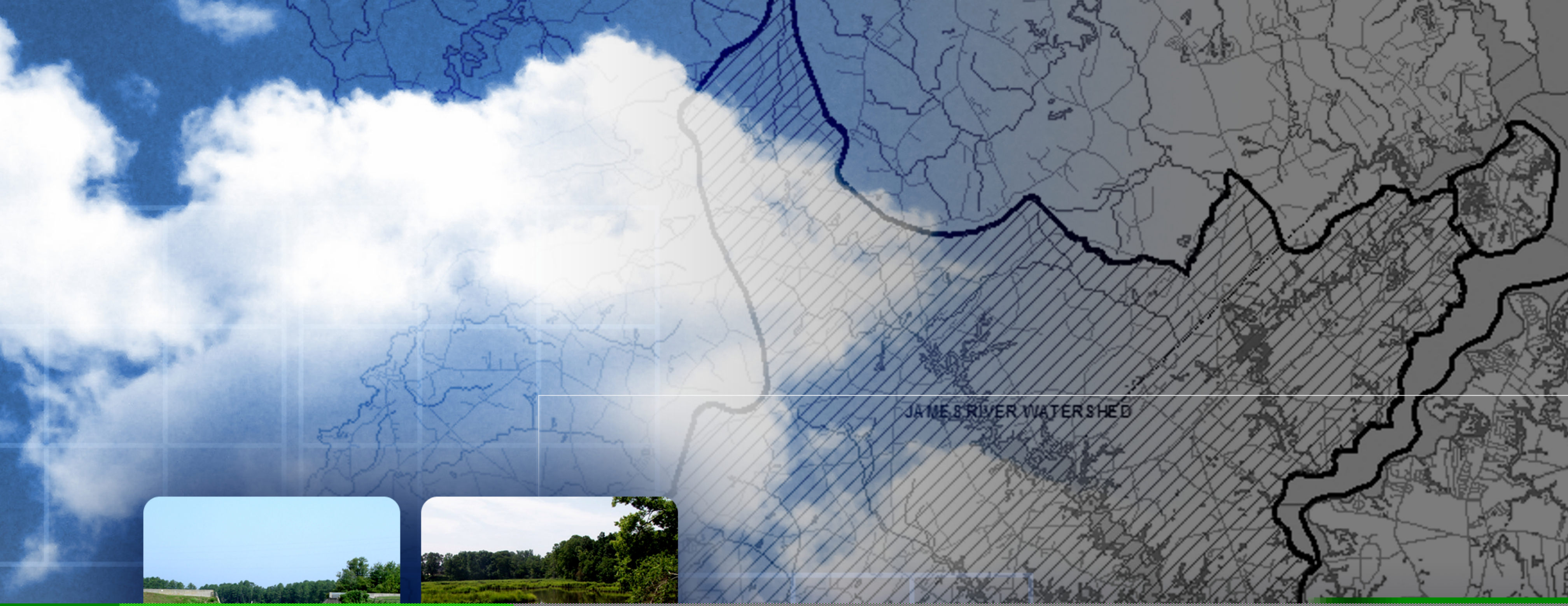

JAMES RIVER WATERSHED

TABLE OF CONTENTS

EXECUTIVE SUMMARY

To assist the City of Suffolk in making stormwater decisions to accommodate future development, Clark Nexsen will perform a study of the three watersheds in Suffolk. These watersheds include the James River Basin, the Chowan River/Blackwater River Basin and the Great Dismal Swamp Basin. The watershed studies will be performed in two phase. The James River Watershed with an approximate drainage area of 96 square miles will be studied under Phase I. The Chowan River/Blackwater River and the Great Dismal Swamp Watersheds with an approximate combined drainage area of 220 square miles, will be studied under Phase II. The purpose of these studies is to develop a stormwater master planning model to assess existing conditions and the impacts future development will have on these stormwater systems based on the projected 2018 land use condition described in Suffolk's Comprehensive Plan.

The studies require the hydrologic and hydraulic analysis of the existing major drainage systems within the regional watersheds. To facilitate the analysis of a complex stormwater network, Expert System's XP-SWMM 2000 (version 9.50) software was used to construct the model. XP-SWMM is a proprietary software based on the USEPA Stormwater and Wastewaster Management Model (SWMM) computer program. The model developed represents the primary stormwater conveyances within the watershed to describe and integrate the relationship of the various land uses and hydraulic controls throughout the watershed. The resulting model will be used to create the stormwater master plan from which the existing conditions and proposed future impacts will be evaluated to assist the City of Suffolk in day-to-day development decisions related to stormwater management concerns. The resulting model was developed using the City's comprehensive GIS mapping and field data to define existing conditions and the 2018 Comprehensive Plan to consider future impacts.

The stormwater network was analyzed for the 2-yr, 10-yr, 25-yr and 100-yr 24-hour, Type II rainfall events. The various storm events were evaluated for the following:

- Identify areas prone to flooding for 10-year storm.
- Evaluate culverts for the 10- and 25-year storms.
- Evaluate bridges and spillways for the 100-year storm.
- Compare existing channel velocities with proposed development channel velocities based on the 2018 Comprehensive Plan.

From the analysis, deficiencies in the stormwater network for the existing condition and future land use condition were determined and recommendations with conceptual construction cost estimates provided to mitigate the problems.
The model also identified that should receive detailed analysis to determine if flood control improvements are warranted. Due to the lack of detailed survey data and the master plan assumptions used, the model should not be used for design applications. Additional data and survey, particularly for roadway culverts, is needed before this model can be used for making specific design decisions and should only be used as a tool preparing the groundwork for more detailed studies and design.

Due to the rural landscape and the planned level of development, designated RE (rural estate with a 16% impervious ratio) and conservation areas for lands draining into the Western Branch Reservoir, relatively few flooding areas were identified for remediation at this stage of the analysis. Table No. ES-1below summarizes the recommended improvements for the deficiencies discussed under Section 3.0 (Existing Condition Model Results) and Section 4.0 (Proposed Condition (2018) Model Results) and associated costs for the respective conditions. Most of the improvements recommended were from inadequate culverts that had the potential to overtop the roadway for the respective 24-hour, Type II rainfall event. The costs listed show the estimated value of improvements based on the existing condition and the proposed condition (2018) land uses. For those improvements where only one cost is listed, there were no additional improvements required to address the proposed condition (2018) land uses. The costs shown are based on 2005 dollars and would be greater in the future due to construction cost escalation.

Table No. ES-1 Summary of Erosion and Flood Control Recommendations		
Location	**Recommendation**	**Cost**
Node CCSTORAGE5, Cherry Grove Rd.	Install two 38" x 24" HERCP culverts parallel to the existing 24" RCP (existing condition land use). A third 38" x 24" HERCP culvert is required for the 2018 land use condition.	$96,974 (existing) $131,572 (2018)
Node NNR362, Godwin Blvd.	Install a 24" RCP and 30" RCP parallel to the existing 24" RCP. The accumulated sediment in the existing 24" RCP should be cleaned out as an immediate remedial measure.	$89,062
Node NNR377, Kings Hwy.	Install two 24" RCP parallel to the existing 24" RCP and provide rip rap conduit outlet protection.	$54,040
Node NNR379, Kings Hwy.	Install a 30" RCP parallel to the existing the 30" HDPE culvert.	$42,708
Node NNR343, Five Mile Rd.	Install two 38" x 24" HERCP culverts parallel to the existing 24" RCP (existing condition landuse). Provide rip rap conduit outlet protection. A third 38" x 24" HERCP culvert is required for the 2018 land use condition.	$73,574 (existing) $102,003 (2018)
Node NNR 331, Lake Prince Dr.	Replace the existing 36" CMP with two 36" RCP culverts.	$52,006
Node NNR361, Godwin Blvd.	Provide erosion control and slope stabilization at the culvert.	$31,139
Node NCC77, Bridge 1802, Godwin Blvd.	Conduct a detailed study of the bridge to determine the if the scour has any impact on the structural integrity of the bridge and to develop the appropriate remedial measures	-

Large scale maps of the major watersheds (GIS-01), XP-SWMM model (GIS-02), flooding deficiencies (GIS-03), and recommended improvements (GIS-04) are located after the appendices.

In addition to the recommendations provided in Table No. ES-1, the following recommendations for supplemental analysis and study are provided:

- Detailed scour analysis for Bridge #1802. Due to complex hydraulics associated with bridges and scour analyses, the stormwater master plan model is not adequate for making appropriate recommendations.

- Detailed channel stability analysis. Because the stormwater master plan model does not focus on variations in channel geometry, roughness, and unique site conditions throughout the channel reach, specific improvement priorities cannot be immediately obtained from the model. At a minimum, outfall channels associated with deficient pipes should be studied in detail.

- Impacts of regional best management practices (BMPs). Due to the limited number of deficient pipes and the limitations associated with making decisions on channel stability, the use of regional BMPs was not addressed in detail. A detailed evaluation in concert with the channel stability analysis should be performed for the outfalls associated with the deficient pipes identified in the study recommendations.

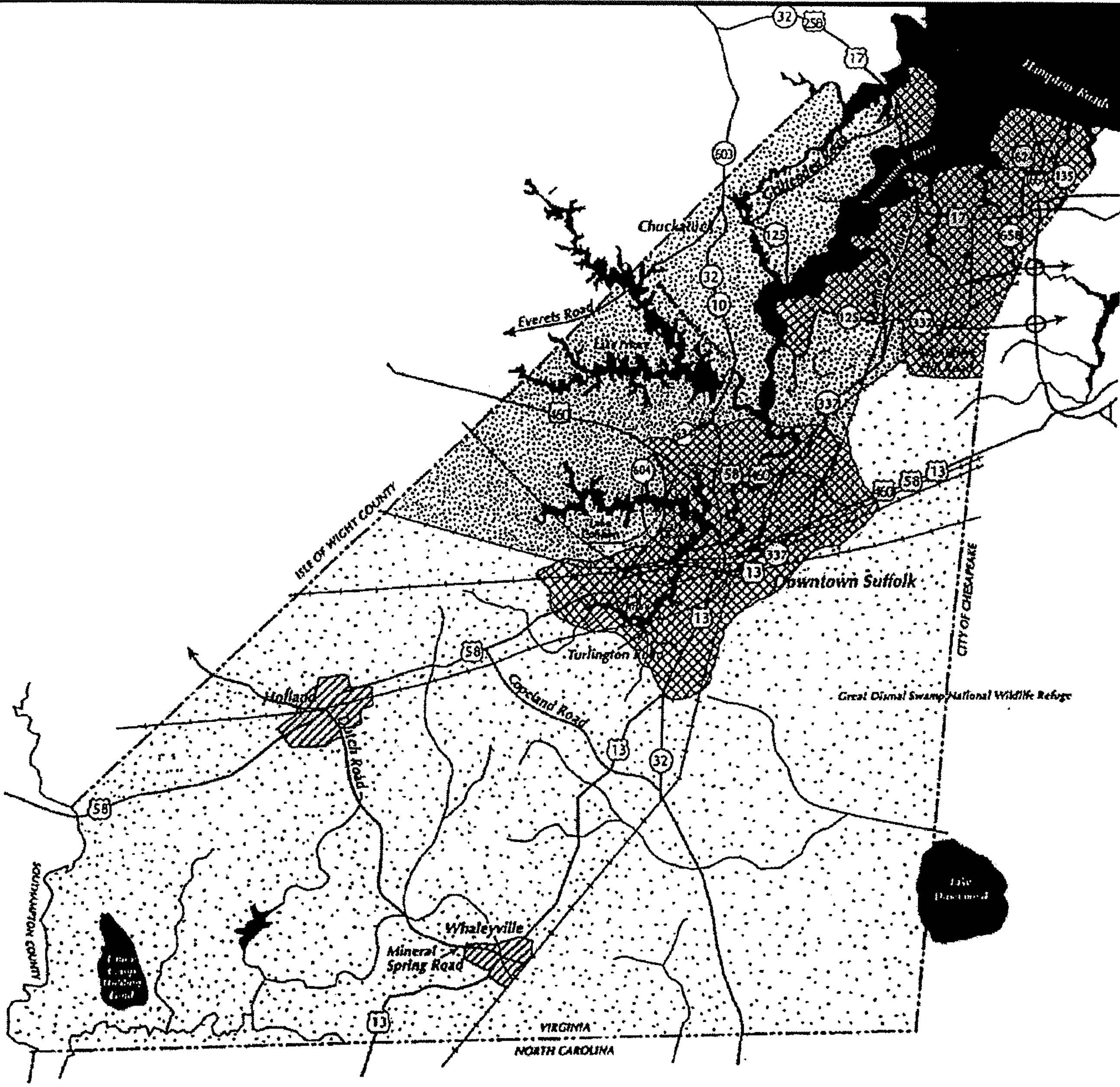
1.0 INTRODUCTION

1.1 CITY OF SUFFOLK OVERVIEW

The City of Suffolk is located in southeastern Virginia and is bordered by the unincorporated areas of Isle of Wight County and Southampton County to the west, the State of North Carolina to the south, the cities of Chesapeake and Portsmouth to the east, and the Lower James and Nansemond Rivers – Hampton Roads to the north. The city has a land area of approximately 430 square miles, which includes a portion of the Great Dismal Swamp. The annual precipitation in Suffolk is approximately 50 inches per year. There are variations in the monthly rainfall averages; however, rainfall is generally distributed throughout the year. Snowfall is infrequent and typically melts in a short period of time.

