Stormwater Master Plan

June 28, 2013

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EXECUTIVE SUMMARY

Project Purpose

The purpose of this Stormwater Master Plan is to identify any deficiencies in the existing stormwater management system and to recommend system improvements to alleviate flooding problems within public right of way areas throughout the study areas. The Stormwater Master Plan will allow the City of Pompano Beach to understand the necessary drainage improvements over the next few years and to budget accordingly. The purpose of this Stormwater Master Plan Update for the City of Pompano Beach can be defined as follows:

- Compile, review, and evaluate stormwater documentation related to the City's existing stormwater management system.
- Compile, review, and evaluate topographic elevation data for the study area.
- Coordinate with City during the development of the Floodplain Management Plan.
- Conduct topographic survey to identify the exact configuration, location, and elevation for all components of the City's stormwater management system.
- Update GIS geodatabase for the existing stormwater system based on survey results along with topographic data collected from South Florida Water Management District (SFWMD), Florida Department of Transportation (FDOT), and Broward County.
- Coordinate with regulatory agencies along with the general public as part of the Floodplain Management Planning Committee (FMPC).
- Conduct water quality data analysis of stormwater runoff within the City.
- Identify system improvement goals and analyze system improvement alternatives.
- Create Stormwater Master Plan which includes conceptual design layouts and cost estimates for system improvement alternatives.
- Prepare CIP implementation plan for stormwater improvements throughout the City.
- Review the City's stormwater utility fee.
- Coordinate with Florida Department of Environmental Protection (FDEP) for inclusion on the funded projects list for state revolving loan funds.
- Coordinate with City on Federal Emergency Management Agency (FEMA) Community Rating System (CRS) documentation.

Data Collection

The initial task for the Stormwater Master Plan was to collect and evaluate all available information on the existing conditions within the City limits and on the existing stormwater management system. The purpose of the information gathering phase is to ensure that the most recent and detailed data on the City's existing stormwater management system is used during the completion of this project. CMA used this new information to create a hydraulic and hydrologic stormwater model of the existing conditions, which allowed us to conduct an analysis of the performance of the existing stormwater management system; to better identify any deficiencies with the existing stormwater management system; and to recommend system improvements.

A task within this Stormwater Master Plan was to complete a full GPS topographic survey to verify the exact configuration, location, and/or elevation for all components of the stormwater management system. CMA retained Stoner and Associates, a licensed land surveyor, to conduct GPS field surveys of all City stormwater structures defined within the City's stormwater atlas. Stoner and Associates obtained the horizontal coordinates and vertical elevation data for each City stormwater structure. CMA updated the GIS Stormwater Geodatabase with all GPS data collected during the field verification of these drainage structures on a monthly basis.

System Requirements

CMA used a level of service (LOS) criteria for stormwater management systems as defined by the regulatory agencies with jurisdiction within the City of Pompano Beach. The level of service criteria for the stormwater management system within the City was defined based on the guidelines from SFWMD and Broward County EPGM. The most stringent requirement from these regulatory agencies was incorporated into the modeling analysis for the study area to ensure a conservative approach to evaluating the existing infrastructure. Please note the LOS criteria only apply to new development and new construction since a large portion of the City was developed prior the implementation of stormwater management regulations. The stormwater model was used to verify whether existing drainage systems meet the following LOS requirements:

- Building Structures – The flood level shall not exceed the finish floor elevation of all building structures within the study area during the 100-year, 3-day storm event.
- Roadways and Parking Lots – Stormwater ponding shall not encroach onto any roadway centerlines during the 10-year, 1-day storm event. Stormwater ponding shall not encroach onto any roadway edge of pavement during the 5-year, 1-day storm event.
- Outfall Discharges – The outfall discharge shall not exceed the allowable peak discharge during a 25-year, 3-day rainfall event as defined by the existing permit requirements. If the existing outfalls do not have a maximum discharge assigned by existing permits, the regulatory agencies will require "pre-development" versus "post-development" discharge analysis to ensure the stormwater discharge into adjacent surface waters does not increase after the proposed construction.

In addition to the quantity of stormwater runoff, the regulatory agencies also define the level of water quality treatment provided to stormwater runoff required prior to discharge via outfalls into adjacent surface water bodies. The water quality criteria for the City are based on the SFWMD and Broward County standards. These standards require treatment of the first inch of stormwater runoff generated from the entire site area or 2.5 inches of stormwater runoff generated from all impervious areas within the site area (whichever is greater). Typical methods for providing water quality treatment of stormwater runoff are the installation of dry retention areas, grass swales, and exfiltration trench to adequate storage volume to meet these requirements.

System Evaluation

The purpose of the Stormwater Master Plan Update is to evaluate the performance of the existing stormwater management system, to determine the most effective methods for minimizing flooding within the City, and to evaluate design alternatives for potential improvements to the stormwater management system. An important component of this project is the creation of a hydraulic and hydrologic computer model of the City's stormwater management system. The system evaluation summarizes the input data, model assumptions, and analysis methods used during the update of the stormwater model. The stormwater model was used to provide a better understanding of the stormwater management system performance under existing conditions and to prioritize the sub-basins through a prioritization formula and identify "study areas" which may require future stormwater improvements, which is further defined within this report.

System Improvement Alternatives

CMA has developed various system improvement alternatives for areas throughout the City in need of stormwater improvements based our analysis of the existing conditions. The goal of these system improvement alternatives is to meet level of service criteria for flood control of the public right of way areas along with providing additional water quality benefits. These various system improvement

alternatives which were considered for these areas include the installation of exfiltration trench systems, the interconnection with the adjacent existing stormwater systems, the upsizing of existing stormwater pipes, the construction of retention areas, the installation of stormwater pump stations, the implementation of backflow prevention devices at existing outfalls, the installation of drainage wells, and the regrading of roadway swale areas. Each of these potential system improvement alternatives were evaluated in multiple configurations within each area to determine which would be the most effective at alleviating the existing flooding within the public right of way areas. For comparison purposes, CMA used the stormwater model to analyze the effectiveness of these system improvement alternatives at improving the performance to the existing stormwater management system within each area of the City. Each system improvement alternative was compared based on its ability to reduce the peak flood stage within the study area and to reduce the flood duration within the study area. This analysis, conceptual design and cost estimates for each study area is further defined within this report.

Recommendations

CMA prepared this Stormwater Master Plan for the entire limits for the City of Pompano Beach. The purpose of this Stormwater Master Plan was to identify any deficiencies in the existing stormwater management system and to recommend system improvements to alleviate flooding issues throughout the City. Based on our analysis with the stormwater model, CMA has developed the recommended system improvement alternatives for each study area within the City of Pompano Beach. CMA has prepared a conceptual layout for the recommended system improvements within each study area along with a preliminary cost estimate for the implementation of each recommended system improvement. The recommended system improvements within each study area are defined within Section 5 of this report.

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SECTION 1 - INTRODUCTION

1.1 GENERAL

The City of Pompano Beach is located within Broward County, Florida. The general geographic boundaries of the City of Pompano Beach are the Atlantic Ocean on the east, Florida Turnpike on the west, McNab Road on the south, and Sample Road on the north, except for another section between Federal Highway and Dixie Highway which extends north to NE 54th Street. The City limits encompass approximately 24.6 square miles. The limits of the Stormwater Master Plan are defined in Figure 1-1 – Project Limits Map within this section. The City of Pompano Beach is fully developed with chiefly residential properties mixed with commercial properties along with some agricultural, industrial, institutional and government owned properties. The project area includes approximately 15,736 acres of land which are separated into 30,011 separate properties. The City of Pompano Beach operates and maintains its own stormwater management facilities within City right-of-way and properties to provide flood control and water quality treatment within the City limits. Existing drainage facilities within the City include catch basins, manholes, control structures, gravity pipes, exfiltration trench, retention areas, outfalls, and canals that connect to the Intracoastal Waterway. The City has a stormwater utility fee in place to provide funding for the operation and maintenance of the existing stormwater system along with funding any regulatory permitting and limited stormwater improvements.

Chen Moore and Associates (CMA) was retained by the City of Pompano Beach in July 2011 to prepare a Stormwater Master Plan for the entire City limits. The purpose of this Stormwater Master Plan is to identify any deficiencies in the existing stormwater management system and to recommend system improvements to alleviate flooding problems within public right of way areas throughout the study area, and address regulatory compliance issues. Within the Stormwater Master Plan CMA shall provide recommendations for improvements to the system that will reduce the flooding issues currently encountered within various right-of-way areas during or after rainfall events. The Stormwater Master Plan will define the existing stormwater management system; summarize the results of the stormwater model for the existing conditions; prioritize the proposed improvements to the stormwater management system; and provide an estimated cost to construct these upgrades to the stormwater management system. The Stormwater Master Plan will allow the City of Pompano Beach to understand the necessary drainage improvements over the next few years and to budget accordingly.

1.2 PROJECT PURPOSE

The purpose of this Stormwater Master Plan for the City of Pompano Beach includes the following tasks:

- Compile, review, and evaluate stormwater documentation related to the City's existing stormwater management system.
- Compile, review, and evaluate topographic elevation data for the study area.
- Coordinate with City during the development of the Floodplain Management Plan.
- Conduct topographic survey to identify the exact configuration, location, and elevation for all components of the City's stormwater management system.
- Update GIS geodatabase for the existing stormwater system based on survey results along with topographic data collected from South Florida Water Management District (SFWMD), Florida Department of Transportation (FDOT), and Broward County.
- Coordinate with regulatory agencies along with the general public as part of the Floodplain Management Planning Committee (FMPC).
- Conduct water quality data analysis of stormwater runoff within the City.
- Identify system improvement goals and analyze system improvement alternatives.
- Create Stormwater Master Plan which includes conceptual design layouts and cost estimates for system improvement alternatives.

- Prepare CIP implementation plan for stormwater improvements throughout the City.
- Review the City's stormwater utility fee.
- Coordinate with Florida Department of Environmental Protection (FDEP) for inclusion on the funded projects list for state revolving loan funds.
- Coordinate with City on Federal Emergency Management Agency (FEMA) Community Rating System (CRS) documentation.

1.3 PROJECT SCHEDULE

The project timeline for the Stormwater Master Plan for the City of Pompano Beach is listed below according to project milestones and deliverable submittals.

- July 5, 2011 — Authorization to Proceed
- August 9, 2011 — Project Kickoff Meeting with City
- October 14, 2011 — Data Collection Technical Memorandum
- November 21, 2011 — Data Collection Review Meeting with City
- February 29, 2012 — Existing Conditions Stormwater Model Technical Memorandum
- March 26, 2012 — Existing Conditions Stormwater Model Review Meeting with City
- April 19, 2012 — Floodplain Management Planning Committee Kickoff Meeting
- May 14, 2012 — Floodplain Management Planning Committee Public Outreach #1
- May 18, 2012 — Existing Conditions Stormwater Model Calibration
- June 8, 2012 — Basin Prioritization Technical Memorandum
- June 26, 2012 — Basin Prioritization Review Meeting with City
- July 23, 2012 — Revised Basin Prioritization Technical Memorandum
- August 8-9, 2012 — Floodplain Management Planning Committee Public Outreach #2
- August 28, 2012 — System Improvement Alternatives Technical Memorandum
- August 28, 2012 — System Improvement Alternatives Conceptual Layouts
- August 28, 2012 — System Improvement Alternatives Preliminary Cost Estimates
- September 7, 2012 — System Improvement Alternatives Review Meeting with City
- September 21, 2012 — Draft Stormwater Master Plan
- November 6, 2012 — Draft Stormwater Master Plan Review Meeting with City
- December 11, 2012 — Final Stormwater Master Plan
- April 9, 2013 — Public Presentation to City Commission
- June 2013 — Adoption of Stormwater Master Plan by City Commission

1.4 PROJECT TEAM

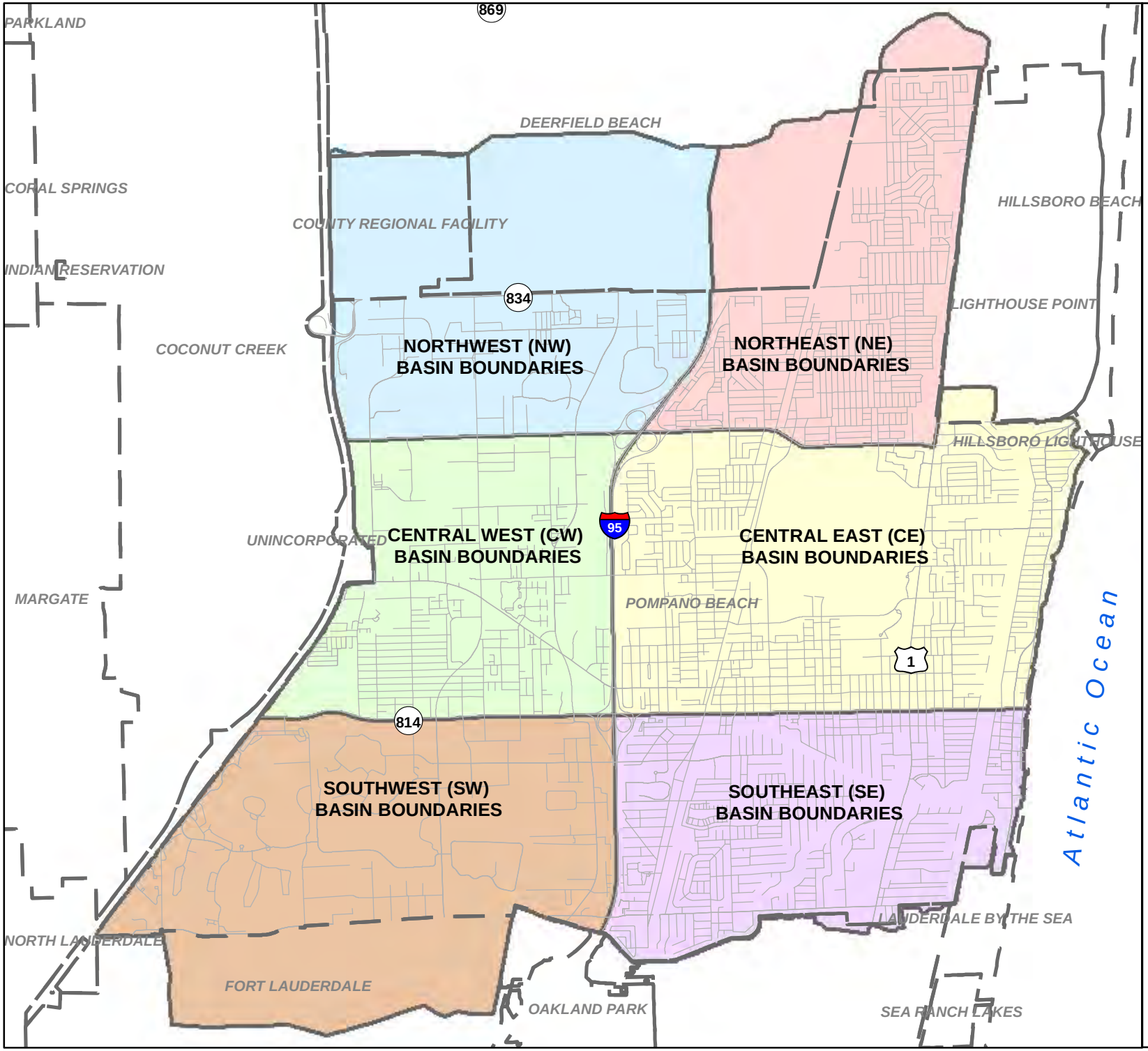
Chen Moore and Associates is the lead consultant for the City of Pompano Beach for this Stormwater Master Plan. The Project Team for Chen Moore and Associates includes the following key individuals:

- Principal in Charge: Peter Moore, P.E., LEED AP
- Project Manager: Jason McClair, P.E., LEED AP, CFM
- Project Engineer: Jennifer Smith, P.E.
- Project Engineer: Gabriel Bacca-Cortes, P.E.
- Project Engineer: Suzanne Dombrowski, P.E.

Chen Moore and Associates has also retained the following subconsultants to be part of the Project Team:

- Topographic Survey:
  Stoner and Associates, Inc.
  4341 SW 62nd Avenue
  Davie, Florida 33314
  Project Contact: James Stoner

- Water Quality Analysis:
  AMEC - BCI Engineers & Scientists, Inc.
  118 West Reynolds Street
  Plant City, FL 33563
  Project Contact: Timothy Kelly



City of Pompano Beach
Stormwater Master Plan

Figure 1-1
Project Limits Map

Legend

- City Boundary
- City Streets
- Basin CE Boundary
- Basin CW Boundary
- Basin NE Boundary
- Basin NW Boundary
- Basin SE Boundary
- Basin SW Boundary

N
W E
S

0 1,500 3,000 6,000
Feet

SECTION 2 – DATA COLLECTION

2.1 OVERVIEW

CMA gathered and evaluated available information on the existing stormwater management system within the City limits. The study area, which includes the City limits along with some adjacent drainage basins in neighboring municipalities, is displayed within Figure 1-1 Project Limits Map. The purpose of the information gathering phase was to ensure that the most recent and detailed data on the City's existing stormwater management system is used during the completion of this project. CMA used this new information to create a hydraulic and hydrologic stormwater model of the existing conditions, which allowed us to conduct an analysis of the performance of the existing stormwater management system; to better identify any deficiencies with the existing stormwater management system; and to recommend system improvements. With the assistance of City staff, CMA collected available information on the existing stormwater management system within the City. The initial task for this project included the collection and evaluation of all available information on the existing conditions within the City limits and on the existing stormwater management system. In order to develop the stormwater model for the existing conditions, CMA collected and evaluated available information, which included the following items:

- 1999 Stormwater Master Plan
- 2002 Alternatives Development and Conceptual Design of Basins Ranked 21-40
- 2010 Stormwater Master Plan Update For Basins 41-60
- SFWMD Rainfall Data
- SFWMD Canal Level Data
- SFWMD Groundwater Level Data
- Relevant stormwater permits from SFWMD and Broward County EPGM
- FEMA Flood Elevation Data
- Natural Resources Soil Conservation (NRCS) Soil Survey Data
- National Oceanic and Atmospheric Administration (NOAA) Tidal Data
- Light Detection and Ranging (LiDAR) Topographic Data
- City Land Use and Zoning Data
- City Flooding Complaint Data
- City Drainage Atlas in Geographic Information Systems (GIS) format
- Previous City Submittals for FEMA CRS
- City NPDES Permit Documentation
- Old Pompano Canal TMDL Data relative to Impaired Waterbody Status
- City Maintenance Records for Drainage Infrastructure
- Available as-built and design drawings for stormwater improvements within the City
- Available stormwater atlases from other agencies (SFWMD, FDOT, Broward County)

The deliverable for Task 1 – Evaluation of Existing Information was a technical memorandum which was submitted to the City on October 14, 2011. The purpose of this technical memorandum was to summarize all available information on the stormwater management system collected during this task. CMA also evaluated any information that seemed irregular or which may have affected the development of the stormwater model. All data collected by CMA is summarized in more detail within the following sections.

2.2 PREVIOUS STORMWATER MASTER PLANS

The original Stormwater Management Master Plan was prepared in August of 1999 by PBS&J. The 1999 Stormwater Management Master Plan identified over 100 drainage basins throughout the City and prioritized each drainage basin based on the need for stormwater improvements. The purpose of the original plan was to identify, rank and prioritize problem areas and identify improvements for the 20 highest priority problem areas, as defined by the prioritization ranking formula. This priority ranking formula (S = 0.4*HDPA + 0.2*(PDCR+FZE+DA) + 0.15*WQ + 0.25*RP) incorporated the following factors for consideration:

- Overall sub-basin score (S)
- Historic drainage problem areas score (HDPA)
- Peak discharge-to-outlet capacity ratio (PDCR)
- Drainage availably score (DA)
- Flood zone elevations score (FZE)
- Water quality score (WQ)
- Recharge potential score (RP)

The 1999 Stormwater Master Plan included gathering information on the existing stormwater management system, which included topographic survey data from City sanitary sewer atlas, City utility atlas map, Broward County aerial maps with outfall locations, City land use map, City resident complaint list, Broward County Water Management District (WMD) control water surface elevations with culvert sizes, NPDES outfall information, SFWMD hydrologic soil groups, and *United States Geological Survey* (USGS) topographic maps. Using the topographic survey information of the City's sanitary sewer system, a total of 112 sub-basins were delineated within the study area. The top 20 priority basins were identified and analyzed further for the development of conceptual layouts and cost estimates for recommended stormwater improvements. The total construction costs for the top 20 sub-basins were estimated to be $4,175,944 in 1999.

The Stormwater Management Master Plan was updated in 2002 by PBS&J with a report titled "Alternatives Development and Conceptual Design of Basins Ranked 21-40," which identified the drainage improvements for these basins. The purpose of this report was to identify improvements for these sub-basins ranked 21-40 in the 1999 Stormwater Master Plan and re-prioritize these basins according to the priority ranking formula for sub-basins 21-40 (S = 0.3*NCA + 0.3*CPS + 0.2*RP + 0.1*PI + 0.05*WQ +0.05*A), which incorporated the following factors:

- Overall sub-basin score (S)
- Number of complaints per area score (NCA)
- Costs per area served score (CPS)
- Recharge Potential score (RP)
- Permitting Issues score (PI)
- Water Quality score (WQ)
- Aesthetics (A)

The 2002 Stormwater Master Plan update used the topographic survey information from the City sanitary sewer system, soil types and land use data collected in the original plan to create conceptual layouts and develop construction cost estimates for sub-basins 21-40. The total construction cost for sub-basins 21-40 was estimated to be $13,816,062.

In 2010, the Stormwater Master Plan was updated by Chen Moore and Associates to identify improvements for the subsequent 20 sub-basins. The purpose of this update to the Stormwater Master

Plan was to identify any deficiencies in the existing stormwater management system and to recommend system improvements to meet regulatory Level of Service (LOS) criteria for the Basins Ranked 41-60 within the 1999 Stormwater Management Master Plan. This update included creating a Triangulated Irregular Network (TIN) of the ground surface elevations, obtaining updated soil parameters, obtaining the existing GIS drainage atlas, calculating updated hydraulic parameters, reviewing complaint data, and interviewing City staff. A Storm Water Management Model (SWMM) was created for each of these basins, which was used to assist with the development of a conceptual design and cost estimates for these sub-basins. The total construction cost was estimated to be $7,208,604 for the sub-basins 41-60.

For further reference, a digital version of these previous stormwater master plan documents (1999 Stormwater Master Plan, 2002 Alternatives Development and Conceptual Design of Basins Ranked 21-40, 2010 Stormwater Master Plan Update for Basin 41-60) has been included in PDF format within Digital Appendix B-3 – Previous Stormwater Master Plan and Updates on a computer disc enclosed with this report.

2.3 BASIN DELINEATION

During the preparation of the 1999 Stormwater Master Plan, PBS&J divided the City into six major drainage basins: Northeast (NE), Northwest (NW), Southeast (SE), Southwest (SW), Central (CT), and North Central (NC). For this project, these major basin designations are maintained for continuity purposes. Based on the available data for the study including a digital elevation model, topographic survey data, aerial photography, and the existing drainage atlas, CMA further divided these six major drainage basins into sub-basin areas to enhance the accuracy of the stormwater model. Any basin and sub-basin boundary was delineated by drawing a basin divide through high terrain. The delineation process commenced with the identification of the major drainage network features, such as large diameter pipes that discharge to canals, marine finger canals, or drainage ditches. Consequently, a basin boundary surrounding these features is delineated to be later subdivided by as many sub-basins as dictated by the existing ground surface topography. In some areas where the drainage network consisted of a single pipe draining only the most adjacent neighborhood area to the Intracoastal Waterway or freshwater canal, the drained area was included in a single sub-basin. Even though these areas are relatively small, each outfall was assigned its own sub-basin per the scope requirements. Other areas that were not part of either major drainage network outfall or immediately adjacent to canals are isolated areas that do not have a drainage system but may or may not drain into either one of the above categories via overland flow depending upon the local topographic conditions. This latter classification makes up for most of the sub-basins delineated within the study area. Considering that topography has no regard for City limits, a few of the sub-basins delineated within the study area extend over the City limits in order to account for potential incoming or outgoing stormwater runoff from and to these areas.

As defined in more detail later in Section 4 this report, CMA delineated a total of 621 sub-basins within the study area, which were incorporated into the stormwater model. The sub-basins within the study area vary in size from 1.5 acres in residential neighborhood areas to up to 420 acres around golf courses. A nomenclature system was established to assign a unique name to each sub-basin within the study area. Each sub-basin name has the following nomenclature structure: **MB_OUT_SB**. Within this nomenclature structure, **MB** is correlated to the major basin classification that was used from the previous stormwater master plan, **OUT** is correlated to a three digit sequential number assigned to each outfall with a sequence maintained within each major basin and assigning an outfall number to the 'isolated' sub-basins category discussed previously, and SB is correlated to a two digit sub-basin number associated to the location of the sub-basin with respect to its outfall assigning a higher number to sub-basins located farther away from the outfall. These sub-basins and names are displayed in Figure 2-1 Topography Map within this section.

2.4 AERIAL PHOTOGRAPHY

High resolution aerial photography was obtained from Broward County for the study area. The aerial photographs can be expected to represent the conditions of the year they were flown, which was during 2011. The aerial photograph for the study area is of 1-foot resolution. The aerial photography was used during this project to assist with verification of imperviousness, land use, curve number assignations, and mapping during the development of the stormwater model.

2.5 TOPOGRAPHIC DATA

The City of Pompano Beach exhibits typical topography for southeast Florida with minimal ground slope, low coastal elevations, and a canal network which provides primary drainage capacity for the area. The topographic elevation information within the City is important for the development of the Stormwater Master Plan since it defines the areas prone to flooding during storm events and controls the type of stormwater management system that is needed to meet regulatory criteria. The topographic information for the City is used to identify low lying areas within the City where stormwater runoff from higher areas will most likely accumulate. This topographic information, together with the existing stormwater network, is also used in the development of a stormwater model to help delineate drainage basins within the study area based on the changes in the elevation of the ground surface around the existing network.

CMA reviewed the 1999 City of Pompano Beach Stormwater Master Plan document to verify the topographic data that was previously used. According to the 1999 Stormwater Master Plan document, topographic data was obtained from SFWMD GIS coverage and City sanitary sewer system topographic data. More accurate elevation data for the entire County has been made available to the general public since the completion of the 1999 Stormwater Master Plan. This new topographic data will enhance the accuracy of the stormwater model for the study area.

CMA obtained digital topographic data within the study area, which was developed using LiDAR technology by the Florida Department of Environmental Protection (FDEP). This technology uses laser pulses from aerial sources to estimate the elevation of a specific point on the ground surface. LiDAR data provides a means to obtain ground elevations in areas where these data were not available before. Typically, there are two versions of LiDAR data, “raw data” and “bare earth data.” The “raw data” version is unmodified data and can reflect the elevation of various aboveground features, such as small vegetation, tree canopy, buildings, and other urban features. The modified version of this “raw data” eliminates the elevation of these aboveground features and replaces them with an estimated ground elevation via interpolation of surrounding elevation points, which is known as “bare earth data.” The estimation of the underlying ground elevation is made by applying one from a wide selection of algorithms that are currently available for this purpose. The accuracy of bare earth LiDAR data is mostly associated with the quality of equipment and procedures used for the data collection and the algorithm used to remove aboveground features. Therefore, it is expected that the accuracy of collected LiDAR data is better in areas where the algorithm is not applied in unobstructed open areas such as roadways, walkways, vacant grassy areas, or any other open land. In order to verify the level of accuracy of this LiDAR data, CMA correlated the LiDAR data against available survey elevation data from various locations throughout the study area. Based on our comparison with available survey elevation data, LiDAR data for the study area was found to have an acceptable level of accuracy for use within the stormwater model.

A Triangulated Irregular Network (TIN) model of the ground surface, which is also known as a digital elevation model (DEM), was created from the bare earth elevation points from the study area at a resolution of 10-foot cell size. The vertical datum of the LiDAR and the DEM elevation data generated from it is the North American Vertical Datum (NAVD) from 1988, which is also known as NAVD88. This topographic data set establishes NAVD88 to be the vertical datum for this project. The DEM was

color coded to show the variation of the ground surface elevation throughout the City as a contour map, which is displayed within Figure 2-1 Topography Map within this section. The DEM of the study area shows that the lowest lying areas are in the southeast and eastern portion of the study area. This DEM was used to help delineate the drainage basins and establish stage-area relationships required by storage elements within each drainage basin, which were incorporated into the stormwater model.

2.6 STORMWATER ATLAS

In 2009, CMA digitized a hard copy of the City's utility atlas into a digital GIS database, which included all stormwater, sewer and watermain facilities. Since the City has updated the database since 2009, the City staff provided the most recent drainage atlas to CMA in GIS format. This stormwater atlas depicted the geographic locations of the catch basin inlets, manhole structures, control structures, drainage piping, and outfalls which are owned and maintained by the City, along with some limited FDOT, Broward, and private drainage infrastructure within the City limit. The stormwater atlas also provides dimensions of some of the existing network elements in the form of pipe diameter and pipe length. The updated stormwater atlas for the City of Pompano Beach is shown in Figure 2-2 Existing Drainage Atlas within this section. CMA has reviewed the City's stormwater atlas and found the information to be comprehensive for the entire City limits. A summary of the City's stormwater management system components included within the updated stormwater atlas is listed within Table 2.1 – Existing Stormwater System Components below.

Table 2.1 – Existing Stormwater System Components

Drainage Component	**Quantity**
Outfalls	399 EA
Total Gravity Main Pipe	525,254 LF 99.5 miles
Gravity Main Pipe (>24-inch)	77,976 LF 14.8 miles
Exfiltration Trench	96,712 LF 18.3 miles
Catch Basin Inlets	5,041 EA
Drainage Manholes	810 EA
Control Structures	17 EA

2.7 OUTFALL LOCATIONS

The information on the positive outfalls from the City's stormwater management system is a crucial component of the existing stormwater model. If an outfall is not properly sized or operating correctly, it will negatively impact the performance of the entire drainage network and could cause additional flooding, regardless of any direct pipe connections to the outfalls. The drainage atlas includes the location of all positive outfalls to surface water bodies within the City limits. According to the City's stormwater atlas, there are 399 outfalls to surface water bodies within the City. The location of the outfalls is displayed on Figure 2-2 Existing Drainage Atlas within this section. The outfall information within the drainage atlas includes a location description and whether or not the outfall remains active at most locations. However, dimensions of the discharging conduit were not included at all locations. During the field investigation task, the location, pipe diameter, and pipe invert of all accessible outfalls were confirmed by the surveyor.

2.8 OTHER STORMWATER FACILITIES

CMA obtained additional as-built drawings and/or design plans for recently constructed drainage projects within the City and incorporated any relevant information on the stormwater infrastructure into the GIS Stormwater Geodatabase. CMA obtained drawings on existing drainage infrastructure within the City limits from the following sources:

- City of Pompano Beach – CMA obtained as-built drawings of drainage improvement projects constructed since the last drainage atlas update from the City for incorporation into the GIS Stormwater Geodatabase. For future reference, these City as-built drawings are included within the digital Appendix B-5 – Past Drainage Project Drawings attached to this report. These as-built drawings were incorporated into the City's GIS Stormwater Database and linked to the stormwater structures within the geodatabase for future reference.
- Broward County Water and Wastewater Services (WWS) – CMA contacted Broward County WWS to collect any available as-built drawings or design plans on any stormwater improvements constructed as part of the neighborhood improvement projects within the City limits. Broward County WWS and their engineering consultants were able to provide some drainage information on the various neighborhood improvement projects completed within the City limits. These neighborhood improvement projects were recently completed or are currently under construction within the recently annexed area in the northern limits of the City. These Broward County drainage project drawings are included in digital Appendix B-6 – Broward County WWS Drainage As-Built Drawings. Any relevant information on the drainage infrastructure, which is either recently installed or currently under construction, was incorporated into the City's GIS Stormwater Database.
- Broward County Highway Construction and Engineering Department (HCED) – CMA contacted Broward County HCED to collect any available drainage atlases or as-built drawings for all County roadways located within the City limits. Broward County provided atlas drawings for Copans Road and portions of Andrews Avenue. These atlas drawings from Broward County are included in digital Appendix B-7 – Broward County HCED Atlas Drawings. Any relevant information on the existing drainage infrastructure within these County right of way areas was incorporated into the City's GIS Stormwater Database for future reference by the City.
- Florida Department of Transportation (FDOT) – CMA contacted Broward County HCED to collect any available drainage atlases or as-built drawings for all County roadways located within the City limits. Although atlas drawings were not provided for all FDOT roadways within the City, a significant amount of drainage information was provided for portions of FDOT roadways, including US1, I-95, USA1A, and Atlantic Boulevard. These atlas drawings are included in digital Appendix B-8 – FDOT Atlas Drawings. Any relevant information on the existing County drainage infrastructure within these County right of way areas was incorporated into the City's GIS Stormwater Database for future reference by the City.

2.9 SURVEY VERIFICATION

In order to verify the exact location and attributes of the City's existing stormwater management system, a full GPS topographic survey is necessary to verify the exact configuration, location, and/or elevation for all components of the stormwater management system. CMA retained Stoner and Associates, a licensed land surveyor, to conduct GPS field surveys of all City stormwater structures defined within the City's stormwater atlas. Stoner and Associates obtained the horizontal coordinates and vertical elevation data for each City stormwater structure. CMA completed routine updates of the GIS Stormwater Geodatabase based on the collected survey data, which was provided by Stoner and Associates in monthly updates for the project duration.

Prior to collecting the drainage as-built elevation data to the appropriate level of accuracy, Stoner and Associates needed to derive quality GPS elevations from the local elevation control. Stoner and Associates conducted level runs throughout the City limits to elevate existing horizontal control as established by both the Broward County Engineering Department (BCED) and the National Geodetic Survey Department (NGS). The Dixie Highway corridor (State Road 811) has an abundance of existing benchmarks set by NGS, which are related to the North American Vertical Datum of 1988 (NAVD 88). There are additional existing benchmarks located along Atlantic Boulevard (State Road 845) and State Road A1A, which are set by NGS. The elevation control was conducted to achieve second order accuracy and meet the ERMS Program. The level run would also turn through existing Broward County GPS points that already meet the monumentation requirements for the ERMS program. Stoner and Associates conducted approximately 45 miles of bench runs to develop adequate elevation control. Once the elevation control was appropriately established, Stoner and Associates conducted the GPS observations to develop the vectors needed to perform extensive square adjustments and to develop local Geoid which would be more accurate than the predetermined National Geoid. The Geoid is the model to be used for the vertical component while collecting a very accurate horizontal position in GPS. Stoner and Associates established 27 new survey benchmarks throughout the City limits, which were certified by the NGS. The location of these 27 new survey benchmarks are defined within the Benchmark Book Certification from Stoner and Associates, which is included within the digital Appendix B-9 – Survey Verification Data attached to this report.

Stoner and Associates conducted a GPS field survey of all City stormwater structures defined within the original City stormwater atlas. According to the original GIS Stormwater Geodatabase, the City's stormwater system should have consisted of 312 outfalls, 3,566 catch basin inlets, 637 drainage manholes, and 9 control structures, which total 4,212 drainage structures. There was an unknown quantity of additional existing stormwater structures within the City system which had not been incorporated into the GIS Stormwater Geodatabase and needed to be located during the survey effort. In order to budget for an unknown quantity of additional drainage structures, the survey scope was increased to include the location of up to 5,200 drainage structures within the City's stormwater system. The collected GPS data includes rim elevations, pipe invert elevations, weir elevations, and horizontal coordinates for each located drainage structure within the City system. The elevation data is accurate to within +/- 0.02 feet. The horizontal data is accurate to within +/- 0.01 feet. All elevation data is in North American Vertical Datum of 1988 (NAVD 88). As feasible, the pipe sizes and pipe materials were also verified at each drainage structure during the survey effort. CMA updated the GIS Stormwater Geodatabase with all GPS data collected during the field verification of these drainage structures on a monthly basis. The relevant survey data is included within the digital Appendix B-9 – Survey Verification Data, which is attached to this report.

2.10 LAND USE

The land use is an important factor used within the stormwater model for estimating the overall stormwater needs of the City. Each land use category typically has a maximum allowable percentage of impervious area on each property. Properties with high percentages of impervious area will contribute more stormwater runoff into public right-of-way areas since there will be less pervious areas which allows stormwater runoff to infiltrate into the ground surface. The stormwater runoff will instead need to be collected for water quality treatment before it can be discharged into adjacent surface water bodies. Properties with a significant amount of green space or that have drainage facilities to retain their stormwater onsite, including graded swales, will reduce the overall discharge of stormwater runoff from the City. The land use categories can also been correlated to pollutant loadings within stormwater runoff from these properties, which impact the water quality of the receiving water bodies.

The land use information was provided by the City of Pompano Beach to CMA in GIS shapefile format. According to this GIS data, there are 19 land use categories found within the City. CMA used this land use data to create updated land use maps for the study area. The current land uses of all properties within the City limits are defined in Figure 2-3 – Land Use Map included within this section. CMA reviewed the land use information for the entire City limits and found the information to be comprehensive for use in the development of this Stormwater Master Plan.

2.11 ZONING

The zoning information was provided by the City of Pompano Beach to CMA in GIS shapefile format. According to this GIS data, there are 44 zoning categories found within the City limits. The current zoning of all properties within the City limits are defined on Figure 2-4 – Zoning Map included within this section. CMA reviewed the zoning information for the entire City limits and found the information to be comprehensive for use in the development of this Stormwater Master Plan.

2.12 IMPERVIOUSNESS

The percentage of ground surface coverage which is considered either impervious versus pervious is an important hydrologic parameter used within a stormwater model. Impervious surfaces, such as roadways, driveways, sidewalks, and parking areas, prevent the infiltration of stormwater runoff while pervious surfaces, such as grass yards, swale areas, parks, and other landscaped areas, will allow stormwater runoff to infiltrate into the ground. Areas with high percentages of impervious area will contribute more stormwater runoff into public right-of-way areas since there will be less pervious areas which allows stormwater runoff to infiltrate into the ground surface. The stormwater runoff from highly impervious areas will instead need to be collected with a stormwater management system to provide transmission capacity to an area with adequate storage volume. Areas with a significant amount of green space will contribute less stormwater runoff into public right-of-way areas due to the infiltration capacity of pervious areas.

The level of imperviousness for a regional study is typically addressed by assuming a range of expected impervious percentage values associated with each land use type. These assumed impervious percentages have been estimated from other previous research and specific studies which have associated typical impervious coverage for each land use category. Since the assumed impervious percentages can significantly vary from the actual conditions due to different local and regional regulations, a more scientific approach will be used to determine this impervious parameter in this study area. The U.S. Geological Survey (USGS) has developed a nationwide impervious percentage database, which is part of a much larger spatial and temporal database of geographical features that has been called SEAMLESS. This impervious percentage database has been put together in a raster format with a cell size of 30 meters. Within this 30-meter cell size, pervious and impervious terrain has been weighted out to come up with a percent imperviousness value representative of the 30-meter cell. The USGS SEAMLESS raster downloaded for the study area is displayed within Figure 2-5 – USGS SEAMLESS Impervious Map within this section. The USGS SEAMLESS data was used to estimate the level of imperviousness for each drainage basin within the study area.

2.13 SOIL CONDITIONS

The soil conditions within the City limits are important for the development of the Stormwater Master Plan since various soil types have different infiltration rates, which will control how quickly stormwater runoff infiltrates at the ground surface within grass swale areas or below ground via exfiltration trenches. The soil survey for Broward County was completed by the NRCS (previously the Soil Conservation Service, SCS) in 1976. A list of the most predominant soil series found within the study area is

summarized within Table 2.2 – NRCS Soil Series Coverage below. According to the soil survey, Immokalee, Urban land, and Paola soil series account for 48 percent of the soil types present in the study area. The soil series distribution for the study area is displayed in Figure 2-6 – USGS Soils Map in this section.

Table 2.2 – NRCS Soil Series Coverage

Soil Series	Total (acres)	Percentage
Arents	389.9	2.0%
Arents – Organic Substratum	896.9	4.6%
Basinger	276.9	1.4%
Beaches	10.4	0.1%
Canaveral	115.9	0.6%
Duette	1,669.4	8.5%
Hallandale	103.1	0.5%
Immokalee	4,535.3	23.1%
Margate	1,098.6	5.6%
Matlacha	784.0	4.0%
Palm Beach	136.5	0.7%
Paola	2,410.7	12.3%
Plantation	48.3	0.2%
Pomello	386.9	2.0%
Pompano	645.3	3.3%
Sanibel	237.4	1.2%
St. Lucie	306.4	1.6%
Udorthents	525.7	2.7%
Udorthents – Shaped	1,625.7	8.3%
Urban Land	2,431.5	12.4%
Water	1,032.2	5.2%
Total	**19,666.9**	**100.0%**

The most common soil series within the study area includes Urban Land with 12.4% coverage, Immokalee Fine Sand with 11.5% coverage, Immokalee – Urban Land with 10.8% coverage, Duette – Urban Land with 8.5% coverage, and Paola – Urban Land with 8.4% coverage. The Urban Land soil type consists of areas that are more than 70% covered with airports, shopping centers, parking lots and other structures with a high level of impermeability. The Immokalee Fine Sand soil type is characterized by being poorly drained and having a shallow water table, but it's their permeability is moderate to moderately rapid. The Paola-Urban Land soil type is characterized by being excessively drained and is mostly used for urban development.

One of the properties identified by the soil survey is the hydrologic soil group (HSG) classification for each soil type. This classification defines the hydrologic property of the soil in terms of the capacity to infiltrate water through available porosity. The hydrologic soil groups are defined by 4 classification categories of A, B, C, and D based on the infiltration capacity. For example, Hydrologic Soil Group A yields more infiltration capacity while Hydrologic Soil Group D has limited to minimal infiltration capacity. Both HSG and antecedent moisture conditions (AMCs) are important in determining an adequate curve number (CN). For a given AMC, a lower CN is assigned to a land use overlying an A-type soil than the same land use overlying a B-type soil. The distribution of hydrologic soil groups within the study area is summarized within Table 2.3 – Hydrologic Soil Group Coverage below.

Table 2.3 – Hydrologic Soil Group Coverage		
Hydrologic Soil Group (HSG)	**Area (Acres)**	**Percentage**
A	8,463.9	43%
A/D	4,725.2	24%
B	784.0	4%
B/D	2,219.7	11%
Water/NA	3,474.2	18%
TOTAL	**19,666.9**	**100%**

A significant percentage of the soils within the study area are classified as HSG Type A, which are soils with low runoff potential when thoroughly wet as water is transmitted freely through the soil. The HSG Type A soils typically have less than 10 percent clay and more than 90 percent sand or gravel. The dual HSG classifications of Type A/D and Type B/D are the next most prevalent soil types within the study area and are typically associated with wet soils. Under these dual classifications, the first hydrologic soil group designates the drained condition while the second hydrologic soil group designates the undrained condition of the soil. In the case of these dual classification groups, certain wet soils are merged with Type D based solely on the presence of a water table within 24 inches of the ground surface even though the saturated hydraulic conductivity may be favorable for water transmission per USDA 2007. The dual hydrologic classification includes soils with high seasonal water table but can be drained via infiltration. For modeling purposes, the dual HSG classifications will be switched to an equivalent of the undrained condition of the soil. For example, Type B/D is equivalent to Type D while Type A/D is equivalent to Type C. Based on this assumption, Table 2.4 shows a modified version of Table 2.3 with an approximate expected infiltration rate associated for each HSG, which was incorporated into the stormwater model.

Table 2.4 – Modified Hydrologic Soil Group Coverage		
Hydrologic Soil Group (HSG)	**Percentage**	**Initial and Final Infiltration Rate (in/hr)**
A	43%	12.0 – 1.0
B	4%	9.0 – 0.5
C	24%	6.0 – 0.25
D	11%	4.0 – 0.10
Water	18%	N/A

The distribution of the modified HSG classification within the study area is depicted within Figure 2-7 HSG Soils Map within this section. As displayed within this map, HSG Type C and Type D are chiefly located west of I-95 and south of Atlantic Boulevard while HSG Type A is predominant to the east of I-95. Based on the HSG coverage within the study area, the eastern portion of the City can be expected to have a higher infiltration rate than the western areas.

2.14 HYDROLOGIC PARAMETERS

The SWMM modeling software requires the input of various hydrologic parameters for each sub-basin or hydrologic unit in order to accurately simulate the flow of stormwater runoff across the ground surface. These hydrologic parameters are used to define the existing conditions of the ground surface within each sub-basin and to establish the flow conditions of surface runoff across these sub-basins into the stormwater management system. Thee hydrologic parameters will be calculated based on the collected data for the study area. The methodology for calculating the hydrologic parameters for each sub-basin

within the study area is defined in detail within Section 4 of this report. The hydrologic parameters to be incorporated into the stormwater model for each sub-basin include the following:

- Total area of ground surface within each sub-basin
- Hydrologic width of each sub-basin
- Slope of each sub-basin
- Percentage of impervious and pervious ground surface within each sub-basin
- Roughness coefficient for the impervious and pervious ground surfaces within each sub-basin.

2.15 WATER LEVEL ELEVATIONS

The water level elevations typically serve as boundary conditions in stormwater models, which can be located upstream and/or downstream from the modeled stormwater system. Under this project, only downstream boundary conditions will be considered due to the nature of the interconnected canal system within the study area. These downstream boundary conditions regulate and control discharge capacity of an outfall due to fluctuation of water levels within the receiving waterbody. Outfalls which discharge to an estuary or a navigable waterway are directly impacted by the water level fluctuations due to the tidal cycles. Outfalls which discharge into freshwater canals are directly impacted by the water level fluctuations which are mechanically regulated by water control structures. Outfalls which discharge into lakes, reservoirs, or retention areas are directly impacted by the water level fluctuations controlled by groundwater elevation and man-made water control structures. The data collected to define the various water level elevations within the study area is defined further in the following sections.

2.15.1 TIDAL WATER LEVELS

CMA collected two tidal datasets for this project, which include a tidal timeseries for the calibration period chosen to validate the stormwater model and a fixed tide elevation for the simulations of one-day and three-day design storms. This tidal information is used to establish the downstream boundary condition for any outfall discharging into tidally influenced waters, such as the Intracoastal Waterway or connected marine canals. In order to determine the fixed tidal elevation to be used in the one-day and three-day design storm simulations, available tidal data was identified in the vicinity of the study area. Two subordinate tidal gauges were found near the Hillsboro Inlet, which include Hillsboro Inlet (Ocean, ID: 8722862) and Hillsboro Inlet (Inside, ID: 8722861). The latter was considered as most representative of the expected tidal fluctuations along the Intracoastal Waterway. Table 2.5 provides a summary of the tidal datum determined by the National Oceanic Atmospheric Administration (NOAA) using as control tide station the Miami Beach (City Pier, ID:8723170) gauge.

Table 2.5 – Summary of Tidal Datums at Hillsboro Inlet, Inside (ID: 8722862) Gauge

Tidal Datums	Elevation (feet NGVD)	Elevation (feet NAVD)
Highest Observed Water Level (10/15/1970)	3.84	1.48
Mean Higher High Water (MHHW)	2.79	0.43
Mean High Water (MHW)	2.65	0.29
Mean Tide Level (MTL)	1.40	-0.96
Mean Sea Level (MSL)	1.40	-0.96
Mean Low Water (MLW)	0.16	-2.20
Mean Lower Low Water (MLLW)	0.00	-2.36
Lowest Observed Water Level (04/26/1971)	-1.12	-3.48

The locations of the subordinate and reference tidal gauges to be used in this project is displayed on Figure 2-8 NOAA Tidal Station within this section. Based on the tidal data presented in Table 2.6 for mean higher high water (MHHW), a boundary condition of +0.45 feet NAVD should be used for during the one-day and three-day design storm simulations with the stormwater model when a static conditions is required.

2.15.2 CANAL WATER LEVELS

The SFWMD DBHYDRO database was used to obtain the recorded water level data at the headwaters of the various water control structures located along the SFWMD canal systems within the study area. Water Control Structures S37A and S37B along the SFWMD C14 Canal and Water Control Structure G57 along the Old Pompano Canal have an impact on the water levels within these canals located in the study area. CMA also obtained the operating criteria for these control structures from SFWMD. A summary of the operating criteria and design conditions for each control structure is presented in Table 2.6 below. The location of these water control structures along the SFWMD C14 Canal and the Old Pompano Canal are shown in Figure 2-9 Rainfall and Canal Level Gauge Locations in this section.

Table 2.6 – Operating Criteria and Design Conditions at Water Control Structures

SFWMD Water Control Structure	Location	Optimum Water Surface (feet NAVD)	Discharge Rate (CFS)	Operating Criteria (HW = Headwater)
S37A	SFWMD C14 Canal (East of Dixie Highway)	1.93	3,890	• When HW = 2.83 feet, then gates begin to open at 6 inches per minute. • When HW rises or falls to 2.43 feet, then gates become stationary. • When HW falls to 1.43 feet, then gates begin to close.
S37B	SFWMD C14 Canal (Palm Aire)	5.43	3,390	• When HW = 5.63 feet, then gates begin to open at 6 inches per minute. • When HW rises or falls to 5.43 feet, then gates become stationary. • When HW falls to 5.23 feet, then gates begin to close. (For dry periods, a different criteria apply)
G57 [3]	Old Pompano Canal (Cypress Road)	2.93	375	• When HW = 3.23 feet, then gates begin to open at 6 inches per minute. • When HW rises or falls to 3.03 feet, then gates become stationary. • When HW falls to 2.73 feet, then gates begin to close.

Regardless of the constraining operating criteria implemented in each water control structure, this data shows that there is a high variability of headwater level that it is skewed to lower water surface elevations below the optimum elevation at each control structure. The high level stages are controlled by gate openings which are controlled by the respective operating criteria. However, external factors seem to play a role in maintaining the water levels at the optimum level. One of those factors is the effect exerted by the existing City of Fort Lauderdale Prospect's Wellfield and the City of Pompano Beach's Wellfields on these neighboring surface waters. The City of Pompano Beach's wellfields are located along the City of Pompano Beach Air Park between Atlantic Boulevard and Copans Road and another located at the northeast corner of Atlantic Boulevard and Florida's Turnpike. During some high potable consumptive

periods, the flow observed at S37A downstream from S37B is less than that observed at S37B. The Old Pompano Canal where structure G57 is located might also be directly influenced by the City of Pompano Beach Air Park Wellfield's cone of depression. A summary of rainfall and headwater level data collected at each water control structure for the peak year is shown within Figure 2-10 Rainfall, Headwater Levels, Optimum Water Surface at SFWMD Water Control Structures below.

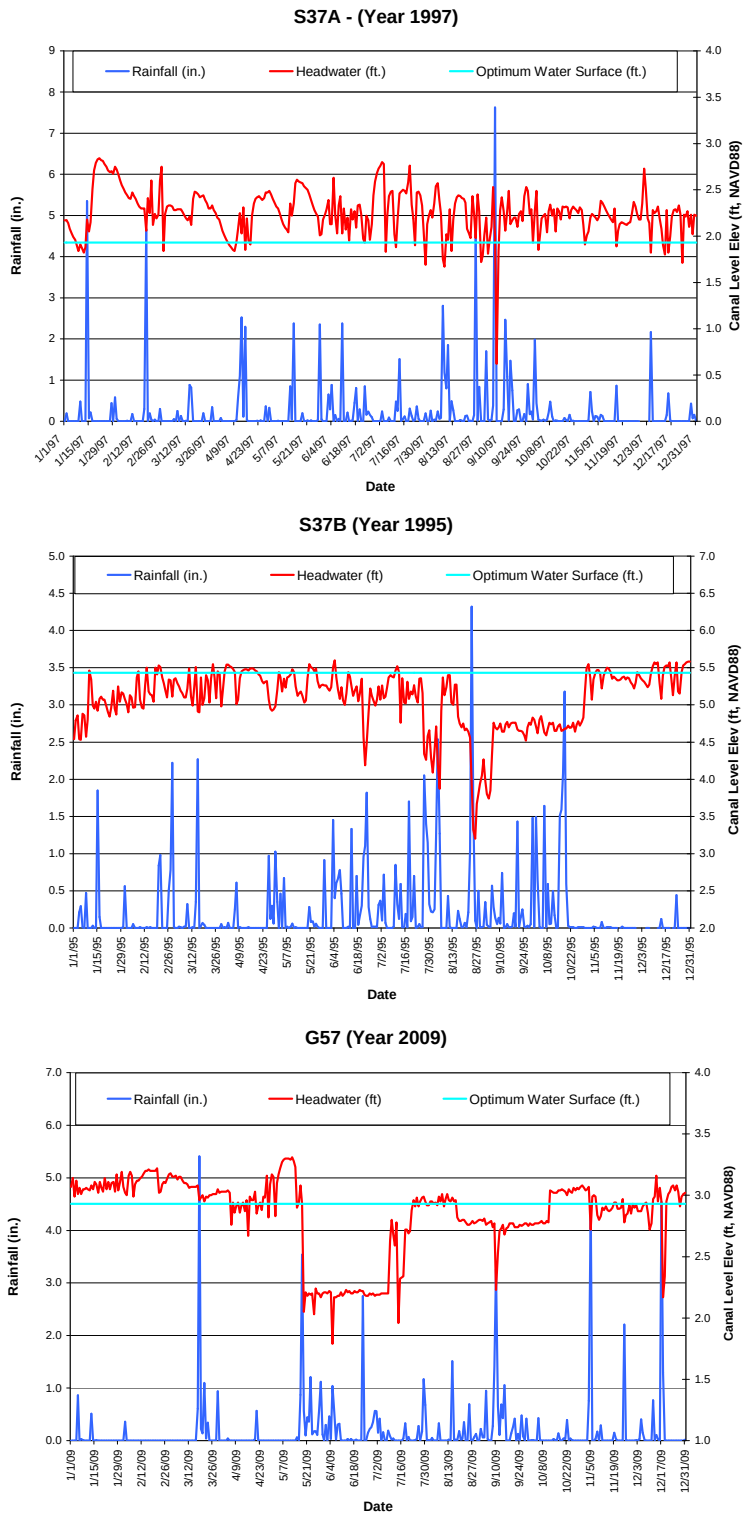


Figure 2-10 Rainfall, Headwater Levels, Optimum Water Surface at SFWMD Water Control Structures

When considering this analysis of the controlled water levels within SFWMD Canals, the incorporation of the optimum water surface elevations as boundary conditions within the stormwater model can be considered a conservative assumption, which is a good modeling practice for these parameters. In conclusion, existing outfalls which directly discharge into the canals regulated by the above water control structures will use the optimum water surface elevation as boundary conditions for tailwater.

2.15.3 Water Levels for Lakes, Ditches, and Retention Areas

Outfalls which discharge into lakes, ditches, or retention areas are affected by the local tailwater conditions. During dry periods, the water level within the lake, ditch, or retention area will be controlled by the local groundwater elevation along with soil type within the area. During wet periods, the water level within lakes, ditches, or retention areas should be assumed to be at or near capacity. Figure 2-11 Average Wet Season Water Table as Determined by USGS (Fish, 1988) shows an insert of the wet season groundwater contour elevation (NGVD29) map published by the USGS for Broward County (Fish, 1988). In order to estimate the water level elevation within these waterbodies for use as boundary condition in the stormwater model, any water control structures at the lakes, ditches, or retention areas will be surveyed for incorporation into the stormwater model. The information to be gathered is the elevation at the overflow weir within these drainage control structures together with identifying the type of connection to a major surface water feature such as a canal. Therefore, the use of a wet season water table elevation data is not necessary as the weir crest at the overflow structure is equal or higher than the high water table.

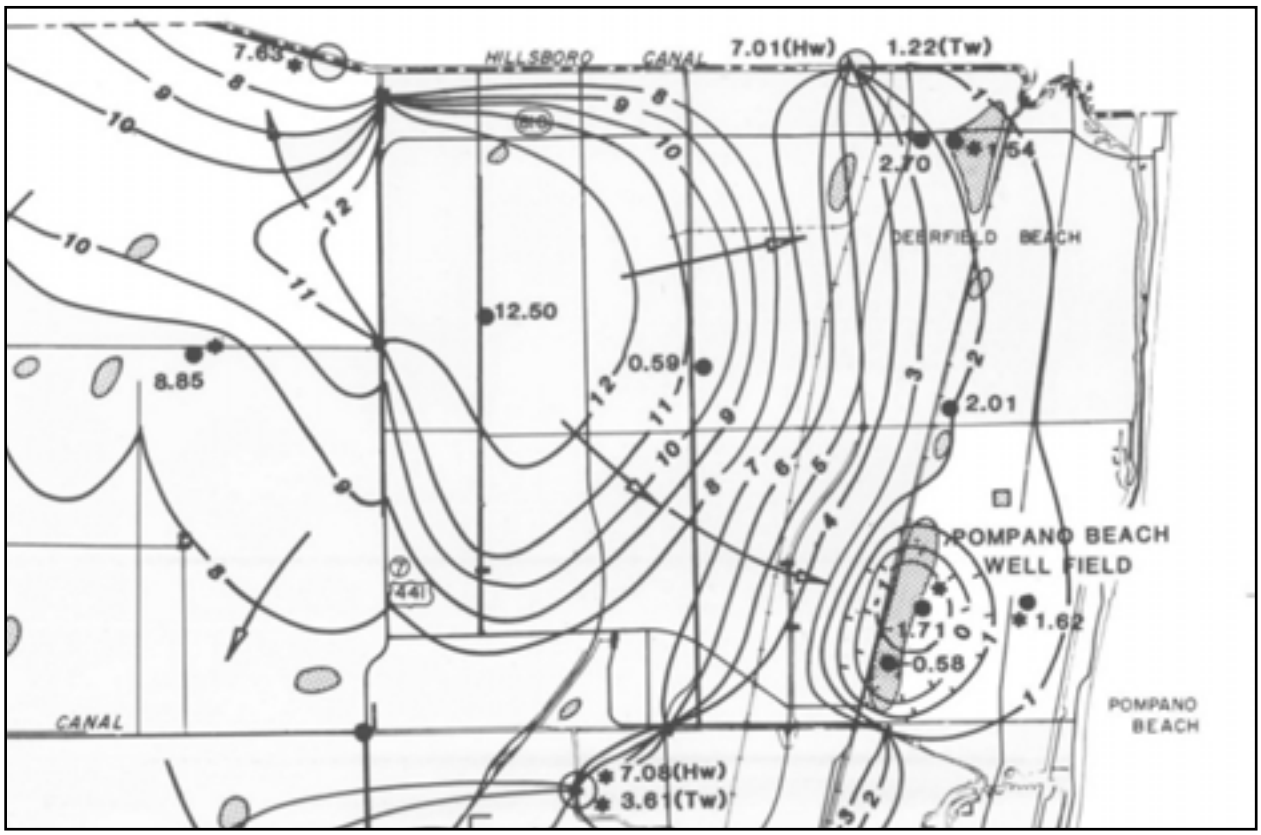


Figure 2-11 Average Wet Season Water Table as Determined by USGS (Fish, 1988)

2.16 RAINFALL

The location of SFWMD rainfall gauges in the study area is displayed within Figure 2-9 Rainfall and Canal Level Gauge Locations within this section. CMA obtained monthly multi-annual rainfall data for Gauge S37A-R, Gauge S37B_R, Gauge G57_R, and Gauge PompanoF_R for use in the development of the stormwater model. A summary of the rainfall data as available from the SFWMD DBHYDRO database within the study area is listed within Table 2.7 – SFWMD Rainfall Gauges Data below.

Table 2.7 – Rainfall Gauges Data from SFWMD

DBKEY	Station Name	Agency	Period	Easting	Northing
PT369*	087254-1	NOAA	1948-2001	934801.69	691411.78
PT370	087254-2	NOAA	2001-2001	937897.78	691432.19
K8628	G57_R	WMD	1997-2011	943261.74	690689.80
VN074		WMD	2007-2011		
VN270	S37A_R	WMD	2007-2011	940882.56	681555.88
K8664		WMD	1997-2011		
16680*		WMD	1991-2011		
K8667	S37B_R	WMD	1997-2011	928113.88	687893.38
16612*		WMD	1991-2011		
VN272		WMD	2007-2011		
06179	POMPANOB_R	NOAA	1941-1998	934877.71	691542.76
05796*	POMPANOF_R	WMD	1957-2002	938066.29	691361.84

In order to compare rainfall data records at these gauges, a box and whisker diagram was created for annual precipitation during their entire record period as shown below.

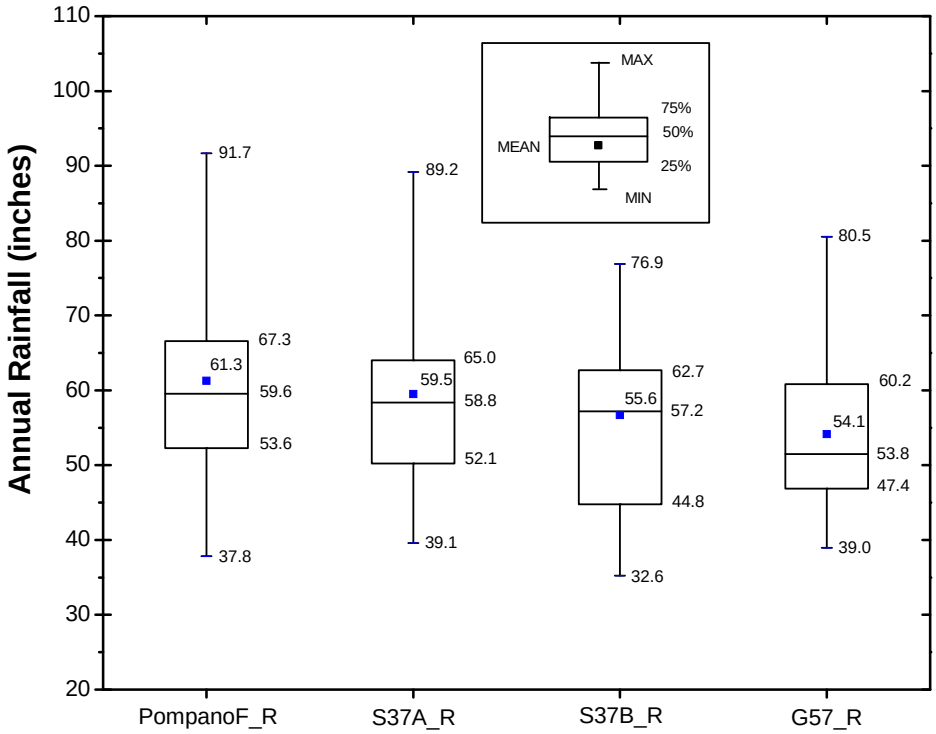


Figure 2-12 Rainfall Gauge Annual Precipitation

2.16.1 RAINFALL FOR WATER QUALITY ANALYSIS

In consideration of its extended recording period along with the quality of collected rainfall data, Gauge PompanoF_R was used for the estimation of annual and seasonal nutrient loads associated to rainfall-runoff events. From the gauges considered for this study area, Gauge Pompano F_R exhibits the greatest variability in annual rainfall, which is expected from the gauge with the longest record period in the study area of 48 years. Although Gauge G57_R shows a variation of almost five inches from Gauge PompanoF_R and Gauge S37A_R, this difference does not represent an error on the collected data at Gauge G57_R. The difference can be directly attributed to the relatively short recording period at Gauge G57_R from 1997 to 2011). A spatial rainfall gradient is not identifiable with the available information. A box and whisker diagram for monthly rainfall data recorded at Gauge PompanoF_R is shown below, which clearly identifies both wet and dry seasons recorded at this station to be May through October for wet season and November through April for dry season.

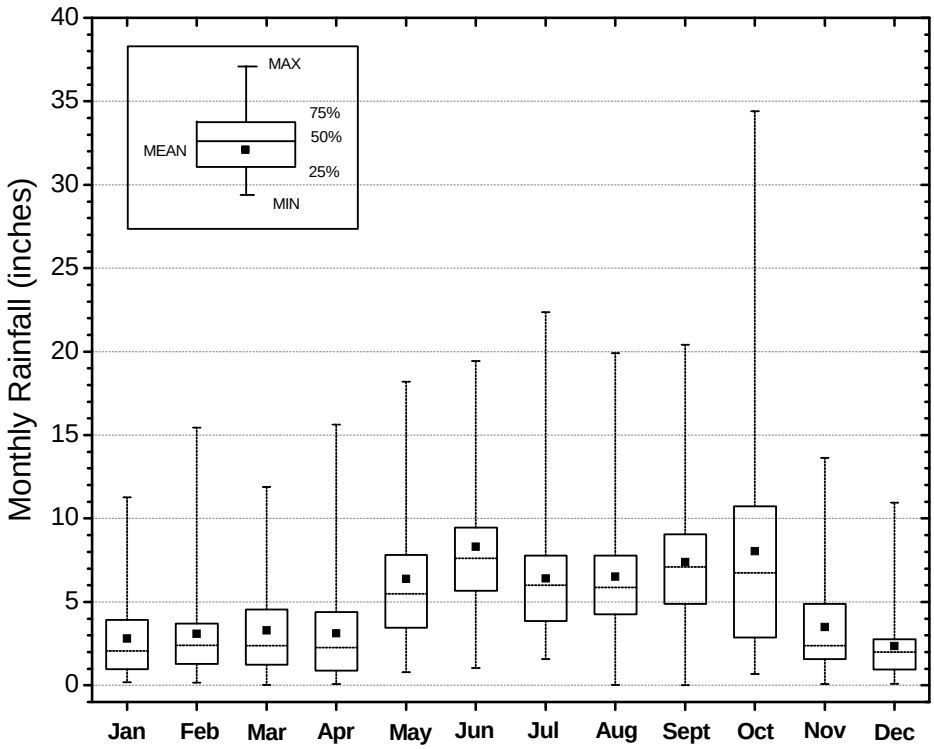


Figure 2-13 Monthly Rainfall Data

2.16.2 RAINFALL FOR MODEL CALIBRATION AND VALIDATION

Hourly precipitation data was also collected for the calibration period chosen for this project during the modeling phase. This rainfall data is provided by SFWMD on special request for incorporation into the stormwater model together with concurrent tidal and freshwater canal elevation data in the next task. This rainfall data as well as other data necessary in the calibration effort will be used to validate the stormwater model by comparing model-estimated stages in neighborhood roads and canals with observed and/or recorded high water marks and water stages in canals for a historical rainfall event.

2.16.3 RAINFALL FOR DESIGN STORMS

In the stormwater model, the use of a synthetic hyetograph for rainfall events is used to evaluate the performance of the existing drainage system and to compare this performance to measurable level of service (LOS) criteria. By simulating these design storms, alternative solutions to identified flooding problems could be made by complying with a previously specified LOS. Synthetic hyetographs have been developed regionally by the SFWMD for one-day and three-day duration design storms. These design storm events were developed by studying the historical distribution of rainfall in the region and represent the regional distribution of one-inch of rainfall. The design storm for a given frequency and duration is determined by multiplying the synthetic hyetograph by the respective rainfall depth, which is found spatially distributed in isohyet maps published by SFWMD in its Basis of Review document. Table 2.8 provides a summary of the rainfall depths to be used in the modeling task of this project.

Table 2.8 – SFWMD Design Storm Depths	
SFWMD Design Storm	Rainfall Depth (inches)
5-year, 24 hour	8.0
10-year, 24-hour	10.0
25-year, 72-hour	15.8
100-year, 72-hour	21.5

An example rainfall map from the SFWMD Basis of Review document is displayed below for the 25-year, 72-hour design storm depth. This rainfall map is an example of the spatial distribution of rainfall observed in the isohyet maps for this region for each frequency and duration. An outstanding feature in these isohyet maps is the confluence of isohyet contours of the highest precipitation values in the region around the area of Boca Raton and north Broward, including portions of the City of Pompano Beach. This data is presented as evidence of the high levels of rainfall expected in the area and to provide an idea of the demands to which the existing drainage system is subjected.

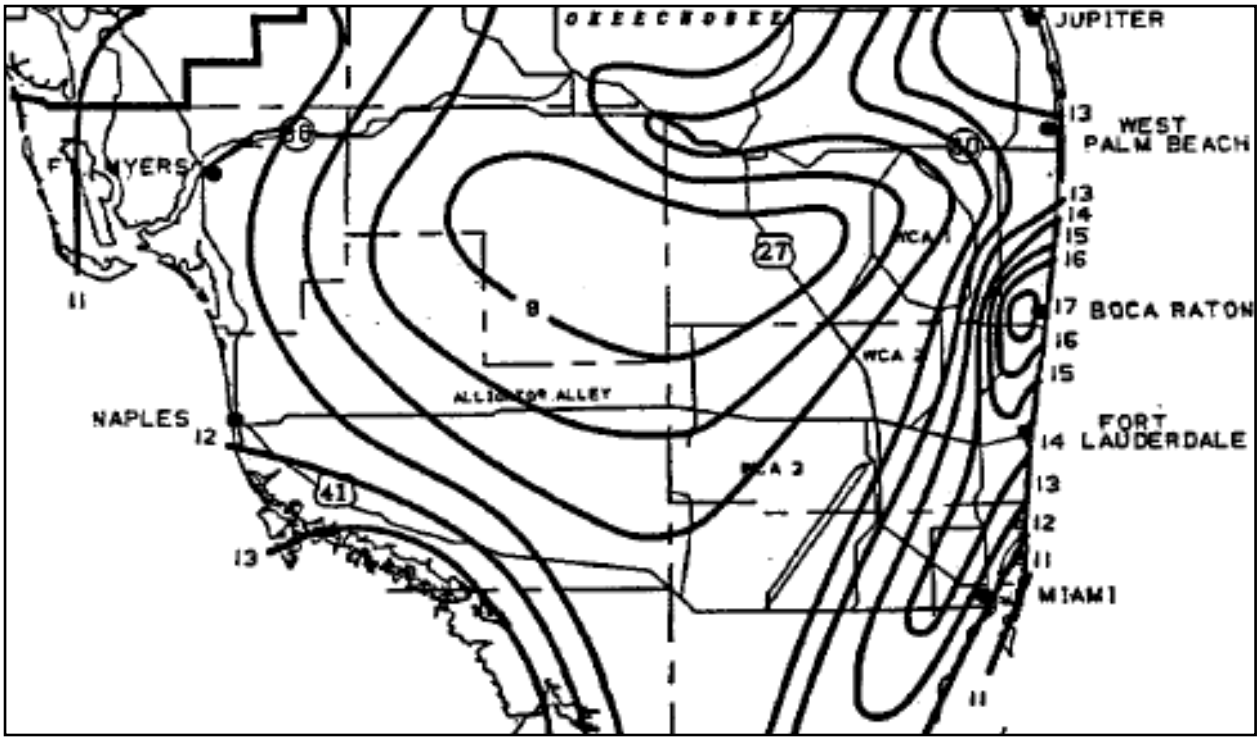


Figure 2-14 SFWMD Rainfall Map

2.17 AVAILABLE WATER QUALITY DATA

The Broward County Water Quality Monitoring Program (BCWQMP) provides the most comprehensive source of water quality data available in the Broward County region. The location of the BCWQMP Station #21FLBROW110 along the Pompano Canal, west of Dixie Highway is shown on Figure 2-9 Rainfall and Canal Level Gauge Locations in this section. This water quality monitoring station was also used by U.S. EPA Region 4 for the development of the proposed total maximum daily load (TMDL) for nutrients in Pompano Canal (US EPA, 2006). The water quality data from this station has been posted on STORET, FDEP's online water quality database, from which the data was downloaded. A summary of the available water quality data in STORET for the Station #21FLBROW is provided within Table 2.9 below. This water quality data includes the number of analysis available and the average concentration for some of the National Pollution Discharge Elimination System (NPDES) water quality parameters.

Table 2.9 Water Quality Data at Station 21FLBROW

Water Quality Parameter	Total Count	Average Concentration	Unit
Fecal Coliform	50	904.29	#/100 ml
Nitrogen, ammonia (NH3) + ammonium (NH4)	50	0.12	mg/l
Nitrogen, Kjeldahl	48	0.81	mg/l
Nitrogen, Nitrite (NO2) + Nitrate (NO3) as N	50	0.09	mg/l
Phosphorus as P	48	0.07	mg/l
Phosphorus, orthophosphate as P	50	0.03	mg/l
Total Coliform	28	5930.72	#/100 ml
Total Organic Carbon (TOC)	46	10.77	mg/l
Total	**370**		

2.18 DRAINAGE COMPLAINT DATA

During the preparation of the 1999 Stormwater Master Plan, the City provided PBS&J with a drainage complaint list. The location of these flooding complaints was displayed on a map within the 1999 Stormwater Master Plan. The 1999 Stormwater Master Plan prioritized the stormwater complaints into three categories: high severity, medium severity and low severity. This complaint data was then used in the ranking of the basins through the prioritization formula. The City staff provided CMA with the recent drainage complaint list. The City continued to collect flooding complaint information from residents throughout the duration of this project. During this project, the City also held two public outreach meeting focused on flooding issues within the City where additional input on existing flooding issues was gathered from concerned citizens. CMA compiled all flooding complaint information, which is included in digital Appendix B-10 – Resident Complaint Data attached to this report. Through September 11, 2012, the City has received a total of 84 flooding complaints from residents within the database. The location of the drainage complaints that have been recorded by the City are displayed on Figure 2-15 Resident Complaints and FEMA Repetitive Losses within this section. During subsequent tasks of this project, these recent complaints were taken into account when identifying areas of the City in need of drainage improvements.

2.19 FEMA FLOOD ZONE ELEVATIONS

The floodplain information within the City limits is important for the development of the Stormwater Master Plan since it can be used to identify low lying areas within the City below the FEMA flood elevation that will experience flooding during heavy rainfall events. These floodplains are geographic

areas that FEMA has defined according to varying levels of flood risk for properties within each area. Within each floodplain area, FEMA has defined maximum flood elevations during specified storms, which can be used to estimate the severity of potential flooding at certain areas within the City and to provide an estimated maximum elevation of flooding during a specified storm event. For example, the 100-year flood elevation describes the elevation of floodwater expected from a 100-year storm event. Typically, finish floor elevation of new construction must be set above the 100-year flood elevation defined by FEMA. The existing floodplain limits defined by FEMA are shown in the Figure 2-16 FEMA Flood Zone Map (Existing) within this section. FEMA has recently released a draft version of the updated flood zones within Broward County. The updated floodplain limits defined by FEMA are shown in the Figure 2-17 FEMA Flood Zone Map (2011 Draft Updates) within this section. A summary of the various flood zones within the study area is included within Table 2.10 – FEMA Flood Zones below.

Table 2.10 – FEMA Flood Zones

FEMA Flood Zone	Percentage	Description
0.2% Annual Chance Flood Hazard	42%	Area of moderate flood hazard, usually the area between the limits of the 100-year and 500-year floods
X	22%	Area of minimal flood hazard, usually depicted on Flood Insurance Rate Maps (FIRMs) as above the 500-year flood level.
AE	13%	The base floodplain where base flood elevations are provided
AH	22%	Areas with a 1% annual chance of shallow flooding, usually in the form of a pond, with an average depth ranging from 1 to 3 feet.
VE	1%	Coastal areas with a 1% or greater chance of flooding and an additional hazard associated with storm waves.

2.20 FEMA REPETITIVE LOSS DATA

The City of Pompano Beach provided CMA with the FEMA repetitive loss data, which is included within digital Appendix B-11 – FEMA CRS Program Documentation. There are a total of 12 repetitive loss property claims within the study area. The location of the repetitive loss claims since 1999 are displayed together with the drainage complaint information in Figure 2-15 Resident Complaints and FEMA Repetitive Losses within this section. During subsequent tasks of this project, these repetitive losses were taken into account when identifying areas of the City in need of drainage improvements.

2.21 REGULATORY DOCUMENTS

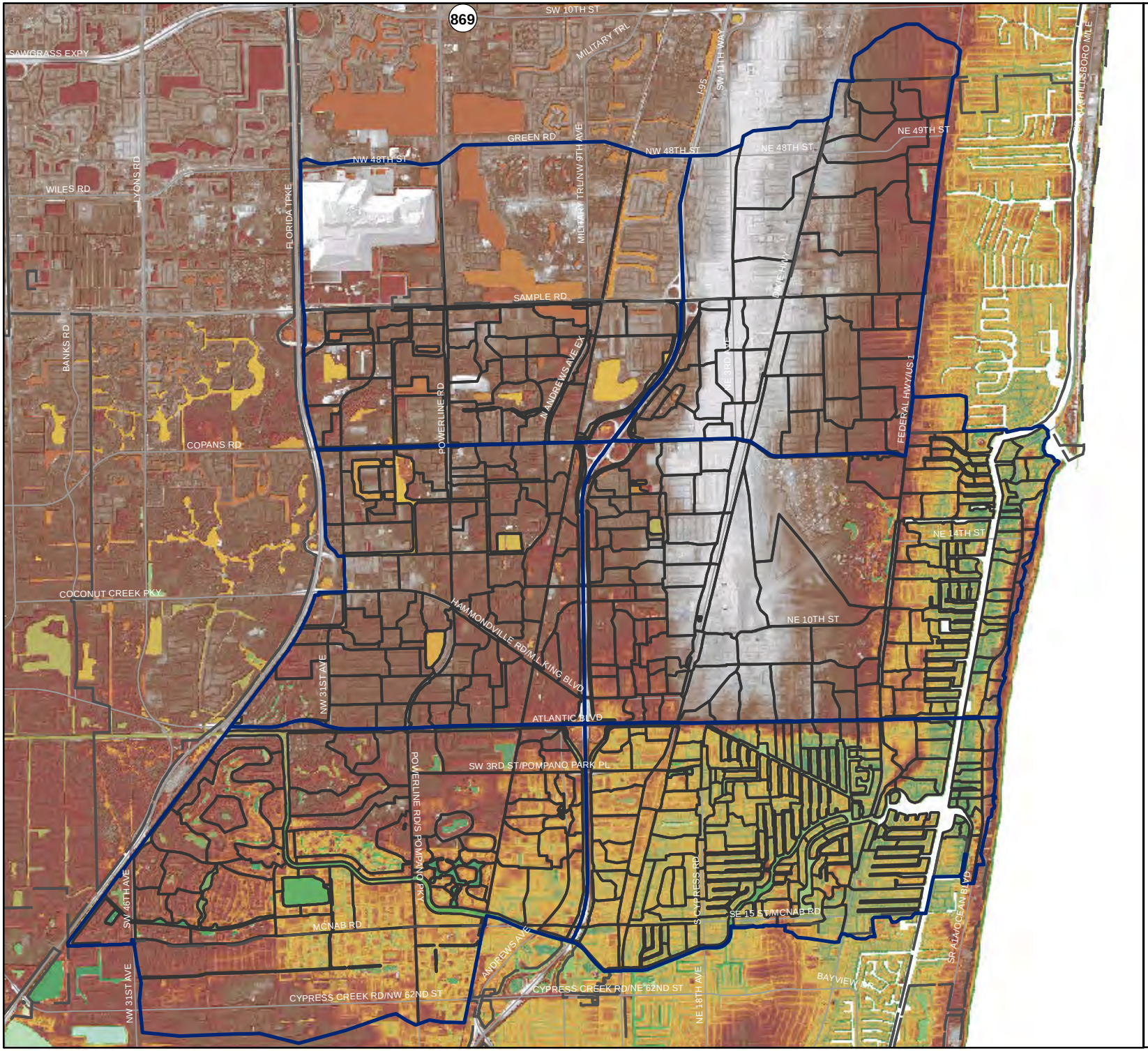
The City of Pompano Beach provided additional regulatory documentation to CMA to incorporate into the Stormwater Master Plan. The City of Pompano Beach maintains records on the existing stormwater management system as required under the various regulatory programs. The City is a co-permittee with Broward County on their NPDES Permit with FDEP, which requires record keeping on the maintenance and operation of the existing stormwater management system. The City must submit annual reports to FDEP to verify that proper Best Management Practices (BMP) have been instituted to minimize the pollutant loads entering surface waterbodies via stormwater runoff. The City also participates in the FEMA Community Rating System (CRS) Program, which require the implementation of policies which will reduce the potential for future flood damage which will result in flood insurance claims. The City must submit recertification documentation to FEMA to verify the City's rating under the CRS Program. The City provided the following documentation on these programs, which have been included in digital Appendix B in PDF format:

- Annual CRS Recertification Package (Appendix B-11)
- NPDES MS4 Annual Reports for Year 2 through Year 8 (Appendix B-13)
- City Stormwater Goals for 2005 through 2012 (Appendix B-13)
- City Daily Inspection Forms for the stormwater management system (Appendix B-14)

During subsequent tasks of the project, CMA reviewed these documents and developed additional backup documentation to further enhance future submittals for the FDEP NPDES Program and FEMA CRS Program.

