	747	80,676	531
TOTAL	31,522	64,444	812,688	1,326	563,713	7.37E+15	1,375	252,976	34,455	5,711,697	21,014

TABLE 6-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS

COLLETON RIVER WATERSHED

Water Quality Basin ID	DHEC Station(s)	Years of Record	No. of samples	Fecal Coliform Concentrations				Trend	Level of Service
				Long-Term Average		Most Recent 3 Year Values			
				Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)		
Colleton River 1	18-05, 18-15	1999-2016	409	2.97	8.19	2.41	4.5	Decreasing	A
Colleton River 2	18-04, 18-06	1999-2016	409	4.24	14	4.88	23.5	Increasing	A
Colleton River 3	18-01, 18-02, 18-03	1999-2016	613	5.89	23	7.28	33	Increasing	A
Okatie River 1	18-07, 18-17, 18-16	1999-2016	611	10.24	49	18.8	79	Increasing	D
Okatie River 2	18-08	1999-2016	202	25.01	170	79.6	350	Increasing	D
Okatie River 3	None	NA	NA	NA	NA	NA	NA	NA	NA
Sawmill Branch 1	None	NA	NA	NA	NA	NA	NA	NA	NA
Sawmill Branch 2	None	NA	NA	NA	NA	NA	NA	NA	NA
Callawassie Creek 1	None	NA	NA	NA	NA	NA	NA	NA	NA
Callawassie Creek 2	None	NA	NA	NA	NA	NA	NA	NA	NA
Colleton River - Tidal Flats	18-09	1999-2016	205	10	49	10.98	47.1	No Trend	C

TABLE 6-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

COLLETON RIVER WATERSHED

Water Quality Basin ID	South WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Colleton River 1	51	2.47E+07	Chechessee River 2	5,688	5,724	180
Colleton River 2	52	2.41E+07	Colleton River 1	3,378	5,375	180
Colleton River 3	53	1.14E+07	Colleton River 2	3,237	6,131	180
Okatie River 1	54	3.39E+06	Colleton River 3	678	5,536	180
Okatie River 2	55	7.59E+05	Okatie River 1	368	3,814	50
Okatie River 3	56	6.94E+04	Okatie River 2	129	1,577	50
Sawmill Branch 1	57	8.59E+05	Colleton River 2	411	1,744	150
Sawmill Branch 2	58	1.54E+05	Sawmill Branch 1	188	1,883	150
Callawassie Creek 1	59	1.42E+06	Colleton River 2	962	1,415	20
Callawassie Creek 2	60	6.98E+05	Callawassie Creek 1	605	1,400	20
Colleton River - Tidal Flats	61	4.02E+05	Colleton River 3	497	1,020	10

TABLE 6-14

.VERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMI

FOR COLLETON RIVER WATER QUALITY BASINS

Water Quality Basin ID	South WASP Segment	EXISTING LAND USE Flow (cfs)	EXISTING LAND USE Fecal Coliform (#/100 ml)
Colleton River 1	51	16.7	1,070
Colleton River 2	52	24.8	1,114
Colleton River 3 Jasper County 3	53	23.0	1,053
Okatie River 1 Jasper County 1 Jasper County 2	54	13.2	868
Okatie River 2	55	3.2	1,054
Okatie River 3	56	9.6	832
Sawmill Branch 1	57	9.7	1,022
Sawmill Branch 2	58	4.2	1,241
Callawassie Creek 1	59	5.4	1,071
Callawassie Creek 2	60	1.9	1,120
Colleton River - Tidal Flats	61	2.6	1,058

TABLE 6-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES

COLLETON RIVER WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs) Existing
Colleton River 1	Chechessee River 2	114
Colleton River 2	Colleton River 1	98
Colleton River 3	Colleton River 2	52
Okatie River 1	Colleton River 3	26
Okatie River 2	Okatie River 1	13.0
Okatie River 3	Okatie River 2	9.6
Sawmill Branch 1	Colleton River 2	14
Sawmill Branch 2	Sawmill Branch 1	4.2
Callawassie Creek 1	Sawmill Branch 2	7.3
Callawassie Creek 2	Callawassie Creek 1	1.9
Colleton River - Tidal Flats	Colleton River 3	2.6

TABLE 6-16

FECAL COLIFORM MODELING RESULTS

COLLETON RIVER WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)	Modeled Level of Service
		Existing	Existing
Colleton River 1	0.7	3.2	A
Colleton River 2	0.7	4.8	A
Colleton River 3	0.7	5.3	A
Okatie River 1	1.0	6.3	A
Okatie River 2	1.0	16.6	D
Okatie River 3	1.0	53.6	D
Sawmill Branch 1	1.0	12.3	D
Sawmill Branch 2	1.0	19.6	D
Callawassie Creek 1	1.0	7.6	B
Callawassie Creek 2	1.0	7.6	B
Colleton River - Tidal Flats	0.5	10.8	D

NOTE: Water quality basins with lower LOS in future are highlighted.

Tables 6-17, 6-18, and 6-19 are not applicable in the update.

TABLE 6-20 (Updated 2017)
PLANNING LEVEL COST ESTIMATES FOR
COLLETON RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
BTC_M-5 *	Road overtopping at Meridian Point Drive Excavate channel section under bridge	$17,000
BTC_M-11	Road overtopping at Fording Island Road (US Hwy 278) Replace existing 1 - 24" RCP with 2 - 6'x4' box culverts	$320,000
BTC_T1-2	Road overtopping at Fording Island Road (US Hwy 278) Add 1 - 48" RCP to existing 1 - 30" RCP	$70,000
CI_M-1 *	Road overtopping at Winding Oak Drive Add 1 - 36" RCP to existing 2 - 15" RCP	$32,000
CSM_M-4	Road overtopping at Camp St. Mary Road Replace existing 1 - 30" RCP and 1 - 24" RCP with 2 - 5'x4' box culverts	$108,000
KC_M-6	Road overtopping at Fording Island Road (US Hwy 278) Replace existing 2 - 42" RCP with 2 - 7'x4' box culverts	$556,000
OW_M-17 *	Road overtopping at Bull Hill Road Add 9 - 36" RCP to existing 2 - 36" RCP	$108,000
OW_M-19	Road overtopping at Okatie Highway (State Hwy 170) Replace existing 1 - 24" RCP with 4 - 6'x4' box culverts	$280,000
PCS_M-4	Road overtopping at Pinkney Colony Road Replace existing 2 - 24" RCP with 8 - 36" RCP	$82,000
PCS_M-8	Road overtopping at Fording Island Road (US Hwy 278) Add 1 - 4'x4' box culvert to existing 2 - 30" RCP	$264,000
PH_M-7	Road overtopping at Graves Road Replace existing 1 - 18" CMP with 4 - 36" RCP	$52,000
RHE_M-10 *	Road overtopping at Clubhouse Drive Replace existing 4 - 36" RCP with 4 - 48" RCP	$102,000
SHLC_M-2 *	Road overtopping at Belfair Oaks Boulevard Replace existing 3 - 42" RCP with 3 - 8'x4' box culverts Add one more weir for a total of three	$211,000
SHLC_M-23 *	Road overtopping at Tower Road Replace existing 1 - 48" CMP with 1 - 8'x5' box culvert	$118,000
SHLC_M-27 *	Road overtopping at Hyon Road Replace existing 1 - 48" CMP with 1 - 8'x5' box culvert	$100,000
SI1_M-1 *	Road overtopping at Spring Island Drive Add 1 - 36" RCP to existing 2 - 36" RCP	$36,000
SMC_M-6	Road overtopping at Fording Island Road (US Hwy 278) Replace existing 3 - 30" RCP with 2 - 9'x5' box culverts	$626,000
SMC_M-8 *	Road overtopping at Mulrain Way Replace existing 1 - 36" RCP with 3 - 7'x4' box culverts	$408,000
W_M-7	Road overtopping at Sawmill Creek Road Replace existing 1 - 18" RCP with 3 - 36" RCP	$55,000
	TOTAL	$3,545,000

* Conduits marked by asterisk are on private land
Costs are in January 2018 dollars.
See Appendix for basis of cost estimates.

Table 6-21 is not applicable in the update.

Figure 6-1

Colleton River Watershed

ATM DESIGN ENGINEERING CONSULTING

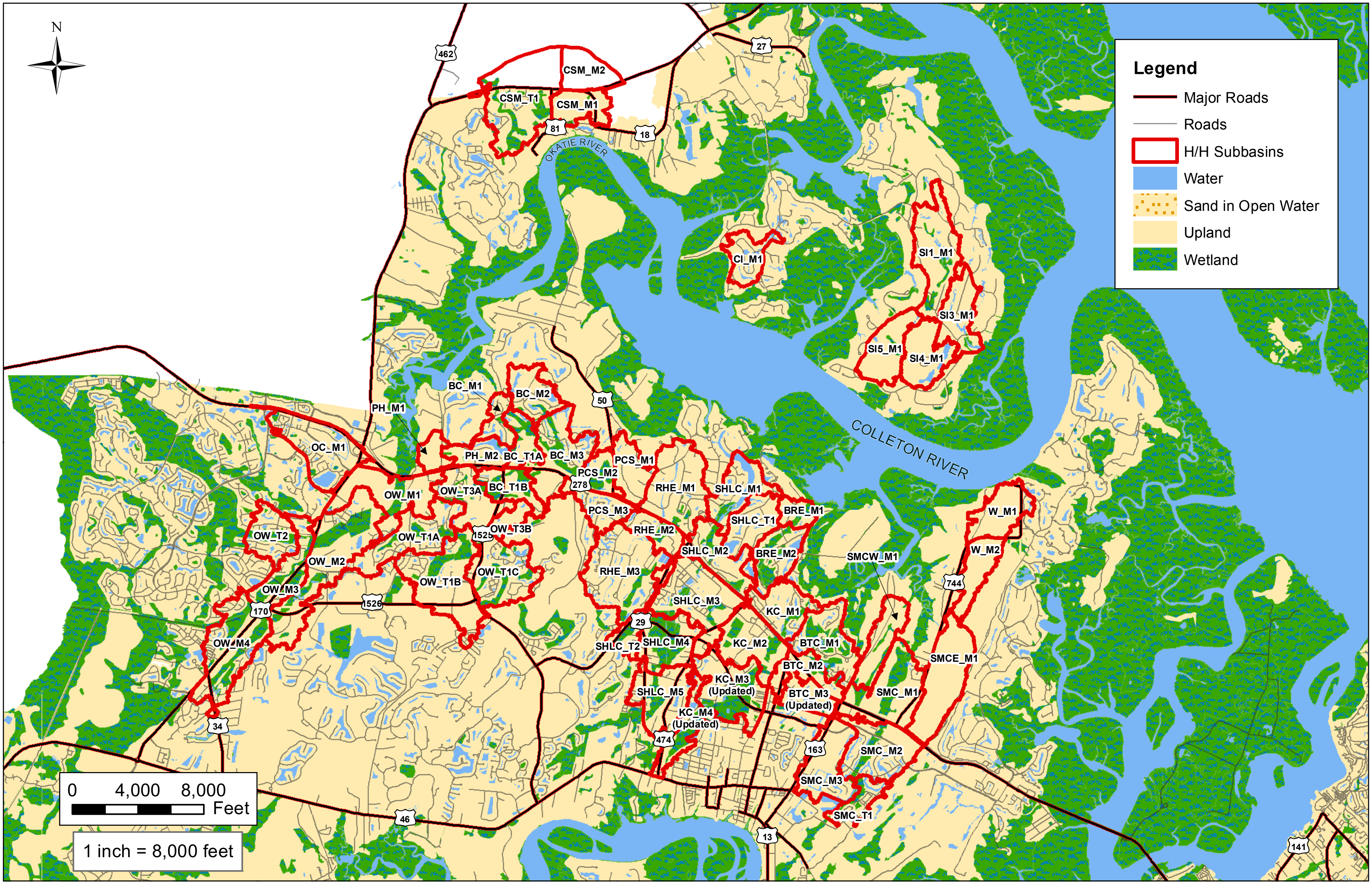


Figure 6-2. Hydrologic Model Subbasins

Colleton River Watershed

ATM DESIGN ENGINEERING CONSULTING

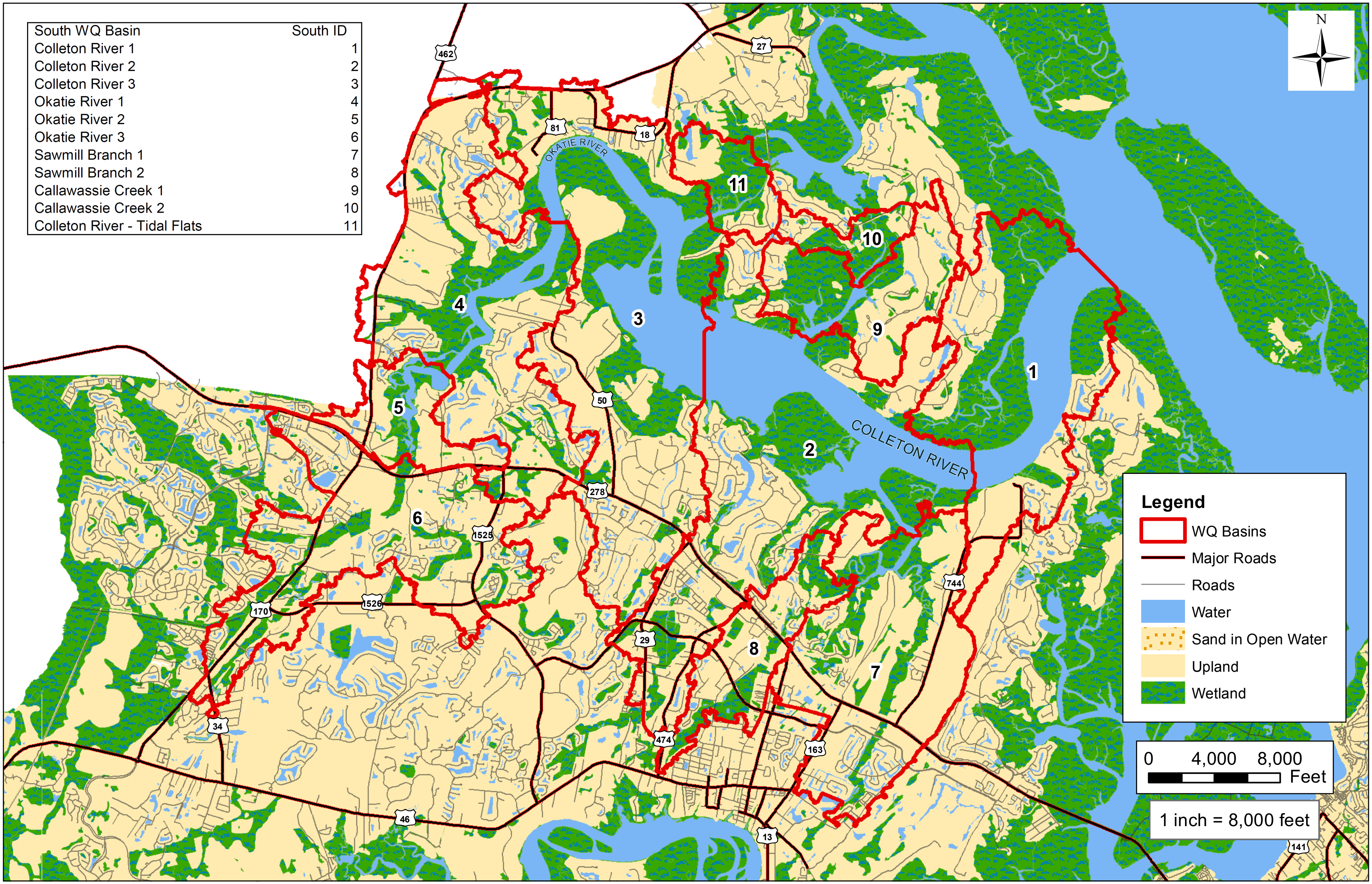
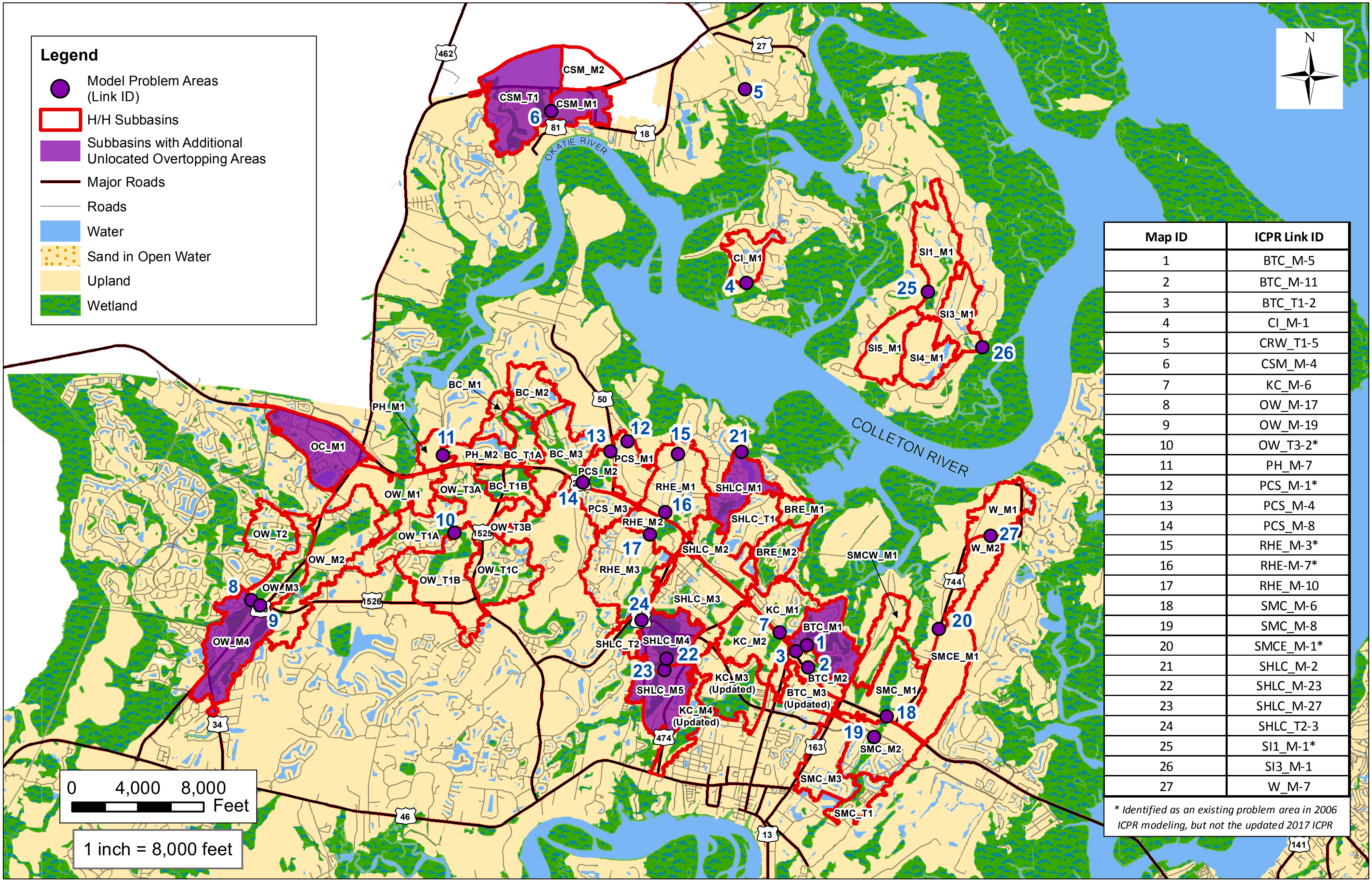


Figure 6-3

Colleton River Watershed
Water Quality Basins

ATM DESIGN ENGINEERING CONSULTING



Map ID	ICPR Link ID
1	BTC_M-5
2	BTC_M-11
3	BTC_T1-2
4	CI_M-1
5	CRW_T1-5
6	CSM_M-4
7	KC_M-6
8	OW_M-17
9	OW_M-19
10	OW_T3-2*
11	PH_M-7
12	PCS_M-1*
13	PCS_M-4
14	PCS_M-8
15	RHE_M-3*
16	RHE-M-7*
17	RHE_M-10
18	SMC_M-6
19	SMC_M-8
20	SMCE_M-1*
21	SHLC_M-2
22	SHLC_M-23
23	SHLC_M-27
24	SHLC_T2-3
25	SI1_M-1*
26	SI3_M-1
27	W_M-7

** Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR*

Figure 6-4. ICPR Identified PSMS Overtopping Problem Areas

Colleton River Watershed

ATM DESIGN ENGINEERING CONSULTING

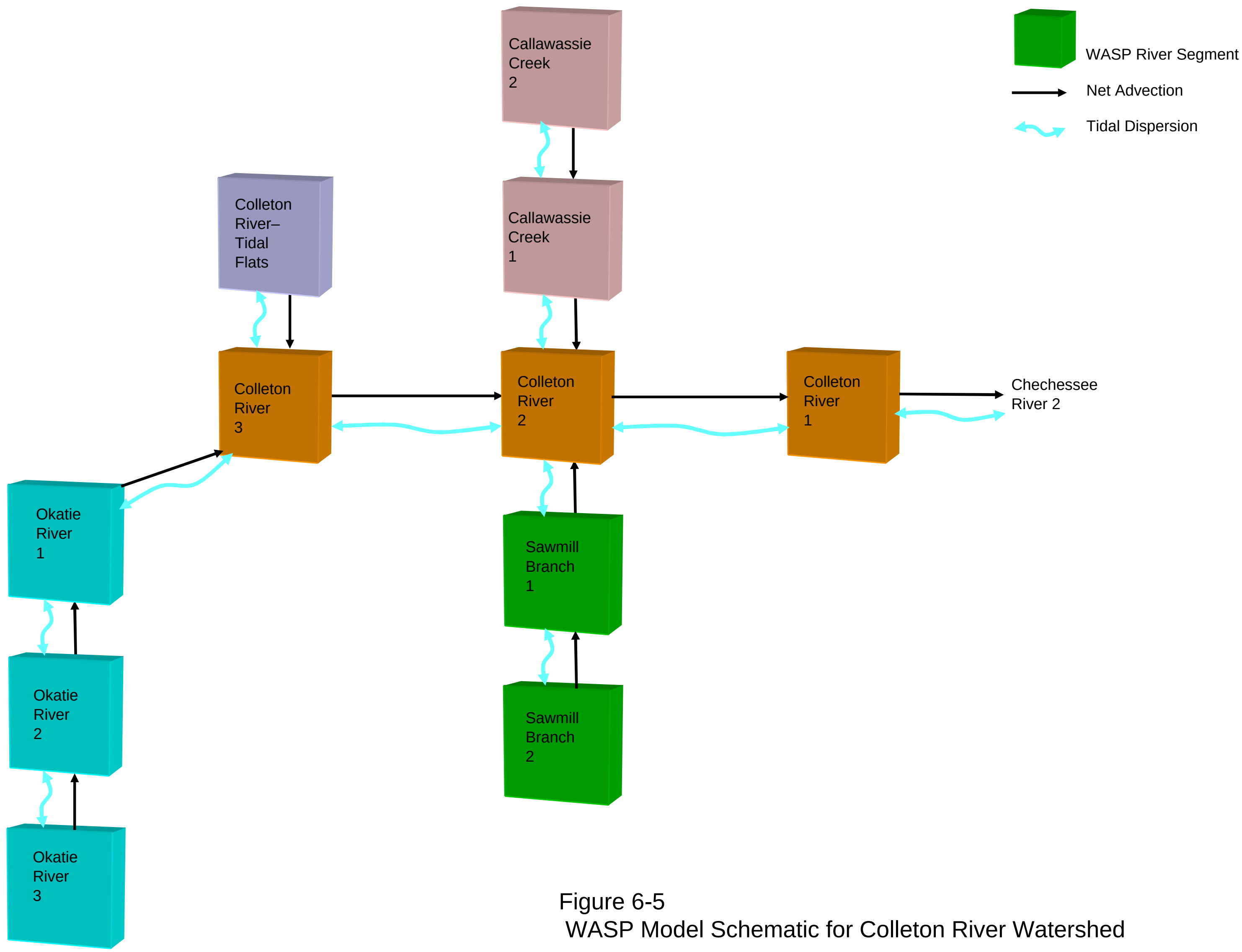


Figure 6-5
WASP Model Schematic for Colleton River Watershed

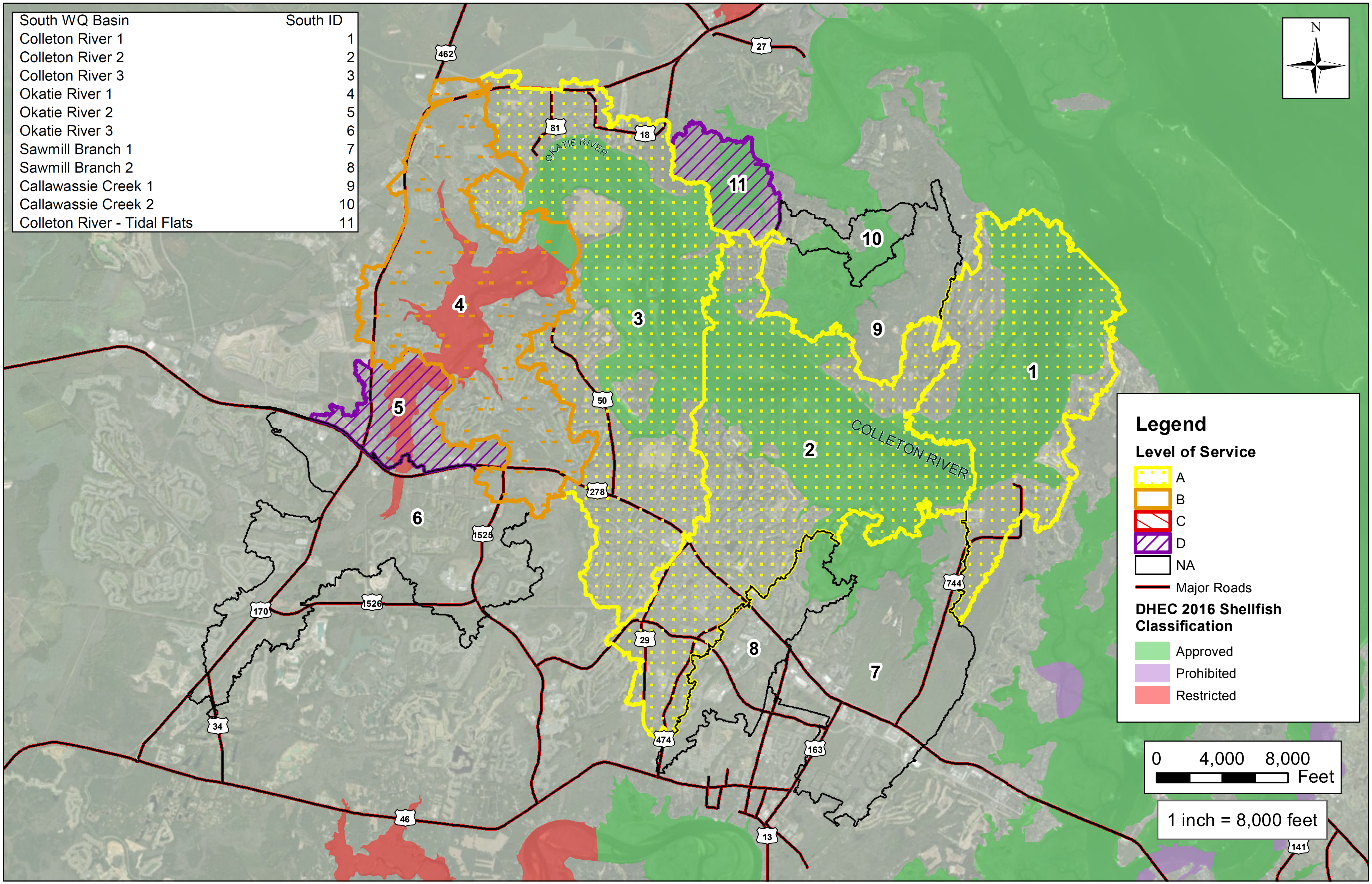


Figure 6-6

Colleton River Watershed
Shellfish Classification and Existing Level of Service

ATM DESIGN ENGINEERING CONSULTING

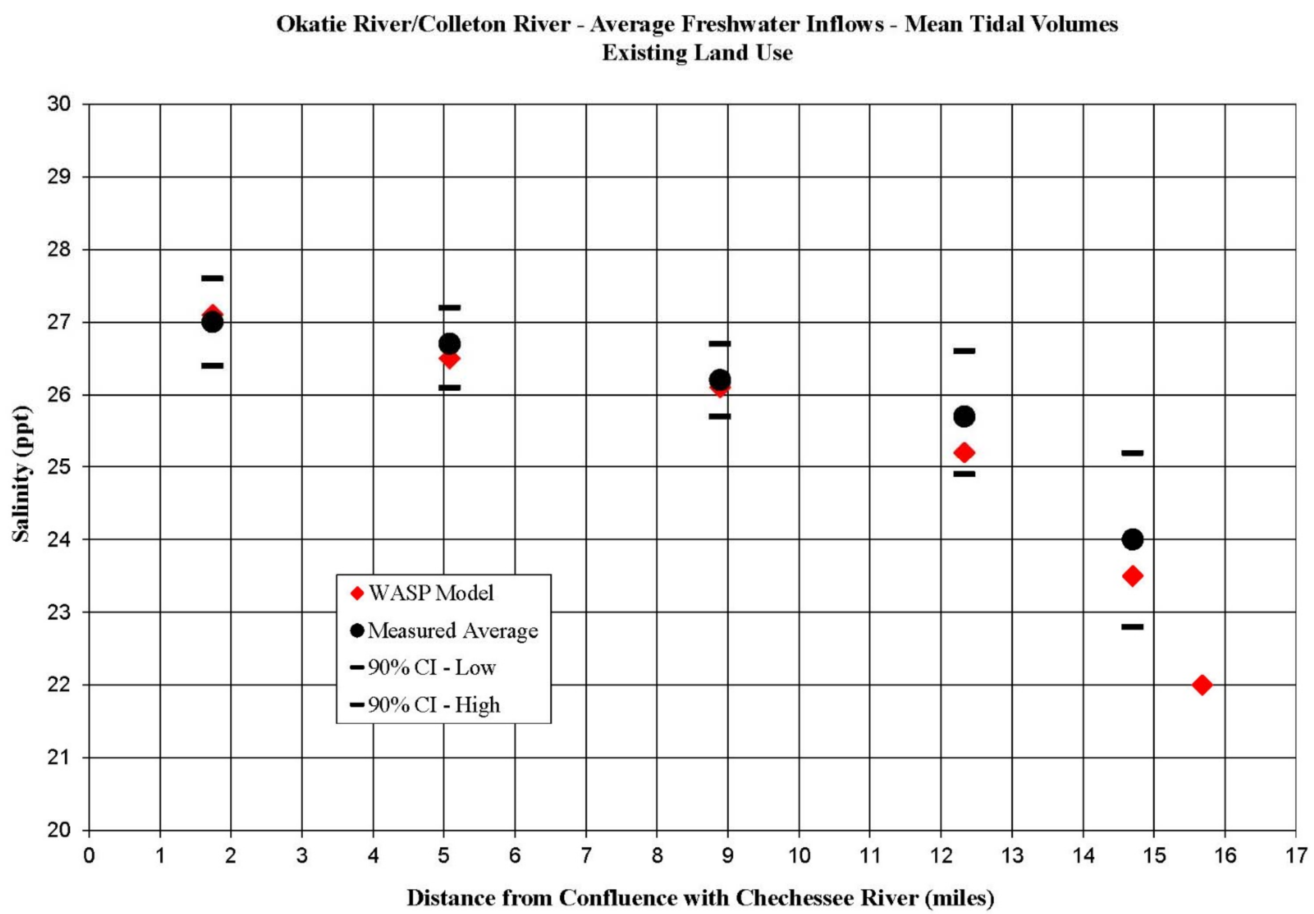


Figure 6-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Colleton River and Okatie River - Salinity.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

ATM DESIGN ENGINEERING CONSULTING

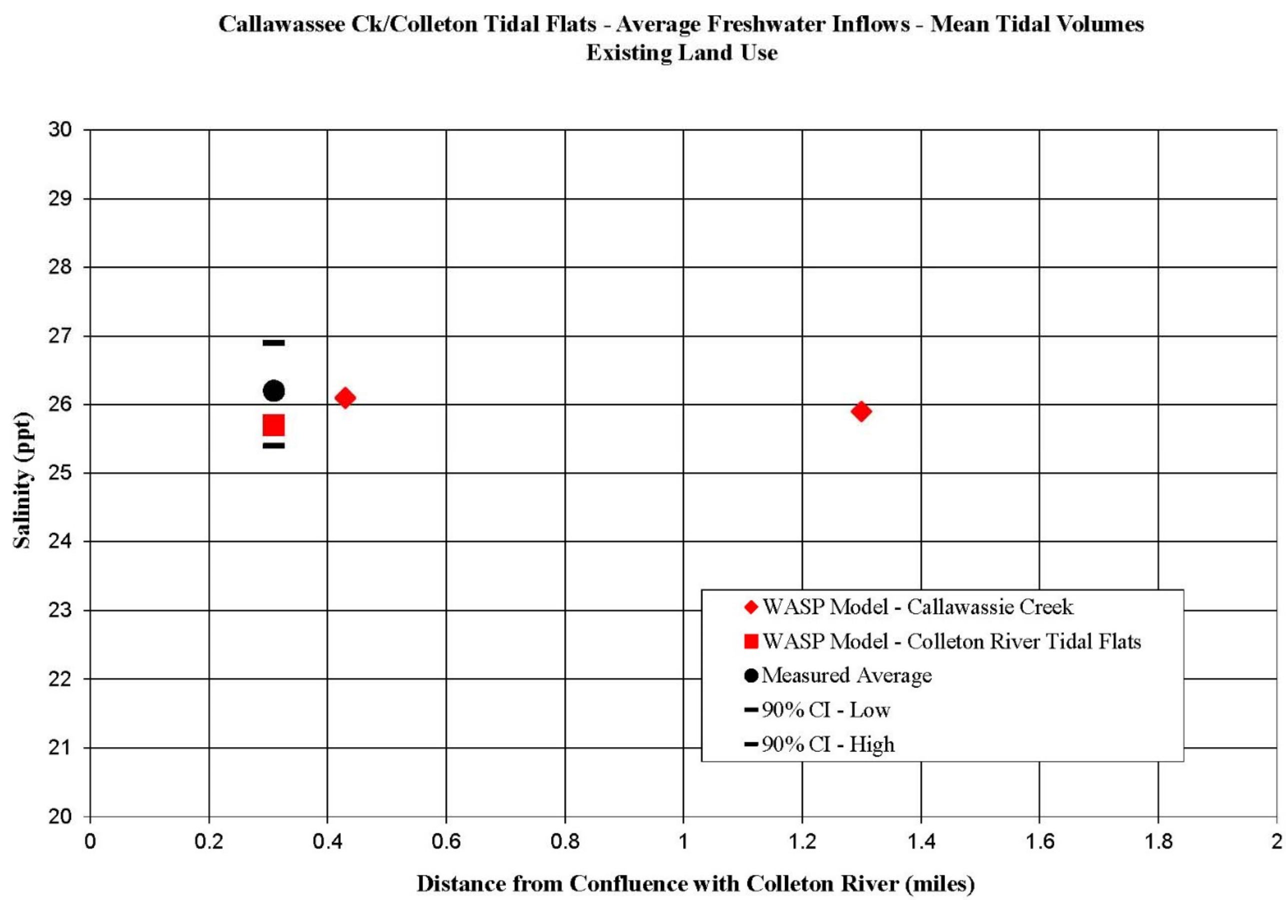


Figure 6-8. Comparison of WASP Model Results with Long-Term Monitoring Data in Colleton River Tributaries - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

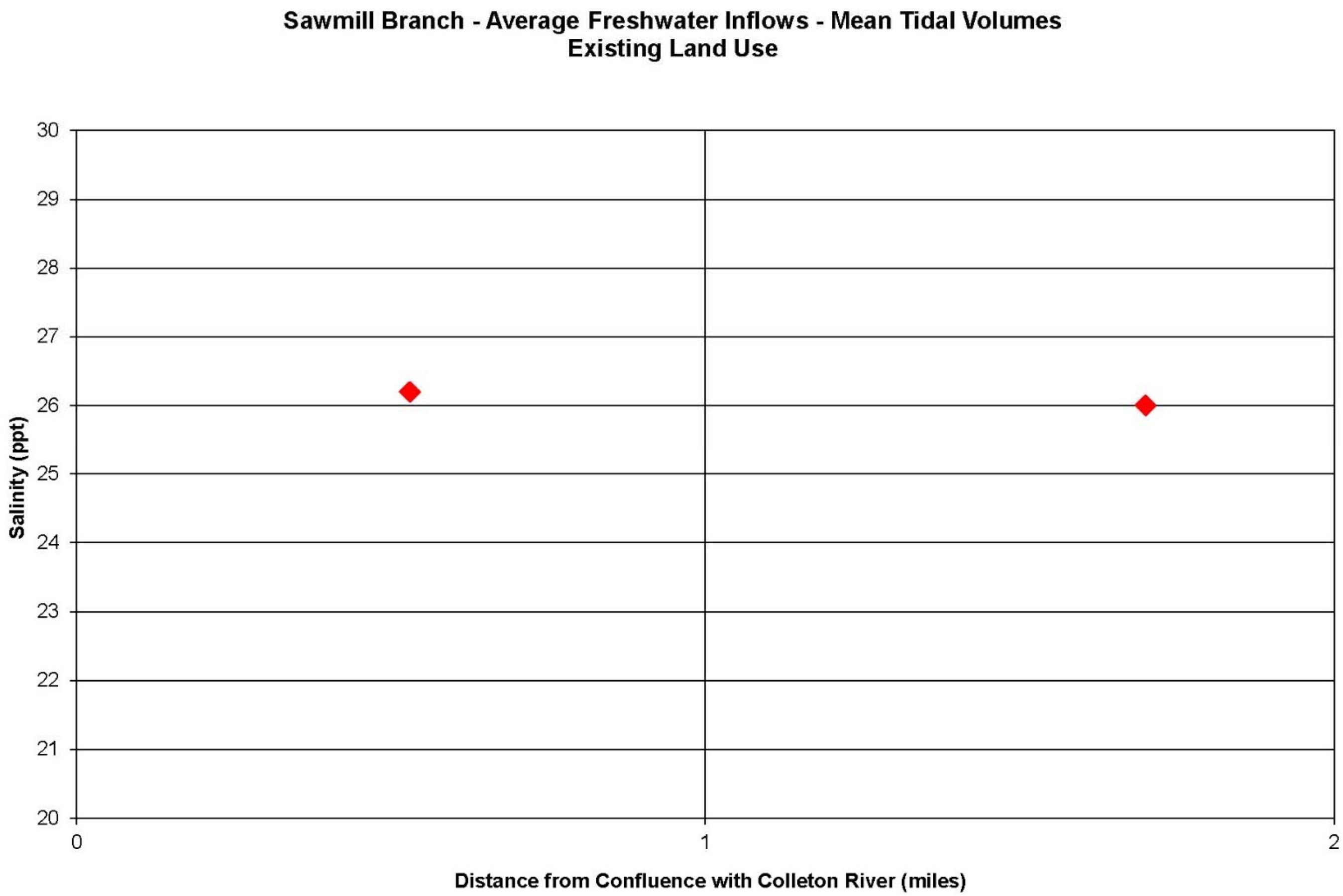


Figure 6-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Sawmill Branch - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

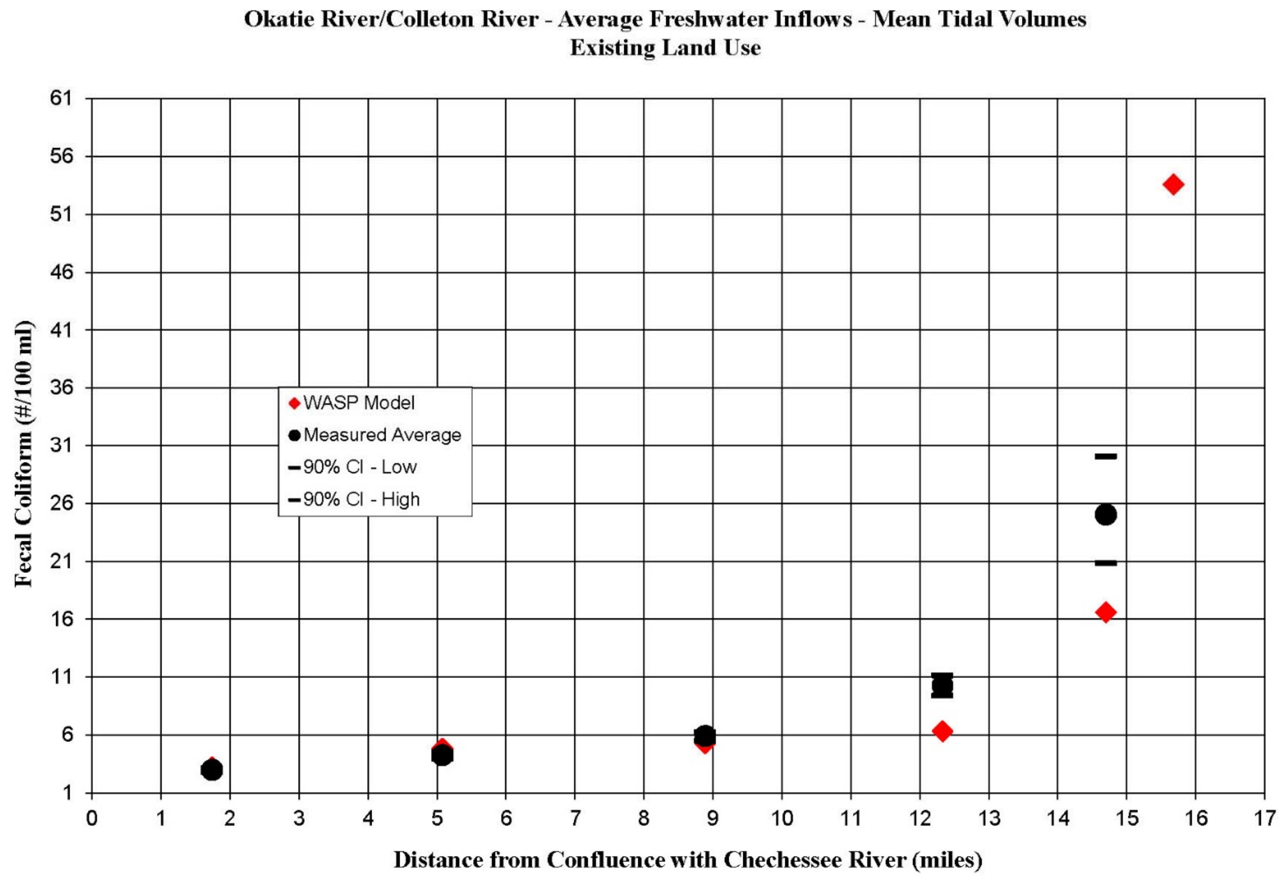


Figure 6-10. Comparison of WASP Model Results with Long-Term Monitoring Data in Colleton River and Okatie River - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

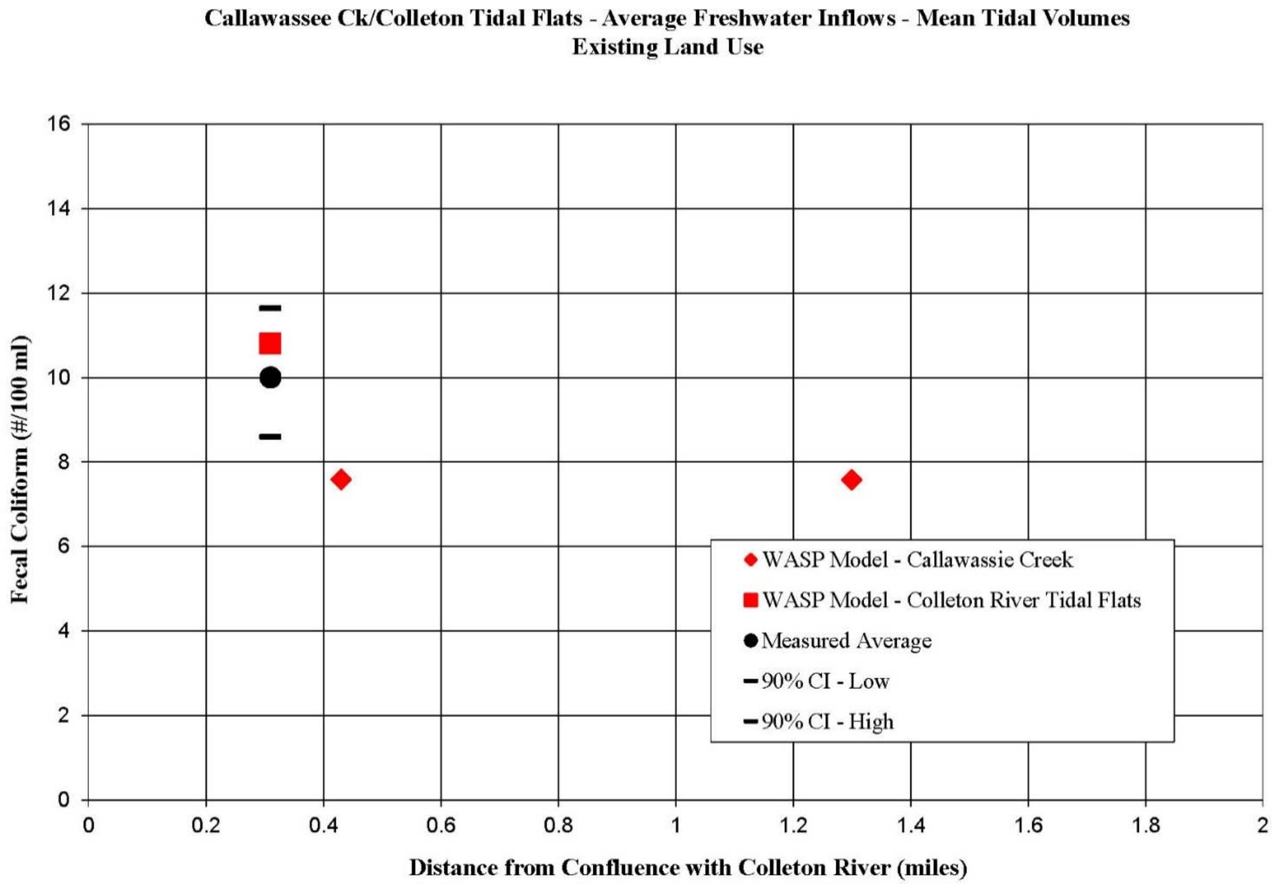


Figure 6-11. Comparison of WASP Model Results with Long-Term Monitoring Data in Colleton River Tributaries - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

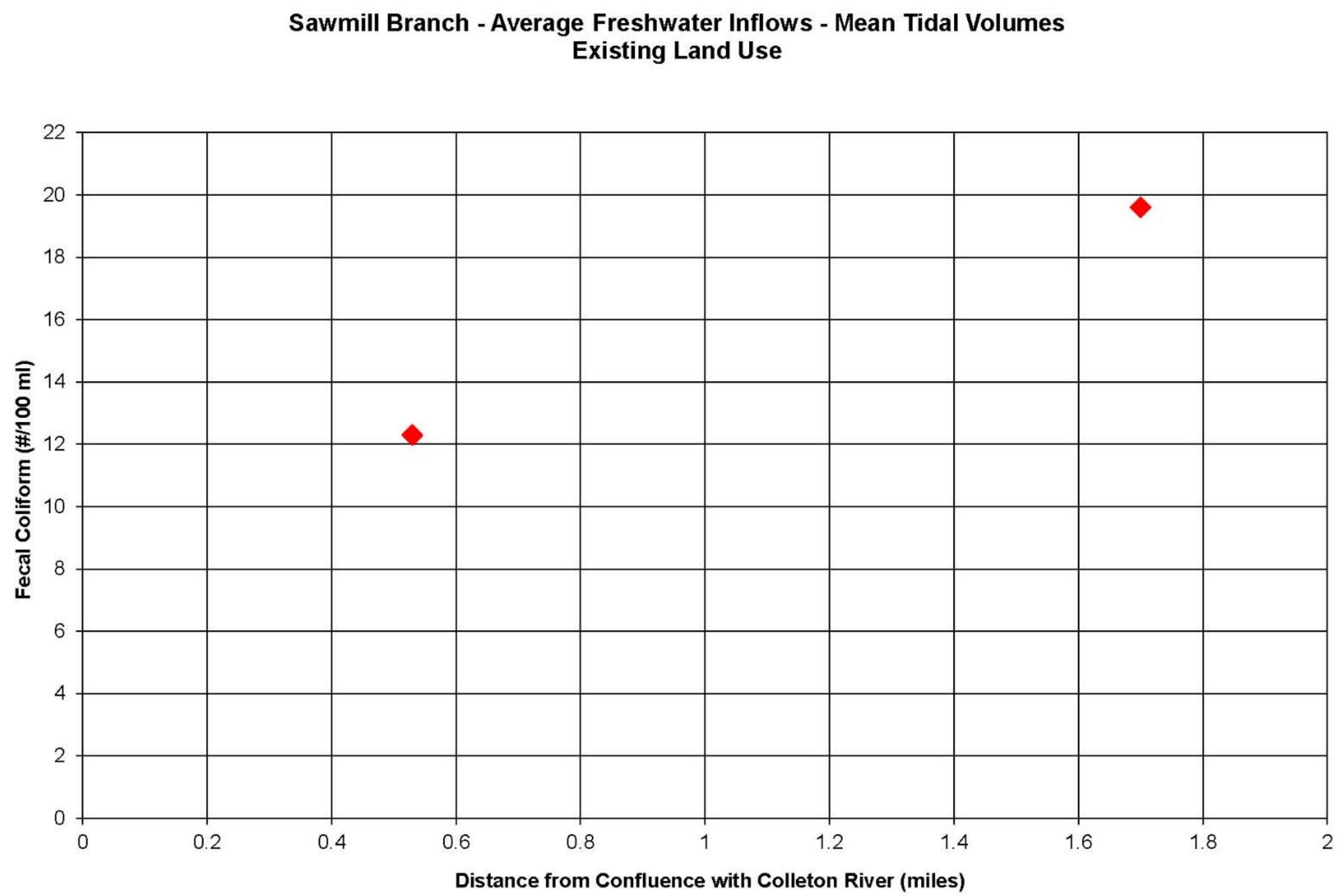
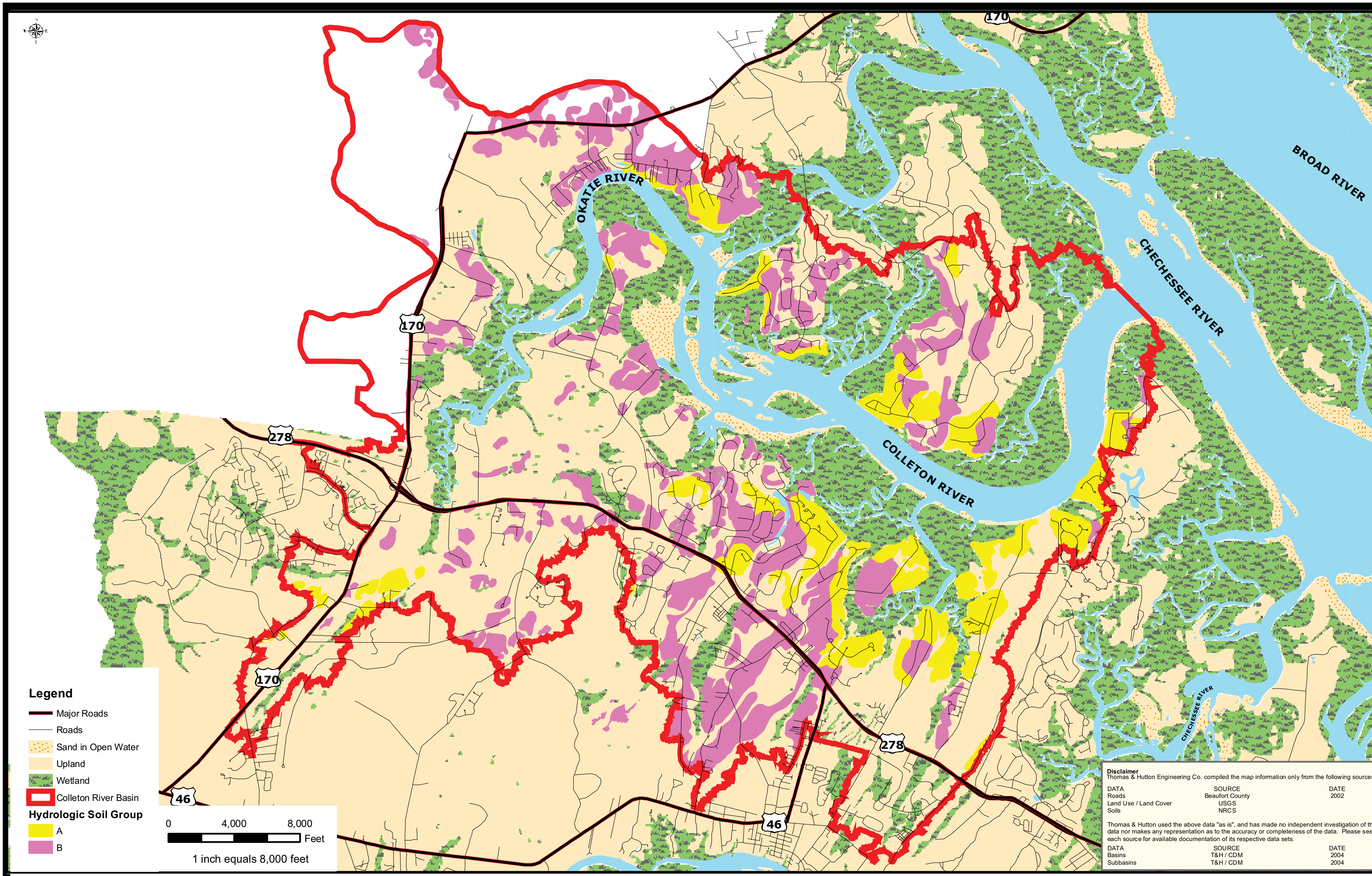


Figure 6-12. Comparison of WASP Model Results with Long-Term Monitoring Data in Sawmill Branch - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

ATM DESIGN ENGINEERING CONSULTING

Figure 6-13 is not applicable in the update.



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Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ColletonRiverSoils_figure6-12.mxd			Vertical Datum: NAVD88

Colleton River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 6-14

CDM Camp Dresser & McKee Inc.

Section 7
New River Watershed Analysis

This section describes the physical features of the New River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

7.1 Overview

The New River watershed is located south of the Broad River (see Figure 7-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area within the County limits, including parts of Bluffton Township, the Town of Bluffton, and Daufuskie Island that are tributary to the New River.

For comparative purposes, the entire tributary area for the New River is presented in Figure 7-2. The figure indicates Beaufort County makes up only a small fraction of the total tributary area to the New River.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several basins. These are listed in Table 7-1 and presented in Figure 7-3. Table 7-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were updated to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into basins. These are listed in Table 7-2 and presented in Figure 7-4. Pollution loads were calculated for each of the water quality basins. Unlike the other watersheds that are south of the Broad River, the vast majority of the New River tributary area is actually located outside of Beaufort County. Because loads from Beaufort County are such a small fraction of the total load to the New River, and loads from outside the County are unknown, tidal river water quality model calculations were not done for the New River.

7.2 Hydrologic and Hydraulic Analysis

The ICPR, Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the New River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP. It was determined that the future analysis previously assumed has not yet been reached for most watersheds.

7.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each New River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include area, curve number, and time of concentration.

Table 7-3 lists the hydrologic parameter values for the New River PSMS subbasins. Each model subbasin is identified by ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the New River PSMS basins is presented in Table 7-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 7-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

7.2.2 Model Results

Tables in Appendix E list summary of the results of the updated study including Updated Areas and CNs for the New River subbasins.

For existing land use, aerial maps generated in the summer of 2016 and local information were used to estimate the percentage of existing urban development.

Appendix E also includes other tables that list the peak water elevation values for model node locations along the New River PSMS.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs, and results showed that the FEMA elevations (based on storm

surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded.

Table 7-6 indicates the road crossings that are being overtopped by the design storm events.
Evaluation of solutions to prevent these problems is discussed in the next section of the report.

7.2.3 Management Strategy Alternatives

The problems areas listed in Table 7-6 were evaluated by reviewing the previous report results and reviewing the culverts in the ICPR hydraulic model. In the original 2006 study, he ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 7-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, circular and box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

7.3 Water Quality Analysis

ATM used WMM for the water quality analysis of the New River watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS.

7.3.1 Land Use and BMP Coverage

Table 7-8 presents the existing land use estimates for the New River water quality basins; collectively, the water quality basins constitute all watershed area within Beaufort County. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 25 percent of the New River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 75 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 8 per cent of the watershed.

Estimates of BMP coverage for existing land use is presented in Table 7-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County, and include areas for which BMPs were designed in accordance with the Beaufort County BMP Manual.

Under existing land use conditions, approximately 4 percent of the urban systems in the watershed (e.g., residential, commercial, golf course) are served by BMPs designed in accordance with the BMP Manual.

7.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in Table 7-10. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water and Sewer Authority. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects what percentage of all urban land in the watershed in served by septic tanks.

For existing land use conditions, 21 percent of the urban systems in the watershed (e.g., residential, commercial) are served by septic.

There is a direct discharge from the Cherry Point WWTP to the Great Swamp. Currently, the discharge is 2.5 mgd, and SCDHEC is currently processing a permit modification to increase the permitted discharge to 7.5 mgd. There are no major indirect discharges in the watershed.

7.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the New River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing land use conditions.

The results are presented in Table 7-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

The direct discharge from the Cherry Point WWTP to the Great Swamp could be a significant source of nutrient loads in the watershed. Currently, the discharge is 2.5 mgd, and SCDHEC is currently processing a permit modification to increase the permitted discharge to 7.5 mgd. Based on the values in Table 2-9, the wastewater load could account for 15 to 20 percent of the TN and TP load at a 2.5 mgd discharge, and 30 to 40 percent of the TN and TP load at a 7.5 mgd discharge. These values consider only the loads from Beaufort County, and do not include the rest of the New River tributary area, which is much larger than Beaufort County's tributary area. Note that the concentration values used in the calculations are not based on actual measured discharge concentrations. Even at the high flow rate of 7.5 mgd, the low bacteria and TSS concentrations of the wastewater would limit the point load contribution to less than 1 percent of the total bacteria and TSS load from Beaufort County.

7.3.4 Management Strategy Alternatives

Besides the enforcement of the BMP Manual requirements for new development (and maintenance of existing BMPs), no specific recommendations are made for the New River watershed. For informational purposes, the areas with "A" and "B" type soils are presented in Figure 7-6. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

Discussion of water quality related recommendations for monitoring in the New River watershed are presented as part of the overall recommended monitoring for Beaufort County contained in the Appendix of this report.

7.4 Planning Level Cost Estimates for Management Alternatives

Table 7-12 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the New River watershed. As shown in the table, the projects are estimated to have a total cost of $0.646 million in January 2017 dollars. Details of the cost estimate for each project are shown in Appendix E.

TABLE 7-1
HYDROLOGIC BASINS
NEW RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Bloody Point	240	1	240
Bluffton Trail	1,081	2	541
Daufuskie South	672	2	336
Eigelberger	43	1	43
Jones Tract North	1,325	3	442
Mungen	281	2	141
New River East	378	2	189
Oak Ridge	703	2	351
Pritchardville West	504	3	168
SC-170/SC-146	567	3	189
TOTAL	5,794	21	276

TABLE 7-2
WATER QUALITY BASINS
NEW RIVER WATERSHED

Basin Name	Tributary Area (acres)
New River 1	6,592
New River 2	10,341
New River 3	5,881
TOTAL	22,815

TABLE 7-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
NEW RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Bloody Point Basin					
BP_M1	240	77	115	79	114
Bluffton Trail Basin					
BT_M1	481	87	164	86	143
BT_M2	600	77	186	79	161
Daufuskie South Basin					
DS_M1	600	77	247	81	218
DS_M2	73	77	85	84	73
Eigelberger Basin					
E_M1	43	77	47	82	43
Jones Tract North Basin					
JTN_M1	659	86	136	90	115
JTN_M2	386	88	111	85	87
JTN_T2	279	90	107	90	89
Mungen Basin					
M_M1	190	64	131	69	118
M_M2	91	66	112	72	98
New River East Basin					
NRE_M1	277	89	127	86	107
NRE_M2	101	90	41	91	41
Oak Ridge Basin					
OR_M1	124	66	105	73	95
OR_M2	579	76	201	83	172
Pritchardville West Basin					
PW_M1	322	83	177	76	147
PW_M2	62	80	67	78	57
PW_M3	120	87	102	77	85
SC-170/SC-146 Basin					
SC170_M1	490	84	136	87	110
SC170_M2	39	89	47	83	36
SC170_T1	38	68	64	72	51
Average	276	80	119	81	103

TABLE 7-4
HYDRAULIC DATA SUMMARY
NEW RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Bloody Point	2	1,853	0	0	0	1	0	0
Bluffton Trail	3	3,624	0	0	0	0	0	0
Daufuskie South	9	9,775	3	4	0	1	2	0
Eigelberger	2	1,095	1	1	0	1	1	0
Jones Tract North	16	17,184	0	0	0	0	1	0
Mungen	5	4,902	2	2	0	1	2	0
New River East	4	5,248	3	6	1	1	1	0
Oak Ridge	9	9,392	3	3	3	1	3	0
Pritchardville West	9	10,492	0	0	0	0	0	0
SC-170/SC-146	10	10,104	2	2	0	3	1	0
TOTAL	69	73,669	14	18	4	9	11	0

TABLE 7-5
CULVERT DATA FOR HYDROLOGIC BASINS
NEW RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Bloody Point Basin						
No road crossings in this basin						
Bluffton Trail Basin						
No road crossings in this basin						
Daufuskie South Basin						
Benjies Point Road	DS_M-2A	36"x36"	30	-0.2	7.2	25
	2B	36"x36"	30	1.7		
Church Road	DS_M-6	30"x30"	45	5.6	11.4	25
Haig Point Road	DS_M-12	24"x24"	45	10.2	17.1	25
Eigelburger Basin						
Prospect Road	E_M-3	15"x15"	25	4.7	7.2	25
Jones Tract North Basin						
No road crossings in this basin						
Mungen Basin						
Prospect Road	M_M-3	24"x24"	30	3.1	5.8	25
School Road	M_M-7	18"x18"	30	7.5	10.2	25
New River East Basin						
Unknown Road	NRE_M-2	Bridge	15	7.0	14.0	25
Unknown Road	NRE_M-4A	36"x36"	65	6.0	12.2	25
	4B	36"x36"	65	5.8		
	4C	36"x36"	65	5.9		
Col. T. Hayward Road	NRE_M-7A	30"x30"	75	9.5	13.9	25
	7B	30"x30"	75	9.1		
	7C	30"x30"	75	9.0		
Oak Ridge Basin						
Prospect Road	OR_M-3	36"x36"	20	2.6	7.0	25
Beach Drive	OR_M-6	18"x18"	36	5.4	8.2	25
Oak Ridge Lane	OR_M-9	30"x30"	60	2.6	9.6	25
Pritchardville West Basin						
No road crossings in this basin						
SC-170/SC-146 Basin						
Okatie Highway (State Hwy 46)	SC170_M-12	24"x24"	42	29.7	34.6	100
Okatie Highway (State Hwy 46)	SC170_T1-2	48"x48"	25	4.4	18.0	100

TABLE 7-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
NEW RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Bloody Point Basin					
No Overtopping Identified					
Bluffton Trail Basin					
No Overtopping Identified					
Daufuskie South Basin					
Benjies Point Road	DS_M-37	7.2	7.2	2	7.5
				10	7.6
				25	7.6
				100	7.6
Eigelburger Basin					
Prospect Road	E_M-12	7.2	7.2	2	7.4
				10	7.8
				25	7.8
				100	7.8
Jones Tract North Basin					
Location Unknown	JTN_M-1	N/A	7.1 (Railroad Crest)	10	7.4
				25	7.5
				100	7.7
No Road Crossing	JTN_T1-13	N/A	7.6	25	7.6
				100	7.9
Mungen Basin					
Prospect Road	M_M-26	5.8	5.8	2	6.2
				25	6.5
				100	6.7
School Road	M_M-50	10.2	10.2	2	10.5
				10	10.6
				25	10.7
				100	10.8
New River East Basin					
No Overtopping Identified					
Oak Ridge Basin					
Prospect Road	OR_M-20	7.0	7.0	10	7.4
				25	7.5
				100	7.7
Beach Drive	OR_M-48	8.2	8.2	2	8.3
				10	8.5
				25	8.5
				100	8.6
Pritchardville West Basin					
No Overtopping Identified					
SC-170/ SC-146 Basin					
No Overtopping Identified					

TABLE 7-7
STREAM CROSSINGS FOR NEW RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Bloody Point Basin			
No improvements required			
Bluffton Trail Basin			
No improvements required			
Daufuskie South Basin			
Benjies Point Road	DS_M-2A 2B	36"x36" 36"x36"	Add seven 36" pipes to existing culverts
Eigelburger Basin			
Prospect Road	E_M-3	15"x15"	Replace culvert with two 36" pipes
Jones Tract North Basin			
No improvements required			
Mungen Basin			
Prospect Road	M_M-3	24"x24"	Raise road from elevation 5.8 ft to 7.6 ft NAVD (length of 360 ft), Replace culvert with four 8 ft by 4 ft box culverts
School Road	M_M-7	18"x18"	Replace culverts with four 36" pipes
New River East Basin			
No improvements required			
Oak Ridge Basin			
Prospect Road	OR_M-3	36"x36"	Raise road from elevation 7.0 ft to elevation 8.0 ft NAVD (length of 260 ft), Add three 36" pipes to existing culvert
Beach Drive	OR_M-6	18"x18"	Raise road from elevation 8.2 ft to elevation 9.0 ft NAVD (length of 170 ft)
Pritchardville West Basin			
No improvements required			
SC-170/ SC-146 Basin			
No improvements required			

TABLE 7-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
NEW RIVER WATERSHED

Land Use Type	New River 1 (acres)	New River 2 (acres)	New River 3 (acres)	TOTAL (acres)
Agricultural/Pasture	0	53	0	53
Commercial	50	22	2	74
Forest/Rural Open	725	2801	1224	4749
Golf Course	322	0	150	472
High Density Residential	2039	552	0	2591
Industrial	309	51	100	460
Institutional	77	0	0	77
Low Density Residential	40	107	414	560
Medium Density Residential	0	1	16	18
Open Water/Tidal	354	2106	3265	5724
Silviculture	0	1031	0	1031
Urban Open	256	996	165	1417
Wetland/Water	2414	2585	535	5535
TOTAL	6584	10307	5871	22762
Urban Imperviousness (%)	20%	4%	2%	8%

TABLE 7-9
WATER QUALITY BASIN BMP COVERAGE
NEW RIVER WATERSHED

Land Use Type	New River 1	New River 2	New River 3	TOTAL
Commercial	15.2%	0.2%	0.0%	10.2%
Golf Course	4.3%	0.0%	0.0%	2.9%
High Density Residential	20.6%	100.0%	0.0%	37.5%
Industrial	2.2%	1.4%	0.0%	1.6%
Institutional	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	3.8%	0.0%	0.7%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%
TOTAL	6.8%	5.4%	0.0%	4.4%

TABLE 7-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
NEW RIVER WATERSHED

Land Use Type	New River 1	New River 2	New River 3	TOTAL
Commercial	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	5.9%	4.4%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%
TOTAL	0.0%	0.0%	0.4%	0.1%

TABLE 7-11

AVERAGE ANNUAL LOADS FOR NEW RIVER WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
New River 1	6584	10043	163000	303	89844	1.49E+15	231	42742	5519	1710000	2077
New River 2	10307	14303	120000	179	122000	1.14E+15	168	51595	5446	694000	3339
New River 3	5871	13946	126000	182	121000	1.28E+15	236	50844	6447	533000	4920
TOTAL	22762	38292	409000	664	332844	3.91E+15	635	145181	17412	2937000	10336

TABLE 7-12 (Updated 2017)

PLANNING LEVEL COST ESTIMATES FOR

NEW RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
DS_M-2	Road overtopping at Benjies Point Road Add 7 - 36" RCP to existing 2 - 36" RCP	$76,000
E_M-3	Road overtopping at Prospect Road Replace existing 1 - 15" CMP with 2 - 36" RCP	$33,000
M_M-3	Road overtopping at Prospect Road Replace existing 1 - 24" CMP with 4 - 8'x4' box culverts Raise road 1.8 feet (length of 360 ft)	$339,000
M_M-7	Road overtopping at School Road Replace existing 1 - 18" RCP with 4 - 36" RCP	$48,000
OR_M-3	Road overtopping at Prospect Road Add 3 - 36" RCP to existing 1 - 36" CMP Raise road 1.0 feet (length of 260 ft)	$45,000
OR_M-6	Road overtopping at Beach Drive Raise road 0.8 feet (length of 170 ft)	$105,000
	TOTAL	$646,000

Costs are in January 2018 dollars.

See Appendix for basis of cost estimates.

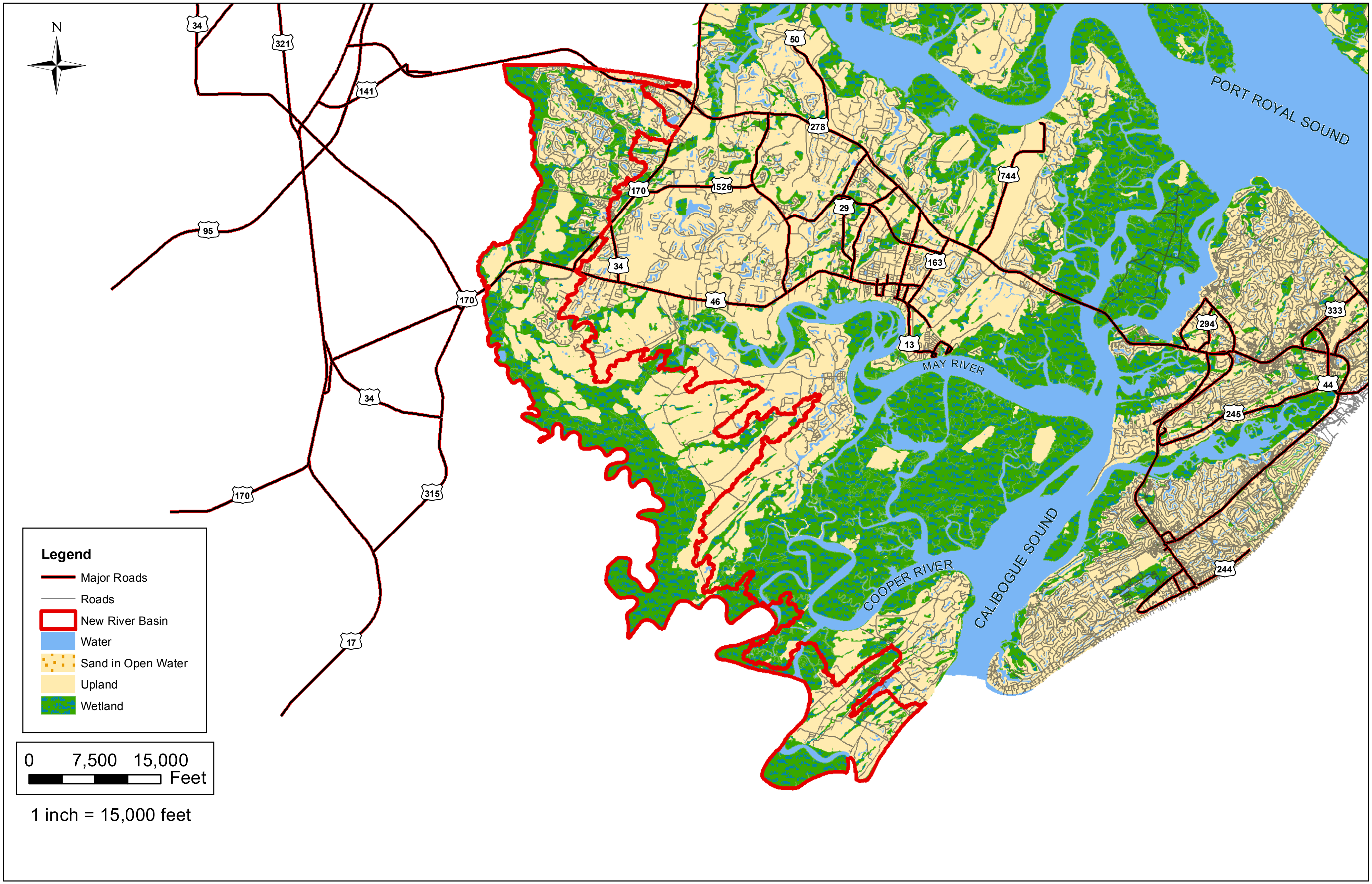
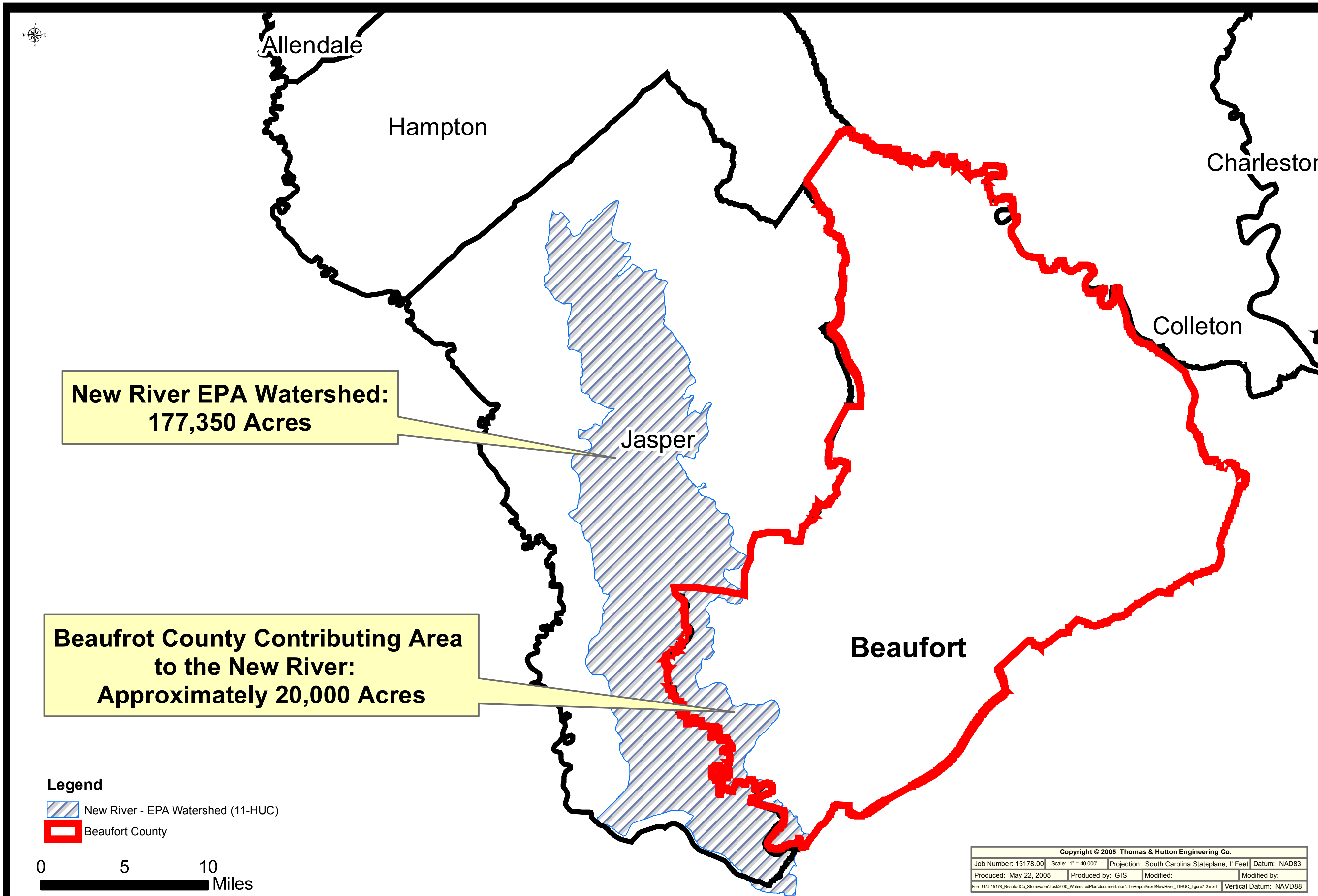


Figure 7-1

New River Watershed (Beaufort County)

ATM DESIGN ENGINEERING CONSULTING



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Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\NewRiver_11HUC_figure7-2.mxd			Vertical Datum: NAVD88

THOMAS & HUTTON ENGINEERING CO.
50 PARK OF COMMERCE WAY
SAVANNAH, GEORGIA 31405
(912) 234-5300

Figure 7-2

New River Watershed
(Total: EPA Watershed 11-HUC)

CDM Camp Dresser & McKee Inc.

Disclaimer
Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Counties	SC DNR	
New River Watershed	EPA	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE

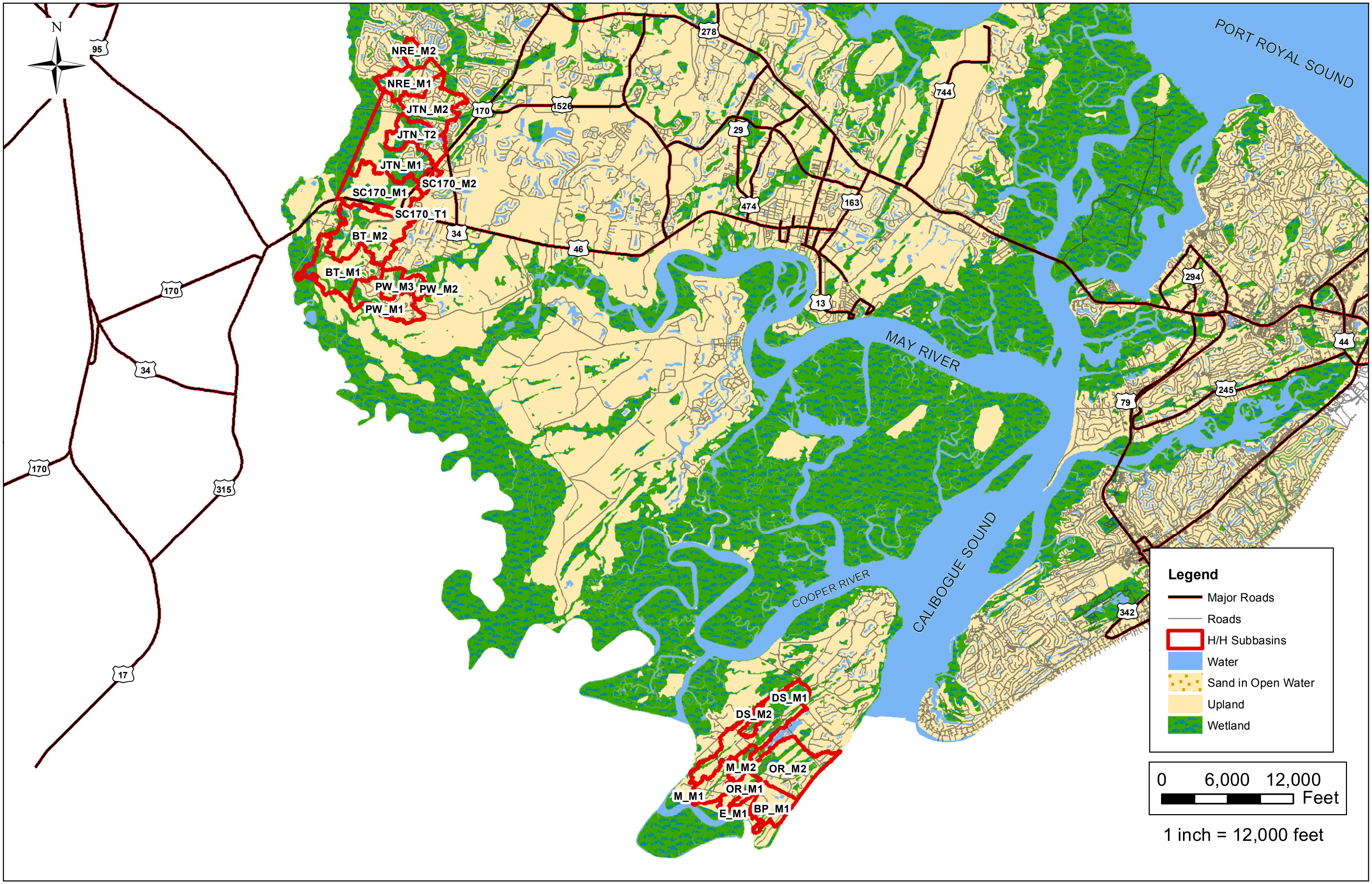

Figure 7-3. Hydrologic Model Subbasins

New River Watershed

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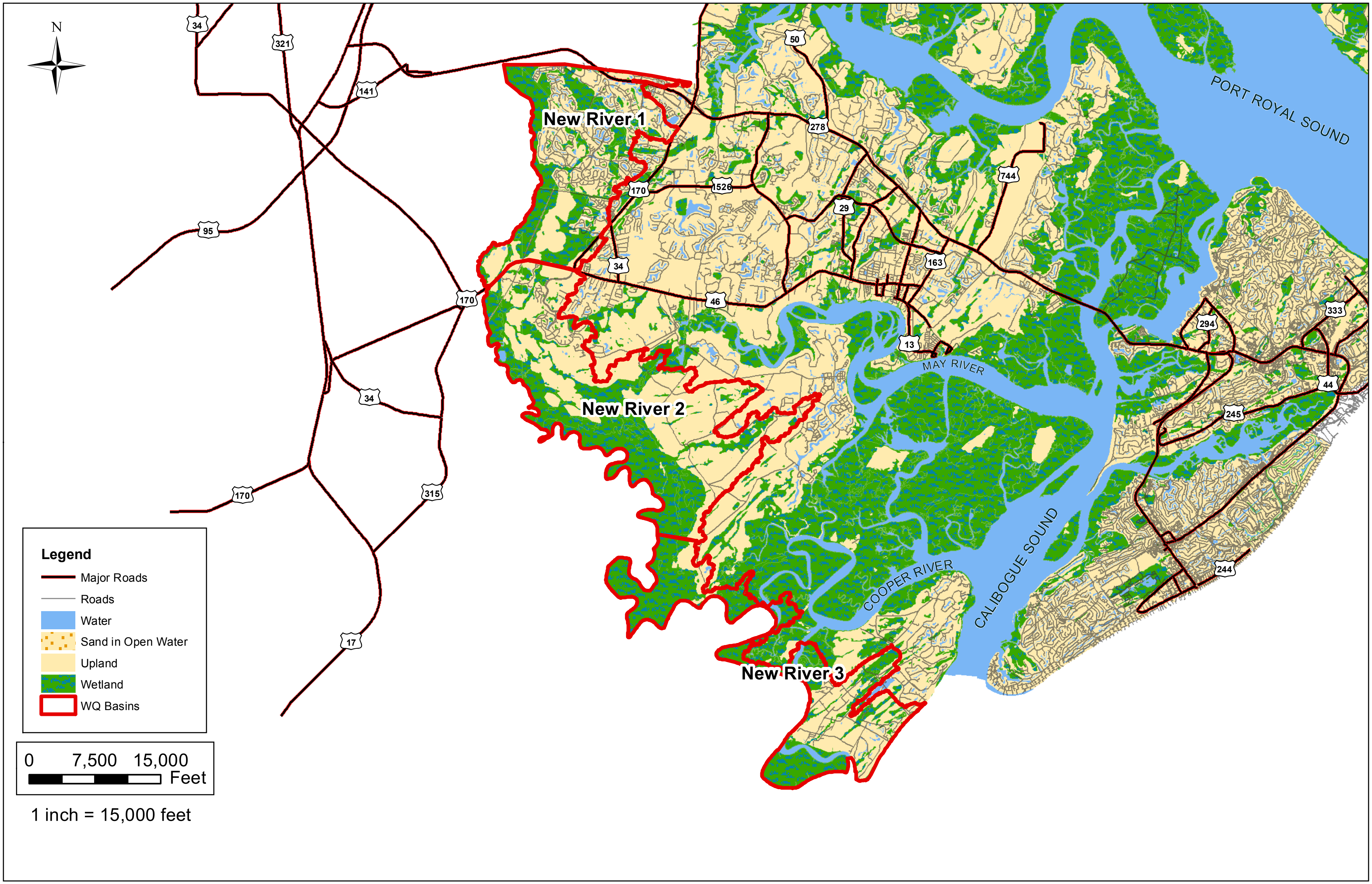
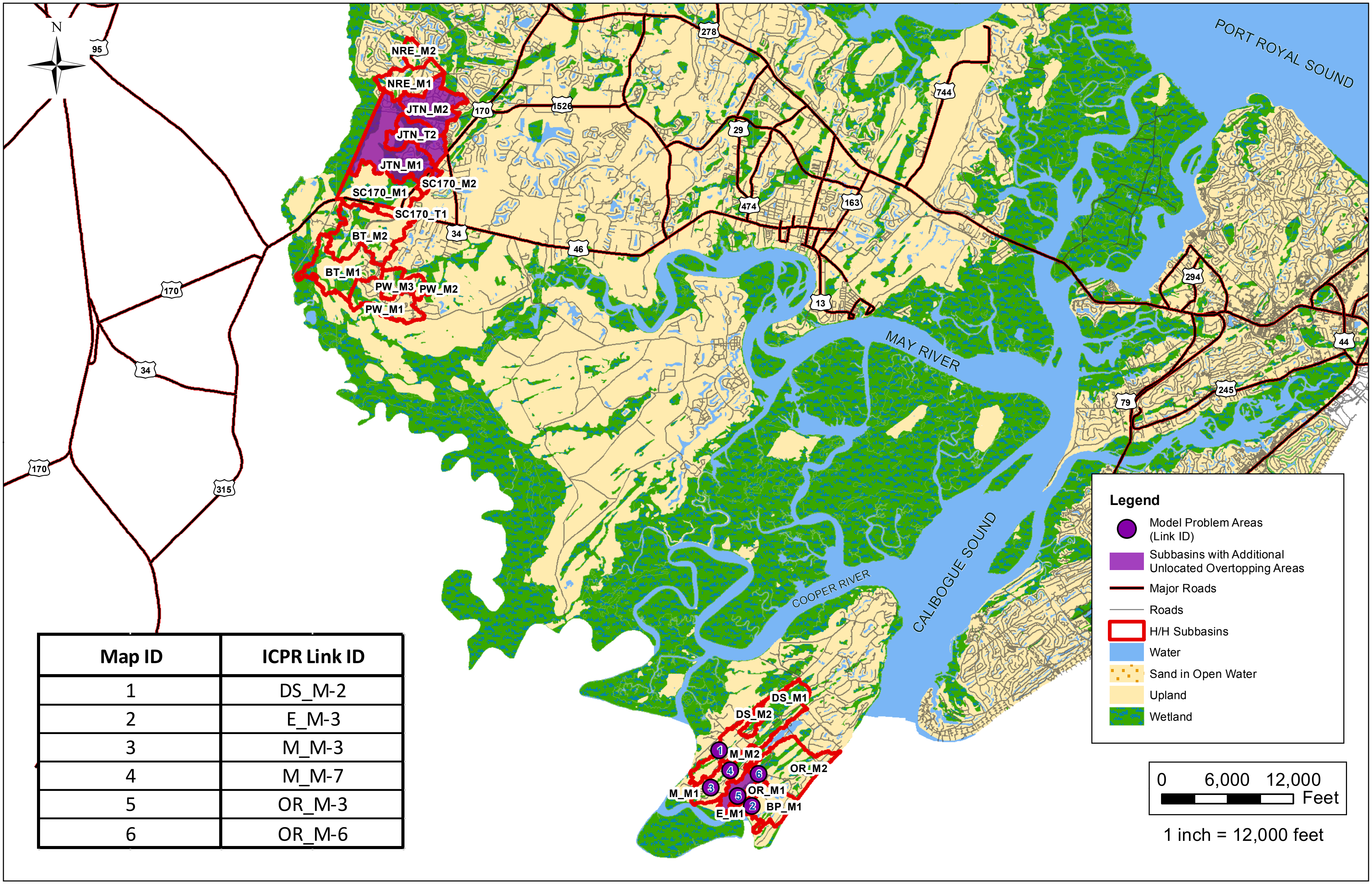


Figure 7-4

New River Watershed
Water Quality Basins

ATM DESIGN ENGINEERING CONSULTING

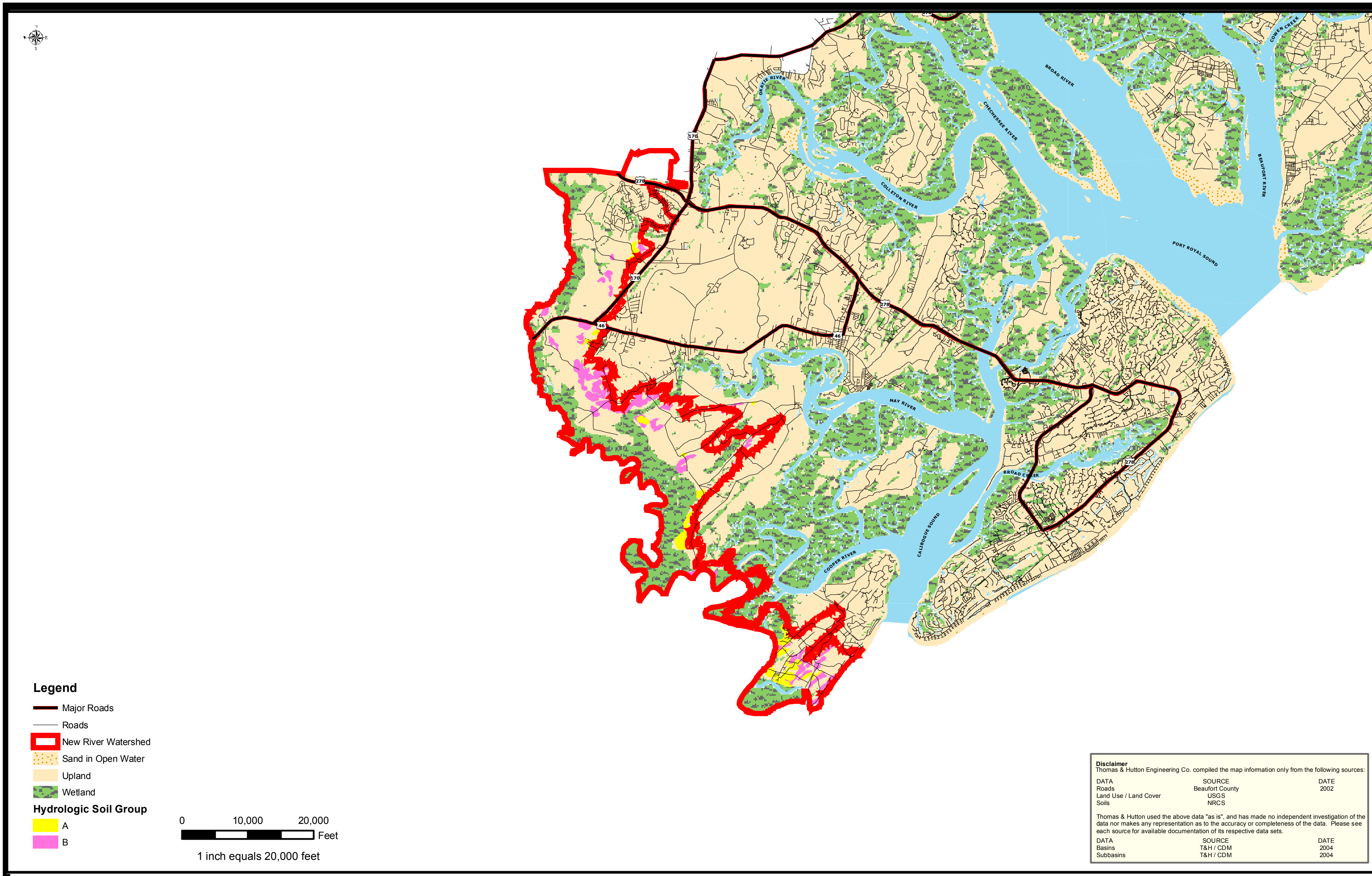


Map ID	ICPR Link ID
1	DS_M-2
2	E_M-3
3	M_M-3
4	M_M-7
5	OR_M-3
6	OR_M-6

Figure 7-5. ICPR Identified PSMS Overtopping Problem Areas

New River Watershed

ATM DESIGN ENGINEERING CONSULTING



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Job Number: 15178.00	Scale: 1" = 20,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\NewRiverSoils_figure7-6.mxd			Vertical Datum: NAVD88

New River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 7-6

CDM Camp Dresser & McKee Inc.

Section 8
Beaufort River Watershed Analysis

This section describes the physical features of the Beaufort River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

8.1 Overview

The Beaufort River watershed is located north of the Broad River (see Figure 8-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in the City of Beaufort, the Town of Port Royal, Port Royal Island, Lady's Island and St. Helena Island that is tributary to the Beaufort River. Major Beaufort River tributaries included in the analysis are Battery Creek, Cowen Creek, Distant Island Creek, Capers Creek, Broomfield Creek and Albergotti Creek.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several "hydrologic" basins. These are listed in Table 8-1 and presented in Figure 8-2. Table 8-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were completed to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into "water quality" basins, and the tidal receiving waters were subdivided into receiving water "segments". These are listed in Table 8-2 and presented in Figure 8-3. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were completed to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

8.2 Hydrologic and Hydraulic Analysis

The ICPR, Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the Beaufort River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP.

8.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Beaufort River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 8-3 lists the hydrologic parameter values for the Beaufort River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the Beaufort River PSMS basins is presented in Table 8-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 8-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

8.2.2 Model Results

Tables in Appendix F list the summary of the results of the updated study including Updated Areas and CNs for the Beaufort River subbasins.

For existing land use, aerial maps generated in the summer of 2016 and local information were used to estimate the percentage of existing urban development.

Appendix F also includes tables that list the peak water elevation values for model node locations along the Beaufort River PSMS.

Specific problem areas identified by the modeling are listed in Table 8-6 and presented in Figure 8-4. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs, and results showed that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded.

Table 8-6 indicates the road crossings that are being overtopped by the design storm events.

Evaluation of solutions to prevent these problems is discussed in the next section of this report.

8.2.3 Management Strategy Alternatives

The problems areas listed in Table 8-6 were evaluated by reviewing the previous reports results and reviewing the culverts in the ICPR hydraulic model. In the original 2006 study, the ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 8-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were typically used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

8.3 Water Quality Analysis

ATM used the WMM and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the Beaufort River watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions.

8.3.1 Land Use and BMP Coverage

Table 8-8 presents the existing land use and future land use estimates for the Beaufort River water quality basins. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 38 percent of the Beaufort River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 62 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 15 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in Table 8-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, less than 1 percent of the urban systems in the watershed are served by BMPs.

8.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in Table 8-10. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water and Sewer Authority. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 37 percent of the urban systems in the watershed are served by septic.

Based on available data, the estimated wastewater discharge under existing conditions is 3.1 mgd of direct discharge to the Beaufort River and Albergotti Creek, and the future discharge is expected to be 3.8 mgd based on increase in residential land between existing and future conditions. There are no indirect discharges (e.g., sprayfields) in the watershed.

8.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Beaufort River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing land use conditions.

The results are presented in Table 8-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

Wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing discharge of wastewater is limited to roughly 3.1 mgd of direct discharge, and the future discharge is expected to be higher (3.8 mgd). Using the values in Table 2-9, the wastewater load for existing conditions accounts for 25 to 35 percent of the total watershed load for nutrients (TN and TP), 5 to 10 percent of the load for BOD and metals, and less than 1 percent of the load for TSS and bacteria. It should be noted that some values (e.g., TN and TP) are not based on actual discharge concentration values.

The Beaufort River was evaluated at part of a total maximum daily load (TMDL) evaluation for DO in the Beaufort River (Conrads et al., 2003). The results suggested that the existing discharges are having a minimal effect on DO concentrations in the Beaufort River.

8.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Beaufort River watershed. The model actually includes Beaufort River, Coosaw River, Whale Branch West and Morgan River watersheds because they are interconnected at several points. Only the Beaufort River will be discussed in this section. A schematic of the model is presented as Figure 8-5.

Existing conditions for bacteria concentrations in the Beaufort River are presented in Table 8-12. For each water quality basin river reach, the table lists the SCDHEC stations for which the bacteria data were analyzed, the concentrations calculated in the analysis, and the LOS associated with these concentrations (as discussed in Section 2.6.2. As shown in the table, SCDHEC data were only available in ten of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations meet the water quality standards in five of the ten monitored segments, and so these segments have an “A” LOS. Segments that do not meet the “A” LOS include Brickyard Creek 1, Battery Creek 1 and 2, Capers Creek 1 and Albergotti Creek 1.

For informational purposes, Figure 8-6 presents a map of the LOS based on the monitoring data analysis, compared to SCDHEC “shellfish classification” (based on the 2016 SCDHEC reports for shellfish area 15). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the LOS, so there may not be a direct relationship between LOS and shellfish classification presented in the map. In general, however, segments with an “A” LOS are expected to have the lowest probability of receiving a “restricted” classification, and segments with a “D” LOS are expected to have the highest probability of receiving a “restricted” classification.

Physical characteristics assigned to the model reaches are presented in Table 8-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the “characteristic length” (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the SCDHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Table 8-14 presents the values used in the existing condition models.

Table 8-15 shows the net advective flows between segments. The hydrodynamic model (SWMM5) indicates that there is a substantial net flow from Port Royal Sound to the Beaufort River, and net flow actually moving upstream (i.e., from the mouth of the Beaufort River toward the river headwaters). It should be noted that data from USGS monitoring stations in the Beaufort River support the premise of net “upstream” flow.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. The calibrated loss-rate coefficients from the 2006 study were used in the updated simulations.

Figure 8-7 is a graph showing a comparison between measured and modeled salinity data along the Beaufort River main. The figure shows that the salinity data calculated by the model is generally within the 90 percent confidence interval of the mean of the salinity data. Adjusting tidal dispersion values further to get a better salinity match tended to make the agreement between measured and modeled bacteria worse, so the results were considered acceptable.

The comparison between measured and modeled salinity data in Battery Creek is presented in Figure 8-8. Both the measured and modeled salinity values show little change between segments, and the modeled values are well within the 90 percent confidence interval of the mean of the salinity data.

Figures 8-9 through 8-11 show the measured and modeled salinity values for Cowen Creek (Figure 8-9) and Distant Island Creek and Capers Creek, which are both tributary to Cowen Creek. Again, the modeled values are always within the 90 percent confidence interval of the measured salinity values. Salinity values in Distant Island Creek tend to be higher than in Capers Creek, which suggests that there is better mixing between Cowen Creek and Distant Island Creek than there is between Cowen Creek and Capers Creek.

The comparison between measured and modeled salinity data in Albergotti Creek is presented in Figure 8-12. The modeled value in Albergotti Creek 1 is within the 90 percent confidence interval of the measured salinity values.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration for the Beaufort River is presented in Figure 8-13. The graph shows very good agreement between the measured values and the model results except for Beaufort River 3 and Brickyard Creek 1, where the modeled value is actually higher than the high end of the 90 percent confidence interval. However, the modeled value of 7/100 mL matches the maximum threshold for the “A” LOS (7/100 mL), so this overestimation is not considered critical.

Figure 8-14 shows the comparison of measured and modeled bacteria concentrations in Battery Creek. The model is underpredicting as compared to the measured geomean values for the segments with monitoring data (Battery Creek 1 and Battery Creek 2).

The comparison of measured and modeled bacteria concentrations for Cowen Creek and its tributaries, Distant Island Creek and Capers Creek, are presented in Figures 8-15 through 8-17. In all cases, the model is underpredicting but were close to the 90 percent confidence interval of the measured bacteria values.

The comparison between measured and modeled bacteria data in Albergotti Creek is presented in Figure 8-18. The modeled value in Albergotti Creek 1 is very close to the geomean of the measured bacteria values. The model is underpredicting but was close to the 90 percent confidence interval of the measured bacteria values.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 8-16. The loss rates ranged from 0.5/day to 2.0/day. The lowest values generally occur at the downstream end of the Beaufort River and major tributaries, with higher values in the upstream end of some tributaries. This makes sense if it is presumed that bacteria loss is in part due to light mortality, because the water depths are much greater at the downstream end of the Beaufort River and major tributaries, and therefore light would be less of a factor relative to the shallower reaches.

Based on water quality sampling data and model results, the following conclusions are:

- Problem basins include Battery Creek 2, 3 and 4, Capers Creek 2 and 3, Broomfield Creek 2, Albergotti Creek 1 and 2
- 3 new regional water quality BMPs are proposed in Battery Creek 2 and Albergotti Creek 2 basins

Discussion of water quality related recommendations for monitoring and regional BMPs in the Beaufort River watershed are presented as part of the overall recommended monitoring and CIP program for Beaufort County contained in the Appendix of this report.

8.3.5 Management Strategy Alternatives

In analyzing the watershed, two feasible regional detention sites were identified. The area tributary to the Albergotti Creek 2 Regional BMP site includes approximately 172 acres of rural and single-family development built prior to stormwater regulations. There are no stormwater best management practices, such as detention facilities, in the area. The project would be to construct a regional wet detention facility adjacent to Roseida Road to provide stormwater runoff water quality treatment and volume reduction. Due to the presence of some wetlands in the area, project design would involve delineation and avoidance of the wetlands.

A new WMM scenario was developed for the Albergotti Creek 2 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Albergotti Creek 2 water quality basin of approximately 4%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Beaufort River:

Parameter	lb/yr removed
Total Nitrogen	270
Total Phosphorus	82
TSS	31,783

The area tributary to the Battery Creek N1 Regional BMP site includes approximately 274 acres of commercial and residential development built prior to volume control stormwater regulations. There are no stormwater best management practices, such as detention facilities, in the area. The project would be to construct a regional wet detention facility adjacent to Salem Road to provide stormwater runoff water quality treatment and volume reduction. Due to the presence of some wetlands in the area, project design would involve delineation and avoidance of the wetlands.

A new WMM scenario was developed for the Battery Creek N1 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Battery Creek 2 water quality basin of approximately 6%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Beaufort River:

Parameter	lb/yr removed
Total Nitrogen	678
Total Phosphorus	128
TSS	79,724

The area tributary to the Battery Creek N2 Regional BMP site includes approximately 67 acres of intense commercial development built prior to volume control stormwater regulations. There are limited stormwater best management practices, such as detention facilities, in the area. The project would be to construct a regional wet detention facility adjacent to Spanish Moss Trail to provide stormwater runoff water quality treatment and volume reduction. Due to the presence of some wetlands in the area, project design would involve delineation and avoidance of the wetlands.

A new WMM scenario was developed for the Battery Creek N2 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Battery Creek 2 water quality basin of approximately 2%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Beaufort River:

Parameter	lb/yr removed
Total Nitrogen	435
Total Phosphorus	79
TSS	53,190

The results of the water quality analysis suggest that the limited amount of future development in the watershed, combined with the effectiveness of required BMPs in reducing bacteria loads from new development, will maintain the existing LOS in all water quality segments. Areas have been identified above for evaluation of measures to improve the existing LOS. These activities would include retrofit of existing development that does not have BMPs, and modification of existing ponds that may not have been designed for water quality control.

For informational purposes, the areas with “A” and “B” type soils are presented in Figure 8-20. In general, these soils are more suitable for infiltration BMPs than areas with “C” and “D” type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

8.4 Planning Level Cost Estimates for Management Alternatives

Table 8-20 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Beaufort River watershed. As shown in the table, the projects are estimated to have a total cost of $3.932 million in January 2018 dollars. Details of the cost estimate for each project are shown in Appendix F.

Three regional CIP projects were identified in the Beaufort River watershed. These two projects are estimated to have a total cost of $2.59 million and are detailed in the CIP in Appendix O.

TABLE 8-1
HYDROLOGIC BASINS
BEAUFORT RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Albergotti Creek	869	3	290
Ballpark Road	304	1	304
Battery Creek East	256	1	256
Battery Creek North	274	1	274
Battery Creek West	468	1	468
Burton Hill	487	2	244
Capers Creek	336	1	336
Capers Road	248	1	248
Grober Hill	431	2	216
Mulligan Creek	281	1	281
Salt Creek	917	3	306
Salt Creek South	343	1	343
Shanklin Road	794	2	397
Southside	412	3	137
Wallace Creek	509	2	254
TOTAL	6,928	25	277

TABLE 8-2
WATER QUALITY BASINS
BEAUFORT RIVER WATERSHED

Basin Name	Tributary Area (acres)
Beaufort River 1	9,468
Beaufort River 2	2,980
Beaufort River 3	6,177
Battery Creek 1	5,792
Battery Creek 2	3,579
Battery Creek 3	391
Battery Creek 4	183
Cowen Creek 1	1,081
Cowen Creek 2	731
Cowen Creek 3	122
Capers Creek 1	2,931
Capers Creek 2	1,268
Capers Creek 3	711
Distant Island Creek 1	745
Distant Island Creek 2	1,536
Distant Island Creek 3	426
Broomfield Creek 1	783
Broomfiled Creek 2	721
Albergotti Creek 1	2,515
Albergotti Creek 2	2,780
Brickyard Creek 1	1,373
TOTAL	46,292

TABLE 8-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
BEAUFORT RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Albergotti Creek Basin					
AC_M1	178	92	62	93	62
AC_M2	260	90	88	90	88
AC_M3	431	90	96	90	96
Ballpark Road Basin					
BR_M1	304	74	165	77	147
Battery Creek East Basin					
BCE_M1	256	78	105	79	100
Battery Creek North Basin					
BYCN_M1	274	84	136	92	89
Battery Creek West Basin					
BYCW_M1	468	76	116	82	105
Burton Hill Basin					
BH_M1	165	79	92	90	63
BH_M2	323	81	128	88	117
Capers Creek Basin					
CC_M1	336	78	132	81	125
Capers Road Basin					
CR_M1	248	75	136	81	126
Grober Hill Basin					
GH_M1	263	84	78	89	74
GH_M2	168	82	66	89	62
Mulligan Creek Basin					
MNC_M1	281	89	83	92	83
Salt Creek Basin					
SC_M1	347	84	118	89	111
SC_M2	153	83	63	88	56
SC_M3	417	82	147	87	127
Salt Creek South Basin					
SCS_M1	343	75	136	82	115
Shanklin Road Basin					
SR_M1	175	81	86	82	83
SR_M2	619	82	142	89	118
Southside Basin					
SHE_M1	100	83	51	83	51
SHE_M2	198	82	83	83	82
SHE_T1	114	85	57	85	56
Wallace Creek Basin					
WC_M1	276	78	120	81	116
WC_M2	233	82	105	84	97
Average	277	82	104	86	94

TABLE 8-4
HYDRAULIC DATA SUMMARY
BEAUFORT RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Albergotti Creek	3	2,730	2	10	0	1	1	0
Ballpark Road	1	823	1	1	0	1	1	0
Battery Creek East	1	140	2	3	0	4	2	0
Battery Creek North	2	1,421	1	1	0	1	0	0
Battery Creek West	2	1,851	1	2	0	1	1	0
Burton Hill	2	2,018	2	1	1	1	2	0
Capers Creek	2	2,093	1	1	0	0	0	0
Capers Road	1	467	0	0	0	0	0	0
Grober Hill	1	858	1	2	0	1	1	0
Mulligan Creek	2	1,475	0	0	0	0	0	0
Salt Creek	7	6,733	2	4	0	1	2	0
Salt Creek South	1	522	1	1	0	1	1	0
Shanklin Road	6	6,792	3	6	0	1	2	0
Southside	7	3,300	5	9	0	3	2	0
Wallace Creek	4	2,333	1	2	0	1	1	0
TOTAL	42	33,556	23	43	1	17	16	0

TABLE 8-5
CULVERT DATA FOR HYDROLOGIC BASINS
BEAUFORT RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Albergotti Creek Basin						
Geiger Road	AC_M-1A	72"x72"	90	-1.6	8.9	25
	1B	72"x72"	90	-1.8		
	1C	72"x72"	90	-1.8		
	1D	72"x72"	90	-2.3		
	1E	72"x72"	90	-2.3		
Rimes Avenue	AC_M-4A	60"x60"	150	1.8	10.8	25
	4B	60"x60"	150	1.7		
	4C	60"x60"	150	2.1		
	4D	60"x60"	150	2.4		
	4E	60"x60"	150	2.6		
Ballpark Road Basin						
Halifax Drive	BR_M-2	24"x24"	40	1.1	5.8	25
Battery Creek East Basin						
Battery Creek Road	BCE_M-1A	36"x36"	60	1.9	7.7	25
	1B	24"x24"	60	0.8		
June Way	BCE_M-3	48"x48"	45	0.8	7.4	25
Battery Creek North Basin						
Robert Smalls Parkway (State Hwy 170)	BYCN_M-3	30"x30"	120	6.5	14.0	100
Battery Creek West Basin						
Parris Island Gateway (State Hwy 802)	BYCW_M-1A	48"x48"	100	0.5	7.5	100
	1B	48"x48"	100	0.7		
Burton Hill Basin						
Old Jerico Road	BH_M-0A	168"x35"	23.3	5.4	10.0	25
	0B	168"x32"	23.3	5.6		
Robert Smalls Parkway (State Hwy 170)	BH_M-2	48"x48"	180	3.4	13.5	100
Capers Creek Basin						
Scott Hill Road	CC_M-1	Bridge	25	1.7	8.4	25
Grober Hill Basin						
Munich Road	GH_M-2A	48"x48"	80	4.3	10.8	25
	2B	48"x48"	80	4.1		
Goethe Hill Road	GH_M-4A	30"x30"	40	7.1	12.8	25
	4B	30"x30"	40	4.8		
Robert Smalls Parkway (State Hwy 170)	GH_M-6	30"x30"	120	6.6	16.0	100
Salt Creek Basin						
Laurel Bay Road	SC_M-4A	36"x36"	100	12.7	19.6	25
	4B	36"x36"	100	13.1		
Shanklin Road Basin	SC_M-7A	42"x42"	80	24.6	35.8	25
	7B	24"x24"	80	25.6		
Salt Creek South Basin						
County Shed Road	SCS_M-1	24"x24"	60	-0.7	6.3	25
Shanklin Road Basin						
Roseida Road	SR_M-3A	48"x48"	50	2.8	10.3	25
	3B	48"x48"	50	3.3		
Fort Sumter Road	SR_M-9A	48"x48"	20	5.6	12.5	25
	9B	48"x48"	20	5.3		
Laurel Bay Road	SR_M-5A	48"x48"	60	5.6	14.5	25
	5B	48"x48"	60	5.1		
Southside Basin						
Battery Creek Road	SHE_M-3A	30"x30"	40	2.5	6.3	25
	3B	30"x30"	40	2.4		
Southside Blvd.	SHE_M-6A	30"x30"	50	2.2	8.2	25
	6B	30"x30"	50	2.1		
	6C	30"x30"	50	2.4		
Broad Street	SHE_M-9	24"x24"	25	4.6	11.7	25
Battery Creek Blvd.	SHE_T1-3	12"x12"	25	4.6	14.0	25
Wallace Creek Basin						
Orange Grove Road	WC_M-2A	30"x30"	40	2.3	8.2	25
	2B	30"x30"	40	1.9		

TABLE 8-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
BEAUFORT RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Ballpark Road Basin					
Halifax Drive	BR_M-11	5.8	5.8	2	6.1
				10	6.1
				25	6.1
				100	6.1
Battery Creek East Basin					
Battery Creek Road	BCE_M-8	7.7	7.7	10	7.9
				25	7.9
				100	7.9
June Way	BCE_M-11	7.4	7.4	2	7.5
				10	7.9
				25	7.9
				100	7.9
Battery Creek North Basin					
Robert Smalls Parkway (State Hwy 170)	BYCN_M-21	14.0	14.0	25	14.1
				100	14.2
Battery Creek West Basin					
Parris Island Gateway (State Hwy 802)	BYCW_M-5	7.5	7.5	10	7.6
				25	7.6
				100	7.6
Burton Hill Basin					
Old Jerico Road	BH_M-7	10.0	8.0	100	8.6
Robert Smalls Parkway (State Hwy 170)	BH_M-21	13.5	13.5	100	13.5
No Road Crossing	BH_M-31	N/A	13.4	25	13.4
				100	13.6
Grober Hill Basin					
Munich Road	GH_M-21	10.8	10.8	10	11.0
				25	11.0
				100	11.1
Goethe Hill Road	GH_M-33	12.8	12.8	100	13.3
Robert Smalls Parkway (State Hwy 170)	GH_M-39	16.0	16.0	25	15.9
				100	16.2
Salt Creek Basin					
Laurel Bay Road	SC_M-59	19.6	19.6	10	19.8
				25	19.8
				100	19.8
Salt Creek South Basin					
No Overtopping Identified					
Shanklin Road Basin					
Roseida Road	SR_M-21	10.3	10.3	2	10.5
				10	10.9
				25	10.9
				100	11.0
Fort Sumter Road	SR_M-34	12.5	12.5	10	12.5
				25	12.5
				100	12.6
Laurel Bay Road	SR_M-36	14.5	14.5	100	14.6
Southside Basin					
Battery Creek Road	SHE_M-5	6.3	6.3	2	6.5
				10	7.1
				25	7.3
				100	7.9
No Road Crossing	SHE_M-8	N/A	6.8	10	7.1
				25	7.3
				100	7.9
Broad Street	SHE_M-28	11.7	11.7	2	10.0
				10	11.1
				25	11.5
				100	12.5
No Road Crossing	SHE_T1-1	N/A	6.9	10	7.1
				25	7.3
				100	7.9
Battery Creek Blvd.	SHE_T1-12	14.0	14.0	100	14.2
Wallace Creek Basin					
Orange Grove Road	WC_M-6	8.2	8.2	10	8.4
				25	8.6
				100	8.7

TABLE 8-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
BEAUFORT RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Ballpark Road Basin			
Halifax Drive	BR_M-2	24"x24"	Raise road from elevation 5.8 ft to elevation 7.6 ft NAVD (length of 1,340 ft), Replace culvert with one 8 ft by 4 ft box culvert
Battery Creek East Basin			
Battery Creek Road	BCE_M-1A 1B	36"x36" 24"x24"	Replace culverts with one 10 ft by 5 ft box culvert
June Way	BCE_M-3	48"x48"	Replace culvert with two 8 ft by 5 ft box culverts
Battery Creek East Basin			
Robert Smalls Parkway (State Hwy 170)	BYCN_M-3	30"x30"	Replace culvert with one 6 ft by 4 ft box culvert
Battery Creek West Basin			
Parris Island Gateway (State Hwy 802)	BYCW_M-1A 1B	48"x48" 48"x48"	Install Tidal Gate & Replace culverts with two 10 ft by 5 ft box culverts
Burton Hill Basin			
Robert Smalls Parkway (State Hwy 170)	BH_M-2	48"x48"	Replace culvert with one 8 ft by 5 ft box culvert
Grober Hill Basin			
Munich Road	GH_M-2A 2B	48"x48" 48"x48"	Replace culverts with three 8 ft by 4 ft box culverts
Goethe Hill Road	GH_M-4A 4B	30"x30" 30"x30"	Replace culverts with two 42" pipes
Robert Smalls Parkway (State Hwy 170)	GH_M-6	30"x30"	Replace culvert with one 5 ft by 4 ft box culvert
Salt Creek Basin			
Laurel Bay Road	SC_M-4A 4B	36"x36" 36"x36"	Add one 48" pipe to existing culverts
Salt Creek South Basin			
*County Shed Road	SCS_M-1	24"x24"	Replace culvert with one 6 ft by 4 ft box culvert
Shanklin Road Basin			
Roseida Road	SR_M-3A 3B	48"x48" 48"x48"	Raise road from elevation 10.3 ft to elevation 12.0 ft NAVD (length of 570 ft), Replace culverts with one 12 ft by 8 ft box culvert, Lower culvert invert to 3.0 ft NAVD
Fort Sumter Drive	SR_M-9A 9B	48"x48" 48"x48"	Replace culverts with one 12 ft by 6 ft box culvert, Lower culvert invert to 5.0 ft NAVD
Laurel Bay Road	SR_M-5A 5B	48"x48" 48"x48"	Add one 48" pipe to existing culverts
Southside Basin			
*Railroad Tracks	SHE_M-1A 1B	36"x36" 24"x24"	Railroad crossing improvements needed to prevent significant backwater effects: Replace culverts with one 4 ft by 4 ft box culvert
Battery Creek Road	SHE_M-3A 3B	30"x30" 30"x30"	Raise road from elevation 6.3 ft to elevation 8.0 ft NAVD (length of 750 ft), Replace culverts with one 6 ft by 4 ft box culvert, Set culvert invert to 2.0 ft NAVD
Wallace Creek Basin			
Orange Grove Road	WC_M-2A 2B	30"x30" 30"x30"	Replace culverts with one 8 ft by 4 ft box culvert

* Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR.