Elevations in the city range from sea level to approximately 85 feet. The greater relief promotes natural drainage for most areas of the city. The topography also supports the creation of six drinking water reservoirs, which include: Lake Cohoon, Lake Meade, Lake Kilby, Speights Run, Lake Prince, and Western Branch Reservoir.

The City of Suffolk is currently undergoing a period of growth and development. To manage the City's future growth the 2018 Comprehensive Plan was adopted by the City in March 1998. The 2018 Comprehensive Plan established long term (20-yr) development strategies, goals, and policies. A map of the proposed landuse for the 2018 Comprehensive Plan is provided in Figure 1-1.



RURAL/AGRICULTURAL CONSERVATION AREA (NO UTILITIES)

RURAL CONSERVATION AREA/LOW INTENSITY RESIDENTIAL (WITH PUBLIC WATER)

SUBURBAN/URBAN DEVELOPMENT (WITH UTILITIES)

RURAL VILLAGE (WITH PACKAGE TREATMENT PLANTS)

NANSEMOND 1646 SUFFOLK 1742
CITY of SUFFOLK, VIRGINIA

JOB TITLE:

THE CONCEPT PLAN FOR 2018
CITY OF SUFFOLK, VIRGINIA

CLARK•NEXSEN
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FIG. 1–1

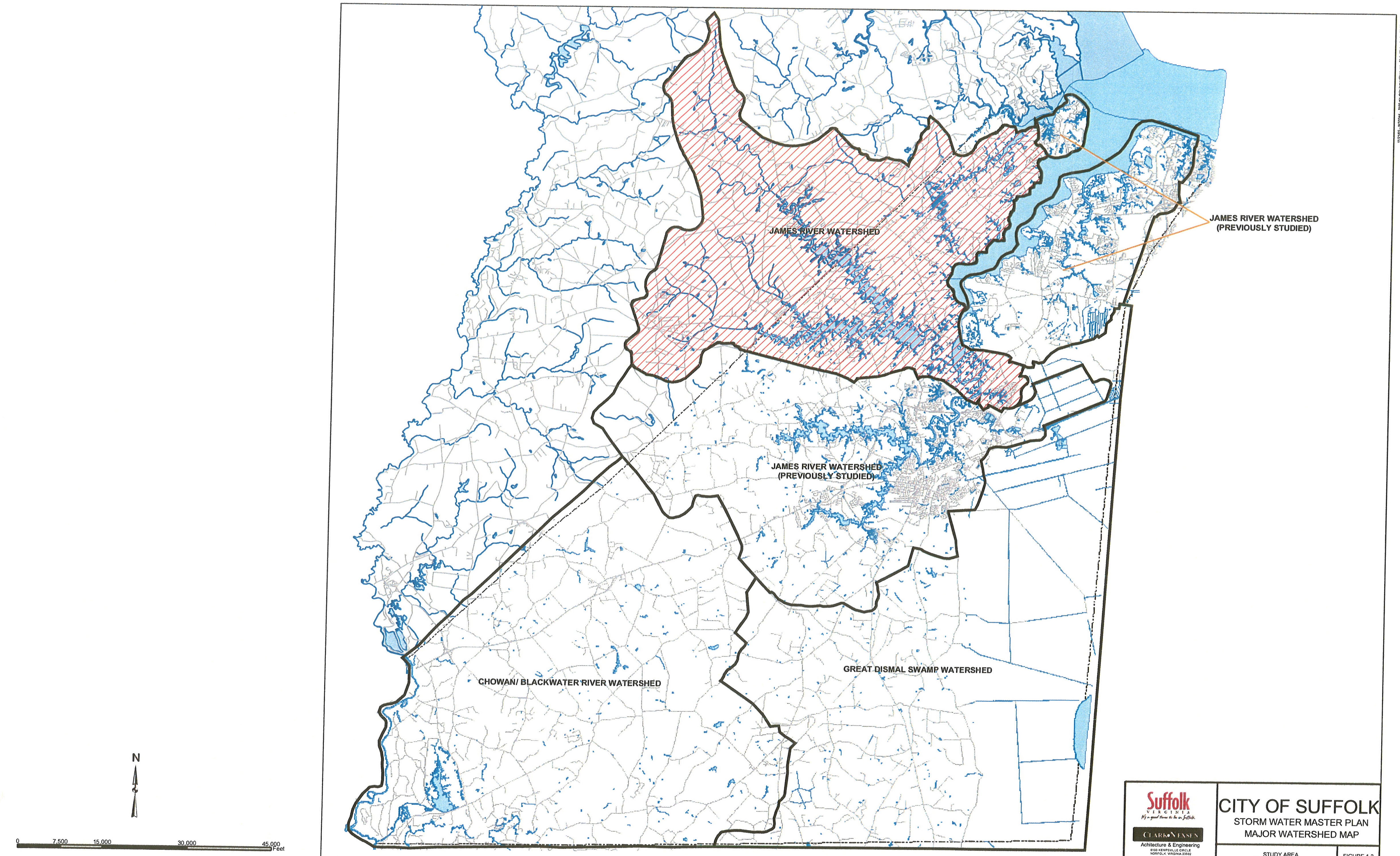
The objectives of the 2018 Comprehensive Plan include, but are not limited to, the following:

- To define and delineate two areas of compact, high-quality urban and suburban development: one around the central city and one in the northeast.
- To allow and promote some low-density, high-quality, large-lot residential development between these two compact areas and in the northwest.
- To preserve the southern half of Suffolk as a rural, agricultural area.
- To make special efforts to protect the watersheds which provide drinking water for Suffolk, Portsmouth, Norfolk, Chesapeake, and Virginia Beach.

To comply with national, state, and local environmental regulations governing water quality, the proposed development in the 2018 Comprehensive Plan must be implemented in accordance with a variety of existing environmental regulations related to the management of stormwater runoff.

1.2 PROJECT OBJECTIVES

Clark Nexsen will perform a study of the three regional watersheds in Suffolk. The watersheds to be studied include the James River Basin, Chowan/Blackwater River Basin and the Great Dismal Swamp Basin. The current study area, the James River Basin, excludes the areas previously studied in the document entitled *Stormwater Master Plan City of Suffolk, Virginia, Final Report May, 2004*, for the U.S. Army Corps of Engineers. The James River Watershed, with an approximate drainage area of 96 sq. mi., will be studied under Phase I. The Chowan/Blackwater River and the Great Dismal Swamp Watersheds with an approximate combined drainage area of 220 sq. mi., will be studied under Phase II. The watershed study will involve the hydrologic and hydraulic analysis of the existing major drainage systems. A map of the major watersheds within the city is presented in Figures 1-2 and GIS-01.



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CITY OF SUFFOLK
STORM WATER MASTER PLAN
MAJOR WATERSHED MAP

STUDY AREA
WATERSHEDS OVERVIEW

FIGURE 1-2

The Phase I Study Area consists of the northwestern and central portions of Suffolk and extends into Isle of Wight. The primary outfalls for this watershed are Chuckatuck Creek and the Nansemond River. The Phase 1 Study Area within Suffolk is generally areas west of the Nansemond River and is bounded by Route 460, Kings Fork Road and Hillpoint Boulevard to the south, Nansemond Parkway to the east, and Moores Point Road and Upton Lane.

The hydraulic grade line within the major conveyances were computed for the existing and future land uses based on current GIS mapping, field observation and the 2018 Comprehensive Land Use Plan to determine the following:

- Identify areas prone to flooding for 10-year storm.
- Evaluate culverts for the 10- and 25-year storms.
- Evaluate bridges and spillways for the 100-year storm.
- Compare existing channel velocities with proposed development channel velocities based on the 2018 Comprehensive Plan.

Once the flood prone areas were determined, stormwater strategies and associated cost estimates were developed to address the deficiencies.

1.3 REPORT ORGANIZATION

The report is organized as follows:

- Section 1 "Introduction," provides a brief overview of the project objectives and of the report contents.
- Section 2 "Stormwater Modeling Approach" describes methods and assumptions used to develop the XP-SWMM model analysis of the primary drainage system.
- Section 3 "Existing Conditions" summarizes the results of the existing condition XP-SWMM model and the field reconnaissance. Evaluations of areas indicated by

the model to be prone to repetitive flooding are provided along with recommendations and cost estimates for potential flood control improvements.

- Section 4 "Proposed Conditions" summarizes the results of the proposed condition XP-SWMM model based on the 2018 Comprehensive Plan land use. Evaluations of areas indicated by the model to be prone to repetitive flooding are provided along with recommendations and cost estimates for potential flood control improvements.
- Section 5 "Conclusions and Recommendations" summarizes the report conclusions, recommendations and cost estimates.

2.0 STORMWATER MODELING APPROACH

2.1 HYDROLOGIC/HYDRAULIC MODELING ANALYSIS

A computer model using XP-SWMM was developed to represent the primary stormwater conveyances within the project study area. The model will assist in accomplishing the following objectives:

- Identify areas prone to flooding for 10-year storm.
- Evaluate culverts for the 10- and 25-year storms.
- Evaluate bridges for the 100-year storm.
- Compare existing channel velocities with proposed development channel velocities based on the 2018 Comprehensive Plan.
- Evaluate the impact of proposed development and future projected development.
- Evaluate other stormwater-related issues as determined by the City.

2.2 SOFTWARE

The computer model utilized as the primary tool for modeling the existing and future (2018) land use condition watershed was XPSWMM-2000 version 9.50. The XP-SWMM model was used for both the hydrologic and hydraulic analyses. XP-SWMM generates runoff hydrographs at specific locations within the system based on specified storm events. The model simulates surface runoff within each sub-area and the subsequent routing through the various drainage system components including pipes, channels and reservoirs. Runoff and routing are simulated based on specific input parameters for each component process. Technical information on model formulation and capabilities is available on-line at **www.xpsoftware.com**.