2.22 WETLAND INVENTORY

Wetlands are helpful in providing flood attenuation and stormwater management, while removing sediments and nutrients from runoff and providing groundwater recharge. CMA obtained the inventory of all wetlands located within the City limits based on the 2004 Broward County Environmental Protection and Growth Management (EPGM) Wetlands Coverage GIS maps. There are a total of 11 wetlands located within the City, which include 55.2 acres of land area. These wetlands are categorized by Broward County as Cypress (35.6 Acres), Marsh (10.5 Acres), or Mitigation Completed (9.1 Acres). The location of these existing wetlands within the City limits is displayed within Figure 2-18 Wetland Inventory at the end of this section. CMA conducted a photo study of each existing wetland, which is displayed within Figure 2-19 Wetland Photo Study at the end of this section.



City of Pompano Beach
Stormwater Master Plan

Figure 2-1
Topography Map

Legend

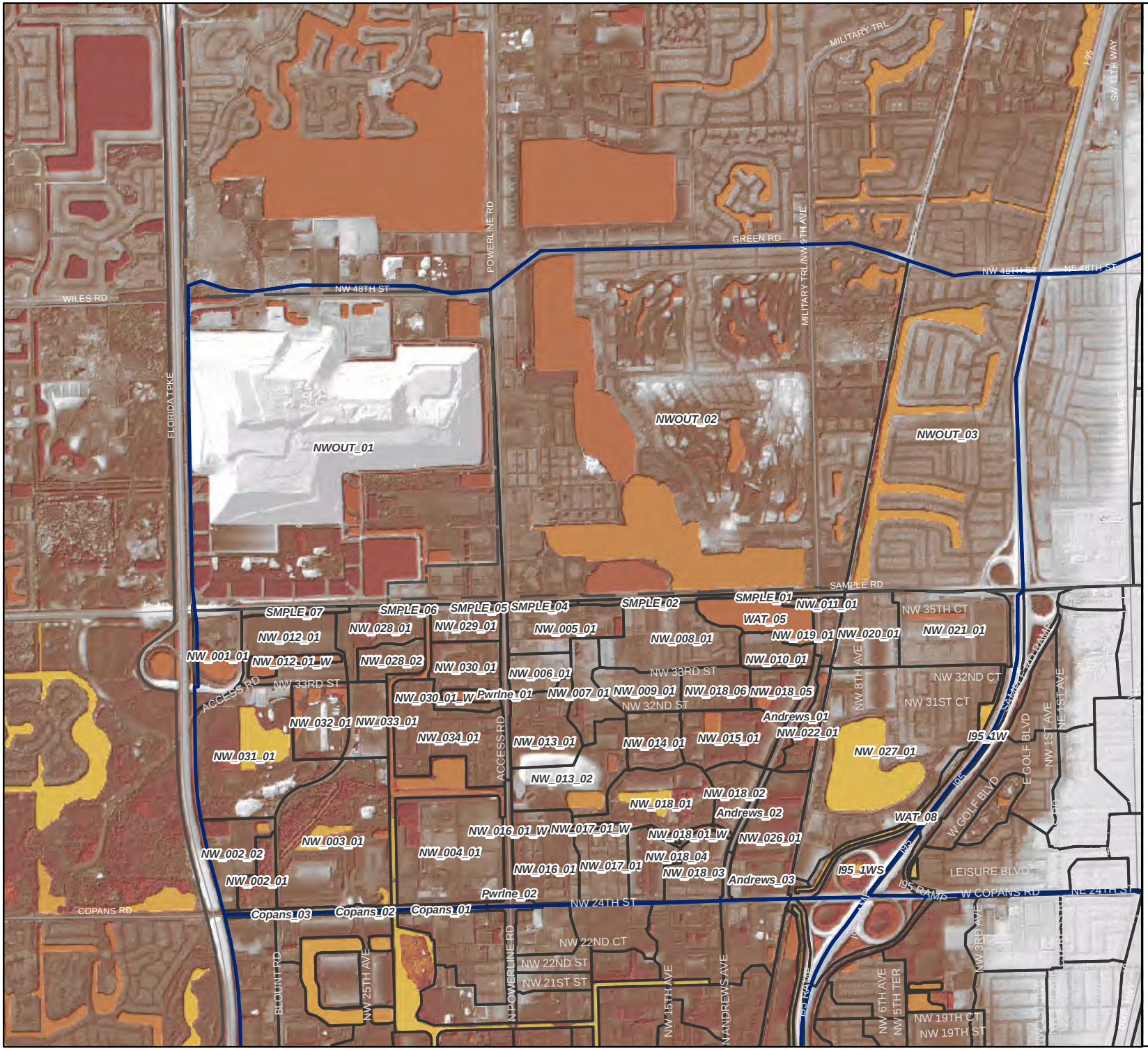
- Major Basin Boundary
- City Boundary
- Sub-Basins

Digital Elevation Model

- High Elev.
- Low Elev.

N
W E
S

0 1,500 3,000 6,000
Feet



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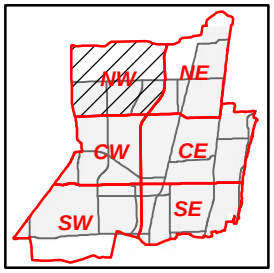
City of Pompano Beach
Stormwater Master Plan

Figure 2-1A
Topography Map
Basin NW



Legend

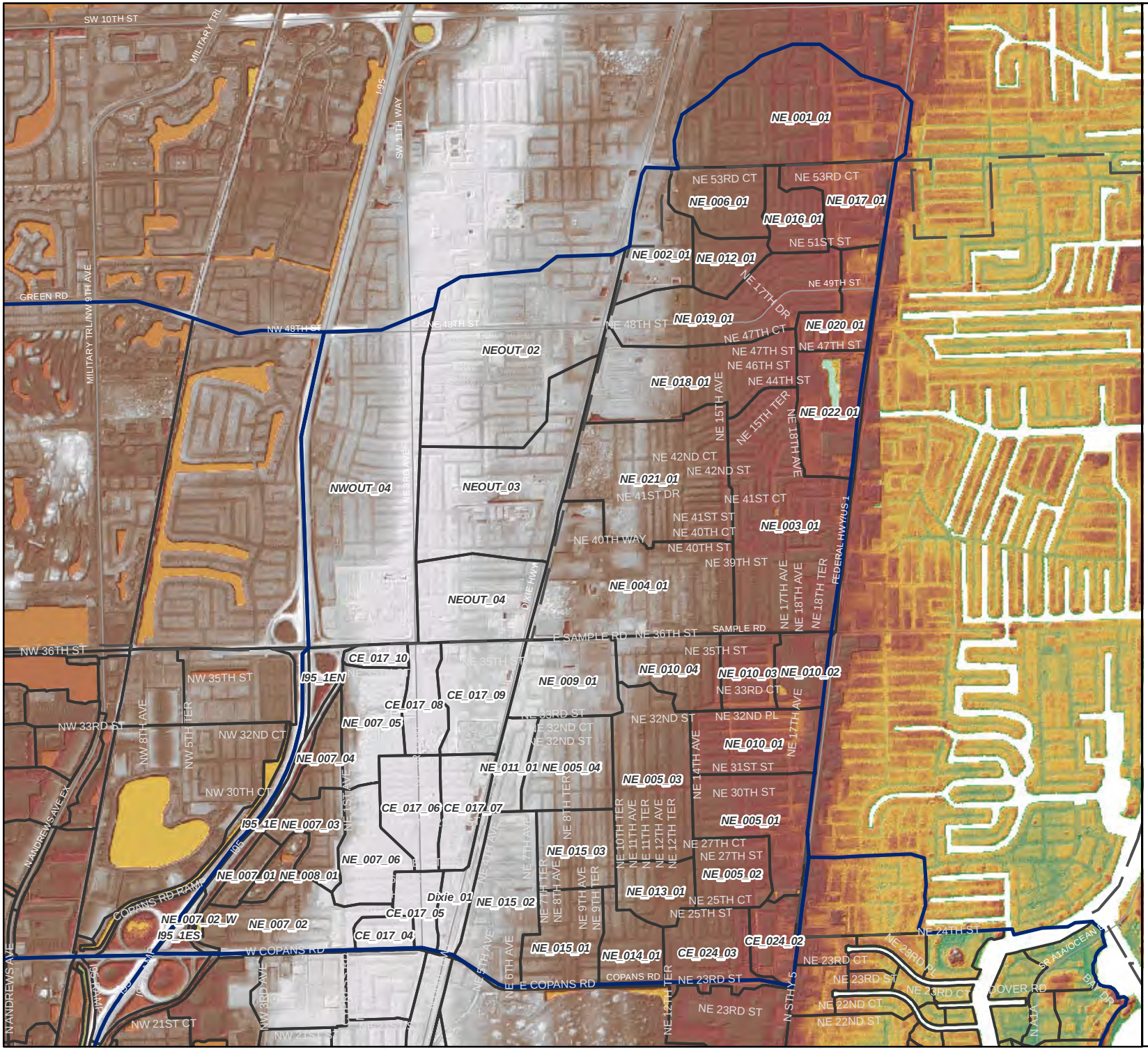
- Major Basin Boundary
- City Boundary
- Sub-Basins

Digital Elevation Model

- High Elev.
- Low Elev.

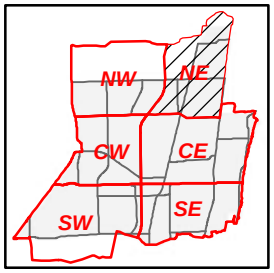
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-1B
Topography Map
Basin NE



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

Digital Elevation Model

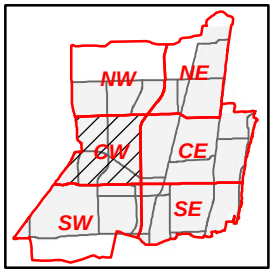
- High Elev.
- Low Elev.

N
W E
S

0 750 1,500 3,000
Feet

City of Pompano Beach
Stormwater Master Plan

Figure 2-1C
Topography Map
Basin CW



Legend

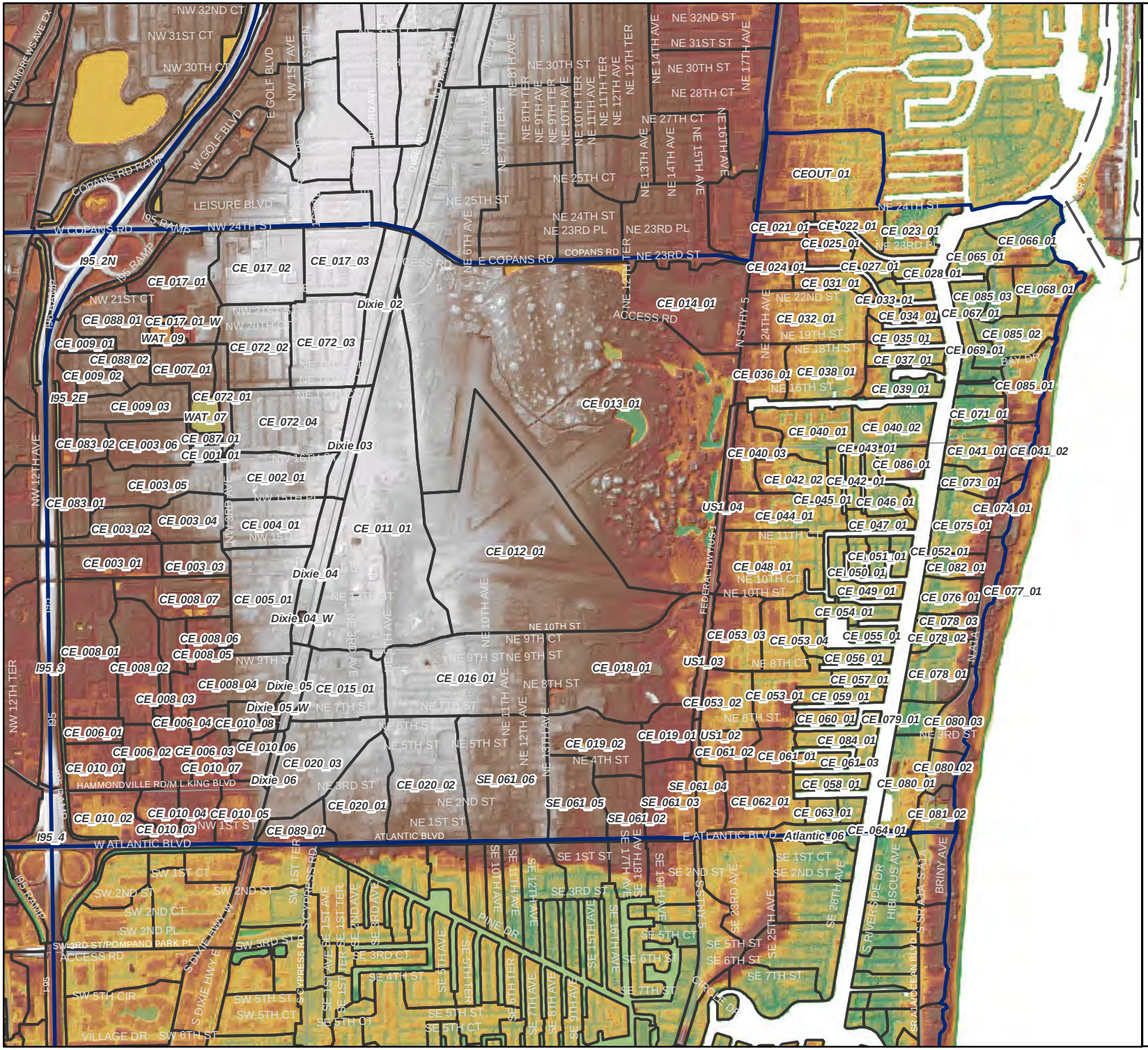
- Major Basin Boundary
- City Boundary
- Sub-Basins

Digital Elevation Model

- High Elev.
- Low Elev.

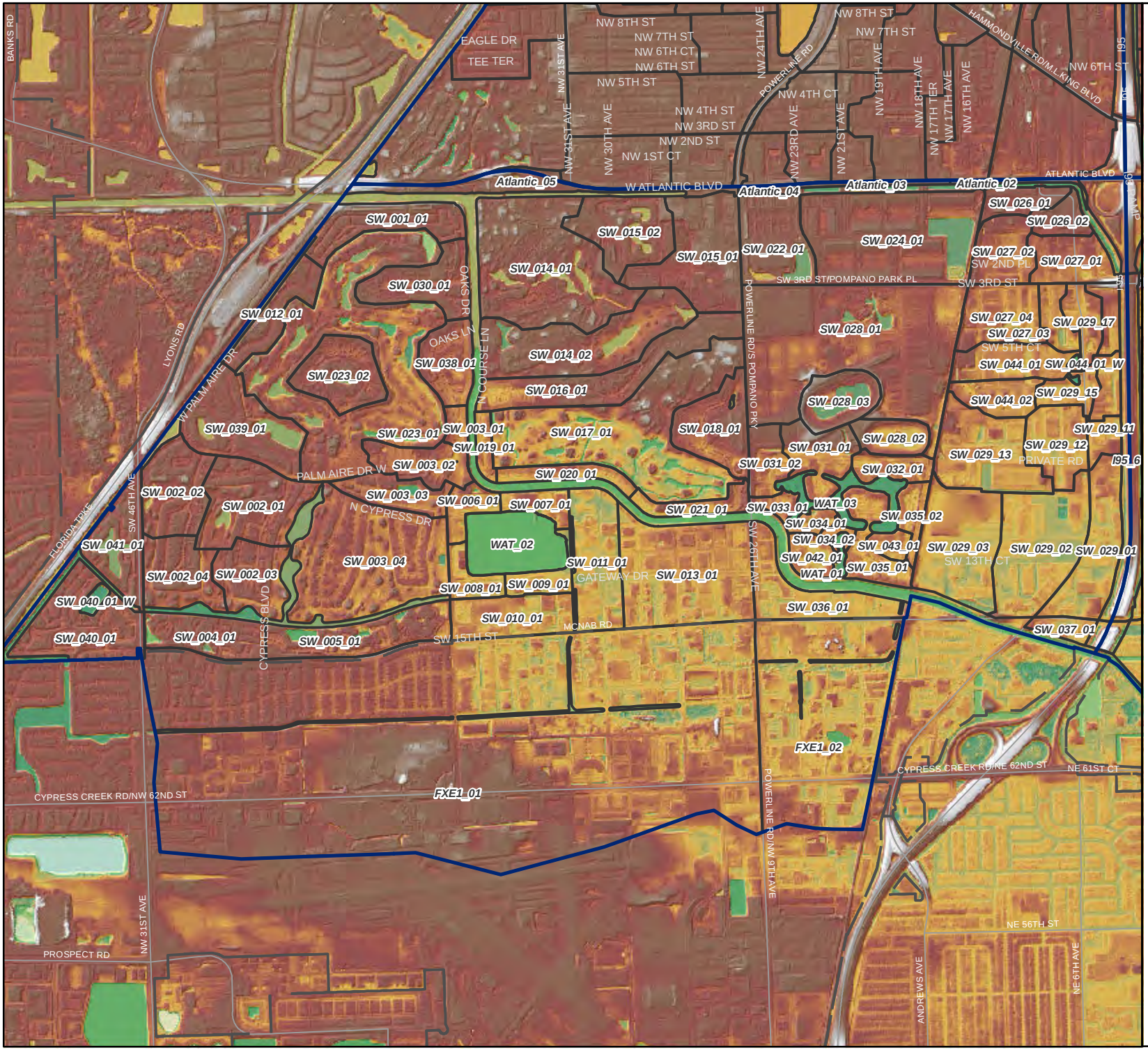
N
W E
S

0 750 1,500 3,000
Feet



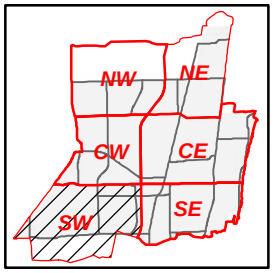
City of Pompano Beach
Stormwater Master Plan

Figure 2-1D
Topography Map
Basin CE



City of Pompano Beach
Stormwater Master Plan

Figure 2-1E
Topography Map
Basin SW



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

Digital Elevation Model

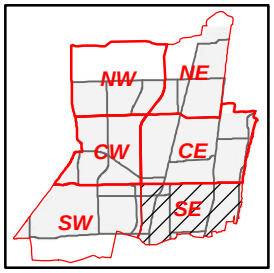
- High Elev.
- Low Elev.

N
W E
S

0 750 1,500 3,000
Feet

City of Pompano Beach
Stormwater Master Plan

Figure 2-1F
Topography Map
Basin SE



Legend

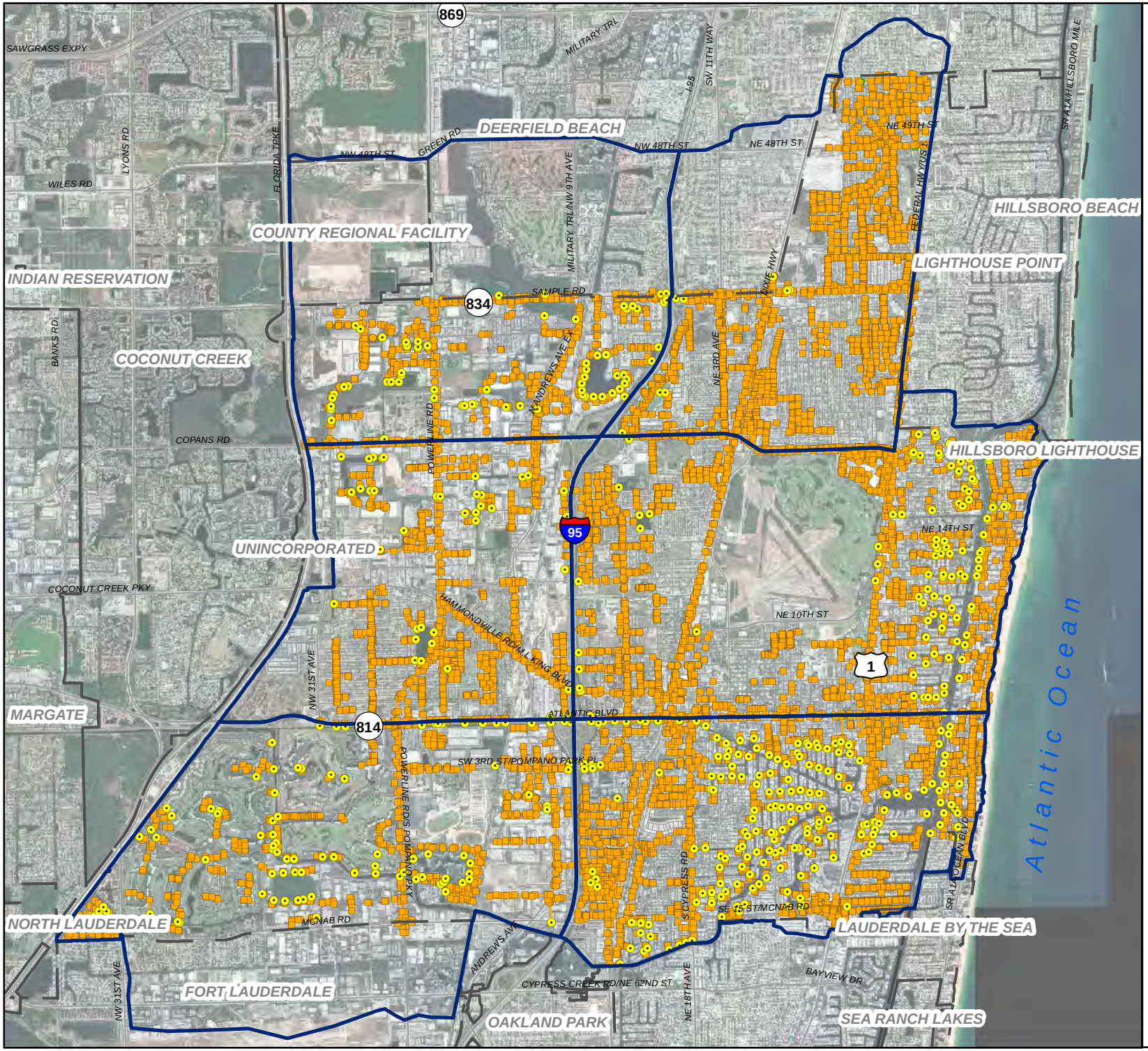
- Major Basin Boundary
- City Boundary
- Sub-Basins

Digital Elevation Model

- High Elev.
- Low Elev.

N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

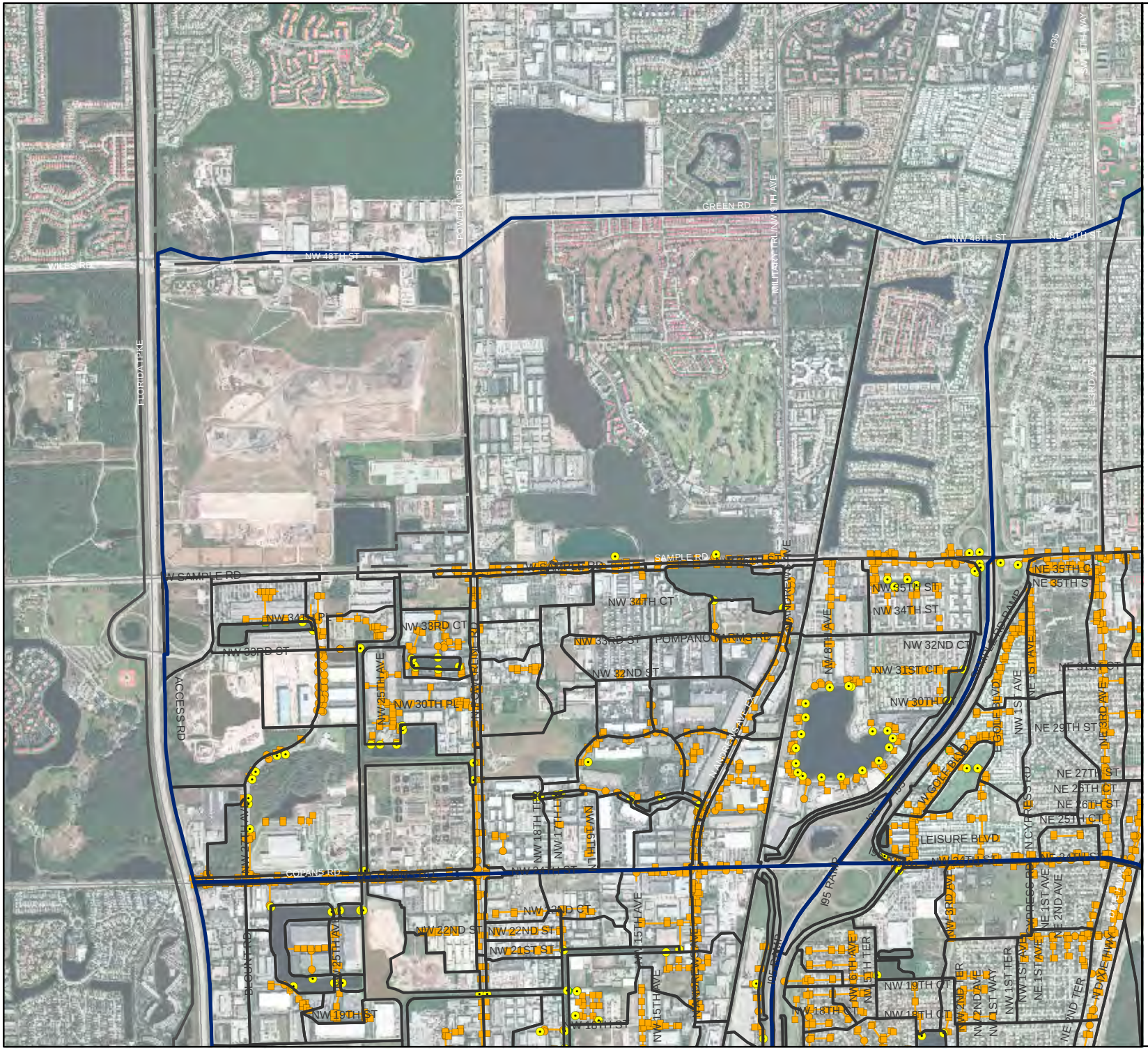
Figure 2-2
Existing Drainage Atlas

Legend

- Major Basin Boundary
- City Boundary
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe
- Streets

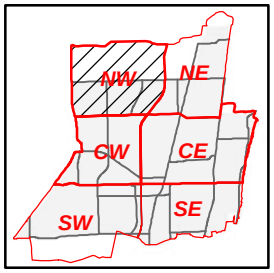
N
W E
S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-2A
Existing Drainage Atlas
Basin NW

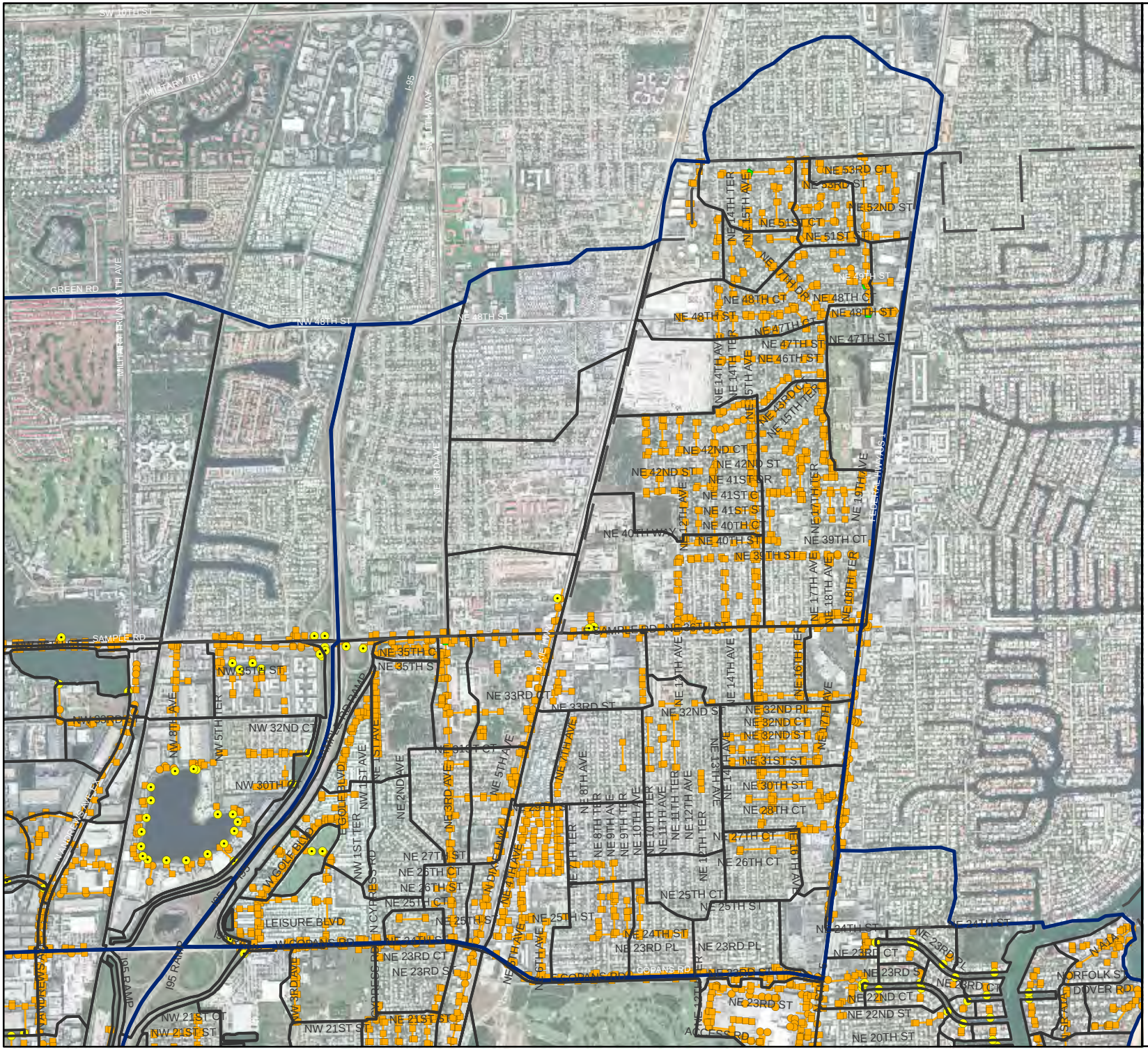


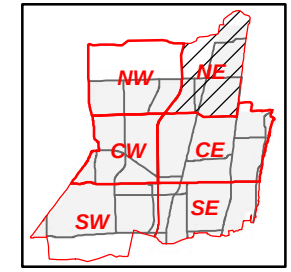
Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe

N
W E
S

0 750 1,500 3,000
Feet





City of Pompano Beach
Stormwater Master Plan

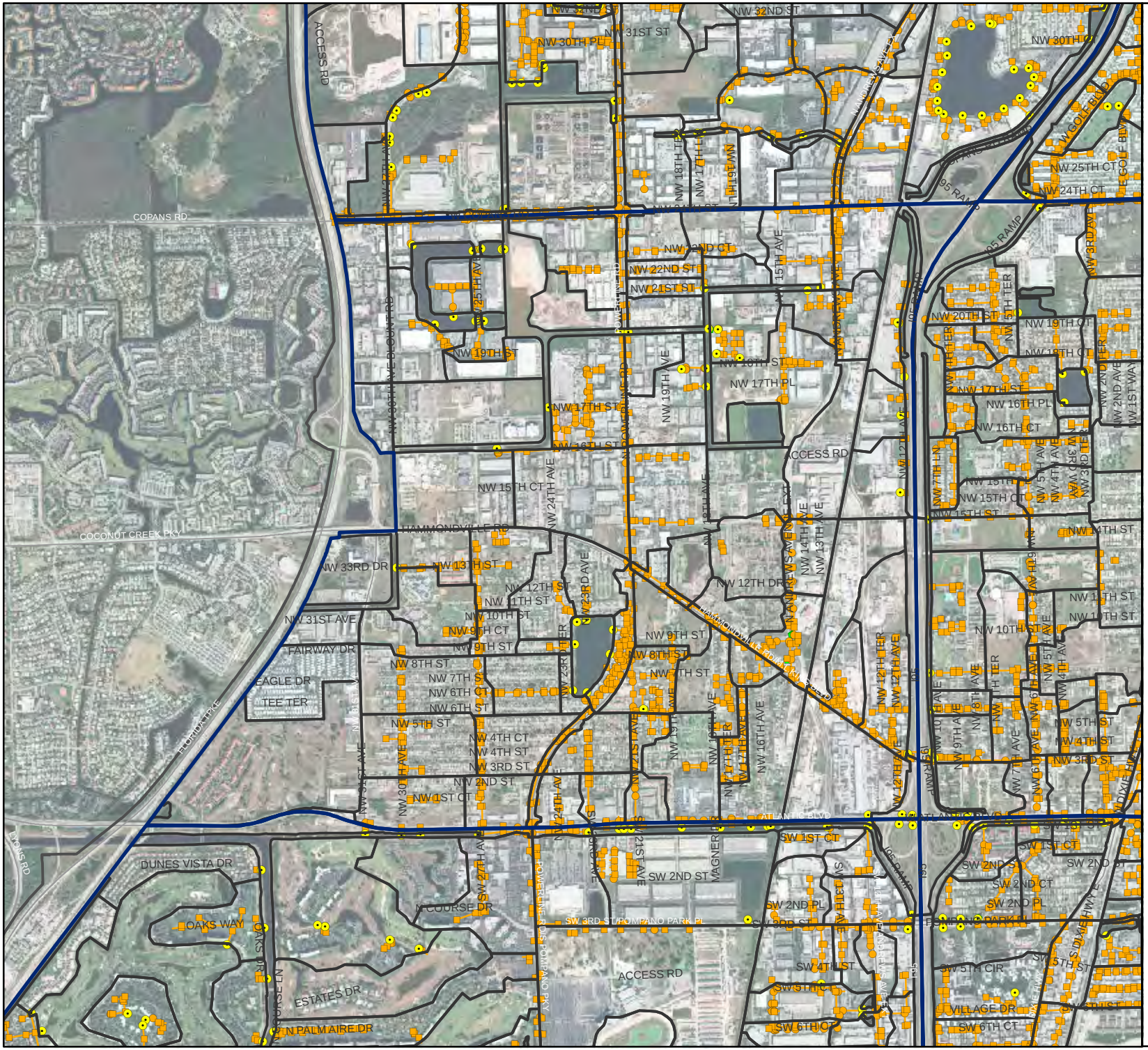
Figure 2-2B
Existing Drainage Atlas
Basin NE

Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe

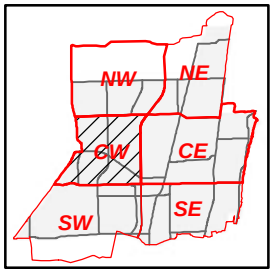
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-2C
Existing Drainage Atlas
Basin CW

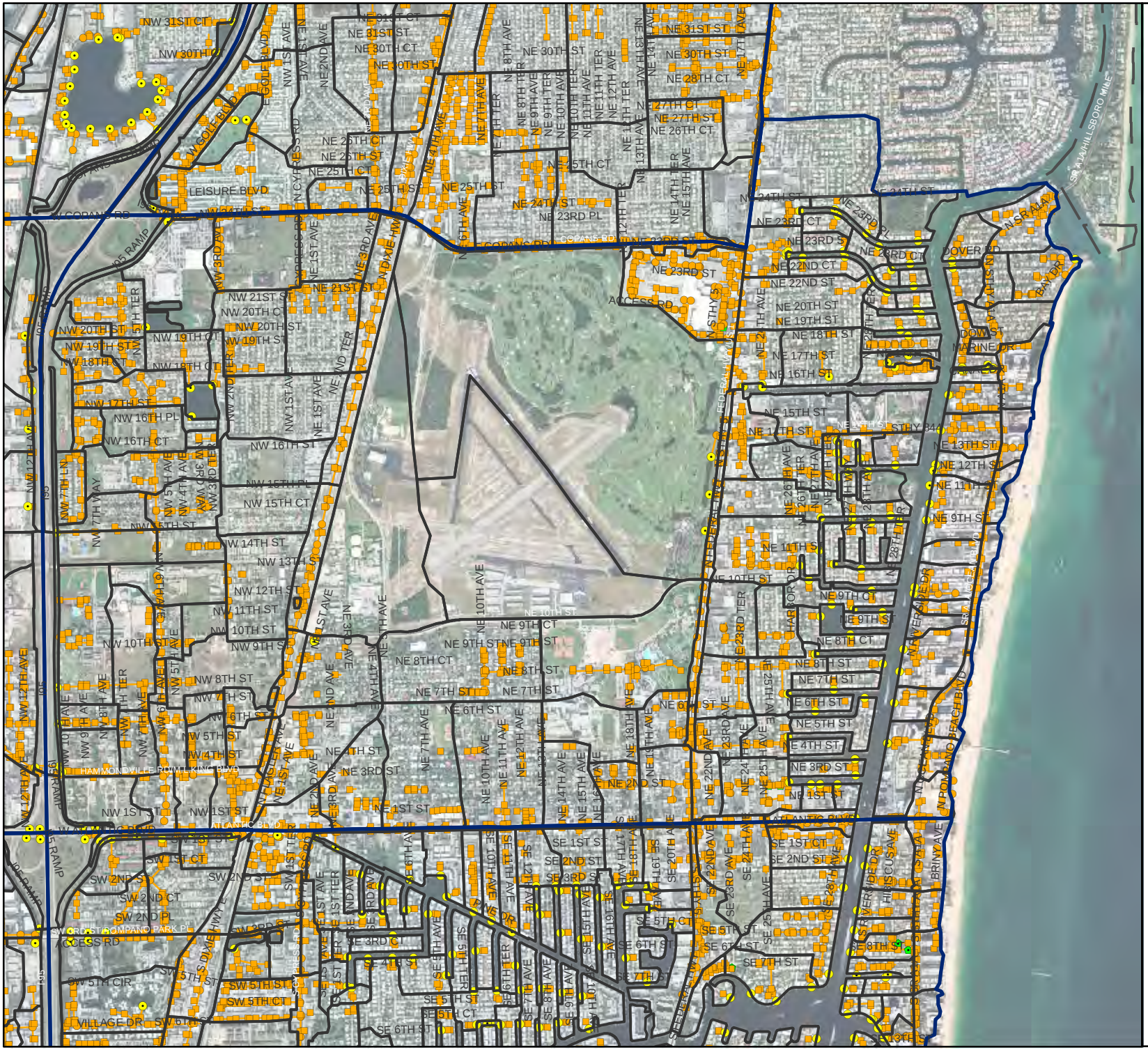


Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe

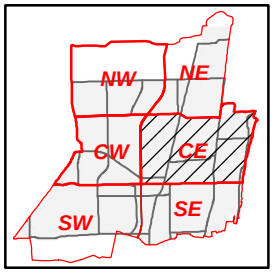
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-2D
Existing Drainage Atlas
Basin CE

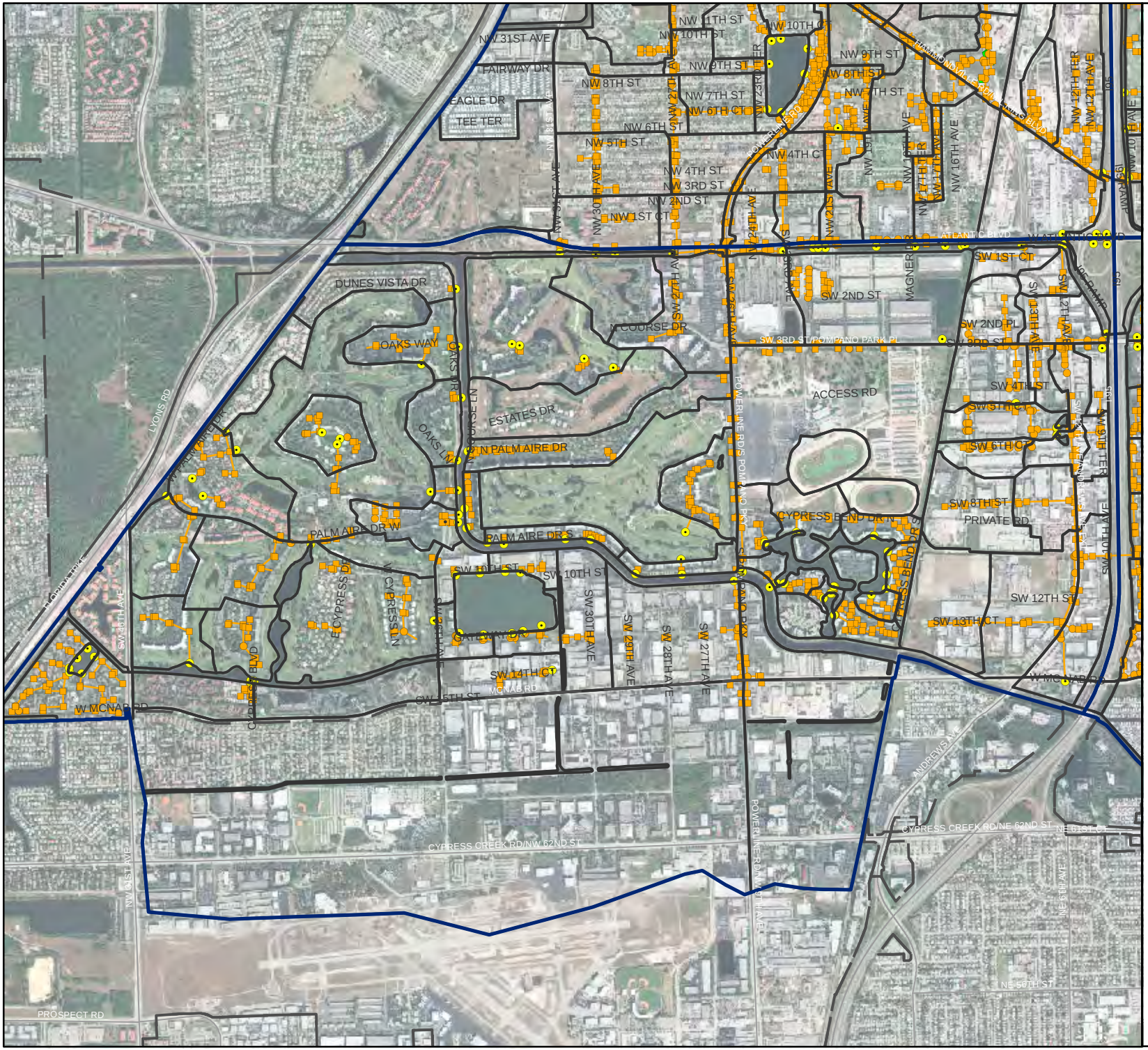


Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe

N
W E
S

0 750 1,500 3,000
Feet



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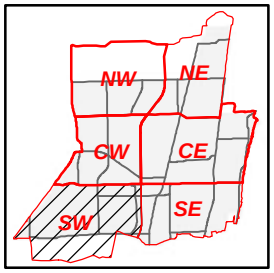
City of Pompano Beach
Stormwater Master Plan

Figure 2-2E
Existing Drainage Atlas
Basin SW

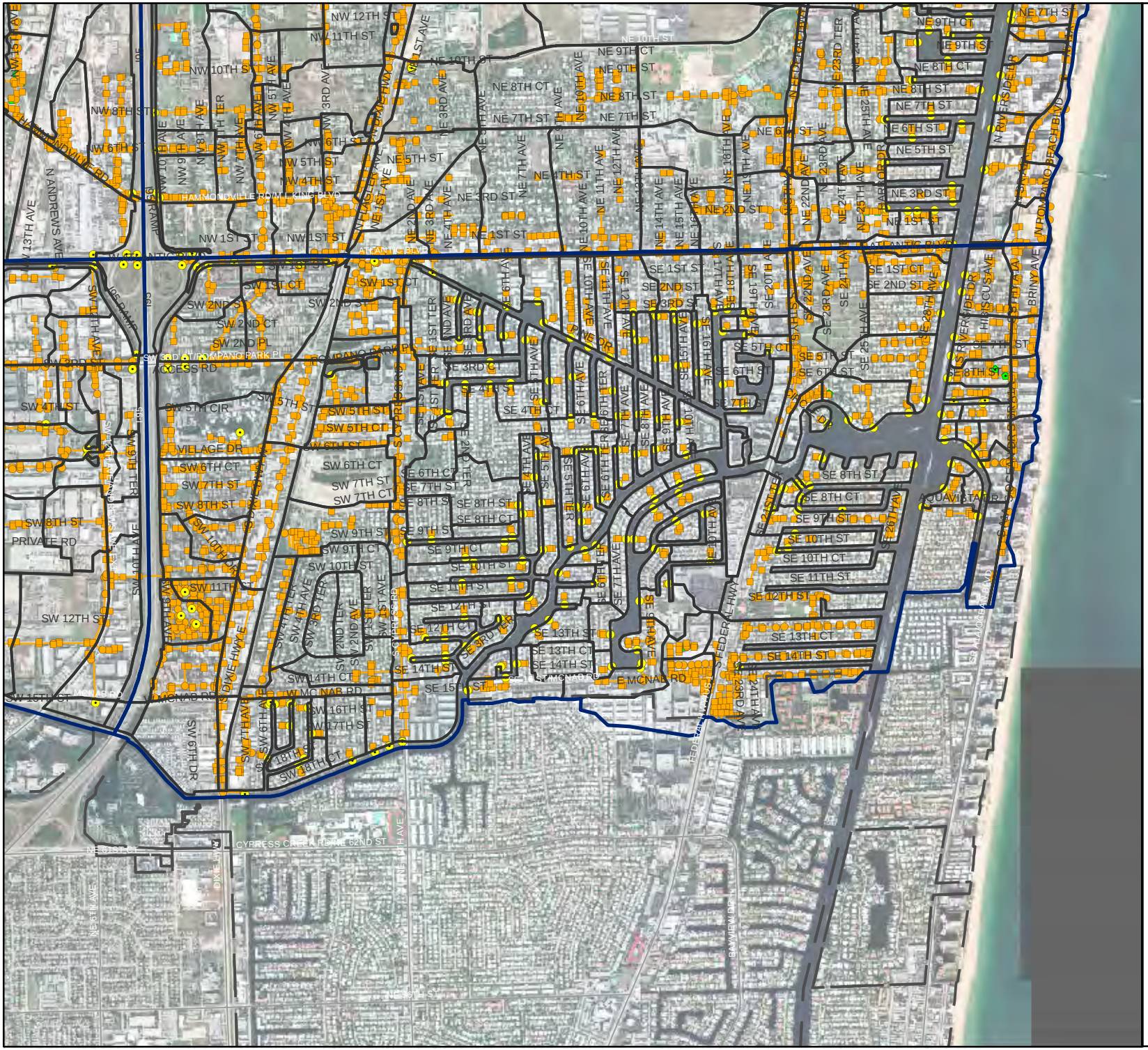


Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe

N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

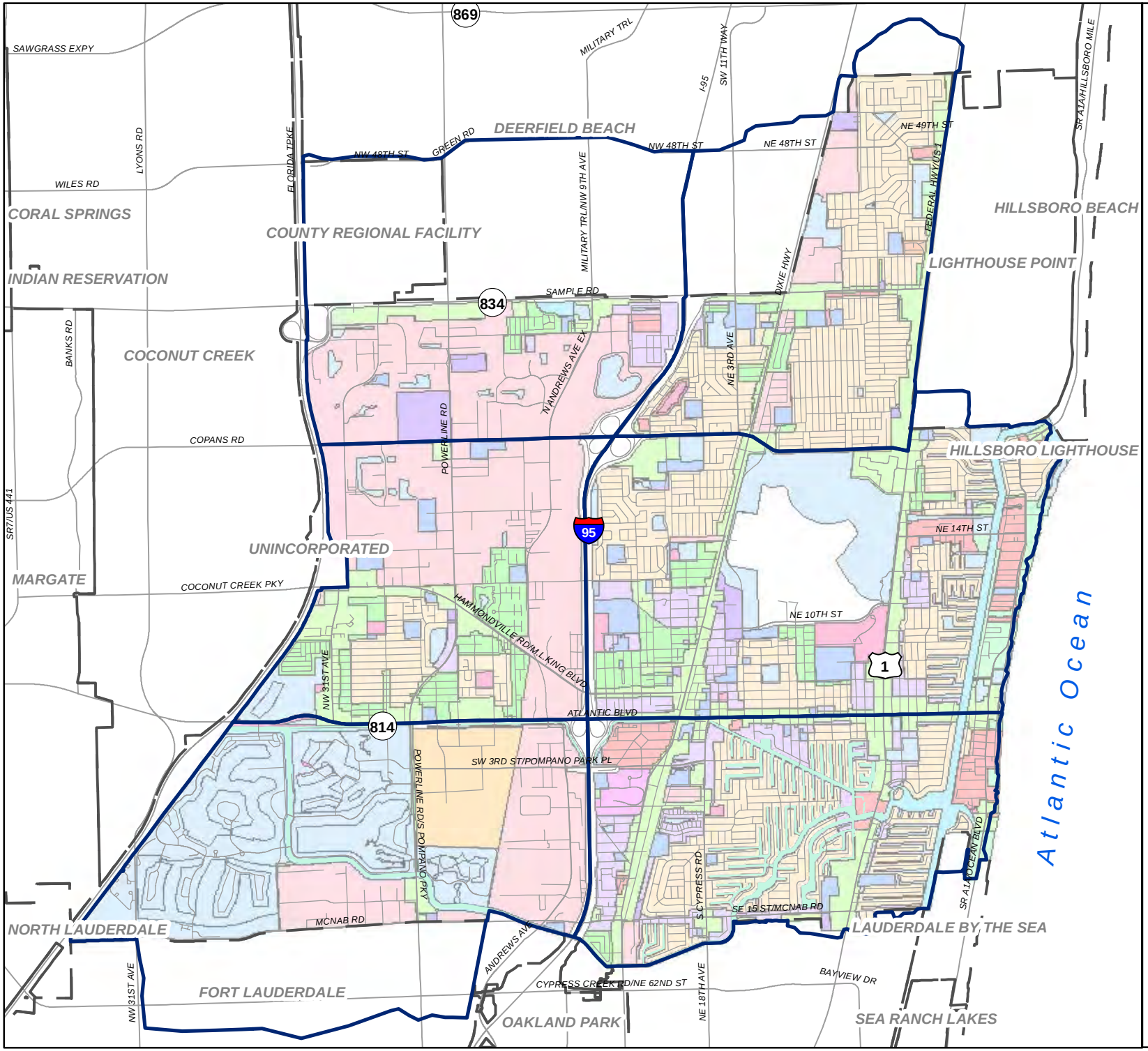
Figure 2-2F
Existing Drainage Atlas
Basin SE

Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins
- Outfall
- Inlet
- Manhole
- Network Structure
- Drainage Pipe

N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-3
Land Use Map

Legend

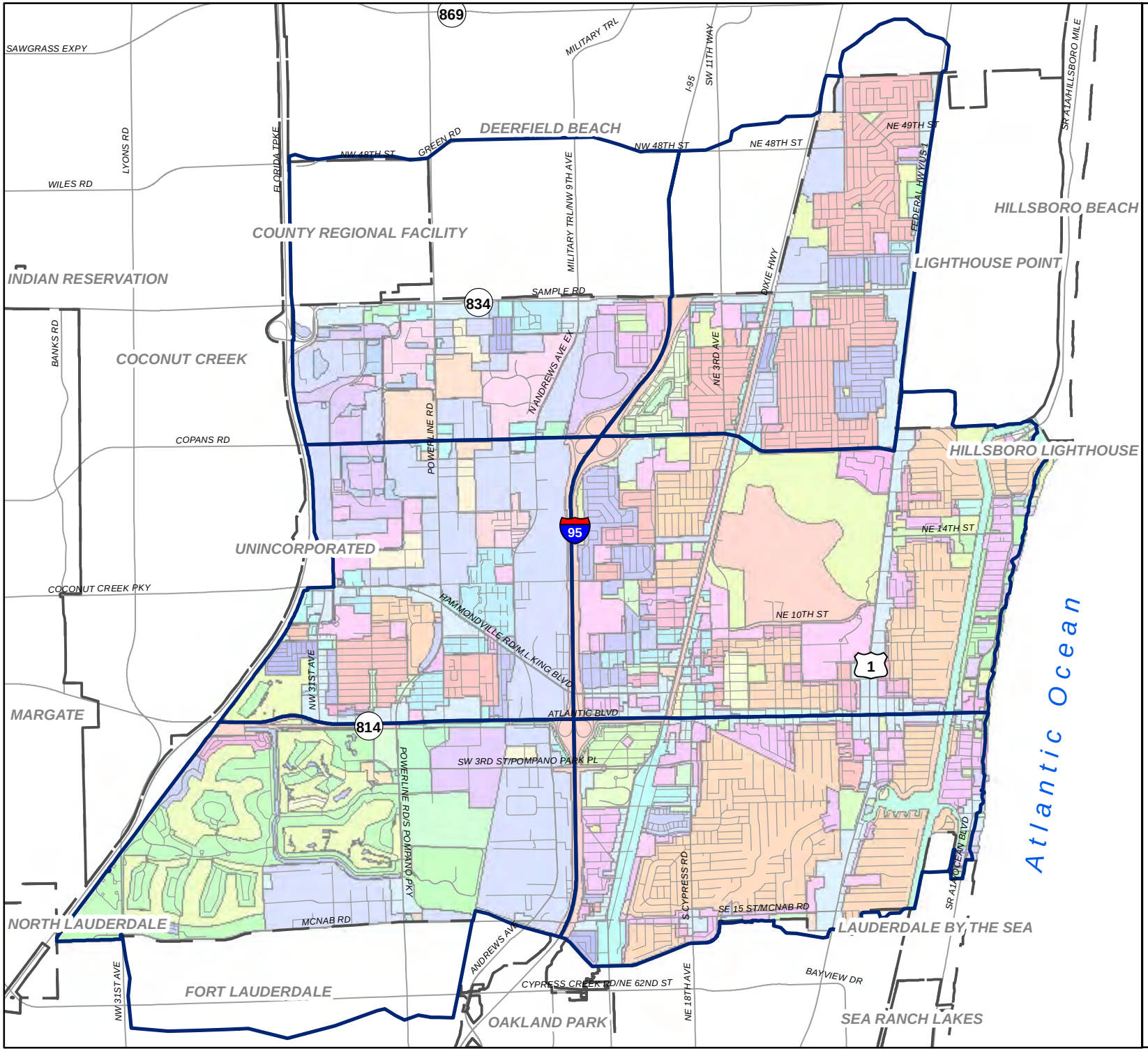
- Major Basin Boundary
- City Boundary
- Streets

LAND USE

- 12
- 36
- C
- CF
- H
- I
- L
- LAC
- LM
- M
- MH
- OR
- RAC
- T
- U
- W

N W E S

0 1,500 3,000 6,000 Feet



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City of Pompano Beach
Stormwater Master Plan

Figure 2-4
Zoning Map

Legend

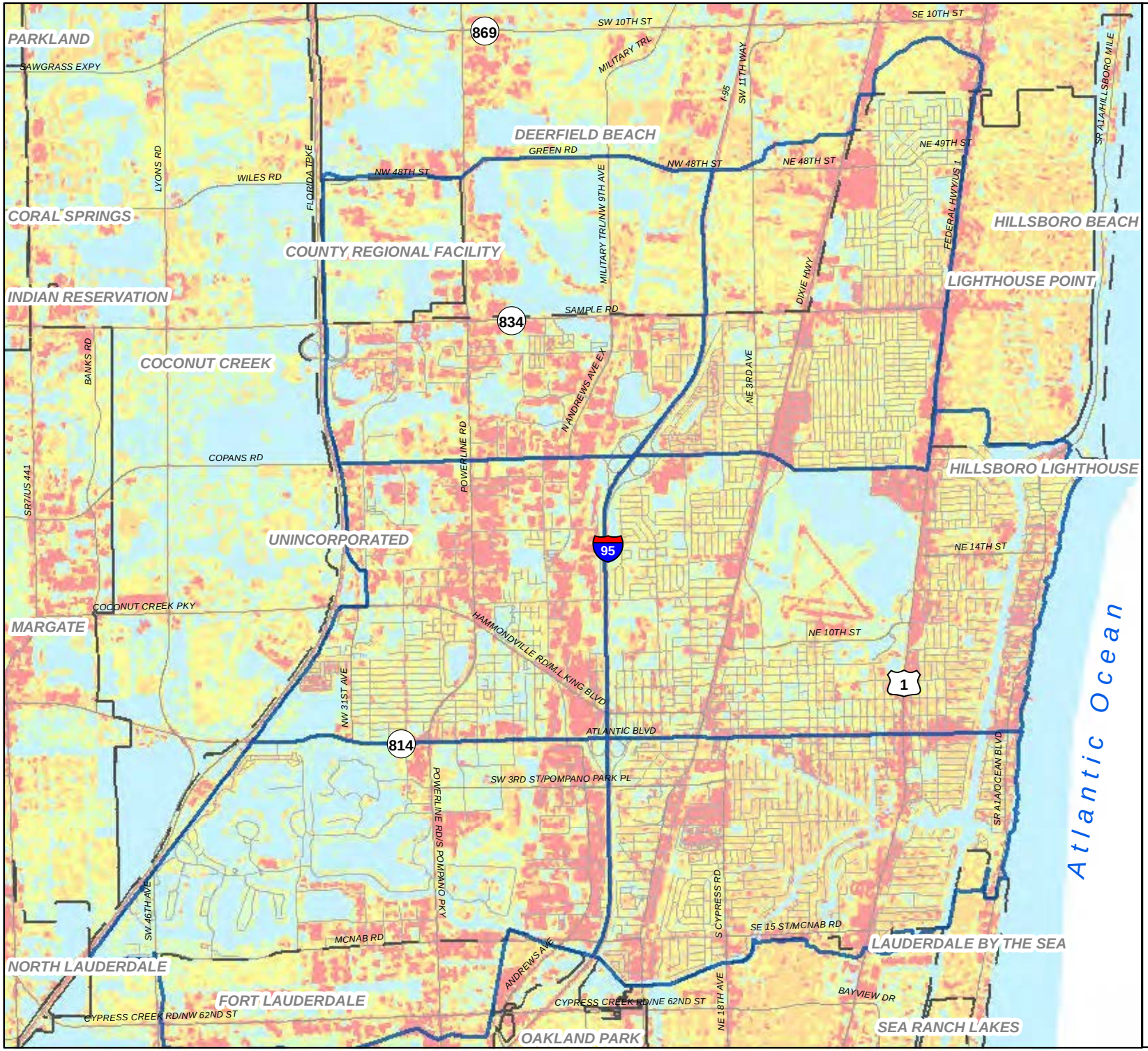
- Major Basin Boundary
- City Boundary
- Streets

ZONING

- AOD
- B-1
- B-2
- B-3
- B-3/PCD
- B-3/PCI
- B-4
- B-4/PCD
- BP
- CF
- CR
- HOD CORE
- HOD TRANS
- I-1
- I-1/PCD
- I-1/PCI
- I-1X
- LAC
- M-1
- M-1/PCD
- M-2
- MH-12
- O-IP
- O-IP/PCD
- O-IP/PCI
- PR
- PU
- RD-1
- RM-12
- RM-20
- RM-30
- RM-45
- RM-45/HR
- RM-45/PCI
- RPUD
- RPUD-6
- RS-1
- RS-2
- RS-3
- RS-4
- RS-L
- T
- W
- Water

N W E S

0 1,500 3,000 6,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-5
USGS SEAMLESS
Impervious Map

Legend

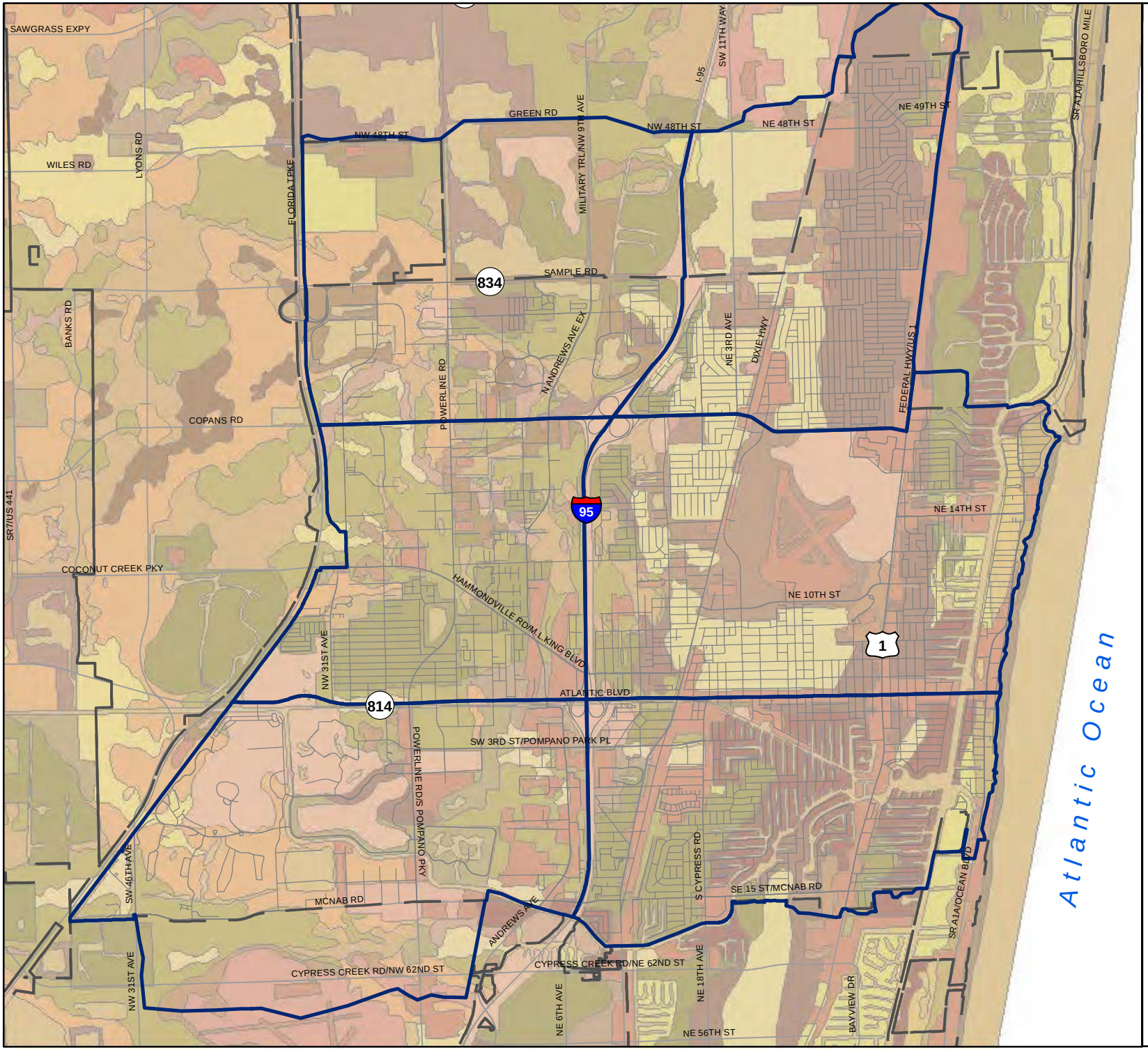
- Major Basin Boundaries
- City Boundary

Percent Impervious

- 0 - 13
- 13.1 - 33
- 33.1 - 51
- 51.1 - 72
- 72.1 - 100

N W E S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6
USGS Soils Map

Legend

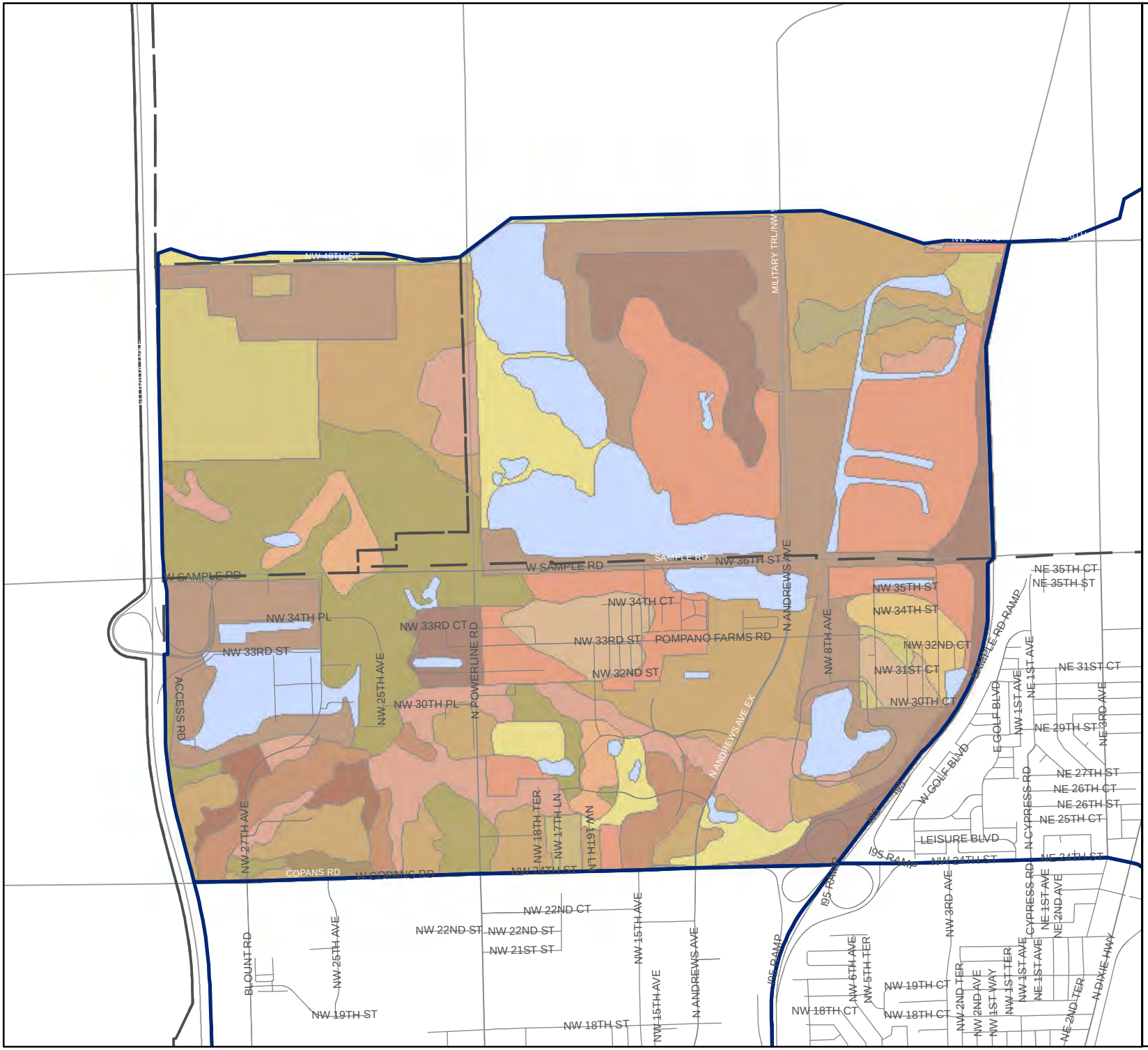
Major Basin Boundary
City Boundary

COMPNAME

- ARENTS - URBAN LAND COMPLEX
- ARENTS, ORGANIC SUBSTRATUM - URBAN LAND COMPLEX
- BASINGER AND MYAKKA SANDS, DEPRESSIONAL
- BASINGER FINE SAND
- BEACHES
- BOCA FINE SAND
- CANAVERAL - URBAN LAND COMPLEX
- DADE - URBAN LAND COMPLEX
- DADE FINE SAND
- DANIA MUCK
- DANIA MUCK, DEPRESSIONAL
- DUETTE - URBAN LAND COMPLEX
- HALLANDALE FINE SAND
- HALLENDALE - URBAN LAND COMPLEX
- HALLENDALE FINE SAND
- IMMOKALEE - URBAN LAND COMPLEX
- IMMOKALEE FINE SAND
- IMMOKALEE, LIMESTONE SUBSTRATUM - URBAN LAND COMPLEX
- LAUDERHILL MUCK
- LAUDERHILL MUCK, DEPRESSIONAL
- MARGATE FINE SAND
- MATLASHA - URBAN LAND COMPLEX
- MATLASHA GRAVELLY FINE SAND, LIMESTONE SUBSTRATUM
- NO DATA
- OKEELANTA MUCK
- PALM BEACH - URBAN LAND COMPLEX
- PALM BEACH - URBAN LAND COMPLEX, 0 TO 8 PCT SLOPES
- PALM BEACH SAND
- PAOLA - URBAN LAND COMPLEX
- PAOLA FINE SAND
- PENNSUCO SILTY CLAY LOAM, DRAINED
- PENNSUCO SILTY CLAY LOAM, TIDAL
- PERRINE SILTY CLAY LOAM, DRAINED
- PERRINE VARIANT SILT LOAM
- PINEDA FINE SAND
- PLANTATION MUCK
- POMELLO FINE SAND
- POMPANO FINE SAND
- SANIBEL MUCK
- ST. LUCIE - PAOLA - URBAN LAND COMPLEX, 0 TO 8 PCT SLOPES
- ST. LUCIE FINE SAND
- TERRA CEIA MUCK, TIDAL
- UDORTHENTS
- UDORTHENTS - URBAN LAND COMPLEX
- UDORTHENTS - WATER COMPLEX
- UDORTHENTS, MARLY SUBSTRATUM - URBAN LAND COMPLEX
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

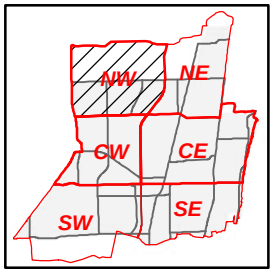
N W E S

0 1,500 3,000 6,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6A
USGS Soils Map
Basin NW



Legend

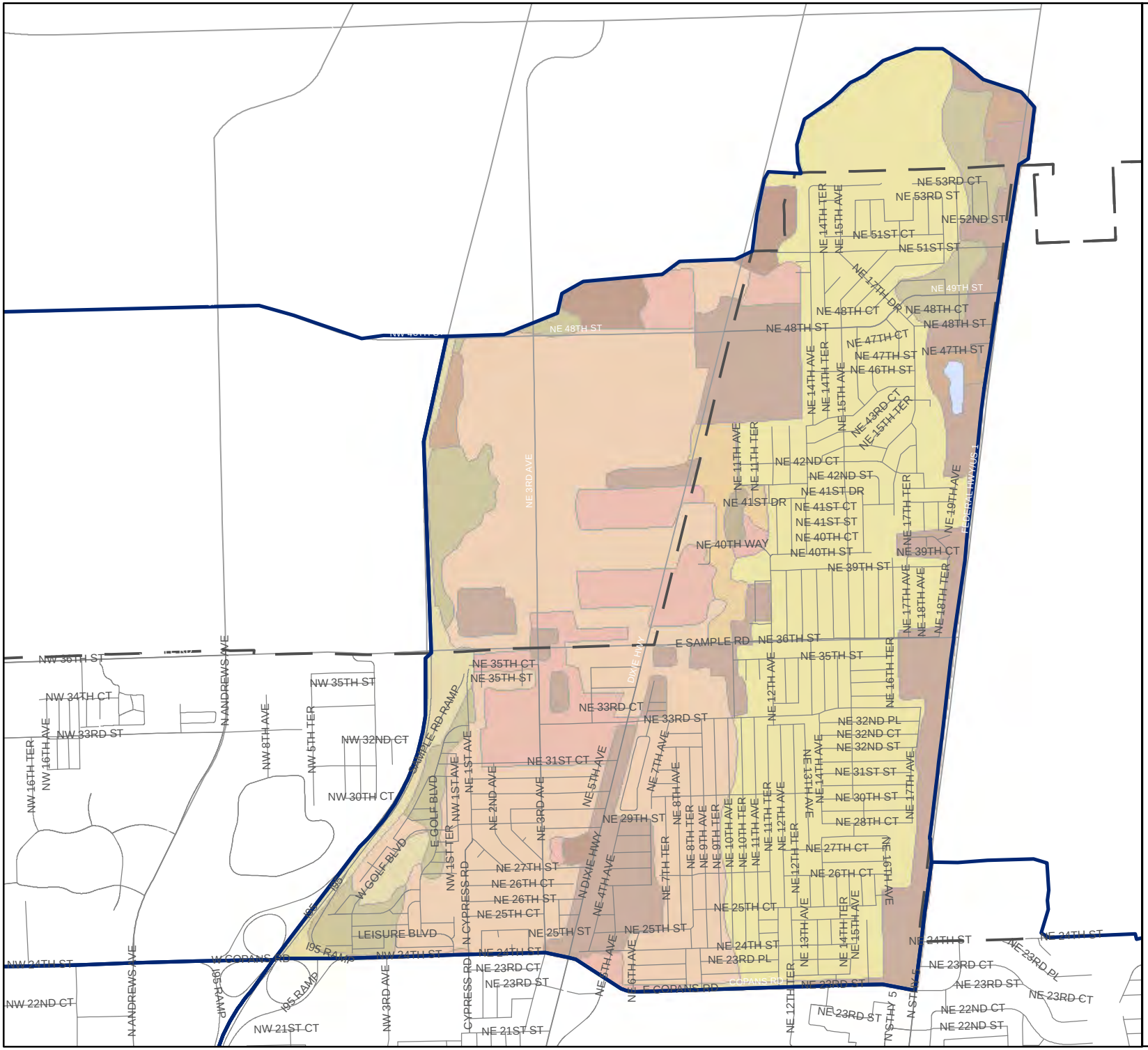
- Major Basin Boundary
- City Boundary

SOILS DESCRIPTION

- ARENTS - URBAN LAND COMPLEX
- BASINGER FINE SAND
- DUETTE - URBAN LAND COMPLEX
- HALLENDALE FINE SAND
- IMMOKALEE - URBAN LAND COMPLEX
- IMMOKALEE FINE SAND
- MARGATE FINE SAND
- MATLASHA GRAVELLY FINE SAND
- PAOLA - URBAN LAND COMPLEX
- PLANTATION MUCK
- POMELLO FINE SAND
- POMPANO FINE SAND
- SANIBEL MUCK
- UDORTHENTS
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

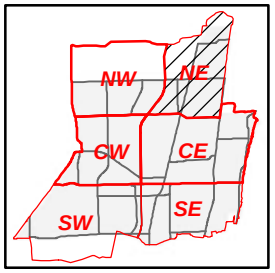
N W E S

0 750 1,500 3,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6B
USGS Soils Map
Basin NE