TABLE 8-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
BEAUFORT RIVER WATERSHED

Land Use Type	Beaufort River 1 (acres)	Beaufort River 2 (acres)	Beaufort River 3 (acres)	Battery Creek 1 (acres)	Battery Creek 2 (acres)	Battery Creek 3 (acres)	Battery Creek 4 (acres)	Cowen Creek 1 (acres)	Cowen Creek 2 (acres)	Cowen Creek 3 (acres)	Capers Creek 1 (acres)	Capers Creek 2 (acres)
Agricultural/Pasture	653	34	0	0	0	0	0	59	75	0	421	17
Commercial	27	64	209	185	377	38	26	5	2	0	13	6
Forest/Rural Open	1238	145	195	393	349	1	4	64	76	2	468	266
Golf Course	34	0	85	0	0	0	0	37	0	0	0	0
High Density Residential	0	333	518	831	368	124	49	0	0	0	0	0
Industrial	833	117	317	510	293	54	43	21	21	9	49	32
Institutional	0	140	46	47	67	15	28	0	17	0	0	9
Low Density Residential	432	82	176	72	213	12	1	55	81	8	543	384
Medium Density Residential	191	52	524	401	364	0	1	89	48	0	0	0
Open Water/Tidal	5859	1959	4027	3107	924	140	11	747	406	94	1279	418
Silviculture	0	0	0	0	170	0	0	0	0	0	0	0
Urban Open	48	34	66	113	80	1	17	2	4	8	37	42
Wetland/Water	153	14	8	115	375	6	2	3	0	0	124	96
TOTAL	9467	2975	6171	5774	3579	391	183	1081	731	121	2935	1270
Urban Imperviousness (%)	8%	13%	14%	19%	24%	36%	48%	4%	6%	6%	4%	6%

TABLE 8-8 (CONTINUED)
WATER QUALITY BASIN LAND USE DISTRIBUTION
BEAUFORT RIVER WATERSHED

Land Use Type	Capers Creek 3 (acres)	Distant Island Creek 1 (acres)	Distant Island Creek 2 (acres)	Distant Island Creek 3 (acres)	Broomfield Creek 1 (acres)	Broomfield Creek 2 (acres)	Albergotti Creek 1 (acres)	Albergotti Creek 2 (acres)	Brickyard Creek 1	TOTAL (acres)
Agricultural/Pasture	54	10	7	0	0	0	0	251	0	1580
Commercial	33	3	43	54	0	91	79	70	0	1325
Forest/Rural Open	146	118	248	34	113	137	97	427	27	4548
Golf Course	0	41	88	33	19	3	0	0	29	367
High Density Residential	0	0	0	0	0	0	60	19	21	2323
Industrial	30	12	48	36	48	33	1164	543	425	4639
Institutional	43	3	4	16	0	0	0	32	0	467
Low Density Residential	176	1	182	64	147	207	31	585	0	3452
Medium Density Residential	10	97	22	27	147	55	32	49	51	2162
Open Water/Tidal	122	460	868	147	294	110	945	229	810	22955
Silviculture	0	0	0	0	0	0	0	42	0	212
Urban Open	62	1	5	5	5	18	26	130	0	705
Wetland/Water	35	0	22	10	9	66	77	404	11	1531
TOTAL	711	745	1537	426	783	720	2511	2780	1373	46265
Urban Imperviousness (%)	13%	5%	7%	22%	11%	19%	38%	20%	24%	15%

TABLE 8-9
WATER QUALITY BASIN BMP COVERAGE
BEAUFORT RIVER WATERSHED

Land Use Type	Albergotti Creek 1	Albergotti Creek 2	Battery Creek 1	Battery Creek 2	Battery Creek 3	Battery Creek 4	Beaufort River 1	Beaufort River 2	Beaufort River 3	Broomfield Creek 1
Commercial	0.9%	0.0%	12.2%	19.4%	0.0%	0.8%	0.0%	0.0%	0.5%	0.0%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.2%	0.0%	1.3%	1.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	1.2%	2.3%	0.0%	0.0%	0.0%	2.2%	2.1%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	15.4%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	6.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	0.0%	0.0%	4.1%	2.8%	0.0%	0.0%	0.0%	0.0%	1.3%	0.0%
TOTAL	0.0%	0.0%	1.0%	3.0%	0.0%	2.4%	0.0%	0.1%	0.2%	0.0%

TABLE 8-9 (C0NTINUED)
WATER QUALITY BASIN BMP COVERAGE
BEAUFORT RIVER WATERSHED

Land Use Type	Broomfield Creek 2	Capers Creek 1	Capers Creek 2	Capers Creek 3	Cowen Creek 1	Cowen Creek 2	Cowen Creek 3	Distant Island Creek 1	Distant Island Creek 2	Distant Island Creek 3	Brickyard Creek 1	TOTAL
Commercial	0.0%	0.0%	0.0%	9.4%	0.0%	0.0%	0.0%	85.6%	5.3%	0.0%	0.0%	7.9%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.7%
Industrial	0.0%	0.0%	4.6%	3.1%	0.0%	0.0%	0.0%	0.0%	12.0%	0.0%	0.0%	0.6%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.9%
Low Density Residential	0.0%	0.0%	1.3%	1.7%	0.0%	0.0%	0.0%	0.0%	4.2%	0.0%	0.0%	1.0%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	37.4%	0.0%	0.0%	1.9%
TOTAL	0.0%	0.0%	0.5%	1.0%	0.0%	0.0%	0.0%	0.3%	1.6%	0.0%	0.0%	0.5%

TABLE 8-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
BEAUFORT RIVER WATERSHED

Land Use Type	Albergotti Creek 1	Albergotti Creek 2	Battery Creek 1	Battery Creek 2	Battery Creek 3	Battery Creek 4	Beaufort River 1	Beaufort River 2	Beaufort River 3	Broomfield Creek 1
Commercial	0.0%	7.5%	0.7%	0.0%	0.0%	0.0%	5.8%	0.1%	1.0%	0.0%
High Density Residential	0.8%	0.0%	0.1%	0.0%	0.2%	0.0%	0.0%	0.3%	0.5%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%	0.0%	0.0%	0.0%
Institutional	0.0%	30.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	4.6%	6.0%	7.9%	1.9%	0.0%	0.0%	5.3%	0.0%	10.6%	3.4%
Medium Density Residential	2.9%	3.1%	0.3%	0.4%	0.0%	0.0%	6.7%	0.2%	2.5%	0.0%
TOTAL	0.1%	1.9%	0.2%	0.2%	0.1%	0.0%	0.4%	0.0%	0.6%	0.6%

TABLE 8-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
BEAUFORT RIVER WATERSHED

Land Use Type	Broomfield Creek 2	Capers Creek 1	Capers Creek 2	Capers Creek 3	Cowen Creek 1	Cowen Creek 2	Cowen Creek 3	Distant Island Creek 1	Distant Island Creek 2	Distant Island Creek 3	Brickyard Creek 1	TOTAL
Commercial	0.0%	0.0%	0.0%	1.0%	0.0%	100.0%	0.0%	0.0%	3.7%	0.1%	0.0%	1.1%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%
Industrial	0.0%	0.0%	1.3%	0.2%	0.0%	0.0%	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	2.1%
Low Density Residential	6.7%	12.3%	16.2%	13.7%	5.7%	2.9%	2.8%	0.0%	7.5%	1.3%	0.0%	8.1%
Medium Density Residential	5.2%	0.0%	0.0%	0.0%	2.4%	0.0%	0.0%	0.1%	3.7%	0.0%	0.0%	1.7%
TOTAL	2.3%	2.3%	5.0%	3.4%	0.5%	0.7%	0.2%	0.0%	1.1%	0.2%	0.0%	0.8%

TABLE 8-11
AVERAGE ANNUAL LOADS FOR BEAUFORT RIVER WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
Beaufort River 1	9,467	25,259	261,000	467	218,000	2.32E+15	480	95,259	12,603	1,360,000	9,327
Beaufort River 2	2,975	8,767	101,000	159	76,759	9.63E+14	184	33,575	4,305	611,000	3,309
Beaufort River 3	6,171	18,182	215,000	340	159,000	2.01E+15	396	69,861	9,185	1,320,000	6,823
Battery Creek 1	5,774	15,901	210,000	368	140,000	1.88E+15	375	61,880	8,177	1,550,000	5,792
Battery Creek 2	3,579	7,424	123,000	215	66,424	1.04E+15	203	30,940	4,074	1,120,000	2,328
Battery Creek 3	391	1,066	19,179	36	9,613	1.58E+14	32	4,670	612	183,000	379
Battery Creek 4	183	397	9,691	21	3,699	7.59E+13	15	1,945	251	108,000	126
Cowen Creek 1	1,081	3,006	29,055	41	26,055	2.91E+14	56	11,154	1,543	121,000	1,133
Cowen Creek 2	731	1,751	18,572	27	15,299	1.93E+14	34	6,695	955	97,670	640
Cowen Creek 3	121	375	3,640	6	3,233	3.29E+13	7	1,375	171	15,449	144
Capers Creek 1	2,935	5,764	58,398	83	50,188	6.03E+14	105	22,067	3,314	310,000	1,985
Capers Creek 2	1,270	2,173	25,364	35	19,207	2.83E+14	45	8,392	1,138	170,000	702
Capers Creek 3	711	996	15,648	25	9,012	1.61E+14	26	4,148	625	145,000	289
Distant Island Creek 1	745	1,936	18,907	27	16,810	1.93E+14	36	7,173	981	84,908	704
Distant Island Creek 2	1,537	3,761	38,557	58	32,639	3.68E+14	73	13,795	1,895	198,000	1,375
Distant Island Creek 3	426	952	14,968	26	8,487	1.25E+14	26	4,027	562	130,000	326
Broomfield Creek 1	783	1,555	21,166	32	13,899	2.22E+14	39	6,211	903	154,000	528
Broomfield Creek 2	720	1,142	20,806	32	10,387	1.87E+14	35	5,054	693	205,000	328
Albergottie Creek 1	2,511	7,182	127,000	333	63,375	8.10E+14	214	31,261	3,836	1,200,000	2,523
Albergottie Creek 2	2,780	11,434	136,000	256	99,791	1.22E+15	242	44,882	5,877	925,000	3,988
Brickyard Creek 1	1,373	4,239	58,669	137	37,003	4.29E+14	104	17,087	2,155	449,000	1,570
TOTAL	46,265	123,262	1,525,620	2,724	1,078,880	1.36E+16	2,727	481,451	63,855	10,457,027	44,319

TABLE 8-12
EXISTING LEVEL OF SERVICE FOR WATER QUALITY BASINS
BEAUFORT RIVER WATERSHED

Water Quality Basin ID	DHEC Station(s)	Years of Record	No. of Samples	Fecal Coliform Concentrations				Trend	Level of Service
				Long-Term Average		Most Recent 3 Year Values			
				Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)		
Beaufort River 1	15-15, 15-17	1999-2016	409	3.64	13	4.17	17	Increasing	A
Beaufort River 2	15-06,15-14	1999-2016	90	3.39	11.1	3.03	8.79	Increasing	A
Beaufort River 3	15-04,15-05	1999-2016	152	4.66	22	5.18	19.34	Increasing	A
Brickyard Creek 1	15-02	1999-2016	203	7.22	23	8.59	33	No Trend	B
Battery Creek 1	15-10, 15-21, 15-24, 15-25, 15-26, 15-27, 15-28, 15-29	1999-2016	1662	7.36	33	10.1	49	Increasing	B
Battery Creek 2	15-19, 15-30, 15-31, 15-32	1999-2016	692	12.02	69.38	15.62	106.52	No Trend	D
Battery Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Battery Creek 4	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cowen Creek 1	15-18	1999-2016	205	4.39	13	5.95	22.8	Increasing	A
Cowen Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cowen Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Distant Island Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Distant Island Creek 2	15-23	1999-2016	206	4.39	13	10.92	33	Increasing	A
Distant Island Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Capers Creek 1	15-20	1999-2016	205	8.67	41.83	8.69	49	No Trend	B
Capers Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Capers Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Broomfield Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Broomfield Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Albergotti Creek 1	15-03, 15-03A, 15-03B	1999-2016	152	11.79	70	16.44	113.74	Increasing	D
Albergotti Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA

TABLE 8-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

BEAUFORT RIVER WATERSHED

Water Quality Basin ID	North WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Beaufort River 1	1	7.30E+07	Port Royal Sound	12,395	7,081	150
Beaufort River 2	2	2.27E+07	Beaufort River 1	4,530	6,855	150
Beaufort River 3	3	2.77E+07	Beaufort River 2	2,248	6,823	10
Brickyard Creek South	4	5.40E+06	Beaufort River 3	776	4,216	0
			Brickyard Creek North	546	2,784	10
Battery Creek 1	5	1.35E+07	Beaufort River 2	1,808	8,079	450
Battery Creek 2	6	2.98E+06	Battery Creek 1	644	6,727	450
Battery Creek 3	7	4.37E+05	Battery Creek 2	218	2,864	150
Battery Creek 4	8	5.54E+04	Battery Creek 3	184	1,110	150
Cowen Creek 1	9	7.57E+06	Beaufort River 1	2,945	3,476	450
Cowen Creek 2	10	1.20E+06	Cowen Creek 1	349	3,219	150
Cowen Creek 3	11	1.63E+05	Cowen Creek 2	354	1,706	150
Distant Island Creek 1	12	2.33E+06	Cowen Creek 1	501	4,924	150
Distant Island Creek 2	13	2.09E+06	Distant Island Creek 1	502	3,347	150
Distant Island Creek 3	14	1.66E+05	Distant Island Creek 2	252	1,126	150
Capers Creek 1	15	4.09E+06	Cowen Creek 1	1,406	3,476	20
Capers Creek 2	16	1.24E+06	Capers Creek 1	1,082	3,733	20
Capers Creek 3	17	1.29E+05	Capers Creek 2	408	2,189	20
Broomfield Creek 1	18	1.02E+06	Beaufort River 3	708	1,867	150
Broomfield Creek 2	19	1.90E+05	Broomfield Creek 1	194	2,060	150
Albergotti Creek 1	20	2.12E+06	Beaufort River 3	714	4,924	900
Albergotti Creek 2	21	2.68E+05	Albergotti Creek 1	186	3,460	150

TABLE 8-14

VERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WM

FOR BEAUFORT RIVER WATER QUALITY BASINS

Water Quality Basin ID	North WASP Segment	EXISTING LAND USE	
		Flow (cfs)	Fecal Coliform (#/100 ml)
Beaufort River 1	1	42.4	1,064
Beaufort River 2	2	14.5	1,170
Beaufort River 3	3	30.1	1,182
Brickyard Creek 1	4	7.0	1,165
Battery Creek 1	5	26.6	1,182
Battery Creek 2	6	13.2	1,185
Battery Creek 3	7	1.8	1,374
Battery Creek 4	8	0.7	1,536
Cowen Creek 1	9	5.0	1,080
Cowen Creek 2	10	3.0	1,081
Cowen Creek 3	11	0.6	1,087
Distant Island Creek 1	12	3.3	1,048
Distant Island Creek 2	13	6.4	1,048
Distant Island Creek 3	14	1.7	1,195
Capers Creek 1	15	10.3	990
Capers Creek 2	16	4.0	1,020
Capers Creek 3	17	1.9	1,060
Broomfield Creek 1	18	2.8	1,155
Broomfield Creek 2	19	2.2	1,159
Albergotti Creek 1	20	11.9	1,216
Albergotti Creek 2	21	7.7	1,060

TABLE 8-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES

BEAUFORT RIVER WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs)
		Existing
Port Royal Sound	Beaufort River 1	388
Beaufort River 1	Beaufort River 2	422
Beaufort River 2	Beaufort River 3	457
Beaufort River 3	Brickyard Creek South	528
Brickyard Creek South	Brickyard Creek North	555
Battery Creek 1	Beaufort River 2	21
Battery Creek 2	Battery Creek 1	8
Battery Creek 3	Battery Creek 2	6
Battery Creek 4	Battery Creek 3	5
Cowen Creek 1	Beaufort River 1	34
Cowen Creek 2	Cowen Creek 1	11
Cowen Creek 3	Cowen Creek 2	10
Distant Island Creek 1	Cowen Creek 1	9
Distant Island Creek 2	Distant Island Creek 1	5
Distant Island Creek 3	Distant Island Creek 2	3
Capers Creek 1	Cowen Creek 1	11
Capers Creek 2	Capers Creek 1	4
Capers Creek 3	Capers Creek 2	3
Broomfield Creek 1	Beaufort River 3	14
Broomfield Creek 2	Broomfield Creek 1	12
Albergotti Creek 1	Beaufort River 3	27
Albergotti Creek 2	Albergotti Creek 1	7

TABLE 8-16

FECAL COLIFORM MODELING RESULTS

BEAUFORT RIVER WATERSHED

Water Quality Basin ID	Modeled Geomean Conc (#/100 ml)	Modeled Level of Service
	Existing	Existing
Beaufort River 1	3.3	A
Beaufort River 2	3.8	A
Beaufort River 3	7.0	A
Brickyard Creek 1	7.0	B
Battery Creek 1	4.3	A
Battery Creek 2	7.1	B
Battery Creek 3	9.2	C
Battery Creek 4	10.0	C
Cowen Creek 1	3.5	A
Cowen Creek 2	5.2	A
Cowen Creek 3	5.4	A
Distant Island Creek 1	3.9	A
Distant Island Creek 2	4.4	A
Distant Island Creek 3	5.4	A
Capers Creek 1	6.0	A
Capers Creek 2	9.1	C
Capers Creek 3	17.4	D
Broomfield Creek 1	7.7	B
Broomfield Creek 2	10.8	D
Albergotti Creek 1	7.9	B
Albergotti Creek 2	20.4	D

NOTE: Water quality basins with lower LOS are highlighted.

Tables 8-17, 8-18, and 8-19 are not applicable in the update.

TABLE 8-20 (Updated 2017)
PLANNING LEVEL COST ESTIMATES FOR
BEAUFORT RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
BR_M-2	Road overtopping at Halifax Drive Replace existing 1 - 24" RCP with 1 - 8'x4' box culvert Raise road 1.8 ft (length of 1,340 ft)	$844,000
BCE_M-1	Road overtopping at Battery Creek Road Replace existing 1 - 36" RCP and 1 - 24" RCP with 1 - 10'x5' box culvert	$148,000
BCE_M-3	Road overtopping at June Way Replace existing 1 - 48" RCP with 2 - 8'x5' box culverts	$229,000
BH_M-2	Road overtopping at Robert Smalls Parkway (State Hwy 170) Replace existing 1 - 48" RCP with 1 - 8'x5' box culvert	$361,000
BYCN_M-3	Road overtopping at Robert Smalls Parkway (State Hwy 170) Replace existing 1 - 30" RCP with 1 - 6'x4' box culvert	$173,000
BYCW_M-1	Road overtopping at Parris Island Gateway (State Hwy 802) Replace existing 2 - 48" RCP with 2 - 10'x5' box culverts	$418,000
GH_M-2	Road overtopping at Munich Road Replace existing 2 - 48" RCP with 3 - 8'x4' box culverts	$368,000
GH_M-4	Road overtopping at Goethe Hill Road Replace existing 2 - 30" RCP with 2 - 42" RCP	$55,000
GH_M-6	Road overtopping at Robert Smalls Parkway (State Hwy 170) Replace existing 1 - 30" RCP with 1 - 5'x4' box culvert	$158,000
SC_M-4	Road overtopping at Laurel Bay Road Add 1 - 48" RCP to existing 2 - 36" RCP	$48,000
SHE_M-3	Road overtopping at Battery Creek Road Replace existing 2 - 30" RCP with 1 - 6'x4' box culvert Raise road 1.7 ft (length of 750 ft)	$462,000
SR_M-3	Road overtopping at Roseida Road Replace existing 2 - 48" RCP with 1 - 12'x8' box culvert Raise road 1.7 ft (length of 570 ft)	$448,000
SR_M-5	Road overtopping at Laurel Bay Road Add 1 - 48" RCP to existing 2 - 4'x4' box culverts	$42,000
SR_M-9	Road overtopping at Fort Sumter Drive Replace existing 2 - 48" RCP with 1 - 12'x6' box culvert	$67,000
WC_M-2	Road overtopping at Orange Grove Road Replace existing 2 - 30" RCP with 1 - 8'x4' box culvert	$111,000
	TOTAL	$3,932,000

Costs are in January 2018 dollars.
See Appendix for basis of cost estimates.

Tables 8-21 is not applicable in the update.

Figure 8-1

Beaufort River Watershed

ATM DESIGN ENGINEERING CONSULTING

Figure 8-2. Hydrologic Model Subbasins

Beaufort River Watershed

ATM DESIGN ENGINEERING CONSULTING

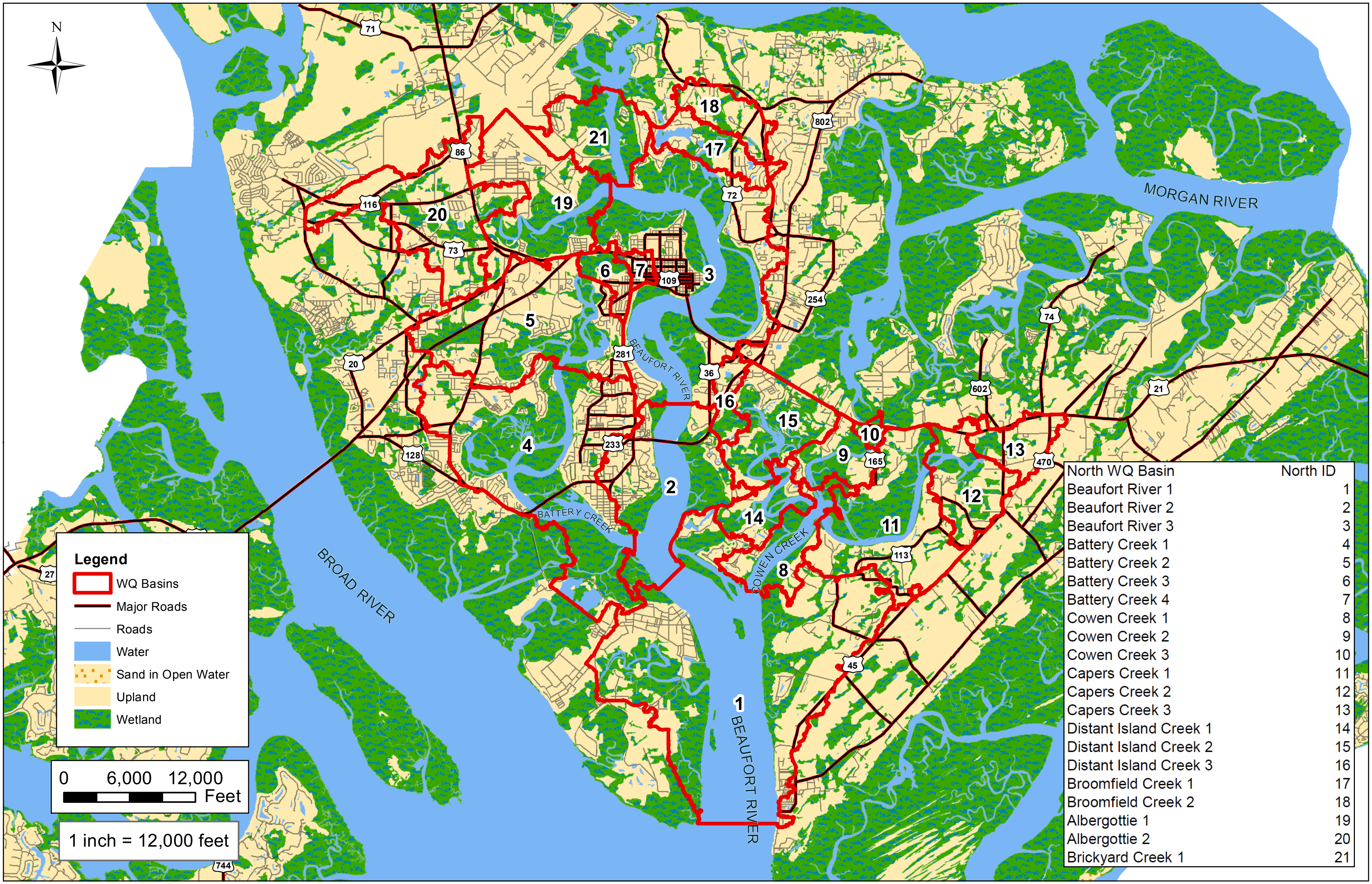
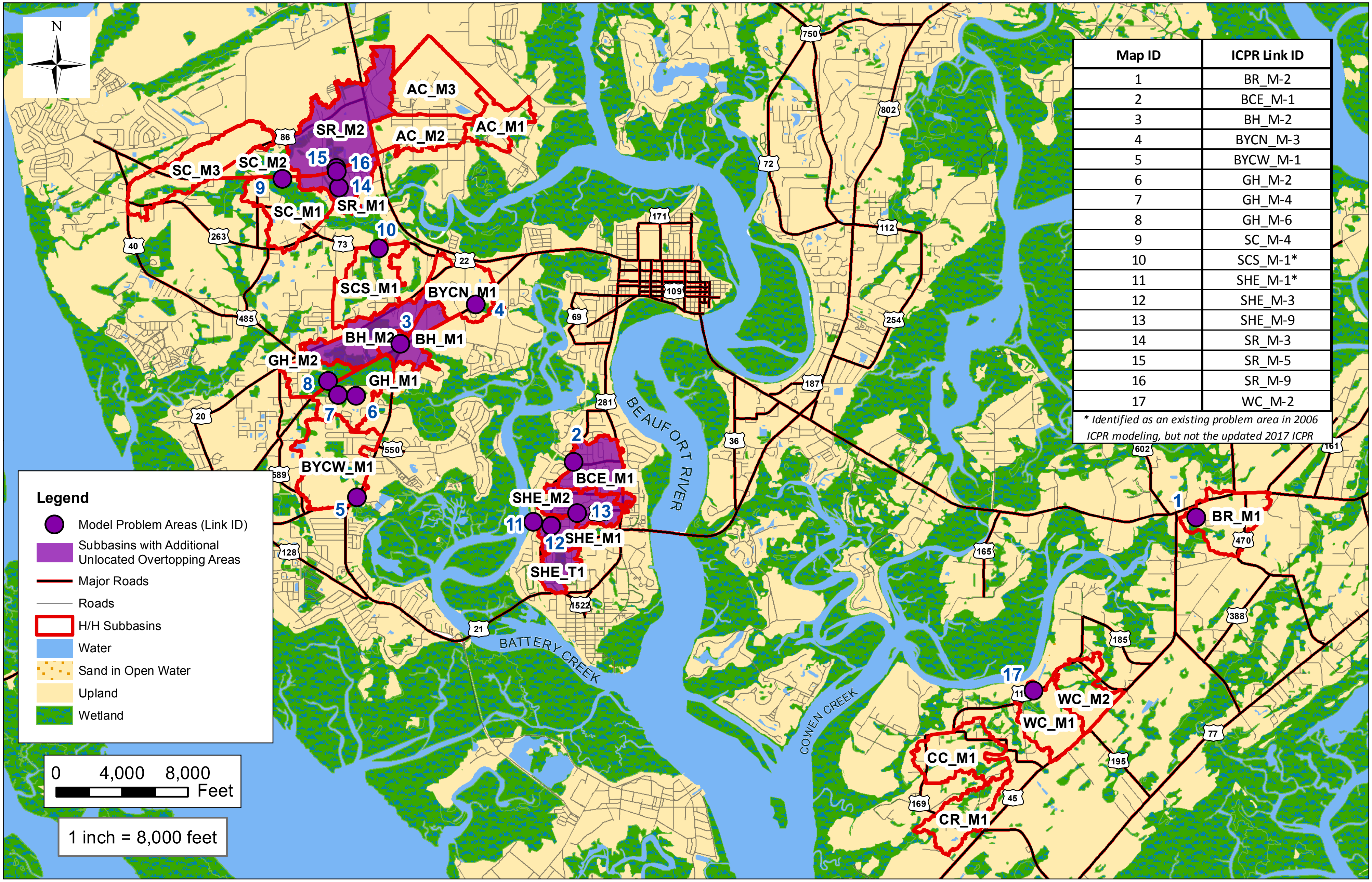


Figure 8-3

Beaufort River Watershed
Water Quality Basins



Map ID	ICPR Link ID
1	BR_M-2
2	BCE_M-1
3	BH_M-2
4	BYCN_M-3
5	BYCW_M-1
6	GH_M-2
7	GH_M-4
8	GH_M-6
9	SC_M-4
10	SCS_M-1*
11	SHE_M-1*
12	SHE_M-3
13	SHE_M-9
14	SR_M-3
15	SR_M-5
16	SR_M-9
17	WC_M-2

** Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR*

Figure 8-4. ICPR Identified PSMS Overtopping Problem Areas

Beaufort River Watershed

ATM DESIGN ENGINEERING CONSULTING

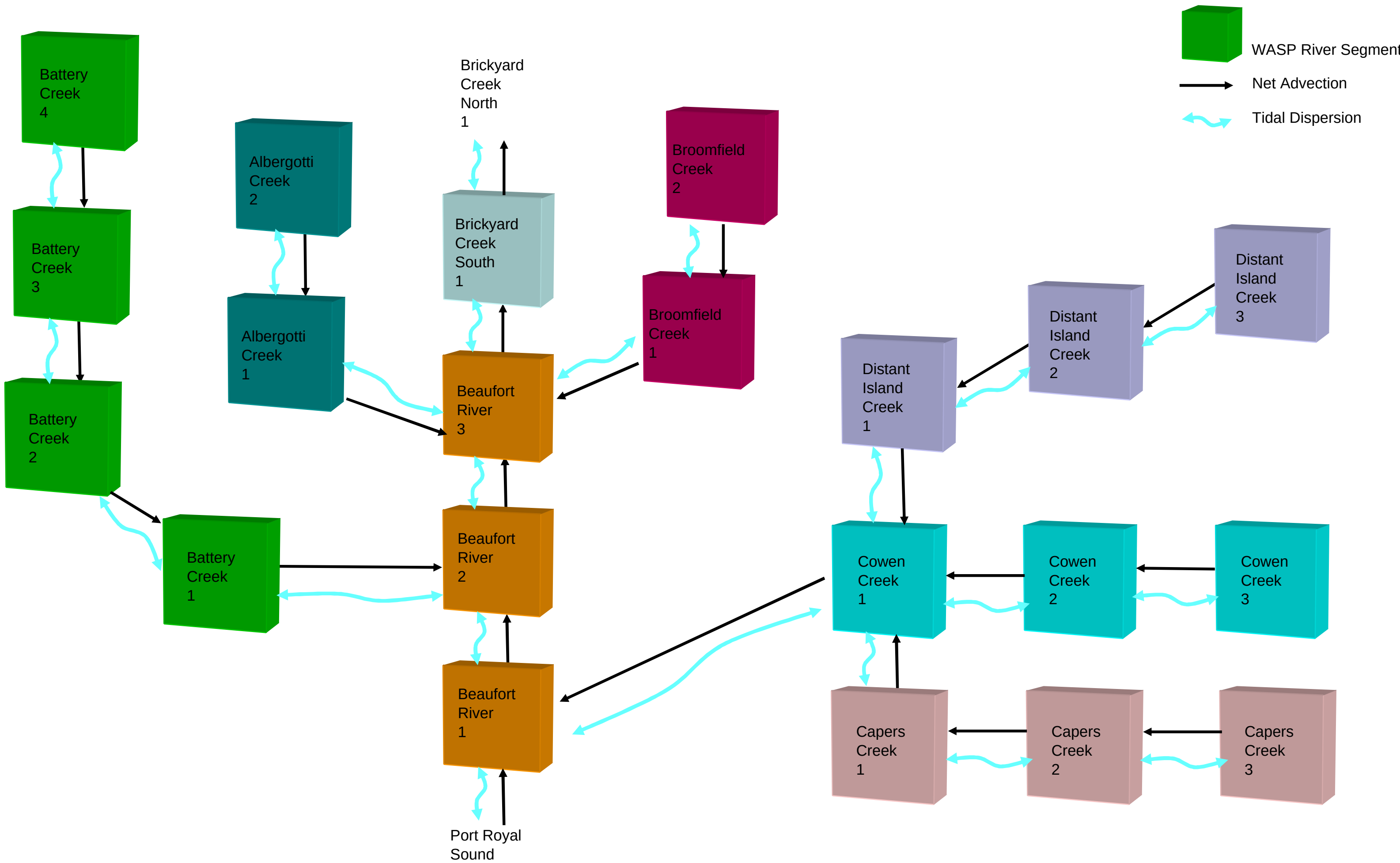


Figure 8-5
WASP Model Schematic for Beaufort River Watershed

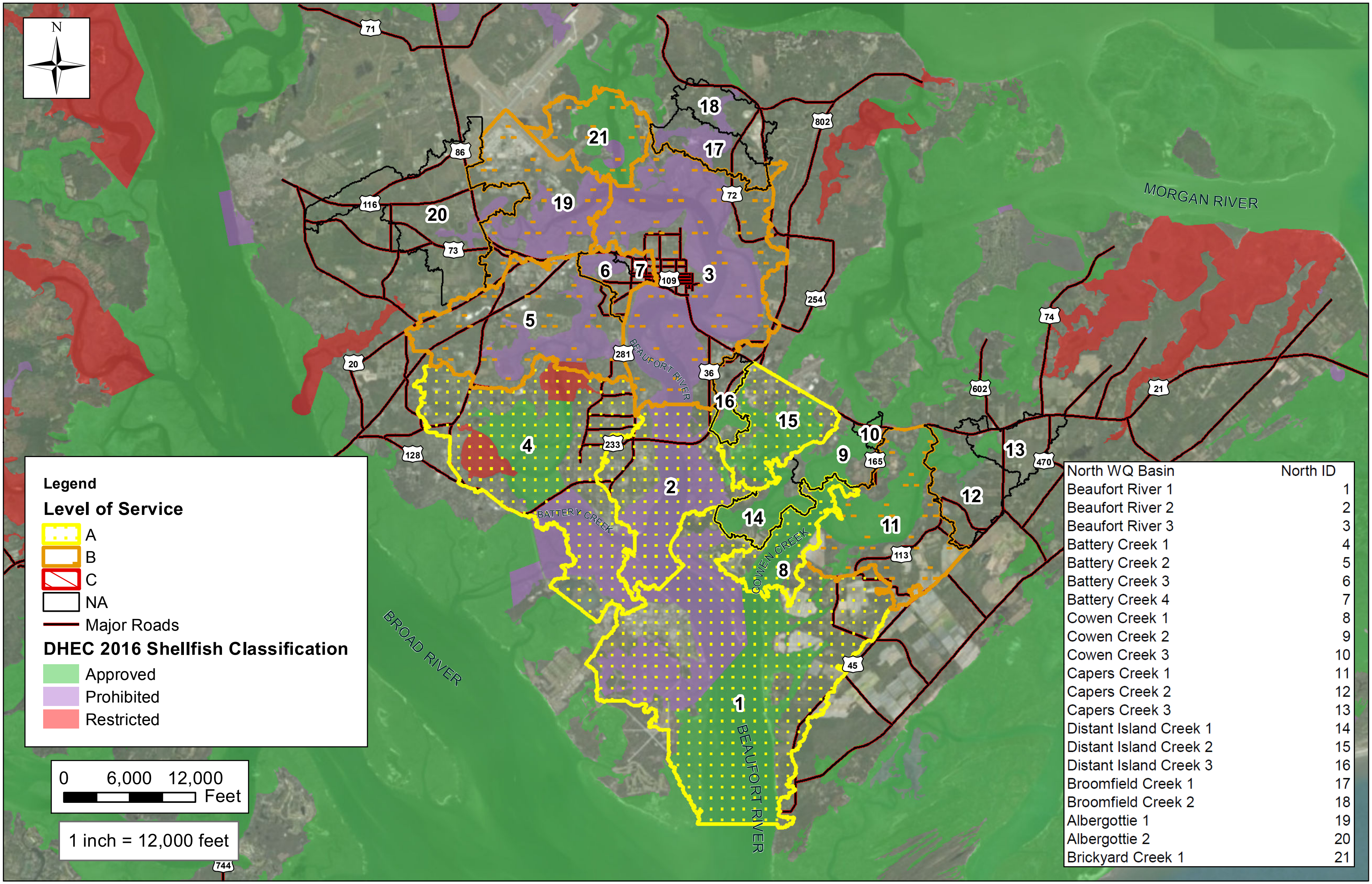


Figure 8-6

Beaufort River Watershed
Shellfish Classification and Existing Level of Service

ATM DESIGN ENGINEERING CONSULTING

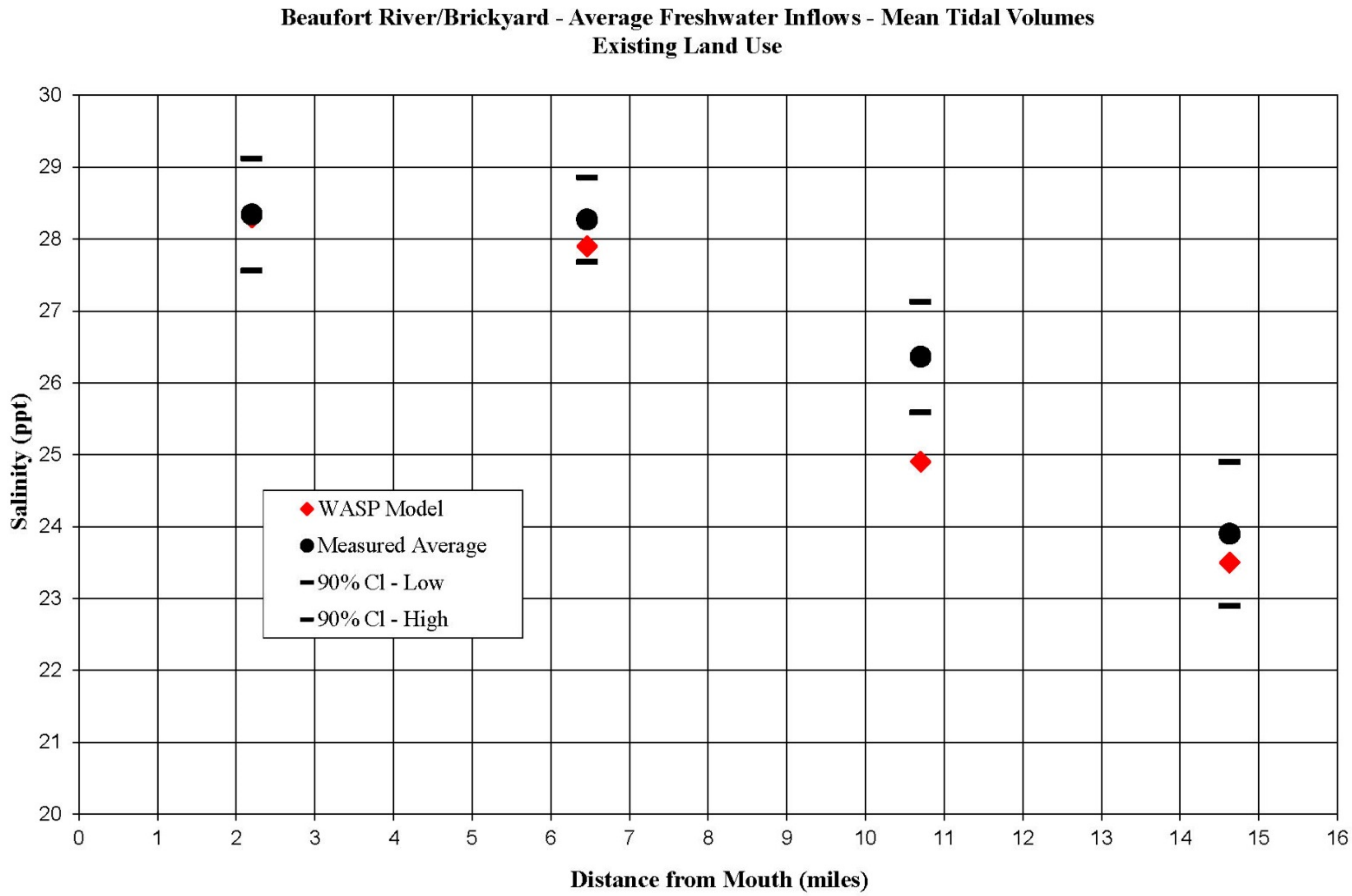


Figure 8-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Beaufort River - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

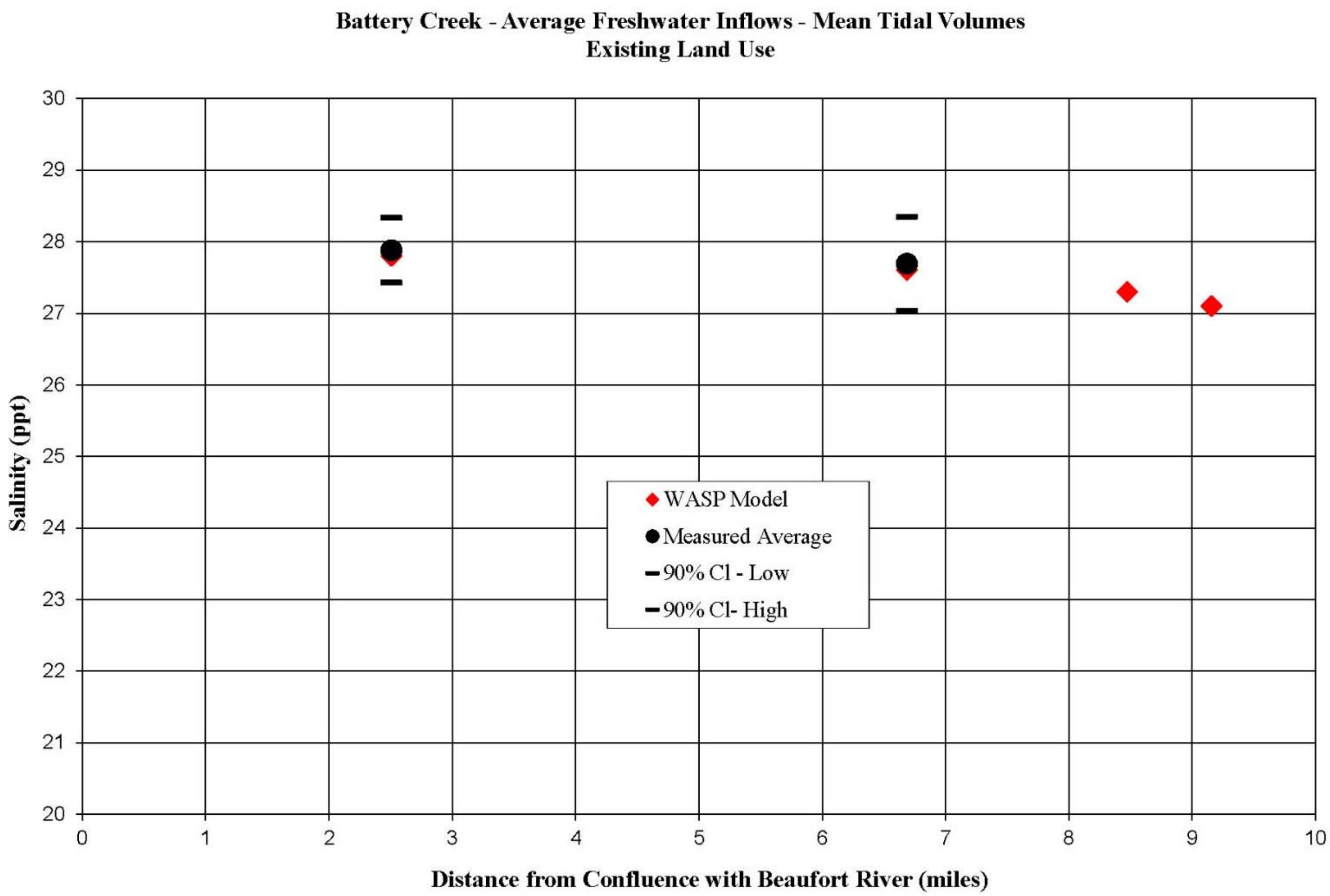


Figure 8-8. Comparison of WASP Model Results with Long-Term Monitoring Data in Battery Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

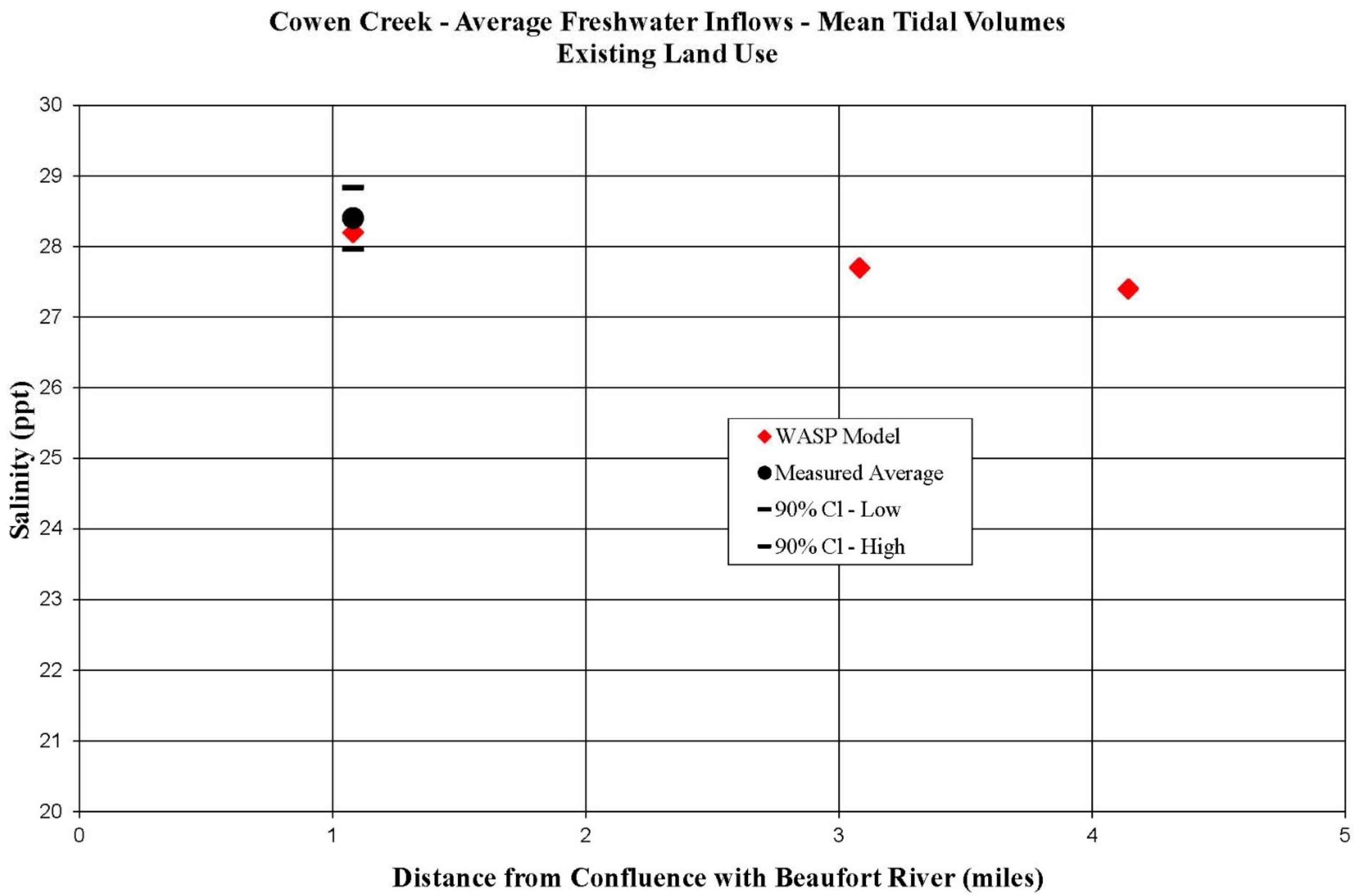


Figure 8-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Cowen Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

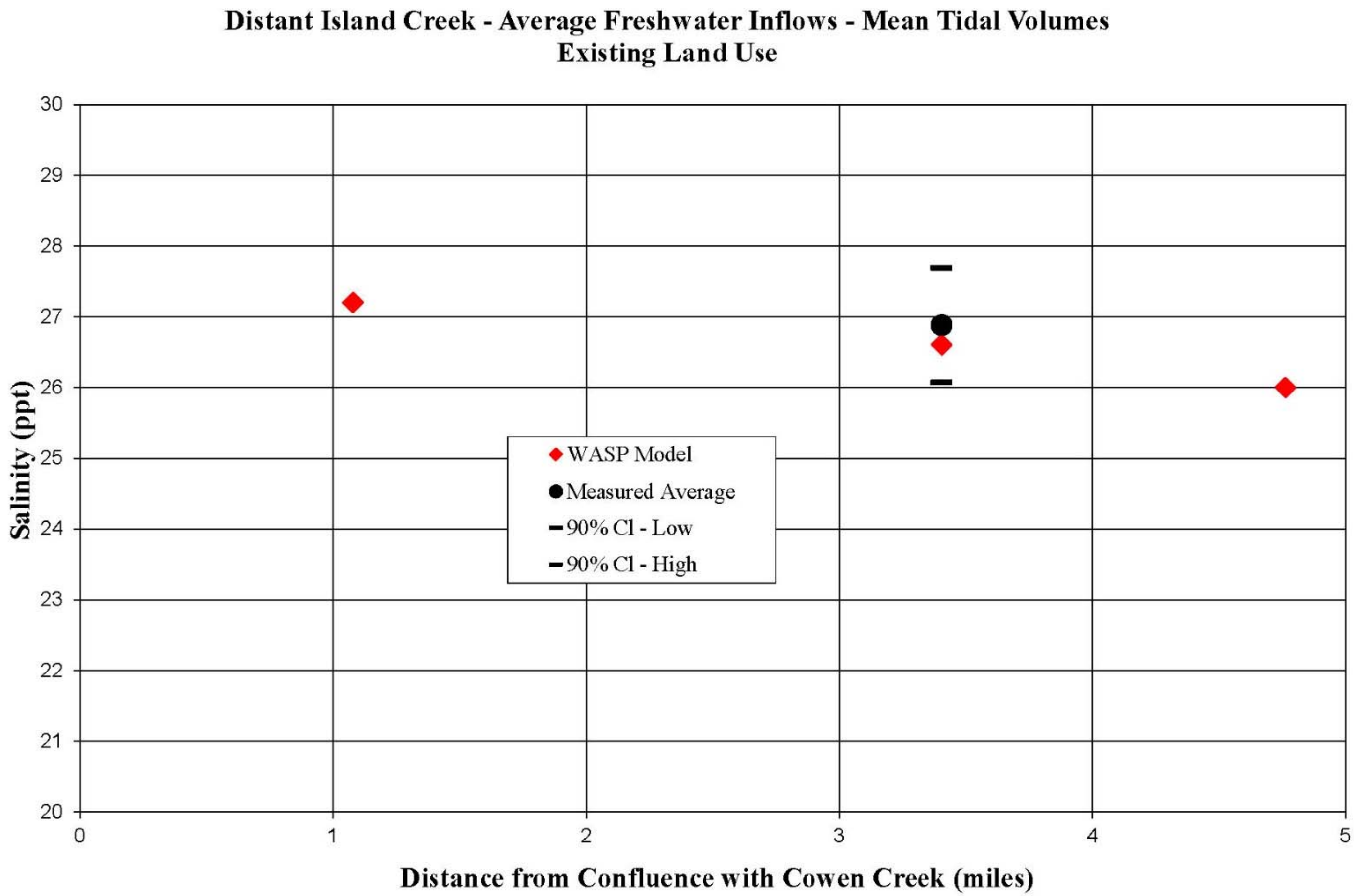


Figure 8-10. Comparison of WASP Model Results with Long-Term Monitoring Data in Distant Island Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

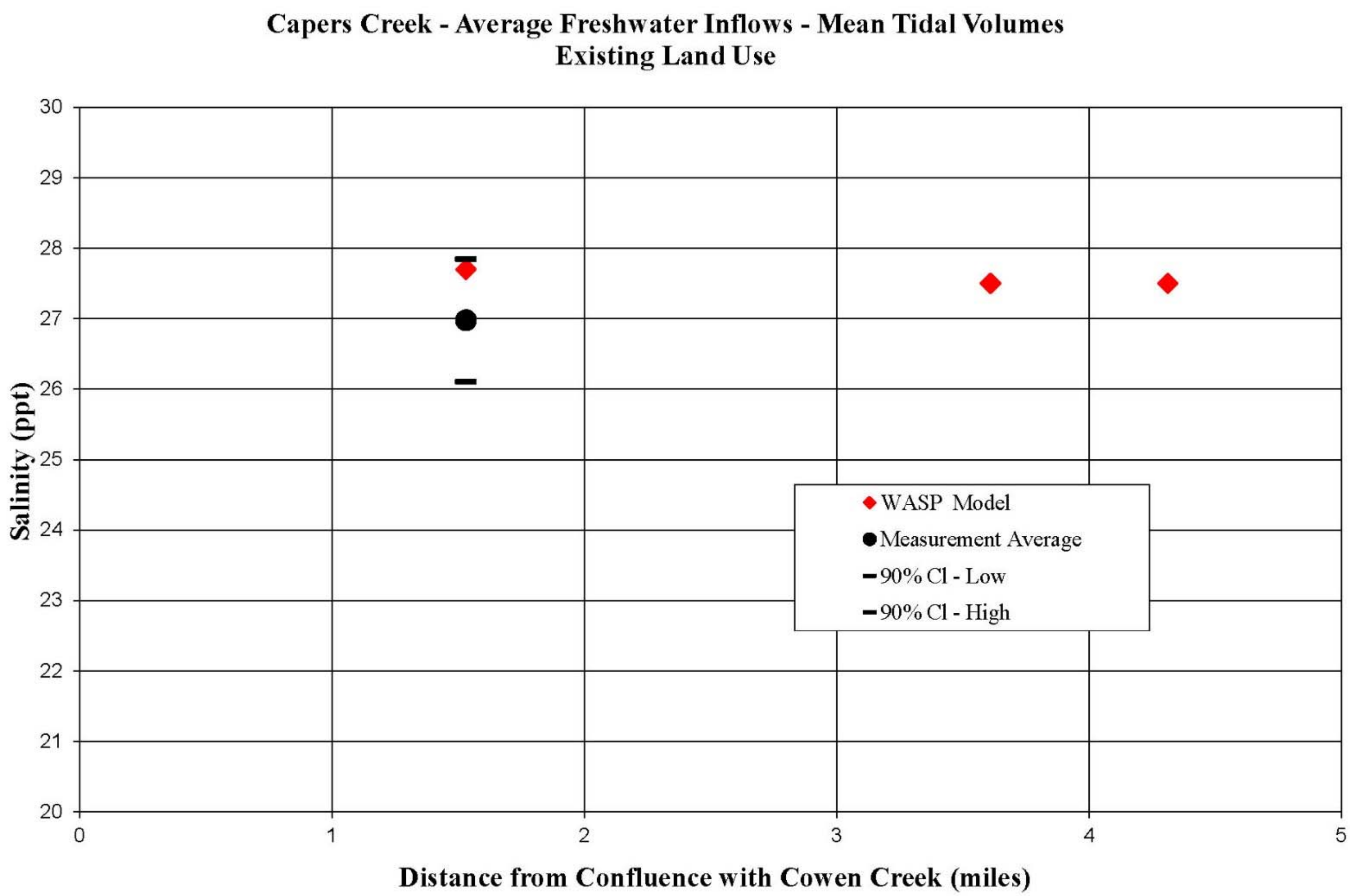


Figure 8-11. Comparison of WASP Model Results with Long-Term Monitoring Data in Capers Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

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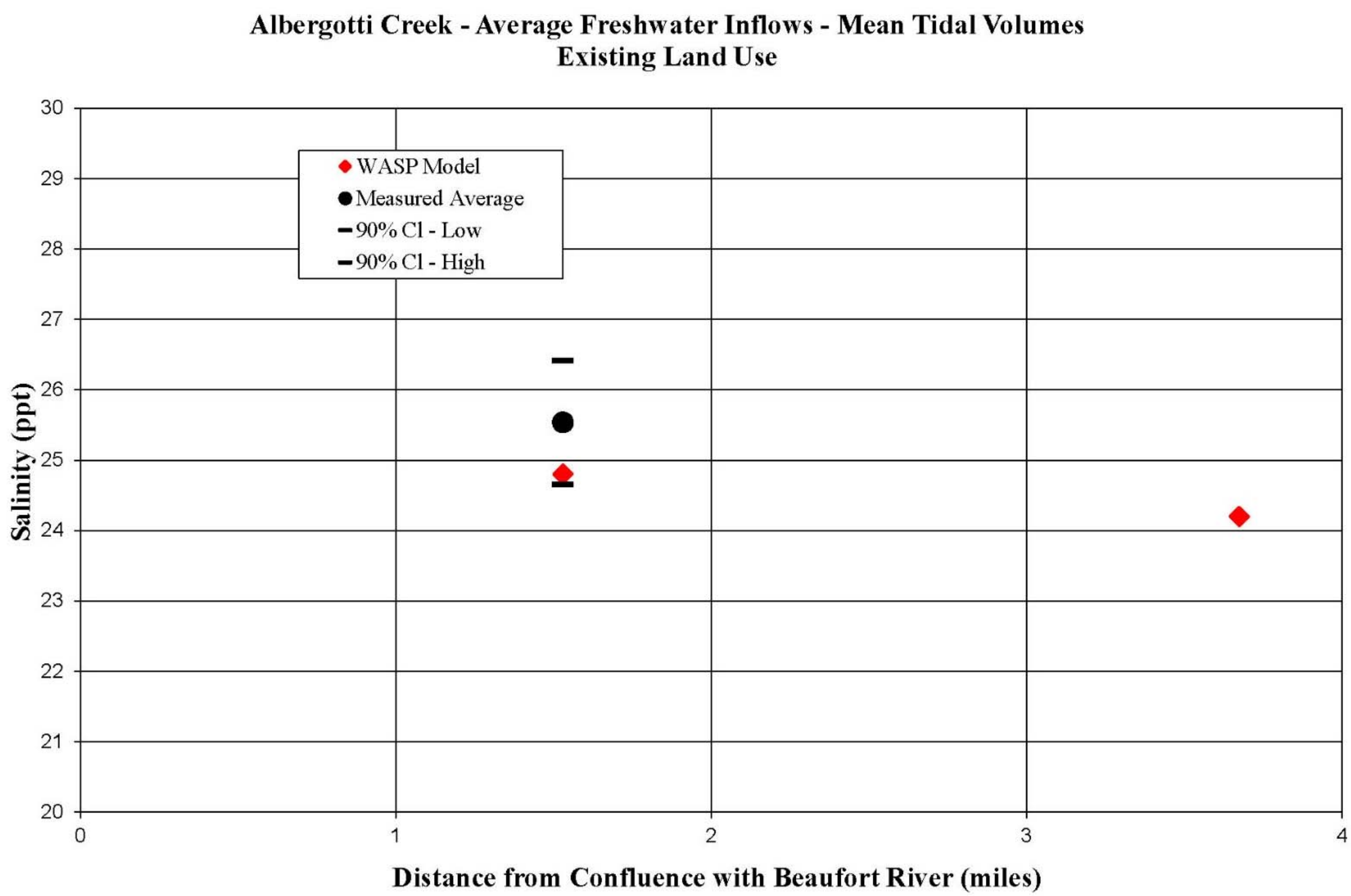


Figure 8-12. Comparison of WASP Model Results with Long-Term Monitoring Data in Albergotti Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

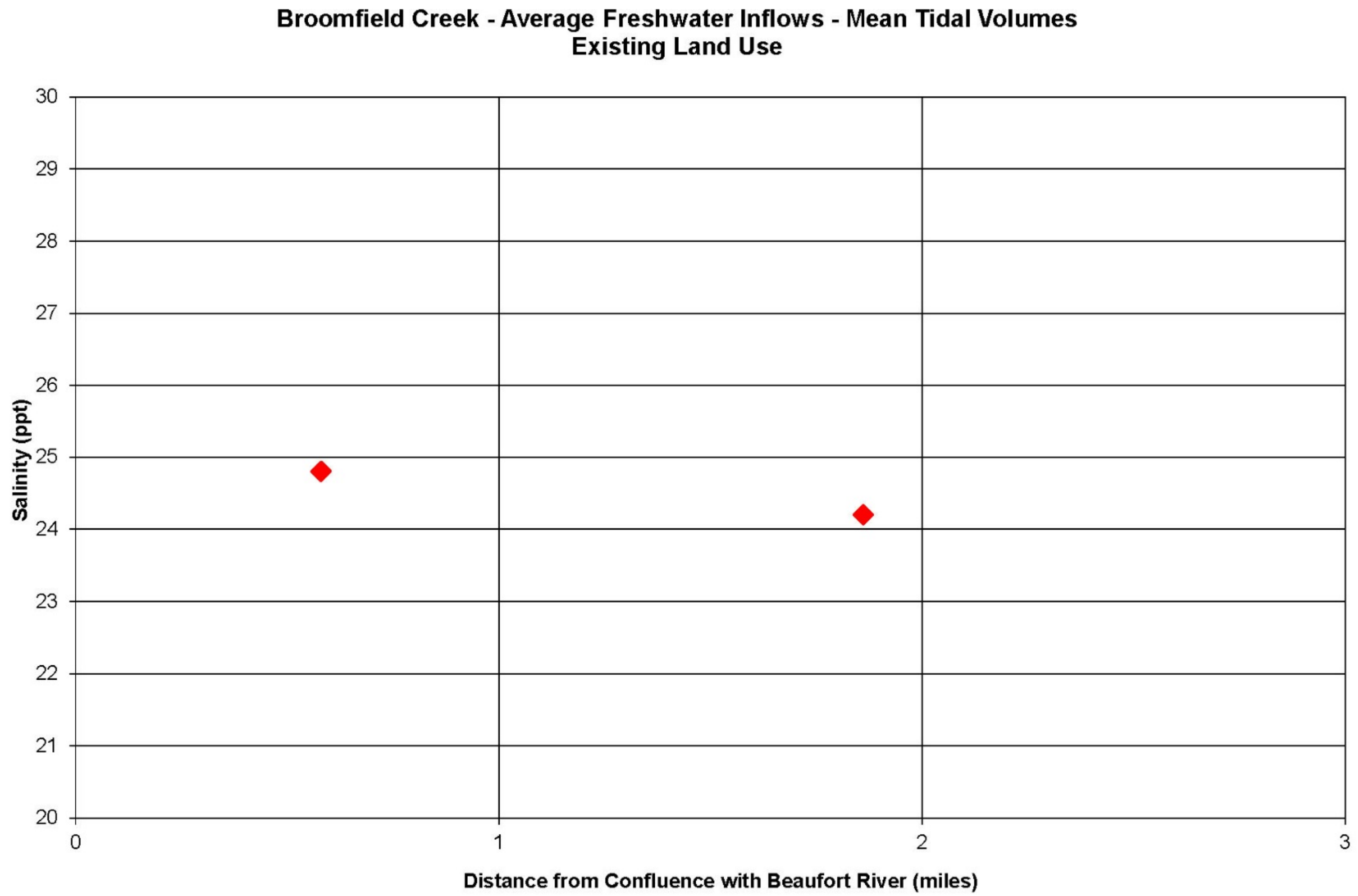


Figure 8-13. Comparison of WASP Model Results with Long-Term Monitoring Data in Broomfield Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

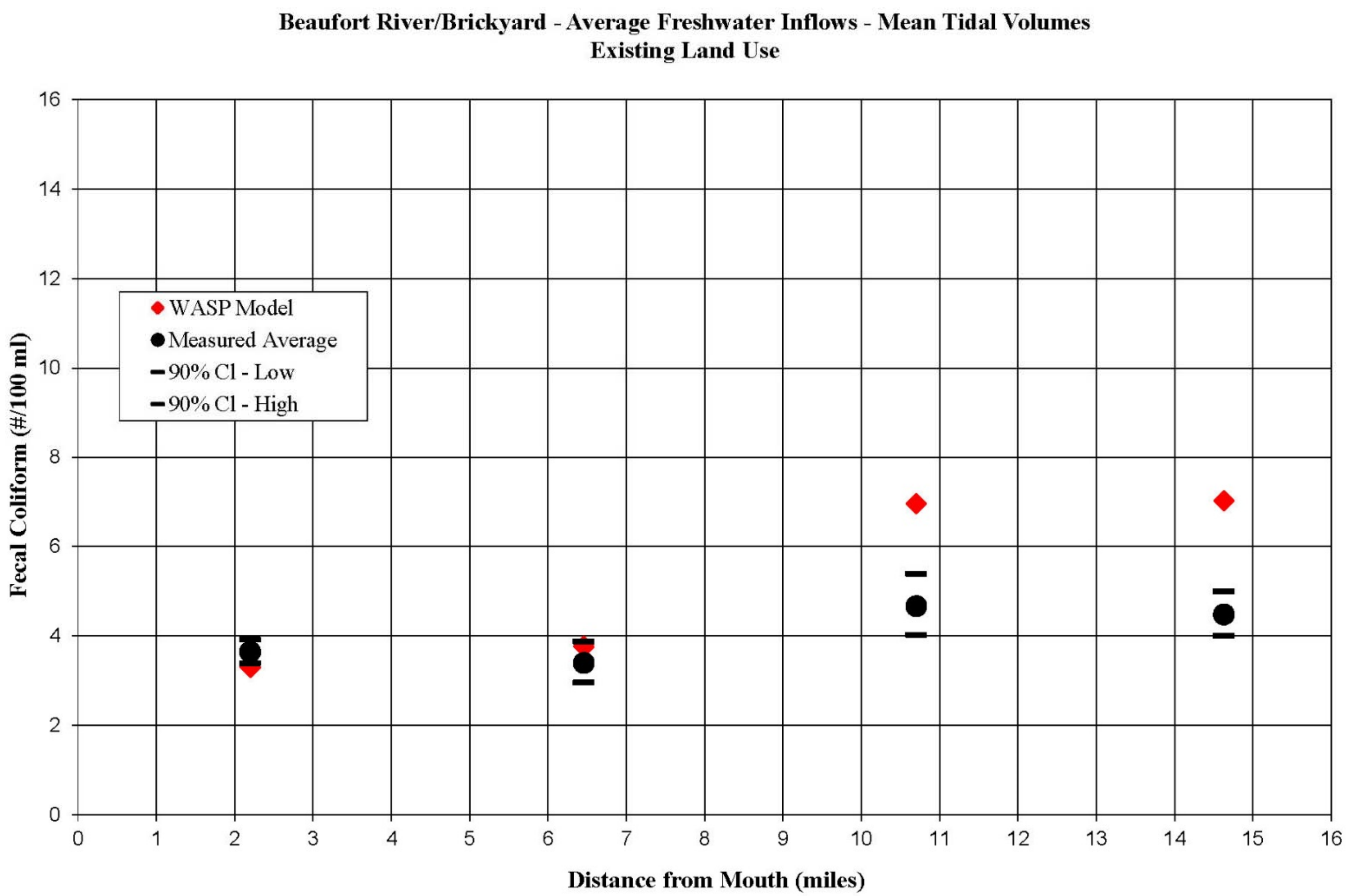


Figure 8-14. Comparison of WASP Model Results with Long-Term Monitoring Data in Beaufort River – Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

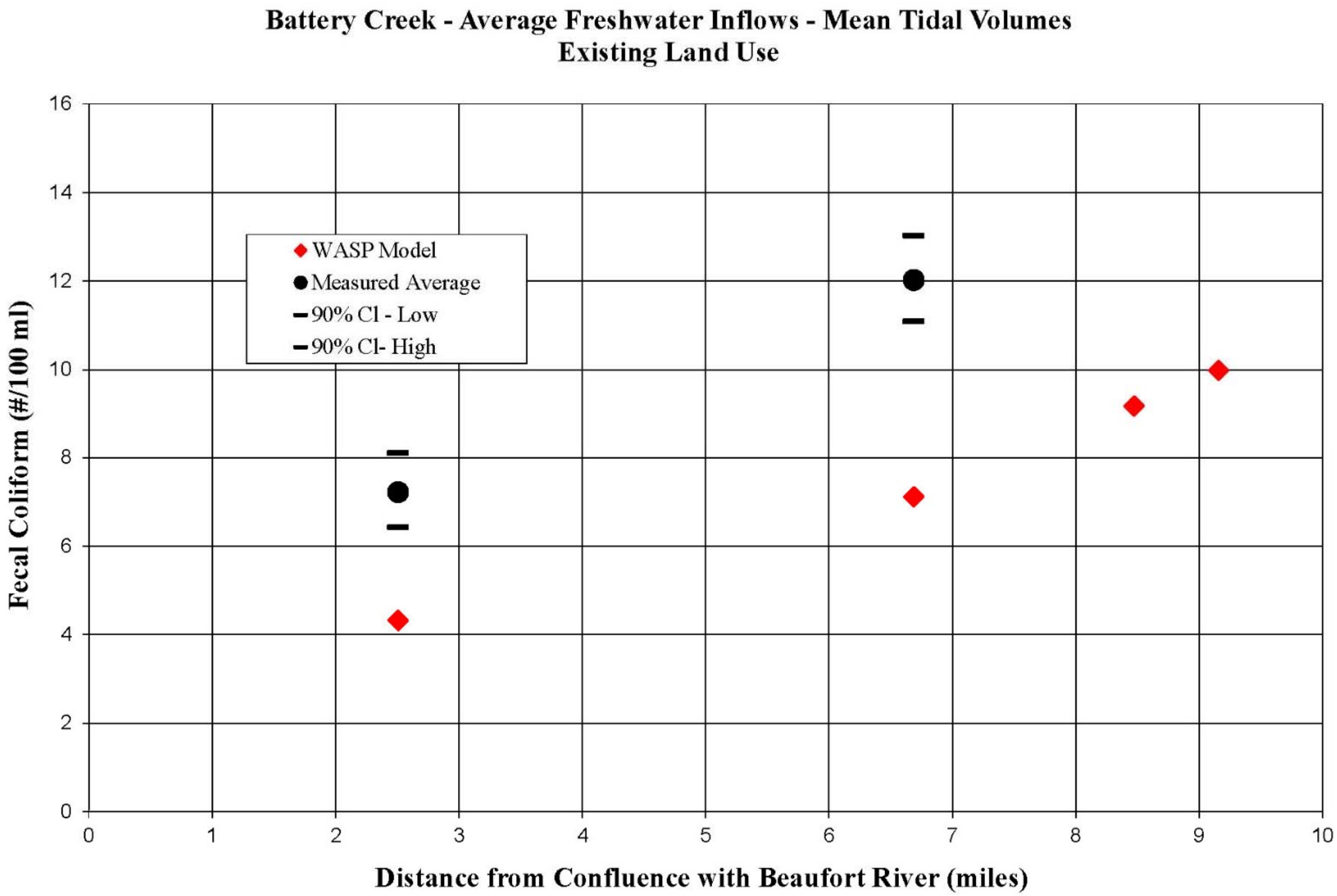


Figure 8-15. Comparison of WASP Model Results with Long-Term Monitoring Data in Battery Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

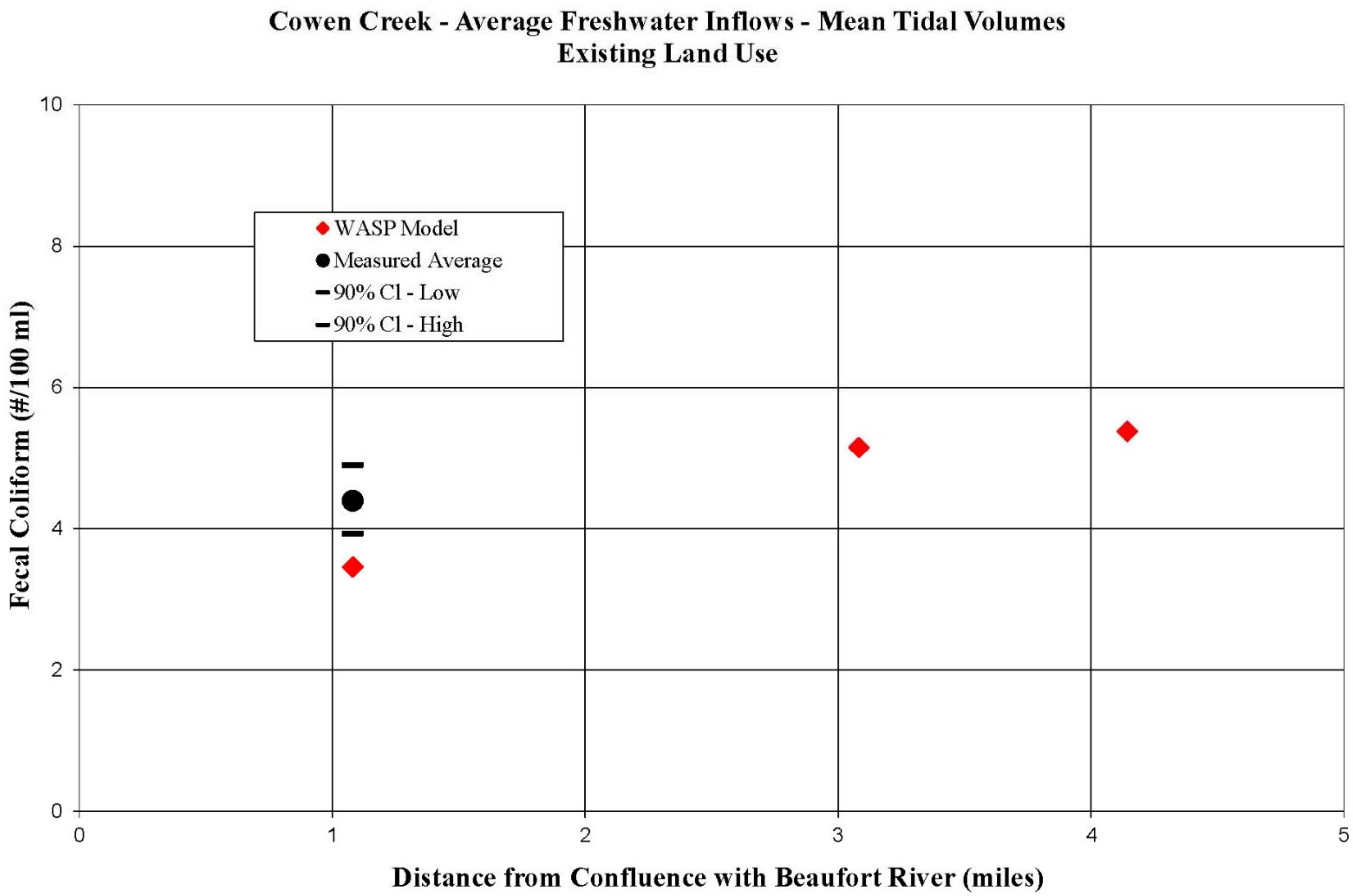


Figure 8-16. Comparison of WASP Model Results with Long-Term Monitoring Data in Cowen Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

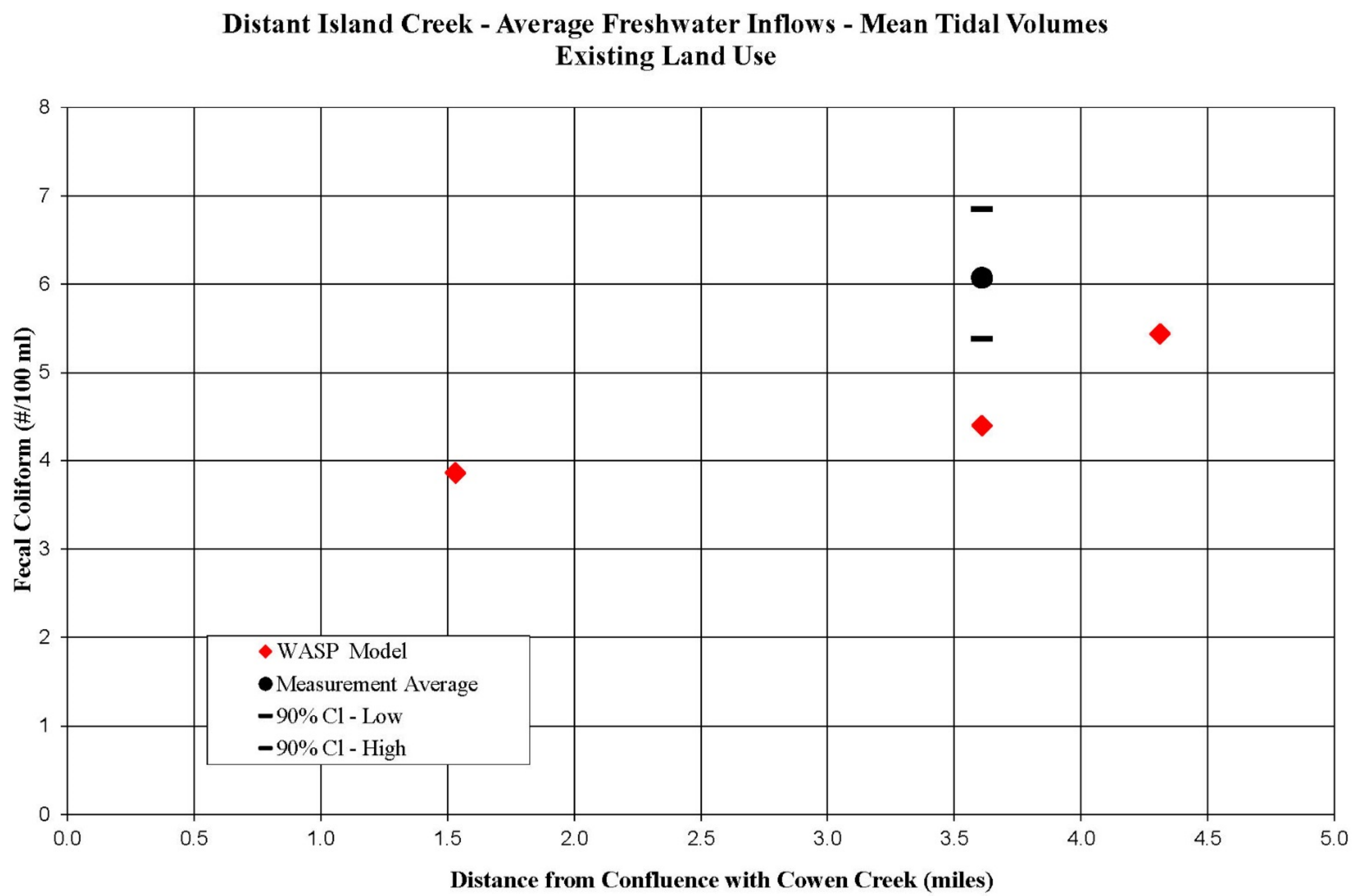


Figure 8-17. Comparison of WASP Model Results with Long-Term Monitoring Data in Distant Island Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

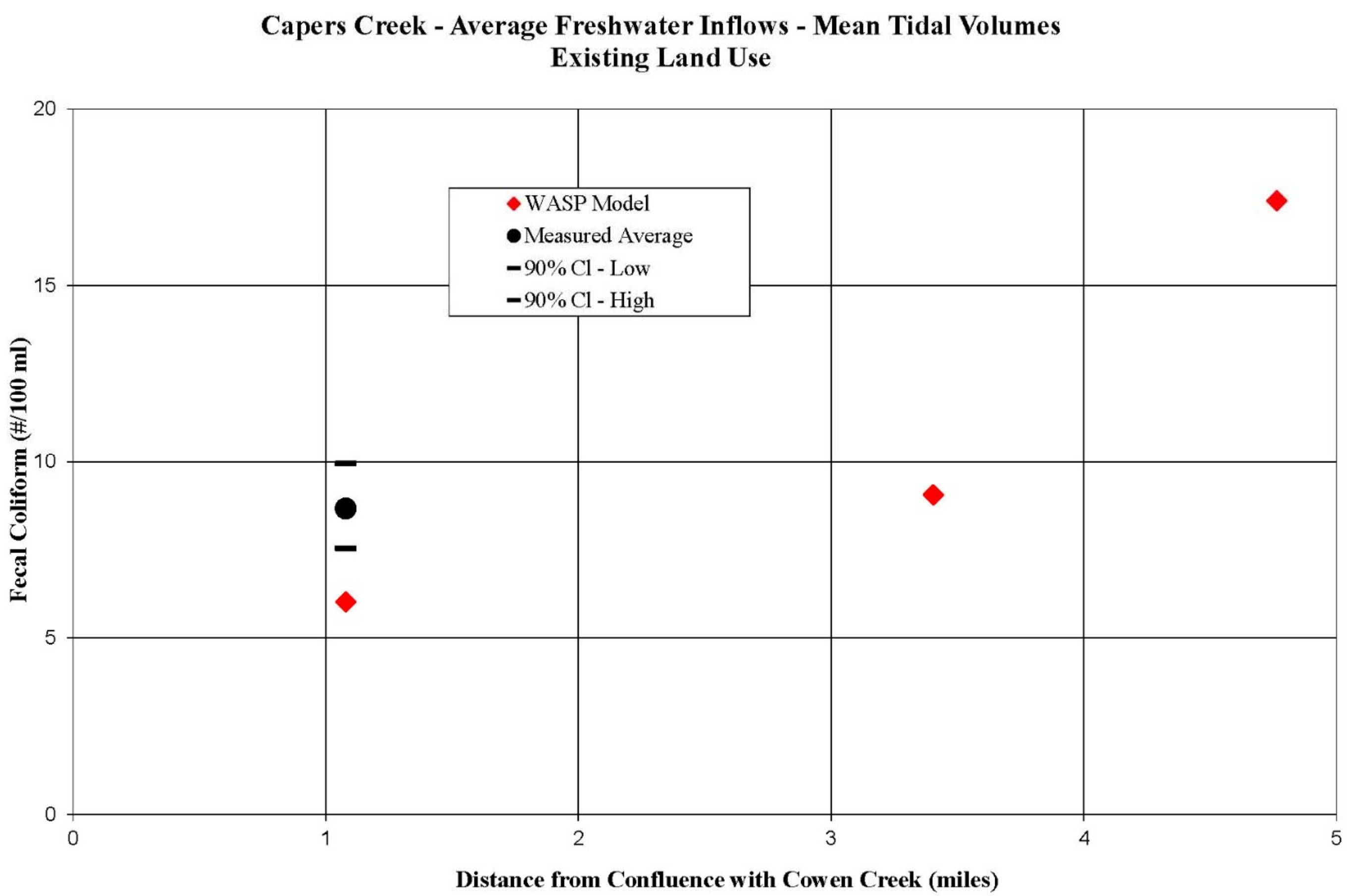


Figure 8-18. Comparison of WASP Model Results with Long-Term Monitoring Data in Capers Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

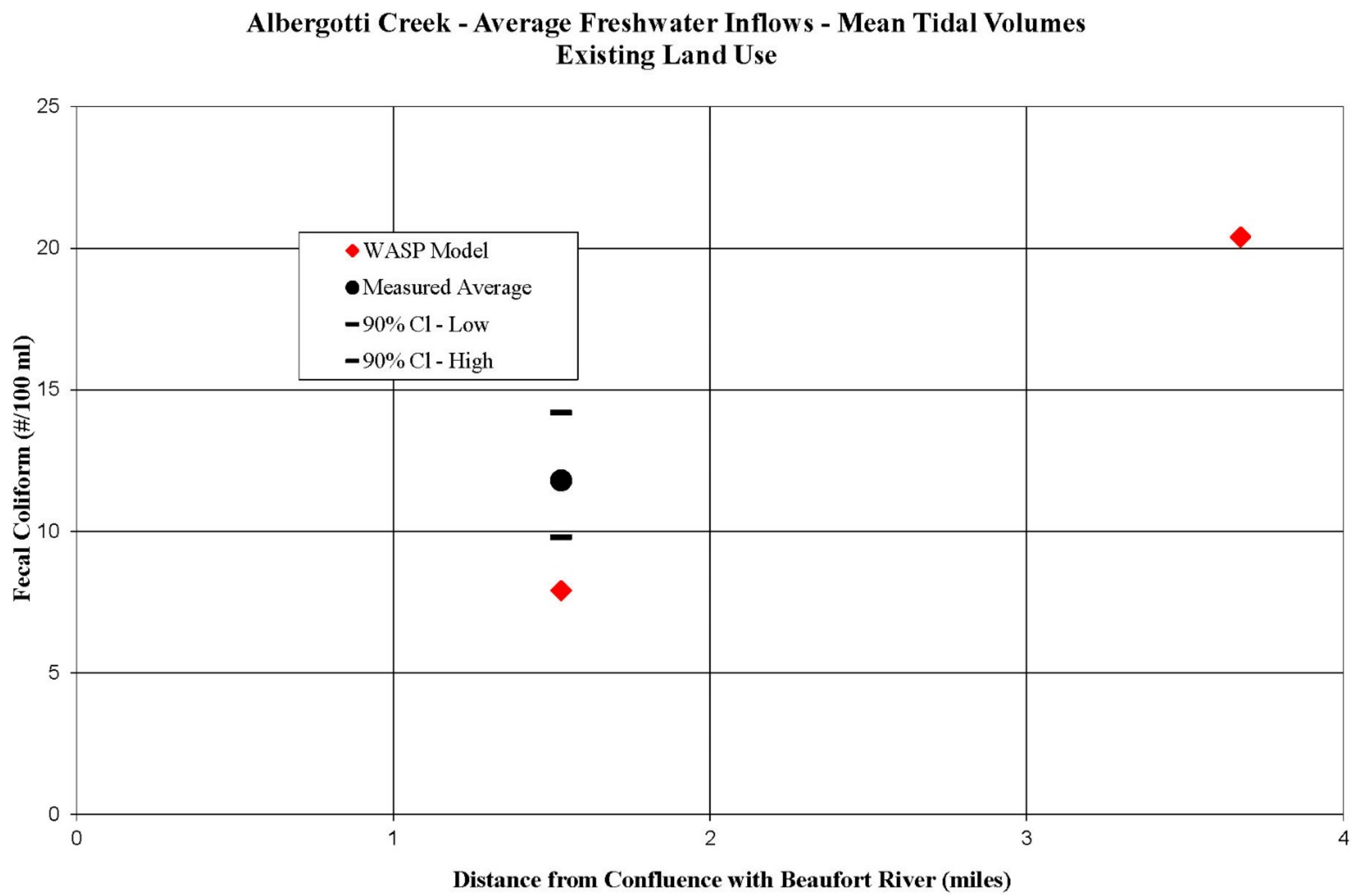


Figure 8-19. Comparison of WASP Model Results with Long-Term Monitoring Data in Albergotti Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

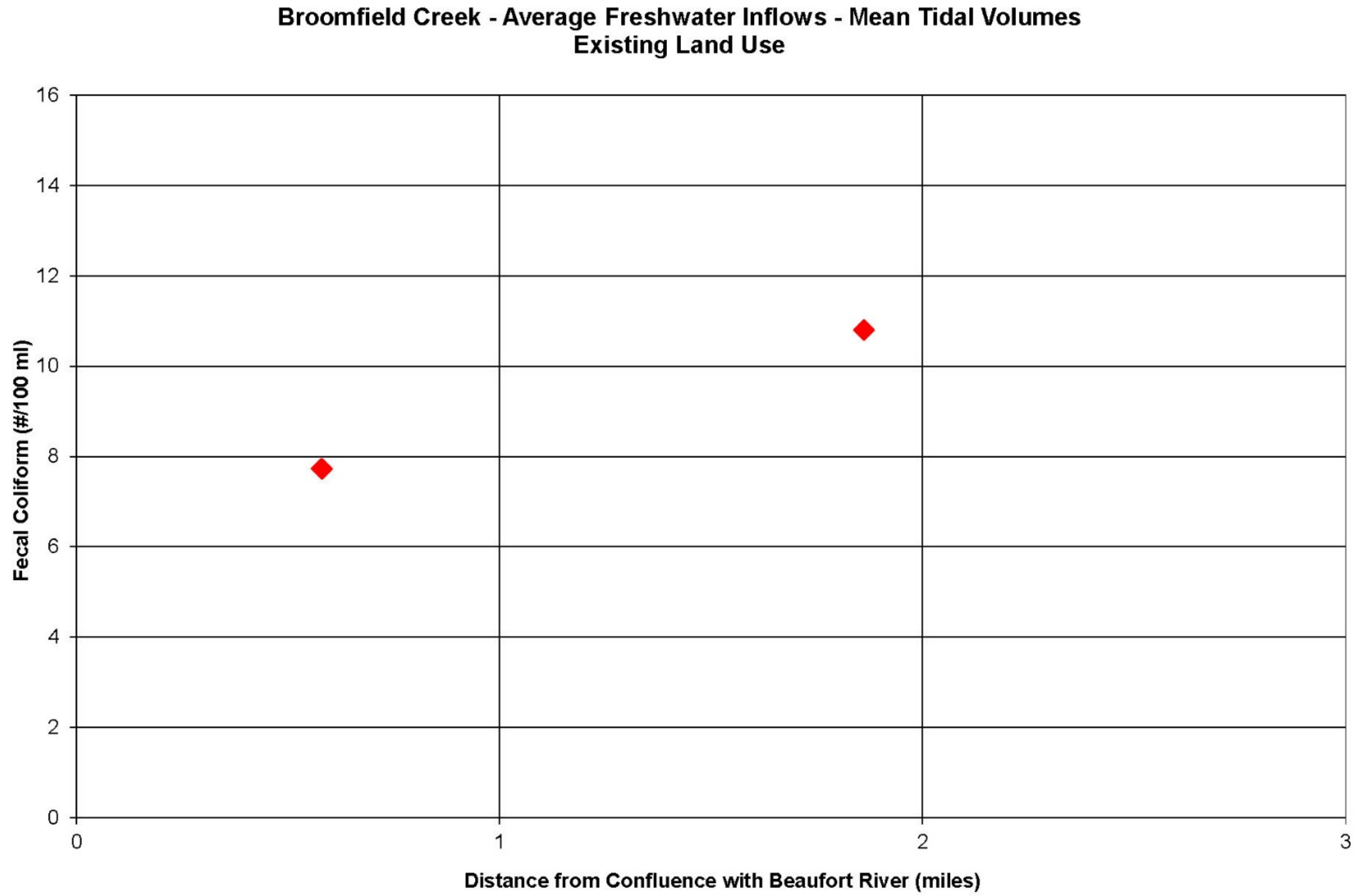
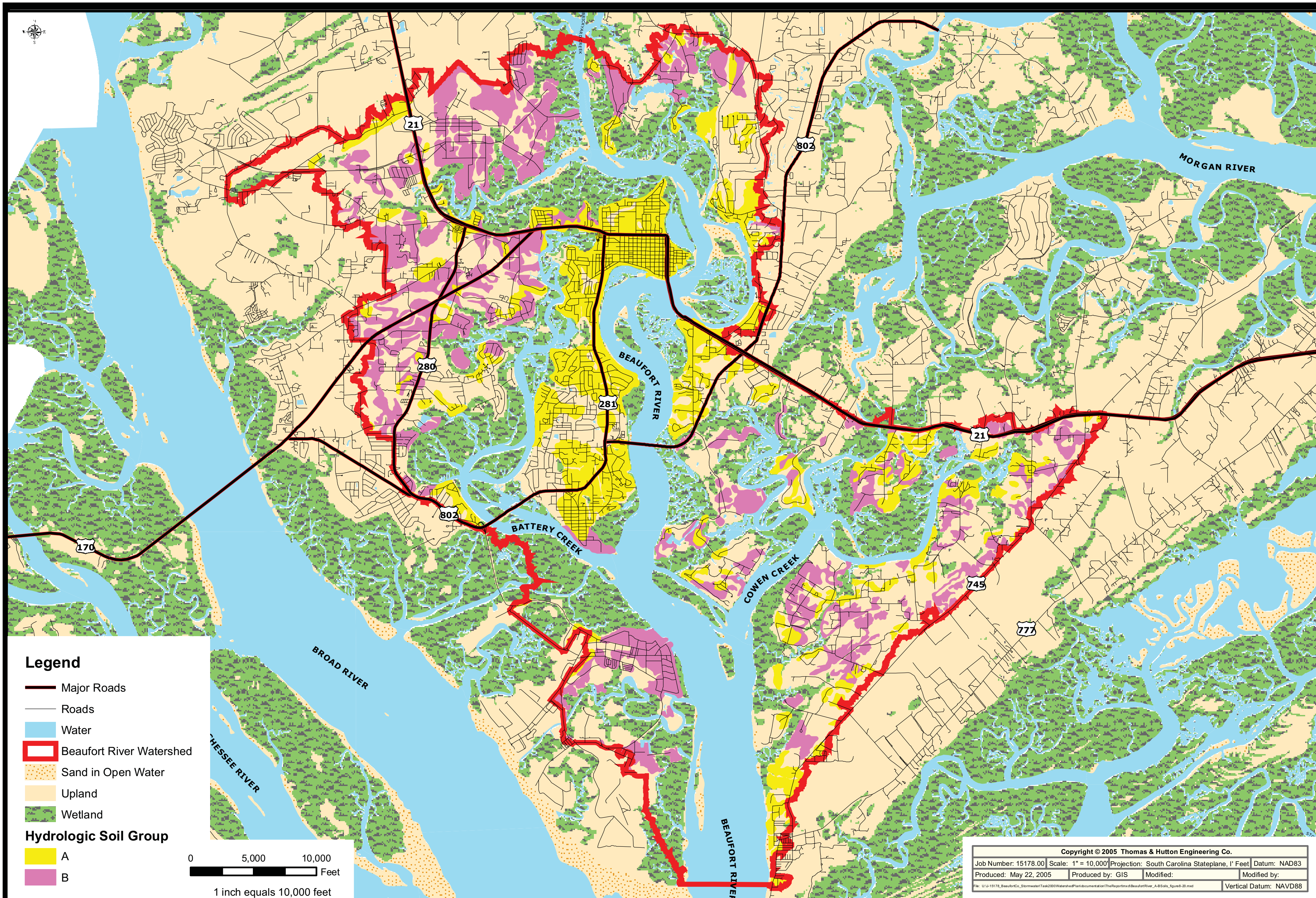


Figure 8-20. Comparison of WASP Model Results with Long-Term Monitoring Data in Broomfield Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Figure 8-21 is not applicable in the update.



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File: U:\J-15178_BeaufortCo_Stormwater\Task2000WatershedPlan\documentation\TheReport\mxd\BeaufortRiver_A-BSoils_figure8-20.mxd			Vertical Datum: NAVD88

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Beaufort River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 8-22

CDM Camp Dresser & McKee Inc.

Disclaimer
Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Section 9
Coosaw River Watershed Analysis

This section describes the physical features of the Coosaw River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

9.1 Overview

The Coosaw River watershed is located north of the Broad River (see Figure 9-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in the City of Beaufort, Sheldon Township, Port Royal Island and Lady's Island that is tributary to the Coosaw River. Major Coosaw River tributaries included in the analysis are Bull River/Wimbee Creek, Lucy Point Creek, South Wimbee Creek, McCalleys Creek, and Brickyard Creek.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several "hydrologic" basins. These are listed in Table 9-1 and presented in Figure 9-2. Table 9-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were updated to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into "water quality" basins, and the tidal receiving waters were subdivided into receiving water "segments". These are listed in Table 9-2 and presented in Figure 9-3. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were completed to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

9.2 Hydrologic and Hydraulic Analysis

CDM and T&H used ICPR, Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the Coosaw River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP.

9.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Coosaw River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 9-3 lists the hydrologic parameter values for the Coosaw River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the Coosaw River PSMS basins is presented in Table 9-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 9-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

9.2.2 Model Results

Tables in Appendix G list the summary of the results of the updated study including Updated Areas and CNs for the Coosaw River subbasins

For existing land use, aerial maps generated in the summer of 2016 and local information were used to estimate the percentage of existing urban development.

Appendix G also includes tables that list the peak water elevation values for model node locations along the Coosaw River PSMS.

Specific problem areas identified by the modeling are listed in Table 9-6 and presented in Figure 9-4. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs, and results showed that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded

Table 9-6 indicates that seventeen road crossings are being overtopped by the design storm events. Most of these areas are in the City of Beaufort and Sheldon Township.

Evaluation of solutions to prevent these problems is discussed in the next section of this report.

9.2.3 Management Strategy Alternatives

The problems areas listed in Table 9-6 were evaluated by reviewing the previous reports results and reviewing the culverts in the ICPR hydraulic model. In the original 2006 study, the ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 9-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were typically used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

9.3 Water Quality Analysis

ATM used the WMM and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the Coosaw River watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions.

9.3.1 Land Use and BMP Coverage

Table 9-8 presents the existing land use and future land use estimates for the Coosaw River water quality basins. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 27 percent of the Coosaw River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 73 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 4 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in Table 9-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 0.1 percent of the urban systems in the watershed are served by BMPs.

9.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing land use is presented in Table 9-10. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water & Sewer Authority. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 82 percent of the urban systems in the watershed are served by septic.

Based on available data, there are no significant wastewater discharges under existing conditions, and therefore none are expected in the future, as new development will primarily be served by septic tanks.

9.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Coosaw River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in Table 9-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

9.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Coosaw River watershed. The model actually includes Beaufort River, Coosaw River, Whale Branch West, and Morgan River watersheds because they are interconnected at several points. Only the Coosaw River will be discussed in this section. A schematic of the model is presented as Figure 9-5.

Existing conditions for bacteria concentrations in the Coosaw River are presented in Table 9-12. For each water quality basin river reach, the table lists the SCDHEC stations for which the 1990s bacteria data were analyzed, the concentrations calculated in the analysis, and the LOS associated with these concentrations (as discussed in Section 2.6.2. As shown in the table, SCDHEC data were available in eight of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations meet the water quality standards in five of the eight monitored segments, and so these segments have an “A” LOS.

For informational purposes, Figure 9-6 presents a map of the LOS based on the monitoring data analysis, compared to SCDHEC “shellfish classification” (based on the 2016 SCDHEC reports for shellfish areas 14, 15 and 16A). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the LOS, so there may not be a direct relationship between LOS and shellfish classification presented in the map. In general, however, segments with an “A” LOS are expected to have the lowest probability of receiving a “restricted” classification, and segments with a “D” LOS are expected to have the highest probability of receiving a “restricted” classification.

Physical characteristics assigned to the model reaches are presented in Table 9-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the "characteristic length" (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the SCDHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Table 9-14 presents the values used in the existing condition models.

Table 9-15 shows the net advective flows between segments. The hydrodynamic model (SWMM5) indicates that there is a net flow from the Beaufort River watershed (via Brickyard Creek) and the Whale Branch West watershed to the Coosaw River headwaters. The results also show a net flow from the Coosaw River south to the Morgan River via Lucy Point Creek and Parrot Creek.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. The calibrated loss-rate coefficients from the 2006 study were used in the updated simulations.

Figure 9-7 is a graph showing a comparison between measured and modeled salinity data along the Coosaw River main stem. The figure shows that the salinity data calculated by the model is very close to the average measured value, and is in all cases within the 90 percent confidence interval of the mean of the salinity data. Both the modeled and measured data show little variability in mean salinity concentrations between the segments.

Measured and modeled salinity data for Lucy Point Creek are shown in Figure 9-8. The modeled salinity data is toward the low end of the 90 percent confidence interval for the measured mean.

Figure 9-9 compares the modeled and measured salinity in the Bull River/Wimbee Creek system. In the one segment where data were measured, the comparison between modeled salinity and the calculated mean salinity value is very good.
The comparison between measured and modeled salinity for McCalleys Creek and Brickyard Creek North is presented in Figure 9-10. In both McCalleys Creek and Brickyard Creek, the modeled salinity tends to be lower than the measured value, though the modeled value is within 1 ppt of the measured mean.

Comparisons between measured and modeled bacteria concentrations are presented in Figures 9-11 through 9-14. The modeled bacteria concentrations are generally close to the measured geomean value, and the 90 percent confidence interval for the measured geomean. At Brickyard Creek and Coosaw River, the modeled values are actually greater than the high end of the 90 percent confidence interval for the measured geomean, but the modeled value (6.0/100 mL) is still substantially lower than the upper threshold for the “A” LOS (7/100 mL). Consequently, the overestimation is not considered critical. At Lucy Point Creek North, the model underestimates bacteria concentrations.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 9-16. The loss rates ranged from 0.5/day to 2.8/day. The lowest values are generally applied to the main stem Coosaw River, and the highest values are applied at the Lucy Point Creek and McCalleys Creek areas.

The graphs show very good agreement between the measured values and the model results for some of the reaches and poor agreement in others. In water quality modeling, most performance metrics indicate a model that predicts a value 45-60% of the observed value is considered fair or satisfactory (Moriasi et. al, 2007, Donigian, 2002). Where predictions are poor, this is likely due to how the hydrodynamics of the systems are being modeled. The approach that has been used to date is based on the net flow advection of the various reaches and is a quasi-steady-state approach. This is an acceptable approach in most cases and has utility in this case as it allows for the comparison of water quality management and their effectiveness. However, given the tide range that exists in the county’s receiving waters and the dynamic salinity regimes present, a detailed 3-dimensional hydrodynamic model, such as the Environmental Fluid Dynamics Code (EFDC), is required to adequately simulate the tidal fluctuations and salinity-density gradients that exist in the receiving waters. Development of a 3-D hydrodynamic model would be a significant effort but would provide the proper hydrodynamic foundation for improved water quality predictions.

Based on water quality sampling data and model results, the following conclusions are:

- Problem basins include Lucy Point Creek North 2, Bull River/Wimbee Creek 3 and 4, South Wimbee Creek 1 and 2
- 1 new regional water quality BMPs is proposed in Lucy Point Creek North 2 basin.

Discussion of water quality related recommendations for monitoring and regional BMPs in the Coosaw River watershed are presented as part of the overall recommended monitoring and CIP program for Beaufort County contained in the Appendix of this report.

9.3.5 Management Strategy Alternatives

In analyzing the watershed, one feasible regional detention site was identified. The area tributary to the Lucy Point Creek Regional BMP site includes approximately 105 acres of rural and single-family development built prior to stormwater regulations. There are limited stormwater best management practices, such as detention facilities, in the area. The project would be to construct modifications to the existing regional wet detention pond including permanent pool expansion, littoral shelf creation and control structure modifications. The project will provide enhanced stormwater runoff water quality treatment and volume reduction. Due to the presence of some wetlands in the area, project design would involve delineation and avoidance of the wetlands.

A new WMM scenario was developed for the Lucy Point Creek Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Lucy Point Creek 2 North water quality basin of approximately 11%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Coosaw River:

Parameter	lb/yr removed
Total Nitrogen	120
Total Phosphorus	30
TSS	13,256

The results of the water quality analysis suggest that the limited amount of future development in the watershed, combined with the effectiveness of required BMPs in reducing bacteria loads from new development, will generally maintain the existing LOS in all watershed reaches.

For informational purposes, the areas with “A” and “B” type soils are presented in Figure 9-16. In general, these soils are more suitable for infiltration BMPs than areas with “C” and “D” type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

9.4 Planning Level Cost Estimates for Management Alternatives

Table 9-18 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Coosaw River watershed. As shown in the table, the projects are estimated to have a total cost of $9.829 million in January 2018 dollars. Details of the cost estimate for each project are shown in Appendix G.

It should be noted that most of the costs in this watershed are associated with the Air Station basin, including substantial required upgrades on the air station itself. Further investigation and discussion with appropriate agencies should be conducted to resolve whether flooding is expected to occur at the air station.

One regional CIP project was identified in the Coosaw River watershed. The project is estimated to have a total cost of $0.438 million and is detailed in the CIP in Appendix O.

TABLE 9-1
HYDROLOGIC BASINS
COOSAW RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Air Station	2,939	9	327
Branford Creek East	6,047	14	432
Briars Creek South	514	1	514
Briars Creek West	1,201	3	400
Brickyard Creek	865	3	288
Browns Island	197	1	197
Coosaw River	338	1	338
Dale	1,546	5	309
Halfmoon Island	78	1	78
Laurel Hill	238	1	238
Lobeco	677	3	226
McCalleys Creek	355	1	355
True Blue Creek North	1,151	4	288
True Blue Creek South	571	2	286
TOTAL	16,718	49	341

TABLE 9-2
WATER QUALITY BASINS
COOSAW RIVER WATERSHED

Basin Name	Tributary Area (acres)
Coosaw River 1	7,455
Coosaw River 2	9,435
Coosaw River 3	3,638
Coosaw River 4	528
Lucy Point Creek North 1	926
Lucy Point Creek North 2	438
Bull River/Wimbee Creek 1	3,118
Bull River/Wimbee Creek 2	6,470
Bull River/Wimbee Creek 3	8,498
Bull River/Wimbee Creek 4	7,836
Williman Creek 1	2,770
Williman Creek 2	1,829
Williman Creek 3	591
Williman Creek Trib.1	1,109
South Wimbee Creek 1	3,401
South Wimbee Creek 2	741
McCalleys Creek 1	6,820
McCalleys Creek 2	1,393
Brickyard Creek North	2,378
TOTAL	69,375

TABLE 9-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
COOSAW RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Air Station Basin					
AS_M1	323	88	127	90	127
AS_M2	308	79	78	91	78
AS_M3	191	87	50	92	50
AS_M4	564	82	102	86	94
AS_M5	172	84	53	86	51
AS_M6	378	80	217	83	169
AS_M7	282	82	159	86	137
AS_T1	392	79	108	92	93
AS_T2	330	70	160	78	127
Branford Creek East Basin					
BDCE_M1	1065	90	184	87	184
BDCE_M2	525	91	135	87	134
BDCE_M3	448	88	121	85	119
BDCE_M4	498	78	135	80	133
BDCE_M5	66	86	55	83	55
BDCE_M6	294	89	175	85	163
BDCE_T1	707	88	160	88	151
BDCE_T1a	533	92	148	86	147
BDCE_T1b	220	83	95	80	95
BDCE_T1c	369	84	180	82	179
BDCE_T1d	479	82	159	81	159
BDCE_T2	168	85	95	85	88
BDCE_T3	431	86	155	85	148
BDCE_T3a	246	86	91	86	86
Briars Creek South Basin					
BCS_M1	514	79	187	83	186
Briars Creek West Basin					
BCW_M1	206	77	137	76	137
BCW_M2	523	79	237	81	220
BCW_T1	471	80	197	81	195
Brickyard Creek Basin					
BC_M1	287	70	158	71	149
BC_M2	291	77	128	81	117
Browns Island Basin					
BI_M1	197	79	104	80	98
Dale Basin					
CWR_M1	338	64	240	65	235
Dale Basin					
DE_M1	208	75	114	78	111
DE_M2	329	81	210	83	201
DE_M3	429	81	183	81	182
DE_M4	269	86	186	88	183
DE_T1	311	84	130	84	126
Halfmoon Island Basin					
HM_M1	78	81	51	87	49
Laurel Hill Basin					
LH_M1	238	69	140	77	126
Lobeco Basin					
LO_M1	259	87	97	88	97
LO_M2	189	79	116	82	104
LO_M3	229	82	131	82	131
McCalleys Creek Basin					
MC_M1	355	73	191	77	166
True Blue Creek North Basin					
TBCN_M1	286	80	104	85	102
TBCN_M2	165	78	85	80	85
TBCN_M3	345	83	134	84	125
TBCN_M4	355	75	203	75	201
True Blue Creek South Basin					
TBCS_M1	263	78	102	80	102
TBCS_M2	308	76	173	79	173
Average	342	81	139	83	133

TABLE 9-4
HYDRAULIC DATA SUMMARY
COOSAW RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Air Station	28	29,421	10	20	1	3	10	0
Branford Creek East	47	50,887	8	8	0	7	6	1
Briars Creek South	3	3,702	0	0	0	0	0	0
Briars Creek West	10	12,655	0	0	0	0	0	0
Brickyard Creek	4	3,662	1	2	0	1	1	0
Browns Island	1	812	2	2	0	1	0	0
Coosaw River	2	660	6	12	0	2	6	0
Dale	15	16,038	2	2	0	2	2	0
Halfmoon Island	1	910	1	3	0	1	0	0
Laurel Hill	2	390	2	3	0	2	3	0
Lobeco	6	4,804	3	4	0	2	2	0
McCalleys Creek	3	2,629	1	2	0	1	1	0
True Blue Creek North	12	11,567	1	1	0	2	1	0
True Blue Creek South	5	5,477	1	1	0	1	1	0
TOTAL	139	143,614	38	60	1	25	33	1

TABLE 9-5
CULVERT DATA FOR HYDROLOGIC BASINS
COOSAW RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Air Station Basin						
R.C. West Road N	AS_M-0A	72"x72"	215	-0.2	8.8	25
	0B	72"x72"	215	-0.5		
Funa Futi Road East	AS_M-7A	66"x66"	140	7.5	18.0	25
	7B	66"x66"	140	7.8		
	7C	66"x66"	140	7.6		
Funa Futi Road West	AS_M-10A	60"x60"	60	13.4	25.1	25
	10B	60"x60"	60	13.3		
T-31	AS_M-12A	60"x60"	1200	16.9	30.0	25
	12B	60"x60"	1200	17.0		
R.C. West Road N	AS_M-14A	48"x48"	120	20.8	27.7	25
	14B	48"x48"	120	21.3		
	14C	60"x38"	61	22.2		
	14D	60"x38"	61	22.2		
Trask Parkway (US Hwy 17)	AS_M-28A	30"x30"	215	20.3	30.0	100
	28B	66"x60"	75	21.8		
Trask Parkway (US Hwy 17)	AS_T1-3	18"x18"	200	27.4	32.3	100
Branford Creek East Basin						
Dike Road	BDCE_M-0	80"x32"	20	-4.3	4.7	25
Charleston Highway (US Hwy 17)	BDCE_M-1	96"x96"	50	-2.4	8.5	100
Big Estate Road	BDCE_M-15	24"x24"	40	2.7	9.3	25
Big Estate Road	BDCE_M-17	24"x24"	40	4.8	11.0	25
Africian Baptist Church Road	BDCE_M-20	36"x36"	60	4.1	9.9	25
Big Estate Road	BDCE_T1-10	72"x72"	45	0.6	7.2	25
Charleston Highway (US Hwy 17)	BDCE_T3-4	60"x 48"	80	3.1	10.5	100
Jacob White Road	BDCE_T3-8	24"x 24"	50	4.5	9.2	25
Charleston Highway (US Hwy 17)	BDCE_T4-3	30"x30"	45	2.6	10.0	100
Brickyard Creek Basin						
Walling Grove Road	BC_M-0A	46"x30"	46	1.5	6.6	25
	0B	46"x30"	46	1.5		

TABLE 9-5
CULVERT DATA FOR HYDROLOGIC BASINS
COOSAW RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Browns Island Basin						
Browns Island Road	BI_M-1A	36"x36"	60	3.7	11.5	25
Keans Neck Road	BI_M-3A	24"x24"	60	4.0	11.4	25
Coosaw River Basin						
Old Plantation Drive	CWR_M-4A	24"x24"	480	6.4	11.5	25
	4B	24"x24"	480	6.4		
	4C	24"x24"	480	6.4		
	4D	24"x24"	480	6.4		
Walling Grove Road	CWR_M-7A	15"x15"	110	7.8	11.5	25
	7B	15"x15"	110	7.9		
	7C	15"x15"	110	8.0		
Dale Basin						
Wimbee Landing Road	DE_M-1	48"x48"	60	0.1	8.0	25
Wimbee Landing Road	DE_T1-2	42"x42"	40	1.1	7.4	25
Laurel Hill Basin						
Gadwell Drive	LH_M-3	15"x15"	30	5.8	8.5	25
Lobeco Basin						
Unknown Road	LO_M-4A	30"x30"	60	4.1	8.7	25
	4B	15"x15"	60	4.6		
Fertile Road	LO_M-6	42"x42"	100	5.1	14.5	25
Keans Neck Road	LO_M-9	30"x30"	40	8.0	14.0	25
McCalleys Creek Basin						
Trask Parkway (US Hwy 17)	MC_M-1A	30"x30"	150	2.7	10.2	100
	1B	30"x30"	150	1.5		
True Blue Creek North Basin						
Stroban Road	TBCN_M-13	36"x36"	50	7.8	14.7	25
True Blue Creek South Basin						
Kinlock Road	TBCS_M-1	30"x30"	40	0.2	8.0	25

TABLE 9-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COOSAW RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Air Station Basin					
R.C. West Road N	AS_M-4	8.8	8.8	10	9.0
				25	9.1
				100	9.1
(T-31)	AS_M-112	30.0	30.0	100	30.0
R.C. West Road N	AS_M-128	27.7	27.7	2	25.2
				10	27.7
				25	28.1
				100	30.0
Trask Parkway (US Hwy 17)	AS_M-130	30.0	30.0	10	30.2
				25	30.3
				100	30.4
No Road Crossing	AS_T1-2	N/A	28.0	2	28.5
				10	28.5
				25	28.5
				100	30.0
Trask Parkway (US Hwy 17)	AS_T1-18	32.3	32.3	100	32.4
Branford Creek East Basin					
Charleston Highway (US Hwy 17)	BDCE_M-92	8.5	6.7	100	7.3
Big Estate Road	BDCE_M-240	9.3	9.1	100	9.5
Big Estate Road	BDCE_M-241	11.0	10.1	100	10.5
African Baptist Church Road	BDCE_M-264	9.9	9.9	25	10.0
				100	10.5
Big Estate Road	BDCE_T1-123	7.2	7.2	10	7.6
				25	7.8
				100	7.9
Brickyard Creek Basin					
Walling Grove Road	BC_M-11	6.6	6.6	2	6.5
				10	6.7
				25	6.7
				100	6.7
Coosaw River Basin					
No Road Crossing	CWR_M-1	N/A	5.6	2	7.4
				10	7.6
				25	7.6
				100	7.7
Old Plantation Drive	CWR_M-17	11.5	11.5	10	12.0
				25	12.0
				100	12.3

TABLE 9-6 (Updated 2017)
OVERTOPPING PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COOSAW RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)
Walling Grove Road	CWR_M-20	11.5	11.5	2	11.5
				10	12.1
				25	12.4
				100	12.2
Dale Basin					
Wimbee Landing Road	DE_M-8	8.0	8.0	25	8.0
				100	8.3
No Road Crossing	DE_M-134	N/A	18.2	25	18.1
				100	18.7
Wimbee Landing Road	DE_T1-11	7.4	7.4	25	7.5
				100	8.3
Laurel Hill Basin					
Gadwell Dr.	LH_M-13	8.5	8.5	2	8.8
				10	8.8
				25	8.8
				100	8.8
Lobeco Basin					
Keans Neck Road	LO_M-60	14.0	14.0	2	14.1
				10	14.1
				25	14.1
				100	14.2
McCalleys Creek Basin					
No Overtopping Identified					
True Blue Creek North Basin					
No Road Crossing	TBCN_M-9	N/A	4.7	10	5.0
				25	5.1
				100	5.1
Stroban Road	TBCN_M-112	14.7	14.7	10	14.8
				25	14.8
				100	14.8
True Blue Creek South Basin					
Kinlock Road	TBCS_M-3	8.0	8.0	2	8.1
				10	8.3
				25	8.3
				100	8.3

TABLE 9-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
COOSAW RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Air Station Basin			
R.C. West Road N	AS_M-0A 0B	72"x72" 72"x72"	Replace culverts with two 12 ft by 6 ft box culverts
*Funa Futi Road East	AS_M-7A 7B 7C	66"x66" 66"x66" 66"x66"	Add two 48" pipes
*'Funa Futi Road West	AS_M-10A 10B	60"x60" 60"x60"	Replace culverts with one 12 ft by 6 ft box culvert
(T-31)	AS_M-12A 12B	60"x60" 60"x60"	Replace culverts with two 12 ft by 6 ft box culverts
R.C. West Road N	AS_M-14A 14B 14C 14D	48"x48" 48"x48" 60"x38" 60"x38"	Replace culverts with two 12 ft by 6 ft box culverts, Raise road from elevation 27.7 ft to elevation 29.0 ft NAVD (length of 1,710 ft)
Trask Parkway (US Hwy 17)	AS_M-28A 28B	30"x30" 66"x60"	Replace culverts with two 14 ft by 7 ft box culverts
Trask Parkway (US Hwy 17)	AS_T1-3	18"x18"	Replace culvert with one 8 ft by 4 ft box culverts
Branford Creek East Basin			
Charleston Highway (US Hwy 17)	BDCE_M-1	96"x96"	Verify entire road elevation is at least 8.0 ft NAVD, and raise to 8.0 ft NAVD if needed. This is a state road crossing and should be referred to SCDOT for improvements.
Big Estate Road	BDCE_T1-10	72"x72"	Replace culvert(s) with 1 - 8'x4' box & Raise Road Elevation to 8.5 ft. Raise road from elevation 7.2 ft to elevation 8.5 ft NAVD (length of 170 ft)
Brickyard Creek Basin			
Walling Grove Road	BC_M-0A 0B	46"x30" 46"x30"	Replace culverts with two 10 ft by 5 ft box culverts
Coosaw River Basin			
Walling Grove Road	CWR_M-7A 7B 7C	15"x15" 15"x15" 15"x15"	Raise road from elevation 11.5 ft to 12.5 ft NAVD
Dale Basin			
Wimbee Landing Road	DE_M-1	48"x48"	Replace culvert with one 6 ft by 4 ft box culvert
Wimbee Landing Road	DE_T1-2	42"x42"	Raise road from elevation 7.4 ft to 8.0 ft NAVD (length of 530 ft)
Halfmoon Island Basin			
Keans Neck Road	HM_M-1A 1B 1C	24"x24" 18"x18" 18"x18"	Replace culverts with two 48" pipes
Laurel Hill Basin			
Gadwell Drive	LH_M-3	15"x15"	Replace culvert with two 36" pipes, set pipe inverts to 5 ft NAVD, Raise road from elevation 8.5 ft to elevation 9.5 ft NAVD (length of 320 ft)
Lobeco Basin			
Keans Neck Road	LO_M-9	30"x30"	Replace culvert with one 10 ft by 5 ft box culvert

TABLE 9-7 (Updated 2017)
RECOMMENDED CULVERT IMPROVEMENTS
COOSAW RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
McCalleys Creek Basin			
*Trask Parkway (US Hwy 17)	MC_M-1A 1B	30"x30" 30"x30"	Replace culverts with one 8 ft by 4 ft box culvert
True Blue Creek North Basin			
Stroban Road	TBCN_M-13	36"x36"	Replace culvert with one 8 ft by 4 ft box culvert
True Blue Creek South Basin			
Kinlock Road	TBCS_M-1	30"x30"	Replace culvert with one 7 ft by 4 ft box culvert

** Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR.*

TABLE 9-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
COOSAW RIVER WATERSHED

Land Use Type	Brickyard Creek North (acres)	Bull River/ Wimbee Creek 1 (acres)	Bull River/ Wimbee Creek 2 (acres)	Bull River/ Wimbee Creek 3 (acres)	Bull River/ Wimbee Creek 4 (acres)	Cosaw River 1 (acres)	Coosaw River 2 (acres)	Coosaw River 3 (acres)	Coosaw River 4 (acres)	Lucy Point Creek North 1 (acres)
Agricultural/Pasture	0	0	1317	206	394	3	22	446	26	1
Commercial	2	0	6	0	1	4	0	14	0	6
Forest/Rural Open	322	31	889	1564	967	139	1242	423	54	166
Golf Course	58	0	0	0	0	0	1	0	0	0
High Density Residential	0	0	0	0	0	0	28	0	0	0
Industrial	508	0	92	103	175	21	180	144	18	38
Institutional	0	0	5	0	6	0	0	14	39	0
Low Density Residential	165	0	450	456	907	130	560	420	16	210
Medium Density Residential	87	0	0	0	0	16	302	130	24	66
Open Water/Tidal	1019	3087	2710	2101	1719	7054	6451	1335	328	383
Silviculture	0	0	157	1801	492	51	215	280	0	0
Urban Open	171	0	261	645	932	13	145	78	21	20
Wetland/Water	46	0	589	1622	2246	25	289	354	3	36
TOTAL	2378	3118	6477	8498	7837	7455	9436	3638	528	926
Urban Imperviousness (%)	17%	0%	2%	2%	3%	1%	3%	6%	7%	8%

TABLE 9-8 (CONTINUED)
WATER QUALITY BASIN LAND USE DISTRIBUTION
COOSAW RIVER WATERSHED

Land Use Type	Lucy Point Creek North 2 (acres)	McCallys Creek 1 (acres)	McCallys Creek 2 (acres)	South Wimbee Creek 1 (acres)	South Wimbee Creek 2 (acres)	Williman Creek 1 (acres)	Williman Creek 2 (acres)	Williman Creek 3 (acres)	Williman Creek Trib (acres)	TOTAL (acres)
Agricultural/Pasture	0	566	25	810	38	0	0	0	0	3854
Commercial	0	84	10	0	2	0	0	0	0	130
Forest/Rural Open	64	1064	179	493	157	92	305	148	0	8298
Golf Course	0	0	0	0	0	0	0	0	0	59
High Density Residential	0	24	0	0	0	0	0	0	0	51
Industrial	29	1135	78	31	29	0	9	6	0	2596
Institutional	1	7	21	0	0	0	0	0	0	92
Low Density Residential	30	467	283	119	152	0	0	0	0	4364
Medium Density Residential	145	485	100	0	0	0	0	0	0	1355
Open Water/Tidal	156	2111	537	1128	170	2679	1501	405	1103	35977
Silviculture	0	0	0	195	107	0	0	0	0	3298
Urban Open	13	188	42	188	24	0	8	0	0	2748
Wetland/Water	1	690	118	437	62	0	14	32	6	6570
TOTAL	438	6820	1393	3401	741	2770	1838	591	1108	69392
Urban Imperviousness (%)	14%	16%	9%	2%	5%	0%	1%	1%	0%	4%

TABLE 9-9
WATER QUALITY BASIN BMP COVERAGE
COOSAW RIVER WATERSHED

Land Use Type	Brickyard Creek North	Bull River/Wimbee Creek 1	Bull River/Wimbee Creek 2	Bull River/Wimbee Creek 3	Bull River/Wimbee Creek 4	Cosaw River 1	Coosaw River 2	Coosaw River 3	Coosaw River 4
Commercial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	61.5%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	7.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.8%	7.1%	0.0%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	3.6%	0.0%	0.0%
TOTAL	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.5%	0.8%	0.0%

TABLE 9-9 (CONTINUED)
WATER QUALITY BASIN BMP COVERAGE
COOSAW RIVER WATERSHED

Land Use Type	Lucy Point Creek North 1	Lucy Point Creek North 2	McCallys Creek 1	McCallys Creek 2	South Wimbee Creek 1	South Wimbee Creek 2	Williman Creek 1	Williman Creek 2	Williman Creek 3	Williman Creek Trib	TOTAL
Commercial	0.0%	0.0%	6.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	4.1%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	33.2%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.5%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	1.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.9%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.8%
TOTAL	0.0%	0.0%	0.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%

TABLE 9-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
COOSAW RIVER WATERSHED

Land Use Type	Brickyard Creek North	Bull River/Wimbee Creek 1	Bull River/Wimbee Creek 2	Bull River/Wimbee Creek 3	Bull River/Wimbee Creek 4	Cosaw River 1	Coosaw River 2	Coosaw River 3	Coosaw River 4	Lucy Point Creek North 1
Commercial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	6.6%	8.0%	0.0%	12.4%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%	0.2%	0.0%	3.7%	2.1%	0.1%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	2.7%	0.0%	0.0%
Low Density Residential	3.6%	0.0%	12.8%	2.4%	7.7%	3.4%	11.2%	15.3%	0.0%	13.4%
Medium Density Residential	6.4%	0.0%	0.0%	0.0%	0.0%	5.4%	12.4%	5.5%	7.4%	2.2%
TOTAL	0.5%	0.0%	0.9%	0.1%	0.9%	0.1%	1.1%	2.1%	0.3%	3.3%

TABLE 9-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
COOSAW RIVER WATERSHED

Land Use Type	Lucy Point Creek North 2	McCallys Creek 1	McCallys Creek 2	South Wimbee Creek 1	South Wimbee Creek 2	Williman Creek 1	Williman Creek 2	Williman Creek 3	Williman Creek Trib	TOTAL
Commercial	0.0%	0.0%	0.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	1.5%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.4%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.4%
Low Density Residential	19.2%	10.5%	6.6%	15.2%	22.1%	0.0%	0.0%	0.0%	0.0%	9.8%
Medium Density Residential	2.1%	5.8%	2.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	6.5%
TOTAL	2.0%	1.1%	1.5%	0.5%	4.5%	0.0%	0.0%	0.0%	0.0%	0.8%

TABLE 9-11
AVERAGE ANNUAL LOADS FOR COOSAW RIVER WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
Coosaw River 1	7,455	25,910	215,000	288	222,000	2.09E+15	428	91,958	11,349	486,000	10,216
Coosaw River 2	9,436	25,795	234,000	330	223,000	2.35E+15	448	93,879	11,702	862,000	9,638
Coosaw River 3	3,638	6,731	72,541	117	58,717	7.29E+14	127	26,434	3,808	458,000	2,189
Coosaw River 4	528	1,493	17,354	24	13,186	1.88E+14	32	5,746	795	105,000	539
Lucy Point Creek 1 (North)	926	1,872	22,326	32	16,522	2.37E+14	41	7,269	975	144,000	643
Lucy Point Creek 2 (North)	438	878	12,852	20	7,919	1.40E+14	23	3,585	522	99,220	293
Bull River / Wimbee Creek 1	3,118	11,219	91,485	122	96,098	8.85E+14	183	39,660	4,879	184,000	4,449
Bull River / Wimbee Creek 2	6,477	12,299	111,000	171	106,000	1.10E+15	194	46,744	7,201	534,000	4,086
Bull River / Wimbee Creek 3	8,498	12,040	105,000	158	104,000	1.07E+15	180	45,604	5,267	656,000	3,314
Bull River / Wimbee Creek 4	7,837	11,316	107,000	171	98,375	1.15E+15	173	43,069	5,395	773,000	2,849
Williman Creek Trib. 1	1,108	4,010	32,692	44	34,347	3.16E+14	65	14,175	1,743	65,804	1,589
Williman Creek 1	2,770	9,763	79,534	106	83,625	7.70E+14	159	34,512	4,241	161,000	3,860
Williman Creek 2	1,838	5,627	45,950	63	48,213	4.45E+14	90	19,932	2,428	107,000	2,172
Williman Creek 3	591	1,590	13,015	19	13,626	1.26E+14	25	5,648	678	36,713	589
South Wimbee Creek 1	3,401	5,530	47,563	76	47,566	4.72E+14	79	21,291	3,435	245,000	1,691
South Wimbee Creek 2	741	1,028	12,132	19	9,052	1.28E+14	21	4,170	563	91,877	302
McCalleys Creek 1	6,820	13,633	193,000	417	120,000	1.61E+15	326	56,395	7,743	1,630,000	4,355
McCalleys Creek 2	1,393	2,806	34,691	54	24,811	3.56E+14	62	10,998	1,485	242,000	937
Brickyard Creek North	2,378	5,611	76,364	171	49,121	6.00E+14	134	22,613	2,891	592,000	1,972
TOTAL	69,392	159,151	1,523,499	2,402	1,376,178	1.48E+16	2,790	593,682	77,100	7,472,614	55,683

TABLE 9-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS

COOSAW RIVER WATERSHED

Water Quality Basin ID	DHEC Station(s)	Years of Record	No. of Samples	Fecal Coliform Concentrations				Trend	Level of Service
				Long-Term Average		Most Recent 3 Year Values			
				Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)		
Coosaw River 1	14-10, 14-16A, 14-11	1999-2016	620	2.77	8	2.83	11	No Trend	A
Coosaw River 2	14-12A	1999-2016	208	2.98	7.86	2.5	4.5	Decreasing	A
Coosaw River 3	14-13, 14-13A, 14-02	1999-2016	471	7.02	33	9.36	33	Increasing	B
Coosaw River 4	NA	NA	NA	NA	NA	NA	NA	NA	NA
Lucy Point Creek North 1	16A-13B	1999-2016	207	8.87	33	10.87	31.83	No Trend	C
Lucy Point Creek North 2	16A-33	2006-2016	123	11.85	41.03	12.45	49	No Trend	D
Bull River/Wimbee Creek 1	14-04	1999-2016	206	2.44	5	2.77	7.80	No Trend	A
Bull River/Wimbee Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bull River/Wimbee Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bull River/Wimbee Creek 4	NA	NA	NA	NA	NA	NA	NA	NA	NA
Williman Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Williman Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Williman Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Williman Creek Trib	NA	NA	NA	NA	NA	NA	NA	NA	NA
South Wimbee Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
South Wimbee Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
McCalleys Creek 1	15-01A, 15-33	1999-2016	207	2.48	5	6.74	47.48	Decreasing	A
McCalleys Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Brickyard Creek North	15-01	1999-2016	203	4.48	17	7.71	33	Increasing	A

TABLE 9-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

COOSAW RIVER WATERSHED

Water Quality Basin ID	North WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Coosaw River 1	22	7.60E+07	St. Helena Sound	8,920	8,915	1,500
			Parrot Creek 2	2,112	1,271	0
Coosaw River 2	23	4.17E+07	Coosaw River 1	4,865	9,688	1,500
Coosaw River 3	24	4.81E+06	Coosaw River 2	1,848	6,373	150
Coosaw River 4	25	1.15E+06	Coosaw River 3	1,026	1,818	150
			Whale Branch West 3	762	2,446	150
Lucy Point Creek North 1	26	1.01E+06	Coosaw River 1	854	1,835	20
Lucy Point Creek North 2	27	4.20E+05	Lucy Point Creek North 1	407	1,368	300
			Lucy Point Creek South 2	249	1,642	300
Bull River/Wimbee Creek 1	28	1.89E+07	Coosaw River 1	3,121	3,832	150
Bull River/Wimbee Creek 2	29	9.76E+06	Bull River/Wimbee Creek 1	841	7,113	150
Bull River/Wimbee Creek 3	30	6.33E+06	Bull River/Wimbee Creek 2	747	7,290	75
Bull River/Wimbee Creek 4	31	1.30E+06	Bull River/Wimbee Creek 3	884	2,897	75
Williman Creek 1	32	1.01E+07	Bull River/Wimbee Creek 1	1,210	4,828	150
Williman Creek 2	33	3.93E+06	Williman Creek 1	1,234	3,991	150
Williman Creek 3	34	1.26E+06	Williman Creek 2	448	2,333	150
Williman Creek Trib	35	1.58E+06	Bull River/Wimbee Creek 1	325	2,929	150
South Wimbee Creek 1	36	1.51E+06	Bull River/Wimbee Creek 2	308	6,823	150
South Wimbee Creek 2	37	1.88E+05	South Wimbee Creek 1	59	4,699	150
McCalleys Creek 1	38	6.99E+06	Coosaw River 2	1,409	8,175	900
McCalleys Creek 2	39	9.46E+05	McCalleys Creek 1	326	6,212	900
Brickyard Creek North	40	1.12E+06	McCalleys Creek 1	825	1,352	900
			Brickyard Creek South	546	2,784	10

TABLE 9-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMM FOR COOSAW RIVER WATER QUALITY BASINS

Water Quality Basin ID	North WASP Segment	EXISTING LAND USE Flow (cfs)	EXISTING LAND USE Fecal Coliform (#/100 ml)
Coosaw River 1	22	41.8	1,063
Coosaw River 2	23	43.2	1,046
Coosaw River 3	24	12.2	979
Coosaw River 4	25	2.3	1,099
Lucy Point Creek North 1	26	3.3	1,082
Lucy Point Creek North 2	27	1.6	1,229
Bull River/Wimbee Creek 1	28	18.0	1,043
Bull River/Wimbee Creek 2	29	22.2	941
Bull River/Wimbee Creek 3	30	23.5	834
Bull River/Wimbee Creek 4	31	21.9	871
Williman Creek 1	32	15.7	1,070
Williman Creek 2	33	9.2	1,035
Williman Creek 3	34	2.7	1,000
Williman Creek Trib	35	6.4	1,077
South Wimbee Creek 1	36	10.4	862
South Wimbee Creek 2	37	2.0	915
McCalleys Creek 1	38	24.3	1,066
McCalleys Creek 2	39	5.0	1,093
Brickyard Creek North	40	9.7	1,089

TABLE 9-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES

COOSAW RIVER WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs)
		Existing
Coosaw River 1	St. Helena Sound	1,832
Coosaw River 1	Lucy Point Creek North 1	287
Coosaw River 1	Parrot Creek 2	374
Coosaw River 2	Coosaw River 1	2,320
Coosaw River 3	Coosaw River 2	1,683
Coosaw River 4	Coosaw River 3	1,670
Whale Branch West 3	Coosaw River 4	1,668
Lucy Point Creek North 1	Lucy Point Creek North 2	290
Lucy Point Creek North 2	Lucy Point Creek South 2	292
Bull River/Wimbee Creek 1	Coosaw River 1	132
Bull River/Wimbee Creek 2	Bull River/Wimbee Creek 1	80
Bull River/Wimbee Creek 3	Bull River/Wimbee Creek 2	45
Bull River/Wimbee Creek 4	Bull River/Wimbee Creek 3	22
Williman Creek 1	Bull River/Wimbee Creek 1	31
Williman Creek 2	Williman Creek 1	25
Williman Creek 3	Williman Creek 2	9.3
Williman Creek Trib	Bull River/Wimbee Creek 1	2.7
South Wimbee Creek 1	Bull River/Wimbee Creek 2	12
South Wimbee Creek 2	South Wimbee Creek 1	2.0
McCalleys Creek 1	Coosaw River 2	594
McCalleys Creek 2	McCalleys Creek 1	5.0
Brickyard Creek North	McCalleys Creek 1	564
Brickyard Creek South	Brickyard Creek North	555

TABLE 9-16

FECAL COLIFORM MODELING RESULTS

COOSAW RIVER WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)	Modeled Level of Service
		Existing	Existing
Coosaw River 1	#REF!	3.3	A
Coosaw River 2	0.0	4.2	A
Coosaw River 3	0.0	7.0	A
Coosaw River 4	0.0	6.8	A
Lucy Point Creek North 1	0.0	5.8	A
Lucy Point Creek North 2	0.0	6.7	A
Bull River/Wimbee Creek 1	0.0	3.2	A
Bull River/Wimbee Creek 2	0.0	5.6	A
Bull River/Wimbee Creek 3	0.0	10.2	D
Bull River/Wimbee Creek 4	0.0	20.1	D
Williman Creek 1	0.0	4.1	A
Williman Creek 2	0.0	5.0	A
Williman Creek 3	0.0	5.0	A
Williman Creek Trib	0.0	7.1	B
South Wimbee Creek 1	0.0	12.3	D
South Wimbee Creek 2	0.0	18.3	D
McCalleys Creek 1	0.0	4.9	A
McCalleys Creek 2	0.0	5.6	A
Brickyard Creek North	0.0	5.2	A

NOTE: Water quality basins with lower LOS are highlighted.