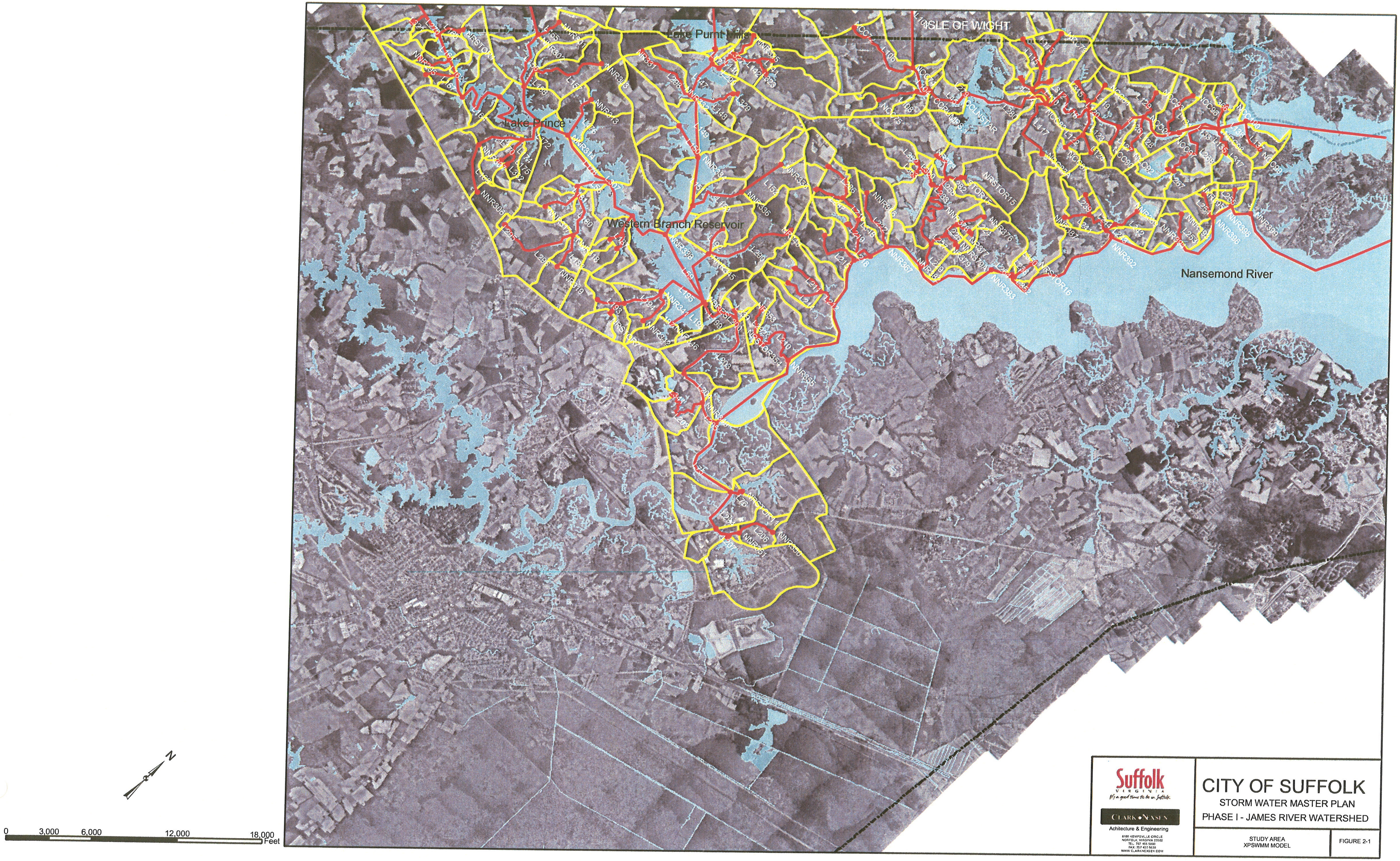
XP-SWMM is a "link-node" model that mathematically represents a drainage system as a series of links and nodes. A "link" represents a hydraulic element for flow transport, such as a pipe, channel, weir, orifice or flow control device. A "node" represents the junction of hydraulic elements (links), as well as a location for input of flow into the drainage system. A node can also represent a storage device such as a lake or pond, or a point junction where link properties change (such as a change in channel slope).

2.3 WATERSHED BOUNDARY DELINEATION

The James River Watershed Phase I Study Area encompasses approximately 96 square miles consisting of the north western portion of Suffolk and extending into Isle of Wight. This area includes Chuckatuck Creek as well as areas draining into the Nansemond River. Agriculture is the predominant land use within the watershed, including the portions within Isle of Wight. Other land use activities include residential homes and developments, light commercial and industrial properties, and conservation areas in the vicinity the reservoirs. The drainage (or sub-catchment) area boundaries within the city for the study area were delineated for primary channels and main receptors as shown in Figures 2-1 and GIS-02.

To construct the model and the associated GIS (ARCInfo 8.1) coverages, various information including current USGS quadrangle maps and City of Suffolk GIS mapping were obtained. Topographic contour data for approximately one-half of the study area located in Isle of Wight was unavailable in electronic format. To provide a means of delineating these areas, Clark Nexsen used digital USGS Quad Maps.

Using the topographic map data, the likely locations (road and rail crossings) of drainage structures (pipe culverts, box culverts, and bridges) serving the primary stormwater conveyances in the delineated watershed boundaries were identified. Clark Nexsen performed field reconnaissance to evaluate and inventory drainage structures to be modeled. Additionally, structure data including material type, shape and dimensions, and photographs were obtained to document existing structures and channel conditions. Several ponding locations (mostly private "farm ponds") were identified on the



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CITY OF SUFFOLK
STORM WATER MASTER PLAN
PHASE I - JAMES RIVER WATERSHED

STUDY AREA
XPSWMM MODEL

FIGURE 2-1

topographic mapping. Many of these ponds are located on private property or were inaccessible and were not included in the field reconnaissance. Where pond outlet structure data was unavailable, a 4' x 4' box riser was assumed with a crest elevation set at the existing water surface elevation as depicted in the GIS mapping.

As-built survey data (structure inverts and associated roadway elevations) from the Virginia Department of Transportation (VDOT) was generally unavailable for the drainage structures identified for the model. Assumptions were made to describe pipe inverts based on field data and GIS mapping. Surveyed channel cross-sections were unavailable. To develop channel sections for the model, Autodesk Civil Design software was used to cut-sections from the existing GIS contour surface. Bridges and railroad trellises were considered as minor obstructions throughout the majority of the modeled area and were therefore removed from further consideration. Tidally influenced stream sections were evaluated using a two year tidal elevation of 3.83 feet (NAVD-88) for storms up to a 25-yr design frequency (2-yr, 10-yr, and 25-ty) and a ten year tidal elevation of 5.00 feet (NAVD-88) for storms above a 25-yr design frequency (50yr and 100-yr).

Due to the lack of detailed survey data for hydraulic structures, roadway elevations, and channel cross sections, hydraulic modeling was only performed to the level of usefulness possible based on available information.

2.4 NODE NAMING CONVENTION

The naming convention used in the model is consistent with the naming convention outlined in the *Stormwater Master Plan City of Suffolk, Virginia, Final Report May, 2004.* The Phase 1 Study Area model consists of two primary subwatersheds, Chuckatuck Creek (CC) and the Nansemond River (NR). The naming convention used for identifying catchments was based on link nodes and storage nodes in the hydraulic model. Sub-catchments that discharge to link nodes are designated with the prefix "N " followed by the sub-catchment identifier and the link node number (e.g. NCC77 = inflow from subcatchments to catchment "77" discharging to the Chuckatuck Creek subwatershed). Sub-catchments that discharge to storage nodes were identified by the primary storage receptor name (e.g. LAKEPRINCE).

2.5 RAINFALL-RUNOFF MODELING

The RUNOFF module of XP-SWMM was used to generate the runoff from each subcatchment. A 30-minute time step was used for the runoff computations. The 30-minute time step was selected since the rainfall distribution is given in 30-minute segments. Runoff is computed from sub-catchments by describing the drainage areas as idealized rectangular areas with the slope of the subcatchment perpendicular to the width.

If overland flow is visualized as running down-slope off an idealized rectangular catchment, then the width of the subcatchment is the physical width of overland flow. Since real subcatchments will not be rectangular with properties of symmetry and uniformity other procedures are required. The width parameters for catchments with drainage channels off-centre, was determined by computing a skew factor:

$$Sk = (A2 - A1) / A$$

$$W = (2 - Sk) * L$$

where

Sk	=	skew factor
A1	=	area to one side of the channel
A2	=	area to other side of the channel
A	=	total area
W	=	subcatchment width
L	=	length of main drainage channel

The width parameter dictates the runoff response time of a sub-catchment and has a significant impact on the size and shape of the hydrograph. A narrower width produces a longer response time, and a wider width produces a shorter runoff response time.

To determine the slope parameter within each sub-area, three measurements were taken at representative locations within each cathcment. These measurements were averaged to determine the overall slope for the sub-catchment.

Impervious surface area estimates for existing land use conditions were developed based on field observation and GIS mapping. Each identified land use category was assigned an impervious surface percentage based on information provided in Suffolk's Unified Development Ordinance (UDO). The normal water surface areas of permanent water bodies, such as Western Branch Reservoir, were modeled as impervious surfaces. From this information, area-weighted averages of impervious surface area were determined for each subcatchment and were used as the approximate baseline percent imperviousness.

For the projected (2018) land use conditions, proposed zoning mapping for the 2018 Comprehensive Plan was obtained from the Suffolk Department of Planning. The proposed sub-catchment zoning coverages were assigned impervious surface ratios based on the Suffolk UDO to create area-weighted estimates of projected impervious surface area. The majority of the Phase 1 Study Area is located in the RE (Residential Estate) Zone. In order to model the projected 2018 land use conditions a minimum 16% impervious coverage, based on the Suffolk UDO, was used.

2.6 Soils Information

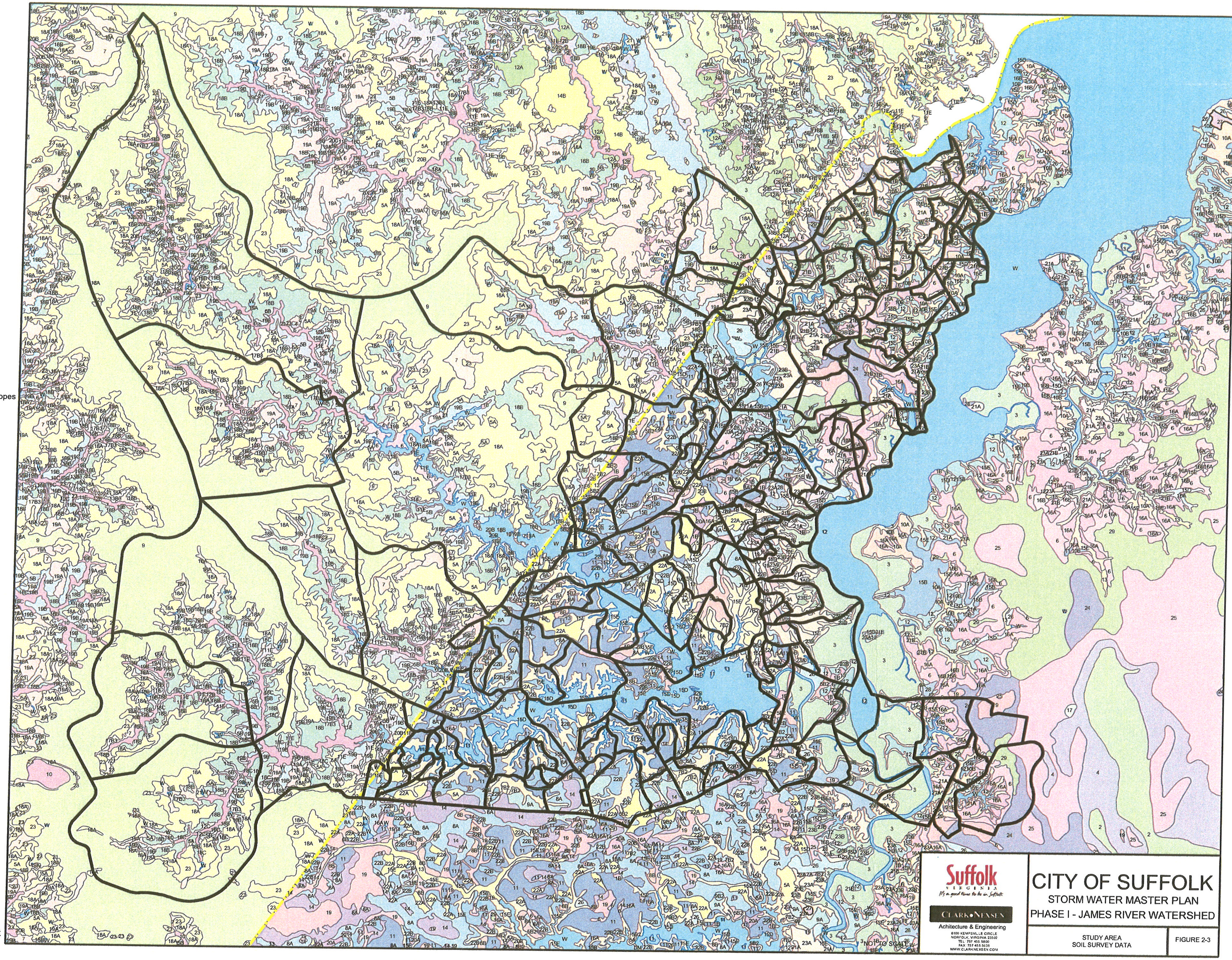
The Soil Survey for the City of Suffolk, published by the United States Department of Agriculture Soil Conservation Service in cooperation with Virginia Polytechnic Institute and State University, provided soil parameters such as hydraulic conductivity, permeability, and initial moisture deficit required for analysis using the Green-Ampt infiltration methodology in XP-SWMM. Additionally, digital soil coverage depicting soil types were obtained from the National Soil Survey Geographic (SSURGO) Database located on-line at **http://www.ftw.nrcs.usda.gov/ssur_data.html**. The SSURGO data, shown in Figure 2-2, was used to develop area-weighted soil parameters for the model. Green-Ampt infiltration parameters were assigned to each soil type. Those soil types were grouped into more generalized parameters to provide a master plan consideration of soil impacts and reduce the time spent developing soil parameters for use in the watershed model.