Legend

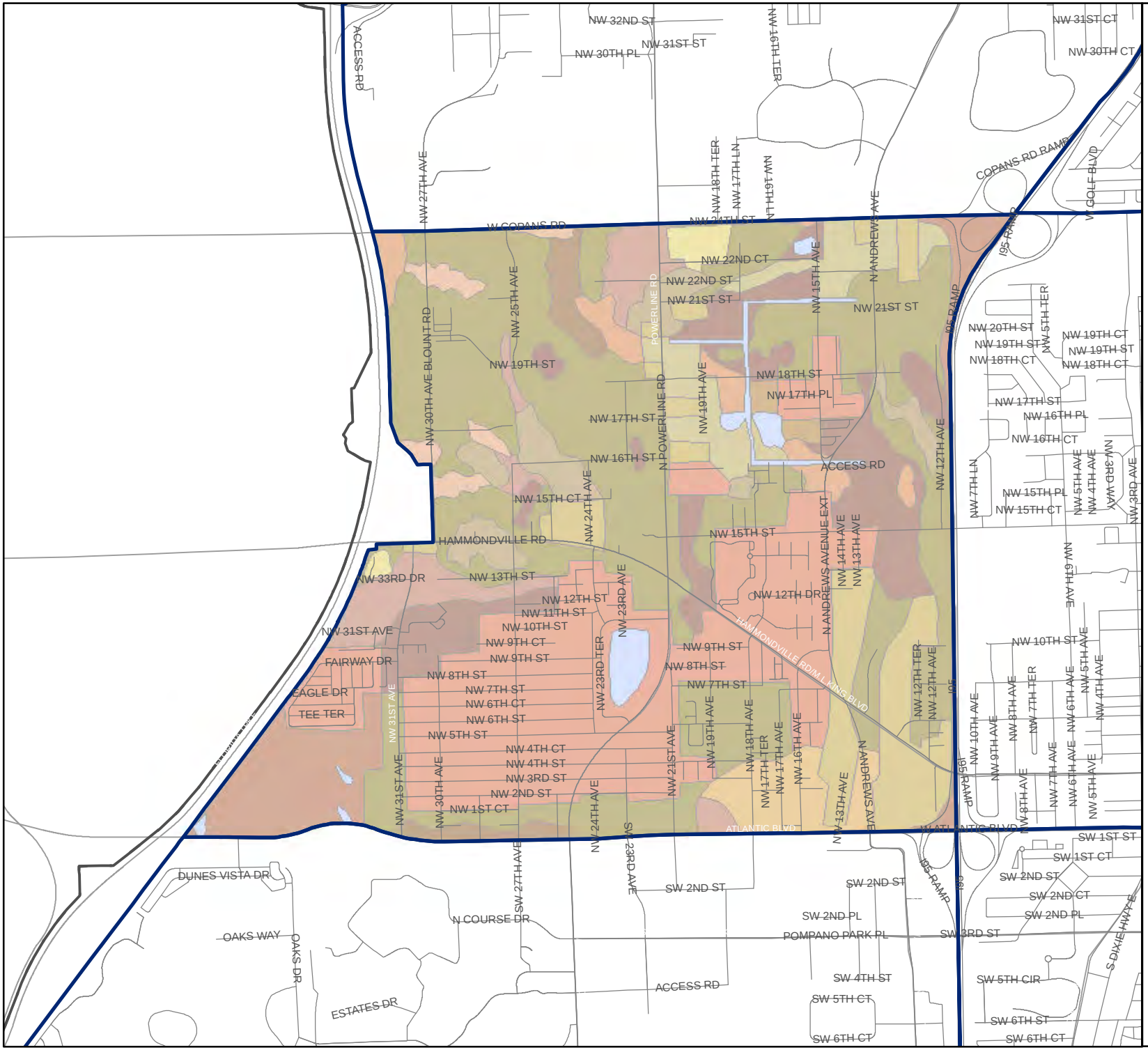
- Major Basin Boundary
- City Boundary

SOILS DESCRIPTION

- ARENTS - URBAN LAND COMPLEX
- DUETTE - URBAN LAND COMPLEX
- IMMOKALEE - URBAN LAND COMPLEX
- PAOLA - URBAN LAND COMPLEX
- PAOLA FINE SAND
- POMELLO FINE SAND
- POMPANO FINE SAND
- ST. LUCIE FINE SAND
- UDORTHENTS
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

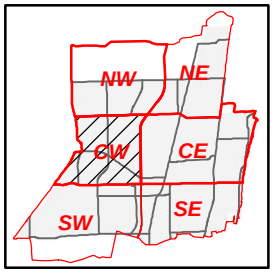
N W E S

0 750 1,500 3,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6C
USGS Soils Map
Basin CW



Legend

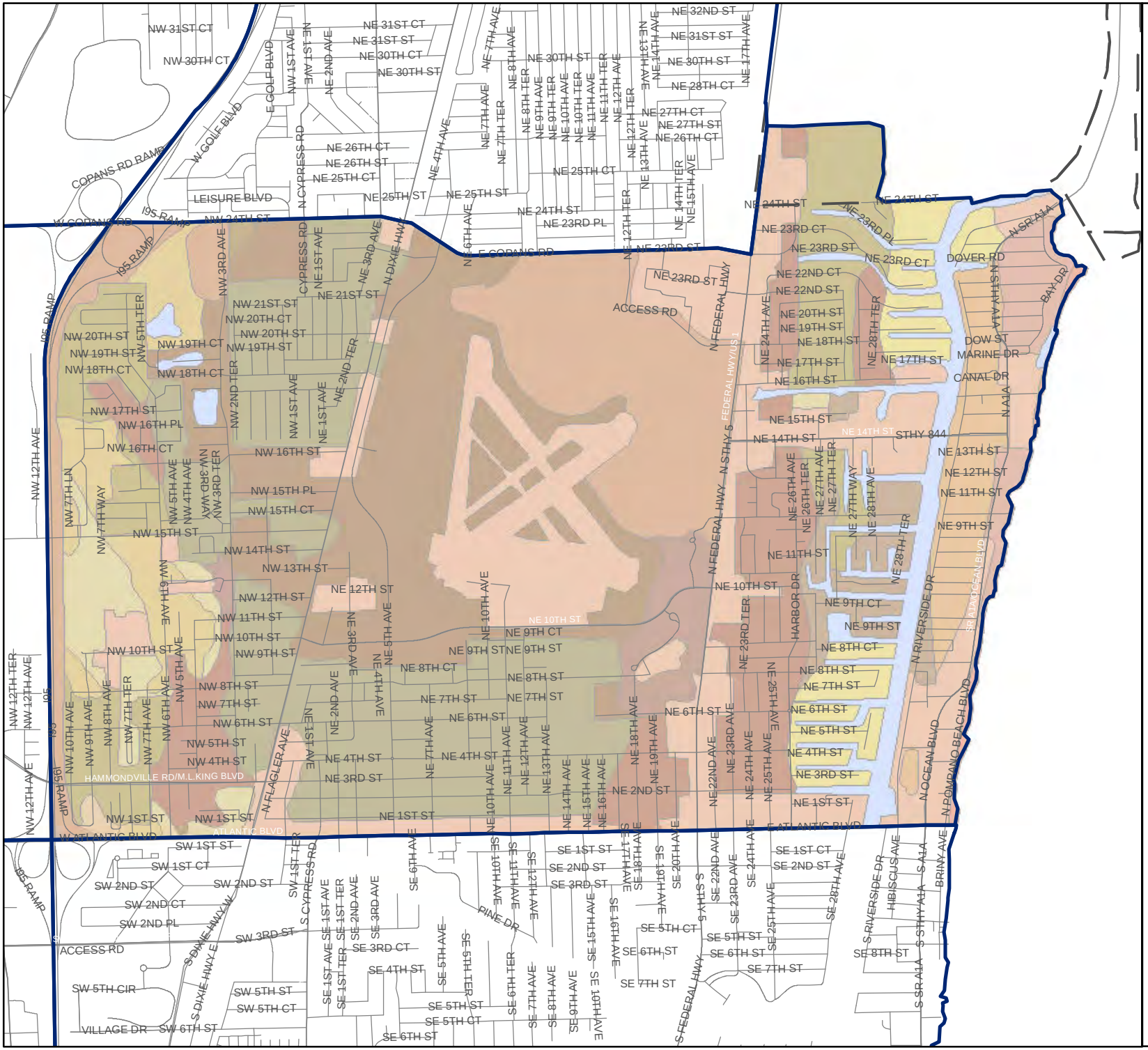
- Major Basin Boundary
- City Boundary

SOILS DESCRIPTION

- BASINGER FINE SAND
- HALLENDALE - URBAN LAND COMPLEX
- HALLENDALE FINE SAND
- IMMOKALEE - URBAN LAND COMPLEX
- IMMOKALEE FINE SAND
- IMMOKALEE, LIMESTONE SUBSTRATUM
- MARGATE FINE SAND
- MATLASHA GRAVELLY FINE SAND, LIMESTONE
- POMELLO FINE SAND
- POMPANO FINE SAND
- SANIBEL MUCK
- UDORTHENTS
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

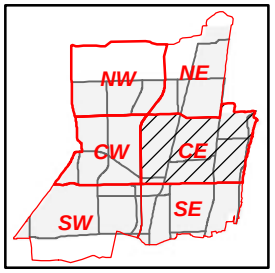
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6D
USGS Soils Map
Basin CE



Legend

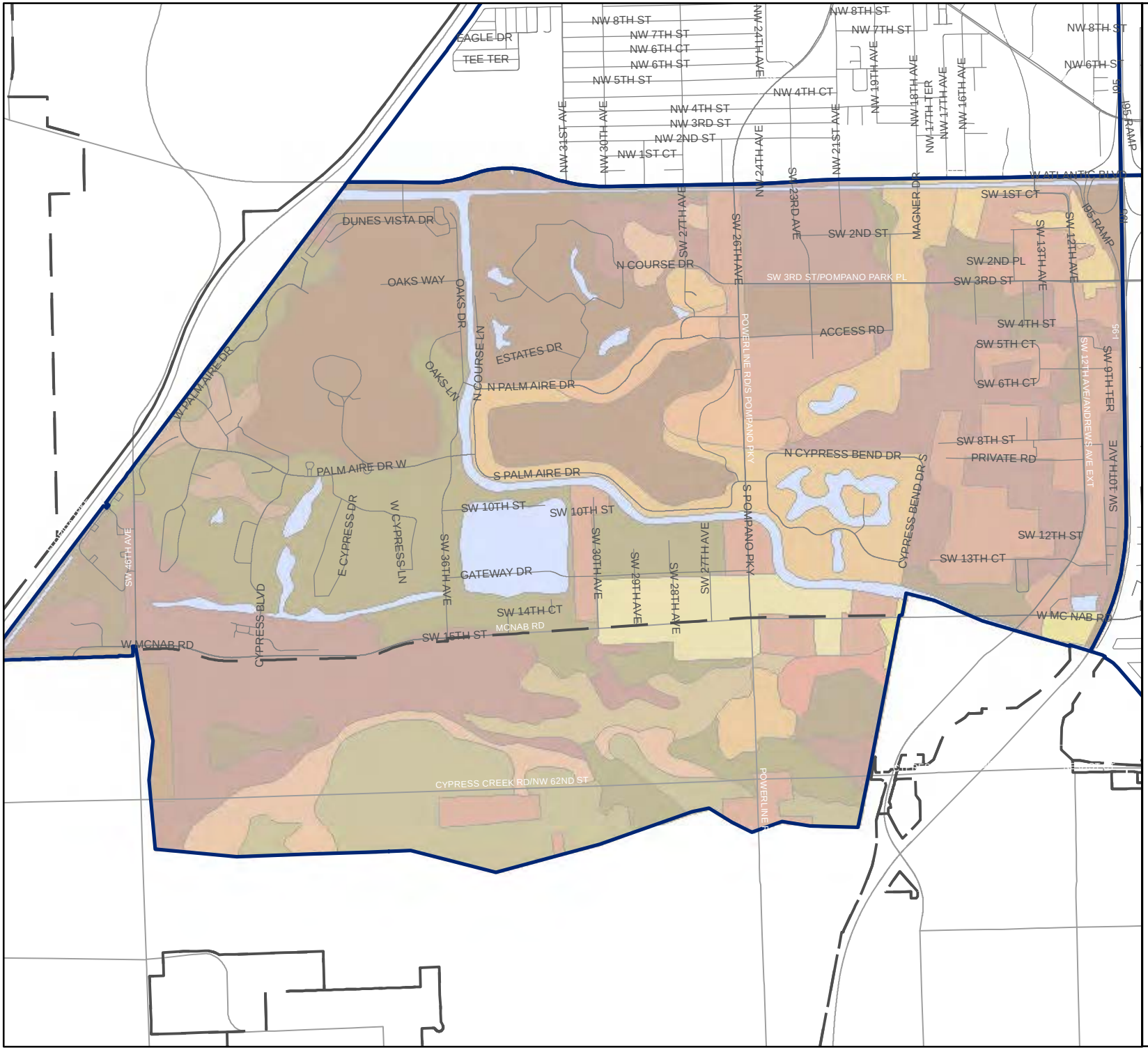
- Major Basin Boundary
- City Boundary

SOILS DESCRIPTION

- ARENTS - URBAN LAND COMPLEX
- ARENTS, ORGANIC SUBSTRATUM
- BEACHES
- CANAVERAL - URBAN LAND COMPLEX
- DUETTE - URBAN LAND COMPLEX
- IMMOKALEE - URBAN LAND COMPLEX
- IMMOKALEE FINE SAND
- MATLASHA - URBAN LAND COMPLEX
- PALM BEACH - URBAN LAND COMPLEX
- PAOLA - URBAN LAND COMPLEX
- PAOLA FINE SAND
- POMELLO FINE SAND
- POMPANO FINE SAND
- ST. LUCIE FINE SAND
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

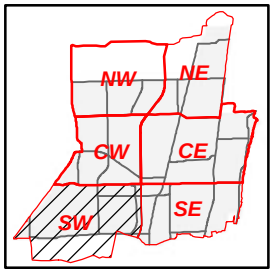
N W E S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6E
USGS Soils Map
Basin SW



Legend

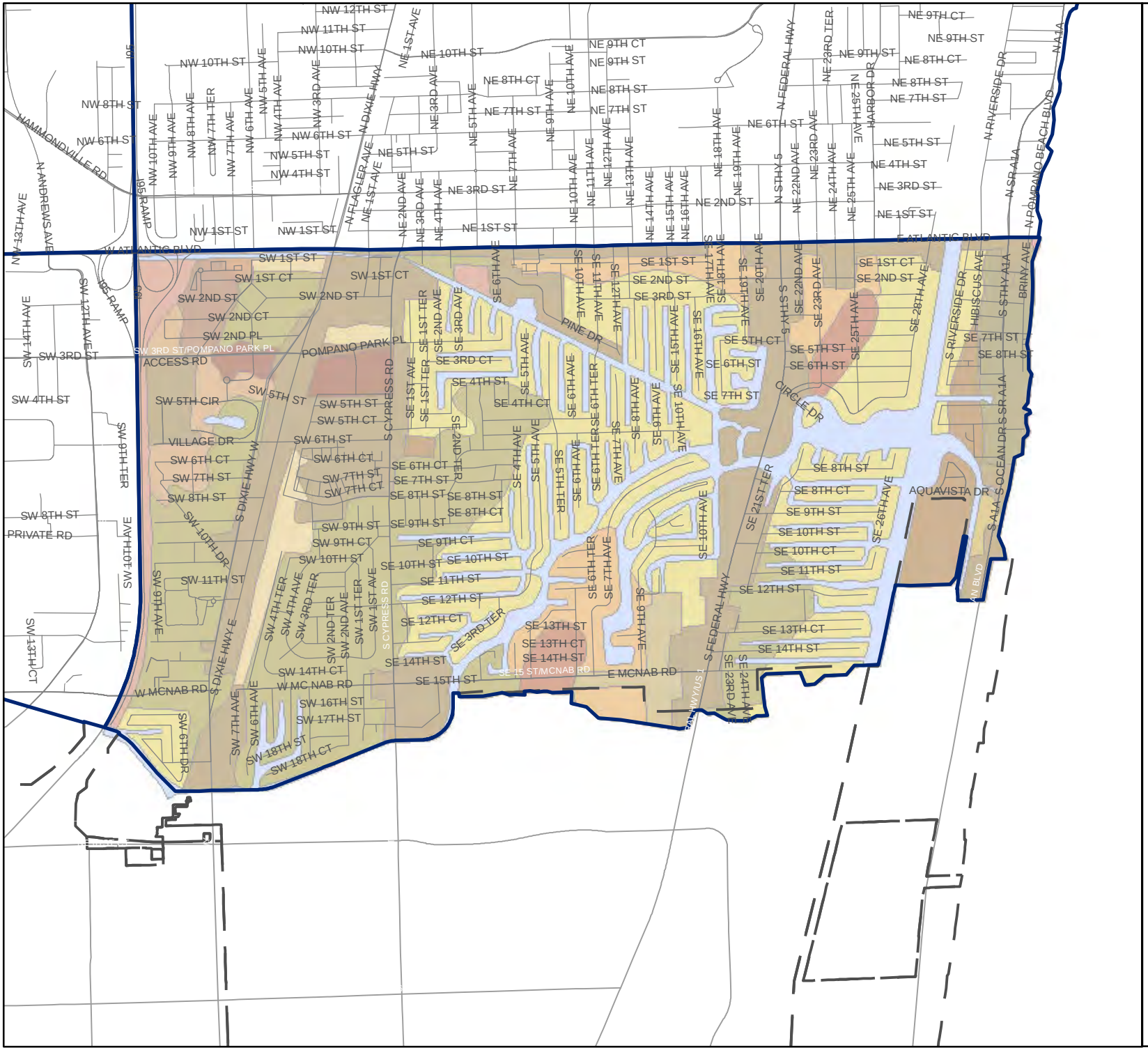
- Major Basin Boundary
- City Boundary

SOIL DESCRIPTION

- ARENTS, ORGANIC SUBSTRATUM
- BASINGER FINE SAND
- DUETTE - URBAN LAND COMPLEX
- HALLENDALE FINE SAND
- IMMOKALEE - URBAN LAND COMPLEX
- IMMOKALEE FINE SAND
- IMMOKALEE, LIMESTONE SUBSTRATUM
- MARGATE FINE SAND
- MATLASHA - URBAN LAND COMPLEX
- MATLASHA GRAVELLY FINE SAND, LIMESTONE
- POMELLO FINE SAND
- POMPANO FINE SAND
- SANIBEL MUCK
- ST. LUCIE FINE SAND
- UDORTHENTS - URBAN LAND COMPLEX
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

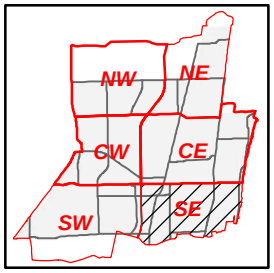
N W E S

0 750 1,500 3,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-6F
USGS Soils Map
Basin SE



Legend

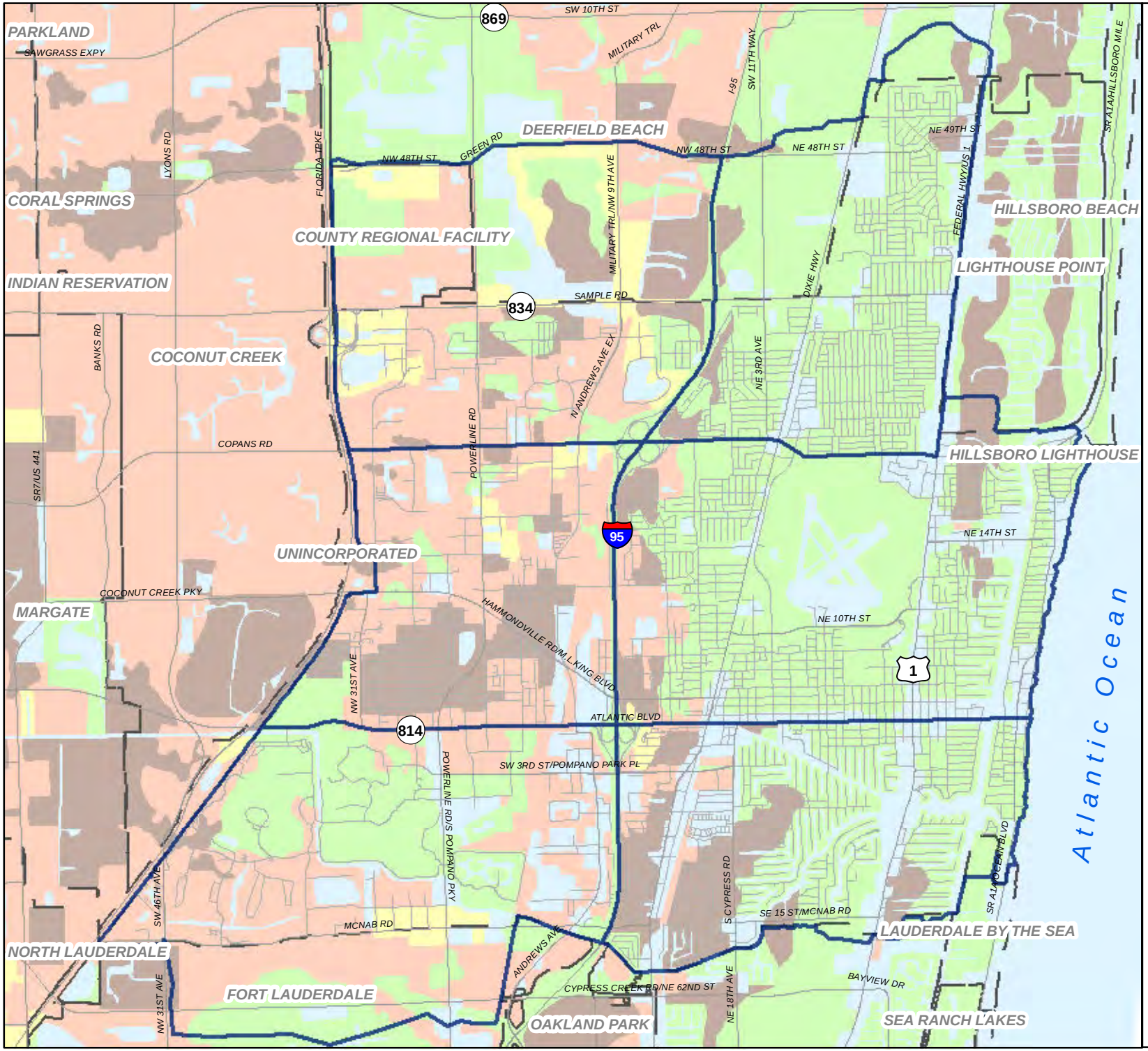
- Major Basin Boundary
- City Boundary

SOILS DESCRIPTION

- ARENTS - URBAN LAND COMPLEX
- ARENTS, ORGANIC SUBSTRATUM
- BASINGER FINE SAND
- BEACHES
- DUETTE - URBAN LAND COMPLEX
- IMMOKALEE - URBAN LAND COMPLEX
- IMMOKALEE FINE SAND
- IMMOKALEE, LIMESTONE SUBSTRATUM
- MATLASHA - URBAN LAND COMPLEX
- PALM BEACH - URBAN LAND COMPLEX
- PALM BEACH SAND
- PAOLA - URBAN LAND COMPLEX
- POMELLO FINE SAND
- POMPANO FINE SAND
- UDORTHENTS, SHAPED
- URBAN LAND
- WATER

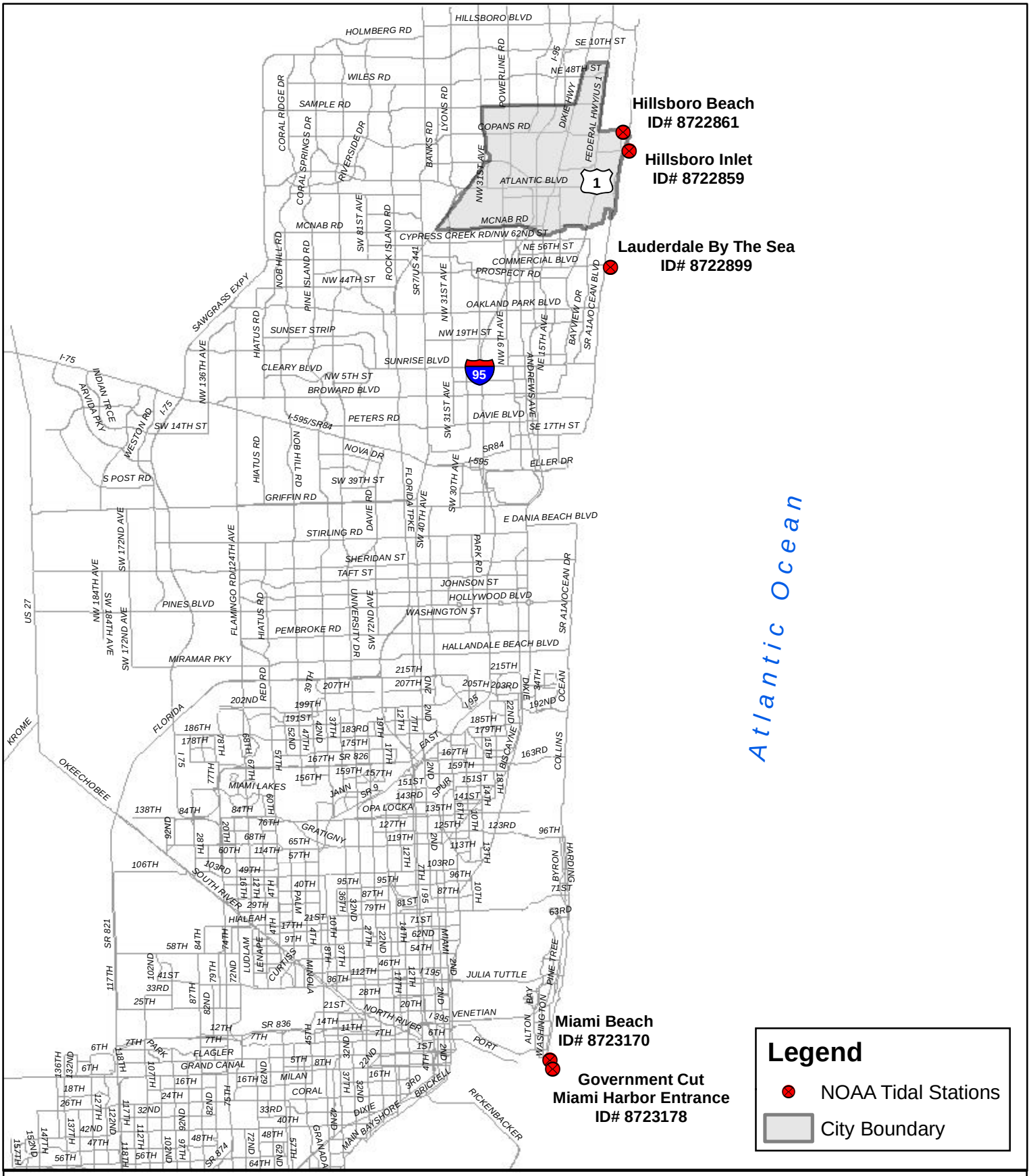
N W E S

0 750 1,500 3,000 Feet



City of Pompano Beach
Stormwater Master Plan

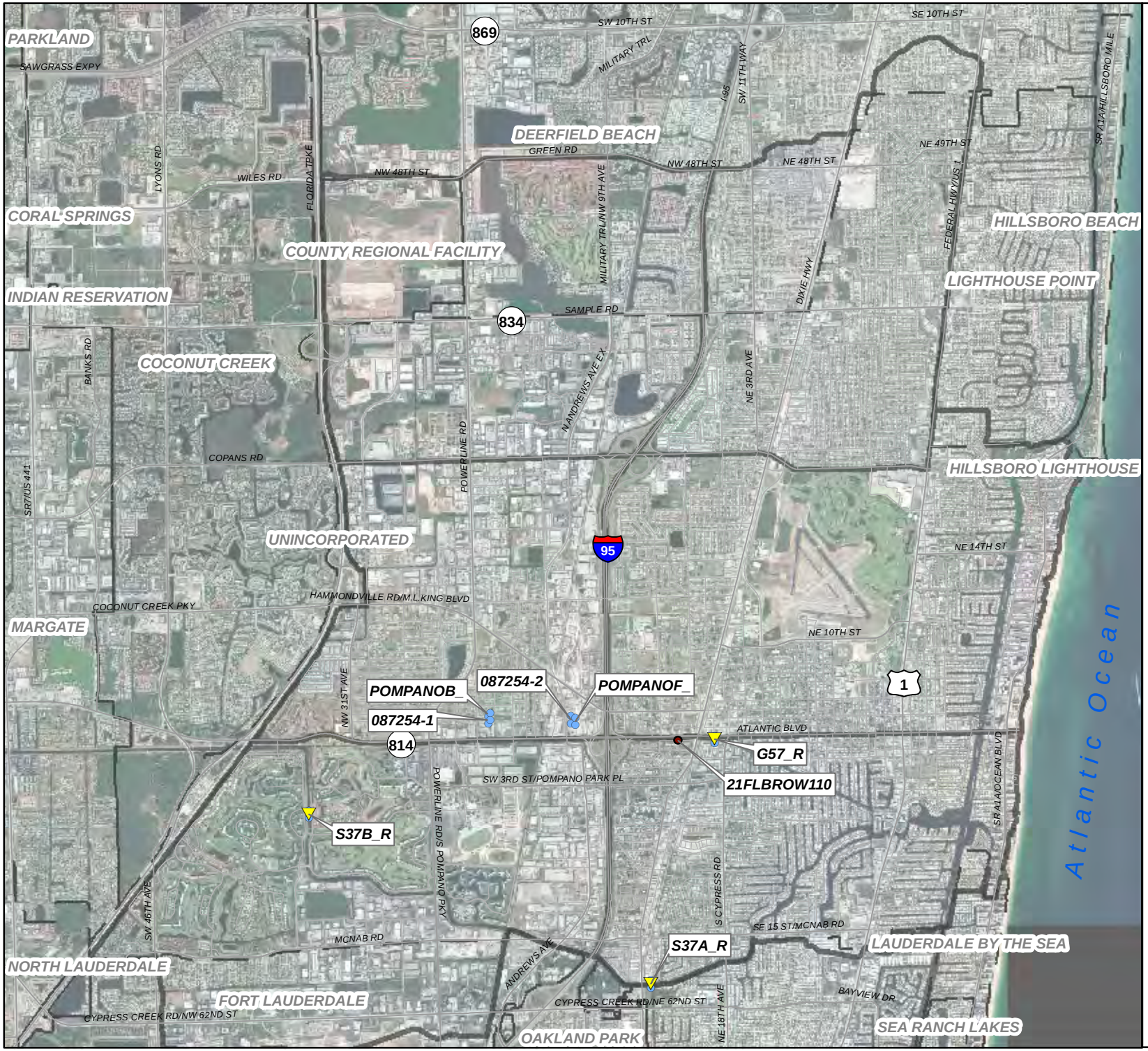
Figure 2-7
HGS Soils Map



City of Pompano Beach
Stormwater Master Plan

Figure 2-8
NOAA Tidal Stations

0 12,50025,000
Feet

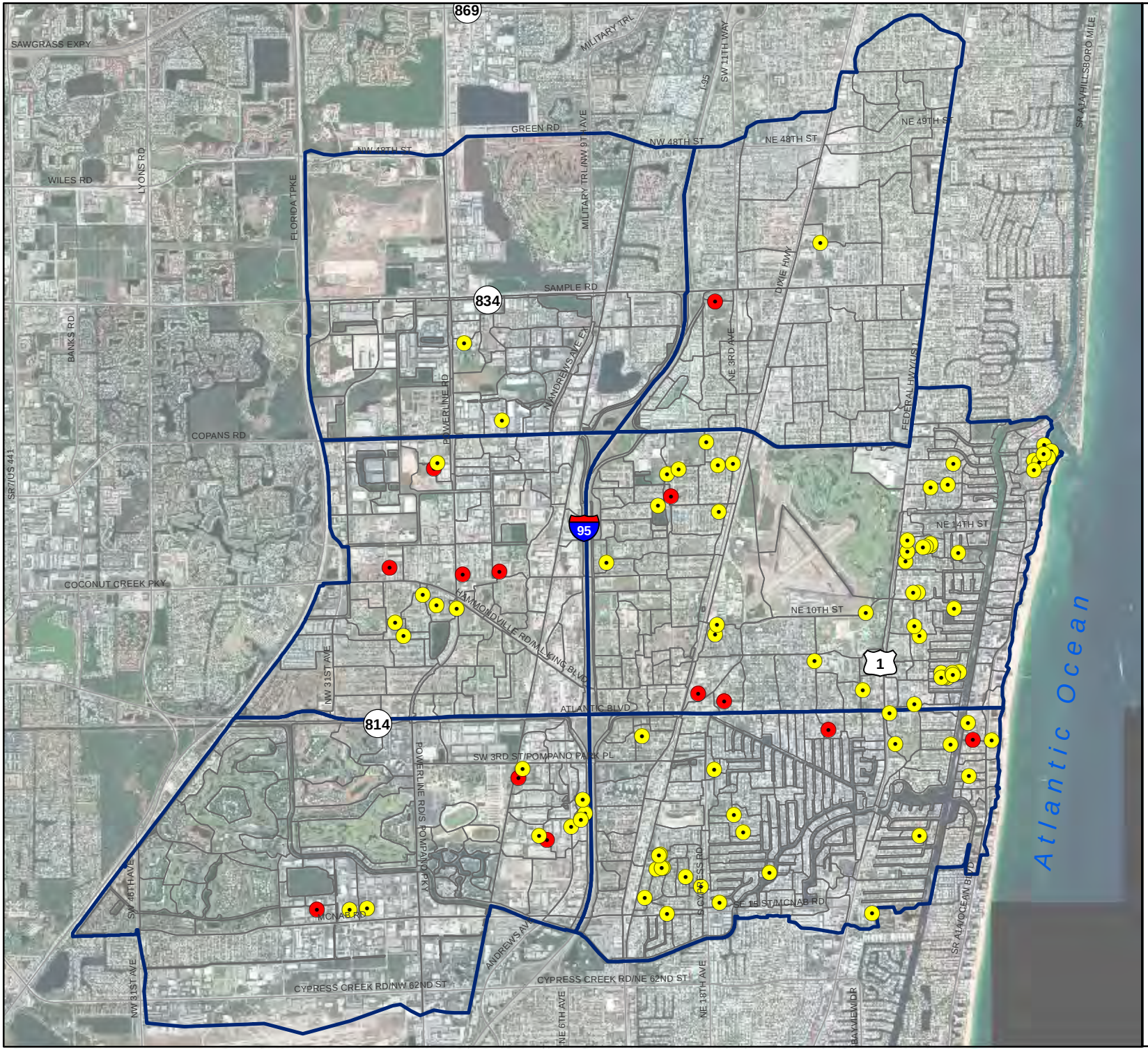


City of Pompano Beach
Stormwater Master Plan

Figure 2-9
Rainfall and Canal Level Gauge Locations

Legend

- Water Quality Gauge
- Canal Level Gauges
- Rainfall Gauges
- City Streets
- Major Basin Boundaries
- City Boundary

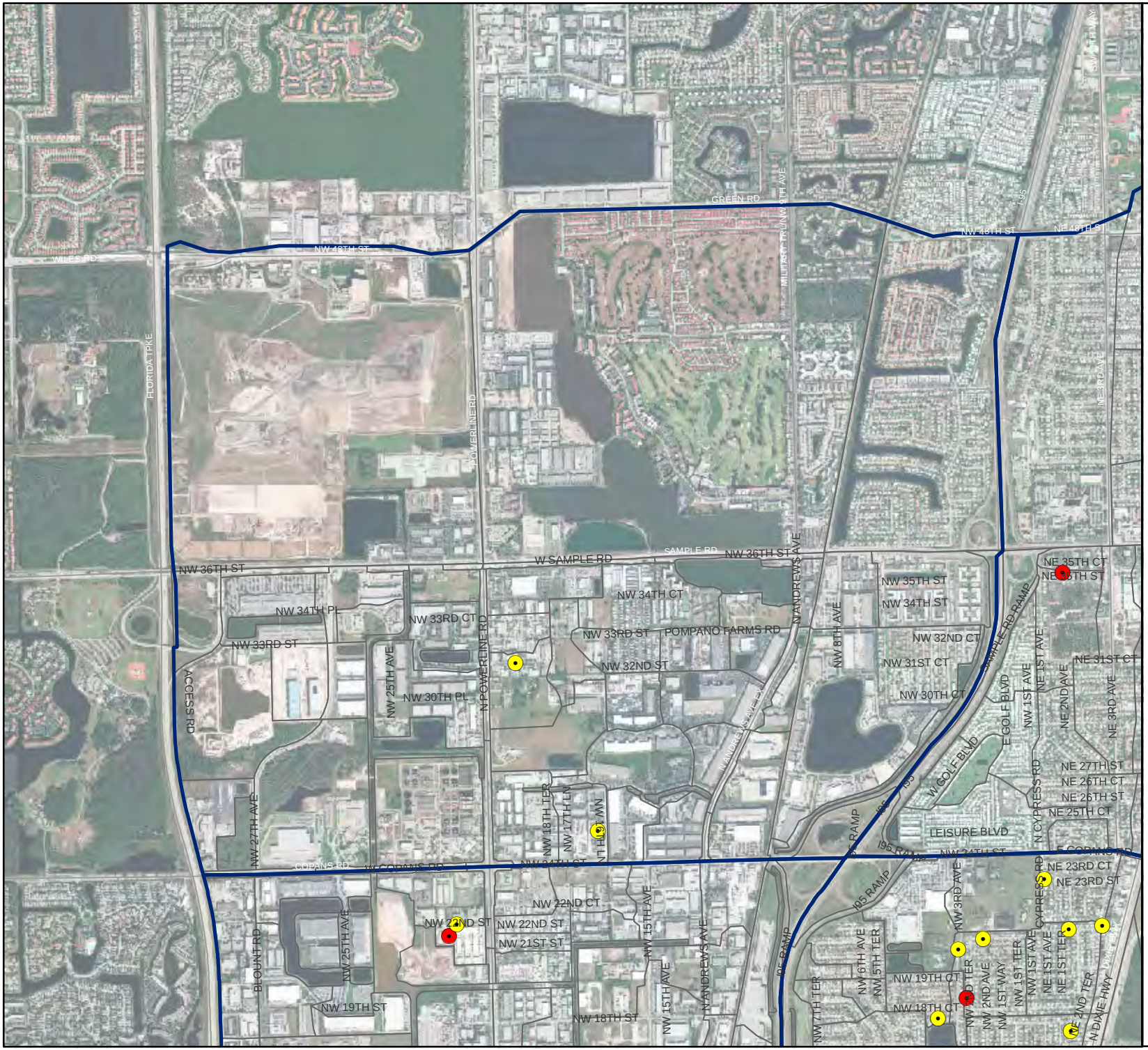


City of Pompano Beach
Stormwater Master Plan

Figure 2-15
Resident Complaints and FEMA Repetitive Losses

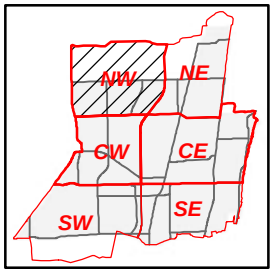
Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins



City of Pompano Beach
Stormwater Master Plan

Figure 2-15A
Resident Complaints and FEMA Repetitive Losses
Basin NW

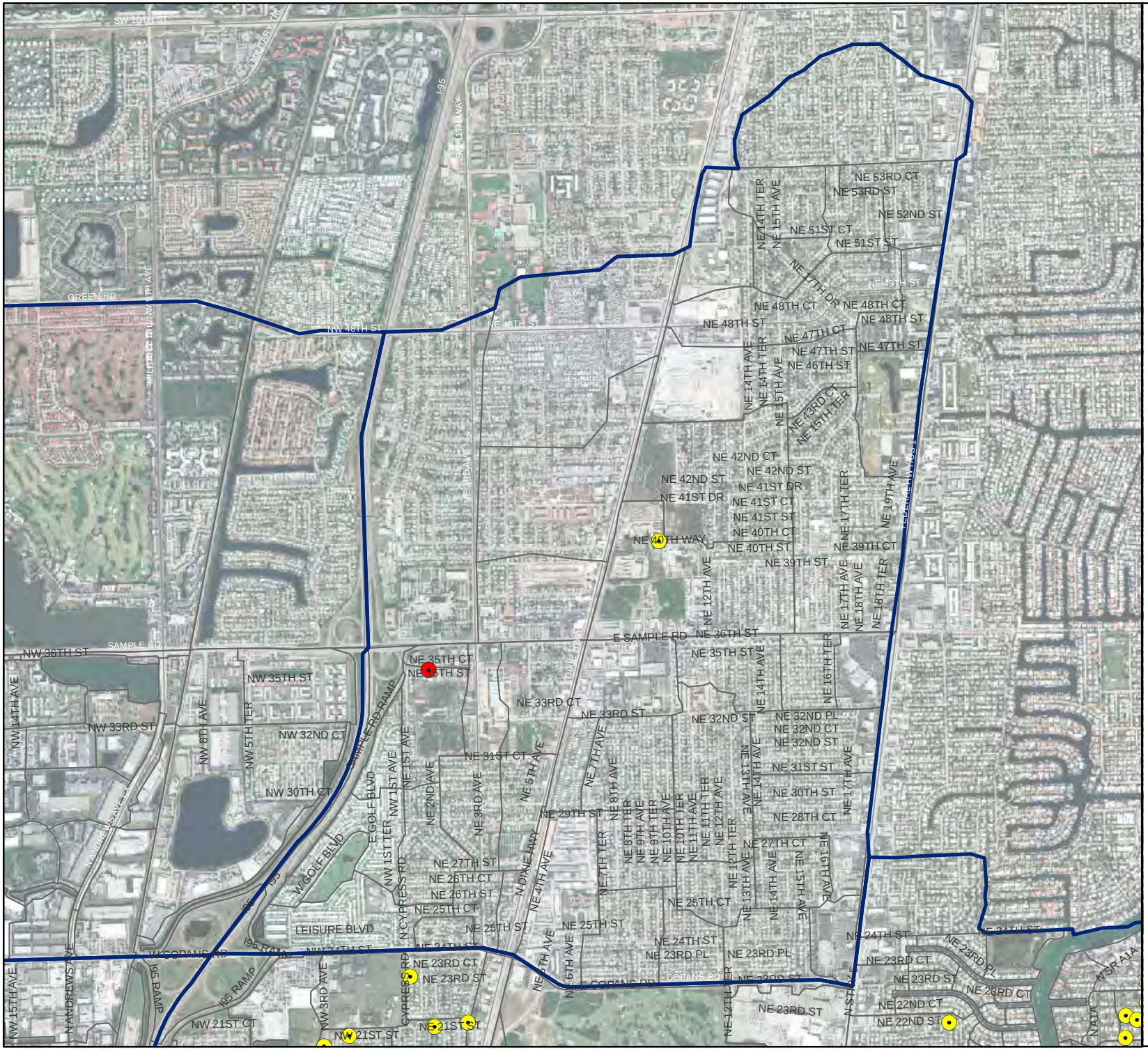


Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins

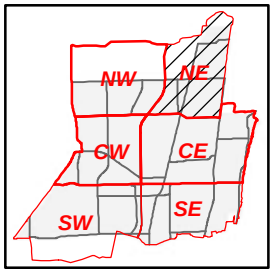
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-15B
Resident Complaints and FEMA Repetitive Losses
Basin NE

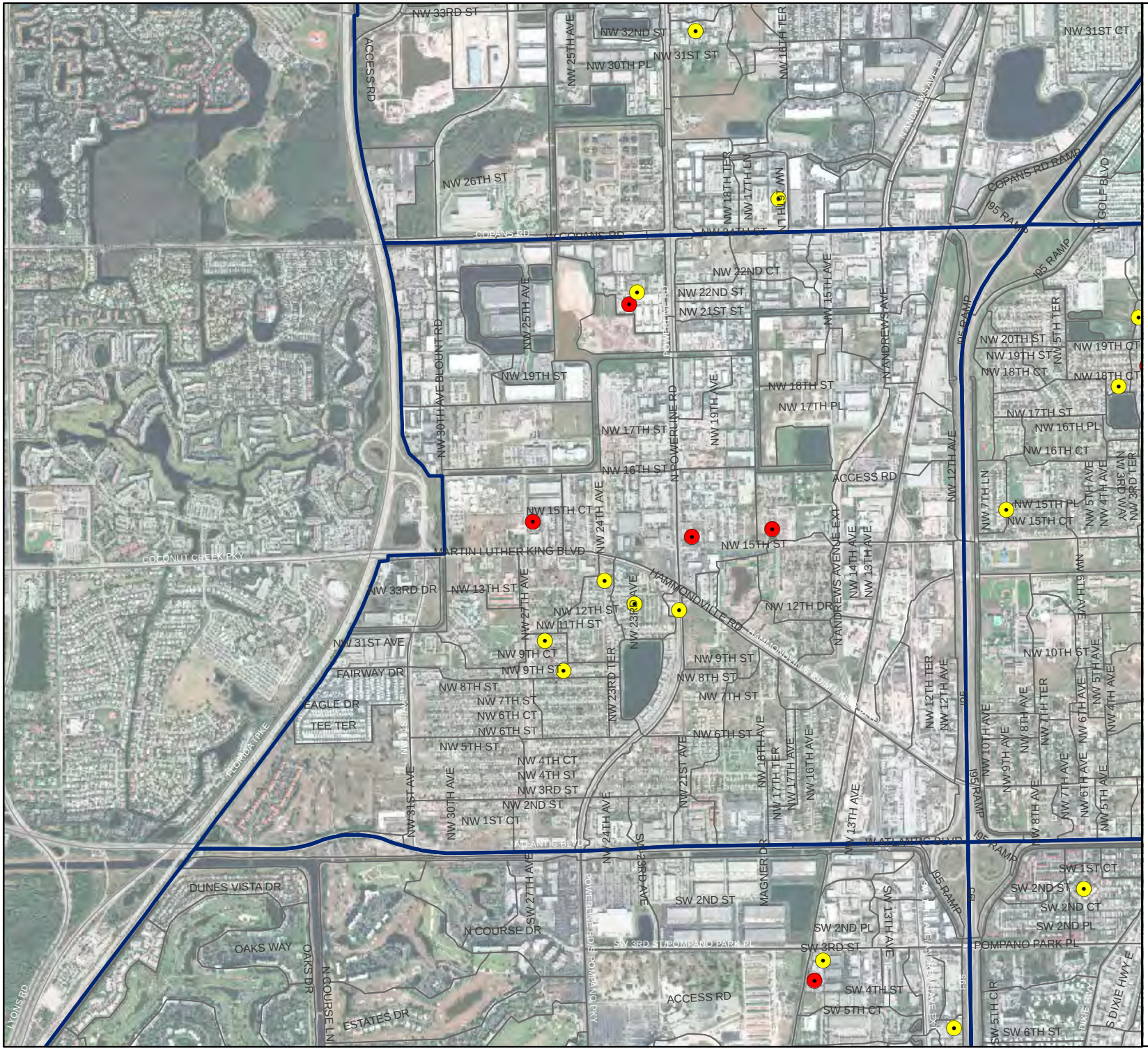


Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins

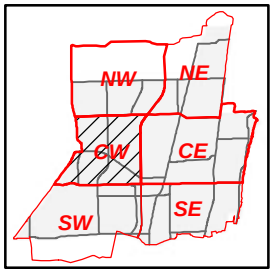
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-15C
Resident Complaints and FEMA Repetitive Losses
Basin CW

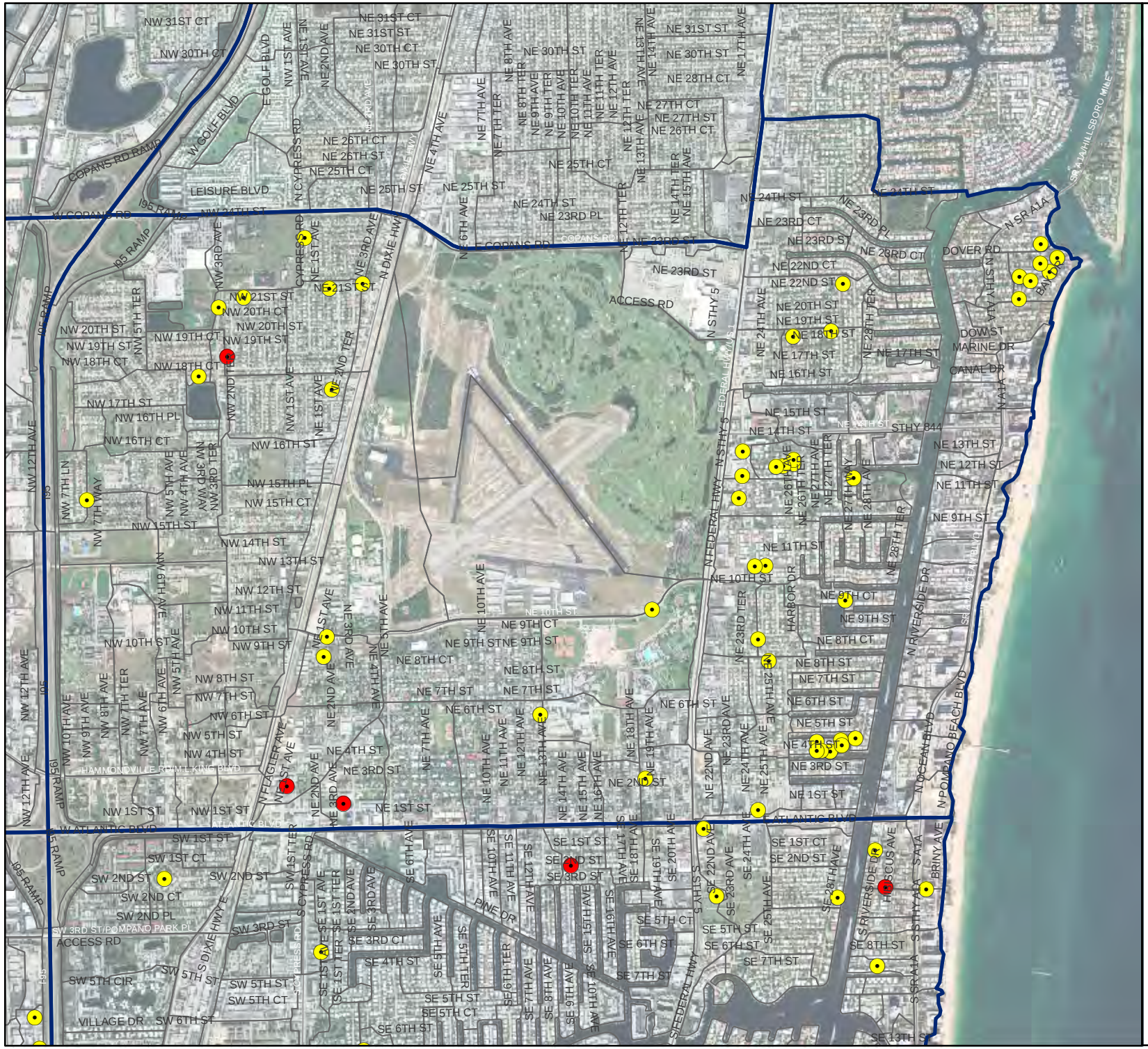


Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins

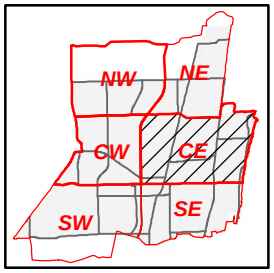
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-15D
Resident Complaints and FEMA Repetitive Losses
Basin CE

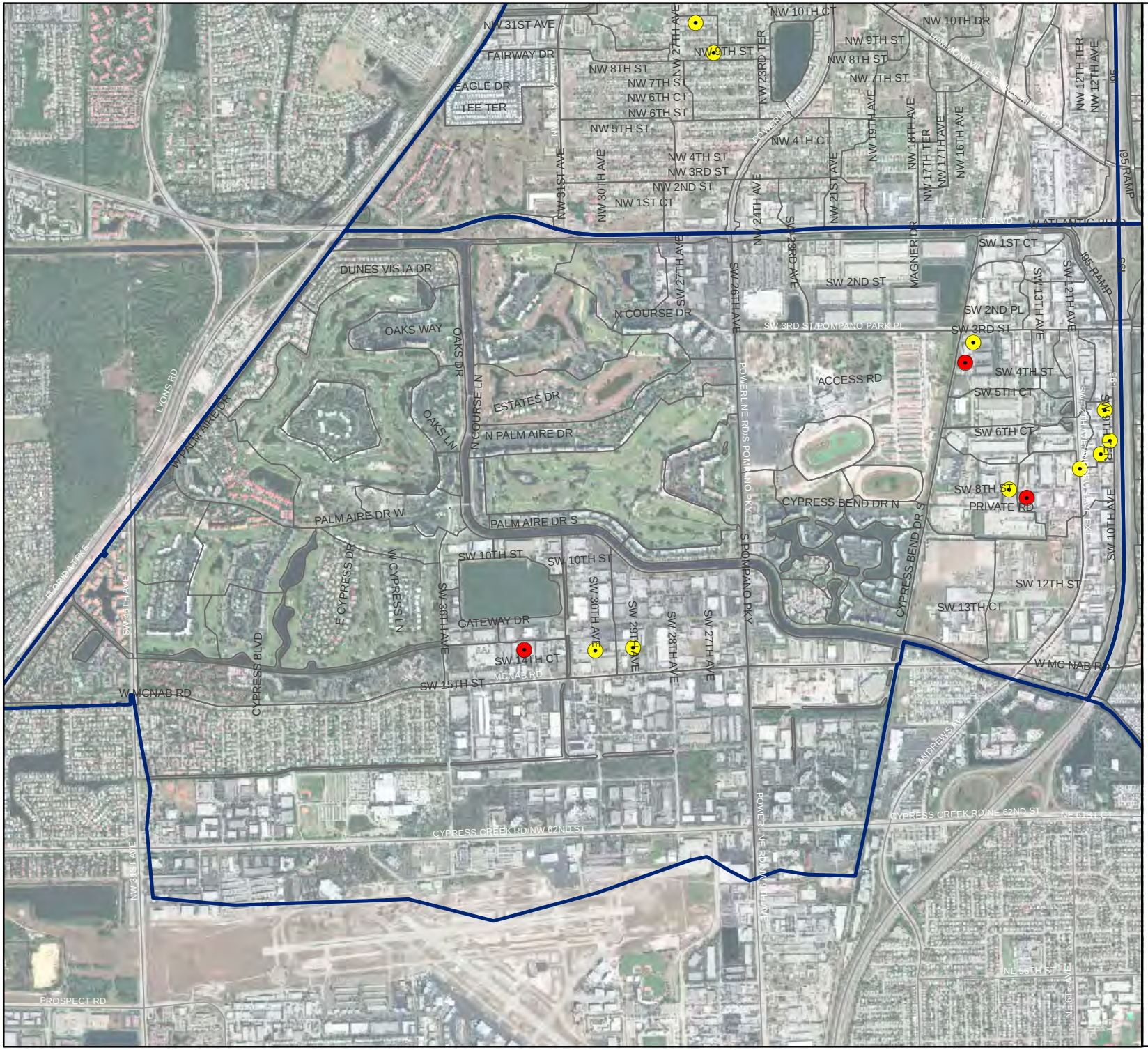


Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins

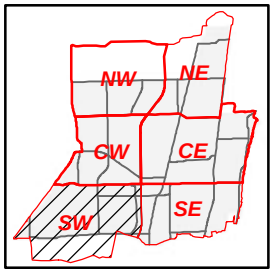
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-15E
Resident Complaints and FEMA Repetitive Losses
Basin SW

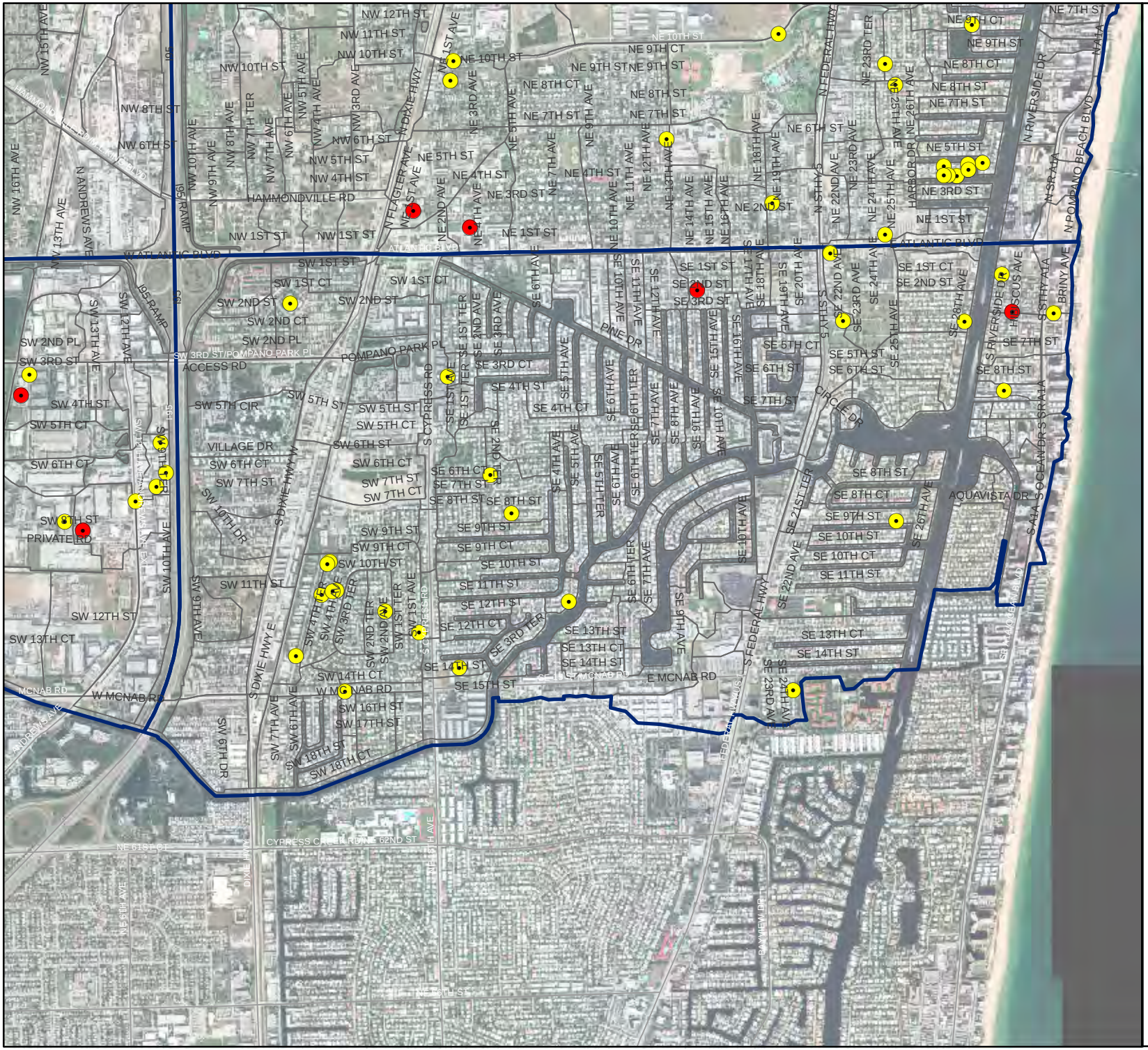


Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins

N
W E
S

0 750 1,500 3,000
Feet



pompano beach

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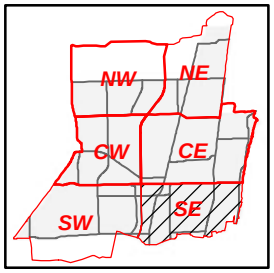
City of Pompano Beach
Stormwater Master Plan

Figure 2-15F
Resident Complaints and FEMA Repetitive Losses
Basin SE

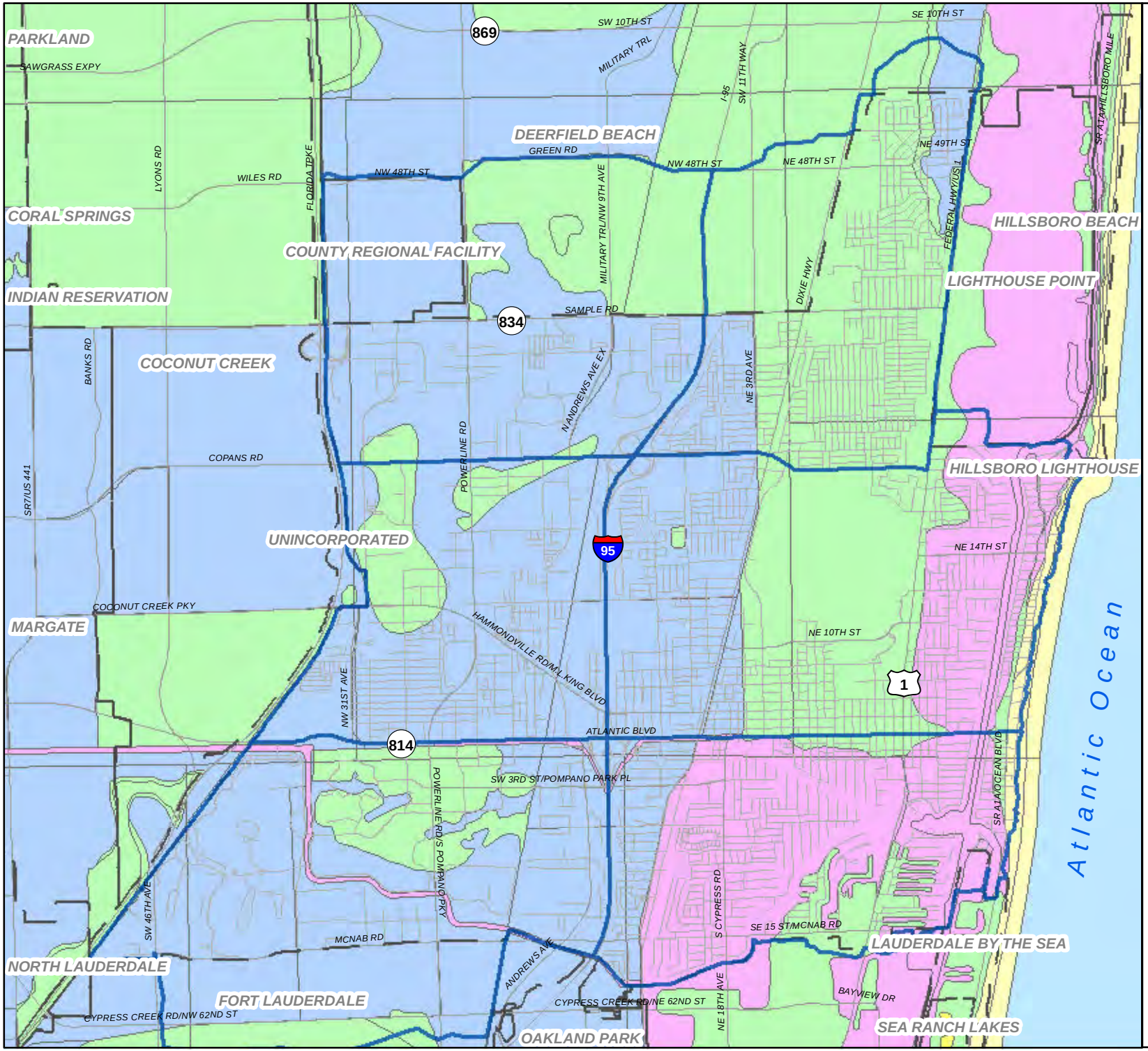


Legend

- Resident Complaints
- FEMA Repetitive Losses
- Major Basin Boundary
- Sub-Basins

N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

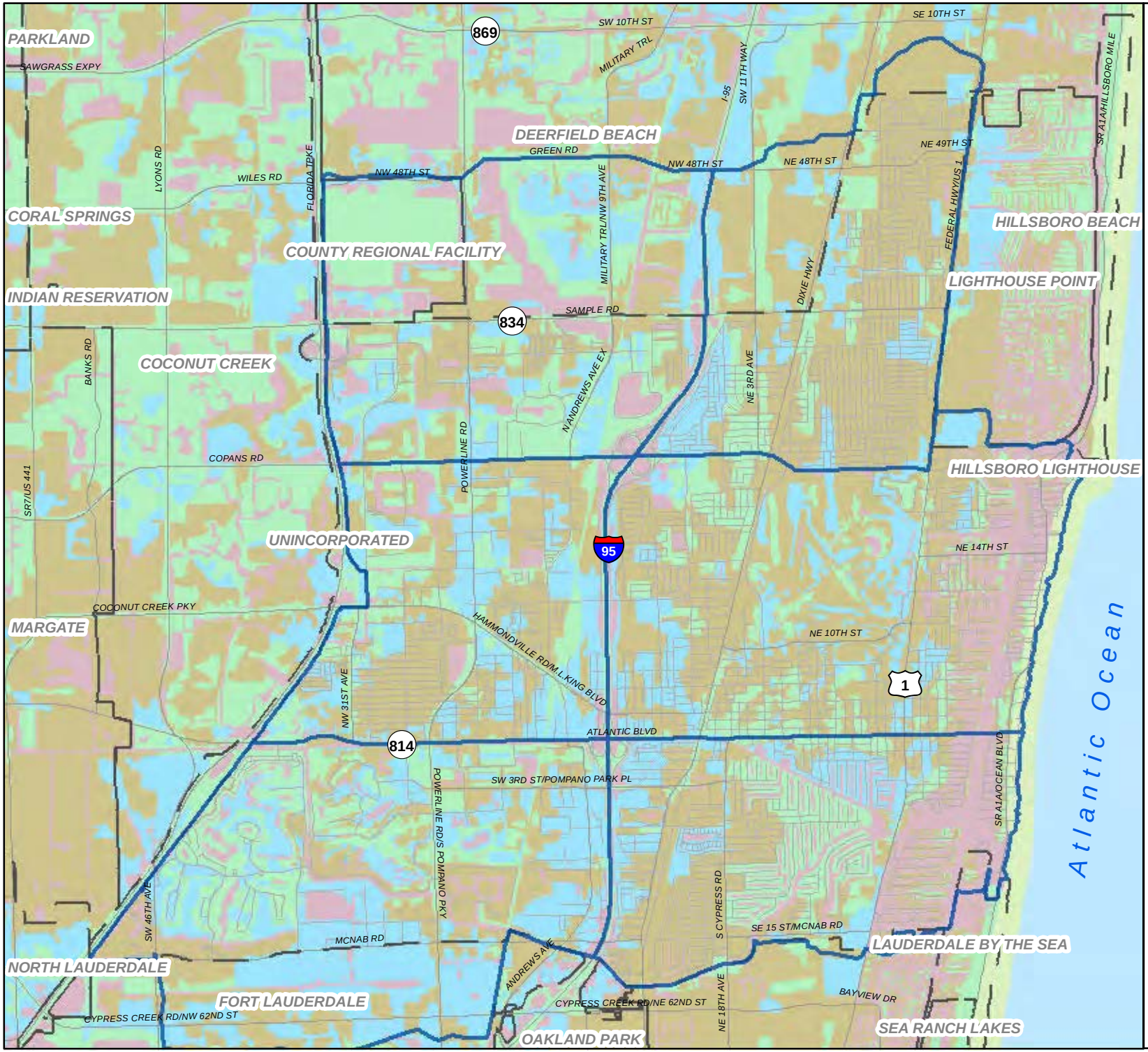
Figure 2-16
FEMA Flood Map
(Existing Standards)

Legend

- Major Basin Boundaries
- City Boundary
- City Streets

FEMAFloodZones

- A
- AE
- AH
- AO
- VE
- X



City of Pompano Beach
Stormwater Master Plan

Figure 2-17
FEMA Flood Map
(2011 Draft Updates)

Legend

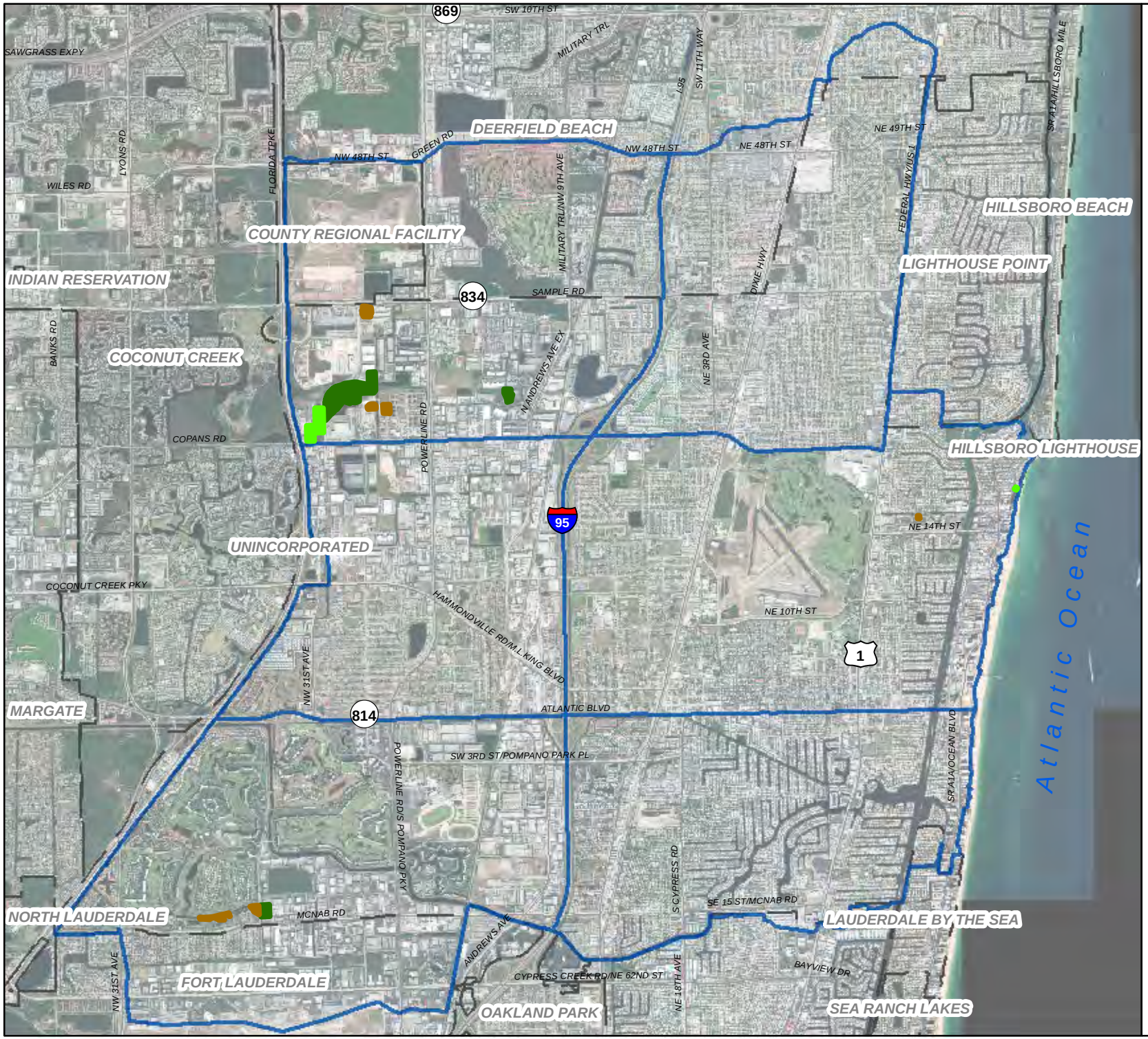
- Major Basin Boundaries
- City Boundary
- City Streets

FLOOD ZONE

- 0.2% ANNUAL CHANCE FLD HAZ
- AE
- AH
- VE
- X

N
W E
S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 2-18
Wetland Inventory

Legend

Wetlands

- Cypress
- Marsh
- Mitigation Completed
- Major Basin Boundaries
- City Boundary

N
W E
S

0 1,500 3,000 6,000
Feet

Figure 2-19
Wetlands Photo Study

0 2,000 4,000 Feet

N W E S

SECTION 3 – SYSTEM REQUIREMENTS

3.1 LEVEL OF SERVICE (LOS) CRITERIA

CMA used a level of service (LOS) criteria for stormwater management systems as defined by the regulatory agencies with jurisdiction within the City of Pompano Beach. CMA reviewed the typical LOS criteria defined by these regulatory agencies. The level of service criteria for the stormwater management system within the City was defined based on the guidelines from SFWMD and Broward County EPGM. The most stringent requirement from these regulatory agencies was incorporated into the modeling analysis for the study area to ensure a conservative approach to evaluating the existing infrastructure. Please note the LOS criteria only apply to new development and new construction since a large portion of the City was developed prior the implementation of stormwater management regulations. The stormwater model was used to verify whether existing drainage systems meet the following LOS requirements:

- Building Structures – The flood level shall not exceed the finish floor elevation of all building structures within the study area during the 100-year, 3-day storm event.

- Roadways and Parking Lots – Stormwater ponding shall not encroach onto any roadway centerlines during the 10-year, 1-day storm event. Stormwater ponding shall not encroach onto any roadway edge of pavement during the 5-year, 1-day storm event.

- Outfall Discharges – The outfall discharge shall not exceed the allowable peak discharge during a 25-year, 3-day rainfall event as defined by the existing permit requirements. If the existing outfalls do not have a maximum discharge assigned by existing permits, the regulatory agencies will require "pre-development" versus "post-development" discharge analysis to ensure the stormwater discharge into adjacent surface waters does not increase after the proposed construction.

In addition to the quantity of stormwater runoff, the regulatory agencies also define the level of water quality treatment provided to stormwater runoff required prior to discharge via outfalls into adjacent surface water bodies. The water quality criteria for the City are based on the SFWMD and Broward County standards. These standards require treatment of the first inch of stormwater runoff generated from the entire site area or 2.5 inches of stormwater runoff generated from all impervious areas within the site area (whichever is greater). Typical methods for providing water quality treatment of stormwater runoff are the installation of dry retention areas, grass swales, and exfiltration trench to adequate storage volume to meet these requirements. The SFWMD provide credits for various Best Management Practices (BMP) types and includes further requirements for various land uses. These issues are fully detailed in SFWMD Environmental Resource Permitting Manual and in Broward County Chapter 27. These water quality requirements are standard among other South Florida municipalities and are consistent with criteria from BCEPGMD.

3.2 REGULATORY COORDINATION

The City's stormwater management system is regulated by various Federal, State, and local regulatory agencies. Any modifications to the existing stormwater management system must be based on the regulatory requirements of these jurisdictional agencies. The various regulatory agencies will jurisdiction over the City's Stormwater Management System are summarized below:

3.2.1 FEDERAL

Environmental Protection Agency (EPA)

The EPA was mandated by Congress through Section 405 of the Water Quality Act of 1987 to manage the National Pollutant Discharge Elimination System (NPDES) Program for stormwater discharges from municipalities. EPA has delegated the NPDES permitting authority to the Florida Department of Environmental Protection (FDEP). As discussed in more detail within Section 3.4 of this report, the City of Pompano Beach is a co-permittee on the NPDES Permit with Broward County, which handles the compliance coordination with FDEP. The updated NDPES permit has been recently issued by FDEP to Broward County. The permit requirements have been factored into the water quality approach in this Stormwater Master Plan. EPA is currently updating the MS4 permit program and the new rule is expected to contain additional requirements for Best Management Practices (BMPs) and documentation on their performance. These potential future needs are included in the recommendations, which include the pending Numeric Nutrient Criteria for all discharges as discussed in further detail in Section 3.3.

Federal Emergency Management Agency (FEMA)

FEMA was established by the Robert T. Stafford Disaster Relief and Emergency Assistance Act (PL 100-707), which was signed into law November 23, 1988 which amended the Disaster Relief Act of 1974 (PL 93-288). This Act provides the Federal Government with the statutory authority for most disaster response activities. FEMA regulates the floodplains and floodways under the National Flood Insurance Program (NFIP). CMA has reviewed the FEMA Flood Insurance Study (FIS) and Flood Insurance Rate Maps (FIRM) for the City of Pompano Beach. These documents establish the flood zone categories throughout the City based on the regional flood hazard modeling by FEMA. CMA used this FEMA information to help identify areas of Pompano Beach are prone to flooding problems. FEMA is currently in the process of adopting new Flood Zone Maps throughout Broward County, which are expected to take effect in late 2012.

United States Geological Survey (USGS)

USGS is a science organization that provides impartial information on the ecosystems, environment, natural hazards, and natural resources within the United States. USGS also provides relevant information on impacts of climate change, which will have a direct impact on the performance of the stormwater management systems. According to the USGS, observational evidence shows that regional climate changes, particularly temperature increases, is impacting water resource issues throughout the United States. CMA has collected relevant climate change information from USGS for use during the sea level rise analysis for this study area.

United States Army Corps of Engineers (USACE)

USACE is the primary federal agency that develops guidance parameters for civil infrastructure design consideration for the potential of sea level rise. In July 2009, USACE published Engineering Circular EC1165-2-211 Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs. This circular provides guidance for incorporating the direct and indirect physical effects of projected future sea- level change in managing, planning, engineering, designing, constructing, operating, and maintaining USACE projects and systems of projects. As indicated in the Circular, potential sea level rises over the next 50 years range from 0.2 to 0.5 ft. CMA has collected relevant sea level rise information from USACE for use during the sea level rise analysis for this study area.

3.2.2 STATE

South Florida Water Management District (SFWMD)

SFWMD is a primary regulatory agency with jurisdiction over stormwater issues in the City of Pompano Beach. Chapter 40E-4 of the Florida Administrative Code (FAC) creates and empowers the SFWMD to regulate all matters related to stormwater management and groundwater withdrawal through the issuance of an Environmental Resource Permit (ERP). Based on these regulations, any future construction project which impacts the stormwater management system in the City will require obtaining a permit from SFWMD. Since SFWMD has jurisdiction within the City of Pompano Beach, their criteria and standards will be used as guidelines for conceptual planning of both water quality and quantity improvements. These guidelines are provided in the South Florida Water Management District ERP Information Manual Volume IV (2010).

Florida Department of Environmental Protection (FDEP)

FDEP regulates environmental programs within the State of Florida and has been delegated as the NPDES MS4 permit authority by EPA. FDEP is responsible for implementing the stormwater element of the Federal NPDES Program throughout the State. The City of Pompano Beach is one of the 29 entities authorized for stormwater discharge under the comprehensive Broward County NPDES MS4 permit (Permit Number FLS000016-003). The City is authorized to discharge to waters of the state per the approved Stormwater Management Programs, effluent limitations, monitoring requirements, and other provisions as set forth in this permit. The City has actively been fulfilling the requirements of the permit related to its existing outfalls. These efforts are documented in annual reports submitted by the City to the FDEP.

Florida Department of Transportation (FDOT)

FDOT owns and maintains several roadways within the City of Pompano Beach, including Powerline Road, I-95, Dixie Highway, US-1, US-A1A, Atlantic Boulevard, Sample Road, and NE 14th Street. As many of the stormwater flooding issues in the City are involved with runoff to and from FDOT right-of-way, coordination of stormwater system improvements with FDOT is critical. This coordination includes identifying interconnectivity of the proposed improvements from FDOT and the City's existing stormwater system. CMA has obtained relevant information from FDOT on their drainage infrastructure along FDOT roadways within the City limits.

3.2.3 LOCAL

Broward County Environmental Protection and Growth Management Department (BCEPGM)

Broward County EPGM is a primary regulatory agency with jurisdiction over stormwater issues in the City of Pompano Beach. Chapter 27, Article V of the Broward County Code of Ordinances empowers the BCEPGMD to regulate all matters related to stormwater management. Based on these regulations, any future construction project which impacts the stormwater management system in the City will require obtaining a permit from BCEPGM. These stormwater regulations are adequate for requiring the appropriate level of flood protection and water quality treatment to be provided by the stormwater management system.