Table 9-17 is not applicable in the update.

TABLE 9-18 (Updated 2017)
PLANNING LEVEL COST ESTIMATES FOR
COOSAW RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
AS_M-0 *	Road overtopping at R.C. West Road N. Replace existing 2 - 72" RCP with 2 - 12'x6' box culverts	$1,014,000
AS_M-12 *	Road overtopping at T-31 Replace existing 2 - 60" RCP with 2 - 12'x6' box culverts	$4,529,000
AS_M-14 *	Road overtopping at R.C. West Road N Replace existing 2 - 48" RCP and 2 - 60"x38" arches with 2 - 12'x6' box culverts Raise road 1.3 ft (length of 1,710 ft)	$1,355,000
AS_M-28	Road overtopping at Trask Parkway (US Hwy 17) Replace existing 1 - 30" RCP and 1 - 5.5'x5' box culvert with 2 - 14'x7' box culverts	$1,279,000
AS_T1-3	Road overtopping at Trask Parkway (US Hwy 17) Replace existing 1 - 18" RCP with 1 - 8'x4' box culvert	$344,000
BC_M-0	Road overtopping at Walling Grove Road Replace existing 2 - 46"x30" box culverts with 2 - 10'x5' box culverts	$226,000
BDCE_T1-10	Road overtopping at Big Estate Road Replace existing 1 - 72" RCP with 1 - 8'x4' box culvert Raise road 1.3 ft (length of 170 ft)	$179,000
CWR_M-7	Road Overtopping at Wailing Grove Road Raise Road from elevation 11.5 ft to 12.5 ft NAVD	$155,000
DE_M-1	Road overtopping at Wimbee Landing Road Replace existing 1 - 48" RCP with 1 - 6'x4' box culvert	$105,000
DE_T1-2	Road overtopping at Wimbee Landing Road Raise road 0.6 feet (length of 530 ft)	$232,000
HM_M-1	Road overtopping at Keans Neck Road Replace existing 1 - 24" RCP and 2 - 18" RCP with 2 - 48" RCP	$55,000
LH_M-3	Road overtopping at Gadwell Drive Replace existing 1 - 15" RCP with 2 - 36" RCP Raise road 1.0 feet (length of 320 ft)	$36,000
LO_M-9	Road overtopping at Keans Neck Road Replace existing 1 - 30" RCP with 1 - 10'x5' box culvert	$114,000
TBCN_M-13	Road overtopping at Stroban Road Replace existing 1 - 36" RCP with 1 - 8'x4' box culvert	$123,000
TBCS_M-1	Road overtopping at Kinlock Road Replace existing 1 - 30" RCP with 1 - 7'x4' box culvert	$83,000
	TOTAL	$9,829,000

* Conduits marked by asterisk are on private land

Costs are in January 2018 dollars.

See Appendix for basis of cost estimates.

Figure 9-1

Coosaw River Watershed

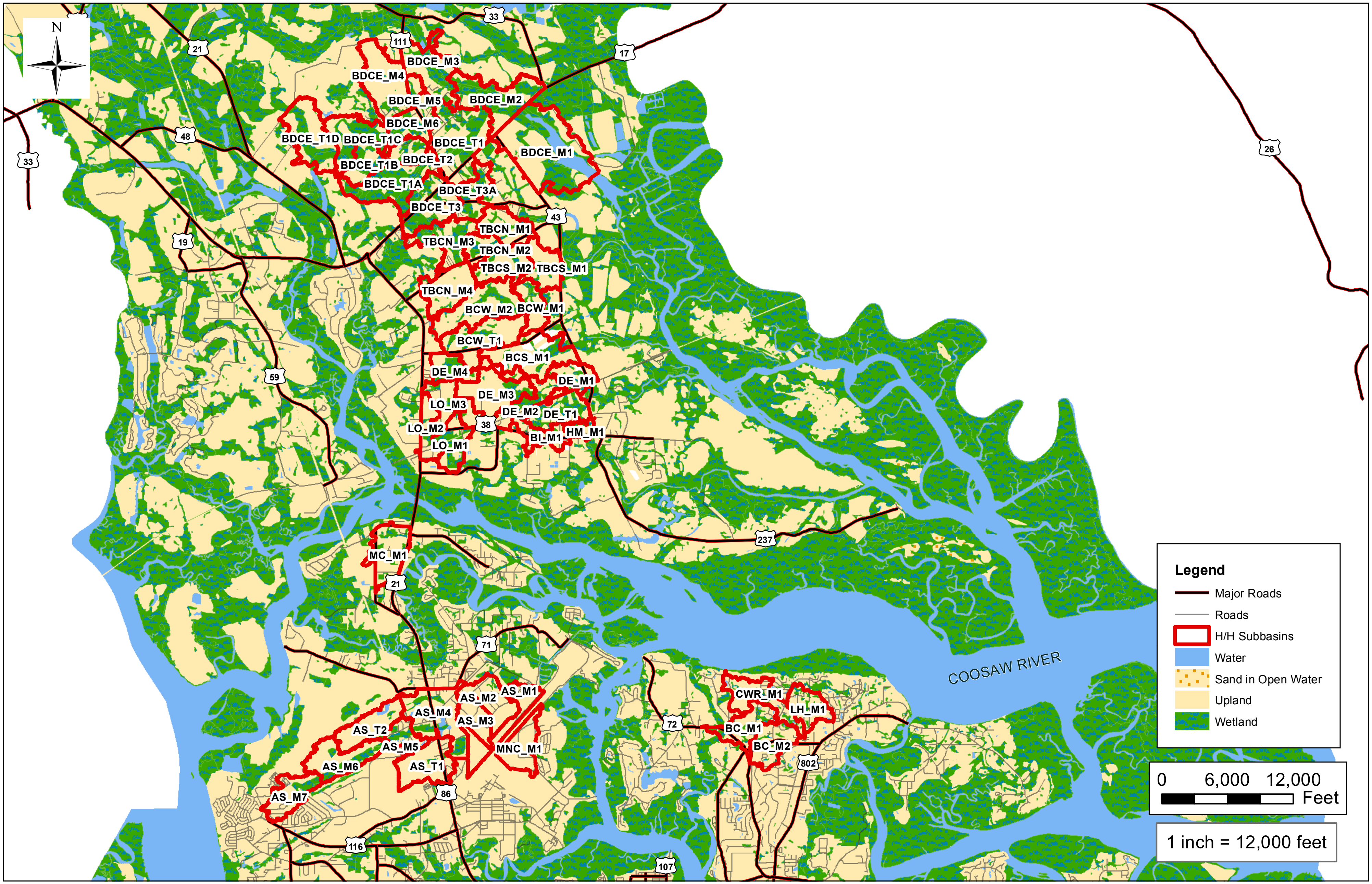

Figure 9-2. Hydrologic Model Subbasins

Coosaw River Watershed

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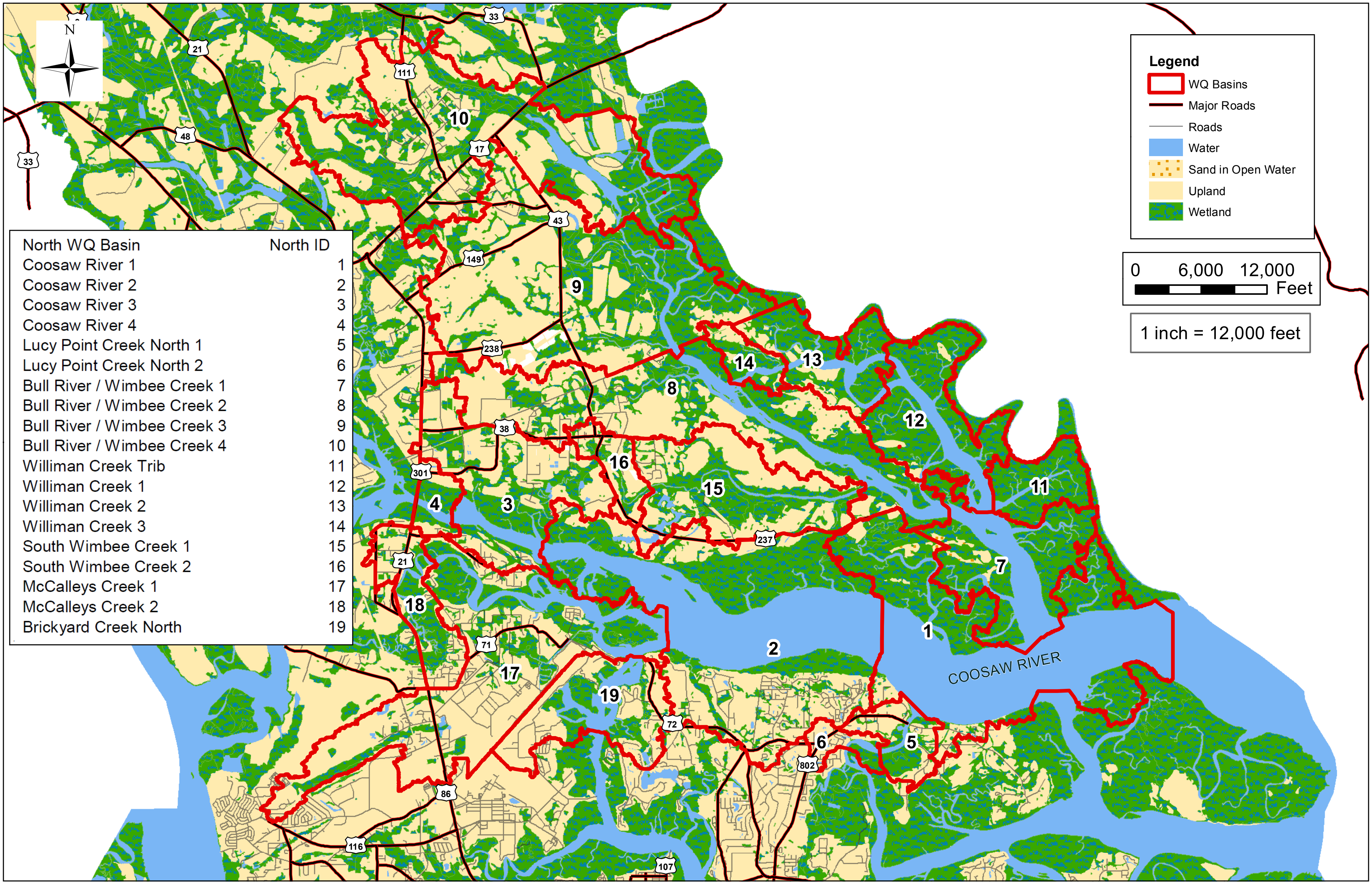
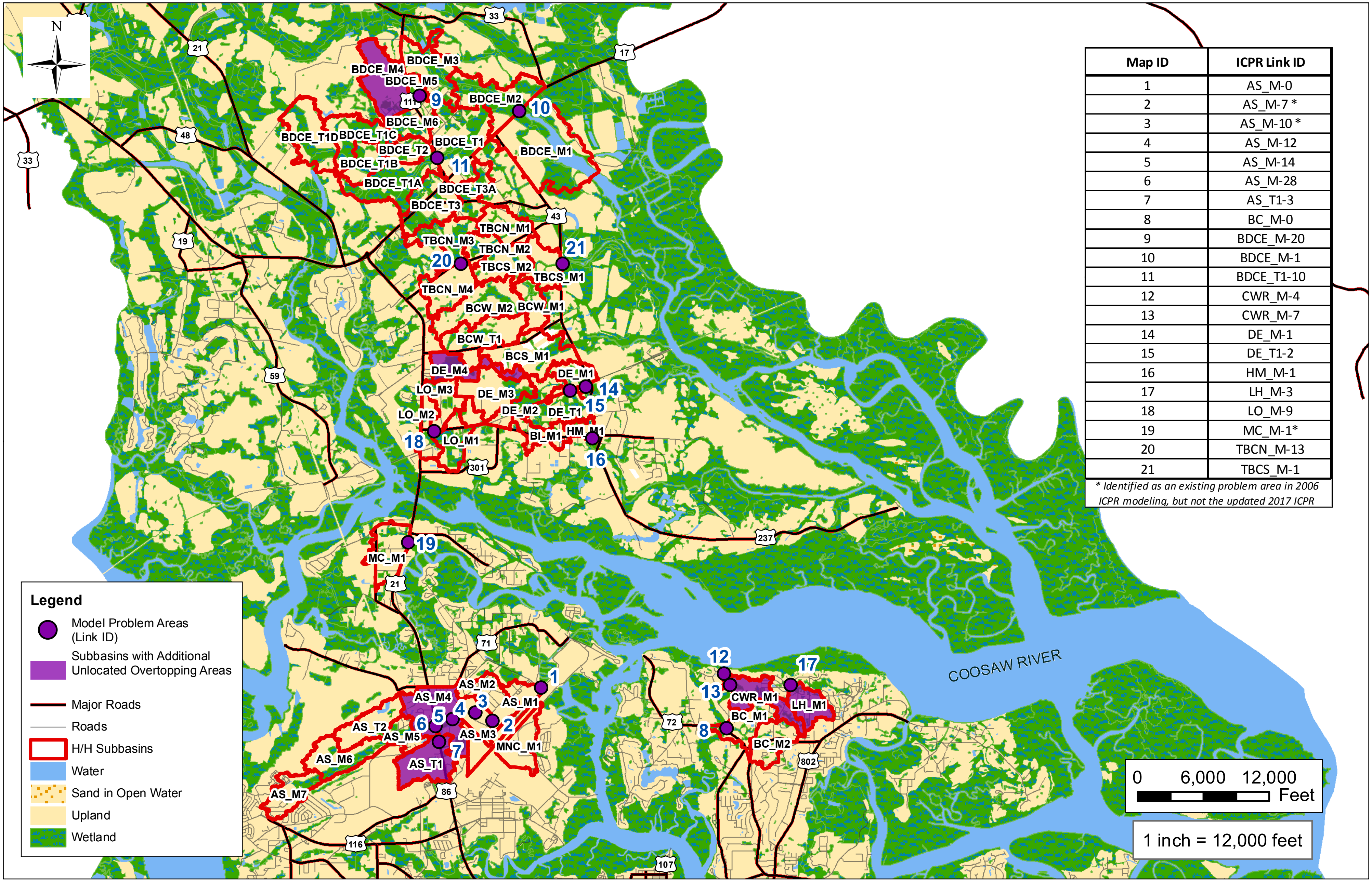


North WQ Basin	North ID
Coosaw River 1	1
Coosaw River 2	2
Coosaw River 3	3
Coosaw River 4	4
Lucy Point Creek North 1	5
Lucy Point Creek North 2	6
Bull River / Wimbee Creek 1	7
Bull River / Wimbee Creek 2	8
Bull River / Wimbee Creek 3	9
Bull River / Wimbee Creek 4	10
Williman Creek Trib	11
Williman Creek 1	12
Williman Creek 2	13
Williman Creek 3	14
South Wimbee Creek 1	15
South Wimbee Creek 2	16
McCalleys Creek 1	17
McCalleys Creek 2	18
Brickyard Creek North	19

Figure 9-3

Coosaw River Watershed
Water Quality Basins

ATM DESIGN ENGINEERING CONSULTING



Map ID	ICPR Link ID
1	AS_M-0
2	AS_M-7 *
3	AS_M-10 *
4	AS_M-12
5	AS_M-14
6	AS_M-28
7	AS_T1-3
8	BC_M-0
9	BDCE_M-20
10	BDCE_M-1
11	BDCE_T1-10
12	CWR_M-4
13	CWR_M-7
14	DE_M-1
15	DE_T1-2
16	HM_M-1
17	LH_M-3
18	LO_M-9
19	MC_M-1*
20	TBCN_M-13
21	TBCS_M-1

** Identified as an existing problem area in 2006 ICPR modeling, but not the updated 2017 ICPR*

Figure 9-4. ICPR Identified PSMS Overtopping Problem Areas

Coosaw River Watershed

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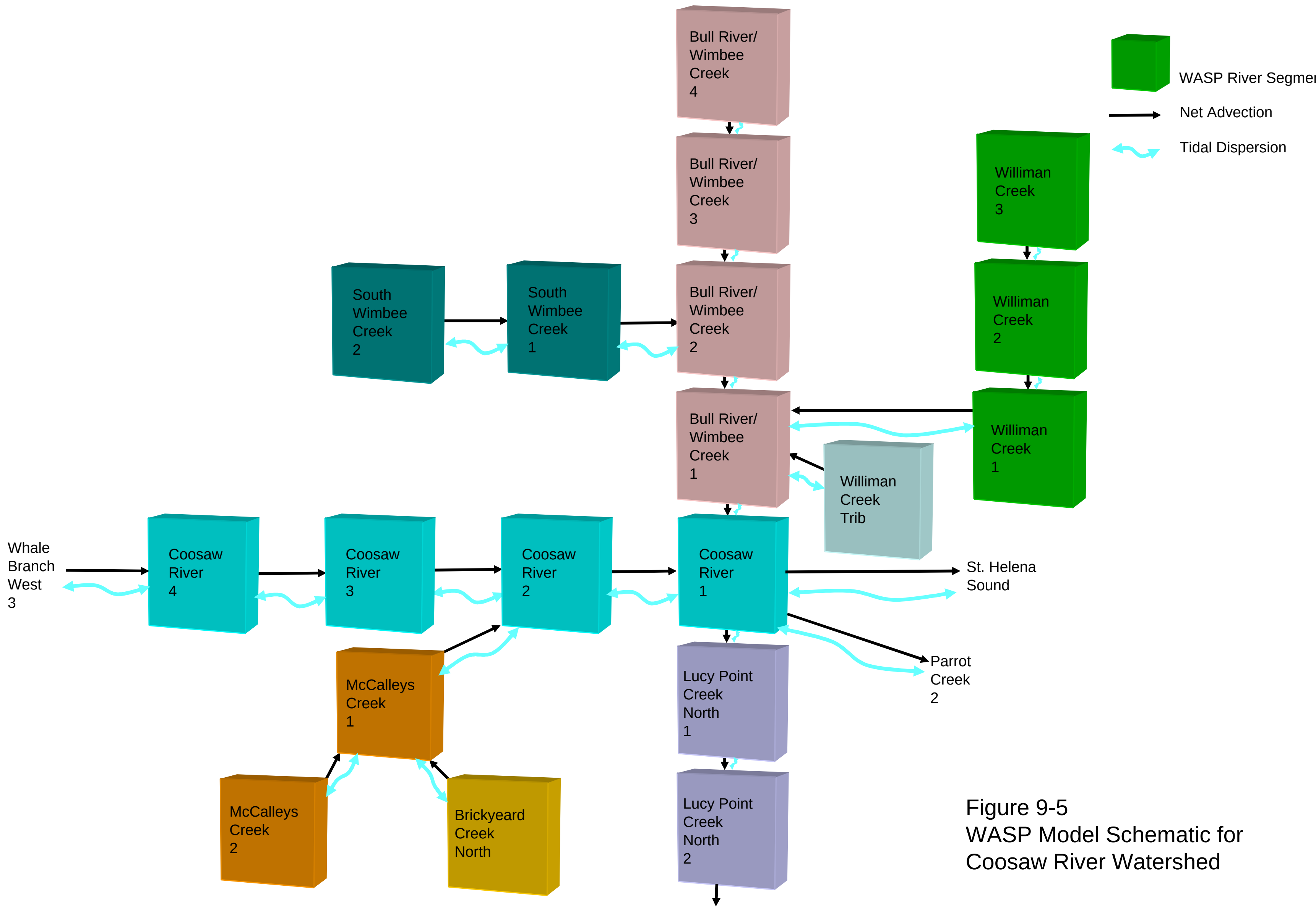

Figure 9-5
WASP Model Schematic for
Coosaw River Watershed

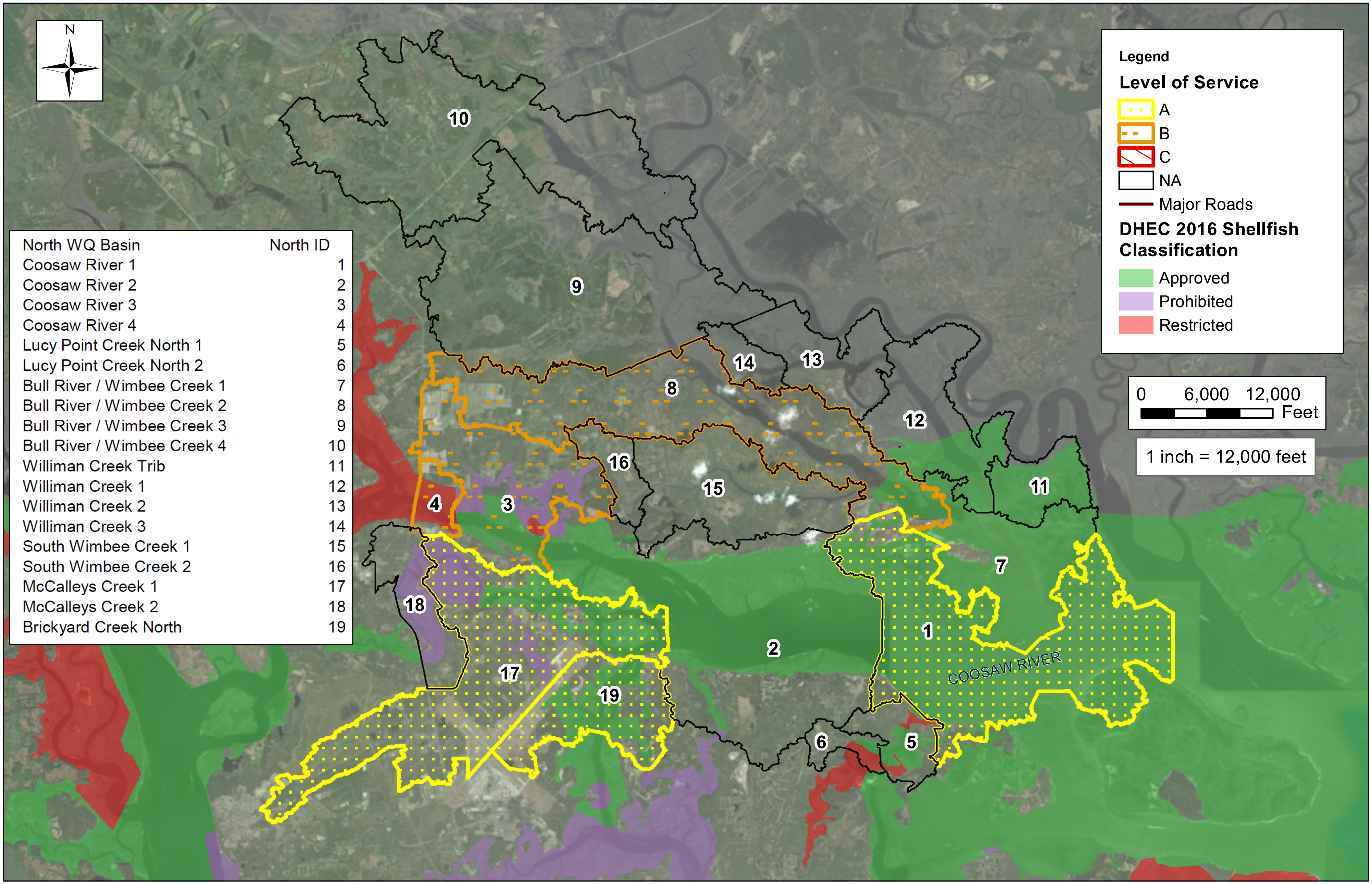

Figure 9-6

Coosaw River Watershed
Shellfish Classification and Existing Level of Service

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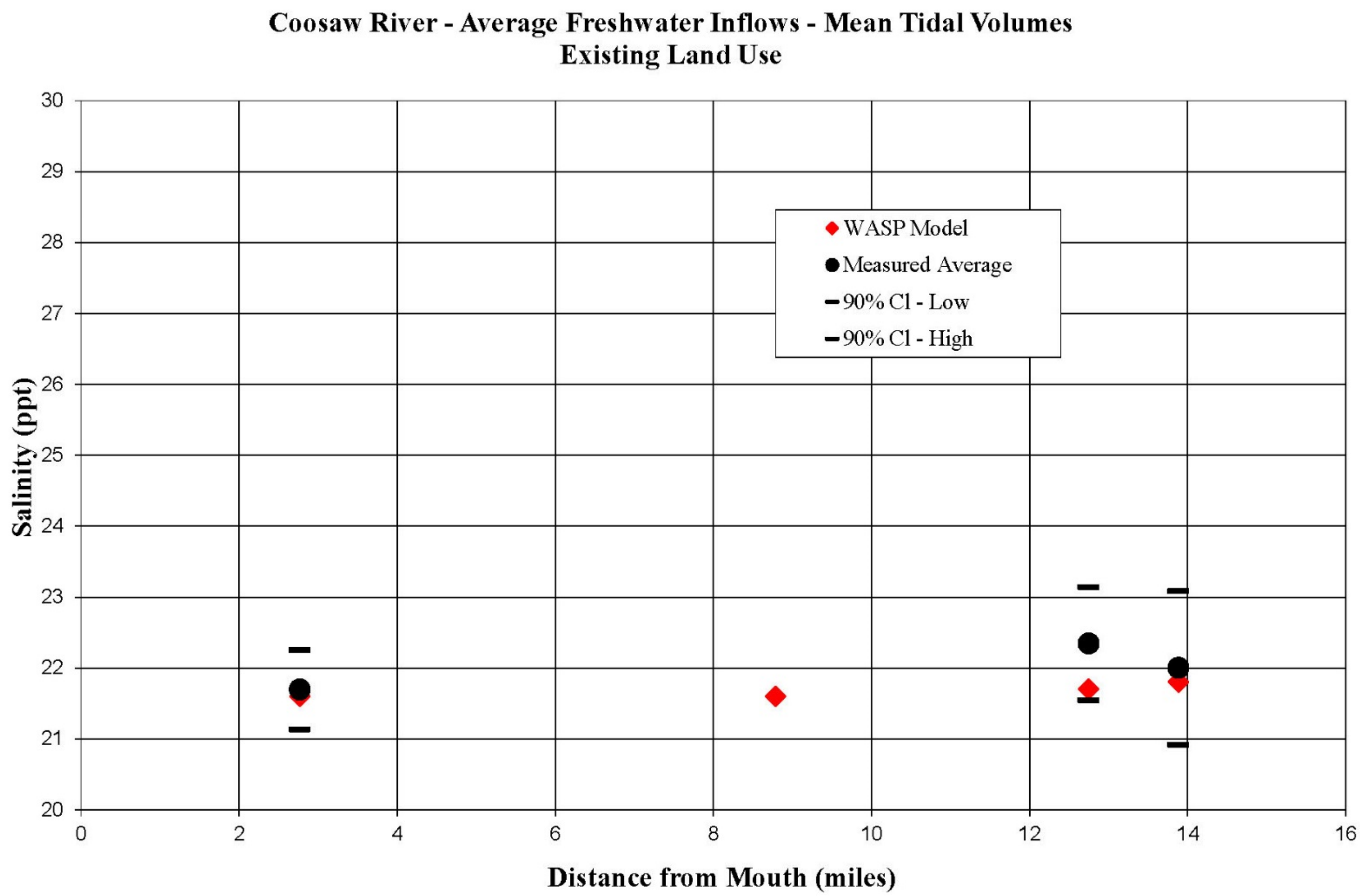

Figure 9-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Coosaw River - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

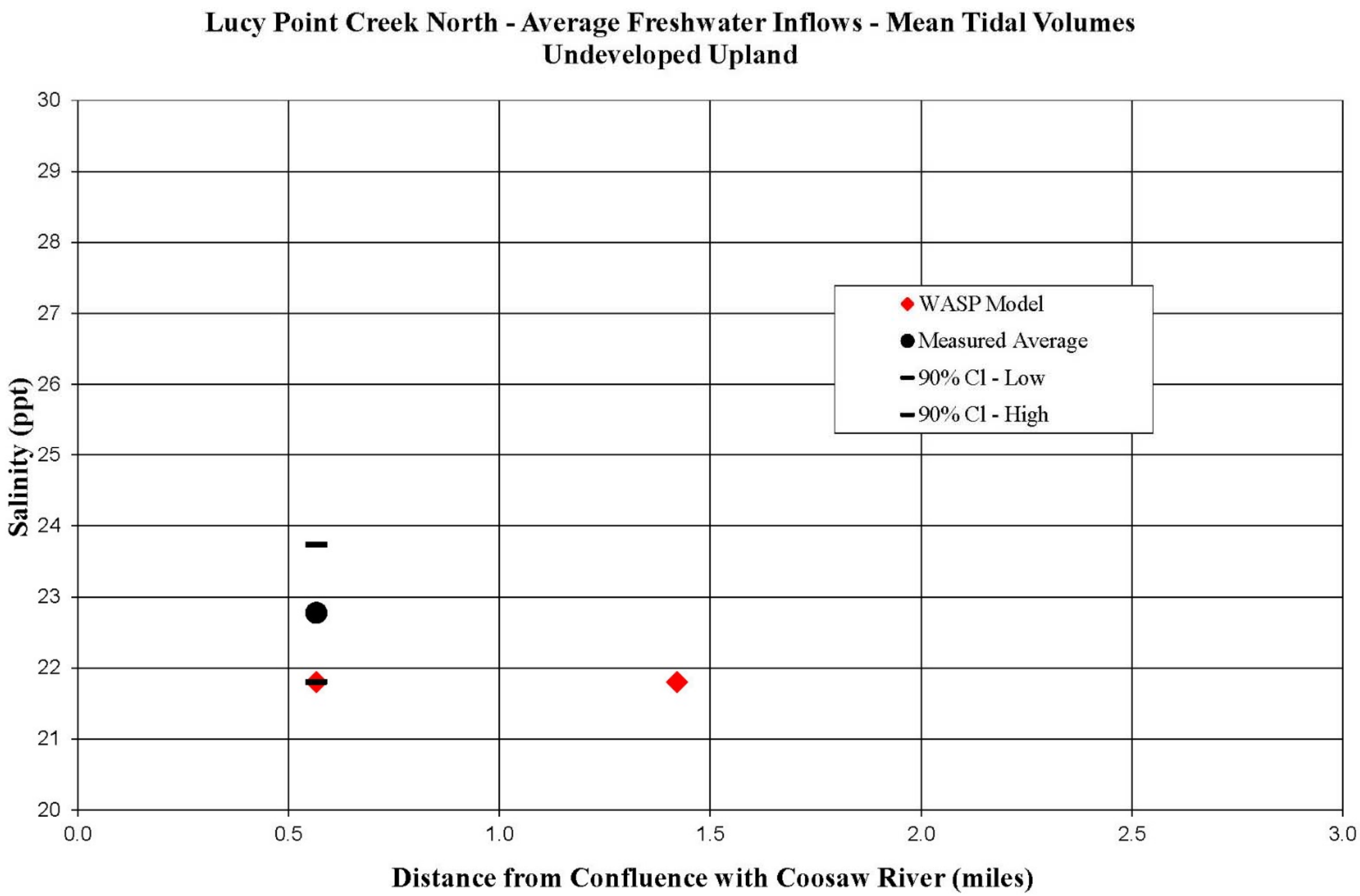

Figure 9-8. Comparison of WASP Model Results with Long-Term Monitoring Data in Lucy Point Creek North- Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

ATM DESIGN ENGINEERING CONSULTING

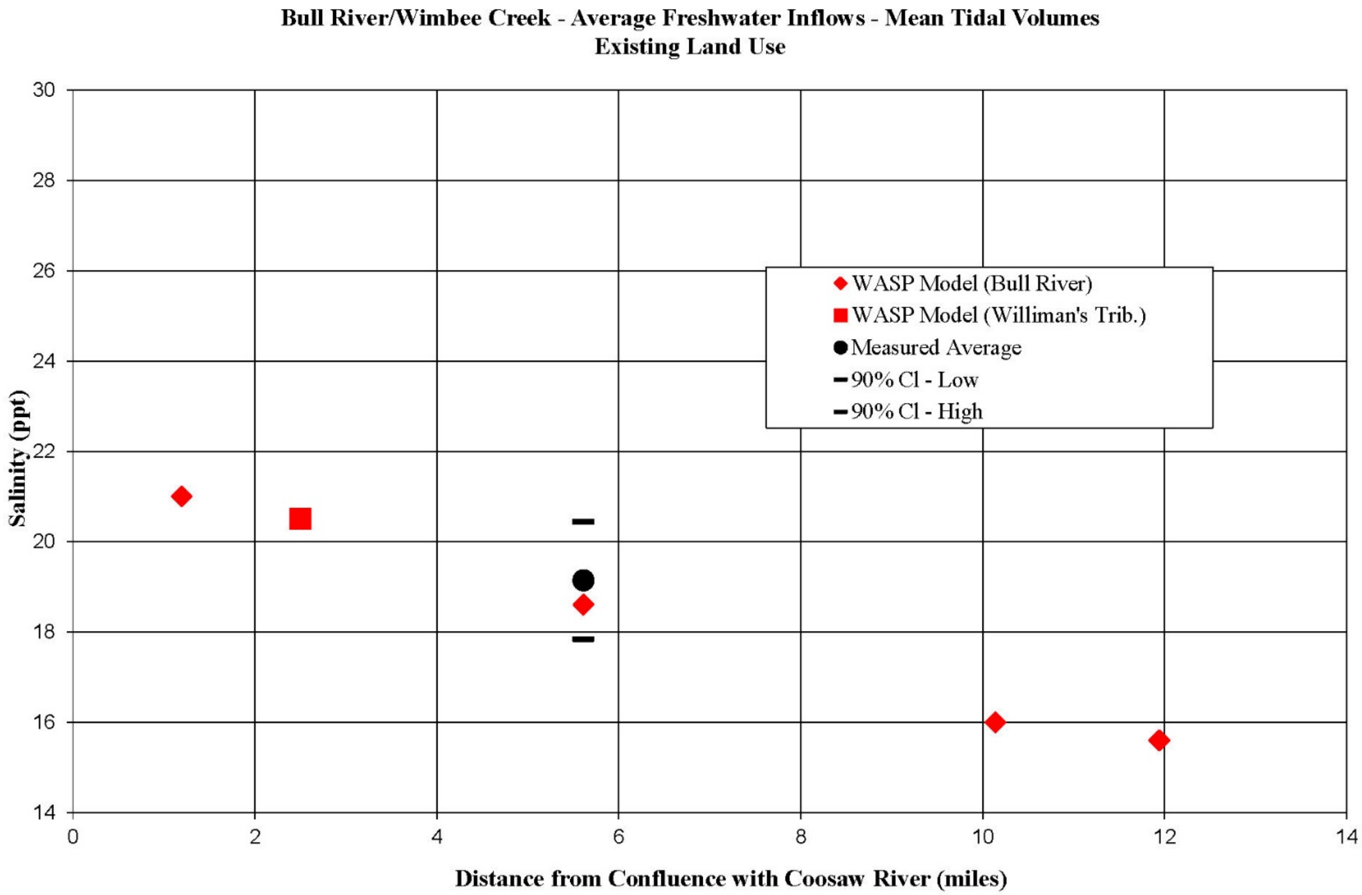

Figure 9-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull River/Wimbee Creek - Salinity.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

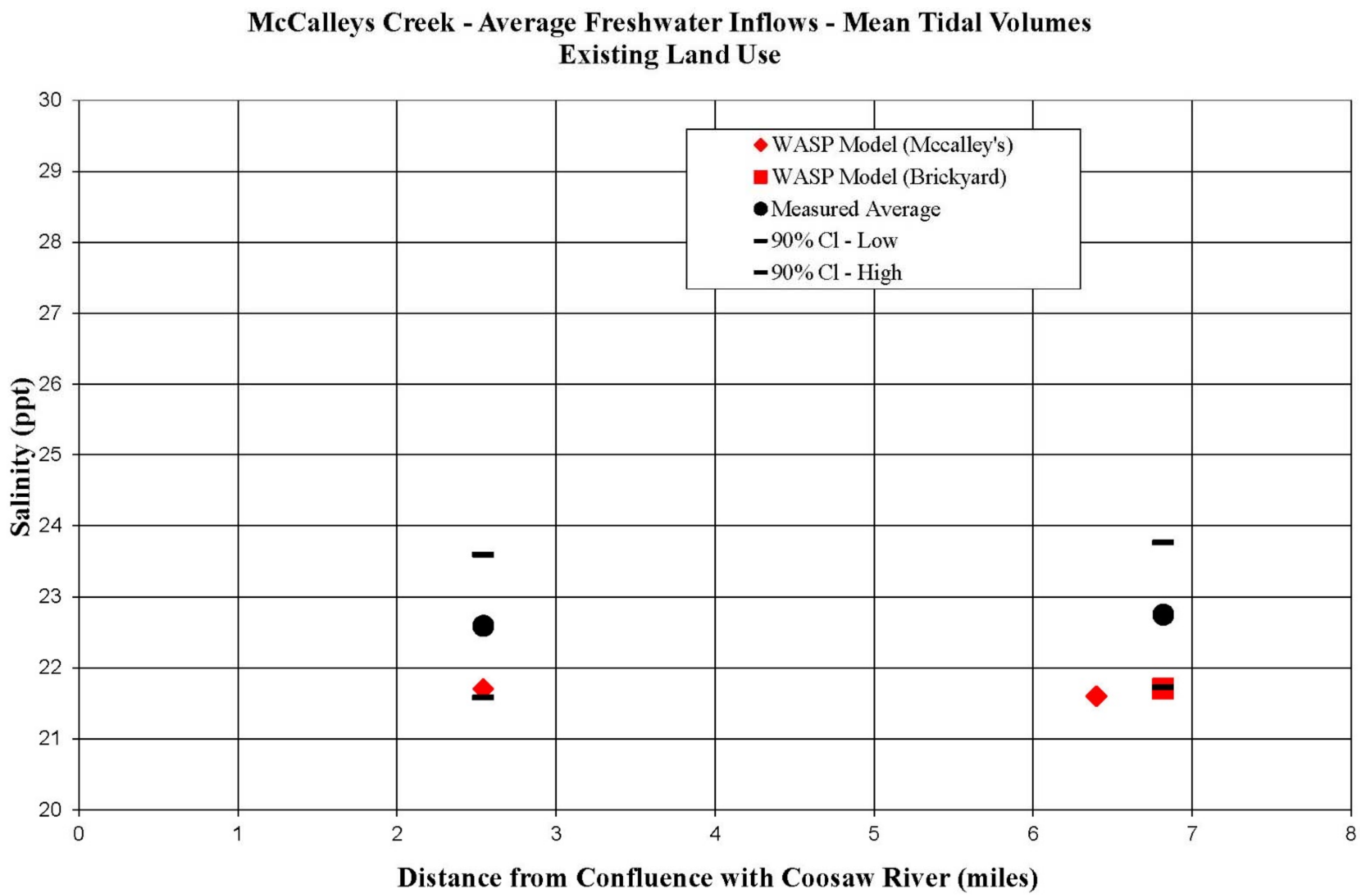

Figure 9-10. Comparison of WASP Model Results with Long-Term Monitoring Data in McCalley's Creek/Brickyard Creek North - Salinity.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

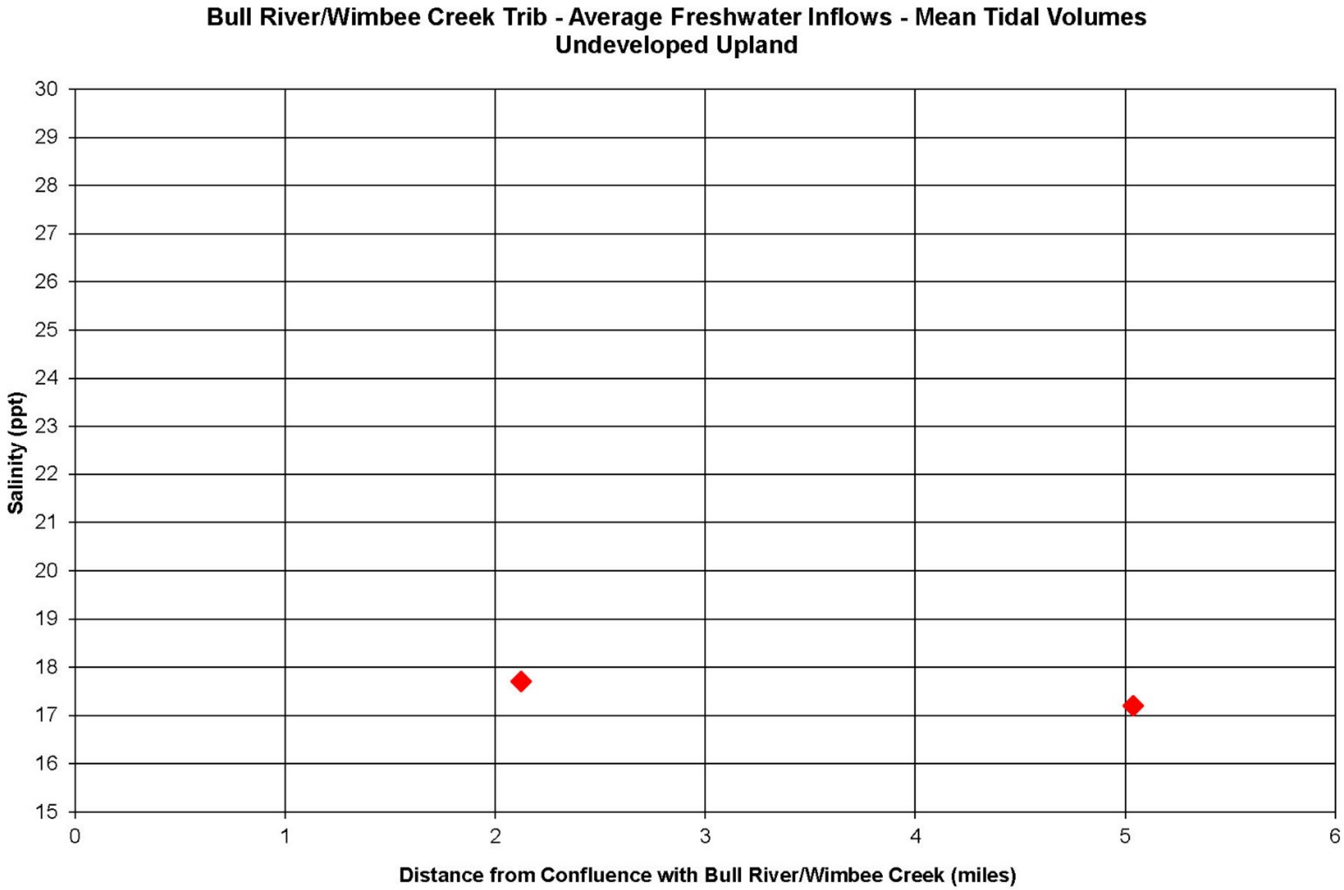

Figure 9-11. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull River and Wimbee Creek Tributary - Salinity.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

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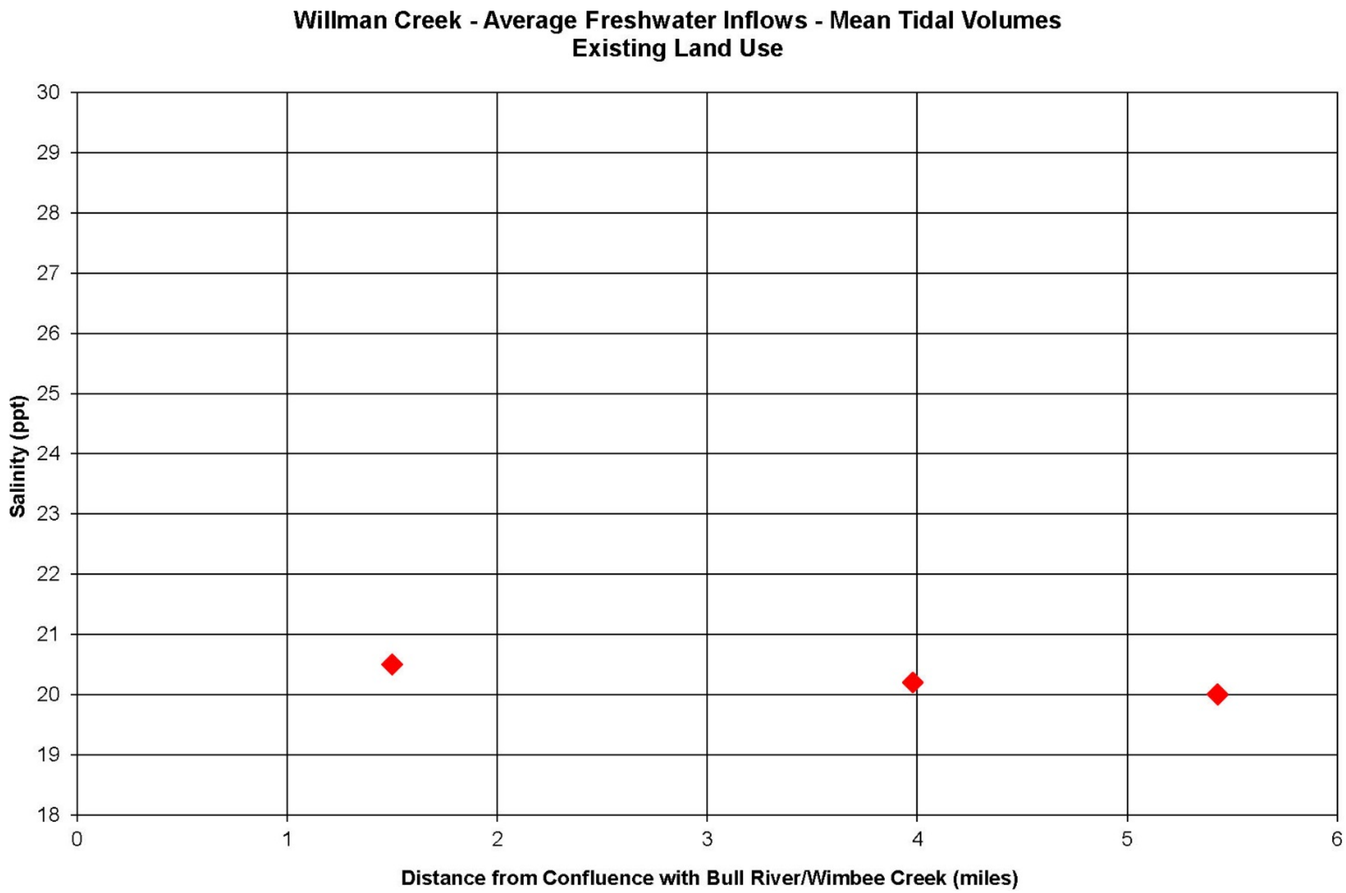

Figure 9-12. Comparison of WASP Model Results with Long-Term Monitoring Data in Willman Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

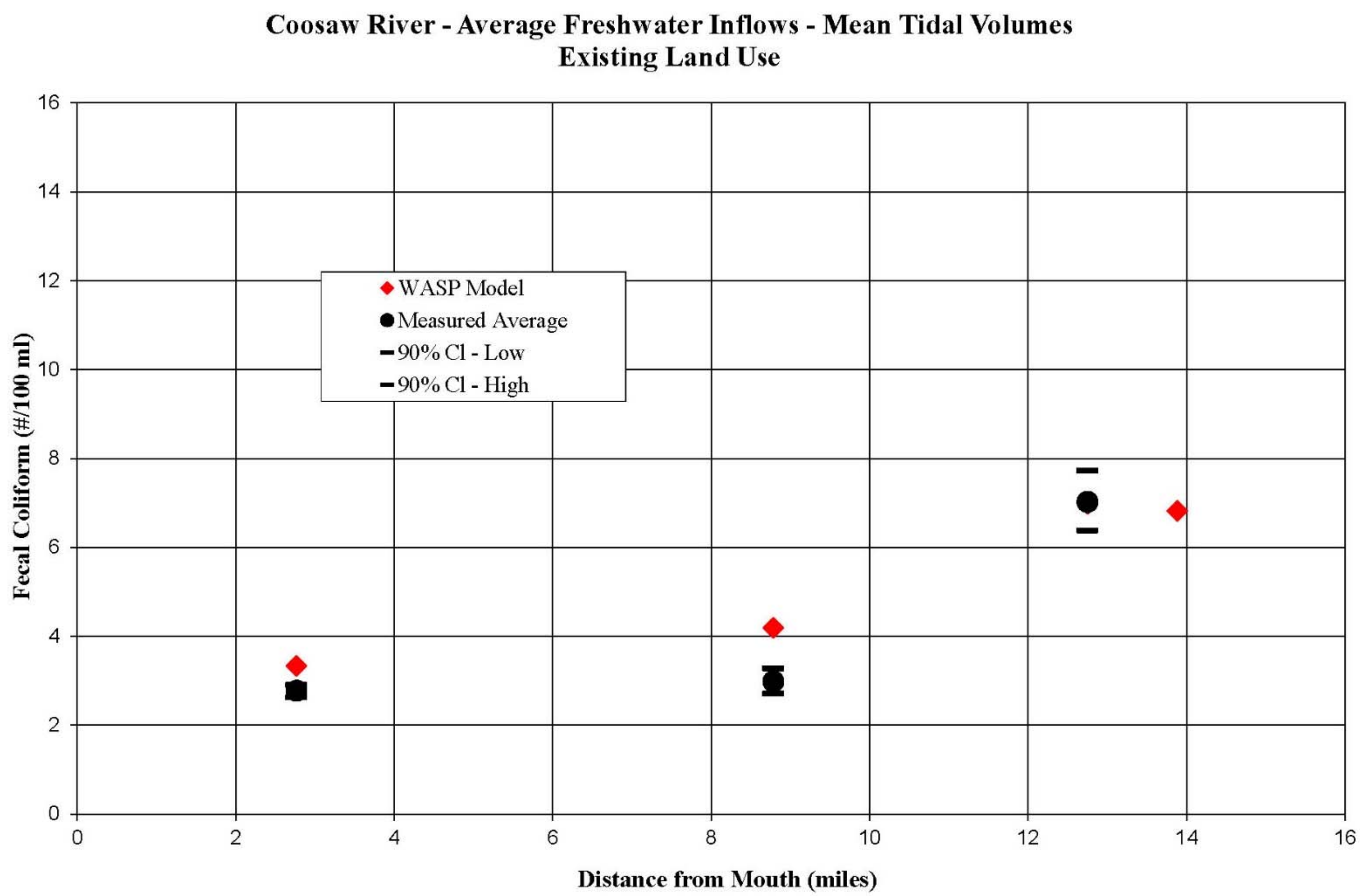

Figure 9-13. Comparison of WASP Model Results with Long-Term Monitoring Data in Coosaw River - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

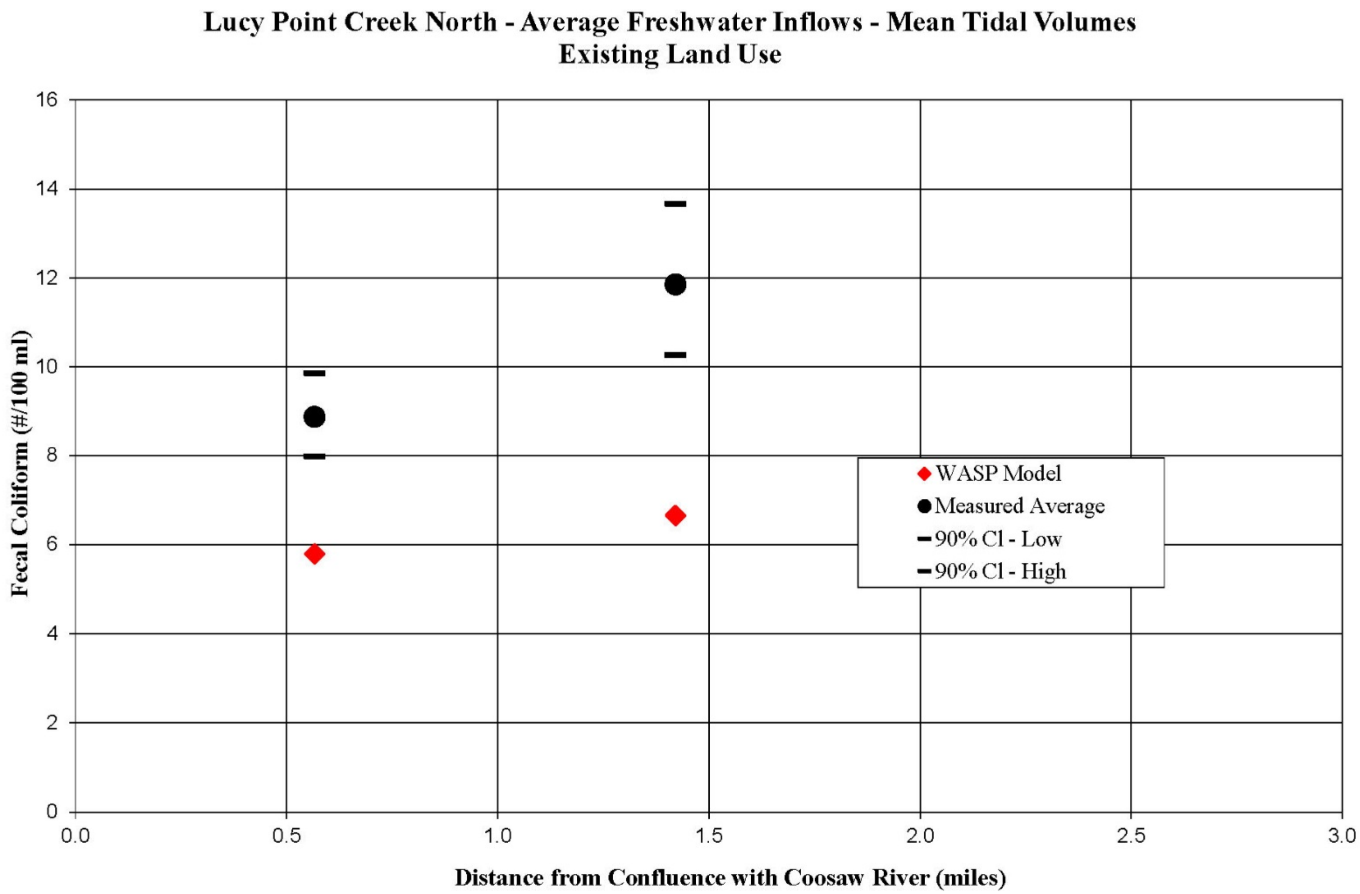

Figure 9-14. Comparison of WASP Model Results with Long-Term Monitoring Data in Lucy Point Creek North - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

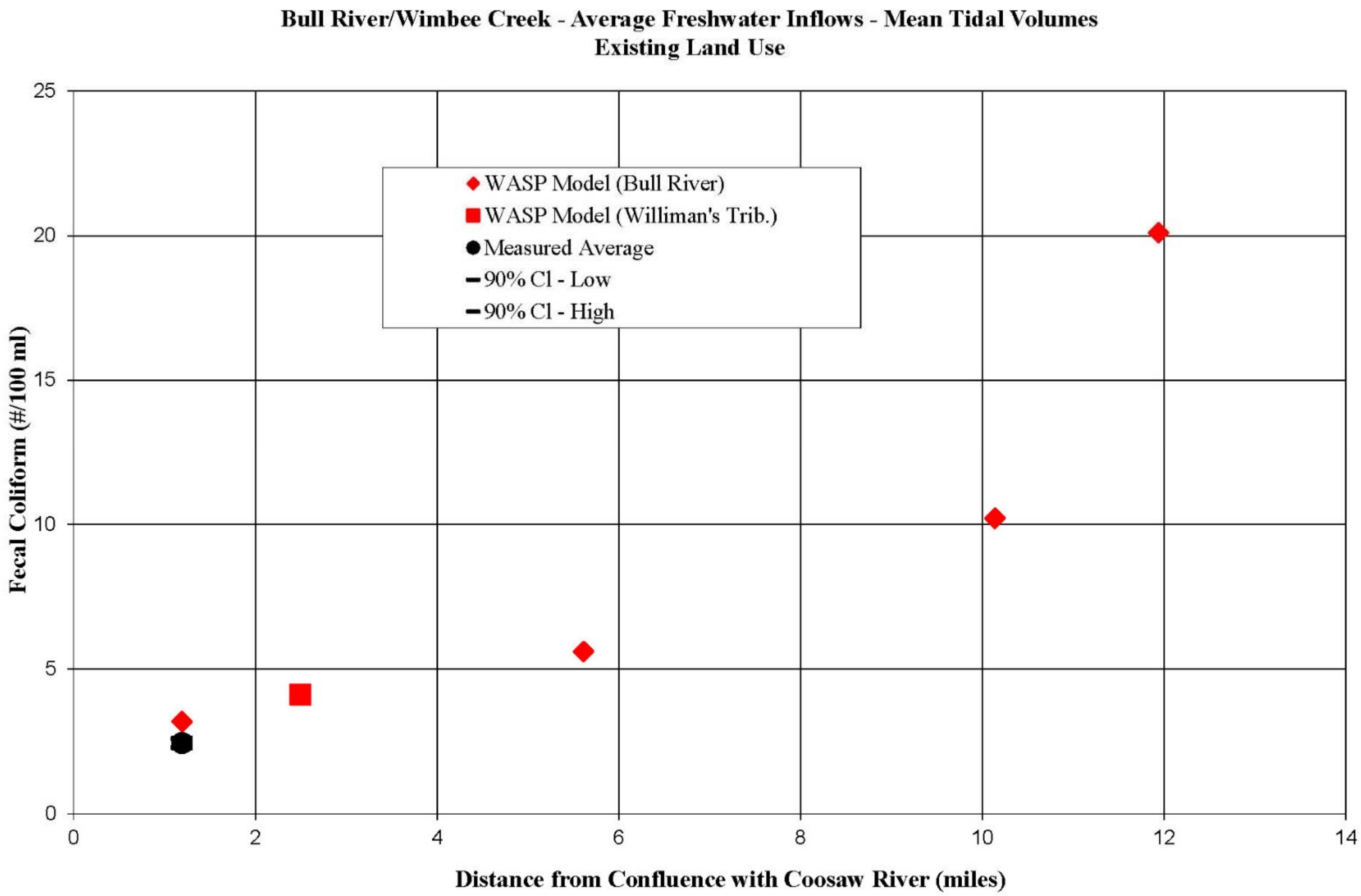

Figure 9-15. Comparison of WASP Model Results with Long-Term Monitoring Data in Bull River/Wimbee Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

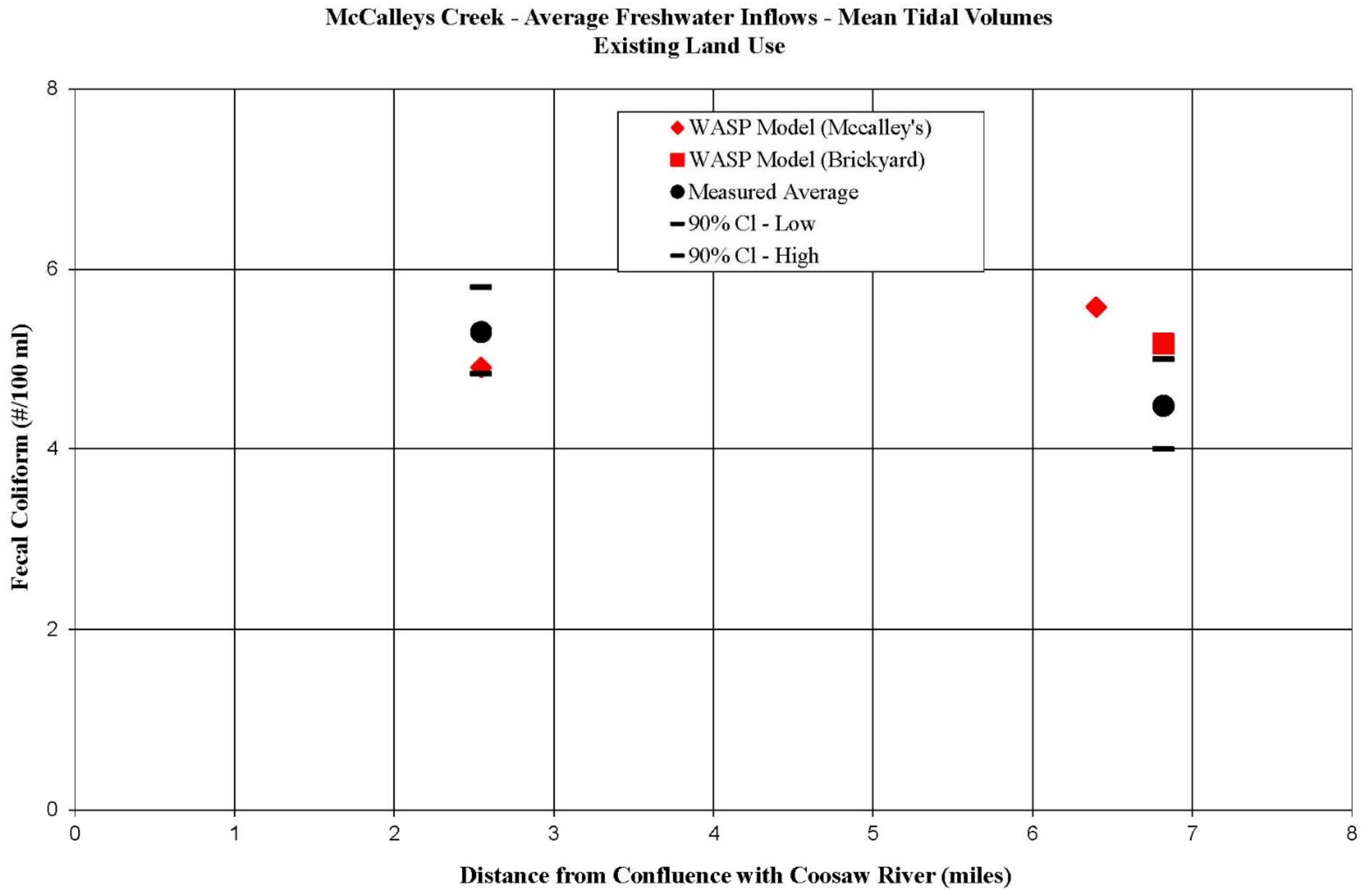

Figure 9-16. Comparison of WASP Model Results with Long-Term Monitoring Data in McCalley's Creek/Brickyard Creek North - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

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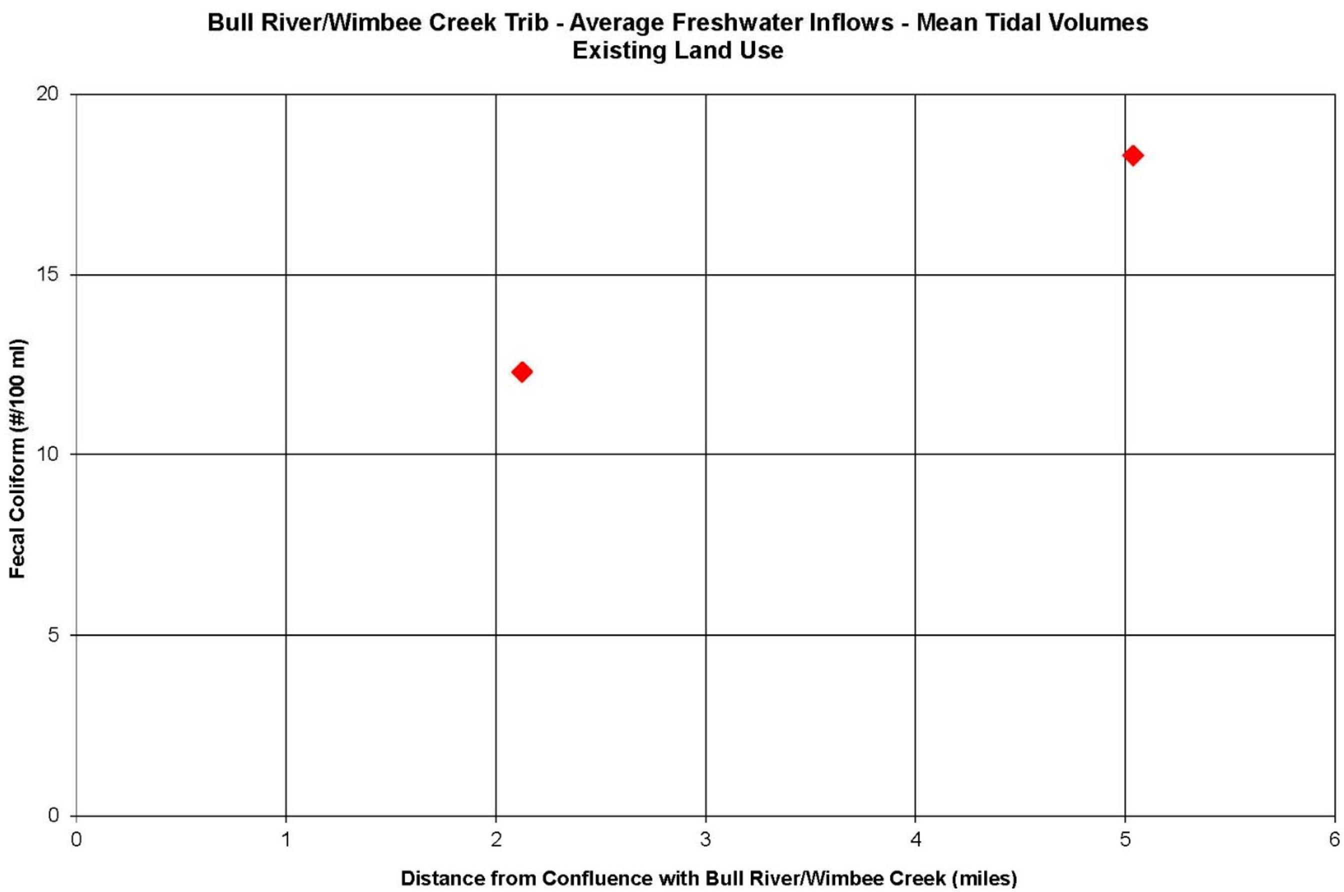

Figure 9-17. Comparison of WASP Model Results with Long-Term Monitoring Data in
Bull River and Wimbee Creek Tributary - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

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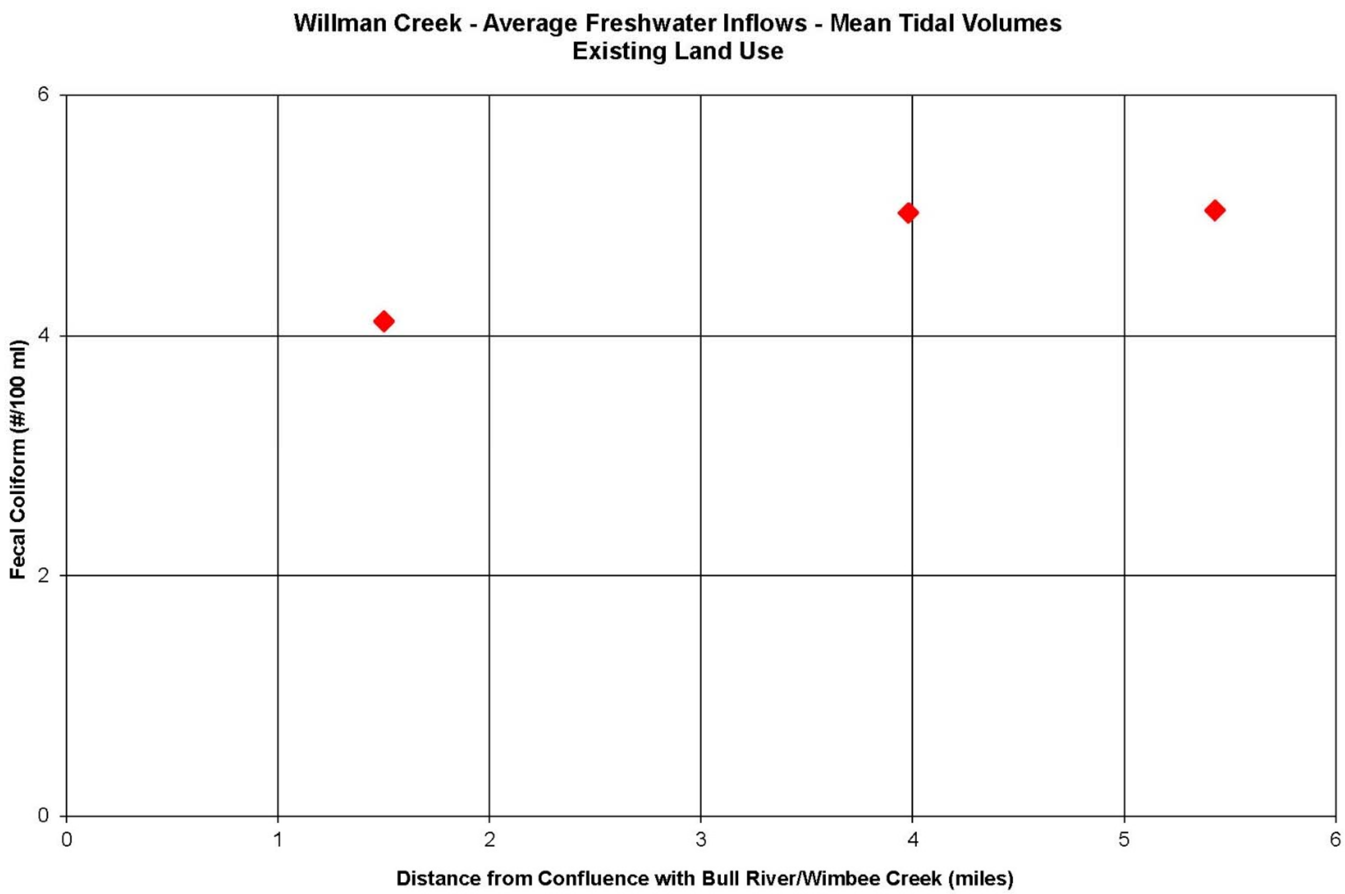

Figure 9-18. Comparison of WASP Model Results with Long-Term Monitoring Data in Willman Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Figure 9-19 is not applicable in the update.

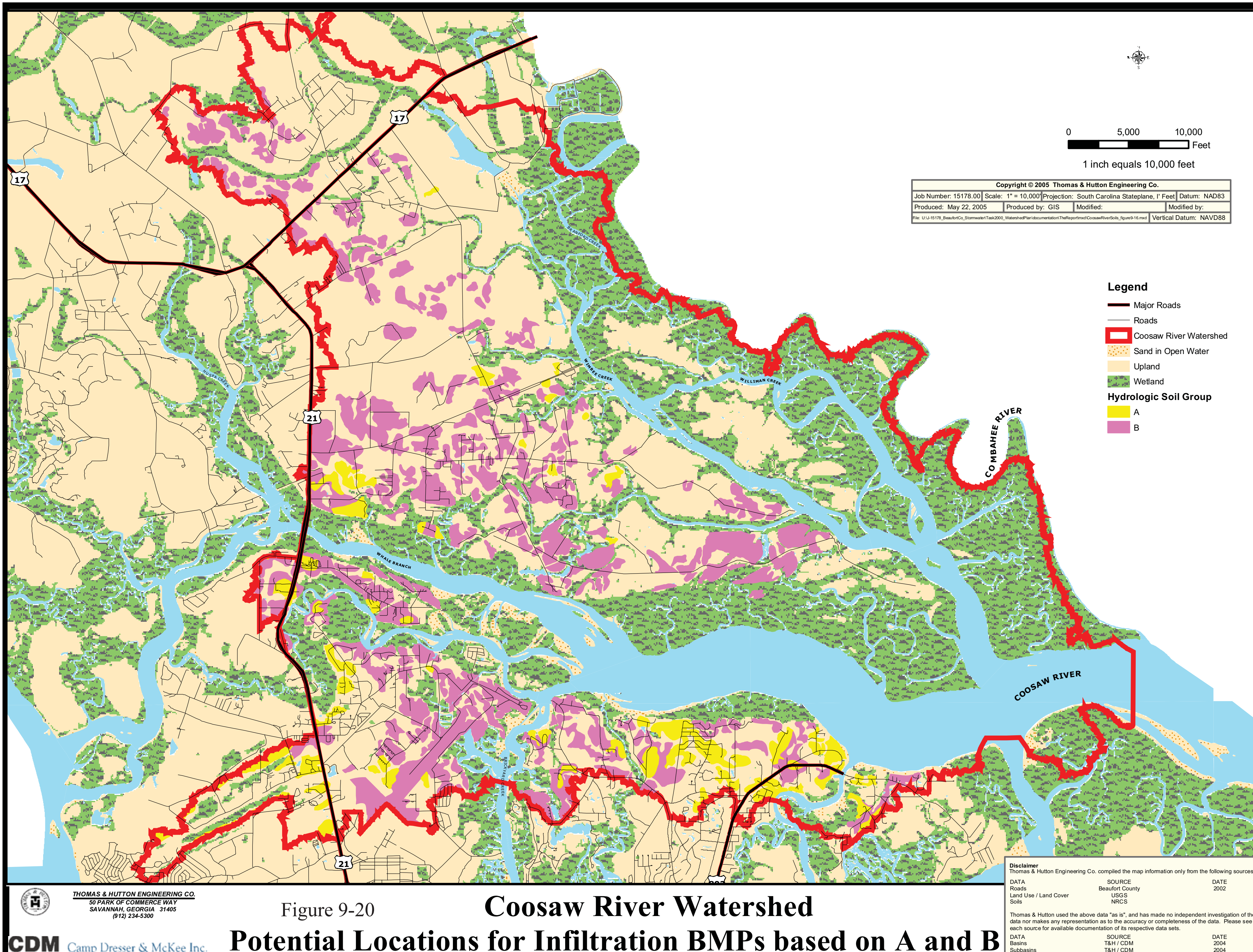

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Figure 9-20

Coosaw River Watershed
Potential Locations for Infiltration BMPs based on A and B

Section 10
Whale Branch West Watershed Analysis

This section describes the physical features of the Whale Branch West watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

10.1 Overview

The Whale Branch West watershed is located north of the Broad River (see **Figure 10-1**). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in Sheldon Township and Port Royal Island that is tributary to the Whale Branch West. Major Whale Branch West tributaries included in the analysis are Middle Creek, Haulover Creek and Huspa Creek.

For the hydrologic and hydraulic analysis of the Primary Stormwater Management System (PSMS), the watershed includes several "hydrologic" basins. These are listed in **Table 10-1**, and presented in **Figure 10-2**. Table 10-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were completed to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into "water quality" basins, and the tidal receiving waters were subdivided into receiving water "segments". These are listed in **Table 10-2**, and presented in **Figure 10-3**. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were completed to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

10.2 Hydrologic and Hydraulic Analysis

CDM and T&H used the Interconnected Pond Routing Model (ICPR), Version 3 for the hydrologic and hydraulic analyses of the PSMS in the Whale Branch West watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were conducted for existing and future land use conditions, with and without alternative management strategies.

The ICPR model is a "link-node" model, representing the PSMS as a series of nodes (stream locations) connected by links (open channels, pipes, culverts). Figures in Appendix H show model schematics of the Whale Branch West PSMS basins, with a separate schematic for each basin.

10.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Whale Branch West basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 10-3 lists the hydrologic parameter values for the Whale Branch West PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development.

Hydraulic summary information for the Whale Branch West PSMS basins is presented in **Table 10-4**. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in **Table 10-5**. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate level of service.

Details regarding specific open channel segments, storage areas, weirs and tide gates are presented in Appendix H.

10.2.2 Model Results

Tables in Appendix H list the peak flow values for the Whale Branch West subbasins. Each table lists peak flows for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak flows are listed by subbasin for various land cover and stormwater management controls, which include the following:

- Undeveloped land
- Existing land use without peak shaving controls
- Existing land use with existing peak shaving controls
- Future land use without peak shaving controls

- Future land use with existing and future peak shaving controls

It should be noted that the tables include values for “uncontrolled” and “controlled” peak flows for the 2-year, 10-year and 25-year design storms. The “uncontrolled” peak flow assumes no peak shaving facilities in the subbasin. In contrast, the “controlled” value accounts for peak shaving facilities in the subbasin.

For existing land use, aerial maps and local information were used to estimate the percentage of existing urban development that is served by peak shaving facilities. The “controlled” peak flow value was then calculated by considering the difference in peak flow between totally undeveloped conditions and existing conditions with no controls. For example, suppose that a subbasin of 100 acres has an undeveloped 2-year peak flow of 20 cfs, and an uncontrolled existing peak flow of 50 cfs, and further suppose that 60 percent of the urban development is controlled by peak shaving facilities. In this case, it is assumed that the existing peak flow is reduced by 60 percent of the difference between undeveloped and developed peak flow (50 – 20 = 30 cfs; 60 percent of 30 cfs = 18 cfs reduction due to peak shaving), and therefore the maximum controlled peak flow will be 32 cfs (50 – 18).

For future land use, the “controlled” peak flow is set equal to the “controlled” peak flow for existing land use, because new development is subject to State and County peak flow regulations. Note, however, that the future condition will still generate more stormwater runoff volume, even though the peak flow is the same. The result is that the peak flow rate will be sustained for a longer period of time under future conditions.

Other tables in Appendix H list the peak water elevation values for model node locations along Whale Branch West PSMS. Each table lists peak stages for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak stages are listed for existing and future land use conditions, with the existing stormwater hydraulic system.

Specific problem areas identified by the modeling are listed in **Table 10-6** and presented in **Figure 10-4**. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

Structural flooding was also considered for the 100-year design storm. In locations where the PSMS evacuation route crossings are overtopped by the 100-year design storm, figures were developed showing the approximate area of inundation upstream of the overtopped road. These figures are presented in Appendix H. In addition, the peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) base flood elevations, and results showed that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA base flood elevations should not be flooded.

Table 10-6 indicates that eight road crossings are being overtopped by the design storm events. Evaluation of solutions to prevent these problems is discussed in the next section of this report.

10.2.3 Management Strategy Alternatives

The problems areas listed in Table 10-6 were evaluated by modifying the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in **Table 10-7**. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were typically used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

For a few locations (e.g., Paige Point Road in Huspa Creek South basin), the proposed solution also included raising the road. This was required to provide a sufficient pipe depth as well as sufficient freeboard between the top of the pipe and the roadway.

10.3 Water Quality Analysis

CDM and T&H used the Watershed Management Model (WMM) and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the Whale Branch West watershed. WMM was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, total nitrogen (total N), total phosphorus (total P), BOD, lead, zinc and total suspended solids (TSS). WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions. The same parameter values were used for evaluation of future conditions, which reflect higher flows and loads from the watershed.

10.3.1 Land Use and BMP Coverage

Table 10-8 presents the existing land use and future land use estimates for the Whale Branch West water quality basins. The existing land use data were gathered from a number of sources, including February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps, plus local knowledge of development completed between February 2002 and June 2003. The future land use map was developed by "filling in" the existing land use map and by replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and Town of Hilton Head Island future land use maps and zoning maps.

Under existing land use conditions, 35 percent of the Whale Branch West watershed area consists of urban systems (e.g., residential, commercial, golf course) and 65 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 6 percent of the watershed.

Under future land use conditions, 44 percent of Whale Branch West watershed area consists of urban systems, and 56 percent consists of natural systems. The major change in land use distribution is the conversion of forest/rural land to low density residential land uses. As a result of projected future development, urban imperviousness increases to about 8 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in **Table 10-9**. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 0 percent of the urban systems in the watershed are served by BMPs. Under future land use conditions, 35 percent of the urban systems are served by BMPs. This increase from existing to future reflects both the increase in urban land use and the 100 percent coverage of the new development with BMPs in accordance with the County BMP Manual.

10.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in **Table 10-10**. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water & Sewer Authority. For future development, areas that are zoned "rural" or "conservation" were assumed to be served by septic tanks, and other areas were assumed to be served by sewer.

Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 96 percent of the urban systems in the watershed are served by septic. Under future land use conditions, 96 percent of the urban systems are served by septic tanks. This reflects the presumption that almost all of the new development in the Whale Branch West watershed will not be sewered.

Based on available data, there are no significant wastewater discharges under existing conditions, and therefore none are expected in the future, as new development will primarily be served by septic tanks.

10.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Whale Branch West water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in **Table 10-11** for existing and future land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

An overall comparison of the WMM modeling results (Table 10-11) indicates that future flows and constituent loads generally increase marginally over their existing counterparts. Specifically, future flow is 2 percent greater than for existing conditions and the increase in loads ranges from 8 percent for BOD to 2 percent for TSS and zinc. The TSS load reduction reflects the fact that BMPs are typically very efficient in removing sediment suspended in stormwater runoff. It should also be noted that the increases for several constituents (e.g., total N, zinc) are limited because direct rainfall on the open water/tidal wetland area provides a significant fraction of the total load to the Whale Branch West.

For individual water quality basins, the greatest changes in flows and loads occur in the Middle Creek 1 and Middle Creek 2 basins. This is because these basins are anticipated to have the greatest amount of future development, and because these basins may also have the smallest fraction of open water and tidal wetland land use. Load increases in these basins are typically 3 to 7 percent, with BOD having the greatest increases (10 to 17 percent) and fecal coliform bacteria showing the smallest load increases (3 percent).

10.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Whale Branch West watershed. The model actually includes Beaufort River, Coosaw River, Whale Branch West and Morgan River watersheds because they are interconnected at several points. Only the Whale Branch West will be discussed in this section. A schematic of the model is presented as **Figure 10-5**.

Existing conditions for bacteria concentrations in the Whale Branch West are presented in **Table 10-12**. For each water quality basin river reach, the table lists the DHEC stations for which the 1990s bacteria data were analyzed, the concentrations calculated in the analysis, and the "level of service" associated with these concentrations (as discussed in Section 2.6.2). As shown in the table, DHEC data were only available in two of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations meet the water quality standards in one of the two segments (Whale Branch West 2), and so that segment has an "A" level of service. The Huspa Creek 1 segment has a "D" level of service.

For informational purposes, **Figure 10-6** presents a map of the level of service based on the monitoring data analysis, compared to the Department of Health and Environmental Control (DHEC) "shellfish classification" (based on the 2002 DHEC reports for shellfish areas 14 and 17). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the level of service, so there may not be a direct relationship between level of service and shellfish classification presented in the map. In general, however, segments with an "A" level of service are expected to have the lowest probability of receiving a "restricted" classification, and segments with a "D" level of service are expected to have the highest probability of receiving a "restricted" classification.

Physical characteristics assigned to the model reaches are presented in **Table 10-1**3. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the "characteristic length" (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the DHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. **Tables 10-14** and **10-15** show the values used in the existing and future condition models.

A review of Table 10-14 shows that there is typically little change in flow or concentration between existing and future land use. For flow, this is because much of the flow to the tidal river segments comes from direct rainfall on the open water and tidal wetlands, as opposed to stormwater runoff and baseflow, and some of the basins have very little change in land use from existing to future conditions. Concentrations remain relatively constant because of the substantial amount of open water/tidal wetland area and the relatively limited development in some basins, as well as the BMPs for new development, which are assumed to have a high level of treatment efficiency.

Table 10-15 shows the net advective flows between segments, which also do not change substantially from existing to future land use. In both cases, the hydrodynamic model (SWMM) indicates that there is a substantial net flow from the Broad River into Whale Branch 1, and the flow continues "upstream" until discharging to the Coosaw River.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. In general, a loss rate of 1.0/day was assumed initially, and values were then adjusted to achieve a better match between modeled and measured data. The final calibration values will be discussed below.

Figure 10-7 is a graph showing a comparison between measured and modeled salinity data along the Whale Branch West main stem. The figure shows that the salinity data calculated by the model is very close to the average measured value.

The measured and modeled salinity in Huspa Creek is compared in **Figure 10-8**. Again, the figure shows that the modeled salinity is very close to the measured mean value.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration for Whale Branch West and Huspa Creek are presented in **Figures 10-9** and **10-10**, respectively. In both cases, the modeled bacteria values are slightly lower than the measured geomean values, but well within the 90 percent confidence intervals for the measured geomean values.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in **Table 10-16**. The loss rates ranged from 0.7/day to 1.0/day. The lowest values are applied at the downstream end of Whale Branch West, and the higher values are applied to the tributaries. This makes sense if it is presumed that bacteria loss is in part due to light mortality, because the water depths are greater in the Whale Branch main stem, and therefore light would be less of a factor relative to the shallower tributary reaches.

After the model was applied for existing conditions, it was the applied for future conditions. The physical characteristics and first-order loss rate from the existing land use model were kept the same in the future land use model. The only changes were the net advective flows and the bacteria loads.

The bacteria concentrations calculated under future land use conditions are presented in Table 10-16 as well. A comparison of concentrations under existing and future land use conditions shows little difference. According to the model, all river reaches will have the same level of service in the future as they do under existing conditions except Middle Creek 1, which goes from a “B” to a “C” level of service. However, the future bacteria concentration (8.7/100 ml) is equal to the threshold value between the “A” and “B” level of service (8.7/100 ml). It should also be noted that the model results in Middle Creek are not calibrated to measured data because there are no bacteria monitoring data for Middle Creek.

In order to estimate the degree to which stormwater management measures are expected to affect instream bacteria concentrations, two sensitivity runs were conducted. The first was run for the existing land use condition, and represents a “best-case” scenario in which all existing development is controlled by BMPs. The second was run for the future land use condition, and represents a “worst-case” condition in which no development is served by BMPs. Analyzing the results of these scenarios indicate the benefits of retrofitting existing development with BMPs, and the potential degradation of river segments if BMPs fail.

The results of the analysis are presented in **Table 10-17**. This table is similar to Table 10-16, in this case showing water quality basin segment fecal coliform concentrations for the “best case” and “worst case” analyses. Segments that show change (e.g., better LOS for the “best case” or degraded LOS for the “worst case”) are highlighted.

A review of the “best-case” scenario indicates that four model segments show improvement in the existing level of service. These include Middle Creek 1 and Middle Creek 2, Haulover Creek 2, and Huspa Creek 1. The Middle Creek 1 and Haulover Creek 2 segments go from a “B” to an “A” level of service, and the Middle Creek 2 and Huspa Creek 1 segments go from a “D” to a “C” level of service. Note that the improvement in Huspa Creek 1 assumes 100 percent BMP coverage in that water quality basin as well as upstream basin Huspa Creek 2, plus all the other basins in the watershed, which results in some improvement in the segment downstream of

Huspa Creek 1 (Whale Branch 3). Consequently, retrofitting existing development only in Huspa Creek 1 would be unlikely to produce a change in the existing level of service in that segment.

A review of the "worst-case" scenario indicates that three model segment show degradation in the future level of service when no BMPs are assumed. These are Whale Branch West 3, Middle Creek 1, and Haulover Creek 1. Whale Branch West 3 and Haulover Creek 1 drop from an "A" to a "B" level of service, though in both cases the "worst-case" concentration (7.3/100 ml in Whale Branch West 3, 7.1/100 ml for Haulover Creek 1) is very close to the threshold for the "A" level of service (7/100 ml). Middle Creek 1 drops from a "C" to a "D" level of service, though again the "worst case" concentration (10.1/100 ml) is very close to the threshold for the "D" level of service (10/100 ml).

Based on water quality sampling data and model results, the following recommendations are made:

- Request that DHEC add bacteria sampling stations in the water quality basins Haulover Creek 1 and Middle Creek 1 so that the model results can be validated

10.3.5 Management Strategy Alternatives

The results of the water quality analysis suggest that the limited amount of future development in the watershed, combined with the effectiveness of required BMPs in reducing bacteria loads from new development, will generally maintain the existing level of service in the watershed reaches. Consequently, no actions are recommended other than additional monitoring to determine if the Middle Creek and Haulover Creek tributaries are meeting the bacteria water quality standards.

Elements of the water quality management plan for the Whale Branch West watershed are presented in **Figure 10-11**. Sampling stations shown in the figure include existing DHEC sites, plus additional requested DHEC open water sampling sites.

For informational purposes, the areas with "A" and "B" type soils are presented in **Figure 10-12**. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

10.4 Planning Level Cost Estimates for Management Alternatives

Table 10-18 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Whale Branch West watershed. As shown in the table, the eight projects are estimated to have a total cost of $1.2 million in December 2004 dollars. Details of the cost estimate for each project are shown in Appendix H.

The prioritization of these projects, and projects identified for other watersheds, is discussed in Section 16 of this report.

TABLE 10-1
HYDROLOGIC BASINS
WHALE BRANCH WEST WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Brewton West	1,356	4	339
Clarendon	243	1	243
Gardens Corner North	618	2	309
Gardens Corner South	669	2	334
Grays Hill North	363	1	363
Haulover Creek East	622	2	311
Huspa Creek East	334	1	334
Huspa Creek North	402	1	402
Huspa Creek South	246	1	246
Huspa Creek West	309	1	309
Laurel Bay North	320	1	320
Scotts Neck East	268	1	268
Scotts Neck South	520	2	260
Sheldon North	260	1	260
Whale Branch East	311	1	311
Whale Branch South	578	1	578
TOTAL	7,419	23	323

TABLE 10-2
WATER QUALITY BASINS
WHALE BRANCH WEST WATERSHED

Basin Name	Tributary Area (acres)
Whale Branch West 1	4,151
Whale Branch West 2	1,543
Whale Branch West 3	1,234
Haulover Creek 1	1,807
Haulover Creek 2	602
Middle Creek 1	888
Middle Creek 2	1,382
Huspa Creek 1	7,617
Huspa Creek 2	5,157
TOTAL	24,379

TABLE 10-3
HYDROLOGIC SUBBASIN CHARACTERISTICS
WHALE BRANCH WEST WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Brewton West Basin					
BW_M1	177	86	96	86	95
BW_M2	226	91	76	91	76
BW_M3	536	91	86	91	86
BW_T1	416	93	94	93	94
Clarendon Basin					
CN_M1	243	62	275	70	221
Gardens Corner North Basin					
GCN_M1	206	74	99	79	86
GCN_M2	412	76	157	77	154
Gardens Corner South Basin					
GCS_M1	386	85	115	86	111
GCS_M2	283	86	125	88	117
Grays Hill North Basin					
GHN_M1	363	65	339	73	275
Haulover Creek East Basin					
HRCE_M1	195	76	115	76	114
HRCE_M2	427	78	122	78	122
Huspa Creek East Basin					
HACE_M1	334	82	106	82	104
Huspa Creek North Basin					
HACN_M1	402	77	166	79	159
Huspa Creek South Basin					
HACS_M1	246	82	117	83	113
Huspa Creek West Basin					
HACW_M1	309	83	104	84	102
Laurel Bay North Basin					
LBN_M1	320	72	142	72	142
Scotts Neck East Basin					
SNE_M1	268	81	105	82	104
Scotts Neck South Basin					
SNS_M1	285	79	118	79	118
SNS_M2	236	81	105	81	105
Sheldon North Basin					
SN_M1	260	87	92	88	87
Whale Branch East Basin					
WBE_M1	311	64	199	72	162
Whale Branch South Baisn					
WBS_M1	578	53	280	65	206
Average	323	78	141	81	128

TABLE 10-4
HYDRAULIC DATA SUMMARY
WHALE BRANCH WEST WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Brewton West	9	11,959	3	4	0	3	2	0
Clarendon	1	434	0	0	0	0	0	0
Gardens Corner North	3	3,512	1	2	0	0	1	0
Gardens Corner South	3	2,205	1	1	0	1	1	0
Grays Hill North	5	3,598	3	3	0	1	3	0
Haulover Creek East	2	2,068	0	0	0	0	0	0
Huspa Creek East	1	1,228	0	0	0	0	0	0
Huspa Creek North	3	3,116	1	1	0	1	1	0
Huspa Creek South	1	367	2	2	0	1	2	0
Huspa Creek West	2	1,219	1	1	0	1	1	0
Laurel Bay North	1	1,434	0	0	0	0	0	0
Scotts Neck East	1	932	1	1	0	1	1	0
Scotts Neck South	5	2,855	2	2	0	0	2	0
Sheldon North	1	617	0	0	0	0	0	0
Whale Branch East	1	870	0	0	0	0	0	0
Whale Branch South	3	2,095	0	0	0	1	0	0
TOTAL	42	38,509	15	17	0	10	14	0

TABLE 10-5
CULVERT DATA FOR HYDROLOGIC BASINS
WHALE BRANCH WEST WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Brewton West Basin						
Cotton Hall Road	BW_M-4	36"x36"	70	1.5	10.8	25
Old Sheldon Church Road	BW_M-10A	24"x24"	60	8.7	12.5	25
	10B	24"x24"	60	8.5		
Gardens Corner North Basin						
Charleston Highway (US Hwy 17)	GCN_M-1A	72"x72"	200	2.5	9.5	100
	1B	72"x72"	200	2.4		
Gardens Corner South Basin						
Trask Parkway (US Hwy 17)	GCS_M-1	48"x48"	160	-0.9	9.0	100
Grays Hill North Basin						
Jonesfield Road	GHN_M-2	24"x24"	60	28.4	32.5	25
Clarendon Road	GHN_M-8	18"x18"	60	35.2	38.2	25
Huspa Creek North Basin						
Old Sheldon Church Road	HACN_M-1	36"x36"	40	4.9	11.5	25
Huspa Creek South Basin						
Paige Point Road	HACS_M-3	30"x30"	40	5.6	9.5	25
Huspa Creek West Basin						
Huspah Court South	HACW_M-1	48"x48"	55	-1.5	8.0	25
Scotts Neck East Basin						
Water Park Road	SNE_M-1	48"x24"	30	3.5	7.7	25

TABLE 10-6
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
WHALE BRANCH WEST WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Brewton West Basin					
Old Sheldon Church Road	BW_M-76	12.5	25	12.8	12.8
Gardens Corner South Basin					
Trask Parkway (US Hwy 17)	GCS_M-5	9.0	100	9.3	9.4
Grays Hill North Basin					
Jonesfield Road	GHN_M-25	32.5	25	32.7	32.8
Clarendon Road	GHN_M-48	38.2	25	38.4	38.5
Huspa Creek North Basin					
Old Sheldon Church Road	HACN_M-15	11.5	25	11.9	11.9
Huspa Creek South Basin					
Paige Point Road	HACS_M-9	9.5	25	10.1	10.1
Huspa Creek West Basin					
Huspah Court South	HACW_M-7	8.0	25	8.8	8.8
Scotts Neck East Basin					
Water Park Road	SNE_M-4	7.7	25	7.9	7.9

TABLE 10-7
RECOMMENDED CULVERT IMPROVEMENTS
WHALE BRANCH WEST WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Brewton West Basin			
Old Sheldon Church Road	BW_M-10A 10B	24"x24" 24"x24"	Replace culverts with two 6 ft by 4 ft box culverts
Gardens Corner South Basin			
Trask Parkway (US Hwy 17)	GCS_M-1	48"x48"	Replace culvert with one 10 ft by 6 ft box culvert
Grays Hill North Basin			
Jonesfield Road	GHN_M-2	24"x24"	Replace culvert with one 8 ft by 4 ft box culverts
Clarendon Road	GHN_M-8	18"x18"	Replace culvert with four 30" pipes
Huspa Creek North Basin			
Old Sheldon Church Road	HACN_M-1	36"x36"	Replace culvert with one 7 ft by 4 ft box culvert
Huspa Creek South Basin			
Paige Point Road	HACS_M-3	30"x30"	Raise road from elevation 9.5 ft to elevation 11.0 ft NAVD (length of 690 ft), Replace culvert with two 6 ft by 4 ft box culverts
Huspa Creek West Basin			
Huspah Court South	HACW_M-1	48"x48"	Raise road from elevation 8.0 ft to elevation 9.5 ft NAVD (length of 460 ft), Replace culvert with one 12 ft by 6 ft box culvert
Scotts Neck East Basin			
Water Park Road	SNE_M-1	48"x24"	Replace culvert with four 36" pipes

TABLE 10-8

WATER QUALITY BASIN LAND USE DISTRIBUTION

WHALE BRANCH WEST WATERSHED

Existing Land Use Type	Haulover Creek 1 (acres)	Haulover Creek 2 (acres)	Middle Creek 1 (acres)	Middle Creek 2 (acres)	Huspa Creek 1 (acres)	Huspa Creek 2 (acres)	TOTAL (acres)
Agricultural/Pasture	30	0	0	0	185	225	509
Commercial	0	0	0	14	166	5	185
Forest/Rural Open	82	71	192	322	870	585	3,586
Golf Course	0	0	0	0	0	0	0
High Density Residential	0	0	0	6	1	0	13
Industrial	19	19	21	107	257	246	778
Institutional	0	0	0	0	13	10	23
Low Density Residential	652	237	14	87	2,468	1,317	5,022
Medium Density Residential	0	0	187	255	0	0	952
Open Water/Tidal	828	254	331	307	2,209	1,183	8,567
Silviculture	0	0	0	0	0	0	0
Urban Open	36	0	12	212	595	341	1,496
Wetland/Water	159	21	131	73	854	1,243	3,248
TOTAL	1,807	602	888	1,382	7,617	5,157	24,379
Urban Imperviousness (%)	4%	6%	7%	12%	8%	6%	6%

Future Land Use Type	Haulover Creek 1 (acres)	Haulover Creek 2 (acres)	Middle Creek 1 (acres)	Middle Creek 2 (acres)	Huspa Creek 1 (acres)	Huspa Creek 2 (acres)	TOTAL (acres)
Agricultural/Pasture	55	71	0	0	214	256	596
Commercial	0	0	0	25	196	5	226
Forest/Rural Open	27	0	0	0	280	168	1,166
Golf Course	0	0	0	0	0	0	0
High Density Residential	0	0	0	6	1	0	14
Industrial	19	19	22	110	264	247	791
Institutional	0	0	0	0	16	24	39
Low Density Residential	696	237	212	592	3,358	1,916	8,266
Medium Density Residential	0	0	193	270	130	111	1,340
Open Water/Tidal	828	253	331	307	2,209	1,183	8,565
Silviculture	0	0	0	0	0	0	0
Urban Open	22	0	0	1	97	5	130
Wetland/Water	159	21	130	72	854	1,242	3,246
TOTAL	1,807	602	888	1,382	7,617	5,157	24,379
Urban Imperviousness (%)	5%	6%	10%	17%	10%	8%	8%

TABLE 10-9

WATER QUALITY BASIN BMP COVERAGE

WHALE BRANCH WEST WATERSHED

Existing Land Use Type	Whale Branch West 1 (%)	Whale Branch West 2 (%)	Whale Branch West 3 (%)	Haulover Creek 1 (%)	Haulover Creek 2 (%)	Middle Creek 1 (%)	Middle Creek 2 (%)	Huspa Creek 1 (%)	Huspa Creek 2 (%)	TOTAL
Commercial	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
Golf Course	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
High Density Residential	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
Industrial	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
Institutional	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
Low Density Residential	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
Medium Density Residential	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
TOTAL	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%

Future Land Use Type	Whale Branch West 1 (%)	Whale Branch West 2 (%)	Whale Branch West 3 (%)	Haulover Creek 1 (%)	Haulover Creek 2 (%)	Middle Creek 1 (%)	Middle Creek 2 (%)	Huspa Creek 1 (%)	Huspa Creek 2 (%)	TOTAL
Commercial	0%	0%	0	0%	0%	0%	45%	15%	0%	18%
Golf Course	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
High Density Residential	0%	0%	0	0%	0%	0%	0%	0%	0%	0%
Industrial	1%	0%	0	0%	0%	1%	1%	3%	1%	1%
Institutional	0%	0%	0	0%	0%	0%	0%	15%	58%	41%
Low Density Residential	100%	18%	1	6%	0%	93%	85%	27%	31%	39%
Medium Density Residential	24%	1%	0	100%	0%	3%	6%	100%	100%	29%
TOTAL	70%	9%	41%	6%	0%	48%	53%	27%	31%	35%

TABLE 10-10

WATER QUALITY BASIN SEPTIC TANK COVERAGE

WHALE BRANCH WEST WATERSHED

Existing Land Use Type	Whale Branch West 1 (%)	Whale Branch West 2 (%)	Whale Branch West 3 (%)	Haulover Creek 1 (%)	Haulover Creek 2 (%)	Middle Creek 1 (%)	Middle Creek 2 (%)	Huspa Creek 1 (%)	Huspa Creek 2 (%)	TOTAL (%)
Commercial	0%	0%	0%	0%	0%	0%	100%	100%	100%	100%
High Density Residential	0%	0%	100%	0%	0%	0%	100%	100%	0%	100%
Industrial	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Institutional	0%	0%	0%	0%	0%	0%	0%	100%	100%	100%
Low Density Residential	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Medium Density Residential	30%	100%	100%	0%	0%	100%	100%	0%	0%	73%
TOTAL	34%	100%	100%	100%	100%	100%	100%	100%	100%	96%

Future Land Use Type	Whale Branch West 1 (%)	Whale Branch West 2 (%)	Whale Branch West 3 (%)	Haulover Creek 1 (%)	Haulover Creek 2 (%)	Middle Creek 1 (%)	Middle Creek 2 (%)	Huspa Creek 1 (%)	Huspa Creek 2 (%)	TOTAL (%)
Commercial	0%	0%	0%	0%	0%	0%	65%	100%	100%	96%
High Density Residential	0%	0%	99%	0%	0%	0%	100%	100%	0%	100%
Industrial	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Institutional	0%	0%	0%	0%	0%	0%	0%	100%	100%	100%
Low Density Residential	100%	100%	89%	100%	100%	100%	100%	100%	100%	100%
Medium Density Residential	27%	100%	100%	100%	0%	100%	96%	100%	100%	73%
TOTAL	73%	100%	92%	100%	100%	100%	98%	100%	100%	96%

TABLE 10-11

AVERAGE ANNUAL LOADS FOR WHALE BRANCH WEST WATERSHED WATER QUALITY BASINS

EXISTING LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Whale Branch West 1	4,151	9,569	85,533	413,000	4,339	34,762	140	2,764	9.30E+14
Whale Branch West 2	1,542	4,194	38,017	169,000	1,990	16,078	67	1,355	4.91E+14
Whale Branch West 3	1,233	3,375	34,994	184,770	1,767	13,893	62	1,150	4.80E+14
Haulover Creek 1	1,806	4,289	45,644	260,000	2,279	17,108	79	1,315	6.16E+14
Haulover Creek 2	601	1,380	15,788	98,095	759	5,825	27	421	2.34E+14
Middle Creek 1	888	2,005	22,012	145,000	1,113	9,063	36	547	4.07E+14
Middle Creek 2	1,382	2,671	35,574	293,000	1,689	14,276	55	638	7.67E+14
Huspa Creek 1	7,617	15,490	185,000	1,350,000	8,886	69,573	293	3,995	2.99E+15
Huspa Creek 2	5,156	9,905	110,000	841,000	5,500	43,244	162	2,168	1.74E+15
TOTAL	24,376	52,878	572,562	3,753,865	28,322	223,822	921	14,353	8.66E+15

FUTURE LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Whale Branch West 1	4,152	9,830	95,914	429,000	4,559	36,341	148	2,837	9.89E+14
Whale Branch West 2	1,543	4,204	38,278	170,000	1,970	16,122	67	1,357	4.95E+14
Whale Branch West 3	1,235	3,429	37,055	187,487	1,777	14,664	64	1,166	5.49E+14
Haulover Creek 1	1,806	4,299	46,147	261,000	2,323	17,216	79	1,318	6.18E+14
Haulover Creek 2	602	1,380	15,970	101,000	863	5,939	27	421	2.34E+14
Middle Creek 1	888	2,057	24,181	148,000	1,160	9,428	38	563	4.21E+14
Middle Creek 2	1,382	2,830	41,664	303,000	1,811	15,237	60	685	7.92E+14
Huspa Creek 1	7,617	15,866	198,000	1,380,000	9,226	72,486	302	4,095	3.09E+15
Huspa Creek 2	5,157	10,133	119,000	855,000	5,742	45,106	168	2,229	1.81E+15
TOTAL	24,381	54,028	616,209	3,834,487	29,431	232,539	953	14,671	9.00E+15
Percent Increase over Existing Land Use		2%	8%	2%	4%	4%	3%	2%	4%

TABLE 10-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS
WHALE BRANCH WEST WATERSHED

Water Quality Basin ID	DHEC Station(s)	Fecal Coliform Concentrations				Level of Service
		Long-Term Average		Maximum 36-Sample Values		
		Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)	
Whale Branch West 1	None	NA	NA	NA	NA	NA
Whale Branch West 2	17- 21	6.3	33	8.7	33	A
Whale Branch West 3	None	NA	NA	NA	NA	NA
Middle Creek 1	None	NA	NA	NA	NA	NA
Middle Creek 2	None	NA	NA	NA	NA	NA
Haulover Creek 1	None	NA	NA	NA	NA	NA
Haulover Creek 2	None	NA	NA	NA	NA	NA
Huspa Creek 1	14-14, 14-18	13.1	49	15.7	69	D
Huspa Creek 2	None	NA	NA	NA	NA	NA

TABLE 10-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS
WHALE BRANCH WEST WATERSHED

Water Quality Basin ID	South WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Whale Branch West 1	41	1.32E+07	Broad River	6,375	4,281	150
Whale Branch West 2	42	5.64E+06	Whale Branch West 1	1,933	3,701	150
Whale Branch West 3	43	3.92E+06	Whale Branch West 2	1,152	3,074	150
			Coosaw River 4	762	2,446	150
Middle Creek 1	44	1.32E+06	Whale Branch West 2	425	2,446	150
Middle Creek 2	45	3.13E+05	Middle Creek 1	291	2,221	150
Haulover Creek 1	46	2.86E+06	Whale Branch West 2	432	3,025	150
Haulover Creek 2	47	5.14E+05	Haulover Creek 1	380	2,253	150
Huspa Creek 1	48	6.18E+06	Whale Branch West 3	490	6,212	450
Huspa Creek 2	49	8.18E+05	Huspa Creek 1	488	4,570	150

TABLE 10-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMM
FOR WHALE BRANCH WEST WATER QUALITY BASINS

Water Quality Basin ID	North WASP Segment	EXISTING LAND USE		FUTURE LAND USE	
		Flow (cfs)	Fecal Coliform (#/100 ml)	Flow (cfs)	Fecal Coliform (#/100 ml)
Whale Branch West 1	41	13.2	1,194	13.6	1,215
Whale Branch West 2	42	5.8	1,334	5.8	1,335
Whale Branch West 3	43	4.7	1,400	4.7	1,420
Middle Creek 1	44	2.8	1,381	2.8	1,402
Middle Creek 2	45	3.7	1,382	3.9	1,407
Haulover Creek 1	46	5.9	1,375	5.9	1,379
Haulover Creek 2	47	1.9	1,405	1.9	1,407
Huspa Creek 1	48	21.4	1,289	21.9	1,346
Huspa Creek 2	49	13.7	1,206	14.0	1,225

TABLE 10-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES
WHALE BRANCH WEST WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs)	
		Existing	Future
Broad River	Whale Branch West 1	1,595	1,593
Whale Branch West 1	Whale Branch West 2	1,608	1,606
Whale Branch West 2	Whale Branch West 3	1,628	1,626
Whale Branch West 3	Coosaw River 4	1,668	1,667
Middle Creek 1	Whale Branch West 2	6.5	6.8
Middle Creek 2	Middle Creek 1	3.7	3.9
Haulover Creek 1	Whale Branch West 2	7.8	7.8
Haulover Creek 2	Haulover Creek 1	1.9	1.9
Huspa Creek 1	Whale Branch West 3	35	36
Huspa Creek 2	Huspa Creek 1	14	14

TABLE 10-16

FECAL COLIFORM MODELING RESULTS
WHALE BRANCH WEST WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)		Modeled Level of Service	
		Existing	Future	Existing	Future
Whale Branch West 1	0.7	4.4	4.5	A	A
Whale Branch West 2	0.7	5.5	5.6	A	A
Whale Branch West 3	1.0	6.5	6.7	A	A
Middle Creek 1	1.0	8.4	8.7	B	C
Middle Creek 2	1.0	13.2	14.0	D	D
Haulover Creek 1	1.0	6.9	6.9	A	A
Haulover Creek 2	1.0	8.0	8.0	B	B
Huspa Creek 1	1.0	11.5	12.1	D	D
Huspa Creek 2	1.0	25.2	26.3	D	D

NOTE: Water quality basins with lower LOS in future are highlighted.

TABLE 10-17

SENSITIVITY ANALYSIS RESULTS
WHALE BRANCH WEST WATERSHED

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)		Modeled Level of Service	
		Best Case	Worst Case	Best Case	Worst Case
Whale Branch West 1	0.7	4.2	4.7	A	A
Whale Branch West 2	0.7	4.8	5.9	A	A
Whale Branch West 3	1.0	5.4	7.3	A	B
Middle Creek 1	1.0	6.5	10.1	A	D
Middle Creek 2	1.0	9.4	17.1	C	D
Haulover Creek 1	1.0	5.7	7.1	A	B
Haulover Creek 2	1.0	6.4	8.2	A	B
Huspa Creek 1	1.0	8.9	13.4	C	D
Huspa Creek 2	1.0	19.2	29.0	D	D

NOTES:
1. Best case represents existing land use with wet detention BMPs serving all existing development.
2. Worst case represents future land use with no BMPs.
3. Water quality segments that show change from base model results (e.g., improved LOS for best case or degraded LOS for worst case) are highlighted.

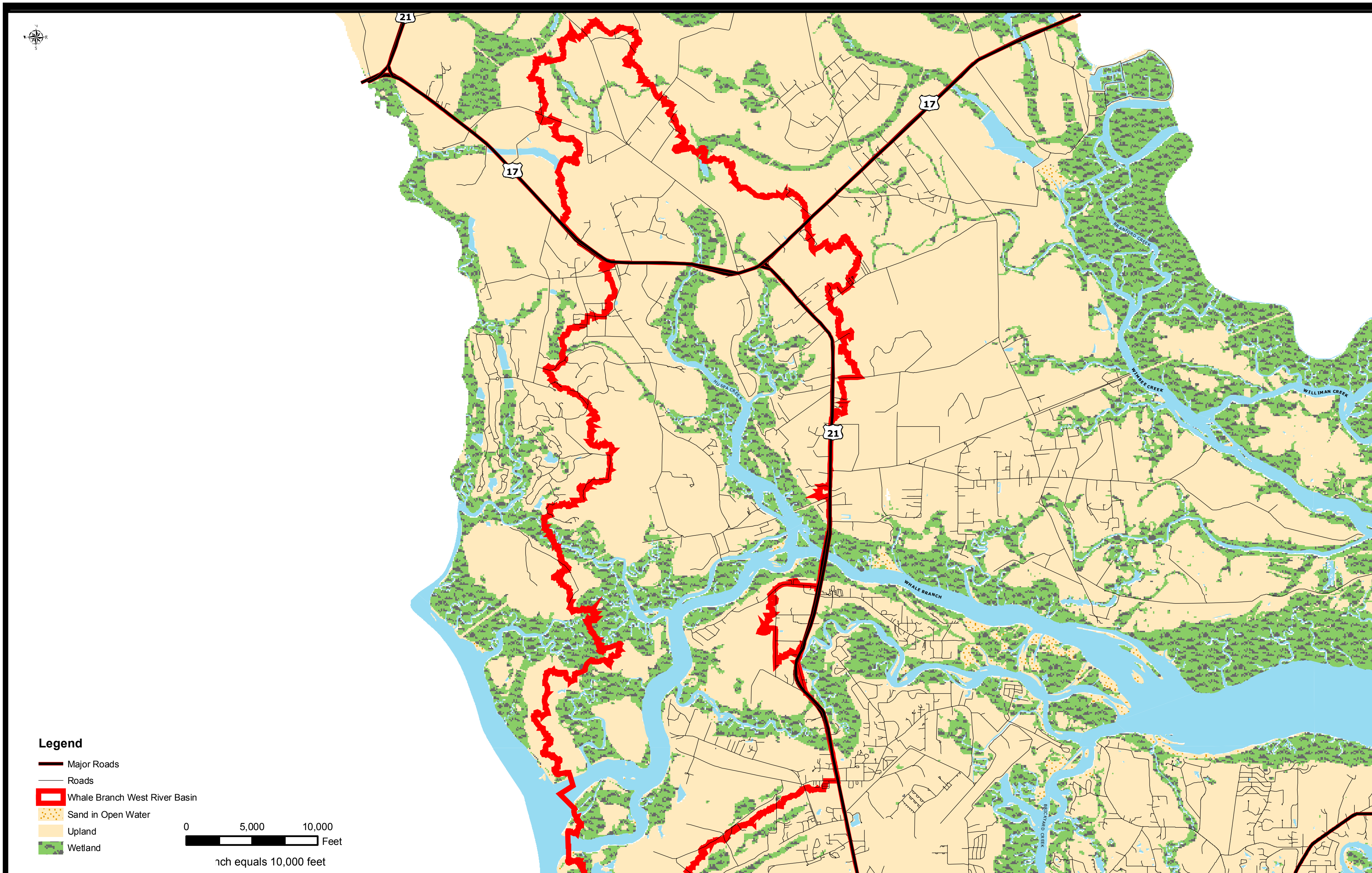
TABLE 10-18

PLANNING LEVEL COST ESTIMATES FOR
WHALE BRANCH WEST WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
BW_M-10	Road overtopping at Old Sheldon Church Road Replace existing 2 - 24" CMP with 2 - 6'x4' box culverts	$121,000
GCS_M-1	Road overtopping at Trask Parkway Replace existing 1 - 48" RCP with 1 - 10'x6' box culvert	$309,000
GHN_M-2	Road overtopping at Jonesfield Road Replace existing 1 - 24" CMP with 1 - 8'x4' box culvert	$90,000
GHN_M-8	Road overtopping at Clarendon Road Replace existing 1 - 18" RCP with 4 - 30" RCP	$38,000
HACN_M-1	Road overtopping at Old Sheldon Church Road Replace existing 1 - 36" RCP with 1 - 7'x4' box culvert	$70,000
HACS_M-3	Road overtopping at Paige Point Road Replace existing 1 - 30" RCP with 2 - 6'x4' box culverts Raise road 1.5 ft (length of 690 ft)	$284,000
HACW_M-1	Road overtopping at Huspah Court South Raise road 1.5 feet (length of 460 ft) Replace existing 1 - 48" RCP with 1 - 10'x5' box culvert	$255,000
SNE_M-1	Road overtopping at Water Park Road Replace existing 1 - 48"x24" box culvert with 6 - 36" RCP	$34,000
	TOTAL	$1,201,000

Costs are in December 2004 dollars.

See Appendix H for basis of cost estimates.