Legend

- "1" Alaga fine sand
- "10" Nawney loam
- "10A" Kalmia fine sandy loam, wet substratum, 0 - 2 % slopes
- "10B" Kalmia fine sandy loam, wet substratum, 2 - 6 % slopes
- "11" Kenansville loamy sand, 0 - 4 % slopes
- "11E" Nevarc and Remlik soils, 15 - 35 % slopes
- "12" Kenansville loamy sand, wet substratum, 0 - 4 % slopes
- "12A" Peawick silt loam, 0 - 2 % slopes
- "12B" Peawick silt loam, 2 - 6 % slopes
- "12C" Peawick silt loam, 6 - 10 % slopes
- "13" Levy silty clay loam
- "13B3" Peawick clay loam, 2 - 6 % slopes, severely eroded
- "14" Lynchburg fine sandy loam
- "14B" Peawick-Slagle complex, 2 - 6 % slopes
- "15" Rappahannock muck
- "15B" Nansemond loamy fine sand, 0 - 6 % slopes
- "15D" Nansemond loamy fine sand, 6 - 15 % slopes
- "15E" Nansemond loamy fine sand, 15 - 30 % slopes
- "16" Rumford loamy sand
- "16A" Nansemond fine sandy loam, 0 - 2 % slopes
- "16B" Nansemond fine sandy loam, 2 - 6 % slopes
- "17" Pactolus loamy fine sand
- "17B3" Slagle sandy loam, 2 - 6 % slopes, severely eroded
- "18" Pungo muck
- "18A" Slagle fine sandy loam, 0 - 2 % slopes
- "18B" Slagle fine sandy loam, 2 - 6 % slopes
- "18C" Slagle fine sandy loam, 6 - 10 % slopes
- "19" Rains fine sandy loam
- "19A" Uchee loamy sand, 0 - 2 % slopes
- "19B" Uchee loamy sand, 2 - 6 % slopes
- "19C" Uchee loamy sand, 6 - 10 % slopes
- "1B" Alaga loamy sand, wet substratum, 2 - 8 % slopes
- "2" Belhaven muck in Suffolk or Bohicket silty clay loam otherwise
- "20A" Rumford loamy fine sand, 0 - 2 % slopes
- "20B" Rumford loamy fine sand in Suffolk, or Uchee-Peawick complex, 2 - 6 % slopes
- "20C" Uchee-Peawick complex, 6 - 10 % slopes"
- "21" Udorthents, loamy
- "21A" State fine sandy loam, 0 - 2 % slopes
- "21B" State fine sandy loam, 2 - 6 % slopes
- "22A" Suffolk loamy sand, 0 - 2 % slopes
- "22B" Suffolk loamy sand, 2 - 6 % slopes
- "23" Yemassee fine sandy loam
- "23A" Tetotum fine sandy loam, 0 - 2 % slopes
- "23B" Tetotum fine sandy loam, 2 - 6 % slopes
- "24" Tomotley loam
- "25" Torhunta loam
- "26" Udorthents, loamy
- "27" Urban land
- "28" Wahee silt loam
- "29" Weston fine sandy loam
- "3" Bohicket silty clay loam in suffolk, or Chickahominy silt loam
- "4" Deloss mucky loam in Suffolk, or Chipley sand elsewhere
- "5A" Dogue fine sandy loamin Suffolk, or Emporia fine sandy loam, 0 - 2 % slopes
- "5B" Emporia fine sandy loam, 2 - 6 % slopes
- "5B2" Dogue fine sandy loam, 2 - 6 % slopes, eroded
- "6" Dragston fine sandy loam in Suffolk or Kenansville loamy sand elsewhere
- "7" Kinston loam
- "7A" Emporia fine sandy loam, 0 - 2 % slopes
- "7B2" Emporia fine sandy loam, 2 - 6 % slopes, eroded
- "8" Leon-Chipley sands
- "8A" Eunola loamy fine sand, 0 - 2 % slopes
- "8B" Eunola loamy fine sand, 2 - 6 % slopes
- "9" Myatt fine sandy loam
- "9A" Goldsboro fine sandy loam, 0 - 2 % slopes
- "9B2" Goldsboro fine sandy loam, 2 - 5 % slopes, eroded
- "DM" Dam
- "M-W" Industrial waste pond
- "W" Water

N

0 3,500 7,000 14,000 21,000 Feet



2.7 TIDAL BOUNDARY CONDITIONS

Tidal boundary conditions were developed to model the tidal influence on stream sections within the coastal floodplain. The 2-yr tidal boundary was used for storms up to a 25-yr design frequency (2-yr, 10-yr, and 25-yr) and a 10-yr tidal boundary was used for storms above a 25-yr design frequency (100-yr). The selection of these tailwaters is based on joint probability as described in the VDOT Drainage Manual. The 2-yr tidal elevation of 3.83 feet (NAVD-88) was used to remain consistent with the *Stormwater Master Plan City of Suffolk, Virginia, Final Report May, 2004*. The 10-yr tidal boundary elevation of 5.00 feet (NAVD-88) was derived form the *Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for the City of Suffolk* dated September 4, 2002.

2.8 DESCRIPTION OF MODEL ACCURACY

Due to the lack of detailed survey data and the master plan assumptions used, the model should not be used for design applications. Additional data and survey needs exist before this model can be used for purposes other than approximate analysis and stormwater generalizations. These additional requirements include:

- Detailed survey of hydraulic structure inverts, channel cross sections, elevations of undersides of bridge decking, crown elevations of bridge decking, and adjacent roadway elevations.

- Identification and inventory of existing small impoundments (ponds and stormwater management facilities), volumetric measurements of each impoundment, and control structure information such as pipe sizes, inverts of pipes, and invert or crest information of any other control structure at each impoundment.

3.0 EXISTING CONDITION MODEL RESULTS

The existing condition model was analyzed to identify flood prone areas throughout the watershed. The different channel reaches and structures (reservoir spillways, culvert crossings and bridges) were analyzed based on the following criteria:

- Existing channel velocities for the 2-yr design storm.
- Roadway culverts for the 10-yr design storm (secondary roadways) and the 25-yr design storm (primary roadways).
- Bridges and spillways for the 100-year storm.

Improvements and associated costs for the identified flooding and erosion problem areas are recommended based on the model results. Detailed cost estimates for each recommendation are located in Appendix A. Detailed XPSWMM model output for the existing condition is located in Appendix B. Mapping of the flood prone areas and proposed improvements, GIS-03 and GIS-04 respectively, are located in the back of the report.

3.1 EVALUATION OF EXISTING CONDITION CHANNEL REACHES

Clark Nexsen performed a field investigation to identify and document the relevant drainage features with in the Phase 1 Study Area. The field study included documenting evidence of erosion at the structures and within the stream channels. Several of the culvert crossing exhibited signs of erosion at the downstream outfall and along the drainage swales which collect runoff from the roadways and direct it to the culverts. The following table summarizes the areas of erosion documented in the field.

Table No. 3-1 Field Observations		
Node Name	**Location**	**Field Observations**
NCC77	Bridge # 1802, Godwin Blvd., Chuckatuck Creek	Evidence of scour (approximately 5-ft deep) beneath the bridge.
NNR377	Culvert crossings at Kings Hwy, near Spring Meadow Ln.	Scour evident at the down stream end of the culvert.
NNR361	Box culvert crossing at Godwin Blvd.	Erosion above the culvert at both the upstream and downstream face.
NNR362	24" RCP crossing Godwin Blvd.	Approximately 6 inches of sediment in pipe.
NNR352	Twin 60" CMP crossing Wilroy Rd.	Approximately 18 inches of sediment in pipe.
NNR343	24" RCP crossing Five Mile Rd.	Scour and erosion at the down stream end of the culvert.

Analysis of the channel velocities from the model does not indicate a severe erosion condition exists within most of the watershed channel sections. However, the model does indicate there are several channel sections with potentially erosive velocities (greater than 2.3 feet-second). The selection of a velocity of 2.3 feet per second or more is based on the general soil characteristics obtained from SSURGO and comparison with Recommended Maximum Water Velocities and Manning's n as a Function of Soil Depth and Flow Type chart contained in the VDOT Drainage Manual. The majority of the soils are classified as A-2 or A-4, which are gravel and sand with mixtures of silt and clay and silts with mixtures of sand and clay, respectively. This is consistent with what was viewed in the field. Many channel bottoms were not vegetated and contained numerous deposits. Several embankments were loose with loamy soils and did not support adequate vegetation for stability due to steep side slopes. The model shows the velocity within several of the conduits contributes to scour at the pipe outfalls, as noted in Table No. 3-1 above. Table No. 3-2 (next page) summarizes the channel and conduit sections that exhibit potentially erosive velocities. Due to the limitations of a general stormwater model typical for master planning purposes, channel velocities could vary just as actual conditions along the channel vary but are not explicitly modeled. Therefore, caution is required to determine if a channel is not adequate based on the results. The list below is general and provided for discussion purposes only.

Table No. 3-2 XP-SWMM Conduit Velocities

Link Name	Conduit Type	Location	2-yr Design Storm Maximum Velocity (f/s)
L113	Natural Channel	DS of NCC79 (Cherry Grove Rd N)	3.53
L152	Natural Channel	DS of NNR335, into Western Branch Reservoir.	3.53
L168	Natural Channel	DS of NNR306, into Lake Prince.	3.02
L279	(3) 8'x10' RCB	Lone Star Lakes	4.00
NR361	4'x4' RCB	Godwin Blvd.	7.81
L286	Natural Channel	DS of NNR361 (Godwin Blvd.)	3.36
L287	Natural Channel	DS of NNR361 (Godwin Blvd.)	2.51
L290	Natural Channel	DS of NNR343 (Five Mile Rd.)	2.61
L296	Natural Channel	DS of NNR320 (Lake Prince Dr.)	2.34
L298	Natural Channel	DS of NNR331 (Lake Prince Dr.)	3.17
L175	Natural Channel	DS of NRSTOR7A (Lake Prince Dr.)	4.28
L173	Natural Channel	DS of NRSTOR7B (Lake Prince Dr.)	3.05
L107	Natural Channel	DS of Crumps Mill Pond Spillway	2.36
L290.2	Natural Channel	DS of NNR343 (Five Mile Rd)	2.98
NR377.1	24"RCP	Kings Hwy, near Spring Meadow Ln.	5.60
NR379.1	30"HDPE	Kings Hwy, near Spring Meadow Ln.	4.81
NR373.1	36" RCP	Lone Star Lakes	12.35
NR362.1	24"RCP	Godwin Blvd.	5.55
NR343.1	24" RCP	Five Mile Rd	10.88
NR352.1	(2) 96" CMP	Wilroy Rd	2.65
NR320	36" RCP	Lake Prince Dr.	4.82
NR331.1	36" CMP	Lake Prince Dr.	6.68
Stor5	(2) 60" CMP	Everets Rd.	4.64
1146.1	24" RCP	Lone Star Lakes	5.58
1155.1	24" RCP	Cherry Grove Rd. N	5.67

3.1.1 Recommendations

In order to reduce the impact of erosion within the watershed the City may wish to consider the following recommendations.