3.3 PENDING REGULATION

CMA is currently tracking potential changes to stormwater regulations which could impact the implementation of the Stormwater Master Plan. All water quality treatment components, such as exfiltration trench, retention areas, or swale areas, were sized based on the current stormwater regulations. Please note the "New Statewide Stormwater Rule" is currently under consideration by the FDEP and SFWMD in conjunction with the EPA Numeric Nutrient Criteria and EPA NPDES Permits. These pending regulations could impact the configuration of the proposed stormwater improvements within the City of Pompano Beach once implemented. If these regulations are implemented, the proposed stormwater improvements which are connected to a positive outfall to a surface water body will need to be reassessed during the design phase to verify adequate water quality components are included. Some of the potential future changes to stormwater regulations include the following items:

3.3.1 FDEP TMDL PROGRAM

As defined by FDEP, a Total Maximum Daily Load (TMDL) is a scientific determination of the maximum amount of a given pollutant that a surface water can absorb and still meet the water quality standards that protect human health and aquatic life. If a surface water body has higher levels of pollutants than the TMDL specifies, FDEP classifies the surface water body as impaired. FDEP has implemented the TMDL Program to satisfy the Clean Water Act. The goal of the TMDL is to work with local stakeholders to identify the impaired waters and create a management plan to improve the water quality. Broward County is responsible for enforcement of the TMDL Program through the NPDES Permit. Although the TMDL designations has not been finalized by FDEP yet, the draft TMDL designations currently in place have been considered during the development of this Stormwater Master Plan.

3.3.2 EPA NUMERIC NUTRIENT CRITERIA

As a result of a lawsuit on behalf of environmental special interests, EPA is attempting to set specific numeric (as opposed to narrative) criteria for nutrient levels in water bodies, such as lakes, streams, canals, estuaries, and marine areas. The proposed levels under discussion could cause many water bodies to fall into an "impaired" classification. This will require that all the areas that contribute to these water bodies will need to reduce their nutrient discharge via stormwater outfalls. The Numeric Nutrient Criteria for lakes and streams within the north and central portions of Florida have been defined on the Federal Registry as of December 2010. The draft Numeric Criteria Standard for South Florida water bodies was published within the Federal Register on December 18, 2012 under the title of "Water Quality Standards for the State of Florida's Estuaries, Coastal Waters, and South Florida Inland Flowing Waters". Interested parties have 60 days from the publication date to submit comments to EPA on the proposed rule. Under the current schedule, the Numeric Criteria Standard for South Florida water bodies could become a rule within the next few years. Depending on the outcome of various legal actions, the FDEP will likely be required to adopt the EPA Numeric Criteria Standard at that time. Some key points of the draft Numeric Nutrient Criteria rule are summarized below:

- Proposed Numeric Nutrient Criteria for Estuaries – The EPA has proposed that the State of Florida's Numeric Nutrient Criteria from the northernmost segment of Biscayne Bay apply to the waters of the Intracoastal Waterway between Biscayne Bay and the Lake Worth Lagoon estuary system. Under the proposed rule, the Numeric Nutrient Criteria for the Intracoastal Waterway would be 0.012 mg/L for total phosphorus (TP), 0.30 mg/L for total nitrogen (TN), and 1.7 ug/L for chlorophyll-A. This draft Numeric Nutrient Criteria could result in additional waterways being designated as impaired once the proposed rule is adopted.
- Proposed Numeric Nutrient Criteria for Coastal Waters – The EPA has proposed a Numeric Nutrient Criteria for chlorophyll-A for Class III waters in coastal regions of Florida. The

proposed Numeric Nutrient Criteria for these coastal waters in Pompano Beach would 0.20 mg/m^3 for chlorophyll-A.

- Proposed Numeric Nutrient Criteria for South Florida Inland Flowing Waters – The EPA has proposed a Numeric Nutrient Criteria for inland flowing waters within South Florida, which would consist solely of the South Florida marine water downstream protection values (DPV).
- Alternative Proposed Numeric Nutrient Criteria for South Florida Inland Flowing Waters – The EPA has proposed an alternative approach for the Numeric Nutrient Criteria in South Florida inland flowing waters. As an alternative to the proposed DPV-only approach for South Florida inland flowing waters, EPA has developed protective instream criteria for TN and TP in Class I and III flowing waters, which includes canals and streams. The EPA's Alternative Numeric Nutrient Criteria for South Florida inland flowing waters would be 2 mg/L for TN and 0.052 mg/L for TP.
- Implementation Schedule – The newly approved WQS include a schedule for future rulemaking whereby FDEP will develop Numeric Nutrient Criteria for additional estuaries by June 30, 2013 and again by June 30, 2015. If FDEP is on schedule toward adoption of protective and approvable standards for their additional estuaries, the EPA may consider delaying the effective date of its final rule to after June 30, 2015 to allow time for FDEP to finalize their proposed rules and for EPA to review the State's Numeric Nutrient Criteria.

3.3.3 FDEP AND SFWMD UNIFIED STATEWIDE STORMWATER RULE

As a result of a lawsuit in the Southwest Florida Water Management District (SWFWMD), the FDEP is working on a rule to define consistent water quality criteria for Environmental Resource Permits throughout the State of Florida. This rule would regulate the discharge of nutrients to pre-development levels, provide credit for BMPs and encourage stormwater reuse. This New State Stormwater Rule was originally set to be released in the Summer of 2009, but has been put on hold pending the Numeric Nutrient Criteria rulemaking. The new Unified Statewide Stormwater Rule is pending the implementation of the Numeric Nutrient Criteria by the EPA according to the tentative schedule noted above.

3.3.4 NATIONAL POLLUTION DISCHARGE ELIMINATION SYSTEM (NPDES) PERMIT CHANGES

Pending the outcome of the EPA Numeric Nutrient Criteria, EPA is now planning to assign further requirements upon drainage outfalls and potentially apply the proposed EPA Numeric Nutrient Criteria. This modification to the NPDES process is expected to be issued in December 2012, and would immediately impact NPDES permit-holders upon adoption.

3.4 NATIONAL POLLUTION DISCHARGE ELIMINATION SYSTEM (NPDES)

The NPDES Program is designed to reduce the discharge of pollutants into water bodies. The City of Pompano Beach participates in the FDEP NPDES Program as a co-permittee with Broward County, who is responsible for compliance with the permit requirements. The requirements of the NDPES Program have been adopted by the City of Pompano Beach. The impacts of the NDPES Permit requirements on the City of Pompano Beach are summarized within the following sections.

3.4.1 PERMIT REQUIREMENTS

The NPDES Permit requires that the City implement a Stormwater Management Program. The requirements of this Stormwater Management Program are summarized below. A new requirement to the NPDES Permit involves a specific inspection and maintenance schedule for the structural controls and roadways within the City, which been outlined within the permit documentation.

<u>Structural Controls and Stormwater Collection Systems Operation</u>

- Provide inventory of structural controls, stormwater collection structures, and outfalls.
- Review Standard Operating Procedures (SOPs).

Areas of New Development and Significant Redevelopment

- Adhere to Comprehensive Plan.
- Maintain documentation of new development and significant redevelopment.
- Review current local codes and land development regulations for possible changes to reduce impact of development on stormwater.

Roadway

- Review litter control program.
- Review street sweeping program.
- Review standard practices to reduce the pollutants in stormwater runoff from areas associated with road repair and maintenance, and from permittee-owned or operated equipment yards and maintenance shops that support road maintenance activities.

Flood Control Projects

- Provide stormwater treatment for all flood control projects.
- Evaluate existing structural control devices to determine if retrofitting could remove more pollutants.

Municipal Waste Treatment, Storage, or Disposal Facilities Not Covered By NPDES Stormwater Permit

- Review procedures for inspections and the implementation of measures to control discharges from facilities that are not otherwise covered by an NPDES stormwater permit.

Pesticides, Herbicides, and Fertilizer Application

- Require proper certification and licensing by the Florida Department of Agriculture and Consumer Services (FDACS) for all pesticide or herbicide applicators.
- Adopt the FDEP's Model Ordinance for Florida-Friendly Fertilizer Use on Urban Landscapes or adopt a Florida-friendly Landscaping Ordinance similar to the one set forth in the document "Florida-friendly Guidance Models for Ordinances, Covenants and Restrictions."
- Implement a public education and outreach program plan to encourage citizens to reduce their use of pesticides, herbicides, and fertilizers.

Illicit Discharges and Improper Disposal

- Inspections, Ordinances, and Enforcement Measures: Strengthen the legal authority to conduct inspections, conduct monitoring, control illicit discharges, illicit connections, illegal dumping and spills into the MS4.
- Inspection and Investigation of Suspected Illicit Discharges and / or Improper Disposal: Implement a written proactive inspection program plan for identifying and eliminating sources of illicit discharges, illicit connections, or dumping to the MS4. Implement a written plan for the training of all appropriate permittee personnel and contractors to identify and report conditions in the stormwater facilities that may indicate the presence of illicit discharges / connections / dumping to the MS4.
- Spill Prevention and Response: Review the spill-prevention / spill-response plan and procedures to prevent, contain, and respond to spills that discharge into the MS4. Train of all appropriate permittee and contractors on proper spill prevention, containment, and response techniques and procedures.
- Public Reporting: Implement a written public education and outreach program plan to promote, publicize, and facilitate public reporting of the presence of illicit discharges and improper disposal of materials into the MS4.
- Oils, Toxics, and Household Hazardous Waste Control: Implement a written public education and outreach program plan to encourage the proper use and disposal of used motor vehicle fluids, leftover hazardous household products, and lead acid batteries.
- Limitation of Sanitary Sewer Seepage: Review the procedures to reduce or eliminate sanitary wastewater contamination into the MS4, including discharges to the MS4 from sanitary sewer overflows (SSOs) and from inflow / infiltration from collection / transmission systems and / or septic tank systems.

Industrial and High Risk Runoff

- Identification of Priorities and Procedures for Inspections: Maintain an up-to-date inventory of all existing high risk facilities discharging into the permittee's MS4. Implement a plan for conducting inspections of high risk facilities to determine compliance with all appropriate aspects of the stormwater program.
- Monitoring for High Risk Industries: Sample the discharge to the stormwater system on an as-needed basis in the event that inspections of high-risk facilities disclose suspected illicit discharges to the MS4.

Construction Site Runoff

- Site Planning and Non-Structural & Structural Best Management Practices: Implement the local codes or land development regulations and the written pre-construction site plan review procedures that require the use and maintenance of appropriate structural and non-structural erosion and sedimentation controls during construction to reduce the discharge of pollutants to the MS4. Review the procedures to notify all new development / redevelopment permit applicants of the need to obtain all required stormwater permits.
- Inspection and Enforcement: Submit a written plan that details the standard operating procedures for implementation of the stormwater, erosion and sedimentation inspection program for construction sites discharging stormwater to the MS4.
- Site Operator Training: implement a written plan for stormwater training / outreach for construction site plan reviewers, site inspectors and site operators.

Monitoring Requirements

- Annual Loadings and Event Mean Concentrations: Provide estimates of the annual pollutant loads and event mean concentrations for each major outfall or major watershed. These estimates shall be provided in the Annual Report for Year 3 of the permit. Compare calculated annual pollutant loadings with the estimates from the previous two permit cycles to determine if the pollutant loadings are increasing or decreasing within these watersheds. If these levels are not decreasing, the Stormwater Management Plan needs to be re-evaluated and revisions shall be submitted in the Annual Report for Year 4 of the NPDES Permit Cycle. The annual pollutant loads and event mean concentrations shall be tabulated for the following constituents:
  a) Biochemical Oxygen Demand (BOD5) (mg/L)
  b) Total Copper (mg/L)
  c) Total Nitrogen (as N) (mg/L)
  d) Total Phosphorus (mg/L)
  e) Total Suspended Solids (TSS) (mg/L)
  f) Total Zinc (mg/L)
- Monitoring Program: The existing monitoring plans that was approved by FDEP on September 18, 2003 under the previous permit cycle shall continue to be implemented by Broward County.

Additional requirements defined within the NPDES Permit for areas that discharge into an impaired waterbody with a designated TMDL, such as the Old Pompano Canal. These additional requirements are discussed in more detail with the Pompano Canal TMDL section of this report.

3.4.2 CITY PARTICIPATION

The City of Pompano Beach has entered into an interlocal agreement with Broward County on July 10, 2012, which defines the City as a "co-permittee" and Broward County as the "lead-permittee" of the NPDES Permit. According to the interlocal agreement, the City implements the various activities in order to meet the NPDES permit requirements while Broward County is responsible for handling compliance with FDEP. There have been several past cycles of the NPDES Permit, which each of which lasts 5 years. NPDES Permit Cycle 3 (Permit #FLS000016-003) was issued by FDEP on September 2, 2011 and is scheduled to expire on September 1, 2016. While these regulations do not directly affect the City's existing NPDES MS4 Permit, the permit requirements did affect industrial activities permits, including

construction activities. The primary change under Permit Cycle 3 was that the threshold for requiring an NPDES Construction Permit dropped from construction sites greater than five acres of disturbed land (or part of a larger common plan activity that will disturb five acres) to construction sites greater than one acre of disturbed land. The balance of the permit conditions and requirements has remained very similar to the original permit.

The City of Pompano Beach is currently in the first year of a NPDES Permit Cycle 3 with Broward County. This permit cycle requires an annual report each year that includes additional information, which includes an outfall inventory, a summary of water quality monitoring program, fiscal analysis, quantifiable stormwater management plan activities, non-quantifiable stormwater management plan activities, and the incorporation of adopted TMDLs. The City has set up a Stormwater Management Utility to address the needs of the community in terms of water quantity and water quality. The following goals have been identified by Stormwater Management Utility:

- Compliance with the NPDES Permit Application Regulations for Stormwater Discharge.
- Coordination with SFWMD, FDOT, Broward County Drainage Districts, and the private industry addressing issues associated with improving water quality in the Old Pompano Canal.
- Coordination with local Water Management Agencies and FDEP addressing issues associated with the new upcoming Nutrient Criteria Rule.
- Receive satisfactory results from NPDES Annual Report
- Coordination with internal City departments along with neighboring municipalities.
- Cross training among all Utility Department staff.
- Implement effective utility management suggestions

FDEP intends to conduct annual site visits/reviews with NPDES MS4 permittees to assess permit activities, compliance, and reporting. These meetings will provide a forum for discussing permit elements that the City feels should be modified in order to improve its stormwater program. As the City proceeds towards the next permit renewal, it should consider that EPA intends to assign further requirements upon drainage outfalls and potentially apply the proposed EPA numeric nutrient criteria to surface waters. If implemented on the schedule defined in the previous section, this requirement could place additional restrictions on stormwater discharges from the City's stormwater management system into surface water bodies. The City will need to follow the changes to the numeric nutrient criteria as it proceeds toward adoption at the Federal and State level. Prior to the next permit renewal, the City should consider adding or expanding the Best Management Practices (BMP) program since they may be become requirements if the numeric nutrient criteria is instituted by FDEP. Some potential BMP programs are further defined in the following section.

3.4.3 OLD POMPANO CANAL TMDL

The Watershed Restoration Act of 1999 (s. 403.067, F. S.) directs the Department of Environmental Protection to scientifically evaluate the quality of Florida's surface waters and promote the mechanisms necessary to clean up pollution. The Act was created specifically to implement the Federal Total Maximum Daily Load (TMDL) Program, which is a systematic approach to establishing how much pollution water bodies can assimilate while still meeting water quality standards. The Act directs the FDEP to report to the Governor and Legislature after five years on the implementation of the TMDL Program and recommend statutory changes necessary to improve it.

On April 16, 2001 Florida's Environmental Regulation Commission approved Chapter 62-303, F.A.C., Identification of Impaired Surface Waters, which establishes the methods by which surface waters are evaluated and the need for TMDLs is determined. The rule provides for developing a planning list to include those waters that may not be meeting water quality standards but for which sufficient scientific

data are not available to judge and a verified list to include waters determined based on sufficient, reliable data to be failing water quality standards because of specific pollutants from human sources. The verified lists are publicly adopted, subject to administrative challenge, for each of the 29 basins into which FDEP has divided the State as part of its watershed management cycle. TMDLs must be developed and adopted for each instance of impairment identified on the verified lists. FDEP must submit the verified lists to EPA for approval. Each water body on the planning list is further monitored to establish whether it truly is impaired. The fundamental distinction between the planning and verified lists is the availability of reliable water quality data to determine impairment. Where data are absent or unreliable, FDEP cannot, by law, list the water as verifiably impaired and must secure additional data.

For the purposes of this study, a review of the Water Body Identification (WBID) extents was conducted for the study area. The Old Pompano Canal (WBID 3271) has been designated as impaired for nutrients and chlorophyll-a. A TMDL addressing the impairments was finalized by FDEP in 2007. Through its wasteload allocation, the TMDL calls for 15.8% reductions in total nitrogen (TN) and 13.6% reductions in total phosphorus (TP) loads within the Old Pompano Canal. These load reductions for the Old Pompano Canal were assigned to the Phase I MS4 permit (# FLS 000016-003). The City has been cooperating with the FDEP for many years and is currently in the process of trying to get WBID 3271 delisted based on new water quality monitoring data after modifications to upstream water control structure, which has increased the average flow to be closer to historical flows.

3.5 FEMA COMMUNITY RATING SYSTEM (CRS)

Management of floodplains has been critical relative to the protection of the health, safety, and welfare of our state and nation's residents. Construction in floodplains and loss of historical floodplain storage are major contributors to flood damage to homes and other personal property. The floodplain also provides benefits relative to maintaining water quality and environmental integrity. From a regulatory standpoint, floodplain management is addressed all the way from the Federal to the local level. The National Flood Insurance Program (NFIP) was established via Congress' passage of the National Flood Insurance Act of 1968. Prior to its passage, the approach to dealing with flood disasters generally involved major flood control projects and post disaster relief to victims. This approach proved to be fairly unsuccessful, resulting in large numbers of flood losses and a tremendous cost to taxpayers. The establishment of the NFIP helped solve this problem by creating an approach to reduce flood damage through local floodplain management ordinances, as well as an insurance premium based method to pay claims for flood losses (rather than burdening the general taxpayer). Flood insurance is required for any construction taking place in a FEMA designated Special Flood Hazard Area (SFHA). These areas are identified and mapped by FEMA. Community participation in the NFIP is voluntary, and by participating, the community enables residents to obtain flood insurance (which is extremely important).

3.5.1 CITY PARTICIPATION

Implemented in 1990, the NFIP's Community Rating System (CRS) is a program which rewards communities for floodplain management programs which exceed minimum standards. These rewards come in the form of adjusted (lower) flood insurance premiums, reflective of the reduced risk of flood damage for communities participating in activities consistent with CRS goals. Credit points are available in 18 different activities which will result in CRS Class and insurance premium reductions. The City of Pompano Beach entered the CRS Program in October 1, 1993. As of July 7, 2012, the City of Pompano Beach has been issued 1,519 credit points under the CRS Program, which results in Class 7 with 15% discounts for SFHA and 5% discounts for non-SFHA. As of May 1, 2011, there were 227 municipalities within Florida receiving premium discounts through the CRS Program. In order to help identify program activities with the potential for a future point increase, the 1,519 credit points earned by the City of Pompano Beach is compared with the average for other municipalities within the State of Florida and nationally in Table 3.1 below. As displayed within the table, the City of Pompano Beach has obtained

more credit points than the average credit points obtained by other participating municipalities both statewide and nationwide. The credit points obtained the City of Pompano Beach is below the Florida average for the following activities: 310 (Elevation Certificates), 420 (Open Space Preservation), 430 (Higher Regulatory Standards), 440 (Flood Data Maintenance), and 450 (Stormwater Management).

Table 3.1 – CRS Credit Points Comparison

Activity Number	Activity Description	Maximum	City of Pompano Beach	Florida Average	National Average
310	Elevation Certificates	162	48	61	69
320	Map Information Services	140	140	134	138
330	Outreach Projects	380	191	87	91
340	Hazard Disclosure	81	10	2	17
350	Flood Protection Information	102	86	29	30
360	Flood Protection Assistance	71	66	24	53
410	Additional Flood Data	1,346	11	7	87
420	Open Space Preservation	900	55	204	194
430	Higher Regulatory Standards	2,740	218	232	235
440	Flood Data Maintenance	239	68	70	102
450	Stormwater Management	670	119	151	122
510	Floodplain Management Planning	359	112	39	120
520	Acquisition and Relocation	3,200	0	6	200
530	Flood Protection	2,800	0	2	97
540	Drainage System Maintenance	330	324	239	220
610	Flood Warning	255	0	57	95
620	Levee Safety	900	0	0	135
630	Dam Safety	175	71	67	69
	Total Credit Points:	14,850	1,519	1,411	1,310

3.5.2 City Activities

The City of Pompano Beach has obtained credit points under various activities defined under the CRS Program. The credit points obtained by the City during the last verification visit on July 7, 2011 are summarized for each activity below:

- Activity 310 – Elevation Certificates (48 points): The Building Department maintains elevation certificates for new and substantially improved buildings, which are available upon request.
- Activity 320 – Map Information Service (140 points): The City provides flood zone information from the current FIRM upon request along with publicizing the service on an annual basis and maintaining current records.
- Activity 330 – Outreach Projects (191 points): The City distributes a community brochure to all properties located within the City on an annual basis. An outreach brochure is also distributed to all properties located in Special Flood Hazard Areas (SFHA) on an annual basis.
- Activity 340 – Hazard Disclosure (10 points): State and local regulations require disclosure of flood hazards.
- Activity 350 – Flood Protection Information (86 points): Floodplain management documentation is made available to the public at the Broward County Library and on the City's website.
- Activity 360 – Flood Protection Assistance (66 points): The City provides technical assistance on flood protection issues to the public along with publicizing this service.

- Activity 410 – Additional Flood Data (11 points): The City has a cooperating technical partnership agreement with FEMA.
- Activity 420 – Open Space Preservation (55 points): The City preserves approximately 748 acres within SFHA as open spaces.
- Activity 430 – Higher Regulatory Standards (218 points): The City enforces regulations that require other higher regulatory standards and state mandated regulatory standards. The City has adopted and implemented the Florida Building Code.
- Activity 440 – Flood Data Maintenance (68 points): The City maintains and uses digital maps in GIS format for the management of the floodplain.
- Activity 450 – Stormwater Management (119 points): The City enforces regulations for stormwater management, freeboard in non-SFHA zones, erosion and soil control, and water quality.
- Activity 510 – Floodplain Management Planning (112 points): The City has 12 repetitive loss properties and is defined as Category C community for CRS purposes. The City has adopted and implemented a Floodplain Management Plan.
- Activity 540 – Drainage System Maintenance (324 points): The Public Works Department inspects the City's drainage system regularly throughout the year and conducts maintenance on an as-needed basis. The City maintains records on all inspections and required maintenance on the City's drainage system. The City has a Capital Improvement Program for the City's drainage system. The City enforces a regulation which prohibits dumping into the City's drainage system.
- Activity 630 – Dam Safety (71 points): All Florida communities receive CRS credit points for the State dam safety program.

The FEMA CRS Program issues credit points for stormwater management regulations, stormwater planning, and water quality management under Activity 450 – Stormwater Management as defined within the CRS Coordinator's Manual. The objective of CRS Activity 450 is to promote flood damage reduction and water quality protection within the City. This Stormwater Master Plan summarizes the City's stormwater management program, which will submitted for review by FEMA for credit under Series 450 – Stormwater Management of the CRS Program. Although the credit documentation for various CRS Activities is included throughout this report, a summary for each item under Activity 450 – Stormwater Management is listed below:

Stormwater Management Regulation (SMR)

The City of Pompano Beach has adopted various ordinances which regulate surface water runoff. These stormwater management regulations are defined within the City of Pompano Beach Code of Ordinances under Chapter 53 – Stormwater Management, Chapter 100 – Streets and Highways, Chapter 152 – Buildings, and Chapter 157 – Development of Land. These stormwater management regulations are further defined in City of Pompano Beach Comprehensive Plan – Drainage Utilities Sub Element, which was adopted by the City on January 26, 2010. For specific information on the City's adopted level of service criteria for stormwater management systems, please refer to Section 3.1 – Level of Service Criteria of this report.

Size of Development (SZ)

The City of Pompano Beach limits the peak stormwater runoff from new development to be no greater than the stormwater runoff under the pre-development conditions. The City of Pompano Beach implements the regulatory requirements of SFWMD and Broward County EPGM which do not allow any increase off-site discharges from a property after development. For specific information on the level of service criteria for stormwater management systems, please refer to Section 3.1 of this report, which was based on the Drainage Element within the City of Pompano Beach Comprehensive Plan.

Design Storms (DS)

The City of Pompano Beach requires the stormwater management system to meet level of service criteria, which is based on design storm events. The City of Pompano Beach implements the regulatory requirements of SFWMD and Broward County EPGM, which are both based on SFWMD design storm events. The SFWMD design storm events include the 5-year 1-day event, 10 year 1 day event, 25 year 3 day event, and 100-year 3 day event. For specific information on the City's adopted level of service criteria related to design storm events for stormwater management systems, please refer to Section 3.1 – Level of Service Criteria of this report.

Public Maintenance (PUB)

The City of Pompano Beach has the authority to inspect all private and public stormwater facilities to ensure adequate maintenance is provided to these systems. For specific information on the existing stormwater management systems within the City, please refer to Section 2.6 – Stormwater Atlas of this report.

Stormwater Master Plan (SMP)

This Stormwater Master Plan has been adopted by the City of Pompano Beach to define the regulatory requirements on the stormwater management system. The Stormwater Master Plan encompasses the entire City limits, which was divided into 6 major drainage basins for modeling purposes. The City of Pompano Beach limits the peak stormwater runoff from new development to be no greater than the stormwater runoff under the pre-development conditions. For specific information on the City's adopted level of service criteria for stormwater management systems, please refer to Section 3.1 – Level of Service Criteria of this report.

Freeboard (FRX)

The City of Pompano Beach limits the lowest finish floor elevation of buildings to be no lower than the elevation for the respective area as depicted on the FEMA National FIRM for flood hazard areas based on the 100-year flood elevation or to be no lower than 18 inches above the crown of the adjacent public street or roadway (whichever is highest). This freeboard requirement is defined within the Chapter 152.20 – Floodplain Management Regulations.

Erosion and Sedimentation Control Regulations (ESC)

The City of Pompano Beach has established erosion and sedimentation control regulations which are defined within the Code of Ordinance Chapter 53.16 under Discharges into the Municipal Stormwater Management System. For additional information on the City's erosion and sedimentation control program, please refer to Section 3.4 of this report on the NPDES Program and Section 4.9.2 of this report on the BMP.

Water Quality Regulations (WQ)

The City of Pompano Beach limits the pollutant loads within stormwater runoff from new development. The City of Pompano Beach implements the regulatory requirements of SFWMD and Broward County EPGM which require the retention of a specified volume of stormwater runoff for water quality treatment. For specific information on the City's adopted level of service criteria related to water quality treatment for stormwater management systems, please refer to Section 3.1 – Level of Service Criteria.

3.5.3 FREEBOARD ANALYSIS

The FEMA CRS program provides credits for City ordinances which require that new development be provided more protection than that of the National Flood Insurance Program's (NFIP's) minimum requirements. The freeboard requirement defines that the finish floor elevation of a new structure must be set at a specified height above the FEMA base flood elevation for that location. The freeboard requirement provides an additional margin of protection to structures from flood hazards. CMA has evaluated the impact of implementing a freeboard standard for finish floor elevations at intervals of 12 inches, 18 inches, and 24 inches above the FEMA flood zone designation for each location. In order to verify the feasibility of the proposed freeboard standard, the elevations were compared to the current City ordinance of setting the finish floor elevation for new construction at least 18 inches above the crown of road elevation. The freeboard analysis will be used to determine the feasibility of converting to a freeboard standard above the FEMA flood elevation at each property location. If the freeboard standard is implemented by the City for new construction, City may obtain additional credit points under the FEMA CRS Program, which could result in a reduction of flood insurance rates for property owners in the City.

The City of Pompano Beach provided CMA with copies of elevation certificates submitted from 2000 to 2012 for properties located throughout the City. CMA reviewed these certificates and randomly selected certificates based on the geographical location of the property for the purpose of reviewing an even distribution of properties throughout the City limits. CMA entered the information from these selected elevation certificates into a GIS geodatabase for comparison with the adjacent ground surface elevation. A total of 168 elevation certificates were inserted into the GIS geodatabase. The map which displays the location of each elevation certificate selected for inclusion in the analysis is enclosed within digital Appendix B-12 attached to this report. For each elevation certificate location, CMA identified the crown of road elevation of the adjacent roadway and identified Draft FEMA Flood Zone Base Flood Elevation for each property location. The crown of road elevation was identified from the Broward County Digital Elevation Model (DEM). A total of 75 properties fell within the FEMA Flood Zone designation of AE, AH, AO, or VE, which have a defined base flood elevation. The remaining properties selected fell within the FEMA Flood Zone designation X or the 0.2% annual chance flood hazard, which do not have a corresponding base flood elevation.

For consistency purposes, the finished floor elevation defined within each elevation certificate was converted to the NAVD 88 datum by subtracting the Broward County standard conversion factor of 1.51 feet. With all elevations defined in the same vertical datum, CMA conducted the free-board analysis comparing the finished floor elevations to 12 inches, 18 inches and 24 inches above the 2011 Draft FEMA Flood Zone Base Flood Elevation. CMA summarized the properties where the finished floor elevations were greater than the FEMA elevations and calculated the percentages of properties able to meet each free-board interval. CMA reviewed freeboard intervals of 0 inches, 12 inches, 18 inches and 24 inches above the 2011 Draft FEMA Flood Zone Base Flood Elevation. For comparison purposes, the complete table of the freeboard analysis results is enclosed within digital Appendix B-12 attached to this report. The results are summarized below:

- 71% Existing finish floors above Draft FEMA Flood Zone Base Flood Elevation
- 28% Existing finish floors above Draft FEMA Flood Zone Base Flood Elevation + 12 inches
- 17% Existing finish floors above Draft FEMA Flood Zone Base Flood Elevation + 18 inches
- 8% Existing finish floors above Draft FEMA Flood Zone Elevation + 24 inches
- 84% Existing finish floors above Crown of Road Elevation + 18 inches for residential properties or + 6 inches for commercial properties

Based on these results, CMA would estimate that less than 30% of properties within the City (in the AE, AH, AO, or VE Flood Zones) would meet a free-board standard of 12 inches above the Draft FEMA

Flood Zone Base Flood Elevation. CMA also reviewed the current standard requirement of 18 inches above the crown of road for residential properties and 6 inches above crown of road for commercial properties and calculated 84% of the existing sampled properties would meet this standard.

3.6 STORMWATER UTILITY FEE

The City established the Stormwater Management Utility to provide a dedicated funding source to address the City's stormwater management concerns. The funding from this Stormwater Utility is utilized to maintain the City's existing stormwater management system and to construct stormwater improvements to address flooding issues. From an organizational and operational perspective, there is no difference between a stormwater management program and a stormwater utility. The stormwater management program was created by City ordinance to implement the functional requirements of the stormwater management system and imposes a stormwater utility fee. These fees provide annual revenues to the stormwater management program enterprise fund. These funds are authorized to be used for the following activities:

- Management Services – design, studies, permit review, plan preparation and development review
- Operation & Maintenance – operation, maintenance, repair and replacement of the stormwater collection, treatment and conveyance infrastructure
- Construction Costs – project costs related to constructing large and small infrastructure
- Administration Costs – administration costs in support of the stormwater management program
- Debt Service – debt service costs related to financing stormwater capital improvements
- Studies – funding of studies related to stormwater management planning

According to a Stormwater Management Billing Rate structure package, the stormwater rates are based on total property area, number of dwelling units, and standard runoff coefficients. Standard runoff coefficients were used for each category, multiplied by the total area in the City for each category, creating weighted total areas. The weighted total areas represent the portion of the budget (based on the percent of weighted area) needed to be produced by each category. For the residential properties, the budget amount to be produced divided by the number of dwelling units (divided by 12 to get a monthly value) defined the residential rate (dollars per month per dwelling unit) and for nonresidential and undeveloped properties, the budget amounts to be produced divided by the respective total areas (also divided by 12) defined the nonresidential and undeveloped rates (dollars per acre per month). The current operating budget for the Stormwater Utility Program along with the total stormwater utility fees collected by the City is summarized within the table below.

Table 3.2 – Operating Budget for Stormwater Utility

REVENUES	**FY2011 (ACTUAL)**	**FY2012 (BUDGET)**	**FY2012 (BUDGET)**
Stormwater Utility Fees	$2,457,922	$2,420,000	$2,420,000
Interest Earnings	$65,575	$110,000	$50,000
Budgetary Retained Earnings	$0	$57,594	$294,784
TOTAL	**$2,523,497**	**$2,587,594**	**$2,764,784**

APPROPRIATIONS	**FY2011 (ACTUAL)**	**FY2012 (BUDGET)**	**FY2012 (BUDGET)**
Personnel Services	$406,086	$508,480	$509,175
Operating Expense	$1,513,342	$598,301	$586,565
Capital Expense	$322,974	$0	$0
Inter-fund Transfers	$155,591	$162,748	$169,044
Working Capital Reserve	$0	$868,065	$500,000
TOTAL	**$2,397,993**	**$2,137,594**	**$1,764,784**

In order to assess the City of Pompano Beach stormwater utility fee, the 2011 Stormwater Utility Survey provided by the Florida Stormwater Association (FSA) was used. The survey reports elements of stormwater utilities within Florida based on survey information collected every other year since 2001. The 2011 FSA survey had 155 respondents, including the City of Pompano Beach. The survey provides a comparison of the current operation of the City's Stormwater Management Program to those of other cities in Florida. The City of Pompano Beach has a current stormwater utility fee of $3.00 per ERU. The municipalities located in Broward County have an average stormwater utility fee of $5.82 per ERU. The average stormwater utility fee within the State of Florida is currently $4.76 per ERU. According to the 2011 FSA survey, the City's stormwater utility fee is 49% lower than the Broward County average and 37% lower the Florida average.

At the completion of this Stormwater Management Master Plan Update, the City will likely reconsider the stormwater capital improvement program (CIP) as well as other stormwater management activities such as operation and maintenance, program management, etc. If the City decides that additional stormwater revenue is needed, especially via the stormwater utility fees, it is suggested that the City conduct an evaluation of the revenue from the Stormwater Utility Fee relative to overall stormwater management program. The additional revenue will likely be required for the implementation of the stormwater improvements recommended within this Stormwater Master Plan, for the additional operation and maintenance effort required for the recently annexed areas of the City, for the additional operation and maintenance effort required for the proposed stormwater improvements, and for additional compliance efforts required to meet new NPDES permit requirements. As suggested to the Florida Stormwater Association, the City should have a standard definition for stormwater management program within the following four categories:

- Program Management (administration, engineering, code review and enforcement and project management)
- NPDES MS4 Compliance (annual reporting, compliance inspection, etc.)
- Operation and Maintenance (O&M) (routine and special maintenance of the stormwater infrastructure and minor repair and replacement)
- Capital Improvements or CIP (the construction of larger stormwater infrastructure projects)

SECTION 4 – EVALUATION OF EXISTING STORMWATER SYSTEM

4.1 OVERVIEW

An important component of this Stormwater Master Plan is the development of the hydraulic and hydrologic (H&H) computer model of the existing conditions within the study area, including the City's entire stormwater management system. This section summarizes the input data, assumptions, and analysis methods used to develop the stormwater model of the existing conditions in the study area. The purpose of the existing conditions stormwater model is to define the expected performance of the City's existing stormwater management system during various design storm events. The stormwater model of the existing conditions was used to identify potential "problem" areas where flooding can be expected during a significant rainfall event. The stormwater model was also used to identify potential "bottlenecks" within the City's existing stormwater management system which restrict the flow of stormwater runoff to system outfalls. Based on the results of the existing conditions stormwater model, CMA identified areas of the City which may require future stormwater improvements in order to meet level of service criteria for flood protection.

CMA developed the H&H computer model for the existing conditions within the City of Pompano Beach. This computer model was created with the Stormwater Management Model (SWMM) software from the Environmental Protection Agency (EPA). CMA created the model based on the comprehensive data collected on the existing conditions, which was defined within Section 2 of this report. This data was used to develop the various model parameters which were incorporated into the stormwater model of the existing conditions, which include the following:

- Drainage Basin Boundaries
- Stage-Storage Elements
- Basin Parameters
- Soil Parameters
- Groundwater Parameters
- Rainfall Maps and Distribution
- Stormwater Management System Pipe Links
- Stormwater Management System Control Structures
- Channel Cross Sections
- Overland Flow Connections
- Boundary Conditions

The development of the existing conditions stormwater model for the City of Pompano Beach is further defined in more detail within the following sections.

4.2 STORMWATER MODEL SELECTION

CMA recommended that the SWMM software from the EPA be used to develop the stormwater model of the existing conditions within the City of Pompano Beach. SWMM provides the ability to conduct a more comprehensive analysis of the performance of the existing stormwater management system. SWMM also provides more flexibility with development of the model since stormwater runoff can be conveyed to a potentially larger number of elements of the local stormwater infrastructure. The SWMM engine simultaneously resolves the continuity and momentum equation using the dynamic wave set of equations that predict the hydraulic behavior in situations where the steady state flow and the kinematic wave equations no longer apply, such as during surcharge, backflow, exit/entry losses, flow reversal, or pressurized flow conditions. The SWMM software can also be integrated with GIS, which allows for a

more visually friendly interface. Using the GIS integration, SWMM can be used to spatially display the link/node components of the stormwater model together with maps of the existing stormwater management system. During the review meeting for the Data Collection Memorandum on November 21, 2011, the City of Pompano Beach agreed with the recommendations to use SWMM for the development of the existing conditions model.

The SWMM software was used to develop a dynamic H&H computer model of the existing conditions within the City of Pompano Beach. SWMM is used to conduct simulations of both stormwater water runoff in overland flow conditions and hydraulic conveyance in channel systems or pipe networks. The hydrologic modeling system within SWMM distributes specified rainfall events across the drainage basins throughout the City limits, estimates stormwater runoff, and conveys runoff to various loading points into the City's stormwater management system. The hydraulic modeling system within SWMM uses a link/node representation to dynamically route the flow of stormwater within the City's stormwater management system. The dynamic flow routing within SWMM can be used to simulate pipe flow, channel flow, free surface flow, and pressure flow within the City's stormwater management system. The capabilities of SWMM will allow CMA to complete all analysis necessary to complete the Stormwater Master Plan.

4.3 DATA EVALUATION

CMA gathered and evaluated available information on the existing stormwater management system within the City limits. The available information on the existing stormwater management system within the City included previous stormwater master planning documents, the City's GIS Stormwater Geodatabase, stormwater atlases from the Florida Department of Transportation (FDOT) and Broward County, drainage as-built drawings on stormwater improvement projects within the City, various aerial photographs, NRCS hydrologic soil coverage, USGS SEAMLESS database for impervious coverage, City land use and zoning categories for all properties, FEMA flood zone elevations, FEMA Repetitive Loss information, National Oceanic Atmospheric Administration (NOAA) tide level data, SFWMD canal level data, U.S. Geological Survey (USGS) groundwater level data, SFWMD rainfall data, SFWMD and Broward County canal cross sections, and bare earth LiDAR topographic data throughout the City. CMA reviewed all collected data to confirm the most recent and most accurate information would be used during the completion of this project. Based on our evaluation of the collected data, CMA confirmed that all information necessary was collected to develop a comprehensive stormwater model for the City of Pompano Beach.

4.4 STORMWATER MODEL DEVELOPMENT

Within the SWMM software, the existing conditions stormwater model consists of two components: hydrology and hydraulics. The hydrologic block in SWMM simulates stormwater water runoff and overland flow conditions throughout all drainage basins within the City. The hydraulic block in SWMM simulates the conveyance of stormwater runoff within all channel systems and pipe networks of the City's stormwater management system. CMA used this comprehensive data collected during Task 1 as the basis for building the hydrology and the hydraulics that define the existing conditions by estimating the various input parameters required by each of these blocks in the SWMM input (INP) file. The methodology used by CMA to develop the existing conditions stormwater model is defined within the following sections.

4.5 HYDROLOGIC MODEL

The hydrologic modeling system within SWMM distributes specified rainfall events across the drainage basins throughout the City limits and determines how much of the rainfall is transformed into stormwater runoff, which is typically called "effective rainfall". The difference in depth between rainfall and effective

rainfall is the lost volume of water to infiltration, impervious and pervious storage, and groundwater recharge. The City of Pompano Beach was divided into major drainage basins and sub-basins. CMA assigned each sub-basin with various hydrologic parameters characteristic of the actual conditions within the tributary area they define. These hydrologic parameters are used to define the typical condition of the ground surface, subsurface soils, and groundwater found within each drainage basin. These hydrologic parameters are used by the SWMM stormwater model to simulate the overland flow of stormwater runoff within each drainage basin under various rainfall scenarios. The hydrologic parameters used for the model simulations are defined in more detail within the following sections.

4.5.1 SUB-BASINS

CMA divided the City into six major drainage basins: Northeast (NE), Northwest (NW), Southeast (SE), Southwest (SW), Central (CT), and North Central (NC). These major basin designations are used within the stormwater model to identify the general location of all sub-basins. Based on the available data collected, including a topographic elevation data, aerial photography, and the existing drainage atlas, a total of 621 sub-basins have been delineated within the study area at this time. The sub-basins vary in size from 1.5 acres in residential neighborhood areas to up to 420 acres around golf courses. The average sub-basin size is 30.4 acres. Since the existing ground surface topography has no regard for City limits, a few of the sub-basins within the study area extend over the City limits in order to account for potential incoming or outgoing stormwater runoff from and to these areas.

The basin delineation process commenced with the identification of the major drainage network features, such as large diameter stormwater pipes, stormwater outfalls, ponds, retention areas, canals or drainage ditches. The land area, which surrounds these features and contributes stormwater runoff to these features, was delineated on a City-wide map. Consequently, each of these land areas were further subdivided into multiple sub-basins based on the existing ground surface topography. The sub-basin boundary was delineated by drawing a basin boundary through high terrain as determined by the available topographic data. Per the scope requirements, each positive outfall from the existing stormwater management system was assigned a sub-basin. Although this scope requirement resulted in some small sub-basins where the drainage network was limited to a single outfall pipe to an adjacent canal, it will enhance the expected accuracy of the stormwater model to estimate the peak discharge from individual outfalls. It should be noted that not all sub-basins included a positive outfall to an adjacent water body, particularly in the western portions of the City. The bulk of the sub-basins did not include either major interconnected drainage network or positive outfall to an adjacent water body. The typical sub-basin within the study area contributes stormwater runoff to a series of independent swale areas or limited drainage facilities dispersed throughout the sub-basin area.

CMA established a nomenclature system to assign a unique name to each sub-basin or hydrologic unit within the study area. Each sub-basin name has the following nomenclature structure: **MB_OUT_SB**. Within this nomenclature structure, **MB** is correlated to the major basin classification of NE, NW, SE, SW, CT, and NC for the region of the City, **OUT** is correlated to a three digit sequential number assigned to each outfall with a sequence maintained within each major basin and assigning an outfall number to the 'isolated' sub-basins category discussed previously and **SB** is correlated to a two digit sub-basin number associated to the location of the sub-basin with respect to its outfall, assigning a higher number to sub-basins located farther away from the outfall. The limits and designation of theses sub-basins within each major basin are displayed in Figure 4-1 Sub-Basin Boundaries within this section.

4.5.2 HYDROLOGIC PARAMETERS

The SWMM requires the input of various hydrologic parameters for each sub-basin or hydrologic unit in order to accurately simulate the flow of stormwater runoff across the ground surface. These hydrologic

parameters are used to define the existing conditions of the ground surface within each sub-basin and to establish the flow conditions of surface runoff across these sub-basins into the stormwater management system. The hydrologic parameters include the total area of ground surface within the sub-basin, the hydrologic width of the sub-basin, the slope of the sub-basin, percentage of impervious and pervious ground surface within the sub-basin, and the roughness coefficient for the impervious and pervious ground surfaces within the sub-basin. The hydrologic parameters for each sub-basin are defined within the SWMM Stormwater Model Input Parameters, which can be found in Appendix A-6 of this report. The methodology for determining the hydrologic parameters for each sub-basin is defined below:

Area

The area of each sub-basin was determined based on the ground surface located within the sub-basin's boundaries delineated by CMA which were based on aerial photographs and topographic data for the City of Pompano Beach. CMA tabulated the ground surface area in acres for each sub-basin within the study area using GIS. Within the study area, the average sub-basin size is 30.4 acres. The area of each sub-basin was included in the model input tables and incorporated into the SWMM stormwater model of the existing conditions. SWMM will use the area of each sub-basin to estimate the amount of stormwater runoff generated within the sub-basin for each rainfall event.

Slope

The slope parameter for each sub-basin was determined based on the elevation differential along the expected flow path for stormwater runoff. Several flow paths were delineated within each sub-basin to calculate the average flow path length and slope. CMA used the Digital Elevation Model (DEM) of the ground surface to determine the elevation at the starting point and loading point of the expected flow path within each sub-basin. CMA tabulated the slope parameter by determining the difference in ground surface elevation between the starting point and the loading point and dividing by the length of the expected flow path within each sub-basin. The average slope for sub-basins within the study area is 0.52%. The slope for each sub-basin was included in the model input tables in percentage format and incorporated into the SWMM stormwater model of the existing conditions. SWMM will use the slope parameter for each sub-basin to estimate the time of concentration (TOC) for stormwater runoff to reach the loading points within the sub-basin. TOC is typically defined as the longest travel time that it takes a particle of water to reach the discharge point of the sub-basin, which is typically representative of the travel time of stormwater runoff.

Width

The width parameter for each sub-basin was determined based on the typical linear width of the expected flow path for stormwater runoff. CMA tabulated the width for each sub-basin by dividing the sub-basin area by the length of the expected flow path. The average width of sub-basins within the study area is 1,392 feet. The width for each sub-basin was included in the model input tables and incorporated into the SWMM stormwater model of the existing conditions. SWMM will use the width parameter for each sub-basin to estimate the time of concentration (TOC) for stormwater runoff to reach the loading points within the sub-basin. TOC is typically defined as the longest travel time that it takes a particle of water to reach the discharge point of the sub-basin, which is typically representative of the travel time of stormwater runoff.

Impervious/Pervious Area

The percentage of ground surface coverage which is considered either impervious versus pervious is an important hydrologic parameter used within a stormwater model. Impervious surfaces, such as roadways,

driveways, sidewalks, and parking areas prevent the infiltration of stormwater runoff while pervious surface, such as grass yards, swale areas, parks, and other landscaped areas will allow stormwater runoff to infiltrate into the ground. Areas with high percentages of impervious area will contribute more stormwater runoff into public right-of-way areas since there will be less pervious areas which allows stormwater runoff to infiltrate into the ground surface. The stormwater runoff from highly impervious areas will instead need to be collected with a stormwater management system to provide transmission capacity to an area with adequate storage volume. Areas with a significant amount of green space will contribute less stormwater runoff into public right-of-way areas due to the infiltration capacity of pervious areas.

CMA determined this impervious parameter for each sub-basin based on available data on the existing conditions within the study area. The USGS has developed a nationwide impervious percentage database, which is part of a much larger spatial and temporal database of geographical features that has been called SEAMLESS. This impervious percentage database has been put together in a raster format with a cell size of 30 meters. Within this 30-meter cell size, pervious and impervious terrain has been weighted out to come up with a percent imperviousness value representative of the 30-meter cell. CMA used the USGS SEAMLESS data to estimate the level of imperviousness for each sub-basin. CMA used GIS to select each 30-meter cell located within each sub-basin boundary and then tabulated the impervious percentage for each sub-basin based on this USGS SEAMLESS data. The average parameters for sub-basins within the study area are 54% impervious coverage and 46% pervious coverage. The breakdown of impervious/pervious area within each sub-basin was included in the model input tables in percentage format. CMA incorporated these impervious/pervious percentages into the SWMM stormwater model of the existing conditions.

SWMM provides the means to differentiate impervious areas that are directly connected to the stormwater system (DCIA) from those are not directly connected (NDCIA). In SWMM, the NDCIA portion of the impervious area is meant to discharge to a pervious area. From previous hydrologic studies in Florida, typical percentages of NDCIA have been established per land use. After applying these NDCIA values to the existing land use coverage obtained from SFWMD, an average of 12% was calculated. Table 4.1 shows a list of typical NDCIA percentage used for this project. SWMM will use the impervious/pervious percentages of each sub-basin to simulate the flow conditions of stormwater runoff across the ground surface, estimate the amount of runoff generated from a sub-basin, estimate the amount of stormwater runoff diverted to infiltration into the ground surface, and estimate the time for stormwater runoff to the reach the loading points.

Table 4.1 – NDCIA and Roughness Coefficient by Land Use

Land Use Category	Impervious N	Pervious N	NDCIA (%)
Open Park	0.015	0.400	4.0
Agricultural/Golf Course	0.015	0.300	4.0
Low Density Residential	0.015	0.250	7.5
Medium Density Residential	0.015	0.250	12.0
High Density Residential	0.015	0.250	17.5
Light Industrial/Commercial	0.015	0.250	9.0
Heavy Industrial	0.015	0.250	10.0
Wetland	0.100	0.400	0.0
Water	0.024	N/A	0.0

Roughness Coefficient

The roughness coefficient of impervious and pervious ground surfaces within each sub-basin was determined based on the ground surface located within the sub-basins boundaries, as delineated by CMA. The Manning's Roughness Coefficient (N) was used to correlate the roughness of the various ground surface coverages found within each sub-basin. The roughness coefficients for impervious and pervious surface were distributed based on land use categories within each sub-basin. Both the land uses and their respective roughness coefficients are listed in Table 4.1 above. For land use coverage, the SFWMD's coverage was applied to the study area as it includes areas that are outside of the City limits. The roughness coefficients within each sub-basin were included in the model input tables and incorporated into the SWMM stormwater model of the existing conditions. SWMM will use the roughness coefficients within each sub-basin to simulate the flow conditions of stormwater runoff across the ground surface and to estimate the time for stormwater runoff to the reach the loading points.

4.5.3 SOIL PARAMETERS

The soil conditions within the City limits are important for the development of the existing conditions stormwater model since various soil types have different infiltration rates, which will control how quickly stormwater runoff infiltrates at the ground surface within pervious areas. The soil survey for Broward County was completed by the NRCS (previously the Soil Conservation Service, SCS) in 1976. One of the properties identified by the soil survey is the hydrologic soil group (HSG) classification for each soil type. This classification defines the hydrologic property of the soil in terms of the capacity to infiltrate water through available porosity. The hydrologic soil groups are defined by four classification categories of A, B, C and D. Each HSG category has been correlated to an expected infiltration rate. The soil parameters for each HSG category, which were used within the stormwater model, are defined within Table 4.2 below.

Table 4.2 – Global Soil Parameters

Soil Type	Maximum Infiltration Rate (in/hr)	Minimum Infiltration Rate (in/hr)	Decay Rate (hr^{-1})	Dry Time (days)	Soil Storage (in)
A	12.0	1.0	2.0016	1.0	6.75
B	9.0	0.50	2.0016	1.0	5.0
C	6.0	0.25	2.0016	1.0	3.8
D	4.0	0.10	2.0016	1.0	1.4

CMA used GIS to map the extents of each HSG classification throughout the study area. CMA used GIS to determine a percentage breakdown of the HSG coverage within each sub-basin boundary. Using the values listed in Table 4.2 for each parameter and HSG GIS coverage, CMA estimated area-weighted soil parameter values per sub-basin. These area-weighted soil parameters were then included in their respective model input blocks. SWMM will use these parameters to simulate the amount of stormwater runoff which infiltrates into the ground surface within each sub-basin as the rainfall event progresses.

4.5.4 GROUNDWATER PARAMETERS

SWMM has the capability to simulate the subsurface movement of stormwater runoff which infiltrates into the ground surface. While the groundwater baseflow can be relatively insignificant within a localized area, the groundwater flow can have an impact on a regional basis, such as the large areas within the City of Pompano Beach. Since various surface water bodies and canals within Pompano Beach intersect the surficial aquifer, a portion of this groundwater baseflow is transferred into the surface water system and should be accounted for within the stormwater model. Based on the topographic distribution within the study area and available information collected from the SFWMD regarding the local surficial hydraulic conditions (i.e. surficial hydraulic conductivity), three aquifers were created in SWMM with similar groundwater characteristics and parameter values except for initial water table elevation and surficial hydraulic conductivity. The SFWMD's DBHYDRO database provided hydraulic conductivity data for the surficial aquifer typically taken within 5 feet through 30 feet below ground. The location of 68 monitoring wells considered for the development of a distribution map of the hydraulic conductivity parameter within the study area is displayed within Figure 4-2 Surficial Hydraulic Conductivity below. Most of these monitoring wells belong to the Broward County Environmental Protection and Growth Management Department (BCEPGM) at which hydraulic pumping tests have been performed to estimate the local hydraulic conductivity. The three aquifers were identified within the study area as EastUS1, NorthATL, and SouthATL due to their respective geographic domain.

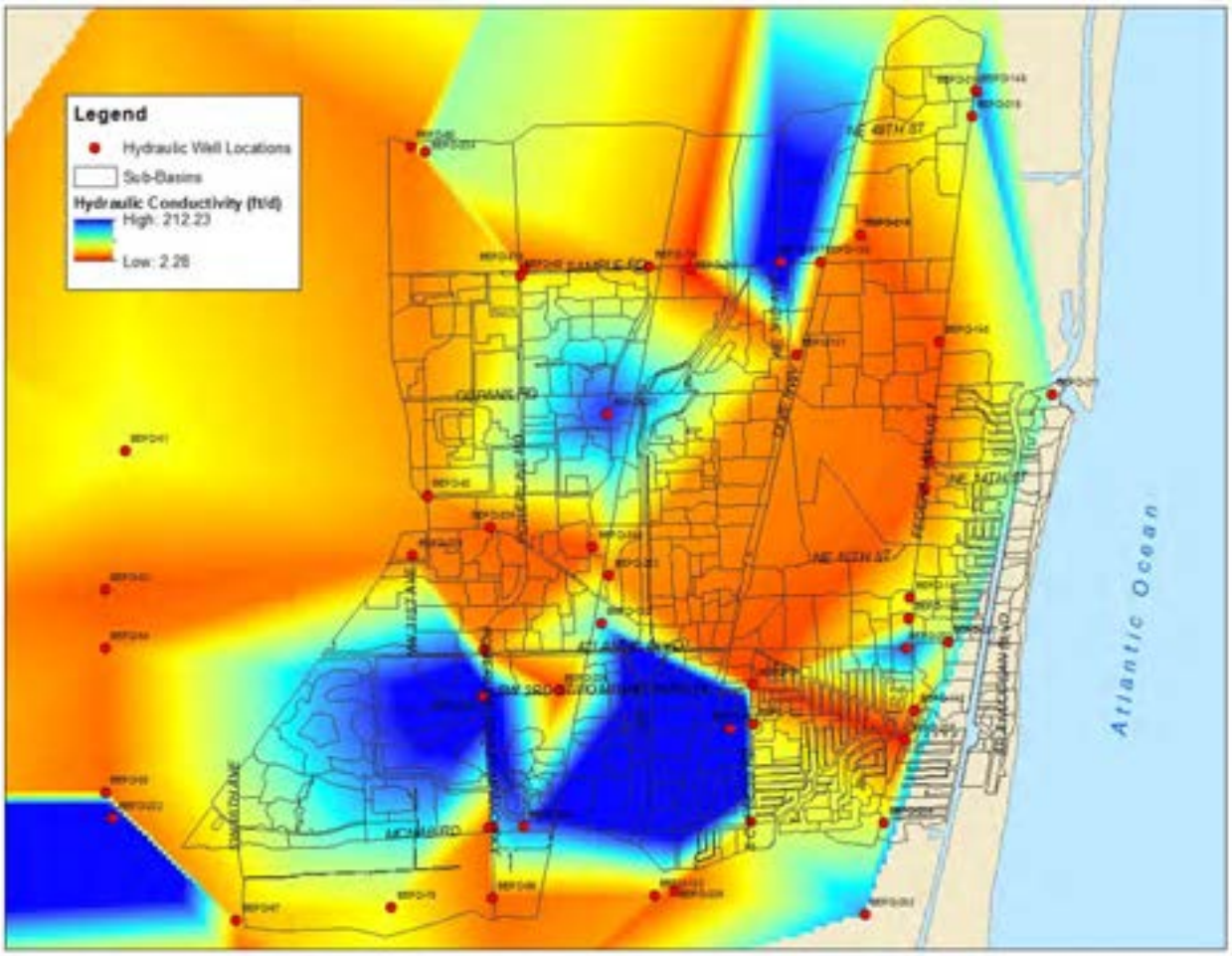


Figure 4-2 Surficial Hydraulic Conductivity (feet/day)

In SWMM, groundwater flow is estimated per sub-basin considering the respective aquifer's characteristics, the sub-basin overlays, the invert of the sub-basin loading node, and the local groundwater characteristics such as hydraulic conductivity and flow path. These characteristics are incorporated into the Dupuit-Forcheimer approximation for flow into a channel that SWMM utilizes for its groundwater routine.

4.5.5 RAINFALL ASSUMPTIONS

Within the stormwater model of the existing conditions, a synthetic hyetograph for design rainfall events is used to evaluate the performance of the existing drainage system and to compare this performance to measurable level of service criteria for flood control. These synthetic hyetographs have been developed regionally by the SFWMD for one-day and three-day duration design storm events by studying the historical distribution of rainfall in the region and representing the regional distribution of one-inch of rainfall. The design storm for a given frequency and duration is determined by multiplying the synthetic hyetograph by the respective rainfall depth, which is found spatially distributed in isohyet maps published by SFWMD. The design storm rainfall volumes for the stormwater model simulations were obtained from the SFWMD Permit Information Manual, Volume 4. The existing conditions stormwater model was used to simulate the 100-year, 72-hour design storm; the 25-year, 72-hour storm event; 10-year, 24-hour storm event; and 5-year, 24-hour storm event. A summary of the rainfall depths to be used in the existing conditions stormwater model is defined within the Table 4.3 below.

Table 4.3 – Design Storm Depths

Design Storm	Rainfall Depth (inches)
5-year, 24 hour	7.8
10-year, 24-hour	9.0
25-year, 72-hour	16.0
100-year, 72-hour	20.0

For the purpose of calibrating the existing conditions stormwater model, historical rainfall data was obtained from rainfall gauges maintained by SFWMD, NOAA, and other water management districts (WMD) throughout the study area. The calibration effort is necessary to validate the accuracy of the existing conditions stormwater model. During the calibration effort, the historical rainfall data for specific rainfall events will be incorporated into the stormwater model to run a simulation under these rainfall conditions. The stormwater model is calibrated by comparing the peak flood stages estimated by the stormwater with observed and/or recorded high water marks and canal water levels during these rainfall events.

4.5.6 INFLOWS

With the purpose of maintaining a realistic water level in the SFWMD C-14 Canal as it enters the City limits and discharges to the Intracoastal Waterway, a 15,000 acre tributary area has been incorporated to the western boundary of the SWMM model to simulate real-world conditions. This tributary area was called "WestC14" due to its geographical location. Due to the extent of this tributary area, which corresponds to 80% of the study area, a land use approach was used to develop the hydrologic characteristics previously discussed in this section, including level of imperviousness (NDCIA) and pervious and impervious roughness coefficients. For similar reasons, another tributary area located south of the SFWMD C-14 Canal and east of the Tri-Rail railroad tracks was also included in the SWMM model as a runoff inflow to the SFWMD Control Structure S-37A. This southern tributary area was defined using a land use approach to identify its respective runoff parameters.

The surface water loss is evident in the stream flow records at SFWMD Control Structures S37A and S37B. This surface water loss is likely associated with the presence of two wellfields (City of Fort Lauderdale Prospect Wellfield adjacent to Fort Lauderdale Executive Airport and Pompano Beach Wellfield at Pompano Beach Municipal Airpark) within or adjacent to the study area, which lead to percolation through the canal bed. In order to account for this loss, an outlet has been added to the existing conditions model with flow withdraws varying with respect to stage elevations at the SFWMD C-14 Canal.

4.6 HYDRAULIC MODEL

The hydraulic model is used to simulate the conveyance of stormwater runoff within all channel systems and pipe networks of the City's stormwater management system. CMA has developed the SWMM hydraulic model of the existing conditions based on topographic data, City GIS Stormwater Geodatabase, SFWMD Canal Cross Sections, Broward County and FDOT stormwater atlases, and various as-built drawings from projects throughout the City. SWMM uses a node/link representation of the primary stormwater management system within the City of Pompano Beach. The primary stormwater management system consists of canals, bridges, culverts, and drainage piping, which transmit stormwater runoff from node to node. The SWMM hydraulic model was configured by establishing nodes at the following locations within the study area:

- Loading point for each hydrologic unit, which is typically the low point of the sub-basin
- End of primary drainage pipes within stormwater management system
- Diameter changes in primary drainage pipes within the stormwater management system
- End of drainage culverts within stormwater management system
- Control structures (pump stations, weirs, and orifices)
- Canal intersections
- Changes in canal geometry

The SWMM stormwater model of the existing conditions consists of 1,090 nodes with 100 outfalls, 630 storage units, and 2,058 links. The configuration of the nodes and links within the SWMM stormwater model is displayed in Figure 4-3 Stormwater Model Schematic Maps for each region of the City of Pompano Beach within this section. The hydraulic components of the stormwater management system which were incorporated into the existing conditions stormwater model are defined in more detail within the following sections.

4.6.1 STAGE-AREA RELATIONSHIP

The stage-area relationships are necessary in stormwater models of relatively flat topography, such as the City of Pompano Beach. An important component of the stormwater model is establishing the available storage within each sub-basin in the study area, which is based on the topographic elevation data. In summary, the stage-area relationship defines the available storage volume for stormwater runoff below each elevation stage interval. In order to calculate the stage-area for each sub-basin, CMA obtained digital topographic data within the City limits, which was developed using LiDAR technology. A Triangulated Irregular Network (TIN) model of the ground surface, which is also known as a digital elevation model (DEM), was created from the bare earth elevation points from the study area at a resolution of 10-foot cell size. This DEM was used to establish stage-area for each sub-basin. A stage interval of 0.5 feet was used to calculate the stage-area for each sub-basin. CMA incorporated the stage-area calculation for each sub-basin into the SWMM existing conditions model. SWMM will use these storage elements of each sub-basin to simulate the flood stages for the duration of the storm event, which,

coupled with the overland flow connectivity discussed in a later section, SWMM will be able to represent inter-basin connectivity characteristic of significant storm events.

4.6.2 Cross Sections

This hydraulic parameter establishes the cross sectional geometry of various links within the stormwater model, including the canals and drainage ditches interconnected with the City's stormwater management system. CMA obtained longitudinal canal depth information from SFWMD for the SFWMD G16 Canal (Old Pompano Canal) to help establish its geometry. Bridge and cross section data for the SFWMD C-14 Canal was included from the MIKE-11 model developed by DHI and CDM Smith for Broward County, which was also used in the Stormwater Master Plan for the City of Fort Lauderdale. Broward County also provided some information on the secondary canal system confirming the cross sections at Water Control Districts (WCD) No.3 and No.4, which are separated by Atlantic Boulevard. A more detailed description of the Broward County WCDs will follow in a later subsection regarding boundary conditions. FDOT provided valuable information on the existing drainage characteristics around the I-95 corridor to which many of the City stormwater management systems discharge, as well as for other FDOT-maintained corridors. For each channel link, the SWMM input data includes shape type, the dimension of the cross-sectional geometry and transects for irregular channel sections. The cross section data for each link was included in the model input tables for the SWMM existing conditions model. SWMM will use the cross section parameter for each link to establish the flow conditions within these canals and estimate the transmission capacity of the system channel for the duration of the model run.

4.6.3 Overland Flow

Overland flow links were incorporated into the SWMM existing conditions model to account for potential inter-basin flow between adjacent sub-basins. Each underground stormwater pipe is accompanied by an overland flow link running parallel to the ground surface. The geometry of typical overland flow links is a trapezoidal cross section with 50 feet of bottom width and 2 feet of depth, which is intends to mimic the right of way interconnections between sub-basins. Another set of overland flow links included in the SWMM model were created specifically for inter-basin connectivity where no underground pipe connects the adjacent sub-basins. For this set of overland links, a transect cross-section was developed on a case by case basis based on existing topography defined by the TIN of the ground surface. These transect cross sections were calculated using a GIS tool that extracts topographic information from the 10-feet DEM created for the study area. For both types of overland links the following was applied:

- Offset elevations were added according to topography between inlet node and outlet node as needed.
- Stage-area relationships were calculated for each overland flow links and subtracted from the sub-basin stage-area relationships in order to avoid an overestimation of storage. In the case of inter-basin overland flow links, half of the length of the overland flow links was considered for the storage adjustment per sub-basin involved.

4.6.4 Pipe Network

CMA has used the City's Stormwater GIS Geodatabase as the basis of the pipe network configuration within the stormwater model. The Stormwater GIS Geodatabase depicts the geographic locations of the catch basin inlets, manhole structures, control structures, drainage piping and outfalls which are owned and maintained by the City. The Stormwater GIS Geodatabase also provides dimensions of some of the existing network elements in the form of pipe diameter and pipe length. As part of this project, CMA updated of the Stormwater GIS Geodatabase for the purpose of incorporating more detailed information into the stormwater model. CMA obtained additional as-built drawings and/or design plans for recently constructed drainage projects from the City and drainage atlases from FDOT and Broward County for

their right-of-way areas within the City limits. This data has been incorporated into the Stormwater GIS Geodatabase and was used in preparation of the stormwater model. In order to verify the location and attributes of the existing stormwater management system, a vertical and as-built survey verified the configuration, location, and elevation for the City's existing stormwater structures. The horizontal and vertical GPS data for each stormwater structure were incorporated into the Stormwater GIS Geodatabase to enhance the accuracy of said database.

CMA used the Stormwater GIS Geodatabase to obtain the various hydraulic parameters which defined the configuration of the components of the stormwater management system, such as the drainage pipes, culverts, drainage structures, control structures, and outfalls. For all drainage pipes defined within the stormwater model, CMA set the pipe lengths according to Stormwater GIS Geodatabase and set the Manning's roughness coefficient (N) according to the pipe material defined within the Stormwater GIS Geodatabase. CMA also set standard entrance and exit losses for pipes within the stormwater model, according to the pipe's conveyance capacity inverse relationship with energy loss. For all drainage structures defined within the stormwater model, the invert elevations were set such that there will be a minimum of 2 feet of cover to the crest of the pipe and that downstream inverts are always at lower elevations to avoid pipes with negative slopes. For all drainage control structures defined within the stormwater model, CMA defined the weir type, crest elevation, discharge coefficient and upstream/downstream junctions based on the available information discussed on this memorandum. Each of these hydraulic parameters were included in the SWMM model input tables. SWMM used the hydraulic parameters for each link to establish the flow conditions within these pipes and estimate the transmission capacity of the drainage pipes for the duration of the model run. In order to avoid continuity errors by having the pipes running dry near a boundary condition, an initial stage elevation of 2.4 feet NAVD at the beginning of the model run was set for those pipes with an invert elevation at or lower than this value. This elevation corresponds to the high water table typically used as water control elevation in WCD No. 4 region by Broward County regulatory activities.

4.6.5 CONTROL STRUCTURES

The existing SFWMD canals which are located within the study have control structures which manage the flow within the channel system. These control structures are typically multiple bay sluice gates that act as a weir from the bottom which can limit the peak flow. CMA obtained the operation criteria and design information for these control structures from SFWMD. The locations of these canal control structures are displayed within Figure 4-4 Control Structure Location Map within this report. Within the SWMM stormwater model, the weir lengths were based on the combined gate widths defined on the SFWMD operating criteria and design drawings and the weir heights were based on the maximum gate opening defined on these drawings. The weir crest was set based on the pipe invert elevation at the bottom of the control gate. Each of these hydraulic parameters for the control structures was included in the SWMM model input tables. SWMM will use the hydraulic parameters for each link to establish the flow conditions within these channels and estimate the peak flow through these control structures for the duration of the model run. This is a list of the water control structures included in the SWMM existing conditions model:

- CS-25R / CS-12: These control structures are part of the WCD No. 3 system controlling upstream waters coming from WCD No. 2 from draining south. Their operating criteria were set to raise the control elevation to facilitate groundwater recharge during the dry season. CS-12 is the last point of control before water reaches the SFWMD C-14 Canal.
- CS-40 / CS-8AR / CS-8 / CS-9 / CS-60 / G-57: These control structures are part of the WCD No. 3 system controlling upstream waters coming from WCD No. 2 from draining south. Their operating criteria were set to raise the control elevation to facilitate groundwater recharge during the dry season. During the dry season, in order to facilitate recharge, CS-8AR will connect water that typically

deviate from CS-40 to CS-25R (east of WCD No. 3) to CS-8, CS-9, CS-60 and finally to the SFWMD G-16 Canal (G-57).

- CS-37B / CS-37A: These control structures are part of the WCD No. 4 system controlling flow from the western portion of Broward County and the Water Conservation area part of the Everglades. These structures were designed to maintain an optimum water elevation on the upstream portion of the canal of 5.4 feet and 1.9 feet NAVD, respectively.

For all the water control structures part of the secondary canal system, the initial water elevation at the beginning of the simulation period was set in SWMM to be its respective wet weather water control elevation minus one foot. This initial water elevation allows the system to in-stream accommodate runoff before discharge through the water control structure occurs.

4.6.6 BOUNDARY CONDITIONS

The information on the positive outfalls from the City's stormwater management system is a crucial component of the existing stormwater model. The City's Stormwater GIS Geodatabase includes the location of all positive outfalls to surface water bodies within the City limits. According to the Stormwater GIS Geodatabase, there are 447 positive outfalls to surface water bodies within the City. The outfall information within the Stormwater GIS Geodatabase includes a location description and whether or not the outfall remains active at most locations. During the field investigation task, the horizontal location, pipe diameter, and pipe invert of all accessible outfalls were confirmed by the surveyor for incorporation into the Stormwater Geodatabase. An important hydraulic parameter within the stormwater model is the water level at each outfall, which is also known as the tailwater elevation. The tailwater elevation can vary due to tides at outfalls into marine canals, control structures at outfalls into SFWMD canals or stormwater runoff at outfalls into stormwater ponds.

This tidal information is used to establish the downstream boundary condition for any outfall discharging into tidally influenced waters. For establishing the boundary conditions at outfalls into the Intracoastal Waterway or the interconnected marine canals, CMA collected tidal datasets from two gauges near the Hillsboro Inlet which are maintained by NOAA. These tidal gauges are representative of the expected tidal fluctuations along the Intracoastal Waterway in the study area. Considering the variability of the tides with respect to the time of response of the entire City watershed to a design storm event, CMA has selected a static boundary condition of 0.45 feet NAVD, which is associated to the mean higher high water (MHHW) at the station. This tailwater elevation was used for running the one-day and three-day design storm simulations with the stormwater model of the existing condition.

For establishing the boundary conditions at outfalls into the SFWMD canals, CMA collected water level data from gauges at canal control structures which are maintained by SFWMD. The SFWMD DBHYDRO database was used to determine the water control elevations through recorded water level data at the headwaters of the water control structure along with the operation criteria used by the water control structures under consideration. The SFWMD Water Control Structures CS-37A, CS-37B and G-57 have an impact on the water levels within canals located in the study area. The high water level stages are controlled by gate openings following established SFWMD operating criteria. The initial water control elevation for these structures was set at 3.4 feet NAVD, which is 12 inches above the WCD No. 4's water control elevation and 18 inches above the optimum design water elevation of Control Structure CS-37A. The initial values at these canal structures will be clearly surpassed during the storm event as runoff from adjacent tributary areas and external tributary areas such as WestC14 and SouthC14 is stored and conveyed in the canal system. For the secondary canals, where, as stated in the previous section, the initial water elevation corresponds to the wet weather water control elevation of the nearest downstream control structure minus one foot, the boundary condition will change accordingly to the rate of discharge of the downstream structure and the amount of runoff discharge into the canal. For both the main and the

secondary canal systems, the SWMM existing conditions model benefits from a near real-world, dynamic, set of boundary conditions that simulate runoff capture and conveyance for a wet weather type of water level conditions throughout the study area.

Outfalls which discharge into lakes, ditches or retention areas are affected by the local tailwater conditions. During dry periods, the water level within the lake, ditch or retention area will be controlled by the local groundwater elevation along with soil type within the area. During wet periods, the water level within lakes, ditches or retention areas should be assumed to be at or near capacity. In order to estimate the water level elevation within these waterbodies for use as a boundary condition in the stormwater model, any water control structures at the lakes, ditches or retention areas were used to set the water level at the associated weir elevation, as available. The use of a wet season water table elevation data is not necessary as the weir crest at the overflow structure is equal to or higher than the high water table.

4.7 EXISTING CONDITIONS MODEL CALIBRATION

The calibration of the existing conditions model includes the adjustment of both hydrologic and hydraulic parameters so that the modeled peak flood stages and/or discharge volumes approximately match the observations and measurements during a historical storm event. The calibration process requires the reproduction of the hydrologic process for a specific historical rainfall event associated with major flooding issues within the study area. Within a properly calibrated stormwater model, the peak flood stages defined by the model should match the peak flood stages observed or recorded by the City staff during or after this historical storm event. Any stormwater model is calibrated by these high water marks which have been recorded for the highest flooding level at specific locations within the study area. The recorded canal water level elevations upstream and downstream from water control structures can also be used to calibrate the stormwater model. CMA has calibrated the existing conditions stormwater model for the study area as summarized below.

4.7.1 DATA COLLECTION

CMA collected the following datasets in order to properly calibrate the existing conditions stormwater model throughout the study area:

- Rainfall data was collected for a 15-minute interval from the SFWMD DBHYDRO database for the significant rainfall event which occurred from October 1st, 1999 to October 21st, 1999 within the study area.
- Tidal data was collected for a 6-minute interval from the NOAA website for two nearby tide stations at Virginia Key and at Lake Worth during the significant rainfall event which occurred from October 1st, 1999 to October 21st, 1999 within the study area.
- Water level data was collected for locations upstream and downstream from the SFWMD Control Structures CS-37A, CS-37B, and G57 during the significant rainfall event which occurred from October 1st, 1999 to October 21st, 1999 within the study area.
- High water marks at various locations within the study area were provided by City staff during this historical rainfall event.

<u>Rainfall Data</u>

For model calibration based on a historical rainfall event, CMA reviewed the historical variability of rainfall at local gauges and selected a significant storm event with adequate rainfall data available at multiple gauges throughout the study area. CMA collected the daily rainfall volume data from the

SFWMD S37A rainfall gauge during the record period from January 1991 until April 2012, which is summarized within Figure 4-5 below. A daily rainfall depth at this rainfall station was recorded to be 1.26 inches on October 14, 1999, 6.36 inches on October 15, 1999 and 5.26 inches on October 16th, 1999. A total of 12.88 inches of rainfall fell at this location during this three day period, which was preceded by wet weather conditions since the beginning of that month. This storm event lingered around the South Florida area for over a period of two weeks. Based on this historical data, the daily rainfall on October 15th, 1999 is the fifth largest daily rainfall depth recorded at the station during the data period. Several factors led to the flooding problems observed within the study area during this time period:

- Wetter than average antecedent soil conditions throughout the study area due to prolonged rainfall
- High rainfall intensity during the peak of the storm
- Higher tidal levels due to the perigean tides during October
- Storm surge associated with the tropical depression storm event

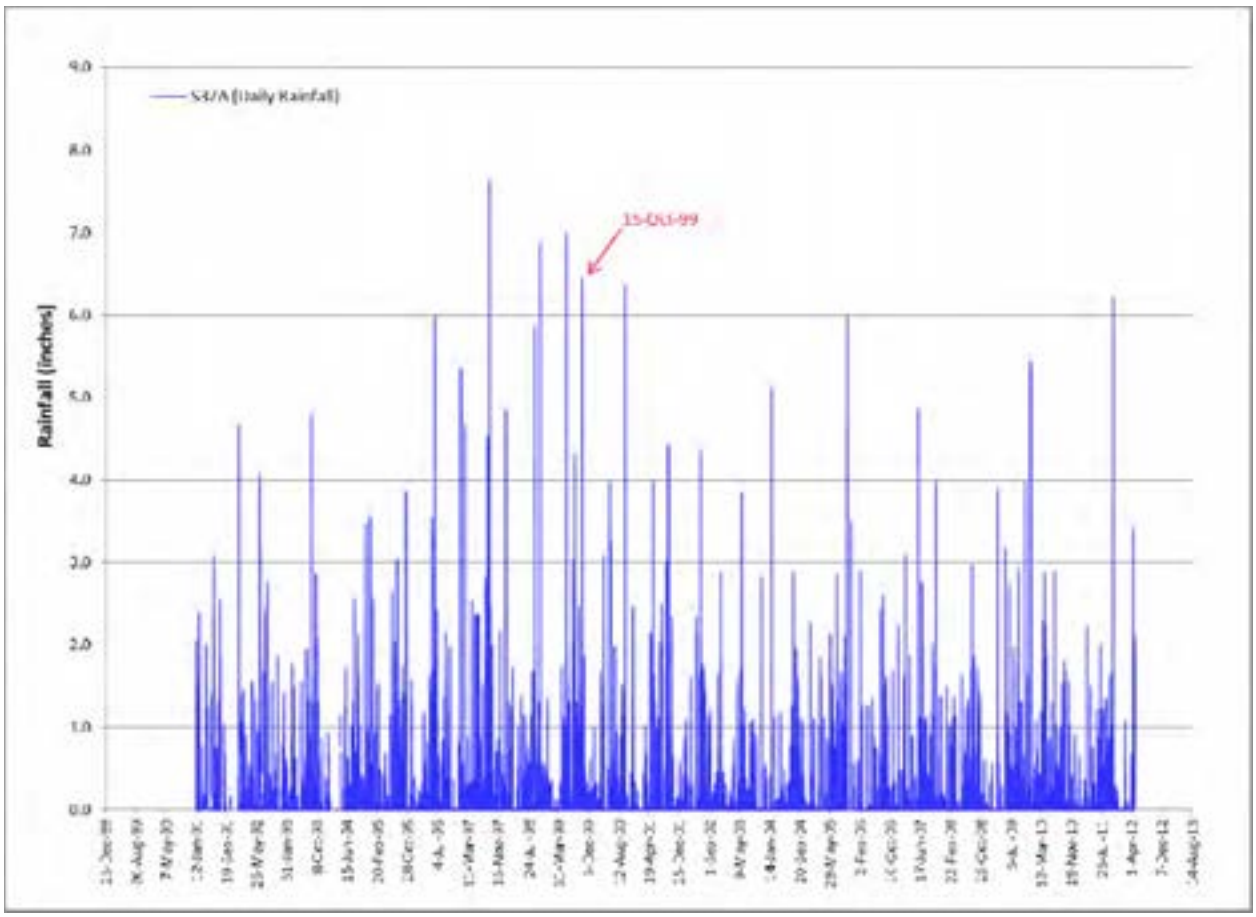


Figure 4-5 Daily Rainfall at SFWMD S37A Gauge

Considering each of the two days with significant amounts of rainfall independently and by determining the return period of each day of rainfall, this two-day rainfall event consists of one independent rainfall event with a 3-year 1-day return period followed by another event with a 2-year 1-day return period. As a three-day event of 12.88 inches of volume rainfall, this rainfall will correspond to an event with a 10-year return period.