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Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\WhaleBranchWestWatershed_figure			Vertical Datum: NAVD88

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Whale Branch West Watershed

Figure 10-1

Disclaimer
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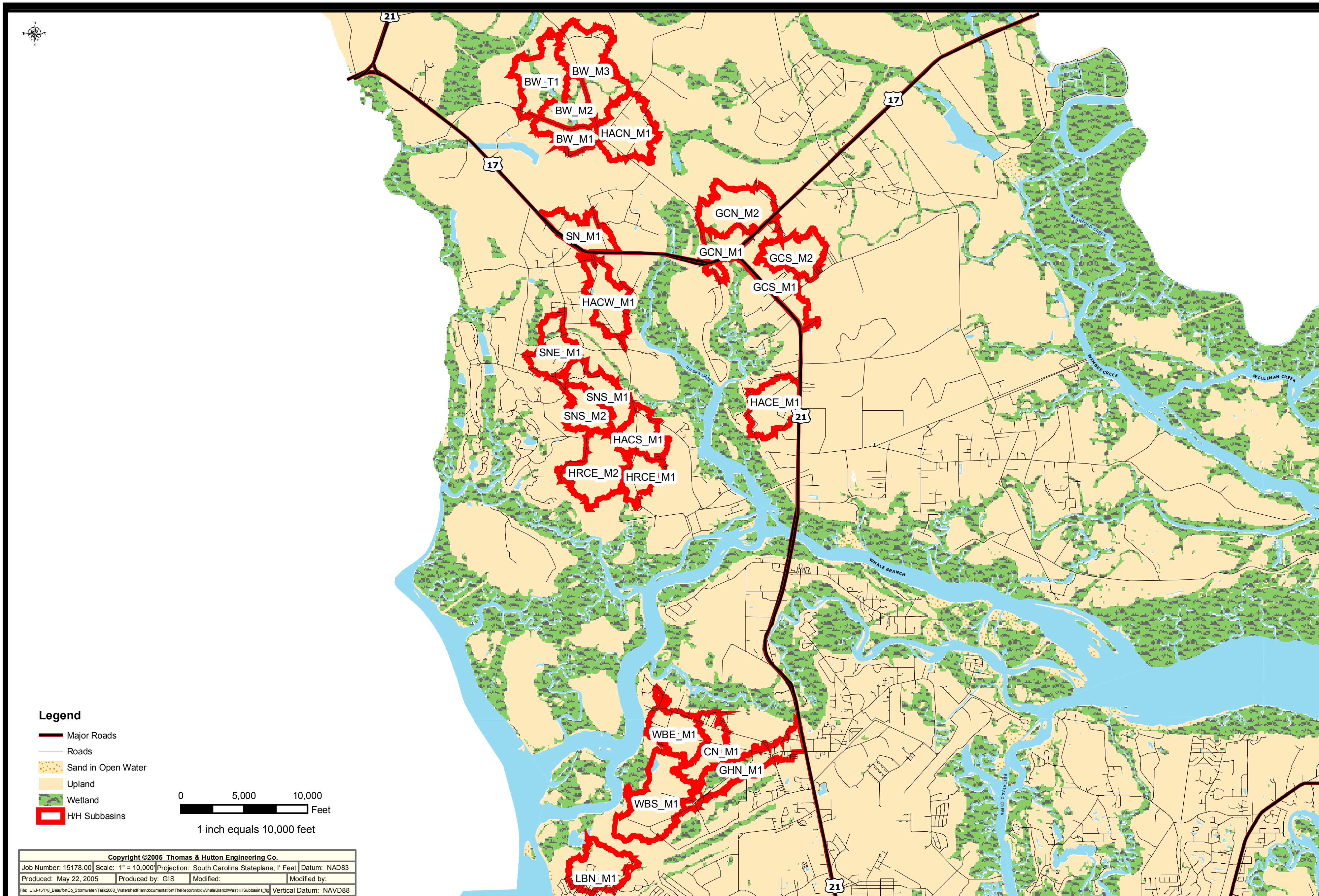
DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Whale Branch West Watershed Hydrologic Model Subbasins

Figure 10-2

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

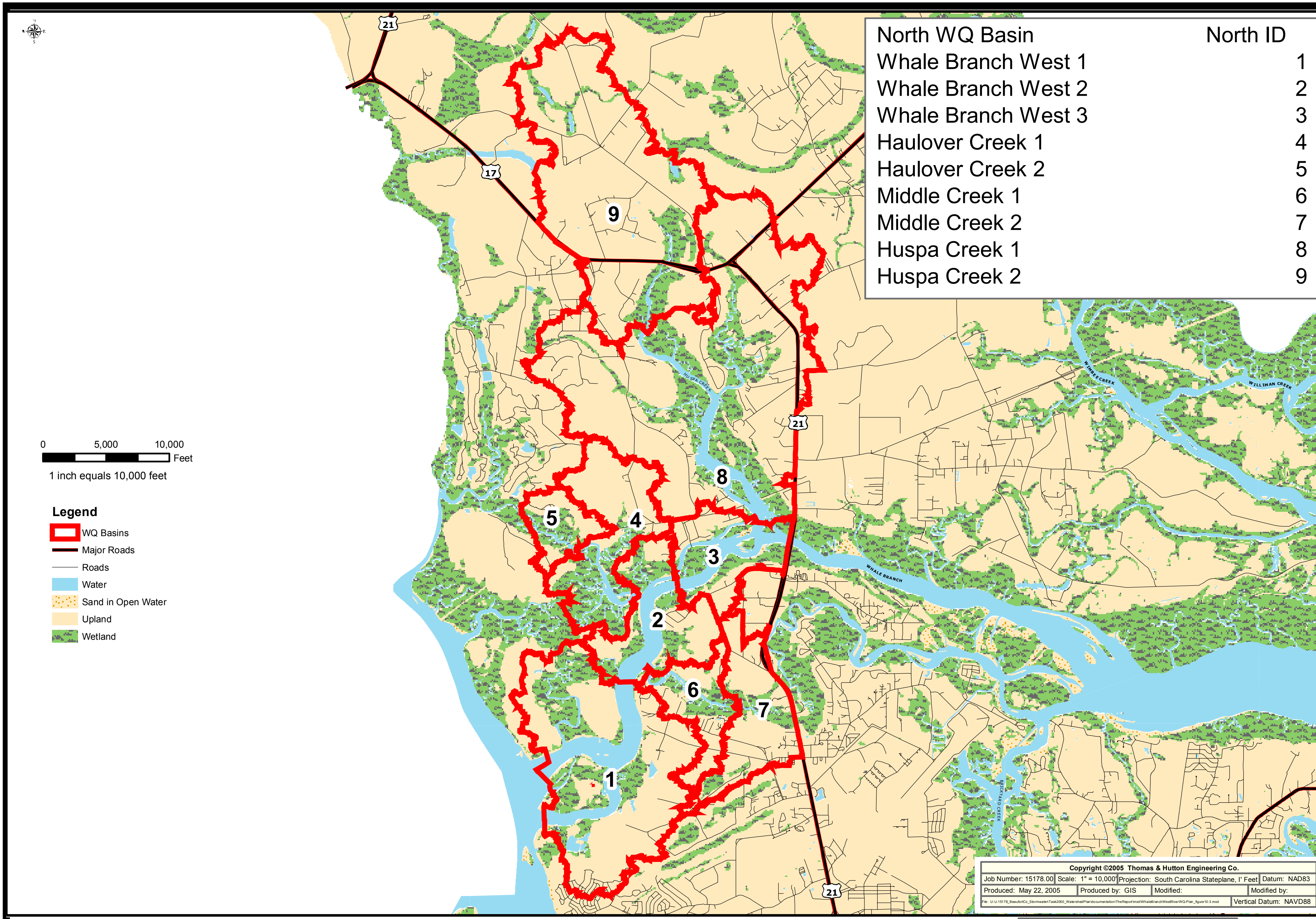

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Figure 10-3

Whale Branch West Watershed Water Quality Basins

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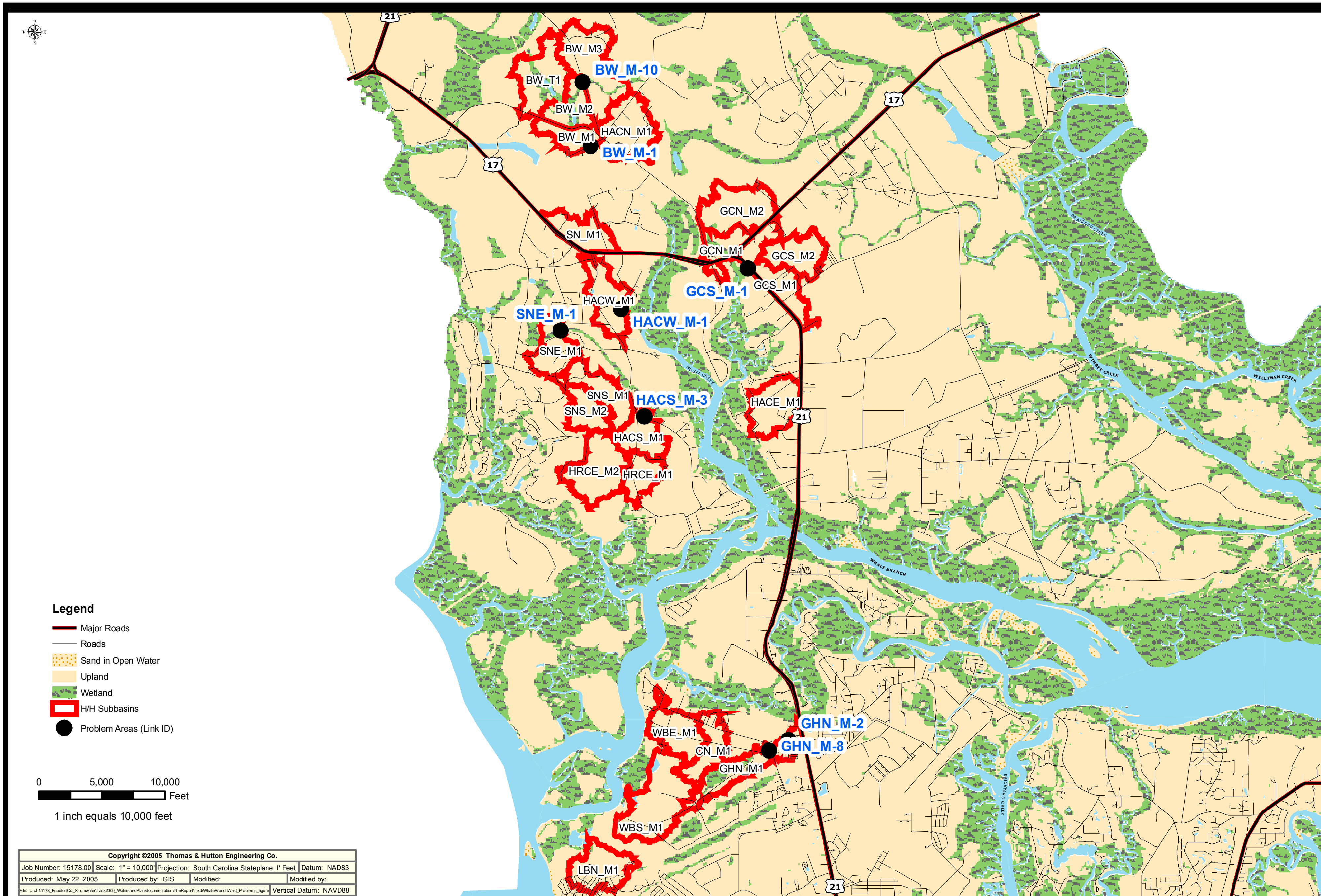
DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



Job Number: 15178.00	Scale: 1" = 10,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\WhaleBranchWest_Problems_figure			Vertical Datum: NAVD88

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Figure 10-4

Whale Branch West Watershed PSMS Problem Areas

Disclaimer
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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

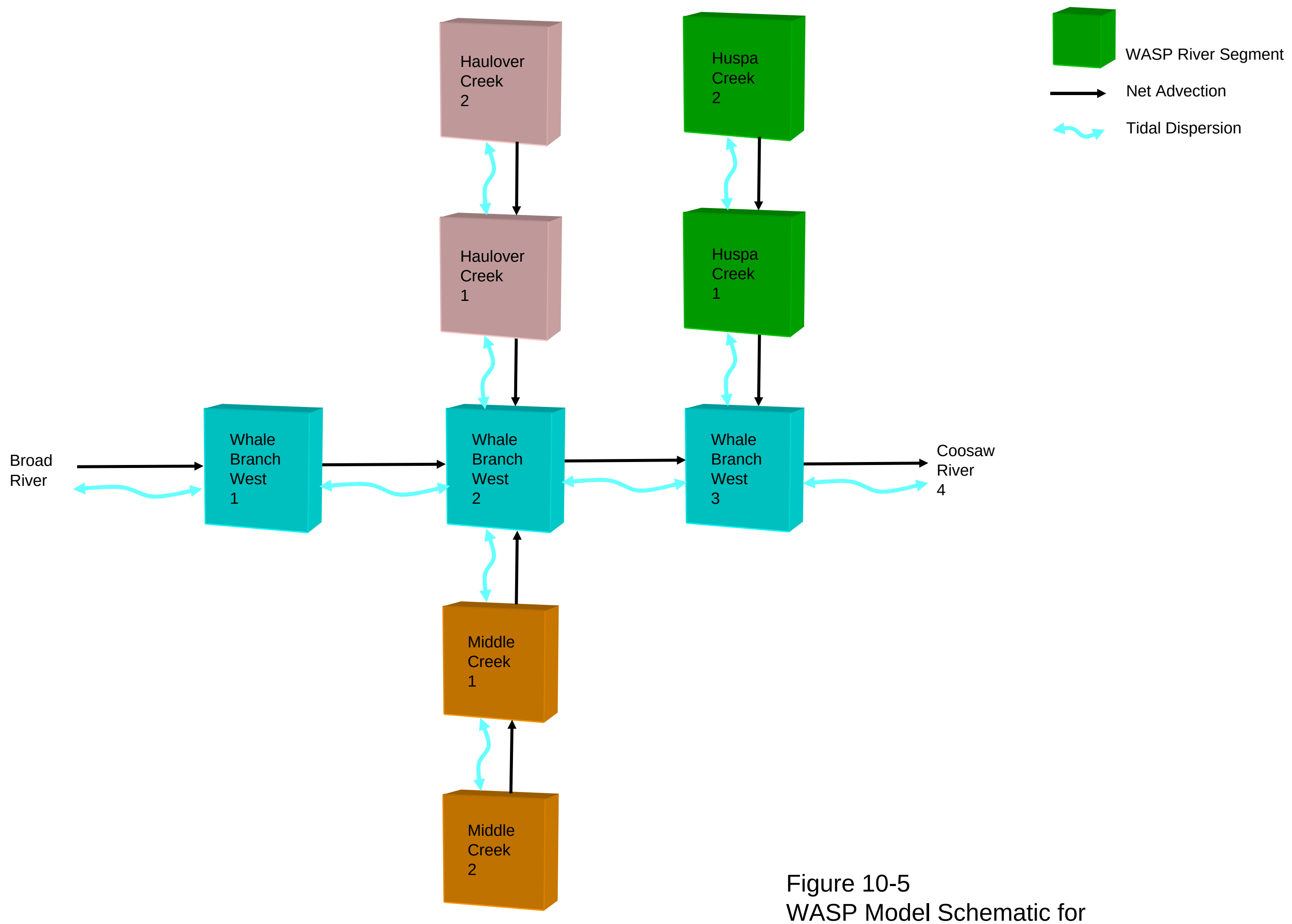

Figure 10-5
WASP Model Schematic for
Whale Branch West Watershed

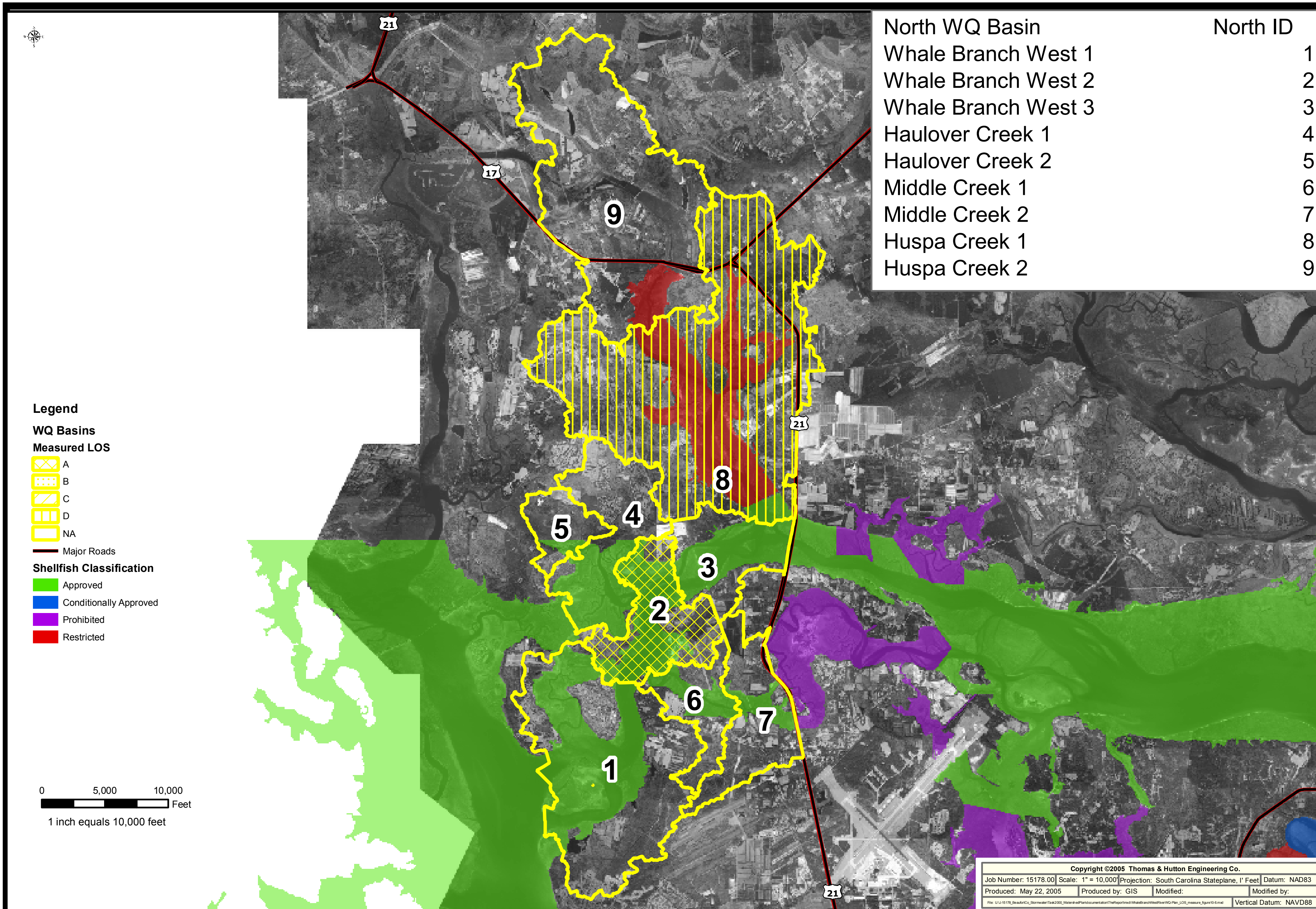

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Figure 10-6

Whale Branch West Watershed
Shellfish Classification and Existing Level of Service

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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Aerial Photography	Beaufort County	2002
Shellfish Classifications	SCDHEC	2004

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

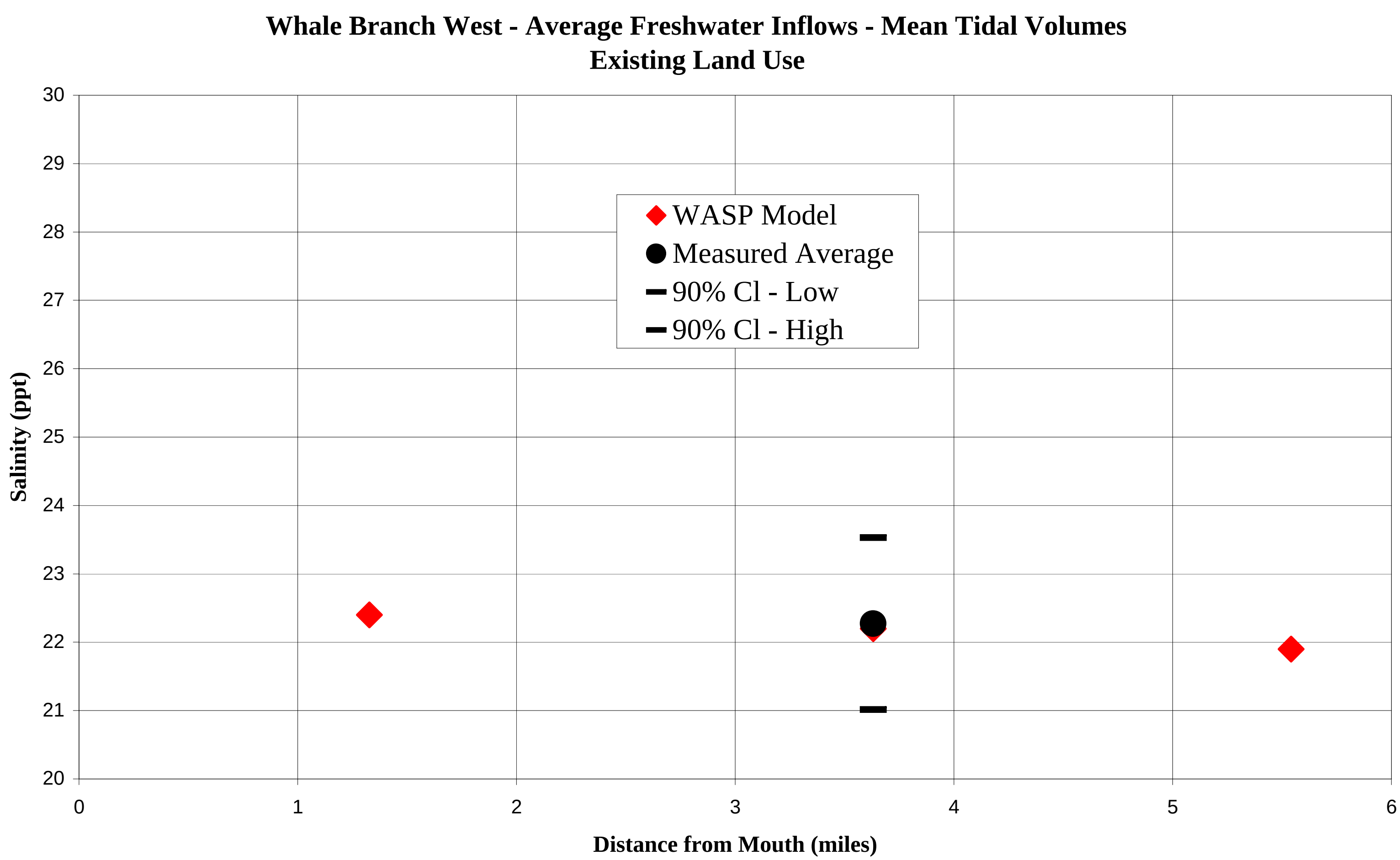

Figure 10-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Whale Branch West - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

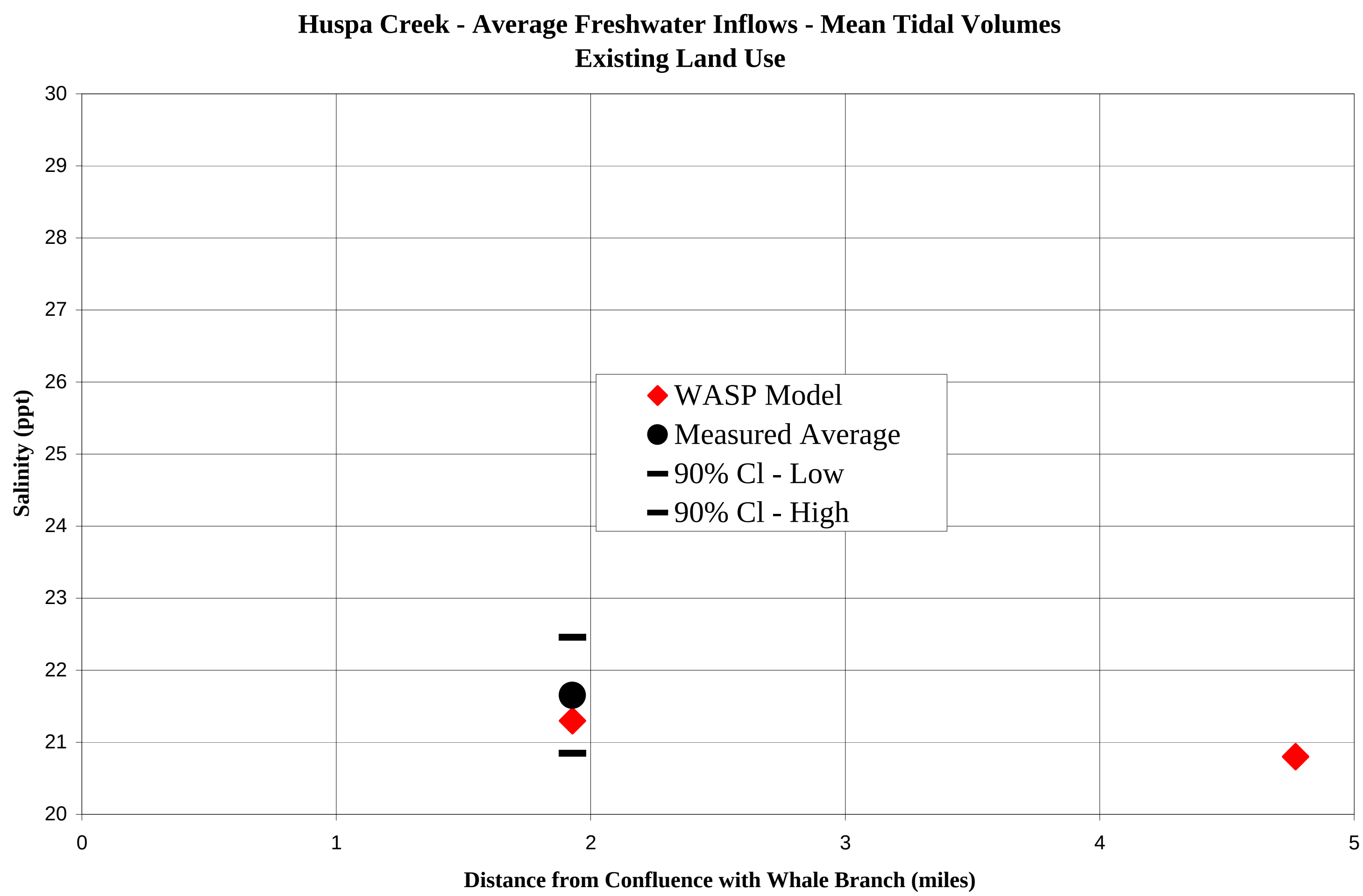

Figure 10-8. Comparison of WASP Model Results with Long-Term Monitoring Data in Huspa Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

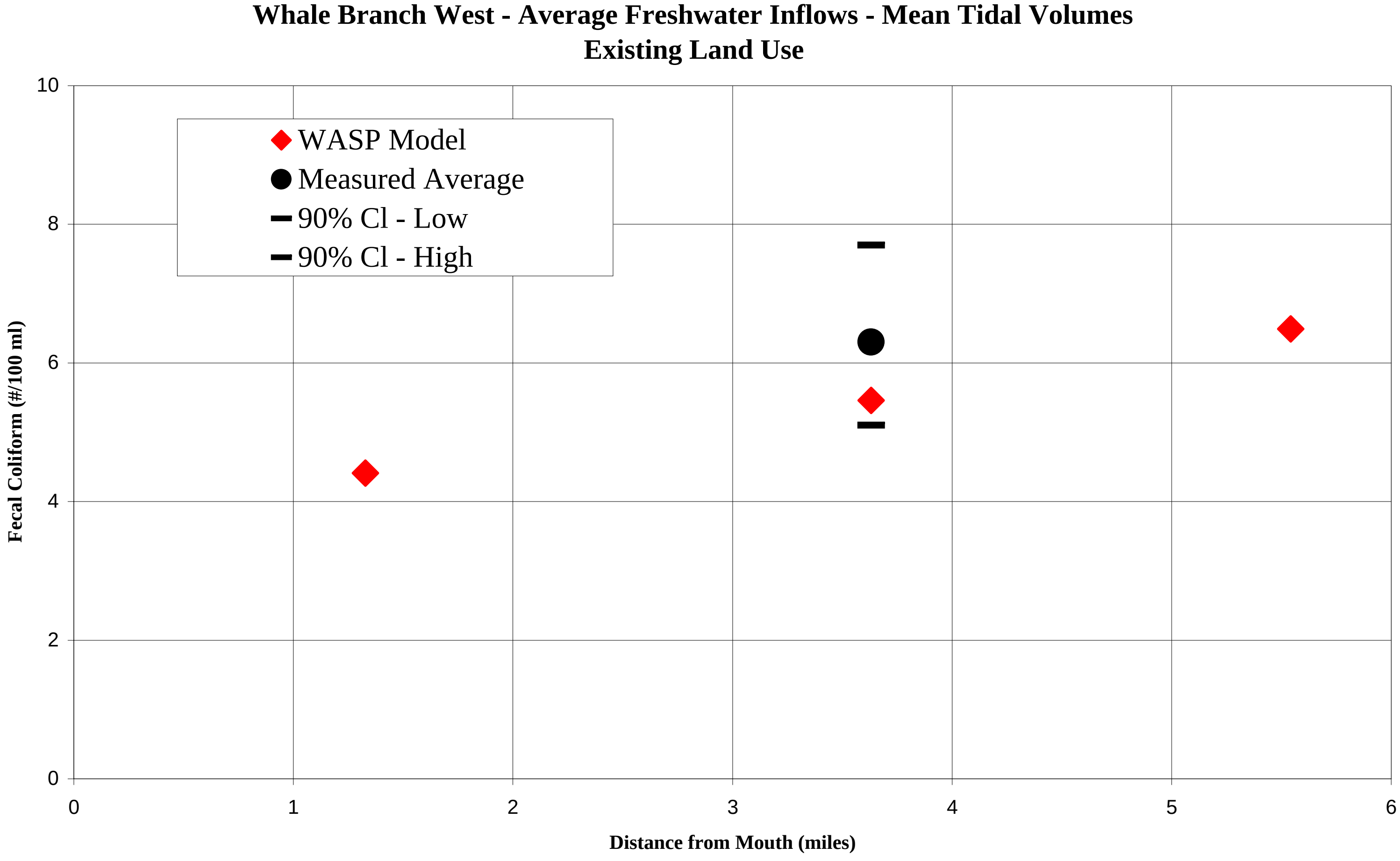

Figure 10-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Whale Branch West - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

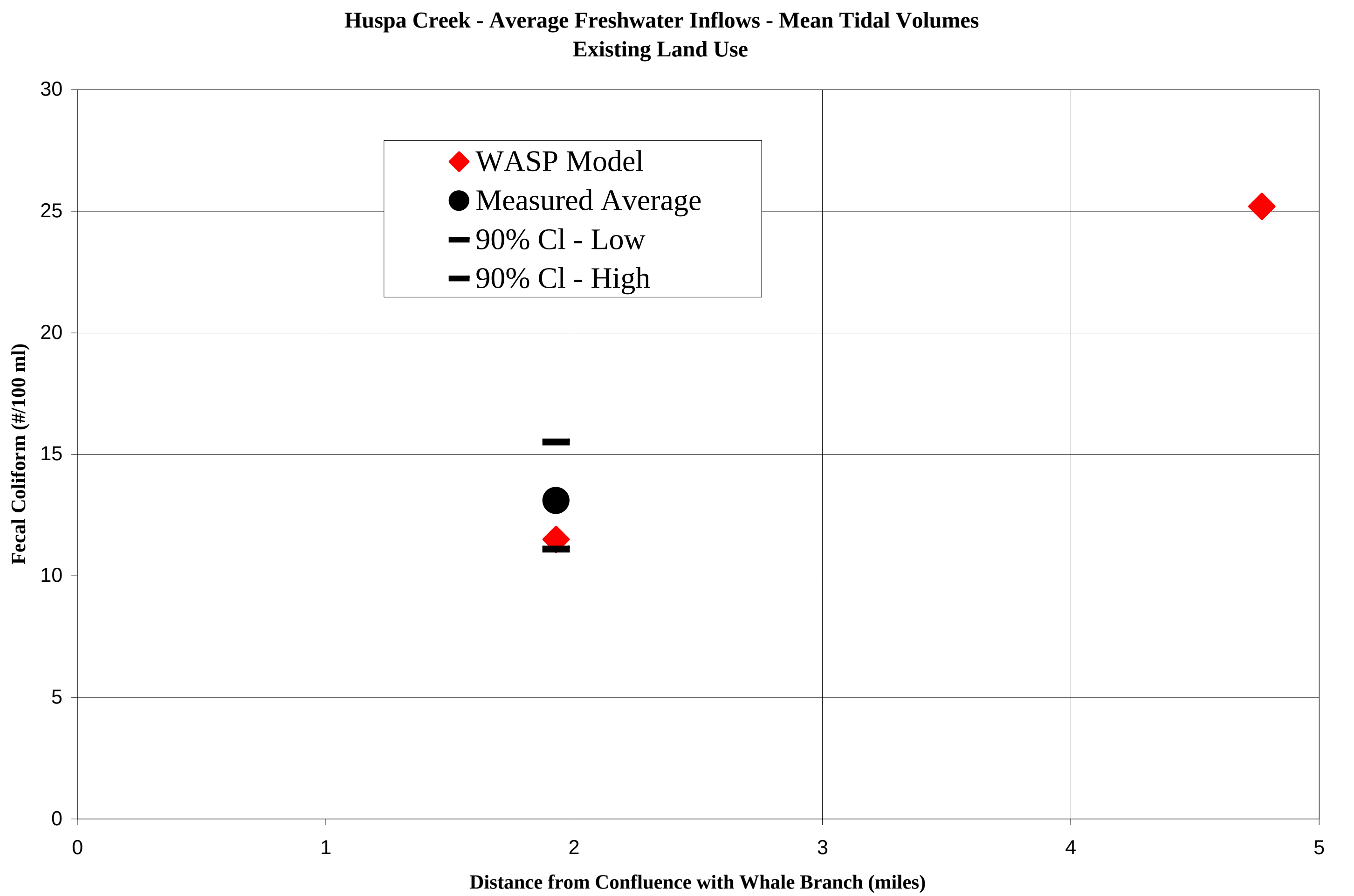

Figure 10-10. Comparison of WASP Model Results with Long-Term Monitoring Data in Huspa Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

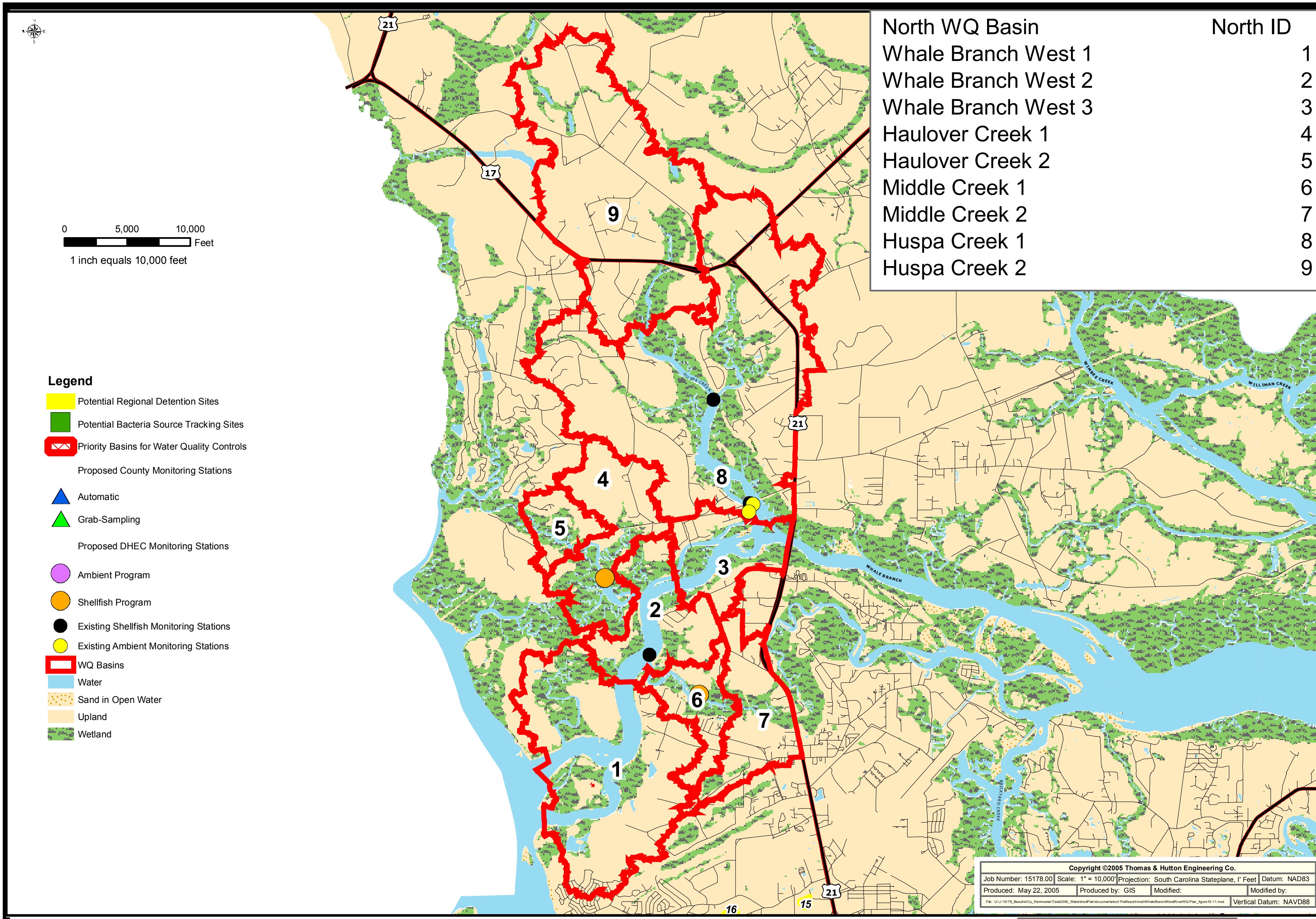

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Figure 10-11

Whale Branch West Watershed Water Quality Plan Elements

Disclaimer
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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Existing WQ Information	SCDHEC	2005

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
WQ Recommendations	T&H / CDM	2004

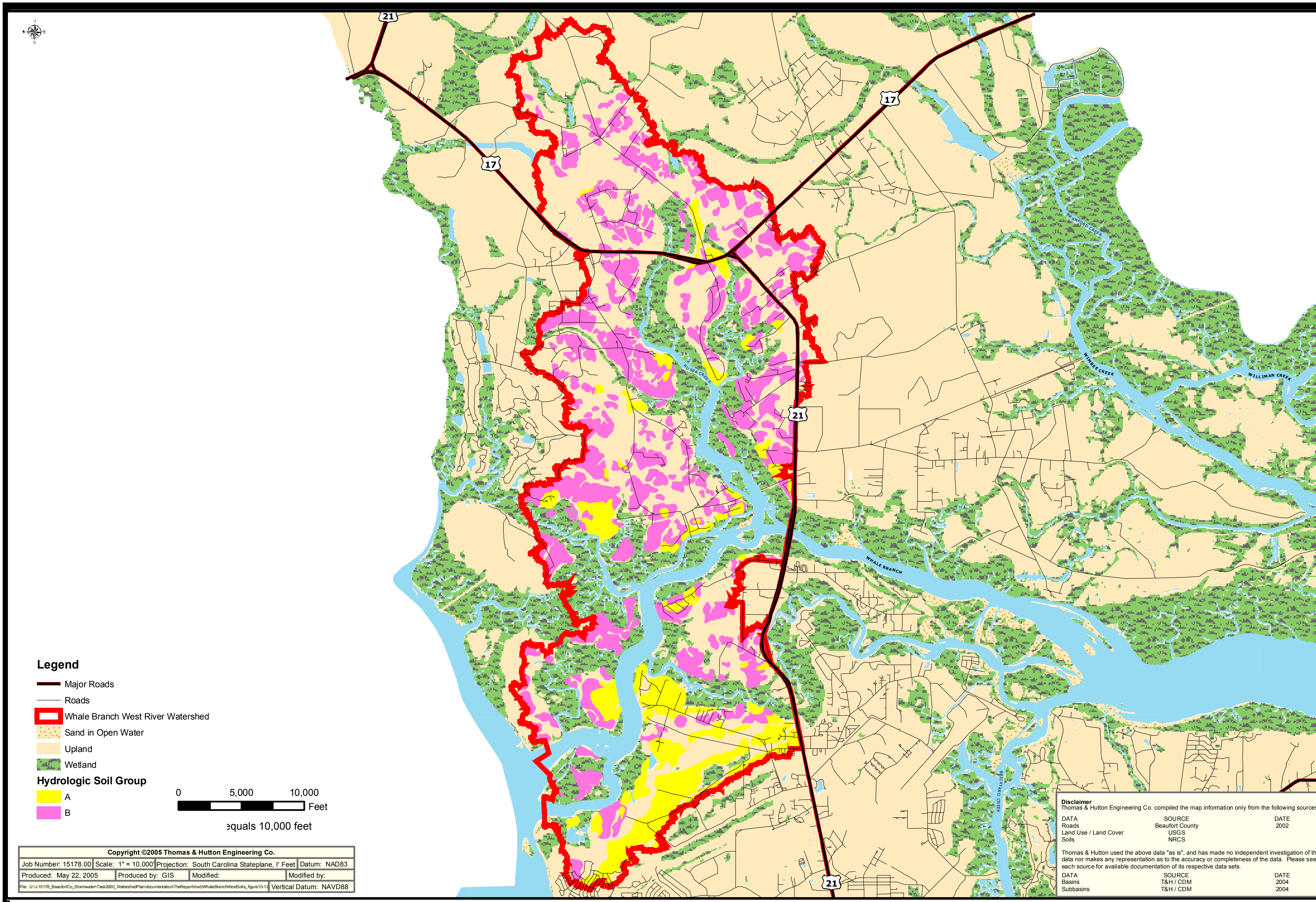

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Figure 10-12

Whale Branch West River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

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Section 11
Morgan River Watershed Analysis

This section describes the physical features of the Morgan River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

11.1 Overview

The Morgan River watershed is located north of the Broad River (see Figure 11-1). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in Lady's Island and St. Helena Island that is tributary to the Morgan River. Major Morgan River tributaries included in the analysis are Coffin Creek, Village Creek, Eddings Point Creek, Jenkins Creek, Parrot Creek, Lucy Point Creek, and Rock Springs Creek.

For the hydrologic and hydraulic analysis of the PSMS, the watershed includes several "hydrologic" basins. These are listed in Table 11-1 and presented in Figure 11-2. Table 11-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were completed to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into "water quality" basins, and the tidal receiving waters were subdivided into receiving water "segments". These are listed in Table 11-2 and presented in Figure 11-3. Pollution loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were completed to evaluate river bacteria concentrations. The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

11.2 Hydrologic and Hydraulic Analysis

The ICPR, Version 3 files previously prepared for the 2006 SWMP were used for the hydrologic and hydraulic analyses of the PSMS in the Morgan River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were updated for current (2016) existing land use conditions and reviewed against the future land use reported in the 2006 SWMP.

11.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Morgan River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 11-3 lists the hydrologic parameter values for the Morgan River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development. In areas where the existing is greater than the future, this indicates where the future condition has been achieved in the watershed compared to the 2006 SWMP model.

Hydraulic summary information for the Morgan River PSMS basins is presented in Table 11-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 11-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate LOS.

11.2.2 Model Results

Tables in Appendix I list the summary of the results of the updated study including Updated Areas and CNs values for the Morgan River subbasins.

For existing land use, aerial maps generated in the summer of 2016 and local information were used to estimate the percentage of existing urban development.

Appendix I also includes tables that list the peak water elevation values for model node locations along Morgan River PSMS.

Specific problem areas identified by the modeling are listed in Table 11-6 and presented in Figure 11-4. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) BFEs, and results showed that the FEMA elevations (based on storm

surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA BFEs should not be flooded.

Table 11-6 indicates that four road crossings are being overtopped by the design storm events. Evaluation of solutions to prevent these problems is discussed in the next section of this report.

11.2.3 Management Strategy Alternatives

The problems areas listed in Table 11-6 were evaluated by reviewing the previous report's results and reviewing the culverts in the ICPR hydraulic model. In the original 2006 study, the ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 11-7. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

11.3 Water Quality Analysis

ATM used the WMM and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of the Morgan River watershed. Land Use/Land Cover, BMP coverage and septic tank coverage was updated in the previously prepared WMM files which was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, TN, TP, BOD, lead, zinc, copper and TSS. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria loss, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria loss rates for existing conditions.

11.3.1 Land Use and BMP Coverage

Table 11-8 presents the existing land use and future land use estimates for the Morgan River water quality basins. The existing land use data were gathered from a number of sources, including July 2016 orthorectified aerials, county existing land use and tax parcel maps, NWI and USGS quadrangle maps and local knowledge of development completed between 2006 and 2016.

Under existing land use conditions, 31 percent of the Morgan River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 69 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 5 percent of the watershed.

Estimates of BMP coverage for existing land use is presented in Table 11-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County. Values are presented for developed urban land uses. The “total” value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall “total” BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 0.2 percent of the urban systems in the watershed are served by BMPs.

11.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing land use is presented in Table 11-10. The existing land use values reflect areas that are not designated as “sewered” areas by the Beaufort-Jasper Water & Sewer Authority. Values are presented for developed urban land uses. The “total” value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall “total” septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 87 percent of the urban systems in the watershed is served by septic.

Based on available data, the estimated wastewater discharge under existing conditions is 0.2 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be 0.6 mgd based on increase in residential land between existing and future conditions. There are no direct discharges to receiving waters in the watershed.

11.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Morgan River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing land use conditions.

The results are presented in Table 11-11 for existing land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

Wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing discharge of wastewater is limited to roughly 0.2 mgd of land application (e.g., golf course irrigation), and the future discharge is expected to be higher (0.6 mgd). Using the values in Table 2-9, the wastewater load for existing conditions accounts for 0.8 to 1.3 percent of the total watershed load for nutrients (TN and TP) and 0.0 to 0.2 percent of the load for other constituents.

11.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the Morgan River watershed. The model actually includes Beaufort River, Coosaw River, Whale Branch West and Morgan River watersheds because they are interconnected at several points. Only the Morgan River will be discussed in this section. A schematic of the model is presented as Figure 11-5.

Existing conditions for bacteria concentrations in the Morgan River are presented in Table 11-12. For each water quality basin river reach, the table lists the SCDHEC stations for which the 1990s bacteria data were analyzed, the concentrations calculated in the analysis, and the LOS associated with these concentrations (as discussed in Section 2.6.2. As shown in the table, SCDHEC data were available in 15 of the 29 river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations meet the water quality standards in eight of the fifteen monitored segments, and so these segments have an “A” LOS. Of the remaining seven monitored segments, one has a “B” LOS, two have a “C” LOS and four have a “D” LOS.

For informational purposes, Figure 11-6 presents a map of the LOS based on the monitoring data analysis, compared to SCDHEC “shellfish classification” (based on the 2016 SCDHEC reports for shellfish areas 16A, 19 and 20). The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the LOS, so there may not be a direct relationship between LOS and shellfish classification presented in the map. In general, however, segments with an "A" LOS are expected to have the lowest probability of receiving a "restricted" classification, and segments with a "D" LOS are expected to have the highest probability of receiving a "restricted" classification.

Physical characteristics assigned to the model reaches are presented in Table 11-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the "characteristic length" (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the SCDHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Table 11-14 presents the values used in the existing condition models.

A review of Table 11-14 shows that there is typically little change in flow or concentration between existing and future land use. For flow, this is because much of the flow to the tidal river segments comes from direct rainfall on the open water and tidal wetlands, as opposed to stormwater runoff and baseflow, and some of the basins have very little change in land use from existing to future conditions. Concentrations remain relatively constant because of the substantial amount of open water/tidal wetland area and the relatively limited development in some basins, as well as the BMPs for new development, which are assumed to have a high level of treatment efficiency.

Table 11-15 shows the net advective flows between segments. The hydrodynamic model (SWMM5) indicates that there is substantial net flow from the Coosaw River to the Morgan River, via Lucy Point Creek and Parrot Creek.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. The calibrated loss-rate coefficients from the 2006 study were used in the updated simulations.

Figure 11-7 is a graph showing a comparison between measured and modeled salinity data along the Morgan River main stem. The figure shows that the salinity data calculated by the model is very close to the average measured value in all of the four segments where monitoring data were available.

Measured and modeled salinity data for Eddings Point Creek are displayed in Figure 11-8. As with the Morgan River, the modeled salinity concentrations are very close to the measured mean values in the segments where monitoring data were available (Eddings Point Creek 1 and Eddings Point Creek 2).

For Parrot Creek, monitoring data were available only in segment Parrot Creek 1. As shown in Figure 11-9, the modeled salinity value there is close to the mean measured salinity, and well within the 90 percent confidence interval of the measured mean.

Figure 11-10 shows the measured and modeled salinity value for Jenkins Creek. The modeled value of salinity in Jenkins Creek 2 is lower than the measured mean, even lower than the low end of the 90 percent confidence interval of the measured mean. However, it should be pointed out that the mean measured salinity in the Jenkins 2 ("upstream") segment is actually higher than the mean measured salinity in the Jenkins 1 ("downstream") segment. Typically, the upstream segment will have a lower salinity because the impact of freshwater inflows is greater, and the influence of the downstream tidal boundary is less. One possible explanation is that there is a connection between the Jenkins Creek headwaters and the headwaters of Cowen Creek (in the Beaufort River watershed) that is not accounted for in the model.

The comparison of measured and modeled salinity for Lucy Point Creek South and its tributary, Rock Springs Creek, is presented in Figure 11-11. The modeled salinities are very close to the measured means.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration for the Morgan River is presented in Figure 11-12. The modeled bacteria values do not match but are very close to the geomean and the 90 percent confidence interval of the bacteria geomean of the measured bacteria data and follow the concentration variation in the river. The modeled values and the measured data are lower than the upper threshold for the "A" LOS (7/100 mL). Consequently, the difference between the modeled and measured values is not considered critical.

Figure 11-13 compares modeled and measured bacteria values for Eddings Point Creek, which discharges to the Morgan River 2 segment. As shown in the figure, the modeled bacteria values are lower than the geomean of the measured bacteria, and again outside the 90 percent confidence interval of the measured geomean. The underestimation of the bacteria in Eddings Point Creek is probably the reason that the Morgan River 2 bacteria concentration is underestimated by the model.

Results for Parrot Creek – the other tributary that discharges to Morgan River 2 – are presented in Figure 11-14. The figure shows that the modeled bacteria is slightly higher than the measured geomean and outside the 90 percent confidence interval for the geomean. The concentrations in Parrot Creek are also significantly lower than those measured in Eddings Point Creek, which again suggests that the underestimation of bacteria from Eddings Point Creek (and possibly underestimation of bacteria loads directly to Morgan River 2) are the causes of the model underestimation of bacteria concentrations in Morgan River 2.

Figure 11-15 shows the comparison of modeled and measured bacteria for Jenkins Creek. The modeled bacteria concentrations are very close to the measured geomean values.

Modeled and measured bacteria values for Lucy Point Creek and Rock Springs Creek are presented in Figure 11-16. The modeled values are generally within the 90 percent confidence interval of the measured bacteria geomean except for Rock Springs Creek 1.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 11-16. The loss rates ranged from 0.5/day to 1.4/day. The lowest values are applied at the downstream end of the Morgan River and downstream end of some tributaries, and in areas where the model was underestimating bacteria concentrations (e.g., Eddings Point Creek). As discussed earlier, even with relatively low loss rates in Eddings Point Creek, the model still underestimates bacteria concentrations, suggesting that the model is underestimating the bacteria loads to the creek.

The graphs show very good agreement between the measured values and the model results for some of the reaches and poor agreement in others. In water quality modeling, most performance metrics indicate a model that predicts a value 45-60% of the observed value is considered fair or satisfactory (Moriasi et. al, 2007, Donigian, 2002). Where predictions are poor, this is likely due to how the hydrodynamics of the systems are being modeled. The approach that has been used to date is based on the net flow advection of the various reaches and is a quasi-steady-state approach. This is an acceptable approach in most cases and has utility in this case as it allows for the comparison of water quality management and their effectiveness. However, given the tide range that exists in the county’s receiving waters and the dynamic salinity regimes present, a detailed 3-dimensional hydrodynamic model, such as the Environmental Fluid Dynamics Code (EFDC), is required to adequately simulate the tidal fluctuations and salinity-density gradients that exist in the receiving waters. Development of a 3-D hydrodynamic model would be a significant effort but would provide the proper hydrodynamic foundation for improved water quality predictions.

Based on water quality sampling data and model results, the following conclusions are:

- Problem basins include Village Creek 2 and 3, Coffin Creek 1 and 2, Eddings Point Creek 1, 2 and 3, Rock Springs Creek 1 and 2.
- 1 new regional water quality BMPs is proposed in Rock Springs Creek 1 basin.

Discussion of water quality related recommendations for monitoring and regional BMPs in the Morgan River watershed are presented as part of the overall recommended monitoring and CIP program for Beaufort County contained in the Appendix of this report.

11.3.5 Management Strategy Alternatives

In analyzing the watershed, one feasible regional detention sites was identified. The area tributary to the Rock Spring Creek 1 Regional BMP site includes approximately 194 acres of golf course and single-family development built prior to volume control stormwater regulations. There are stormwater best management practices, such as detention facilities, in the area. The project would be to construct modifications to the existing regional wet detention pond including permanent pool expansion, littoral shelf creation and control structure modifications. The project will provide enhanced stormwater runoff water quality treatment and volume reduction.

A new WMM scenario was developed for the Rock Spring Creek 1 Regional BMP and its contributing basin using the updated WMM database. Land cover estimates were made using 2016 aerial photographs. The receiving water quality parameter of focus is fecal coliform. Based on 80% reduction of fecal coliform loads from the contributing basin in the proposed wet detention pond, this would result in an overall fecal coliform load reduction in the Rock Spring Creek 1 water quality basin of approximately 10%. Based on the removal efficiencies in WMM, the proposed pond is anticipated to also provide the following pollutant load reductions to the Morgan River:

Parameter	lb/yr removed
Total Nitrogen	361
Total Phosphorus	144
TSS	33,583

The results of the water quality analysis suggest that the limited amount of future development in the watershed, combined with the effectiveness of required BMPs in reducing bacteria loads from new development, will maintain the existing high LOS in many of the watershed reaches. Areas have been identified above for evaluation of measures to improve the existing LOS. These activities could include retrofit of existing development that does not have BMPs, and modification of existing ponds that may not have been designed for water quality control.

For informational purposes, the areas with “A” and “B” type soils are presented in Figure 11-18. In general, these soils are more suitable for infiltration BMPs than areas with “C” and “D” type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

11.4 Planning Level Cost Estimates for Management Alternatives

Table 11-20 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Morgan River watershed. As shown in the table, the projects are estimated to have a total cost of $0.721 million in January 2018 dollars. Details of the cost estimate for each project are shown in Appendix I.

One regional CIP project was identified in the Morgan River watershed. The project is estimated to have a total cost of $0.431 million and is detailed in the CIP in Appendix O.

TABLE 11-1
HYDROLOGIC BASINS
MORGAN RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Coffin Creek	376	2	188
Factory Creek	444	2	222
Lucy Point	361	1	361
Rock Springs Creek	468	2	234
Village Creek	1,572	4	393
TOTAL	3,221	11	293

TABLE 11-2
WATER QUALITY BASINS
MORGAN RIVER WATERSHED

Basin Name	Tributary Area (acres)
Morgan River 1	1,178
Morgan River 2	2,407
Morgan River 3	1,901
Morgan River 4	2,231
Morgan River 5	2,693
Morgan River 6	184
Village Creek 1	1,735
Village Creek 2	1,407
Village Creek 3	2,061
Coffin Creek 1	1,001
Coffin Creek 2	594
Parrot Creek 1	1,161
Parrot Creek 2	386
Bass Creek 1	733
Bass Creek 2	197
Eddings Point Creek 1	860
Eddings Point Creek 2	1,064
Eddings Point Creek 3	545
Eddings Point Creek Trib. 1	696
Boatswain Pond Creek	512
Jenkins Creek 1	1,373
Jenkins Creek 2	1,804
Doe Point Creek 1	356
Lucy Point Creek South 1	697
Lucy Point Creek South 2	426
Rock Springs Creek 1	1,398
Rock Springs Creek 2	1,188
Jenkins Creek Warsaw Flats 1	568
Jenkins Creek Warsaw Flats 2	1,230
TOTAL	32,585

TABLE 11-3 (Updated 2017)
HYDROLOGIC SUBBASIN CHARACTERISTICS
MORGAN RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Coffin Creek Basin					
CNC_M1	64	74	72	77	66
CNC_M2	312	73	192	80	159
Factory Creek Basin					
FC_M1	171	73	158	75	149
FC_M2	274	79	138	80	130
Lucy Point Basin					
LP_M1	361	77	148	78	145
Rock Springs Creek Basin					
RSC_M1	194	78	119	79	116
RSC_M2	273	81	122	82	120
Village Creek Basin					
VC_M1	378	72	114	81	107
VC_M2	535	77	169	79	160
VC_T1	318	72	186	74	166
VC_T2	341	71	204	76	179
Average	293	75	148	78	136

TABLE 11-4
HYDRAULIC DATA SUMMARY
MORGAN RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges
Coffin Creek	3	1,830	3	5	0
Factory Creek	5	5,421	1	1	0
Lucy Point	4	2,027	2	2	0
Rock Springs Creek	3	989	3	3	0
Village Creek	12	14,165	1	1	0
TOTAL	27	24,432	10	12	0

Basin Name	Other Features		
	Storage Nodes	Weirs	Drop Structures
Coffin Creek	1	2	0
Factory Creek	2	1	0
Lucy Point	1	2	0
Rock Springs Creek	1	4	1
Village Creek	1	1	0
TOTAL	6	10	1

TABLE 11-5
CULVERT DATA FOR HYDROLOGIC BASINS
MORGAN RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Coffin Creek Basin						
Shed Road	CNC_M-1A	48"x48"	40	-1.5	6.0	25
	1B	48"x48"	40	-1.2		
Sea Island Parkway (US Hwy 21)	CNC_M-3A	42"x42"	60	2.6	7.4	100
	3B	48"x48"	60	1.4		
Langford Road	CNC_M-6	24"x24"	20	1.8	5.5	25
Factory Creek Basin						
Holly Hall Road	FC_M-3	24"x24"	40	5.3	9.5	25
Lucy Point Basin						
Pine Run Trail	LP_M-2	72"x72"	72	8.6	15.4	25
Sams Point Road (State Hwy 802)	LP_M-4	72"x72"	72	11.3	19.8	100
Rock Springs Creek Basin						
Sams Point Road (State Hwy 802)	RSC_M-3	36"x36"	40	9.1	17.9	100
Wade Hampton Drive	RSC_M-5	24"x24"	40	13.9	18.3	25
Village Creek Basin						
Hickory Hall Road	VC_T1-4	30"x30"	50	14.1	23.4	25

TABLE 11-6 (Updated 2017)
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
MORGAN RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Warning Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Area Located in GIS	Recommended in 2006 CIP
Coffin Creek Basin							
Langford Road	CNC_M-22	5.5	5.5	2	5.7	Yes	Yes
				10	5.8		
				25	5.8		
				100	5.8		
Factory Creek Basin							
Holly Hall Road	FC_M-23	9.5	9.5	2	10.0	Yes	Yes
				10	10.1		
				25	10.1		
				100	10.1		
Rock Springs Creek Basin							
Sams Point Road (State Hwy 802)	RSC_M-13	17.9	17.9	10	18.0	Yes	Yes
				25	18.0		
				100	18.0		
Location Unknown (Wade Hampton Drive)	RSC_M-18	Unknown	18.4	10	18.5	No	No
				25	18.5		
				100	18.5		
Wade Hampton Drive	RSC_M-21	18.3	18.3	10	18.7	Yes	Yes
				25	18.7		
				100	18.7		

TABLE 11-7
RECOMMENDED CULVERT IMPROVEMENTS
MORGAN RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Coffin Creek Basin			
Langford Road	CNC_M-6	24"x24"	Raise road from elevation 5.5 ft to elevation 6.6 ft NAVD (length of 620 ft), Replace culvert with one 8 ft by 4 ft box culvert
Factory Creek Basin			
Holly Hall Road	FC_M-3	24"x24"	Replace culvert with three 8 ft by 4 ft box culverts
Rock Springs Creek Basin			
Sams Point Road (State Hwy 802)	RSC_M-3	36"x36"	Replace culvert with one 8 ft by 6 ft box culvert
Wade Hampton Drive	RSC_M-5	24"x24"	Replace culvert with one 8 ft by 4 ft box culvert

TABLE 11-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
MORGAN RIVER WATERSHED

Land Use Type	Bass Creek 1 (acres)	Bass Creek 2 (acres)	Boatswain Pond (acres)	Coffin Creek 1 (acres)	Coffin Creek 2 (acres)	Doe Point Creek 1 (acres)	Eddings Point Creek 1 (acres)	Eddings Point Creek 2 (acres)	Eddings Point Creek 3 (acres)	Eddings Creek Trib 1 (acres)	Jenkings Creek 1 (acres)	Jenkins Creek 2 (acres)	Jenkins Creek Warsaw Flats 1 (acres)	Jenkins Creek Warsaw Flats 2 (acres)
Agricultural/Pasture	0	0	0	0	0	0	0	11	184	0	0	86	0	0
Commercial	0	0	0	0	2	0	1	0	0	0	1	50	0	5
Forest/Rural Open	18	0	126	20	139	22	105	67	68	58	2	168	17	30
Golf Course	0	0	0	0	0	0	0	0	0	0	89	34	0	0
High Density Residential	0	0	0	0	0	6	0	0	0	0	3	16	0	0
Industrial	0	0	10	27	22	16	12	30	9	3	35	54	14	28
Institutional	0	0	0	0	1	1	0	0	0	0	0	25	0	4
Low Density Residential	0	0	79	246	165	63	89	371	83	10	38	215	49	281
Medium Density Residential	0	0	0	0	0	0	0	0	0	0	112	42	51	2
Open Water/Tidal	714	197	277	573	82	218	582	515	151	626	1080	933	390	595
Silviculture	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Urban Open	0	0	20	129	160	31	60	44	12	0	11	146	45	253
Wetland/Water	0	0	1	4	22	0	11	25	38	0	1	35	1	31
TOTAL	733	197	512	1001	594	356	860	1064	545	696	1373	1804	568	1230
Urban Imperviousness (%)	0%	0%	3%	5%	6%	6%	2%	6%	3%	1%	4%	7%	5%	5%

TABLE 11-8 (CONTINUED)
WATER QUALITY BASIN LAND USE DISTRIBUTION
MORGAN RIVER WATERSHED

Land Use Type	Lucy Point Creek South 1 (acres)	Lucy Point Creek South 2 (acres)	Morgan River 1 (acres)	Morgan River 2 (acres)	Morgan River 3 (acres)	Mogan River 4 (acres)	Morgan River 5 (acres)	Morgan River 6 (acres)	Parrot Creek 1 (acres)	Parrot Creek 2 (acres)	Rock Springs Creek 1 (acres)	Rock Springs Creek 2 (acres)	Village Creek 1 (acres)	Village Creek 2 (acres)	Village Creek 3 (acres)	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	0	0	0	0	0	0	0	0	0	0	113	394
Commercial	2	0	0	5	0	0	69	2	0	0	0	5	0	4	2	149
Forest/Rural Open	63	8	38	194	61	105	134	7	193	0	9	147	276	307	129	2510
Golf Course	0	0	0	0	64	102	0	0	0	0	222	0	0	0	0	511
High Density Residential	0	0	0	0	0	0	0	0	0	0	12	58	0	0	0	95
Industrial	8	4	0	10	18	43	184	6	10	0	92	84	21	37	58	835
Institutional	0	0	0	0	0	0	44	0	0	0	0	58	0	2	0	134
Low Density Residential	17	31	0	117	124	0	196	18	47	0	10	207	156	468	774	3854
Medium Density Residential	33	11	0	0	61	209	358	0	0	0	591	294	0	0	0	1765
Open Water/Tidal	341	308	1140	2011	1530	1719	1563	140	858	386	258	86	1183	397	141	18992
Silviculture	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Urban Open	57	8	0	64	43	38	117	12	49	0	48	91	73	183	537	2232
Wetland/Water	177	56	0	6	0	15	30	0	4	0	158	158	25	9	307	1115
TOTAL	696	426	1178	2407	1901	2231	2693	184	1161	386	1398	1188	1735	1407	2061	32586
Urban Imperviousness (%)	3%	2%	0%	1%	2%	4%	12%	4%	1%	0%	16%	18%	2%	6%	6%	5%

TABLE 11-9
WATER QUALITY BASIN BMP COVERAGE
MORGAN RIVER WATERSHED

Land Use Type	Bass Creek 1	Bass Creek 2	Boatswain Pond	Coffin Creek 1	Coffin Creek 2	Doe Point Creek 1	Eddings Point Creek 1	Eddings Point Creek 2	Eddings Point Creek 3
Commercial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
TOTAL	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

TABLE 11-9 (CONTINUED)
WATER QUALITY BASIN BMP COVERAGE
MORGAN RIVER WATERSHED

Land Use Type	Eddings Creek Trib 1	Jenkings Creek 1	Jenkins Creek 2	Jenkins Creek Warsaw Flats 1	Jenkins Creek Warsaw Flats 2	Lucy Point Creek South 1	Lucy Point Creek South 2	Morgan River 1	Morgan River 2	Morgan River 3	Mogan River 4
Commercial	0.0%	0.0%	30.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	11.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	9.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	6.5%
TOTAL	0.0%	0.0%	2.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	1.0%

TABLE 11-9 (CONTINUED)
WATER QUALITY BASIN BMP COVERAGE
MORGAN RIVER WATERSHED

Land Use Type	Morgan River 5	Morgan River 6	Parrot Creek 1	Parrot Creek 2	Rock Springs Creek 1	Rock Springs Creek 2	Village Creek 1	Village Creek 2	Village Creek 3	TOTAL
Commercial	12.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	16.0%
Golf Course	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.9%
Institutional	3.9%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	1.3%
Low Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.5%
Medium Density Residential	0.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	1.5%
TOTAL	0.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%

TABLE 11-10
WATER QUALITY BASIN SEPTIC TANK COVERAGE
MORGAN RIVER WATERSHED

Land Use Type	Bass Creek 1	Bass Creek 2	Boatswain Pond	Coffin Creek 1	Coffin Creek 2	Doe Point Creek 1	Eddings Point Creek 1	Eddings Point Creek 2	Eddings Point Creek 3
Commercial	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.9%	0.0%	0.0%	0.2%	0.0%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	0.0%	7.9%	7.7%	9.9%	10.0%	17.6%	5.8%	1.6%
Medium Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
TOTAL	0.0%	0.0%	1.2%	1.9%	2.8%	1.8%	1.8%	2.0%	0.2%

TABLE 11-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
MORGAN RIVER WATERSHED

Land Use Type	Eddings Creek Trib 1	Jenkings Creek 1	Jenkins Creek 2	Jenkins Creek Warsaw Flats 1	Jenkins Creek Warsaw Flats 2	Lucy Point Creek South 1	Lucy Point Creek South 2	Morgan River 1	Morgan River 2	Morgan River 3	Mogan River 4
Commercial	0.0%	0.0%	0.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
High Density Residential	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Industrial	0.0%	0.0%	0.0%	0.0%	0.6%	0.0%	0.1%	0.0%	4.8%	0.6%	0.0%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Low Density Residential	0.0%	15.6%	9.9%	7.7%	18.1%	2.7%	13.6%	0.0%	4.7%	6.4%	0.0%
Medium Density Residential	0.0%	1.6%	7.6%	0.0%	0.0%	0.0%	19.4%	0.0%	0.0%	0.0%	1.2%
TOTAL	0.0%	0.6%	1.4%	0.7%	4.2%	0.1%	1.5%	0.0%	0.3%	0.4%	0.1%

TABLE 11-10 (CONTINUED)
WATER QUALITY BASIN SEPTIC TANK COVERAGE
MORGAN RIVER WATERSHED

Land Use Type	Morgan River 5	Morgan River 6	Parrot Creek 1	Parrot Creek 2	Rock Springs Creek 1	Rock Springs Creek 2	Village Creek 1	Village Creek 2	Village Creek 3	TOTAL
Commercial	8.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	4.2%
High Density Residential	0.0%	0.0%	0.0%	0.0%	2.7%	3.2%	0.0%	0.0%	0.0%	2.3%
Industrial	0.0%	4.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%	0.2%
Institutional	0.0%	0.0%	0.0%	0.0%	0.0%	1.0%	0.0%	0.0%	0.0%	0.4%
Low Density Residential	1.9%	0.0%	0.0%	0.0%	16.8%	5.7%	5.0%	11.6%	4.4%	7.8%
Medium Density Residential	6.3%	0.0%	0.0%	0.0%	5.4%	4.3%	0.0%	0.0%	0.0%	4.3%
TOTAL	1.2%	0.1%	0.0%	0.0%	2.4%	2.3%	0.4%	3.8%	1.6%	1.2%

TABLE 11-11
AVERAGE ANNUAL LOADS FOR MORGAN RIVER WATERSHED WATER QUALITY BASINS

Water Quality Basin ID	Area (acres)	Flow (ac-ft/yr)	BOD (lbs/yr)	Cu (lbs/yr)	FC Geomean Log (lbs/yr)	F-Coli (counts/yr)	Pb (lbs/yr)	Total N (lbs/yr)	Total P (lbs/yr)	TSS (lbs/yr)	Zn (lbs/yr)
Morgan River 1	1,178	4,156	33,856	45	35,596	3.28E+14	68	14,690	1,805	68,692	1,643
Morgan River 2	2,407	7,548	63,915	86	64,830	6.27E+14	126	26,909	3,329	168,000	2,931
Morgan River 3	1,901	5,840	51,956	71	50,346	5.17E+14	102	21,106	2,737	164,000	2,259
Morgan River 4	2,231	6,734	62,180	89	58,191	6.19E+14	122	24,415	3,240	227,000	2,573
Morgan River 5	2,693	7,182	85,439	137	63,051	8.42E+14	158	27,483	3,639	532,000	2,628
Morgan River 6	184	550	5,164	8	4,750	4.93E+13	10	2,007	251	19,853	211
Village Creek 1	1,735	4,652	41,083	56	40,083	4.08E+14	79	16,742	2,081	138,000	1,750
Village Creek 2	1,407	2,123	26,355	35	18,873	2.94E+14	47	8,346	1,137	185,000	684
Village Creek 3	2,061	1,947	29,552	41	17,785	3.48E+14	48	8,299	1,293	293,000	385
Coffin Creek 1	733	2,406	24,780	34	20,975	2.55E+14	47	8,959	1,169	120,000	890
Coffin Creek 2	197	644	8,749	13	5,775	9.64E+13	15	2,614	348	75,868	168
Parrot Creek 1	1,161	3,286	27,791	38	28,223	2.72E+14	54	11,719	1,439	78,406	1,253
Parrot Creek 2	386	1,400	11,425	15	11,996	1.10E+14	23	4,951	609	22,849	556
Bass Creek 1	733	2,601	21,198	28	22,281	2.05E+14	42	9,195	1,130	42,871	1,030
Bass Creek 2	197	715	5,834	8	6,125	5.64E+13	12	2,528	311	11,667	284
Eddings Point Creek 1	860	2,298	20,581	28	19,820	2.07E+14	40	8,313	1,036	72,088	864
Eddings Poin Creek 2	1,064	2,307	26,066	34	20,292	2.77E+14	49	8,785	1,201	149,000	828
Eddings Point Creek 3	545	793	8,235	13	6,915	8.34E+13	13	3,227	624	53,407	239
Eddings Point Creek Trib. 1	696	2,311	19,077	26	19,812	1.85E+14	38	8,193	1,007	43,412	906
Boatswain Pond Creek	512	10,228	85,020	114	87,723	8.28E+14	169	36,303	4,483	196,000	4,023
Jenkins Creek 1	1,373	4,235	40,014	58	36,631	3.95E+14	78	15,572	2,091	153,000	1,630
Jenkins Creek 2	1,804	4,202	44,874	68	36,609	4.36E+14	83	15,237	2,165	254,000	1,509
Doe Point Creek 1	356	914	9,436	14	7,948	9.29E+13	18	3,406	437	46,427	341
Lucy Point Creek South 1	696	1,584	13,858	20	13,672	1.43E+14	25	5,729	685	64,382	515
Lucy Point Creek South 2	426	1,238	10,868	15	10,679	1.12E+14	21	4,475	553	38,097	456
Rock Springs Creek 1	1,398	2,244	38,375	63	20,688	4.51E+14	69	10,127	1,783	353,000	627
Rock Springs Creek 2	1,188	1,570	31,708	52	14,803	3.68E+14	53	7,271	1,082	331,000	383
Jenkins Creek Warsaw Flats 1	568	1,582	15,723	22	13,747	1.59E+14	30	5,824	756	68,950	596
Jenkins Creek Warsaw Flats 2	1,230	2,626	27,329	37	22,935	2.90E+14	51	9,869	1,275	143,000	936
TOTAL	31,921	89,916	890,441	1,268	781,154	9.05E+15	1,690	332,294	43,696	4,112,969	33,098

TABLE 11-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS

MORGAN RIVER WATERSHED

Water Quality Basin ID	DHEC Station(s)	Years of Record	No. of Samples	Fecal Coliform Concentrations				Trend	Level of Service
				Long-Term Average		Most Recent 3 Year Values			
				Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)		
Morgan River 1	16A-08, 16A-27	1999-2016	412	7.02	33	8.92	33	Increasing	B
Morgan River 2	16A-09	1999-2016	207	6.13	33	8.2	49	Increasing	A
Morgan River 3	16A-11	1999-2016	207	4.12	12.39	3.85	7.8	No Trend	A
Morgan River 4	16A-39, 16A-35	2006-2016	231	5.36	22	4.81	15.69	Decreasing	A
Morgan River 5	NA	NA	NA	NA	NA	NA	NA	NA	NA
Morgan River 6	NA	NA	NA	NA	NA	NA	NA	NA	NA
Village Creek 1	16A-38, 16A-32	1999-2016	197	9.96	49	12.76	49	Increasing	C
Village Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Village Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Coffin Creek 1	16A-28	1999-2010	140	21.16	140	33.66	127.85	Increasing	D
Coffin Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Eddings Point Creek 1	16A-23	1999-2016	207	11.91	49	16.84	75.32	Increasing	D
Eddings Point Creek 2	16A-18	1999-2010	141	14.76	70	23.94	91.79	Increasing	D
Eddings Point Creek 3	NA	NA	NA	NA	NA	NA	NA	NA	NA
Eddings Point Creek Trib. 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Parrot Creek 1	16A-10	1999-2016	204	2.93	7	2.83	7.8	No Trend	A
Parrot Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bass Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Bass Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Jenkins Creek 1	16A-14, 16A-24	1999-2016	413	5.29	17	5.18	13	No Trend	A
Jenkins Creek 2	16A-30, 16A-37	1999-2016	206	6.21	22	5.05	13	Decreasing	A
Doe Point Creek 1	NA	NA	NA	NA	NA	NA	NA	NA	NA
Boatswain Pond	NA	NA	NA	NA	NA	NA	NA	NA	NA
Jenkins Creek Warsaw Flats 1	16A-25, 16A-36	1999-2016	328	6.66	23	5.87	20.36	Decreasing	A
Jenkins Creek Warsaw Flats 2	NA	NA	NA	NA	NA	NA	NA	NA	NA
Lucy Point Creek South1	16A-13A	1999-2016	206	6.56	28.93	5.88	26.57	Decreasing	A
Lucy Point Creek South 2	16A-13, 16A-34	1999-2016	330	9.79	34.81	7.43	20.36	Decreasing	C
Rock Springs Creek 1	16A-19	1999-2016	206	22.91	110	21.08	130	Decreasing	D
Rock Springs Creek 2	NA	NA	NA	NA	NA	NA	NA	NA	NA

TABLE 11-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

MORGAN RIVER WATERSHED

Water Quality Basin ID	North WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Morgan River 1	50	2.08E+07	St. Helena Sound	8,367	2,736	300
Morgan River 2	51	2.90E+07	Morgan River 1	4,895	3,637	75
Morgan River 3	52	1.16E+07	Morgan River 2	2,581	3,653	150
Morgan River 4	53	9.34E+06	Morgan River 3	2,432	3,138	900
Morgan River 5	54	3.76E+06	Morgan River 4	1,312	3,170	900
Morgan River 6	55	3.62E+05	Morgan River 5	293	2,559	150
Village Creek 1	56	3.50E+06	Morgan River 1	720	7,177	150
Village Creek 2	57	7.11E+05	Village Creek 1	703	5,262	150
Village Creek 3	58	5.86E+04	Village Creek 2	117	1,915	150
Coffin Creek 1	59	9.25E+05	Morgan River 1	475	4,924	150
Coffin Creek 2	60	9.56E+04	Coffin Creek 1	247	2,752	150
Eddings Point Creek 1	61	1.89E+06	Morgan River 2	927	3,025	900
Eddings Point Creek 2	62	1.41E+06	Eddings Point Creek 1	465	2,929	25
Eddings Point Creek 3	63	2.51E+05	Eddings Point Creek 2	215	2,044	25
Eddings Point Creek Trib. 1	64	1.07E+06	Eddings Point Creek 1	194	4,828	20
Parrot Creek 1	65	2.82E+06	Morgan River 2	1,653	1,352	75
Parrot Creek 2	66	2.65E+06	Parrot Creek 1	1,535	1,271	75
			Coosaw River 1	2,112	1,271	0
Bass Creek 1	67	1.96E+06	Parrot Creek 1	363	2,575	150
Bass Creek 2	68	1.79E+05	Bass Creek 1	409	1,239	150
Jenkins Creek 1	69	5.58E+06	Morgan River 3	823	5,439	150
Jenkins Creek 2	70	1.90E+06	Jenkins Creek 1	898	5,311	150
Doe Point Creek 1	71	3.40E+05	Jenkins Creek 1	281	1,320	150
Boatswain Pond	72	1.96E+05	Morgan River 3	231	1,899	150
Jenkins Creek Warsaw Flats 1	73	1.36E+06	Morgan River 4	641	2,189	75
Jenkins Creek Warsaw Flats 2	74	7.97E+05	Jenkins Creek Warsaw Flats 1	710	1,754	75
Lucy Point Creek South 1	75	1.46E+06	Morgan River 4	879	1,802	75
Lucy Point Creek South 2	76	1.66E+06	Lucy Point Creek South 1	582	1,915	75
			Lucy Point Creek North 2	249	1,642	300
Rock Springs Creek 1	77	5.81E+05	Lucy Point Creek 2	280	3,379	300
Rock Springs Creek 2	78	9.80E+04	Rock Springs Creek 1	92	2,205	300

TABLE 11-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMM FOR MORGAN RIVER WATER QUALITY BASINS

Water Quality Basin ID	North WASP Segment	EXISTING LAND USE Flow (cfs)	EXISTING LAND USE Fecal Coliform (#/100 ml)
Morgan River 1	50	6.7	1,070
Morgan River 2	51	12.4	1,057
Morgan River 3	52	9.6	1,075
Morgan River 4	53	11.1	1,085
Morgan River 5	54	12.1	1,153
Morgan River 6	55	0.9	1,072
Village Creek 1	56	7.8	1,032
Village Creek 2	57	4.1	997
Village Creek 3	58	4.3	902
Coffin Creek 1	59	4.1	1,070
Coffin Creek 2	60	1.4	886
Eddings Point Creek 1	61	3.9	1,036
Eddings Point Creek 2	62	4.0	1,090
Eddings Point Creek 3	63	1.5	886
Eddings Point Creek Trib. 1	64	3.8	1,061
Parrot Creek 1	65	5.5	1,027
Parrot Creek 2	66	2.2	1,075
Bass Creek 1	67	4.2	1,071
Bass Creek 2	68	1.1	1,078
Jenkins Creek 1	69	7.0	1,098
Jenkins Creek 2	70	7.3	1,054
Doe Point Creek 1	71	1.5	1,079
Boatswain Pond	72	1.9	1,300
Jenkins Creek Warsaw Flats 1	73	2.6	1,100
Jenkins Creek Warsaw Flats 2	74	4.6	1,039
Lucy Point Creek South1	75	2.7	992
Lucy Point Creek South 2	76	2.1	1,060
Rock Springs Creek 1	77	4.2	1,261
Rock Springs Creek 2	78	3.1	1,285

TABLE 11-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES

MORGAN RIVER WATERSHED

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs) Existing
Morgan River 1	St. Helena Sound	804
Morgan River 2	Morgan River 1	775
Morgan River 3	Morgan River 2	362
Morgan River 4	Morgan River 3	335
Morgan River 5	Morgan River 4	13
Morgan River 6	Morgan River 5	0.9
Village Creek 1	Morgan River 1	16
Village Creek 2	Village Creek 1	8.4
Village Creek 3	Village Creek 2	4.4
Coffin Creek 1	Morgan River 1	5.5
Coffin Creek 2	Coffin Creek 1	1.4
Eddings Point Creek 1	Morgan River 2	13
Eddings Point Creek 2	Eddings Point Creek 1	6.4
Eddings Point Creek 3	Eddings Point Creek 2	4.2
Eddings Point Creek Trib. 1	Eddings Point Creek 1	1.2
Parrot Creek 1	Morgan River 2	388
Parrot Creek 2	Parrot Creek 1	379
Coosaw River 1	Parrot Creek 2	374
Bass Creek 1	Parrot Creek 1	5.3
Bass Creek 2	Bass Creek 1	3.8
Jenkins Creek 1	Morgan River 3	16
Jenkins Creek 2	Jenkins Creek 1	7.0
Doe Point Creek 1	Jenkins Creek 1	7.3
Boatswain Pond	Morgan River 3	1.6
Jenkins Creek Warsaw Flats 1	Morgan River 4	4.8
Jenkins Creek Warsaw Flats 2	Jenkins Creek Warsaw Flats 1	2.1
Lucy Point Creek South 1	Morgan River 4	306
Lucy Point Creek South 2	Lucy Point Creek South 1	302
Lucy Point Creek North 2	Lucy Point Creek South 2	292
Rock Springs Creek 1	Lucy Point Creek South 2	7.3
Rock Springs Creek 2	Rock Springs Creek 1	4.6

TABLE 11-16

FECAL COLIFORM MODELING RESULTS

MORGAN RIVER WATERSHED

Water Quality	Bacteria	Modeled Geomean Conc (#/100 ml)	Modeled Level of Service
Basin ID	Loss Rate (1/day)	Existing	Existing
Morgan River 1	0.0	5.4	A
Morgan River 2	0.0	4.2	A
Morgan River 3	0.0	5.1	A
Morgan River 4	0.0	5.4	A
Morgan River 5	0.0	5.8	A
Morgan River 6	0.0	7.4	B
Village Creek 1	0.0	5.9	A
Village Creek 2	0.0	11.5	D
Village Creek 3	0.0	21.7	D
Coffin Creek 1	0.0	8.9	C
Coffin Creek 2	0.0	10.6	D
Eddings Point Creek 1	0.0	4.5	A
Eddings Point Creek 2	0.0	11.8	D
Eddings Point Creek 3	0.0	12.3	D
Eddings Point Creek Trib. 1	0.0	8.8	C
Parrot Creek 1	0.0	4.1	A
Parrot Creek 2	0.0	3.6	A
Bass Creek 1	0.0	5.4	A
Bass Creek 2	0.0	5.8	A
Jenkins Creek 1	0.0	6.2	A
Jenkins Creek 2	0.0	9.1	C
Doe Point Creek 1	0.0	6.8	A
Boatswain Pond	0.0	7.9	B
Jenkins Creek Warsaw Flats 1	0.0	6.8	A
Jenkins Creek Warsaw Flats 2	0.0	8.6	B
Lucy Point Creek South1	0.0	7.2	B
Lucy Point Creek South 2	0.0	8.7	C
Rock Springs Creek 1	0.0	15.5	D
Rock Springs Creek 2	0.0	22.9	D

NOTE: Water quality basins with lower LOS are highlighted.

Tables 11-17, 11-18, and 11-19 are not applicable in the update.

TABLE 11-20 (Updated 2017)

PLANNING LEVEL COST ESTIMATES FOR

MORGAN RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
CNC_M-6	Road overtopping at Langford Road Replace existing 1 - 24" RCP with 1 - 8'x4' box culvert Raise road 1.1 ft (length of 620 ft)	$267,000
FC_M-3	Road overtopping at Holly Hall Road Replace existing 1 - 24" RCP with 3 - 8'x4' box culverts	$226,000
RSC_M-3 *	Road overtopping at Sams Point Road Replace existing 1 - 36" RCP with 1 - 8'x6' box culvert	$117,000
RSC_M-5	Road overtopping at Wade Hampton Drive Replace existing 1 - 24" RCP with 1 - 8'x4' box culvert	$111,000
	TOTAL	$721,000

* Conduits marked by asterisk are on private land

Costs are in January 2018dollars.

See Appendix for basis of cost estimates.

Table 11-21 is not applicable in the update.

Figure 11-1

Morgan River Watershed

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Figure 11-2. Hydrologic Model Subbasins

Morgan River Watershed

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Figure 11-3

Morgan River Watershed
Water Quality Basins

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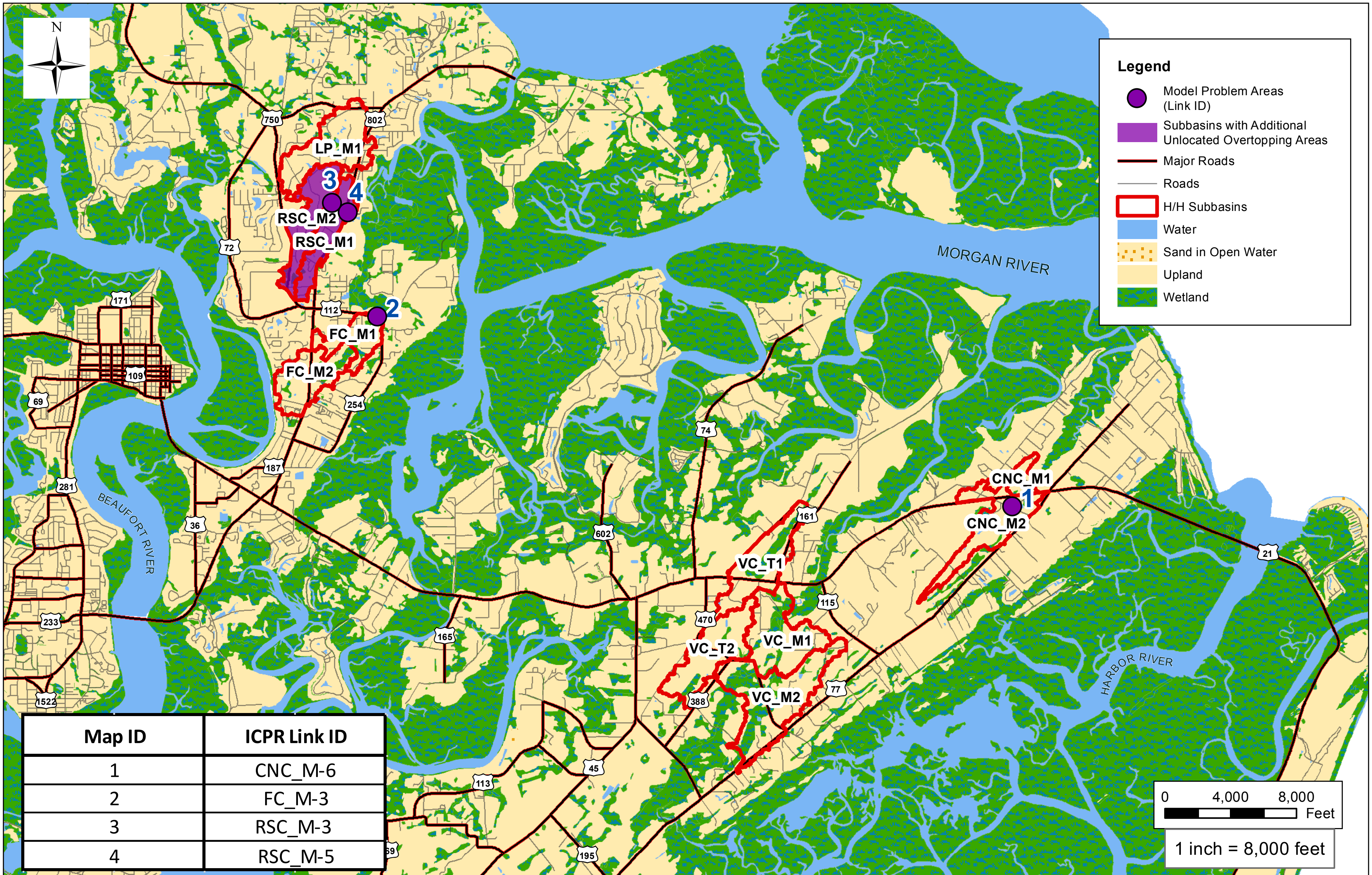

Map ID	ICPR Link ID
1	CNC_M-6
2	FC_M-3
3	RSC_M-3
4	RSC_M-5

Figure 11-4. ICPR Identified PSMS Overtopping Problem Areas

Morgan River Watershed

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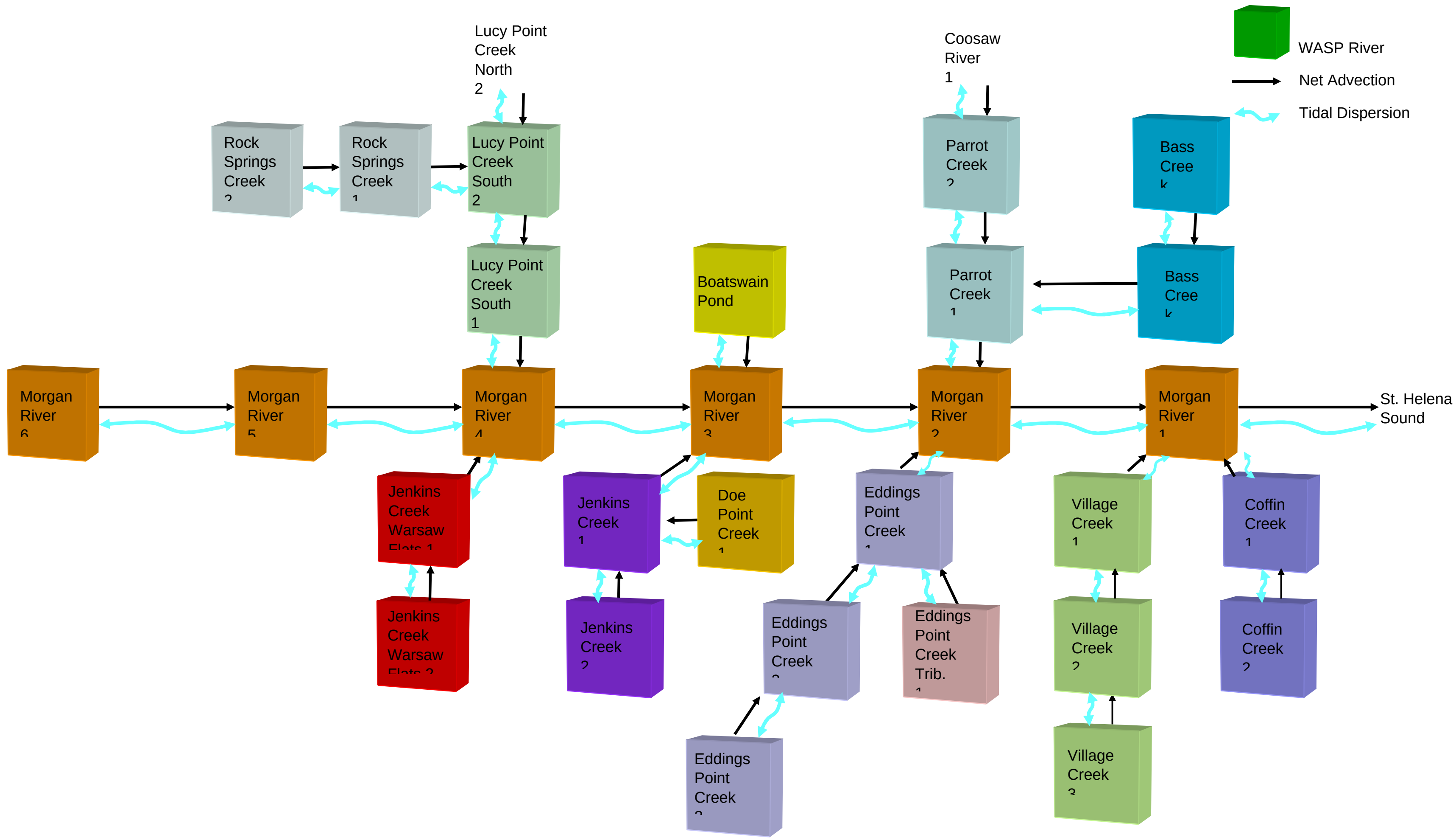

Figure 11-5
WASP Model Schematic for Morgan River Watershed

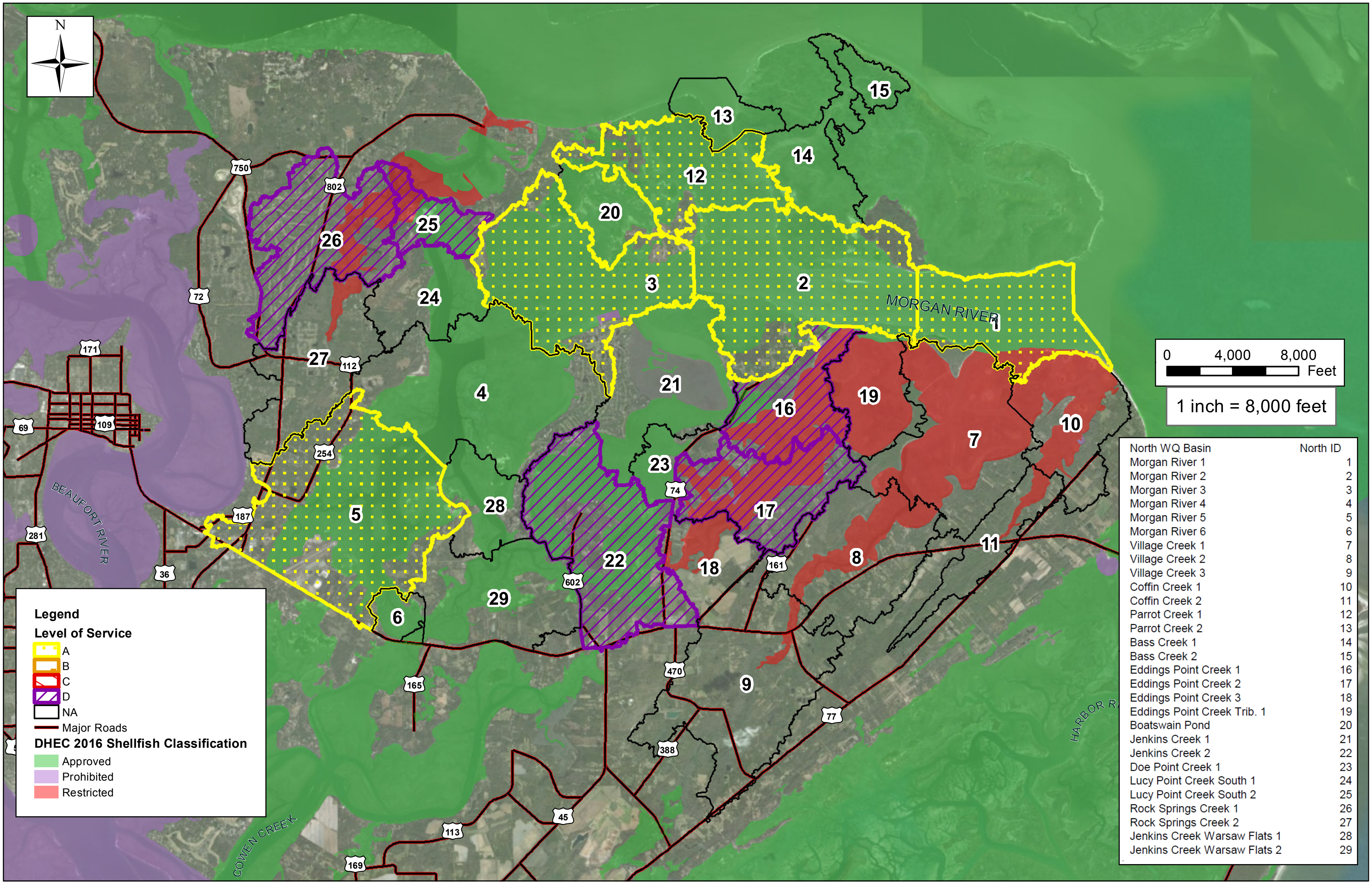

Figure 11-6

Morgan River Watershed
Shellfish Classification and Existing Level of Service

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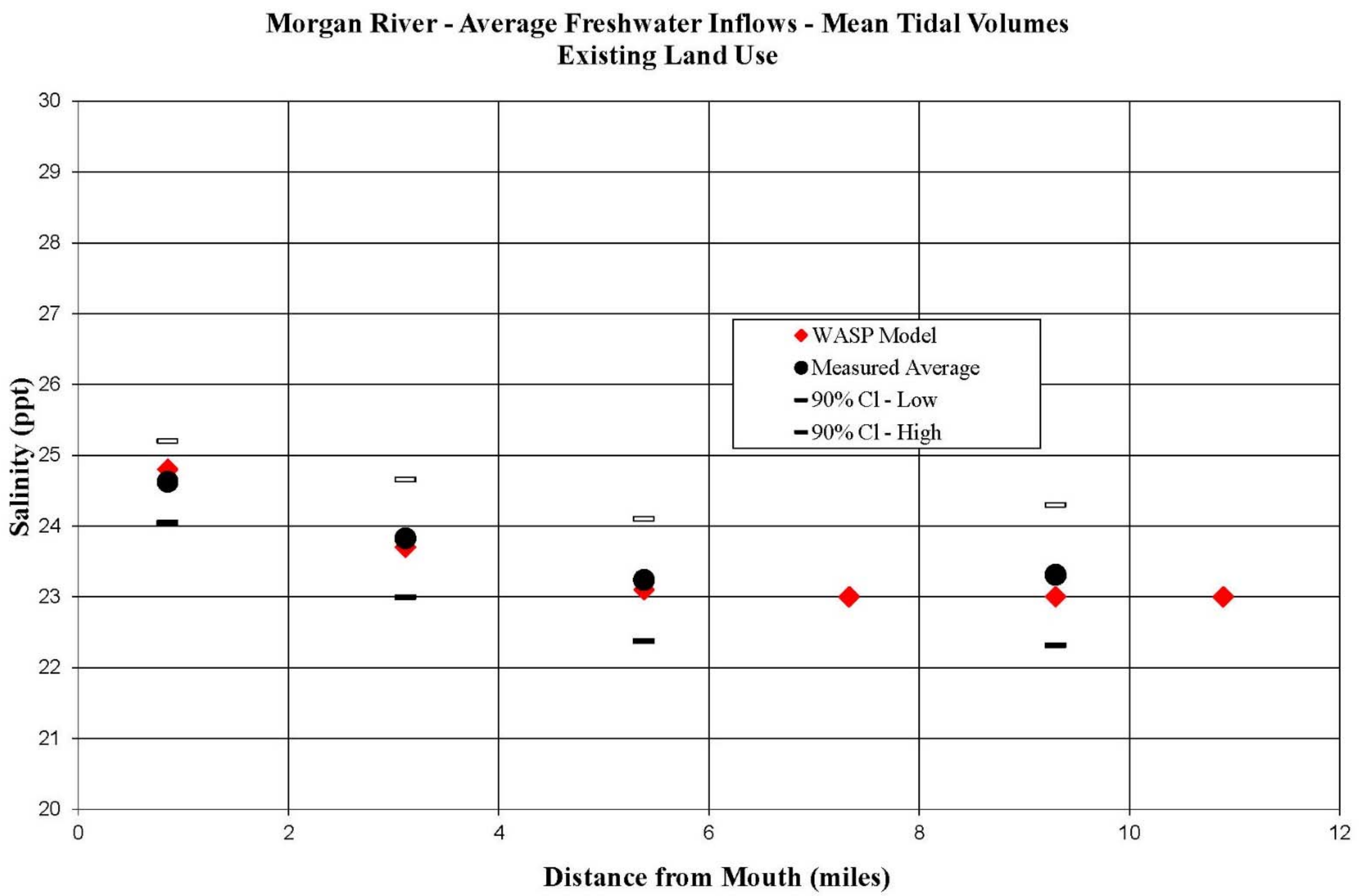

Figure 11-7. Comparison of WASP Model Results with Long-Term Monitoring Data in Morgan River - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

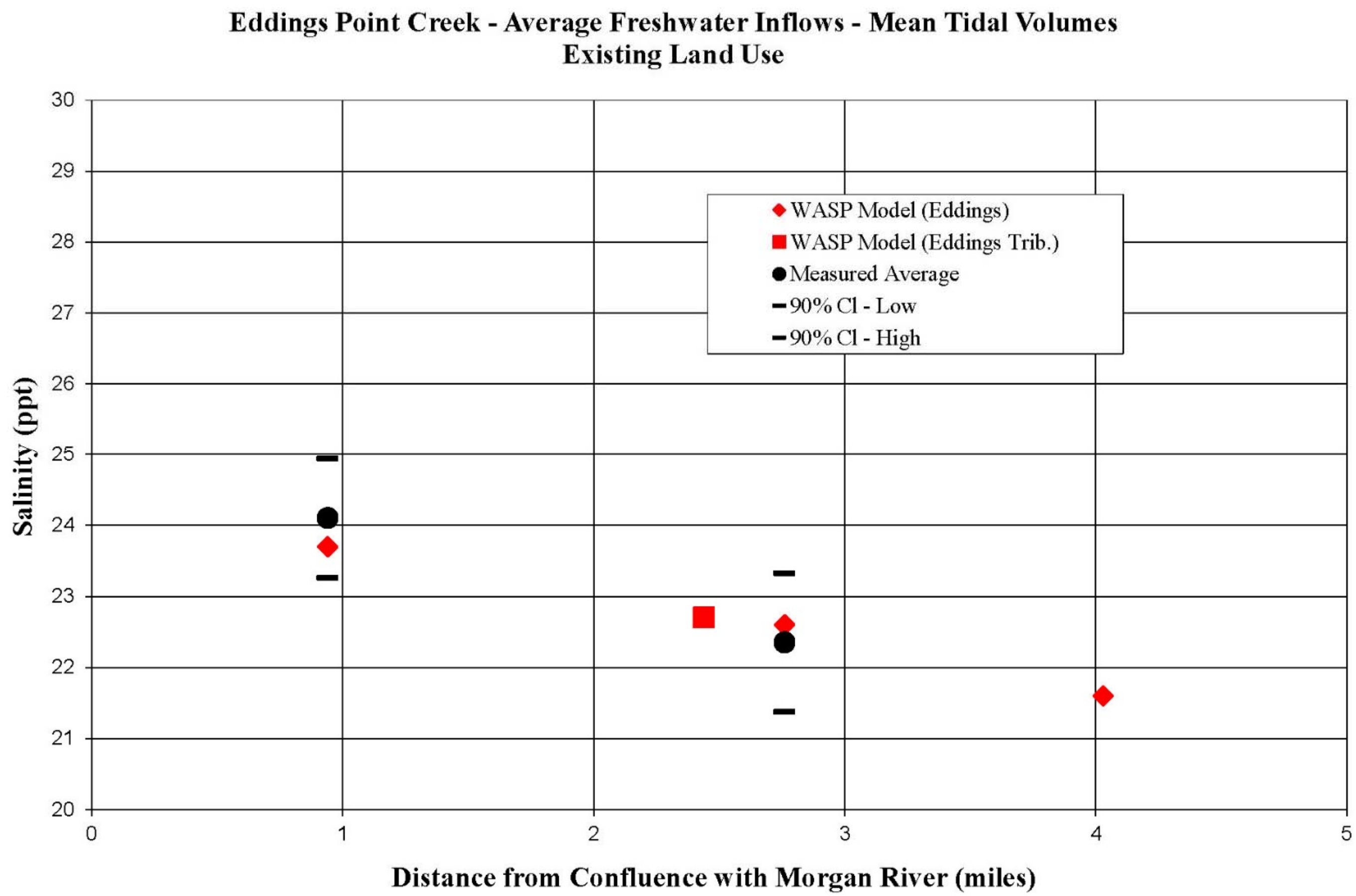

Figure 11-8. Comparison of WASP Model Results with Long-Term Monitoring Data in Eddings Point Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

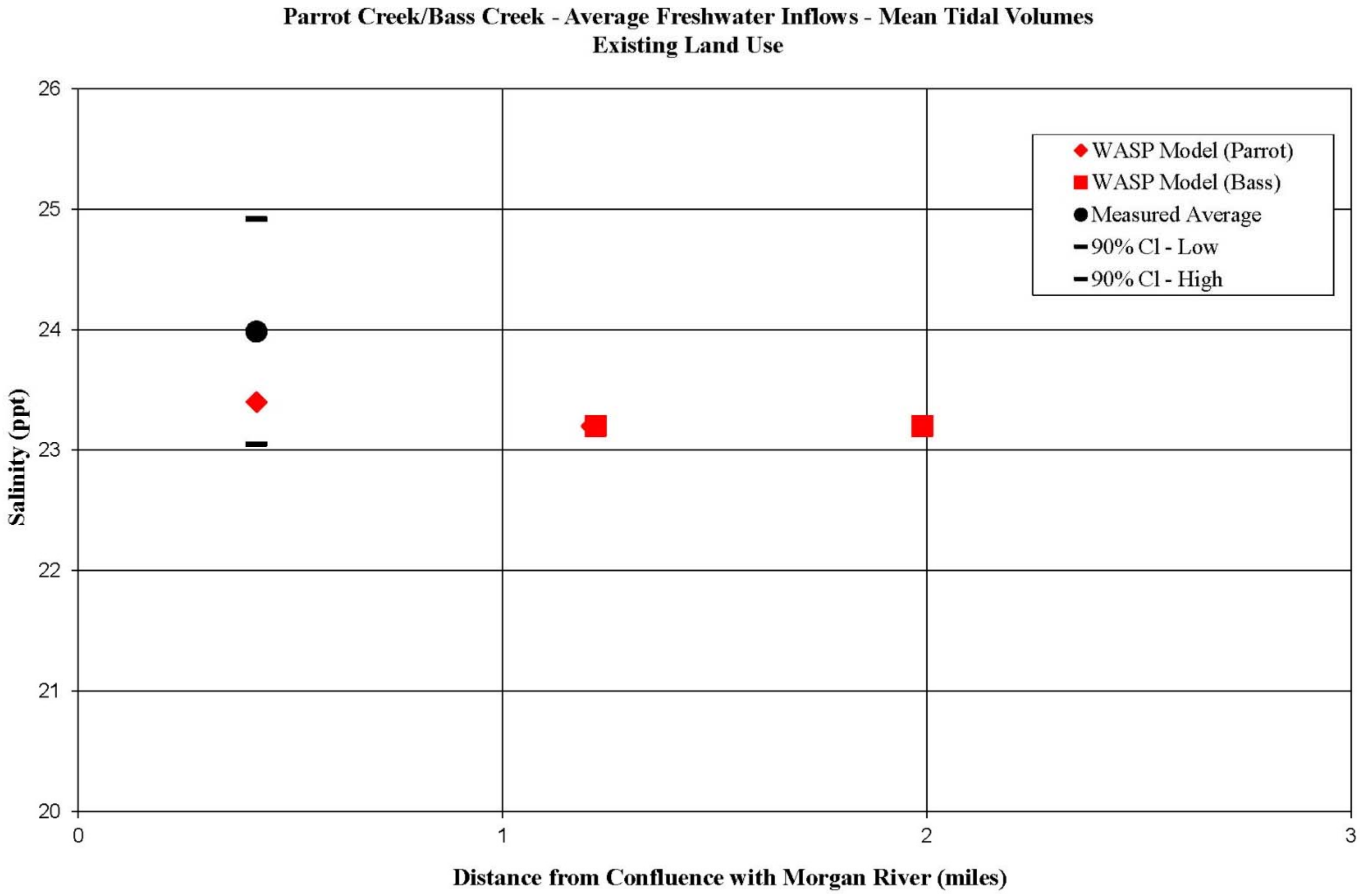

Figure 11-9. Comparison of WASP Model Results with Long-Term Monitoring Data in Parrot Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

ATM DESIGN ENGINEERING CONSULTING

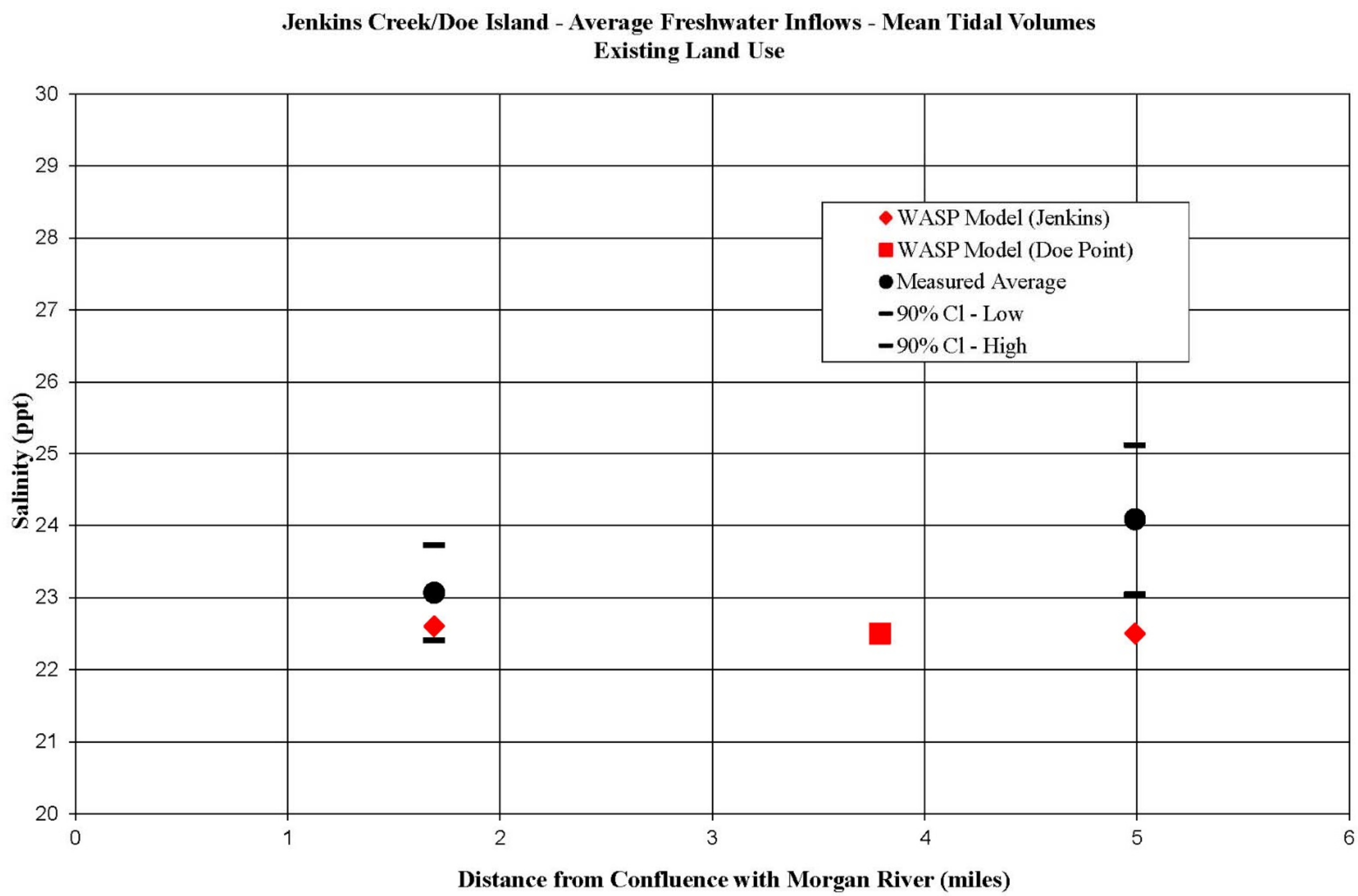

Figure 11-10. Comparison of WASP Model Results with Long-Term Monitoring Data in Jenkins and Doe Point Creeks - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

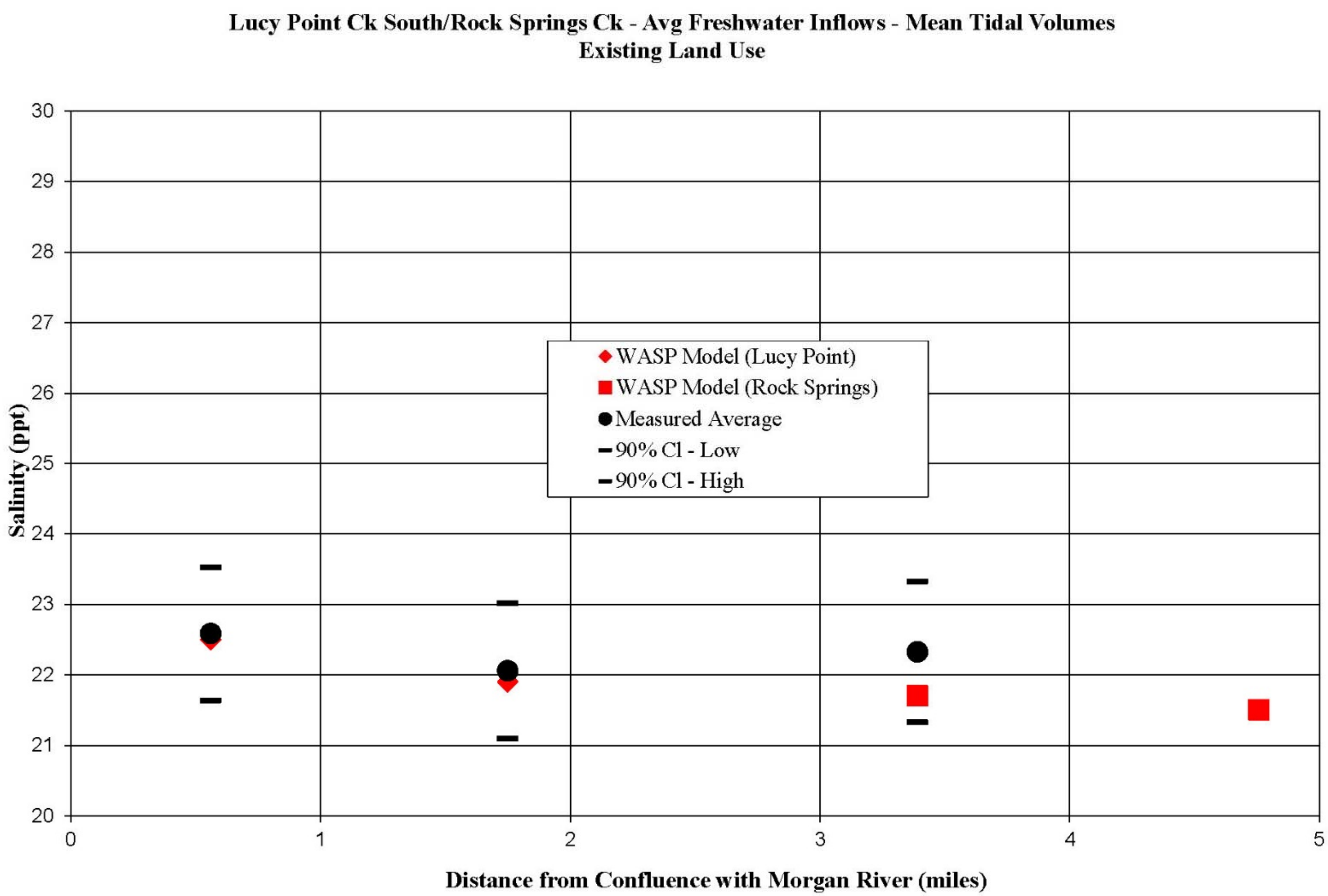

Figure 11-11. Comparison of WASP Model Results with Long-Term Monitoring Data in Lucy Point South and Rock Springs Creeks - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data

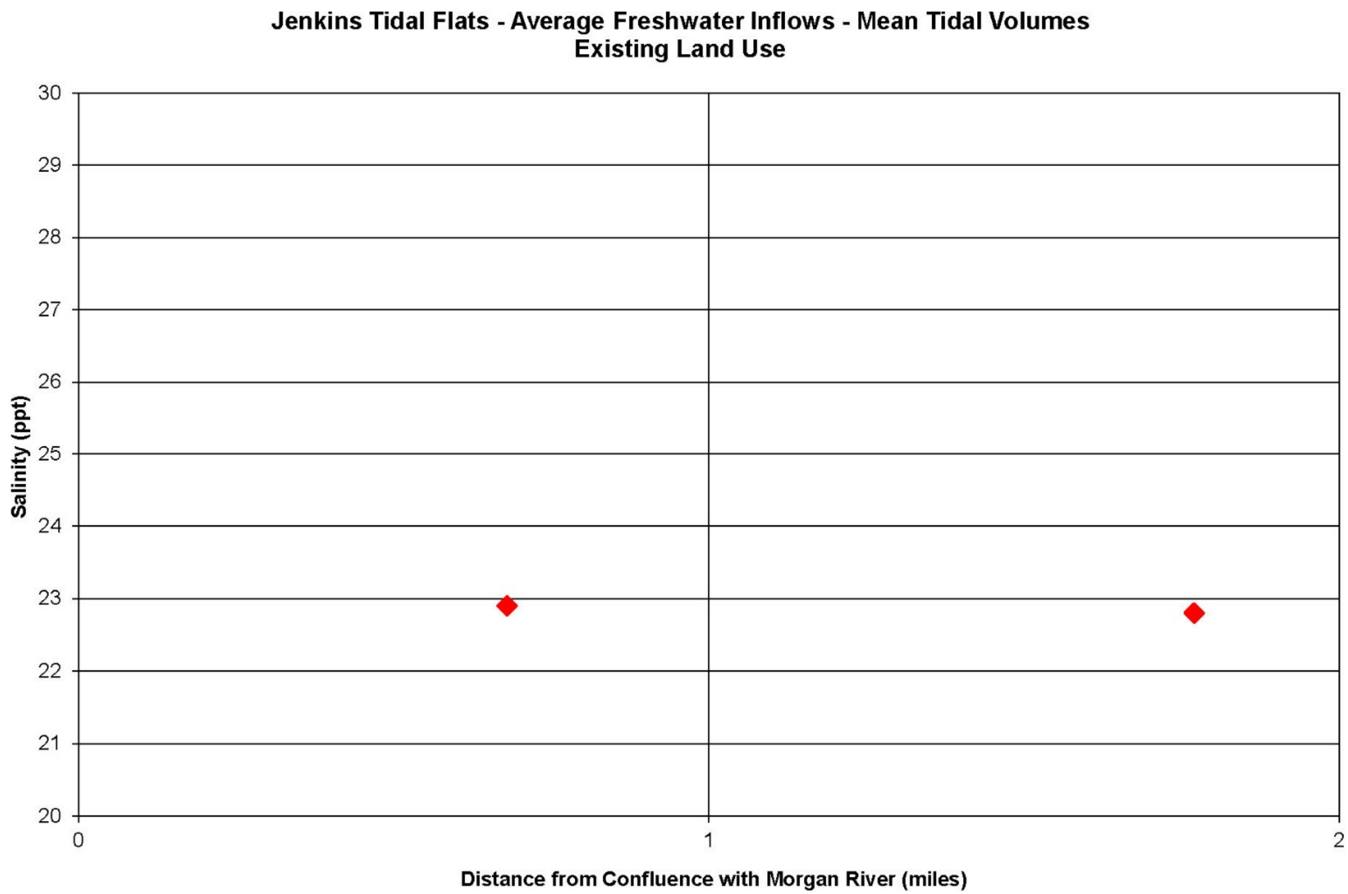

Figure 11-12. Comparison of WASP Model Results with Long-Term Monitoring Data in Jenkins Tidal Flats - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data

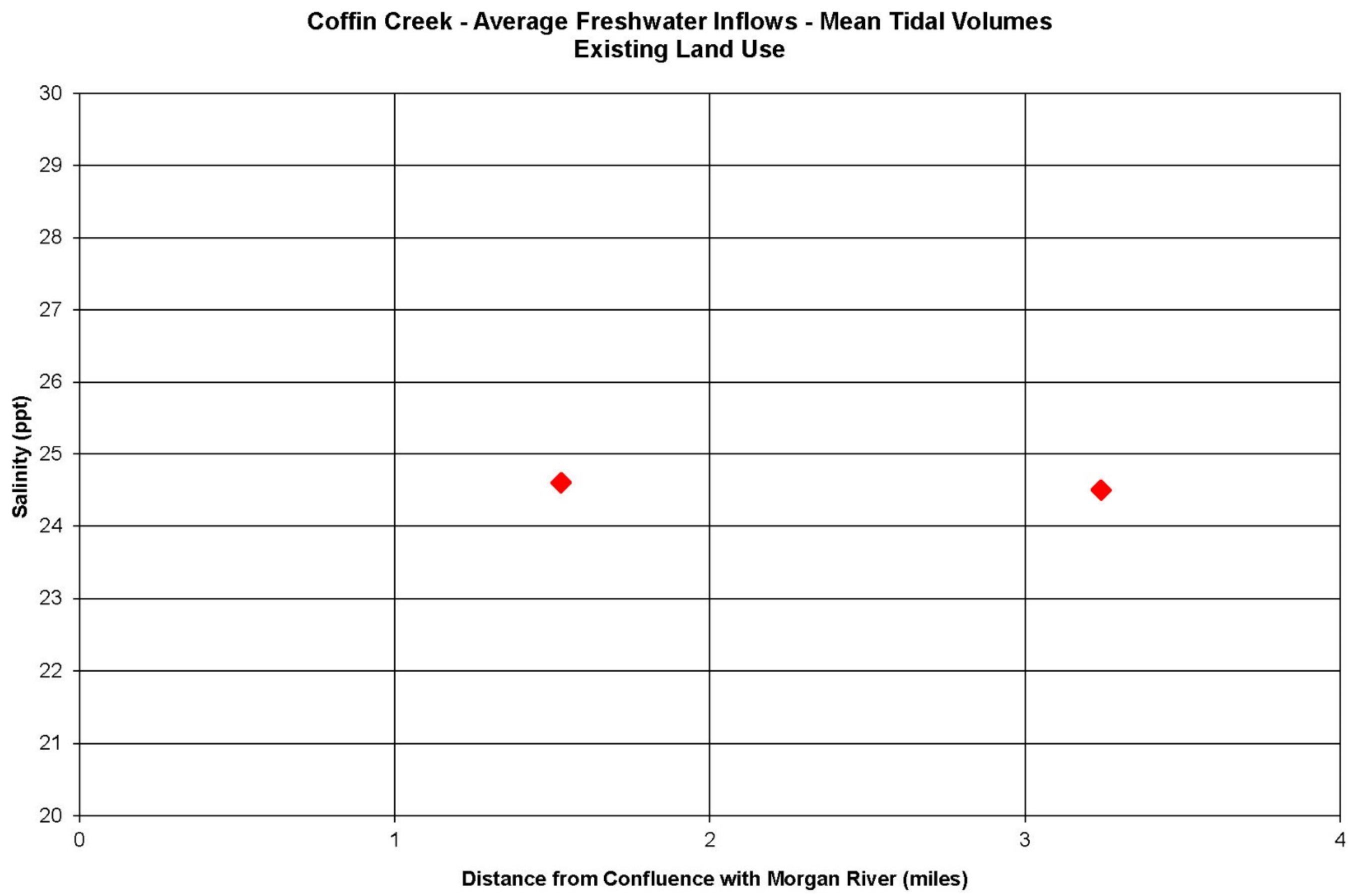

Figure 11-13. Comparison of WASP Model Results with Long-Term Monitoring Data in Coffin Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data