- Bridge Scour: Significant scouring, approximately 5-ft below the normal channel bottom elevation, was noted beneath Bridge # 1802 on Godwin Blvd. (node NCC77). A detailed study of the bridge should be performed to determine if the scour has any impact on the structural integrity of the bridge and to develop the appropriate remedial measures. No additional recommendations will be made under this report.

- Pipe Scour: The culverts at nodes NNR377, and NNR343 listed in Table No. 3-1 and 3-2 above exhibit evidence of scour at the downstream pipe inverts. Rip rap conduit outlet protection should be provided at each outfall. In the case of node NNR343, additional rip rap protection is needed to protect the trees immediately downstream of the culvert from scour. The costs for the rip rap improvements are included in section 3.2.

- Pipe Sedimentation: The accumulated sediment should be removed within the culverts at nodes NNR362 (24" RCP @ Godwin Blvd.) and NNR352 (Twin 60" CMP at Wilroy Rd.). Cleaning of the culverts would be performed by the City Department of Public Works as part of the roadway maintenance program.

- Erosion Control: Erosion is evident at the drainage swales along Godwin Blvd draining to the box culvert at node NNR361 and the slope above the culvert. These eroded areas should be stabilized and re-graded where needed. Erosion control matting should be installed to stabilize the soil and provide a base for establishing permanent vegetation. The cost for the stabilization measures is approximately $31,139.

- Channel Stabilization: Several channel sections within the XP-SWMM model exhibit potentially erosive velocities. The July 2005 field investigations did not include a detailed survey of the channel sections. The channels were modeled as generalized estimates of channel cross-sections to define their hydraulic model parameters. The generalized sections may not reflect actual channel conditions that could result in an erosive condition. No specific recommendations for channel stability, beyond the items mentioned above, are provided at this time.

3.2 EVALUATION OF EXISTING CONDITION ROADWAY CULVERTS

The roadway culverts analysis was based on the design criteria in the VDOT Drainage Manual, Chapter 8, Culverts. Primary and arterial roadways were evaluated for flooding using the 25-yr design storm and secondary roadways were evaluated using the 10-yr design storm. Table No. 3-3 below summarizes the peak design storm flood elevation at each culvert.

Table No. 3-3 Culvert Peak Design Storm Flood Elevations					
Node Name	**Conduit Type**	**Road Name**	**Roadway Designation**	**Approximate Top of Road Elevation (ft)**	**Design Flood Elevation (ft)**
CCSTORAGE5	24" RCP	Cherry Grove Rd.	Secondary	5.50	5.87
CCLNSTAR	8'x10' RCB	Lone Star Lakes	Secondary	15.00	6.30
NRSTOR373	36" RCP	Lone Star Lakes	Secondary	--	--
NNR361	4'x4' RCB	Godwin Blvd.	Primary	63.00	56.90
NNR362	24"RCP	Godwin Blvd.	Primary	65.00	65.22
NNR377	24"RCP	Kings Hwy.	Primary	21.00	21.07
NNR379	30"HDPE	Kings Hwy.	Primary	21.00	20.68
NRSTOR15	36" RCP & 36" HDPE	Kings Hwy.	Primary	16.30	14.26
NNR343	24" RCP	Five Mile Rd.	Secondary	53.40	53.64
NNR353	24" CMP	SBS Greenhouses	Secondary	17.00	15.86
NNR352	96" CMP (x2)	Wilroy Rd.	Secondary	5.50	5.07
NNR320	36" RCP	Lake Prince Dr.	Secondary	71.00	69.39
NNR331	36" CMP	Lake Prince Dr.	Secondary	69.00	69.14
NRSTOR5	60" CMP (x2)	Everets Rd.	Secondary	24.60	24.44
NRSTOR11	22' SPILLWAY	Long Point Ln.	Secondary	9.50	9.53

The table above indicates that the design flood elevation for culvert crossing at nodes CCSTORAGE5, NRSTOR373, NNR362, NNR377, NNR343, NNR331, and NRSTOR11 overtop their respective roadways. The design flood elevation for culvert crossing at nodes NNR379, NNR352, and NRSTOR5 are within one foot of the top of their respective roadways.

3.2.1 Recommendations

In order to reduce the design flood elevation for the culverts, the City may wish to consider the following recommendations.

- CCSTORAGE5: The existing 24" RCP culvert does not have the required capacity to pass the 10-yr design storm without overtopping Cherry Grove Road.

Though the road is overtopped; the level of ponding at the culvert is not expected to impact nearby residential structures. To convey the runoff beneath the road, two 38" x 24" elliptical pipes should be installed parallel to the existing 24" RCP. The cost of the culvert improvements would be approximately $96,974.

- NRSTOR373: The existing 36" RCP is located within Lone Star Lakes Park. The model indicates the roadway may be flooded for the 10-yr design storm. Improvements are not recommended at this time since the path does not convey general residential, commercial or industrial traffic and is located within a recreational park area. Further study on the interaction of the various lakes is required to determine if improvements are warranted.

- NNR362: The 24" RCP crosses beneath Godwin Boulevard, which is a primary roadway. The roadway is flooded at the culvert for the 25-yr design storm. Flooding at this elevation could impact nearby residential structures. Installing a parallel 24" RCP and 30" RCP would lower the 25-yr design flood headwater elevation to reduce the flooding in the vicinity of the residential structures and prevents the road from being overtopped. The cost of the culvert improvements would be approximately $89,062.

- NNR377: The 24" RCP at this node crosses Kings Highway. The model indicates the roadway is flooded for the 25-yr design storm however; the level of ponding at the culvert is not expected to impact nearby residential structures. Adding two parallel 24" RCP next to the existing culvert would lower the 25-yr peak design flood elevation and prevent the road from being flooded. The cost of the culvert improvements would be approximately $54,040.

- NNR379: The 30" HDPE at this node crosses Kings Highway. The model indicates the roadway is flooded for the 25-yr design storm however; the level of ponding at the culvert is not expected to impact nearby residential structures. Adding a parallel 30" RCP next to the existing 30" HDPE would lower the 25-yr peak design flood elevation and prevent the road from being flooded. The cost of the culvert improvements would be approximately $42,708.

- NNR343: The existing 24" RCP culvert does not have the required capacity to convey the 10-yr design storm without overtopping Five Mile Road. Flooding at this level may have an impact on nearby residential structures. To convey the runoff beneath the road without overtopping, two 38"x 24" horizontal elliptical pipes should be installed parallel to the existing 24" RCP. This lowers the headwater elevation which would reduce the flooding in the vicinity of the existing residential structures and prevent the road from being overtopped. The cost of the culvert improvements would be approximately $73,574.

- NNR352: This crossing is being identified for possible improvements since the 10-yr design storm headwater elevation for the two existing 96" CMP culverts at Wilroy Road is within six inches of overtopping the road. Overtopping of Wilroy Road is a concern due to the high tidal conditions that could exist and the limited upstream storage due to development. However, since development best management practices have not been modeled, no recommendations for improvements will be made at this time. A more detailed analysis should be performed for this crossing to determine if improvements are warranted.

- NNR331: The existing 36" CMP culvert does not have the required capacity to convey the 10-yr design storm without overtopping Lake Prince Drive. Though the road could overtop, the level of ponding of the culvert headwaters does not impact nearby residential structures. To convey the runoff without overtopping, a parallel 36" RCP should be installed. The cost of the culvert improvements would be approximately $52,006.

- NRSTOR5: The model indicates that Everets Road at this node may be flooded during the 10-yr design storm event. However, field reconnaissance was not performed for the privately owned ponds upstream of node NRSTOR5 and will have an impact on the crossing at Everets Road. A more detailed study of the relationship between the upstream ponds and the culvert should be performed prior to developing any improvement recommendation; therefore no recommendations for this culvert are being presented at this time.

- NRSTOR11: The model indicates that Long Point Lane at this node may be overtopped during the 10-yr design storm event. However, since Long Point Lane is a private road and the BMP privately owned, no recommendations for improvements are being presented at this time.

Table No. 3-4 summarizes the existing and proposed improvements peak headwater elevations for the recommendations provided above.

Table No. 3-4 Comparison of Existing and Improved Culvert Peak Headwater Elevations – Existing Condition Model

Node Name	Road Name	Roadway Designation	Approximate Top of Road Elevation (ft)	Existing Culvert Peak Headwater Elevation (ft)	Improved Culvert Peak Headwater Elevation (ft)
CCSTORAGE 5	Cherry Grove Rd.	Secondary	5.50	5.87	5.30
NNR362	Godwin Blvd.	Primary	65.00	65.22	63.40
NNR377	Kings Hwy.	Primary	21.00	20.68	19.40
NNR379	Kings Hwy.	Primary	21.00	20.68	19.40
NNR343	Five Mile Rd.	Secondary	53.40	53.64	52.10
NNR331	Lake Prince Dr.	Secondary	29.50	69.14	66.30

3.3 Evaluation of Existing Condition Bridges and Spillways

The flood elevations at the bridges and dam spillways were evaluated using the 100-yr design storm. Table No. 3-5 below summarizes the 100-yr design flood elevation at the various bridges within the watershed.

Table No. 3-5 Bridge 100-yr Design Storm Flood Elevations

Node	Location	Bridge Number	Low Chord Elevation (ft)	Top of Road Elevation (ft)	100-yr Design Storm Elevation(ft)
NCC77	Godwin Blvd.	1802	9.50	12.00	10.32
NNR347	Godwin Blvd.	1800	13.10	19.80	5.11
NNR315	Girl Scout Rd.	8009	38.00	41.60	29.31
LAKEPRINCE	Girl Scout Rd.	8008	25.50	28.00	21.93
NNR312	Lake Prince Dr.	8007	27.50	30.00	29.31
NNR300	Gardner Ln.	8015	29.50	33.00	29.31
NNR305	Exeter Dr.	8010	31.50	33.00	29.31
NNR329	Everets Rd.	8001	23.50	26.00	21.93

The existing condition model shows that the 100-yr design flood elevations at the bridges do not overtop the roadways. However, for Bridge #1802 on Godwin Boulevard and Bridge # 8007 on Lake Prince Drive, the 100-yr design flood elevations are above low chord. Table No. 3-6 below summarizes the 100-yr design flood elevation at the reservoir spillways within the watershed. The model requires further evaluation to determine if this condition is reasonable prior to making recommendations.

Table No. 3-6 Spillway 100-yr Design Storm Flood Elevations

Node	Location	Weir Length (ft)	Weir Elevation (ft)	Parapet Elevation (ft)	100-yr Design Storm Elevation (ft)
WBRES	Western Branch Reservoir	267	20.00	28.00	21.93
BURNTMILLS	Lake Burnt Mills	165	34.00	43.50	38.30
LAKEPRINCE	Lake Prince	174	26.00	30.00	29.31

The existing condition model shows that the spillways for each reservoir can pass the 100-yr design storm without overtopping the parapet. The analysis does not account for any additional pumping facilities that may be employed at each dam.