Tidal Data

A timeseries for the tide levels within the study area during this storm event was derived from the recorded tide levels at the NOAA tidal gauges in Virginia Key and Lake Worth. The tidal data for a 15-minute interval during the period between October 1st, 1999 and October 21st, 1999 is displayed within Figure 4-6 below. This figure also displays the rainfall data recorded at SFWMD Station S37A over 15-minute intervals during this period along with a cumulative representation. According to this rainfall data, a cumulative rainfall of 16.8 inches had occurred by the peak of the storm on October 15th, 1999. The peak of the storm event occurred during the typical high perigean tides of October along with a significant storm surge of about 1-foot higher than typical high tides during this time period.

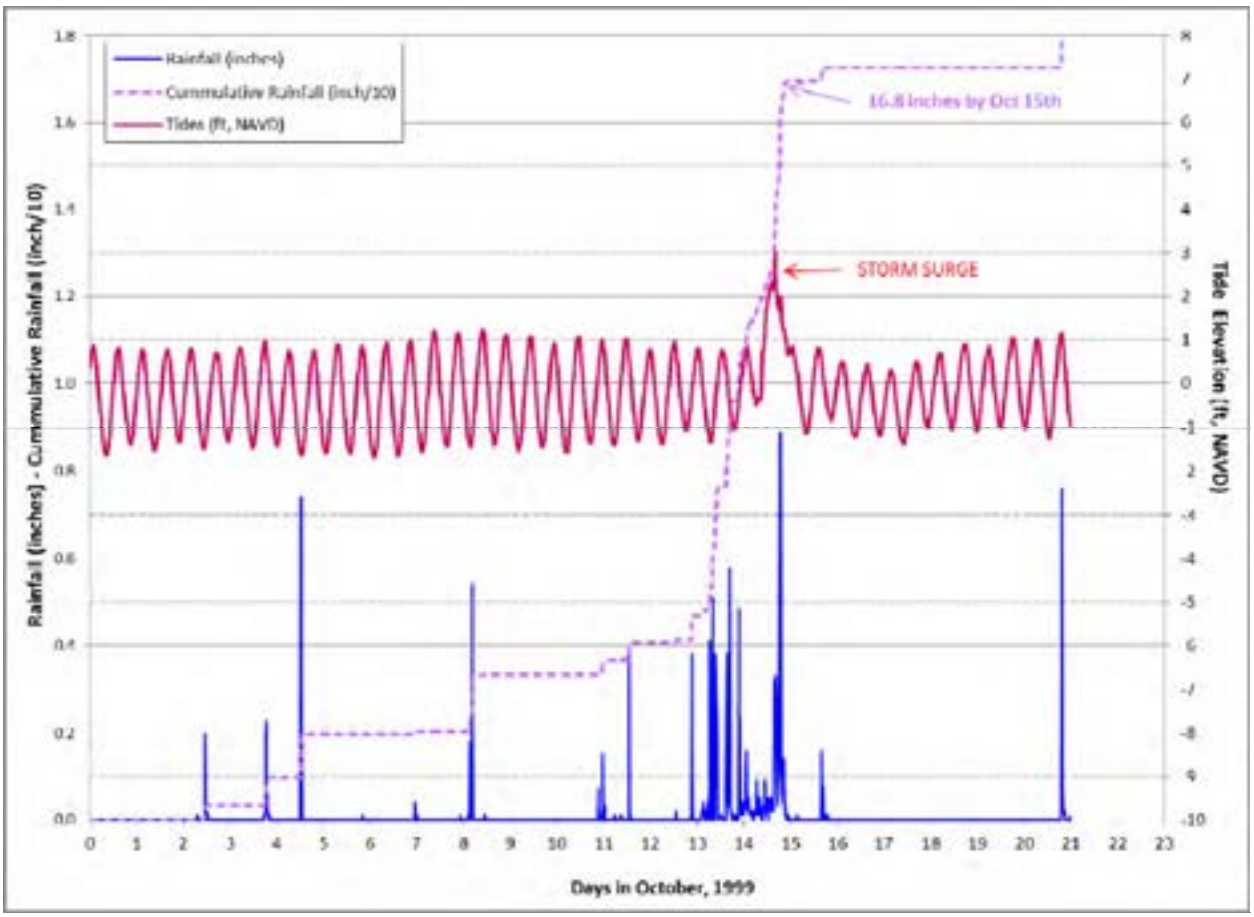


Figure 4-6 NOAA Tidal Data

High Water Marks

At the time of this evaluation, no information on records for depth of flooding throughout the study area were available from the City. Therefore, the stormwater model was not calibrated for observed roadway flooding. Due to the lack of high water marks throughout the City, the model calibration effort focused primarily on reproducing peak canal levels at the water control structures throughout the study area.

4.7.2 METHODOLOGY

The existing conditions stormwater model for the study area was calibrated for this historical rainfall event based on the measured tailwater elevations during this time period. During the model calibration,

the available tidal data for this time period was used to define the tailwater elevation for drainage basins which discharged into the Intracoastal Waterway or the connected tidally influenced canals. During the model calibration, the available canal level data was used to define the tailwater elevation for drainage basins which discharge into the SFWMD Canal system. Since the tailwater elevations within the receiving waterbodies impact the performance of the stormwater systems, the historical water level data must be incorporated into the stormwater model during the calibration stage. Due to the location of the receiving waterbodies along with the existing SFWMD water control structures within the study area, the model calibration effort was separated into multiple sections:

- Intracoastal Waterway: Based on the available tidal data during this time period, the stormwater model was calibrated for the eastern portions of the study area based on these historical tide measurements. The portion of study area not impacted by tidal levels within the Intracoastal Waterway is generally limited to areas north of Atlantic Boulevard and east of Dixie Highway. These areas discharge into the Intracoastal Waterway or other connected marine canals, which create a tidally influenced boundary condition within the stormwater model.
- SFWMD C-14 Canal: Based on the available data from the SFWMD water control structures CS-37A and CS-37B, the stormwater model was calibrated to account for the canal flow generated on the west portion of the C-14 Basin before it discharges through the CS-37B structure in the eastern portion of the study area.
- Old Pompano Canal: Based on the available data from the SFWMD water control structure G-57, the stormwater model was calibrated to match the measured water levels along the Old Pompano Canal, which drains the area north of Atlantic Boulevard and east of Powerline Road.

SFWMD C-14 Canal

The drainage basin which discharges into the SFWMD C-14 Canal was divided in two portions within the stormwater model. The portion of the SFWMD C-14 Canal Basin located east of the Florida Turnpike is modeled in detail since it is located within the study area. The portion of the SFWMD C-14 Canal Basin located west of the Florida Turnpike is located within adjacent municipalities, such as Coconut Creek, North Lauderdale, Margate, Tamarac, and Coral Springs. The water levels within the SFWMD C-14 Canal are controlled by Control Structure CS-37A and Control Structure CS-37B, which are both operated by the SFWMD. Control Structure CS-37B regulates the incoming flow from the western portion of the SFWMD C-14 Basin and from the Broward County Water Control District No. 3 (WCD No. 3) secondary canal, which discharges into the SFWMD C-14 Canal near the intersection of the Florida Turnpike and Atlantic Boulevard. The WCD No. 3 secondary canals drainage area is located west of Powerline Road between Atlantic Boulevard and Sample Road. Flow through these canals is controlled by three control structures (CS-40, CS-25R and CS-12), which are all included in the stormwater model. Control Structure CS-12 is the last water control structure prior to discharging into the SFWMD C-14 Canal. Control Structure CS-37A regulates the water levels downstream of Control Structure CS-37B between I-95 and the Florida Turnpike along Atlantic Boulevard. These control structures basically control the amount of stormwater runoff being discharged from the existing drainage network into the canal segments between CS-37A and CS-37B.

The SFWMD C-14 Canal segment located between CS-37B and CS-37A receives stormwater runoff from both highly pervious and impervious areas. The pervious areas correspond to the Palm Aire Country Club properties The impervious areas correspond to the commercial development areas located north of the WCD No.4 Canal and the Fort Lauderdale Executive Airport Airpark located south of the WCD No. 4 Canal. The west portion of the SFWMD C-14 Canal Basin was delineated based on the published SFWMD basin delineation and corresponded to an area of about 14,000 acres. As mentioned in previous sections of this report, this watershed was included in the stormwater model to provide adequate inflow from generated runoff and groundwater interflow during any given storm event.

The goal of the model calibration effort is to maintain reliable water levels upstream of CS-37B and CS-37A within the stormwater model. The parameters adjusted along the SFMWD C-14 Canal in order to match the observed stages upstream and downstream of CS-37A and CS-37B during the historical storm event period are summarized below:

- A significant amount of soil storage was added to the west portion of the SFWMD C-14 Canal watershed to account for the groundwater interflow and baseflow incoming from the Everglades Water Conservation Area, which is expected to exert an influence on the surface water near the coastal areas before discharging into the Atlantic Ocean. The flows within the SFWMD C-14 Canal were very sensitive to the changes in soil storage value assigned to this watershed, which typically contributed to elongate the peak in the hydrograph creating a wider declining limb.
- The width of the west portion of the SFWMD C-14 Canal watershed was also adjusted to accommodate the peak runoff contribution of this area in relation to the measured water level data at CS37B.
- Additional stormwater flow is also generated from the southern portion of the SFWMD C-14 Basin located between Andrews Avenue and US-1. Stormwater runoff from this area actually discharges through several existing canals into the SFWMD C-14 canal. However, the stormwater model defined the discharge to occur at a single location upstream of structure CS37A. The width of this watershed was adjusted during the calibration process in order to match stages observed at structure CS37A.
- The operational criteria of Control Structures CS37A and CS37B were modified to reflect measured stages around the most intense period of the storm event from October 14th, 1999 to October 15th, 1999. Due to the high intensity of this storm event, the operating criteria at the control structures were adjusted to evacuate as much water as possible throughout the watershed before and after the peak of the storm event.
- ♦ The Manning's roughness coefficient (N) within the canal cross section was adjusted within the stormwater model during the calibration process. In general, higher roughness coefficients were assigned to the most downstream segments of the SFWMD C-14 Canal to account for various canal obstructions.

<u>Old Pompano Canal (SFWMD G-16 Canal)</u>

The SFWMD G-16 Canal receives stormwater runoff from areas south of Atlantic Boulevard prior to the G-65 control structure. The tributary area for the SFWMD G-16 Canal consists of chiefly commercial land use and mixed land use located adjacent to Atlantic Boulevard. The SFWMD G-16 Canal Basin receives stormwater runoff from areas north of Atlantic Boulevard, including significant flows from the FDOT drainage system along I-95. The FDOT drainage system along I-95 starts next to Morningstar Lake north of Copans Road before discharging into the SFWMD G-16 Canal. The FDOT drainage system along I-95 receives stormwater runoff from City stormwater systems. This existing drainage ditch along the east side of I-95 receives stormwater discharge from the City's system serving the residential and commercial areas located between Dixie Highway and I-95. The drainage canal along I-95 also receives stormwater runoff from an area located north of Martin Luther King Boulevard and west of I-95.

Water control structure G-65 connects the SFWMD C-14 Canal with the SFWMD G-16 Canal with an adjustable weir structure connected to a 5 foot x 5 foot box culvert. However, this structure was disconnected approximately 20 years ago but was recently reactivated to address water quality concerns outlined in the Nutrient TMDL for this waterbody (WBID 3271, EPA, 2007). In order to replicate the flow conditions existing during the 1999 storm event period, the G-65 control structure was not included in the calibration model since it was not active at the time of the storm event. However, control structure G-65 is included within the existing conditions stormwater model. In order to match the observed canal

levels upstream and downstream of Control Structure G57 during the historical storm event period, the parameters adjusted along the SFMWD G-16 Canal are summarized below:

- The operational criteria of Control Structure G57 was adjusted to reflect measured stages around the most intense period of the storm event from October 14th, 1999 to October 15th, 1999. The water control structure provides a larger discharge capacity than the tributary area could ever generate for a given storm. This control gate was not been modified after the rerouting of the SFWMD C-14 Canal through control structures CS-37B and CS-37A.
- The Manning's roughness coefficient (N) was adjusted within the stormwater model during the calibration process for the canal cross sections located upstream from the control structure and for the receiving canal waterbody.

4.7.3 ANALYSIS

CMA conducted a comparison between the measured water levels and the water levels predicted by the stormwater model for headwater (upstream) and tailwater (downstream) conditions for the canal control structures during the storm event from October 1st, 1999 through October 21st, 1999. The objective of this calibration effort was to simulate the peak stages observed during the most intense period of this storm event and to follow the trend of water level fluctuation for the duration of the storm. The water elevations predicted by the stormwater model before and after the peak stage on October 15th, 1999 are within an acceptable range throughout the duration of the storm. The recorded timeseries depicts more variable water level elevations in both upstream and downstream of the existing control structures, due to the following factors:

- Contribution of groundwater baseflow which is not directly modeled by SWMM
- Influence of tidal variations caused by storm surges

The capabilities of SWMM are limited to model interflow, which is the groundwater response to a particular or a series of rainfall events. However, baseflow is a more temporarily retarded and a more extensive type of contribution spatially that only models with comprehensive groundwater modeling capabilities will be able to simulate. The next generation of SWMM under development by EPA will include such baseflow capabilities. This stormwater model will be ready to be upgraded to that level of integrated watershed modeling when it becomes available from EPA. The major source of this baseflow into the study area is the Everglades Water Conservation area located in west Broward County. The Everglades Water Conservation area exerts a significant pressure over the coastline's surficial aquifer and to the surface waters.

The peak stages estimated by the calibrated stormwater model at all control structures along SFWMD C14 Canal and SFWMD G16 Canal are considered satisfactory for the calibration effort. The stormwater model parameters adjusted during the calibration process chiefly consisted of Manning's roughness coefficients, soil storage, watershed widths, saturated hydraulic conductivity, and water control structures operating criteria. By controlling and adjusting hydrologic and hydraulic parameters, the calibrated stormwater model accurately predicted the water level trends before and after the storm event and matched the peak stages at each of the six calibration points of CS-37A (upstream, downstream), CS-37B (upstream, downstream), and G57 (upstream, downstream). The measured water levels during the storm event period from October 1st, 1999 through October 21st, 1999 and the water levels predicted by the calibrated stormwater model are displayed within the various figures below for comparison purposes. The water levels immediately upstream and downstream of each canal control structure along the SFWMD C14 Canal and the SFWMD G16 Canal are summarized within the following figures below:

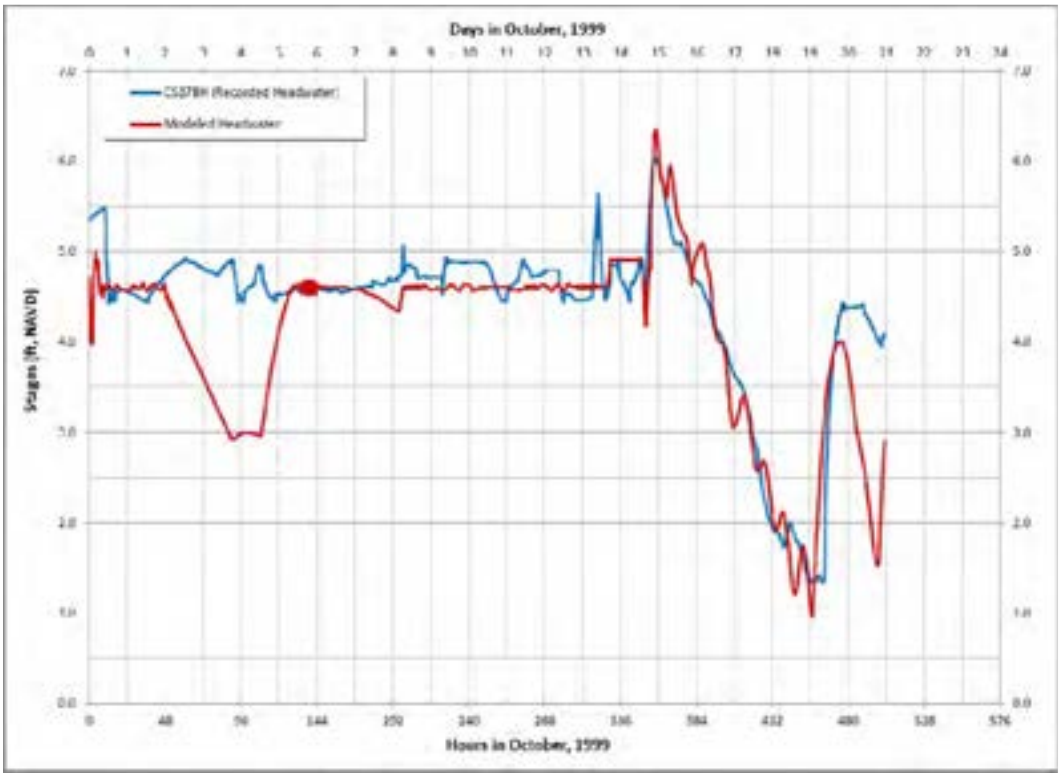


Figure 4-7 SFWMD C14 Canal Level Data - Upstream of Control Structure CS37B

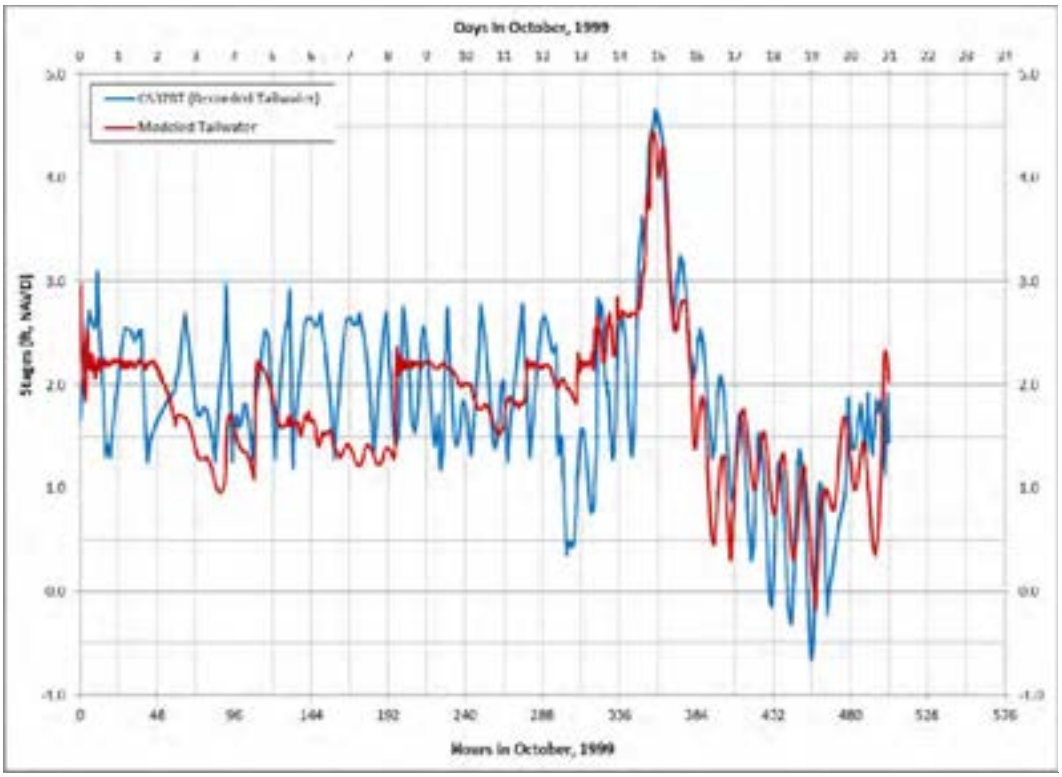


Figure 4-8 SFWMD C14 Canal Level Data - Downstream of Control Structure CS37B

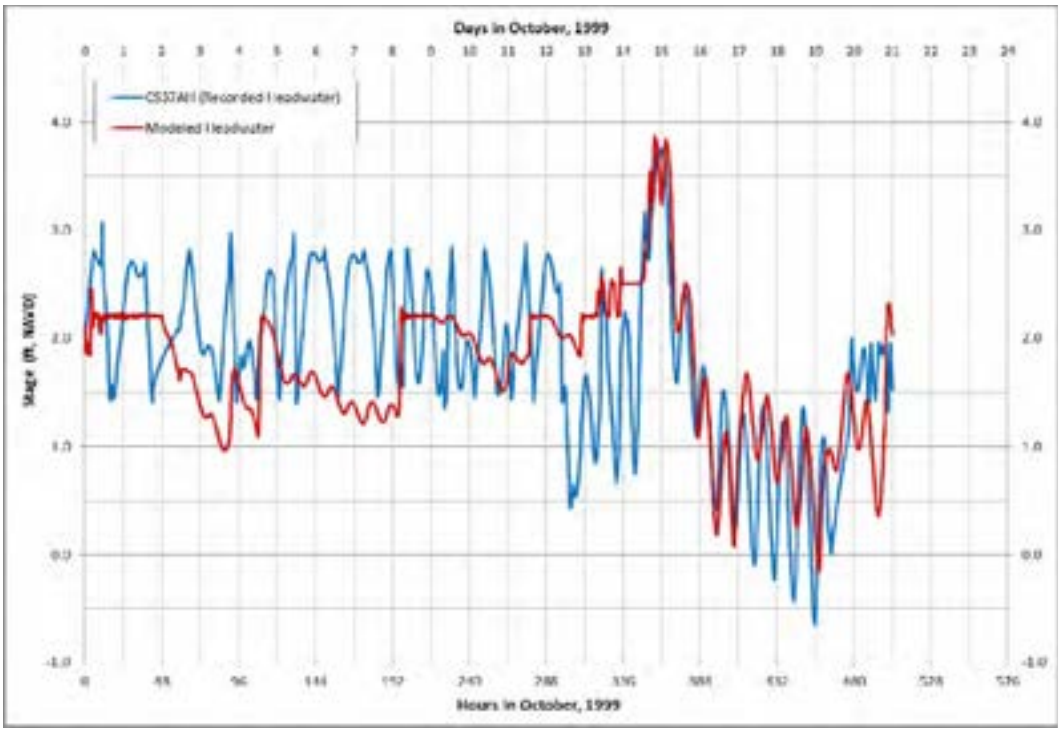


Figure 4-9 SFWMD C14 Canal Level Data - Upstream of Control Structure CS37A

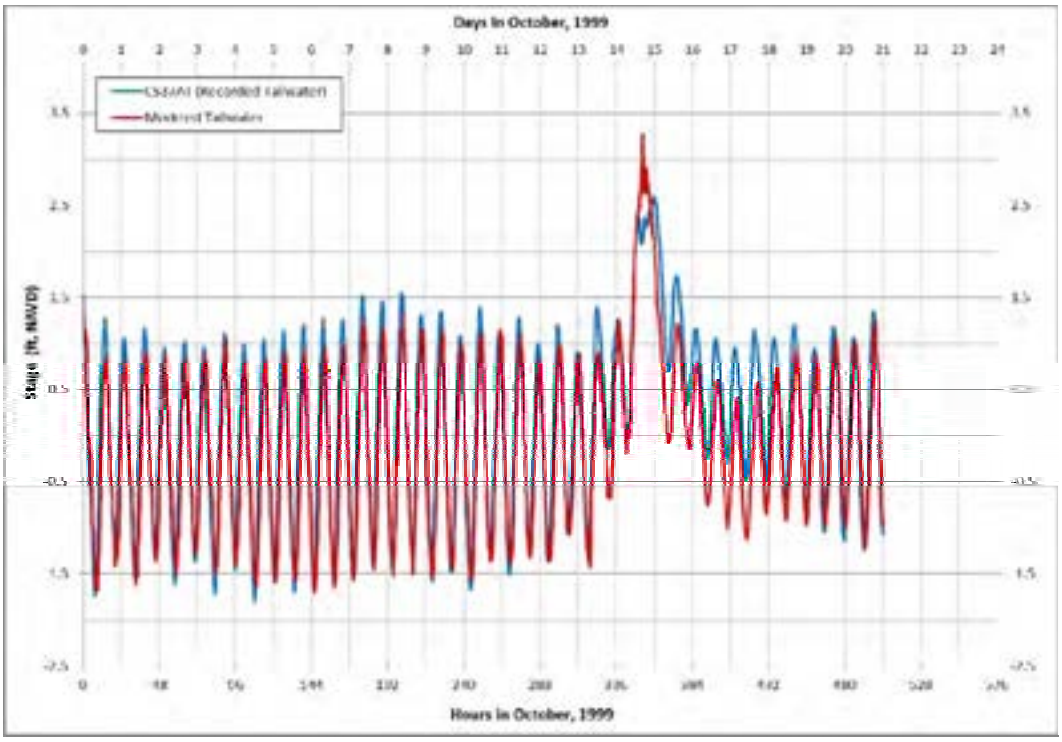


Figure 4-10 SFWMD C14 Canal Level Data - Downstream of Control Structure CS37A

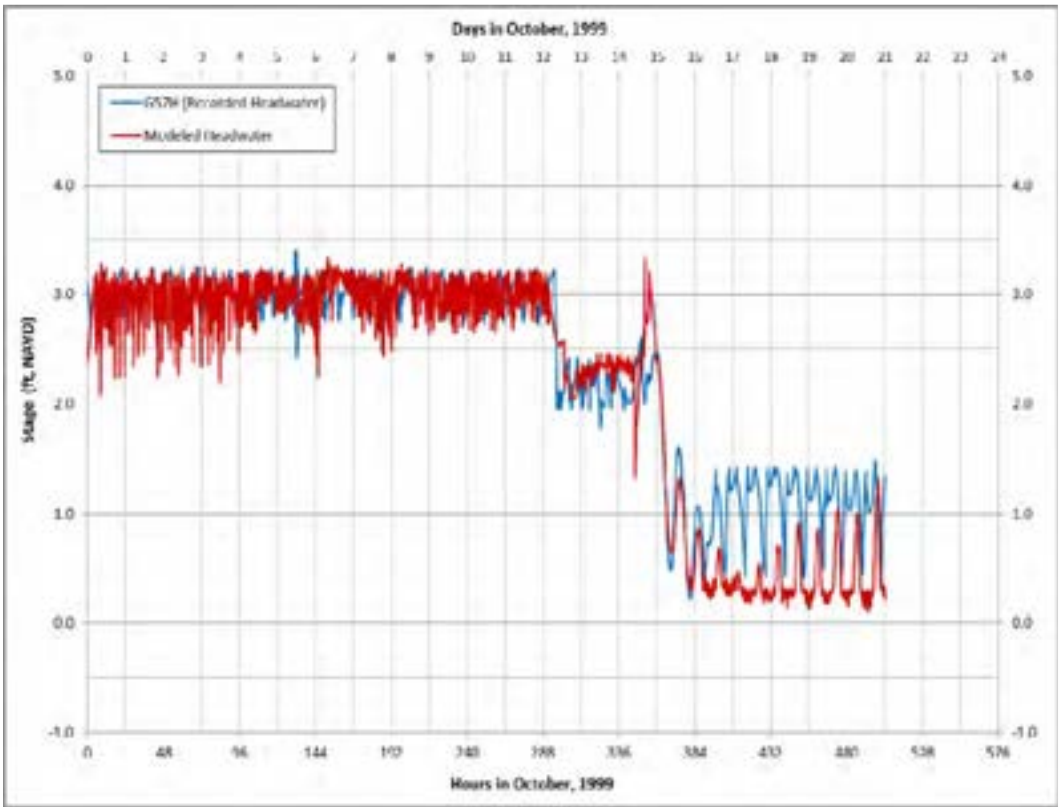


Figure 4-11 SFWMD G16 Canal Level Data - Upstream of Control Structure G57

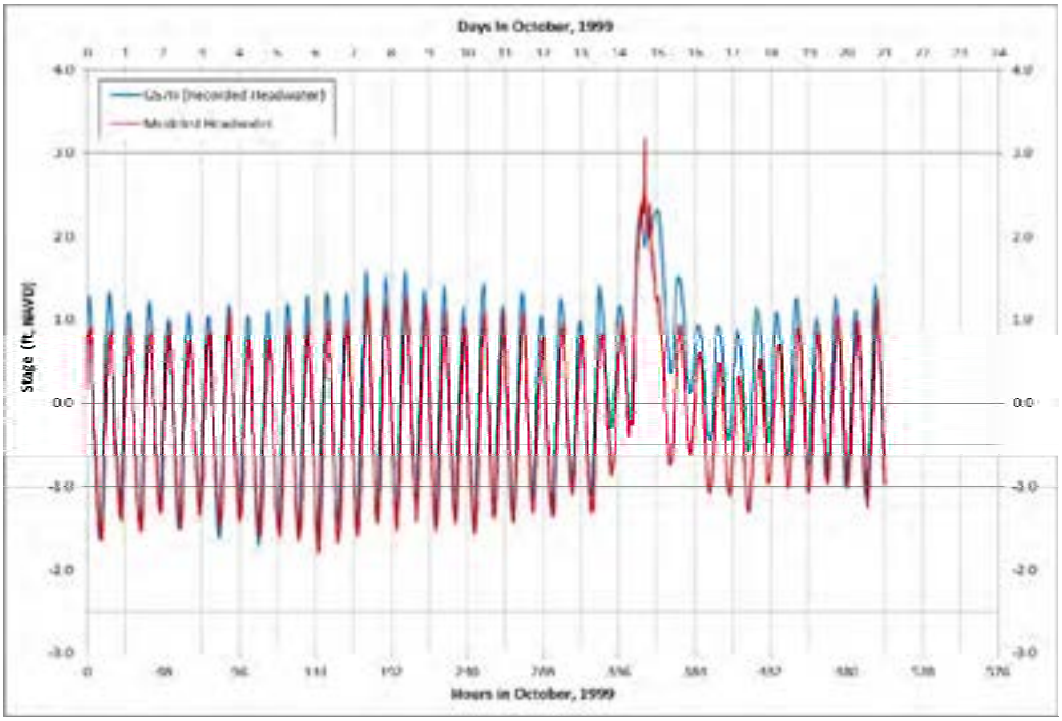


Figure 4-12 SFWMD G16 Canal Level Data - Downstream of Control Structure G57

4.8 EXISTING CONDITIONS MODEL RESULTS

The stormwater model used the SWMM software to conduct simulations of the routing of stormwater runoff between basins via the link connections, which mimics the performance of the stormwater management system within the City of Pompano Beach. The SWMM stormwater model of the existing conditions was run under four design storm scenarios (5-year/24-hour; 10year/24-hour; 25-year/72-hour; 100-year/72-hour). The results from the stormwater model as generated by the SWMM software for all scenarios are included within Appendix A-7 – SWMM Existing Conditions Stormwater Model Results. The key results of the stormwater model are the peak flood elevation and the peak discharge rate via existing outfalls into the receiving water bodies. These model results will be used to identify potential flooding "problem areas" within the City.

4.8.1 PEAK FLOOD STAGE

The SWMM existing conditions model simulated the peak flood stage within every sub-basin in the study area during four design storms (5-year/24-hour; 10year/24-hour; 25-year/72-hour; 100-year/72-hour). The peak flood stage for all sub-basins during each design storm is summarized in tabular format in Appendix A-7 – SWMM Existing Conditions Stormwater Model Results to allow for comparison. The peak flood stage can be used to identify the potential flooding areas through a comparison with the ground surface elevations within each sub-basin. The purpose of this comparison is to identify areas of the City which may require future stormwater improvements in order to meet level of service criteria for flood protection.

The City of Pompano Beach has a level of service criteria for providing flood protection to new public roadways. During the design stage, the stormwater system for a public roadway must be designed to ensure the roadway crown is not flooded during the 10-year storm event and travel lanes are not flooded during the 5-year storm event. The results of the existing conditions stormwater model can used to identify City roadways which do not meet the level of service criteria under the existing conditions. The maximum flood stage in each sub-basin can be compared with the DEM to verify it did not exceed the lowest pavement elevation for any roadways located in the sub-basin and to ensure ponding did not encroach across any roadway. Basically, the low point of the stormwater management system along the roadways within each sub-basin was identified according to the DEM. CMA prepared Figure 4-13 Potential Flooding Maps, which are included within this section. These Potential Flooding Maps display areas in all sub-basins with ground surface elevations lower than the peak flood stage during the 5-year storm event.

The City of Pompano Beach has a level of service criteria for providing flood protection to new building structures. During the design stage, the stormwater system for a site development must be designed to ensure the finish floor elevation of the new building structure is not flooded during the 100-year storm event. The results of the existing conditions stormwater model can be used to identify existing structures which do not meet the level of service criteria under the existing conditions. The maximum flood stage in each sub-basin can be compared with the DEM to verify it did not exceed the approximate finish floor elevation for any existing building structures located in the sub-basin. Although the actual finish floor elevations for existing building structures located within the City are not currently available, assumptions can be made to estimate the lowest finish floor elevation within each sub-basin using the LiDAR elevation data. The lowest finish floor elevation within each sub-basin can be assumed to be 6 inches above the ground surface elevation of the lowest private property area within the sub-basin.

4.8.2 PEAK DISCHARGE

Typically, the maximum allowable discharge via an existing outfall cannot exceed the peak discharge rate allowed under the original stormwater permit requirements. If the existing outfalls do not have a

maximum discharge assigned by existing permits, the regulatory agencies will require "pre-development" versus "post-development" discharge analysis to ensure the stormwater discharge into adjacent surface waters does not increase after the proposed construction. During the permitting stage for any future infrastructure improvements, the City of Pompano Beach will have to ensure that the peak discharge from the outfalls does not exceed the peak discharge under the 25-year / 72-hour design storm established by the existing conditions stormwater model. The summary of the peak discharge per outfall for the 25-year / 72-hour design storm is included for the existing conditions in Appendix A-7 – SWMM Existing Conditions Stormwater Model Results.

4.9 WATER QUALITY ANALYSIS

CMA retained with AMEC Environment & Infrastructure, Inc. to estimate the stormwater pollutant loads and fecal coliform loads generated from the City of Pompano Beach's drainage basins. The complete reports from AMEC are enclosed within this report in Appendix A-8 –Stormwater Pollutant Load Estimates Report and Appendix A-9 – Fecal Coliform Impairments Report. The summary of this water quality analysis for the City of Pompano Beach is outlined within the following sections.

4.9.1 POLLUTANT LOADING ESTIMATES

AMEC used a land use methodology for estimating the pollutant loads generated from each sub-basin within the study area. The land use methodology was based on FDEP's Evaluation of Current Stormwater Design Criteria in the State of Florida (June 2007, Harper & Baker, ERD) and Water Management Districts' Environmental Resource Permit Stormwater Quality Applicant's Handbook (Draft, March 2010). The land use was categorized with the Florida Land Use Code and Classification System (FLUCCS). Each category of land use was assigned a corresponding directly connected impervious area (DCIA) value based on the USDA's Technical Release 55 (TR-55). The curve number (CN) for each sub-basin was interpolated using TR-55 based on land use category(s) and the hydrologic soil group(s) found within the sub-basin. Each combination of sub-basin, land use, and hydrologic soil group was assigned a non-DCIA CN and DCIA value within the calculations. The runoff coefficients were calculated using a table from "Mean Annual Runoff Coefficients as a Function of DCIA Percentage and Non-DCIA Curve Number" (Harper 2007).

The event mean concentration (EMC) parameters for each pollutant were obtained from the Environmental Resource Permit Stormwater Quality Applicant's Handbook (FDEP and Water Management Districts, March 2010 Draft) and the Evaluation of Current Stormwater Design Criteria within the State of Florida (Harper, Baker, June 2007). The EMC parameters which were used for the pollutant load calculations are displayed within Table 4.4 below. The runoff coefficients along with the EMC parameters were used to estimate the pollutant loading for each sub-basin, land use, soil group combination. These values were summed within each sub-basin to provide the total pollutant loading generated by each sub-basin.

Table 4.4 – Event Mean Concentrations (mg/L)							
Land Use	**TN**	**TP**	**BOD**	**TSS**	**Cu**	**Pb**	**Zn**
Low-Density Residential	1.5	0.18	4.7	23	0.008	0.002	0.031
Single-Family	1.85	0.31	7.9	37.5	0.016	0.004	0.062
Multi-Family	1.91	0.48	11.3	77.8	0.009	0.006	0.086
Low-Intensity Commercial	0.93	0.16	7.7	57.5	0.018	0.005	0.094
High-Intensity Commercial	2.48	0.23	11.3	69.7	0.015		0.16
Light Industrial	1.14	0.23	7.6	60	0.003	0.002	0.057
Mining/Extractive	1.18	0.15	7.6	60	0.003	0.002	0.057
Open Land	1.15	0.055	1.4	8.4	0	0	0
Row Crops	2.47	0.51		19.8	0.022	0.004	0.03
Feeding Operations	2.48	0.7	5.1	94.3	0	0	0
General Agriculture	2.42	0.46	3.8	43.2	0.013	0.003	0.021
Undeveloped/Rangeland/Forest	1.15	0.055	1.4	8.4	0	0	0
Waterways	0	0	0	0	0	0	0
Open Water / Lake	0	0	0	0	0	0	0
Wetland	1.01	0.09	2.63	11.2	0.001	0.001	0.006
Highway	1.37	0.17	5.2	37.3	0.032	0.011	0.126
Utilities	1.37	0.17	5.2	37.3	0.032	0.011	0.126

4.9.2 BEST MANAGEMENT PRACTICES (BMP)

Numerous regulatory programs have been dedicated to the issue of developing and refining BMPs aimed at minimizing the water quality impacts of urban stormwater runoff on surface water bodies. As the NPDES Program and TMDL Program continue to place limitations on the pollutants within stormwater runoff, the use of BMPs in reducing pollutants will become even more important to the City. This section provides an overview of typical BMPs, which apply to specific conditions within the City of Pompano Beach. BMPs can be defined as either structural (point source) or non-structural (programmatic). These BMPs can be implemented independently or in combination by the City of Pompano Beach to meet certain water quality objectives. The goal of BMPs is typically aimed at the prevention (source control) or reduction/treatment of pollutants within stormwater runoff. The selection of the BMP is typically made based upon numerous factors including:

- Physical conditions
- Pollutant characteristics
- Water quality goals
- Appropriate/available technology
- Economics
- Public perceptions/input

Broward County has adopted a BMP Manual which utilizes practices advocated by FDEP and Broward County EPGM. The City of Pompano Beach has already implemented many of these BMPs for stormwater management and should continue to use in the future as part of the NPDES Permit Compliance. A summary of the various structural and non-structural BMPs are listed below.

Non-Structural BMPs

Non-structural BMPs are often referred to as source controls because they are aimed at eliminating or reducing potential pollutants at the source. These non-structural BMPs are preemptive strategies while structural BMPs are generally aimed at diverting or treating the problems (pollutants) once they have already been introduced into a surface water body. These non-structural BMPs are typically more cost effective than structural BMPs. These non-structural BMPs can be considered good housekeeping procedures, which are a vital element in reducing urban stormwater pollution. Some general examples of non-structural BMPs are listed below:

- Public education, outreach and involvement
- Conservation and land management
- Planning and regulatory controls
- Maintenance and good housekeeping

The City currently has implemented various non-structural BMPs and should continue to utilize these non-structural BMPs in its effort to improve the quality of stormwater discharges. The various non-structural activities which the City currently implements or could consider adding for the purpose of enhancing the water quality of stormwater runoff are summarized below.

- Storm Drainage Stenciling – The City has instituted a program of stenciling storm drains with notification of the importance of not discharging harmful materials to the drains.
- Erosion and Sediment Control Training – The City has instituted a program of periodic training for appropriate City staff to help ensure proper erosion and sediment control is implemented during all construction related activities throughout the City.
- Vacuum Sweeping Operations – As part of the operations and maintenance procedures, the City should continue to routinely vacuum sweep asphalt parking areas for all City owned facilities.
- Reclaimed Water Limitations – The City currently prohibits the application of reclaimed water to impervious surfaces that allow flow into surface water bodies.
- Fertilizer Management – The City currently trains appropriate staff members on proper fertilizer and pesticide application, storage, and training requirements. The City is currently in the process of adopting a fertilizer management ordinance for the purpose of limiting the types of fertilizer applications throughout the City.
- Public Education Web Links – The City should utilize its website to provide links to other educational information for the general public related to stormwater management and pollution prevention. For example, FDEP, SFWMD, and Broward County have specific web pages targeted at activities for individual residents and specific interest groups. There is specific BMP/Pollution Prevention guidance targeted for boatyards/marine facilities, vehicle maintenance facilities, dry cleaners, printers, solid waste, and other industries from SFWMD and Broward County. In addition, the SFWMD has developed a more general "Turf and Landscape – Best Management Practices for the C-11 West Canal Basin" web page. This is a more general document which addresses vegetation management, fertilizer management, stormwater management, irrigation management, and general good housekeeping.
- Florida Yards and Neighborhoods (FYN) – This program was developed by the University of Florida Institute of Food and Agricultural Sciences Extension to help address the issues of pollution prevention and loss of the natural environment and habitat concurrently. The program offers unique community educational and outreach activities, and is available in Broward County through the Broward County Agricultural and Extension Education Division.

- Land Development Manual – The City can create a standard manual which defines the various procedures which should be required during the development of all properties within the City. The Land Development Manual should be used to define any requirements related to erosion and sediment controls at construction sites, maintaining minimum on-site storage for stormwater runoff, limiting any obstruction to existing natural or manmade flow patterns of stormwater runoff, prohibiting any increase of stormwater runoff offsite, limiting amount of new impervious surfaces, promoting materials which promote the infiltration of stormwater runoff into the ground surface, encouraging the reuse of stormwater runoff for the irrigation of landscaping, and defining appropriate landscaping which limit irrigation and fertilizer needs.
- Golf Course Management – Daily operations and maintenance at golf courses are critical to minimizing the potential for stormwater pollution for the facilities. While there is not currently an agency which regulates/monitors these activities, there are BMPs and educational guidance available in this area. FDEP published a document entitled "BMPs for Golf Course Maintenance Departments" which addresses numerous areas including pesticide, herbicide, and fertilizer handling and application. The document is aimed at preventing pollution of surface and ground waters.
- Greenspace Management Program – The City should consider creating a Greenspace Management Program for providing natural drainage, clean water, wildlife habitat, and passive recreational activities within the City.

There are numerous other BMPs both structural and non-structural (programmatic) which could be utilized by the City. An excellent resource relative to this issue is the Non-Point Source (NPS) Stormwater Toolkit, which is a large watershed-based education package. The toolkit is intended to be available free of charge to municipal governments.

Structural BMPs

Structural BMPs are usually associated with the construction of some type of drainage system or facility for controlling/treating pollution associated with stormwater runoff. Structural BMPs may be implemented anywhere between the pollutants' point of introduction to the stormwater management system and their point of discharge into the receiving water of concern. Historically in Florida, structural BMPs have generally involved some type of retention or detention strategy (either online or offline). While these are still the most common approaches, other structural BMPs are under development to work in combination with these strategies or as alternatives in special situations as often encountered in retrofit type situations). The following discussion of some typical structural BMPs utilized in South Florida includes:

- Retention systems (wet and dry)
- Detention systems (wet and dry)
- Grassed swales
- Exfiltration trenches
- Water quality inlets/separation devices
- Chemical treatment

These systems typical retain stormwater runoff on-site up to a specified design volume with no means of immediate discharge. Volume recovery is primarily by infiltration into the ground surface or subsurface soils, which requires the presence of well drained soils. Because there is no discharge from these systems for the design storm event, they are very effective at removing pollutants for rainfall events equal to or less than the design event. These systems provide the added benefit of maximizing groundwater recharge. Some retention systems are intended to provide water quality treatment and some design storm event discharge attenuation. These systems have some overflow mechanism to a downstream system or

receiving waters. Other retention systems are designed to provide total on site retention up to the maximum design storm event. These systems are typically used only when there is no means of positive discharge for bleed down and/or overflow and require a significant amount of storage capacity and excellent soils for volume recovery.

Retention and detention systems come in online and offline arrangements. Online systems are placed directly in the primary conveyance path of the overall surface water management/drainage system. This type of system has a major drawback in that for events exceeding the design storm rainfall, flushing of accumulated pollutants into downstream systems and/or receiving waters may occur. Offline systems are typically preferable in that they divert the first pollutant laden flush of runoff offline from the primary conveyance course. If the design event rainfall is exceeded, the additional runoff subsequent to the first flush bypasses the treatment area, which avoids or minimizes the flushing effect. The typical advantages, disadvantages, and requirements for retention systems are defined in the table below:

Table 4.5 – Summary for Retention Systems

Advantages	• Reduces pollutant loads in stormwater runoff • Maximizes groundwater recharge • Reduces runoff volume/rates • Long service life • Low maintenance requirements
Disadvantages	• Potential standing water which has safety/aesthetic concerns • Reduction of infiltration rates over time • Vulnerable to seasonal rainy season • Space requirements
Requirements	• Significant available space for adequate storage volume • Soil types with high permeability • Minimum depth to water table

Unlike retention systems, detention systems provide a mechanism for bleeding off the design storage volume through positive discharge. The system functions to detain or slow down the discharge of runoff so that pollutants may be removed primarily through the settlement of suspended solids. Wet detention ponds with vegetated littoral zones may also remove dissolved pollutants, including nutrients, through biological activity. Because they do allow a limited discharge, detention systems generally are less effective than retention systems at pollutant removal. However, the effectiveness of both types of systems is heavily dependent on design retention/detention volumes and detention times, system maintenance, system configuration, and other factors. The typical advantages, disadvantages, and requirements for detention systems are defined in table above.

Dry detention ponds are designed to stay dry between storm events. The dry detention systems are effective at removing suspended solids and associated pollutants, but are typically ineffective at removing soluble pollutants, such as nutrients. Care should be taken in the design of these facilities to ensure that they are able to dry out between rainfall events to prevent the introduction of soggy nuisance areas, which are difficult to maintain. As such, they should be designed so that the control elevation (wet season water table) is at least one foot below the pond bottom.

Wet detention ponds are designed to maintain a permanent pool of water and often include a shallow littoral shelf with aquatic vegetation. Like dry detention ponds, the wet detention ponds are intended to slow down the discharge of runoff, to allow the removal of pollutants, and to attenuate the peak rate of runoff for flood protection purposes. Unlike dry detention areas, wet detention areas, which can include constructed wetlands, are effective at removing nutrients through biological processes. These systems are

more forgiving from a design standpoint since they do not require as precise a determination relative to matters such as soil conditions and water table depth. By their nature, wet detention systems also introduce a greater element of concern relative to safety, requiring special attention to side slope design and sometimes access control.

Table 4.6 – Summary for Detention Systems	
Advantages	• Moderately effective at reducing pollutant load • Not dependent on soil types with high permeability • More reliable volume recovery • Requires less storage volume than retention systems • Relatively long service life • Relatively low maintenance (except constructed wetlands) • Creation of wetland habitat • Applicable in areas with higher water table
Disadvantages	• Generally not as effective at pollutant removal as retention • Ineffective at removing soluble nutrients (unless wet pond with vegetation) • Safety issues related to permanent water pool • Space requirements
Requirements	• Wet ponds require vegetation harvesting for ultimate nutrient removal • Constructed wetlands require removal of nuisance species

Grassed swales are shallow, vegetated trenches, ditches, or depressions which can be used for water quality treatment and/or conveyance of stormwater runoff. Swale areas have very mild side slopes and longitudinal slopes which are dependent upon whether their purpose is primarily for conveyance or treatment. Because they have limited storage capabilities, their use is typically limited to low density areas and fairly small drainage areas per unit length of swale. Their effectiveness in reducing pollutants is limited, especially if they are used for conveyance. The use of raised inlets and check dams in swales may help to slow flow velocities and improve treatment capabilities. They are common in residential areas because of their low capital costs, limited space requirements within public right-of-way, and relatively low maintenance. The typical advantages, disadvantages, and requirements for swale systems are defined in the table below:

Table 4.7 – Summary for Grassed Swales	
Advantages	• Low capital costs • Additional green space and groundwater recharge area • Typically maintained by adjacent property owner • Linear configuration fits into limited spaces • Attenuation of stormwater runoff
Disadvantages	• Objections from some residents • Standing water during extended wet periods • Limited pollutant reduction capabilities • Alteration by adjacent property owners
Requirements	• Soils types with high permeability • Moderately deep water table

Exfiltration trenches provide an alternative means for the provision of retention, typically for the purpose of providing water quality treatment. An exfiltration trench is basically a perforated pipe placed in a trench backfilled with course aggregate. Runoff is exfiltrated out of the pipe, through the trench, and then recharges the groundwater. Exfiltration trenches are most effective in areas with highly permeable soils

and a fairly deep water table. They are also more effective, and longer lasting, if they are equipped with upstream devices for removing some of the suspended solids and floating debris associated with the stormwater runoff. Otherwise, these facilities are highly susceptible to clogging and will lose effectiveness. Exfiltration trenches are typically intended for the treatment of first flush runoff only. Extrapolating their use for flood control storage should only be considered in special situations, and the ability of the facilities to perform in this capacity must be well carefully assessed and documented. Exfiltration trenches are extremely popular in commercial and high density residential applications within South Florida because they can be placed under the pavement, preserving valuable developable spaces. Use of this BMP should consider long term maintenance, reliability, and the implications of diminishing effectiveness. When designed and operating properly, exfiltration trenches are effective at reducing pollutant loads similar to other forms of retention. The typical advantages, disadvantages, and requirements for exfiltration systems are defined in the table below:

Table 4.8 – Summary for Exfiltration Trenches	
Advantages	• Effective at pollutant load reduction • Recharge groundwater • Preservation of land for other uses • Easily incorporated for retrofit situations • Typical method throughout South Florida region
Disadvantages	• Limited longevity • High maintenance requirements • Subject to failure due to plugging
Requirements	• Permeable subsurface soils • Deep water table

As attention to urban stormwater management has increased, new technologies and associated BMPs have been developed for water quality structures and separation devices. Some examples include baffle boxes, oil/grit separators, sediment sumps, skimmer devices, and various other devices which utilize vortex action and hydrocarbon filters. The majority of these devices operate on the basis of separating sediments and floating debris from the stormwater runoff. These water quality structures are primarily used to retrofit older systems without existing water quality treatment provisions to provide some net water quality improvement in locations where more traditional BMPs would not be feasible to incorporate due to space restrictions. These water quality structures may also be used in combination with other traditional BMPs in an overall “treatment train” configuration. Effectiveness of these devices is heavily dependent upon specific site and design conditions. These devices are generally applicable only for small drainage areas. Although these devices are not currently accepted by local regulatory agencies to meet minimum water quality requirements for new development, the regulatory agencies are currently testing these devices for future use in urban stormwater management. The typical advantages, disadvantages, and requirements for water quality structures are defined in the table below:

Table 4.9 – Summary for Water Quality Structures/Separation Devices	
Advantages	• Limited space requirements • Ideal for retrofit projects • Relatively long life • Unaffected by soil conditions
Disadvantages	• Serves only small drainage area • High maintenance • Not currently accepted by SFWMD or Broward County EPGM

Chemical treatment is not commonly used in South Florida for urban stormwater runoff treatment. Chemical treatment typically involves the injection of alum into the stormwater runoff stream, promoting coagulation and thus settling out of finer suspended materials. This approach is useful when the detention pond size/configuration does not allow sufficient settling naturally. Issues typically associated with chemical treatment include high capital cost of these facilities, additional maintenance costs and requirements, and the removal and disposal of settled materials. The typical advantages, disadvantages, and requirements for chemical treatment are defined in the table below:

Table 4.10 – Summary for Chemical Treatment	
Advantages	• Reduces detention time requirements • Works for relatively large areas • Unaffected by soil conditions
Disadvantages	• Solid waste disposal issues • High maintenance

4.9.3 EXISTING BMP LOAD REDUCTION ESTIMATES

The water quality treatment provided by the existing BMPs was taken into account when calculating the net pollutant loading per sub-basin. The SFWMD permits were reviewed to determine the type of stormwater treatment BMPs currently in place throughout the study area. The removal efficiency of each type of BMP was obtained from the Appendix F, Zone 5 of the "Water Management Districts' Environmental Resource Permit Stormwater Quality Applicant's Handbook" (Draft, March 2010) and "Evaluation of Current Stormwater Design Criteria within the State of Florida". The BMP removal efficiencies are summarized in Table 4.11 below.

Table 4.11 – BMP Removal Efficiency (%)

Parameter	**Wet Detention (14 days)**	**Dry Retention (1-inch)**	**Dry Retention (1.25-inch)**	**Exfiltration Trench**
TN	33.2%	**	**	50%
TP	61.76%	**	**	50%
BOD	74%	99%	99.9%	70%
TSS	77%	99%	99.9%	90%
Cu	69%	99%	99.9%	90%
Pb	84%	99%	99.9%	90%
Zn	85%	99%	99.9%	90%

The yield of the estimated pollutant load per acre was calculated for each sub-basin. This pollutant yield provides a means for comparing the pollutant loads among sub-basins throughout the study area. The pollutant yield for each major basin within the study area is summarized within Table 4.12 below. These values have been adjusted to include the BMPs. For comparison purposes, the range of pollutant yields for each sub-basin throughout the study area is graphically depicted within Figure 4-14 for total nitrogen (TN) and Figure 4-15 for total phosphorus (TP), which are included within this section.

Table 4.12 – Estimated Stormwater Pollutant Yields								
Major Basin Name	**Major Basin Area (acres)**	**Estimated Existing Annual Yield (lbs/ac)**						
		TN	**TP**	**BOD**	**TSS**	**Cu**	**Pb**	**Zn**
CE	3,810	4.535	0.850	20.968	135.836	0.036	0.012	0.191
CW	2,330	4.869	0.838	25.746	180.857	0.051	0.015	0.271
I95	321	7.595	0.954	28.531	204.635	0.169	0.058	0.670
NE	2,345	1.689	0.342	2.622	15.988	0.006	0.002	0.025
NW	3,534	4.532	0.826	20.341	144.723	0.035	0.013	0.200
SE	2,311	6.219	1.198	32.668	207.051	0.056	0.018	0.297
SW	3,848	5.353	1.093	28.158	205.349	0.047	0.017	0.274

4.9.4 FECAL COLIFORM ANALYSIS

The City of Pompano Beach currently has three water bodies identified by FDEP as containing elevated portions of fecal coliform bacteria. These levels are greater than the standard of 400 colony forming units (CFU) specified in Chapter 62-302.250 F.A.C., which is in place to comply with the Federal Clean Water Act. Hydrologic basins corresponding to these three water bodies are designated with a water body identification numbers (WBIDs) by FDEP. The WBIDs with elevated fecal coliform levels within the City of Pompano Beach are listed below:

- WBID 3271 (Pompano Canal)
- WBID 3270 (Pompano Canal)
- WBID 3274 (C-13 East/Middle River)

To date, a Total Maximum Daily Load (TMDL) designation for fecal coliform has not been formally established by FDEP within the City of Pompano Beach. A TMDL designation was established by FDEP in 2007 for WBID 3271 (Old Pompano Canal), which is defined in more detail in Section 3.4 of this report. This existing TMDL designation for the Old Pompano Canal is for nutrients impairment but not fecal coliform impairment. A proposed TMDL for WBID 3270, which has not yet been finalized by FDEP, recommends a 22% reduction in fecal coliform loading. A proposed TMDL for WBID 3274, which has not yet been finalized by FDEP, recommends a 67% reduction in fecal coliform loading. The limits of the the impaired WBID boundaries within the City of Pompano Beach is displayed on Figure 4-16 within this section. The impaired classification for these three WBID basins is pending until finalized by FDEP. The City should note that although WBID 3226G1 (North Broward Intracoastal Waterway) has not been listed by FDEP as impaired for fecal coliform, elevated values of fecal coliform have been recorded within the EPA STORET database.

The “Implementation Guidance for the Fecal Coliform Total Daily Maximum Loads” adopted by the FDEP provides direction for local stakeholders to address fecal coliform impairments in recreational waters. This guidance document offers a series of steps which will need to be implemented when these tree WBIDs are officially designated as impaired for fecal coliform in the future:

- Understanding the Basin: Evaluate existing TMDLs, sewer infrastructure, sanitary sewer overflow database, private sewer infrastructure, stormwater infrastructure, septic tanks, soils, land use, hydrology, water quality and rainfall data.
- Identifying and Assembling the Appropriate Stakeholders: Impacted MS4 permittees, Public Works Departments, Utility Departments, Broward County Health Department, FDEP, Florida Department

of Agriculture and Consumer Services, Florida Department of Health, and Florida Department of Transportation.

- Identifying Source: Typical sources of fecal coliform include sewer infrastructure, onsite sewage treatment and disposal systems, urban stormwater, nonpoint sources and wildlife. Stormwater may contribute through illicit connections of sanitary sewer systems to the stormwater system. Nonpoint sources can include pet waste, homeless populations, and certain types of agricultural operations.
- Walk the Water Body: This is a field effort which involves walking the water body to identify the contributing tributaries, location of infrastructure, and potential sources of the fecal coliform.
- Decision Matrix and Ranking Tool: This is a tool that uses the fecal coliform levels, presence and magnitude of human fecal contamination and other sources of human pathogens to determine the level of impairment of the water body.
- Wildlife Survey: Record instances of wildlife to help correlate potential sources with fecal coliform levels.
- Evaluate Data Sufficiency: Evaluate the data gathered above to determine if additional data is necessary.
- Management Actions: If additional work is necessary to reduce the fecal coliform, develop a management plan outlining the actions to take place and a timeline for completion.
- Structural Activities: Implement flood control projects to control the nonpoint source pollution and overflow of sanitary sewer facilities. Wet ponds and swales are common BMPs used to treat stormwater and can reduce the fecal coliform loading. Upgrading sanitary sewer systems so they are less likely to leak or fail can also reduce the fecal coliform loading in the stormwater system.
- Nonstructural Activities: Programs can be put into place to reduce the fecal coliform loading including Fats, Oils and Grease (FOG) Program, Sanitary Sewer Overflow (SSO) Root Cause Program, Wastewater Capacity, Management, Operations, and Maintenance (CMOM) Program, Utilities inspection and maintenance program, litter removal program, public education and outreach program and local ordinances.
- Reporting Management Actions: FDEP encourages stakeholders to summarize the management actions to aid in matching the efforts with the potential fecal coliform source they address.

If FDEP designates these WBIDs as impaired for fecal coliform in the future, the City should investigate having FDEP adopt their management plan as a formal BMAP (Basin Management Action Plan). The benefits of having a BMAP include providing an enforcement mechanism to FDEP and opening up sources of funding through the state loan and grant programs and water management district funds.

4.10 SUB-BASIN PRIORITIZATION

CMA has developed a method for ranking all sub-basins for the purpose of identifying potential “problem areas” of the City which are most in need of stormwater improvements. CMA prioritized the need for potential system improvement alternatives throughout the City based on various factors. The prioritization formula for each sub-basin within the City of Pompano Beach was developed based on the information compiled for each basin. This prioritization formula was based on a combination of stormwater modeling results, historical observations of City staff, flooding complaints received from residents and business owners, FEMA Repetitive Losses, and water quality impacts on surface water bodies. The basin prioritization formula simply assigns a value for each parameter and adds them together, as displayed below:

Basin Prioritization Formula = M + H + C + F + WQ

M = Model Results per Sub-Basin
H = Historical Observations by City Staff per Sub-Basin
C = Public Complaints per Sub-Basin
F = FEMA Repetitive Loss Properties per Sub-Basin
WQ = Water Quality Impacts per Sub-Basin

This basin prioritization formula assigns a value for each sub-basin, which was used to rank the drainage sub-basins most in need of improvements to the existing stormwater management system. The results of the Basin Prioritization Formula were defined in the Drainage Basin Prioritization Memo submitted on July 23, 2012. The results of the sub-basin prioritization formula are summarized within the results table in Appendix A-4 – Basin Prioritization Data Tables of this report and are graphically displayed on Figure 4-17 Sub-Basin Prioritization Map at the end of this section. Each parameter within the basin prioritization formula is defined in more detail below.

4.10.1 Stormwater Model Results

The stormwater model results parameter within the basin prioritization formula is based on a standardized estimate of the expected length of roadway flooding divided by the tributary area of the respective sub-basin. The purpose of this stormwater model results parameter within the basin prioritization formula is to incorporate the objective results of the computer model. As previously noted, CMA has prepared potential flooding area maps, which display the extent of expected flooding during the 5-year storm event. As displayed on these maps, the limits of expected flooding extend into roadway areas throughout the City. Since the locations where the expected flooding extended to the roadway centerline can be considered an appropriate metric for identifying the worst potential flooding "problem areas", CMA used the length of the roadway centerline within the expected flooding area to rank the sub-basins for flooding potential.

Using a GIS intersection tool, CMA tabulated the length of roadway centerline within the expected flooding area for each sub-basin. In order to develop the ranking for flooding potential, CMA divided the length of flooded roadway centerline within each sub-basin by the total area of tributary area within each sub-basin. This standardized parameter is an appropriate metric for comparing the extent of potential flooding within each sub-basin. Please note this ratio is just an indicator which can be used to compare the flooding potential of all sub-basins within the study area. CMA has assigned points to all 621 sub-basins for the stormwater model results parameter (M) of the basin prioritization formula according to the following scale:

- 3 points per sub-basin ranked above the 95th percentile
- 2 points per sub-basin ranked between the 90th percentile and the 95th percentile
- 1 point per sub-basin ranked between the 80th percentile and the 90th percentile
- 0.5 points per sub-basin displaying flooding ranked below the 80th percentile

The points assigned to each sub-basin under the stormwater model results (M) parameter within the basin prioritization formula are summarized within the results table in Appendix A-4 of this report

4.10.2 Historical Flooding Observations

During multiple review meetings during the development of this Stormwater Master Plan, CMA obtained input from City staff on various areas of the City right of way which have been observed to have historical flooding problems during and after heavy rainfall events. During each review meeting, CMA

presented the potential flooding maps to City Utility Department staff and Engineering staff to confirm which City right of way areas experience the worst flooding problems based on their historical observations of past flooding problems. The City Utility Department staff and Engineering staff marked up the potential flooding maps with their input. Based on feedback from City Utility Department staff and Engineering staff, 52 sub-basins within the City were identified to have historical flooding problems. All sub-basins where historical flooding problems have been observed by City staff were assigned a numeric value of three points within the prioritization formula. The points assigned to each sub-basin under the historical observations (H) parameter within the basin prioritization formula are summarized within the results table in Appendix A-4 of this report.

4.10.3 Resident Complaints

All flooding complaints submitted by the general public to the City were taken into account within this basin prioritization formula. The City maintains a database which tracks all flooding complaints received from the general public, which was provided to CMA for incorporation into the basin prioritization formula. The additional feedback on flooding problems was received from the public during two public outreach events conducted during the development of this Stormwater Master Plan. At these public outreach meetings, residents completed a questionnaire to describe any past flooding that has occurred within their neighborhood. CMA compiled all flooding complaint information, which is included in digital Appendix B-10 – Resident Complaint Data attached to this report. Through September 11, 2012, the City has received a total of 84 flooding complaints from residents within the database. The location of the drainage complaints that have been recorded by the City are displayed on Figure 2-15 Resident Complaints and FEMA Repetitive Losses within Section 2. All complaints received through September 11, 2012 are included in this prioritization formula. Each flooding complaint was assigned a numeric value of one point per complaint and tabulated for each sub-basin. The points assigned to each sub-basin under the resident complaints (C) parameter within the basin prioritization formula are summarized within the results table in Appendix A-4 of this report.

4.10.4 FEMA Repetitive Loss Properties

All properties that have filed at least two insurance claims for flood damage to the property within any ten year period have been designated as a FEMA Repetitive Loss Property. There are a total of 13 FEMA Repetitive Loss Properties with claims since 1999 which have been included within the basin prioritization formula. The locations of the FEMA Repetitive Loss Properties are displayed on Figure 2-15 Resident Complaints and FEMA Repetitive Losses within Section 2. Each FEMA Repetitive Loss Property was assigned a numeric value of one point per property location and tabulated for each sub-basin. The points assigned to each sub-basin under the FEMA Repetitive Loss Properties (F) parameter within the basin prioritization formula are summarized within the results table in Appendix A-4 of this report.

4.10.5 Water Quality Impacts

All sub-basins within the City were assigned water quality points based on the potential pollutant loads within stormwater runoff generated from each sub-basin along with the proximity of the sub-basin to an impaired water body, such as the Old Pompano Canal. The purpose of this water quality impact parameter within the basin prioritization formula is to provide additional weight to sub-basins which negatively impact the water quality of stormwater runoff and negatively impact the water quality within the Old Pompano Canal. The water quality parameter within the basin prioritization formula is a combination of the estimated pollutant load and the proximity to impaired waterbody (WBID). The water quality parameter (WQ) of the basin prioritization parameter is calculated by the following equation:

WQ = CDP + WBIDP + PLP

Canal Discharge Point (CDP) parameter was assigned a value of 1 for sub-basins that discharge directly via a positive outfall pipe into the Old Pompano Canal or tributary canal west of Control Structure G57. Otherwise, all other sub-basins were assigned a value of 0. The purpose of this component is to provide more weight to sub-basins which directly discharge stormwater runoff and potential pollutants into the Old Pompano Canal which is impaired.

Impaired Waterbody Proximity (WBIDP) parameter was assigned based on the percentage of sub-basin area which falls within the impaired WBID boundary for the Old Pompano Canal. The sub-basins which were entirely located within the WBID boundary were assigned a value of 1. The sub-basins which were partially located within the WBID boundary were assigned a ratio based on the percentage of the sub-basin located in the WBID. The purpose of this component is to provide more weight to sub-basins which are located in WBID boundary for the Old Pompano Canal, which is impaired.

Estimated Pollutant Load (PLP) parameter is a combination of the estimated pollutant loads of total nitrogen (TN) and total phosphorous (TP) generated from each sub-basin. The methodology for calculating this parameter includes sorting all sub-basins based on the estimated total pollutant load (lb) for both TN and TP generated by each sub-basin. All sub-basins were ranked separately for TN and TP with the highest ranking corresponding to the highest total load of each corresponding pollutant generated from the sub-basin. Based on the average rank of estimated pollutant load of both TN and TP, each sub-basin was assigned point value as defined within Table 4.13 below.

Table 4.13 – PLP Parameter Point Assignments

Sub-Basin Estimated Pollutant Load Rank (Average of TN and TP)	Points
1-100	1
101-200	0.8
201-300	0.6
301-400	0.4
401-500	0.2
501-622	0

The points assigned to each sub-basin under the Water Quality Impacts (WQ) parameter within the basin prioritization formula are summarized within the results table in Appendix A-4 of this report.

4.11 SEA LEVEL RISE ANALYSIS

CMA has used the stormwater model to analyze the impact of potential future sea level rise on the level of service provided by the City's stormwater management system. The purpose of our analysis is to identify areas which will be prone to additional flooding under various sea level rise scenarios. The Broward County Climate Change Action Plan has requested that various agencies assess the public infrastructure to determine vulnerability to potential sea level rise. In order to identify any public infrastructure which may be impacted by potential sea level rise, inundation maps of the City of Pompano Beach have been prepared to show the approximate limits of expected flooding during various storm events under various sea level rise scenarios. The approximate limits of potential flooding shown on these inundation maps will allow the City to identify any public infrastructure located within these flooded areas.

The Broward County Climate Change Task Force previously prepared inundation maps on a regional basis throughout Broward County, including the areas within the City. These inundation maps are intended to display areas of Broward County with ground surface elevations below various sea level rise scenarios. These maps were designed for planning purposes to identify potentially vulnerable areas and were not intended to forecast actual inundation areas in detail for localized areas. Although these inundation maps displayed areas prone to flooding due to sea level rise only, the maps did not address the combined impact of sea level rise during various storm events. The City requested CMA develop more detailed inundation maps which display the impacts of rainfall during the various sea level rise scenarios. CMA also used more detailed topographic data available to improve the accuracy of the inundation maps. The methodology and assumptions used for the development of these inundation maps is summarized below.

4.11.1 SEA LEVEL RISE PROJECTIONS

<u>Sea Level Rise Projections</u>

Southeast Florida Regional Climate Change Compact Work Group has developed regional sea level rise projections for South Florida. The Compact Work Group reviewed the existing projections along with current scientific literature related to sea level rise in order to develop the regional sea level rise projections. Based on this research, the Compact Work Group recommended using the regional sea level projections on the US Army Corps of Engineers July 2009 Guidance Document Engineering Circular 1165-2-211 (USACE, 2009). The Compact Work Group used the historical tidal data at Key West from 1913-1999 as the basis for the regional sea level rise projection. The Unified Southeast Florida Sea Level Rise Projection is displayed below within Figure 4-18 below. As displayed within the graph, range of 3 sea level rise projections is defined based on the Historic Rise Rate, Modified NRC Curve I, and Modified NRC Curve III.

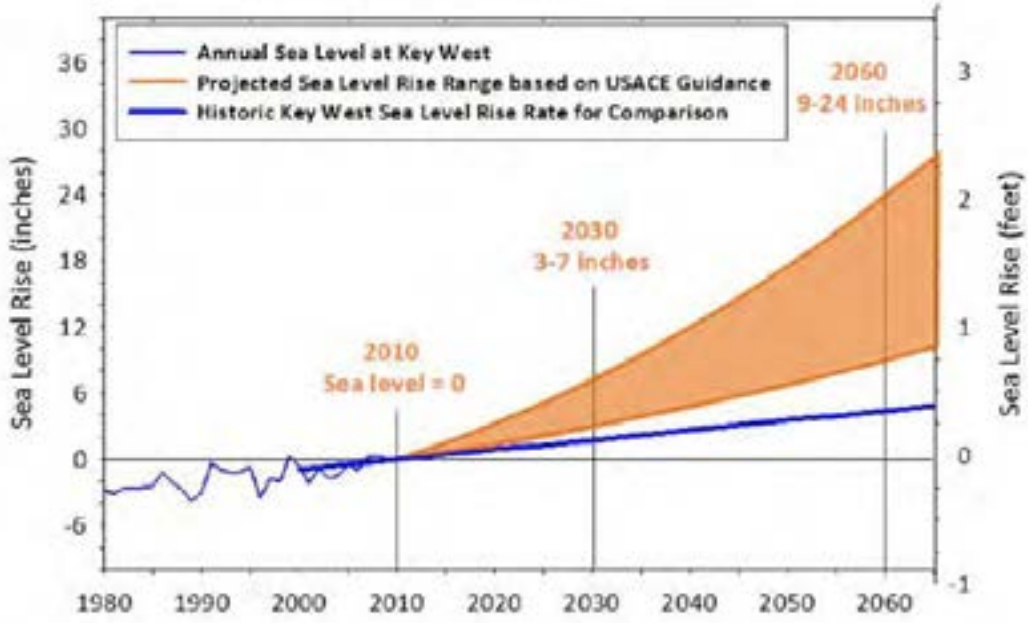


Figure 2. Unified Southeast Florida Sea Level Rise Projection for Regional Planning Purposes. This projection uses historic tidal information from Key West and was calculated by Kristopher Esterson from the United States Army Corps of Engineers using USACE Guidance (USACE 2009) intermediate and high curves to represent the lower and upper bound for projected sea level rise in Southeast Florida. Sea level measured in Key West over the past several decades is shown. The rate of sea level rise from Key West over the period of 1913 to 1999 is extrapolated to show how the historic rate compares to projected rates.

Figure 4-18 Unified Southeast Florida Sea Level Rise Projection for Regional Planning Purposes

The Broward County Climate Change Task Force had previously prepared inundation maps on a regional basis throughout Broward County for potential sea level rise scenarios of 1 foot, 2 feet, and 3 feet above the current tidal levels. CMA used these sea level rise scenarios to prepare the detailed inundation maps for the City limits based on the same potential sea level rise scenarios of 1 foot, 2 feet, and 3 feet above the current tidal levels. According to the Unified Southeast Florida Sea Level Rise Projections defined by the Southeast Florida Regional Climate Change Compact, these sea level rise scenarios can be projected to occur within the following timeframes depending on the sea level rise projection method used:

- 1 foot sea level rise: Years 2040-2070
- 2 feet sea level rise: Years 2060-2115
- 3 feet sea level rise: Years 2075-2150

Based on the regional sea level rise projections, sea level rise scenarios of 1 foot rise and 2 feet rise could be occur within the 50-year planning horizon. The City of Pompano Beach should focus on the inundation maps under these scenarios to identify vulnerable facilities within the City in order to plan for any potential infrastructure modifications.

4.11.2 MODEL ASSUMPTIONS

Stormwater Model

CMA used hydrologic and hydraulic computer model of the City's limits, which can be used to analyze the performance of the City's existing stormwater management system. The stormwater model encompasses the entire limits of City along with adjacent areas which are hydraulically interconnected. The stormwater model can be used to estimate the peak flood stage within every drainage basin throughout the City under various rainfall events. Based on the peak flood stage results from the stormwater model along with the topographic elevation data, CMA is able to develop inundation maps which display expected limits of flooding under the various rainfall scenarios.

For the purposes of this sea level rise analysis, CMA was able to use the stormwater model to analyze the performance of the stormwater management system during various rainfall events under various future sea level rise scenarios. Since the significant portions of the City's stormwater management system is directly connected to the tidally influenced waterways via multiple outfalls, the performance of the stormwater management system would be negatively impacted by any future sea level rises. In order to analyze the impact of various future sea level rise scenarios, the assumptions for the tailwater conditions within the stormwater model was modified from the estimated existing tidal levels within the Intracoastal Waterway to the projected future tidal levels under each sea level rise scenario. Basically, the assumed tailwater elevations at each outfall was increased within the stormwater model by increments of 12-inches, 24-inches, and 36-inches in order to estimate the peak flood stage within each drainage basin under each sea level rise scenario. Based on these peak flood stage results from the stormwater model, CMA is able to develop inundation maps which display expected limits of flooding under the combined scenario of rainfall events during the various sea level rise projections.

Tidal Assumptions

CMA established the existing tailwater assumptions within the stormwater model for the outfalls into the Intracoastal Waterway based on available tidal data from nearby measurement stations. Since the Intracoastal Waterway is a tidally influence waterbody, the tailwater assumptions were defined as time series data to represent the water level fluctuations due to the tides throughout the analysis period. CMA gathered tidal data from three nearby tide stations, which are maintained by National Oceanic and Atmospheric Administration (NOAA). The available tide data for current epoch (1983-2001) included mean higher high water (MHHW), mean high water (MHW), mean tide level (MTL), mean sea level (MSL), mean low water (MLW), and mean lower low water (MLLW), which are summarized in Table 4.14 below in NAVD88 vertical datum:

Table 4.14 – NOAA Tidal Elevation Data (NAVD)

NOAA Tide Station	MHHW	MHW	MTL	MSL	MLW	MLLW
South Dania Sound	0.41	0.32	-0.78	-0.80	-1.89	-2.05
South Port Everglades	0.50	0.38	-0.85	-0.86	-2.10	-2.28
North Dania Sound	0.49	0.37	-0.86	-0.86	-2.10	-2.27
Average	**0.47**	**0.36**	**-0.83**	**-0.84**	**-2.03**	**-2.20**

CMA used the average tidal elevations from all three NOAA tidal stations to define the existing tailwater elevations within the stormwater model. The time series data for the existing conditions was established to match tide patterns of high tide and low tide occurring every 12 hours during the entire model run period. Within the time series for the existing tidal conditions, the tidal elevations were defined to be +0.36 feet NAVD for high tide and -2.03 feet NAVD for low tide, which are each equivalent to the

average tidal elevations from the three nearby NOAA tidal stations. In order to establish the tailwater conditions for various sea level rise scenarios, CMA shifted the time series elevation data up by increments of 1 foot, 2 feet, and 3 feet. The time series data for the assumed tidal elevations within the Intracoastal Waterway is displayed in Figure 4-19 below by each hour of the model run duration for each sea level rise scenario:

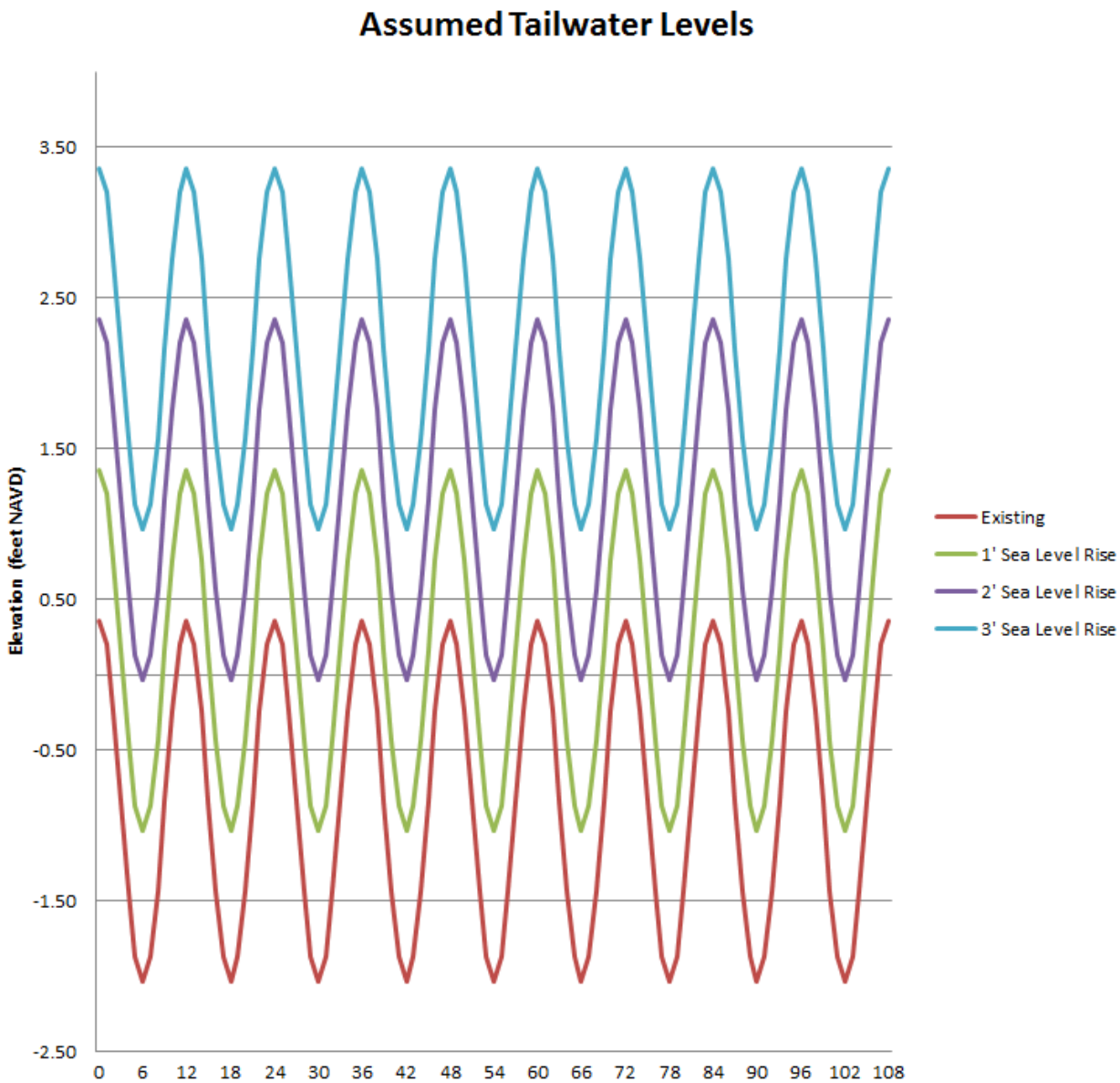


Figure 4-19 Intracoastal Waterway Time-Series Data

Rainfall Assumptions

Within the stormwater model of the existing conditions, design rainfall events are used to evaluate the performance of the existing drainage system and to compare this performance to measurable LOS criteria for flood control. These design storm events have been developed regionally by the SFWMD for one-day and three-day duration design storm events by studying the historical distribution of rainfall in the region. The design storm rainfall volumes for the stormwater model simulations were obtained from the SFWMD Permit Information Manual, Volume 4. The existing conditions stormwater model was used to simulate

the 100-year, 72-hour design storm; the 25-year, 72-hour storm event; 10-year, 24-hour storm event and 5-year, 24-hour storm event. CMA completed the model runs for all four design storms but only prepared inundation maps for the 5-year, 24-hour storm event. A summary of the rainfall depths to be used in the existing conditions stormwater model is defined within the table below.