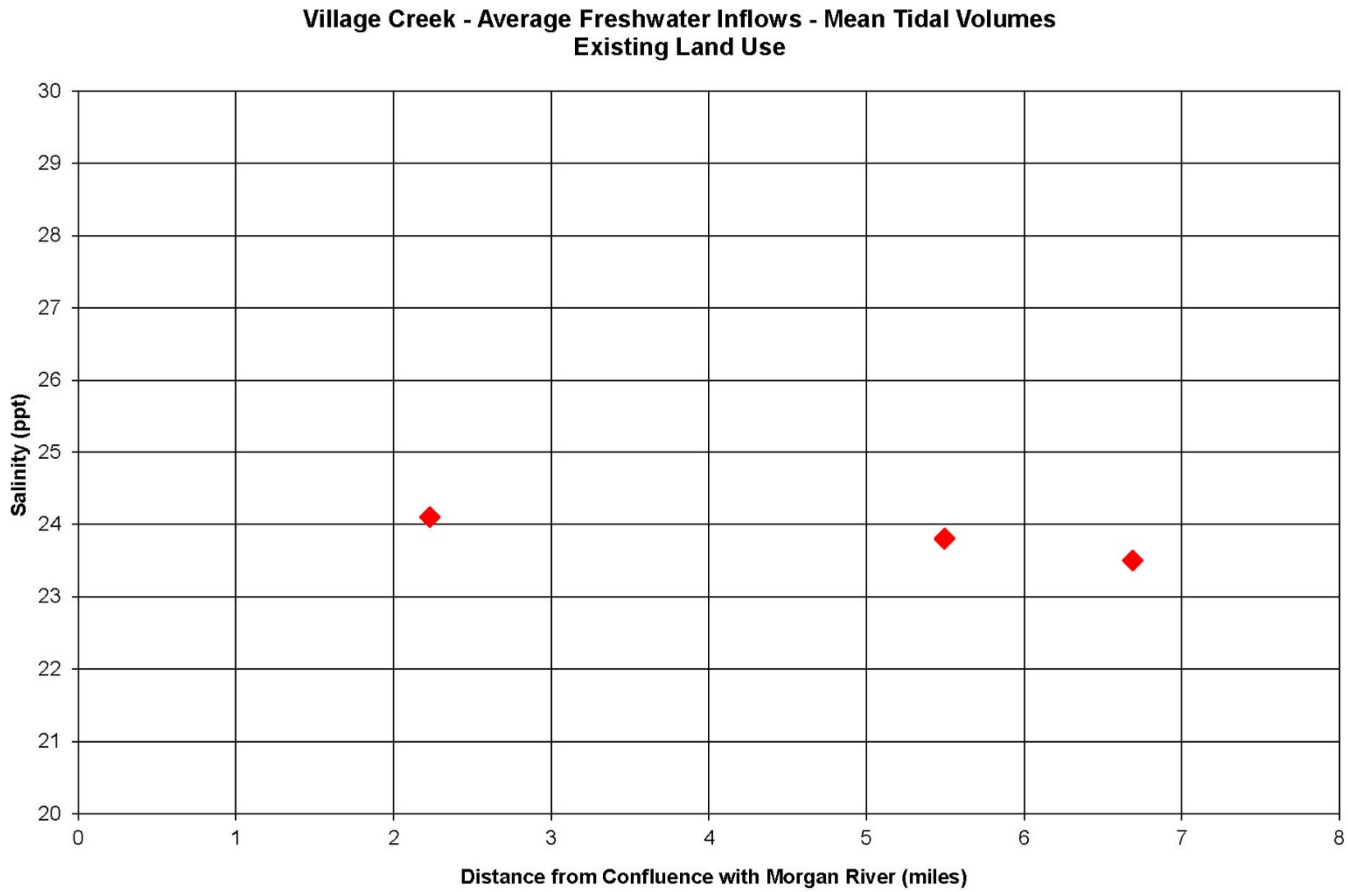

Figure 11-14. Comparison of WASP Model Results with Long-Term Monitoring Data in Village Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data

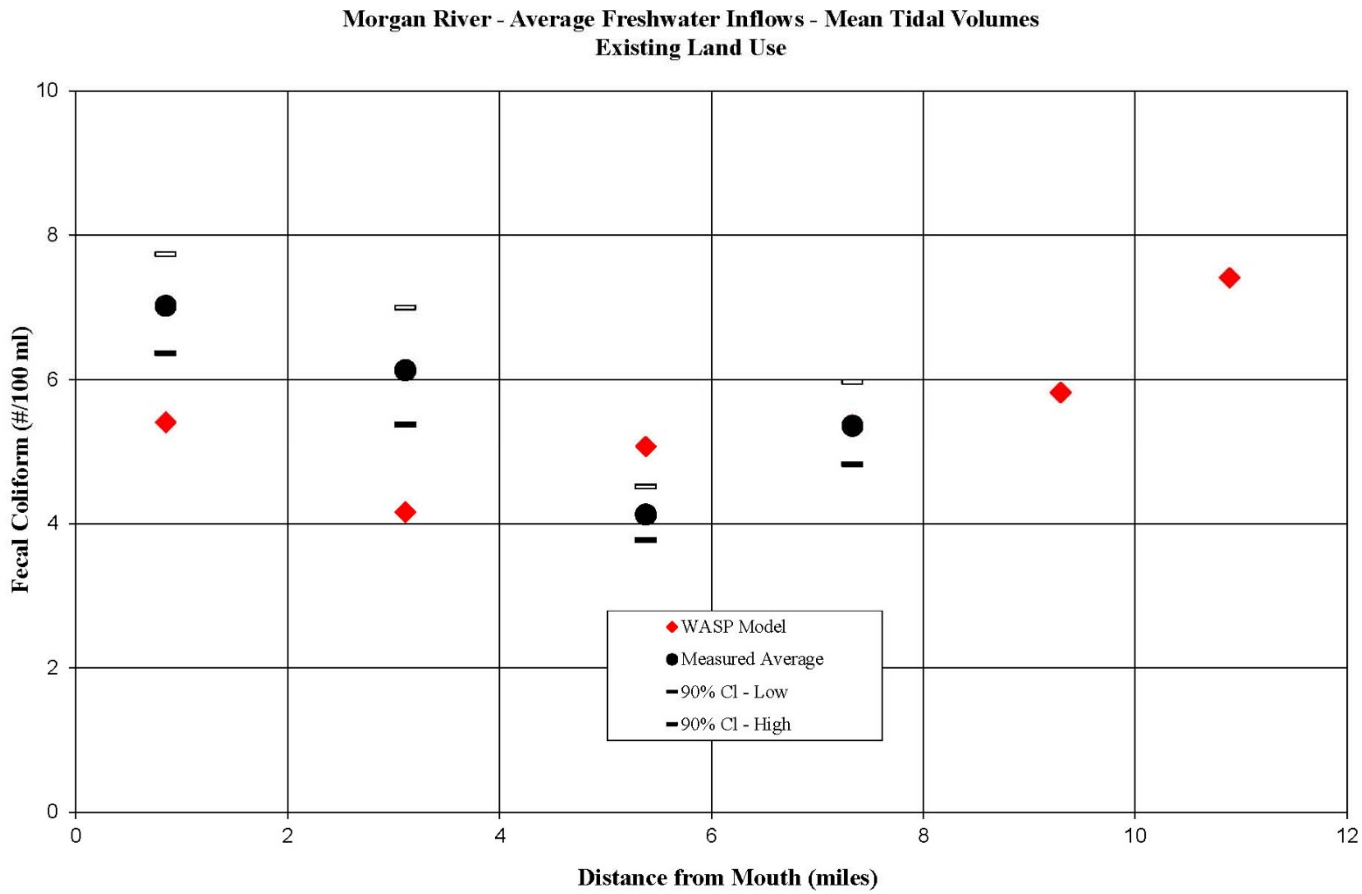

Figure 11-15. Comparison of WASP Model Results with Long-Term Monitoring Data in Morgan River - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

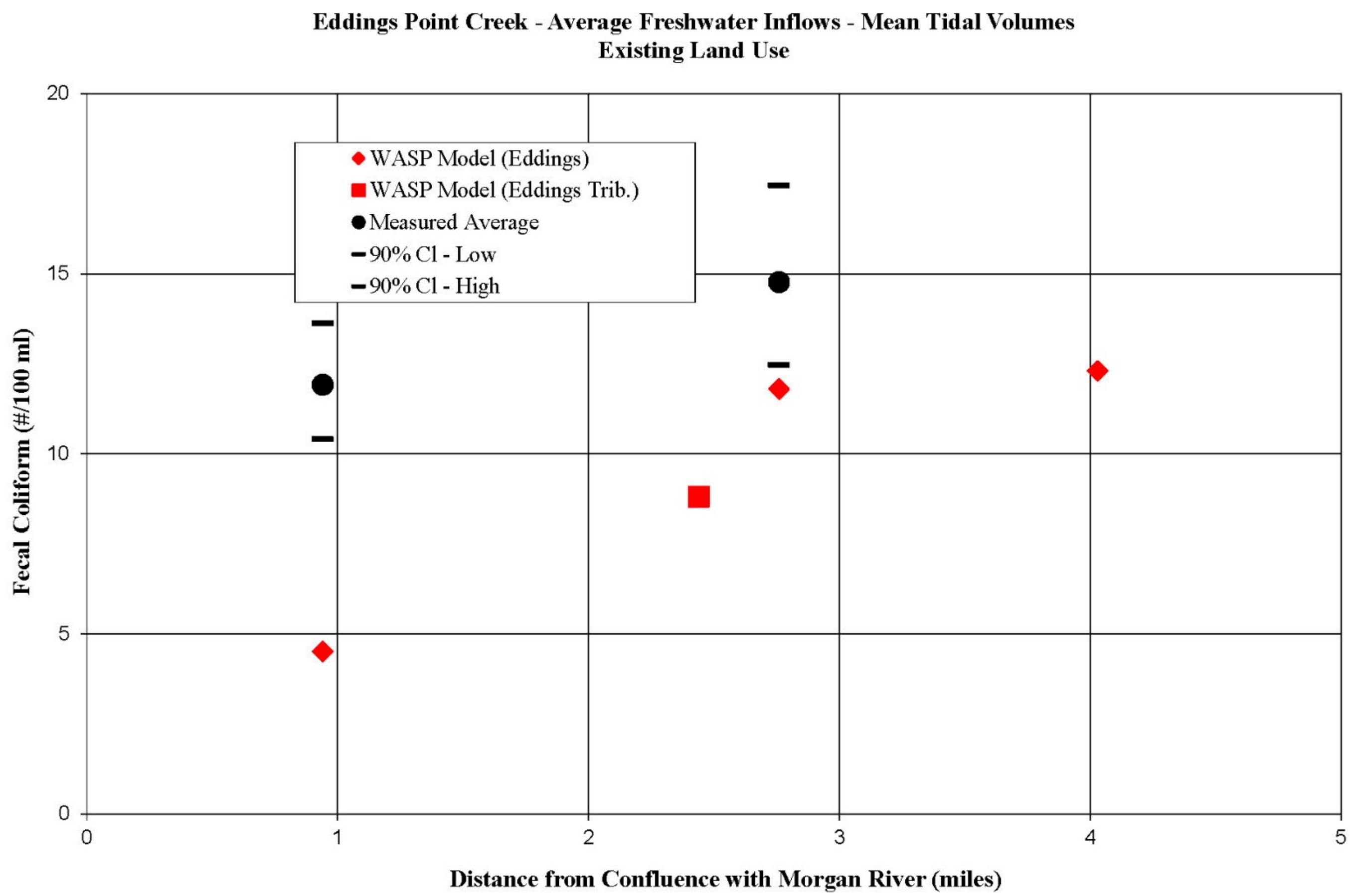

Figure 11-16. Comparison of WASP Model Results with Long-Term Monitoring Data in Eddings Point Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

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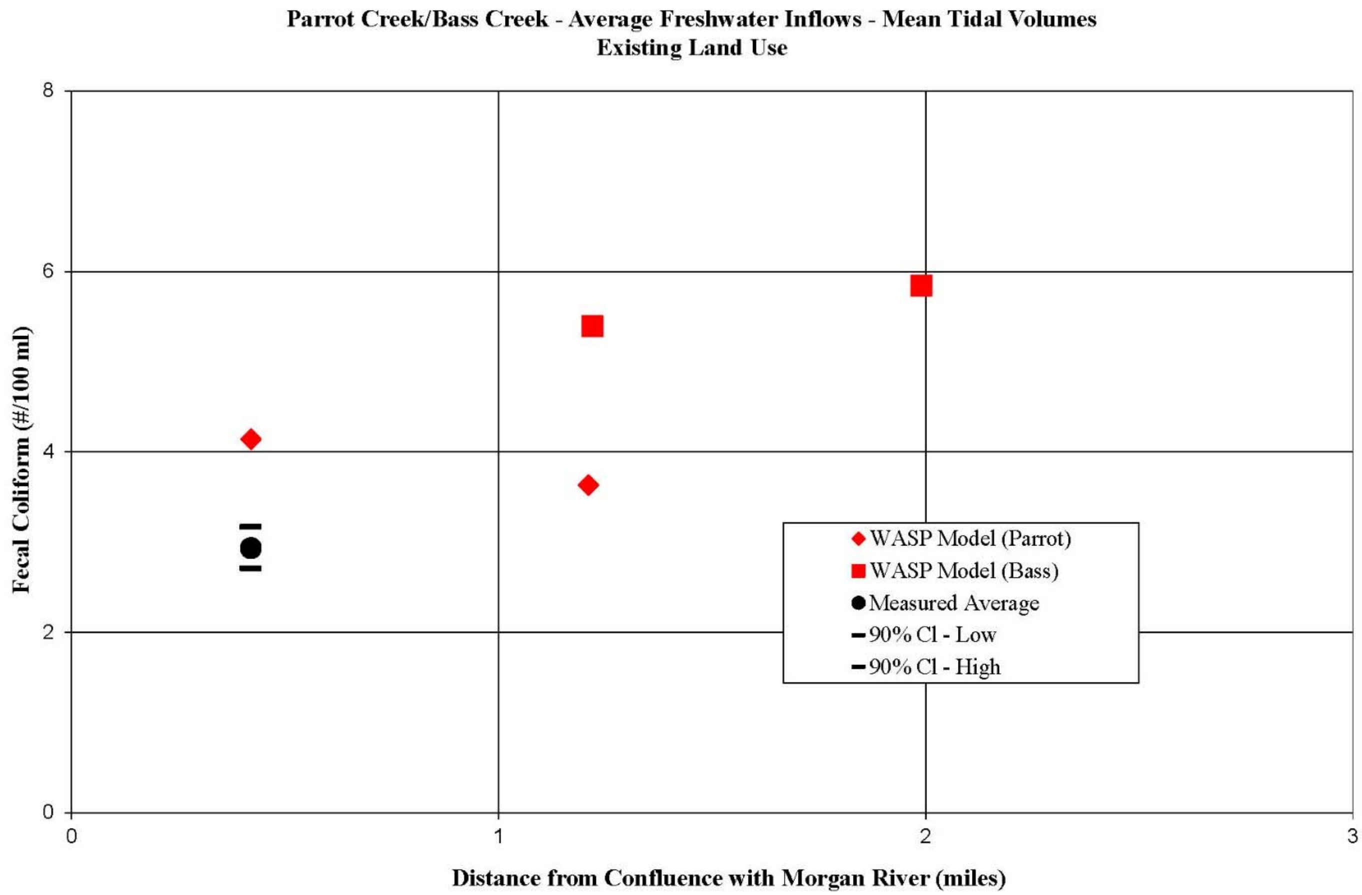


Figure 11-17. Comparison of WASP Model Results with Long-Term Monitoring Data in Parrot Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

ATM DESIGN ENGINEERING CONSULTING

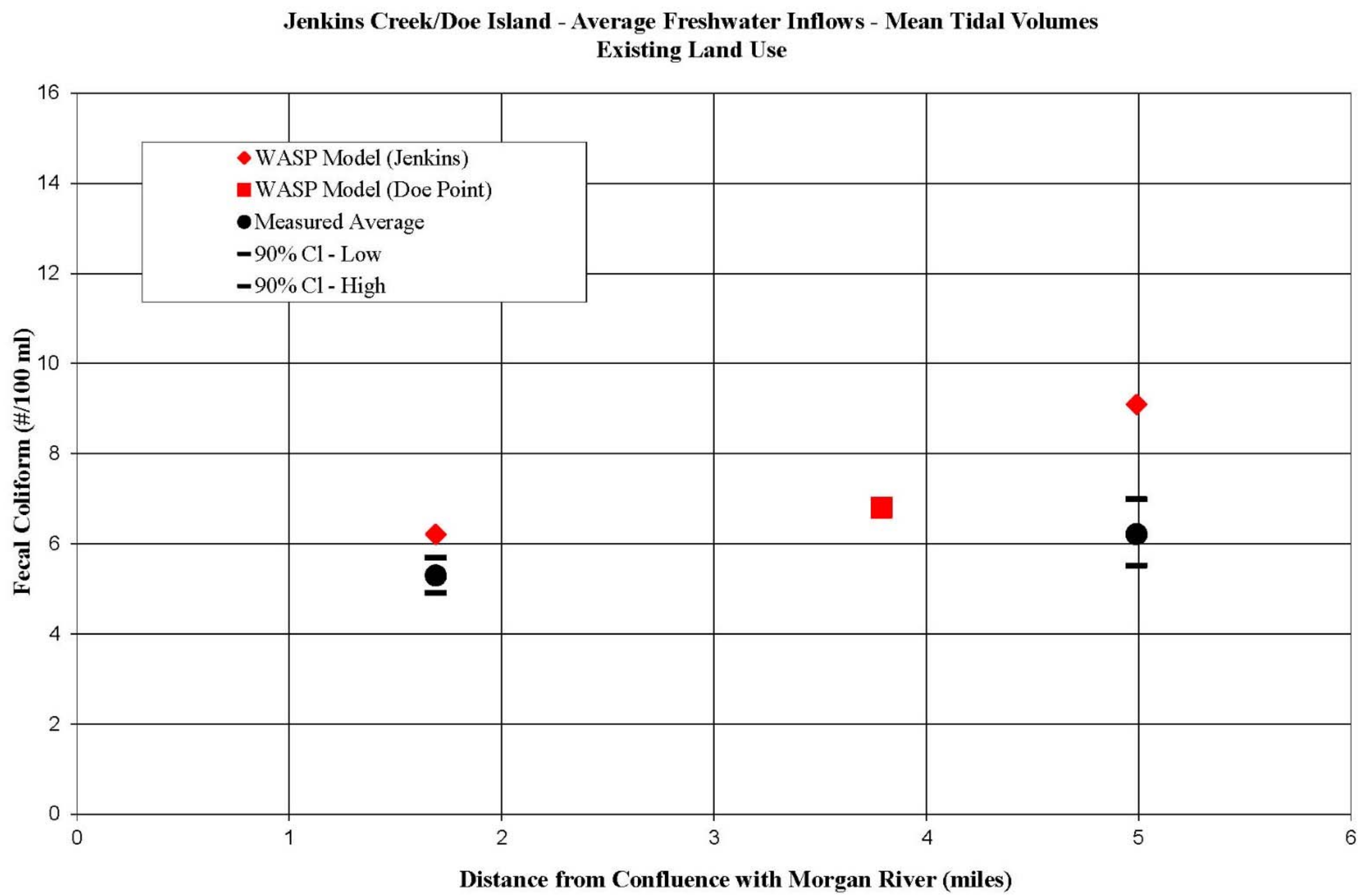


Figure 11-18. Comparison of WASP Model Results with Long-Term Monitoring Data in Jenkins and Doe Point Creeks - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

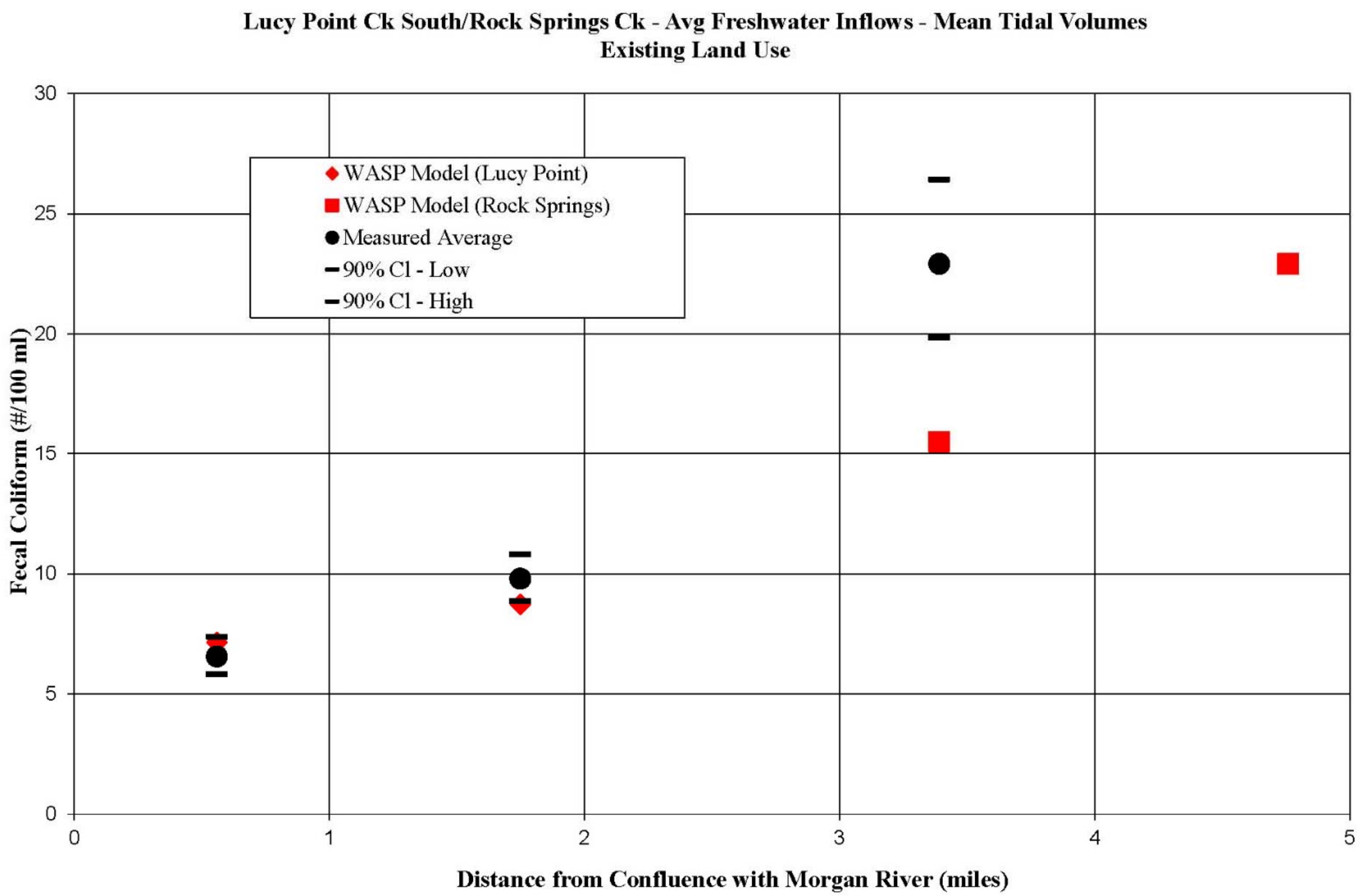


Figure 11-19. Comparison of WASP Model Results with Long-Term Monitoring Data in Lucy Point South and Rock Springs Creeks - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

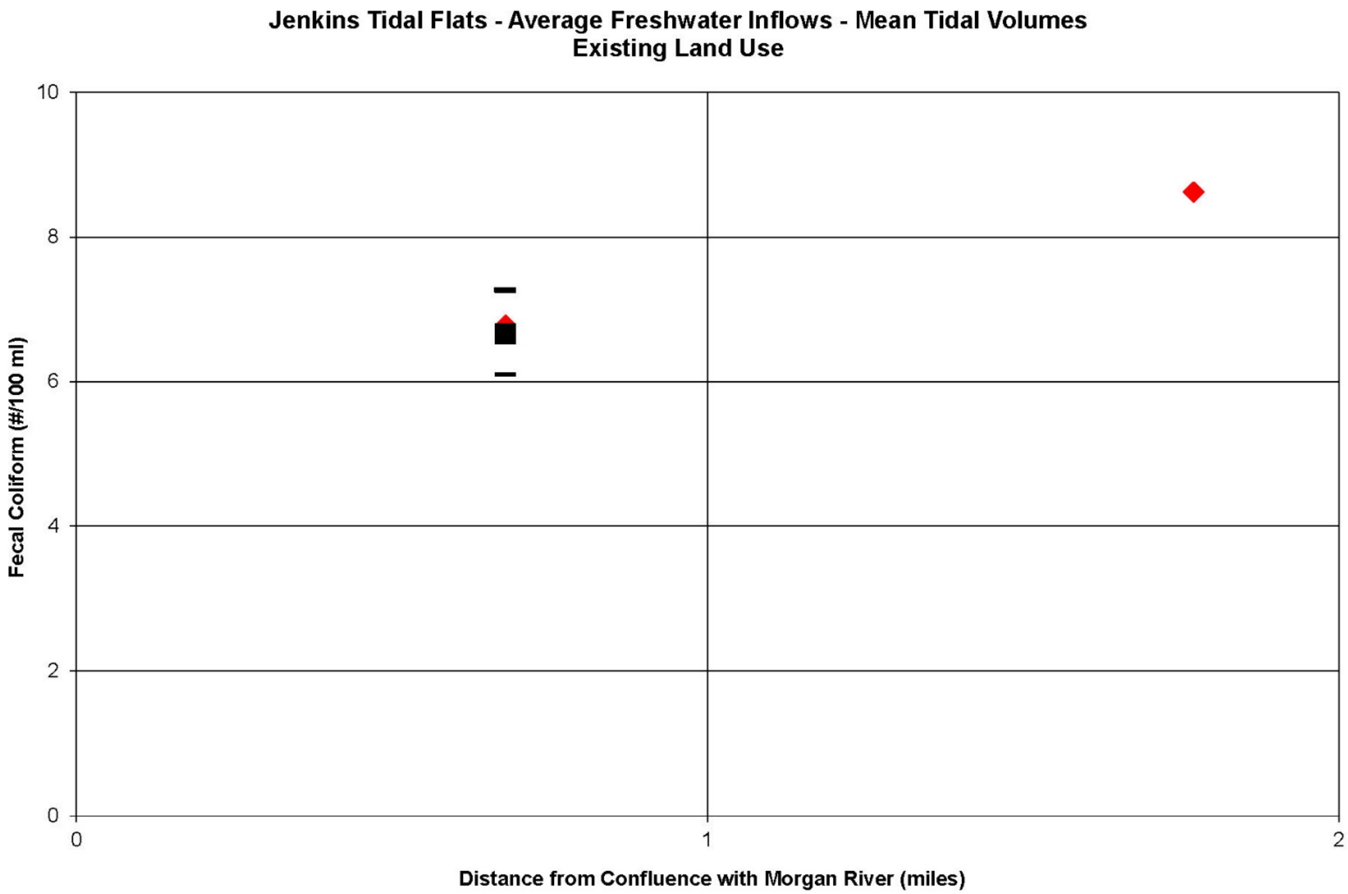


Figure 11-20. Comparison of WASP Model Results with Long-Term Monitoring Data in Jenkins Tidal Flats - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

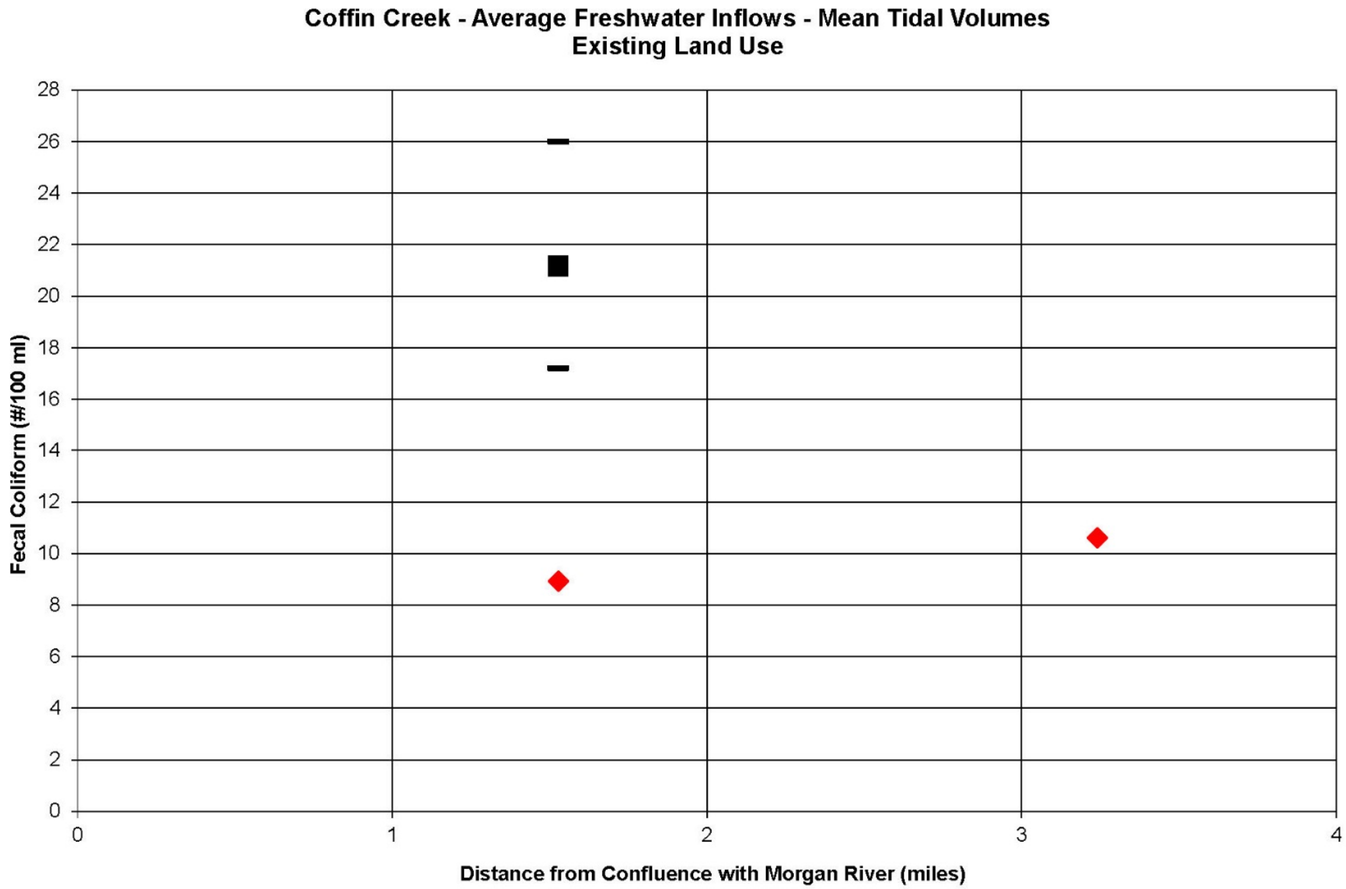


Figure 11-21. Comparison of WASP Model Results with Long-Term Monitoring Data in Coffin Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

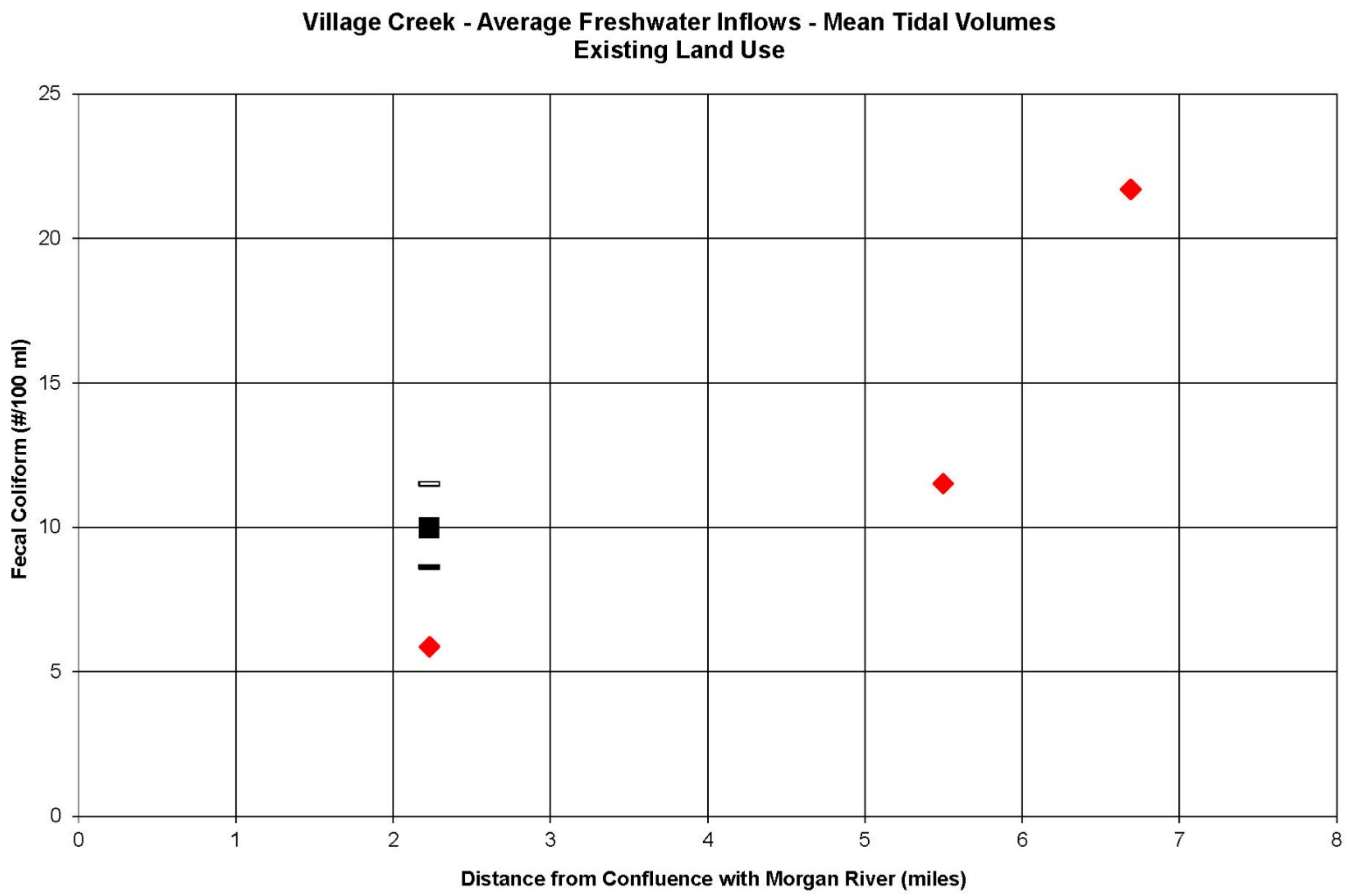
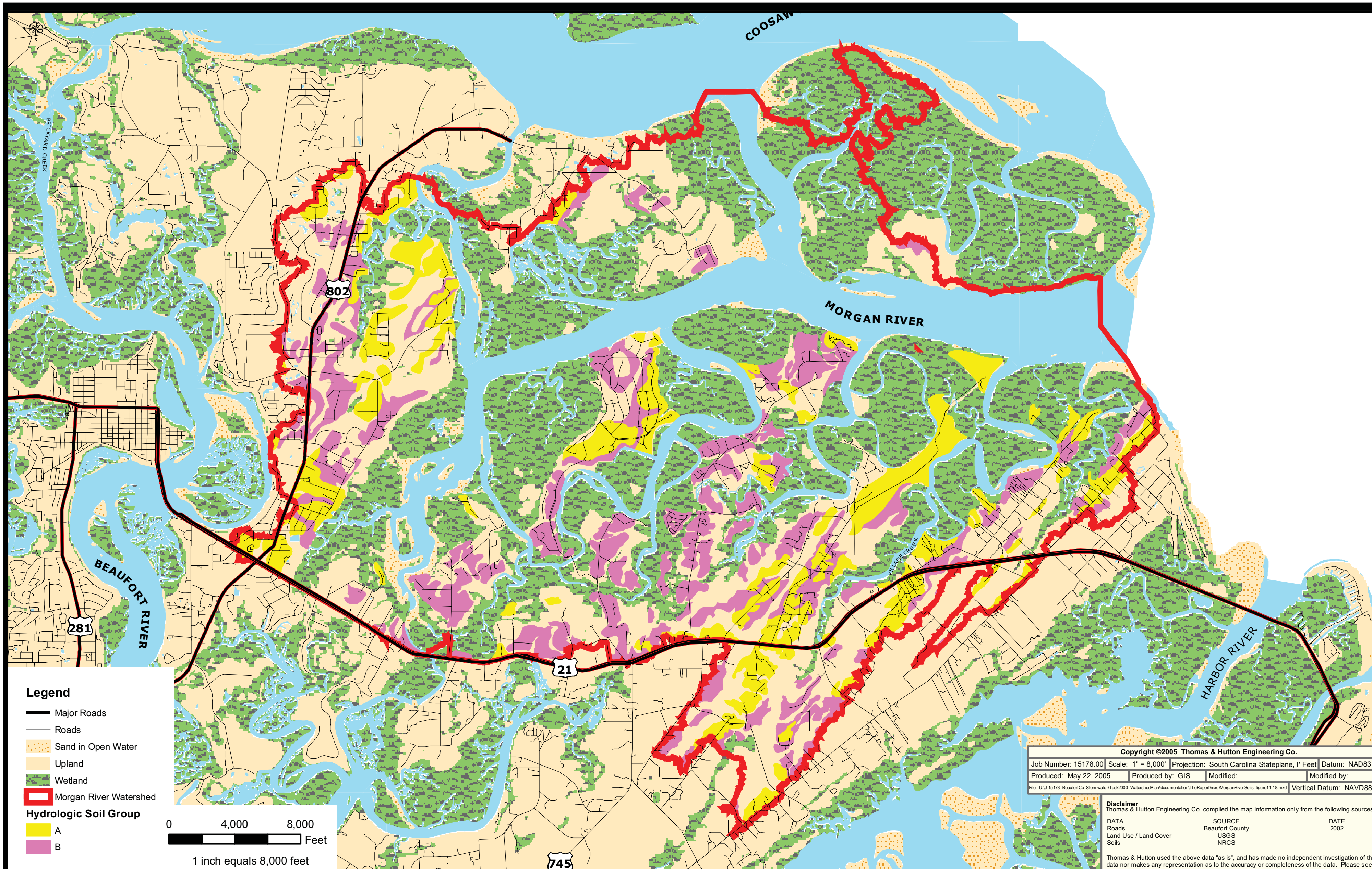


Figure 11-22. Comparison of WASP Model Results with Long-Term Monitoring Data in Village Creek - Bacteria

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

Figure 11-23 is not applicable in the update.



Disclaimer
Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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SAVANNAH, GEORGIA 31405
(912) 234-5300

Morgan River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

CDM Camp Dresser & McKee Inc.

Figure 11-24

Section 12
Broad River Watershed Analysis

This section describes the physical features of the Broad River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

12.1 Overview

For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area in Sheldon Township, Port Royal Island, Bluffton Township and the Town of Hilton Head Island that is tributary to the Broad River (see **Figure 12-1**).

For comparative purposes, the entire tributary area for the New River is presented in **Figure 12-2**. The figure indicates Beaufort County makes up only a small fraction of the total tributary area to the Broad River.

For the hydrologic and hydraulic analysis of the Primary Stormwater Management System (PSMS), the watershed includes several "hydrologic" basins. These are listed in **Table 12-1**, and presented in **Figure 12-3**. Table 12-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were completed to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into "water quality" basins. These are listed in **Table 12-2**, and presented in **Figure 12-4**. Pollution loads were calculated for each of the water quality basins. Unlike some of the other watersheds, the vast majority of the Broad River tributary area is actually located outside of Beaufort County. Because loads from Beaufort County are such a small fraction of the total load to the Broad River, and loads from outside the County are unknown, tidal river water quality model calculations were not done for the Broad River.

12.2 Hydrologic and Hydraulic Analysis

CDM and T&H used the Interconnected Pond Routing Model (ICPR), Version 3 for the hydrologic and hydraulic analyses of the PSMS in the Broad River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were conducted for existing and future land use conditions, with and without alternative management strategies.

The ICPR model is a "link-node" model, representing the PSMS as a series of nodes (stream locations) connected by links (open channels, pipes, culverts). Figures in Appendix J show model schematics of the Broad River PSMS basins, with a separate schematic for each basin.

12.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Broad River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include hydrologic basin area, curve number, and time of concentration.

Table 12-3 lists the hydrologic parameter values for the Broad River PSMS subbasins. Each model subbasin is identified by an ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development.

Hydraulic summary information for the Broad River PSMS basins is presented in **Table 12-4**. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in **Table 12-5**. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate level of service.

Details regarding specific open channel segments, storage areas, weirs and tide gates are presented in Appendix J.

12.2.3 Model Results

Tables in Appendix J list the peak flow values for the Broad River subbasins. Each table lists peak flows for one of the return periods analyzed in this study, which include 2-year, 12-year, 25-year, and 100-year return periods. In each of the tables, the peak flows are listed by subbasin for various land cover and stormwater management controls, which include the following:

- Undeveloped land
- Existing land use without peak shaving controls
- Existing land use with existing peak shaving controls
- Future land use without peak shaving controls

- Future land use with existing and future peak shaving controls

It should be noted that the tables include values for "uncontrolled" and "controlled" peak flows for the 2-year, 12-year and 25-year design storms. The "uncontrolled" peak flow assumes no peak shaving facilities in the subbasin. In contrast, the "controlled" value accounts for peak shaving facilities in the subbasin.

For existing land use, aerial maps and local information were used to estimate the percentage of existing urban development that is served by peak shaving facilities. The "controlled" peak flow value was then calculated by considering the difference in peak flow between totally undeveloped conditions and existing conditions with no controls. For example, suppose that a subbasin of 100 acres has an undeveloped 2-year peak flow of 20 cfs, and an uncontrolled existing peak flow of 50 cfs, and further suppose that 60 percent of the urban development is controlled by peak shaving facilities. In this case, it is assumed that the existing peak flow is reduced by 60 percent of the difference between undeveloped and developed peak flow (50 – 20 = 30 cfs; 60 percent of 30 cfs = 18 cfs reduction due to peak shaving), and therefore the maximum controlled peak flow will be 32 cfs (50 – 18).

For future land use, the "controlled" peak flow is set equal to the "controlled" peak flow for existing land use, because new development is subject to State and County peak flow regulations. Note, however, that the future condition will still generate more stormwater runoff volume, even though the peak flow is the same. The result is that the peak flow rate will be sustained for a longer period of time under future conditions.

Other tables in Appendix J list the peak water elevation values for model node locations along Broad River PSMS. Each table lists peak stages for one of the return periods analyzed in this study, which include 2-year, 12-year, 25-year, and 100-year return periods. In each of the tables, the peak stages are listed for existing and future land use conditions, with the existing stormwater hydraulic system.

Specific problem areas identified by the modeling are listed in **Table 12-6** and presented in **Figure 12-5**. For each area, the table identifies the road crossing, associated model ID, design storm, "critical elevation" (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

Structural flooding was also considered for the 100-year design storm. Structural flooding was also considered for the 100-year design storm. In locations where the PSMS road crossings classified as evacuation routes are overtopped by the 100-year design storm, figures were developed showing the approximate area of inundation upstream of the overtopped road. These figures are presented in Appendix J. In addition, the modeled peak 100-year water elevations were compared to Federal Emergency Management Agency (FEMA) base flood elevations, and results showed that the FEMA elevations (based on storm surge) are always greater than the modeled 100-year peak stages, suggesting that structures built in accordance with the FEMA base flood elevations should not be flooded.

Table 12-6 indicates that seventeen road crossings are being overtopped by the design storm events. Most of the problem areas are located on Port Royal Island, particularly in the Laurel Bay South and Broad River Boulevard basins.

Evaluation of solutions to prevent these problems is discussed in the next section of this report.

12.2.4 Management Strategy Alternatives

The problems areas listed in Table 12-6 were evaluated by modifying the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in **Table 12-7**. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

12.3 Water Quality Analysis

CDM and T&H used the Watershed Management Model (WMM) for the water quality analysis of the Broad River watershed. WMM was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, total nitrogen (total N), total phosphorus (total P), BOD, lead, zinc and total suspended solids (TSS).

12.3.1 Land Use and BMP Coverage

Table 12-8 presents the existing land use and future land use estimates for the Broad River water quality basins. The existing land use data were gathered from a number of sources, including February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps, plus local knowledge of development completed between February 2002 and June 2003. The future land use map was developed by "filling in" the existing land use map and by replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and the Town of Hilton Head Island future land use maps and zoning maps.

Under existing land use conditions, 21 percent of the Broad River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 79 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 7 percent of the watershed.

Under future land use conditions, 24 percent of Broad River watershed area consists of urban systems, and 76 percent consists of natural systems. The major change in land use distribution is the conversion of forest/rural land to urban land uses. As a result of projected future development, urban imperviousness increases to about 9 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in **Table 12-9**. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by BMPs.

Under existing land use conditions, 23 percent of the urban systems in the watershed are served by BMPs. Under future land use conditions, 42 percent of the urban systems are served by BMPs. This increase from existing to future reflects both the increase in urban land use and the 100 percent coverage of the new development with BMPs in accordance with the County BMP Manual.

12.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in **Table 12-10**. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water & Sewer Authority. For future development, areas that are zoned "rural" or "conservation" were assumed to be served by septic tanks, and other areas were assumed to be served by sewer.

Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 55 percent of the urban systems in the watershed are served by septic. Under future land use conditions, 46 percent of the urban systems are served by septic tanks.

Based on available data, the estimated wastewater discharge under existing conditions is 0.9 million gallons per day (mgd) of land application (e.g., golf course irrigation) and 0.5 mgd of direct discharge, and the future discharge is expected to be 1.1 mgd of indirect discharge and 0.6 mgd of direct discharge, based on increase in residential land between existing and future conditions.

12.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Broad River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in **Table 12-11** for existing and future land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

An overall comparison of the WMM modeling results (Table 12-11) indicates that future flows and constituent loads generally increase marginally over their existing counterparts. Specifically, future flow is 2 percent greater than for existing conditions and the increase in loads ranges from 4 percent for BOD to -1 percent (decrease) for fecal coliform bacteria. It should also be noted that the increases for several constituents (e.g., total N, zinc) are limited because direct rainfall on the open water/tidal wetland area provides a significant fraction of the total load to the Broad River.

12.3.4 Management Strategy Alternatives

Besides the enforcement of the BMP Manual requirements for new development (and maintenance of existing BMPs), no specific recommendations are made for the Broad River watershed. There is only a small increase in impervious cover and annual loads when comparing existing and future conditions.

For informational purposes, the areas with "A" and "B" type soils are presented in **Figure 12-6**. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

12.4 Planning Level Cost Estimates for Management Alternatives

Table 12-12 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Broad River watershed. As shown in the table, the 17 projects are estimated to have a total cost of $3.3 million in December 2004 dollars. Details of the cost estimate for each project are shown in Appendix J.

The prioritization of these projects, and projects identified in other watersheds, is discussed in Section 16 of this report.

TABLE 12-1
HYDROLOGIC BASINS
BROAD RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Baynard	490	2	245
Brays Island East	397	1	397
Broad River Blvd	1,126	3	375
Habersham Creek North	1,490	4	373
Habersham Creek South	244	1	244
Habersham Creek West	414	2	207
Laurel Bay South	1,588	5	318
Pocotaligo South	317	1	317
Scotts Neck North	654	2	327
Scotts Neck West	376	1	376
Tomotley	1,997	8	250
Yemassee West	1,682	5	336
TOTAL	10,775	35	308

TABLE 12-2
WATER QUALITY BASINS
BROAD RIVER WATERSHED

Basin Name	Tributary Area (acres)
Broad River 1	15,549
Broad River 2	19,034
Broad River 3	18,572
Broad River 4	15,939
TOTAL	69,094

TABLE 12-3
HYDROLOGIC SUBBASIN CHARACTERISTICS
BROAD RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use Curve Number	Existing Land Use Time of Concentration (minutes)	Future Land Use Curve Number	Future Land Use Time of Concentration (minutes)
Baynard					
BD_M1	252	82	95	86	84
BD_M2	238	84	86	86	80
Brays Island East					
BIE_M1	397	83	103	83	103
Broad River Blvd					
BRB_M1	509	84	112	88	98
BRB_M2	293	89	80	93	69
BRB_M3	324	82	94	86	82
Habersham Creek North					
HCN_M1	306	79	131	81	122
HCN_M2	278	74	114	77	105
HCN_M3	389	80	146	84	129
HCN_M4	233	79	121	84	104
HCN_T1	283	80	139	84	119
Habersham Creek South					
HCS_m1	244	76	128	80	114
Habersham Creek West					
HCW_M1	132	72	126	78	105
HCW_M2	282	78	98	82	88
Laurel Bay South					
LBS_M1	206	70	110	73	102
LBS_M2	204	76	101	79	92
LBS_M3	166	86	73	88	68
LBS_M4	442	82	116	91	84
LBS_M5	570	84	120	90	95
Pocotaligo South					
PS_M1	317	83	134	83	134
Scotts Neck North					
SNN_M1	460	80	126	82	116
SNN_M2	194	87	68	88	66
Scotts Neck West					
SNW_M1	376	78	166	78	166
Tomotley					
TY_M1	386	90	119	90	117
TY_M2	310	89	96	89	96
TY_M3	251	94	62	95	61
TY_M4	416	88	100	90	92
TY_T1	114	95	52	95	51
TY_T1a	232	80	115	80	115
TY_T1b	62	79	52	79	52
SS_M1	226	78	127	80	118
Yemassee West					
YW_M1	493	90	117	90	117
YW_M2	386	89	118	89	118
YW_M3	289	91	65	93	59
YW_T1	161	84	105	84	105
YW_T1a	354	89	80	89	80
Average	299	83	105	85	98

TABLE 12-4
HYDRAULIC DATA SUMMARY
BROAD RIVER WATERSHED

Basin Name	Open Channels		Stream Crossings			Other Features		
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Baynard	3	1,770	2	4	0	2	2	0
Brays Island East	1	1,336	1	1	0	2	1	0
Broad River Blvd	8	7,119	3	5	0	1	3	0
Habersham Creek North	10	12,432	3	10	1	3	3	0
Habersham Creek South	0	0	0	0	0	0	0	0
Habersham Creek West	2	1,561	1	1	0	1	1	0
Laurel Bay South	18	16,265	9	12	0	3	9	2
Pocotaligo South	3	3,474	0	0	0	0	0	0
Scotts Neck North	3	2,726	1	2	0	1	0	0
Scotts Neck West	1	1,151	0	0	0	0	0	0
Tomotley	10	12,883	4	8	0	6	6	0
Yemassee West	12	15,360	2	2	0	1	0	0
TOTAL	71	76,077	26	45	1	20	25	2

TABLE 12-5
CULVERT DATA FOR HYDROLOGIC BASINS
BROAD RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Baynard Basin						
Baynard Road	BD_M-1A	48"x48"	55	-1.3	7.5	25
	1B	42"x42"	55	1.3		
	1C	42"x42"	55	1.6		
Savannah Highway (State Hwy 802)	BD_M-3	42"x42"	60	-0.9	8.0	100
Brays Island East Basin						
Pinkney Landing Road	BIE_M-1	36"x36"	60	0.72	8.0	25
Broad River Blvd. Basin						
Savannah Highway (State Hwy 802)	BRB_M-1A	36"x36"	100	-0.3	11.0	100
	1B	48"x48"	100	1.4		
Grober Hill Road	BRB_M-3A	24"x24"	40	2.2	7.8	25
	3B	36"x36"	40	1.7		
Robert Smalls Parkway (State Hwy 170)	BRB_M-9	48"x48"	220	7.1	12.3	100
Habersham Creek North Basin						
Joe Frazier Road	HCN_M-0A	180"x53"	30	2.4	8.6	25
	0B	204"x57"	30	2.0		
Burton Wells Road	HCN_M-4A	30"x30"	60	5.5	9.5	25
	4B	30"x30"	60	3.9		
	4C	30"x30"	60	4.5		
	4D	30"x30"	60	5.3		
	4E	30"x30"	60	3.9		
	4F	30"x30"	60	4.6		
Pine Grove Road	HCN_T1-3A	36"x36"	60	8.8	13.0	25
	3B	36"x36"	60	7.5		
Habersham Creek West Basin						
Cherokee Farms Road	HCW_M-2	36"x36"	30	4.1	10.0	25

TABLE 12-5
CULVERT DATA FOR HYDROLOGIC BASINS
BROAD RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Laurel Bay South Basin						
Heronwyck Plantation Road	LBS_M-1	30"x30"	35	-2.3	8.8	25
Unknown Road	LBS_M-3	30"x30"	35	3.1	14.1	25
Morrell Drive	LBS_M-6A	42"x42"	60	9.3	16.4	25
	6B	42"x42"	60	9.7		
Joe Frazier Road	LBS_M-9A	48"x48"	60	14.7	23.6	25
	9B	48"x48"	60	14.4		
Laurel Bay Road	LBS_M-12A	48"x48"	50	15.7	26.4	25
	12B	48"x48"	50	15.8		
Mroz Road	LBS_M-17	48"x48"	90	20.4	29.6	25
Schein Loop	LBS_M-20	48"x48"	65	21.5	30.0	25
Schein Road	LBS_M-21	48"x48"	50	20.9	30.2	25
Parker Drive	LBS_M-25	48"x48"	60	23.6	32.1	25
Scotts Neck North Basin						
William Campbell Road	SNN_M-1A	30"x30"	45	1.2	7.5	25
	1B	30"x30"	45	1.2		
Tomotley Basin						
Stony Creek Cemetary Road	TY_M-0	72"x60"	50	-3.4	6.5	25
Trask Parkway (US Hwy 17)	TY_M-1	96"x60"	160	-3.3	8.1	100
Cotton Hill Road	TY_M-5A	48"x48"	50	1.1	8.3	25
	5B	48"x48"	50	0.5		
Trask Road (Us Hwy 21)	TY_T1-4A	24"x24"	150	8.6	14.8	100
	4B	36"x36"	150	10.6		
Stony Creek Cemetary Road	TY_T1a-3	96"x72"	40	-2.5	7.4	25
Stony Creek Cemetary Road	TY_T1b-3	42"x42"	50	-0.7	7.4	25
Yemassee West Basin						
Frampton Road	YW_M-6	60"x60"	60	1.2	9.8	25
Castle Hall Road	YW_T1-5	24"x24"	50	10.7	15.4	100

TABLE 12-6
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
BROAD RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Baynard Basin					
Baynard Road	BD_M-1	7.5	25	7.5	7.6
Brays Island East Basin					
Pinkney Landing Road	BIE_M-4	8.0	25	8.3	8.3
Broad River Blvd. Basin					
Savannah Highway (State Hwy 802)	BRB_M-6	11.0	100	11.4	11.5
Grober Hill Road	BRB_M-11	7.8	25	10.6	11.0
Robert Smalls Parkway (State Hwy 170)	BRB_M-59	12.3	100	13.0	13.0
Habersham Creek North Basin					
Burton Wells Road	HCN_M-37	9.5	25	10.3	10.5
Pine Grove Road	HCN_T1-15	13.0	25	12.9	13.2
Habersham Creek West Basin					
Cherokee Farms Road	HCW_M-21	10.0	25	10.9	10.9
Laurel Bay South Basin					
Heronwyck Plantation Road	LBS_M-12	8.8	25	9.4	9.7
Morrell Drive	LBS_M-49	16.4	25	17.1	17.5
Joe Frazier Road	LBS_M-74	23.6	25	24.0	24.4
Laurel Bay Road	LBS_M-89	26.4	25	26.7	27.1
Mroz Road	LBS_M-129	29.6	25	29.8	30.0
Schein Loop	LBS_M-144	30.0	25	30.7	30.9
Schein Road	LBS_M-145	30.2	25	30.7	30.8
Scotts Neck North Basin					
William Campbell Road	SNN_M-2	7.5	25	7.5	7.5
Tomotley Basin					
Cotton Hill Road	TY_M-72	8.3	25	8.9	8.9

TABLE 12-7
RECOMMENDED CULVERT IMPROVEMENTS
BROAD RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Baynard Basin			
Baynard Road	BD_M-1A	48"x48"	Replace culverts with one 10 ft by 5 ft box culvert
	1B	42"x42"	
	1C	42"x42"	
Brays Island East Basin			
Pinkney Landing Road	BIE_M-1	36"x36"	Replace culvert with one 8 ft by 5 ft box culvert
Broad River Blvd. Basin			
Savannah Highway (State Hwy 802)	BRB_M-1A	36"x36"	Replace culverts with one 16 ft by 8 ft box culvert
	1B	48"x48"	
Grober Hill Road	BRB_M-3A	24"x24"	Replace culverts with three 10 ft by 5 ft box culverts, Raise road from elevation 7.8 ft to elevation 9.0 ft NAVD (length of 400ft)
	3B	36"x36"	
Robert Smalls Parkway (State Hwy 170)	BRB_M-9	48"x48"	Replace culvert with three 8 ft by 4 ft box culverts
Habersham Creek North Basin			
Burton Wells Road	HCN_M-4A	30"x30"	Replace culverts with three 7 ft by 4 ft box culverts, Raise road from 9.5 ft to 11.0 ft NAVD
	4B	30"x30"	
	4C	30"x30"	
	4D	30"x30"	
	4E	30"x30"	
	4F	30"x30"	
Pine Grove Road	HCN_T1-3A	36"x36"	Add one 36" pipe to existing culverts
	3B	36"x36"	
Habersham Creek West Basin			
Cherokee Farms Road	HCW_M-3	36"x36"	Replace culvert with two 8 ft by 4 ft box culverts, Raise road from elevation 10.0 ft to elevation 11.0 ft NAVD (length of 290 ft)
Laurel Bay South Basin			
Heronwyck Plantation Road	LBS_M-1	30"x30"	Replace culvert with two 11 ft by 7 ft box culverts, Replace weir with four horizontal rectangular 6 ft by 6 ft weirs
Morrell Drive	LBS_M-6A	42"x42"	Replace culverts with two 12 ft by 6 ft box culverts
	6B	42"x42"	
Joe Frazier Road	LBS_M-9A	48"x48"	Replace culverts with two 8 ft by 4 ft box culverts
	9B	48"x48"	
Laurel Bay Road	LBS_M-12A	48"x48"	Replace culverts with one 10 ft by 6 ft box culvert
	12B	48"x48"	
Mroz Road	LBS_M-17	48"x48"	Replace culvert with two 12 ft by 6 ft box culverts
Schein Loop	LBS_M-20	48"x48"	Replace culvert with two 12 ft by 8 ft box culverts
Schein Road	LBS_M-21	48"x48"	Replace culvert with two 9 ft by 6 ft box culvert
Scotts Neck North Basin			
William Campbell Road	SNN_M-1A	30"x30"	Replace culverts with one 6 ft by 4 ft box culvert
	1B	30"x30"	
Tomotley Basin			
Cotton Hill Road	TY_M-5A	48"x48"	Replace culverts with one 12 ft by 6 ft box culvert
	5B	48"x48"	

TABLE 12-8

WATER QUALITY BASIN LAND USE DISTRIBUTION

BROAD RIVER WATERSHED

Land Use Type	Broad River 1 Existing	Broad River 2 Existing	Broad River 3 Existing	Broad River 4 Existing	TOTAL
Agricultural/Pasture	188	403	0	0	591
Commercial	23	68	115	194	401
Forest/Rural Open	2,921	1,191	59	75	4,246
Golf Course	0	0	0	965	965
High Density Residential	10	975	695	1,652	3,333
Industrial	287	556	2,020	634	3,497
Institutional	0	167	7	17	190
Low Density Residential	2,902	469	4	0	3,375
Medium Density Residential	0	503	0	9	511
Open Water/Tidal	5,188	12,149	14,951	11,501	43,789
Silvaculture	0	0	0	0	0
Urban Open	349	1,059	357	759	2,524
Wetland/Water	3,680	1,496	365	133	5,674
TOTAL	15,549	19,034	18,572	15,939	69,095
Urban Imperviousness (%)	3%	6%	10%	9%	7%

Land Use Type	Broad River 1 Future	Broad River 2 Future	Broad River 3 Future	Broad River 4 Future	TOTAL
Agricultural/Pasture	934	34	0	0	968
Commercial	25	167	223	202	616
Forest/Rural Open	1,916	57	2	7	1,982
Golf Course	0	0	6	1,214	1,220
High Density Residential	10	975	695	1,655	3,335
Industrial	325	1,131	2,059	761	4,276
Institutional	6	192	42	42	282
Low Density Residential	3,181	468	4	0	3,653
Medium Density Residential	189	2,064	209	122	2,583
Open Water/Tidal	5,191	12,147	14,960	11,497	43,795
Silvaculture	0	0	0	0	0
Urban Open	92	304	8	307	711
Wetland/Water	3,680	1,496	365	133	5,674
TOTAL	15,549	19,034	18,572	15,939	69,095
Urban Imperviousness (%)	4%	11%	11%	10%	9%

TABLE 12-9

WATER QUALITY BASIN BMP COVERAGE

BROAD RIVER WATERSHED

Land Use Type	Broad River 1 Existing	Broad River 2 Existing	Broad River 3 Existing	Broad River 4 Existing	TOTAL
Commercial	0%	0%	0%	16%	8%
Golf Course	0%	0%	0%	100%	100%
High Density Residential	0%	0%	0%	87%	43%
Industrial	0%	0%	0%	69%	12%
Institutional	0%	0%	0%	6%	1%
Low Density Residential	0%	0%	0%	0%	0%
Medium Density Residential	0%	0%	0%	0%	0%
TOTAL	0%	0%	0%	82%	23%

Land Use Type	Broad River 1 Future	Broad River 2 Future	Broad River 3 Future	Broad River 4 Future	TOTAL
Commercial	8%	59%	48%	19%	40%
Golf Course	95%	0%	100%	100%	100%
High Density Residential	0%	0%	0%	87%	43%
Industrial	13%	50%	2%	85%	31%
Institutional	100%	13%	84%	62%	33%
Low Density Residential	9%	0%	0%	0%	8%
Medium Density Residential	100%	76%	100%	93%	80%
TOTAL	14%	45%	13%	87%	42%

TABLE 12-10

WATER QUALITY BASIN SEPTIC TANK COVERAGE

BROAD CREEK WATERSHED

Land Use Type	Broad River 1 Existing	Broad River 2 Existing	Broad River 3 Existing	Broad River 4 Existing	TOTAL
Commercial	100%	96%	39%	21%	43%
High Density Residential	100%	89%	89%	10%	50%
Industrial	100%	63%	10%	28%	29%
Institutional	0%	51%	84%	9%	49%
Low Density Residential	100%	80%	100%	0%	97%
Medium Density Residential	0%	7%	100%	100%	9%
TOTAL	100%	65%	31%	16%	55%

Land Use Type	Broad River 1 Future	Broad River 2 Future	Broad River 3 Future	Broad River 4 Future	TOTAL
Commercial	100%	39%	20%	20%	28%
High Density Residential	100%	89%	90%	10%	50%
Industrial	100%	34%	10%	24%	26%
Institutional	100%	45%	13%	3%	35%
Low Density Residential	100%	80%	100%	0%	97%
Medium Density Residential	100%	2%	0%	7%	9%
TOTAL	100%	36%	27%	14%	46%

TABLE 12-11

AVERAGE ANNUAL LOADS FOR BROAD RIVER WATERSHED WATER QUALITY BASINS

EXISTING LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Broad River 1	15,549	32,957	316,000	1,940,000	15,862	128,000	487	8,296	4.11E+15
Broad River 2	19,036	55,078	538,000	2,650,000	27,363	225,000	958	19,039	7.70E+15
Broad River 3	18,573	63,128	652,000	3,270,000	29,974	249,000	1,207	23,916	7.19E+15
Broad River 4	15,939	50,115	468,000	1,550,000	22,167	182,000	792	17,780	4.14E+15
TOTAL	69,097	201,278	1,974,000	9,410,000	95,366	784,000	3,444	69,031	2.31E+16

FUTURE LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Broad River 1	15,549	33,232	325,000	1,980,000	17,118	131,000	492	8,362	4.18E+15
Broad River 2	19,035	57,386	593,000	2,790,000	27,522	231,000	994	19,544	7.50E+15
Broad River 3	18,573	63,631	663,000	3,300,000	30,121	251,000	1,214	24,034	7.19E+15
Broad River 4	15,939	50,456	474,000	1,510,000	22,275	182,000	789	17,820	4.07E+15
TOTAL	69,096	204,705	2,055,000	9,580,000	97,036	795,000	3,489	69,760	2.29E+16
Percent Increase over Existing Land Use		2%	4%	2%	2%	1%	1%	1%	-1%

TABLE 12-12

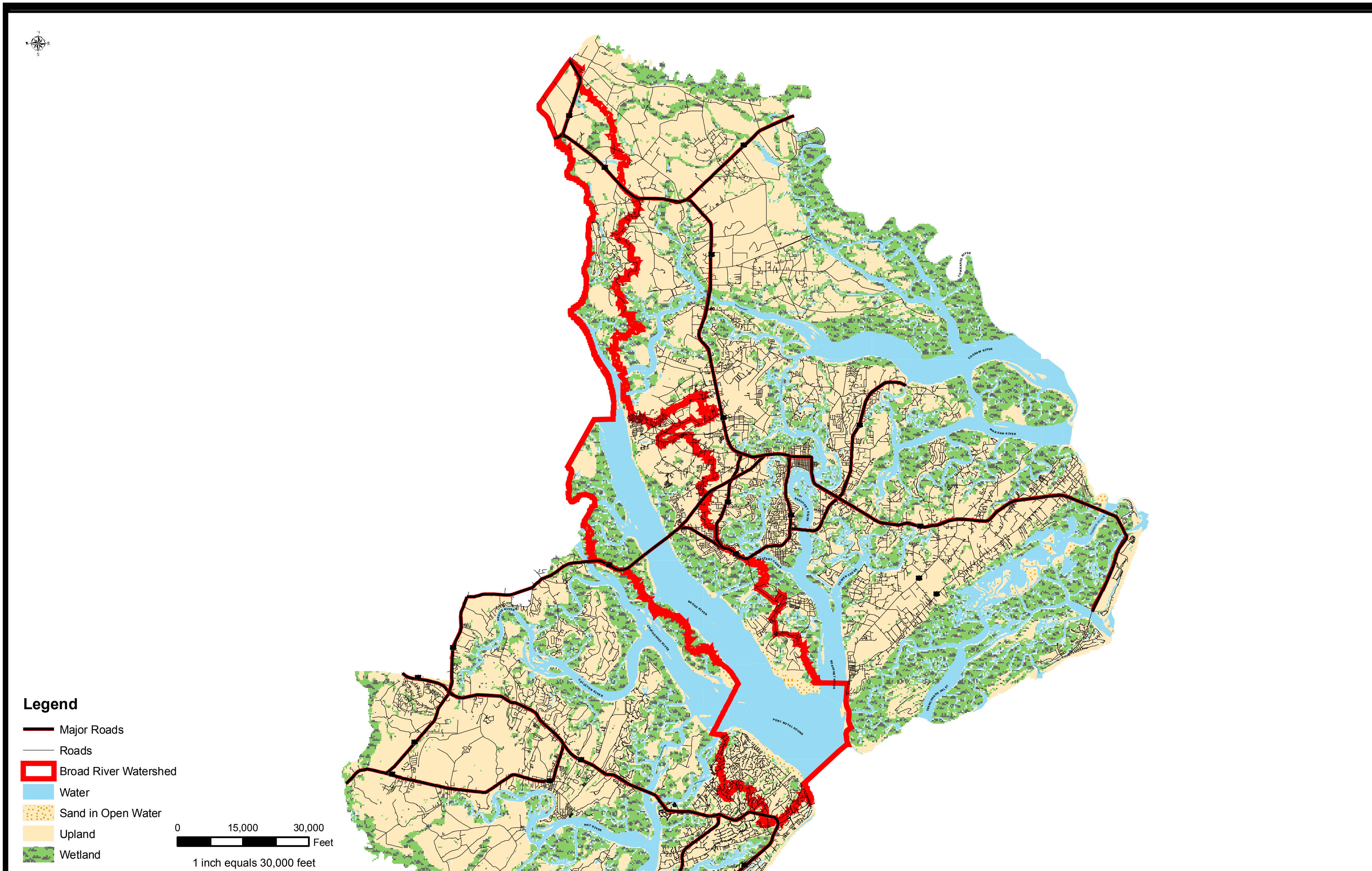
PLANNING LEVEL COST ESTIMATES FOR
BROAD RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
BD_M-1	Road overtopping at Baynard Road Replace existing 1 - 48" RCP and 2 - 42" RCP with 1 - 10'x5' box culvert	$92,000
BIE_M-1	Road overtopping at Savannah Highway (State Hwy 802) Replace existing 1 - 36" CMP with 1 - 8'x5' box culvert	$95,000
BRB_M-1	Road overtopping at Savannah Highway (State Hwy 802) Replace existing 1 - 36" RCP and 1 - 48" RCP with 1 - 16'x8' box culvert	$281,000
BRB_M-3	Road overtopping at Grober Hill Road Replace existing 1 - 24" RCP and 1 - 36" RCP with 3 - 10'x5' box culverts Raise road 1.2 ft (length of 400 ft)	$296,000
BRB_M-9	Road overtopping at Robert Smalls Parkway (State Hwy 170) Replace existing 1 - 48" RCP with 3 - 8'x4' box culverts	$580,000
HCN_M-4	Road overtopping at Burton Wells Road Replace existing 6 - 30" RCP with 3 - 7'x4' box culverts Raise road 1.5 ft (length of 570 ft)	$331,000
HCN_T1-3	Road overtopping at Pine Grove Road Add 1 - 36" RCP to existing 2 - 36" RCP	$21,000
HCW_M-3	Road overtopping at Cherokee Farms Road Replace existing 1 - 36" RCP with 2 - 8'x4' box culverts Raise road 1.0 ft (length of 290 ft)	$162,000
LBS_M-1*	Road overtopping at Heronwyck Plantation Road Replace existing 1 - 30" RCP with 2 - 11'x7' box culverts Replace existing 1 - 36"x36" horizontal weir riser with 4 - 72"x72" horizontal weir risers	$151,000
LBS_M-6	Road overtopping at Morrell Drive Replace existing 2 - 42" RCP with 2 - 12'x6' box culverts	$202,000
LBS_M-9	Road overtopping at Joe Frazier Road Replace existing 2 - 48" RCP with 2 - 8'x4' box culverts	$164,000
LBS_M-12	Road overtopping at Laurel Bay Road Replace existing 2 - 48" RCP with 1 - 10'x6' box culvert	$100,000
LBS_M-17	Road overtopping at Mroz Road Replace existing 1 - 48" RCP with 2 - 12'x6' box culverts	$281,000
LBS_M-20	Road overtopping at Schein Loop Replace existing 1 - 48" CMP with 2 - 12'x8' box culverts	$286,000
LBS_M-21	Road overtopping at Schein Road Replace existing 1 - 48" CMP with 2 - 9'x6' box culverts	$149,000
SNN_M-1	Road overtopping at William Campbell Road Replace existing 2 - 30" RCP with 1 - 6'x4' box culvert	$58,000
TY_M-5	Road overtopping at Cotton Hill Road Replace existing 2 - 48" RCP with 1 - 12'x6' box culvert	$100,000
	TOTAL	$3,349,000

* Conduits marked by asterisk are on private land

Costs are in December 2004 dollars.

See Appendix J for basis of cost estimates.



Broad River Watershed
(Beaufort County)

Figure 12-1

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Roads	Beaufort County	2002
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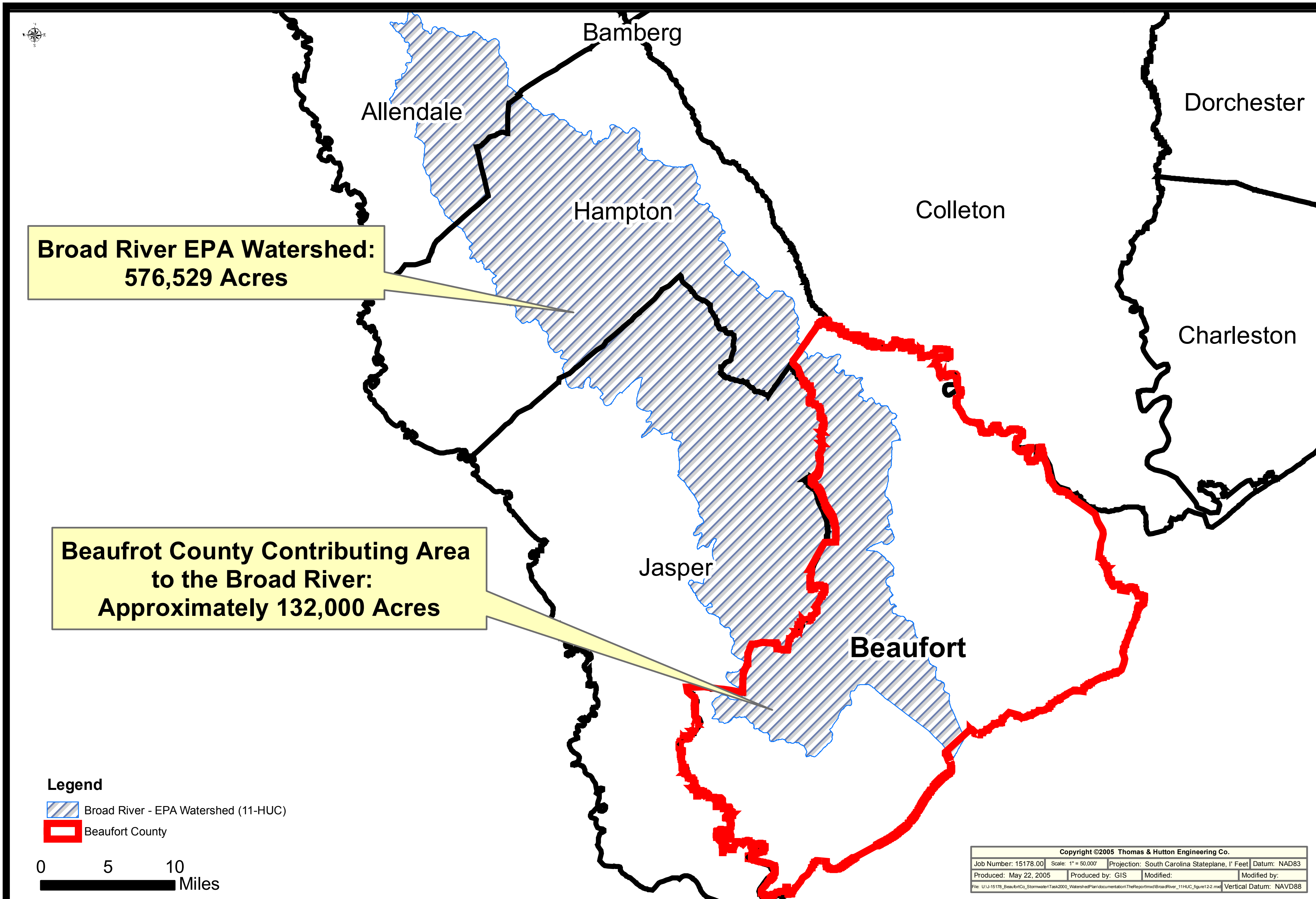
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Broad River Watershed
(Total: EPA Watershed 11-HUC)
Figure 12-2

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Counties	SC DNR	
Broad River Watershed	EPA	

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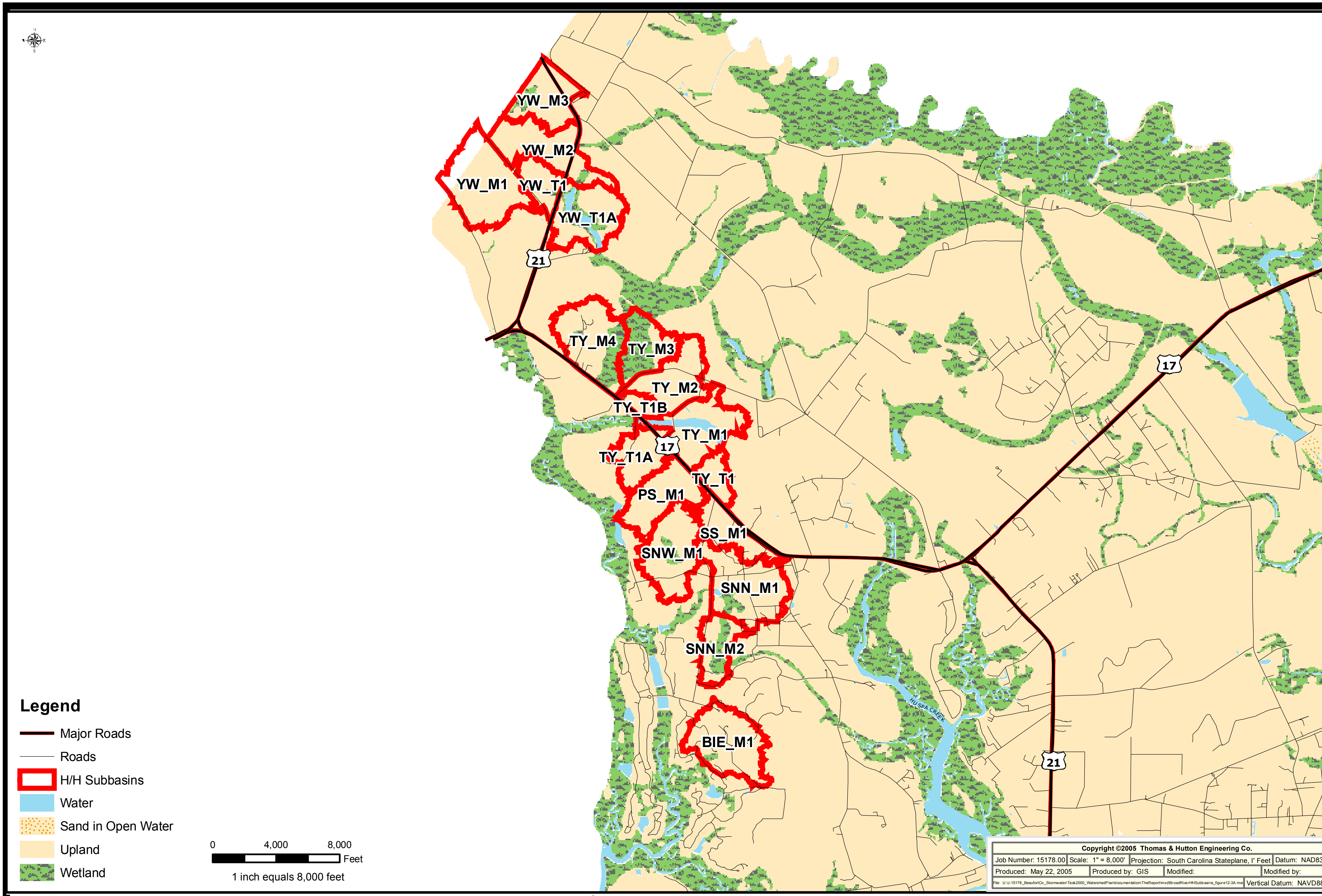
DATA	SOURCE	DATE



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Broad River Watershed
Hydrologic Model Subbasins
Figure 12-3A

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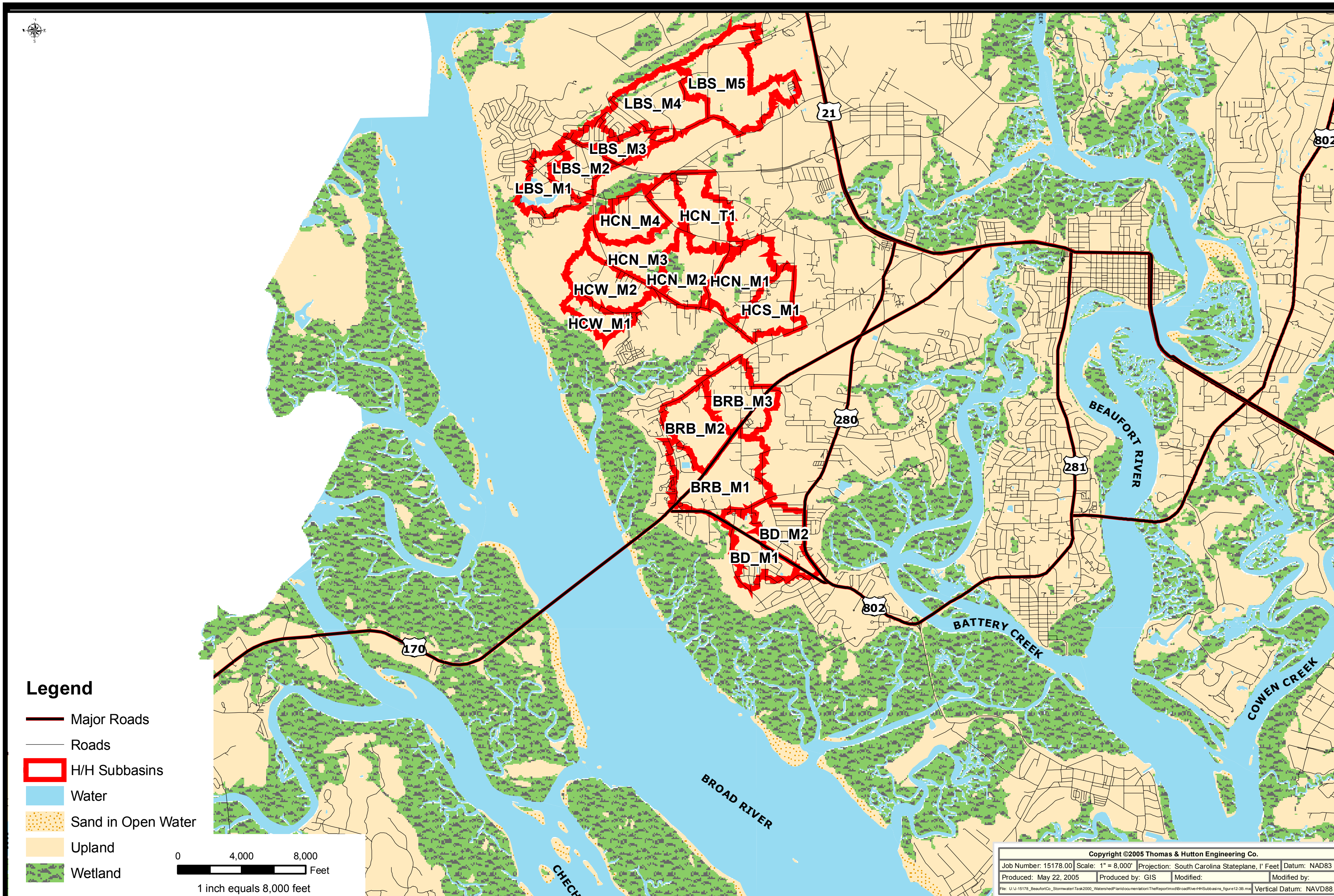
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Broad River Watershed
Hydrologic Model Subbasins

Figure 12-3B

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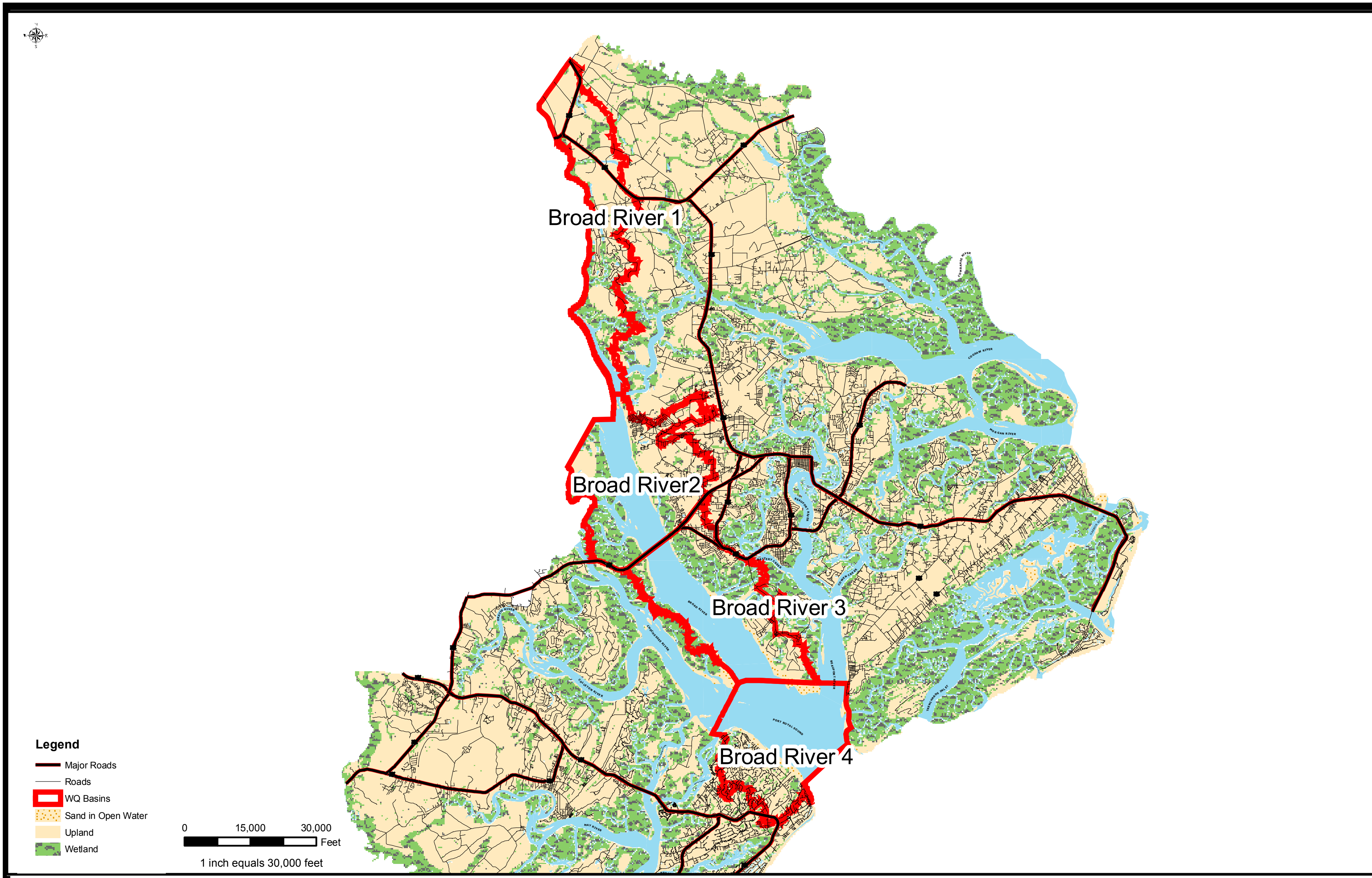
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Broad River Watershed
Water Quality Basins

Figure 12-4

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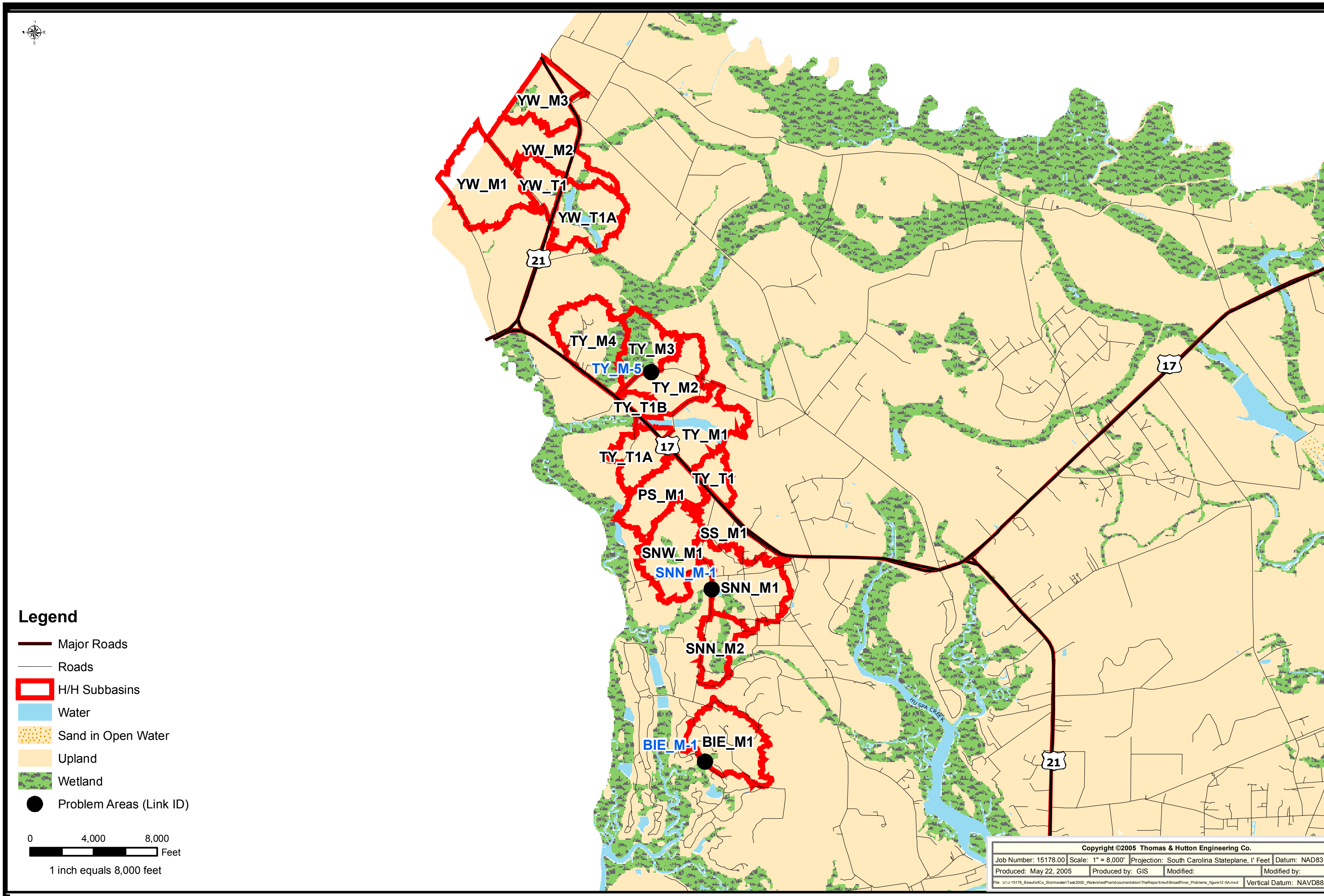
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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Broad River Watershed
PSMS Problem Areas
Figure 12-5A

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Land Use / Land Cover	USGS	

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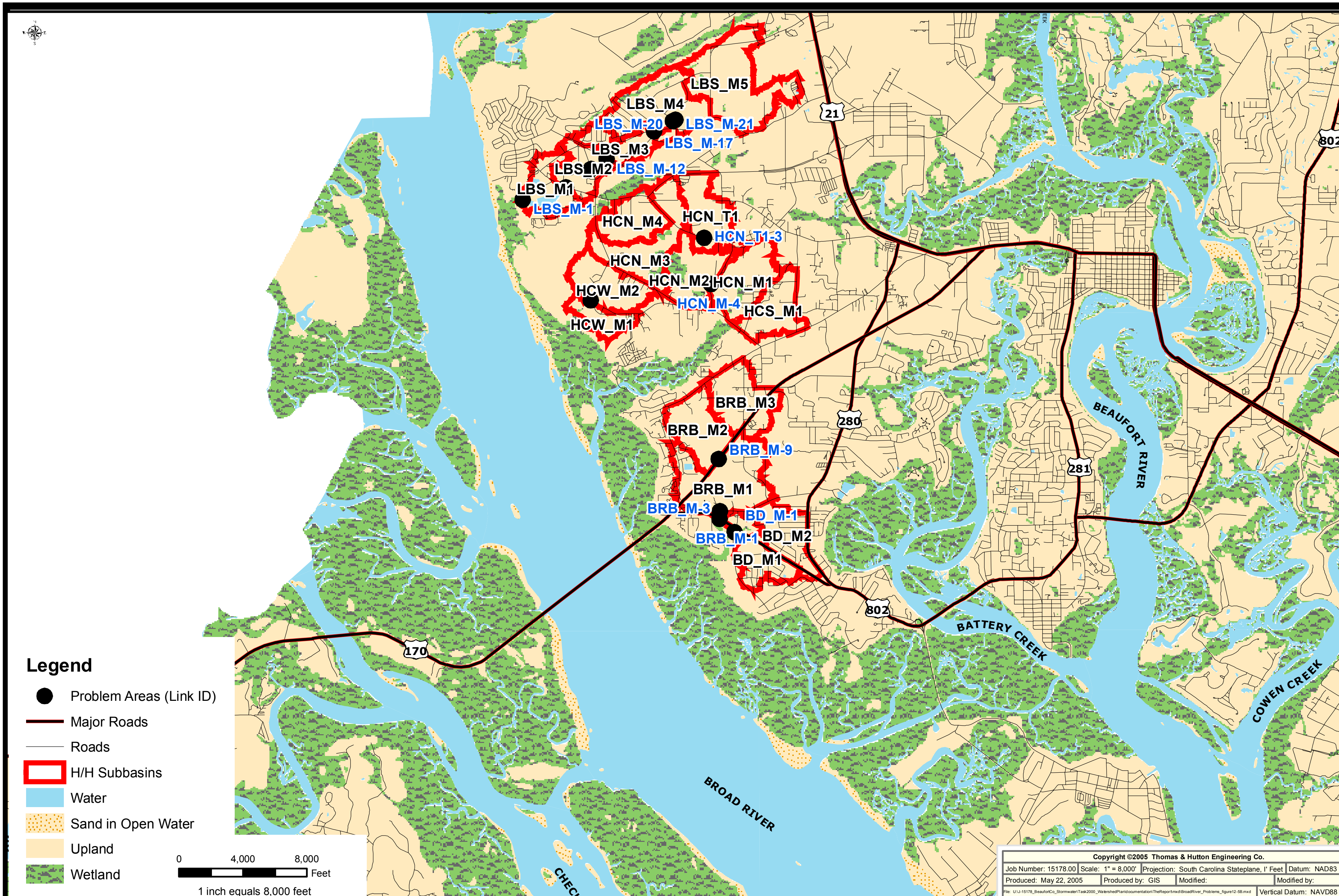
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Broad River Watershed
PSMS Problem Areas
Figure 12-5B

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Land Use / Land Cover	USGS	

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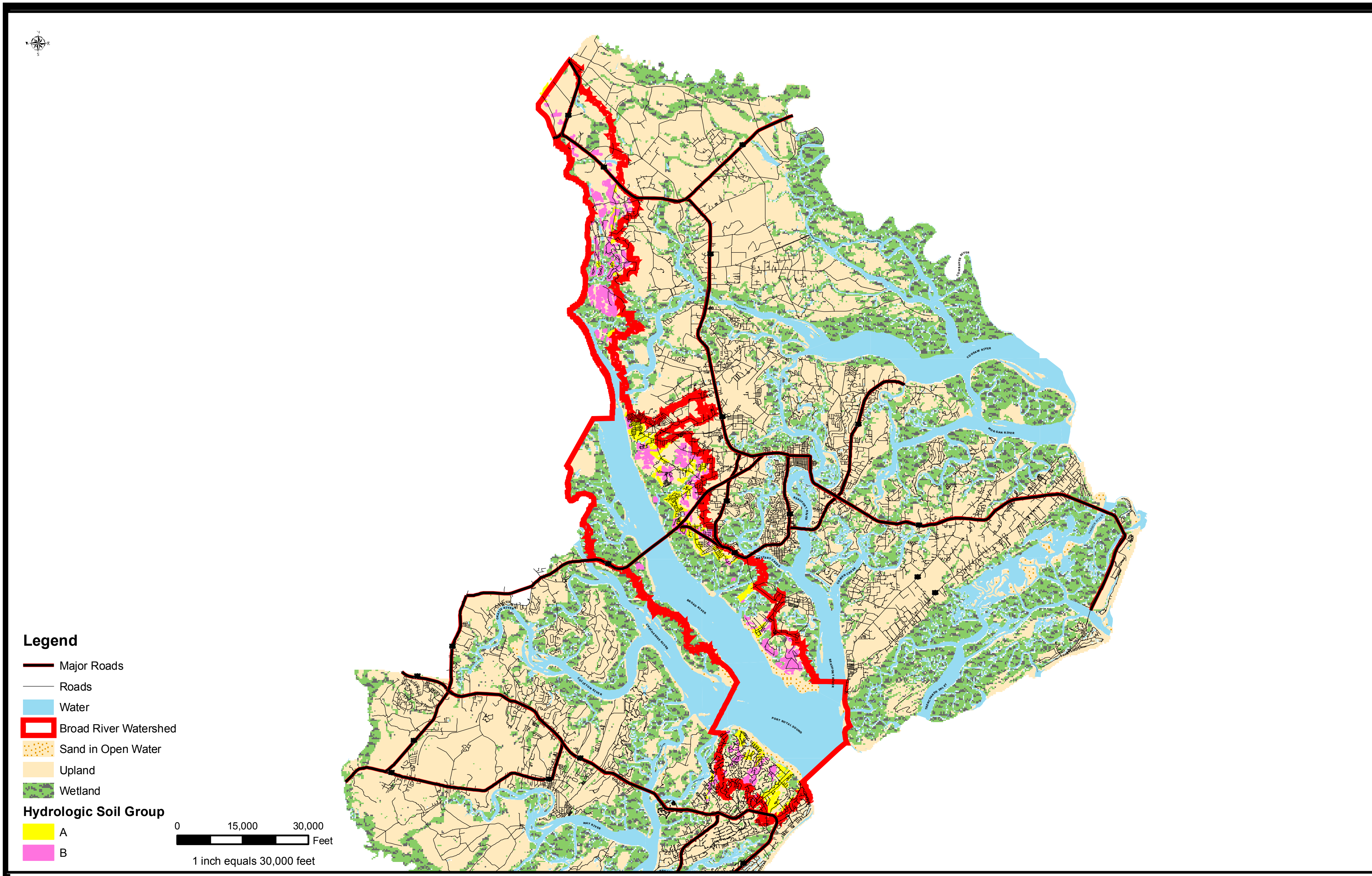
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Broad River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

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Figure 12-6

Job Number: 15178.00	Scale: 1" = 30,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Section 13
Combahee River Watershed Analysis

This section describes the physical features of the Combahee River watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

13.1 Overview

The Combahee River runs along the north border of Beaufort County (see **Figure 13-1**). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area from Sheldon Township that is tributary to the Combahee River.

For comparative purposes, the entire tributary area for the Combahee River is presented in **Figure 13-2**. The figure indicates Beaufort County makes up only a small fraction of the total tributary area to the Combahee River.

For the hydrologic and hydraulic analysis of the Primary Stormwater Management System (PSMS), the watershed includes several basins. These are listed in **Table 13-1**, and presented in **Figure 13-3**. Table 13-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were done to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into basins. These are listed in **Table 13-2**, and presented in **Figure 13-4**. Pollution loads were calculated for each of the water quality basins. Unlike the other watersheds that are north of the Broad River, the vast majority of the Combahee River tributary area is actually located outside of Beaufort County. Because loads from Beaufort County are such a small fraction of the total load to the Combahee River, and loads from outside the County are unknown, tidal river water quality model calculations were not done for the Combahee River.

13.2 Hydrologic and Hydraulic Analysis

CDM and T&H used the Interconnected Pond Routing Model (ICPR), Version 3 for the hydrologic and hydraulic analyses of the PSMS in the Combahee River watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were conducted for existing and future land use conditions, with and without alternative management strategies.

The ICPR model is a "link-node" model, representing the PSMS as a series of nodes (stream locations) connected by links (open channels, pipes, culverts). Appendix K includes model schematics of the Combahee River PSMS basins, with a separate schematic for each basin.

13.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Combahee River basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include area, curve number, and time of concentration.

Table 13-3 lists the hydrologic parameter values for the Combahee River PSMS subbasins. Each model subbasin is identified by ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development.

Hydraulic summary information for the Combahee River PSMS basins is presented in **Table 13-4**. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in **Table 13-5**. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate level of service.

Details regarding specific open channel segments, storage areas, weirs and tide gates are presented in Appendix K.

13.2.2 Model Results

Tables in Appendix K list the peak flow values for the Combahee River subbasins. Each table lists peak flows for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak flows are listed by subbasin for various land cover and stormwater management controls, which include the following:

- Undeveloped land
- Existing land use without peak shaving controls
- Existing land use with existing peak shaving controls
- Future land use without peak shaving controls

- Future land use with existing and future peak shaving controls

It should be noted that the tables include values for “uncontrolled” and “controlled” peak flows for the 2-year, 10-year and 25-year design storms. The “uncontrolled” peak flow assumes no peak shaving facilities in the subbasin. In contrast, the “controlled” value accounts for peak shaving facilities in the subbasin.

For existing land use, aerial maps and local information were used to estimate the percentage of existing urban development that is served by peak shaving facilities. The “controlled” peak flow value was then calculated by considering the difference in peak flow between totally undeveloped conditions and existing conditions with no controls. For example, suppose that a subbasin of 100 acres has an undeveloped 2-year peak flow of 20 cfs, and an uncontrolled existing peak flow of 50 cfs, and further suppose that 60 percent of the urban development is controlled by peak shaving facilities. In this case, it is assumed that the existing peak flow is reduced by 60 percent of the difference between undeveloped and developed peak flow (50 – 20 = 30 cfs; 60 percent of 30 = 18 cfs reduction due to peak shaving), and therefore the maximum controlled peak flow will be 32 cfs (50 – 18).

For future land use, the “controlled” peak flow is set equal to the “controlled” peak flow for existing land use, because new development is subject to State and County peak flow regulations. Keep in mind, however, that the future condition will still generate more stormwater runoff volume, even though the peak flow is the same. The result is that the peak flow rate will be sustained for a longer period of time under future conditions.

Other tables in Appendix K list the peak water elevation values for model node locations along the Combahee River PSMS. Each table lists peak stages for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak stages are listed for existing and future land use conditions, with the existing hydraulic system.

Specific problem areas identified by the modeling are listed in **Table 13-6** and presented in **Figure 13-5**. For each area, the table identifies the road crossing, associated model ID, design storm, “critical elevation” (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm

Table 13-6 indicates that two road crossings are being overtopped by the design storm events. Problem areas were identified in the Combahee East and Combahee West basins.

Evaluation of solutions to prevent these problems is discussed in the next section of the report.

13.2.4 Management Strategy Alternatives

The problems areas listed in Table 13-6 were evaluated by modifying the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in **Table 13-7**. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

13.3 Water Quality Analysis

CDM and T&H used the Watershed Management Model (WMM) for the water quality analysis of the Combahee River watershed. WMM was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, total nitrogen (total N), total phosphorus (total P), BOD, lead, zinc and total suspended solids (TSS).

13.3.1 Land Use and BMP Coverage

Table 13-8 presents the existing land use and future land use estimates for the Combahee River water quality basins; collectively, the water quality basins constitute all watershed area within Beaufort County. The existing land use data reflects a number of sources, including February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps, plus local knowledge of development completed between February 2002 and June 2003. The future land use map was developed by "filling in" the existing land use map, replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and Town of Hilton Head Island future land use maps and zoning maps.

Under existing land use conditions, 20 percent of the Combahee River watershed area consists of urban systems (e.g., residential, commercial, golf course) and 80 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh).

Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 2 per cent of the watershed.

Under future land use conditions, the split between urban systems and natural systems is roughly the same, at 20 percent urban systems, and 80 percent natural systems. What little develop is expected to occur will primarily be from forest/rural land to low density residential and industrial land uses. As a result of limited projected future development, urban imperviousness stays at 2 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in **Table 13-9**. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County, and include areas for which BMPs were designed in accordance with the Beaufort County BMP Manual. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects what percentage of all urban land in the watershed in served by BMPs

Under existing land use conditions, none of the urban systems in the watershed (e.g., residential, commercial, golf course) are served by BMPs designed in accordance with the BMP Manual. Under future land use conditions, 1 percent of the urban systems are served by BMPs. This small increase from existing to future reflects the limited amount of expected future development.

13.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in **Table 13-10**. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water & Sewer Authority. For future development, areas that are zoned "rural" or "conservation" were assumed to be served by septic tanks, and other areas were assumed to be served by sewer.

Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects what percentage of all urban land in the watershed in served by septic tanks.

For existing land use conditions, 100 percent of the urban systems in the watershed (e.g., residential, commercial) are served by septic. Under future land use conditions, 100 percent of the urban systems are also served by septic tanks. This reflects the presumption that all of the new development will be served by septic tanks.

There are no known direct or indirect discharges of wastewater in the watershed.

13.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Combahee River water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in **Table 13-11** for existing and future land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

An overall comparison of the WMM modeling results (Table 13-11) indicates that future flows and constituent loads increase over their existing counterparts. Specifically, future flow is 2 percent greater than for existing conditions and the increase in loads ranges from 5 percent for zinc to -1 percent (decrease) for TSS.

13.3.4 Management Strategy Alternatives

Besides the enforcement of the BMP Manual requirements for new development (and maintenance of existing BMPs), no specific recommendations are made for the Combahee River watershed. There is only a very small increase in impervious cover and annual loads when comparing the exiting and future conditions. Even in the future build-out condition, the overall urban imperviousness of the watershed is only 2 percent.

For informational purposes, the areas with "A" and "B" type soils are presented in **Figure 13-6**. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

13.4 Planning Level Cost Estimates for Management Alternatives

Table 13-12 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Combahee River watershed. As shown in the table, the two projects are estimated to have a total cost of $0.2 million in December 2004 dollars. Details of the cost estimate for each project are shown in Appendix K.

The prioritization of these projects, and projects identified for other watersheds, is discussed in Section 16 of this report.

TABLE 13-1
HYDROLOGIC BASINS
COMBAHEE RIVER WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
Combahee East	1,068	3	356
Combahee Middle	207	1	207
Combahee North	359	1	359
Combahee West	3,633	11	330
Yemassee East	447	2	224
TOTAL	5,714	18	317

TABLE 13-2
WATER QUALITY BASINS
COMBAHEE RIVER WATERSHED

Basin Name	Tributary Area (acres)
Combahee River 1	13,669
Combahee River 2	8,869
TOTAL	22,538

TABLE 13-3
HYDROLOGIC SUBBASIN CHARACTERISTICS
COMBAHEE RIVER WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use Curve Number	Existing Land Use Time of Concentration (minutes)	Future Land Use Curve Number	Future Land Use Time of Concentration (minutes)
Combahee East Basin					
CE_M1	344	84	149	84	149
CE_M2	411	88	142	88	142
CE_M3	314	83	144	83	144
Combahee Middle Basin					
CM_M1	207	92	108	92	108
Combahee North Basin					
CHN_M1	358	88	125	88	125
Combahee West Basin					
CW_M1	121	92	66	92	66
CW_M2	326	82	120	82	120
CW_M3	91	84	67	84	67
CW_M4	661	88	117	88	117
CW_M5	362	91	92	91	92
CW_M6	319	90	95	90	95
CW_M7	257	91	83	91	83
CW_T1	314	93	127	93	127
CW_T2	412	84	154	84	153
CW_T3	450	85	136	85	136
CW_T4	320	94	97	94	97
Yemassee East Basin					
YE_M1	167	92	66	92	66
YE_M2	281	83	110	83	110
Average	318	88	111	88	111

TABLE 13-4
HYDRAULIC DATA SUMMARY
COMBAHEE RIVER WATERSHED

	Open Channels		Stream Crossings			Other Features		
Basin Name	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
Combahee East	5	5,548	1	1	0	0	1	1
Combahee Middle	0	0	2	2	1	2	0	0
Combahee North	2	1,590	1	2	0	0	0	0
Combahee West	26	31,522	3	3	1	1	4	0
Yemassee East	2	1,840	2	2	0	1	2	0
TOTAL	35	40,500	9	10	2	4	7	1

TABLE 13-5
CULVERT DATA FOR HYDROLOGIC BASINS
COMBAHEE RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
Combahee East Basin						
River Road	CE_M-0	36"x36"	56	-3.3	6.0	25
Combahee Middle Basin						
River Road	CM_M-1	Bridge	30	-0.5	5.2	25
Big Estate Road	CM_M-2A	24"x24"	45	0.3	6.1	25
	2B	24"x24"	45	0.4		
Combahee North Basin						
River Road	CHN_M-1A	48"x48"	65	-0.2	7.4	25
	1B	48"x48"	65	0.2		
Combahee West Basin						
River Road	CW_M-1	Bridge	30	-2.0	8.8	25
Old Sheldon Church Road	CW_M-25	72"x72"	40	3.2	9.1	25
Twickenham Plantation Road	CW_T2-2A	36"x36"	25	2.5	7.7	25
	2B	36"x36"	25	2.7		
Yemassee East Basin						
Old Sheldon Church Road	YE_M-4	36"x36"	40	2.7	9.4	25

TABLE 13-6
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COMBAHEE RIVER WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Combahee East Basin					
River Road	CE_M-5	6.0	25	6.2	6.2
Combahee West Basin					
Twickenham Plantation Road	CW_T2-14	7.7	25	8.2	8.2

TABLE 13-7
RECOMMENDED CULVERT IMPROVEMENTS
COMBAHEE RIVER WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Combahee East Basin			
River Road	CE_M-0	36"x36"	Replace culvert with one 6 ft by 6 ft box culvert; Replace drop structure with 2 risers, each with three vertical weirs measuring 4 ft by 4 ft and one horizontal weir measuring 4 ft by 4 ft
Combahee West Basin			
Twickenham Plantation Road	CW_T2-2A 2B	36"x36" 36"x36"	Replace culverts with three 8 ft by 5 ft box culverts

TABLE 13-8

WATER QUALITY BASIN LAND USE DISTRIBUTION

COMBAHEE RIVER WATERSHED

Land Use Type	Combahee River 1 Existing	Combahee River 2 Existing	TOTAL Existing
Agricultural/Pasture	136	0	136
Commercial	11	0	11
Forest/Rural Open	1,139	160	1,299
Golf Course	0	0	0
High Density Residential	0	0	0
Industrial	259	28	287
Institutional	0	0	0
Low Density Residential	2,584	624	3,208
Medium Density Residential	0	0	0
Open Water/Tidal	3,956	7,985	11,941
Silvaculture	0	0	0
Urban Open	1,024	0	1,024
Wetland/Water	4,558	72	4,631
TOTAL	13,669	8,869	22,538
Urban Imperviousness (%)	3%	1%	2%

Land Use Type	Combahee River 1 Future	Combahee River 2 Future	TOTAL Future
Agricultural/Pasture	136	0	136
Commercial	12	0	12
Forest/Rural Open	1,122	161	1,283
Golf Course	0	0	0
High Density Residential	0	0	0
Industrial	272	27	299
Institutional	3	0	3
Low Density Residential	2,590	625	3,215
Medium Density Residential	24	0	24
Open Water/Tidal	3,951	7,983	11,934
Silvaculture	0	0	0
Urban Open	1,002	0	1,002
Wetland/Water	4,556	73	4,629
TOTAL	13,669	8,869	22,538
Urban Imperviousness (%)	3%	1%	2%

TABLE 13-9

WATER QUALITY BASIN BMP COVERAGE

COMBAHEE RIVER WATERSHED

Land Use Type	Combahee River 1 Existing	Combahee River 2 Existing	TOTAL
Commercial	0%	0%	0%
Golf Course	0%	0%	0%
High Density Residential	0%	0%	0%
Industrial	0%	0%	0%
Institutional	0%	0%	0%
Low Density Residential	0%	0%	0%
Medium Density Residential	0%	0%	0%
TOTAL	0%	0%	0%

Land Use Type	Combahee River 1 Future	Combahee River 2 Future	TOTAL
Commercial	9%	0%	9%
Golf Course	0%	0%	0%
High Density Residential	0%	0%	0%
Industrial	4%	0%	4%
Institutional	100%	0%	100%
Low Density Residential	0%	0%	0%
Medium Density Residential	100%	0%	100%
TOTAL	1%	0%	1%

TABLE 13-10

WATER QUALITY BASIN SEPTIC TANK COVERAGE

COMBAHEE RIVER WATERSHED

Land Use Type	Combahee River 1 Existing	Combahee River 2 Existing	TOTAL
Commercial	100%	100%	100%
High Density Residential	0%	0%	0%
Industrial	100%	100%	100%
Institutional	0%	100%	0%
Low Density Residential	100%	100%	100%
Medium Density Residential	0%	100%	0%
TOTAL	100%	100%	100%

Land Use Type	Combahee River 1 Future	Combahee River 2 Future	TOTAL
Commercial	100%	100%	100%
High Density Residential	0%	0%	0%
Industrial	100%	100%	100%
Institutional	100%	100%	100%
Low Density Residential	100%	100%	100%
Medium Density Residential	100%	100%	100%
TOTAL	100%	100%	100%

TABLE 13-11

AVERAGE ANNUAL LOADS FOR COMBAHEE RIVER WATERSHED WATER QUALITY BASINS

EXISTING LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Combahee River 1	13,669	28,219	268,000	1,760,000	13,364	109,000	398	6,450	3.52E+15
Combahee River 2	8,869	30,128	257,000	675,000	13,494	109,000	502	11,631	2.66E+15
TOTAL	22,538	58,347	525,000	2,435,000	26,858	218,000	900	18,081	6.18E+15

FUTURE LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Combahee River 1	13,669	29,449	281,000	1,740,000	14,030	114,000	431	7,313	3.65E+15
Combahee River 2	8,869	30,121	257,000	674,000	13,489	109,000	502	11,627	2.66E+15
TOTAL	22,537	59,570	538,000	2,414,000	27,519	223,000	933	18,940	6.31E+15
Percent Increase over Existing Land Use		2%	2%	-1%	2%	2%	4%	5%	2%

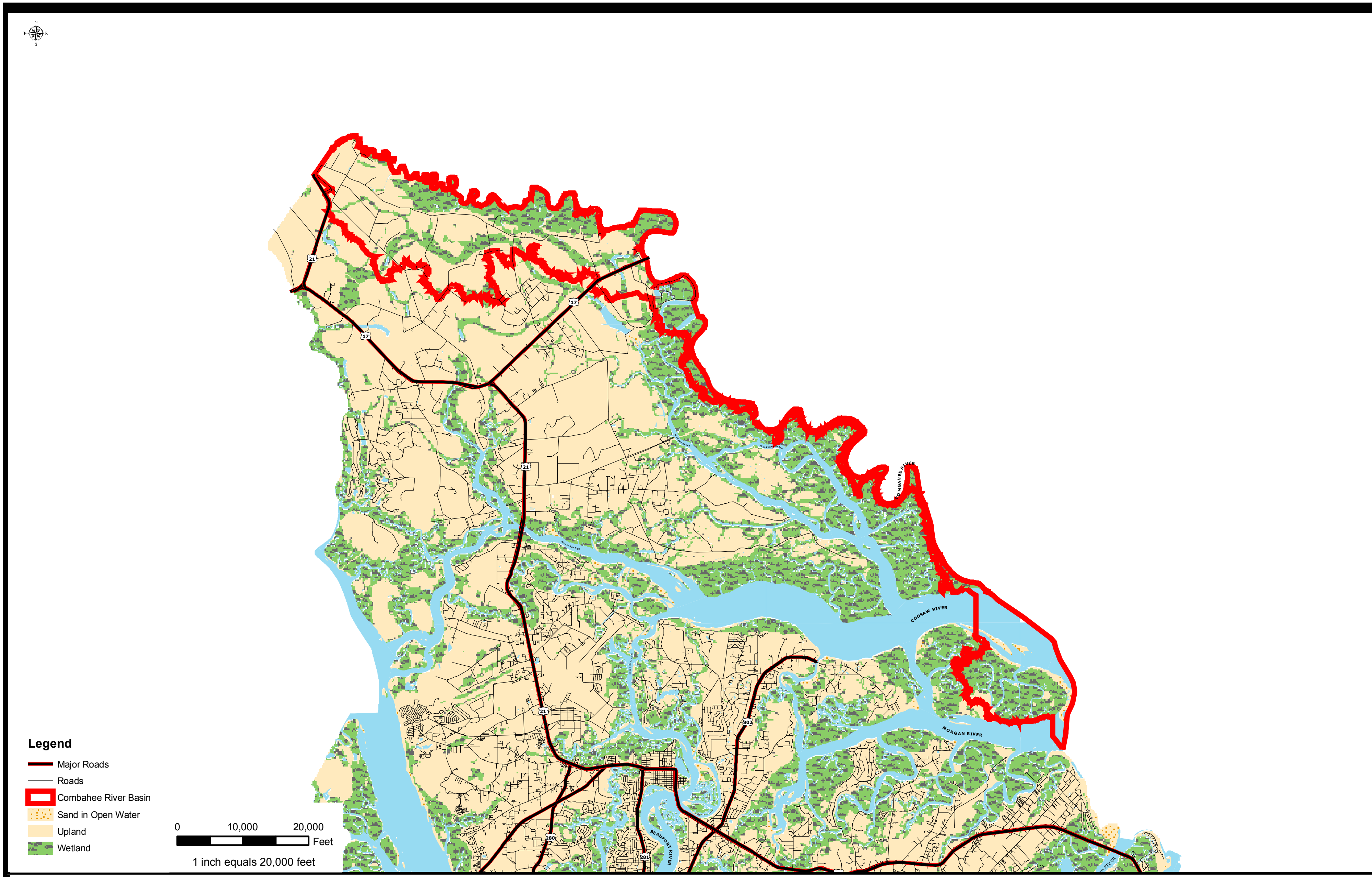
TABLE 13-12

PLANNING LEVEL COST ESTIMATES FOR
COMBAHEE RIVER WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
CE_M-0	Road overtopping at River Road Replace existing 1 - 36" RCP with 1 - 6'x6' box culvert Replace existing 1 - 3'x3' vertical weir drop structure with 2 drop structures, each with 3 - 4'x4' vertical weirs and 1 - 4'x4' horizontal weir	$88,000
CW_T2-2	Road overtopping at Twickenham Plantation Road Replace existing 2 - 36" CMP with 3 - 8'x5' box culverts	$114,000
	TOTAL	$202,000

Costs are in December 2004 dollars.

See Appendix K for basis of cost estimates.