3.3.1 Recommendations

Based on the results of the existing condition model regarding flood elevations, no recommendations for the bridges and spillways are being presented at this time. The model will need to be more closely evaluated to determine if there are possible flooding concerns at the bridges.

4.0 PROPOSED CONDITION (2018) MODEL RESULTS

The existing condition model was modified to reflect the future planned development in the watershed as outlined in the 2018 Comprehensive Plan. The proposed sub-catchment zoning coverages were assigned impervious surface ratios based on the Suffolk UDO to create area-weighted estimates of projected impervious surface area. The majority of the Phase 1 Study Area is located in the RE (Residential Estate) Zone. In order to model the future (2018) land use conditions a minimum 16% impervious coverage, based on the Suffolk UDO, was used for the sub-catchments within the model. The proposed model was analyzed to identify flood prone areas. The different channel reaches and structures (reservoir spillways, culvert crossings and bridges) were analyzed based on the following criteria, similar to the existing condition model:

- Proposed channel velocities for the 2-yr design storm.
- Roadway culverts for the 10-yr design storm (secondary roadways) and the 25-yr design storm (primary roadways).
- Bridges and spillways for the 100-year storm.

The improvements for the identified flooding and erosion problem areas recommended in Section 3 of the report were analyzed for the 2018 land use condition. Most of the improvements cited in Section 3 will be adequate for the 2018 land use condition. Detailed cost estimates for each recommendation are located in Appendix A. Detailed XPSWMM model output for the 2018 condition is located in Appendix C. Mapping of the flood prone areas and proposed improvements, GIS-03 and GIS-04 respectively, are located in the back of the report.

4.1 EVALUATION OF PROPOSED CONDITION CHANNEL REACHES

As with the existing condition model, analysis of the channel velocities does not indicate a severe erosion condition exists within most of the watershed channels incorporated into the model. However, there are some channels with potentially erosive velocities. The model also shows the flow velocity within several conduits may contribute to scour at the pipe outfall. Table No.4-1 below summarizes the channels and conduits that exhibit potentially erosive velocities.

Table No. 4-1 XP-SWMM Conduit Velocities

Link Name	Conduit Type	Location	2-yr Design Storm Maximum Velocity (f/s)
L113	Natural Channel	DS of NCC79 (Cherry Grove Rd N)	3.48
L152	Natural Channel	DS of NNR335, into Western Branch Reservoir.	2.77
L168	Natural Channel	DS of NNR306, into Lake Prince.	3.46
L206	Natural Channel	DS of NNR350 (Nansemond Pkwy.)	2.72
L279	(3) 8'x10' RCB	Lone Star Lakes	4.30
NR361	4'x4' RCB	Godwin Blvd.	8.72
L286	Natural Channel	DS of NNR361 (Godwin Blvd.)	3.65
L287	Natural Channel	DS of NNR361 (Godwin Blvd.)	2.56
L290	Natural Channel	DS of NNR343 (Five Mile Rd)	3.14
L296	Natural Channel	DS of NNR320 (Lake Prince Dr.)	2.31
L298	Natural Channel	DS of NNR331 (Lake Prince Dr.)	3.30
L175	Natural Channel	DS of NRSTOR7A (Lake Prince Dr.)	4.22
L173	Natural Channel	DS of NRSTOR7B (Lake Prince Drive)	3.09
L107	Natural Channel	DS of Crumps Mill Pond Spillway	2.46
L290.2	Natural Channel	DS of NNR343 (Five Mile Rd)	3.36
L113.1	Natural Channel	DS of NCC79 (Cherry Grove Rd N)	2.29
NR377.1	24"RCP	Kings Hwy, near Spring Meadow Ln.	7.19
NR379.1	30"HDPE	Kings Hwy, near Spring Meadow Ln.	5.64
NR373.1	36" RCP	Lone Star Lakes	12.74
NR362.1	24"RCP	Godwin Blvd.	5.83
NR343.1	24" RCP	Five Mile Rd	11.09
NR352.1	(2) 96" CMP	Wilroy Rd	2.87
NR320	36" RCP	Lake Prince Dr.	4.75
NR331.1	36" CMP	Lake Prince Dr.	7.67
Stor5	(2) 60" CMP	Everets Rd.	4.88
1146.1	24" RCP	Lone Star Lakes	5.66
1155.1	24" RCP	Cherry Grove Rd. N	5.70

4.1.1 Recommendations

Section 3 of this report summarized the stabilization recommendations for the outfalls and channels based on the exiting condition model and field reconnaissance. The table above indicates there is a slight increase in channel velocities for the proposed condition. However, based on the level of detail of the channel sections as discussed in Section 3, no additional recommendations for the channel reaches are being presented at this time.

4.2 Evaluation of Proposed Condition Roadway Culverts

The proposed condition roadway culverts were analyzed using the same design criteria as the existing condition model based on the data in the VDOT Drainage Manual. Primary and arterial roadways were evaluated for flooding using the 25-yr design storm and secondary roadways were evaluated using the 10-yr design storm. Table No. 4-2 below summarizes the peak design storm flood elevation at each culvert modeled. The deficient pipes from the existing condition model were the only deficient pipes for the 2018 condition model. No new culverts were identified with roadway overtopping for the appropriate design storm.

Table No. 4-2 Culvert Peak Design Storm Flood Elevations

Node Name	Conduit Type	Road Name	Roadway Designation	Approximate Top of Road Elevation (ft)	Design Flood Elevation (ft)
CCSTORAGE5	24" RCP	Cherry Grove Rd.	Secondary	5.50	5.89
CCLNSTAR	8'x10' BOX	Lone Star Lakes	Secondary	15.00	6.63
NRSTOR373	36" RCP	Lone Star Lakes	Secondary	11.00	12.38
NNR361	4'x4' BOX	Godwin Blvd.	Primary	63.00	57.98
NNR362	24"RCP	Godwin Blvd.	Primary	65.00	65.38
NNR377	24"RCP	Kings Hwy.	Primary	21.00	21.08
NNR379	30"HDPE	Kings Hwy.	Primary	21.00	20.91
NRSTOR15	36" RCP & 36" HDPE	Kings Hwy.	Primary	16.30	14.71
NNR343	24" RCP	Five Mile Rd.	Secondary	53.40	53.66
NNR353	24" CMP	SBS Greenhouses	Secondary	17.00	16.29
NNR352	96" CMP (x2)	Wilroy Rd.	Secondary	5.50	5.13
NNR320	36" RCP	Lake Prince Dr.	Secondary	71.00	69.62
NNR331	36" CMP	Lake Prince Dr.	Secondary	69.00	69.13
NRSTOR5	60" CMP (x2)	Everets Rd.	Secondary	24.60	24.54
NRSTOR11	22' SPILLWAY	Long Point Ln.	Secondary	9.50	9.57

Table No. 4-2 indicates that the design flood elevation for culvert crossing at nodes CCSTORAGE5, NRSTOR373, NNR362, NNR377, NNR343, NNR331, and NRSTOR11 overtop their respective roadways. The design flood elevation for culvert crossing at nodes NNR379, NNR352, and NRSTOR5 are within one foot of the top of their respective roadways. These results are similar to the existing condition model, no additional roadway culverts indicate flooding for the 2018 condition.

4.2.1 Recommendations

The existing condition recommendations for the culverts were analyzed for the 2018 land use condition. ***Generally, improvements recommended under the existing condition were adequate for the proposed condition (2018).*** The total costs for areas that require additional improvements are adjusted to account for escalation in prices when considering the timeframe of proposed improvements. The following summarizes the proposed improvements for the 2018 model.

- CCSTORAGE5: : In addition to the improvements recommended under Section 3, a third parallel 38" x 24" elliptical pipe should be installed to convey the 10-yr peak discharge. The adjusted cost of the culvert improvements would be approximately $131,572.

- NRSTOR373: The improvements proposed under Section 3 are adequate to address the 2018 land use condition.

- NNR362: The improvements recommended under Section 3 are adequate to convey the 25-yr design storm without overtopping the road or flooding nearby residential structures.

- NNR377: The improvements recommended under Section 3 are adequate to convey the 25-yr design storm without overtopping the road or flooding nearby residential structures.

- NNR379: The improvements recommended under Section 3 are adequate to convey the 25-yr design storm without overtopping the road or flooding nearby residential structures.

- NNR343: In addition to the improvements recommended under Section 3, a third 38" x 24" elliptical pipe should be installed to convey the 10-yr peak discharge. The adjusted cost of the culvert improvements would be approximately $102,003.

- NNR331: The improvements recommended under Section 3 are adequate to convey the 25-yr design storm without overtopping the road or flooding nearby residential structures.

- NRSTOR5: A more detailed study of the relationship between the upstream ponds and the culvert should be performed prior to developing any improvement strategies; therefore no recommendations for this culvert are being presented at this time.

- NRSTOR11: Since the pond is privately owned, no recommendations for improvements are being presented at this time.

- Regional Best Management Practices (BMPs): The comparative analysis of constructing a BMP versus making roadway drainage improvements indicates that a significant storage volume is typically required to attenuate sufficient flow to avoid making roadway drainage improvements. Because these costs would far exceed making roadway improvements, no recommendations are made to use storage ponds upstream of roadways. The cost of regional BMPs would be greater than the cost associated with upgrading the roadway culverts. Additionally, construction of regional BMPs reduces the amount of developable land and would require substantial right-of-way to construct and maintain.

Table No. 4-3 summarizes the existing and proposed improvements peak headwater elevations for the recommendations provided above.

Table No. 4-3 Comparison of Existing and Improved Culvert Peak Headwater Elevations – Proposed Condition (2018) Model

Node Name	Road Name	Roadway Designation	Approximate Top of Road Elevation (ft)	Existing Culvert Peak Headwater Elevation (ft)	Improved Culvert Peak Headwater Elevation (ft)
CCSTORAGE 5	Cherry Grove Rd.	Secondary	5.50	5.89	5.50
NNR362	Godwin Blvd.	Primary	65.00	65. 38	63.50
NNR377	Kings Hwy.	Primary	21.00	21.08	19.80
NNR379	Kings Hwy.	Primary	21.00	20.91	19.80
NNR343	Five Mile Rd.	Secondary	53.40	53.66	52.10
NNR331	Lake Prince Dr.	Secondary	29.50	69.13	67.20

4.3 EVALUATION OF PROPOSED CONDITION BRIDGES AND SPILLWAYS

The flood elevations at the bridges and dam spillways were evaluated using the 100-yr design storm. Table No. 4-4 below summarizes the 2018 condition 100-yr design flood elevation at the various bridges within the watershed.

Table No. 4-4 Bridge 100-yr Design Storm Flood Elevations

Node	Location	Bridge Number	Low Chord Elevation (ft)	Top of Road Elevation (ft)	100-yr Design Storm Elevation (ft)
NCC77	Godwin Blvd.	1802	9.50	12.00	10.60
NNR343	Godwin Blvd.	1800	13.10	19.80	5.13
NNR315	Girl Scout Rd.	8009	38.00	41.60	29.40
LAKEPRINCE	Girl Scout Rd.	8008	25.50	28.00	22.02
NNR312	Lake Prince Dr.	8007	27.50	30.00	29.41
NNR300	Gardner Ln.	8015	29.50	33.00	29.41
NNR305	Exeter Dr.	8010	31.50	33.00	29.41
NNR329	Everets Rd.	8001	23.50	26.00	22.02

As previously determined from the existing condition model, Bridge #1802 on Godwin Boulevard and Bridge # 8007 on Lake Prince Drive indicate the 100-yr design flood elevations are above low chord. Table No. 4-5 below summarizes the 100-yr design flood elevation at the reservoir spillways within the watershed. The model requires further evaluation to determine if this condition is plausible prior to making recommendations.