Table 4.15 – Depths of Design Storm Events	
Design Storm	Rainfall Depth (inches)
5-year, 24 hour	7.8
10-year, 24-hour	9.0
25-year, 72-hour	16.0
100-year, 72-hour	20.0

Ground Surface Elevations

Prior to preparing the inundation maps, CMA used the recent topographic data for the City limits collected for this Stormwater Master Plan. This elevation data has a better level of accuracy over the LIDAR data previously used by Broward County Climate Change Task Force for their regional inundation mapping. CMA used the elevation point files to prepare a Digital Elevation Model (DEM) for each individual drainage basin defined within the SWMM Stormwater Model. The DEM defines the elevation grades within each drainage basin as a contour map of the ground surface. CMA used this updated DEM of each drainage basin to prepare an inundation map for each sea level rise scenario, which shows the areas with ground surface elevations below the peak flood stage under each sea level rise scenario.

4.11.3 INUNDATION MAPPING

First, CMA prepared updated inundation maps to show the sea level rise impacts on City without any rainfall under the existing conditions. Since the existing stormwater management system is directly connected to the tidally influenced waterways via multiple outfalls, surface waters could possibly backflow via the outfalls and inundate the low lying areas of the City as water level rises under each sea level rise projection. These updated inundation maps were prepared to show areas of the City which could be impacted by sea level rise without any rainfall. The inundation maps were prepared based on the sea level rise scenarios of 1 foot, 2 feet, and 3 feet above the current high tide water level within the Intracoastal Way, which was established to be +0.36 feet NAVD. Under the various sea level rise scenarios, these low lying areas of the City could be inundated by just the backflow from the existing outfalls into tidally influenced waterways, such as the Intracoastal Waterway or other marine canals which are tidally influenced. All areas with ground surface elevations below each sea level rise scenario were highlighted within the maps. The inundation maps for each sea level rise scenario without any rainfall are within Figure 4-20 Sea Level Rise Vulnerability Map at the end of this section.

CMA used the SWMM stormwater model to determine the peak flood stages within each drainage basin for various storm events under each sea level rise scenario. CMA used the existing conditions stormwater model. The SWMM stormwater model was used to analyze the impact of the 5-year 1-day storm event during each sea level rise scenario. Within the SWMM Stormwater Model, the tailwater elevation adjusted higher based on the sea level rise scenarios of 1 foot, 2 feet, and 3 feet above current MHW level. The stormwater model was used to calculate the peak flood stages within all drainage sub-basins throughout the City during multiple design storm events at various sea level rise projections. The peak flood stages within each drainage basin were determined by the SWMM Stormwater Model for each sea level rise scenario. CMA prepared inundation maps for each sea level rise scenario combined with the 5-year, 24-hour design storm event as displayed on Figure 4-21 Inundation Map (5-year Storm + 1 feet

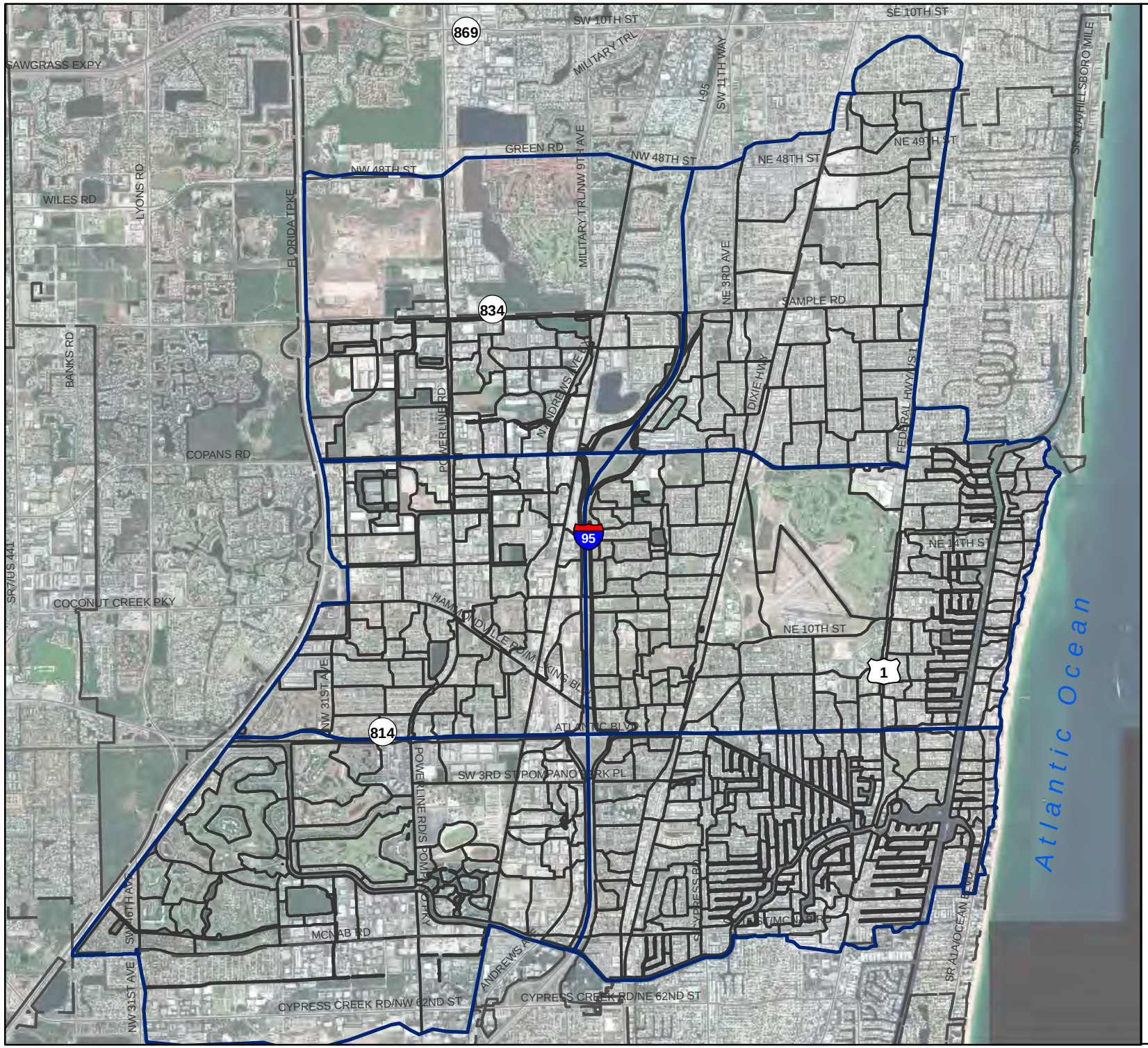
SLR), Figure 4-22 Inundation Map (5-year Storm + 2 feet SLR), and Figure 4-23 Inundation Map (5-year Storm + 3 feet SLR), which are enclosed at the end of this section.

As expected, the peak flood stages throughout the City increased as the sea level increased at the outfalls from the stormwater management system. It should be noted that the peak flood stage did not increase uniformly within all drainage sub-basins as the sea level increased. The primary reason for the differential increases in peak flood stages is the relative ground surface elevation of each drainage basin. For example, significantly more ponding can be expected to be encountered in drainage basins with lower ground surface elevations. Drainage basins with higher ground surface elevations will drain quicker due to higher ground storage capacity in subsurface soils and higher head differential, which will allow stormwater runoff to continue flowing out of the drainage basin via the stormwater piping network. The effectiveness of the existing stormwater management system in these drainage basins with higher ground surface elevations was not significantly impacted by higher tailwater elevations and remained relatively effective at transmitting stormwater runoff via the existing outfalls. In order to effectively display the results of the SWMM Stormwater Model for each scenario, the limits of expected ponding or standing water would need to be graphically displayed on localized maps, which would allow the City to estimate the potential impact on existing and planned facilities. Based on the results of the stormwater model for peak flood stages within each basin, CMA prepared updated inundation maps for each rainfall event under each sea level rise scenario. These inundation maps show the areas of the City with ground surface elevations below the peak flood stage determined by the SWMM Stormwater Model for each drainage basin.

4.11.4 VULNERABLE AREAS

As displayed on the enclosed inundation maps, potential sea level rise alone will not dramatically impact the existing facilities throughout the City. The substantial impact on the City would be encountered during significant rainfall events with higher sea levels. Stormwater ponding would likely extend further and deeper throughout the City when significant rainfall occurred under these potential sea level rise scenarios. Many City right of way areas could be vulnerable under the various sea level rise scenarios since the effectiveness of the stormwater management system would diminish as the tailwater elevation rises in the Intracoastal Waterway and its tributary canals. From a strictly stormwater perspective, the feasibility of the following system improvement alternatives should be investigated to determine if they would protect very low lying areas of the City from sea level rise:

- Eliminate the open connection into the tidally influenced waterways by converting the existing gravity outfalls into to a pumped system which can continue to discharge stormwater under the various sea level rise scenarios.
- Install backflow prevention devices at the existing outfalls into tidally influenced waterways to prevent the backflow of surface water into the stormwater management system under the various sea level rise scenarios.



City of Pompano Beach
Stormwater Master Plan

Figure 4-1
Subbasin Boundary

Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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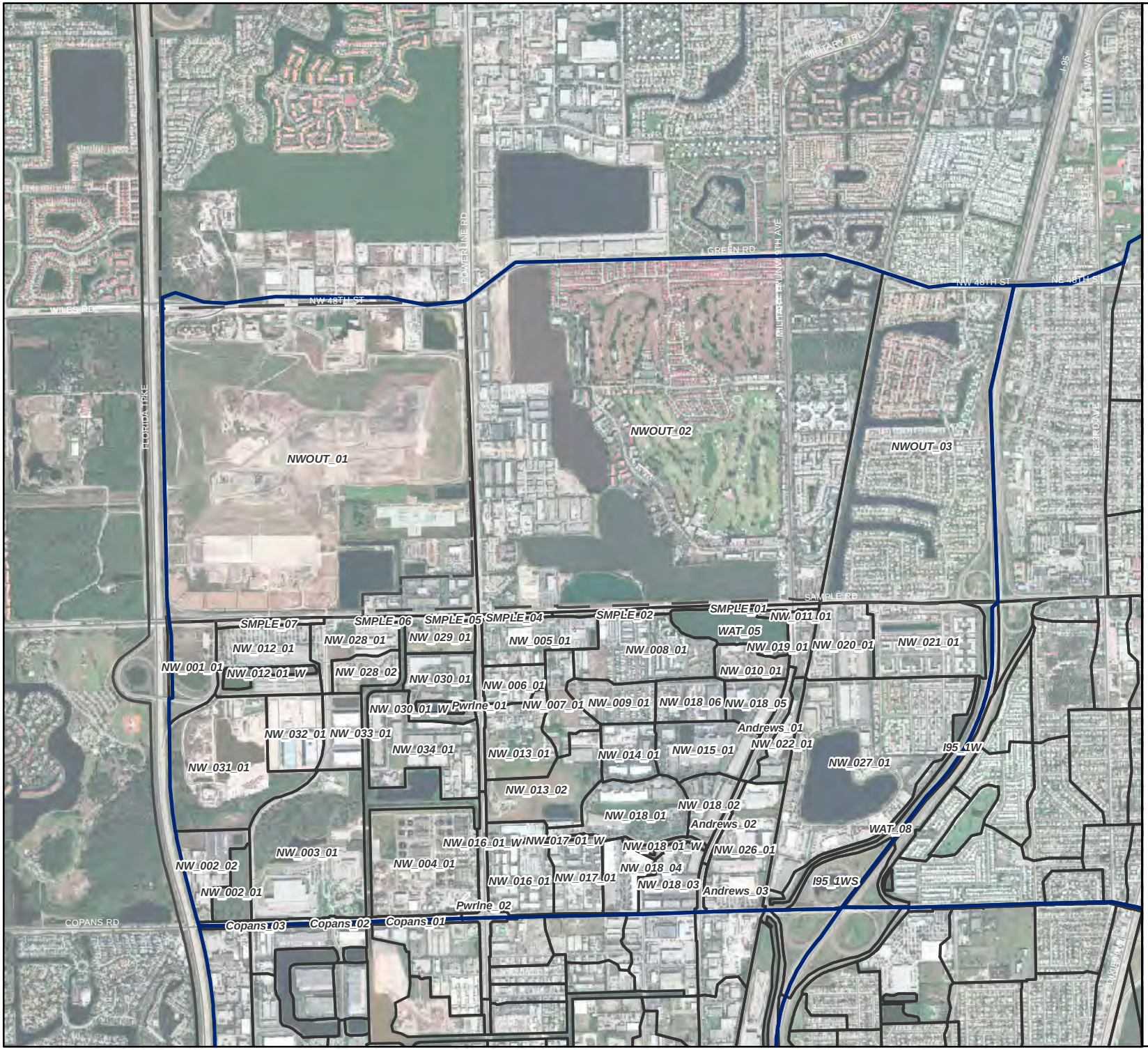
Feet



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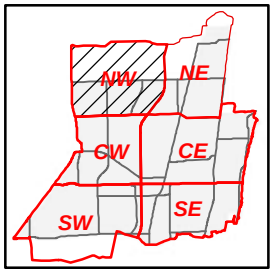
City of Pompano Beach
Stormwater Master Plan

Figure 4-1A
Subbasin Boundary
Basin NW



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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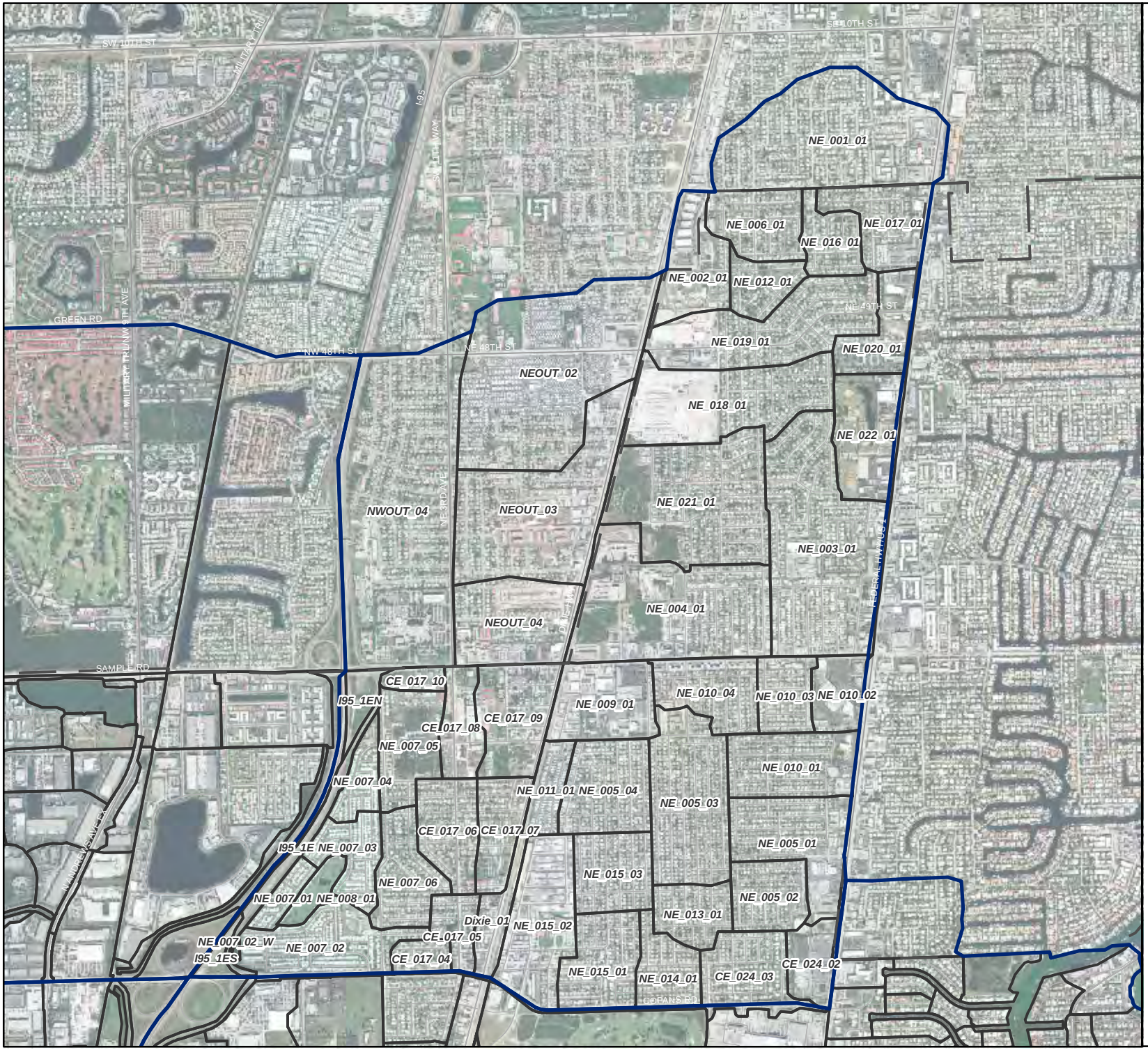
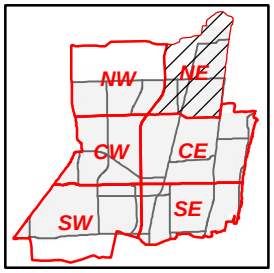


City of Pompano Beach
Stormwater Master Plan

Figure 4-1B
Subbasin Boundary
Basin NE



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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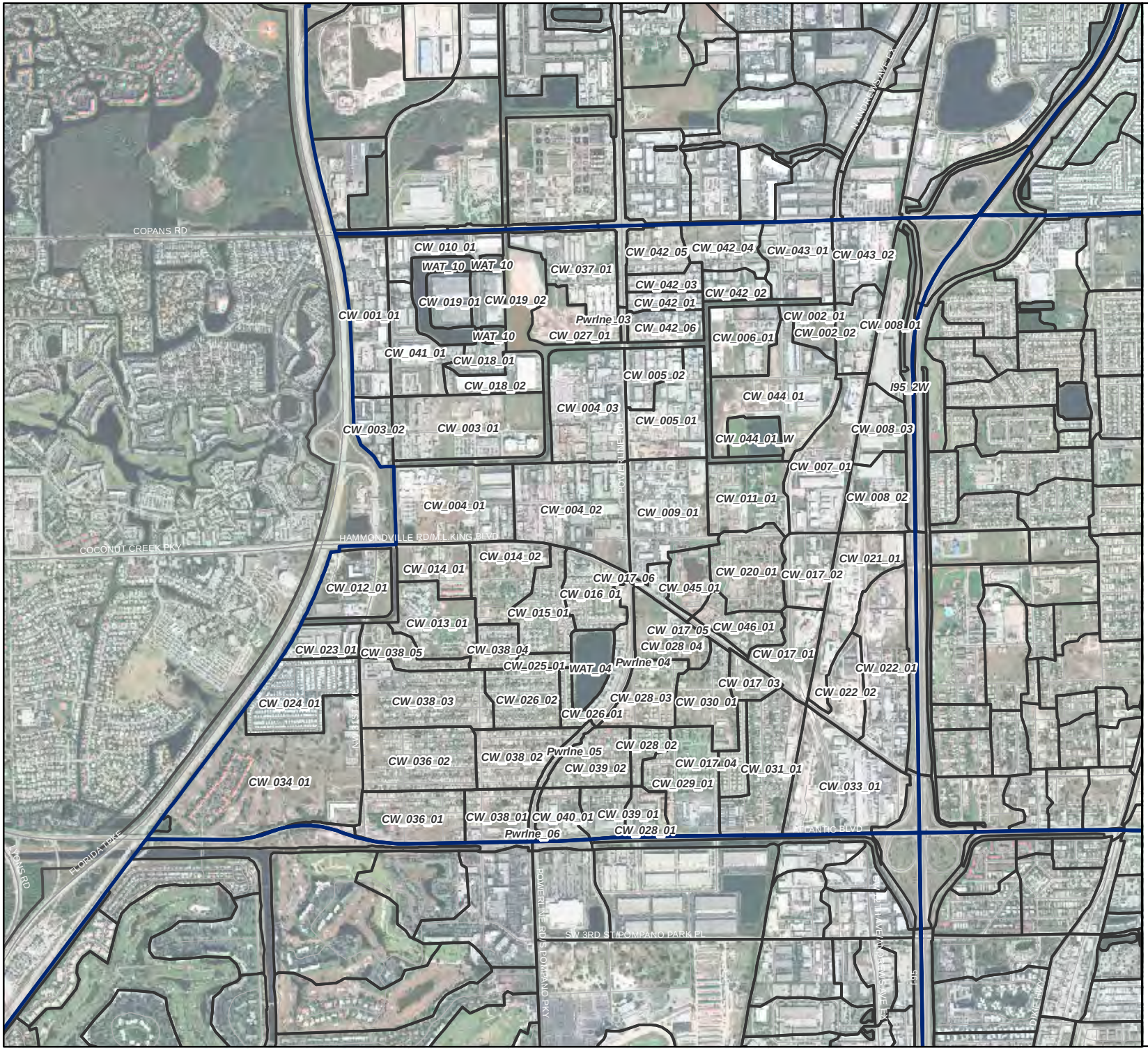
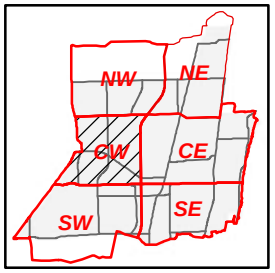


City of Pompano Beach
Stormwater Master Plan

Figure 4-1C
Subbasin Boundary
Basin CW



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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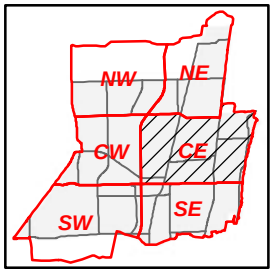
City of Pompano Beach
Stormwater Master Plan

Figure 4-1D
Subbasin Boundary
Basin CE



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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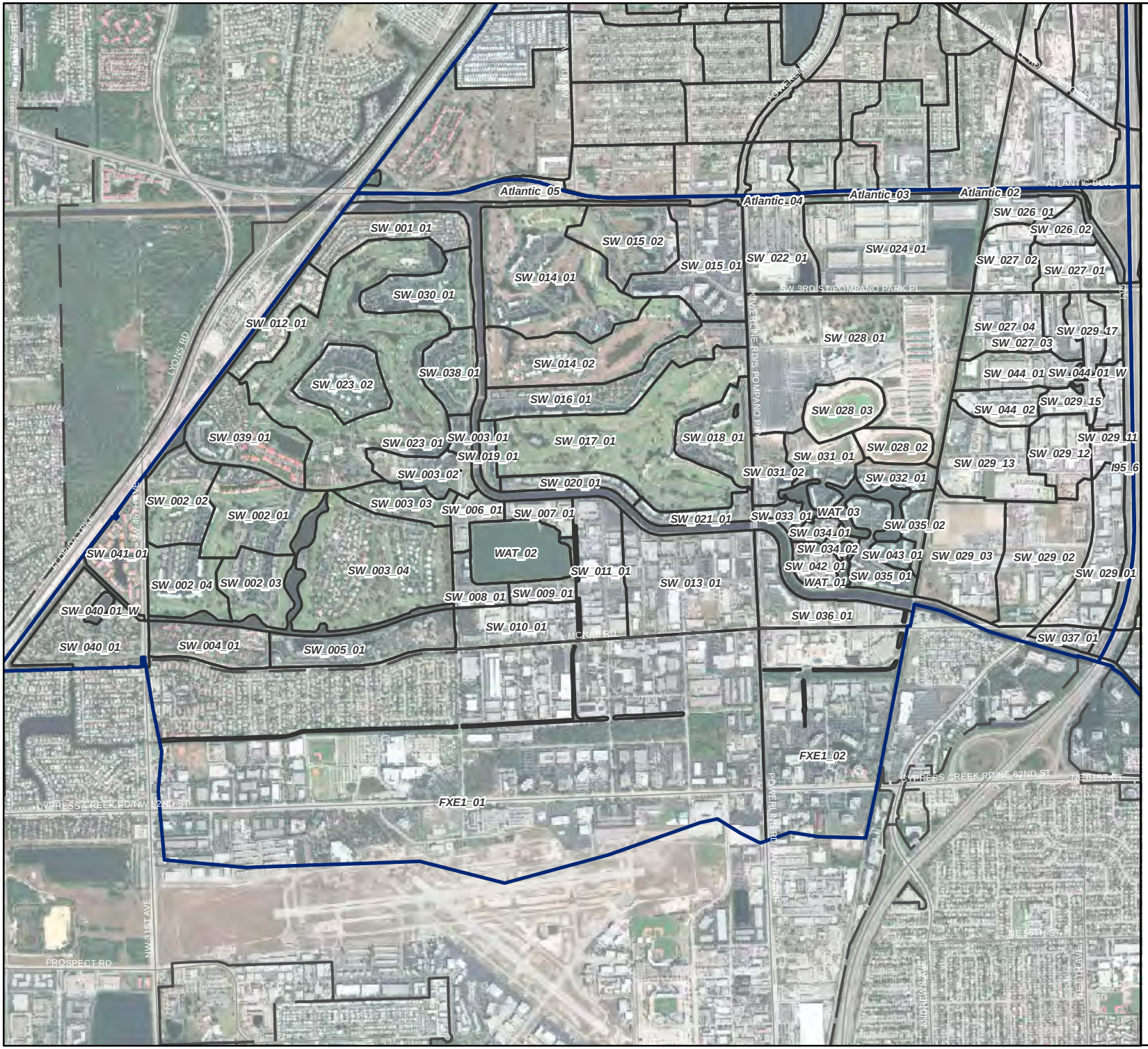
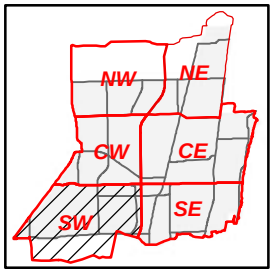


City of Pompano Beach
Stormwater Master Plan

Figure 4-1E
Subbasin Boundary
Basin SW



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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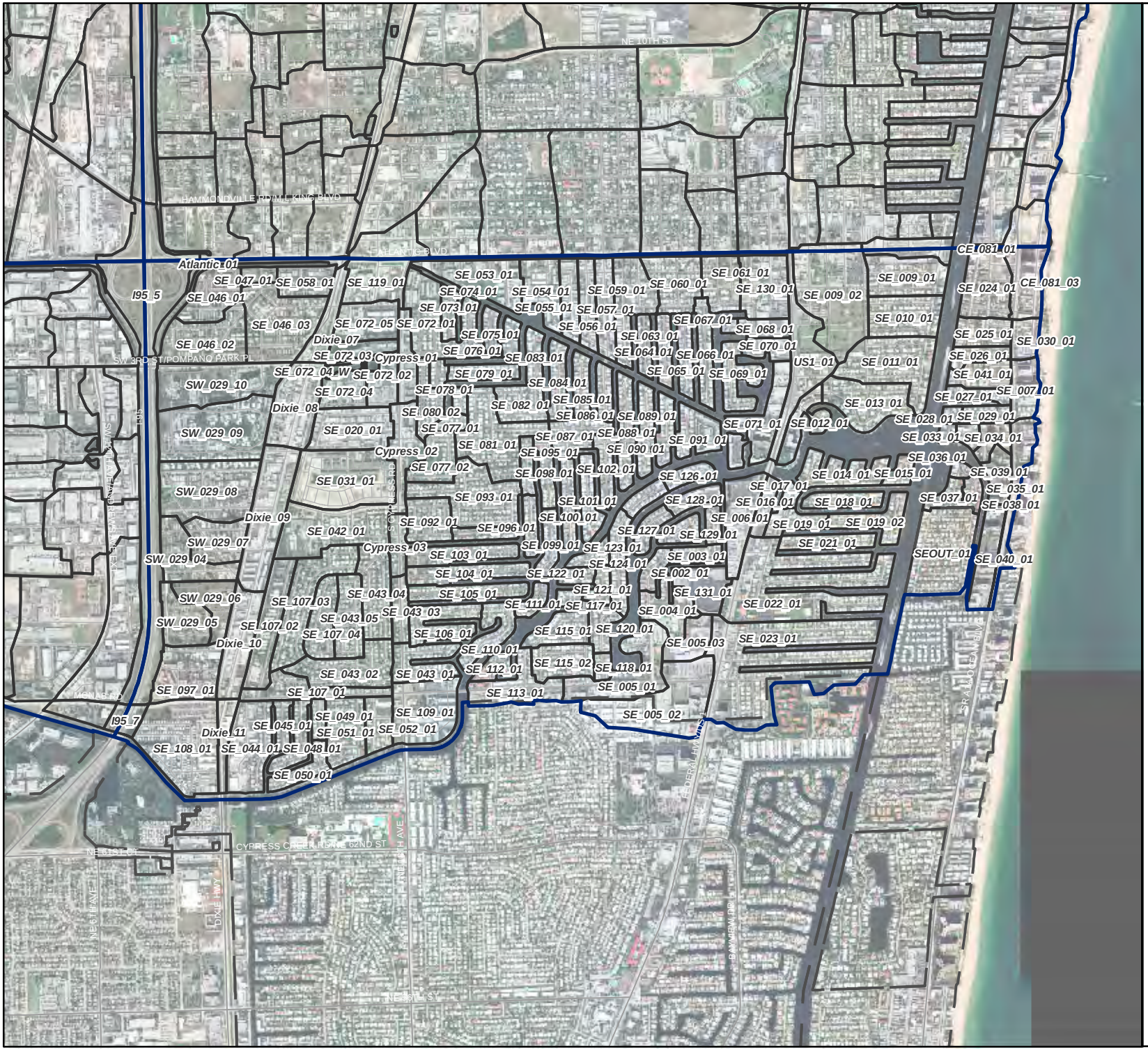

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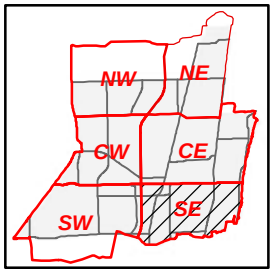
City of Pompano Beach
Stormwater Master Plan

Figure 4-1F
Subbasin Boundary
Basin SE



Legend

- Major Basin Boundary
- City Boundary
- Sub-Basins

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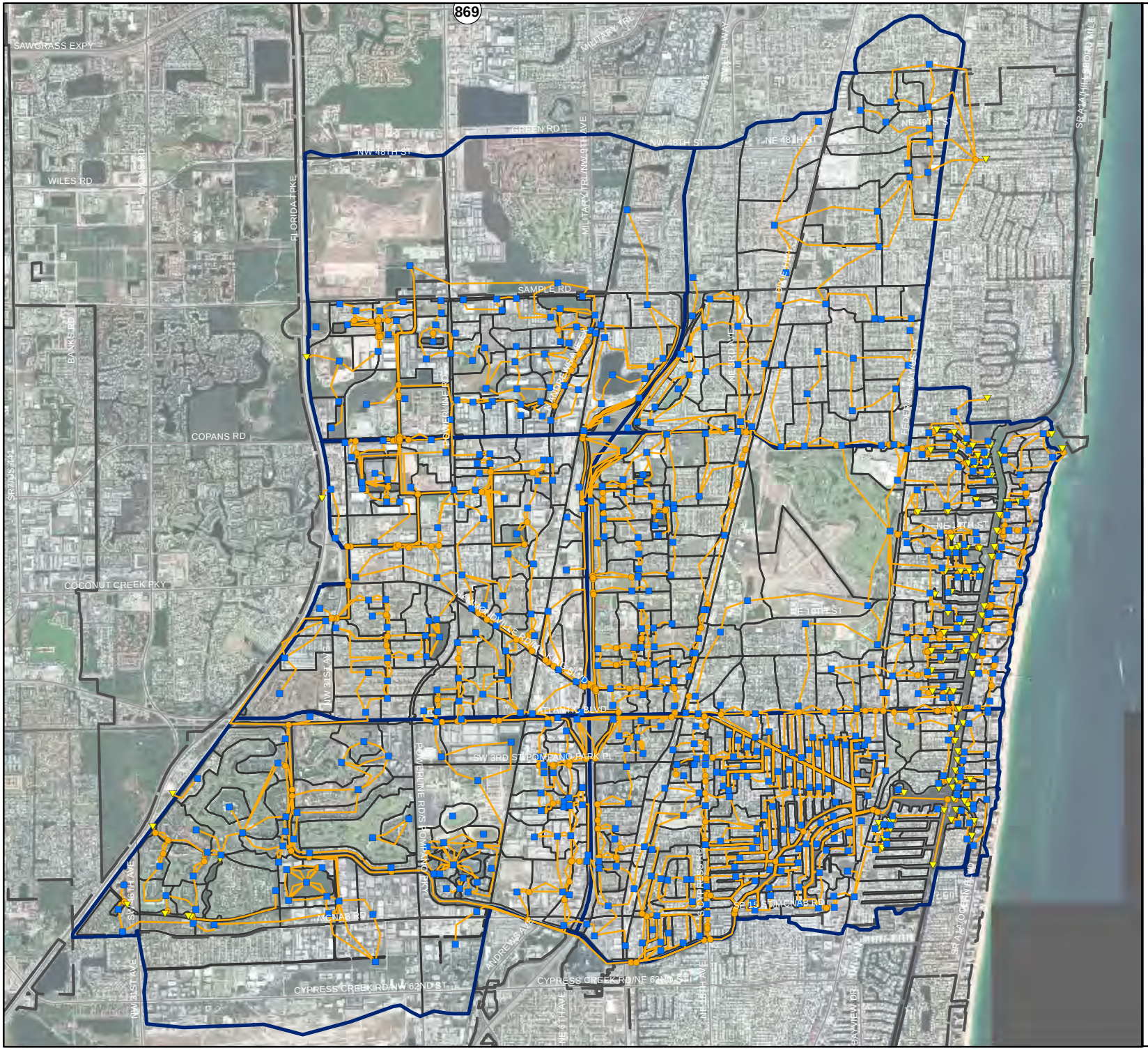

City of Pompano Beach
Stormwater Master Plan

Figure 4-3
Stormwater Model Schematic Map

Legend

MODEL NODES

- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

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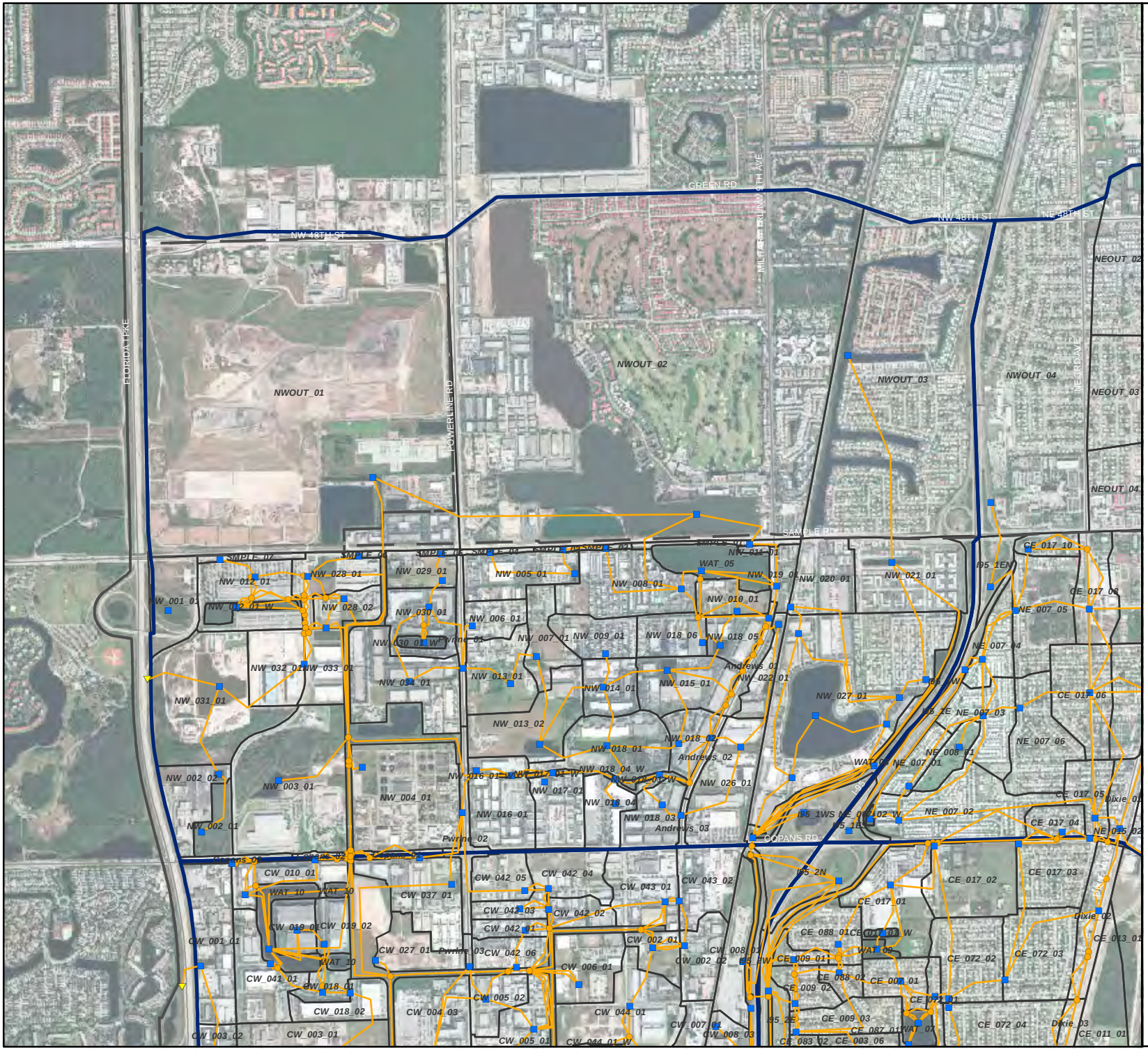
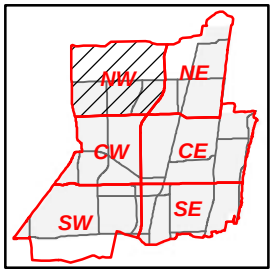


City of Pompano Beach
Stormwater Master Plan

Figure 4-3A
Stormwater Model Schematic Map
Basin NW



Legend

MODEL NODES

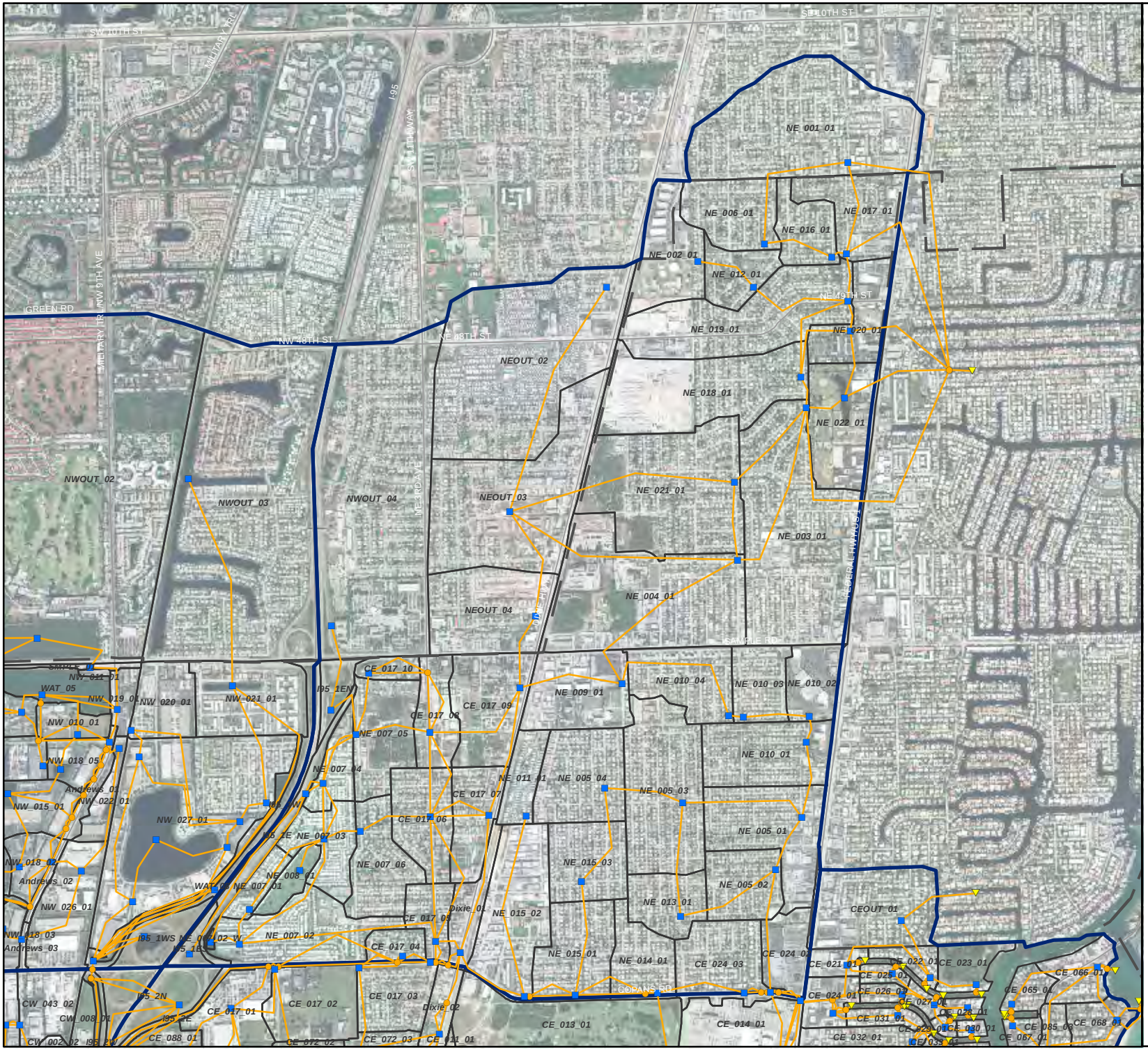
- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

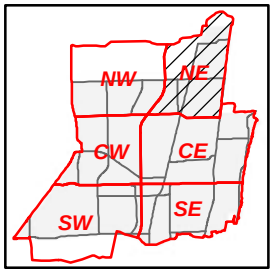
N W E S

0 750 1,500 3,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-3B
Stormwater Model
Schematic Map
Basin NE



Legend

MODEL NODES

- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

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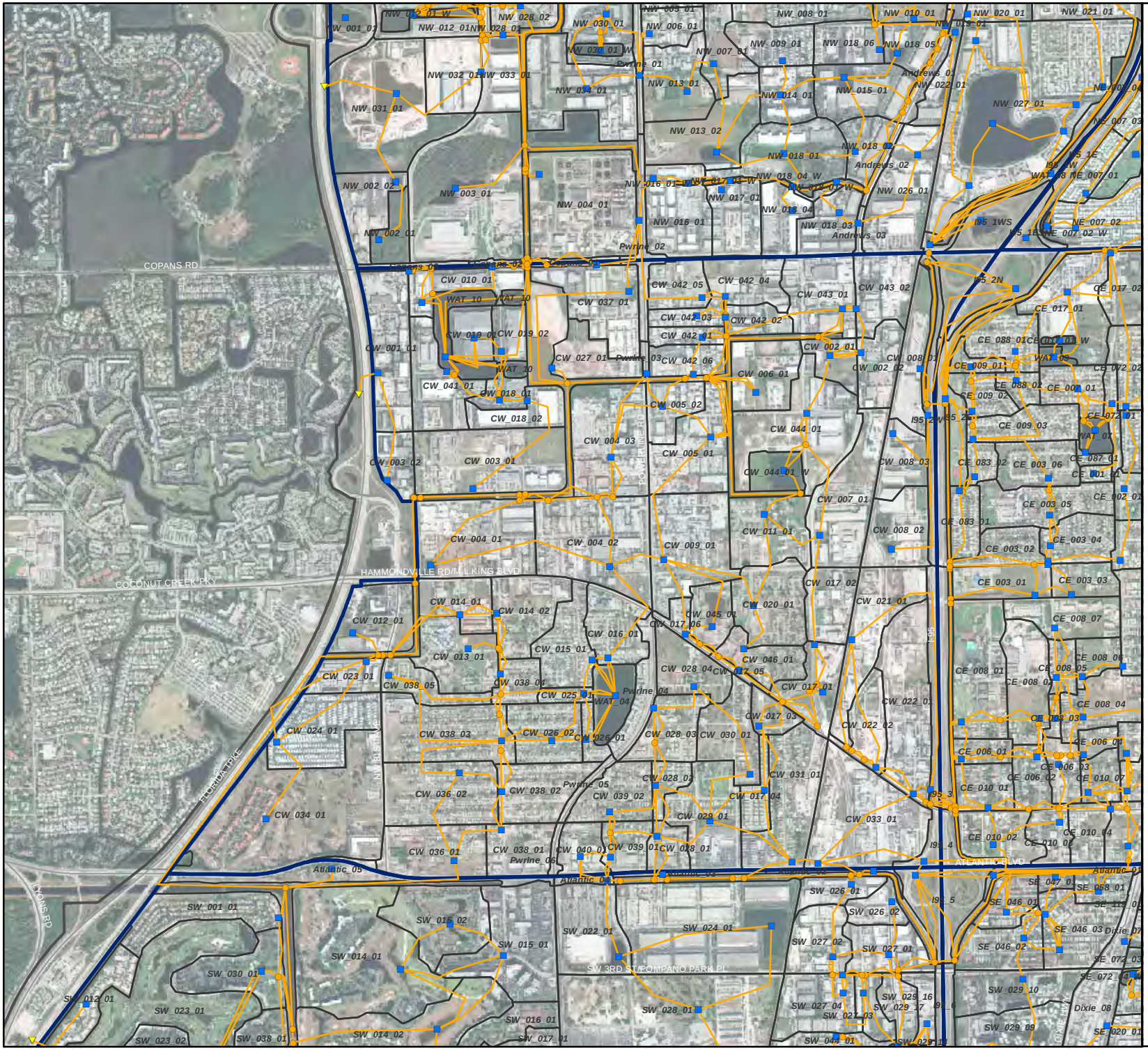

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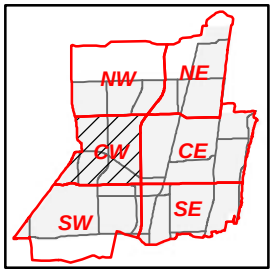
City of Pompano Beach
Stormwater Master Plan

Figure 4-3C
Stormwater Model Schematic Map
Basin CW



Legend

MODEL NODES

- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

N W E S

0 750 1,500 3,000 Feet

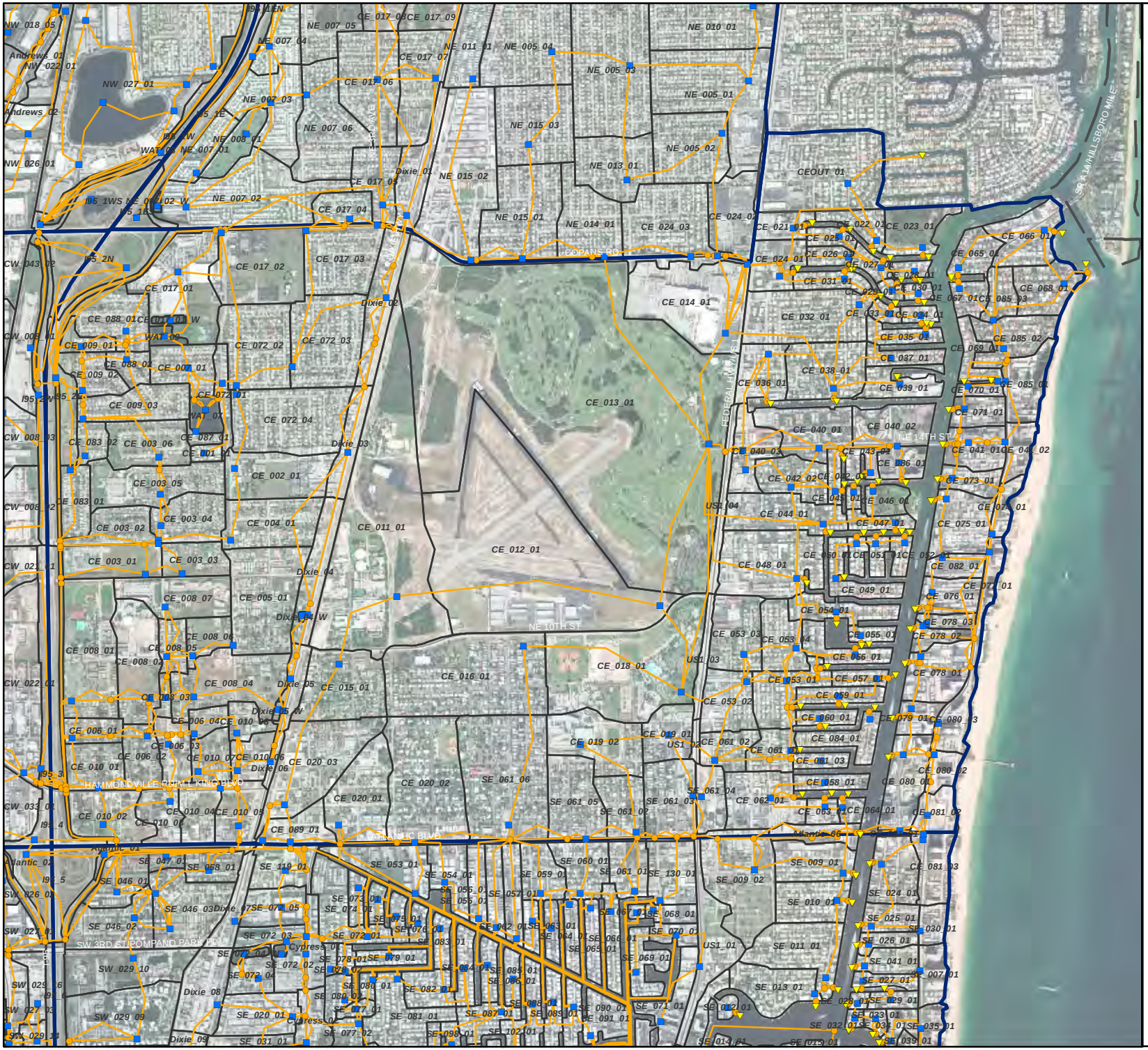


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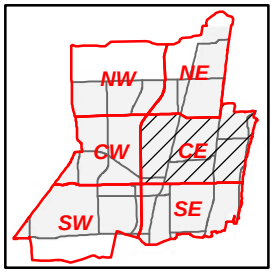
Figure 4-3D
Stormwater Model Schematic Map
Basin CE



Legend

MODEL NODES

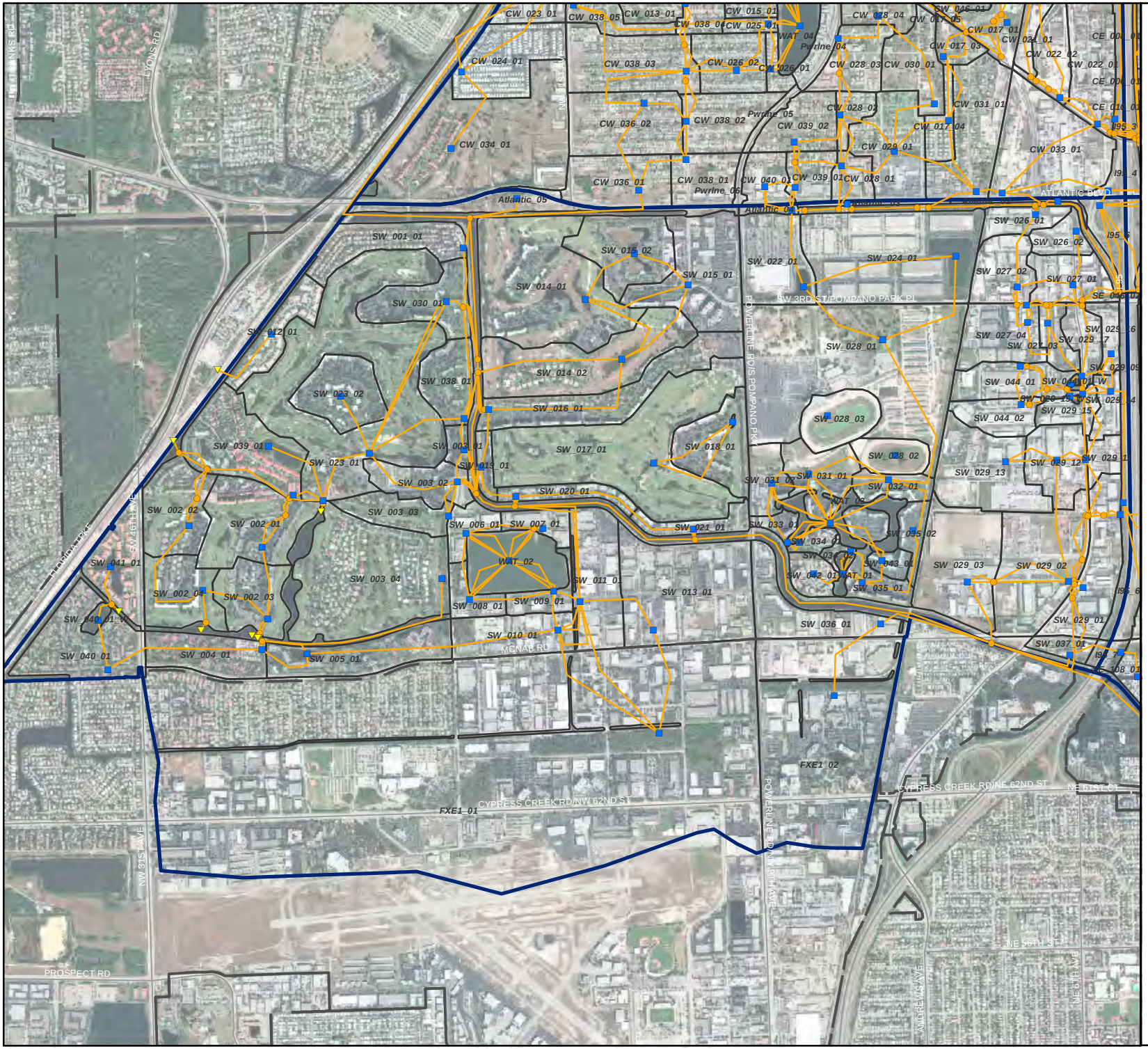
- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

N W E S

0 750 1,500 3,000 Feet



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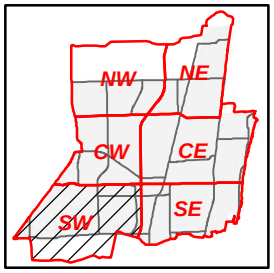
Stormwater Master Plan

Figure 4-3E
Stormwater Model Schematic Map
Basin SW



Legend

MODEL NODES

- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

N
W E
S

0 750 1,500 3,000
Feet

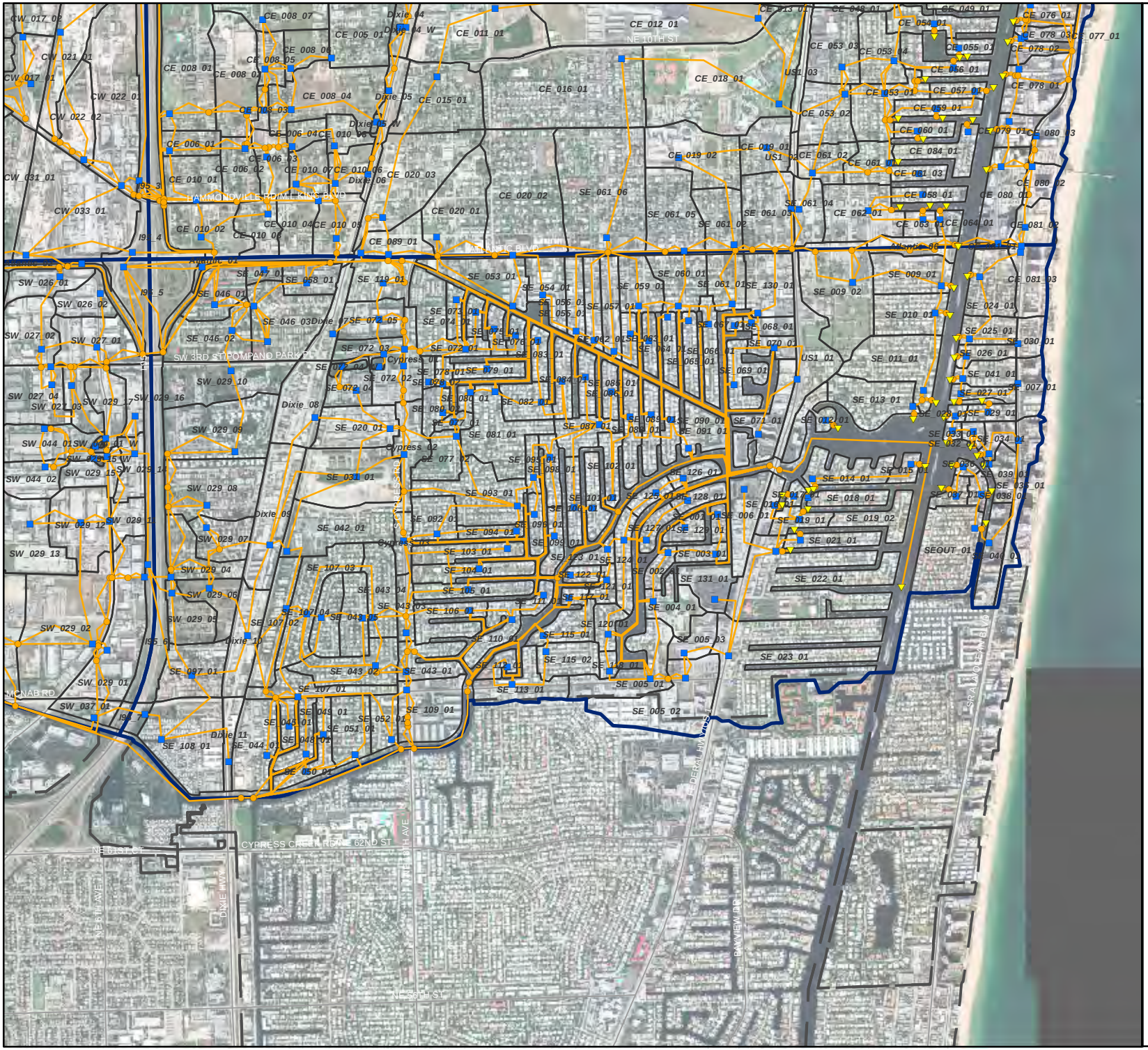


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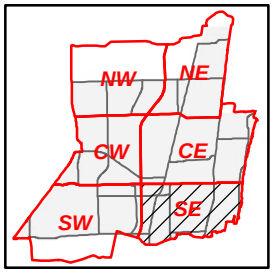
Figure 4-3F
Stormwater Model Schematic Map
Basin SE



Legend

MODEL NODES

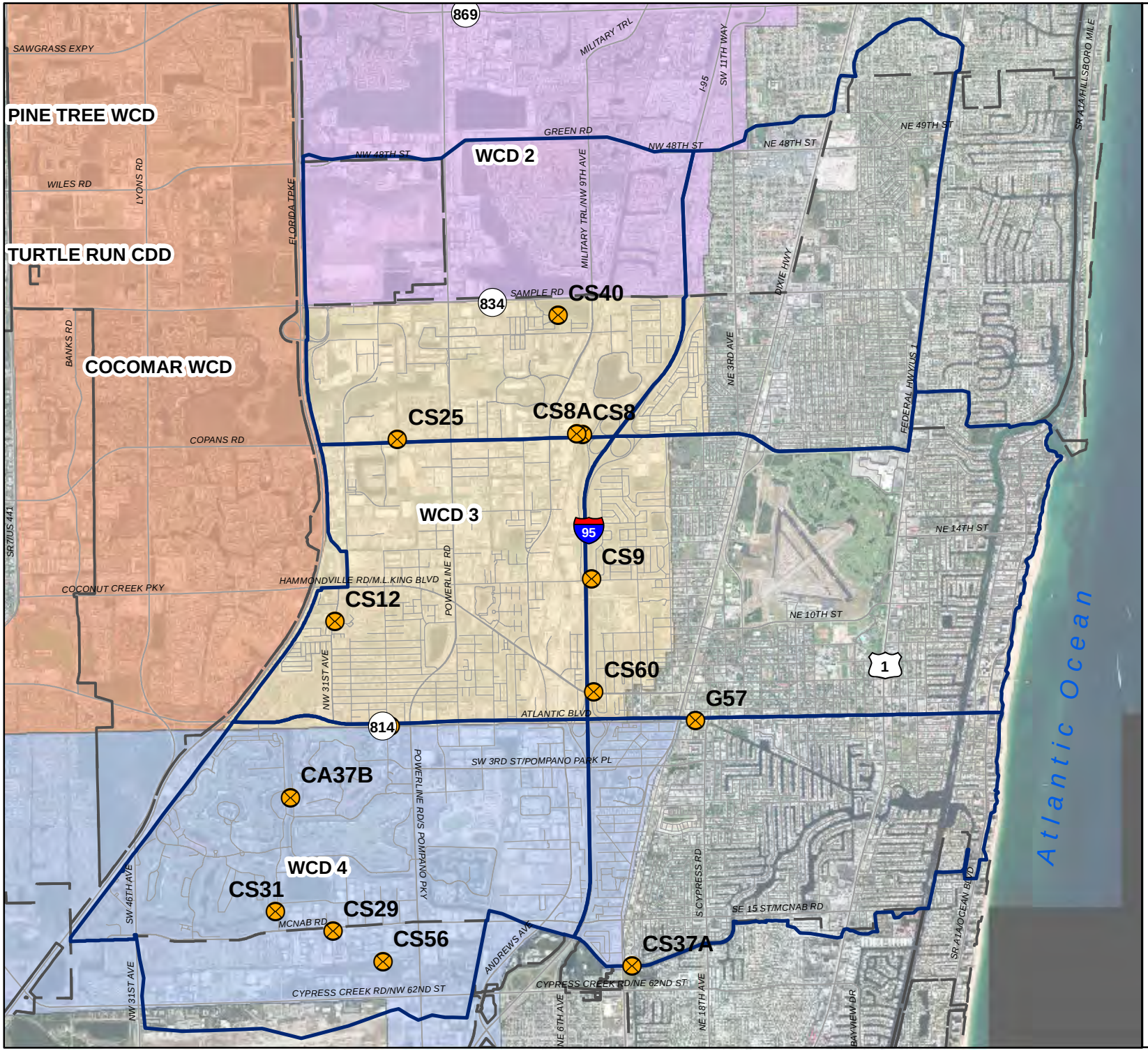
- JUNCTION
- OUTFALL
- STORAGE
- UNDERGROUND CONDUIT

pShape

- BRIDGE
- OVERLAND FLOW
- WEIR
- OVERLAND FLOW
- Major Basin Boundary
- City Boundary
- Sub-Basins

N W E S

0 750 1,500 3,000 Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-4
Control Structure Location Map

Legend

- Control Structures
- Major Basin Boundary
- City Boundary
- Streets

DISTRICT

- Cocomar WCD
- Broward WCD 2
- Broward WCD 3
- Broward WCD 4

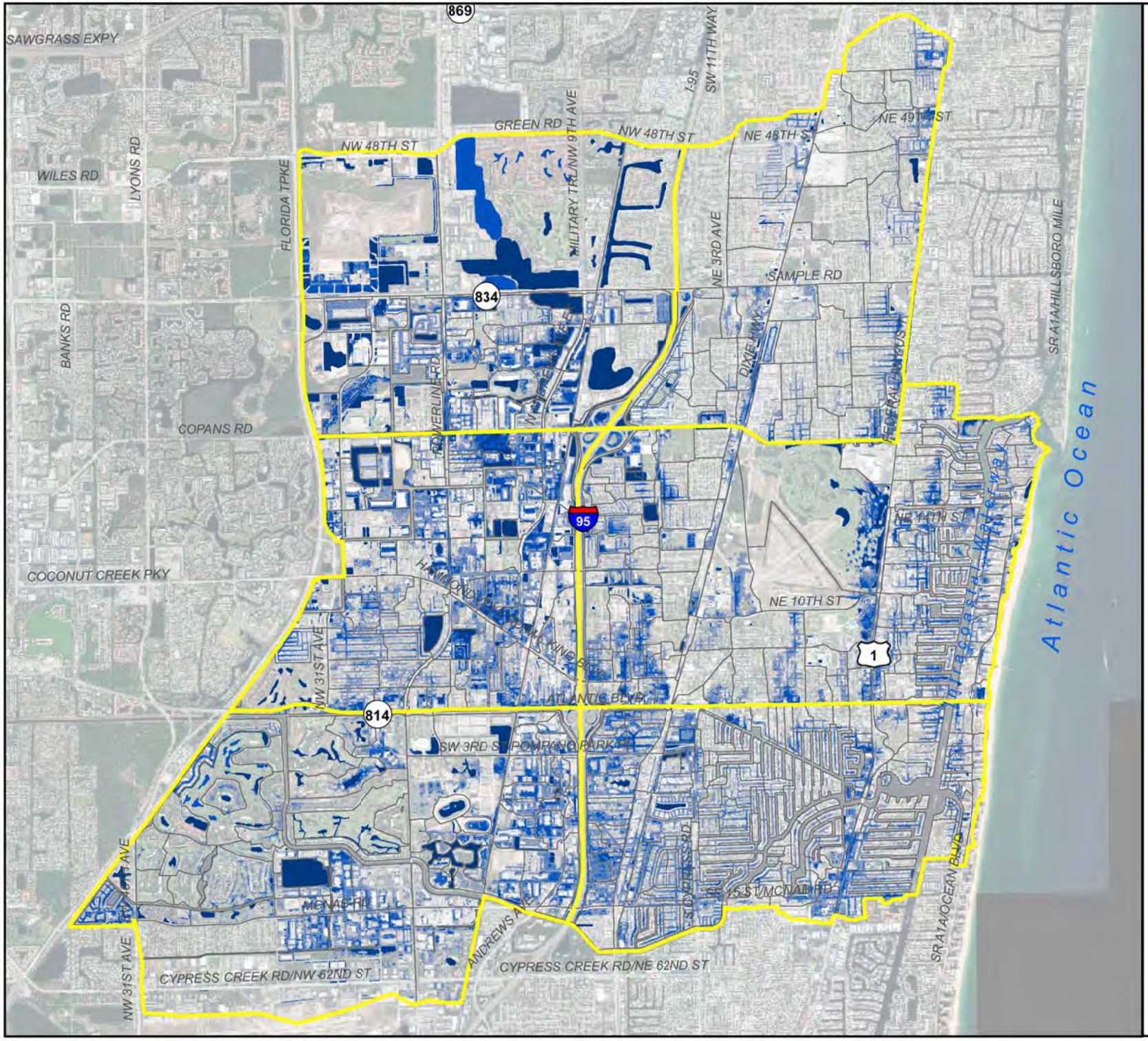


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Figure 4-13
Potential Flooding Map

Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

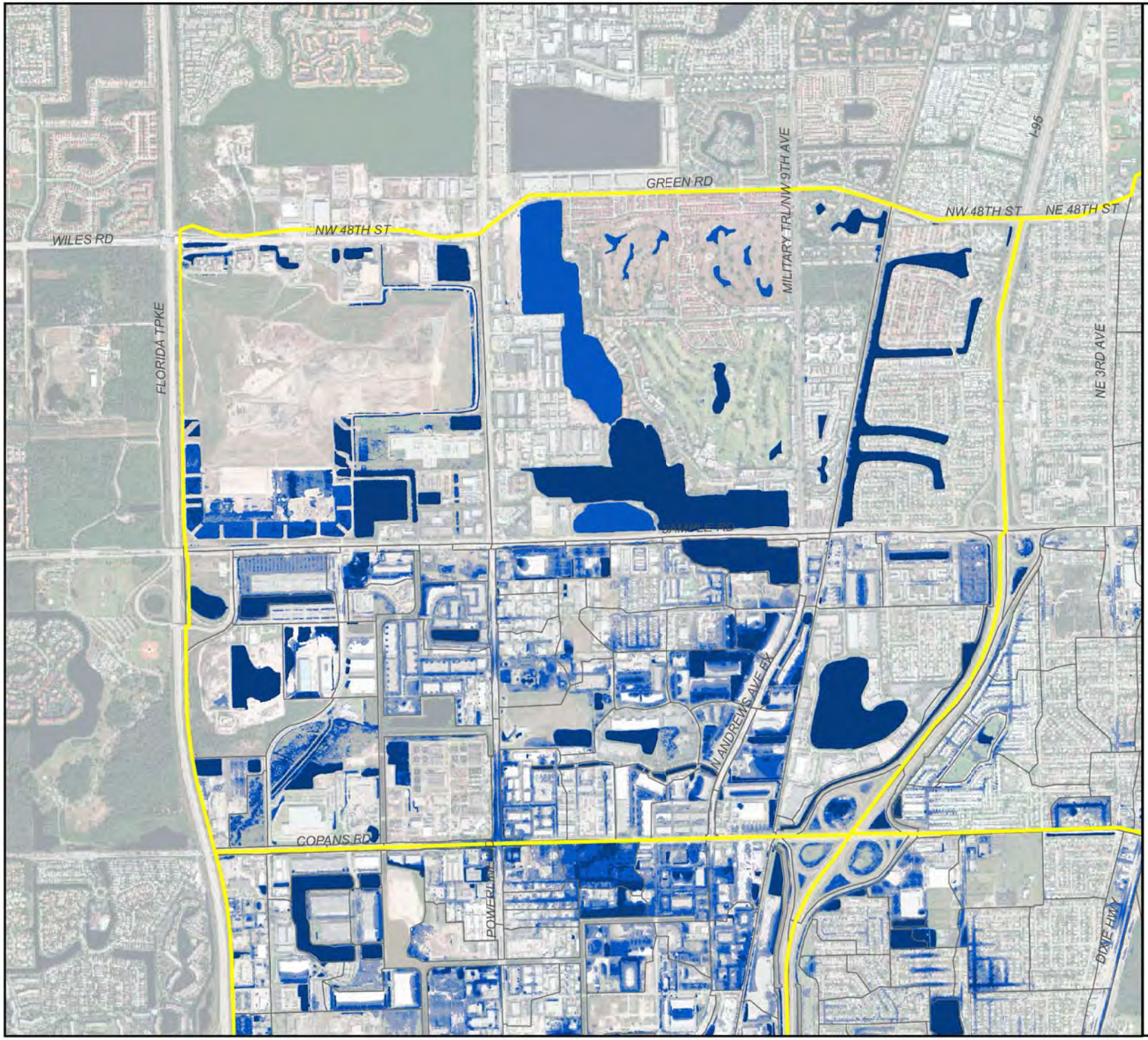

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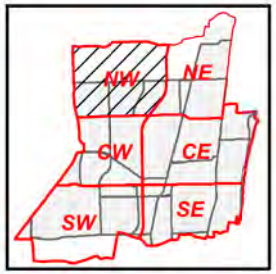
Figure 4-13A
Potential Flooding Map
Basin NW



Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

N
W E
S

0 750 1,500 3,000
Feet

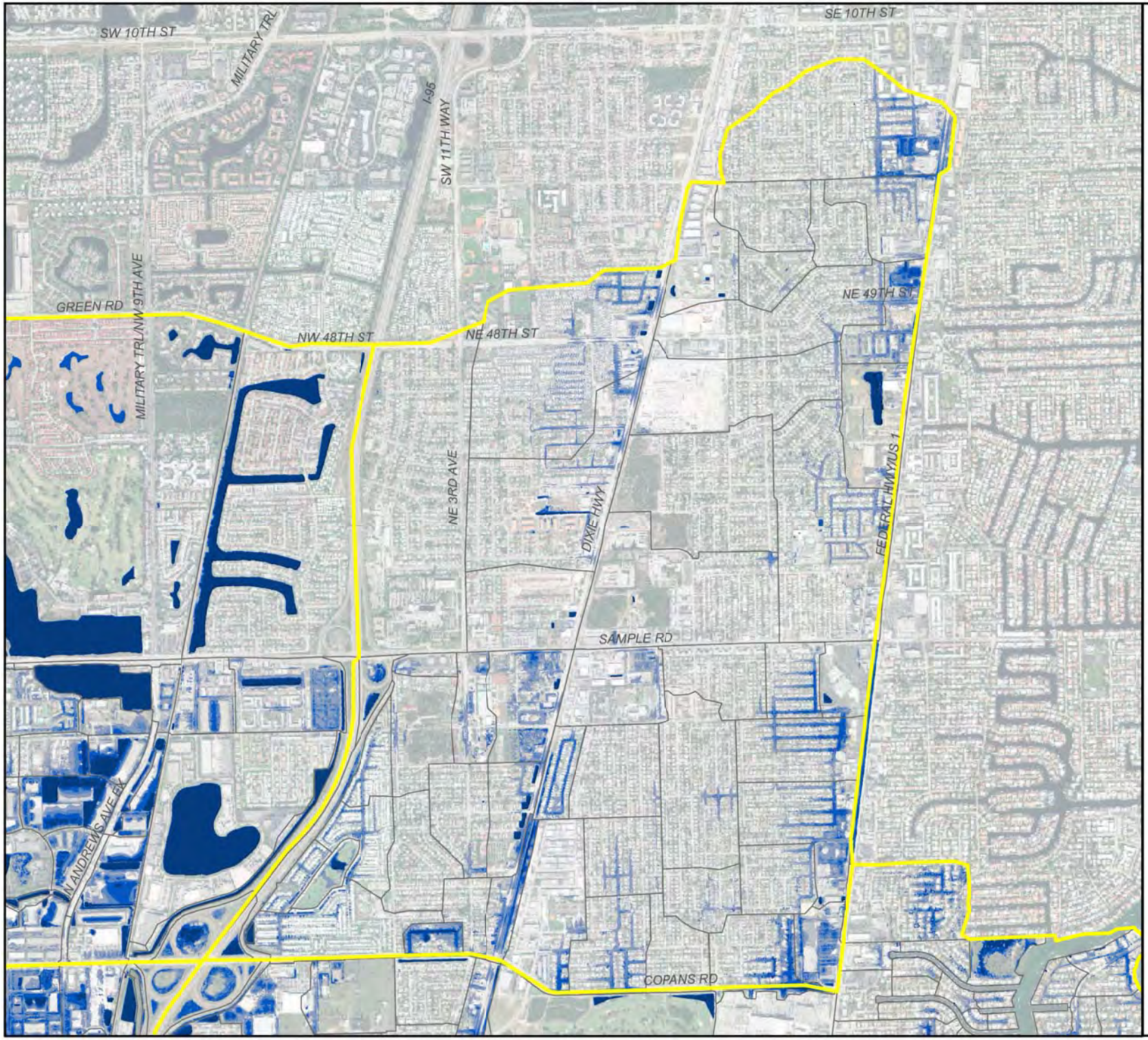


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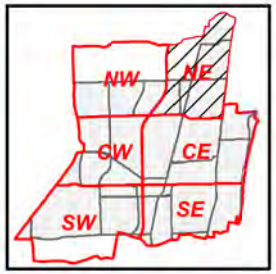
Figure 4-13B
Potential Flooding Map
Basin NE



Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

N
W E
S

0 750 1,500 3,000
Feet

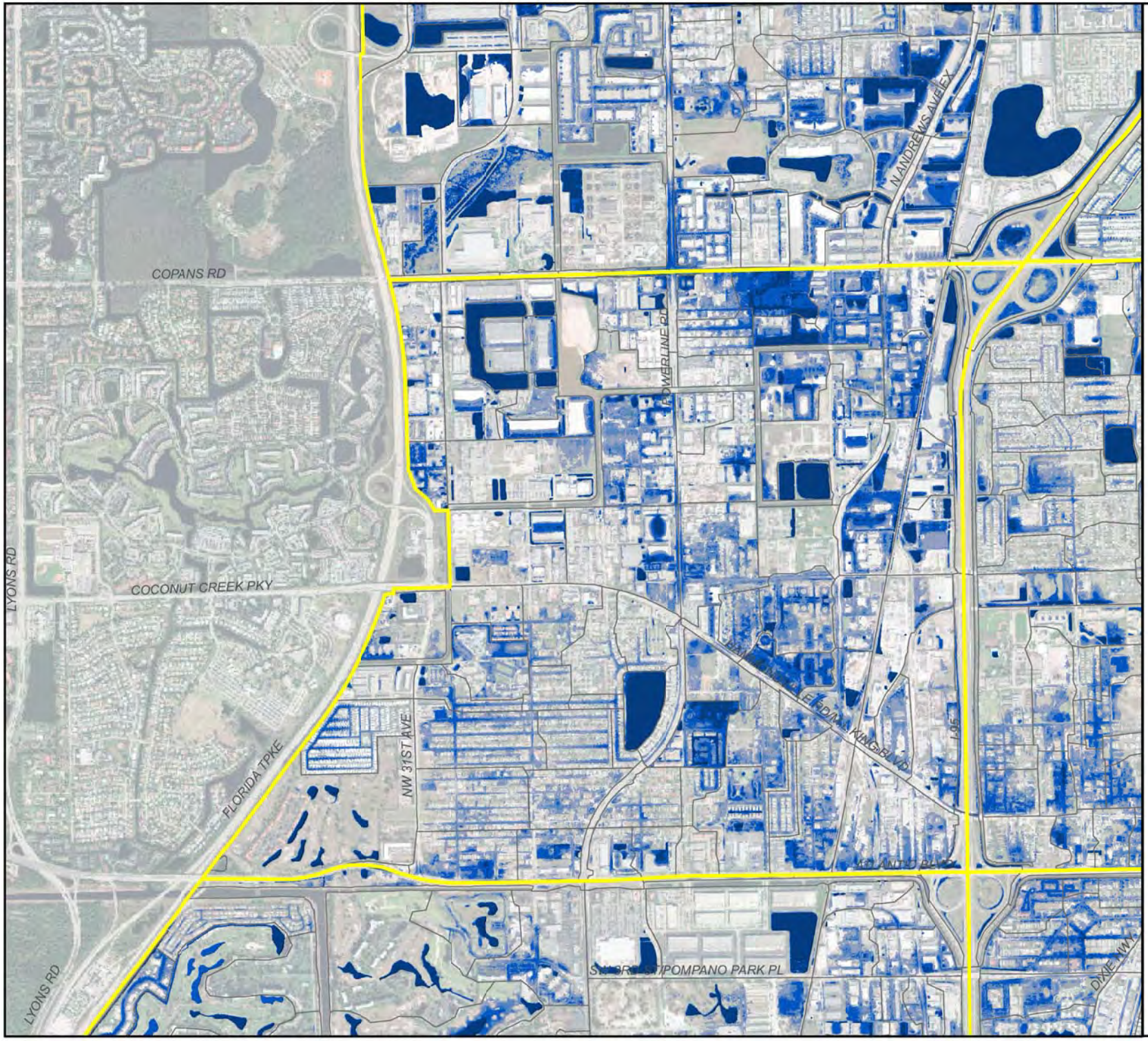


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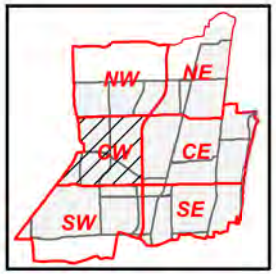
Figure 4-13C
Potential Flooding Map
Basin CW



Legend

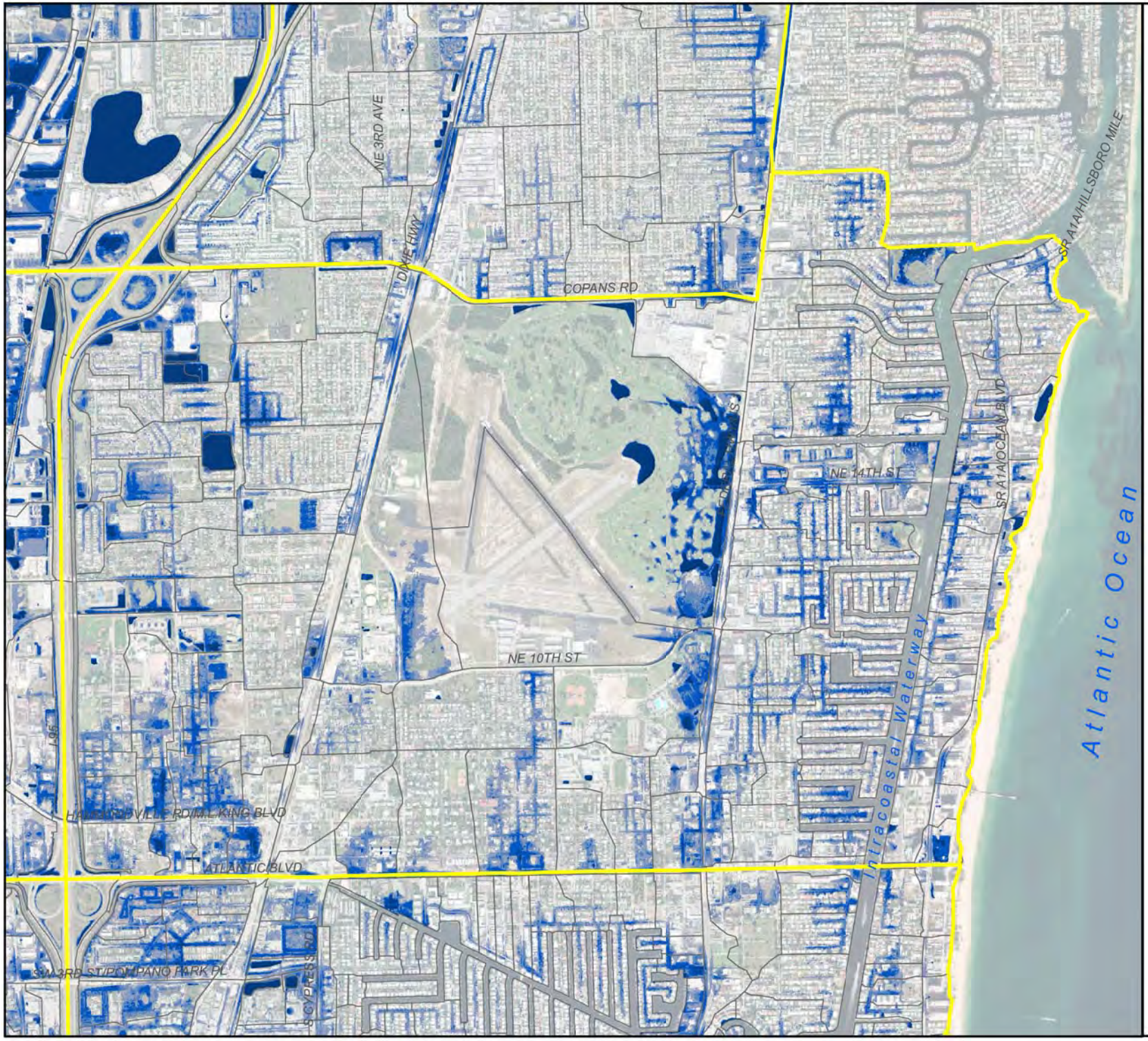
- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

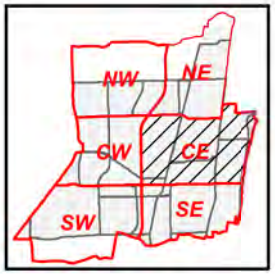
N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-13D
Potential Flooding Map
Basin CE



Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

N
W E
S

0 750 1,500 3,000
Feet

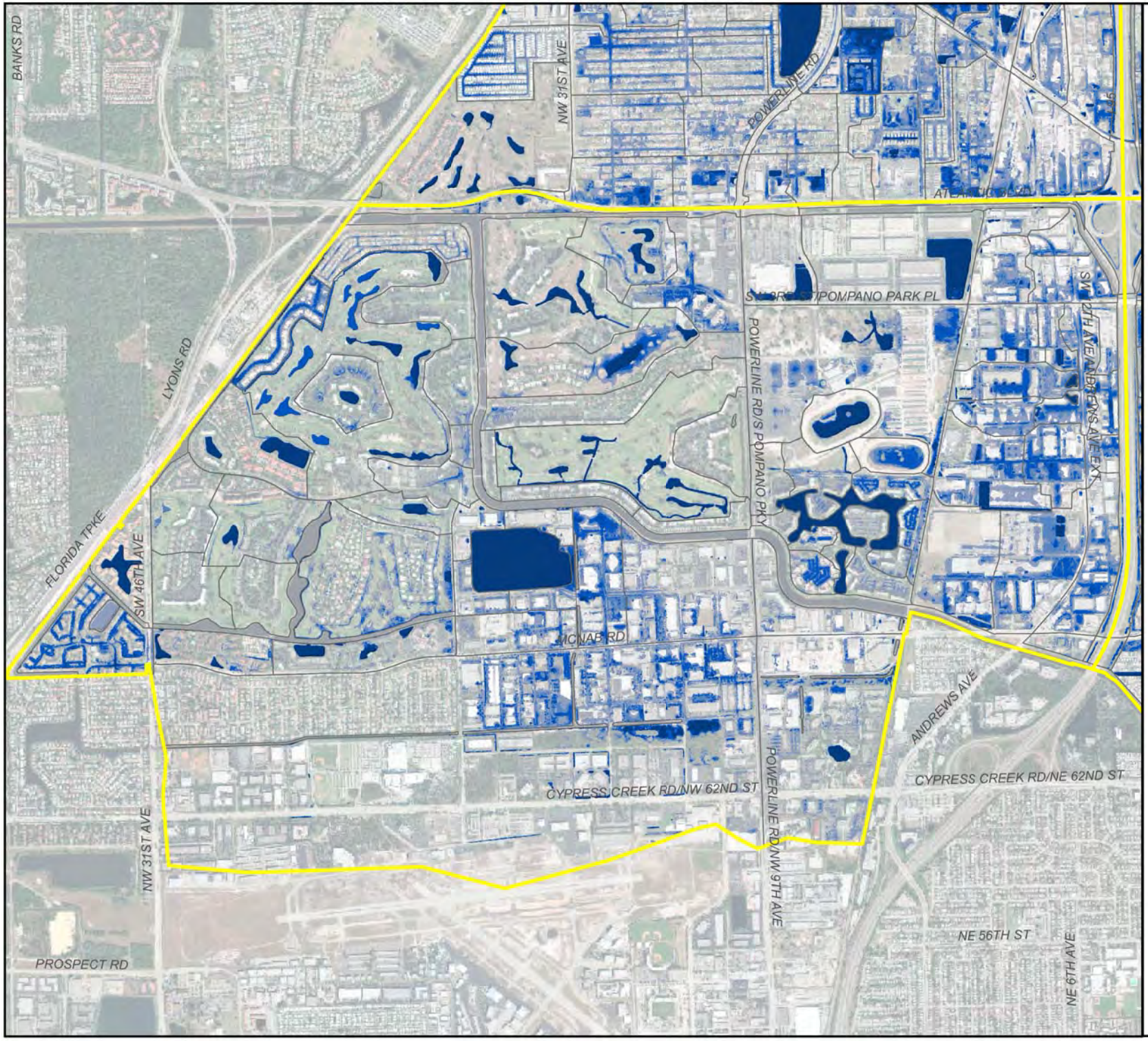


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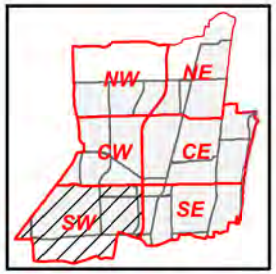
Figure 4-13E
Potential Flooding Map
Basin SW



Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

N
W E
S

0 750 1,500 3,000
Feet

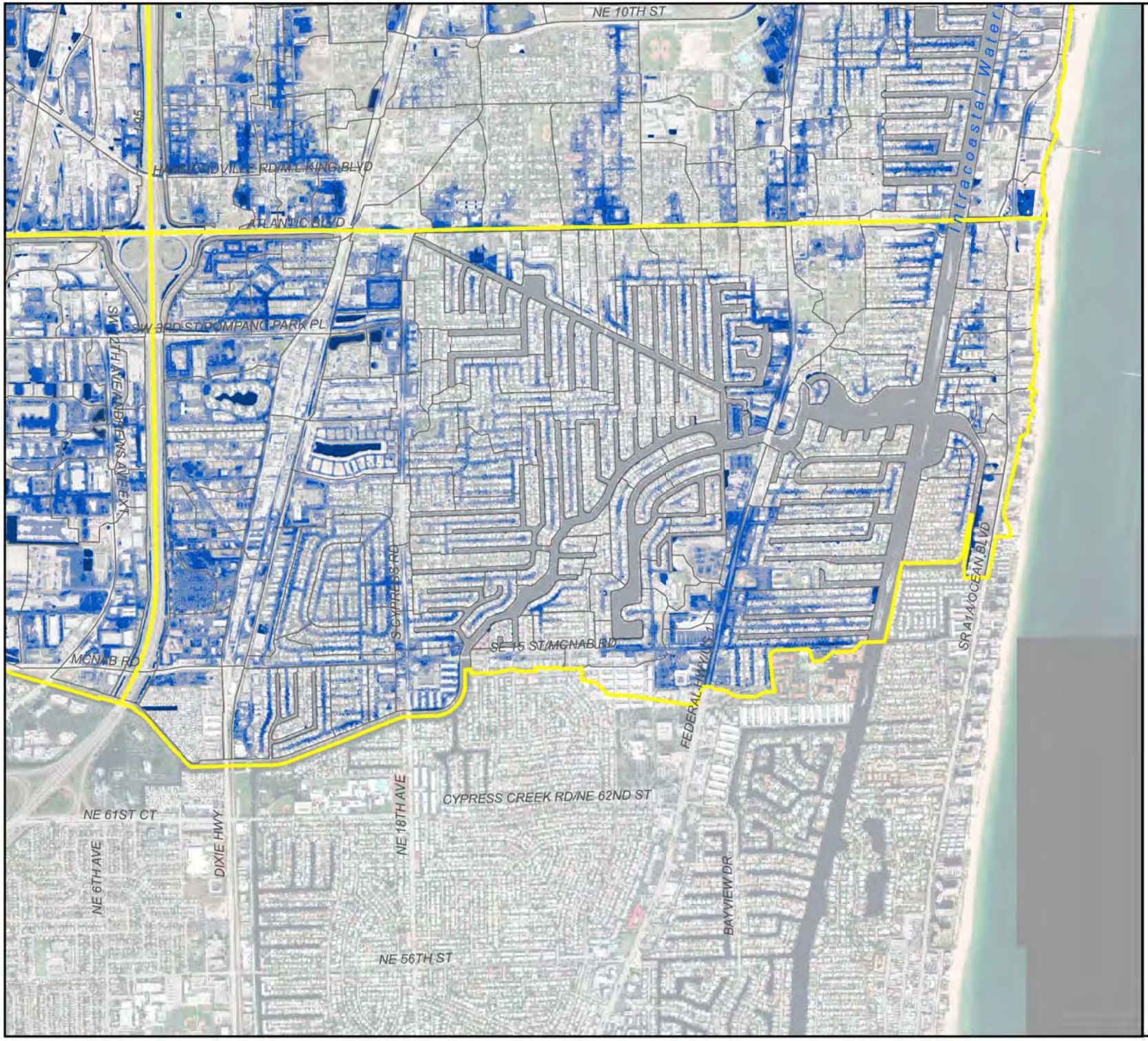


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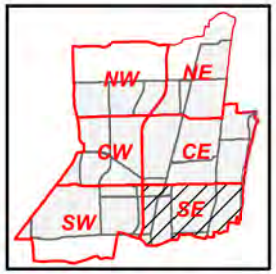
Figure 4-13F
Potential Flooding Map
Basin SE



Legend

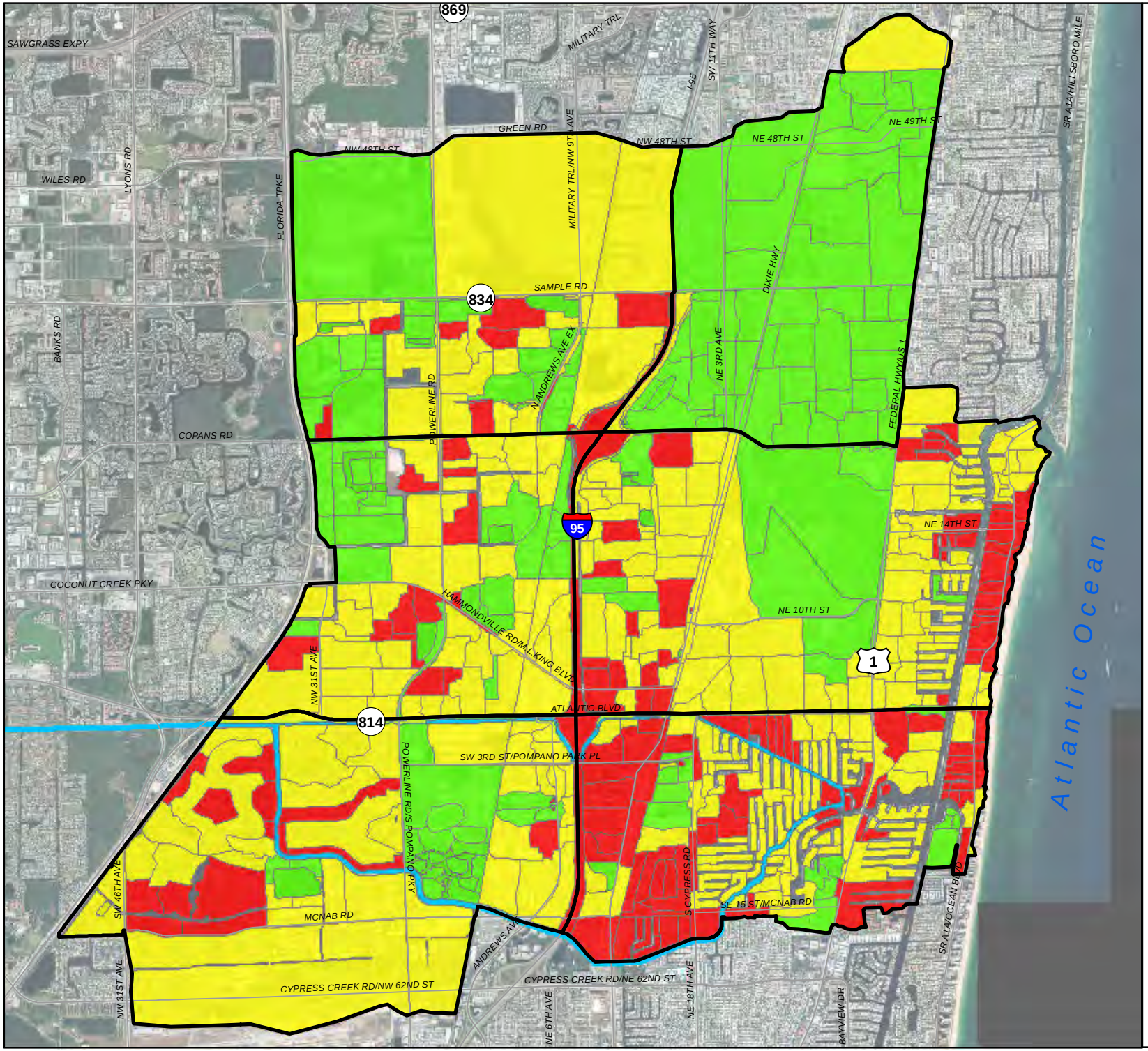
- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

N
W E
S

0 750 1,500 3,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-14
Estimated TN Yield (lb/Ac)

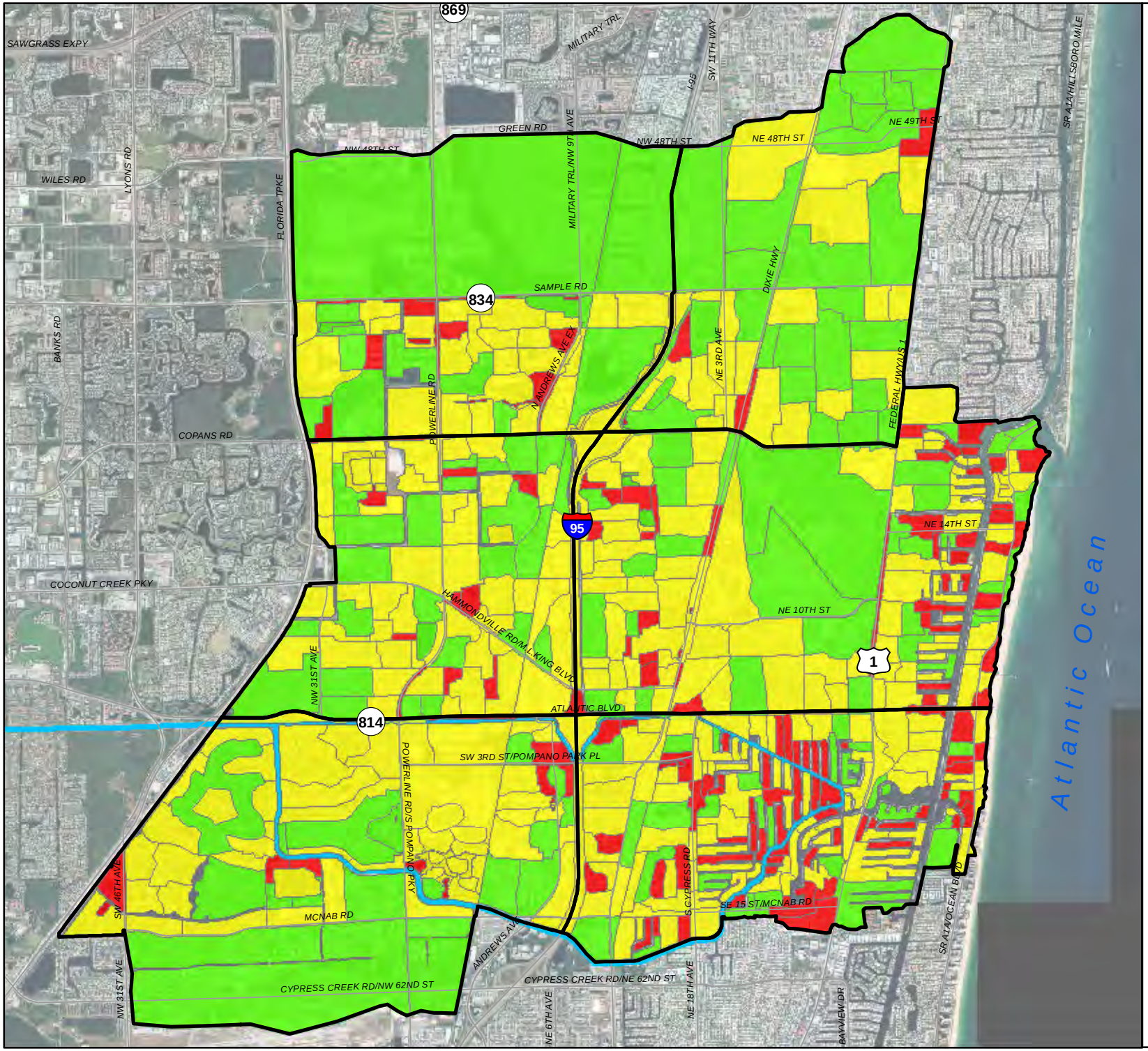
Legend

TN Yield (lb/Ac)

- < 25th Percentile (1.92)
- 25th to 75th Percentile (1.92 - 6.82)
- > 75th Percentile (6.82)
- Major Basins
- Canal

N
W E
S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-15
Estimated TP Yield (lb/Ac)

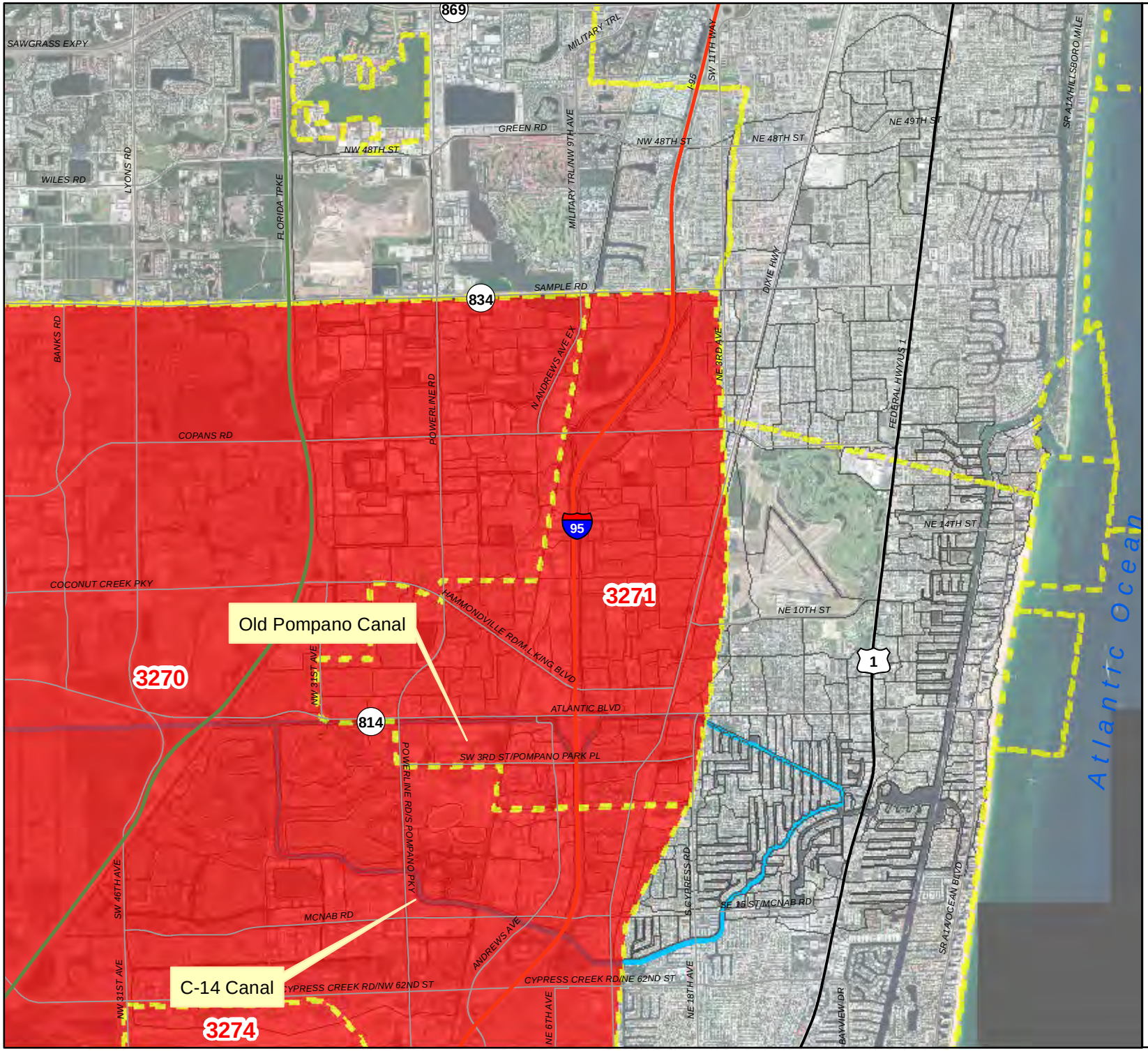
Legend

TN Yield (lb/Ac)

- < 25th Percentile (0.15)
- 25th to 75th Percentile (0.15 - 1.88)
- > 75th Percentile (1.88)
- Major Basins
- Canal

N
W E
S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

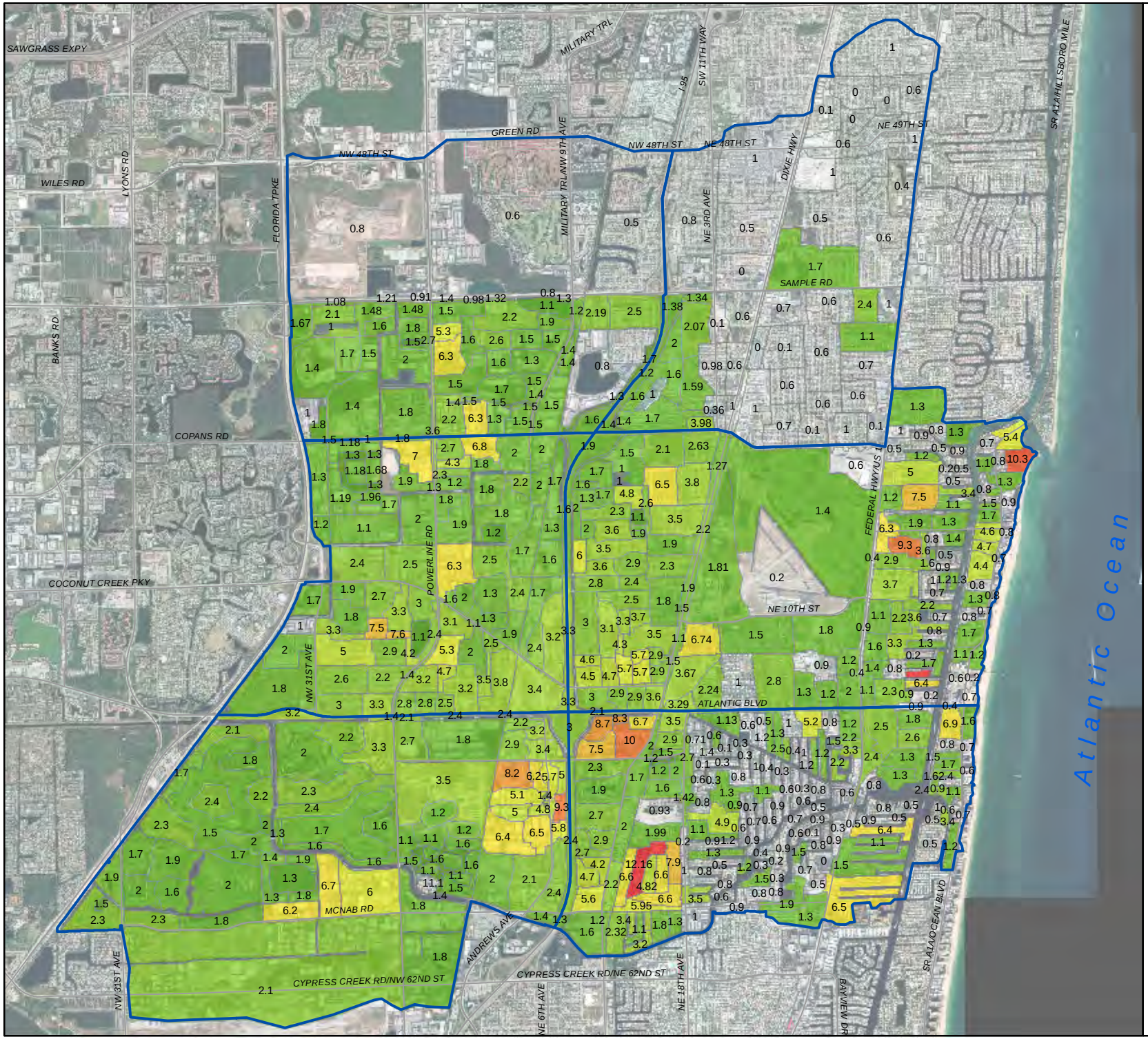
Figure 4-16
WBID Map

Legend

- WBID Boundary
- Impaired WBID
- Subbasins
- Canal

N
W E
S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-17
Sub-Basin
Prioritization Map

Legend

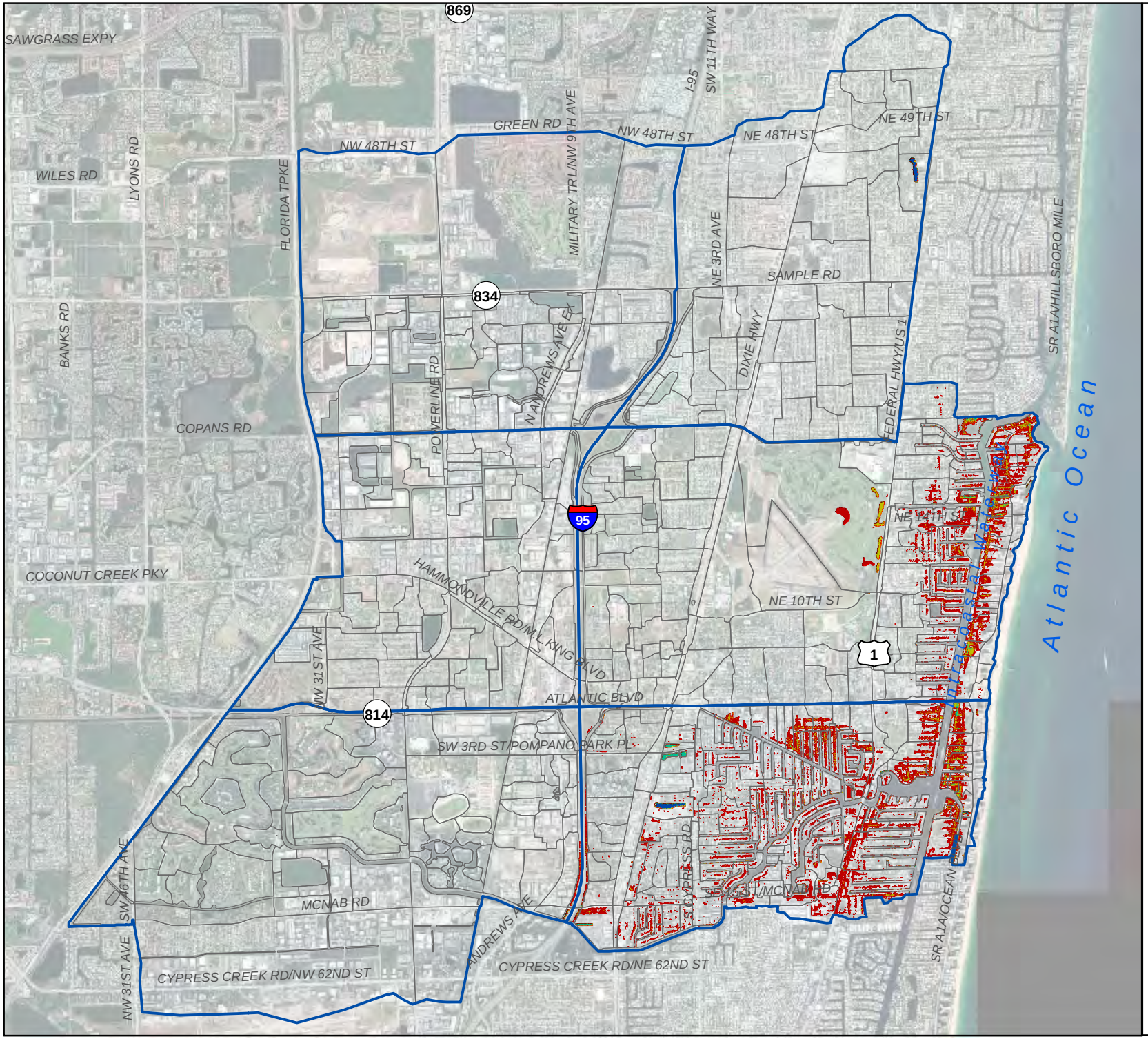
Major Basins

Sub-Basin Prioritization Value

- 0.0 - 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.1
- 4.2 - 5.1
- 5.2 - 6.1
- 6.2 - 7.1
- 7.2 - 8.1
- 8.2 - 9.1
- 9.2 - 10.1
- 10.2 - 11.1
- 11.2 - 12.2

N
W E
S

0 1,500 3,000 6,000
Feet



City of Pompano Beach
Stormwater Master Plan

Figure 4-20
Sea Level Rise
Vulnerability Map

Legend

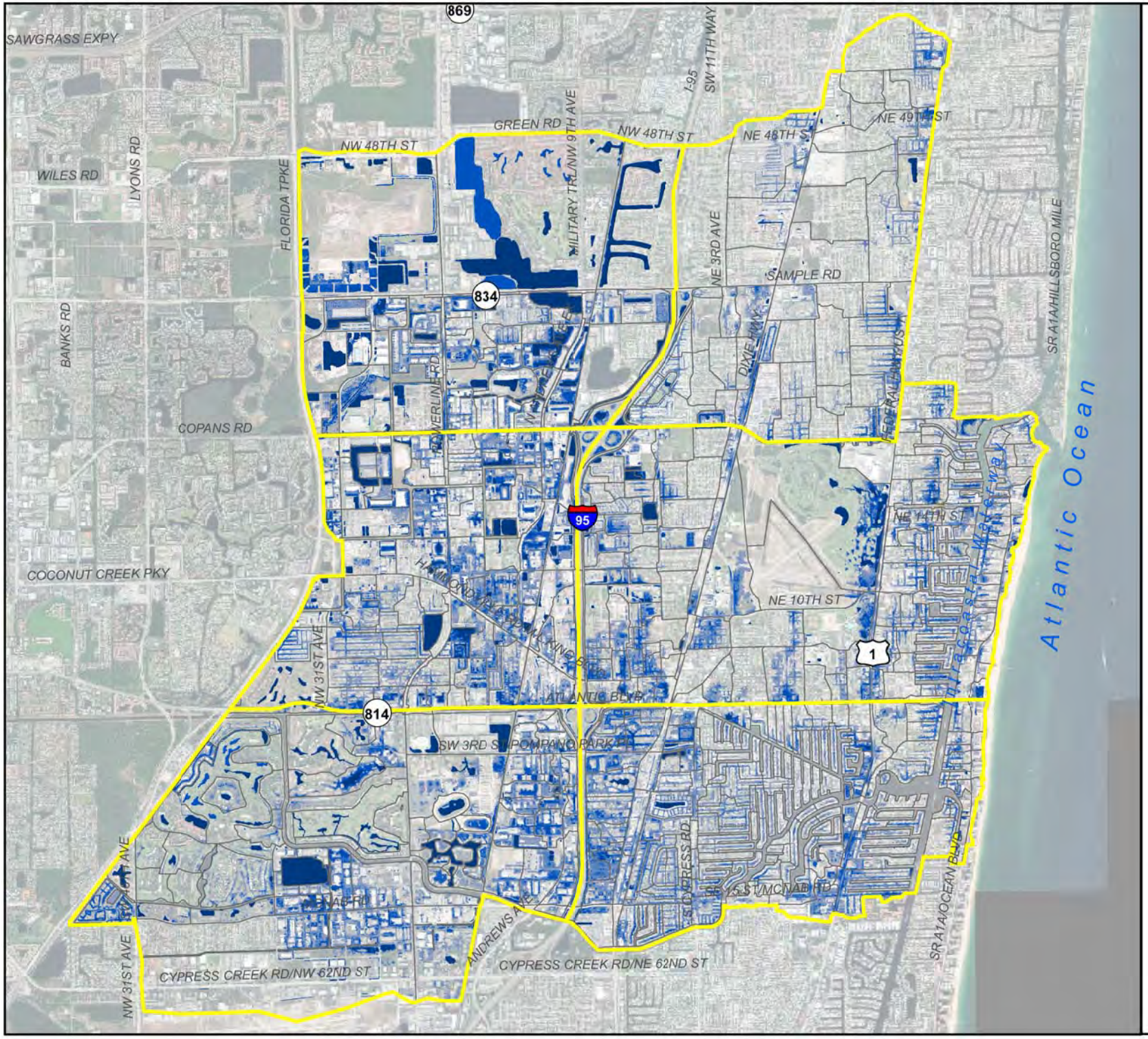
- Major Basins Boundary
- Sub-Basins

Elevation

- 3 Ft Sea Level Rise (+3.4 NAVD)
- 2 Ft Sea Level Rise (+2.4 NAVD)
- 1 Ft Sea Level Rise (+1.4 NAVD)
- Current Sea Level (+0.4 NAVD)

N
W E
S

0 1,500 3,000 6,000
Feet



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Stormwater Master Plan

Figure 4-21
Sea Level Rise
Inundation Map
(5-year Storm + 1 foot SLR)

Legend

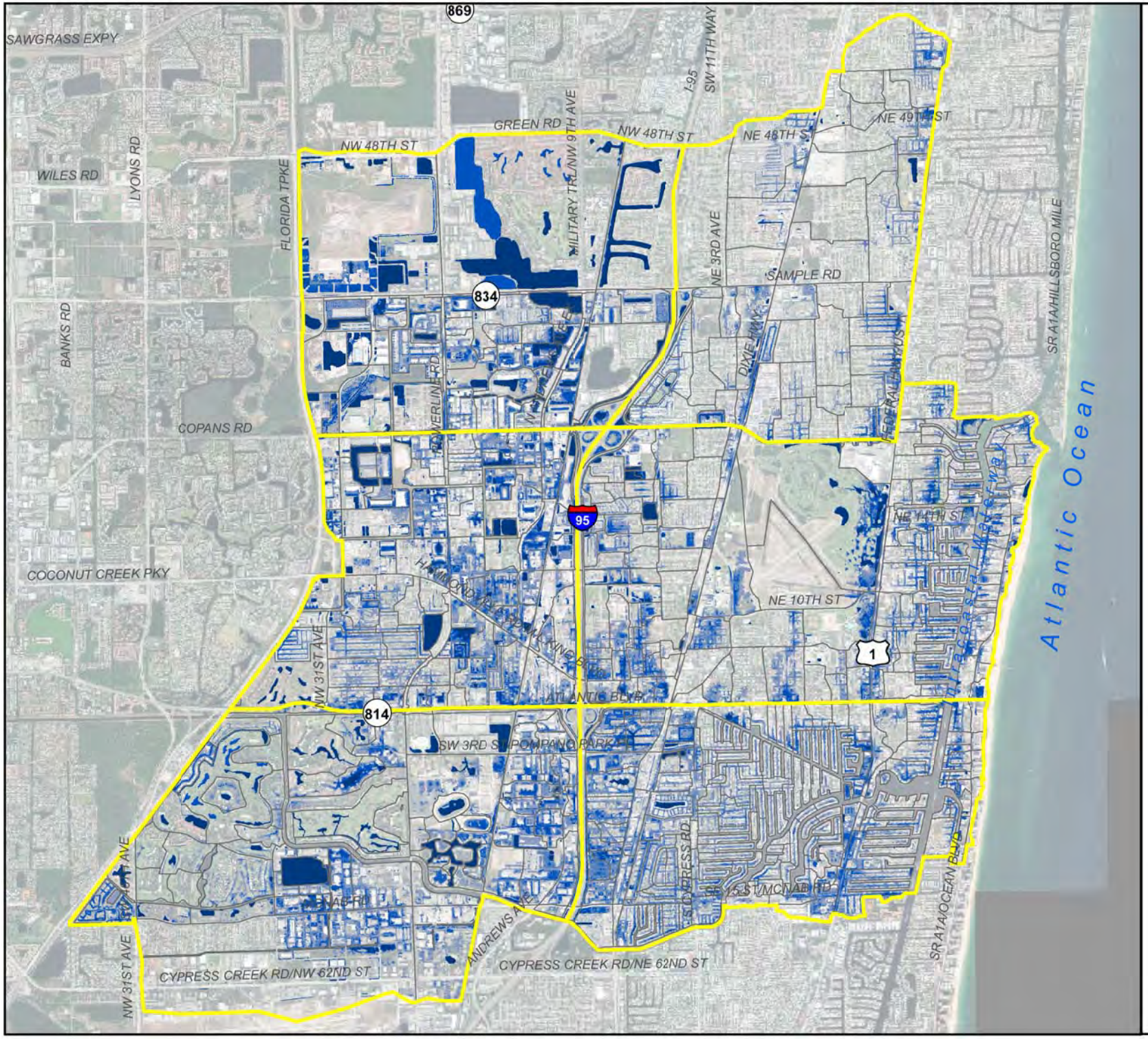
- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

N
W E
S

0 1,500 3,000 6,000
Feet



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Stormwater Master Plan

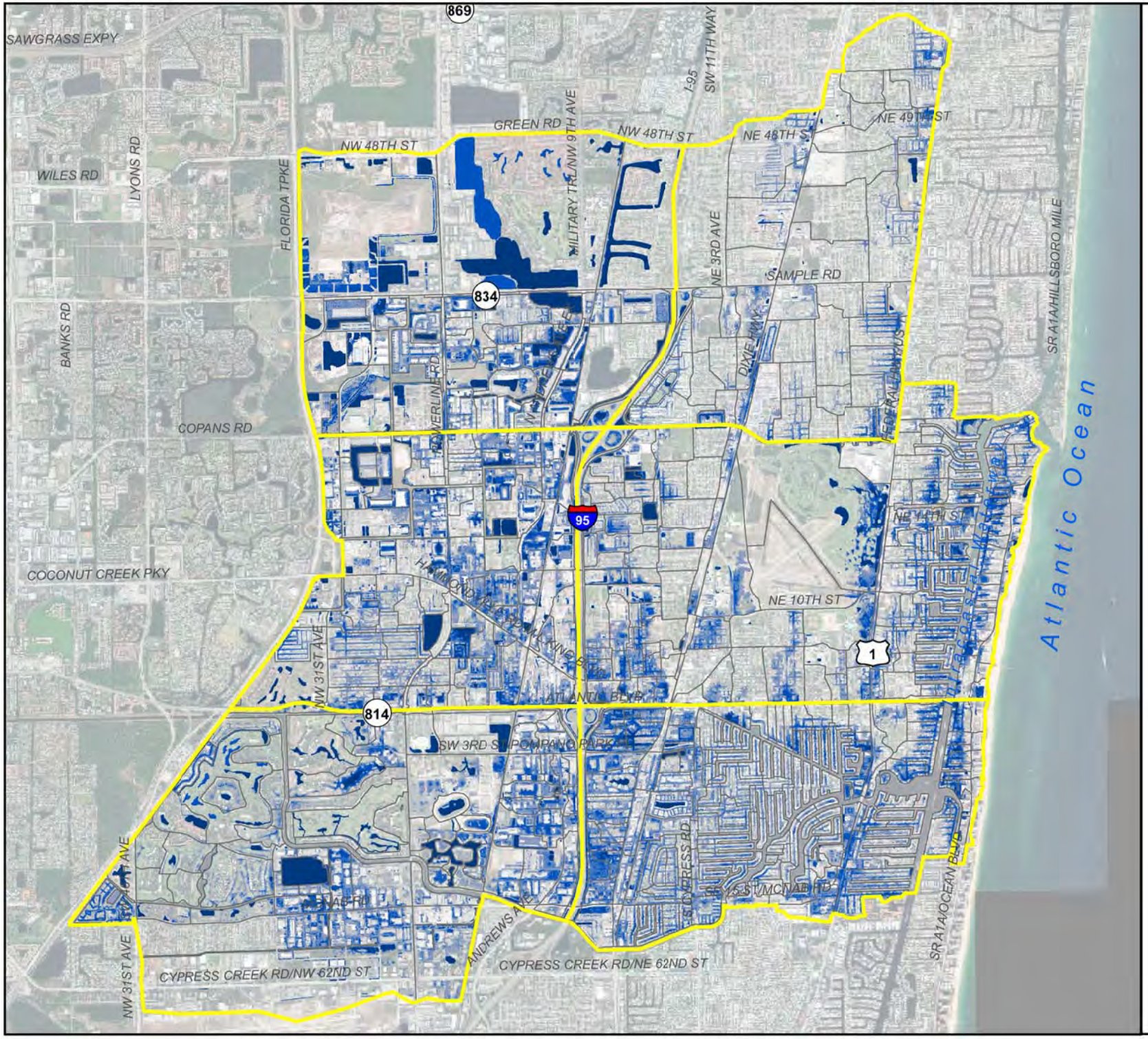
Figure 4-22
Sea Level Rise
Inundation Map
(5-year Storm + 2 feet SLR)

Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5



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City of Pompano Beach
Stormwater Master Plan

Figure 4-23
Inundation Map
(5-year Storm + 3 feet SLR)

Legend

- Sub-Basins
- Major Basins Boundary

Flood Depth

- > 2
- 1 - 2
- 0.5 - 1
- 0 - 0.5

SECTION 5 – ANALYSIS OF SYSTEM IMPROVEMENT ALTERNATIVES

5.1 OVERVIEW

CMA has developed various system improvement alternatives for areas throughout the City in need of stormwater improvements based our analysis of the existing conditions. The goal of these system improvement alternatives is to meet level of service criteria for flood control of the public right of way areas along with providing additional water quality benefits. These various system improvement alternatives which were considered for these areas include the installation of exfiltration trench systems, the interconnection with the adjacent existing stormwater systems, the upsizing of existing stormwater pipes, the construction of retention areas, the installation of stormwater pump stations, the implementation of backflow prevention devices at existing outfalls, the installation of drainage wells, and the regrading of roadway swale areas. Each of these potential system improvement alternatives were evaluated in multiple configurations within each area to determine which would be the most effective at alleviating the existing flooding within the public right of way areas. For comparison purposes, CMA used the stormwater model to analyze the effectiveness of these system improvement alternatives at improving the performance to the existing stormwater management system within each area of the City. Each system improvement alternative was compared based on its ability to reduce the peak flood stage within the study area and to reduce the flood duration within the study area. Our analysis of each system improvement alternative has been summarized within the following sections.

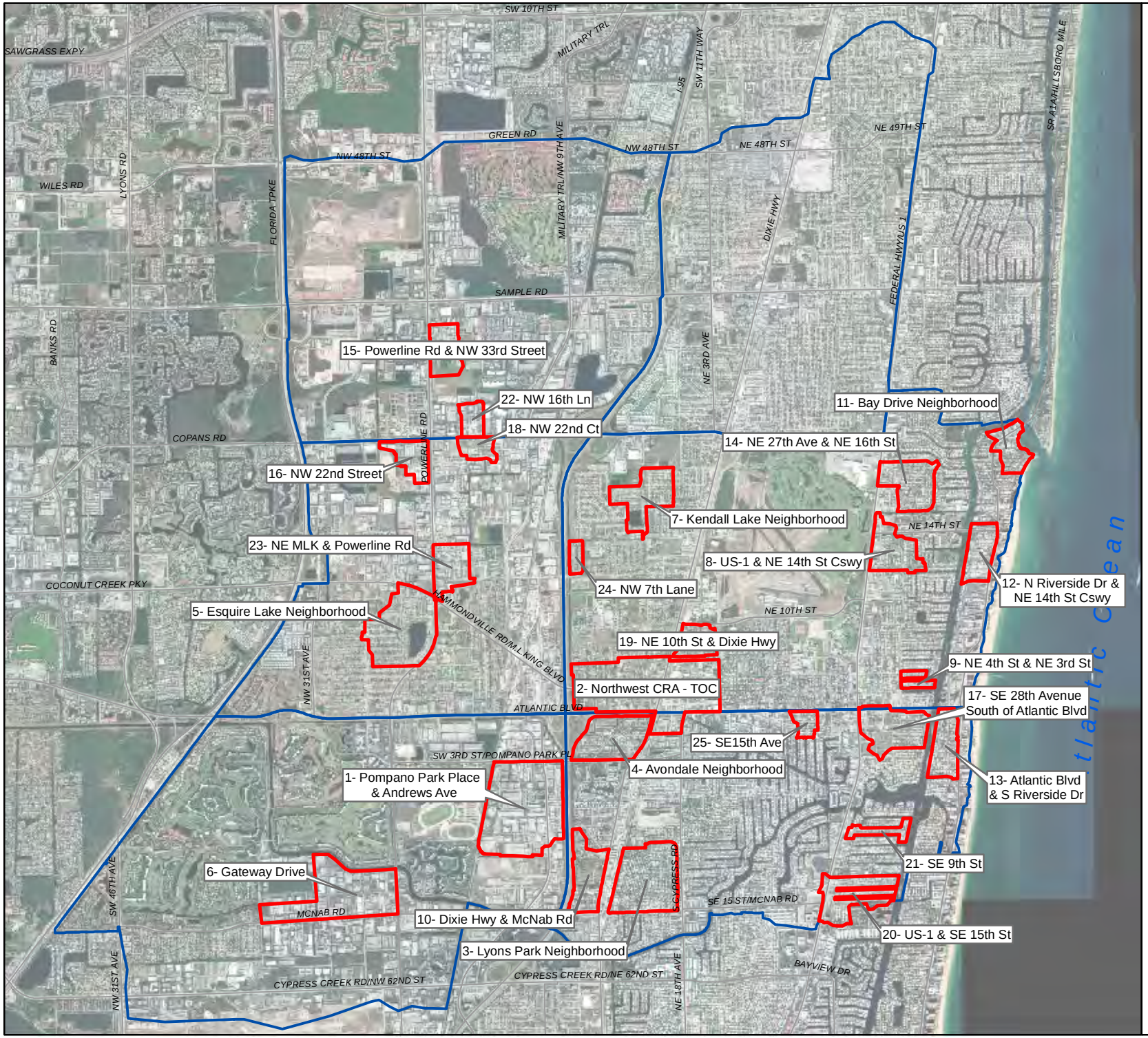
5.2 STUDY AREAS

As previously discussed in Section 4.10 of this report, all sub-basins within the City were ranked for the purpose of identifying potential flooding problem areas of the City. Based on the results of the basin prioritization formula, all sub-basins within the City were ranked to identify which are most in need of stormwater improvements. Any sub-basin within the City with a basin prioritization formula score of 4.5 or greater was identified to a priority sub-basin most in need of stormwater improvements. CMA closely reviewed the topography to verify whether these priority sub-basins are part of a larger drainage basin which can allow the flow of stormwater runoff between adjacent sub-basins. CMA also reviewed the extent of the existing stormwater system within these priority sub-basins to verify any direct pipe connections with adjacent sub-basins. CMA conducted field visits for each priority sub-basin during various rainfall events for the purpose of identifying flood prone areas. Photographs were taken of localized street flooding to document our observations. A photo study of the priority basin can be found in Appendix A-5. These site visits confirmed that flooding does occur in areas which were identified as flood prone by the existing conditions model.

Depending on the ground surface topography and the extent of the existing stormwater system, these priority sub-basins were often grouped together with adjacent sub-basins into major study areas for more effective analysis with the stormwater model. Sub-basins which can be considered to be hydraulically isolated from other adjacent sub-basins were analyzed individually with the stormwater model. The limits of these priority study areas are displayed on Figure 5 Priority Study Area Locations within this section. These study areas were ranked by tabulating the basin prioritization formula score for all sub-basins included within the boundaries of these study areas. Each study area has an individual section of this report, which includes a topographic map of the study area, potential flooding map of the study area, the model results for the existing conditions and each system improvement alternative, and a recommendation of the most effective system improvement alternative. The ranking of the study areas along with the associated prioritization score is summarized within Table 5 below.

Table 5 – Study Area Ranking		
Priority	**Study Area Name**	**Prioritization Total**
1	Pompano Park Place and Andrews Avenue	70.80
2	Northwest CRA - TOC	61.80
3	Lyons Park Neighborhood	50.63
4	Avondale Neighborhood	41.20
5	Esquire Lake Neighborhood	24.50
6	Gateway Drive	18.90
7	Kendall Lake Neighborhood	18.60
8	US-1 and NE 14th Street Causeway	18.50
9	NE 4th Street and NE 3rd Street	17.60
10	Dixie Highway and McNab Road	17.20
11	Bay Drive Neighborhood	15.70
12	North Riverside Drive and NE 14th Street Causeway	13.70
13	South Riverside Drive and Atlantic Boulevard	13.30
14	NE 27th Avenue and NE 16th Street	12.50
15	Powerline Road and NW 33rd Street	11.60
16	NW 22nd Street	7.00
17	SE 28th Avenue South of Atlantic Boulevard	6.90
18	NW 22nd Court	6.80
19	NE 10th Street and Dixie Highway	6.70
20	US-1 and SE 15th Street	6.50
21	SE 9th Street	6.40
22	NW 16th Lane	6.30
23	NE MLK and Powerline Road	6.30
24	NW 7th Terrace	6.00
25	SE 15th Avenue	5.20

For comparison purposes, CMA used the stormwater model to analyze the effectiveness of various system improvement alternatives at reducing flooding within each study area. The purpose of this analysis is to identify which type of stormwater system improvement is most effective at reducing the peak flood stage and flooding duration within each study area. CMA also compared the estimated implementation costs for each system improvement within each study area. Based on the model results and estimated implementation costs, CMA identified which system improvement alternative is most effective for each study area. Once the system improvement alternative was selected for each study area, CMA developed conceptual layouts and preliminary cost estimates for the proposed improvements within each study area, which are included within Appendix A. Our analysis of the system improvement alternatives for each study area is summarized within the following sections.



City of Pompano Beach
Stormwater Master Plan

Figure 5
Priority Study Area
Location Map

Legend

- Project Areas
- Major Basins

5.2.1 STUDY AREA 1 – POMPANO PARK PLACE AND ANDREWS AVENUE

This study area is located on the west side of the I-95, east of Andrews Avenue and south of Pompano Park Place. This study area mainly consists of industrial and commercial properties with high amounts of impervious ground surface. The majority of these commercial properties have their own on-site drainage system or along the private roadways. Some properties located at the center of the study area discharge to two large lakes at SW 6th Street and Andrews Avenue which overflow through a control structure into the Andrews Avenue stormwater system.

The existing conditions stormwater model was used to evaluate performance of the existing stormwater system within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The topography of the study area as well as the model schematics are displayed Figure 5-1A. within this section The project area is enclosed by the sub-basins SW_027_03, SW_027_04, SW_029_11, SW_029_12, SW_029_13, SW_029_14, SW_029_15, SW_029_15W, SW_029_16, SW_029_17, SW_044_01, SW_044_01W, and SW_044_02. As displayed in Figure 5-1B within this section, the many roadways and parcels within this study area have flooding depth greater than one inch. Due to the connectivity of this study area and the Andrews Avenue system, some improvement alternatives included options of improving the Andrews Avenue system. We understand that this may not be a feasible option since the Andrews Avenue system is not owned or maintained by the City. However, these alternatives were analyzed to determine the best solution for reducing flooding within the study area.

Alternative 1: Pipe Size Upgrades

Alternative 1 includes upgrading the pipe sizes along Andrews Avenue from the connection with the lakes at SW 6th Street south to the existing outfall near McNab Road in order to increase transmission capacity. The existing drainage pipes to be removed and upsized include 5,675 linear feet of pipe. The diameter of existing pipe to be removed ranges from 42-inch to 84-inch. Alternative 1 also interconnects this stormwater system with the existing stormwater system east of I-95. CMA conducted model runs of multiple pipe configurations until the outfall size would be limited by regulatory and constructability restrictions.

Under Alternative 1, the proposed improvements include 6,125 linear feet of pipe replacement. The proposed pipe diameters range from 42-inch RCP to 96-inch RCP at the outfall location. The estimated design and construction costs for this pipe size upgrades alternative are approximately $3,602,000. As displayed within Table 5.1.1 below, Alternative 1 is effective in reducing the peak flood stage throughout the study area, with an average reduction of approximately 0.25 feet throughout the study area. At the critical nodes within the study area (Node IN_1037, Node IN_1026, and Node IN_0402), the average peak flood depth was reduced from 1.40 feet under the existing conditions to 1.18 feet under Alternative 1.

Table 5.1.1 – Alternative 1 Peak Flood Stage Summary					
Nodes	**Existing Conditions**			**Alternative 1**	
	Peak Stage (feet)	**Ground Elevation (feet, NAVD)** [5]	**Flood Depth (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**
Key Nodes – West of I-95					
IN_0402	6.50	5.65	0.85	6.49	-0.01
IN_0392	6.47	5.70	0.77	6.48	+0.01
IN_1026	6.26	4.40	1.86	5.89	-0.37
MH_0153	6.38	5.30	1.08	6.12	-0.26
IN_1020	6.42	5.90	0.52	6.36	-0.06
IN_1037	6.24	4.74	1.50	5.97	-0.27
IN_1043	6.24	4.67	1.57	5.96	-0.28
MH_0151	6.20	5.70	0.50	5.57	-0.63
MH_0128	5.09	6.10	0.00	4.85	-0.24
MH_0123	4.64	8.10	0.00	4.59	-0.05
MH_0120	4.26	10.10	0.00	4.30	+0.04
BANDMNB_TW	3.72	11.10	0.00	3.78	+0.05
Key Nodes – East of I-95					
SE09701	5.62	5.20	0.42	5.27	-0.35
IN_0948	5.62	4.50	1.12	5.26	-0.36
MH_0131	5.61	4.60	1.01	5.23	-0.38
MH_0128	5.09	6.10	0.00	4.85	-0.24
I956	3.96	N/A	N/A	4.79	N/A

Under Alternative 1, the reduction of flood duration through the study area is summarized within Table 5.1.2 below. Alternative 1 is effective at reducing the flood duration within the study area. At the critical nodes within the study area (Node IN_1037, Node IN_1026, and Node IN_0402), the average flood duration was reduced from 15.1 hours under the existing conditions to 7.23 feet under Alternative 1.

Table 5.1.2 – Alternative 1 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 1**	**Reduction (%)**
IN_1037	4.74	17.5	8.3	53
IN_1043	4.67	17	7.6	56
MH_0151	5.70	9.8	0	100
IN_1026	4.40	17.5	4.0	77
IN_0402	5.65	10.2	9.4	8
IN_0948	4.50	16.6	7.5	55
MH_0131	4.60	15.1	4.3	72

Alternative 2: Exfiltration Trench

Alternative 2 includes the installation of new exfiltration trench within available right-of-way areas throughout the study area in order to provide additional storage and infiltration capacity to the existing stormwater system. Under Alternative 2, the installation of exfiltration trench was proposed throughout the study area along available public roadways with a ground elevation greater than +5.0 feet NAVD, which results in a maximum potential installation of 7,344 linear feet of exfiltration trench. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,843,000. The total length of proposed exfiltration trench under Alternative 2 is summarized by sub-basin within Table 5.1.3 below.

Table 5.1.3 – Alternative 2 Proposed Exfiltration Trench Summary		
Sub-Basin	**Exfiltration Trench Length (LF)**	**Mean Ground Elevation (feet NAVD)**
SW_027_03	244	+6.23
SW_027_04	155	+6.12
SW_029_11	816	+5.98
SW_029_12	1,409	+5.45
SW_029_13	1,245	+5.91
SW_029_16	211	+5.91
SW_029_17	2,026	+6.33
Total	**7,344**	**+5.98**

CMA conducted an analysis with the stormwater model of Alternative 2 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. The following proposed design parameters were assumed within the stormwater model for the evaluation of Alternative 2:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} cfs/ft^2-ft head

Based on our analysis of Alternative 2 with the stormwater model, the reduction in peak flood stage through the study area is summarized within Table 5.1.4 below. As displayed within the table below, Alternative 2 results in minimal reductions in peak flood stage within the problem area of the study area. At the critical nodes within the study area (Node IN_1037, Node IN_1026, and Node IN_0402), the average peak flood depth was reduced from 1.40 feet under the existing conditions to 1.38 feet under Alternative 2.

Table 5.1.4 – Alternative 2 Peak Flood Stage Summary					
Nodes	**Existing Conditions**			**Alternative 2**	
	Peak Stage (feet)	**Ground Elevation (feet, NAVD)[5]**	**Flood Depth (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**
Key Nodes – West of I-95					
IN_0402	6.50	5.65	0.85	6.47	-0.03
IN_0392	6.47	5.70	0.77	6.48	+0.01
IN_1026	6.26	4.40	1.86	6.25	-0.01
MH_0153	6.38	5.30	1.08	6.73	+0.35
IN_1020	6.42	5.90	0.52	6.40	-0.02
IN_1037	6.24	4.74	1.50	6.23	-0.01
IN_1043	6.24	4.67	1.57	6.23	-0.01
MH_0151	6.20	5.70	0.50	6.19	-0.01
MH_0128	5.09	6.10	0.00	5.08	-0.01
MH_0123	4.64	8.10	0.00	4.62	-0.02
MH_0120	4.26	10.10	0.00	4.22	-0.04
BANDMNB_TW	3.72	11.10	0.00	3.68	-0.04
Key Nodes – East of I-95					
SE09701	5.62	5.2	0.42	5.61	-0.01
IN_0948	5.62	4.5	1.12	5.61	-0.01
MH_0131	5.61	4.6	1.01	5.61	0.00
MH_0128	5.09	6.1	0.00	5.08	-0.01
I956	3.96	N/A	N/A	3.90	-0.06

Based on our analysis of Alternative 2 with the stormwater model, the reduction in flood duration is summarized in Table 5.1.5 below, which is minimal for the problem areas of this study area. At the critical nodes within the study area (Node IN_1037, Node IN_1026, and Node IN_0402), the average flood duration was reduced from 15.1 hours to 14.5 feet under Alternative 2.

Table 5.1.5 – Alternative 2 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 2**	**Reduction (%)**
IN_1037	4.74	17.5	17	3
IN_1043	4.67	17	16.4	4
MH_0151	5.70	9.8	8.8	10
IN_1026	4.40	17.5	17.5	0
IN_0402	5.65	10.2	9.0	12
IN_0948	4.50	16.6	N/A	--
MH_0131	4.60	15.1	N/A	--

N/A : no exfiltration systems were proposed here because of low-lying ground.

Alternative 3: Pump Station + Additional Dry Retention Storage

Alternative 3 includes the construction of a pump station at the south end of SW 14th Avenue in order to pump stormwater runoff to new dry retention area(s) throughout the study area in order to provide additional storage volume for stormwater runoff. The pump station would transmit all collected stormwater runoff into a new retention area(s) at a location to be determined by the City in the future. Under this analysis, the proposed storage retention area(s) would encompasses a total area of 1.00 acres. The average ground surface elevation of this study area is about +9.0 feet NAVD. The proposed retention area was assumed to have a perimeter berm elevated up to +12.0 feet NAVD and with a 3:1 internal side slope to the bottom of the retention area at +3.5 feet NAVD. The proposed retention area(s) would need to be interconnected with a weir-type control structure to the existing stormwater system to allow the overflow of stormwater runoff. The control structure was assumed with a weir elevation of +10.5 feet NAVD, which provides a retention volume of 1.6 acre-feet. Based on the configuration of the existing stormwater system along with the contributory area, the proposed pump capacity was assumed to be 15 CFS to transmit to the proposed retention area(s). The estimated design and construction costs for this pump station alternative are approximately $3,088,000. Based on our analysis of Alternative 3 with the stormwater model, the reduction in peak flood stages within the study area is summarized within Table 5.1.6 below, which shows minimal reductions in peak flood stages. At the critical nodes within the study area (Node IN_1037, Node IN_1026, and Node IN_0402), the average peak flood depth was not reduced from the existing conditions under Alternative 3.

Table 5.1.6 – Alternative 3 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 3	
	Peak Stage (feet)	Ground Elevation (feet, NAVD) [5]	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
Key Nodes – West of I-95					
IN_0402	6.50	5.65	0.85	6.52	+0.02
IN_0392	6.47	5.70	0.77	7.89	+1.42
IN_1026	6.26	4.40	1.86	6.27	+0.01
MH_0153	6.38	5.30	1.08	6.38	0.00
IN_1020	6.42	5.90	0.52	6.4	-0.02
IN_1037	6.24	4.74	1.50	6.22	-0.02
IN_1043	6.24	4.67	1.57	6.22	-0.02
MH_0151	6.20	5.70	0.50	6.19	-0.01
MH_0128	5.09	6.10	0.00	5.08	-0.01
MH_0123	4.64	8.10	0.00	4.62	-0.02
MH_0120	4.26	10.10	0.00	4.22	-0.04
BANDMNB_TW	3.72	11.10	0.00	3.67	-0.05
Key Nodes – East of I-95					
SE09701	5.62	5.20	0.42	5.61	-0.01
IN_0948	5.62	4.50	1.12	5.62	0.00
MH_0131	5.61	4.60	1.01	5.61	0.00
MH_0128	5.09	6.10	0.00	5.08	-0.01
I956	3.96	N/A	N/A	3.88	-0.08

Based on our analysis of Alternative 3 with the stormwater model, the reduction in flood duration throughout the study area is summarized in Table 5.1.7 below. As displayed in the model results, Alternative 3 results in minimal reductions in flood duration within the problem area nodes of the study area, with a maximum value of 24% peak reduction. At the critical nodes within the study area (Node IN_1037, Node IN_1026, and Node IN_0402), the average flood duration was reduced from 15.1 hours to 13.73 feet under Alternative 3.

Table 5.1.7 – Alternative 3 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
IN_1037	4.74	17.5	16.8	4
IN_1043	4.67	17	16.4	4
MH_0151	5.70	9.8	8.6	12
IN_1026	4.40	17.5	16.5	6
IN_0402	5.65	10.2	7.8	24
IN_0948	4.50	16.6	16.6	0
MH_0131	4.60	15.1	14.9	1

Alternative Comparison

Refer to Table 5.1.8 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.1.8 refer to the average of the critical problem areas in the study area, which correspond to Node IN_1037, Node IN_1026, and Node IN_0402 within the stormwater model. Based on our analysis with the stormwater model, only Alternative 1 can significantly reduce the peak flood stages and the flood duration within the study area. However, Alternative 1 should be eliminated from consideration since the existing downstream piping in need of upsizing is located within the right-of-way of Andrews Avenue, which is the responsibility of Broward County, not the City of Pompano Beach. Alternative 3 should also be eliminated from consideration since using private property in this study area for stormwater retention is not feasible from a cost standpoint.

Table 5.1.8 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.22	7.87	$3,602,000
Alternative 2	0.02	0.6	$1,843,000
Alternative 3	0.00	1.37	$3,088,000

Based on the ownership of existing drainage facilities within the study area, the extensive existing underground utilities within public right of way areas, and the lack of available property for stormwater storage, a scaled back version of Alternative 2 is the recommended option for improving the stormwater system within this study area. Although Alternative 2 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 2 does provide limited benefits which alleviate the localized flooding problems within the areas with historical flooding and public complaints. Instead of the installation of new exfiltration trench throughout all City

right of way areas, CMA recommends the installation of exfiltration trench in targeted City right-of-ways which address isolated flooding problem locations within the study area. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along SW 9th Terrace and along SW 16th Terrace to address localized flooding problems in this area. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 2 will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed.

If the City wishes to implement comprehensive basin-wide stormwater improvements which would meet level of service throughout all City right-of-way areas within this study area, the only option would be the construction of a new centralized stormwater system which serves the entire study area. This new stormwater system for this study area would need flow into new dry retention area(s) to provide significantly more storage capacity for stormwater runoff. Based on our preliminary calculations, the new dry retention area would need to provide at least 10 acre-feet in additional storage capacity in order to meet level of service criteria within City right-of-way areas. Depending on the configuration of the proposed dry retention area, the City would need between 6 to 12 acres of available land area. Based on our review of the study area, there is not adequate City owned or vacant land available for use for a dry retention area.

In order to implement this option, the City would need to purchase property within the study area, demolish any existing structure, and construct a new dry retention area. Since the bulk of the existing stormwater system flows toward the Andrews Avenue system which is maintained by Broward County, the ideal location of the dry retention area would be in the southeast portion of the study area. Due to the discontinuous nature of the right-of-ways within this study area along with the extensive existing development, the installation of a new pump station would also be required in order to transmit stormwater runoff to the new dry retention area(s). In order for this option to be economically feasible, the proposed improvements would likely need to be partially funded through a special drainage district inclusive of all property owners within this study area. Due to extensive obstacles to option, CMA does not recommend this option without a more detailed feasibility study of this area.

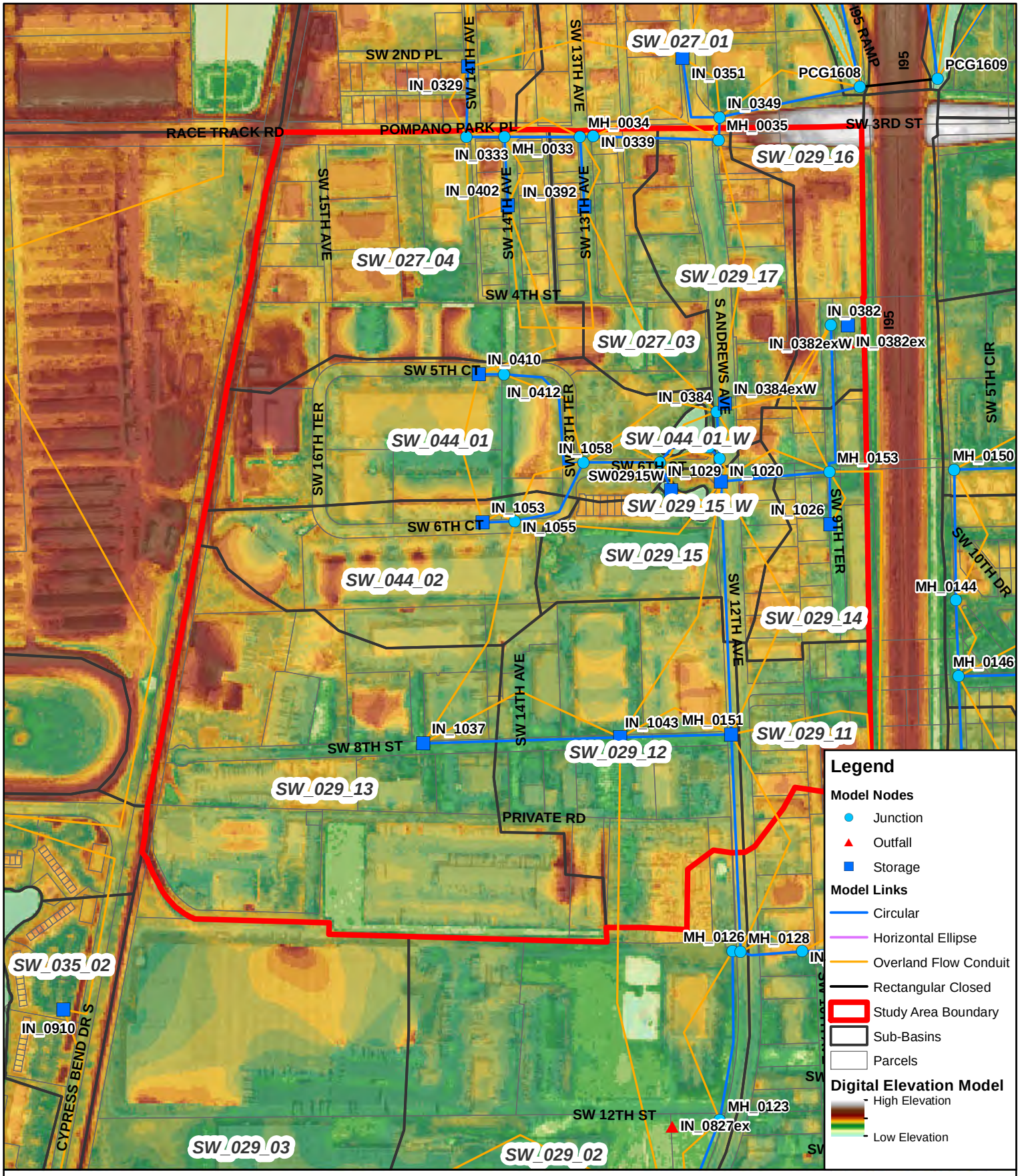


Figure 5-1A
Pompano Park Place & Andrews Avenue
Existing Topography & Model Schematics

N
W E
S

0 300 600
Feet

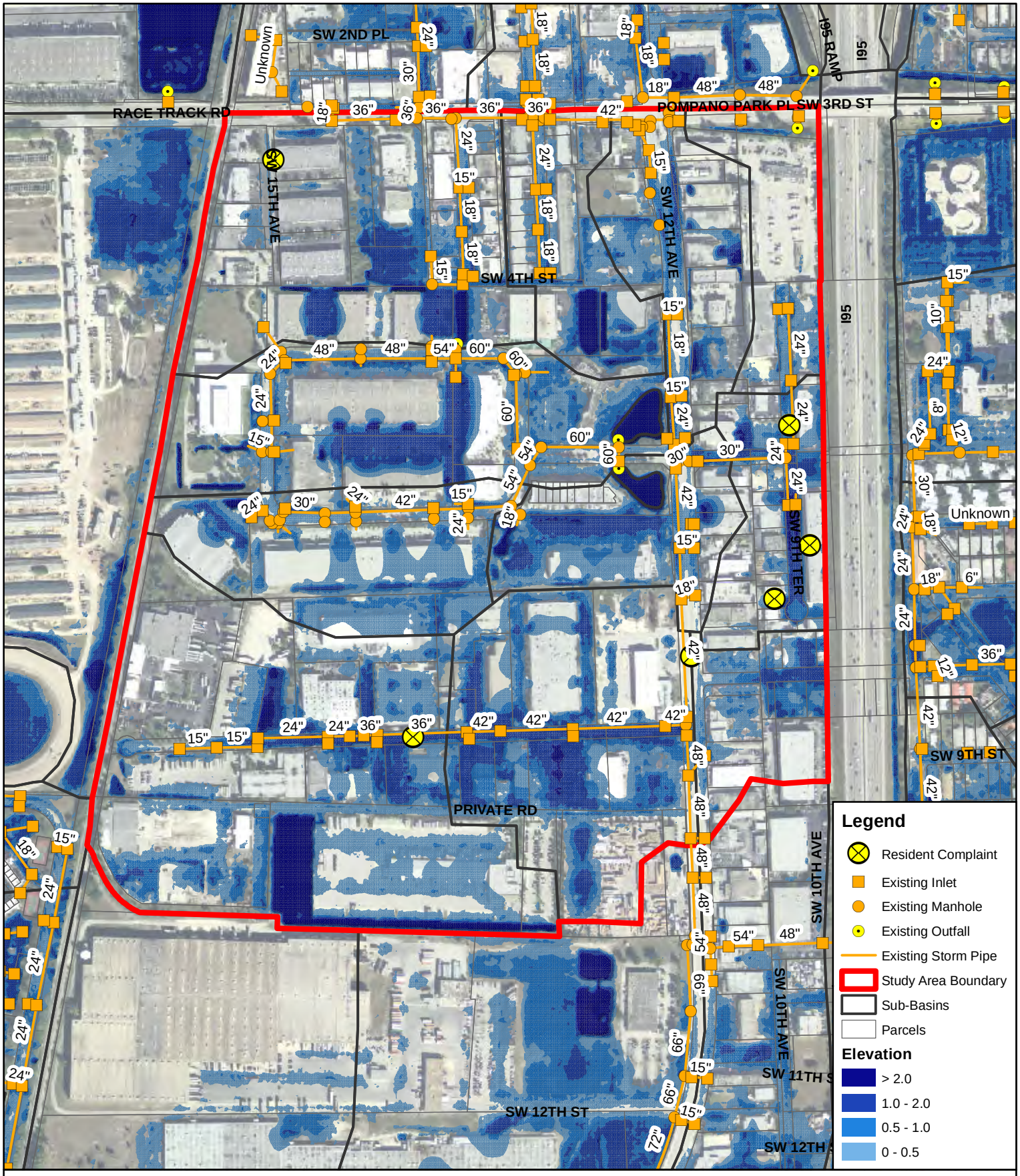


Figure 5-1B
Pompano Park Place & Andrews Avenue
Potential Flooding Depth

N
W E
S

0 300 600
Feet

5.2.2 STUDY AREA 2 – NORTHWEST CRA TOC

The Northwest CRA Transit Oriented Corridor (TOC) Neighborhood was identified as a priority drainage basin in need of stormwater system improvements based on results of the basin prioritization formula. The study area for the Northwest CRA TOC Neighborhood has general boundaries of NW 6th Street on the north, West Atlantic Boulevard on the south, I-95 on the west, and NE 5th Street on the east. The NW CRA TOC Neighborhood typically experiences flooding throughout the area during heavy rainfall events. This study area is also located directly within the WBID basin for the Old Pompano Canal and is considered to have an impact on water quality within this impaired waterbody.