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File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\CombaheeRiverWatershed_figure13			Vertical Datum: NAVD88

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Combahee River Watershed

Figure 13-1

Disclaimer
Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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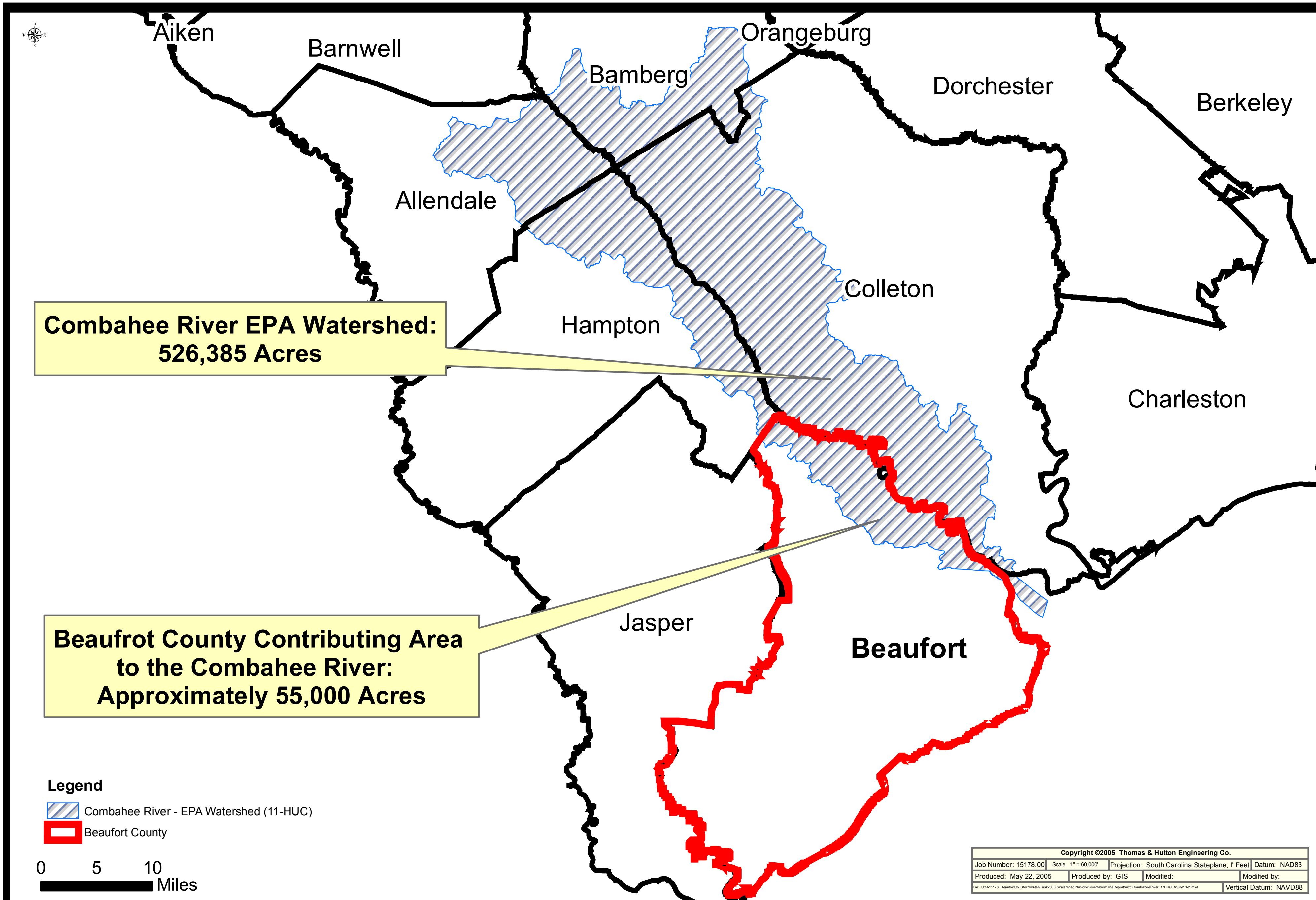

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Figure 13-2

Combahee River Watershed
(Total: EPA Watershed 11-HUC)

Disclaimer
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DATA	SOURCE	DATE
Counties	SC DNR	
Combahee River Watershed	EPA	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA SOURCE DATE

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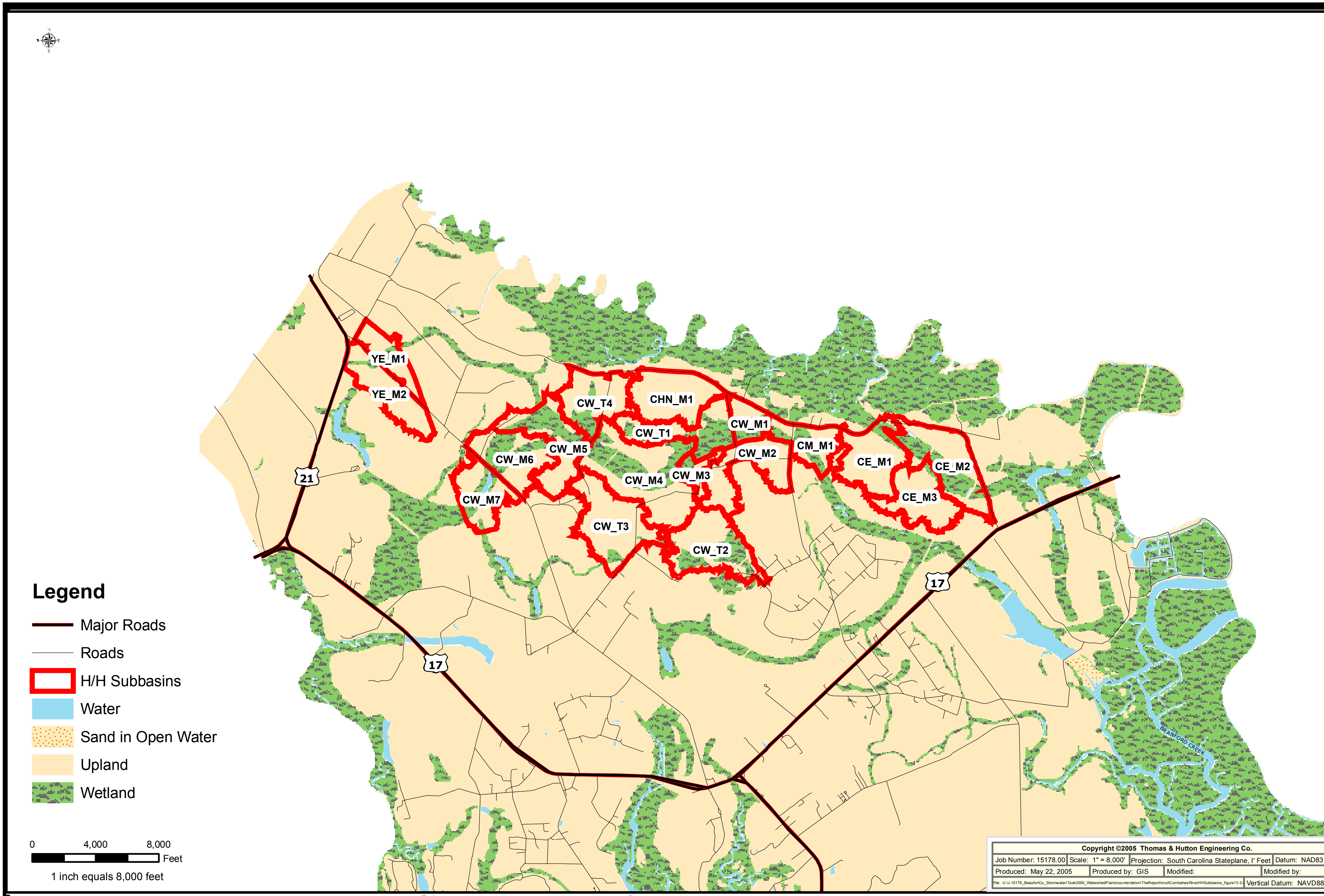

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Combahee River Watershed
Hydrologic Model Subbasins
Figure 13-3

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Land Use / Land Cover	USGS	

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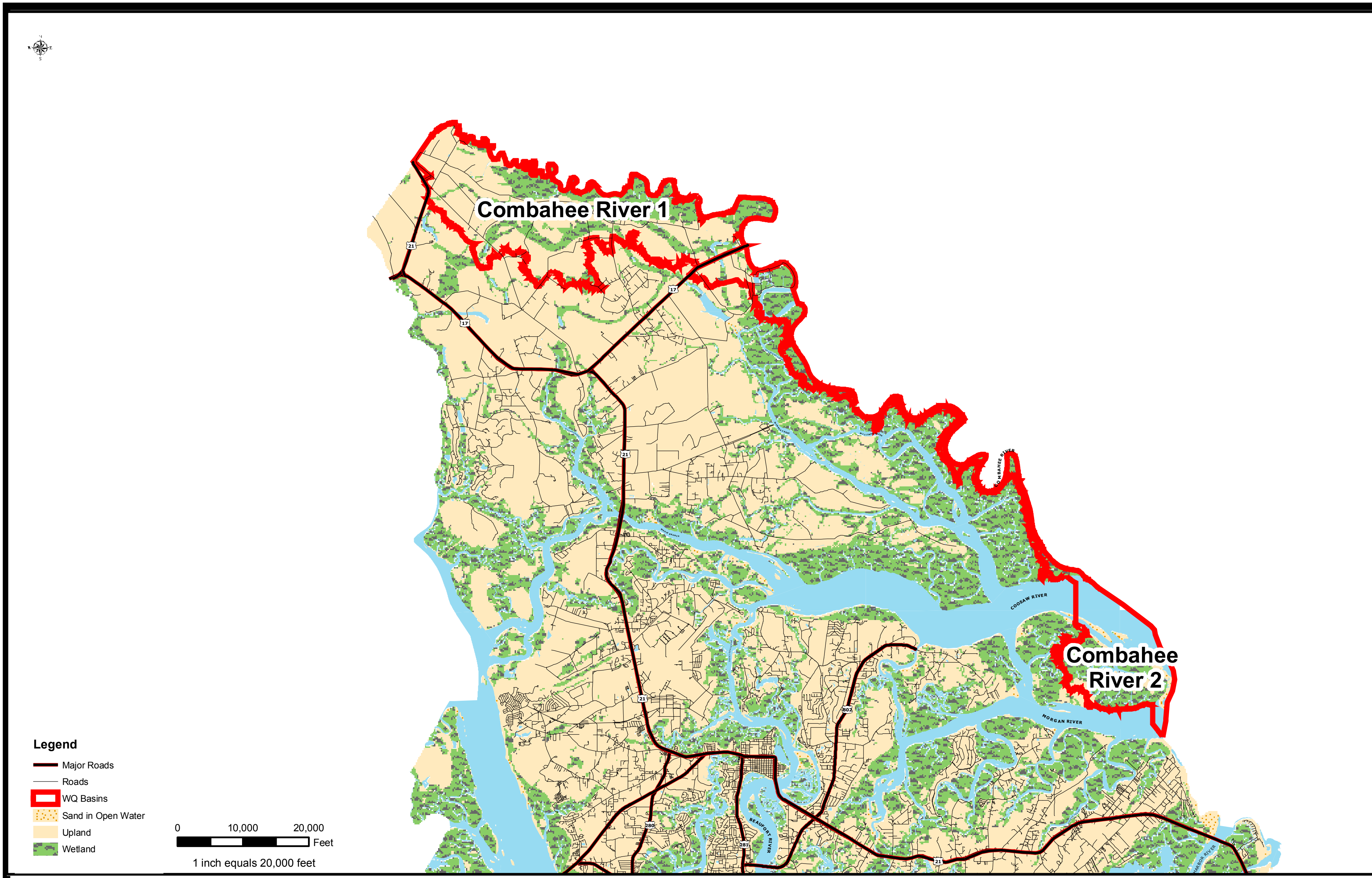
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



Combahee River Watershed Water Quality Basins

Figure 13-4

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Job Number: 15178.00	Scale: 1" = 20,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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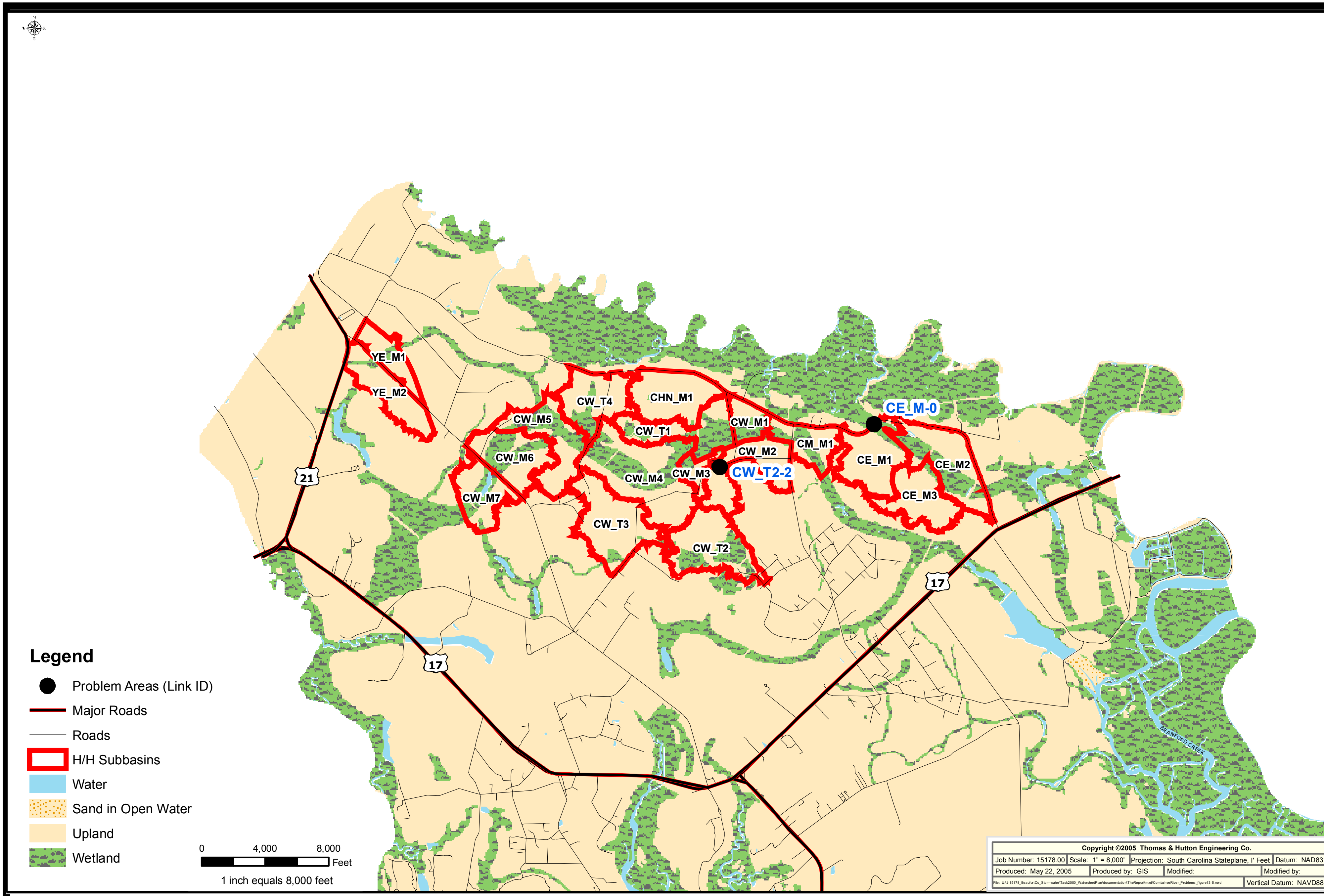
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Combahee River Watershed
PSMS Problem Areas
Figure 13-5

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Land Use / Land Cover	USGS	

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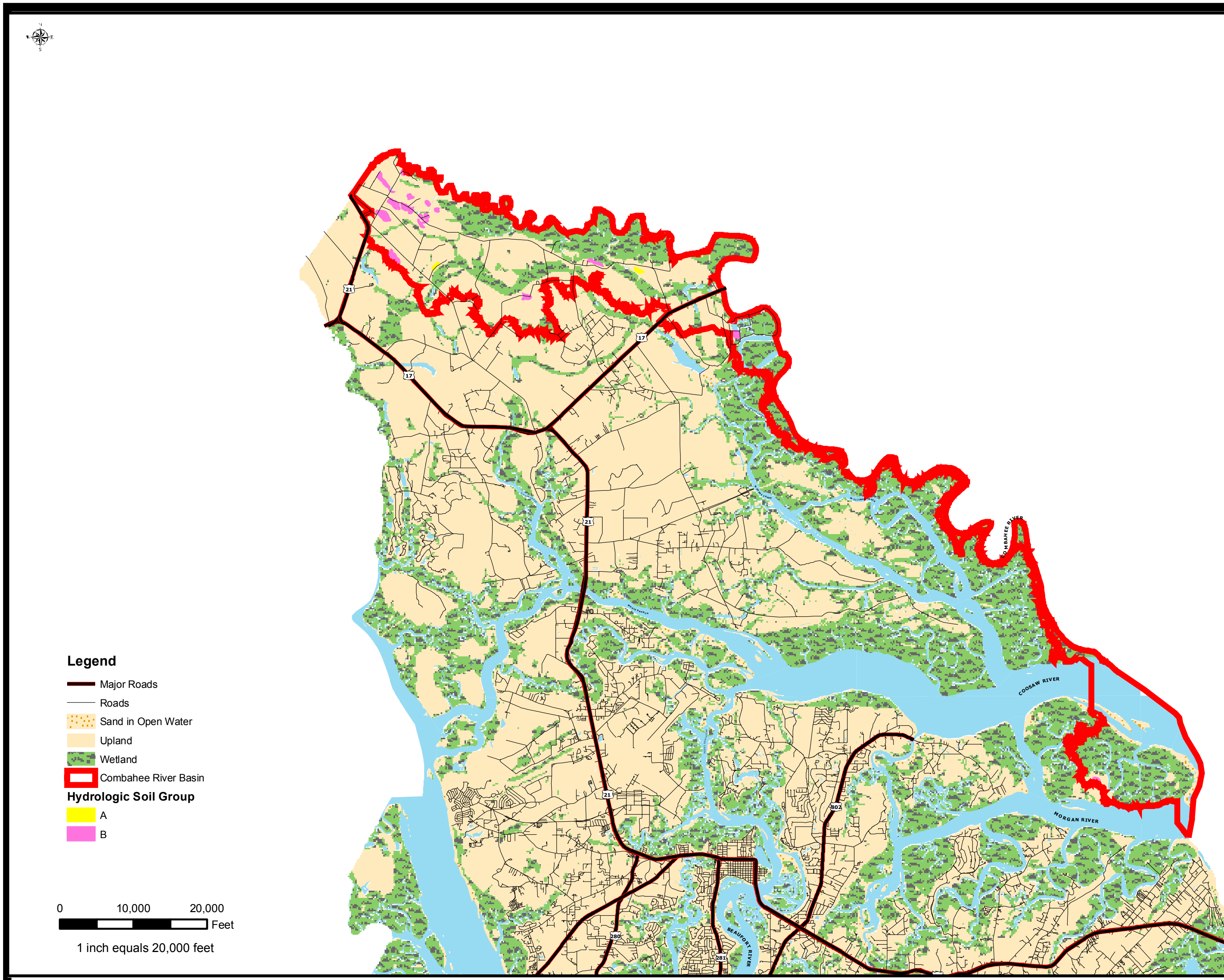
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Job Number: 15178.00	Scale: 1" = 20,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\CombaheeRiverSoils_figure13-6.mxd			Vertical Datum: NAVD88

Combahee River Watershed

Potential Locations for Infiltration BMPs based on A and B Soils

Figure 13-6

CDM Camp Dresser & McKee Inc.

Section 14
Coastal Area Watershed Analysis

This section describes the physical features of the Coastal Area watershed, water quantity and water quality problems, modeling results, alternatives evaluation, and recommendations.

14.1 Overview

The Coastal Area watershed is located in eastern Beaufort County (see **Figure 14-1**). For the purposes of this study, the area included in the watershed analysis includes open water, tidal marsh and upland area on St. Helena Island that is tributary to the Coastal Area.

For the hydrologic and hydraulic analysis of the Primary Stormwater Management System (PSMS), the watershed includes several basins. These are listed in **Table 14-1**, and presented in **Figure 14-2**. Table 14-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were done to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the watershed was subdivided into basins. These are listed in **Table 14-2**, and presented in **Figure 14-3**. Pollution loads were calculated for each of the water quality basins. Unlike the other watersheds that are north of the Broad River, the vast majority of the Coastal Area tributary area is actually located outside of Beaufort County. Because loads from Beaufort County are such a small fraction of the total load to the Coastal Area, and loads from outside the County are unknown, tidal river water quality model calculations were not done for the Coastal Area.

14.2 Hydrologic and Hydraulic Analysis

CDM and T&H used the Interconnected Pond Routing Model (ICPR), Version 3 for the hydrologic and hydraulic analyses of the PSMS in the Coastal Area watershed. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were conducted for existing and future land use conditions, with and without alternative management strategies.

The ICPR model is a "link-node" model, representing the PSMS as a series of nodes (stream locations) connected by links (open channels, pipes, culverts). Appendix L includes model schematics of the Coastal Area PSMS basins, with a separate schematic for each basin.

14.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Coastal Area basin consisted of one of more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include area, curve number, and time of concentration.

Table 14-3 lists the hydrologic parameter values for the Coastal Area PSMS subbasins. Each model subbasin is identified by ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated future development.

Hydraulic summary information for the Coastal Area PSMS basins is presented in **Table 14-4**. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments, and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in **Table 14-5**. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate level of service.

Details regarding specific open channel segments, storage areas, weirs and tide gates are presented in Appendix L.

14.2.2 Model Results

Tables in Appendix L list the peak flow values for the Coastal Area subbasins. Each table lists peak flows for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak flows are listed by subbasin for various land cover and stormwater management controls, which include the following:

- Undeveloped land
- Existing land use without peak shaving controls
- Existing land use with existing peak shaving controls
- Future land use without peak shaving controls
- Future land use with existing and future peak shaving controls

It should be noted that the tables include values for "uncontrolled" and "controlled" peak flows for the 2-year, 10-year and 25-year design storms. The "uncontrolled" peak flow assumes no peak shaving facilities in the subbasin. In contrast, the "controlled" value accounts for peak shaving facilities in the subbasin.

For existing land use, aerial maps and local information were used to estimate the percentage of existing urban development that is served by peak shaving facilities. The "controlled" peak flow value was then calculated by considering the difference in peak flow between totally undeveloped conditions and existing conditions with no controls. For example, suppose that a subbasin of 100 acres has an undeveloped 2-year peak flow of 20 cfs, and an uncontrolled existing peak flow of 50 cfs, and further suppose that 60 percent of the urban development is controlled by peak shaving facilities. In this case, it is assumed that the existing peak flow is reduced by 60 percent of the difference between undeveloped and developed peak flow (50 – 20 = 30 cfs; 60 percent of 30 = 18 cfs reduction due to peak shaving), and therefore the maximum controlled peak flow will be 32 cfs (50 – 18).

For future land use, the "controlled" peak flow is set equal to the "controlled" peak flow for existing land use, because new development is subject to State and County peak flow regulations. Keep in mind, however, that the future condition will still generate more stormwater runoff volume, even though the peak flow is the same. The result is that the peak flow rate will be sustained for a longer period of time under future conditions.

Other tables in Appendix L list the peak water elevation values for model node locations along the Coastal Area PSMS. Each table lists peak stages for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak stages are listed for existing and future land use conditions, with the existing hydraulic system.

Specific problem areas identified by the modeling are listed in **Table 14-6** and presented in **Figure 14-4**. For each area, the table identifies the road crossing, associated model ID, design storm, "critical elevation" (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm

Table 14-6 indicates that three of the six evaluated road crossings are being overtopped by the design storm events. Problem areas were identified in the Scott Creek, South Frogmore and Station Creek basins.

Evaluation of solutions to prevent these problems is discussed in the next section of the report.

14.2.3 Management Strategy Alternatives

The problems areas listed in Table 14-6 were evaluated by modifying the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed a small fraction of the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in **Table 14-7**. The table presents the size of the existing culverts, plus the size of the added or replacement culvert(s). For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the depth of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The depth of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

14.3 Water Quality Analysis

CDM and T&H used the Watershed Management Model (WMM) for the water quality analysis of the Coastal Area watershed. WMM was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, total nitrogen (total N), total phosphorus (total P), BOD, lead, zinc and total suspended solids (TSS).

14.3.1 Land Use and BMP Coverage

Table 14-8 presents the existing land use and future land use estimates for the Coastal Area water quality basins; collectively, the water quality basins constitute all watershed area within Beaufort County. The existing land use data reflects a number of sources, including February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps, plus local knowledge of development completed between February 2002 and June 2003. The future land use map was developed by "filling in" the existing land use map, replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and Town of Hilton Head Island future land use maps and zoning maps.

Under existing land use conditions, 11 percent of the Coastal Area watershed area consists of urban systems (e.g., residential, commercial, golf course) and 89 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh).

Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 2 per cent of the watershed.

Under future land use conditions, the split between urban systems and natural systems is nearly the same, at 13 percent urban systems, and 87 percent natural systems. What little develop is expected to occur will primarily be from forest/rural land to low density and medium density residential land uses. As a result of limited projected future development, urban imperviousness increases only slightly to 3 percent of the watershed.

Estimates of BMP coverage for existing and future land use in presented in **Table 14-9**. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs in Beaufort County, and include areas for which BMPs were designed in accordance with the Beaufort County BMP Manual. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects what percentage of all urban land in the watershed in served by BMPs

Under existing land use conditions, none of the urban systems in the watershed (e.g., residential, commercial, golf course) are served by BMPs designed in accordance with the BMP Manual. Under future land use conditions, 42 percent of the urban systems are served by BMPs. This increase from existing to future reflects the 100 percent coverage of new development by BMPs, and the limited amount of existing development.

14.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use in presented in **Table 14-10**. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water & Sewer Authority. For future development, areas that are zoned "rural" or "conservation" were assumed to be served by septic tanks, and other areas were assumed to be served by sewer.

Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects what percentage of all urban land in the watershed in served by septic tanks.

For existing land use conditions, 77 percent of the urban systems in the watershed (e.g., residential, commercial) are served by septic. Under future land use conditions, 86 percent of the urban systems are also served by septic tanks. This reflects the presumption that most of the new development will be served by septic tanks.

Based on available data, the estimated wastewater discharge under existing conditions is 0.1 million gallons per day (mgd) of land application (e.g., golf course irrigation), and the future discharge is expected to be 0.2 mgd based on increase in residential land between existing and future conditions. There are no direct discharges to receiving waters in the watershed.

14.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Coastal Area water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in **Table 14-11** for existing and future land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

An overall comparison of the WMM modeling results (Table 14-11) indicates that future flows and constituent loads are typically equal to or less than the existing loads. There is a 1 percent flow increase from existing to future land use conditions, and loads show increases of 1 to 2 percent.

14.3.4 Management Strategy Alternatives

Besides the enforcement of the BMP Manual requirements for new development (and maintenance of existing BMPs), no specific recommendations are made for the Coastal Area watershed. There is only a very small increase in impervious cover and the model actually projects a small reduction in annual loads when comparing the exiting and future conditions. Even in the future build-out condition, the overall urban imperviousness of the watershed is only 3 percent.

For informational purposes, the areas with "A" and "B" type soils are presented in **Figure 14-5**. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

14.4 Planning Level Cost Estimates for Management Alternatives

Table 14-12 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Coastal Area watershed. As shown in the table, the three projects are estimated to have a total cost of $0.3 million in December 2004 dollars. Details of the cost estimate for each project are shown in Appendix L.

The prioritization of these projects, and projects identified for other watersheds, is discussed in Section 16 of this report.

TABLE 14-1
HYDROLOGIC BASINS
COASTAL WATERSHED

Basin Name	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
County Landing	761	2	381
Harbor River	650	2	325
Longwood	680	3	227
Scott Creek	452	1	452
Sod Farm	513	2	257
South Frogmore	512	1	512
Station Creek	546	2	273
TOTAL	4,114	13	316

TABLE 14-2
WATER QUALITY BASINS
COASTAL WATERSHED

Basin Name	Tributary Area (acres)
Coastal Area	50,647
TOTAL	50,647

TABLE 14-3
HYDROLOGIC SUBBASIN CHARACTERISTICS
COASTAL WATERSHED

ICPR Subbasin ID	Tributary Area (acres)	Existing Land Use Curve Number	Existing Land Use Time of Concentration (minutes)	Future Land Use Curve Number	Future Land Use Time of Concentration (minutes)
County Landing Basin					
CL_M1	374	86	131	85	133
CL_M2	387	82	158	85	144
Harbor River Basin					
HR_M1	379	72	192	75	177
HR_M2	271	76	199	82	167
Longwood Basin					
LD_M1	367	82	169	86	146
LD_T1	313	72	164	78	142
Scott Creek Basin					
STC_M1	452	82	134	83	127
Sod Farm Basin					
SF_M1	149	76	102	76	101
SF_M2	364	80	125	83	115
South Frogmore Basin					
SHF_M1	513	79	140	81	132
Station Creek Basin					
SNC_M1	286	81	156	81	152
SNC_M2	260	85	121	85	121
Average	376	81	147	83	139

TABLE 14-4
HYDRAULIC DATA SUMMARY
COASTAL WATERSHED

	Open Channels		Stream Crossings			Other Features		
Basin Name	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Weirs	Drop Structures
County Landing	7	6,171	1	1	0	0	0	0
Harbor River	3	3,172	1	1	0	0	0	0
Longwood	4	5,353	1	0	1	0	0	0
Scott Creek	3	1,833	1	2	0	0	3	0
Sod Farm	2	1,762	0	0	0	0	0	0
South Frogmore	3	3,121	1	2	0	0	1	0
Station Creek	5	5,412	1	1	0	0	1	0
TOTAL	27	26,824	6	7	1	0	5	0

TABLE 14-5
CULVERT DATA FOR HYDROLOGIC BASINS
COASTAL WATERSHED

Road Crossing	ICPR Model Link ID	Culvert Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Roadway Elevation (ft NAVD)	Level of Service
County Landing Basin						
Seaside Road	CL_M-1	54" x 54"	60	-0.9	10.6	25
Harbor River Basin						
Seaside Road	HR_M-1	36"x36"	60	1.3	12.3	25
Longwood Basin						
Seaside Road	LD_M-1	Bridge	30	1.1	7.6	25
Scott Creek Basin						
Seaside Road	STC_M-1A	42"x42"	60	0.6	8.1	25
	1B	42"x42"	60	0.7		
Sod Farm Basin						
No road crossings in this basin						
South Frogmore Basin						
Club Bridge Road	SHF_M-1A	36"x36"	40	-1.8	5.9	25
	1B	36"x36"	40	-2.1		
Station Creek Basin						
Seaside Road	SNC_M-0	36"x36"	50	0.9	7.9	25

TABLE 14-6
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
COASTAL WATERSHED

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Scott Creek Basin					
Seaside Road crossing	STC_M-2	8.1	25	8.9	9.0
South Frogmore Basin					
Club Bridge Road crossing	SHF_M-1	5.9	25	6.1	6.2
Station Creek Basin					
Seaside Road crossing	SNC_M-5	7.9	25	8.4	8.4

TABLE 14-7
RECOMMENDED CULVERT IMPROVEMENTS
COASTAL WATERSHED

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements
Scott Creek Basin			
Seaside Road crossing	STC_M-1	2 - 42"x42"	Replace culverts with one 12 ft by 7 ft box culvert
South Frogmore Basin			
Club Bridge Road crossing	SHF_M-1	2 - 36"x36"	Replace culverts with two 5 ft by 5 ft box culverts
Station Creek Basin			
Seaside Road crossing	SNC_M-0	36"x36"	Replace culverts with one 7 ft by 6 ft box culvert

TABLE 14-8

WATER QUALITY BASIN LAND USE DISTRIBUTION

COASTAL WATERSHED

Land Use Type	Coastal Area Existing	Coastal Area Future
Agricultural/Pasture	1,921	2,314
Commercial	34	42
Forest/Rural Open	4,043	2,645
Golf Course	191	191
High Density Residential	742	743
Industrial	553	554
Institutional	6	66
Low Density Residential	1,953	3,802
Medium Density Residential	90	707
Open Water/Tidal	37,391	37,530
Silvaculture	0	0
Urban Open	1,965	290
Wetland/Water	1,759	1,758
TOTAL	50,647	50,644
Urban Imperviousness (%)	2%	3%

TABLE 14-9

WATER QUALITY BASIN BMP COVERAGE

COASTAL WATERSHED

Land Use Type	Coastal Area Existing	Coastal Area Future
Commercial	0%	19%
Golf Course	0%	0%
High Density Residential	0%	0%
Industrial	0%	1%
Institutional	0%	91%
Low Density Residential	0%	49%
Medium Density Residential	0%	87%
TOTAL	0%	42%

TABLE 14-10

WATER QUALITY BASIN SEPTIC TANK COVERAGE

COASTAL WATERSHED

Land Use Type	Coastal Area Existing	Coastal Area Future
Commercial	70%	67%
High Density Residential	17%	17%
Industrial	71%	71%
Institutional	96%	97%
Low Density Residential	100%	100%
Medium Density Residential	100%	96%
TOTAL	77%	86%

TABLE 14-11

AVERAGE ANNUAL LOADS FOR COASTAL WATERSHED WATER QUALITY BASINS

EXISTING LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Coastal Area	50,648	153,000	1,330,000	4,490,000	71,304	559,000	2,462	55,296	1.39E+16

FUTURE LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Coastal Area	50,642	154,000	1,360,000	4,550,000	72,787	568,000	2,491	55,723	1.42E+16
Percent Increase over Existing Land Use		1%	2%	1%	2%	2%	1%	1%	2%

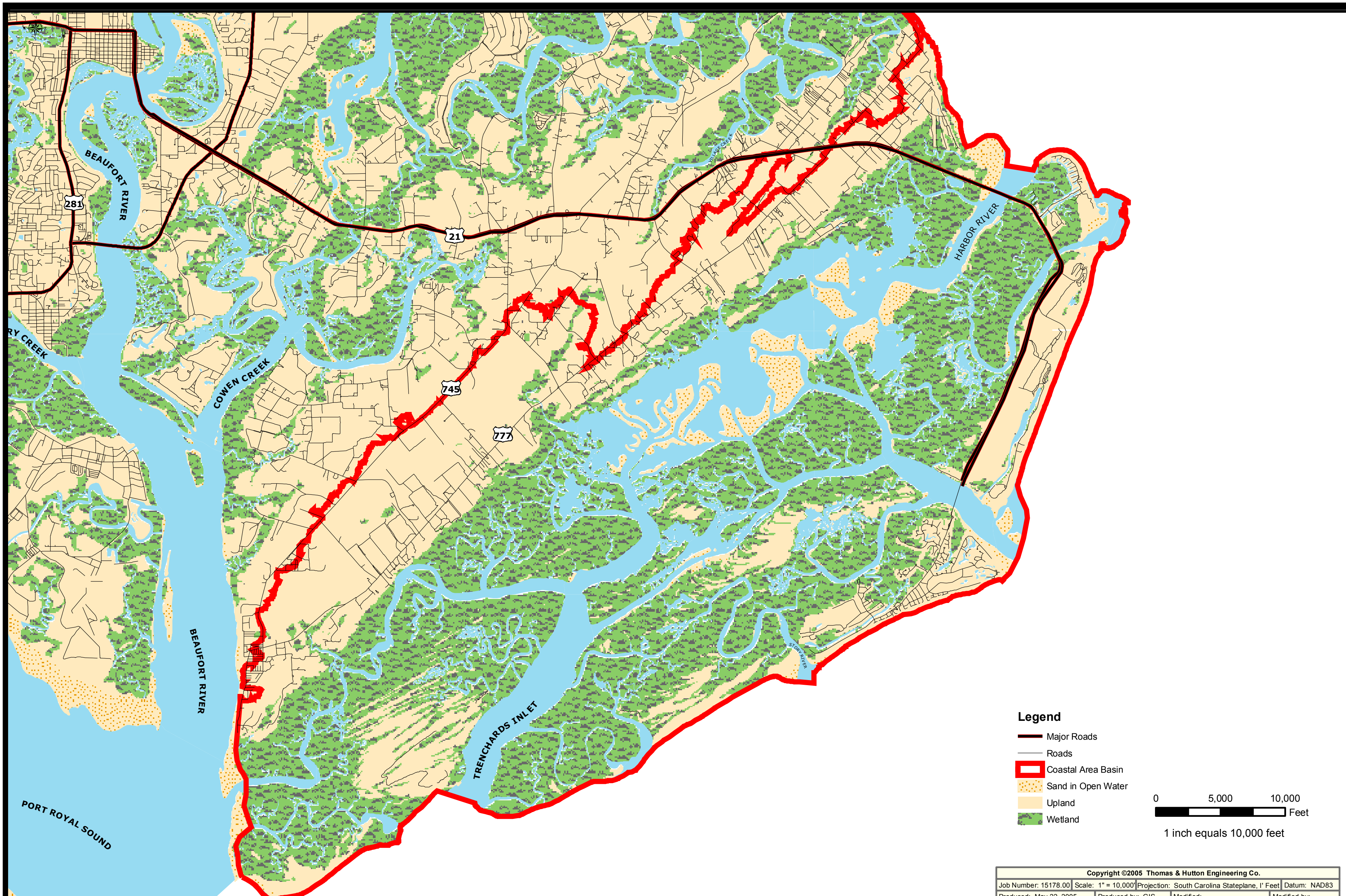
TABLE 14-12

PLANNING LEVEL COST ESTIMATES FOR
COASTAL WATERSHED

MODEL CONDUIT	PROJECT	ESTIMATED COST
STC_M-1	Road overtopping at Seaside Road Replace existing 2 - 42" RCP with 1 - 12'x7' box culvert	$133,000
SHF_M-1	Road overtopping at Club Bridge Road Replace existing 2 - 36" RCP with 2 - 5'x5' box culverts	$78,000
SNC_M-0	Road overtopping at Seaside Road Replace existing 1 - 36" RCP with 1 - 7'x6' box culverts	$81,000
	TOTAL	$292,000

Costs are in December 2004 dollars.

See Appendix L for basis of cost estimates.



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Coastal Area Watershed

Figure 14-1

Disclaimer
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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	

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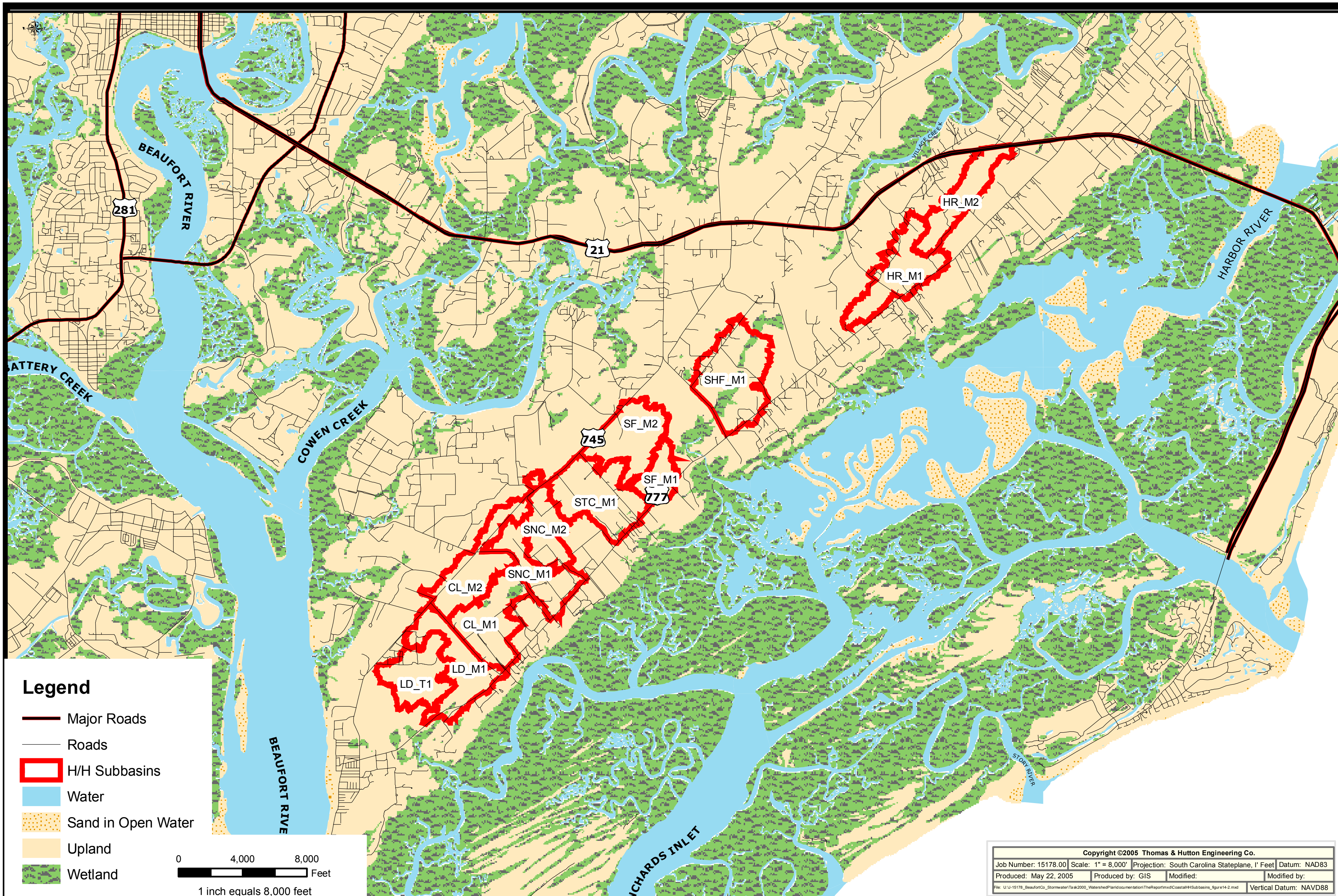
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Coastal Area Hydrologic Model Subbasins

Figure 14-2

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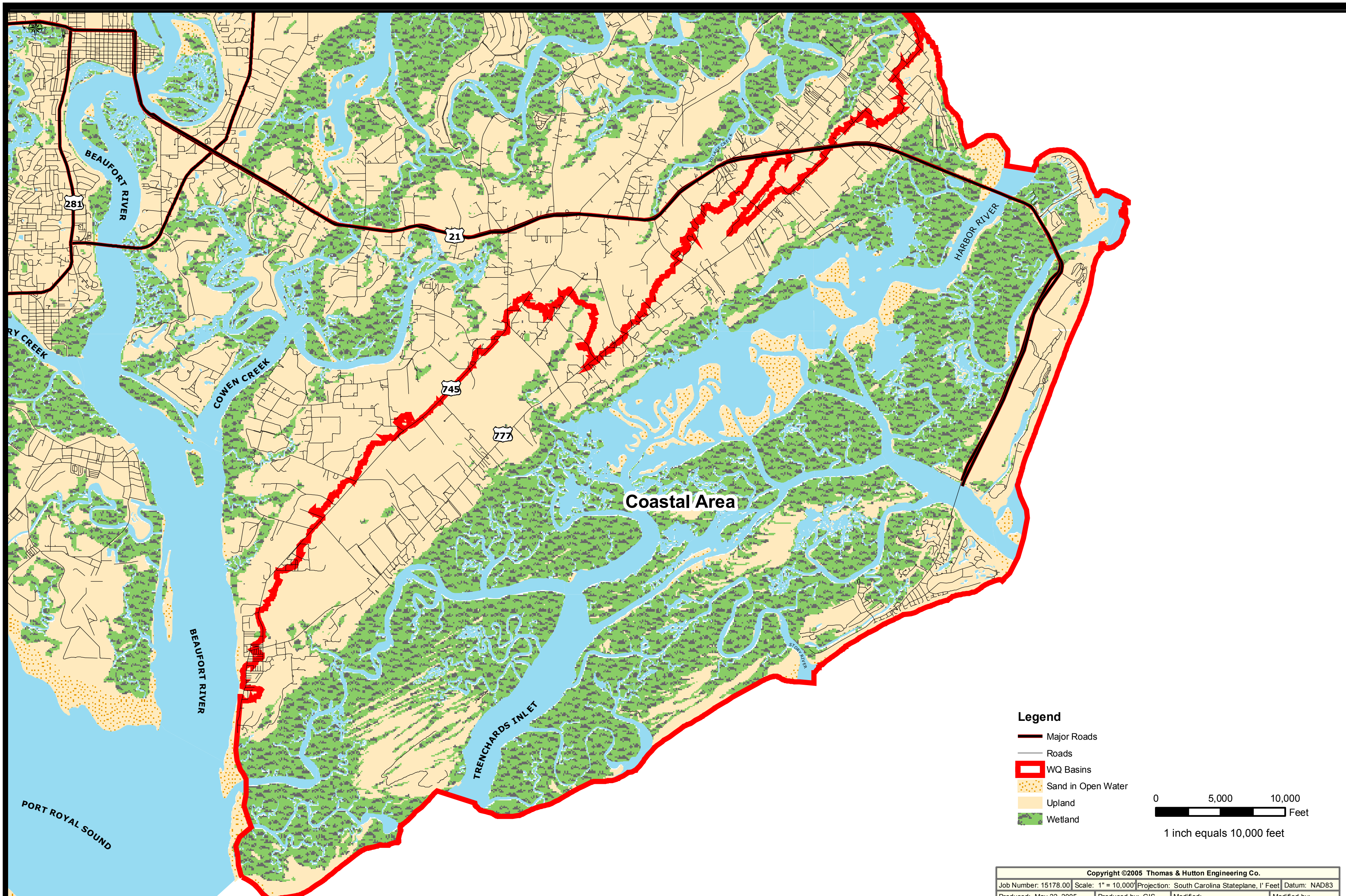
Subbasins	T&H / CDM	2004



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Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\CoastalWatershed_figure14-1.mxd			Vertical Datum: NAVD88

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Coastal Area Watershed Water Quality Basins

Figure 14-3

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Basins	T&H / CDM	2004
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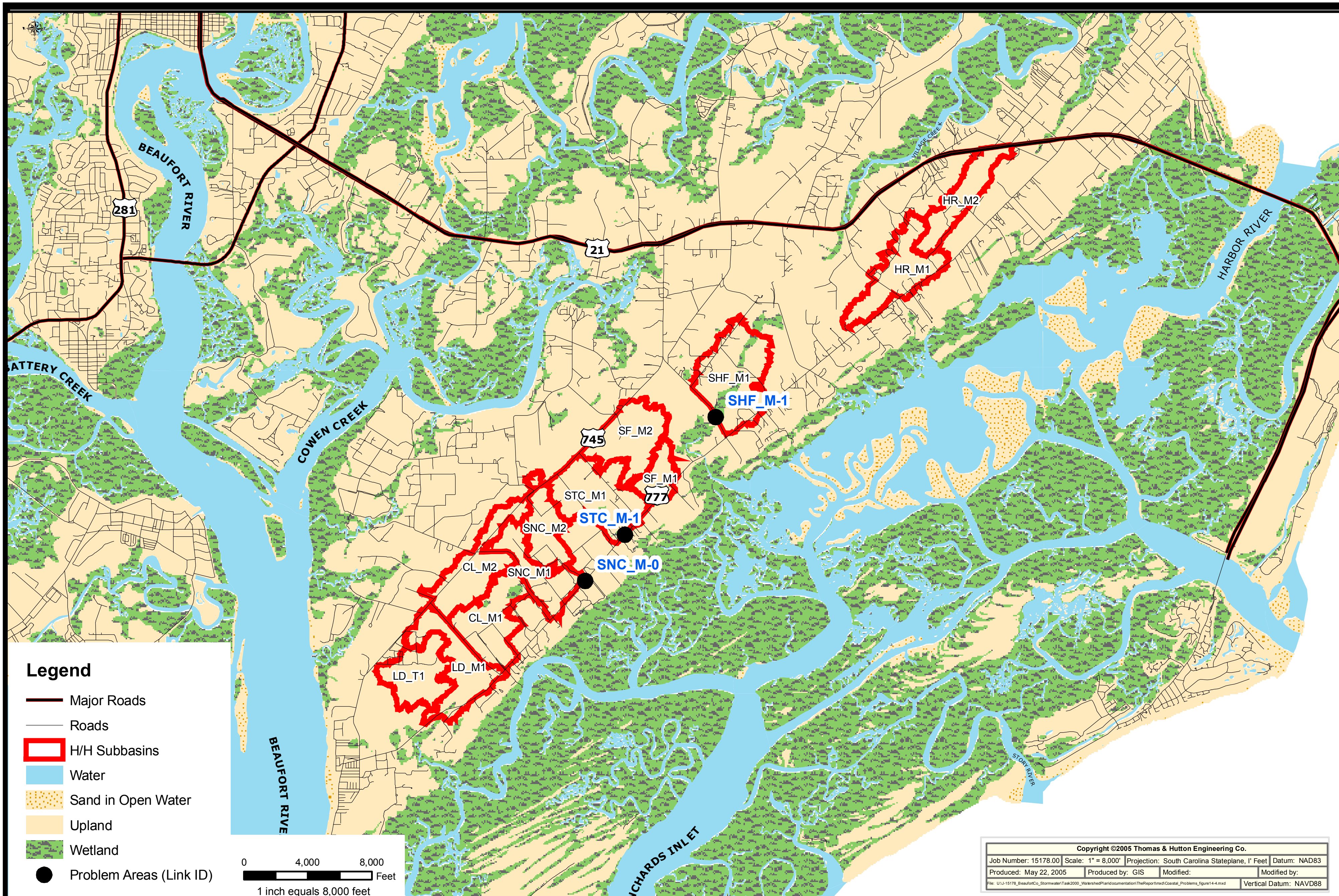

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Coastal Area Watershed
PSMS Problem Areas
Figure 14-4

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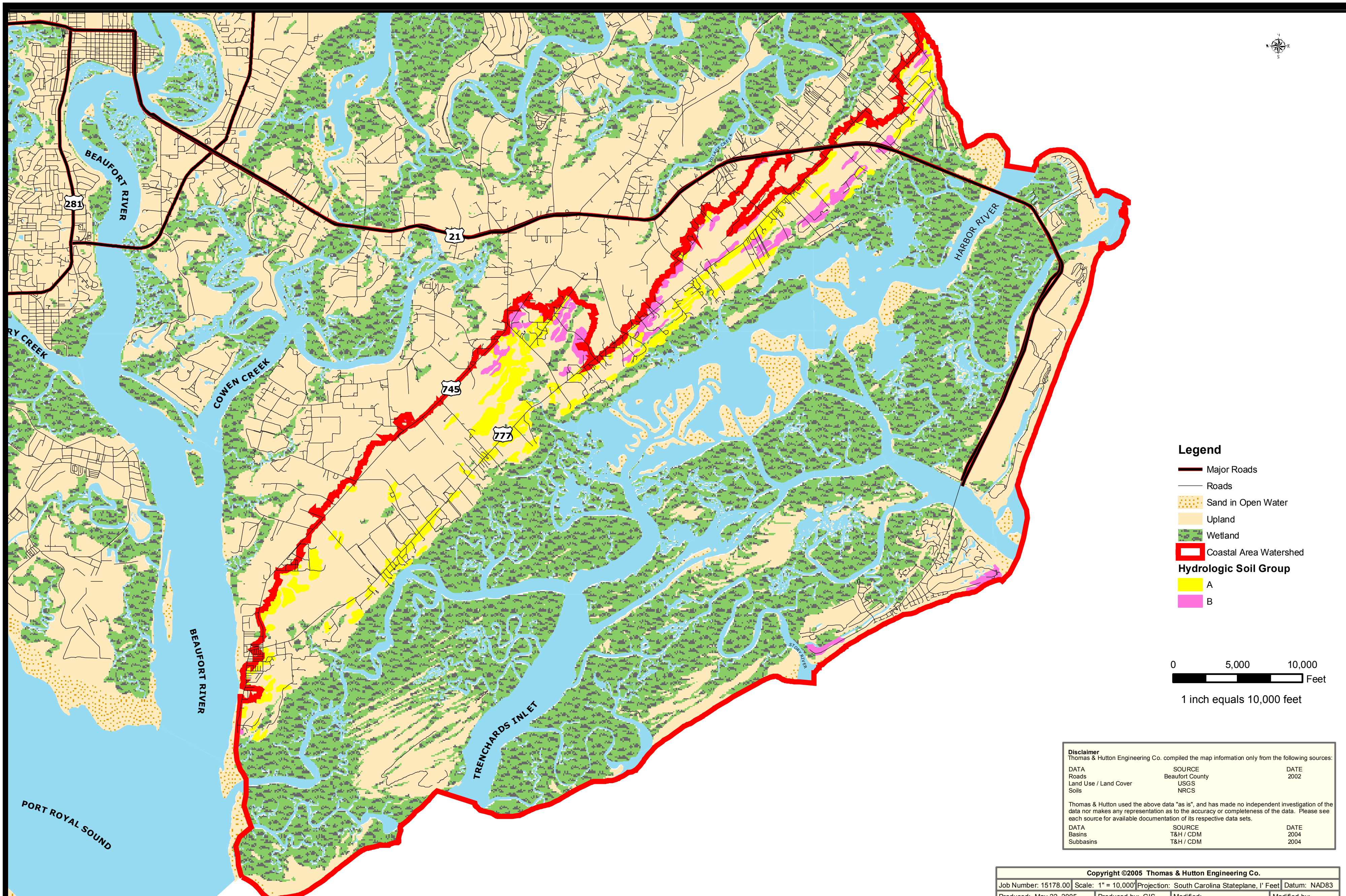
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Coastal Area Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 14-5

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Section 15
Hilton Head Island Analysis

This section describes the physical features of Hilton Head Island's watersheds, the potential water quantity problems, the modeling results, the alternative evaluations, and recommended actions.

It must be noted that the Town of Hilton Head Island performed a detailed Island Wide Drainage Study (IWDS) in August 1995. The purposes of the IWDS were to prepare an island-wide drainage inventory (including primary and secondary drainage systems), identify flood prone area, and present corrective actions to eliminate the flooding for a 25-year storm. Since 1995, numerous drainage improvements have been installed on the island, and the majority of the flooding problems have been eliminated for the 25-year, 24-hour storm. There are several advantages of the current study as compared to the 1995 IWDS. The advantages are listed below.

- The current study utilizes the 2002 LiDAR topography, NAVD88 datum. This topography is +/- six inches (vertical accuracy) and is more accurate than the 1986 Hargray-sponsored topographic maps utilized in the 1995 study.

- Hydrologic parameters for the 2006 study were generated via GIS from the digital soils survey, LiDAR topography, existing land use plan and digital elevation model and are electronically reproducible.

- Information provided to the Town of Hilton Head Island in the 1995 Study was delivered in the form of a hard copy. The information provided in the 2006 study consists of GIS derived hydrologic basins and subbasins, soils, land use coverages, topography and hydro-reinforced topography in electronic files, and electronic hydrologic and hydraulic models. By using the electronic information, the Town of Hilton Head Island may modify the models to reflect any modifications in hydrologic parameters, secondary drainage characteristics and record drawing information.

- Record drawing information provided by the County and Town of Hilton Head Island is now electronically incorporated in this study.

The 1995 Study was utilized for inventory background information along with the other drainage studies provided by the Town of Hilton Head Island. Thus, a great deal of the inventory information in the 1995 study is either repeated or utilized in the current study.

15.1 Overview

Hilton Head Island is a barrier island that occupies approximately 21,000 acres, has over twelve miles of beach, and has a population of approximately 34,000 people (U.S. Census 2000). It is part of the Carolina Low Country located at the southernmost tip of the South Carolina coastline. Access to the Island is by U.S. Highway 278, the Intracoastal Waterway, the Atlantic Ocean and the Hilton Head Island Airport.

The majority of the Island is fully developed and has been developed since the 1950s as planned communities. Within the Island, there are eleven planned communities and twenty-one championship golf courses. The planned communities of Hilton Head Island consist of:

A.	Hilton Head Plantation	4,000 ± acres
B.	Indigo Run	1,700 ± acres
C.	Long Cove Club	650 ± acres
D.	Palmetto Dunes	1,600 ± acres
E.	Palmetto Hall	750 ± acres
F.	Port Royal Plantation	950 ± acres
G.	Sea Pines	5,000 ± acres
H.	Shipyard Plantation	900 ± acres
I.	Spanish Wells Plantation	350 ± acres
J.	Wexford Plantation	500 ± acres
K.	Yacht Cove	125 ± acres

The terrain characteristic of Hilton Head Island is extremely flat with very little topographic relief. The low elevation of the island, compounded with the flat characteristic of the land, causes storm drainage to be a complex and critical issue. Flooding of Hilton Head Island can be the result of either rainfall, tides, storm surge or any combination of these events.

From the January 17, 1991, Beaufort County Flood Insurance Study, the anticipated Atlantic Ocean surges for the Hilton Head Island Area are listed below:

STILLWATER ELEVATION
FEET NATIONAL GEODETIC VERTICAL DATA NAVD (NGVD)

10-YEAR	50-YEAR	100-YEAR	500-YEAR
10.1 (11.0)	**11.9(12.8)**	**13.1 (14.0)**	**14.4 (15.3)**

The stillwater elevation is the ocean water elevation with no wave action. In addition to the stillwater elevation there is an increase in water elevation due to wave effects. For the 100-year frequency storm surge event, the Federal Emergency Management Agency (FEMA) states the wave crest elevation is 20.7 (21.6 NGVD) for Hilton Head Island. With this is mind, most of the island would be inundated in a wave crest analysis since the majority of the island is at elevation 13.1 (14 NGVD) or lower. The highest elevation is 27.1 (28.0 NGVD) and is located within Hilton Head Plantation.

The Hilton Head Island drainage facilities are not capable of accommodating a storm surge from the Atlantic Ocean. Within the planned communities of the Island, the drainage networks typically consist of a complex arrangement of inter-connected lagoons that ultimately discharge to the Atlantic Ocean. These lagoons serve as a storm water management tool, a water quality improvement tool, as well as an aesthetically pleasing environment for home owners, tourists and golfers. The complexity of the lagoon system is created by accumulation of storm water runoff within each lagoon. This accumulated runoff becomes a driving force (head) that controls the rate at which water releases from the system. The water accumulation in each lagoon impoundment provides a driving energy head that forces the flow of water from higher to lower areas. The rate at which the water flows from the higher to lower areas is dependent on the driving force (head). Thus, the interaction between the lagoons acts as an intricate and sensitive drainage network.

The drainage systems of the majority of the unplanned communities are comprised of sporadically placed piping, ditching and detention. The systems have not been designed with comprehensive planning to consider the drainage network efficiency, but have been developed as assemblies of quick relief treatments. These systems are generally less efficient systems that may have non-uniform longitudinal slope (and adverse slopes). Thus, drainage systems comprised of sporadically placed piping and ditching results in a less hydraulically efficient drainage network.

The delicacy of the Island drainage system is due to the ocean's tides and surges having a direct effect on the system's efficiency. For example, if a high tide or a surge coincides with a heavy rainfall event, flooding potential is substantially increased.

For the hydrologic and hydraulic analysis of the Primary Stormwater Management System (PSMS), the area includes several basins. These are listed in Table 15-1. Table 15-1 lists the basin names, tributary areas, number of subbasins, and average subbasin size. Hydrologic and hydraulic model calculations were done to evaluate peak flows and water elevations within the PSMS. The model results were compared to critical water elevations (e.g., roadway elevations) to identify potential problem areas and evaluate alternative management strategies.

For the analysis of pollution loads and receiving water quality, the island and tidal waters receiving stormwater runoff and baseflow from the island were subdivided into water quality basins, and the tidal receiving waters were subdivided into receiving segments. The water quality basins chiefly associated with the island are listed in Table 15-2, and presented later in this section as Figures 15-21, 15-22 and 15-23. Multiple figures are presented because the island and its receiving waters are actually in three watersheds – Calibogue Sound, Chechessee River, and Broad River – that were analyzed separately in Sections 3, 5 and 12 of this report, respectively. For one of the water quality basins (Broad River 4), the land area excludes the open water and tidal marshland land use that was included in the original Broad River 4 water quality basin, including only the upland area that is part of the island.

Pollutant loads were calculated for each of the water quality basins. For fecal coliform bacteria, tidal river water quality model calculations were done to evaluate river bacteria concentrations.

The model results were compared to the tidal river bacteria standards to identify potential problem areas and evaluate alternative management strategies.

15.2 Hydrologic and Hydraulic Analysis

CDM and T&H used the Interconnected Pond Routing Model (ICPR), Version 3 for the hydrologic and hydraulic analyses of the PSMS for the Hilton Head Island watersheds. The analyses included modeling of 24-hour design storms with return periods of 2 years, 10 years, 25 years, and 100 years. Analyses were conducted for existing and future land use conditions, with and without alternative management strategies.

The ICPR model is a "link-node" model, representing the PSMS as a series of nodes (stream locations) connected by links (open channels, pipes, culverts). Figures in Appendix M show model schematics of the Hilton Head Island PSMS basins, with separate sets of schematic for each basin.

The ICPR model was originally calibrated by the USGS regression equations. However, the equations are based upon large undeveloped tracts and not applicable to the Town of Hilton Head Island. Unfortunately, there is no applicable method to realistically calibrate the Hilton Head Island portion of the water quantity model. Since the Beaufort County PSMS consists of large sub basins, it is not comparable to more detailed studies performed in the past (i.e. 1995 IWDS). Since the 2006 study incorporates the cumulative area of lagoons on the secondary drainage systems and places the total drainage area on the primary system, this study does not include the effects of varying water release rates/elevations of secondary lagoons and hydraulic effects of secondary pipes. Thus, resulting stages from the primary drainage system study will be more conservative than high water mark elevations on the primary system.

15.2.1 Hydrologic and Hydraulic Parameters

In the hydrologic model development, each Hilton Head Island major basin consisted of one or more subbasins. Section 2.2 of this report describes how appropriate parameter values were developed for model subbasins. These parameters include area, curve number, and time of concentration.

Table 15-3 lists the hydrologic parameter values for the Hilton Head Island PSMS subbasins. Each model subbasin is identified by ICPR model ID number. Curve number and time of concentration values are presented for existing land use and future land use conditions. The future land use values generally show a higher curve number and lower time of concentration than the existing land use as a result of anticipated development. For Hilton Head Island, the future land use values generally result in unrealistic higher curve numbers and lower times of concentration. *These curve numbers could be applicable in areas where existing homes are being demolished and replaced with larger homes.* Hilton Head Island consists of numerous master planned unit developments, many of which have a lower density than specified in the existing land use plan. Presently, an amendment to the master plan must be approved in order to increase dwelling units per acre or modify a land use within a master planned subdivision. Per the current Town of Hilton Head Island Land Management Ordinance, any redevelopment would require on-site detention and retention. Theoretically, this would mean no increase in storm water runoff due to any future revised land use. If further analysis is to be completed, we recommend the approved planned unit development master plan be utilized for land use in the master planned unit developments. This methodology is not utilized in this study because the majority of Beaufort County is not master planned unit developments. To be consistent, reproducible and remain within the scope of this study, the same methodology was used for every watershed in this report.

Hydraulic summary information for the Hilton Head Island PSMS basins is presented in Table 15-4. For each basin, the table lists data regarding open channel sections, stream crossings, and other hydraulic features. Open channel data includes the number of defined open channel segments and the total length of the channel segments. Stream crossing data includes the number of stream crossings, the total number of culverts associated with those crossings, and the number of crossings that are actually bridge openings rather than culverts. Other features data includes the number of storage nodes, weirs, and tide gates that are part of the PSMS. Note that the number of weirs includes actual weir structures (e.g., inline weir across channel) as well as roadways that act as weirs if road overtopping is occurring.

Details regarding the stream crossings are presented in Table 15-5. For each stream crossing, the table presents the road name, ICPR model link ID, culvert dimensions and length, invert elevation, roadway elevation, and appropriate level of service.

Details regarding specific open channel segments, storage areas, weirs and tide gates are presented in Appendix M.

15.2.2 Hilton Head Island Tailwater Boundary Conditions

For the Beaufort County Storm water Management Plan, a mean annual high tide 5.6 NAVD88 (6.5 NGVD) is utilized as the tailwater for areas south of the Broad River. The mean annual high tide is derived by averaging the highest annual tide over the number of years that records are available (56). The 5.6 feet NAVD88 (6.5 NGVD) tailwater is for the Fort Pulaski National Monument at the mouth of the Savannah River. However, for the portion of the study pertaining to Hilton Head Island, it is recommended to utilize a tailwater of 3.0 NAVD88 (3.9 NGVD). Reasoning for this recommendation is discussed in detail below.

Design requirements for storm drainage systems are typically established by local and state agencies. For Hilton Head Island, the design requirements have been set by Beaufort County and the Town of Hilton Head Island. As typical for governing bodies, the design requirements have evolved, and each evolution produces more stringent requirements. Town of Hilton Head Island's design criteria have evolved from providing protection against a flood of 2.8 inches of rainfall in one hour, to the present 8 inches of rainfall in twenty-four hours. Requirements for drainage systems' tidal tailwater condition have not been addressed in any Town of Hilton Head Island/Beaufort County ordinances or design guidelines.

For Hilton Head Island, development started in the 1950's. Many of the roads, parking lots and existing developments are at elevations well below the mean annual high tide. Also, HHI is relatively fully developed, and the majority of the Island's lagoons water levels are at elevations 3.1 (4.0 NGVD) or lower. In contrast to the remainder of Beaufort County, the drainage outfalls for Hilton Head Island drain directly to tidal creeks and marshes. The majority of the remainder of Beaufort County is higher in elevation than Hilton Head Island and drains through a series of long wetlands that eventually empty into tidal outfalls. Direct connects into tidal areas, as opposed to draining through a series of wetlands, are much more effective and efficient in preventing flooding. To retrofit Hilton Head Island's existing drainage systems to comply with the mean annual high tide (5.6 NAVD88; 6.5 NGVD) tailwater boundary condition will require substantial drainage system upgrades, elevating roads and constructing dikes within some areas.

The 1995 Town of Hilton Head Island drainage study utilized the 25-year storm with a tailwater elevation of 3.0 NAVD88 (3.9 NGVD) as the boundary condition. The 3.0 NAVD88 tailwater condition is an average of the Mean Higher High Water and the Mean High Water and was determined appropriate and practical by Town staff and Thomas & Hutton for the 1995 study. This tailwater elevation was determined to be "reasonable" due to the island's low elevations, direct discharge from outfalls to the marsh, and its stage of development. The ocean storm surge was not considered as part of the 1995 Island Wide Drainage Study project scope and is not considered in this study either. Since a tailwater of 3.0 NAVD has been justifiably implemented in past studies and designs for the Town and no historical flooding if designs implementing this tailwater have been documented, it is recommended for Hilton Head Island to utilize a tailwater elevation of 3.0 NAVD88 (3.9 NGVD). As history demonstrates, construction of drainage systems originally designed with tidal tailwater elevations of 3.0 NAVD (3.9 NGVD) have yielded a safe, economical and practical engineering solution to discharging storm water on Hilton Head Island.

15.2.3 Model Results

Tables in Appendix M list the peak flow values for the Hilton Head Island sub basins. Each table lists peak flows for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak flows are listed by sub basins for various land cover and stormwater management controls, which include the following:

- Undeveloped land
- Existing land use without peak shaving controls
- Existing land use with existing peak shaving controls
- Future land use without peak shaving controls
- Future land use with peak shaving controls

It should be noted that the tables include values for "uncontrolled" and "controlled" peak flows for the 2-year, 10-year and 25-year design storms. The "uncontrolled" peak flow assumes no peak shaving facilities in the subbasin. In contrast, the "controlled" value accounts for peak shaving facilities in the subbasin.

For existing land use, aerial maps and local information were used to estimate the percentage of existing urban development that is served by peak shaving facilities. The "controlled" peak flow value was then calculated by considering the difference in peak flow between totally undeveloped conditions and existing conditions with no controls. For example, suppose that a subbasin of 100 acres has an undeveloped 2-year peak flow of 20 cfs, and an uncontrolled existing peak flow of 50 cfs, and further suppose that 60% of the urban development is controlled by peak shaving facilities. In this case, it is assumed that the existing peak flow is reduced by 60% of the difference between undeveloped and developed peak flow (50 – 20 = 30 cfs; 60% of 30 = 18 cfs reduction due to peak shaving), and therefore the maximum controlled peak flow will be 32 cfs (50 – 18).

For future land use, the "controlled" peak flow is set equal to the "controlled" peak flow for existing land use, because new development is subject to State and County peak flow regulations. Keep in mind, however, that the future condition will still generate more stormwater runoff volume, even though the peak flow is the same. The result is that the peak flow rate will be sustained for a longer period of time under future conditions.

Tables in Appendix M list the peak water elevation values for model node locations along the Hilton Head Island PSMS. Each table lists peak stages for one of the return periods analyzed in this study, which include 2-year, 10-year, 25-year, and 100-year return periods. In each of the tables, the peak stages are listed for existing and future land use conditions, with the existing hydraulic system.

Specific problem areas identified by the modeling are presented in Tables 15-6. For each area, the table identifies the road crossing, associated model ID, design storm, "critical elevation" (e.g., top-of-road elevation), and maximum water elevation for the listed design storm. As discussed earlier in Section 2, roads considered evacuation routes were evaluated with the 100-year design storm, and other roads were evaluated for the 25-year design storm.

The 2006 Stormwater Management Plan is a conservative plan which reflects potential flooding. Due to the conservative nature of the study, prior to installing any improvements, a detailed drainage analysis is recommended to be performed to size the improvements. Thus, due to the general modeling approach, there are various areas within Sea Pines Plantation and Hilton Head Plantation (HHP) that the ICPR model indicates are flooding. However, based upon the detailed analysis of the 1995 study, some of these areas do not flood during a 25-year storm event if the drainage system is serviceable.

The pseudo-flooding in this study is caused by several factors listed below.

- Watershed Basin Size Restraints – It is the scope of this study to include watershed basins with the smallest sub-basins having an average area of 0.5 square miles. For Hilton Head Island, the **largest** watershed is less than 0.4 square miles. Due to the extensive lagoon system, to delineate Hilton Head Island sub-basins by each lagoon watershed is beyond the scope. Since the secondary system is not modeled, the watershed basins being discharged into the primary system cause pseudo-staging.

- Secondary drainage system – It is beyond the scope of this study to model lagoons and drainage features located on the secondary system. The HHP drainage system, for example, is composed of many lagoons inter-connected by pipes or weir/pipe combinations. In this study, all the storage capacity of secondary lagoons was cumulatively added to a node located on the primary system (cumulative storage capacity per basin). This causes pseudo-flooding in the model because all the runoff is discharged into the primary system at once. This does not account for delays in flow reaching the primary system due to differing storage potentials or differing runoff release times in the inter-connected secondary lagoons (delayed discharge). For example, an outfall pipe may delay the discharge from a secondary lagoon, such that the volume of discharge of the secondary lagoon would not enter the primary system until the primary system water elevations have receded.

- One hydrograph per basin – The hydrograph for the entire sub-basin discharges into the primary system at one time. Since all sub-basins within HHP are 0.4 square miles or less, it is not within the scope of this study to further divide the sub-basin into smaller areas. Dividing sub-basins further would allow the model to account for differing storage potentials and release times. Also, it would decrease the volume of water entering the primary system at the peak of the storm event.

Evaluation of solutions to correct these problems is discussed in the next section of the report.

15.2.4 Management Strategy Alternatives

The problem areas listed in Table 15-6 were evaluated by modifying the culverts in the ICPR hydraulic model. The ICPR model for existing conditions was modified to either add one or more culverts to the existing culvert(s), or to replace the existing culvert(s) with one or more new culverts. Replacement was typically considered if the model results showed that the existing culvert or culverts passed less than half the peak flow, and most of the peak flow passed over the road for the design storm. In contrast, addition of one or more culverts was typically assumed in cases where the existing system was able to pass most of the peak flow, and a small fraction of the peak flow is passed over the road.

The resulting improvements are presented in Table 15-7. The table presents the sizes of the existing culverts, plus the sizes of the added or replacement culverts. For the analysis, box culverts were used as the added or replacement culverts. There is no reason that a different culvert shape could not be used, as long as the conveyance capacity of the culvert(s) remains the same. Also, the depth of the added or replacement culverts was usually assumed to be equal to the height of the existing culvert(s), because there was often little freeboard between the crown of the existing culvert(s) and the top of the road. The height of the added or replacement culvert(s) was greater than that of the original culvert(s) only when there was sufficient freeboard.

15.3 Water Quality Analysis

CDM and T&H used the Watershed Management Model (WMM) and the Water Quality Analysis Simulation Program (WASP) for the water quality analysis of Hilton Head Island. WMM was used to calculate average annual flows and average annual loads of various water quality constituents, including fecal coliform bacteria, total nitrogen (total N), total phosphorus (total P), BOD, lead, zinc and suspended solids. WMM was also used to calculate the geometric mean bacteria concentration of the flows from the watershed to the tidal river system. The flow and geometric mean concentration data were used as input to the WASP model, which accounted for tidal mixing and bacteria die-off, to evaluate bacteria concentrations in the tidal river system for existing and future conditions. Measured salinity and bacteria concentrations were used to calibrate key model parameters such as tidal mixing coefficients and bacteria die-off rates for existing conditions. The same parameter values were used for evaluation of future conditions, which reflect higher flows and loads from the watershed.

15.3.1 Land Use and BMP Coverage

Table 15-8 presents the existing land use and future land use estimates for Hilton Head Island water quality basins. The existing land use data reflects a number of sources, including February 2002 aerials, County existing land use and tax parcel maps, National Wetlands Inventory (NWI) and USGS quadrangle maps, plus local knowledge of development. The future land use map was developed by "filling in" the existing land use map, replacing undeveloped area with anticipated urban development. The anticipated future development was characterized based on the Beaufort County and Town of Hilton Head Island future land use maps and zoning maps.

Under existing land use conditions, 71 percent of the PSMS tributary area consists of urban systems (e.g., residential, commercial, golf course) and 29 percent consists of natural systems (e.g., forest, water/wetlands, tidal open water/marsh). Based on the imperviousness values assigned to urban land uses, urban impervious area covers about 26 per cent of the watershed.

Under future land use conditions, 72 percent of the PSMS tributary area consists of urban systems, and 28 percent consists of natural systems. The changes in land use distribution reflect the conversion of forest/rural and urban open land to golf course, medium density residential, commercial, industrial and institutional land uses. As a result of projected future development, urban imperviousness increases to about 28 percent of the watershed.

Estimates of BMP coverage for existing and future land use is presented in Table 15-9. The existing land use values reflect local knowledge of development with respect to the implementation of BMPs on Hilton Head Island. Future BMP coverage was estimated presuming that all new development would be treated by BMPs in accordance with the County BMP Manual. Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by BMPs relative to the total urban land area. The overall "total" BMP coverage (lower right corner value in the two tables) reflects what percentage of all urban land in the watershed in served by BMPs

Under existing land use conditions, it is estimated that 69 percent of the urban systems in the watershed (e.g., residential, commercial, golf course) are served by BMPs. Under future land use conditions, 73 percent of the urban systems are served by BMPs. Thee increase from existing to future reflects the expectation that 100% of the new development will be treated with BMPs in accordance with the County BMP Manual.

15.3.2 Septic Tanks and Point Sources

Estimates of septic tank usage for existing and future land use are presented in Table 15-10. The existing land use values reflect areas that are not designated as "sewered" areas by the Beaufort-Jasper Water & Sewer Authority or the Public Service Districts (PSDs) on the Town of Hilton Head Island. For future development, areas that are zoned "rural" or "conservation" were assumed to be served by septic tanks, and other areas were assumed to be served by sewer.

Values are presented for developed urban land uses. The "total" value for each water quality basin is based on the total urban area served by septic tanks relative to the total urban land area. The overall "total" septic tank coverage (lower right corner value in the two tables) reflects the percentage of all urban land in the watershed that is served by septic tanks.

For existing land use conditions, 14 percent of the urban systems in the watershed are served by septic tanks. Under future land use conditions, 13 percent of the urban systems are served by septic tanks. This decrease reflects the presumption that new development will be sewered.

Wastewater discharges are roughly 4 million gallons per day (MGD) of land application (e.g., golf course irrigation), and the future discharge is expected to be slightly higher (between 4 and 5 mgd). There are no direct discharges to receiving waters in the watershed.

15.3.3 Model Annual Pollution Load Results

Average annual constituent loads were calculated for the Hilton Head Island water quality basins using the methodology described in Section 2.4 of the report. Loads were calculated for existing and future (build-out) land use conditions. The loads were tabulated and compared to evaluate the relative changes in loads due to new development, assuming that the new development is controlled by BMPs in accordance with the County BMP Manual.

The results are presented in Table 15-11 for existing and future land use conditions. For each water quality basin and land use condition, the table lists the basin tributary area, total average annual flow in acre-feet, and the average annual loads for each of the seven constituents considered in the study. With the exception of fecal coliform bacteria, the loads are presented in units of pounds per year. Fecal coliform results are presented in units of counts per year (#/yr).

An overall comparison of the WMM modeling results (Table 15-11) indicates that future flows and constituent loads generally increase marginally over their existing counterparts; however, in the case of fecal coliform bacteria loads, a very small decrease is experienced. Specifically, future flow is 2 percent greater than for existing conditions and the increase in loads ranges from 3 percent for BOD to -2 percent (slight reduction in load) for fecal coliform bacteria. The fecal coliform load reflects the fact that BMPs are typically very efficient in removing bacteria in stormwater runoff. In addition, all of the basins have relatively small changes in percent urban imperviousness from existing to future conditions.

Direct and indirect wastewater discharges account for a very small fraction of the total watershed load for all constituents, particularly fecal coliform bacteria. As shown previously in Table 2-9, the existing indirect discharge of wastewater for the Calibogue Sound and Broad River watersheds (which reflects the discharges on the island) are limited to roughly 5 million gallons per day (MGD) of land application (e.g., golf course irrigation), and the future discharge is expected to be slightly higher (between 5 and 6 mgd). Using the values in Table 2-9, the wastewater load accounts for 7 to 13 percent of the total island load for nutrients (total nitrogen and total phosphorus) and less than 2% of the load for other constituents.

15.3.4 Model Tidal River Water Quality Results

The WASP model was applied to evaluate geomean concentrations of fecal coliform bacteria in the receiving waters of Hilton Head Island. The model actually includes Calibogue Sound, May River, Colleton River, and Chechessee River watersheds because they are interconnected at several points. Only the island receiving waters will be discussed in this section. A schematic of the model is presented as Figure 15-24.

Existing conditions for bacteria concentrations in the island receiving waters are presented in Table 15-12. For each water quality basin river reach, the table lists the DHEC stations for which the 1990s bacteria data were analyzed, the concentrations calculated in the analysis, and the "level of service" associated with these concentrations (as discussed in Section 2.6.2. As shown in the table, DHEC data were only available in eight of the river model segments. For both the long-term and the 36-sample maximum values, the geomean and 90th percentile bacteria concentrations in eight of the twelve segments meet the water quality standards, and so these segments have an "A" level of service. Segments that do not meet the "A" level of service include three segments in Broad Creek , and Broad Creek 4 (the headwater segment in Broad Creek) is unlikely to meet the "A" level of service if Broad Creek 3 does not.

For informational purposes, Figures 15-25 and 15-26 present maps of the level of service based on the monitoring data analysis, compared to the Department of Health and Environmental Control (DHEC) "shellfish classification" (based on the 2002 DHEC reports for shellfish areas 16A, 17, 18, 19 and 20) for the Calibogue Sound and Chechessee River watersheds, respectively. The shellfish classification is based on data from a specific 3-year monitoring period that is different than the period of data used to develop the level of service, so there may not be a direct relationship between level of service and shellfish classification presented in the map. In general, however, segments with an "A" level of service are expected to have the lowest probability of receiving a "restricted" classification, and segments with a "D" level of service are expected to have the highest probability of receiving a "restricted" classification.

Physical characteristics assigned to the model reaches are presented in Table 15-13. The average segment volume is listed, as well as tidal dispersion information. This information includes the segments between which mixing is simulated, and parameters used to calculate dispersion, such as the cross-sectional area, the "characteristic length" (typically the distance between segment midpoints) and a dispersion coefficient. The area and length are based on physical data (e.g., bathymetric data), whereas the dispersion coefficient was established through calibration of the modeled salinity to average salinity values calculated from the DHEC monitoring data.

Other key model input includes the average flows and geomean bacteria concentrations, and net advective flows between river segments. Tables 15-14 and 15-15 show the values used in the existing and future condition models.

A review of Table 15-14 shows that there is typically little change in flow or concentration between existing and future land use. For flow, this is because much of the flow to the tidal river segments comes from direct rainfall on the open water and tidal wetlands, as opposed to stormwater runoff and baseflow, and the basins have very little change in land use from existing to future conditions. Concentration remain relatively constant because of the substantial amount of open water/tidal wetland area and the relatively limited development in some basins, as well as the BMPs for new development, which are assumed to have a high level of treatment efficiency.

Table 5-15 shows the net advective flows between segments, which also do not change substantially from existing to future land use.

The final key input parameter for bacteria modeling is the first-order loss rate. The value of this parameter was adjusted so that the measured geomean concentrations and modeled geomean concentrations were in agreement, for those river segments that had measured data. In general, a loss rate of 1.0/day was assumed initially, and values were then adjusted to achieve a better match between modeled and measured data. The final calibration values will be discussed below.

Figure 15-27 is a graph showing a comparison between measured and modeled salinity data along the Calibogue Sound main stem. The figure shows that the salinity data calculated by the model is very close to the average measured value, and is in all cases well within the 90 percent confidence interval of the mean of the salinity data. Measured salinity values do not vary much along the main stem.

Figures 15-28 and 15-29 are graphs showing a comparison between measured and modeled salinity data for Broad Creek and for Old House Creek/Jarvis Creek, respectively. These are tributaries whose contributing area is entirely within the Town of Hilton Head Island. The figures show that the salinity data calculated by the model is very close to the average measured value, and is in all cases well within the 90 percent confidence interval of the mean of the salinity data. Measured and modeled salinity values drop noticeably at the upstream segments of Broad Creek, whereas the measured and modeled salinity values do not vary much in Old House Creek/Jarvis Creek.

Figure 15-30 is a graph showing a comparison between measured and modeled salinity data along Skull Creek. The figure shows that the salinity data calculated by the model is very close to the average measured value, and is in all cases well within the 90 percent confidence interval of the mean of the salinity data. Measured salinity values do not vary much along the main stem.

The comparison of measured geomean bacteria concentrations and modeled bacteria concentration for the same waters are presented in Figures 15-31 through 15-34. The graphs generally show the same type of results as the salinity plots. Results for Calibogue Sound (Figure 15-31) , Old House Creek/Jarvis Creek (Figure 15-33) and Skull Creek (Figure 15-34) show very good agreement between the measured values and the model results. For Broad Creek (Figure 15-32), the model is not able to replicate the high bacteria concentration measured in the Broad Creek 3 segment, which may be due to the underestimation of bacteria loads in that basin and the upstream Broad Creek 4 basin. Nevertheless, both the measured and modeled results suggest a "D" level of service there.

The first-order loss rates assigned to the river segments, and the concentrations calculated by the model, are presented in Table 15-16. The loss rates ranged from 0.5/day to 2.0/day. The lowest values are typically applied at the downstream end of the main stem and major. This makes sense if it is presumed that bacteria loss is in part due to light mortality, because the water depths are much greater at the downstream end of the main stem and major tributaries, and light would penetrate less of the total depth in those areas.

After the model was applied for existing conditions, it was then applied for future conditions. The physical characteristics and first-order loss rate from the existing land use model were kept the same in the future land use model. The only changes were the net advective flows and the bacteria loads.

The bacteria concentrations calculated under future land use conditions are presented in Table 15-16 as well. A comparison of concentrations under existing and future land use conditions shows little difference. According to the model, all of the river reaches will have the same level of service in the future as they do under existing conditions.

In order to estimate the degree to which stormwater management measures are expected to affect instream bacteria concentrations, two sensitivity runs were conducted. The first was run for the existing land use condition, and represents a "best-case" scenario in which all existing development is controlled by BMPs. The second was run for the future land use condition, and represents a "worst-case" condition in which no development is served by BMPs. Analyzing the results of these scenarios indicate the benefits of retrofitting existing development with BMPs, and the potential degradation of river segments if BMPs fail.

The results of the analysis are presented in Table 15-17. This table is similar to Table 15-16, in this case showing water quality basin segment fecal coliform concentrations for the "best case" and "worst case" analyses. Segments that show change (e.g., better LOS for the "best case" or degraded LOS for the "worst case") are highlighted.

A review of the "best-case" scenario indicates that three model segments show improvement in the existing level of service. These include Broad Creek 2, Broad Creek 3, and Jarvis Creek 2. The Jarvis Creek 2 segment shows the greatest improvement, going from a "D" to a "B" level of service. Note that the improvement in Broad Creek 2 and 3 assumes 100% BMP coverage in those water quality basins as well as upstream water quality basin Broad Creek 4. Similarly, the improvement in Jarvis Creek 2 assumes 100% BMP coverage in that water quality basin as well as the downstream water quality basin Jarvis Creek 1, which reduces the bacteria load to Jarvis Creek 2 from Jarvis Creek 1 on the incoming tide.

A review of the “worst-case” scenario indicates that three model segments show degradation in the future level of service when no BMPs are assumed. These include Broad Creek 1 and Broad Creek 2. Broad Creek 1 drops from an “A” to a “C” level, though the change in geomean concentration (from 6.7/100 ml to 8.8/100 ml) is small. Broad Creek 2 drops from a “B” to a “D” level of service.

Based on water quality sampling data and model results, the following recommendations are made:

- Request that DHEC add bacteria sampling stations in the water quality basin Jarvis Creek 2, to validate model results

- Evaluate opportunities for retrofit BMPs or modification of existing ponds in the Broad Creek water quality basins to the maximum extent practicable.

- Consider monitoring major stormwater outfall locations to the Broad Creek water quality basins (the Town of Hilton Head Island is already doing this)

- Consider bacterial source tracking (BST) to identify the sources of unexpectedly-high bacteria levels in Broad Creek 3 and 4

More discussion of the overall recommended monitoring program for Beaufort County is presented in Section 16 of this report.

15.3.5 Management Strategy Alternatives

The results of the water quality analysis suggest that several areas (e.g., Broad Creek) do not meet the bacteria water quality standards under existing conditions, and a few other segments may have degradation in level of service based on future conditions. Areas such as Broad Creek appear to be affected by urban development, and it is appropriate to evaluate measures that could be taken to meet the water quality standards, or perhaps more realistically, to improve the existing level of service. As discussed above, these activities would include retrofit of existing development that does not have ponds, and modification of existing ponds that may not have been designed for water quality control.

Elements of the water quality management plan for the Calibogue Sound and Chechessee River watersheds – the watersheds with receiving waters affected by the island - are presented in Figures 15-35 and 15-36, respectively. Sampling stations shown in the figure include existing DHEC sites, as well as the additional open water sites that are recommended as discussed in Section 15.3.4 above. Also identified are “priority” water quality basins. Sensitivity analysis results suggest that load changes in these basins are most likely to result in an improved or degraded LOS in the receiving waters.

For informational purposes, the areas with "A" and "B" type soils for the Calibogue Sound and Chechessee River watersheds are presented in Figures 15-37 and 15-38, respectively. In general, these soils are more suitable for infiltration BMPs than areas with "C" and "D" type soils, though high water table conditions may still limit the effectiveness of infiltration BMPs in these areas. The figure is provided to indicate areas where new development BMP design should consider infiltration BMPs as a primary or secondary treatment method.

15.4 Planning Level Cost Estimates for Management Alternatives

Table 15-18 lists potential projects identified in the hydrologic and hydraulic analysis of the PSMS in the Hilton Head Island watershed. As shown in the table, the projects are estimated to have a total cost of $1.8 million in December 2004 dollars. Details of the cost estimate for each project are shown in Appendix M.

The prioritization of these projects identified for other watersheds, is discussed in Section 16 of this report. Most of the proposed improvements are located within private developments and are considered to be low priority. Also, based on our knowledge of historical rainfalls and flooding, most of the modeled flooding would not occur. **It is recommended that any areas indicated to flood be modeled with the modeling extended into the secondary systems to reflect stage/storage and varying discharge release rates.**

TABLE 15-1
HYDROLOGIC BASINS
HILTON HEAD ISLAND WATERSHEDS

Basin Names	Tributary Area (acres)	Number of Subbasins	Average Subbasin Size (acres)
BA-SPP-01	698	11	63
BA-SPP-02	163	5	33
BA-SPP-03	177	4	44
BC-SPP-01	77	2	38
BR-CHP-01	263	4	66
BR-IRP-01	935	10	93
BR-IRP-02	679	7	97
BR-LCC-01	618	8	77
BR-LCC-02	9	1	9
BR-PCT-01	70	6	12
BR-PCT-02	31	2	16
BR-PDP-01	1,610	14	115
BR-PRP-01	967	14	69
BR-WEX-01	1,390	15	93
BR-WEX-02	135	4	34
BR-XNG-01	161	3	54
CA-SPP-01	84	1	84
CA-SPP-02	41	1	41
FH-AIR-01	453	4	113
FH-PRP-01	687	6	115
JV-GUM-01	222	1	222
JV-HHP-01	1,080	11	98
JV-IRP-01	278	3	93
LC-SPP-01	1,778	11	162
OH-SPW-01	137	3	46
PA-HHP-01	839	6	140
PC-SPP-01	297	5	59
PC-SPP-02	128	4	32
PR-HHP-01	687	4	172
PR-HHP-02	178	3	59
PR-PHP-01	885	8	111
SK-GUM-01	266	3	89
SK-HHP-01	229	4	57
SK-HHP-02	108	3	36
TOTAL	16,357	191	86

TABLE 15-2
WATER QUALITY BASINS
HILTON HEAD ISLAND

Basin Name	Tributary Area (acres)
Broad Creek 1	4,219
Broad Creek 2	7,846
Broad Creek 3	750
Broad Creek 4	1,417
Old House Creek	288
Jarvis Creek 1	927
Jarvis Creek 2	1,924
Broad River 4 *	4,438
TOTAL	21,809

* excludes open water/tidal marshland that was included in the Broad River 4 basin in Section 12

TABLE 15-3N
SUBBASIN HYDROLOGIC CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (NORTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Chaplan Area - Broad Creek Outfall - Major Basin 1 BR-CHP-01					
BR-CHP-01-001	39.5	84	41	87	37
BR-CHP-01-002	35.2	81	42	89	32
BR-CHP-01-003	10.3	92	19	94	17
BR-CHP-01-004	178.2	88	63	88	63
Indigo Run - Broad Creek - Major Basin 1 BR-IRP-01					
BR-IRP-01-001	48.0	83	60	87	53
BR-IRP-01-002	56.1	78	65	78	65
BR-IRP-01-003	265.0	78	145	78	145
BR-IRP-01-004	65.9	91	62	91	62
BR-IRP-01-005	124.3	85	89	91	71
BR-IRP-01-006	60.9	88	59	89	57
BR-IRP-01-007	21.2	86	31	88	29
BR-IRP-01-008	109.6	85	87	85	87
BR-IRP-01-009	28.6	81	40	81	40
BR-IRP-01-010	155.0	76	82	79	75
Indigo Run - Broad Creek - Major Basin 2 BR-IRP-02					
BR-IRP-02-001	25.2	74	46	74	46
BR-IRP-02-002	144.0	81	94	82	91
BR-IRP-02-003	44.2	79	62	79	62
BR-IRP-02-004	81.6	81	79	81	79
BR-IRP-02-005	102.7	79	107	81	101
BR-IRP-02-006	115.0	84	98	85	95
BR-IRP-02-007	166.3	82	92	82	92
Airport - Fish Haul Creek - Major Basin 1 FH-AIR-01					
FH-AIR-01-001	92.7	74	92	79	80
FH-AIR-01-002	85.2	81	62	92	41
FH-AIR-01-003	58.3	82	80	85	72
FH-AIR-01-004	216.7	80	108	85	92
Gum Tree - Jarvis Creek - Major Basin 1 JV-GUM-01					
JV-GUM-01-001	222.1	81	95	83	89

TABLE 15-3N

SUBBASIN HYDROLOGIC CHARACTERISTICS

HILTON HEAD ISLAND WATERSHEDS (NORTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Hilton Head Plantation - Jarvis Creek - Major Basin 1 JV-HHP-01					
JV-HHP-01-001	170.2	79	101	80	97
JV-HHP-01-002	19.7	90	59	92	54
JV-HHP-01-003	128.9	87	89	88	86
JV-HHP-01-004	102.5	87	65	90	58
JV-HHP-01-005	151.6	87	68	89	63
JV-HHP-01-006	94.0	87	70	87	70
JV-HHP-01-007	101.6	85	91	85	91
JV-HHP-01-008	72.4	85	54	85	54
JV-HHP-01-009	99.9	82	86	82	86
JV-HHP-01-010	27.5	71	68	72	66
JV-HHP-01-011	112.0	85	59	87	55
Indigo Run - Jarvis Creek - Major Basin 1 JV-IRP-01					
JV-IRP-01-001	35.4	85	45	86	43
JV-IRP-01-002	99.1	73	148	73	148
JV-IRP-01-003	143.0	69	146	69	146
Spanish Wells Plantation - Old House Creek - Major Basin 1 OH-SPW-01					
OH-SPW-01-001	37.1	66	61	66	61
OH-SPW-01-002	67.6	69	78	69	78
OH-SPW-01-003	32.2	63	62	63	62
Hilton Head Plantation - Park Creek - Major Basin 1 PA-HHP-01					
PA-HHP-01-001	219.8	84	75	84	75
PA-HHP-01-002	86.8	84	65	84	65
PA-HHP-01-003	124.5	82	67	82	67
PA-HHP-01-004	87.8	85	52	85	52
PA-HHP-01-005	187.9	87	50	87	50
PA-HHP-01-006	132.6	87	75	87	75
Hilton Head Plantation - Port Royal Sound - Major Basin 1 PR-HHP-01					
PR-HHP-01-001	94.2	78	62	78	62
PR-HHP-01-002	81.1	80	65	80	65
PR-HHP-01-003	357.4	82	100	82	100
PR-HHP-01-004	153.8	88	89	88	89
Hilton Head Plantation - Port Royal Sound - Major Basin 2 PR-HHP-02					
PR-HHP-02-001	22.6	83	35	83	35
PR-HHP-02-002	63.5	86	55	86	55
PR-HHP-02-003	91.5	79	63	79	63

TABLE 15-3N
SUBBASIN HYDROLOGIC CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (NORTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Palmetto Hall - Port Royal Sound - Major Basin 1 PR-PHP-01					
PR-PHP-01-001	94.0	70	98	77	81
PR-PHP-01-002	110.7	77	93	78	91
PR-PHP-01-003	79.9	85	57	85	57
PR-PHP-01-004	158.2	86	86	86	86
PR-PHP-01-005	101.9	81	89	82	86
PR-PHP-01-006	158.6	80	79	80	79
PR-PHP-01-007	80.3	77	70	77	70
PR-PHP-01-008	101.5	78	89	80	84
Gum Tree - Skull Creek - Major Basin 1 SK-GUM-01					
SK-GUM-01-001	79.7	83	69	85	65
SK-GUM-01-002	93.0	86	58	86	58
SK-GUM-01-003	93.2	84	82	88	71
Hilton Head Plantation - Skull Creek - Major Basin 1 SK-HHP-01					
SK-HHP-01-001	52.5	80	88	81	86
SK-HHP-01-002	11.8	78	44	78	44
SK-HHP-01-003	54.7	77	66	77	66
SK-HHP-01-004	109.8	89	66	89	66
Hilton Head Plantation - Skull Creek - Major Basin 2 SK-HHP-02					
SK-HHP-02-001	41.4	80	43	80	43
SK-HHP-02-002	38.1	75	46	75	46
SK-HHP-02-003	28.1	77	49	77	49

TABLE 15-3S
HYDROLOGIC SUBBASIN CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (SOUTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Sea Pines - Baynard Cove Outfall - Major Basin 1 BA-SPP-01					
BA-SPP-01-001	51.50	75	59	75	59
BA-SPP-01-002	97.93	70	93	70	93
BA-SPP-01-003	42.42	76	50	76	50
BA-SPP-01-004	34.16	82	44	82	44
BA-SPP-01-005	13.94	76	37	76	37
BA-SPP-01-006	47.64	64	96	64	96
BA-SPP-01-007	57.23	67	199	67	199
BA-SPP-01-008	82.68	65	168	65	168
BA-SPP-01-009	170.19	70	116	70	116
BA-SPP-01-010	91.07	61	187	61	187
BA-SPP-01-011	9.15	79	55	79	55
Sea Pines - Baynard Cove Outfall - Major Basin 2 BA-SPP-02					
BA-SPP-02-001	31.23	70	61	70	61
BA-SPP-02-002	47.22	72	74	72	74
BA-SPP-02-003	45.75	74	56	74	56
BA-SPP-02-004	11.43	77	49	77	49
BA-SPP-02-005	27.73	64	67	68	60
Sea Pines - Baynard Cove Outfall - Major Basin 3 BA-SPP-02					
BA-SPP-03-001	40.66	79	59	79	59
BA-SPP-03-002	62.09	72	56	72	56
BA-SPP-03-003	60.95	61	90	61	90
BA-SPP-03-004	13.01	81	34	81	34
Sea Pines - Braddock Cove Outfall - Major Basin 1 BC-SPP-01					
BC-SPP-01-001	47.39	78	46	78	46
BC-SPP-01-002	29.35	79	32	79	32
Long Cove Club - Broad Creek - Major Basin 1 BR-LCC-01					
BR-LCC-01-001	32.8	80	49	80	49
BR-LCC-01-002	101.9	82	64	82	64
BR-LCC-01-003	68.4	82	64	86	56
BR-LCC-01-004	114.3	77	102	77	102
BR-LCC-01-005	58.6	81	46	81	46
BR-LCC-01-006	180.2	76	80	77	78
BR-LCC-01-007	31.3	81	47	81	47
BR-LCC-01-008	30.3	63	58	63	58

TABLE 15-3S
HYDROLOGIC SUBBASIN CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (SOUTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Long Cove Club - Broad Creek - Major Basin 2 BR-LCC-02					
BR-LCC-02-001	8.6	76	24	76	24
Point Comfort - Broad Creek - Major Basin 1 BR-PCT-01					
BR-PCT-01-001	5.0	84	15	84	15
BR-PCT-01-002	4.7	89	16	89	16
BR-PCT-01-003	8.5	83	18	84	18
BR-PCT-01-004	2.9	84	17	87	15
BR-PCT-01-005	27.4	80	44	84	39
BR-PCT-01-006	21.3	85	29	85	29
Point Comfort - Broad Creek - Major Basin 2 BR-PCT-02					
BR-PCT-02-001	12.6	84	27	85	26
BR-PCT-02-002	18.5	82	45	82	45
Palmetto Dunes - Broad Creek - Major Basin 1 BR-PDP-01					
BR-PDP-01-001	36.6	83	41	84	40
BR-PDP-01-002	171.1	87	104	87	104
BR-PDP-01-003	21.3	89	44	89	44
BR-PDP-01-004	146.6	81	132	81	132
BR-PDP-01-005	117.8	74	86	74	86
BR-PDP-01-006	97.9	79	96	79	96
BR-PDP-01-007	73.4	71	98	71	98
BR-PDP-01-008	12.0	68	33	68	33
BR-PDP-01-009	273.2	65	155	65	155
BR-PDP-01-010	179.9	71	171	71	171
BR-PDP-01-011	138.5	77	120	77	120
BR-PDP-01-012	162.9	65	131	65	131
BR-PDP-01-013	55.6	81	53	81	53
BR-PDP-01-014	122.7	66	72	66	72
Port Royal Plantation - Broad Creek - Major Basin 1 BR-PRP-01					
BR-PRP-01-001	34.7	85	46	87	43
BR-PRP-01-002	89.6	79	61	80	59
BR-PRP-01-003	24.6	68	53	68	53
BR-PRP-01-004	68.5	71	90	71	90
BR-PRP-01-005	198.1	66	129	66	129
BR-PRP-01-006	24.0	69	49	69	49
BR-PRP-01-007	106.0	60	113	60	113

TABLE 15-3S
HYDROLOGIC SUBBASIN CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (SOUTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
BR-PRP-01-008	104.3	77	112	77	112
BR-PRP-01-009	9.4	51	138	51	138
BR-PRP-01-010	162.2	64	110	64	110
BR-PRP-01-011	88.0	61	117	61	117
BR-PRP-01-012	19.9	51	91	51	91
BR-PRP-01-013	17.0	87	30	87	30
BR-PRP-01-014	20.8	83	45	83	45

TABLE 15-3S
HYDROLOGIC SUBBASIN CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (SOUTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Wexford Plantation - Broad Creek - Major Basin 1 BR-WEX-01					
BR-WEX-01-001	73.7	79	60	79	60
BR-WEX-01-002	32.5	73	44	73	44
BR-WEX-01-003	129.1	79	76	79	76
BR-WEX-01-004	100.1	75	69	78	63
BR-WEX-01-005	184.2	77	103	79	97
BR-WEX-01-006	36.2	84	49	84	49
BR-WEX-01-006A	63.9	90	40	90	40
BR-WEX-01-007	74.1	76	81	76	81
BR-WEX-01-007A	114.6	73	91	73	91
BR-WEX-01-007B	112.9	70	142	70	142
BR-WEX-01-008	142.1	70	94	71	92
BR-WEX-01-009	119.8	67	94	68	92
BR-WEX-01-009A	14.8	59	54	59	54
BR-WEX-01-010	89.9	74	96	75	93
BR-WEX-01-011	102.5	64	104	64	104
Wexford Plantation - Broad Creek - Major Basin 2 BR-WEX-02					
BR-WEX-02-001	44.6	82	38	82	38
BR-WEX-02-002	14.0	62	70	62	70
BR-WEX-02-003	49.4	74	61	74	61
BR-WEX-02-004	26.8	83	48	83	48
Crossings - Broad Creek - Major Basin 1 BR-XNG-01					
BR-XNG-01-001	44.3	95	27	95	27
BR-XNG-01-002	87.3	80	72	84	63
BR-XNG-01-003	29.5	81	53	87	43
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01					
CA-SPP-01-001	83.8	78	78	79	76
Sea Pines - Calibogue Sound - Major Basin 2 CA-SPP-02-001					
CA-SPP-02-001	40.9	77	45	77	45
Port Royal Plantation - Fish Haul Creek - Major Basin 1 FH-PRP-01					
FH-PRP-01-001	137.3	80	67	80	67
FH-PRP-01-002	168.8	74	107	78	95
FH-PRP-01-003	21.2	83	32	85	30
FH-PRP-01-004	55.8	79	42	80	41

TABLE 15-3S
HYDROLOGIC SUBBASIN CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (SOUTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
FH-PRP-01-005	196.9	74	100	75	97
FH-PRP-01-006	107.0	88	56	89	54

TABLE 15-3S
HYDROLOGIC SUBBASIN CHARACTERISTICS
HILTON HEAD ISLAND WATERSHEDS (SOUTH)

ICPR Subbasin Name	Tributary Area (acres)	Existing Land Use		Future Land Use	
		Curve Number	Time of Concentration (minutes)	Curve Number	Time of Concentration (minutes)
Sea Pines - Lawton Canal - Major Basin 1 LC-SPP-01					
LC-SPP-01-001	255.4	75	185	75	185
LC-SPP-01-002	98.4	73	143	73	143
LC-SPP-01-003	113.9	68	157	68	157
LC-SPP-01-004	52.3	74	87	74	87
LC-SPP-01-005	48.6	72	83	72	83
LC-SPP-01-006	278.5	79	134	79	134
LC-SPP-01-007	35.6	81	68	83	64
LC-SPP-01-008	226.1	96	113	97	107
LC-SPP-01-009	494.1	77	139	79	131
LC-SPP-01-010	90.3	81	67	82	65
LC-SPP-01-011	84.5	75	89	75	89
Sea Pines - Point Comfort Creek - Major Basin 1 PC-SPP-01					
PC-SPP-01-001	50.6	74	65	74	65
PC-SPP-01-002	115.7	85	63	86	61
PC-SPP-01-003	28.2	76	48	76	48
PC-SPP-01-004	51.2	92	40	92	40
PC-SPP-01-005	51.2	89	41	89	41
Sea Pines - Point Comfort Creek - Major Basin 2 PC-SPP-02					
PC-SPP-02-001	3.3	72	22	72	22
PC-SPP-02-002	52.5	82	55	82	55
PC-SPP-02-003	28.8	85	46	85	46
PC-SPP-02-004	43.5	91	42	91	42

TABLE 15-4
SUMMARY HYDRAULIC BASIN DATA
HILTON HEAD ISLAND WATERSHEDS

Basin Name	Open Channels		Culvert Crossings			Other Features			
	Number	Length (feet)	Number	Number of Culverts	Number of Bridges	Storage Nodes	Pump Stations	Weirs	Drop Structures
BA-SPP-01	25	13,976	32	32	0	20	0	28	1
BA-SPP-02	7	2,670	9	9	0	5	0	10	1
BA-SPP-03	6	2,120	7	7	0	6	0	7	1
BC-SPP-01	0	0	1	1	0	2	0	2	2
BR-CHP-01	9	2,675	9	13	0	2	0	8	0
BR-IRP-01	0	0	16	24	0	14	0	14	2
BR-IRP-02	3	500	5	5	0	7	0	3	0
BR-LCC-01	0	0	11	11	0	15	0	17	7
BR-LCC-02	0	0	1	1	0	1	0	1	0
BR-PCT-01	0	0	7	9	0	4	0	7	0
BR-PCT-02	0	0	6	6	0	2	0	7	0
BR-PDP-01	73	35,320	18	9	15	17	0	0	0
BR-PRP-01	30	12,165	14	24	0	12	0	14	0
BR-WEX-01	54	17,880	16	21	3	19	2	18	2
BR-WEX-02	9	3,847	2	2	0	4	0	3	0
BR-XNG-01	14	5,500	2	6	0	5	0	3	1
CA-SPP-01	0	0	2	2	0	2	0	2	0
CA-SPP-02	0	0	1	1	0	1	0	2	0
FH-AIR-01	1	1,000	3	4	0	4	0	5	0
FH-PRP-01	14	7,700	5	6	0	4	0	5	0
JV-GUM-01	0	0	2	3	0	2	0	2	0
JV-HHP-01	2	1,550	16	20	0	15	1	16	6
JV-IRP-01	1	450	3	5	0	3	0	3	1
LC-SPP-01	45	21,661	15	17	0	22	1	17	5
OH-SPW-01	1	500	6	6	0	7	0	6	1
PA-HHP-01	0	0	6	10	1	8	0	9	2
PC-SPP-01	17	5,590	8	11	0	4	0	6	1
PC-SPP-02	3	1,005	2	2	0	3	0	2	0
PR-HHP-01	5	2,100	6	6	0	8	0	9	4
PR-HHP-02	0	0	5	5	0	4	0	3	1
PR-PHP-01	3	1,950	16	20	0	15	0	13	7
SK-GUM-01	10	3,042	18	18	0	1	0	12	0
SK-HHP-01	0	0	6	6	0	4	0	4	1
SK-HHP-02	0	0	3	3	0	4	0	5	1
TOTAL	332	143,201	279	325	19	246	4	263	47

TABLE 15-5N
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

		Culvert			Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Culvert Invert (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Chaplan Area - Broad Creek Outfall - Major Basin 1 BR-CHP-01								
Broad Creek Outfall	BRCHP01-P1	2 - 36 x 36	185	0.13	--	--	7	25
Driveway	BRCHP01-P10	36 x 36	48	5.08	--	--	9	25
Driveway	BRCHP01-P10A	36 x 36	49	5.74	--	--	9	25
Mingo Green Road	BRCHP01-P2	2 -36 x 36	35	1.88	--	--	7	25
Driveway	BRCHP01-P3	2 -36 x 36	192	2.24	--	--	6	25
Marshland Drive	BRCHP01-P4	2 -36 x 36	70	2.41	--	--	7	25
Driveway	BRCHP01-P5	36 x 36	60	4.08	--	--	8	25
Driveway	BRCHP01-P6	48 x 48	45	4.28	--	--	8	25
Indigo Run - Broad Creek Outfall - Major Basin 1 BR-IRP-01								
Broad Pointe Road	BRIRP01-P1	3 - 48 x 48	42	2.01	--	--	10	25
Gardner Drive	BRIRP01-P10	48 x 48	63	1.94	--	--	12	25
Gardner Drive	BRIRP01-P10A	30 x 30	76	2.47	--	--	12	25
US 278	BRIRP01-P11	24 x 24	172	8.64	--	--	13	100
US 278	BRIRP01-P11A	24 x 24	156	7.45	--	--	13	100
Northridge Preserve Causeway	BRIRP01-P12	36 x36	28	6.83	--	--	12	25
Owners Club #2	BRIRP01-P2	2 - 60 x 60	42	1.28	--	--	10.5	25
Owners Club #1	BRIRP01-D1	2 - 60 x 60	41	3.74	7.05	13	12	25
Aberdeen Ct	BRIRP01-P3	2 - 60 x 60	204	0.86	--	--	14	25
Marshland Road	BRIRP01-D2	2 - 54 x 54	356	4.17	9.13	30	14	25
Sunningdale Road	BRIRP01-P4A	2 - 36 x 36	105	2.78	--	--	13	25
Sunningdale Road	BRIRP01-P4B	60 x 60	105	2.17	--	--	13	25
Wentworth Place	BRIRP01-P5	60 x 60	366	2.55	--	--	12	25
Doral Lane	BRIRP01-P6	60 x 60	174	2.75	--	--	13	25
Mead Lane	BRIRP01-P7	54 x 54	118	2.64	--	--	12	25
Leg O' Mutton Drive	BRIRP01-P8	3 - 24 x 24	113	6.04	--	--	13	25
Leg O' Mutton Drive	BRIRP01-P8A	30 x 30	112	5.64	--	--	13	25
Crossing "The Preserve"	BRIRP01-P9	2 - 48 x 48	44	6.93	--	--	13	25
Indigo Run - Broad Creek Outfall - Major Basin 1 BR-IRP-02								
Marshland Road	BRIRP02-P1	60 x 60	100	-0.91	--	--	10	25
Colonial Drive	BRIRP02-P2	54 x 54	322	0.71	--	--	10	25
Golf Hole No. 2	BRIRP02-P3	36 x 36	675	1.58	--	--	**NA**	**NA**
Golf Hole No. 1	BRIRP02-P4	36 x 36	1520	1.65	--	--	**NA**	**NA**
Colonial Drive	BRIRP02-P5	36 x 36	2235	0.95	--	--	**NA**	**NA**
Gum Tree - Jarvis Creek Outfall - Major Basin 1 JV-GUM-01								
US 278	JVGUM01-P1	72 x 72	104	0.08	--	--	11.00	100
Marshside Drive	JVGUM01-P2	2 - 60 x 38	50	0.08	--	--	7	25

TABLE 15-5N
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

Road Crossing	ICPR Model Link ID	Culvert: Dimensions (in x in)	Culvert: Culvert Length (ft)	Culvert: Culvert Invert (ft NAVD)	Weir: Invert (ft NAVD)	Weir: Length (ft)	Roadway: Lowest Adjacent Elevation (ft NAVD)	Roadway: Level of Service
Hilton Head Plantation - Jarvis Creek Outfall - Major Basin 1 JV-HHP-01								
Jarvis Pump Station Gravity Outfall	JVHHP01-P1	2 - 72 x 60	80	-0.92	--	--	**NA**	**NA**
US 278	JVHHP01-P2	96 x 144	200	-2.92	--	--	13	100
US 278	JVHHP01-P2A	60 x 60	200	1.08	--	--	13	100
Main Street	JVHHP01-D1	3 - 60 x 60	66	0.36	5.48	4.42	9	25
Parkwood Drive	JVHHP01-P3	2 - 48 x 48	85	-0.34	--	--	12	25
Parkwood Drive	JVHHP01-P3A	72 x 72	85	-2	--	--	12	25
Whooping Crane Way/ Big Woods Way	JVHHP01-D4A	60 x 60	715	-0.4	--	--	12	25
Knollwood Drive	JVHHP01-P4	2 - 48 x 48	85	-0.34	--	--	12	25
Knollwood Drive	JVHHP01-P4A	48 x 48	85	-2	--	--	12	25
Headlands Drive	JVHHP01-P5	2 - 42 x 42	352	0.91	--	--	11	25
Headlands Drive	JVHHP01-P5A	72 x 72	352	-2			11	25
Crooked Pond Drive	JVHHP01-P6	36 x 36	151	3.95	--	--	12	25
Whooping Crane	JVHHP01-D4	42 x 42	110	4.17	8.93	6	12	25
Fallen Arrow Drive	JVHHP01-P7	24 x 24	388	3.2	--	--	10	25
Fallen Arrow Drive	JVHHP01-P7A	48 x 48	427	-0.19			10	25
Whooping Crane Way/ Big Woods Way	JVHHP01-P9	30 x 30	1867	10.58	--	--	14	25
Whooping Crane	JVHHP01-P10	30 x 30	232	10.58	--	--	15	25
Open Space	JVHHP01-P11	24 x 24	625	10.68	--	--	**NA**	**NA**
Golf Course	JVHHP01-D5	18 x 18	515	10.7	13.07	3.5	**NA**	**NA**
Summer Breeze Court	JVHHP01-D6	18 x 18	415	5.08	14.08	3.50	16	25
Open Space	JVHHP01-P12	24 x 24	1200	10.7	--	--	**NA**	**NA**
Open Space	JVHHP01-D100	18 x 18	333	0	--	--	**NA**	**NA**
Indigo Run - Jarvis Creek Outfall - Major Basin 1 JV-IRP-01								
Cross Island Parkway Outfall	JVIRP01-P1	3 - 24 x 24	100	1.08	--	--	15	100
Unknown	JVIRP01-P2	42 x 42	100	0.08	--	--	**NA**	**NA**
Linden Place	JVIRP01-P3	36 x 36	853	-1.42	--	--	9	25
Control Structure	JVIRP01-D1	54 x 54	250	-0.92	5.20	8.00	**NA**	**NA**
Spanish Wells Plantation - Old House Creek Outfall - Major Basin 1 OH-SPW-01								
Spanish Wells Road	OHSPW01-P1	24 x 24	70	5.46	--	--	11	25
Spanish Wells Road	OHSPW01-P1A	36 x 36	70	5.18	--	--	11	25
Golf Course - Hole 1	OHSPW01-D1	18 x 18	70	8.71	10.37	24	**NA**	**NA**
Golf Course - Hole 1	OHSPW01-P2	18 x 18	40	9.93	--	--	**NA**	**NA**
Golf Course - Hole 1	OHSPW01-P3	12 x 12	30	8.92	--	--	**NA**	**NA**
Golf Course - Hole 1	OHSPW01-P4	18 x 18	70	10.82	--	--	**NA**	**NA**
McIntosh Road	OHSPW01-P5	18 x 18	70	10.03	--	--	15	25
Hilton Head Plantation - Park Creek Outfall - Major Basin 1 PA-HHP-01								
Dolphin Head Drive	PAHHP01-P1	3 - 54 x 54	90	-0.93	--	--	8	25
Seabrook Drive	PAHHP01-P2	3 - 54 x 54	90	-0.85	--	--	10	25
Seabrook Drive / Golden Hind Drive	PAHHP01-P3	2 - 54 x 54	850	3.6	--	--	10	25
Golf Course	PAHHP01-D1	2 - 42 x 42	487	1.42	10.83	5.00	**NA**	**NA**
Golf Course	PAHHP01-P4	Cart Bridge	30	0	--	--	**NA**	**NA**
Golf Course	PAHHP01-P5	36 x 36	949	7.8	--	--	**NA**	**NA**
Golf Course	PAHHP01-P6	48 x 48	400	0	--	--	**NA**	**NA**
Seabrook Drive	PAHHP01-D2	30 x 30	112	-0.75	5.65	3.75	**NA**	**NA**

TABLE 15-5N
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

		Culvert			Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Culvert Invert (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Hilton Head Plantation - Port Royal Sound Outfall - Major Basin 1 PR-HHP-01								
Hickory Forest Drive	PRHHP01-P1	2 - 42 x 42	100	0.52	--	--	11	25
Hickory Forest Drive	PRHHP01-P1A	60 x 60	100	0			11	25
Oyster Reef Drive	PRHHP01-P2	42 x 42	730	3.58	--	--	11	25
Oyster Reef Drive	PRHHP01-P2A	48 x 48	850	0			11	25
Lagoon Outfall	PRHHP01-P3	15 x 15	39	4.07	--	--	11.00	25
Oyster Reef Drive	PRHHP01-D4	36 x 36	380	0.3	8.63	7.5	12	25
Golf Course	PRHHP01-P4	24 x 24	444	5.16	--	--	**NA**	**NA**
Adjacent to High Bluff Road	PRHHP01-D1	42 x 42	171	3.24	7.09	5	**NA**	**NA**
Open Space	PRHHP01-D2	42 x 42	459	3.05	8.68	5	**NA**	**NA**
Wetland / Oyster Reef Drive	PRHHP01-D3	2 -30 x 30	280	5.08	12.41	7.5	12	25
Hilton Head Plantation - Port Royal Sound Outfall - Major Basin 2 PR-HHP-02								
High Bluff Road	PRHHP02-P1	48 x 48	60	4.68	--	--	14	25
Golf Course	PRHHP02-P2	30 x 30	260	-0.92	--	--	**NA**	**NA**
China Cockle Way	PRHHP02-P3	42 x 42	1180	4.5	--	--	14	25
China Cockle Way	PRHHP02-P4	36 x 36	116	7.31	--	--	14	25
China Cockle Way	PRHHP02-P5	18 x 18	270	7.94	--	--	15	25
Outfall	PRHHP02-D1	2- 48 x 48	300	0	10.42	120	**NA**	**NA**
Palmetto Hall - Port Royal Sound - Major Basin 1 PR-PHP-01								
Mitchellville Road	PRPHP01-P1	36 x 36	36	2.52	--	--	7	25
Mitchellville Road	PRPHP01-P1A	36 x 36	36	2.48	--	--	7	25
Mitchellville Road	PRPHP01-P1B	36 x 36	36	2.36	--	--	7	25
Mitchellville Road	PRPHP01-P1C	24 x 24	36	2.36	--	--	7	25
Mitchellville Road	PRPHP01-P1D	24 x 24	36	2.13	--	--	7	25
Fish Haul Road	PRPHP01-D1	24 x 24	225	5.5	9	9	7	25
Fish Haul Road	PRPHP01-P2	42 x 42	44	5.58			10	25
Fish Haul Road	PRPHP01-P2-1	42 x 42	44	4.99			10	25
Adjacent to Fish Haul Road	PRPHP01-P2A	30 x 30	20	4.87			10	25
Adjacent to Fish Haul Road	PRPHP01-P2B	30 x 30	5	4.87			10	25
Adjacent to Fish Haul Road	PRPHP01-P2C	30 x 30	55	4.87			10	25
Adjacent to Fish Haul Road	PRPHP01-D1A	24 x 24	220	5.12			**NA**	**NA**
Adjacent to Fish Haul Road	PRPHP01-D1D	30 x 30	85	5.19			**NA**	**NA**
Open Space	PRPHP01-D2	24 x 24	945	3.08	9.09	9	**NA**	**NA**
Fort Howell Drive	PRPHP01-P3	24 x 24	110	4.08	--	--	12	
Adjacent to Port Howell Drive (Golf Course)	PRPHP01-P4	24 x 24	815	4.08			**NA**	**NA**
Adjacent to Port Howell Drive (Golf Course)	PRPHP01-P5	24 x 24	1025	4.06			**NA**	**NA**
Fort Howell Drive	PRPHP01-D3	24 x 24	730	5.08	11.08	9	10.6	25
Driveway	PRPHP01-P6	24 x 24	170	6.08	--	--	14	25
Golf Course	PRPHP01-D4	24 x 24	535	6.08	13.08	9	**NA**	**NA**
Golf Course	PRPHP01-P10	24 x 24	250	2.08	--	--	**NA**	**NA**
Access Road	PRPHP01-P11	24 x 24	1020	2.08	--	--	15	?
Fort Howell Drive	PRPHP01-P12	24 x 24	440	2.08	--	--	13	25
Sedge Fern Drive	PRPHP01-P13	24 x 24	575	1.83	--	--	16	25
Open Space	PRPHP01-D10	24 x 24	945	2.08	11.08	9	**NA**	**NA**
Adjacent to Clyde Lane	PRPHP01-P14	24 x 24	1200	5.08	--	--	15	25
Clyde Lane	PRPHP01-P15	24 x 24	910	5.08	--	--	15	25

TABLE 15-5N
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

		Culvert			Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Culvert Invert (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Airport - Fish Haul Creek Outfall - Major Basin 1 FH-AIR-01								
Dillon Road	FHAIR01-P1	60 x 36	70	-3.31	--	--	7	25
Dillon Road	FHAIR01-P1A	60 x 36	70	-2.96	--	--	7	25
Runway	FHAIR01-P2	48 x 48	334	2.73	--	--	12	25
Runway	FHAIR01-P3	54 x 54	392	2.73	--	--	12	25
Gum Tree - Skull Creek Outfall - Major Basin 1 SK-GUM-01								
Wild Horse Road	SKGUM01-P1	144 x 36	176	1.3	--	--	7	25
Maintenance Causeway	SKGUM01-P2	144 x 36	36	1.46	--	--	**NA**	**NA**
Maintenance Causeway	SKGUM01-P3	120 x 48	36	1.04	--	--	**NA**	**NA**
Maintenance Causeway	SKGUM01-P4	144 x 36	12	1.37	--	--	**NA**	**NA**
Gum Tree Road	SKGUM01-P5	4- 42 x 42	65	1.38	--	--	9	25
Chinaberry Lane	SKGUM01-P6	4 - 36 x 36	48	2.35	--	--	11	25
Kings Court	SKGUM01-P7	3 - 36 x 36	384	4.66	--	--	10	25
Squiresgate Road	SKGUM01-P8	3 - 36 x 36	78	6.08	--	--	12	25
Hilton Head Plantation - Skull Creek Outfall - Major Basin 1 SK-HHP-01								
Seabrook Drive	SKHHP01-P1	54 x 54	1010	3.64	--	--	16	25
Birdsong Lane	SKHHP01-P2	48 x 48	240	3.83	--	--	11	25
Open Space	SKHHP01-P2A	42 x 42	210	5.08	--	--	**NA**	**NA**
Birdsong Lane	SKHHP01-D1	42 x 42	366	0.12	7.08	5.25	11	25
Open Space	SKHHP01-P4	36 x 36	220	-0.92	--	--	**NA**	**NA**
Birdsong / Meadowlark Lane	SKHHP01-P5	30 x 30	763	5.08	--	--	12	25
Connector Pipe	SKHHP01-P100	36 x 36	2000	0	--	--	12	25
Hilton Head Plantation - Skull Creek Outfall - Major Basin 2 SK-HHP-02								
Old Fort Way	SKHHP02-D1	48 x 48	710	-0.04	7.08	6.00	14	25
Santa Maria Drive	SKHHP02-P1	48 x 48	1020	3.08	--	--	10	25
Country Club	SKHHP02-P2	42 x 42	230	4.98	--	--	**NA**	**NA**
Country Club Court	SKHHP02-P3	42 x 42	565	4.77	--	--	14	25