Table No. 4-5 Spillway 100-yr Design Storm Flood Elevations

Node	Location	Weir Length (ft)	Weir Elevation (ft)	Parapet Elevation (ft)	100-yr Design Storm Elevation (ft)
WBRES	Western Branch Reservoir	267	20.00	28.00	22.02
BURNTMILLS	Lake Burnt Mills	165	34.00	43.50	38.42
LAKEPRINCE	Lake Prince	174	26.00	30.00	29.41

The 100-yr design flood elevation at the bridges is a few tenths of a foot higher than the existing condition. The proposed 2018 land use does not seem to have a significant impact on the design flood elevations for the bridges or spillways. As with the existing condition model, the analysis does not account for any additional pumping facilities that may be employed at each dam.

4.3.1 Recommendations

Based on the results of the existing condition model and the need for a more detailed analysis regarding flood elevations for bridges and spillways, no recommendations are being presented at this time.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The recommendations proposed in the report are summarized in Table No. 5-1 below with associated construction cost estimates. These recommendations are based on model computations only and certain conditions discerned from the field investigation. Other conditions could exist that were not modeled due to the scope of the project and the intended usefulness of a stormwater master planning model. The improvements are not in a specific order. Most of the improvements recommended were from inadequate culverts that had the potential to overtop the roadway for the respective 24-hour rainfall event. The costs listed show the estimated value of improvements based on the existing condition and the proposed condition (2018)land uses. For those improvements where only one cost is listed, there were no additional improvements required to address the proposed condition (2018) land uses. The costs shown are based on 2005 dollars and would be greater in the future due to construction cost escalation.

Table No. 5-1 Summary of Erosion and Flood Control Recommendations

Location	Recommendation	Cost
Node CCSTORAGE5, Cherry Grove Rd.	Install two 38" x 24" HERCP culverts parallel to the existing 24" RCP (existing condition land use). A third 38" x 24" HERCP culvert is required for the 2018 land use condition.	$96,974 (existing) $131,572 (2018)
Node NNR362, Godwin Blvd.	Install a 24" RCP and 30" RCP parallel to the existing 24" RCP. The accumulated sediment in the existing 24" RCP should be cleaned out as an immediate remedial measure.	$89,062
Node NNR377, Kings Hwy.	Install two 24" RCP parallel to the existing 24" RCP and provide rip rap conduit outlet protection.	$54,040
Node NNR379, Kings Hwy.	Install a 30" RCP parallel to the existing the 30" HDPE culvert.	$42,708
Node NNR343, Five Mile Rd.	Install two 38" x 24" HERCP culverts parallel to the existing 24" RCP (existing condition landuse). Provide rip rap conduit outlet protection. A third 38" x 24" HERCP culvert is required for the 2018 land use condition.	$73,574 (existing) $102,003 (2018)
Node NNR 331, Lake Prince Dr.	Replace the existing 36" CMP with two 36" RCP culverts.	$52,006
Node NNR361, Godwin Blvd.	Provide erosion control and slope stabilization at the culvert.	$31,139
Node NCC77, Bridge 1802, Godwin Blvd.	Conduct a detailed study of the bridge to determine the if the scour has any impact on the structural integrity of the bridge and to develop the appropriate remedial measures	-

The model identified areas within the watershed that require a more detailed analysis to determine what flood control improvements are warranted. Due to the lack of detailed survey data and the master plan assumptions used, the model should not be used for design applications. Additional data and survey, particularly for roadway culverts, is needed before this model can be used for making specific design decisions and should only be used as a tool preparing the groundwork for more detailed studies and design.

Due to the nature of scour critical bridges, such as that which was seen in the field for Bridge #1802, a detailed bridge scour analysis should be performed. Due to the complex hydraulics associated with bridges, the stormwater master plan model is not adequate for making appropriate recommendations. Therefore, no cost is provided, only the need for a more detailed study.

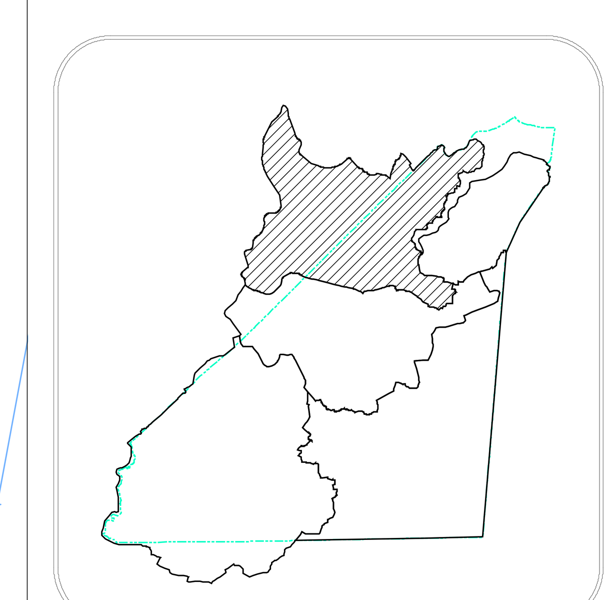
Similarly, channel improvements due to erosive velocities are provided only as general conditions that exist within the model. Since the model does not focus on variations in channel geometry, roughness, and unique site conditions throughout the channel reach, specific improvement priorities cannot be immediately obtained from the model. However, where recommendations for improvements are adequate is from the analysis of the pipes that show increased velocities at the outfalls and that the condition exists due to signs of erosion viewed in the field investigation. The cost of these improvements is included in the cost of the culvert improvements.

Regional best management practices (BMPs) were not closely evaluated at this stage of the model. Due to the limited number of deficient pipes and the limitations associated with making decisions on channel stability, the use of regional BMPs was not addressed in detail. Additionally, the cost for constructing a regional BMP would be greater versus the costs of making the pipe improvements. This increased cost is associated with land acquisition and large excavation and disposal hauling costs. The cost of constructing a regional BMP was therefore deemed unwarranted at this time. However, if a more detailed study of channel stability indicates that severe erosive condition exists along a major channel, the use of a regional BMP may be warranted over the cost for making extensive channel improvements and the associated cost in easement acquisition, property impacts and permitting. At this stage, we recommend a more detailed study be performed for those channels downstream of culverts recommended for improvements. This consideration is based on the increased flow volume released with the upgraded pipe systems and may therefore require downstream improvements beyond the distance assumed for initial cost of making the pipe improvements. A summary of the existing regional ponds input into the SWMM model is provided on Table No. 5-2.

Table No. 5-2 Summary of Modeled Regional Ponds - James River Watershed

Node	Location	Tributary Drainage Area (acres)	Normal Water Surface Elevation (ft)	Max Storage Capacity (acft)	10yr Storm Storage Volume (acft)	Outlet Configuration
CCRUMPS	Crumps Millpond	2,736	16	1390	680	45 ft weir
CCSTORAGE2	Off of Cherry Grove Rd	51	12	17	5	4-ft box and 100-ft weir
CCSTORAGE3	Off of Cherry Grove Rd	83	14	30	10	4-ft box and 45-ft weir
CCSTORAGE4	Off of Crittenden Rd	62	6	24	10	16-ft weir
CCSTORAGE6	Off of Crittenden Rd	84	8	38	21	4-ft box and 100-ft weir
NNR353	Off of Sack Point Rd	86	12	5	3	6 ft weir
NRCEDARLK	Cedar Lake	693	9	439	82	4-ft box and 79-ft weir
NRSTOR1	Off of Exeter Dr	105	48	77	9	4-ft box and 70-ft weir
NRSTOR1A	Off of Ennis Mill Rd	83	50	21	6	4-ft box and 38-ft weir
NRSTOR10	Hillpoint Commons	173	12	154	37	4-ft box and 100-ft weir
NRSTOR11	Long Point Lane	250	8	262	48	22 ft weir
NRSTOR12	Off of Godwin Blvd	50	22	5	0.3	25 ft weir
NRSTOR13	Off of Godwin Blvd	59	16	59	7	4-ft box and 16-ft weir
NRSTOR14	Off of Godwin Blvd	324	12	51	30	4-ft box and 119-ft weir
NRSTOR15	Saint Johns Church	246	12	23	7	12 ft weir
NRSTOR16	Ferry Point Rd	346	6	432	40	4-ft box and 200-ft weir
NRSTOR17	Ferry Point Rd	402	2	332	134	4-ft box and 162-ft weir
NRSTOR354	Off of Sack Point Rd	159	5	49	14	4-ft box and 285-ft weir
NRSTOR5	Kirk Pond	1,086	20	62	44	Twin 60" CMP
NRSTOR7	Kings Fork MS	73	50	44	5	4-ft box and 175-ft weir
NRSTOR7A	Lake Prince Farms	35	52	7	2	4-ft box and 26-ft weir
NRSTOR7B	Lake Prince Farms	40	53	23	4	4-ft box and 166-ft weir
NRSTOR9	Off of Godwin Blvd	61	9	44	5	4-ft box and 90-ft weir

Many of these ponds are located on private property or were inaccessible and were not included in the field reconnaissance. Where pond outlet structure data was unavailable, a 4' x 4' box riser was assumed with a crest elevation set at the existing water surface elevation as depicted in the GIS mapping. Weir dimensions were derived from GIS mapping.



Legend

- SUB BASIN BOUNDARY
- MAJOR_WATERSHEDS
- CITY OF SUFFOLK BOUNDARY

0 3,000 6,000 12,000 18,000 24,000 Feet

1"=6,000'

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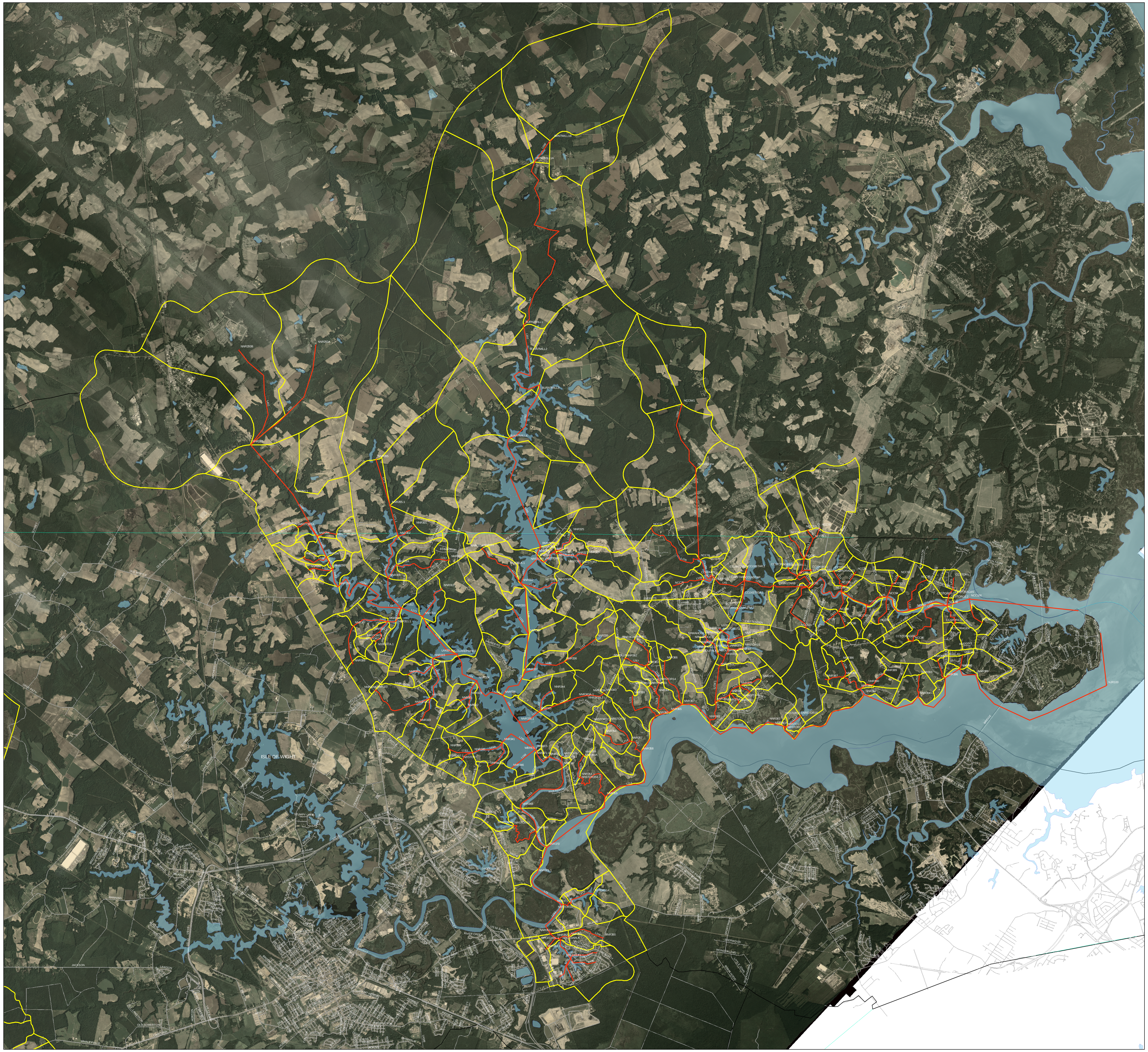
CITY OF SUFFOLK, VIRGINIA
STORM WATER MASTER PLAN
JAMES RIVER WATERSHED
STUDY AREA XP-SWWM MODEL

CN NO: N1955.2 E
DATE: 10/22/08
DESIGN: JPP
DRAWN: KRH
REVIEW: RAS
REVISIONS:

NO.	DATE	DESCRIPTION	BY

GIS-01

SHEET 1 OF 4



Legend

SWMM MODEL LINK

SWMM MODEL NODE

DRAINAGE_AREAS

MAJOR WATERSHED BOUNDARY

City of Suffolk Boundary

0 1,000 2,000 4,000 6,000 8,000 Feet

1"=2000'

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STORM WATER MASTER PLAN

PHASE I - JAMES RIVER WATERSHED

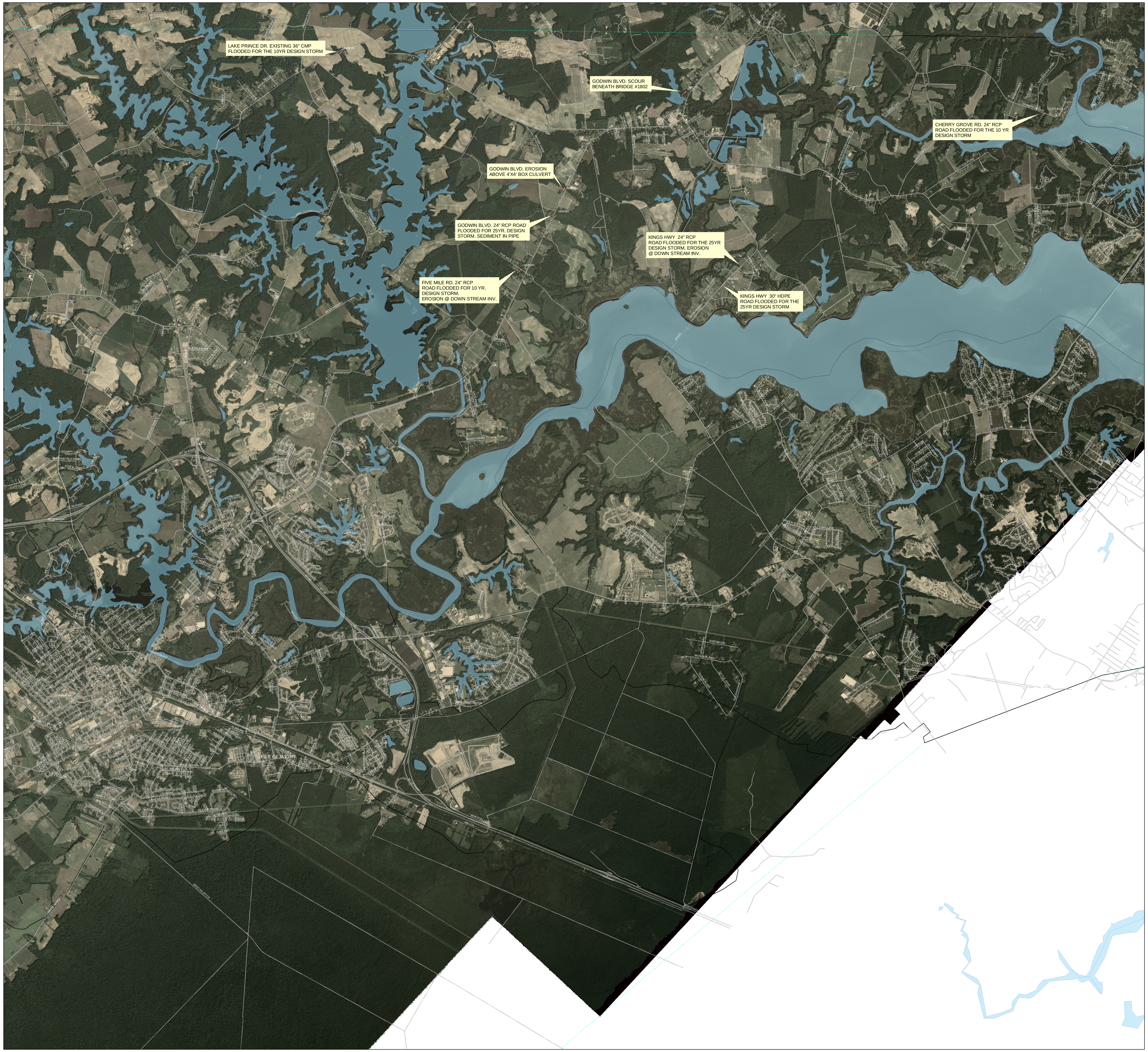
STUDY AREA XPSWMM MODEL

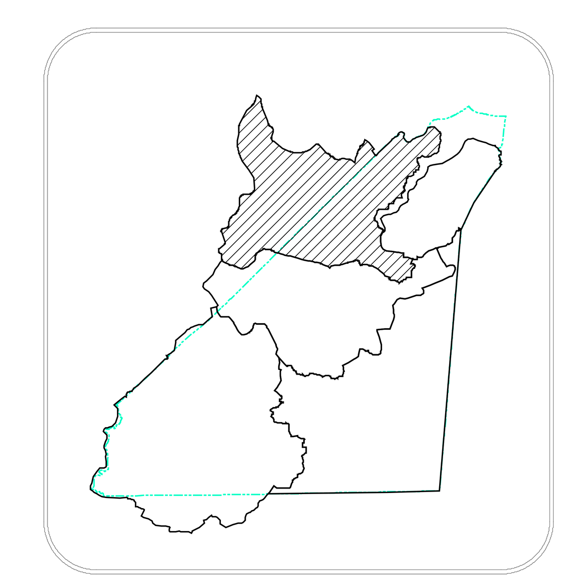
CN NO: N1955.2 D
DATE: 12/12/08
DESIGN: JPP
DRAWN: KRH
REVIEW: RAS
REVISIONS:

NO.	DATE	DESCRIPTION	BY

GIS-02

SHEET 2 OF 4





Legend

DEFFICIENCY_IMPROVEMENT

MAJOR WATERSHEDBOUNDARY

CITY OF SUFFOLK BOUNDARY

1"=2000'

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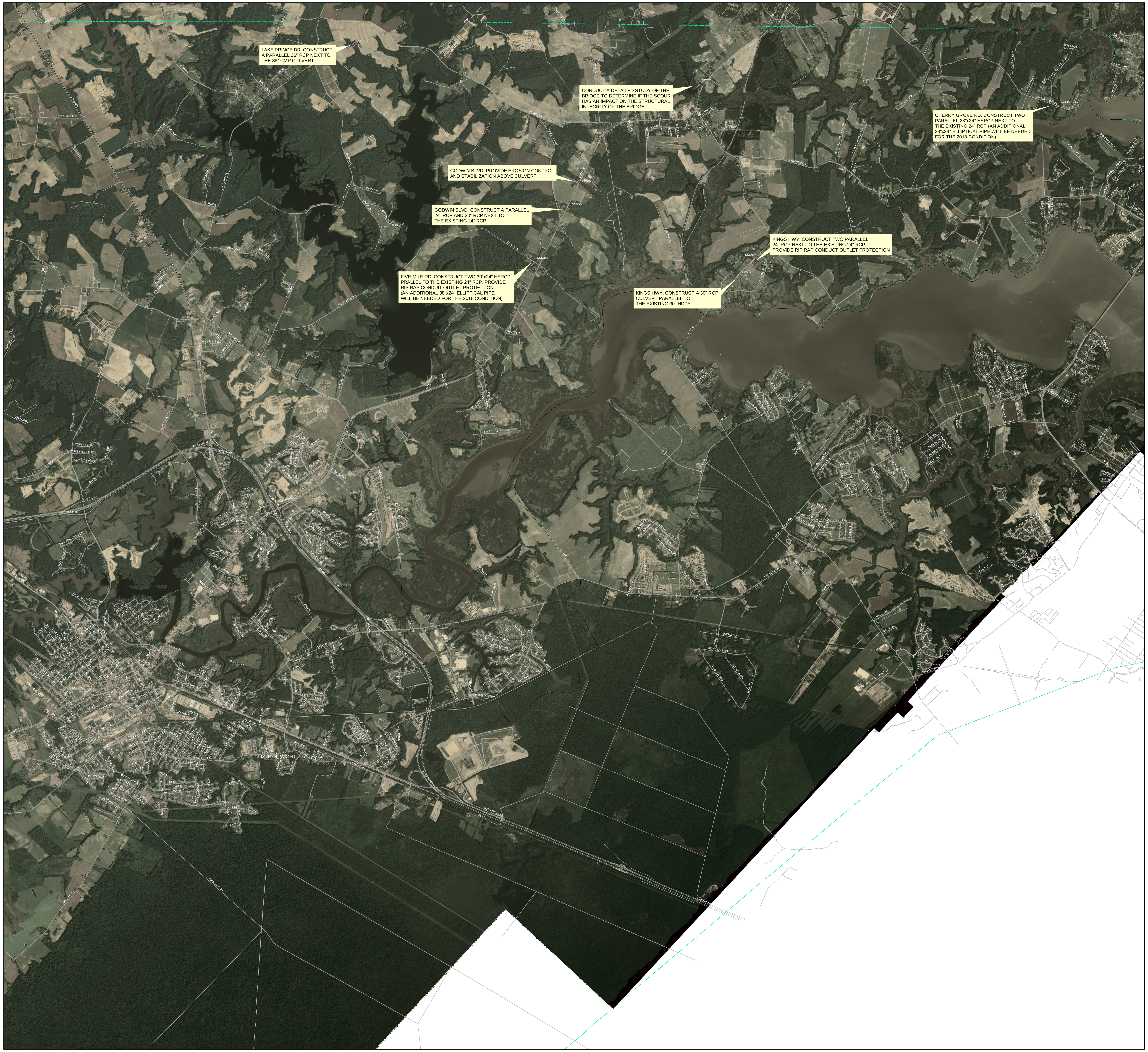
CITY OF SUFFOLK, VIRGINIA
STORM WATER MASTER PLAN
PHASE I - JAMES RIVER WATERSHED
STUDY AREA - DEFICIENY AREAS

CN NO: N1955.2 D
DATE: 12/12/08
DESIGN: JPP
DRAWN: KRH
REVIEW: RAS
REVISIONS:

NO.	DATE	DESCRIPTION	BY

GIS-03



Legend

- MAJOR WATERSHED
- CITY OF SUFFOLK BOUNDARY

0 1,000 2,000 4,000 6,000 8,000 Feet

1"=2000'

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CITY OF SUFFOLK, VIRGINIA
STORM WATER MASTER PLAN
PHASE I - JAMES RIVER WATERSHED
STUDY AREA - PROPOSED IMPROVEMENTS

CN NO: N1955.2 D
DATE: 12/12/08
DESIGN: JPP
DRAWN: KRH
REVIEW: RAS
REVISIONS:

NO.	DATE	DESCRIPTION	BY

GIS-04