TABLE 15-5S
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

	Culvert				Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Port Royal Plantation - Broad Creek - Major Basin 1 BR-PRP-01								
Mathews Drive	BRPRP01-P1	72 x 72	55	-2.72	--	--	6.0	25
Mathews Drive	BRPRP01-P2	60 x 60	55	-2.72	--	--	6.0	25
US 278	BRPRP01-P3	72 x 72	210	-0.67	--	--	11.1	100
US 278	BRPRP01-P4	60 x 60	210	-0.67	--	--	11.1	100
Barony Lane	BRPRP01-P5	2 - 72 x 72	80	-1.07	--	--	11.0	25
Golf Course	BRPRP01-P5A	2- 72 x 72	100	-1.07	--	--	9.5	25
Grasslawn Avenue	BRPRP01-P6	48 x 48	120	1.61	--	--	9.0	25
Grasslawn Avenue	BRPRP01-P7	2 - 83 x 57	120	-0.42	--	--	9.0	25
Scarborough Head Road	BRPRP01-P8	2 - 48 x 48	40	1.55	--	--	9.0	25
Golf Course	BRPRP01-P9	48 x 48	30	0.42	--	--	**NA**	**NA**
Golf Course	BRPRP01-P10	48 x 48	30	0.08	--	--	**NA**	**NA**
Fairway Winds Place	BRPRP01-P11	88 x 54	155	-1.2	--	--	6.0	25
Coggins Point Place	BRPRP01-P12	48 x 48	900	-0.59	--	--	6.5	25
Doubloon Drive	BRPRP01-P13	30 x 30	60	5.08	--	--	12.0	25
Century drive	BRPRP01-P40	2 - 48 x 48	50	1.03	--	--	8.0	25
Golf Course	BRPRP01-P46	48 x 48	30	0.79	--	--	**NA**	**NA**
Audobon Place	BRPRP01-P47	30 x 30	330	0.79	--	--	8.0	25
South Port Royal Drive	BRPRP01-P61	36 x 36	50	2.15	--	--	9.0	25
South Port Royal Drive	BRPRP01-P66	36 x 36	50	2.88	--	--	9.0	25
Port Royal Plantation - Fish Haul Creek - Major Basin 1 FH-PRP-01								
Oak Creek Drive	FHPRP01-P1	48 x 48	75	-2.02	--	--	8.0	25
Market Place Drive	FHPRP01-P2	60 x 60	75	0.08	--	--	6.0	25
Union Cemetery Road	FHPRP01-P3	48 x 48	40	0.18	--	--	8.0	25
Golf Course	FHPRP01-P3A	24 x 24	330	2.58	--	--	**NA**	**NA**
US 278	FHPRP01-P4	36 x 36	186	2.6	--	--	11.0	100
US 278	FHPRP01-P5	36 x 36	160	2.6	--	--	11.0	100
Palmetto Dune - Broad Creek - Major Basin 1 BR-PDP-01								
US 278 / Shelter Cove	BRPDP01-P1A	5 - 72 x 72	100	-7.1	--	--	13.0	100
US 278 / Shelter Cove	BRPDP01-P1B	2 - 72 x 72	200	-8.13	--	--	13.0	100
Low Water Drive	BRPDP01-P2	Bridge	30	-5.1	--	--	12.0	25
Port Tack	BRPDP01-P3	Bridge	40	-4.56	--	--	9.0	25
Port Tack	BRPDP01-P4	Bridge	40	-4.32	--	--	9.6	25
Sea Lane	BRPDP01-P5	Bridge	30	-4.02	--	--	10.0	25
Queens Folly Road	BRPDP01-P6	Bridge	100	-4.86	--	--	11.5	25
Driveway	BRPDP01-P7	Bridge	40	-4.6	--	--	8.0	25
Queens Way	BRPDP01-P8	Bridge	40	-4.49	--	--	9.7	25
Leamington Lane	BRPDP01-P9	Bridge	50	-4.24	--	--	9.5	25
Haul Away	BRPDP01-P10	Bridge	60	-4.44	--	--	10.2	25
Mooring Buoy	BRPDP01-P11	Bridge	50	-4.32	--	--	9.0	25
Ocean Lane	BRPDP01-P11A	Bridge	50	-4.54	--	--	9.0	25
Carnoustie Road	BRPDP01-P12	Bridge	30	-4.55	--	--	9.3	25
Queens Way	BRPDP01-P13	Bridge	60	-4.17	--	--	8.0	25
Starboard Tack	BRPDP01-P14	Bridge	30	-4.59	--	--	9.8	25
Causeway	BRPDP01-P15	2 - 60 x 60	100	-7.88	--	--	**NA**	**NA**
Mooring Buoy	BRPDP01-P100	36 x 36	60	-7.71	--	--	4.2	25

TABLE 15-5S
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

	Culvert				Weir		Roadway	
	ICPR Model	Dimensions	Culvert Length	Invert Elevation	Invert	Length	Lowest Adjacent Elevation	Level of
Road Crossing	Link ID	(in x in)	(ft)	(ft NAVD)	(ft NAVD)	(ft)	(ft NAVD)	Service
Wexford Plantation - Broad Creek - Major Basin 1 BR-WEX-01								
Outfall Causeway	BRWEX01-P1	72 x 60	60	-2.92	--	--	**NA**	**NA**
Harrogate Drive	BRWEX01-P2	Bridge	30	-5.66	--	--	8.5	25
Wexford Drive	BRWEX01-P3	Bridge	40	-4.84	--	--	11.9	25
Dunnagan's Alley	BRWEX01-P3A	144 x 60	40	-2	--	--	8.0	25
US 278	BRWEX01-P4	3 - 60 x 60	150	0.88	--	--	9.0	100
Bridgeport Lane	BRWEX01-P5	Bridge	32	-2.26	--	--	9.0	25
Cart Bridge	BRWEX01-P7A	Bridge	10	5.6				
Cordillo Parkway	BRWEX01-P7	2 - 60 x 60	90	-3.07	--	--	8.0	25
Shipyard Drive	BRWEX01-P60	2 - 60 x 60	80	-4.38	--	--	7.0	25
Open Space	BRWEX01-P61	36 x 36	40	0.54	--	--	**NA**	**NA**
Kingston Road	BRWEX01-P62	48 x 48	80	-2.04	--	--	5.0	25
Open Space	BRWEX01-P63	36 x 36	80	1.04	--	--	**NA**	**NA**
Colonnade Road	BRWEX01-D60	24 x 24	70	1.76	4.08	3	9.0	25
Open Space	BRWEX01-P64	36 x 36	50	-2.38	--	--	**NA**	**NA**
Gloucester Road	BRWEX01-P49	1 - 24 x 24	260	1.97			6.0	25
Gloucester Road	BRWEX01-P50A	48 x 48	260	0			6.0	25
Gloucester Road	BRWEX01-D50	36 x 36	100	0.78	3.98	9	6.0	25
Open Space	BRWEX01-P70	60 x 60	115	-0.92	--	--	**NA**	**NA**
Wexford Plantation - Broad Creek - Major Basin 2 BR-WEX-02								
Wexford Club Drive	BRWEX02-P1	30 x 30	210	-3.02	--	--	7	25
Wexford Club Drive	BRWEX02-P2	30 x 30	200	-0.92	--	--	7	25
The Crossings - Broad Creek - Major Basin 1 (Haig Point) BR-XNG-01								
Haig Point Court	BRXNG01-P1	3 - 60 x 60	60	1.48			7	25
D/Sof Palmetto Bay Business Park	BRXNG01-D1	2 - 60 x 60	160	2.08			7	25
Palmetto Bay Business Park	BRXNG01-P2	3 - 60 x 60	160	-3.77			7	25
Point Comfort - Broad Creek - Major Basin 1 BR-PCT-01								
Open Space	BRPCT01-P1	24 x 24	150	2.68			**NA**	**NA**
Freshwater Lane	BRPCT01-P2	24 x 24	140	2.68			7	25
Open Space	BRPCT01-P3	2 - 24 x 24	70	2.68			**NA**	**NA**
Shoreline Drive	BRPCT01-P4	36 x 36	48	-0.92			7	25
Tide Pointe Way	BRPCT01-P5	36 x 36	48	-0.92			8	25
Spruce Court	BRPCT01-P6	2 - 24 x 24	68	2.08			6	25
Tide Pointe Way	BRPCT01-P7	2- 24 x 24	22	2.08			6.4	25
Point Comfort - Broad Creek - Major Basin 2 BR-PCT-02								
Open Space	BRPCT02-P1	42 x 42	150	0.08			**NA**	**NA**
Ashton Cove Drive	BRPCT02-P2	42 x 42	47	0.08			6	25
Open Space	BRPCT02-P3	42 x 42	165	0.2			**NA**	**NA**
Open Space	BRPCT02-P4	30 x 30	135	0.78			**NA**	**NA**
Ashton Cove Drive	BRPCT02-P5	30 x 30	45	0.78			6	25
Open Space	BRPCT02-P6	30 x 30	160	0.51			**NA**	**NA**
Sea Pines - Point Comfort - Major Basin 1 PC-SPP-01								
Golf Course Club House Drive	PCSPP01-P1	2 - 42 x 42	88	-1.42			10	25
Golf Course Club House Drive	PCSPP01-P1A	48 x 48	88	-1.42			10	25
Club Course Drive	PCSPP01-P2	2 - 48 x 48	72	-0.78			6	25
Open Space	PCSPP01-P3	24 x 24	30	-1.3			**NA**	**NA**
Isle of Pines Road	PCSPP01-P4	24 x 24	30	2.58			7	25
Otter Road	PCSPP01-D1	36 x 36	63	0.22	3.88	4.5	7	25
Market Place	PCSPP01-P5	54 x 54	230	0.62			**NA**	**NA**
Greenwood Drive	PCSPP01-P6	48 x 48	160	1.09			10	25

TABLE 15-5S
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

	Culvert				Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Sea Pines - Point Comfort - Major Basin 2 PC-SPP-02								
Club Course Drive	PCSPP02-P1	42 x 42	80	-1.24			7	25
Club Course Drive / Open Space	PCSPP02-P2	30 x 30	700	1.08			7	25
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01								
Stoney Creek Road	CASP01-P1	42 x 42	65	-0.82			6.0	25
Open Space	CASP01-P2	42 x 42	65	-0.82			**NA**	**NA**
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01								
Stoney Creek Road	CASP02-P1	24 x 24	60	1.78			6.0	25
Sea Pines - Baynard Cove - Major Basin 1 BA-SPP-01								
Baynard Cove Road	BASP01-D1	48 x 48 2 - 66 x 66	264	-2.52	1.88	6	8	25
Wagon Road	BASP01-P1	48 x 48 66 x 66	100	-2.62			7.0	25
Greenwood Drive	BASP01-P2	48 x 48	85	-2.62			6.5	25
Greenwood Drive	BASP01-P3	60 x 60	95	-2.62			6.5	25
Open Space	BASP01-P4	48 x 48 60 x 60	100	-2.92			**NA**	25
Woodbine Place	BASP01-P5	58 x 91	55	-3.92			4	25
Woodbine Place	BASP01-P6	60 x 60	40	-2.92			4	25
Lighthouse Road	BASP01-P7	48 x 48	50	-2.55			4	25
Lighthouse Road	BASP01-P8	48 x 48	170	-2.45			4	25
Lighthouse Road	BASP01-P8A	48 x 48	60	-3.46			4	25
Open Space	BASP01-P9	42 x 42	80	-3.07			**NA**	**NA**
Lighthouse Road	BASP01-P10	42 x 42	380	-2.55			**4.0**	25
Open Space	BASP01-P11	36 x 36	50	-1.42			**NA**	**NA**
North/South Live Oak Road	BASP01-P12	42 x 42	60	-1.65			4.0	25
North/South Live Oak Road	BASP01-P13	42 x 42	30	-1.65			4.0	25
Old Military Road	BASP01-P14	48 x 48	65	-1.52			10.0	25
Forest Drive	BASP01-P15	24 x 24	110	1.08			6.0	25
North Sea Pines Drive	BASP01-P16	42 x 42	100	-3.42			5.0	25
Open Space	BASP01-P17	48 x 48	30	-3.78			**NA**	**NA**
Open Space	BASP01-P18	48 x 48	30	-3.71			**NA**	**NA**
North Sea Pines Drive	BASP01-P19	42 x 42	72	-3.6			5.0	25
Open Space	BASP01-P20	42 x 42	100	-3.57			**NA**	**NA**
Open Space	BASP01-P21	30 x 30	40	-3.94			**NA**	**NA**
Open Space	BASP01-P22	36 x 36	40	-3.57			**NA**	**NA**
Open Space	BASP01-P23	42 x 42	80	-3.12			**NA**	**NA**
South Live Oak Road	BASP01-P24	36 x 36	40	-1.92			6.0	25
Beach Lagoon Road	BASP01-P25	42 x 42	150	-3.42			4.0	25
South Beach Lagoon Road	BASP01-P26	42 x 42	150	-3.17			5.0	25
Parallel South Beach Lagoon Road	BASP01-P27	30 x 30	100	-2.95			4.0	25
Parallel South Beach Lagoon Road	BASP01-P28	24 x 24	100	-2.91			5.0	25
North Sea Pines Drive	BASP01-P29	24 x 24	600	-2.9			6.0	25
Open Space	BASP01-P59A	36 x 36	40	-1.04			**NA**	**NA**
Open Space	BASP01-P59B	24 x 24	36	-1.04			**NA**	**NA**
Sea Pines - Baynard Cove - Major Basin 2 BA-SPP-02								
Baynard Park Road	BASP02-D1	48 x 48	62	-4.22	3.28	28	6	25
Turnberry Lane	BASP02-P1	48 x 48	62	-1.42			6	25
Open Space	BASP02-P1A	48 x 48	51	-1.12			**NA**	**NA**
Bayanrd Cove Road	BASP02-P2	48 x 48	62	-0.02			6	25
Open Space	BASP02-P3	30 x 30	270	-0.42			**NA**	**NA**
Heritage Road	BASP02-P4	30 x 30	80	0.68			5	25
Open Space	BASP02-P5	36 x 36	255	-0.52			**NA**	**NA**
Saint Andrews Place	BASP02-P6	24 x 24	41	0.58			6	25
Open Space	BASP02-P7	18 x 18	244	0.38			**NA**	**NA**
Muirfield Road	BASP02-P8	24 x 24	172	1.38			6	25

TABLE 15-5S
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

	Culvert				Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Sea Pines - Baynard Cove - Major Basin 3 BA-SPP-03								
Outfall	BASP03-D1	30 x 30	250	-0.77	3.28	12	**NA**	**NA**
Bayanrd Cove Road	BASP03-P1	48 x 48	51	-0.67			6	25
Heritage Road	BASP03-P2	42 x 42	150	-0.92			6	25
Open Space	BASP03-P3	30 x 30	100	-0.88			**NA**	**NA**
Open Space	BASP03-P4	30 x30	100	-0.84			**NA**	**NA**
Harleston Green Road	BASP03-P5	30 x 30	100	-0.81			8	25
Harleston Green Road	BASP03-P6	30 x 30	550	-0.92			7	25
Forest Drive	BASP03-P7	42 x 42	45	1.28			7	25
Sea Pines - Braddock Cove - Major Basin 1 BC-SPP-01								
South Sea Pines Drive	BCSP01-D1	30 x 30	150	0.84	2.08	5	6	25
Sprum Pond Road	BCSP01-P1	18 x 18	100	0.48			7	25
Sprum Pond Road	BCSP01-D2	18 x 18	250	0.48	1.32	1.5	7	25
Sea Pines - Lawton Canal - Major Basin 1 LC-SPP-01								
Greenwood Drive	LCSP01-P1	2 - 60 x 60	90	-3.97			7	25
Open Space	LCSP01-P2	2 - 60 x 60	60	-2.74			**NA**	**NA**
Open Space	LCSP01-P3	60 x 60	66	-2.52			**NA**	**NA**
Open Space	LCSP01-P31	30 x 30	65	-1.46			**NA**	**NA**
Open Space	LCSP01-P32	30 x 30	20	-1.42			**NA**	**NA**
Open Space	LCSP01-P4	72 x 72	70	-0.06			**NA**	**NA**
Open Space	LCSP01-P51	48 x 48	30	-3.92			**NA**	**NA**
Open Space	LCSP01-P52	18 x 18	28	-1.04			**NA**	**NA**
Open Space	LCSP01-P53	24 x 24	120	0.98			**NA**	**NA**
Open Space	LCSP01-P54	24 x 24	178	1.78			**NA**	**NA**
Open Space	LCSP01-P71	42 x 42	48	-3.92			**NA**	**NA**
Open Space	LCSP01-P72	42 x 42	32	-1.92			**NA**	**NA**
Open Space	LCSP01-P73	42 x 42	48	-0.89			**NA**	**NA**
Open Space	LCSP01-P73A	42 x 42	48	-0.92			**NA**	**NA**
Open Space	LCSP01-P85	24 x 24	90	-1.42			**NA**	**NA**
Open Space	LCSP01-D1	24 x 24	180	-0.46	2.08	3	**NA**	**NA**
School Driveway	LCSP01-D4	132 x 60	50	-2.92	0.00	8	5	25
Greenwood Drive	LCSP01-D50	48 x 48	58	-3.92	1.48	6	5	25
Open Space	LCSP01-D51	30 x 30	20	-3.72	1.48	4	**NA**	**NA**
Willow Oak Road	LCSP01-D70	36 x 36	48	-0.83	1.88	3.5	5	25
Long Cove Club - Broad Creek - Major Basin 1 BR-LCC-01								
Open Space	BRLCC01-P1	15 x 15	60	1.07			**NA**	**NA**
Strawberry Hill Road	BRLCC01-P10	24 x 24	70	-2.13			7.0	25
Turnbridge Drive	BRLCC01-P11	24 x 24	60	-2.1			6.0	25
Long Cove Drive	BRLCC01-P12	36 x 36	104	-4.42			5.0	25
Open Space	BRLCC01-P2	24 x 24	75	0.08			NA	NA
Retreat Lane	BRLCC01-P3	30 x 30	55	0.08			7.0	25
Turnbridge Drive	BRLCC01-P5	30 x 30	115	-1.73			7.0	25
Long Cove Drive	BRLCC01-P6	24 x 24	70	0.08			6.0	25
Turnbridge Drive	BRLCC01-P7	15 x 15	70	3.08			6.0	25
Open Space	BRLCC01-P8	30 x 30	230	-3.92			**NA**	**NA**
Combahee Road	BRLCC01-P9	30 x 30	90	-3.92			5.0	25
Outfall	BRLCC01-D1	2 - 30 x 30	50	-0.92	2.68	8.5	NA	NA
Long Cove Drive	BRLCC01-D2	24 x 24	60	-1.92	2.58	3.5	6.0	25
Long Cove Drive	BRLCC01-D3	15 x 15	100	-1.92	3.58	3	6.0	25
Long Cove Drive	BRLCC01-D3A	30 x 30	100	-1.92	3.58	9	8.0	25
Long Cove Drive	BRLCC01-D4	30 x 30	110	-3.92	2.38	5	7.0	25
Open Space	BRLCC01-D5	30 x 30	50	-1.92	1.48	5	**NA**	**NA**
Open Space	BRLCC01-D6	30 x 30	64	-2.42	1.48	5	**NA**	**NA**

TABLE 15-5S
CULVERT / STRUCTURE DATA FOR HYDROLOGIC BASINS, HILTON HEAD ISLAND (SOUTH)

	Culvert				Weir		Roadway	
Road Crossing	ICPR Model Link ID	Dimensions (in x in)	Culvert Length (ft)	Invert Elevation (ft NAVD)	Invert (ft NAVD)	Length (ft)	Lowest Adjacent Elevation (ft NAVD)	Level of Service
Long Cove Club - Broad Creek - Major Basin 2 BR-LCC-02								
Outfall Causeway	BRLCC02-P1	18 x 18	100	-0.92			**NA**	**NA**

TABLE 15-6N
PROBLEM AREAS IDENTIFIED BY ICPR MODEL
HILTON HEAD ISLAND WATERSHEDS (NORTH)

Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Chaplan Area - Broad Creek Outfall - Major Basin 1 BR-CHP-01					
No Problem Areas					
Indigo Run - Broad Creek Outfall - Major Basin 1 BR-IRP-01					
Wentworth Place	BRIRP01-5	11.5	25	12.34	12.49
Doral Lane	BRIRP01-6	11.0	25	12.64	12.7
Mead Lane	BRIRP01-7	11.1	25	12.64	12.7
Leg O Mutton Road	BRIRP01-9	12.0	25	12.86	12.95
Gardner Drive U/S	BRIRP01-10	11.0	25	12.86	12.95
U.S. 278	BRIRP01-11	13.0	100	13.66	13.78
Indigo Run - Broad Creek Outfall - Major Basin 2 BR-IRP-02					
No Problem Areas					
Gum Tree - Jarvis Creek Outfall - Major Basin 1 JV-GUM-01					
No Problem Areas					
Hilton Head Plantation - Jarvis Creek Outfall - Major Basin 1 JV-HHP-01					
Parkwood Drive	JVHHP01-14	9.5	25	9.65	9.73
Knollwood Drive	JVHHP01-15	9.2	25	9.85	9.94
Headlands/Fallen Arrow Drive	JVHHP01-16	10.0	25	10.21	10.24
Whooping Crane Way	JVHHP01-17	11.7	25	12.15	12.2
Summer Breeze Court	JVHHP01-23	17.0	25	17.97	17.97
Golf Course	JVHHP01-25	18.0	25	20.59	20.59
Indigo Run - Jarvis Creek Outfall - Major Basin 1 JV-IRP-01					
No Problem Areas					
Spanish Wells Plantation - Old House Creek Outfall - Major Basin 1 OH-SPW-01					
No Problem Areas					
Hilton Head Plantation - Park Creek Outfall - Major Basin 1 PA-HHP-01					
Dolphin Head Road	PAHHP01-1A	8.5	25	8.66	8.66
Seabrook / Golden Hind Drive	PAHHP01-3	11.0	25	12.99	12.99
Golf Course	PAHHP01-4	14.0	25	15.02	15.02
Golf Course	PAHHP01-5	14.0	25	15.02	15.02
Seabrook Drive	PAHHP01-8	6.6	25	8.65	8.65

TABLE 15-6N PROBLEM AREAS IDENTIFIED BY ICPR MODEL HILTON HEAD ISLAND WATERSHEDS (NORTH)					
Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Hilton Head Plantation - Port Royal Sound Outfall - Major Basin 1 PR-HHP-01					
Hickory Forest Drive	PRHHP01-2	10.0	25	9.99	10.15
Oyster Reef Drive	PRHHP01-8	11.0	25	12.04	11.53
High Bluff Road	PRHHP01-9	11.0	25	12.63	12.58
Open Space	PRHHP01-11	10.6	25	12.38	12.14
Golf Course / Oyster Reef Drive	PRHHP01-12	12.0	25	12.71	12.66
Hilton Head Plantation - Port Royal Sound Outfall - Major Basin 2 PR-HHP-02					
High Bluff Road	PRHHP02-2	13.0	25	14.14	14.14
Towhee Road/ Golf Course	PRHHP02-3	15.6	25	15.89	15.89
China Cockle Way	PRHHP02-7	13.5	25	14.72	14.72
Palmetto Hall - Port Royal Sound - Major Basin 1 PR-PHP-01					
Fish Haul Road	PRPHP01-5	10.0	25	13.16	13.17
Fort Howell Drive	PRPHP01-6	10.6	25	14.1	14.12
Fort Howell Drive	PRPHP01-7	11.5	25	14.13	14.15
Golf Course	PRPHP01-8	14.5	25	15.52	15.57
Fort Howell Drive	PRPHP01-9	11.6	25	15.57	15.61
Fort Howell Drive	PRPHP01-10	13.0	25	15.57	15.62
Fort Howell Drive	PRPHP01-12	12.6	25	15.57	15.62
Fort Howell Drive	PRPHP01-14	12.1	25	12.44	12.49
Fort Howell Drive	PRPHP01-15	11.7	25	14.79	14.79
Sedge Fern Drive	PRPHP01-15A	12.6	25	13.51	13.52
Fort Howell Drive	PRPHP01-16	13.0	25	18.85	18.89
Golf Course	PRPHP01-17	13.5	25	18.85	18.88
Golf Course	PRPHP01-18	13.5	25	18.85	18.88
Airport - Fish Haul Creek Outfall - Major Basin 1 FH-AIR-01					
No Problem Areas					
Gum Tree - Skull Creek Outfall - Major Basin 1 SK-GUM-01					
No Problem Areas					
Hilton Head Plantation - Skull Creek Outfall - Major Basin 1 SK-HHP-01					
Bird Song Way	SKHHP01-1A	12.7	25	13.22	13.23
Bird Song Way	SKHHP01-2	11.0	25	13.28	13.28
Hilton Head Plantation - Skull Creek Outfall - Major Basin 2 SK-HHP-02					
Santa Maria Drive	SKHHP02-2	12.0	25	12.13	12.13

TABLE 15-6S PROBLEM AREAS IDENTIFIED BY ICPR MODEL HILTON HEAD ISLAND WATERSHEDS (SOUTH)					
Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Port Royal Plantation - Fish Haul Creek - Major Basin 1 FH-PRP-01					
No Problem Areas					
Palmetto Dunes - Broad Creek - Major Basin 1 BR-PDP-01					
No Problem Areas					
Wexford Plantation - Broad Creek - Major Basin 1 BR-WEX-01					
No Problem Areas					
Wexford Plantation - Broad Creek - Major Basin 2 BR-WEX-02					
No Problem Areas					
The Crossings - Broad Creek - Major Basin 1 (Haig Point) BR-XNG-01					
No Problem Areas					
Point Comfort - Broad Creek - Major Basin 1 BR-PCT-01					
No Problem Areas					
Point Comfort - Broad Creek - Major Basin 2 BR-PCT-02					
No Problem Areas					
Sea Pines - Point Comfort - Major Basin 1 PC-SPP-01					
Club Course Drive	PCSPP01-7	7.0	25	7.88	7.90
Upstream Club Course Drive	PCSPP01-8	7.0	25	7.91	7.93
Upstream Club Course Drive	PCSPP01-9	6.2	25	7.97	7.99
Upstream Club Course Drive	PCSPP01-10	6.5	25	7.98	8.00
Upstream Club Course Drive	PCSPP01-11	6.5	25	7.99	8.00
Upstream Club Course Drive	PCSPP01-12	6.5	100	8.73	8.76
Otter Road	PCSPP01-19	6.2	25	7.41	7.43
Publix	PCSPP01-21	8.0	25	8.37	8.39
	PCSPP01-22	7.5	25	8.41	8.42
Greenwood Drive	PCSPP01-23	7.5	25	9.4	9.42
Sea Pines - Point Comfort - Major Basin 2 PC-SPP-02					
Club Course Drive	PCSPP02-5	7.0	25	8.97	8.97
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01					
No Problem Areas					

TABLE 15-6S PROBLEM AREAS IDENTIFIED BY ICPR MODEL HILTON HEAD ISLAND WATERSHEDS (SOUTH)					
Road Crossing	ICPR Model Node ID	Roadway Elevation (ft NAVD)	Level of Service	Existing Peak Water Elevation (ft NAVD)	Future Peak Water Elevation (ft NAVD)
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01					
No Problem Areas					
Sea Pines - Baynard Cove - Major Basin 1 BA-SPP-01					
No Problem Areas					
Sea Pines - Baynard Cove - Major Basin 2 BA-SPP-02					
Golf Course	BASP02-12	5.9	25	6.29	6.29
Heritage Road	BASP02-13	5.9	25	6.29	6.3
Open Space	BASP02-15	5.6	25	6.11	6.12
St. Andrews Place	BASP02-16	5.6	25	6.11	6.12
Sea Pines - Baynard Cove - Major Basin 3 BA-SPP-03					
Outfall Pipe	BASP03-2	6.0	25	6.14	6.14
Baynard Cove Road	BASP03-4	6.0	25	6.29	6.29
Baynard Cove Road	BASP03-5	6.0	25	6.29	6.29
Heritage Road	BASP03-6	6.0	25	6.29	6.29
Heritage Road	BASP03-8	6.0	25	6.29	6.29
Open Space	BASP03-9	6.0	25	6.29	6.29
Open Space	BASP03-10	6.0	25	6.3	6.3
Open Space	BASP03-11	6.0	25	6.29	6.29
Open Space	BASP03-12	6.0	25	6.22	6.22
Harleston Green Road	BASP03-14	6.0	25	6.21	6.21
Sea Pines - Braddock Cove - Major Basin 1 BC-SPP-01					
Sprunt Pond Road	BCSP01-2	4.8	25	5.5	5.5
Sprunt Pond Road	BCSP01-3	5.0	25	7.08	7.08
Long Cove Club - Broad Creek - Major Basin 1 BR-LCC-01					
Outfall Pipe	BRLCC01-1	5.0	25	5.12	5.16
Long Cove Drive	BRLCC01-2	5.6	25	6.23	6.35
Open Space	BRLCC01-3	5.7	25	6.24	6.31
Long Cove Drive	BRLCC01-4	5.8	25	6.71	6.73
Open Space	BRLCC01-5	5.9	25	6.89	6.92
Retreat Lane	BRLCC01-6	5.9	25	6.87	6.89
Long Cove Drive	BRLCC01-7	6.0	25	6.88	6.91
Long Cove / Turnbridge Drive	BRLCC01-8	5.2	25	5.6	5.63
Long Cove / Turnbridge / Strawberry Hill	BRLCC01-12	5.1	25	5.2	5.21
Long Cove Club - Broad Creek - Major Basin 2 BR-LCC-02					
No Problem Areas					
Sea Pines - Lawton Canal - Major Basin 1 LC-SPP-01					
No Problem Areas					

TABLE 15-7N
RECOMMENDED CULVERT IMPROVEMENTS, HILTON HEAD ISLAND (NORTH)

<table>
<tr><th>Road Crossing</th><th>ICPR Model Link ID</th><th>Existing Culvert Dimensions (in x in)</th><th>Recommended Improvements</th><th>Priority</th></tr>
<tr><td colspan="5">Chaplan Area - Broad Creek Outfall - Major Basin 1
BR-CHP-01</td></tr>
<tr><td colspan="5">No Improvements Necessary</td></tr>
<tr><td colspan="5">Indigo Run - Broad Creek Outfall - Major Basin 1
BR-IRP-01</td></tr>
<tr><td>Wentworth Place</td><td>BRIRP01-P5</td><td>60 x 60</td><td rowspan="6">Add new outfall from Node BRIRP01-6 to Broad Creek; Outfall Structure should have minimum weir length of 21 feet at elevation 6.5; Lower all affected lagoon elevations to 6.5 NAVD; Additional 48" RCP Leg O Mutton; Additional 42" RCP Mead Lane; Additional 42" RCP at the Preserve Crossing; Additional 24" under US 278 at Lowest Invert Possible</td><td rowspan="6">1</td></tr>
<tr><td>Doral Lane</td><td>BRIRP01-P6</td><td>60 x 60</td></tr>
<tr><td>Mead Lane</td><td>BRIRP01-P7</td><td>54 x 54</td></tr>
<tr><td>Crossing "The Preserve"</td><td>BRIRP01-P9</td><td>2 - 48 x 48</td></tr>
<tr><td>Gardner Drive U/S</td><td>BRIRP01-P10/
BRIRP01-P10A</td><td>48 x 48 / 30 x 30</td></tr>
<tr><td>U.S. 278</td><td>BRIRP01-P11</td><td>2 - 24 x 24</td></tr>
<tr><td colspan="5">Indigo Run - Broad Creek Outfall - Major Basin 2
BR-IRP-02</td></tr>
<tr><td colspan="5">No Improvements Necessary</td></tr>
<tr><td colspan="5">Gum Tree - Jarvis Creek Outfall - Major Basin 1
JV-GUM-01</td></tr>
<tr><td colspan="5">No Improvements Necessary</td></tr>
<tr><td colspan="5">Hilton Head Plantation - Jarvis Creek Outfall - Major Basin 1
JV-HHP-01</td></tr>
<tr><td>Parkwood Drive</td><td>JVHHP01-P3</td><td>2 - 48 x 48</td><td rowspan="6">Further analysis/modeling beyond the scope of this study is recommended.</td><td rowspan="6">5</td></tr>
<tr><td>Knollwood Drive</td><td>JVHHP01-P4</td><td>2 - 48 x 48</td></tr>
<tr><td>Headlands/Fallen Arrow Drive</td><td>JVHHP01-P5</td><td>2 - 42 x 42</td></tr>
<tr><td>Whooping Crane Way</td><td>JVHHP01-D4</td><td>42 x 42</td></tr>
<tr><td>Summer Breeze Court</td><td>JVHHP01-D5</td><td>18 x 18</td></tr>
<tr><td>Golf Course</td><td>JVHHP01-P12</td><td>24 x 24</td></tr>
<tr><td colspan="5">Indigo Run - Jarvis Creek Outfall - Major Basin 1
JV-IRP-01</td></tr>
<tr><td colspan="5">No Improvements Necessary</td></tr>
<tr><td colspan="5">Spanish Wells Plantation - Old House Creek Outfall - Major Basin 1
OH-SPW-01</td></tr>
<tr><td colspan="5">No Improvements Necessary</td></tr>
<tr><td colspan="5">Hilton Head Plantation - Park Creek Outfall - Major Basin 1
PA-HHP-01</td></tr>
<tr><td>Dolphin Head Road</td><td>PAHHP01-P1</td><td>3 - 54 x 54</td><td rowspan="5">Due to watershed basin size restrictions levied by the scope of this study, further analysis/modeling beyond the scope of this study is recommended.</td><td rowspan="5">5</td></tr>
<tr><td>Seabrook / Golden Hind Drive</td><td>PAHHP01-P3</td><td>2 - 54 x 54</td></tr>
<tr><td>Golf Course</td><td>PAHHP01-D1</td><td>2 - 42 x 42</td></tr>
<tr><td>Golf Course</td><td>PAHHP01-P4</td><td>Cart Bridge</td></tr>
<tr><td>Seabrook Drive</td><td>PAHHP01-D2</td><td>30 x 30</td></tr>
</table>

TABLE 15-7N
RECOMMENDED CULVERT IMPROVEMENTS, HILTON HEAD ISLAND (NORTH)

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements	Priority
Hilton Head Plantation - Port Royal Sound Outfall - Major Basin 1 PR-HHP-01				
Hickory Forest Drive	PRHHP01-P1	2 - 42 x 42	Further analysis/modeling beyond the scope of this study is recommended.	5
Oyster Reef Drive	PRHHP01-P3	15 x 15		
High Bluff Road	PRHHP01-P2	42 x 42		
Open Space	PRHHP01-P4	24 x 24		
Golf Course / Oyster Reef Drive	PRHHP01-D2	42 x 42		
Hilton Head Plantation - Port Royal Sound Outfall - Major Basin 2 PR-HHP-02				
High Bluff Road	PRHHP02-P1	48 x 48	Due to watershed basin size restrictions levied by the scope of this study, further analysis/modeling beyond the scope of this study is recommended.	5
Towhee Road/ Golf Course	PRHHP02-P5	18 x 18		
China Cockle Way	PRHHP02-P3	42 x 42		
Palmetto Hall - Port Royal Sound - Major Basin 1 PR-PHP-01				
Fish Haul Road	PRPHP01-D1	3 - 36 x 36	Due to watershed basin size restrictions levied by the scope of this study, recommend further watershed basin analysis, breakdown and detailed analysis of this drainage system. From discussions with Palmetto Hall Property Owners Association, Mr. Terry Ennis, nuisance road flooding does occur. There is no known residential structural flooding.	3
Fish Haul Road	PRPHP01-D1A	4 - 36 x 36		
Fort Howell Drive	PRPHP01-D2	24 x 24		
Fort Howell Drive	PRPHP01-P3	24 x 24		
Golf Course	PRPHP01-P4	24 x 24		
Fort Howell Drive	PRPHP01-P5	24 x 24		
Fort Howell Drive	PRPHP01-D3	24 x 24		
Fort Howell Drive	PRPHP01-P6	24 x 24		
Fort Howell Drive	PRPHP01-P11	24 x 24		
Fort Howell Drive	PRPHP01-P12	24 x 24		
Sedge Fern Drive	PRPHP01-P13	24 x 24		
Fort Howell Drive	PRPHP01-D10	24 x 24		
Golf Course	PRPHP01-P14	24 x 24		
Golf Course	PRPHP01-P15	24 x 24		
Airport - Fish Haul Creek Outfall - Major Basin 1 FH-AIR-01				
No Improvements Necessary				
Gum Tree - Skull Creek Outfall - Major Basin 1 SK-GUM-01				
No Improvements Necessary				
Hilton Head Plantation - Skull Creek Outfall - Major Basin 1 SK-HHP-01				
Bird Song Way	SKHHP01-P2	48 x 48	Due to watershed basin size restrictions levied by the scope of this study, further analysis/modeling beyond the scope of this study is recommended.	5
Bird Song Way	SKHHP01-P2A	42 x 42		
Hilton Head Plantation - Skull Creek Outfall - Major Basin 2 SK-HHP-02				
Santa Maria Drive	SKHHP02-P1	48 x48	Due to watershed basin size restrictions levied by the scope of this study, further analysis/modeling beyond the scope of this study is recommended.	5

TABLE 15-7S RECOMMENDED CULVERT IMPROVEMENTS, HILTON HEAD ISLAND (SOUTH)				
Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements	Priority
Port Royal Plantation - Broad Creek - Major Basin 1 **BR-PRP-01**				
No Improvements Necessary				
Port Royal Plantation - Fish Haul Creek - Major Basin 1 **FH-PRP-01**				
No Improvements Necessary				
Palmetto Dunes - Broad Creek - Major Basin 1 **BR-PDP-01**				
No Improvements Necessary				
Wexford Plantation - Broad Creek - Major Basin 1 **BR-WEX-01**				
No Improvements Necessary				
Wexford Plantation - Broad Creek - Major Basin 2 **BR-WEX-02**				
No Improvements Necessary				
The Crossings - Broad Creek - Major Basin 1 (Haig Point) **BR-XNG-01**				
No Improvements Necessary				
Point Comfort - Broad Creek - Major Basin 1 **BR-PCT-01**				
No Improvements Necessary				
Point Comfort - Broad Creek - Major Basin 2 **BR-PCT-02**				
No Improvements Necessary				
Sea Pines - Point Comfort - Major Basin 1 **PC-SPP-01**				
Club Course Drive	PCSPP01-P2	2 - 48 x 48	Flooding within this area has not been as severe as depicted in this study. Further analysis/modeling beyond the scope of this study has been completed. Refer to Club Course Drive Culvert Improvements for Detailed Study and Solutions.	1
Channel	PCSPP01-C6	NA		
Channel	PCSPP01-C7	NA		
Channel	PCSPP01-C8	NA		
Channel	PCSPP01-C9	NA		
Channel	PCSPP01-C10	NA		
Otter Road	PCSPP01-D1	36 x 36		
Publix	PCSPP01-P5	54 x 54		
Channel	PCSPP01-C15	NA		
Greenwood Drive	PCSPP01-P6	48 x 48		
Sea Pines - Point Comfort - Major Basin 2 **PC-SPP-02**				
Club Course Drive	PCSPP02-P2	30 x 30	Flooding within this area of Sea Pines has not been an issue in the recent past. Further analysis/modeling beyond the scope of this study is recommended.	5

TABLE 15-7S
RECOMMENDED CULVERT IMPROVEMENTS, HILTON HEAD ISLAND (SOUTH)

Road Crossing	ICPR Model Link ID	Existing Culvert Dimensions (in x in)	Recommended Improvements	Priority
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01				
No Improvements Necessary				
Sea Pines - Calibogue Sound - Major Basin 1 CA-SPP-01				
No Improvements Necessary				
Sea Pines - Baynard Cove - Major Basin 1 BA-SPP-01				
No Improvements Necessary				
Sea Pines - Baynard Cove - Major Basin 2 BA-SPP-02				
Open Space	BASP02-P3	30 x 30	Install additional 36" downstream of Turnberry Lane; Additional 30" at Turnberry Lane; Remove and Replace Existing 30" with 42" downstream crossing of Heritage Road; Remove and Replace Existing 30" with 36" at Heritage Road.	3
Heritage Road	BASP02-P4	30 x 30		
Open Space	BASP02-P5	36 x 36		
Saint Andrews Place	BASP02-P6	24 x 24		
Sea Pines - Baynard Cove - Major Basin 3 BA-SPP-03				
Outfall Pipe	BASP03-D1	30 x 30	Install additional 4' x 4' Drop Structure with a 30" barrel at outfall. Weir Elevation 3.28	3
Baynard Cove Road	BASP03-P1	48 x 48		
Channel	BASP03-C3	NA		
Channel	BASP03-C4	NA		
Heritage Road	BASP03-P2	42 x 42		
Channel	BASP03-C6	NA		
Open Space	BASP03-P3	30 x 30		
Channel	BASP03-C7	NA		
Open Space	BASP03-P4	30 x30		
Harleston Green Road	BASP03-P5	30 x 30		
Sea Pines - Braddock Cove - Major Basin 1 BC-SPP-01				
Sprunt Pond Road	BCSP01-P1	18 x 18	Replace existing outfall control structure with new control structure (42" pipe/12' min weir length/Weir Elevation 3.0) at South Sea Pines Drive; Replace existing drop structure with 36" RCP Pipe (No Weir) under Sprunt Pond Road	4
Sprunt Pond Road	BCSP01-D2	18 x 18		
Sea Pines - Lawton Canal - Major Basin 1 LC-SPP-01				
No Improvements Necessary				
Long Cove Club - Broad Creek - Major Basin 1 BR-LCC-01				
Outfall Pipe	BRLCC01-D1	2 - 30 x 30	Flooding within Long Cove has not been an issue in the recent past. Further analysis/modeling beyond the scope of this study is recommended.	5
Long Cove Drive	BRLCC01-D2	24 x 24		
Open Space	BRLCC01-P1	15 x 15		
Long Cove Drive	BRLCC01-D3	15 x 15		
Open Space	BRLCC01-P2	24 x 24		
Retreat Lane	BRLCC01-P3	30 x 30		
Long Cove Drive	BRLCC01-D3A	30 x 30		
Long Cove / Turnbridge Drive	BRLCC01-P5	30 x 30		
Long Cove / Turnbridge / Strawberry Hill	BRLCC01-P10	24 x 24		
Long Cove Club - Broad Creek - Major Basin 2 BR-LCC-02				
No Improvements Necessary				

TABLE 15-8
WATER QUALITY BASIN LAND USE DISTRIBUTION
HILTON HEAD ISLAND WATER QUALITY BASINS

Exisitng Land Use Type	Broad Creek 1 (acres)	Broad Creek 2 (acres)	Broad Creek 3 (acres)	Broad Creek 4 (acres)	Old House Creek (acres)	Jarvis Creek 1 (acres)	Jarvis Creek 2 (acres)	Broad River 4 Existing	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	0	0	0	0	0
Commercial	193	397	53	58	16	15	86	194	1,013
Forest/Rural Open	0	46	2	4	1	3	43	75	174
Golf Course	223	1,214	7	248	0	0	194	965	2,850
High Density Residential	962	2,196	91	527	0	57	285	1,652	5,770
Industrial	370	890	53	167	42	22	268	634	2,447
Institutional	13	27	0	8	0	0	103	17	169
Low Density Residential	7	0	0	0	0	0	0	0	7
Medium Density Residential	14	170	0	0	88	47	206	9	534
Open Water/Tidal	1,432	1,881	480	170	108	707	284	0	5,061
Silviculture	0	0	0	0	0	0	0	0	0
Urban Open	645	665	62	128	33	63	341	759	2,695
Wetland/Water	360	360	2	107	0	12	113	133	1,088
TOTAL	4,219	7,846	750	1,417	288	927	1,924	4,438	21,809
Urban Imperviousness (%)	22%	27%	17%	31%	23%	7%	26%	33%	26%

Future Land Use Type	Broad Creek 1 (acres)	Broad Creek 2 (acres)	Broad Creek 3 (acres)	Broad Creek 4 (acres)	Old House Creek (acres)	Jarvis Creek 1 (acres)	Jarvis Creek 2 (acres)	Broad River 4 Future	TOTAL (acres)
Agricultural/Pasture	0	0	0	0	0	0	0	0	0
Commercial	206	481	58	71	16	16	130	202	1,180
Forest/Rural Open	0	2	0	0	0	0	0	7	9
Golf Course	244	1,328	7	248	0	0	227	1,214	3,267
High Density Residential	985	2,216	91	527	0	57	286	1,655	5,816
Industrial	371	891	54	168	49	22	270	761	2,585
Institutional	63	43	0	24	0	2	121	42	294
Low Density Residential	7	0	0	0	0	0	0	0	7
Medium Density Residential	75	278	4	8	106	94	321	122	1,008
Open Water/Tidal	1,430	1,881	480	171	108	708	284	0	5,062
Silviculture	0	0	0	0	0	0	0	0	0
Urban Open	478	366	53	94	9	17	172	307	1,497
Wetland/Water	360	360	2	107	0	12	113	133	1,089
TOTAL	4,219	7,846	750	1,417	288	927	1,924	4,443	21,814
Urban Imperviousness (%)	23%	29%	18%	32%	26%	9%	30%	36%	28%

excludes Broad River open water/tidal area that was included in the Broad River 4 water quality basin in the Broad River analysis (Section 12)

TABLE 15-9
WATER QUALITY BASIN BMP COVERAGE
HILTON HEAD ISLAND

Existing Land Use Type	Broad Creek 1 (%)	Broad Creek 2 (%)	Broad Creek 3 (%)	Broad Creek 4 (%)	Old House Creek (%)	Jarvis Creek 1 (%)	Jarvis Creek 2 (%)	Broad River 4 Existing	TOTAL (%)
Commercial	18%	27%	3%	23%	0%	15%	15%	16%	20%
Golf Course	100%	94%	86%	100%	0%	0%	100%	100%	97%
High Density Residential	67%	85%	86%	80%	0%	97%	97%	87%	83%
Industrial	10%	32%	38%	57%	0%	12%	37%	69%	40%
Institutional	0%	0%	0%	0%	0%	0%	0%	6%	1%
Low Density Residential	0%	0%	0%	0%	0%	0%	0%	0%	0%
Medium Density Residential	0%	60%	0%	0%	0%	0%	0%	0%	19%
TOTAL	53%	71%	52%	77%	0%	43%	51%	82%	69%

Future Land Use Type	Broad Creek 1 (%)	Broad Creek 2 (%)	Broad Creek 3 (%)	Broad Creek 4 (%)	Old House Creek (%)	Jarvis Creek 1 (%)	Jarvis Creek 2 (%)	Broad River 4 Future	TOTAL (%)
Commercial	23%	39%	11%	38%	0%	22%	44%	19%	31%
Golf Course	100%	94%	85%	100%	0%	0%	100%	100%	98%
High Density Residential	68%	85%	86%	80%	0%	97%	97%	87%	83%
Industrial	11%	32%	38%	58%	14%	13%	37%	85%	47%
Institutional	79%	38%	0%	65%	0%	100%	15%	62%	43%
Low Density Residential	0%	0%	0%	0%	0%	0%	0%	0%	0%
Medium Density Residential	81%	76%	100%	100%	17%	50%	36%	93%	57%
TOTAL	57%	73%	54%	78%	15%	58%	59%	87%	73%

TABLE 15-10

WATER QUALITY BASIN SEPTIC TANK COVERAGE
HILTON HEAD ISLAND

Existing Land Use Type	Broad Creek 1 (%)	Broad Creek 2 (%)	Broad Creek 3 (%)	Broad Creek 4 (%)	Old House Creek (%)	Jarvis Creek 1 (%)	Jarvis Creek 2 (%)	Broad River 4 (%)	TOTAL (%)
Commercial	0%	2%	6%	39%	100%	83%	9%	21%	10%
High Density Residential	0%	2%	62%	14%	0%	1%	0%	10%	6%
Industrial	0%	15%	36%	19%	74%	26%	42%	28%	21%
Institutional	0%	0%	0%	0%	0%	0%	2%	9%	2%
Low Density Residential	100%	0%	0%	0%	0%	0%	0%	0%	100%
Medium Density Residential	100%	97%	0%	0%	73%	81%	80%	100%	85%
TOTAL	1%	9%	40%	17%	77%	40%	30%	16%	14%

Future Land Use Type	Broad Creek 1 (%)	Broad Creek 2 (%)	Broad Creek 3 (%)	Broad Creek 4 (%)	Old House Creek (%)	Jarvis Creek 1 (%)	Jarvis Creek 2 (%)	Broad River 4 (%)	TOTAL (%)
Commercial	0%	1%	5%	32%	100%	76%	6%	20%	9%
High Density Residential	0%	2%	62%	14%	0%	1%	0%	10%	6%
Industrial	0%	15%	35%	19%	65%	22%	42%	24%	20%
Institutional	0%	0%	0%	0%	0%	0%	1%	3%	1%
Low Density Residential	100%	0%	0%	0%	0%	0%	0%	0%	100%
Medium Density Residential	19%	59%	0%	0%	61%	41%	51%	7%	45%
TOTAL	1%	9%	38%	16%	66%	30%	25%	14%	13%

TABLE 15-11

AVERAGE ANNUAL LOADS FOR HILTON HEAD ISLAND WATER QUALITY BASINS

EXISTING LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Broad Creek 1	4,219	10,643	137,000	942,000	4,936	39,737	194	3,002	9.44E+14
Broad Creek 2	7,846	18,630	253,000	1,650,000	8,899	71,321	313	4,638	1.74E+15
Broad Creek 3	750	2,354	26,172	136,000	1,077	9,118	43	818	2.26E+14
Broad Creek 4	1,417	3,080	44,158	297,000	1,489	12,650	47	623	3.49E+14
Old House Creek	288	745	11,488	96,570	505	4,281	20	232	2.39E+14
Jarvis Creek 1	926	2,974	28,242	106,000	1,364	11,206	51	1,083	3.10E+14
Jarvis Creek 2	1,924	4,060	61,078	482,000	2,191	18,281	79	898	7.08E+14
Broad River 4	4,438	3,728	127,431	868,863	4,003	34,420	111	1,206	8.5E+14
TOTAL	21,808	46,214	688,569	4,578,433	24,464	201,014	858	12,500	5.36E+15

FUTURE LAND USE

Water Quality Basin ID	Area (acres)	Flow (acre-feet)	BOD (lb/yr)	TSS (lb/yr)	Total P (lb/yr)	Total N (lb/yr)	Lead (lb/yr)	Zinc (lb/yr)	Fecal Coliform (#/yr)
Broad Creek 1	4,219	10,788	140,000	950,000	4,986	40,128	196	3,034	9.45E+14
Broad Creek 2	7,846	18,928	259,000	1,670,000	9,050	72,202	318	4,709	1.73E+15
Broad Creek 3	750	2,368	26,512	137,000	1,105	9,626	43	822	2.58E+14
Broad Creek 4	1,417	3,133	45,251	299,000	1,501	12,792	47	635	3.47E+14
Old House Creek	288	769	12,091	98,271	509	4,301	20	237	2.27E+14
Jarvis Creek 1	927	3,010	29,170	107,000	1,375	11,237	51	1,091	2.98E+14
Jarvis Creek 2	1,924	4,251	65,584	494,000	2,256	18,693	82	940	6.78E+14
Broad River 4	4,438	4,069	133,431	828,863	4,111	34,420	108	1,246	7.8E+14
TOTAL	21,809	47,316	711,039	4,584,134	24,893	203,399	865	12,714	5.26E+15
Percent Increase over Existing Land Use		2%	3%	0%	2%	1%	1%	2%	-2%

TABLE 15-12

EXISTING LEVEL OF SERVICE IN WATER QUALITY BASINS

HILTON HEAD ISLAND

Water Quality Basin ID	DHEC Station(s)	Fecal Coliform Concentrations				Level of Service
		Long-Term Average		Maximum 36-Sample Values		
		Geomean (#/100 ml)	90th Percentile (#/100 ml)	Geomean (#/100 ml)	90th Percentile (#/100 ml)	
Broad Creek 1	20-15A	8.8	43	14.7	63	C
Broad Creek 2	20-18	9.1	43	11.0	60	C
Broad Creek 3	20-16, 20-16A	22.6	116	29.9	215	D
Broad Creek 4	None	NA	NA	NA	NA	NA
Old House Creek	None	NA	NA	NA	NA	NA
Jarvis Creek 1	20-23	4.6	15	4.6	15	A
Jarvis Creek 2	None	NA	NA	NA	NA	NA

TABLE 15-13

TIDAL RIVER SEGMENT PHYSICAL CHARACTERISTICS

HILTON HEAD ISLAND

Water Quality Basin ID	South WASP Segment	Volume (m^3)	Exchange with Water Quality Basin ID	Tidal Dispersion Values Area (m^2)	Length (m)	Coefficient (m^2/s)
Broad Creek 1	6	7.02E+06	Calibogue Sound 1	1,606	4,408	180
Broad Creek 2	7	7.03E+06	Broad Creek 1	834	5,262	300
Broad Creek 3	8	1.33E+06	Broad Creek 2	700	4,023	20
Broad Creek 4	9	1.27E+05	Broad Creek 3	346	1,143	20
Old House Creek	18	1.61E+05	Calibogue Sound 2	314	1,184	150
Jarvis Creek 1	19	1.34E+06	Calibogue Sound 3	649	3,454	450
Jarvis Creek 2	20	2.26E+05	Jarvis Creek 1	293	1,851	150

TABLE 15-14

AVERAGE FLOWS AND GEOMEAN FECAL COLIFORM CONCENTRATIONS FROM WMM
FOR HILTON HEAD ISLAND WATER QUALITY BASINS

Water Quality Basin ID	South WASP Segment	EXISTING LAND USE		FUTURE LAND USE	
		Flow (cfs)	Fecal Coliform (#/100 ml)	Flow (cfs)	Fecal Coliform (#/100 ml)
Broad Creek 1	6	14.7	1,188	14.9	1,184
Broad Creek 2	7	25.7	1,001	26.1	1,027
Broad Creek 3	8	3.2	1,322	3.3	1,334
Broad Creek 4	9	4.3	896	4.3	896
Old House Creek	18	1.0	1,785	1.1	1,745
Jarvis Creek 1	19	4.1	1,374	4.2	1,375
Jarvis Creek 2	20	5.6	1,129	5.9	1,113

TABLE 15-15

TIDAL RIVER ADVECTIVE FLOW EXCHANGES
HILTON HEAD ISLAND

From Water Quality Basin ID	To Water Quality Basin ID	Net Advective Flow (cfs)	
		Existing	Future
Broad Creek 1	Calibogue Sound 1	48	49
Broad Creek 2	Broad Creek 1	33	34
Broad Creek 3	Broad Creek 2	7.5	7.6
Broad Creek 4	Broad Creek 3	4.3	4.3
Old House Creek	Calibogue Sound 2	1.0	1.1
Jarvis Creek 1	Calibogue Sound 3	9.7	10
Jarvis Creek 2	Jarvis Creek 1	5.6	5.9

TABLE 15-16

FECAL COLIFORM MODELING RESULTS
HILTON HEAD ISLAND

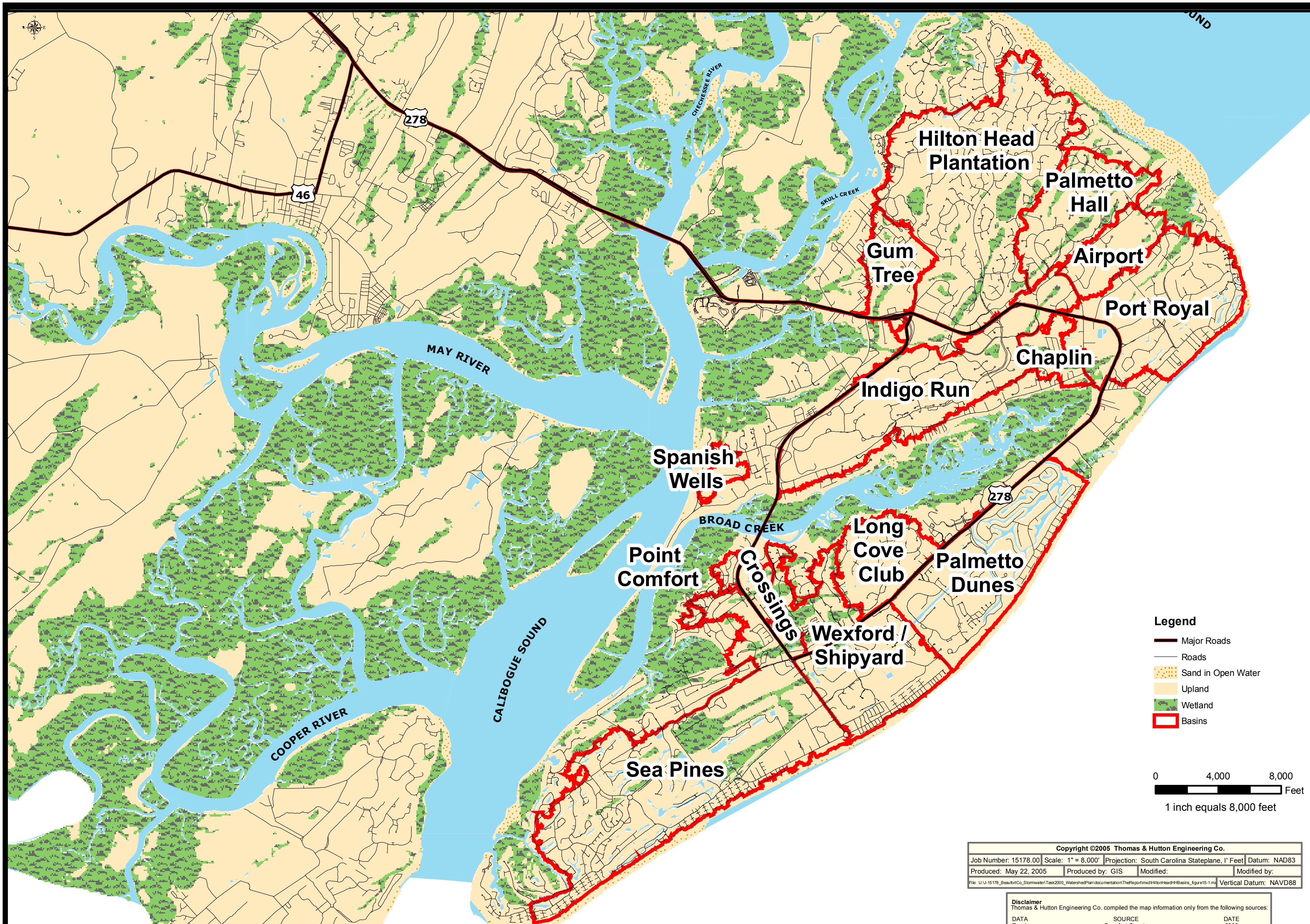
Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)		Modeled Level of Service	
		Existing	Future	Existing	Future
Broad Creek 1	0.7	6.6	6.7	A	A
Broad Creek 2	1.0	8.1	8.4	B	B
Broad Creek 3	1.0	11.7	11.9	D	D
Broad Creek 4	1.0	23.4	23.8	D	D
Old House Creek	1.0	4.7	4.7	A	A
Jarvis Creek 1	2.0	5.2	5.3	A	A
Jarvis Creek 2	2.0	10.4	10.7	D	D

TABLE 15-17

SENSITIVITY ANALYSIS RESULTS

HILTON HEAD ISLAND

Water Quality Basin ID	Bacteria Loss Rate (1/day)	Modeled Geomean Conc (#/100 ml)		Modeled Level of Service	
		Best Case	Worst Case	Best Case	Worst Case
Broad Creek 1	0.7	5.3	8.8	A	C
Broad Creek 2	1.0	6.3	12.5	A	D
Broad Creek 3	1.0	9.4	17.7	C	D
Broad Creek 4	1.0	18.1	41.7	D	D
Old House Creek	1.0	3.9	5.0	A	A
Jarvis Creek 1	2.0	4.5	6.2	A	A
Jarvis Creek 2	2.0	7.7	15.1	B	D



Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHeadHHBasins_figure15-1.mx			Vertical Datum: NAD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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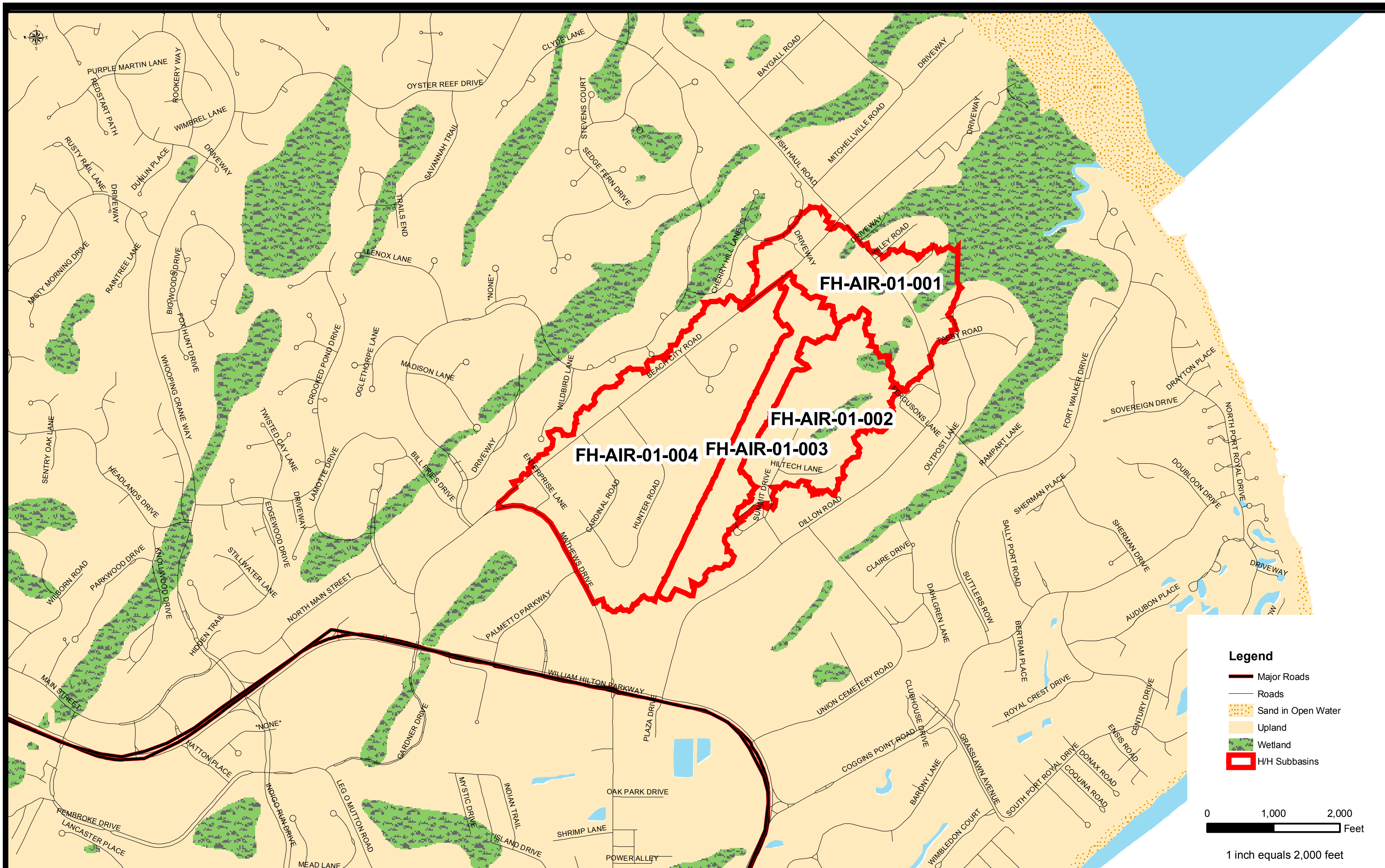
CDM Camp Dresser & McKee Inc.

Figure 15-1

Town of Hilton Head Island Hydrologic Model Basins



Town of Hilton Head Island
Airport
Model Subbasins

Figure 15-2

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Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHeadAirport_figure15-2.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Town of Hilton Head Island
Chaplin Basin
Model Subbasins

Figure 15-3

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CDM Camp Dresser & McKee Inc.

Job Number: 15178.00	Scale: 1" = 1,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHeadChaplin_figure15-3.mxd			Vertical Datum: NAVD88

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Land Use / Land Cover	USGS	

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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CDM Camp Dresser & McKee Inc.

Town of Hilton Head Island
Crossings Basin
Model Subbasins

Figure 15-4

Job Number: 15178.00	Scale: 1" = 1,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHeadCrossings_figure15-4.mx			Vertical Datum: NAVD88

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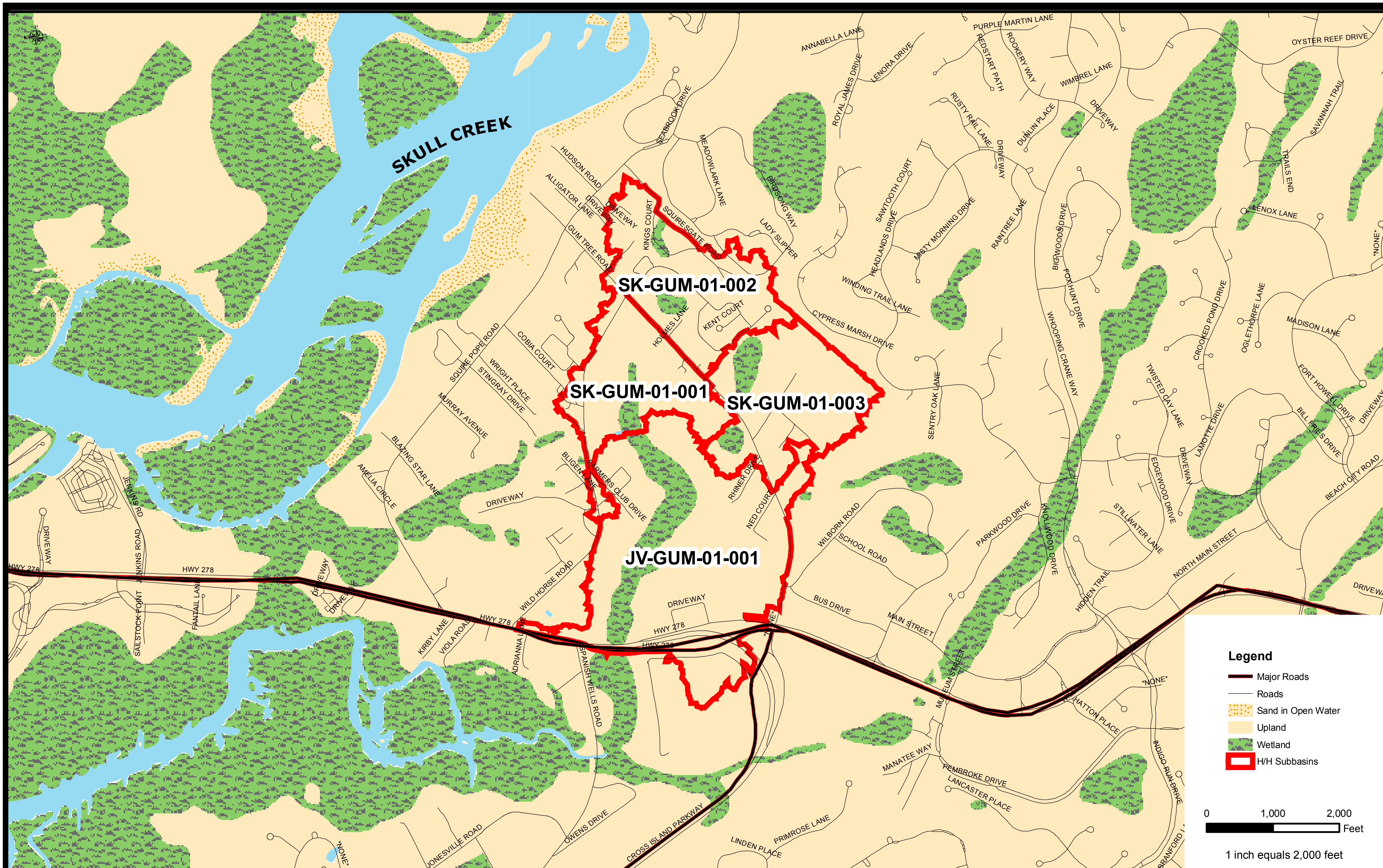
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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CDM Camp Dresser & McKee Inc.

Town of Hilton Head Island
Gum Tree Basin
Model Subbasins

Figure 15-5

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHeadGumTree_figure15-5.mx			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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CDM Camp Dresser & McKee Inc.

Town of Hilton Head Island
Hilton Head Plantation Basin
Model Subbasins

Figure 15-6

Job Number: 15178.00	Scale: 1" = 3,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:

Vertical Datum: NAVD88

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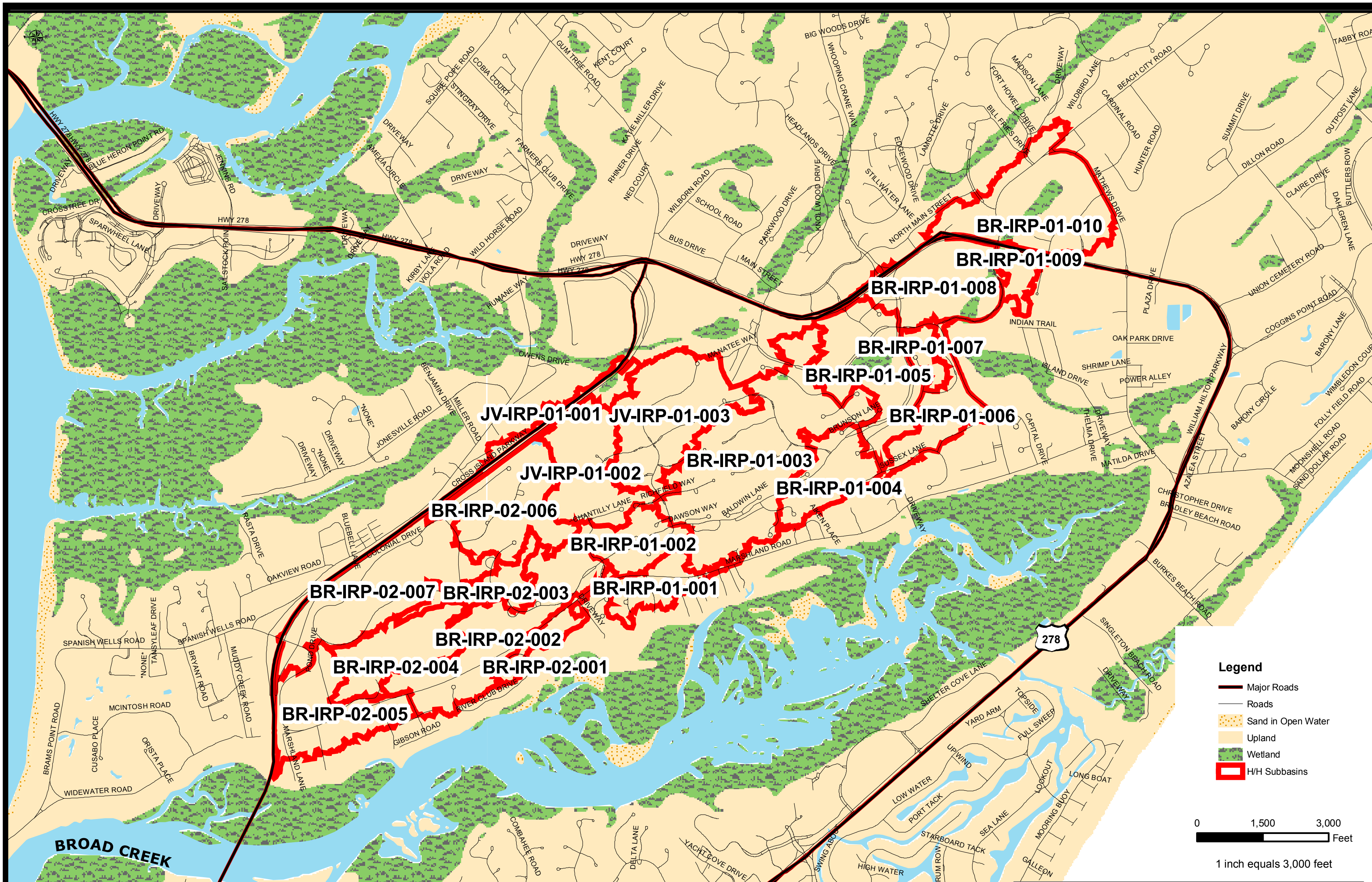
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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CDM Camp Dresser & McKee Inc.

Town of Hilton Head Island
Indigo Run Basin
Model Subbasins

Figure 15-7

Job Number: 15178.00	Scale: 1" = 3,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-IndigoRun_figure15-7.m			Vertical Datum: NAVD88

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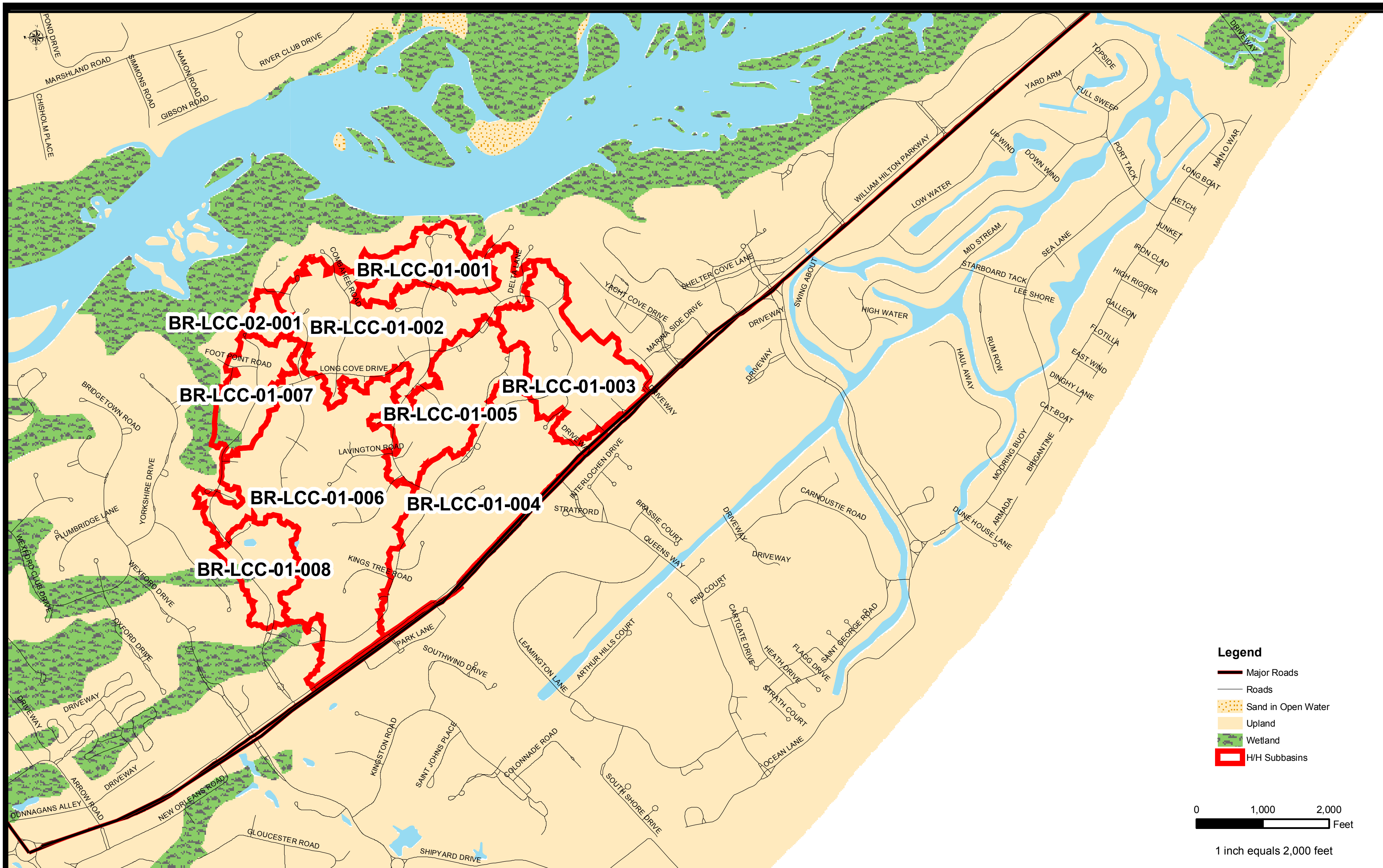
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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CDM Camp Dresser & McKee Inc.

Town of Hilton Head Island
Long Cove Club Basin
Model Subbasins

Figure 15-8

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-LongCove_figure15-8.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Job Number: 15178.00	Scale: 1" = 3,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-PalmettoDunes_figure1			Vertical Datum: NAVD88

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Town of Hilton Head Island
Palmetto Dunes Basin
Model Subbasins

Figure 15-9

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Town of Hilton Head Island
Palmetto Hall Basin
Model Subbasins

Figure 15-10

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
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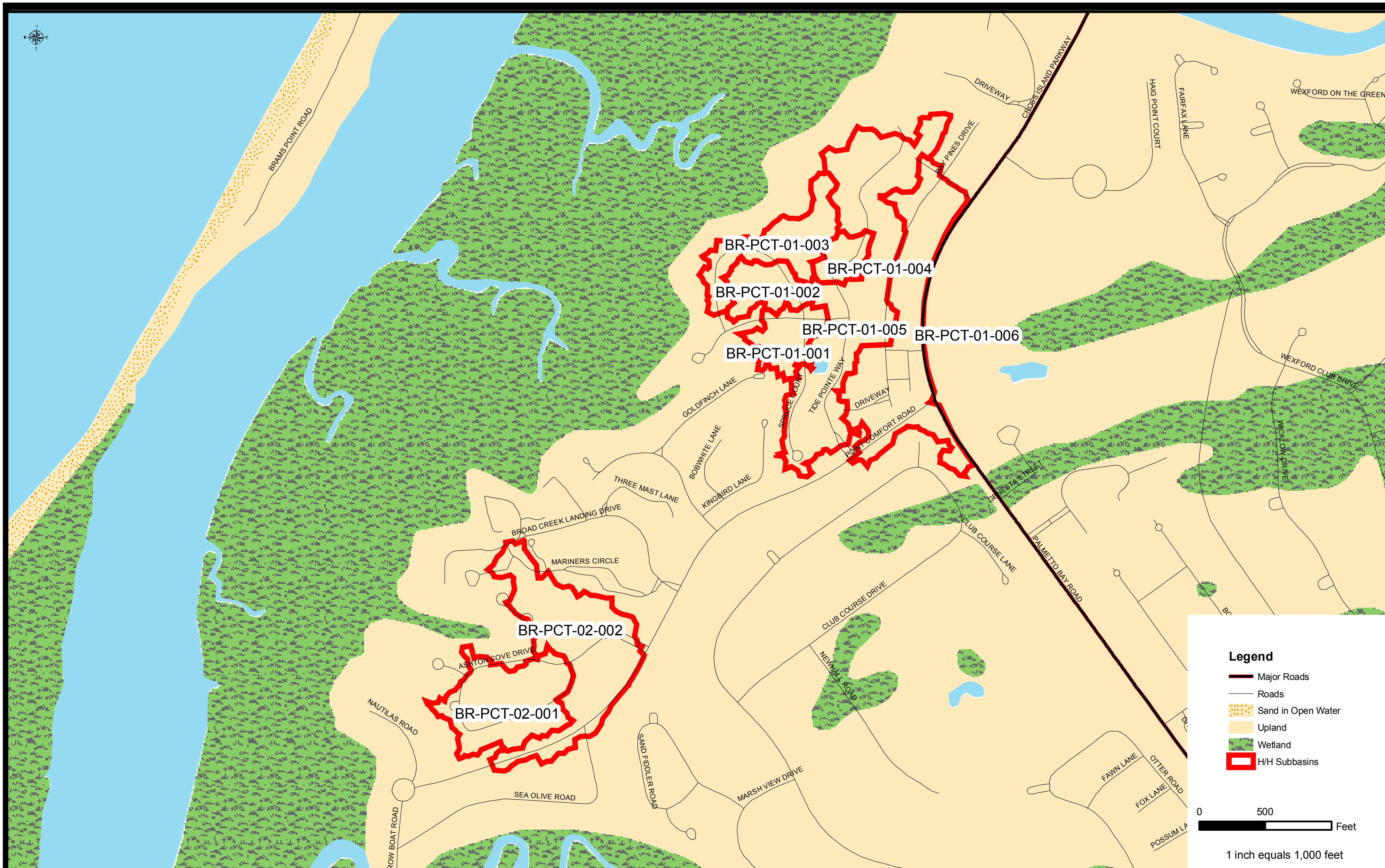

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Town of Hilton Head Island
Point Comfort Basin
Model Subbasins

Figure 15-11

CDM Camp Dresser & McKee Inc.

Job Number: 15178.00	Scale: 1" = 1,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-PointComfort_figure15-1			Vertical Datum: NAVD88

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Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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CDM Camp Dresser & McKee Inc.

Town of Hilton Head Island
Port Royal Plantation Basin
Model Subbasins

Figure 15-12

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:

Vertical Datum: NAVD88

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Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Job Number: 15178.00	Scale: 1" = 4,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-SeaPines_figure15-13.m			Vertical Datum: NAVD88

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Town of Hilton Head Island
Sea Pines Plantation Basin
Model Subbasins

Figure 15-13

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Town of Hilton Head Island
Spanish Wells Basin
Model Subbasins

Figure 15-14

Job Number: 15178.00	Scale: 1" = 1,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-SpanishWells_figure15-			Vertical Datum: NAVD88

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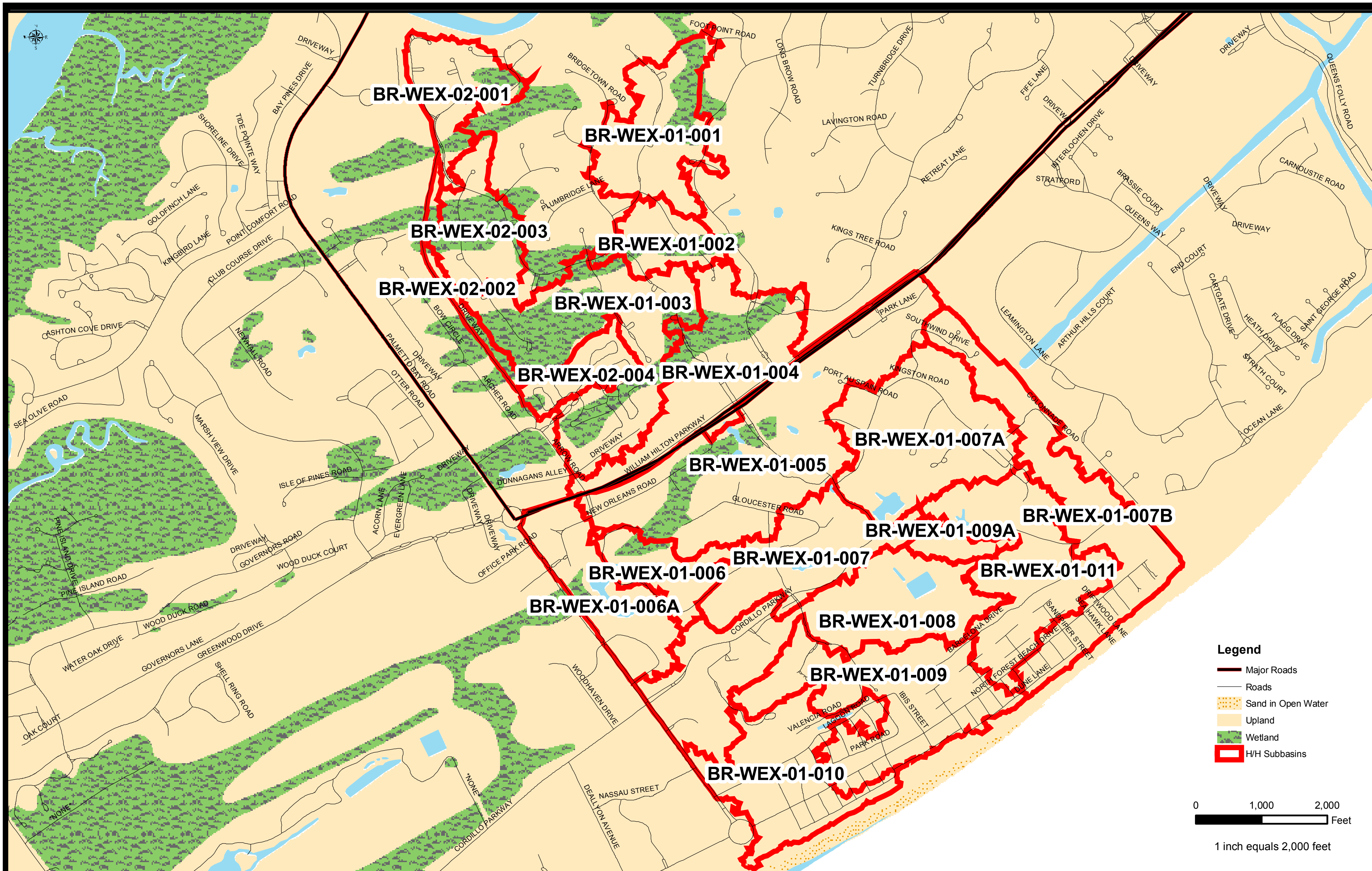
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Town of Hilton Head Island
Wexford / Shipyard Basin
Model Subbasins

Figure 15-15

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-Wexford_figure15-15.m			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Job Number: 15178.00	Scale: 1" = 3,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHeadHHP-Problems_figure15-16.mxd			Vertical Datum: NAVD88

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Town of Hilton Head Island
Hilton Head Plantation Basin - PSMS Problem Areas

Figure 15-16

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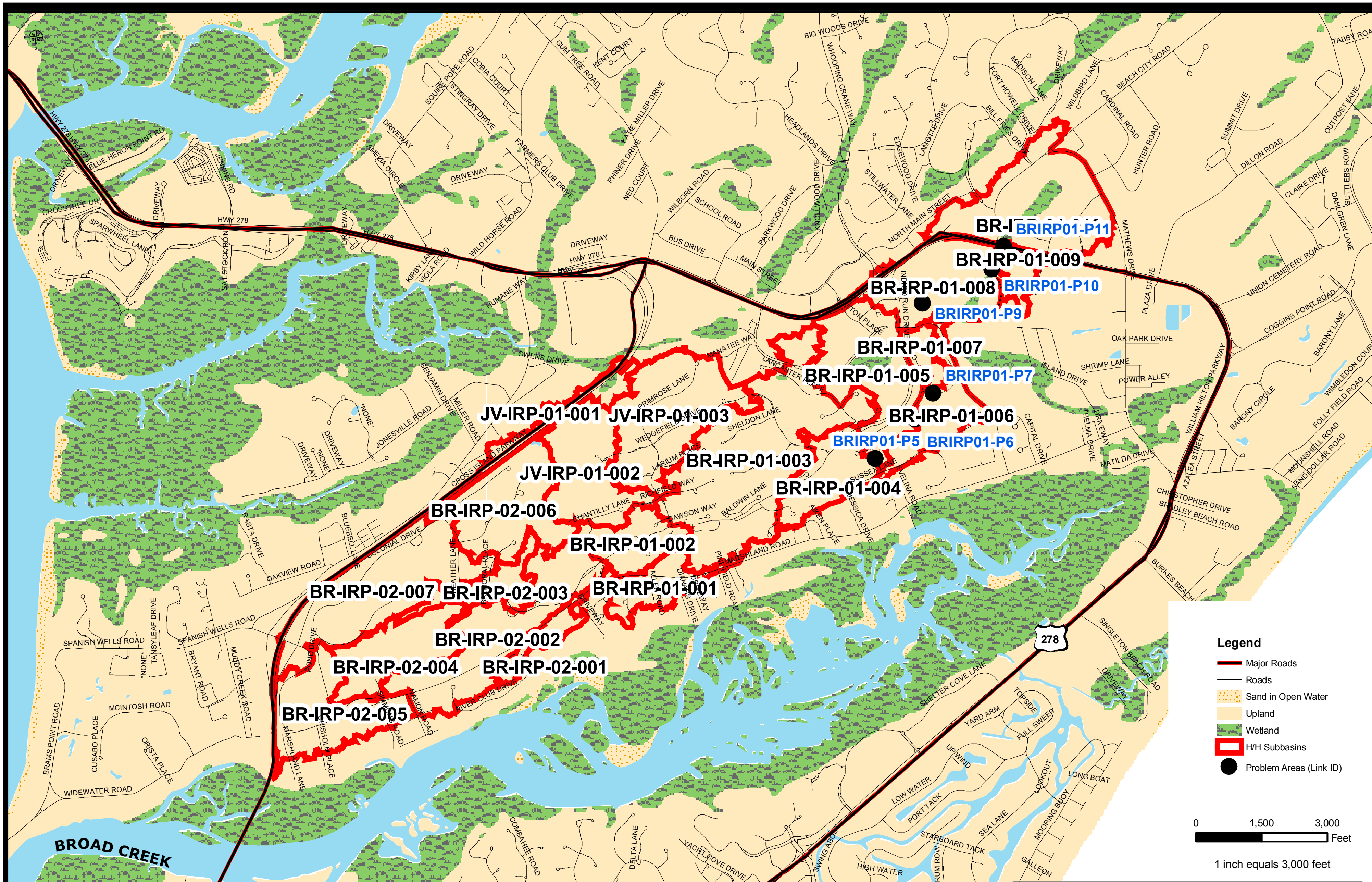
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Town of Hilton Head Island
Indigo Run Basin
PSMS Problem Areas

Figure 15-17

Job Number: 15178.00	Scale: 1" = 3,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-IndigoRun-Problems_fig			Vertical Datum: NAVD88

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Land Use / Land Cover	USGS	

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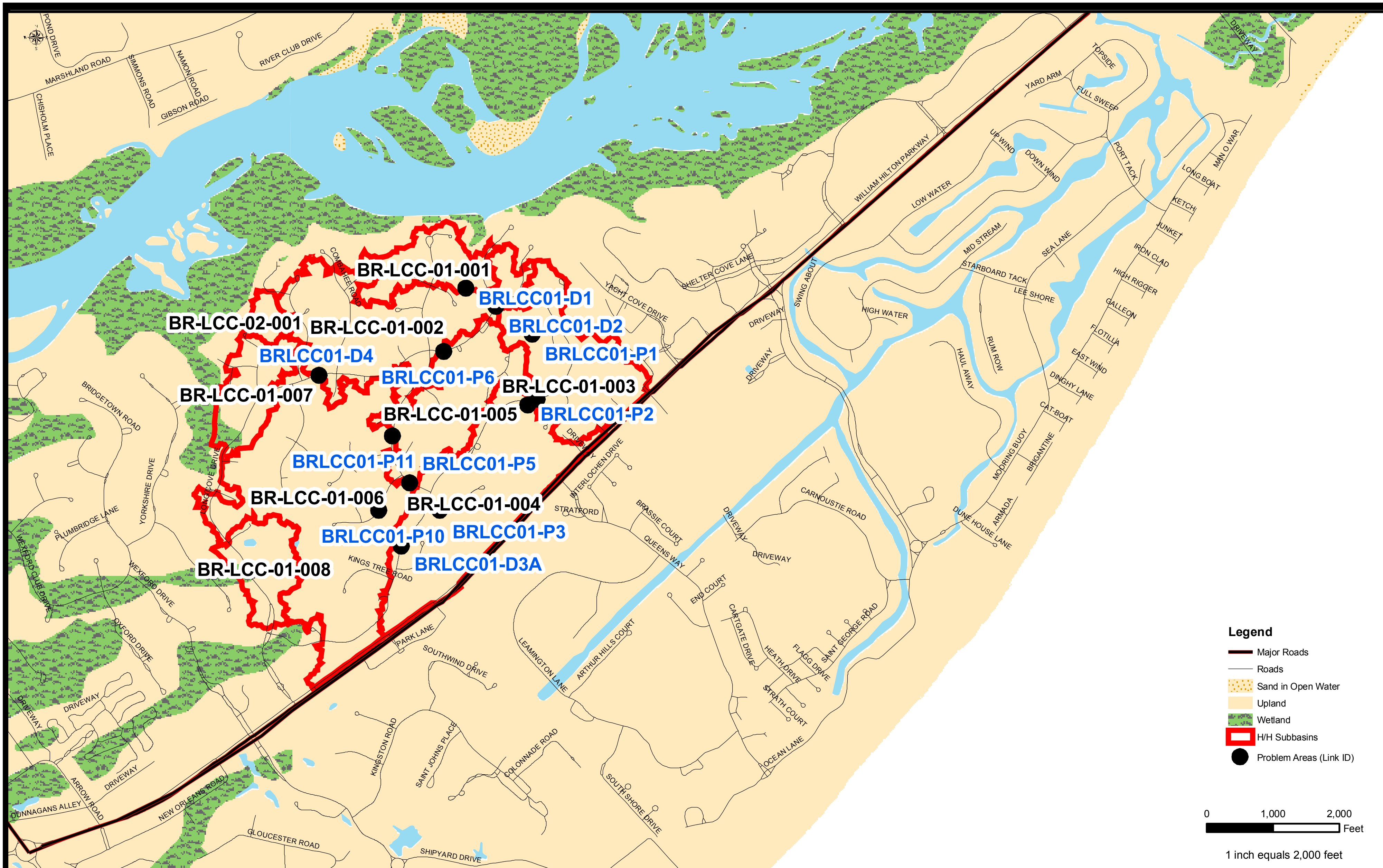
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Town of Hilton Head Island
Long Cove Club Basin
PSMS Problems Areas

Figure 15-18

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-LongCove-Problems_fig			Vertical Datum: NAVD88

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Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Town of Hilton Head Island
Palmetto Hall Basin
PSMS Problem Areas

Figure 15-19

Job Number: 15178.00	Scale: 1" = 2,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-PalmettoHall-Problems_			Vertical Datum: NAVD88

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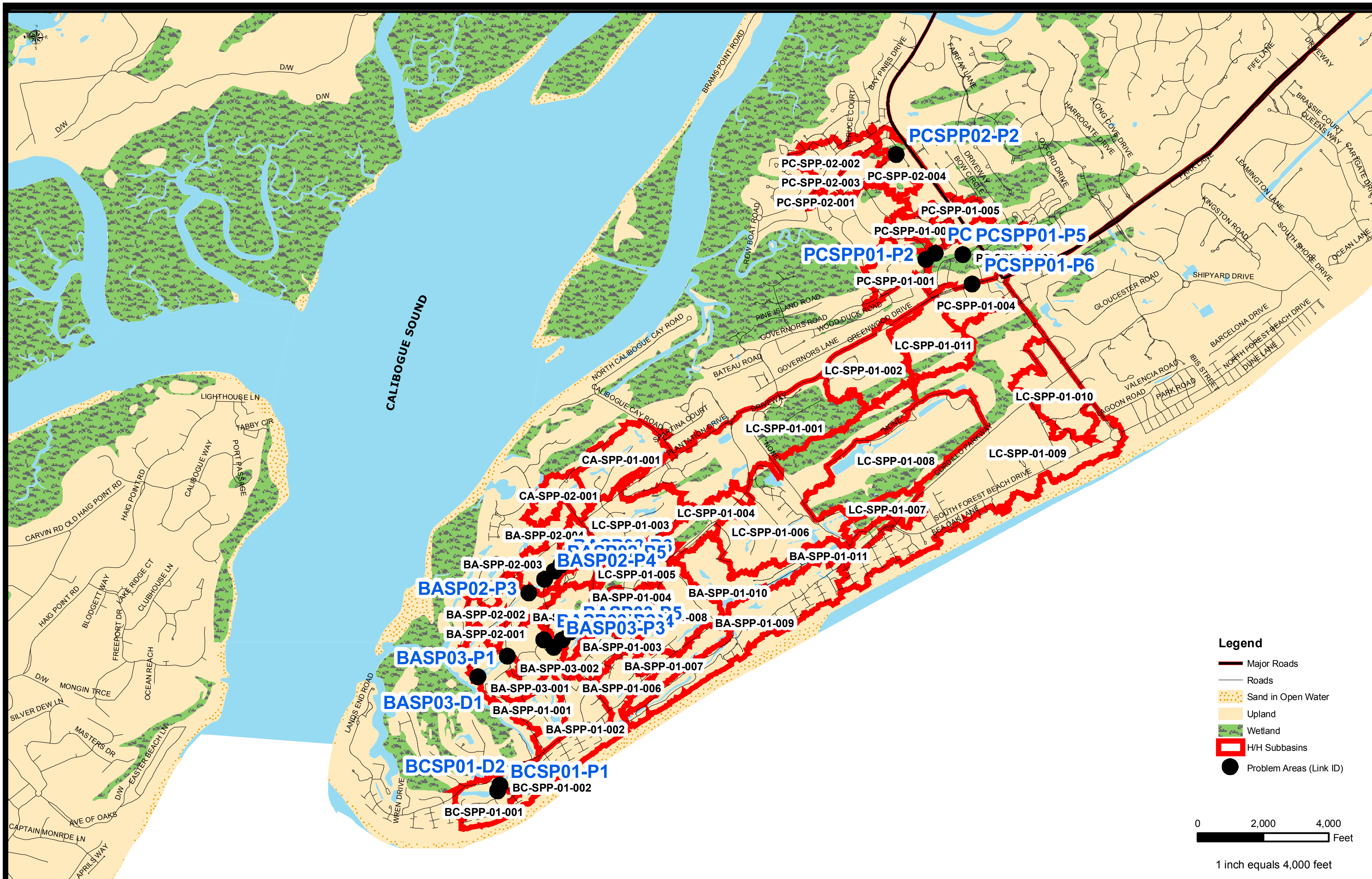
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Town of Hilton Head Island
Sea Pines Plantation Basin
PSMS Problem Areas

Figure 15-20

Job Number: 15178.00	Scale: 1" = 4,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
			Vertical Datum: NAVD88

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Land Use / Land Cover	USGS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

South WQ Basin	South ID
Calibogue Sound 1	1
Calibogue Sound 2	2
Calibogue Sound 3	3
Calibogue Sound 4	4
Calibogue Sound 5	5
Broad Creek 1	6
Broad Creek 2	7
Broad Creek 3	8
Broad Creek 4	9
Cooper River 1	10
Cooper River 2	11
Cooper River 3	12
Cooper River Trib	13
Bull Creek/Cooper 1	14
Bull Creek/Cooper 2	15
Bull Creek/Cooper 3	16
Hoophole Creek	17
Old House Creek	18
Jarvis Creek 1	19
Jarvis Creek 2	20
Skull Creek South 1	21
Skull Creek South 2	22
Mackays Creek South	23
Bryan Creek 1	24
Bryan Creek 2	25
Savage Creek 1	26
Savage Creek 2	27

Job Number: 15178.00	Scale: 1" = 10,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-CalibogueWQ-Plan_figure15-22.mxd			Vertical Datum: NAVD88

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Figure 15-21

Calibogue Sound Watershed Water Quality Basins

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

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Figure 15-22

Chechessee River Watershed
Water Quality Basins

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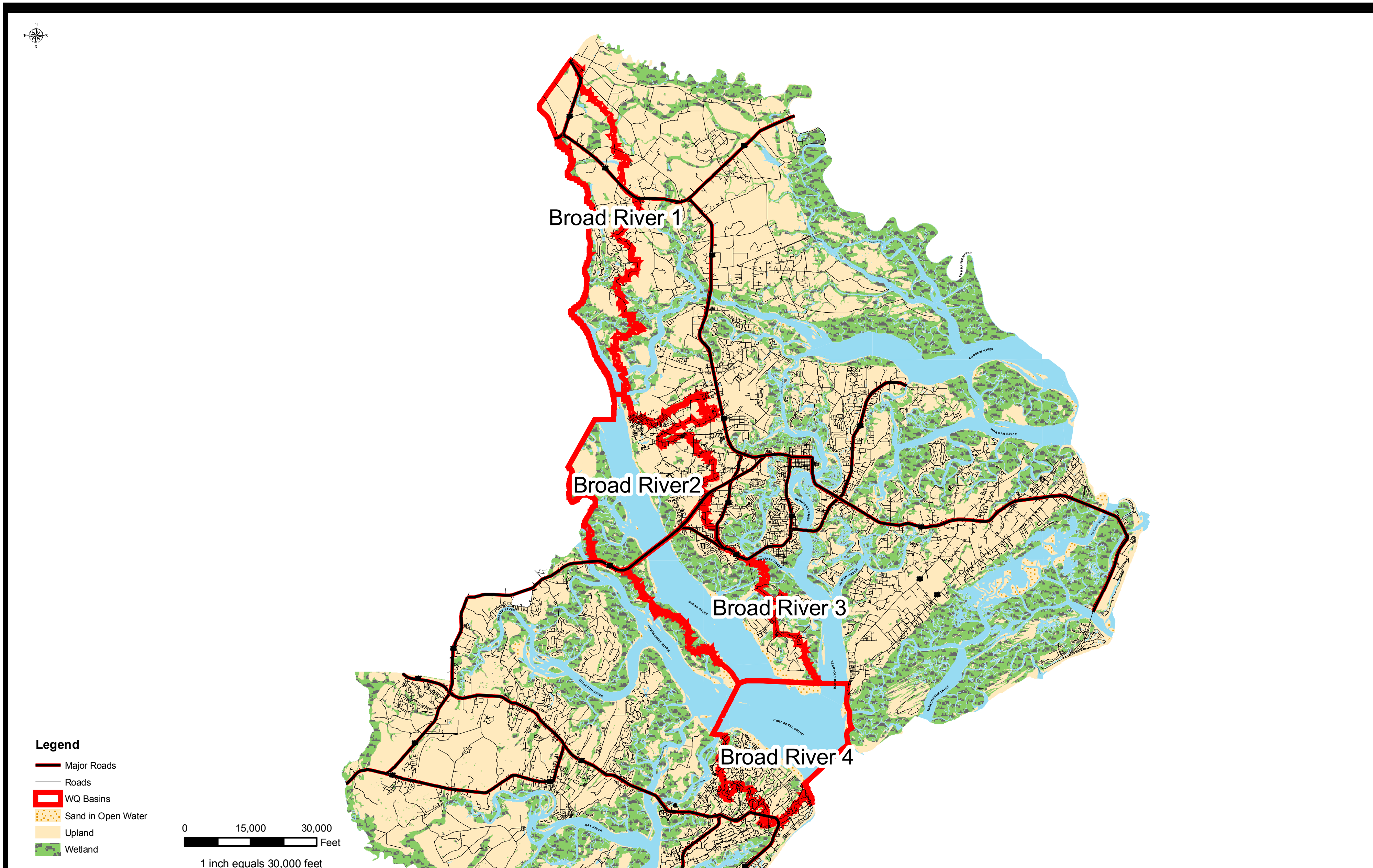
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



Broad River Watershed
Water Quality Basins

Figure 15-23

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Job Number: 15178.00	Scale: 1" = 30,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-BroadRiverWQ_figure15-24.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

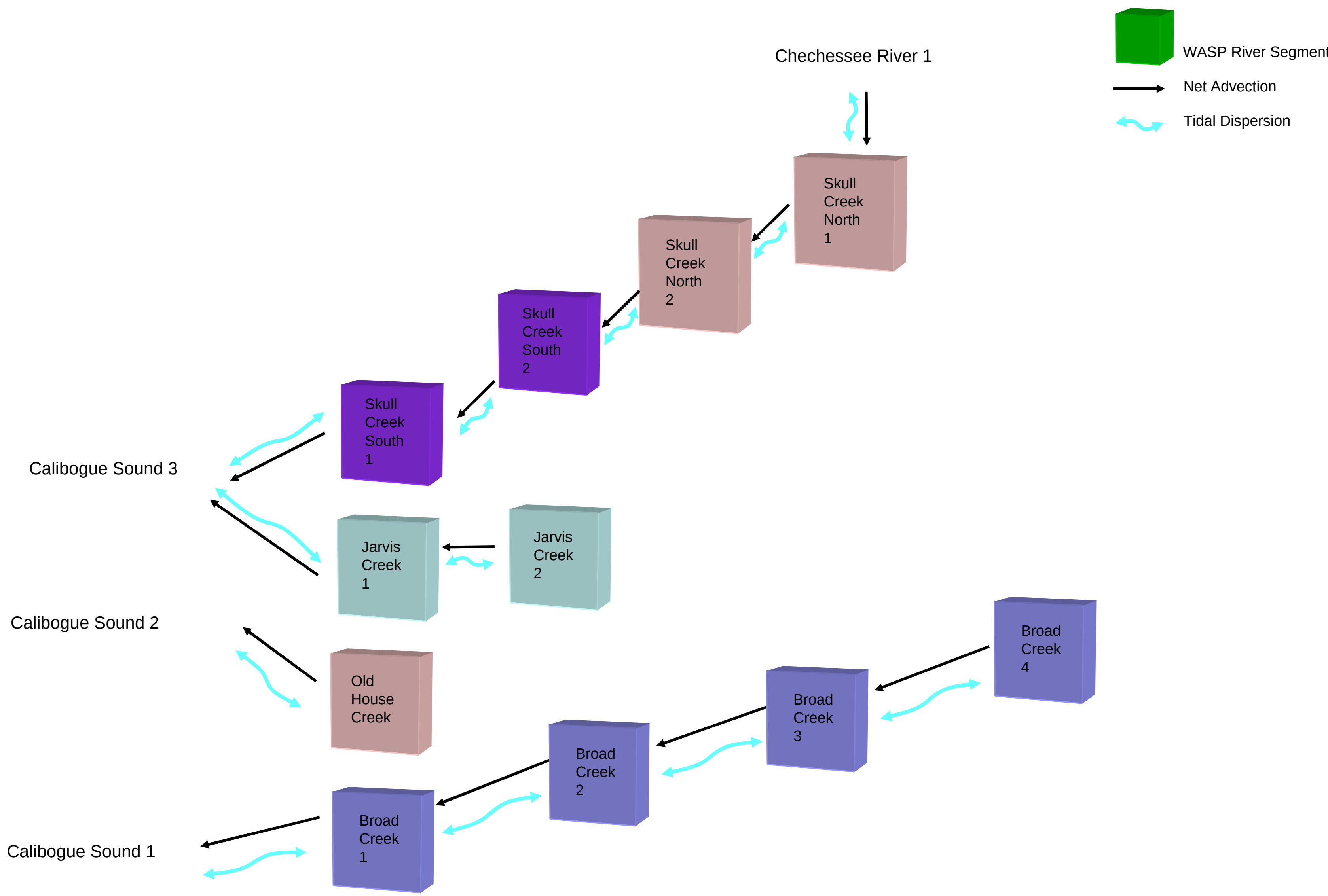
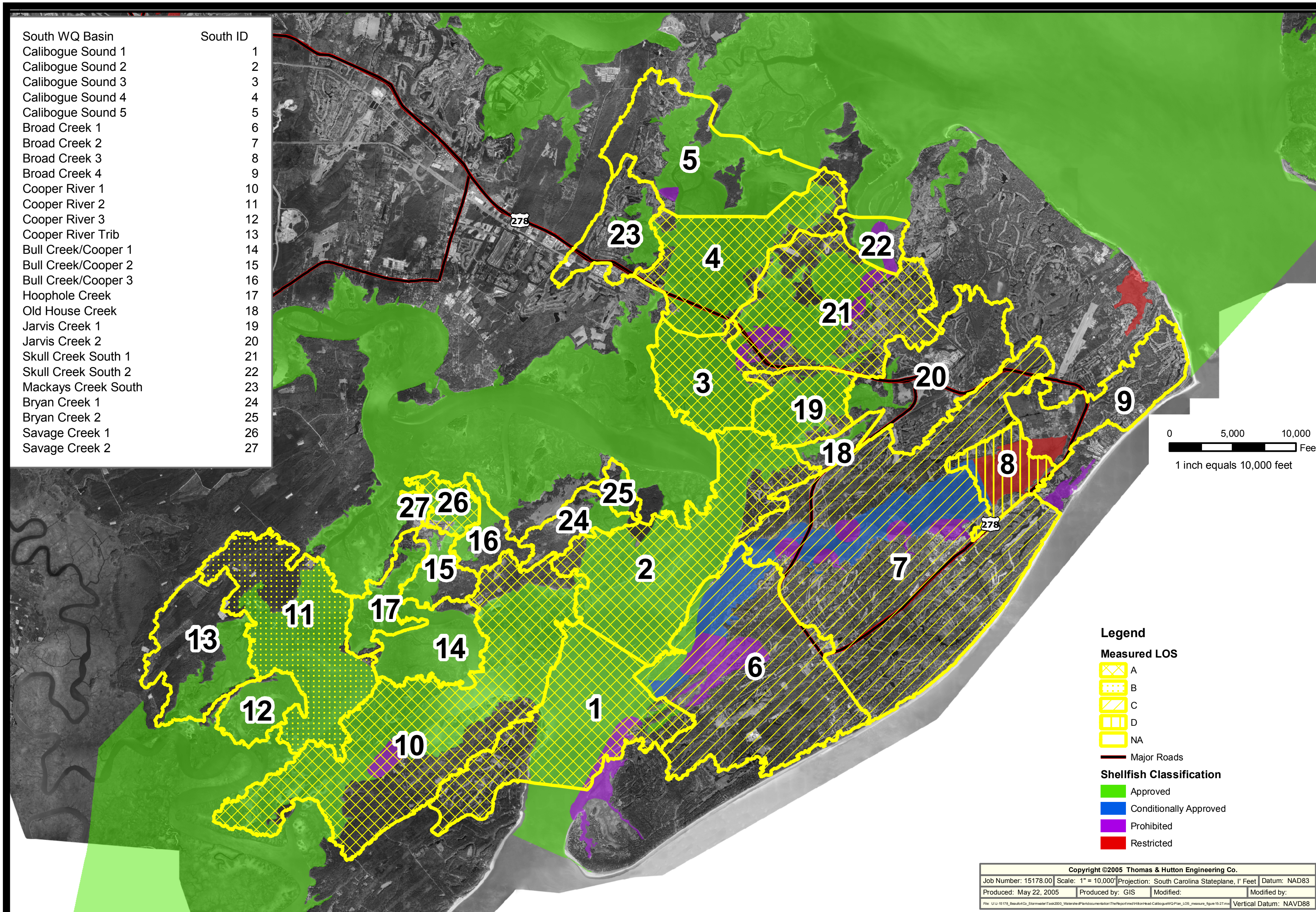


Figure 15-24
WASP Model Schematic for Hilton Head Island



Job Number: 15178.00	Scale: 1" = 10,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-CalibogueWQ-Plan_LOS_measure_figure15-27.mxd			Vertical Datum: NAVD88

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Figure 15-25

Calibogue Sound Watershed
Shellfish Classifications and Existing Level of Service

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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Aerial Photography	Beaufort County	2002
Shellfish Classifications	SCDHEC	2004

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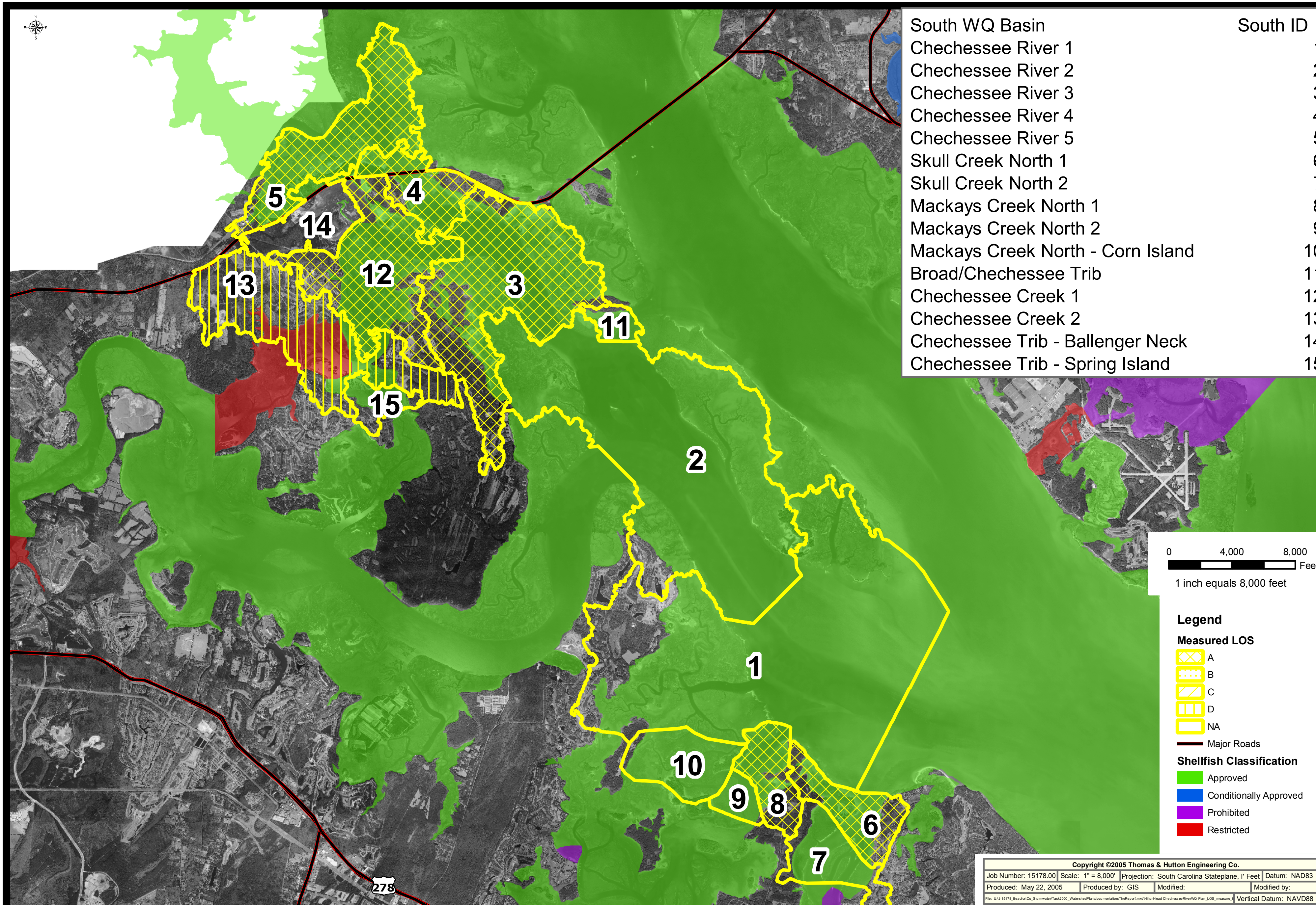
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
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File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-ChechesseeRiverWQ-Plan_LOS_measure_f			Vertical Datum: NAVD88

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Figure 15-26

Chechessee River Watershed
Shellfish Classification and Existing Level of Service

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Aerial Photography	Beaufort County	2002
Shellfish Classifications	SCDHEC	2004

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

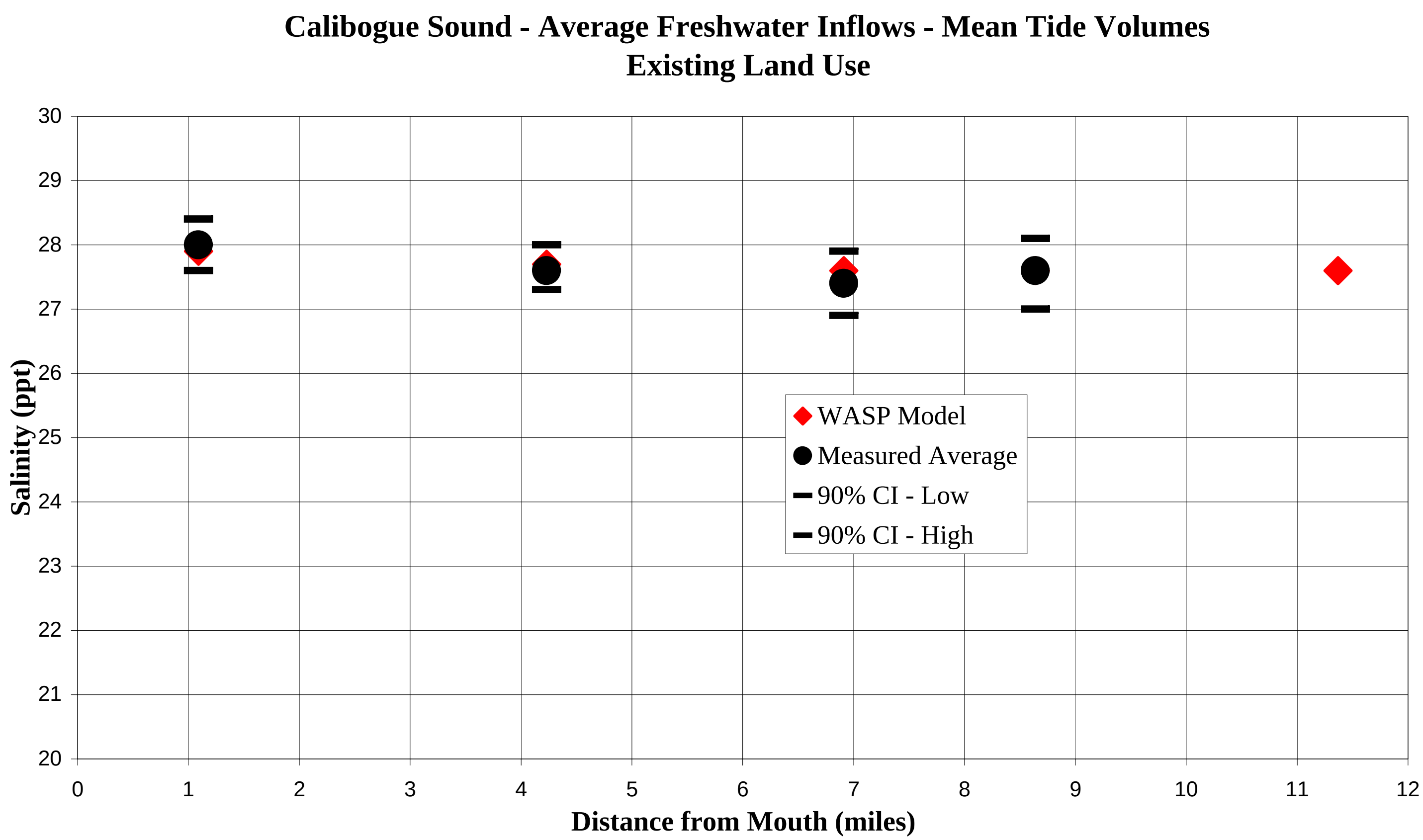


Figure 15-27 Comparison of WASP Model Results with Long-Term Monitoring Data in Calibogue Sound - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

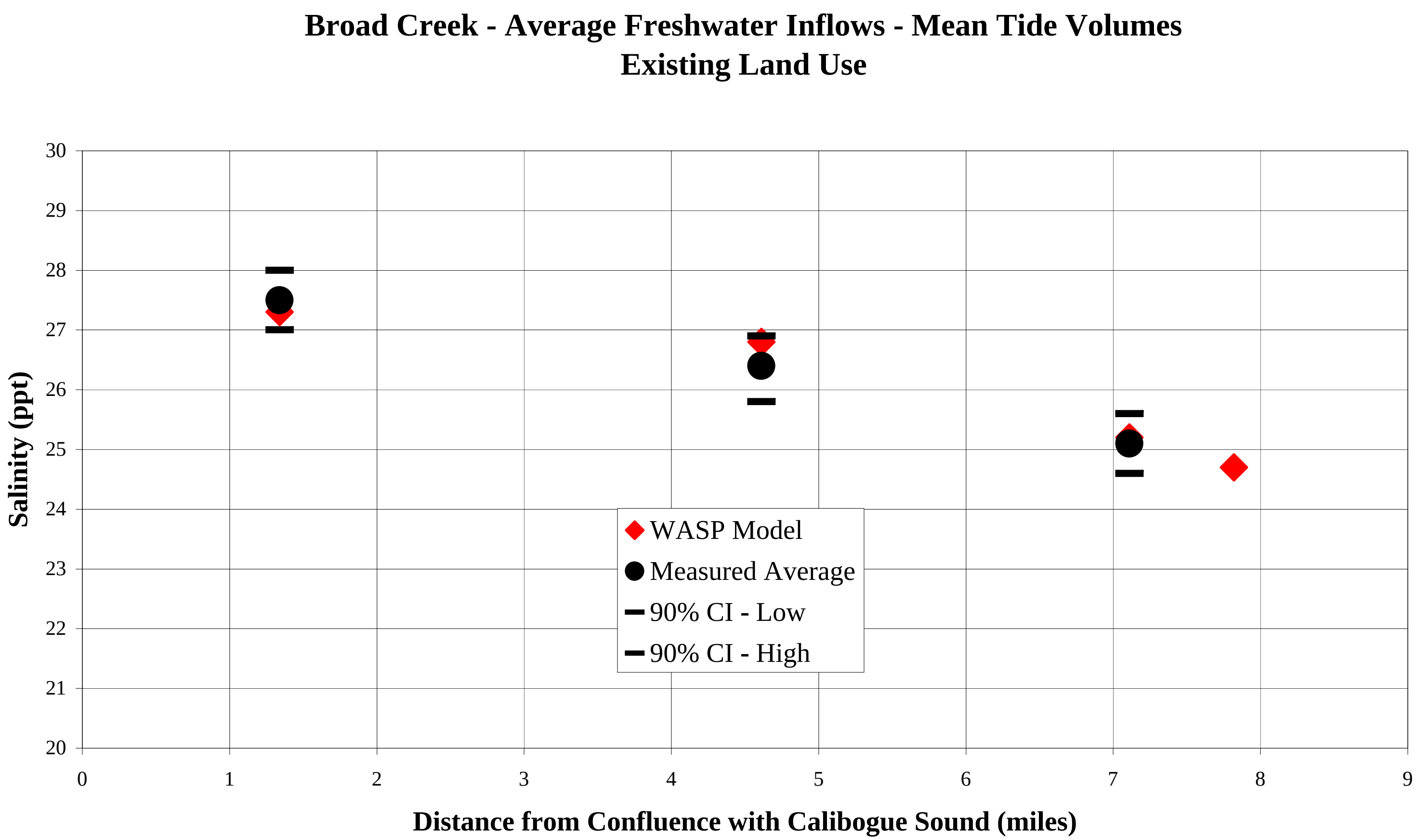


Figure 15-28 Comparison of WASP Model Results with Long-Term Monitoring Data in Broad Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

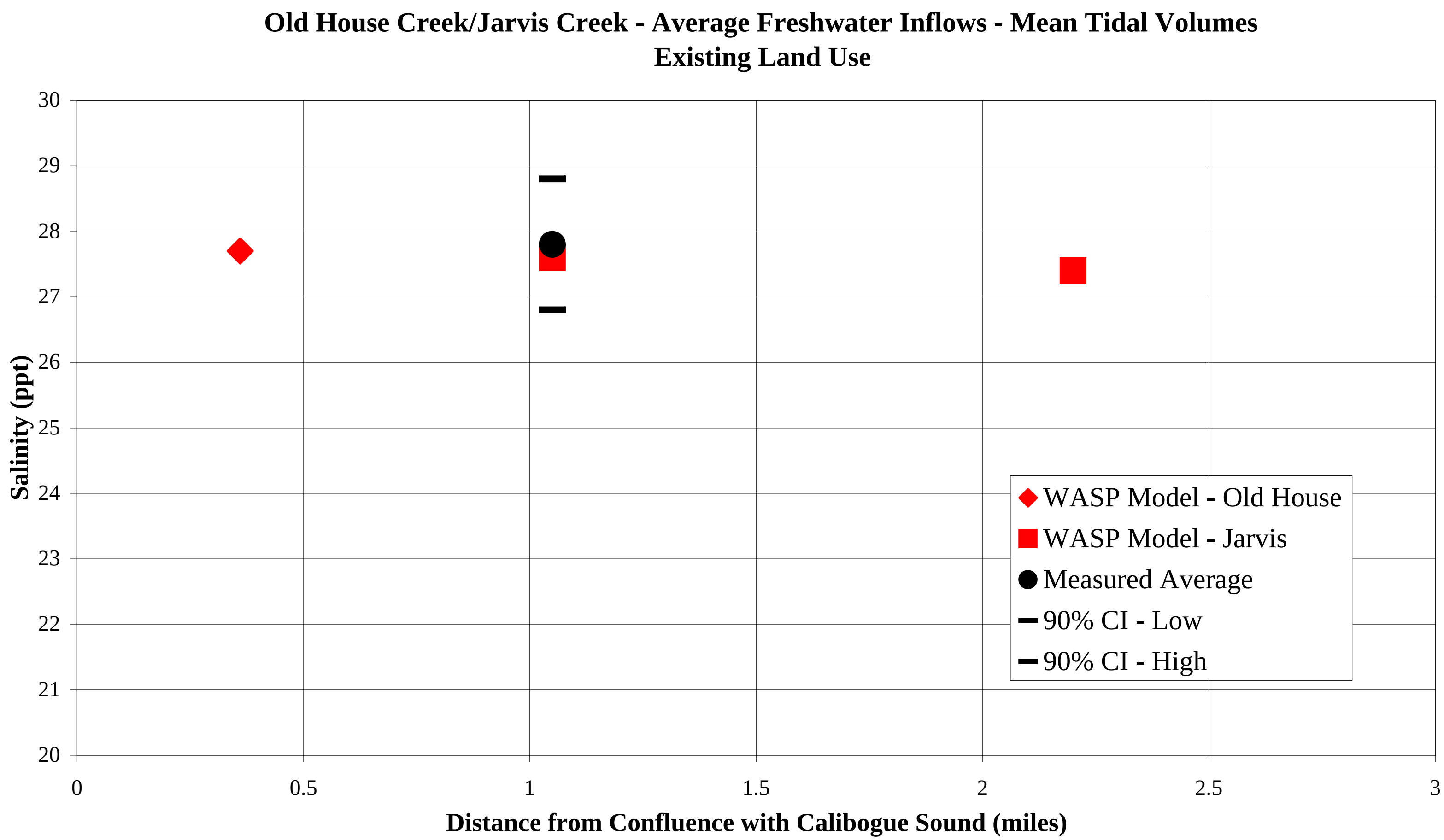


Figure 15-29 Comparison of WASP Model Results with Long-Term Monitoring Data in Old House and Jarvis Creeks - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

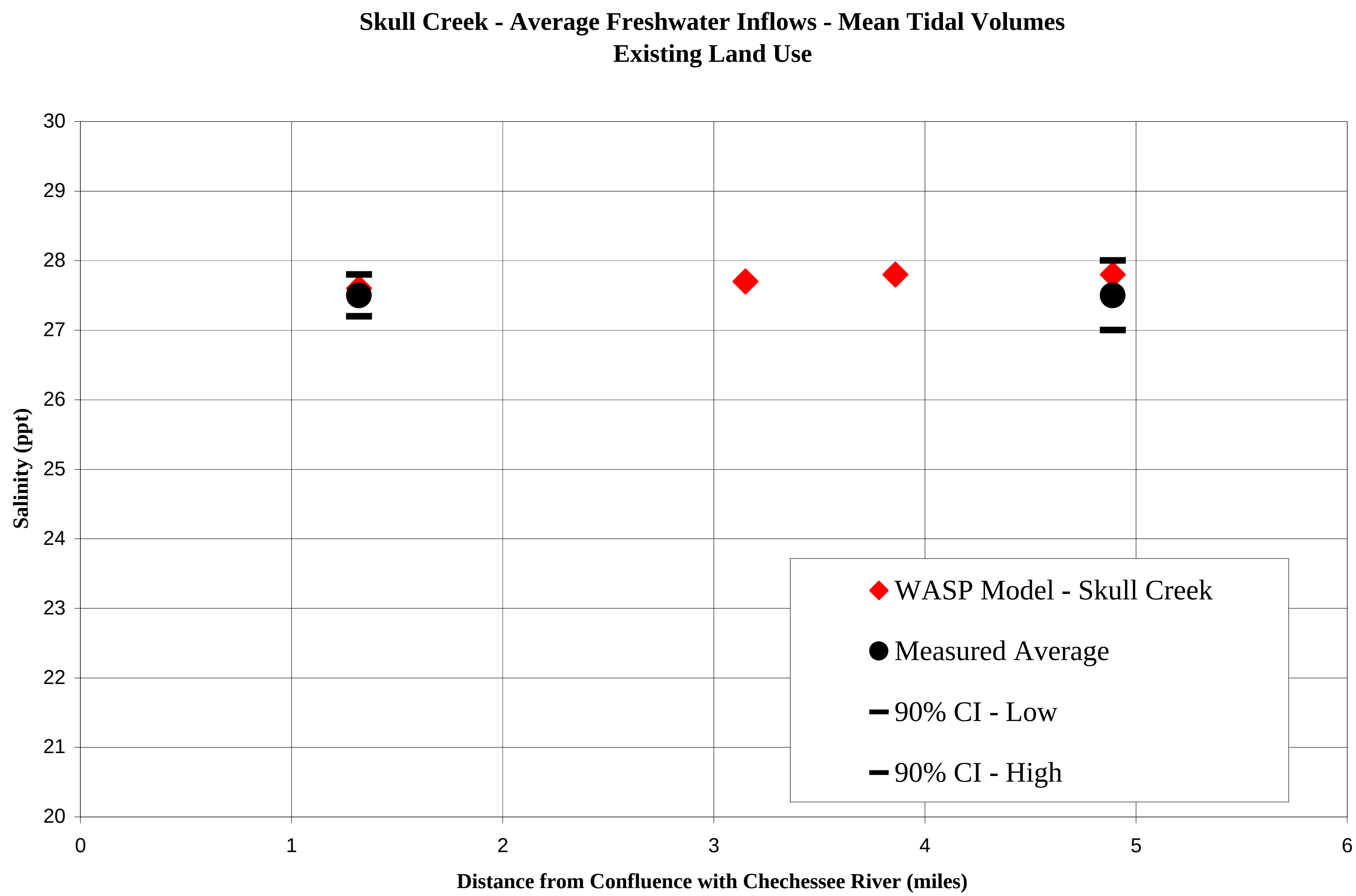


Figure 15-30 Comparison of WASP Model Results with Long-Term Monitoring Data in Skull Creek - Salinity

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

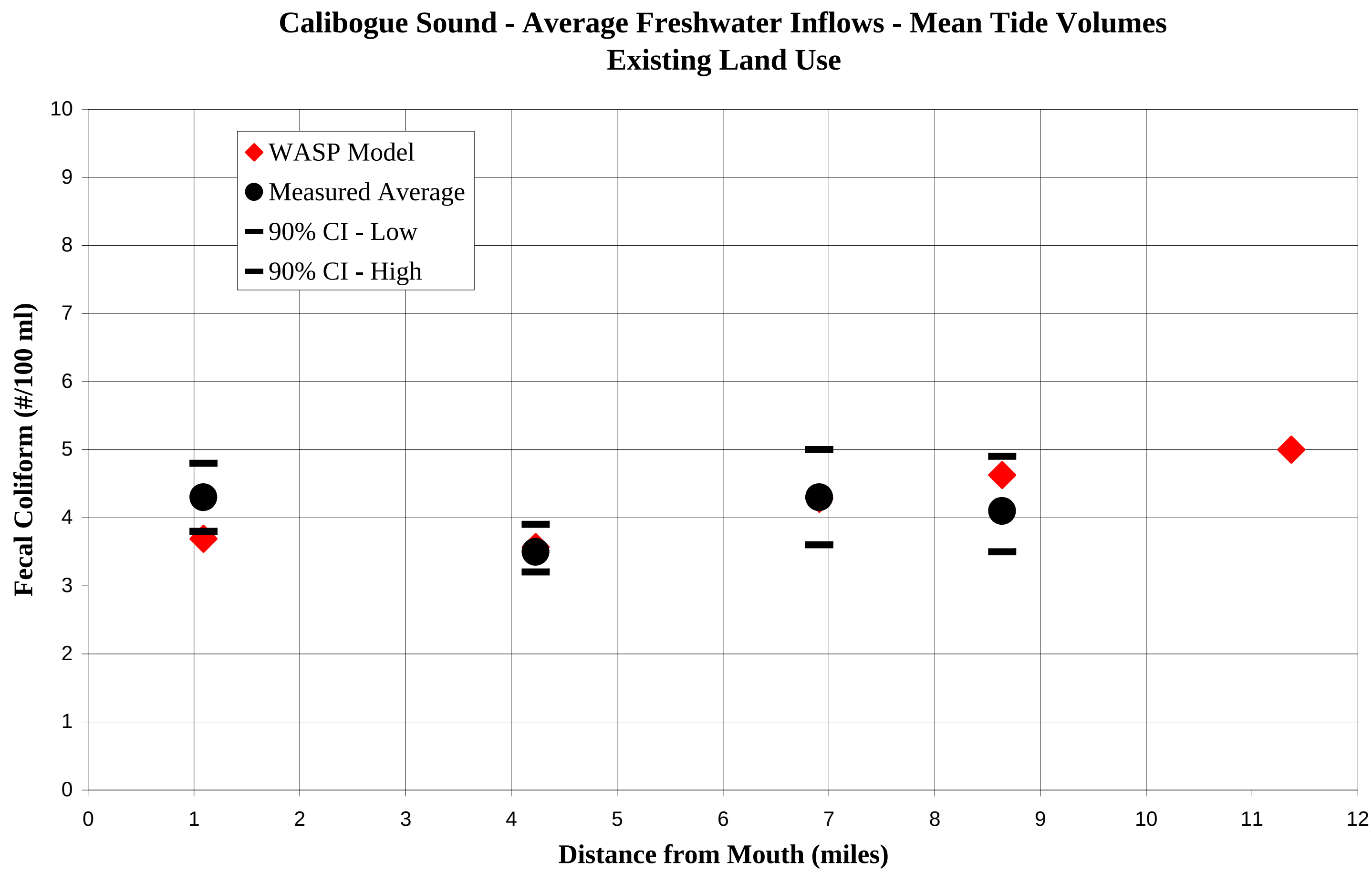


Figure 15-31 Comparison of WASP Model Results with Long-Term Monitoring Data in Calibogue Sound - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

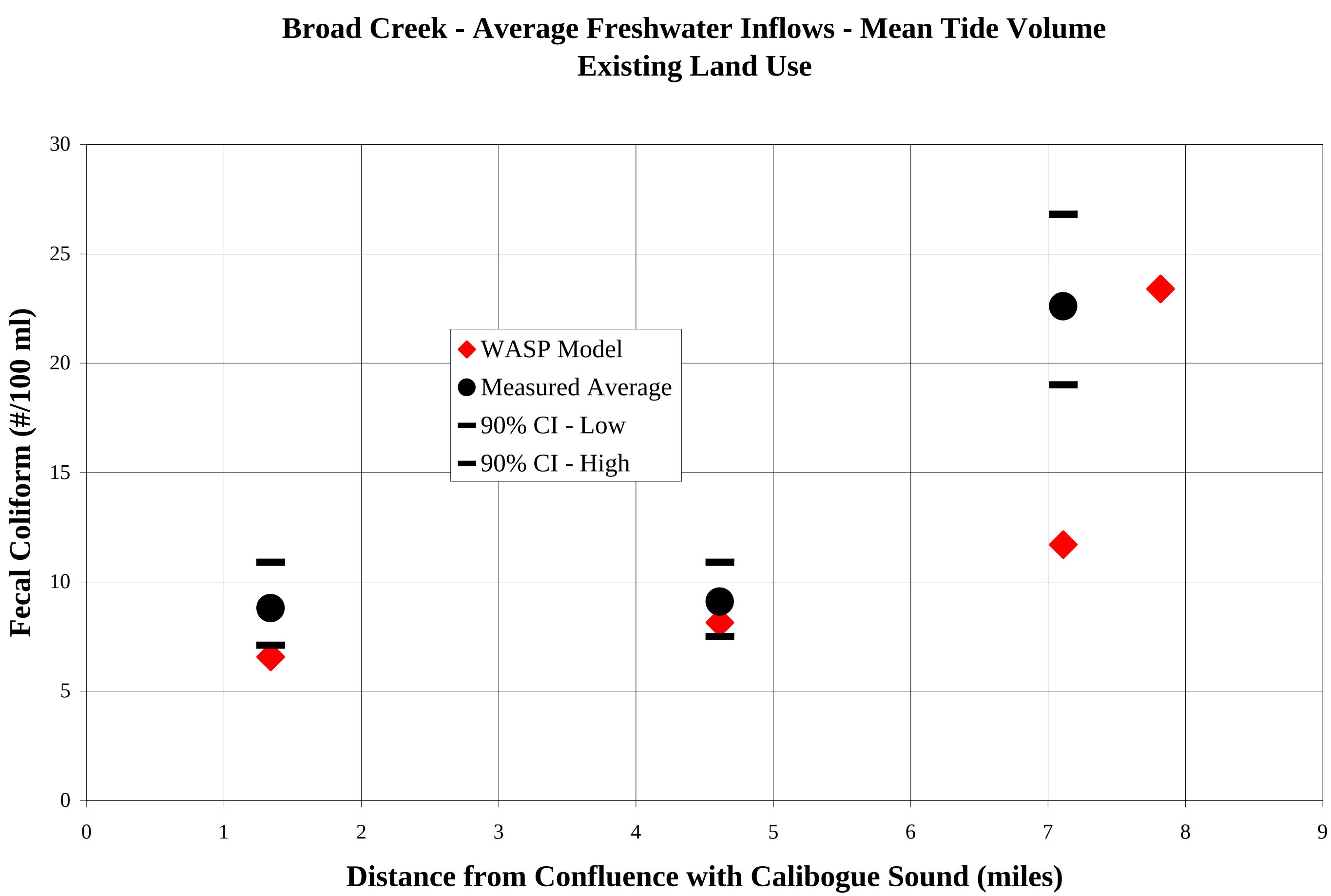


Figure 15-32 Comparison of WASP Model Results with Long-Term Monitoring Data in Broad Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

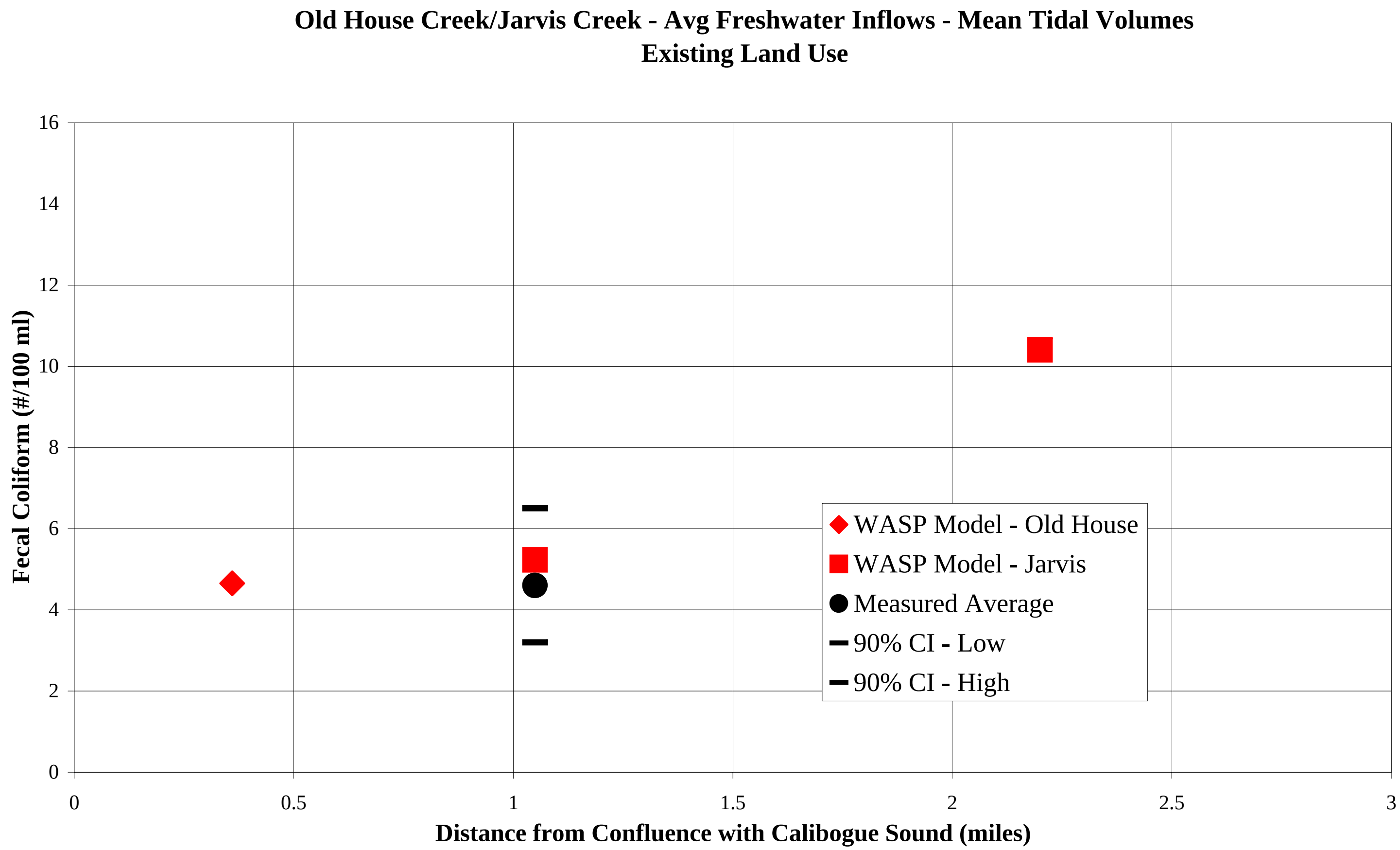


Figure 15-33 Comparison of WASP Model Results with Long-Term Monitoring Data in Old House and Jarvis Creeks - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.

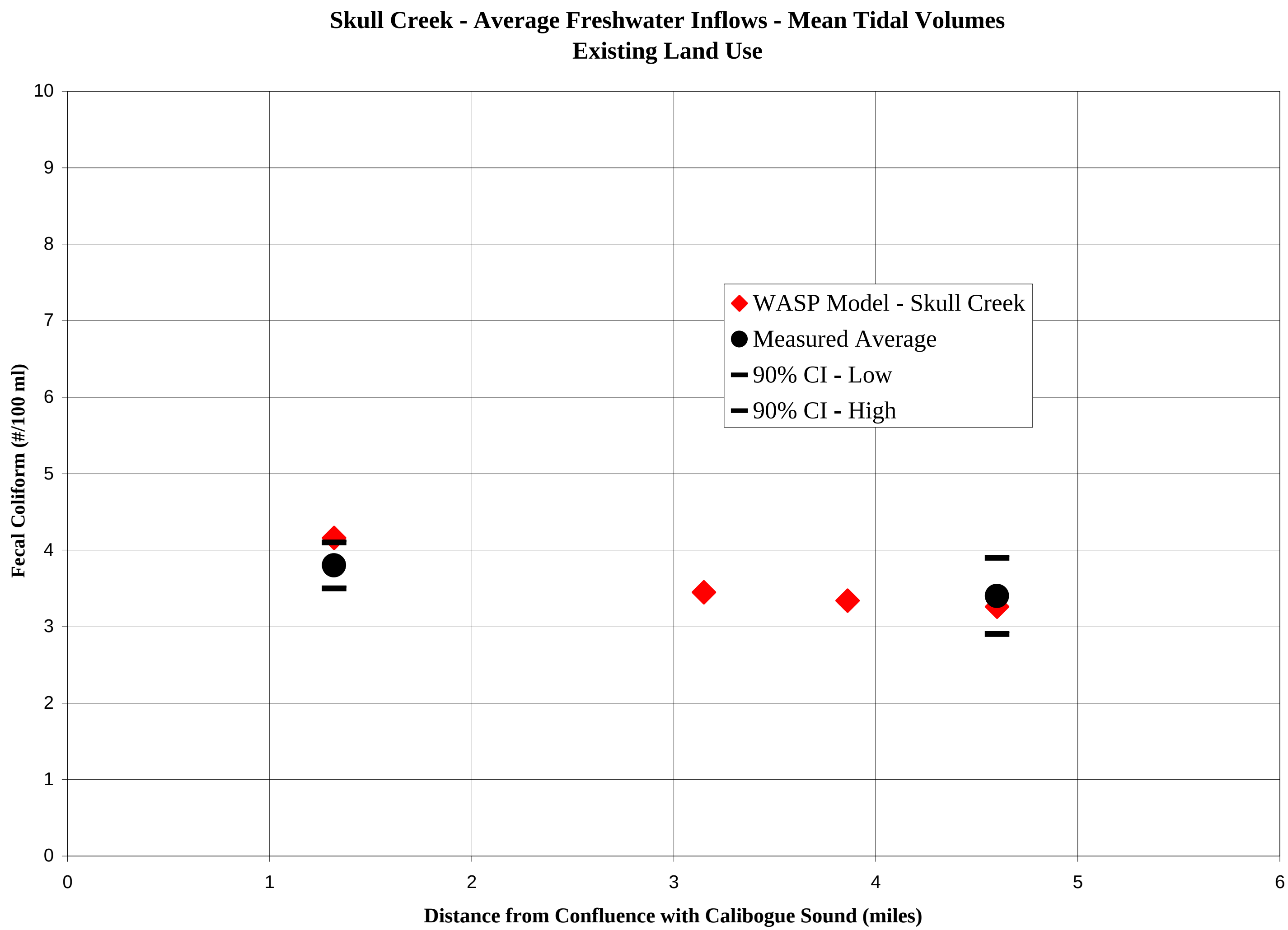
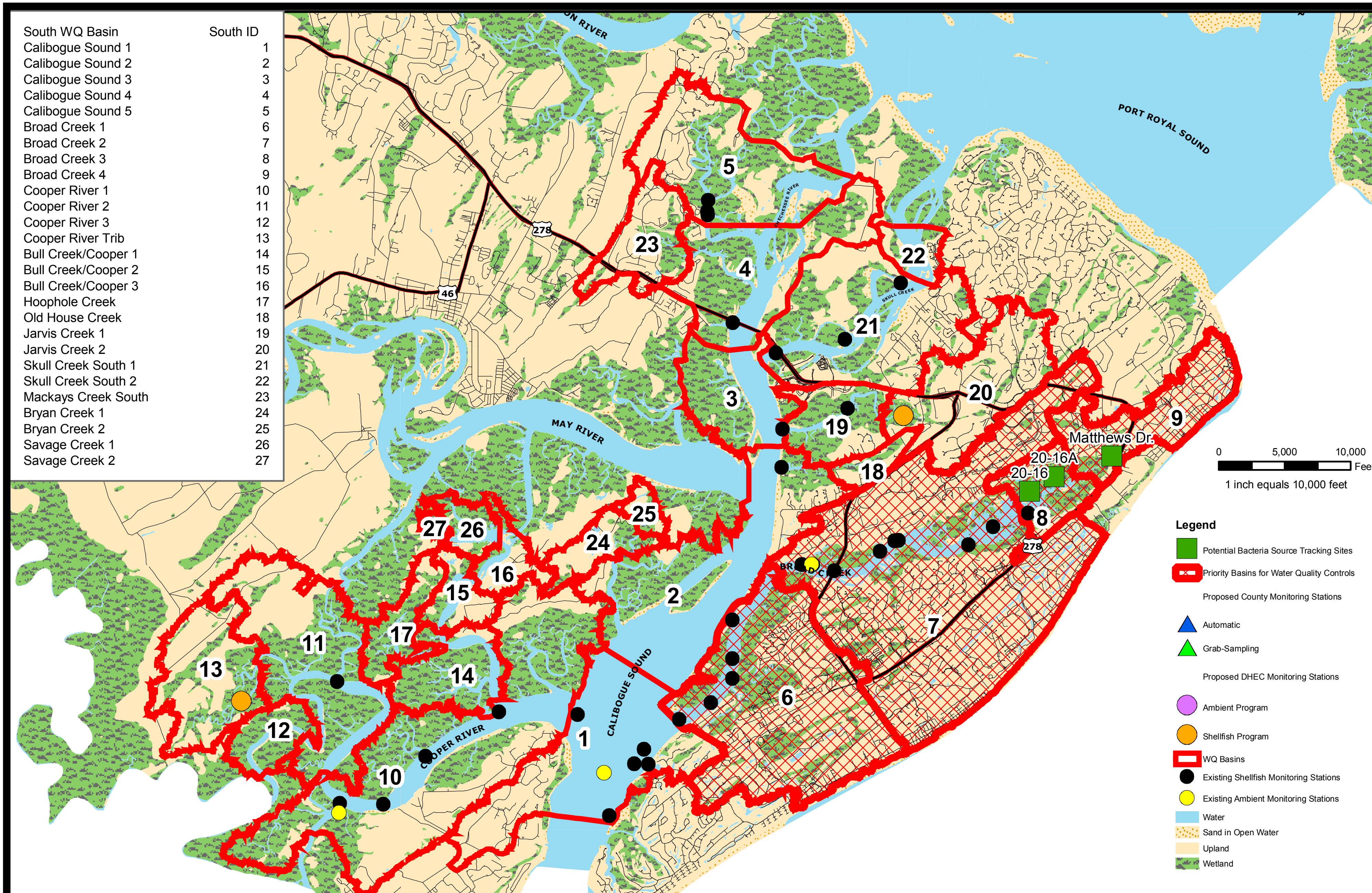


Figure 15-34 Comparison of WASP Model Results with Long-Term Monitoring Data in Skull Creek - Bacteria.

Note: 90% CI = 90% confidence interval for the measured mean based on statistical analysis of monitoring data.



South WQ Basin	South ID
Calibogue Sound 1	1
Calibogue Sound 2	2
Calibogue Sound 3	3
Calibogue Sound 4	4
Calibogue Sound 5	5
Broad Creek 1	6
Broad Creek 2	7
Broad Creek 3	8
Broad Creek 4	9
Cooper River 1	10
Cooper River 2	11
Cooper River 3	12
Cooper River Trib	13
Bull Creek/Cooper 1	14
Bull Creek/Cooper 2	15
Bull Creek/Cooper 3	16
Hoophole Creek	17
Old House Creek	18
Jarvis Creek 1	19
Jarvis Creek 2	20
Skull Creek South 1	21
Skull Creek South 2	22
Mackays Creek South	23
Bryan Creek 1	24
Bryan Creek 2	25
Savage Creek 1	26
Savage Creek 2	27

Job Number: 15178.00	Scale: 1" = 10,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-CalibogueWQ-Plan_figure15-36.mxd			Vertical Datum: NAVD88

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Figure 15-35

Calibogue Sound Watershed Water Quality Plan Elements

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Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Existing WQ Information	SC DHEC	2005

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
WQ Recommendations	T&H / CDM	2004

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Figure 15-36

Chechessee River Watershed Water Quality Plan Elements

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Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Existing WQ Information	SC DHEC	2005

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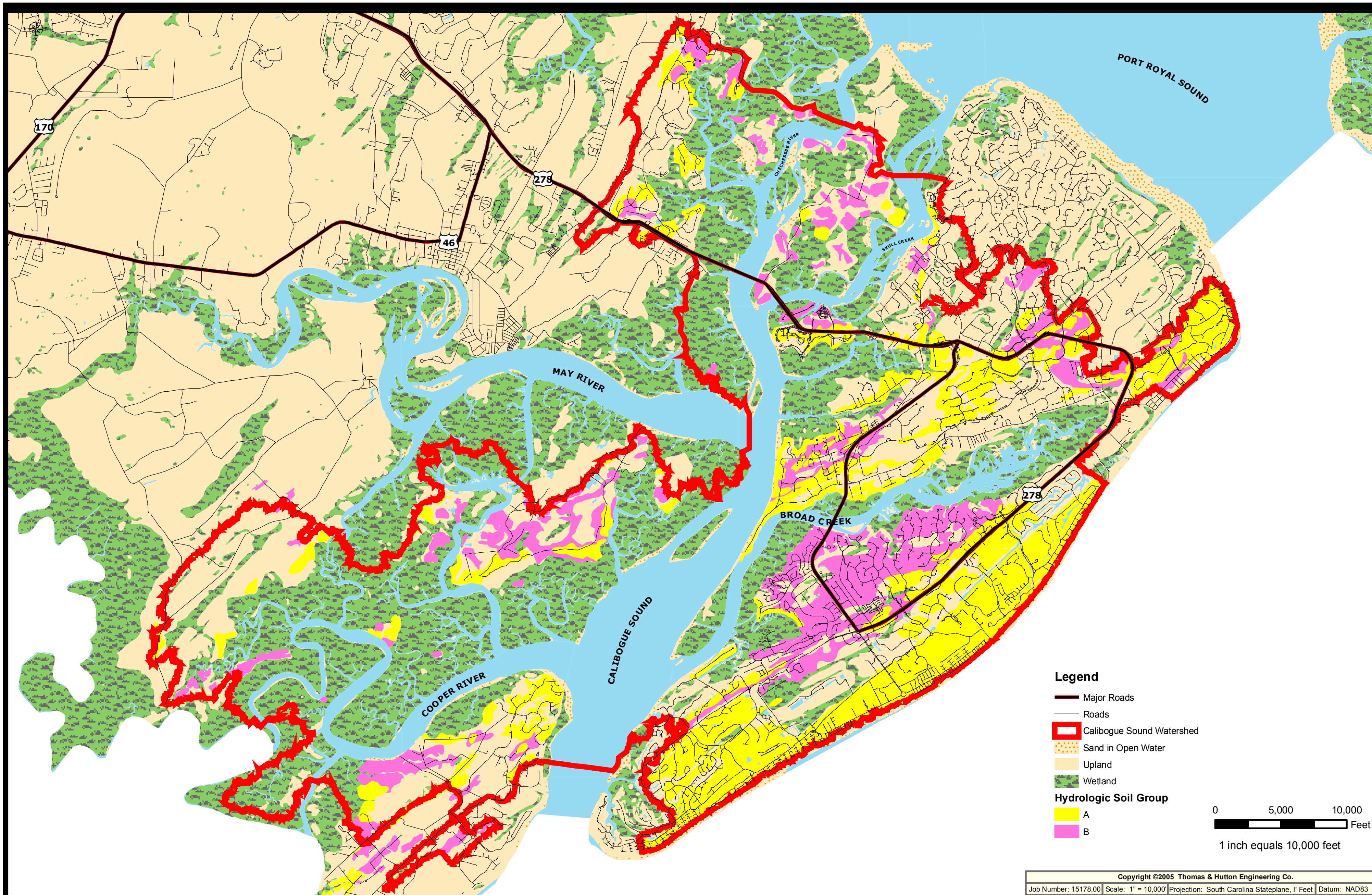
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
WQ Recommendations	T&H / CDM	2004



Job Number: 15178.00	Scale: 1" = 10,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-CalibogueSoundA-BSoils_figure15-38.mxd			Vertical Datum: NAVD88

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Calibogue Sound Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 15-37

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DATA	SOURCE	DATE
Roads	Beaufort County	2002
Land Use / Land Cover	USGS	
Soils	NRCS	

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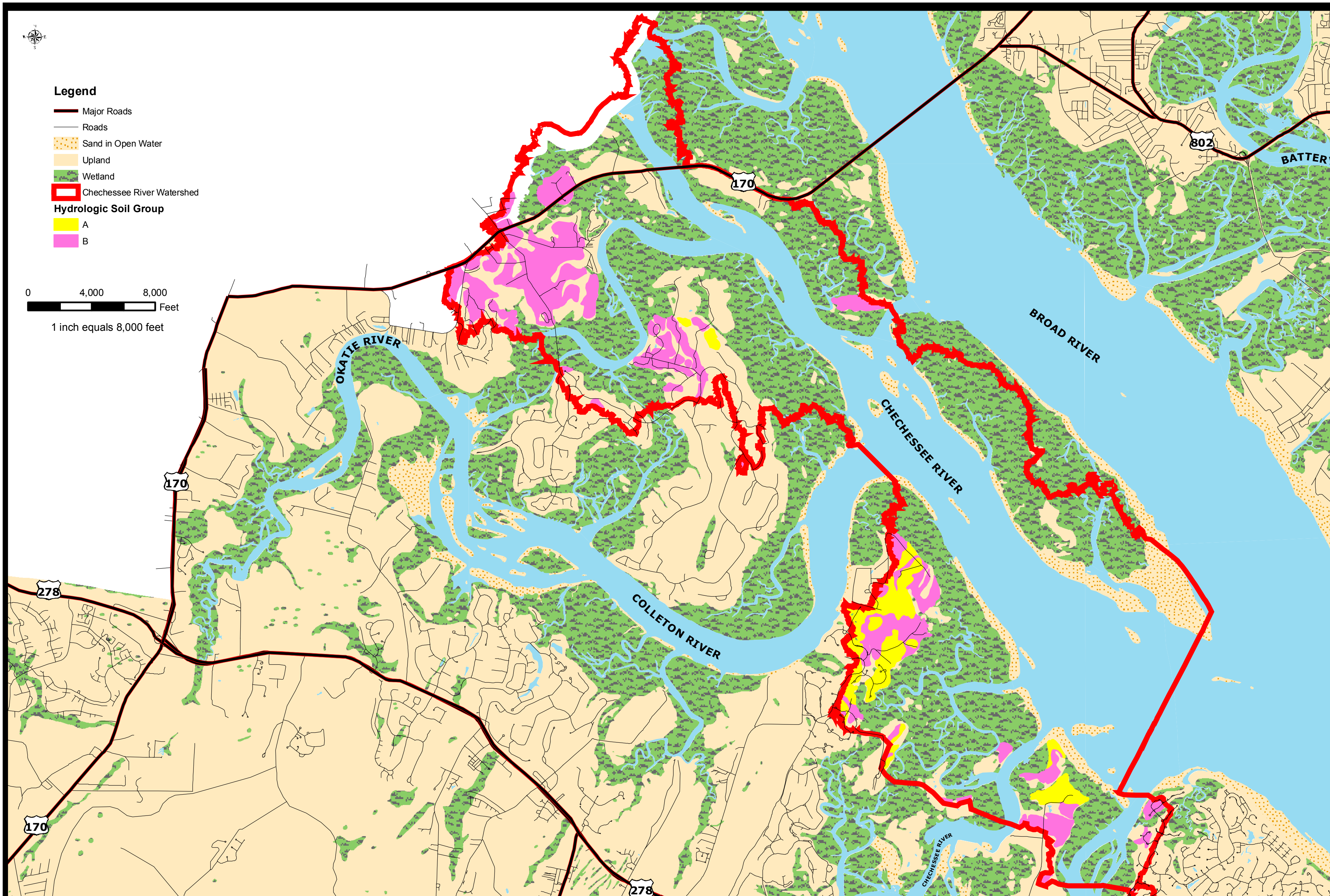
DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004



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Chechessee River Watershed
Potential Locations for Infiltration BMPs based on A and B Soils

Figure 15-38

CDM Camp Dresser & McKee Inc.

Job Number: 15178.00	Scale: 1" = 8,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\HiltonHead-ChechesseeRiverSoils_figure15-39.mxd			Vertical Datum: NAVD88

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DATA	SOURCE	DATE
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Land Use / Land Cover	USGS	
Soils	NRCS	

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DATA	SOURCE	DATE
Basins	T&H / CDM	2004
Subbasins	T&H / CDM	2004

Section 16
Recommended County Stormwater Management Plan

This section summarizes the recommended stormwater management plan for Beaufort County. Recommendation in this section is based primarily upon the findings presented in Sections 3 through 15 of the report. Section 16.1 describes the elements of the plan, and the planning level cost estimates for each element are presented in Section 16.2.

16.1 Recommended Watershed Management Plan

The recommended plan includes the following elements:

- Stormwater control regulations
- Primary stormwater management system (PSMS) enhancements
- Water quality controls for existing development
- Water quality monitoring
- Operations and maintenance (O&M) of the PSMS and secondary stormwater management systems
- Inventory of the secondary stormwater management system
- Additional and on-going study and analysis
- Public information

For each plan element, the discussion below identifies objectives and recommended activities.

16.1.1 Stormwater Control Regulations

Beaufort County ordinances require the control of the quantity and quality of stormwater discharges from new development. For both quantity and quality, the County requirements are more stringent than State requirements.

For water quantity, County ordinances require that the post-development peak flow from new development must be limited to the pre-development peak flow for design storms with return periods of 25 years or less (e.g., 2-year, 10-year and 25-year design storms). In contrast, the State requirements are limited to the 2-year and 10-year design storms only.

For water quality, the County has a Stormwater Manual for Best Management Practices (BMPs) that provides guidance in the selection of appropriate BMPs, and provides sizing and design criteria to enhance the effectiveness of the BMPs in removing stormwater pollutants. The selection and sizing is based on an "anti-degradation" goal, using total phosphorus and fecal coliform bacteria as "indicator" pollutants. The "anti-degradation" goals limit new development phosphorus loads to the load that would be produced by a development of 10 percent imperviousness with no BMPs. The 10 percent level was selected because several studies have suggested that adverse impacts begin in watersheds when imperviousness reaches 10 percent to 20 percent. The goals also limit new development bacteria loads to the load that would be produced by development of 5 percent imperviousness with no BMPs. The lower threshold of 5 percent was selected based on limited analysis of bacteria concentrations in Beaufort County receiving waters and associated imperviousness levels.

The results of the hydrologic and hydraulic analysis suggest that the existing ordinances are sufficient to control peak discharges from extreme storm events. Model results for future land use conditions, which accounted for the existing peak-shaving requirements by limiting future subbasin peak flows to the peak flow under existing land use conditions, indicated that there were few road crossings that flooded under future conditions that did not flood under existing conditions, and peak stages for future conditions were typically the same or only 0.1 foot higher.

Similarly, the water quality modeling analysis suggests that the BMP requirements for new development, in conjunction with land use planning that requires low density development in much of the County, are sufficient to control stormwater pollution loads. When the models were applied to future conditions, with wet detention pond BMPs for new development, the overall watershed loads for future conditions typically increased by 10 percent or less compared to existing loads (see **Table 16-1**). In addition, bacteria concentrations calculated for modeled receiving waters under future conditions meet an "anti-degradation" standard. This means that the existing and future "level of service" (ability to meet the bacteria water quality standards) is essentially the same (see **Table 16-2**).

Consequently, additional requirements for new development controls are not recommended. For water quantity control, reducing post-development peak flows to pre-development levels for the 100-year storm could reduce the size of some of the recommended stream crossing upgrades for evacuation routes, so that could be considered on a case-by-case basis.

The results of the modeling for the Colleton River watershed indicate that inflows from Jasper County may have an adverse impact on the water quality level of service in the Okatie River and Colleton River. This is due to the relatively high imperviousness of the projected future development, and the presumption that the new development in Jasper County will be served by less-efficient BMPs (extended-dry detention) which would be sufficient to meet State requirements. It would be desirable to discuss the issue with Jasper County to determine if lower density future development and/or more efficient BMPs could be required for those areas of Jasper County that are tributary to the Colleton River watershed.

16.1.2 PSMS Enhancements

As a result of the hydrologic and hydraulic analyses, a total of 119 locations for stream crossing overtoppings were identified. These results were developed by analyzing evacuation routes for the 100-year design storm, and analyzing all other roads for the 25-year design storm. Locations of the problem areas are presented in **Figure 16-1**.

The evaluation of solutions for road overtopping focused primarily on the upgrade of culverts at the stream crossings. Road overtopping is eliminated by increasing the conveyance capacity of the culverts. In some cases, the culvert upgrade was supplemented by raising the road, particularly in locations where the road elevation was at or near the design downstream boundary water elevation, which was defined as the mean annual high tide.

Efforts were made to identify locations for regional detention along the primary stormwater management system (PSMS). In general, potential regional sites were located in areas of existing wetlands, which would require the implementation of "off-line" detention facilities primarily excavated from upland areas outside of the existing wetlands. At the sites that were evaluated, the costs of constructing regional detention and purchasing the land for the facilities were much greater than any cost savings associated with eliminating or reducing the magnitude of the PSMS enhancements downstream.

It may be useful to look for detention storage sites in the secondary drainage system, particularly for systems that have several road overtopping problems along the PSMS. Though the storage may not solve the road overtopping problems, it may reduce the size of the culvert upgrades to the point that the detention storage is cost-effective. Another advantage of detention is the potential for water quality treatment.

The study of Hilton Head Island found that many problems identified in the 1990s have been eliminated because of extensive drainage improvements implemented since 1995 (based on a 1995 storm drainage study). In the current study described in this report, the stormwater system on the island was analyzed using recent LiDAR topography and the current stormwater system, for existing and future land use conditions. Several improvements are recommended for the island stormwater system.

16.1.3 Water Quality Controls for Existing Development

The water quality analysis identified a number of water quality basins in the County where treatment of runoff from existing development could improve the potential for meeting bacteria water quality standards. These areas are presented in **Figure 16-2**.

Again, efforts were made to identify locations for regional detention along the primary stormwater management system (PSMS) in these basins. In general, potential regional sites were located in areas of existing wetlands, which would require the implementation of "off-line" detention facilities primarily excavated from upland areas outside of the existing wetlands.

A total of 17 alternative sites were evaluated. The evaluation included a review of the sites with County staff, evaluation of potential wetlands impact based on the National Wetland Inventory (NWI), determination of site tributary area and existing land use, sizing of the pond permanent pool based on Beaufort County BMP standards, and evaluation of construction costs, land acquisition costs and benefits (annual bacteria load reduction). Based on the evaluation, 8 of the 17 sites are recommended as part of the master plan, and the locations of the proposed facilities are shown in Figure 16-2. The other sites were not recommended because they were not cost-effective relative to other potential methods of water quality control, as discussed below.

Other options for these areas would include the enhancement of existing stormwater controls, or retrofit of existing development with no stormwater controls. In the case of existing controls, there may be areas in which the stormwater controls are designed for water quantity (peak shaving) control only. These facilities can be enhanced to provide water quality benefits as well (e.g., add a permanent pool of water to dry detention facilities, convert dry facility to extended dry detention, modify permanent pool size or other design characteristics to enhance treatment). In areas with no controls, retrofit BMPs would be required, subject to availability of land area. Devices such as Stormceptors may be the most easily implemented retrofit, but they are not expected to be effective in removing bacteria or other pollutants that are primarily in the dissolved form. "Rain gardens" (bioretention) or more effective manufactured systems such as the Stormtreat device would be a more effective choice for bacteria removal.

16.1.4 Water Quality Monitoring

In general, a water quality monitoring program can serve a number of purposes in a stormwater management program. These could include:

- Establish baseline water quality
- Identify water quality trends
- Develop data to support water quality modeling

A monitoring program has been designed to achieve these purposes.

Figure 16-2 shows recommended monitoring stations, which were discussed in Sections 3 through 15 of this report. Sampling sites are discussed further in Section 16.2.4.

The plan considers that Beaufort County would sample the major tributary areas to the tidal creeks and rivers modeled in this study, and that the DHEC would conduct the sampling in the open water tidal areas. Discussion with DHEC staff indicates that DHEC is willing to consider the additional open water sampling, and has provided the County with the costs necessary to conduct this sampling for the County. This sampling will be discussed later in this section.

A total of 18 major tributary area sampling stations, to be monitored by the County, are recommended. Fourteen of the 18 stations are expected to be grab sampling stations, where samples will be taken monthly for most parameters (quarterly for metals). Parameters that will be sampled in the tributaries as part of the grab sampling program include the following:

- Fecal coliform bacteria
- Total suspended solids (TSS)
- Conductivity
- Salinity
- Water temperature
- Dissolved oxygen (DO)
- pH
- Turbidity
- Biochemical oxygen demand (BOD)
- Ammonia nitrogen
- Nitrite and nitrate nitrogen
- Total Kjeldahl nitrogen (TKN)
- Total phosphorus
- Chlorophyll-a
- Total organic carbon (TOC) quarterly
- Metals (cadmium, chromium, copper, iron, lead, manganese, mercury, nickel and zinc) quarterly

The other four stations would be automatic sampling stations, at which sampling will be activated during storm events so that stormwater runoff sampling can be reliably conducted. The four sites were selected to represent runoff quality from different urban land use types (e.g., industrial, residential/golf course). In general, the same parameters will be sampled. Measurements of rainfall, stage, velocity and flow rate will also be made at the automatic sampling stations.

Given that the effectiveness of BMPs in removing bacteria and other pollutants is a critical factor in the evaluation of bacteria loads under future conditions, the plan also recommends monitoring a minimum of two wet detention ponds. Ideally, the ponds would have a single inlet and outlet point to facilitate the monitoring. It is expected that automatic sampling would be required to reliably measure pond inflows and outflows during storm events, monitoring the same parameters described above. By monitoring ponds for 1-2 years, and then moving the monitoring to another pond, data can be collected at ponds with varying characteristics (e.g., with/without littoral shelf, residence time, depth, length:width ratio) over the 10-year planning period.

The plan also identifies stations that are recommended for addition to DHEC's existing ambient (nutrients, metals, chlorophyll-a) and shellfish (bacteria) monitoring programs. The ambient stations are located in water quality segments that the modeling showed would be most sensitive to controls of existing development and/or sensitive to BMP effectiveness. The bacteria stations are located in water quality segments to provide long-term data that can be used to validate or refine the water quality models developed in this study.

The recommended monitoring also includes some Bacteria Source Tracking (BST) studies. BST analysis is used to identify the sources of bacteria (e.g., human, animal) in samples. This sampling is recommended in locations where the monitored bacteria levels are higher than expected based on the water quality modeling.

The recommendations above are based on collecting data to validate values used in the planning level modeling, and to assess compliance with existing water quality standards (e.g., fecal coliform bacteria). Further monitoring may be desirable to assess issues such as habitat changes in the tributaries. This was an observation of a team that conducted an independent peer review of the SWMP report (SAIC, 2005).

Based on the uncertainties in the desired objectives and scope of this additional modeling, no immediate changes have been made to the base monitoring program. Further study and discussion should be conducted to clearly establish the objectives of this additional modeling, as well as program details (e.g., number of stations, method of sampling, guidelines for prioritizing potential sampling locations).

16.1.5 Operation and Maintenance

For the PSMS, operations and maintenance would primarily include maintenance of culverts and bridges, and maintenance of open channels. Activities at culverts and bridges would generally include removal of silt or other obstructions. For open channels, activities would also include silt and debris removal, and may also include periodic mowing.

The PSMS for this study (excluding the Town of Hilton Head Island) include 232 stream crossings and 141 miles of open channel. It should be noted that roughly 2/3 of the open channel consists of wetland channels that would likely see little or no maintenance, while the remaining 1/3 has more of a defined channel and would require maintenance.

16.1.6 Inventory of Secondary Stormwater Management System

This master plan study focused on the PSMS, and an inventory of the PSMS has been developed as part of the study. The PSMS includes the major drainage systems in the County, typically including any conveyance with a tributary area of 320 acres or more.

Future efforts should focus on the inventory of the secondary stormwater management system, which conveys the stormwater to the PSMS. In areas such as the City of Beaufort and the Town of Port Royal, drainage system maps are not current, and often show information that is not accurate. An accurate and complete inventory will be useful in evaluating the stormwater management system and evaluating the extent of required maintenance in those areas.

16.1.7 Additional and On-Going Study and Analysis

One of the major recommendations for further analysis is the development of an up-to-date structure GIS coverage with finished first-floor elevation data, and flood inundation mapping. The modeling in this study developed peak water elevation data for the various design storms evaluated, including the 100-year design storm. However, the current version of the ICPR model does not include the capability of automated flood inundation mapping. Furthermore, the County structure database is not current, and does not include finished first-floor elevations. Consequently, the model results and LiDAR topographic data may suggest that the ground surface near a structure is inundated, but there is no way to confirm whether or not the structure itself is flooded or not (e.g., is it elevated to prevent flooding). Specific activities would include updating and maintaining the structure database and GIS coverage, and to evaluate finished first-floor elevations, by building certificates or survey.

Additional recommendations based on peer review comments (SAIC, 2005) include additional evaluation of the water quality models, and consideration of additional sampling. The additional water quality modeling would consider validation (i.e., applying the model to data independent of the calibration data set) and sensitivity and/or uncertainty analysis. The sensitivity and uncertainty analyses would indicate how model results change as a result of changes in model input parameters (e.g., BMP efficiency, runoff concentrations), and which input parameters most affect the variability in model results in each water quality segment.

16.1.8 Public Information

Public information is another aspect of a comprehensive stormwater master plan. The most important function of public information is to get residents involved and make them aware of the links between their actions and the quality of the water bodies in the County.

There are a number of approaches that can be taken. Media campaigns (e.g., advertisements/public service announcements, direct mailings, newsletters) are one way of connecting with residents. Interactive training outreach activities can also be used as a follow-up on the media campaign. Examples of interactive training outreach could include:

- Workshops/meetings
- Stream walks, facility site visits
- Volunteer stream monitoring program
- Storm drain stenciling program
- Speakers Bureau for civic associations and other local meetings

This report does not recommend how Beaufort County should specifically approach this issue, but does recommend allocation resources for it.

16.2 Planning Level Costs for Plan Components

Conceptual costs have been estimated for each of the items discussed above. In some cases, such as the culvert upgrades, the cost is specified as a total cost in 2005 dollars. In contrast, other costs such as operations and maintenance are expressed as an annual cost.

16.2.1 Stormwater Control Regulations

No specific changes to stormwater regulations have been recommended. However, a cost of $100,000 has been estimated for the inspection of BMPs in the County.

16.2.2 PSMS Enhancements

The conceptual probable capital cost for the improvements was presented in the watershed sections of this report. The total cost was $22.9 million ($1.8 million for the Town of Hilton Head Island and $21.1 million for the rest of the County).

Further analysis has been done in order to prioritize the improvements based on the type of road and the depth of road overtopping for the design storm event. The following criteria were used to set priorities from 1 to 5, with 1 being the highest priority:

- Priority 1 – Road overtopping of 0.1 feet or more on evacuation routes (100-year design storm.

- Priority 2 – Road overtopping of 0.1 feet or more on non-evacuation routes (25-year storm) for major roads with no convenient alternative route.
- Priority 3 - Road overtopping of 0.1 feet or more on non-evacuation routes (25-year storm) for major roads with a convenient alternative route or a major neighborhood road with no alternative route.
- Priority 4 - Road overtopping of 0.1 feet or more on non-evacuation routes (25-year storm) for neighborhood roads with a convenient alternative route or minor neighborhood roads; with 100-year flooding greater than 0.5 feet OR 100-year road overflow velocity greater than 1 foot per second.
- Priority 5 - Road overtopping of 0.1 feet or more on non-evacuation routes (25-year storm) for neighborhood roads with a convenient alternative route or minor neighborhood roads (same as Priority 4); with 100-year flooding less than 0.5 feet AND 100-year road overflow velocity less than 1 foot per second.

In addition, the projects were classified by flooding depth as follows:

- Level A: Flood depth of more than 9 inches
- Level B: Flood depth of 6 to 9 inches
- Level C: Flood depth of 3 to 6 inches
- Level D: Flood depth of less than 3 inches

Consideration was also given to "public" versus "private" improvements, where "private" improvements would be in developments that would not be considered part of the "public" PSMS. This review indicated that the total projected cost for public projects is $15.3 million, and the projected cost of private projects is $7.9 million.

In subsequent sections, the discussion of PSMS improvements will focus on the public projects. Several of the private projects are located in areas such as the Parris Island Airfield, which is beyond the jurisdiction of the County and other jurisdictions, and others are in subdivisions and gated communities that are expected to address flooding issues internally.

Table 16-3 presents the projected cost of the public PSMS improvements by priority and flood depth category, and **Table 16-4** presents cumulative projected costs based on priority and overtopping category. For example, the value in Table 16-4 corresponding to priority 2 and flooding category B represents the cost of all projects having a priority of 1 or 2, and flooding category of A or B. This table may be useful in determining the phasing of the PSMS improvements.

The public projects identified through the analysis are listed in **Tables 16-5** and **16-6** for areas south and north of the Broad River, respectively. For each project, the tables list the watershed, hydrologic basin, jurisdiction, stream crossing name, priority, flood depth category, and projected cost. The projects in each table are arranged based on priority (highest priority listed first), and within each priority level, the projects are listed in order of flood depth category, with the greatest flooding listed first. In all, the total cost of projects south of the Broad River is $5.1 million, and the cost is $10.3 million for the projects north of the Broad River.

In general, the jurisdiction was determined based on the location of the hydrologic basin. In some cases, there is more than one jurisdiction associated with the project. For these projects, it is likely that both jurisdictions contribute stormwater discharges to the project location, so it is anticipated that the jurisdictions will share in the cost of the improvements.

16.2.3 Water Quality Controls for Existing Development

The water quality controls for existing development focuses on the implementation of regional detention facilities strategically located in areas with existing development that is not controlled by BMPs. The conceptual probable capital cost for the improvements was presented in the watershed sections of this report. The total cost was $14.4 million, which includes the construction cost plus the land acquisition cost.

Table 16-7 summarizes the analysis for the regional detention pond sites. The recommended pond sites are listed by watershed, in order of overall effectiveness. Results indicated that the implementation of the regional facilities in the Beaufort River watershed would improve the level of service in several water quality basins. In the Colleton River and Morgan River watersheds, the geomean bacteria concentrations calculated by the model were reduced slightly, but did not result in an improved LOS in any water quality segments.

16.2.4 Water Quality Monitoring

As outlined in Section 16.1, the monitoring program includes tributary and BMP monitoring that would be conducted by the County, plus open water monitoring that would be conducted by DHEC. For Beaufort County, the monitoring would include 14 grab sample (ambient) station, plus 8 automatic samplings (4 on watershed tributary areas, 4 on BMP inflow and outflow points). The DHEC sampling would include ambient sampling at 12 stations, with nutrient and metals data collected at four of the stations, and bacteria data collected at all 12 stations. Bacterial source tracking (BST) monitoring is also recommended, and 5 stations have been identified.

Information about the recommended tributary stations is presented in **Table 16-8**. For each location, the table lists the watershed, hydrologic basin, sampling method, purpose of data collection, and tributary area characteristics.

As shown in the table, four stations are recommended for automatic sampling. These locations typically have one dominant land use type in the tributary area, and a high level of existing urbanization (small future change in percent imperviousness and percent urban area). Storm event water quality data collected at these stations can be compared to the event mean concentration (EMC) values used in the watershed water quality modeling, to either validate the values used or to refine those values if the local results are substantially different than the EMC values used. This sampling should also satisfy any potential NPDES Phase II requirements for single land use storm event sampling.

Eight of the stations are recommended for grab sampling to evaluate existing water quality. These are located in areas where water quality controls for existing development could facilitate an improvement in the water quality level of service (LOS). Consequently, comparison of the water quality data across stations may be useful in identifying areas where retrofits of existing development would be most effective. Note also that many of the stations are located downstream of potential regional BMP sites. If the BMP is constructed, the data collected at the downstream station would provide some insight into how the BMP is affecting water quality.

Six of the stations are recommended for grab sampling to evaluate water quality trends. Generally, these are located in areas where the percent of urban land and percent imperviousness is expected to increase dramatically in the future. Sampling data collected at these stations over a long period of time can be used to evaluate how water quality has changed, and is changing, as development occurs upstream.

Recent experience suggests that the County ambient station monitoring is expected to cost $5,000/year per station for the sample collection and laboratory analysis, and that the automatic sampling stations are expected to cost $25,000/year for the first year, which would include the purchase and installation of the equipment ($10,000) plus station maintenance, sample pickup and transport to the laboratory, and laboratory analysis. For 14 ambient and 8 automatic stations (year 1), the annual cost would be $270,000 per year. Addition cost would be incurred to entering the data into a database, analyzing the data, and presenting data summaries/reports. Consequently, the annual estimate is increased to $300,000 per year for year 1. Subsequent years would have a lower cost, though in the future, some of the automatic samplers may be re-located to other locations or need to be replaced.

The locations of the recommended open water sampling stations are presented in **Table 16-9**. At four of the stations, classified “ambient” stations, monthly grab sampling will collect data on bacteria, nutrients and metals. These are located in water quality basins that are expected to be sensitive to the implementation of water quality controls, based on the water quality model sensitivity analysis. Data collected at these stations over time can be used to see whether any obvious trends in improved or degraded water quality are apparent. At the other eight stations, classified “shellfish” stations, bacteria and salinity data will be collected. The objective is to use the collected data for comparison to the water quality model results, to determine if the model parameters provided a reasonable simulation of bacteria conditions, or whether the model should be refined with adjusted mixing and first-order loss parameter values.

DHEC has provided cost estimates for the open water sampling (Berry, 2005). Based on the information provided, the laboratory cost estimates are $2,500 per station for the nutrient/metals/bacteria stations, and $500 per station for the stations where only bacteria data are collected. Using these values, the overall cost to the County for the DHEC sampling would be $14,000. This estimate may be low because it did not include the laboratory costs for several parameters (chlorophyll-a, TKN) and it is not clear whether addition costs would be charged for sample collection and transport.

Table 16-10 lists the location of potential stations for the BST analyses. These stations have been located where existing monitoring results show bacteria concentrations that are higher than expected based on the water quality modeling of the watersheds and tidal rivers. The table lists the locations of the existing stations (DHEC or Town of Hilton Head Island stations) where the high bacteria values have been observed.

Cost estimates for the BST analyses presume that the approach will be designed to identify the sources of the bacteria. In general, BST methods can be described as "library" or "non-library". In the "library" approach, fecal samples of humans, birds, pets and wildlife are collected and analyzed, and a "library" of characteristics for each particular species is established. Samples are then analyzed against this library to determine the relative sources of bacteria in the samples. In contrast, the "non-library" approach would be capable of determining human versus non-human source of bacteria, but not the species that have contributed.

Personal communication (Falco, 2005) suggests that planning level costs for creating the library is $50,000, and monthly sampling and laboratory analysis would be $10,000 per year per station. On the basis of 5 stations, the total cost would be $50,000 per year, assuming that the library is established in the first year and sampling begins in the second year.

The annual tributary and BMP sampling costs may vary depending upon several factors:

- Number of samples per year (expected costs include 12 events per year for automatic samplers; for tributary stations, monthly grab samples for bacteria and nutrients, quarterly sampling for metals).

- Number of water quality constituents that are measured

Thus, within a $300,000 per year framework, there is flexibility to adjust the number of stations, number of samples and number of constituents measured, or to reduce the overall monitoring costs if necessary.

As discussed earlier, the issue of additional monitoring beyond the base recommended program should be evaluated further. This additional analysis is discussed in Section 16.2.7 of this report.

16.2.5 Operation and Maintenance

The consideration of operation and maintenance costs for this study focused on the maintenance of the PSMS (excluding the Town of Hilton Head Island). Additional costs will be incurred for maintenance of the primary system in the Town of Hilton Head Island, and secondary system maintenance throughout the County.

Previous studies have used a value of $2 per linear foot as a unit cost for open channel maintenance, and $1500 per stream crossing as the unit cost for stream crossing maintenance. When the unit cost for stream crossing maintenance is applied to the 232 crossings that are part of the primary stormwater system (excluding the Town of Hilton Head Island), the conceptual maintenance cost is about $350,000 per year. The channel maintenance cost was calculated based on the observation that roughly 2/3 of the open channel conveyance in the primary stormwater system is actually wetlands with no defined channel, and 1/3 of the open channel was more representative of the channel type that would be maintained at an annual cost of $2 per foot. Consequently, the unit cost of $2 per foot was applied to 1/3 of the 141 miles of open channel in the primary system, to yield a conceptual channel maintenance cost of about $500,000 per year. The sum of the channel and crossing maintenance is $850,000 per year.

Additional maintenance costs have been established by staff from Beaufort County and the Town of Hilton Head Island. County staff has estimated that the cost of maintaining the secondary system would be $2.0 million per year. Town of Hilton Head Island staff has estimated a cost of $300,000 per year for maintenance.

Consequently, the total estimated cost for operation and maintenance of the entire stormwater system (PSMS plus secondary) is $3.2 million per year.

16.2.6 Inventory of Secondary Stormwater Management System

Experience with inventory data collection on other projects suggests that typical costs range from $10,000 to $20,000 per square mile, depending upon the level of development. More highly developed areas would have more features to inventory, and thus would have a higher cost. These values were used to develop cost estimates for inventory of each jurisdiction. The resulting cost estimate is a total of $3.1 million, distributed as follows:

- City of Beaufort: $ 120,000
- Town of Bluffton: $ 450,000
- Town of Hilton Head Island: $ 400,000
- Town of Port Royal: $ 120,000

- Unincorporated County: $2,000,000

These costs include the data collection and entry into a database and GIS.

16.2.7 Additional and On-Going Study and Analysis

The major activity included in this category is the development of inundated area and evaluation of structural finished floor elevations. For this task, a projected cost of $300,000 was developed, based on the following tasks:

- Develop software to automate inundation mapping from ICPR hydraulic model output - $40,000
- Inundation analysis - $150,000
- Update GIS coverage of structures in the inundated area (overlay inundated area with aerial photographs) - $25,000
- Obtain first-floor elevations for structures in the inundated area - $50,000
- Engineering Analysis - $20,000

Another on-going activity to consider is the update of the models developed for this study. An annual cost of $50,000 per year has been estimated for this activity. It is not clear whether this should be done annually, or periodically in conjunction with updates to land use databases or other databases. Regardless, data required for model update such as land use and PSMS upgrades should be compiled as they occur to facilitate the model updates.

To further evaluate the water quality models, additional studies such as model validation, and sensitivity and uncertainty analysis, is recommended. Model validation would require that the models are applied to a second data set, independent of the data used for calibration (1990s bacteria data). Sensitivity and uncertainty analysis would involve applying the model with changes to the input parameter values, and evaluating how the change in the input value affects the model output (expected geomean bacteria concentration in the receiving waters).

For model validation, data are now available for the years 2000 through 2004, and these data could be used for the validation. Analysis of the more recent data would yield geomean and 90th percentile values for a second, independent time period for comparison with the WMM/WASP output. The data could also be combined with the 1990s data to re-evaluate the level of service criteria (i.e., refine the values marking threshold between various levels of services).

The sensitivity and uncertainty analysis should focus on model input values for WMM (loads) and SWMM/WASP models (receiving water impacts). Input parameters for WMM could include the following:

- BMP efficiency
- Runoff concentrations (EMCs)
- Runoff coefficients for pervious and impervious land
- Septic tank failure rate
- Ratio of failing septic tank load to surface runoff load for various land uses

SWMM and WASP input parameters could include the following:

- Boundary concentration
- Tidal mixing coefficient
- First-order bacteria loss rate
- Average tidal range

Sensitivity analysis would indicate how model results (expected geomean bacteria concentrations) would change based on change in one of the input parameter values. Uncertainty analysis would help to determine which parameter(s) are most important to predicted values in each water quality segment. For example, results would likely show that the boundary concentration is a major factor in determining concentrations in segments near the boundary, but have little or no impact for segments further from the boundary.

The actual cost of the additional modeling and data analysis would depend upon the actual scope. Looking at fecal coliform only, the cost of performing the tasks listed above is expected to cost at least $50,000. Evaluating other constituents (e.g., total N, total P, TSS) would require additional cost.

If additional monitoring beyond the base program is desired, the scope of this monitoring should be studied. This study would identify locations and frequency of sampling for methods such as continuous monitoring with probes, and benthic/sediment sampling. Considerations should include potential sampling sites (e.g., outlet of identified hydrologic basins), criteria for prioritizing the sites (e.g., intensity of development, LOS of water quality basin), and frequency of sampling. The study should also weigh the costs of the additional sampling against the

Again, it is difficult to evaluate a cost for such a study. For planning purposes, a value of $25,000 will be used.

16.2.8 Public Information

It is very difficult to estimate a cost for public information without identifying specific activities that will be part of the public information program. For purposes of this report, a cost of $100,000 per year is suggested. This is roughly $1 per billing unit. It is possible that the cost in initial years would be higher, and could be less in subsequent years as less information and education is required.

16.3 Implementation of the Plan Components

The implementation of the master plan will depend upon the costs required to implement the recommendations, as compared to the available funds being generated by the Storm Water Utility. As discussed in Section 16.2, the plan includes some activities that will be done annually (e.g., maintenance, monitoring), and other activities that would be one-time expenses (e.g., PSMS enhancements, regional detention land cost and construction).

Table 16-11 is an example of how the plan could be implemented over the first ten years. For each plan element, the table shows the level to which each element would be implemented, with associated cost. In the case of one-time expenses, the total cost has been divided by 10 years to yield an equivalent annual cost. The annual cost for the one-time expense would actually be higher if interest was considered. All of the annual costs are summed to calculate the total annual cost, which is compared to the amount of revenue that the utility would generate at a base rate of $40 per year, based on the tiered rate structure recommended by CDM (CDM, 2005) and May 2005 projections.

The logic behind the 10-year cost estimate and the distribution between jurisdictions is discussed below:

- Stormwater control regulations: the estimated annual cost is split among all jurisdictions based upon the jurisdiction's share of the total anticipated revenue.

- PSMS enhancements: The total cost is based on the presumption that the locations with priority 1, 2 or 3 AND flood depth category A or B (6 inches or more of flooding) will be designed and constructed. If so, the total cost would be $7.9 million. Priority 2 and priority 3 projects costs were assigned to the jurisdiction or jurisdictions where the project is located. For priority 1 (evacuation route), a different allocation approach was taken. For priority 1 projects south of the Broad River, the cost of each evacuation route project was shared between the unincorporated County, the Town of Bluffton and the Town of Hilton Head Island. The cost was split based on the relative revenues of the towns and the portion of unincorporated County revenue collected from properties south of the Broad River. Similarly, priority 1 projects north of the Broad River were allocated between the City of Beaufort, Town of Port Royal and unincorporated County.

- Water quality controls for existing development: It is presumed that land purchase at the regional sites (before the land is developed) is a high priority, and therefore all sites are purchased in the 10-year period. Three of the eight regional detention facilities (the most effective) are constructed. The cost is split among all jurisdictions based upon the jurisdiction's share of total anticipated revenue, because water quality is considered to be a Countywide issue.

- Water quality monitoring: The annual costs of the base recommended sampling program are included, and split among all jurisdictions based upon the jurisdiction's share of total anticipated revenue, because water quality is considered to be a Countywide issue. One-time costs related to monitoring (automatic sampler purchase and installation, creation of BST "library") are addressed in another part of the table.

- Annual maintenance: The annual costs of PSMS maintenance is allocated between the City of Beaufort, Town of Bluffton, Town of Port Royal and unincorporated County based on the number of hydrologic model subbasins in each of those jurisdictions. The secondary system maintenance for those same jurisdictions was distributed based upon the jurisdiction's share of total anticipated revenue. For the Town of Hilton Head Island, the anticipated cost of maintenance provided by Town staff was entered as secondary system maintenance.

- Inventory of secondary stormwater management system: The values in the table reflect estimates by jurisdiction based on the land area and extent of development, and presumes that this will be completed during the 10-year time frame.

- Additional and ongoing study and analysis: All of these costs were split among all jurisdictions based upon the jurisdiction's share of total anticipated revenue, and presumes the one-time tasks will be completed in the 10-year time frame.

- Public information: The annual cost was split among all jurisdictions based upon the jurisdiction's share of total anticipated revenues.

- Bonded debt service: The Town of Hilton Head Island has implemented a number of stormwater management system improvements that were recommended in a previous master plan study (T&H, 1995), and financed these improvements with bonds. The annual debt service cost of $1.2 million is included here for the Town.

- Utility administration: The administrative cost of the program is calculated as 8 percent of the total cost, again split among all jurisdictions based upon the jurisdiction's share of total anticipated revenue.

The annual total cost for all of the activities described above, in a 10-year time frame, is $7.5 million per year.

As shown in the table, the projected revenue of the utility (provided that the recommended tiered rate structure is applied with a base rate of $40 per year) is $4.8 million. This is about two-thirds of the required revenue based on the example 10-year planning horizon outlined in the table.

Table 16-12 shows an example 10-year planning horizon that has costs roughly equivalent to the anticipated revenue. Changes incorporated into the reduced-cost example include the following:

- Reduce the PSMS project list to priority 1, flood categories A and B, and priority 2, flood category A
- Reduce the number of regional sites purchased and regional facilities constructed
- Reduce the number of storm event sampling stations from 8 (4 tributary, 4 BMP inflow or outflow) to 5 (3 tributary, 2 BMP inflow or outflow)
- Reduce the frequency for tributary, open water and BST sampling from monthly to bi-monthly during the "non-growing" season (October through March)
- Reduce the maintenance budget to 60 percent of original estimate
- Limit secondary system inventory to the City of Beaufort, Town of Bluffton and Town of Port Royal
- Reduce public information budget to $50,000 (reduce by half)

Though the total expenditure and total revenue are comparable, the expenditures and revenues for specific jurisdictions may not be in balance. This can potentially be resolved by modifying the cost-sharing methodologies for the various plan components.

Stakeholders (jurisdiction staff, citizens, politicians, Storm Water Utility Advisory Board) will need to evaluate the various ways of establishing a balance between the utility revenues and expenditures, in total and by jurisdiction. As shown, expenditures can be reduced in a number of ways, such as completing less projects, reducing the number of stations and/or frequency of sampling, and providing less frequent maintenance.

Since the development of these tables, the jurisdictions have decided to raise the base rate beyond the $40 previously assumed. Therefore, the actual annual revenue will likely be higher than the $4.9 million used as the basis for demonstration.

TABLE 16-1

ANNUAL LOADS FOR BEAUFORT COUNTY WATERSHEDS

WATERSHED	Total Phosphorus (lb/yr)			Fecal Coliform Bacteria (#/yr)		
	Existing	Future	% Change	Existing	Future	% Change
Calibogue Sound	61,529	62,391	1%	1.28×10^{16}	1.27×10^{16}	-1%
May River	28,815	31,092	8%	6.70×10^{15}	6.90×10^{15}	3%
Chechessee River	28,968	28,874	0%	5.65×10^{15}	5.57×10^{15}	-1%
Colleton River *	38,989	40,837	5%	8.71×10^{15}	9.28×10^{15}	7%
New River *	20,014	21,620	8%	4.30×10^{15}	4.21×10^{15}	-2%
Beaufort River	69,156	69,956	1%	2.02×10^{16}	2.02×10^{16}	0%
Coosaw River	89,084	91,792	3%	2.14×10^{16}	2.21×10^{16}	3%
Whale Branch West	28,322	29,431	4%	8.66×10^{15}	9.00×10^{15}	4%
Morgan River	43,613	45,580	5%	1.09×10^{16}	1.16×10^{16}	6%
Broad River *	95,366	97,036	2%	2.31×10^{16}	2.29×10^{16}	-1%
Combahee River *	26,858	27,519	2%	6.18×10^{15}	6.31×10^{15}	2%
Coastal	71,304	72,787	2%	1.39×10^{16}	1.42×10^{16}	-13%
TOTAL	602,018	618,915	3%	1.43×10^{17}	1.45×10^{17}	2%

* Does not include tributary area outside of Beaufort County

TABLE 16-2

WATER QUALITY LOS BASED ON MODEL RESULTS

	Number of Segments Having Level of Service							
	Model - Existing Land Use				Model - Future Land Use			
WATERSHED	A	B	C	D	A	B	C	D
Calibogue Sound	21	2	1	3	21	2	0	4
May River	7	0	0	1	7	0	0	1
Chechessee River	12	0	1	2	12	0	1	2
Colleton River	3	3	0	5	3	2	0	6
New River	---	---	---	---	---	---	---	---
Beaufort River	10	2	3	6	10	2	3	6
Coosaw River	11	4	0	4	10	5	0	4
Whale Branch West	4	2	0	3	4	1	1	3
Morgan River	11	6	4	8	10	5	3	11
Broad River	---	---	---	---	---	---	---	---
Combahee River	---	---	---	---	---	---	---	---
Coastal	---	---	---	---	---	---	---	---
TOTAL	79	19	9	32	77	17	8	37
% OF TOTAL	57%	14%	6%	23%	55%	12%	6%	27%

TABLE 16-3

PLANNING LEVEL COST ESTIMATES FOR PSMS IMPROVEMENTS
BY PRIORITY AND FLOODING CATEGORY -
PUBLIC PROJECTS ONLY

PRIORITY	FLOODING CATEGORY				
	A	B	C	D	TOTAL
1	$1,751,000	$1,879,000	$1,258,000	$1,080,000	$5,968,000
2	$772,000	$942,000	$843,000	$153,000	$2,710,000
3	$2,202,000	$317,000	$467,000	$183,000	$3,169,000
4	$1,042,000	$1,301,000	$576,000	$402,000	$3,321,000
5	$0	$0	$0	$185,000	$185,000
TOTAL	$5,767,000	$4,439,000	$3,144,000	$2,003,000	$15,353,000

TABLE 16-4

CUMULATIVE PLANNING LEVEL COST ESTIMATES FOR PSMS IMPROVEMENTS
BY PRIORITY AND FLOODING CATEGORY -
PUBLIC PROJECTS ONLY

PRIORITY	FLOODING CATEGORY			
	A	B	C	D
1	$1,751,000	$3,630,000	$4,888,000	$5,968,000
2	$2,523,000	$5,344,000	$7,445,000	$8,678,000
3	$4,725,000	$7,863,000	$10,431,000	$11,847,000
4	$5,767,000	$10,206,000	$13,350,000	$15,168,000
5	$5,767,000	$10,206,000	$13,350,000	$15,353,000

Note: Cumulative cost value reflects the cost of completing all projects with equal or higher priority and flooding category.
Example: Value of $4,418,000 for priority 2, flooding category B includes costs of all priority 1 and priority 2 projects with A or B flooding levels.

Costs are based on December 2004 dollars.

TABLE 16-5

PSMS IMPROVEMENTS - PUBLIC - SOUTH OF BROAD RIVER

Watershed	Basin	Jurisdiction	Stream Crossing	Priority	Flood Depth Category	Cost
Colleton River	Okatie West	UCS	Okatie Highway (State Hwy 170)	1	A	$185,000
Town of Hilton Head Island	Sea Pines	THHI	Club Course Drive	1	A	$314,000
Colleton River	Kitty's Crossing	TB/UCS	Fording Island Road (US Hwy 278)	1	A	$367,000
Town of Hilton Head Island	Indigo Run	THHI	Lagoon near Marshland Road	1	A	$885,000
Colleton River	Burnt Church	UCS	Fording Island Road (US Hwy 278)	1	B	$211,000
Colleton River	Sawmill Creek	UCS	Fording Island Road (US Hwy 278)	1	B	$244,000
Colleton River	Pinkney Colony South	UCS	Fording Island Road (US Hwy 278)	1	C	$174,000
Colleton River	Burnt Church	UCS	Fording Island Road (US Hwy 278)	1	D	$46,000
Colleton River	Rose Hill East	UCS	Fording Island Road (US Hwy 278)	1	--- [1]	$389,000
Colleton River	Sawmill Creek East	UCS	Sawmill Creek Road	2	A	$90,000
Colleton River	Waddell	UCS	Sawmill Creek Road	2	B	$36,000
May River	Ulmer	UCS	Alljoy Road	2	B	$140,000
May River	Alljoy Landing	UCS	Ulmer Road	2	B	$499,000
Chechessee River	Callawassee Road West	UCS	Callawassee Drive	2	C	$29,000
May River	May River	TB	Palmetto Bluff Road	2	C	$44,000
Colleton River	Pinkney Colony South	UCS	Pinkney Colony Road	2	C	$54,000
New River	Eigelberger	UCS	Prospect Road	3	A	$22,000
Colleton River	Pepper Hall	UCS	Graves Road	3	A	$34,000
May River	Bluffton East	TB/UCS	Bruin Road (State Hwy 46)	3	A	$103,000
New River	Mungen	UCS	Prospect Road	3	A	$244,000
New River	Oak Ridge	UCS	Prospect Road	3	B	$30,000
May River	Ulmer	UCS	Confederate Avenue	3	B	$114,000
New River	Mungen	UCS	School Road	3	C	$32,000
New River	Oak Ridge	UCS	Beach Drive	3	C	$69,000
Colleton River	Camp St. Mary's	UCS	Camp St. Mary Road	3	C	$71,000
New River	Daufuskie South	UCS	Benjies Point Road	4	A	$50,000
Calibogue Sound	Webb Tract	UCS	Freeport Road	4	A	$232,000
Calibogue Sound	Webb Tract	UCS	Cooper River Landing Road	4	A	$343,000
TOTAL						$5,051,000

TB: Town of Bluffton

THHI: Town of Hilton Head Island

UCS: Unincorporated County, South of Broad River

[1] US Hwy 278 in Rose Hill East does not flood, but improvement is recommended to eliminate upstream flooding

Costs are based on December 2004 dollars.

TABLE 16-6

PSMS IMPROVEMENTS - PUBLIC - NORTH OF BROAD RIVER

Watershed	Basin	Jurisdiction	Stream Crossing	Priority	Flood Depth Category	Cost
Broad River	Broad River Boulevard	CB/UCN	Robert Smalls Parkway (State Hwy 170)	1	B	$580,000
Coosaw River	Air Station	CB/UCN	Trask Parkway (US Hwy 21)	1	B	$844,000
Beaufort River	Grober Hill	CB/UCN	Robert Smalls Parkway (State Hwy 170)	1	C	$104,000
Beaufort River	Battery Creek North	CB/UCN	Robert Smalls Parkway (State Hwy 170)	1	C	$114,000
Beaufort River	Battery Creek West	UCN	Parris Island Gateway (State Hwy 802)	1	C	$276,000
Broad River	Broad River Boulevard	CB/UCN	Savannah Highway (State Hwy 802)	1	C	$281,000
Whale Branch West	Gardens Corner South	UCN	Trask Parkway (US Hwy 21)	1	C	$309,000
Coosaw River	McCalleys Creek	UCN	Trask Parkway (US Hwy 21)	1	D	$180,000
Coosaw River	Air Station	CB/UCN	Trask Parkway (US Hwy 21)	1	D	$227,000
Beaufort River	Burton Hill	CB/UCN	Robert Smalls Parkway (State Hwy 170)	1	D	$238,000
Beaufort River	Salt Creek South	UCN	County Shed Road	2	A	$69,000
Coastal	Scott Creek	UCN	Seaside Road	2	A	$133,000
Morgan River	Factory Creek	UCN	Holly Hall Road	2	A	$149,000
Broad River	Habersham Creek North	UCN	Burton Wells Road	2	A	$331,000
Coosaw River	Branford Creek East	UCN	Big Estate Road	2	B	$118,000
Coosaw River	Brickyard Creek	UCN	Walling Grove Road	2	B	$149,000
Coosaw River	True Blue Creek South	UCN	Kinlock Road	2	C	$55,000
Whale Branch West	Huspa Creek North	UCN	Old Sheldon Church Road	2	C	$70,000
Coosaw River	Lobeco	UCN	Keans Neck Road	2	C	$75,000
Combahee River	Combahee West	UCN	Twickenham Plantation Road	2	C	$114,000
Whale Branch West	Brewton West	UCN	Old Sheldon Church Road	2	C	$121,000
Broad River	Laurel Bay South	UCN	Mroz Road	2	C	$281,000
Broad River	Habersham Creek North	UCN	Pine Grove Road	2	D	$21,000
Beaufort River	Grober Hill	CB/UCN	Goethe Hill Road	2	D	$36,000
Whale Branch West	Grays Hill North	UCN	Clarendon Road	2	D	$38,000
Broad River	Scotts Neck North	UCN	William Campbell Road	2	D	$58,000

TABLE 16-6

PSMS IMPROVEMENTS - PUBLIC - NORTH OF BROAD RIVER

Watershed	Basin	Jurisdiction	Stream Crossing	Priority	Flood Depth Category	Cost
Beaufort River	Southside	CB	Railroad	3	A	$54,000
Broad River	Laurel Bay South	UCN	Joe Frazier Road	3	A	$164,000
Morgan River	Coffin Creek	UCN	Langford Road	3	A	$176,000
Broad River	Laurel Bay South	UCN	Morrell Drive	3	A	$202,000
Broad River	Laurel Bay South	UCN	Schein Loop	3	A	$286,000
Beaufort River	Shanklin Road	CB/UCN	Roseida Road	3	A	$296,000
Broad River	Broad River Boulevard	CB/UCN	Grober Hill Road	3	A	$296,000
Beaufort River	Southside	CB	Battery Creek Road	3	A	$325,000
Morgan River	Rock Springs Creek	UCN	Wade Hampton Road	3	B	$73,000
Broad River	Laurel Bay South	UCN	Laurel Bay Road	3	B	$100,000
Beaufort River	Salt Creek	UCN	Laurel Bay Road	3	C	$32,000
Coastal	South Frogmore	UCN	Club Bridge Road	3	C	$78,000
Whale Branch West	Grays Hill North	UCN	Jonesfield Road	3	C	$90,000
Broad River	Brays Island East	UCN	Savannah Highway (State Hwy 802)	3	C	$95,000
Beaufort River	Shanklin Road	CB/UCN	Laurel Bay Road	3	D	$28,000
Whale Branch West	Scotts Neck East	UCN	Water Park Road	3	D	$34,000
Beaufort River	Shanklin Road	CB/UCN	Fort Sumter Drive	3	D	$44,000
Morgan River	Rock Springs Creek	UCN	Sams Point Road	3	D	$77,000
Broad River	Habersham Creek West	UCN	Cherokee Farms Road	4	A	$162,000
Whale Branch West	Huspa Creek West	UCN	Huspah Court South	4	A	$255,000
Coosaw River	Laurel Hill	UCN	Gadwell Drive	4	B	$24,000
Coosaw River	Halfmoon Island	UCN	Keans Neck Road	4	B	$36,000
Broad River	Tomotley	UCN	Cotton Hill Road	4	B	$100,000
Broad River	Laurel Bay South	UCN	Schein Road	4	B	$149,000
Beaufort River	Battery Creek East	CB	June Way	4	B	$151,000
Whale Branch West	Huspa Creek South	UCN	Paige Point Road	4	B	$284,000
Beaufort River	Ballpark Road	UCN	Halifax Drive	4	B	$557,000
Beaufort River	Wallace Creek	UCN	Orange Grove Drive	4	C	$73,000
Coosaw River	True Blue Creek North	UCN	Stroban Road	4	C	$81,000
Coastal	Station Creek	UCN	Seaside Road	4	C	$81,000
Beaufort River	Battery Creek East	CB	Battery Creek Road	4	C	$98,000
Beaufort River	Grober Hill	CB/UCN	Munich Road	4	C	$243,000
Coosaw River	Dale	UCN	Wimbee Landing Road	4	D	$69,000
Combahee River	Combahee East	UCN	River Road	4	D	$88,000
Broad River	Baynard	CB/UCN	Baynard Road	4	D	$92,000
Coosaw River	Dale	UCN	Wimbee Landing Road	4	D	$153,000
Morgan River	Rock Springs Creek	UCN	Golf Course	5	D	$185,000
TOTAL						$10,302,000

CB: City of Beaufort

UCN: Unincorporated County, North of Broad River

Costs are based on December 2004 dollars.

TABLE 16-7

POTENTIAL SITES FOR REGIONAL DETENTION BMPs FOR TREATMENT OF RUNOFF FROM EXISTING DEVELOPMENT

Site ID	Water Quality Basin	Hydrologic Subbasin	Tributary Area (acres)	Land Acquisition Cost (million $)	Construction and Land Acquisition Cost (million $)	Fecal Coliform Bacteria Reduction (counts/year)	Fecal Coliform Bacteria Reduction % of water quality basin load
Beaufort River Watershed							
9	Battery Creek 1	Battery Creek West M1	367	0.3	1.8	7.5×10^{13}	2%
11	Battery Creek 2	Grober Hill M2	116	0.2	0.6	1.2×10^{14}	5%
12	Battery Creek 2	Burton Hill M2	239	0.3	1.2	1.8×10^{14}	7%
14	Albergotti Creek 2	Salt Creek South M1	311	0.2	1.8	2.2×10^{14}	11%
15	Albergotti Creek 2	Shanklin Road M2	587	0.8	2.5	4.6×10^{14}	22%
Colleton River Watershed							
4	Okatie River 3	Okatie West T3-A	277	0.2	1.3	3.4×10^{13}	5%
8	Colleton River 3	Camp St. Mary's M2	233	0.2	1.3	5.7×10^{13}	3%
Morgan River Watershed							
17	Rock Springs Creek 2	Factory Creek M2	274	0.4	1.3	1.1×10^{14}	16%
TOTAL				2.6	11.8		---

TABLE 16-8

RECOMMENDED TRIBUTARY SAMPLING LOCATIONS - BEAUFORT COUNTY

Watershed	Hydrologic Basin	% Urban - Future Land Use	% Impervious - Future Land Use	Future Increase in % Urban	Future Increase in % Impervious	Sampling Method	Purpose
Beaufort River	Southside	92%	51%	2%	1%	Automatic	High Density Residential Runoff
Beaufort River	Albergotti Creek	93%	67%	0%	0%	Automatic	Industrial Runoff
Colleton River	Camp St. Marys	48%	8%	16%	2%	Automatic	Low Density Residential Runoff [1]
Morgan River	Rock Springs Creek	96%	22%	7%	2%	Automatic	Medium Density Residential Runoff
Beaufort River	Burton Hill	71%	43%	19%	13%	Grab	Existing quality [1]
Beaufort River	Grober Hill	53%	25%	12%	3%	Grab	Existing quality [1]
Beaufort River	Salt Creek	75%	27%	35%	13%	Grab	Existing quality
Beaufort River	Salt Creek South	78%	30%	41%	11%	Grab	Existing quality [1]
Beaufort River	Shanklin Road	81%	49%	31%	21%	Grab	Existing quality [1]
Colleton River	Berkeley Creek	67%	18%	15%	5%	Grab	Existing quality
Morgan River	Factory Creek	84%	25%	15%	5%	Grab	Existing quality [1]
Morgan River	Lucy Point	95%	21%	6%	1%	Grab	Existing quality
Beaufort River	Battery Creek North	90%	67%	55%	43%	Grab	Trend Analysis
Beaufort River	Battery Creek West	82%	28%	50%	10%	Grab	Trend Analysis [1]
Colleton River	Okatie West	83%	25%	58%	19%	Grab	Trend Analysis [1]
May River	Rose Dhu Creek	91%	22%	54%	13%	Grab	Trend Analysis
May River	Stoney Creek	72%	12%	51%	8%	Grab	Trend Analysis
Morgan River	Coffin Creek	87%	22%	59%	14%	Grab	Trend analysis

[1] Sampling station is downstream of potential regional detention site, and therefore may provide data for prioritizing the construction of ponds and evaluating benefits (if pond is built)

TABLE 16-9

RECOMMENDED OPEN WATER SAMPLING LOCATIONS - BEAUFORT COUNTY/SCDHEC

Watershed	Water Quality Basin	Station Type	Purpose
Calibogue Sound	Jarvis Creek 2	Shellfish	Validate model results
Calibogue Sound	Cooper River Trib	Shellfish	Validate model results
May River	May River 4	Ambient	Trend analysis
Colleton River	Okatie River 1	Ambient	Trend analysis
Colleton River	Sawmill Creek 1	Shellfish	Validate model results
Colleton River	Callawassie Creek 1	Shellfish	Validate model results
Beaufort River	Battery Creek 2	Ambient	Trend analysis
Beaufort River	Bloomfield Creek 1	Shellfish	Validate model results
Beaufort River	Albergotti Creek 1	Ambient	Trend analysis
Beaufort River	Albergotti Creek 1	Shellfish	Validate model results
Whale Branch West	Haulover Creek 1	Shellfish	Validate model results
Whale Branch West	Middle Creek 1	Shellfish	Validate model results

TABLE 16-10

RECOMMENDED BACTERIA SOURCE TRACKING SAMPLING LOCATIONS - BEAUFORT COUNTY/SCDHEC

Watershed	Water Quality Basin	Station Type	Existing Station
Morgan River	Morgan River 2	Open Water	SCDHEC 16A-09
Morgan River	Eddings Point Creek 1	Open Water	SCDHEC 16A-23
Morgan River	Eddings Point Creek 2	Open Water	SCDHEC 16A-18
Town of Hilton Head Island	Broad Creek 3	Open Water	SCDHEC 20-16 or 20-16A
Town of Hilton Head Island	Broad Creek 4	Tributary	THHI - Matthews Drive

TABLE 16-11

EXAMPLE OF ANNUAL COSTS BY JURISDICTION BASED ON 10-YEAR PLANNING HORIZON

	COSTS BY JURISDICTION						
MASTER PLAN ELEMENT	CITY OF BEAUFORT	TOWN OF BLUFFTON	TOWN OF HILTON HEAD ISLAND	TOWN OF PORT ROYAL	UNINCORPORATED COUNTY	COUNTY-WIDE	TOTAL
STORMWATER CONTROL REGULATIONS							
BMP Inspections	$9,426	$4,434	$29,711	$4,515	$51,914	$100,000	$100,000
ANNUAL SUBTOTAL	$9,426	$4,434	$29,711	$4,515	$51,914	$100,000	$100,000
PSMS ENHANCEMENTS							
Priority 1 - A (public)	$0	$158,357	$1,061,070	$0	$531,574		$1,751,000
Priority 1 - B (public)	$263,335	$41,149	$275,721	$126,137	$1,172,658		$1,879,000
Priority 2 - A (public)	$0	$0	$0	$0	$772,000		$772,000
Priority 2 - B (public)	$0	$0	$0	$0	$942,000		$942,000
Priority 3 - A (public)	$499,116	$23,641	$0	$0	$1,679,243		$2,202,000
Priority 3 - B (public)	$0	$0	$0	$0	$317,000		$317,000
TOTAL	$762,451	$223,147	$1,336,790	$126,137	$5,414,474		$7,863,000
ANNUAL SUBTOTAL	$76,245	$22,315	$133,679	$12,614	$541,447		$786,300
WATER QUALITY CONTROLS FOR EXISTING DEVELOPMENT							
Purchase Land for 9 Sites	$245,070	$115,287	$772,482	$117,389	$1,349,771		$2,600,000
Construct 3 Sites	$518,418	$243,877	$1,634,097	$248,322	$2,855,286		$5,500,000
TOTAL	$763,488	$359,164	$2,406,579	$365,711	$4,205,057		$8,100,000
ANNUAL SUBTOTAL	$76,349	$35,916	$240,658	$36,571	$420,506		$810,000
WATER QUALITY MONITORING							
Tributary Sampling - County	$17,909	$8,425	$56,451	$8,578	$98,637		$190,000
Open Water Sampling - County/DHEC	$5,655	$2,660	$17,827	$2,709	$31,149		$60,000
Bacteria Source Tracking - County/DHEC	$4,713	$2,217	$14,855	$2,257	$25,957		$50,000
ANNUAL SUBTOTAL	$28,277	$13,302	$89,133	$13,544	$155,743	$0	$300,000
ANNUAL MAINTENANCE							
Primary System	$24,000	$27,000	$0	$3,000	$846,000		$900,000
Secondary System	$268,000	$127,000	$300,000	$129,000	$1,479,000		$2,303,000
ANNUAL SUBTOTAL	$292,000	$154,000	$300,000	$132,000	$2,325,000		$3,203,000
INVENTORY OF SECONDARY STORMWATER MANAGEMENT SYSTEM							
Inventory	$120,000	$450,000	$400,000	$120,000	$2,000,000		$3,090,000
TOTAL	$120,000	$450,000	$400,000	$120,000	$2,000,000	$0	$3,090,000
ANNUAL SUBTOTAL	$12,000	$45,000	$40,000	$12,000	$200,000		$309,000

TABLE 16-11

EXAMPLE OF ANNUAL COSTS BY JURISDICTION BASED ON 10-YEAR PLANNING HORIZON

MASTER PLAN ELEMENT	COSTS BY JURISDICTION CITY OF BEAUFORT	TOWN OF BLUFFTON	TOWN OF HILTON HEAD ISLAND	TOWN OF PORT ROYAL	UNINCORPORATED COUNTY	COUNTY-WIDE	TOTAL
ADDITIONAL/ON-GOING STUDY AND ANALYSIS							
Structure GIS Database/Inundation Mapping	$28,277	$13,302	$89,133	$13,545	$155,743		$300,000
Bacterial Source Tracking Library Creation	$4,713	$2,217	$14,855	$2,257	$25,957		$50,000
Autosampler Purchase/Installation	$7,541	$3,547	$23,769	$3,612	$41,531		$80,000
Water Quality Model Validation/Sensitivity Studies	$4,713	$2,217	$14,855	$2,257	$25,957		$50,000
Analysis for Additional Tributary Modeling	$2,356	$1,109	$7,428	$1,129	$12,979		$25,000
TOTAL	$40,531	$19,066	$127,757	$19,414	$223,231	0	$430,000
Model Updates (annual cost)	$4,713	$2,217	$14,855	$2,257	$25,957		$50,000
ANNUAL SUBTOTAL	$8,766	$4,124	$27,631	$4,198	$48,280	$0	$92,999
PUBLIC INFORMATION							
ANNUAL SUBTOTAL	$9,000	$4,000	$30,000	$5,000	$52,000	$100,000	$100,000
BONDED DEBT SERVICE							
ANNUAL SUBTOTAL	$0	$0	$1,200,000	$0	$0		$1,200,000
OVERALL SUBTOTAL	$512,063	$283,091	$2,090,812	$220,442	$3,794,890		$6,901,298
UTILITY ADMINISTRATION (8%)							
ANNUAL SUBTOTAL	$41,000	$22,600	$167,300	$17,600	$303,600		$552,100
ANNUAL TOTAL	$553,063	$305,691	$2,258,112	$238,042	$4,098,490	$0	$7,453,398
PROJECTED REVENUE	$456,209	$214,612	$1,438,009	$218,524	$2,512,657		$4,840,011

NOTES:

1. System improvement costs for category 1 (evacuation routes) is split between jurisdictions based on projected SFU.
2. Priority 1 costs south of the Broad River are split between the Town Of Hilton Head Island, the Town of Bluffton and the unincorporated County south of Broad River.
3. Priority 1 costs north of the Broad River are split between the City of Beaufort, the Town of Port Royal and the unincorporated County north of Broad River.
4. Priority 2 and 3 costs are assigned to jurisdiction(s) where the problem area and tributary area are located.
5. Primary system maintenance cost was split between jurisdictions (excluding Town of Hilton Head Island) based on number of primary system subbasins associated with each jurisdiction.
6. Secondary system maintenance costs (excluding Town of Hilton Head Island) are based on total provided by County staff, split between jurisdictions based on projected SFU.
7. Detention facility, monitoring, public information and additional study were split between jurisdictions based on projected SFU.
8. Projected revenue is from Table 3-10 of the draft Stormwater Utility Report (April 20, 2005).
9. Secondary system annual maintenance costs include miscellaneous system upgrades
10. Costs are based on December 2004 dollars.

TABLE 16-12

EXAMPLE OF ANNUAL COSTS BY JURISDICTION BASED ON 10-YEAR PLANNING HORIZON

RESTRICTED TO ANTICIPATED REVENUE

	COSTS BY JURISDICTION					
MASTER PLAN ELEMENT	CITY OF BEAUFORT	TOWN OF BLUFFTON	TOWN OF HILTON HEAD ISLAND	TOWN OF PORT ROYAL	UNINCORPORATED COUNTY	TOTAL
STORMWATER CONTROL REGULATIONS						
BMP Inspections	$9,426	$4,434	$29,711	$4,515	$51,914	$100,000
ANNUAL SUBTOTAL	$9,426	$4,434	$29,711	$4,515	$51,914	$100,000
PSMS ENHANCEMENTS						
Priority 1 - A (public)	$0	$158,357	$1,061,070	$0	$531,574	$1,751,000
Priority 1 - B (public)	$263,335	$41,149	$275,721	$126,137	$1,172,658	$1,879,000
Priority 2 - A (public)	$0	$0	$0	$0	$772,000	$772,000
TOTAL	$263,335	$199,506	$1,336,790	$126,137	$2,476,231	$4,402,000
ANNUAL SUBTOTAL	$26,334	$19,951	$133,679	$12,614	$247,623	$440,200
WATER QUALITY CONTROLS FOR EXISTING DEVELOPMENT						
Purchase Land for Beaufort River Sites	$169,664	$79,814	$534,796	$81,269	$934,457	$1,800,000
Construct 1 Facility	$235,645	$110,853	$742,772	$112,874	$1,297,857	$2,500,000
TOTAL	$405,309	$190,667	$1,277,568	$194,143	$2,232,314	$4,300,000
ANNUAL SUBTOTAL	$40,531	$19,067	$127,757	$19,414	$223,231	$430,000
WATER QUALITY MONITORING						
Tributary Sampling - County	$10,368	$4,878	$32,682	$4,966	$57,106	$110,000
Open Water Sampling - County/DHEC	$4,242	$1,995	$13,370	$2,032	$23,361	$45,000
Bacteria Source Tracking - County/DHEC	$3,535	$1,663	$11,142	$1,693	$19,468	$37,500
ANNUAL SUBTOTAL	$18,145	$8,536	$57,194	$8,691	$99,935	$192,500
ANNUAL MAINTENANCE						
Primary System	$14,000	$16,000	$0	$2,000	$507,000	$540,000
Secondary System	$158,000	$75,000	$200,000	$76,000	$873,000	$1,381,800
ANNUAL SUBTOTAL	$172,000	$91,000	$200,000	$78,000	$1,380,000	$1,921,800
INVENTORY OF SECONDARY STORMWATER MANAGEMENT SYSTEM						
Inventory	$120,000	$450,000	$0	$120,000	$0	$690,000
TOTAL	$120,000	$450,000	$0	$120,000	$0	$690,000
ANNUAL SUBTOTAL	$12,000	$45,000	$0	$12,000	$0	$69,000

TABLE 16-12

EXAMPLE OF ANNUAL COSTS BY JURISDICTION BASED ON 10-YEAR PLANNING HORIZON

RESTRICTED TO ANTICIPATED REVENUE

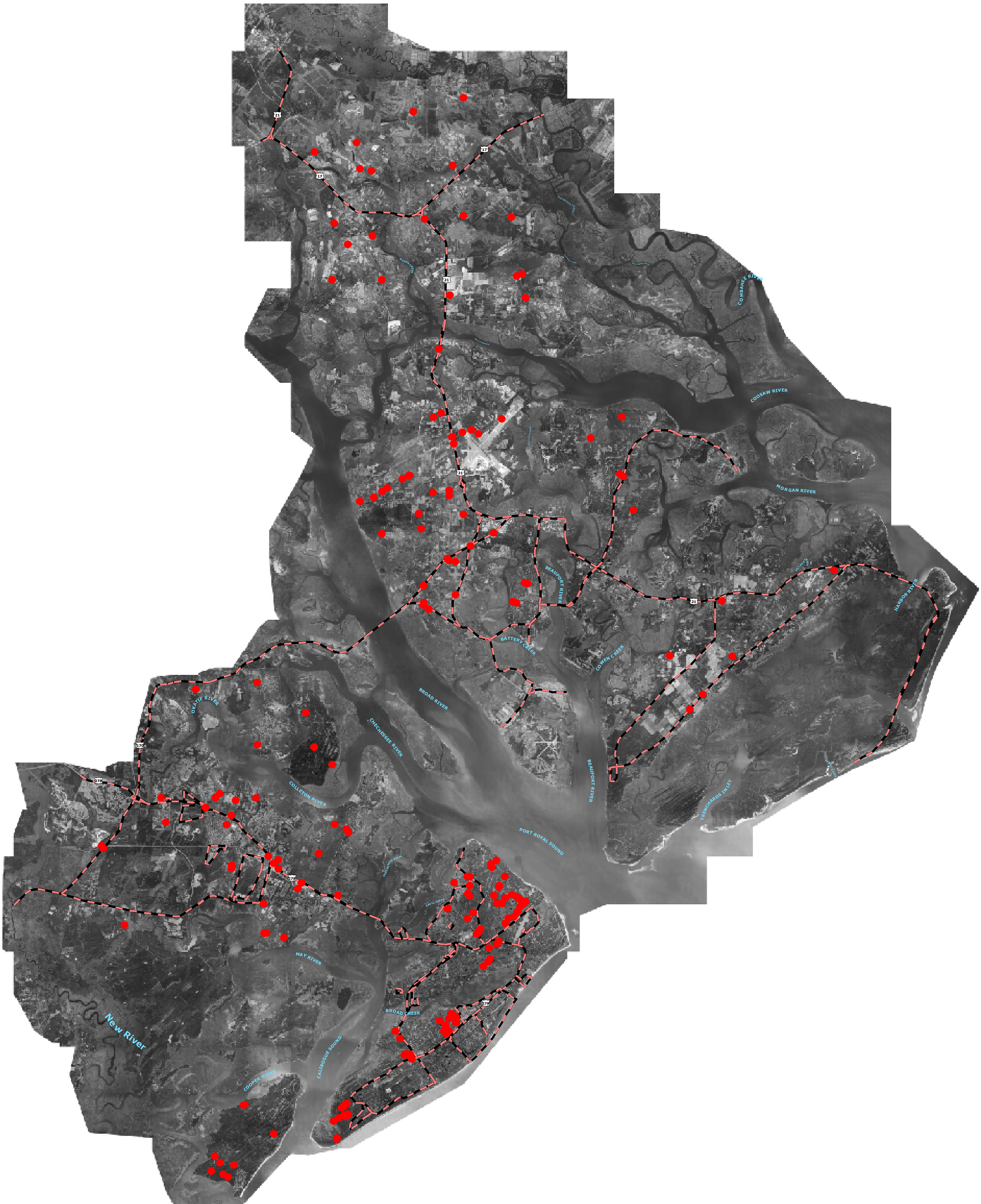
	COSTS BY JURISDICTION					
MASTER PLAN ELEMENT	CITY OF BEAUFORT	TOWN OF BLUFFTON	TOWN OF HILTON HEAD ISLAND	TOWN OF PORT ROYAL	UNINCORPORATED COUNTY	TOTAL
ADDITIONAL/ON-GOING STUDY AND ANALYSIS						
Structure GIS Database/Inundation Mapping	$28,277	$13,302	$89,133	$13,545	$155,743	$300,000
Bacterial Source Tracking Library Creation	$4,713	$2,217	$14,855	$2,257	$25,957	$50,000
Autosampler Purchase/Installation	$7,541	$3,547	$23,769	$3,612	$41,531	$80,000
Water Quality Model Validation/Sensitivity Studies	$4,713	$2,217	$14,855	$2,257	$25,957	$50,000
Analysis for Additional Tributary Modeling	$2,356	$1,109	$7,428	$1,129	$12,979	$25,000
TOTAL	$40,531	$19,066	$127,757	$19,414	$223,231	$430,000
Model Updates (annual cost)	$4,713	$2,217	$14,855	$2,257	$25,957	$50,000
ANNUAL SUBTOTAL	$8,766	$4,124	$27,631	$4,198	$48,280	$92,999
PUBLIC INFORMATION						
ANNUAL SUBTOTAL	$9,000	$4,000	$30,000	$5,000	$52,000	$50,000
BONDED DEBT SERVICE						
ANNUAL SUBTOTAL	$0	$0	$1,200,000	$0	$0	$1,200,000
OVERALL SUBTOTAL	$296,202	$196,111	$1,805,972	$144,432	$2,102,984	$4,545,700
UTILITY ADMINISTRATION (8%)						
ANNUAL SUBTOTAL	$23,700	$15,700	$144,500	$11,600	$168,200	$363,700
ANNUAL TOTAL	$319,902	$211,811	$1,950,472	$156,032	$2,271,184	$4,909,400

PROJECTED REVENUE	$456,209	$214,612	$1,438,009	$218,524	$2,512,657	$4,840,011

NOTES:

1. System improvement costs for category 1 (evacuation routes) is split between jurisdictions based on projected SFU.
2. Priority 1 costs south of the Broad River are split between the Town Of Hilton Head Island, the Town of Bluffton and the unincorporated County south of Broad River.
3. Priority 1 costs north of the Broad River are split between the City of Beaufort, the Town of Port Royal and the unincorporated County north of Broad River.
4. Priority 2 and 3 costs are assigned to jurisdiction(s) where the problem area and tributary area are located.
5. Primary system maintenance cost was split between jurisdictions (excluding Town of Hilton Head Island) based on number of primary system subbasins associated with each jurisdiction.
6. Secondary system maintenance costs (excluding Town of Hilton Head Island) are based on total provided by County staff, split between jurisdictions based on projected SFU.
7. Detention facility, monitoring, public information and additional study were split between jurisdictions based on projected SFU.
8. Projected revenue is from Table 3-10 of the draft Stormwater Utility Report (April 20, 2005).
9. Secondary system annual maintenance costs include miscellaneous system upgrades
10. Costs are based on December 2004 dollars.

Beaufort County Location of Road Overtopping Problems



Legend

● Model - Potential Problem Areas

0 10,000 20,000 Feet

1 inch equals 20,000 feet

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Figure 16-1

Job Number: 15178.00	Scale: 1" = 20,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ThePlan_HH-Problems_figure16-1.mxd			Vertical Datum: NAVD88

Disclaimer

Thomas & Hutton Engineering Co. compiled the map information only from the following sources:

DATA	SOURCE	DATE
Roads	Beaufort County	2002
Aerials	Beaufort County	2002

Thomas & Hutton used the above data "as is", and has made no independent investigation of the data nor makes any representation as to the accuracy or completeness of the data. Please see each source for available documentation of its respective data sets.

DATA	SOURCE	DATE
Modeled Problem Areas	CDM / T&H	2004

Beaufort County Potential Locations for Water Quality Improvements and Monitoring

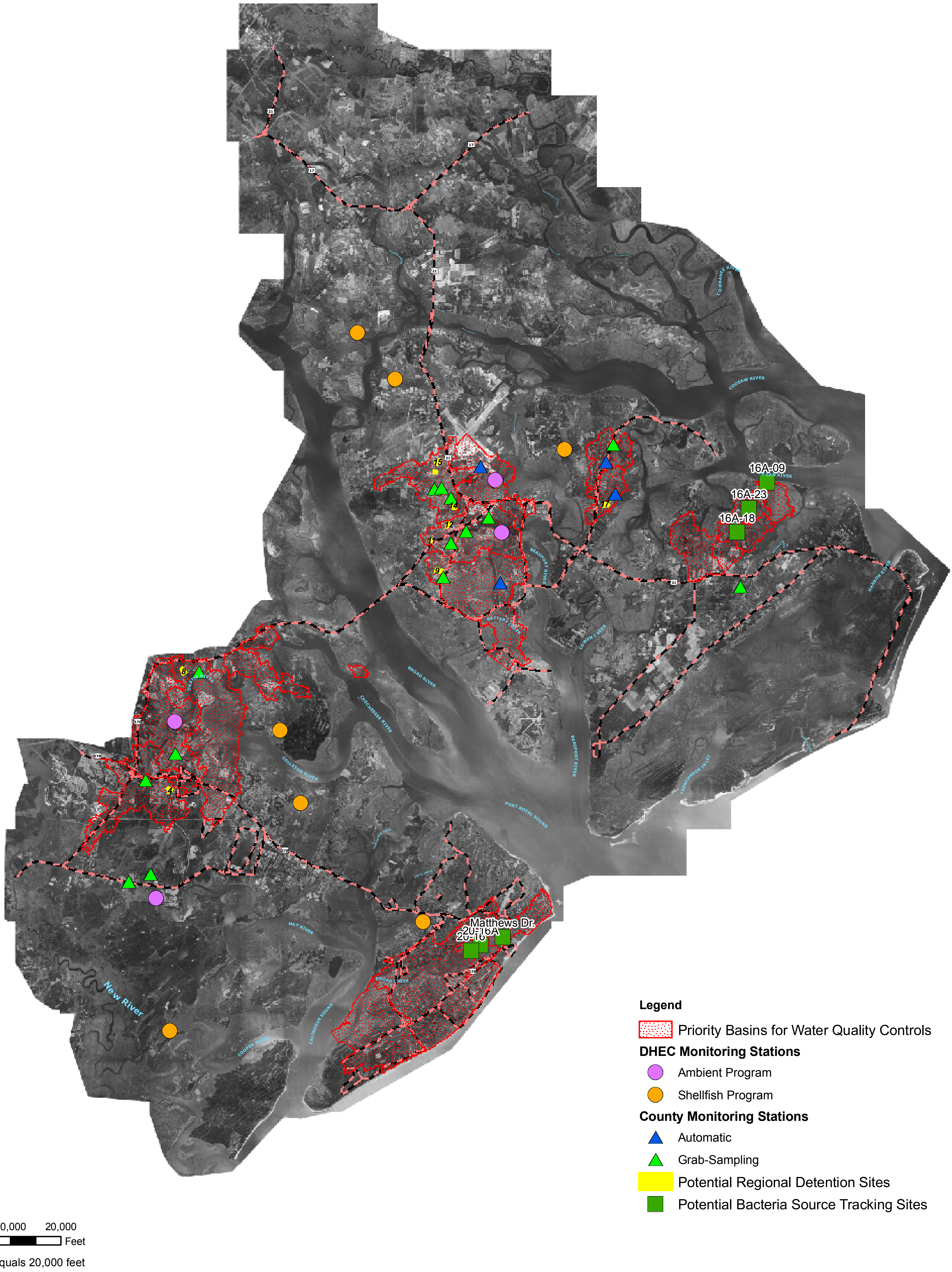

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Figure 16-2

Job Number: 15178.00	Scale: 1" = 20,000'	Projection: South Carolina Stateplane, I' Feet	Datum: NAD83
Produced: May 22, 2005	Produced by: GIS	Modified:	Modified by:
File: U:\J-15178_BeaufortCo_Stormwater\Task2000_WatershedPlan\documentation\TheReport\mxd\ThePlan_WQMonitoring_figure16-2.mxd			Vertical Datum: NAVD88

Section 17
2018 Stormwater Implementation Guide Recommendations

This section summarizes the recommendations generated from the updated SWMP. Recommendations in this section are based upon the findings presented in Updated Sections 3,4,6,7,8,9, & 11 of the report. Section 17.1 describes the elements of the guide, and the planning level cost estimates are presented in Section 17.2.

17.1 Recommended Watershed Management Plan

The recommended implementation guide includes the following elements:

- PSMS enhancements
- Water quality monitoring
- Operations and maintenance (O&M) of the PSMS and secondary stormwater management systems
- Inventory of the secondary stormwater management system
- Additional and on-going study and analysis

For each plan element, the following sections identify objectives and recommended activities.

17.1.1 PSMS Enhancements

As a result of the updated to the hydrologic and hydraulic analyses, a total of 76 locations for improvements to mitigate overtopping were identified. These results were developed by analyzing evacuation routes for the 100-year design storm and analyzing all other roads for the 25-year design storm. Locations of the problem areas are presented in tables 3-6, 4-6, 6-6, 7-6, 8-6, 9-6, & 11-6 and Figures 3-4, 4-4, 6-4, 7-4, 8-4, 9-4, & 11-4 in this update.

The evaluation of solutions for overtopping focused primarily on comparing original 2006 models to current information and continued to focus on the upgrade of culverts at the stream crossings. Overtopping is mitigated by increasing the conveyance capacity of the culverts. In some cases, the culvert upgrade was supplemented by raising the road, particularly in locations where the road elevation was at or near the design downstream boundary water elevation, which was defined as the mean annual high tide.

Originally the 2006 SWMP considered regional detention along PSMS and concluded that the cost of detention was prohibitive compared to upgrading culverts or raising roads. It is recommended as part of this guide that where CIP water quality locations are considered, the effect of that detention should attempt to take into account any local overtopping that could be mitigated by the CIP project.

In general, consideration should be made for additional detention along all drainage systems wherever feasible to add more volume to the system, thus mitigating potential flooding for smaller storm events, and reducing duration of flooding for larger events.

17.1.2 Water Quality Controls for Existing Development

The water quality analysis identified a number of water quality basins in the County where treatment of runoff from existing development could improve the potential for meeting bacteria and other water quality standards.

In general, potential regional sites were located in areas of existing wetlands, which require the implementation of "off-line" detention facilities primarily excavated from upland areas outside of the existing wetlands.

A total of nine sites were recommended for regional BMPs. The evaluation included a review of the sites with participating jurisdictions staff, evaluation of potential wetlands impact, determination of site tributary area and existing land use, general order of magnitude sizing of the pond, and evaluation of construction costs, land acquisition costs and benefits (bacteria, TP, TN, and TSS load reduction). The locations of the proposed facilities are shown in and further described in Appendix O in the CIP recommendations.

17.1.3 Water Quality Monitoring

A monitoring program was in place in the County as a result of previous 2006 SWMP recommendations and other activities in the watershed. As part of this implementation guide, this data was evaluated, and a new set of monitoring locations as recommended. Details of this are located in Appendix Q of this guide.

The goals of the program include the following:

- Characterize baseline water quality via ambient (grab) sampling
- Identify seasonal trends and overall trends over time using long-term ambient sampling data
- Evaluate dry weather (ambient) and wet weather (automatic sampling) water quality in selected areas for comparison to pollutant concentration values used in the watershed water quality modeling effort
- Evaluate sources of bacteria (human, bird, pets, wildlife) in locations where measured bacteria levels are substantially higher than expected, based on the watershed and receiving water quality modeling

It is recommended that Beaufort County staff be responsible for monitoring on the tributaries to the major open water tidal river segments and BMP monitoring. Where coordination with other municipalities is occurring, this should be continued. This monitoring will be done in conjunction with SCDHEC's existing monitoring programs.

Water quality data from Beaufort County, the Town of Bluffton and Hilton Head Island were collected and analyzed for standard statistical parameters and for trends. The identification of appropriate sampling sites for grab sampling and automatic storm event sampling was based on the water quality statistical analysis, the current LOS for water quality segments, and the existing land use distribution. In all, four sites were selected for automatic sampling, and 52 sites were selected for grab sampling. These sites are provided on Figure ES-6U in the Executive Summary of this guide.

Sampling would be conducted on a monthly basis. Sampling events will note weather conditions, flow conditions, and tidal condition (ebb and flood). Field parameters monitored during each sampling event include temperature, dissolved oxygen (DO), conductivity/salinity, pH and turbidity. Samples will be collected and analyzed for the following parameter list:

- *Enterococci* (saltwater)
- *Escherichia coli* (*E. coli*) (freshwater)
- Fecal coliform bacteria
- Total suspended solids (TSS)
- Biochemical oxygen demand (BOD)
- Ammonia nitrogen
- Nitrite and nitrate nitrogen
- Total Kjeldahl nitrogen (TKN)
- Total phosphorus
- Chlorophyll-a
- Total organic carbon (TOC) quarterly
- Metals (cadmium, chromium, copper, iron, lead, manganese, mercury, nickel and zinc) quarterly
- Hardness, quarterly

Samples collected will be characterized as either "dry" or "wet" samples, based on the amount of precipitation received over the 72 hours preceding sample collection. If less than 0.1 inch of rain fell in the 72 hours before the time of sampling, the samples will be classified as dry weather samples. If 0.1 inch of rain or more fell during the previous 72-hour period, the sample will be categorized as a wet weather sample. By identifying the weather conditions preceding each sampling event, it is hoped that contaminant concentrations can be linked to base- or low-flow conditions, or high-flow associated with stormwater runoff, thus providing valuable diagnostic information regarding potential source(s) of pollution.

Results from the laboratory analysis and field-collected parameters will be compared to the applicable water quality standards and criteria contained in SCDHEC Rule R.61-68, Water Classifications and Standards. Modifications to the plan, including stations to be sampled and observed concentrations, will occur based on the results obtained. Recommended statistical evaluations include standard descriptive statistics including data distribution, trend analysis (Kendall-Tau) and inter-station comparison (Mann Whitney, Wilcoxon).

Four stations would also include automatic sampling stations, so that sampling will be activated during storm events and stormwater runoff sampling can be reliably conducted. The four sites will be selected to represent runoff quality from different urban land use types (e.g., industrial, residential/golf course) and observed receiving water quality. In general, the same parameters will be sampled. Measurements of rainfall, stage, velocity and flow rate will also be made at the automatic sampling stations. The purpose of this sampling is to provide additional information to better define relationships be runoff event mean concentrations (EMCs) and receiving water quality. Preliminary pollutant loading modeling has revealed locations where resultant fecal coliform loads from the model were not excessive as compared to other areas but associated receiving waters were known "hot spots" based on evaluation of water quality data (i.e., tidal creek areas of May River and Okatie River). Other factors such as salinity regime changes, flushing, etc., also have an effect on observed fecal coliform levels in receiving waters. In addition to providing local EMC data to support future modeling efforts, this also provides insights to the importance of the various factors that affect receiving quality. It is anticipated that 12 or more storm event samples will need to be collected at each location to estimate EMCs with a reasonable confidence (95%). The actual number will depend on the variability of the data record at each location.

SCDHEC stations, classified as "shellfish" stations, will be evaluated concurrently for bacteria and salinity data. The objective is to use the collected data for comparison to the water quality model results and to determine if the model parameters provided a reasonable simulation of bacteria conditions or whether the model should be refined with adjusted mixing and first-order loss parameter values.

In general, there was good agreement between the measured values and the model results. However, some of the reaches did not have good agreement. This is likely due to how the hydrodynamics of the systems are being modeled. The approach that has been used to date is based on the net flow advection of the various reaches and is a quasi-steady-state approach. This is an acceptable approach in most cases. However, given the tide range that exists in the county's receiving waters and the dynamic salinity regimes present, a detailed 3-dimensional hydrodynamic model, such as the Environmental Fluid Dynamics Code (EFDC), is required to adequately simulate the tidal fluctuations and salinity-density gradients that exist in the receiving waters. Development of a 3-D hydrodynamic model would be a significant effort but would provide the proper hydrodynamic foundation for improved water quality predictions.

17.1.4 Operation and Maintenance

Operations and Maintenance recommendations have not changed from the original recommendations in the 2006 study. For the PSMS, operations and maintenance would primarily include maintenance of culverts and bridges, and maintenance of open channels. Activities at culverts and bridges would generally include removal of silt or other obstructions. For open channels, activities would also include silt and debris removal, and may also include periodic mowing.

17.1.5 Inventory of Secondary Stormwater Management System

Both this guide and the original 2006 SWMP focused on the PSMS, and an inventory of the PSMS was reviewed and updated as part of the study. The PSMS includes the major drainage systems in the County, typically including any conveyance with a tributary area of 320 acres or more.

Future efforts should focus on improving the data associated with the PSMS to include inverts, culvert sizes, GPS/GIS location data, and efforts should be made to improve the accessibility of that data within the County and jurisdictions GIS models.

The inventory of the secondary stormwater management system, which conveys the stormwater to the PSMS should also begin to be assembled. In many areas, drainage system maps are not current, and often show information that is not accurate. An accurate and complete inventory will be useful in evaluating the stormwater management system and evaluating the extent of required maintenance in those areas.

17.1.6 Additional and On-Going Study and Analysis

One of the major recommendations for further analysis is the continued development and improvement of an up-to-date structure GIS coverage with finished first-floor elevation data, and flood inundation mapping as well as PSMS and secondary stormwater systems. The modeling in this study developed peak water elevation data for the various design storms evaluated, including the 100-year design storm. However, the current version of the ICPR model does not include the capability of automated flood inundation mapping. Furthermore, while additional information and data was gathered as part of the update, the County database can use additional improvements. Consequently, the model results and LiDAR topographic data may suggest that the ground surface near a structure is inundated, but there is no way to confirm whether or not the structure itself is flooded or not (e.g., is it elevated to prevent flooding). Specific activities would include updating and maintaining the structure database and GIS coverage, and to evaluate finished first-floor elevations, by building certificates or survey.

Additional recommendations based on the update include updating the models to current versions of the software or considering migration to other platforms. For example, ICPR3.0 that was utilized for this analysis is now no longer supported by the vendor, and there is a new ICPR4.0 version which adds functionality and would be beneficial to consider updating the model to this platform. Additionally, the WMM and other water quality models are designed to run on older operating systems, and updated versions or new platforms should be considered for future work on this data.

17.2 Planning Level Costs for Plan Components

Conceptual costs have been estimated for some of the items discussed above as part of the implementation guide. In some cases, such as the water quality CIP projects and culvert upgrades, the cost is specified as a total cost in 2018 dollars. In contrast, other costs such as operations and maintenance are expressed as an annual cost.

17.2.1 PSMS Enhancements

The cost for the recommended improvements was presented in the watershed sections of this report. The total cost for updated watersheds was $22.2 million.

Analysis in the original 2006 SWMP had been done in order to prioritize the improvements based on the type of road and the depth of road overtopping for the design storm event. This criterion was not changed for this update analysis.

Consideration was also given to “public” versus “private” improvements, where “private” improvements would be in developments that would not be considered part of the “public” PSMS. This review indicated that the total projected cost for public projects as a result of this update is $9.2 million, and the projected cost of private projects is $12.9 million. This is shown in Table ES-6U in the Executive Summary of this guide.

17.2.2 Water Quality Controls for Existing Development

The water quality controls for existing development focuses on the implementation of regional BMP and detention facilities strategically located in areas with existing development that is not controlled by BMPs. The conceptual probable capital cost for the improvements was presented in the watershed sections and is further identified in Appendix O of this report. The total cost was $10.0 million, which includes the construction cost plus the land acquisition cost.

17.2.3 Additional and On-Going Study and Analysis

The major activity included in this category is the development of inundated area and evaluation of structural finished floor elevations. Cost of this task will need to be determined based on current state of the GIS and available FEMA data as well as other factors. For budgetary purposes, an allowance for $300,000 should be allocated towards this task.

Another on-going activity to consider is the update of the models developed for this study. An annual cost of $50,000 per year was previously stated in the 2006 study and is suitable for ongoing update costs to keep models current and prepare for future updates to the document and models. It is recommended to do this annually and coordinate this work in conjunction with updates to land use databases or other databases. Data required for model update such as land use and PSMS upgrades should be compiled as they occur to facilitate the model updates. This cost will not eliminate the need for future large-scale updates to models, it will only assist in maintenance of the models and keep costs down since the data will be better organized and available.

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