The flooding problems within the NW CRA TOC Neighborhood are created primarily by a combination the topography within the study area and the lack of extensive existing stormwater infrastructure. The ground surface elevation within the public right-of-way areas typically range between 6.1 feet and 9.2 feet NAVD. The existing ground surface elevations are displayed within the Existing Topography Map on Figure 5-2A. The existing drainage system within the NW CRA TOC Neighborhood consists of gravity pipes collecting stormwater runoff from the public right of way areas. The existing stormwater system within the study area ranges from 15-inches to 72-inches in diameter. Although there are limited existing stormwater facilities within the NW CRA TOC Neighborhood, it does not provide an adequate level of service to some right-of-way areas within the study area. The stormwater model was used to evaluate the performance of the existing stormwater management system within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. Based on the results of the stormwater model, the extent of estimated flooding within the NW CRA TOC Neighborhood is displayed in Figure 5-2B at the end of this section. Based on the results of the existing conditions stormwater model, the problem area within the NW CRA TOC Neighborhood is primarily located within the sub-basin CE_006_02 along NW 7th Avenue.

CMA has conducted a supplemental stormwater study of this study area on behalf of the Pompano Beach CRA, which considers both the existing conditions along with the future conditions defined by the CRA. In addition to identifying locations within the study area with existing flooding problems, the purpose of this supplemental stormwater study of the Northwest CRA Area is to analyze the extent of future development which would be feasible within the TOC Area upon the implementation of a new master stormwater system to serve these future development conditions. The proposed stormwater improvements within the NW CRA TOC Neighborhood will need to be implemented to meet level of service criteria under both the existing conditions and the future conditions. Since there is a history of past flooding within portions of the study area, the CRA is interested in implementing a master stormwater management system for the entire TOC area for the purpose of promoting future development.

Due to regulatory restrictions and site conditions, any future redevelopment could be limited depending on the available storage volume for stormwater runoff within the TOC Area along with the stormwater discharge rate via existing outfalls into adjacent surface water bodies. This stormwater storage volume must be available for both water quality treatment of stormwater runoff and flood attenuation during storm events. Any available stormwater storage within the stormwater management system will help limit the peak flood level within the drainage basin during a storm event. Adequate stormwater storage must be provided to ensure that the peak flood levels do not extend across public roadways or up to inhabited structures during heavy storm events. According to regulatory requirements, adequate stormwater storage must also be provided to retain stormwater runoff on-site for water quality treatment purposes. The retention of stormwater runoff will reduce the concentration of suspended solids and various pollutants for the purpose of enhancing the water quality of the stormwater runoff before it is discharged into the receiving water body. The required volume of stormwater storage directly corresponds to the total area of the drainage basin and the area of impervious ground surface within the drainage basin, which is typically increased during higher density redevelopment.

The CRA wishes to eventually construct a master stormwater system for the TOC Area, which would maximize the storage volume for any stormwater runoff from future development. The proposed master stormwater system would be constructed in advance of redevelopment within the TOC Area. The proposed master stormwater system could be used to promote development within the TOC Area since additional drainage infrastructure would not be needed within each site, which would reduce the development costs. Under this master stormwater system concept, the amount of stormwater storage capacity for each site would be deducted from the total volume of available stormwater storage capacity within the master stormwater management system as development occurs within the TOC Area. The CRA could then charge any developers of available properties within the TOC Area an impact fee for using a portion of the master stormwater management system. This would allow developers to complete their site development without the need for an extensive on-site drainage system.

Future Development within TOC Area

The CRA has developed a long term plan to guide the redevelopment within the Northwest Redevelopment District. The CRA wishes to promote public/private partnerships to attract private investment for future development within the northwest redevelopment district. The CRA has identified the TOC, which is located within the Northwest Redevelopment District, as an area it wishes to promote future development. The TOC Area is a mixture of commercial and low/medium density residential land uses under the existing conditions. The CRA is promoting future redevelopment with higher density mixed uses within the TOC Area. The CRA goals for future redevelopment within the TOC Area include the following conceptual plans, which defined the targeted areas where redevelopment should be promoted:

- Old Pompano Historical Commercial District at NE 1st Street and NE 1st Avenue
- Blanche Ely Retail/Office Center at MLK Boulevard and NW 6th Avenue
- Hammondville Village Concept along MLK Boulevard
- Downtown Civic Campus Concept along Atlantic Boulevard

Figure 5-2C – Old Pompano Historical Commercial District at NE 1st Street and NE 1st Avenue

Figure 5-2D – Blanche Ely Retail/Office Center at MLK Boulevard and NW 6th Avenue

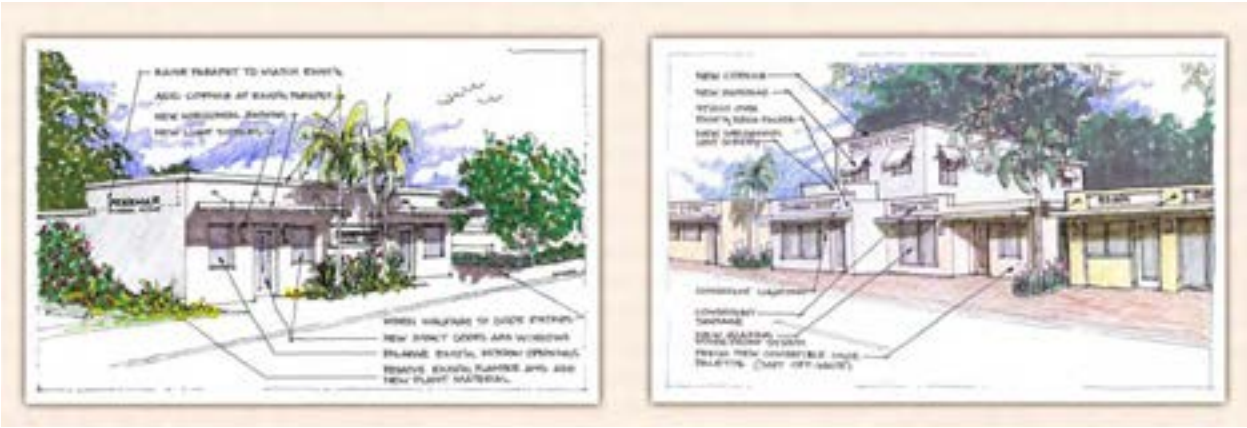


Figure 5-2E – Hammondville Village Concept along MLK Boulevard

Figure 5-2F – Downtown Civic Campus Concept along Atlantic Boulevard

For these conceptual redevelopment plans to become a reality, the CRA would like to promote future redevelopment within the TOC Area by offering any feasible incentives to developers. The CRA has already purchased properties throughout the TOC Area for the purpose of property consolidation. These CRA owned properties can be used for future development and/or the master stormwater management

system. The implementation of a master stormwater system within the TOC Area would promote future development in the area through development incentives. The master stormwater system would limit the amount of stormwater infrastructure that a developer would need to construct on site since all stormwater runoff within the TOC Area would be handled by the master stormwater system. The master stormwater system would also allow developers to maximize the amount of impervious area and building area upon redevelopment since extensive on-site stormwater retention areas would not be required.

Future Stormwater Impacts – Flood Control

A purpose of the existing stormwater management system within the TOC Area is to alleviate any flooding throughout the neighborhood under the future conditions during heavy rainfall events. The existing stormwater management system consists of drainage structures interconnected with underground drainage piping, which collect stormwater runoff throughout the neighborhood and transmit the stormwater via the pipe network to the system outfalls into the receiving water body. The existing stormwater management system within the TOC Area west of Dixie Highway discharges into the Pompano Canal via a 72-inch outfall at MLK Boulevard and via a 36-inch outfall at NW 6th Street. The existing stormwater system within the TOC Area includes 1,000 linear feet of existing exfiltration trench which provides flood attenuation and water treatment for surface runoff.

Any future development within the TOC Area will impact the performance of the existing stormwater management system from a flood control perspective. The additional impervious ground surface associated with this future development will reduce the amount of stormwater runoff which can infiltrate into the ground surface and will cause stormwater runoff to flow quicker toward low lying areas of the TOC Area. Any future development would likely also modify the ground surface grading at the development site, which could cause more stormwater runoff to collect in low lying areas of the TOC. If the future development does not include improvements to the existing stormwater management system, additional flooding within the TOC Area could result due to this development. Any recommendations for improvements to the existing stormwater management system are based on level of service criteria for flood control, which was established by the City based on regulatory requirements. The level of service criteria for the stormwater management system within the City was defined based on the guidelines from SFWMD and Broward County EPGM.

Future Stormwater Impacts – Water Quality

Another purpose of the existing stormwater management system within the TOC Area is also to provide water quality treatment to stormwater runoff prior to discharge into the Pompano Canal. The water quality treatment is typically accomplished by retaining stormwater runoff in grass swale areas, dry retention areas, wet detention ponds, and/or exfiltration trenches. The retention of stormwater runoff within these systems allows various sediments suspended within the stormwater to settle, which reduces the amount of pollutants within the stormwater entering the Pompano Canal via the existing stormwater outfalls. Since the Pompano Canal is currently designated as an impaired waterbody by Florida DEP, the reduction of pollutant loading within stormwater discharges from the TOC Area is a high priority for the City.

Any future development within the TOC Area will impact the performance of the existing stormwater management system from a water quality perspective. The additional impervious ground surface associated with this future development will typically increase the amount of pollutants within the stormwater runoff. For example, stormwater runoff from impervious surfaces, such as asphalt roadways and parking lots, typically has higher pollutant load than stormwater runoff from pervious surfaces, such natural grass areas. If the future development does not include improvements to the existing stormwater management system, stormwater runoff from the TOC Area can be expected to have higher pollutant loading after development.

The purpose of this supplemental study on the TOC Area is to determine any impacts of future development on the water quality treatment performance of the existing stormwater management system and to recommend any improvements to the stormwater management system to ensure adequate water quality treatment under the future development conditions. Any recommendations for improvements to the existing stormwater management system are based on level of service criteria for water quality treatment, which was established by the City based on regulatory requirements. The level of service criteria for the stormwater management system within the City was defined based on the guidelines from SFWMD and Broward County EPGM. The regulatory agencies define the level of water quality treatment provided to stormwater runoff required prior to discharge via outfalls into adjacent surface water bodies, which are summarized below.

- Provide stormwater storage volume equivalent to the first inch of stormwater runoff generated from the entire TOC Area (whichever is greater).
- Provide stormwater storage volume equivalent to 2.5 inches of stormwater runoff generated from all impervious areas within the entire TOC Area (whichever is greater).

The typical methods for providing water quality treatment of stormwater runoff are the installation of dry retention areas, grass swales and exfiltration trench to provide adequate storage volume to meet these requirements. Additional water quality upgrades to the existing stormwater management system will be necessary to account for any future redevelopment within the TOC Area.

Coordination with Regulatory Agencies

On October 31, 2012, staff members from CMA and the CRA met with staff at Broward County Environmental Protection and Growth Management (BCEPGM) to discuss the feasibility of the CRA implementing a master stormwater system in advance of future development along with possible improvement options to the existing stormwater management system within the TOC Area. CMA explained that the proposed master stormwater system for the TOC Area will consist of various improvements to the existing stormwater management system. The proposed improvements will be a combination of dry detention areas and exfiltration trench, which is interconnected to the existing system.

CMA noted the intention to obtain a conceptual surface water permit from Broward County EPGM to establish the master stormwater system for the TOC Area along with defining the total available stormwater storage volume of the proposed master system. The CRA would then obtain a permit modification to the conceptual permit to construct the proposed improvements to the existing stormwater system in advance of any development. As future development occurs within the TOC Area, the developer will need to obtain a permit modification to the conceptual permit for each project. The amount of stormwater storage volume required for each development site would then be deducted from the total volume of available stormwater storage volume defined within the conceptual permit for the master stormwater management system. Broward County EPGM confirmed that obtaining a conceptual permit is the appropriate plan for the establishment of a master stormwater system for the TOC Area. Broward County EPGM noted that the conceptual permit would need to be obtained from the South Florida Water Management District (SFWMD) if any of the proposed improvements impact Broward County owned properties.

Broward County EPGM also noted that the TOC Area is located within drainage basin (WBID #3271) of the Pompano Canal, which has been identified as an impaired waterbody by FDEP. The limits of Pompano Canal WBID #3271 was displayed on Figure 4-16. Since the Pompano Canal has been assigned Total Maximum Daily Load (TMDL) for nutrient pollutants (total nitrogen and total phosphorus) by FDEP, Broward County EPGM noted that nutrient analysis and nutrient load reduction calculations will

likely be required during the conceptual permitting process due to the interconnection with the Pompano Canal. In general, Broward County EPGM did not have any objections to the proposed master stormwater management system for the TOC Area since the proposed improvements are intended to reduce the peak discharge into the Pompano Canal and to reduce pollutant loads within the stormwater discharge into the Pompano Canal.

Flood Control Analysis

Based on the results of the stormwater model, CMA identified areas within the TOC Area which may require future stormwater improvements in order to meet level of service criteria for flood protection. Under the existing conditions within the TOC Area, potential flooding during heavy storm events can be expected in the areas displayed on Figure 5-2B – Potential Flooding Map. The general areas of the TOC Area with potential flooding during significant rainfall events include the following locations:

- NW 9th Avenue between MLK Boulevard and NW 6th Street
- NW 7th Avenue between NW 1st Street and NW 6th Street
- NW 6th Avenue between NW 4th Street and NW 6th Street
- NW 4th Avenue between NW 4th Street and NW 6th Street
- NW 3rd Avenue between NW 1st Street and MLK Boulevard
- NW 4th Street between NW 3rd Avenue and NW 4th Avenue
- NE 4th Avenue and NE 2nd Street

CMA targeted the proposed improvements to the existing stormwater management system to reduce the peak flood stage and flood duration within these areas. CMA conducted an iterative analysis with the stormwater model of various system improvement alternatives for the purpose of identifying the most effective option for reducing the peak flood stage and flood duration within the TOC Area. The stormwater model was modified until the recommended system improvements were identified based on the model results. The model results under the existing conditions and future conditions are displayed within the tables on the following pages to compare the potential reduction in flood stage, flood duration, and peak discharge due to these system improvements.

Table 5.2.1 – Peak Flood Stage Model Results

Basin	Nodes	Ground Elevation	Existing		Proposed		Peak Stage Reduction (feet)
			Peak Stage (ft)	Flood Depth (ft)	Peak Stage (ft)	Flood Depth (ft)	
CE_006_01	IN_1104	7.47	8.21	0.74	8.27	0.80	-0.06
	IN_1109	8.60	8.31	-0.29	8.3	-0.30	0.01
CE_006_02	IN_1094	8.89	9.57	0.68	9.05	0.16	0.52
	IN_1095	7.60	8.98	1.38	8.86	1.26	0.12
	IN_1099	7.92	8.8	0.88	9.18	1.26	-0.38
	MH_0225	9.16	9.17	0.01	8.57	-0.59	0.60
CE_006_03	IN_1129	8.05	8.94	0.89	8.85	0.8	0.09
	IN_1127	7.75	8.93	1.18	8.86	1.11	0.07
	IN_1136	8.15	8.93	0.78	8.86	0.71	0.07
	MH_0178	8.41	8.93	0.52	8.85	0.44	0.08
CE_006_04	IN_1137	8.15	8.93	0.78	8.86	0.71	0.07
CE_010_01	IN_1276	7.49	8.28	0.79	8.29	0.8	-0.01
CE_010_02	CE01002	4.84	5.91	1.07	5.8	0.96	0.11
CE_010_03	MH_0219	9.71	9.03	-0.68	8.86	-0.85	0.17
	MH_0223	8.95	9.02	0.07	8.99	0.04	0.03
	IN_5322	8.73	5.66	-3.07	5.56	-3.17	0.1
CE_010_04	MH_0215	8.58	8.81	0.23	8.86	0.28	-0.05
	IN_5330	7.94	6.06	-1.88	5.6	-2.34	0.46
CE_010_05	MH_0198	8.41	8.69	0.28	8.65	0.24	0.04
	IN_1215	7.09	8.67	1.58	8.59	1.5	0.08
	MH_1420	7.98	8.65	0.67	8.58	0.6	0.07
CE_010_06	IN_1164	8.75	8.93	0.18	8.91	0.16	0.02
CE_010_07	IN_1154	7.03	8.93	1.9	8.86	1.83	0.07
CE_010_08	IN_1143	11.68	11.67	-0.01	11.57	-0.11	0.10
CE_089_01	MH_0208	12.98	9.02	-3.96	8.98	-4.00	0.04
	IN_1242	7.18	8.97	1.79	4.16	-3.02	4.81
CE_020_01	MH_0210	13.78	14.02	0.24	14.04	0.26	-0.02
	MH_0211	13.41	6.59	-6.82	6.70	-6.71	-0.11
CE_020_03	MH_0204	14.86	11.81	-3.05	11.65	-3.21	0.16
	MH_0205	14.66	12.91	-1.75	12.71	-1.95	0.20
SE_119_01	IN_2319	4.26	5.54	1.28	5.64	1.38	-0.10
	MH_0401	6.09	3.58	-2.51	3.44	-2.65	0.14

Table 5.2.2 – Peak Flood Duration Model Results

Basin	Nodes	Existing Flood Duration (hours)	Proposed Flood Duration (hours)	Percent Reduction
CE_006_01	IN_1104	23.5	7.6	68%
CE_006_02	IN_1095	23.5	11.6	51%
	IN_1099	22.2	10.0	55%
CE_006_03	IN_1129	23.0	11.7	49%
	IN_1127	24.7	13.3	46%
	IN_1136	23.0	12.9	44%
	MH_0178	18.8	8.3	56%
CE_006_04	IN_1137	23.3	13.3	43%
CE_010_01	IN_1276	23.5	7.4	69%
CE_010_02	CE01002	45.0	5.4	88%
CE_010_03	MH_0223	0.9	0.7	22%
CE_010_04	MH_0215	11.2	1.2	89%
CE_010_05	MH_0198	22.7	10.3	55%
	IN_1215	22.4	22.8	-2%
CE_010_06	IN_1164	20.2	4.9	76%
CE_010_07	IN_1154	45.0	22.5	50%
CE_010_08	IN_1143	0	0	N/A
CE_089_01	IN_1242	2.1	0.0	100%
CE_020_01	MH_0210	1.1	1.0	9%
CE_020_03	MH_0205	13.8	0.0	100%
SE_119_01	IN_2319	13.7	11.5	16%

Table 5.2.3 – Outfall Peak Discharge Model Results

Street Name	Pipe Diameter (in)	Link	Existing Peak Discharge (CFS)	Proposed Peak Discharge (CFS)	Peak Discharge Reduction (CFS)
NW 6 Street	36	IN_1104_1	54.96	52.2	2.76
MLK Boulevard	72	IN_1276_1	65.94	46.24	19.7
Flagler Avenue	30	MH_0208	24.86	10.95	13.91
SE 3rd Avenue	54	MH_0211_1	82.28	86.21	-3.93

Based on this analysis, CMA developed recommended system improvements to improve the flood control under the future conditions within the TOC Area. Since the minimum system improvements required to address flood control within the TOC Area did not provide adequate stormwater storage for water quality treatment purposes, the recommended system improvements had to be supplemented with additional treatment elements, such as exfiltration trench and dry retention areas. The water quality analysis along with the recommended improvements is summarized further within the sections below.

Water Quality Analysis

The water quality analysis was conducted to determine the required improvements to the stormwater management system within the TOC Area to ensure adequate storage volume for water quality treatment under the future development conditions. The extent of any improvements to the existing stormwater management system will be directly related to the water quality treatment requirements, which are chiefly based on the area of impervious ground surface. Under the future conditions, the amount of impervious ground surface within the TOC Area is expected to significantly increase as future development occurs. CMA reviewed multiple future development scenarios within the TOC Area to determine the amount of water quality storage volume required under each option. The various future conditions scenarios are summarized further below.

Future Conditions Scenarios

Under each future development scenario, CMA analyzed the impact to the existing stormwater management system if specific properties within the TOC Area were redeveloped to the maximum allowable impervious coverage. The purpose of this analysis was to determine the amount of additional storage volume for water quality treatment due to potential future development within the TOC Area. CMA met with CRA staff to establish the various future development scenarios by identifying individual properties within the TOC Area which will be likely candidates for future development. All of the properties within the TOC Area are displayed within Figure 5-2G – Project Limits and Properties Map to show each property which is owned by the Pompano Beach CRA, the City of Pompano Beach, Broward County or the State of Florida. The properties which are owned by the Pompano Beach CRA and those properties located immediately adjacent were deemed to be the most likely candidates for future redevelopment. Each development scenario included different combinations of various properties within the TOC Area to be redeveloped under the future conditions.

Under each future development scenario, CMA also considered various future impervious ground coverage for each property combination. CMA considered impervious ground coverage of 80%, 85%, 90%, 95%, and 100% for each property combination identified for future redevelopment while the remaining properties within TOC Area maintained the same impervious ground coverage as under the existing conditions. CMA calculated the existing impervious ground coverage for each property within the TOC Area by digitizing a recent aerial photograph. For each future development scenario, CMA calculated the total storage volume required for water quality treatment for both the west basin and east basins of the TOC Area. The analysis of each future development scenario is summarized within Table 5.2.4 for the West Basin and Table 5.2.5 for the East Basin on the following page. The assumptions for each future development scenario are defined within the sections below.

Table 5.2.4 - West Basin Future Development Scenarios

CRA Owned Properties

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL
Total Area (ac)	28.75	150.65	28.75	150.65	28.75	150.65	28.75	150.65	28.75	150.65	28.75	150.65
Building Area (ac)	2.75	14.15	0.00	11.40	0.00	11.40	0.00	11.40	0.00	11.40	0.00	11.40
Impervious Area (ac)	1.51	45.60	28.75	72.84	27.31	71.40	25.88	69.97	24.44	68.53	23.00	67.09
Pervious Area (ac)	24.49	90.90	0.00	66.41	1.44	67.85	2.88	69.29	4.31	70.72	5.75	72.16
Water Quality Volume (ac-in)		150.65		182.10		178.51		174.91		171.32		167.73

All TOC Properties

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL
Total Area (ac)	117.19	150.65	117.19	150.65	117.19	150.65	117.19	150.65	117.19	150.65	117.19	150.65
Building Area (ac)	14.15	14.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Impervious Area (ac)	18.84	45.60	117.19	143.95	111.33	138.09	105.47	132.23	99.61	126.37	93.75	120.51
Pervious Area (ac)	84.21	90.90	0.00	6.69	5.86	12.55	11.72	18.41	17.58	24.27	23.44	30.13
Water Quality Volume (ac-in)		150.65		359.88		345.23		330.58		315.93		301.29

CRA Owned Properties, Vacant & Commercial Zoned Properties

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL
Total Area (ac)	75.88	150.65	75.88	150.65	75.88	150.65	75.88	150.65	75.88	150.65	75.88	150.65
Building Area (ac)	8.37	14.15	0.00	5.78	0.00	5.78	0.00	5.78	0.00	5.78	0.00	5.78
Impervious Area (ac)	11.54	45.60	75.88	109.94	72.09	106.15	68.29	102.35	64.50	98.56	60.70	94.76
Pervious Area (ac)	55.97	90.90	0.00	34.93	3.79	38.72	7.59	42.52	11.38	46.31	15.18	50.11
Water Quality Volume (ac-in)		150.65		274.85		265.37		255.88		246.40		236.91

CRA Owned Properties & Properties Along the Major Corridors

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL
Total Area (ac)	56.55	150.65	56.55	150.65	56.55	150.65	56.55	150.65	56.55	150.65	56.55	150.65
Building Area (ac)	4.96	14.15	0.00	9.19	0.00	9.19	0.00	9.19	0.00	9.19	0.00	9.19
Impervious Area (ac)	5.10	45.60	56.55	97.05	53.72	94.22	50.90	91.39	48.07	88.56	45.24	85.74
Pervious Area (ac)	46.49	90.90	0.00	44.42	2.83	47.25	5.66	50.07	8.48	52.90	11.31	55.73
Water Quality Volume (ac-in)		150.65		242.61		235.54		228.48		221.41		214.34

Table 5.2.5 - East Basin Future Development Scenarios

CRA Owned Properties

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL	CRA Parcels	TOTAL
Total Area (ac)	0.95	116.63	0.95	116.63	0.95	116.63	0.95	116.63	0.95	116.63	0.95	116.63
Building Area (ac)	0.00	19.27	0.00	19.27	0.00	19.27	0.00	19.27	0.00	19.27	0.00	19.27
Impervious Area (ac)	0.00	38.78	0.95	39.73	0.90	39.68	0.85	39.64	0.81	39.59	0.76	39.54
Pervious Area (ac)	0.95	58.58	0.00	57.63	0.05	57.68	0.09	57.73	0.14	57.77	0.19	57.82
Water Quality Volume (ac-in)		116.63		116.63		116.63		116.63		116.63		116.63

All TOC Properties

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL	All Parcels	TOTAL
Total Area (ac)	77.29	116.63	77.29	116.63	77.29	116.63	77.29	116.63	77.29	116.63	77.29	116.63
Building Area (ac)	19.27	19.27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Impervious Area (ac)	20.32	38.78	77.29	95.75	73.43	91.89	69.57	88.03	65.70	84.16	61.84	80.30
Pervious Area (ac)	37.70	58.58	0.00	20.88	3.86	24.74	7.73	28.61	11.59	32.47	15.46	36.34
Water Quality Volume (ac-in)		116.63		239.39		229.73		220.06		210.40		200.74

CRA Owned Properties, Vacant & Commercial Zoned Properties

	Existing		100% Impervious		95% Impervious		90% Impervious		85% Impervious		80% Impervious	
	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL	Main Parcels	TOTAL
Total Area (ac)	29.62	116.63	29.62	116.63	29.62	116.63	29.62	116.63	29.62	116.63	29.62	116.63
Building Area (ac)	5.32	19.27	0.00	13.95	0.00	13.95	0.00	13.95	0.00	13.95	0.00	13.95
Impervious Area (ac)	9.33	38.78	29.62	59.07	28.14	57.59	26.66	56.11	25.18	54.63	23.70	53.15
Pervious Area (ac)	14.97	58.58	0.00	43.61	1.48	45.09	2.96	46.57	4.44	48.05	5.92	49.53
Water Quality Volume (ac-in)		116.63		147.67		143.97		140.27		136.57		132.86

Scenario 1 – CRA Owned Properties

Under this scenario, all CRA owned properties within the TOC Area will be assumed to be redeveloped under the future conditions. The CRA owned properties within the TOC Area are highlighted in Figure 5-2H below. The total water quality storage volume required for each TOC basin was calculated based on the assumption that all CRA owned properties were redeveloped with various impervious ground coverage while the remaining properties in the TOC Area maintained the same impervious ground coverage as the existing conditions. Under this scenario, the total storage volume required if all of the CRA owned properties in the TOC Area were to be developed to 100% impervious coverage would be 182 acre-inches in the West Basin and116 acre-inches in the East Basin. The total storage volume for both the West Basin and East Basin is summarized in Table 5.2.6 below for each level of future impervious ground coverage.

Figure 5-2H – Scenario 1 Properties Map

Table 5.2.6 – Scenario 1 Future Impervious Coverage

	Water Quality Volume Required (ac-in)	
Percent Impervious	**West Basin**	**East Basin**
100%	182.10	116.63
95%	178.51	116.63
90%	174.91	116.63
85%	171.32	116.63
80%	167.73	116.63

Scenario 2 – All Properties

Under this scenario, all available properties within the TOC Area will be assumed to be redeveloped under the future conditions. The scenario assumed that all properties within the TOC Area are available for future redevelopment. The available properties within the TOC Area are highlighted in Figure 5-2I below. The total water quality storage volume required for each TOC basin was calculated based on the assumption that all available properties were redeveloped with various impervious ground coverages while the remaining properties in the TOC Area maintained the same impervious ground coverage as the existing conditions. Under this scenario, the total storage volume required if all of the available properties in the TOC Area were to be developed to 100% impervious coverage would be 360 acre-inches in the West Basin and 239 acre-inches in the East Basin. The total storage volume for both the West Basin and East Basin is summarized in Table 5.2.7 below for each level of future impervious ground coverage.

Figure 5-2I – Scenario 2 Properties Map

Table 5.2.7 - Scenario 2 Future Impervious Coverage

	Water Quality Volume Required (ac-in)	
Percent Impervious	**West Basin**	**East Basin**
100%	359.88	239.39
95%	345.23	229.73
90%	330.58	220.06
85%	315.93	210.40
80%	301.29	200.74

Scenario 3 – CRA Owned Properties + Vacant Properties + Commercial Zoned Properties

Under this scenario, all CRA owned properties, all vacant properties and all properties with commercial zoning designations within the TOC Area will be assumed to be redeveloped under the future conditions. The scenario assumed that these designated properties within the TOC Area are available for future redevelopment. The properties within the TOC Area which are assumed to be redeveloped under the future conditions are highlighted in Figure 5-2J below. The total water quality storage volume required for each TOC basin was calculated based on the assumption that these designated properties were redeveloped with various impervious ground coverages while the remaining properties in the TOC Area maintained the same impervious ground coverage as the existing conditions. Under this scenario, the total storage volume required if all of these designated properties in the TOC Area were to be developed to 100% impervious coverage would be 275 acre-inches in the West Basin and 148 acre-inches in the East Basin. The total storage volume for both the West Basin and East Basin is summarized in Table 5.2.8 below for each level of future impervious ground coverage.

Figure 5-2J – Scenario 3 Properties Map

Table 5.2.8 - Scenario 3 Future Impervious Coverage

	Water Quality Volume Required (ac-in)	
Percent Impervious	**West Basin**	**East Basin**
100%	274.85	147.67
95%	265.37	143.97
90%	255.88	140.27
85%	246.40	136.57
80%	236.91	132.86

Scenario 4 – CRA Owned Properties + All Available Properties Located Along the Major Corridors

Under this scenario, all CRA owned properties and all available properties along the major corridors of MLK Boulevard and NW 6th Avenue will be assumed to be redeveloped under the future conditions. The scenario assumed that these designated properties within the TOC Area are available for future redevelopment. The properties within the TOC Area which are assumed to be redeveloped under the future conditions are highlighted in Figure 5-2K below. The total water quality storage volume required for each TOC basin was calculated based on the assumption that these designated properties were redeveloped with various impervious ground coverages while the remaining properties in the TOC Area maintained the same impervious ground coverage as the existing conditions. Under this scenario, the total storage volume required if all of these designated properties in the TOC Area were to be developed to 100% impervious coverage would be 249 acre-inches in the West Basin. The total storage volume for the West Basin is summarized in Table 5.2.9 below for each level of future impervious ground coverage.

Figure 5-2K – Scenario 4 Properties Map

Table 5.2.9 – Scenario 4 Future Impervious Coverage

	Water Quality Volume Required (ac-in)	
Percent Impervious	**West Basin**	**East Basin**
100%	242.61	147.67
95%	235.54	143.97
90%	228.48	140.27
85%	221.41	136.57
80%	214.34	132.86

System Improvement Alternatives (Future Conditions)

CMA has investigated various system improvement alternatives for the implementation of a master stormwater management system for the entire TOC study area. The proposed master stormwater management system will be sized to meet the regulatory requirements related to flood protection and water quality treatment of stormwater runoff for the entire TOC area under the future build out conditions. The system improvement alternatives considered as feasible for the TOC Area include the construction of exfiltration trench and/or dry retention areas which are interconnected with the existing stormwater management system. The replacement of existing outfalls with larger diameter pipe or the installation of stormwater pumps stations were not considered as a feasible system improvement alternative since neither provides the additional water quality storage volume required for the reduction of pollutant loading within stormwater runoff under the future conditions. The installation of larger outfall pipes or stormwater pump stations would likely increase the peak discharge into the Pompano Canal, which would likely not be allowed due to regulatory restrictions. The installation of gravity drainage wells is not an option due to the lack of brackish groundwater at the bottom of the surficial aquifer below the TOC Area since the boundary of this brackish groundwater is located just east of Dixie Highway. The installation of Class V drainage wells is not allowed outside of this zone by the regulatory agencies. The assumptions used for proposed exfiltration trench and dry retention areas within the TOC Area are summarized below.

Exfiltration Trench

The goal of this system improvement alternative is to maximize the storage and infiltration capacity below ground within proposed exfiltration trench throughout the TOC Area. The proposed exfiltration trench would be aligned within the public right-of-way area and would be interconnected with the existing stormwater management system. The proposed exfiltration trench system would need to be aligned along streets that do not currently have existing drainage infrastructure. In order for an exfiltration trench system to be effective, the existing ground surface should be greater than +5.0 feet NAVD to ensure adequate below ground storage above the water table. Exfiltration trench systems located in areas with ground surface elevations below this elevation are not likely to provide tangible benefits since the groundwater table will be located relatively close to the ground surface and would eliminate the storage capacity within the exfiltration trench. The proposed exfiltration trenches were aligned in public right–of-way areas with existing ground surface elevations above +5.0 feet NAVD, which is the case throughout the TOC Area. The average ground surface elevation throughout the TOC Area is approximately +12.0 feet NAVD in the East Basin and approximately +8.0 feet NAVD in the West Basin. While analyzing the effectiveness of this system improvement alternative, the following design parameters were assumed during our analysis of the proposed exfiltration trench:

- Water Table Elevation 2.5 feet NAVD
- Trench Width: 5.5 feet
- Trench Height: 4.5 feet
- Perforated Pipe Diameter: 24-inch
- Hydraulic conductivity: 6.5×10^{-5} CFS/ft^2-ft head

Dry Retention Area

The goal of this system improvement alternative is also to maximize the storage and infiltration capacity within proposed dry retention areas located within available properties throughout the TOC Area. In order for dry retention area to be effective, the existing ground surface should be greater than +5.0 feet NAVD to ensure adequate storage capacity is provided within the retention area. Dry retention areas located in

areas with ground surface elevations below this elevation are not likely to provide tangible benefits since the groundwater table will be located relatively close to the ground surface and would limit the available aboveground storage capacity. With a high water table elevation of +2.5 feet NAVD within the TOC Area, dry retention areas would require the bottom elevation to be set at or above +3.5 feet NAVD. The following design parameters were assumed during our analysis of the proposed dry retention areas:

- Setback Distance 4 feet from the parcel boundaries
- Maximum Side Slope 4:1
- Bottom Elevation +3.5 feet NAVD

Master Stormwater System for Future Conditions

CMA developed the proposed master system improvements to the existing stormwater management system within the TOC Area with the goal of maximizing the stormwater storage volume as feasible under the site constraints. By maximizing the additional stormwater storage volume within the proposed stormwater improvements, the CRA will be able to maximize the amount of new impervious area associated with new development within the TOC Area. Based on the assumptions outlined within the previous section, CMA tabulated the amount of additional stormwater storage capacity that would be provided by the master stormwater system improvements. CMA then compared this additional stormwater storage capacity with the minimum stormwater storage required under each future development scenario. Based on this comparison, the CRA will be able to identify the extent of future development within the TOC Area which could be feasibly served by the proposed master stormwater system. The proposed master system improvements have been divided into the West Basin and East Basin of the CRA TOC Area.

West Basin

In order to maximize any new exfiltration trench, CMA developed a preliminary layout of the proposed stormwater improvements within the TOC West Basin. CMA assumed new exfiltration trench would be installed within all public right-of-ways within the TOC West Basin which are currently not served by existing drainage infrastructure. The proposed exfiltration trench would be interconnected with the existing stormwater system to allow the proposed improvements to provide additional storage capacity to all portions of the TOC West Basin. Based on the assumptions outlined within the previous section, CMA tabulated the amount of additional stormwater storage capacity that would be provided by the proposed exfiltration trench within the West Basin. The proposed exfiltration trench includes approximately 14,708 linear feet, which corresponds to a volume of approximately 104 acre-inches in additional stormwater storage. Within the existing stormwater system in the TOC West Basin, there are approximately 963 linear feet of existing exfiltration trench, which corresponds to approximately 7.4 acre-inches of existing stormwater storage. The proposed master stormwater system within the TOC West Basin would provide a total stormwater storage volume of 111 acre-inches within the existing and proposed exfiltration trench for the West Basin. A detailed inventory of the additional stormwater storage volume per basin can be found in Table 5.2.10 within this section.

In order to maximize any new dry retention area, CMA developed a preliminary layout of the proposed stormwater improvements within the TOC West Basin. CMA assumed new dry retention areas would be constructed at properties currently owned by the CRA within the TOC West Basin. The proposed dry retention areas were also targeted to be near existing stormwater outfalls into the Pompano Canal, which would allow them to serve the entire TOC West Basin. The proposed dry retention areas would be interconnected with the existing and new stormwater improvements via control structures, which would allow the master stormwater system to overflow into dry retention areas prior to discharge into the Pompano Canal. Due to the topography of the TOC West Basin, the western limits of the study area have

lower ground surface elevations, which naturally cause stormwater runoff to typically flow from east to west. The existing stormwater outfalls into the Pompano Canal are also located in the western portion of the TOC West Basin. The proposed dry retention areas were located within properties which are currently owned by the CRA and were identified by the CRA to be the most feasible options for stormwater facilities. The proposed dry retention areas encompass a total area of 3.95 acres of property. The proposed dry retention areas would provide 129.5 acre-inches in additional stormwater storage within the master stormwater system for the TOC West Basin. A detailed inventory of the additional stormwater storage volume per basin can be found in Table 5.2.10 below.

Table 5.2.10 – West Basin Stormwater Storage per Basin

Basin	Existing Exfiltration Length (ft)	Existing Volume (ac-in)	Proposed Exfiltration Length (ft)	Proposed Exfiltration Volume (ac-in)	Proposed Retention Area (ac)	Proposed Retention Volume (ac-in)
CE_006_01			303.80	2.12		
CE_006_02			830.50	6.20		
CE_006_03	300.00	2.27	1,029.00	7.80		
CE_006_04			925.90	7.23		
CE_010_01			2,223.80	14.71	1.18	49.80
CE_010_02			3,165.20	18.90	2.77	79.70
CE_010_03	663.00	5.10	871.00	6.70		
CE_010_04			1,244.00	9.71		
CE_010_05			1,577.50	11.59		
CE_010_06			600.10	4.68		
CE_010_07			1,270.40	8.88		
CE_010_08			666.70	5.20		
TOTAL	**963.00**	**7.37**	**14,707.90**	**103.72**	**3.95**	**129.50**

The proposed master stormwater system for the TOC West Basin would consist of a combination of the existing drainage infrastructure, the proposed exfiltration trench, and the proposed dry retention areas. All of these components would be interconnected to form the proposed master stormwater system, which would serve the entire TOC West Basin under the future conditions. CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

Based on our analysis, the proposed master stormwater system for the West Basin would be sufficient to handle stormwater runoff from the TOC Area under the future development conditions similar to Scenario 1 or a modified version of Scenario 4. The total volume of stormwater storage required under each scenario with either 80% or 100% impervious coverage is summarized within Table 5.2.11 below. Table 5.2.12 also shows the total volume of stormwater storage required under each scenario compared to the total stormwater storage volume available within the proposed master stormwater improvements for the West Basin. Under Scenario 1, the proposed master stormwater improvements would provide sufficient additional stormwater storage for the redevelopment of all CRA owned properties to 100% impervious coverage with an additional 58.5 acre-inches of volume available for the redevelopment of additional parcels. Under Scenario 2, the proposed master stormwater improvements would not provide sufficient additional stormwater storage for the redevelopment of all properties to 100% impervious coverage since an additional 119 acre-inches of storage volume would be required. Under Scenario 3, the proposed master stormwater improvements would not provide sufficient additional stormwater storage for the

redevelopment of all CRA owned, vacant, and commercial zoned properties to 100% impervious coverage since an additional 34 acre-inches of storage volume would be required. Under Scenario 4, the proposed master stormwater improvements would almost provide sufficient additional stormwater storage for the redevelopment of all CRA owned properties and parcels along major corridors to 100% impervious coverage since only an additional 2.02 acre-inches of storage volume would be required.

Table 5.2.11 – West Basin Scenario Analysis

Scenario	Volume Required (ac-in)		Master System Improvements Volume Available (ac-in)
	100% Impervious	80% Impervious	
1	182.10	167.73	240.59
2	359.88	301.29	240.59
3	274.85	236.91	240.59
4	242.61	214.34	240.59

The extent of future development within the TOC West Basin will be limited by the available stormwater storage capacity within the proposed master stormwater system improvements, which is directly related to the area of impervious ground surface in the West Basin. Due to regulatory restrictions, the proposed master stormwater system improvements could only handle stormwater runoff from a limited area of impervious ground surface within the TOC West Basin. The capacity of the proposed master stormwater system improvements will limit the amount of additional impervious ground surface constructed within the TOC West Basin. The maximum allowable area of impervious ground surface under the future conditions throughout the TOC West Basin is outlined within Table 5.2.12 below.

Table 5.2.12 – Maximum Allowable Impervious Area

	West Basin
Maximum Allowable Impervious Area (acre)	97.05
Existing Impervious Area to Remain (acre)	40.54
Additional Impervious Area Available (acre)	56.55

East Basin

CMA developed a preliminary layout for a proposed master stormwater system within the TOC East Basin. CMA assumed new exfiltration trench would be installed along all public right-of-ways within the TOC East Basin. Due to the limited number of CRA owned parcels available to construct a dry retention area, the installation of a dry retention system was not a feasible option for the TOC East Basin. Based on the assumptions outlined within the previous section, CMA tabulated the amount of additional stormwater storage capacity that would be available within a proposed master stormwater system for the TOC East Basin. The proposed exfiltration trench includes approximately 13,970 linear feet in the East Basin, which provides approximately 100.8 acre-inches in additional stormwater storage volume. A detailed inventory of the additional stormwater storage volume by basin can be found in Table 5.2.13 below.

Table 5.2.13 – East Basin Stormwater Storage per Basin

Basin	Existing Exfiltration Length (ft)	Existing Volume (ac-in)	Proposed Exfiltration Length (ft)	Proposed Exfiltration Volume (ac-in)
CE_089_01	0	0	2,063.61	16.11
CE_020_01	0	0	5,853.14	45.69
CE_020_03	0	0	3,971.89	31.00
SE_119_01	0	0	2,079.18	7.99
TOTAL	**0**	**0**	**13,967.81**	**100.79**

Based on our analysis, the proposed master stormwater system for the East Basin would not be a feasible option to handle stormwater runoff from the TOC Area under any of the future development conditions. The total volume of stormwater storage required under each scenario with either 80% or 100% impervious coverage is summarized within Table 5.2.14 below. Table 5.2.14 also shows the total volume of stormwater storage required under each scenario compared to the total stormwater storage volume available within the proposed master stormwater improvements for the East Basin. The implementation of a proposed master stormwater system for the TOC East Basin would provide approximately 100.8 acre-inches in additional storage volume for the purpose of flood control and water quality treatment of stormwater runoff. However, the total stormwater storage volume provided by a master stormwater system within the East Basin does not meet the minimum volume required under the existing conditions or any of the future development scenarios.

Table 5.2.14 – East Basin Scenario Analysis

Scenario	Volume Required (ac-in)		Master System Improvements Volume Available (ac-in)
	100% Impervious	80% Impervious	
1	116.63	116.63	100.79
2	239.39	200.74	100.79
3	147.67	132.86	100.79
4	147.67	132.86	100.79

Recommended Improvements

East Basin

CMA does not recommend any stormwater improvements within the TOC East Basin due to the lack of flooding problems in this area under the existing conditions. CMA also does not recommend the implementation of a master stormwater system within the TOC East Basin to serve the future redevelopment within the area. The implementation of a master stormwater system for the East Basin of the TOC Area would provide up to 100.8 acre-inches of additional stormwater storage volume, which does not meet the minimum volume required under the existing conditions or any of the future development scenarios. Therefore, the implementation of a master stormwater system within the TOC East Basin is not a feasible option for future redevelopment. Any future redevelopment within the TOC East Basin will need to provide on-site stormwater facilities and will be required to retain stormwater runoff on-site for water quality treatment purposes according to regulatory requirements.

West Basin

CMA recommends the implementation of a master stormwater system for the TOC West Basin to promote future redevelopment within the TOC Area. The implementation of the master stormwater system improvements for the TOC West Basin would provide a total of 240.6 acre-inches in storage volume for the purpose of flood control and water quality treatment of stormwater runoff. This total stormwater storage volume provided by the proposed master stormwater system exceeds the minimum volume required for the existing conditions by current regulatory standards. The excess storage capacity within the proposed master stormwater system could be used to handle runoff from future redevelopment within the West Basin of the TOC Area.

Please note that the recommended master stormwater system for this study area would be required in order to meet future development scenarios defined by the CRA. The estimated implementation costs for the master stormwater system serving the future redevelopment conditions within the West Basin is $8,150,000, which would be funded through the CRA. Any stormwater improvements required to address only the existing flooding problems under the existing conditions within this study area would be limited to the installation of exfiltration trench along the streets with localized flooding issues, which include NW 9th Avenue, NW 7th Avenue, NW 4th Avenue, and NW 2nd Street. The estimated implementation costs for the stormwater improvements serving just the existing conditions within the West Basin, is $1,982,000. CMA has prepared a preliminary cost estimate for both the master stormwater system for the future development conditions and the stormwater improvements for the existing conditions, which are enclosed within Appendix A-2.

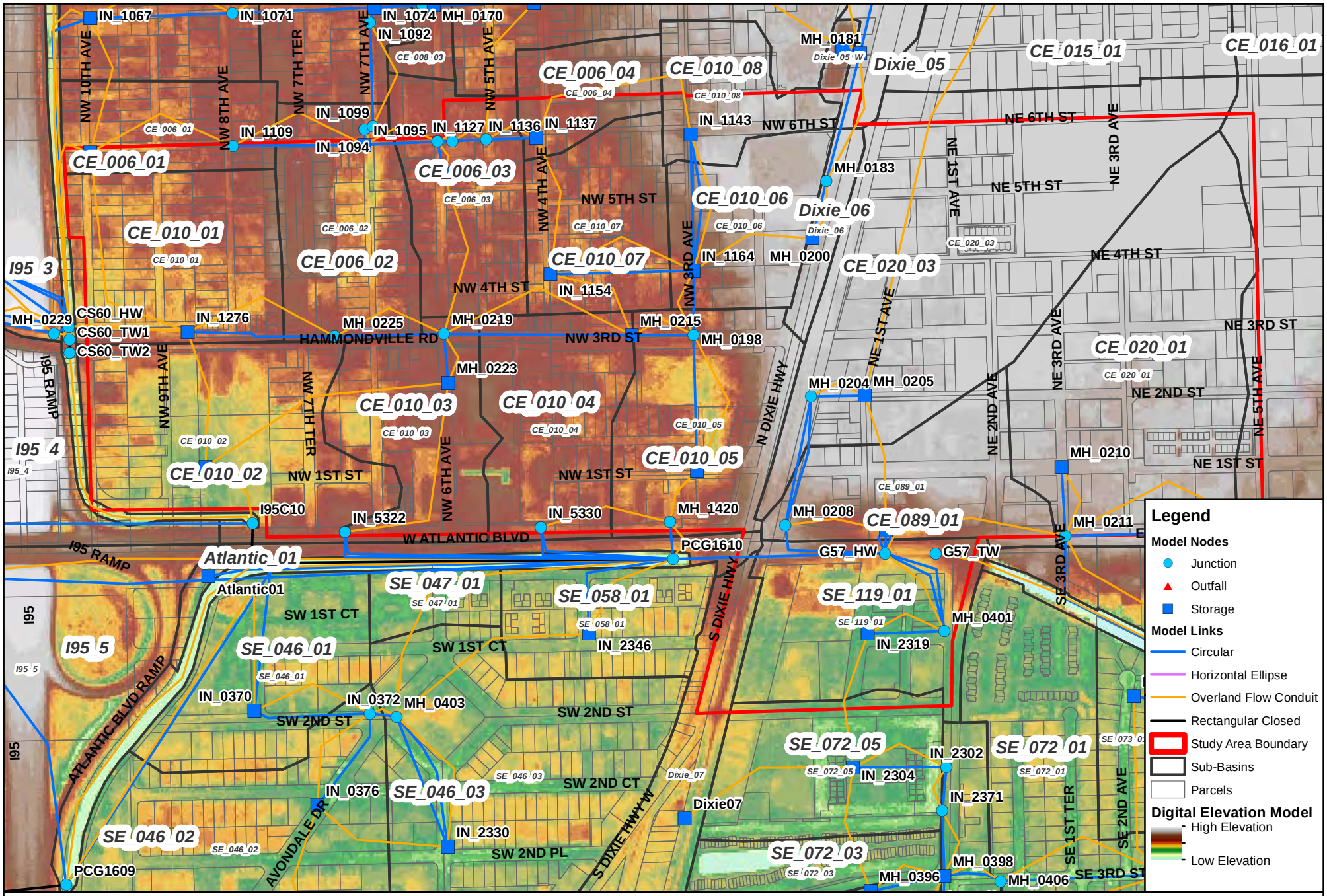


Figure 5-2A
Northwest CRA - TOC
Existing Topography & Model Schematics

0 250 500 Feet

N W E S

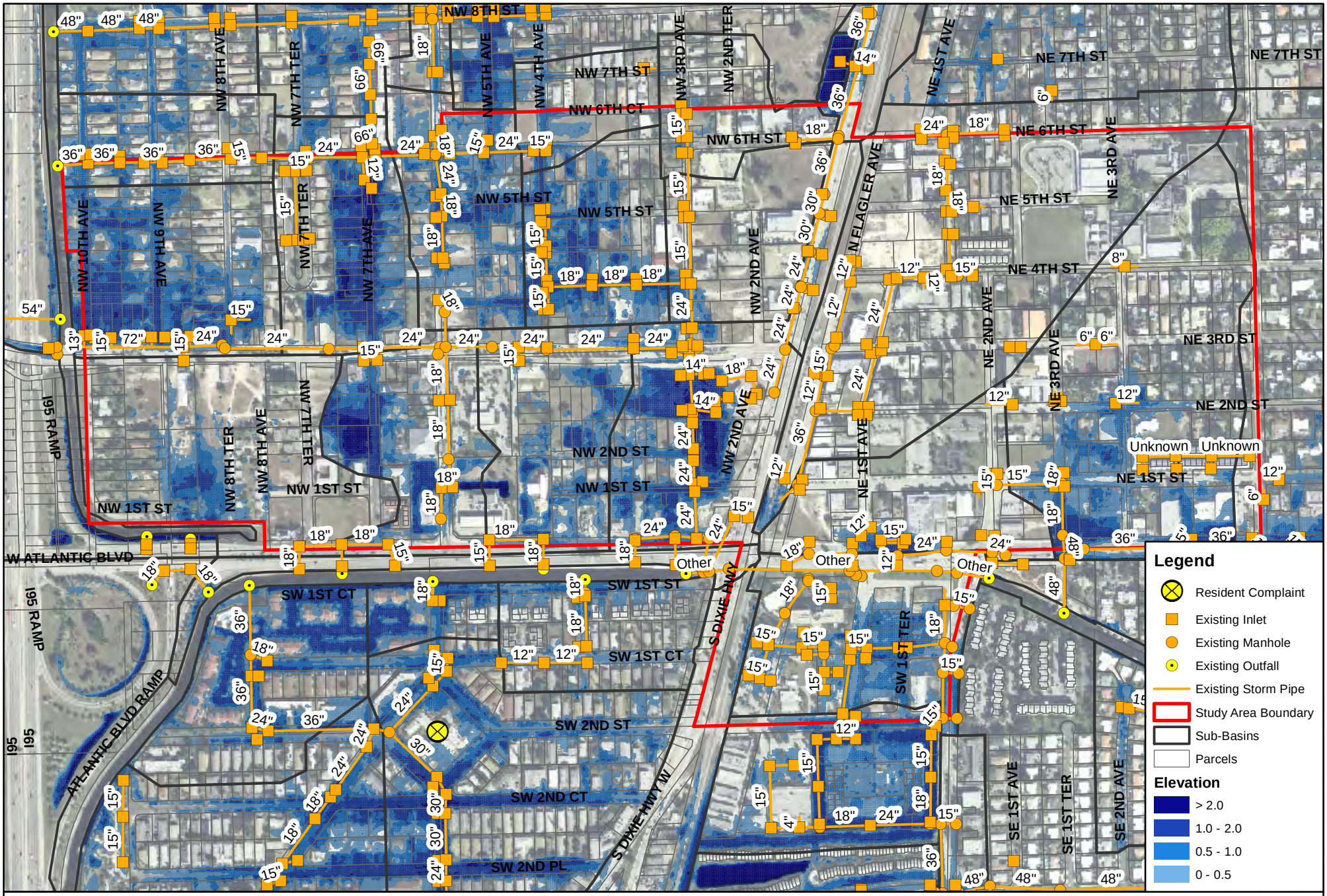


Figure 5-2B
Northwest CRA - TOC
Potential Flooding Depth

0 250 500 Feet

N W E S

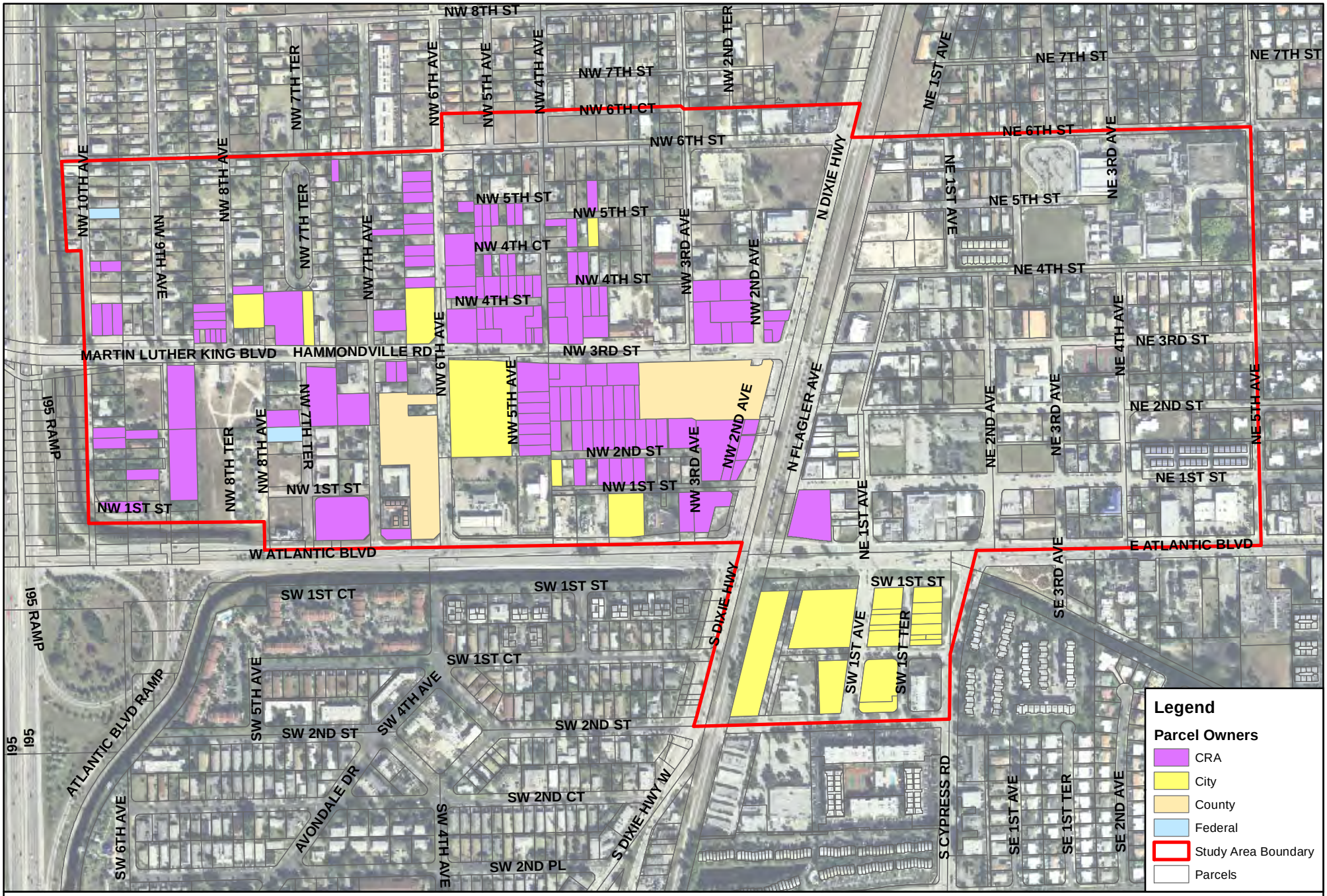


Figure 5-2G
Northwest CRA - TOC
Project Limits and Properties Map

0 250 500 Feet

N W E S

5.2.3 STUDY AREA 3 – LYONS PARK NEIGHBORHOOD

The Lyons Park Neighborhood is located west of South Cypress Road, north of McNab Road, east of South Flagler Avenue and south of Southwest 8th Street. This study area is a residential neighborhood with chiefly single family homes, which is served by an existing stormwater collection system. The existing stormwater management system consists of four independent systems which are interconnected with other stormwater systems outside of the neighborhood. Under the existing stormwater system, the Lyons Park neighborhood can be divided into four separate service areas. The stormwater runoff from these service areas within the Lyons Park neighborhood is eventually discharged via multiple positive outfalls located to the south of West McNab Road and to the east of South Cypress Road. The locations of the existing positive outfalls are summarized below:

- Southeast of SW 6th Avenue and West McNab Road
- Southeast of SW 5th Avenue and West McNab Road
- Southeast of South Cypress Road and SE 13th Street
- South Cypress Road

The first three positive outfalls discharge to finger canals adjacent to the SFWMD C-14 Canal, while the Cypress Road outfall discharges directly into SFWMD C-14 Canal. The system configuration of independent outfalls allows the opportunity to analyze the flooding within each independent service area and to identify which independent system does not provide adequate discharge capacity to the respective service area. Within the existing conditions stormwater model, the study area is includes sub-basins SE_043_02, SE_043_04, SE_043_05, SE_107_01, SE_107_02, SE_107_03, and SE_107_04. The existing ground surface topography of the Lyons Park neighborhood along with nodal schematic map of the stormwater model is displayed within Figure 5-3A. The existing condition stormwater model was used to analyze the performance of the existing stormwater management system during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. As displayed in Figure 5-3B, Potential Flooding Map, enclosed at the end of this section, the Lyons Park Neighborhood experiences significant flooding throughout the entire study area according to the results of the existing conditions stormwater model. Based on our analysis, the peak flooding depth can reach greater than 2 feet in low lying areas of the neighborhood, such as along the eastern end of SE 12th Street within the study area.

The stormwater model was used to evaluate and compare the benefits associated to the various system improvement alternatives within the Lyons Park Neighborhood. The results of the stormwater model are used to compare the reduction in peak flood stage and the reduction of flood duration under each system improvement alternative within the Lyons Park neighborhood. Since the Lyons Park neighborhood is served by an existing drainage system, various pipe upgrades to this existing system are evaluated with the stormwater model to optimize the pipe sizing of the stormwater system. Additional system improvement alternative, such as drainage wells, pump stations, storage/retention areas, and/or a combination of these alternatives, were also analyzed with the stormwater model to compare the effectiveness in reducing the peak flood stage and flood duration. The installation of exfiltration trench was eliminated from consideration as system improvement alternative due to low surface elevations throughout the study area which would limit the effectiveness of this option. The analysis of each system improvement alternative is summarized within the following sections.

Alternative 1: Pipe Size Upgrades

The stormwater model was used to conduct multiple simulations of various scenarios of upsizing the drainage pipe throughout the study area. Under Alternative 1, the purpose of the pipe upgrades is to increase the transmission capacity of the existing system. The proposed pipe size upgrades were located at specific locations within the existing stormwater system where additional conveyance capacity of a larger pipe cross section is needed. The potential pipe upsizing locations were identified by examining the energy grade line throughout the existing pipe network during peak flow conditions. Typically, the peaks of the energy grade line correspond to pipe segments where a change in diameter will be the most effective in maximizing the transmission capacity of the drainage system. The most effective locations for potential pipe sizing are typically at the downstream end of the existing stormwater system near or at the positive outfall. If the pipes are upsized near the outfalls, it will allow the targeted upsizing of drainage pipe further upstream to optimize the transmission capacity. Under Alternative 1, the existing pipe would be upsized along the primary routes to the existing outfalls, which include upgrading along South Cypress Road to 42-inch diameter and along Flagler Avenue to 48-inch diameter.

Based on the analysis of Alternative 1 with the stormwater model, the reduction in peak flood stage from the existing conditions is summarized within Table 5.3.1 below. The results of our analysis show the effectiveness of Alternative 1 in reducing the peak flood stage throughout the Lyons Park neighborhood. Within the west service area, a significant reduction in peak flood stage was predicted by the stormwater model along West McNab Road (Node IN_2115, Node IN_2134, and Node MH_0369), which is located immediately upstream from existing outfalls. However, the reduction in peak flood stage were minimal within upstream areas of the drainage system (Node IN_2168, Node IN_2166, Node IN_2163) on the west side of the study area where the worst flooding has been encountered within the Lyons Park neighborhood. Alternative 1 does significantly reduce the peak flood stage within the targeted areas of the eastern service area with the most significant flooding problems under the existing conditions (Node MH_0374 and Node IN_2171). According the results of the stormwater model, Alternative 1 only reduces the peak flood stages by 0.1 and 0.2 feet, respectively, from the existing conditions.

Table 5.3.1 – Alternative 1 Peak Flood Stage Summary

Nodes	Existing			Alternative 1	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
West Service Area – Dual Outfalls south of West McNab Road					
IN_2168*	4.78	3.98	0.80	4.77	-0.01
IN_2166*	4.75	3.88	0.87	4.62	-0.13
IN_2163*	4.58	3.98	0.60	4.49	-0.09
MH_0369	4.59	5.36	0.00	4.12	-0.47
IN_2125**	4.55	3.74	0.81	3.52	-1.03
IN_2134**	4.41	4.11	0.30	3.75	-0.66
CS37A_TW	2.31	N/A	N/A	2.36	N/A
Northeast Service Area – Outfall west of South Cypress Road					
MH_0379	4.78	4.57	0.21	4.76	-0.02
MH_0374*	4.78	3.44	1.34	4.34	-0.44
MH_0375	4.65	5.36	0.00	4.14	-0.51
MH_0373	5.08	5.00	0.08	4.10	-0.98
MH_0372	4.83	4.66	0.17	3.86	-0.97
MH_0409**	2.40	5.66	0.00	3.15	N/A
BMCNAB_TW1	2.15	N/A	N/A	2.2	N/A

Southeast Service Area – Outfall along South Cypress Road					
IN_2171*	4.76	3.84	0.92	4.56	-0.20
IN_2176	4.77	4.97	0.00	4.27	-0.50
MH_0371	4.79	4.91	0.00	4.19	-0.60
IN_2155	4.62	4.22	0.40	4.25	-0.37
IN_2150	3.82	4.66	0.00	3.26	-0.60
IN_2151	2.69	5.05	0.00	2.80	N/A
IN_2149**	2.40	4.99	0.00	2.51	N/A
BCYPR_TW	2.20	N/A	N/A	2.26	N/A

*Critical flooded nodes.

**Upstream nodes from outfalls.

Based on the analysis of Alternative 1 with the stormwater model, the reduction in flood duration from the existing conditions is summarized within Table 5.3.2 below. Although Alternative 2 does not effectively reduce the peak flood stage within the problem areas of the Lyon Park neighborhood, it does significantly reduce the flooding duration throughout all areas of the study area. Based on the analysis with the stormwater model, Alternative 1 results in an average reduction in flood duration of -64% from the existing conditions.

Table 5.3.2 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_2168*	3.98	>33	12.9	61
IN_2166*	3.88	>33	13	73
IN_2163*	3.98	26.5	6.7	59
MH_0374*	3.44	24.3	3.1	74
IN_2171*	3.84	13	2.9	55

Since Alternative 1 does not significantly reduce the peak flood stages within the Lyons Park neighborhood, it does not provide adequate flood protection to meet the level of service criteria for the flooding of public roadways. The lack of significant reduction in peak flood stages is caused by the low topographic conditions of the study area. The Lyons Park neighborhood has low-lying streets surrounded by higher ground on all sides, which leads to stormwater runoff flowing into the neighborhood from the perimeter. Since the downstream drainage system is located along these higher perimeter areas, the flow of stormwater runoff from the neighborhood within the drainage system can be limited by the hydraulics of stormwater runoff entering the upstream system from higher elevations along the perimeter. Basically, stormwater runoff from the neighborhood does not effectively flow toward the outfalls until the higher perimeter areas have been adequately drained by the upstream system. The estimated design and construction costs for the pipe size upgrades under Alternative 1 are $4,766,000. Please note that additional regulatory considerations would need to be addressed during the detailed design of these improvements. The regulatory agencies will limit the peak discharge via the upsized outfall pipes along with requiring some form of water quality treatment, likely regraded swale areas. The constructability of Alternative 1 would also be limited by the site conditions within the right-of-way areas where the outfall pipe is located.

Alternative 2: Drainage Wells

Under Alternative 2, the stormwater model was used to iteratively estimate the impact of installing a variable number of stormwater drainage wells throughout the Lyons Park neighborhood. The purpose of Alternative 2 is to create additional discharge capacity via new drainage wells. The locations of proposed drainage wells were targeted to optimize the flooding reduction within the low lying problem areas of the Lyons Park neighborhood. All proposed drainage wells will be interconnected with a drainage pipe network to maintain a consistent driving head within each well, which will help distribute the stormwater discharge throughout the study area. In order to maximize the reductions in flooding, 25 drainage wells have been considered under Alternative 2. The proposed drainage wells will be distributed throughout each service area within Lyon Park, as summarized below:

- Southeast Service Area (Node IN_2171): 5 proposed drainage wells
- Northeast Service Area (Node MH_0374): 7 proposed drainage wells
- West Service Area (Node IN_2168): 3 proposed drainage wells
- West Service Area (Node IN_2166) 5 proposed drainage wells
- West Service Area (Node IN_2163) 5 proposed drainage wells

During the analysis of Alternative 2 with the stormwater model, the seasonal high water table was assumed to be at +1.5 feet NAVD, as defined by the Seasonal High Water Table (SHWT) map for Broward County. The minimum driving head was assumed to be 1.5 feet above the SHWT within the stormwater model. Based on historical information within the area, the discharge rate of the proposed drainage wells was assumed to be 450 GPM per foot of head within the stormwater model, which is approximately equivalent to 1.0 CFS per foot of head.

Based on the analysis of Alternative 2 with the stormwater model, the reduction in peak flood stage from the existing conditions is summarized within Table 5.3.3 below. The results of our analysis show the effectiveness of Alternative 2 in reducing the peak flood stage throughout the Lyons Park neighborhood. In order to identify the impact of increasing the number of proposed drainage wells within each service area, the model results for another simulation where the number of drainage wells was doubled are also presented in Table 5.3.3 below. The proposed drainage wells do not significantly reduce the peak flood stages in the Lyons Park neighborhood from existing conditions. The increase in drainage wells within each service area does not reduce the peak flood stages enough to meet an acceptable level of service.

Table 5.3.3 – Alternative 2 Peak Flood Stage Summary

Nodes	Existing			Alternative 2 Drainage Wells (25)		Alternative 2a Drainage Wells (50)	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)	Peak Stage (feet)	Peak Reduction (feet)
West Service Area – Dual Outfalls south of West McNab Road							
IN_2168*	4.78	3.98	0.80	4.73	-0.05	4.69	-0.09
IN_2166*	4.75	3.88	0.87	4.60	-0.15	4.47	-0.28
IN_2163*	4.58	3.98	0.60	4.55	-0.03	4.51	-0.07
MH_0369	4.59	5.36	0.00	4.56	-0.03	4.52	-0.07
IN_2125**	4.55	3.74	0.81	4.52	-0.03	4.49	-0.06
IN_2134**	4.41	4.11	0.30	4.28	-0.13	4.21	-0.20
CS37A_TW	2.31	N/A	N/A	2.30	-0.01	2.29	-0.02

Northeast Service Area – Outfall west of South Cypress Road							
MH_0379	4.78	4.57	0.21	4.71	-0.07	4.69	-0.09
MH_0374*	4.78	3.44	1.34	4.70	-0.08	4.49	-0.29
MH_0375	4.65	5.36	0.00	4.64	-0.01	4.56	-0.09
MH_0373	5.08	5.00	0.08	5.08	0.00	5.07	-0.01
MH_0372	4.83	4.66	0.17	4.83	0.00	4.82	-0.01
MH_0409**	2.40	5.66	0.00	2.40	0.00	2.40	0.00
BMCNAB_TW1	2.15	N/A	N/A	2.13	-0.02	2.13	-0.02
Southeast Service Area – Outfall along South Cypress Road							
IN_2171*	4.76	3.84	0.92	4.60	-0.16	4.50	-0.26
IN_2176	4.77	4.97	0.00	4.73	-0.04	4.70	-0.07
MH_0371	4.79	4.91	0.00	4.77	-0.02	4.76	-0.03
IN_2155	4.62	4.22	0.40	4.62	0.00	4.62	0.00
IN_2150	3.82	4.66	0.00	3.79	-0.03	3.78	-0.04
IN_2151	2.69	5.05	0.00	2.61	-0.08	2.61	-0.08
IN_2149**	2.40	4.99	0.00	2.40	0.00	2.40	0.00
BCYPR_TW	2.20	N/A	N/A	2.20	0.00	2.19	-0.01

*Critical flooded nodes.
**Upstream nodes from outfalls.

Based on the analysis of Alternative 2 with the stormwater model, the reduction in flood duration from the existing conditions is summarized within Table 5.3.4 below for critical flooded nodes only. Although Alternative 2 does not effectively reduce the peak flood stage within the problem areas of the Lyon Park neighborhood, it does significantly reduce the flooding duration throughout all areas of the study area. Based on the analysis with the stormwater model, Alternative 2 results in an average reduction in flood duration of -68% from the existing conditions.

Table 5.3.4 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_2168*	3.98	>33	7	79
IN_2166*	3.88	>33	9	73
IN_2163*	3.98	26.5	11	59
MH_0374*	3.44	24.3	6.2	74
IN_2171*	3.84	13	5.8	55

Since Alternative 2 does not significantly reduce the peak flood stages within the Lyons Park neighborhood, it does not provide adequate flood protection to meet the level of service criteria for the flooding of public roadways. The lack of significant reduction in peak flood stages is caused by the low topographic conditions of the study area, which limits the discharge capacity of each drainage well. Alternative 2 does provide significant reduction in the expected flood duration within the study area, which compares well with the other system improvement alternatives considered for Lyons Park. The estimated design and construction costs for the pipe size upgrades under Alternative 2 are $3,659,000.

Alternative 3: Pump Stations

Alternative 3 includes the proposed construction of a stormwater pump station along with associated drainage pipe upgrades in various configurations. The purpose of Alternative 3 is to provide additional discharge capacity to the existing system, especially during high tide. Due to the layout of the existing drainage system along with the large size of the Lyons Park neighborhood, the construction of a single pump station would not be capable of servicing the entire neighborhood. Three system configurations were developed and analyzed to determine the best options for reducing the peak flood stage and the flooding duration within the Lyons Park neighborhood. The results of our analysis with the stormwater model are summarized below for the three layout options for Alternative 3.

Layout 1

Under this layout option, the proposed construction includes two new pump stations at existing outfalls in order to provide additional hydraulic head on the downstream end of the system to increase the discharge capacity of the system. This additional discharge capacity from the stormwater pump stations will draw down the flooding within the upstream areas of the Lyons Park neighborhood quicker than the existing conditions. Under this layout option, the proposed pump stations are located within the west service area (Node IN_2125) and within the east service area (Node MH_0409). Pipe improvements to the existing stormwater system are also proposed to efficiently transmit stormwater runoff to the proposed pump stations in order to maximize the pump operational capacity. The estimated design and construction costs for pump station Alternative 3 – Layout 1 are approximately $3,980,000. The components associated with each pump station are listed below.

West Service Area Pump Station:

- Install a new 24-inch discharge pipe from pump station to existing outfall location.
- Install a new flap gate at the point of discharge to minimize tidal influences on the system performance.
- A wet well will have a total footprint of approximately 150 square feet and a depth of 8-feet.
- A proposed pump will have a discharge capacity of 34 CFS, which is equivalent to the peak discharge from existing outfalls during low tide conditions.
- Replace existing pipe downstream from Node IN_2166 with a new 21-inch RCP pipe.
- Replace existing pipe downstream from Node MH_0369 with a new 24-inch RCP pipe.

East Service Pump Station:

- Install a new 24-inch discharge pipe from the pump station to the existing outfall location at the finger canal south of SE 13th Street.
- Install a new flap gate at the point of discharge to minimize tidal influences on the system performance.
- A wet well will have a total footprint of approximately 150 square feet and a depth of 8 feet.
- The proposed pump will have a discharge capacity of 24 CFS, which is equivalent to the peak discharge of the existing system under the existing conditions.
- Replace existing pipe downstream from Node Mh_0372 with a new 36-inch RCP pipe.

Based on the analysis with the stormwater model, the potential reduction of peak flood stages are summarized within Table 5.3.8 for the Layout 1, Layout 2 and Layout 3 of Alternative 3. The results for Layout 1 show reductions in peak flood stage at nodes adjacent to the proposed pump station for the west service area (Node MH_0369 and Node IN_2125). There are no other nodes in the problem areas of the Lyons Park neighborhood that receive any benefit in peak flood stage reductions from this pump station layout. The lack of success in reducing peak stages in the problem areas of the Lyons Park neighborhood

suggests that the location of proposed pump stations do not exert any influence in the upstream portions of the existing stormwater system. The construction of a pump station system closer to the problem areas of Lyons Park will likely be more effective in reducing the peak flood stages.

For Layout 1, the reduction of flood duration to five hours or less can be considered an acceptable level of improvement. The reduction in flood duration within the Lyons Park neighborhood from the existing conditions is summarized in Table 5.3.5 below for Layout 1. The estimated reduction in flood duration from Alternative 3 – Layout 1 is relatively minimal when compared to Alternative 2 with drainage wells or Alternative 1 pipe size upgrades.

Table 5.3.5 – Alternative 3 (Layout 1) Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3 Layout 1	Reduction (%)
IN_2168*	3.98	>33	27.4	17
IN_2166*	3.88	>33	28.3	14
IN_2163*	3.98	26.5	24	9
MH_0374*	3.44	24.3	14.8	39
IN_2171*	3.84	13	11	15

The reduction of peak flood stages and flood duration within the Lyons Park neighborhood are minimal for this Alternative 3 – Layout 1. Only areas immediately adjacent to the proposed pump stations receive any significant benefits under this alternative, which are not considered problem areas for the Lyons Park neighborhood. Alternative 3 (Layout 1) should be eliminated from consideration as a potential solution since other system improvement alternatives considered for this study area provide better flood control benefits.

Layout 2

Under this layout option, the proposed construction includes two new pump stations near the problem area of Lyons Park in order to increase the discharge capacity of the stormwater system. This additional discharge capacity from the stormwater pump stations will draw down the flooding within the problem areas of the Lyons Park neighborhood quicker than the existing conditions. Under this layout option, the proposed pump stations are located within the west service area approximately 500 feet north of the intersection of West McNab Road and South Flagler Avenue at City inlet IN_2157 and within the east service area at the problem area (Node MH_0374) on the southeast corner of SW 12th Street and SW 1st Avenue. The western pump station is located approximately midway between the problem area and the existing outfall. Pipe improvements to the existing stormwater system are also proposed to efficiently transmit stormwater runoff to the proposed pump stations in order to maximize the pump operational capacity and to reduce the possibility of creating flooding issues downstream of the proposed pump station. The estimated design and construction costs for this pump station alternative (Layout 2) are approximately $5,107,000. The components associated with each pump station are listed below.

West Service Area:

- The proposed wet well will have a total footprint of about 100 square feet and a depth of 8 feet.
- The proposed pump will have a discharge capacity of 40 CFS. Larger discharge capacities were attempted but this discharge rate was selected as it is more feasible due to space constraints.
- Install a new force main from the pump station to the Node IN_2125, which is just upstream from the existing outfall.
- Replace existing pipe downstream from Node IN_2125 with a 36-inch RCP pipe.

* Replace existing pipe downstream from Node IN_2168 with a 24-inch RCP pipe.
* Replace existing pipe downstream from Node IN_2166 with a 24-inch RCP pipe.
* Replace existing pipe downstream from Node IN_2163 with a 24-inch RCP, which is the terminal run of drainage pipe into the proposed wet well.

East Service Area:

* The proposed wet well will have a total footprint of about 100 square feet and a depth of 8 feet at Node MH_0374.
* The proposed pumps will have a discharge capacity of 50 CFS. Lower discharge capacities were simulated providing proportional peak flood reductions. Space constraints may limit the applicability of this pump capacity.
* The installation of a new 30-inch discharge pipe from the pump station to Node MH_0375.
* Replace existing pipe downstream from Node MH_0375 with a 36-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0373 with a 36-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0372 with a 36-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0409 with a 36-inch RCP pipe.
* Connect with newly proposed pipes at Node IN_2171 and Node MH_0374 with a 24-inch RCP pipe along SW 1st Avenue in order to extend the benefit of the pump station to the problem area in the south (Node IN_2171).

The estimated reduction in peak flood stage is summarized within Table 5.3.8 for this alternative layout. The western problem areas of Lyons Park, which are represented by Node IN_2168, Node IN_2166, and Node IN_2163 within the stormwater model, show a low reduction in peak flood stages for a 40 CFS pump capacity scenario. The eastern problem areas of Lyons Park, which are represented by Node MH_0374 and Node IN_2171 within the stormwater model, show a moderate reduction in peak flood stages for a 50 CFS pump capacity scenario. The largest peak reduction is estimated at Node MH_0374 with a reduction of 0.56 feet from existing conditions. A reduction of 0.25 feet was estimated at Node MH_0374 for a simulation with a 35 CFS pump capacity. However, even with this significant reduction in peak flood stage, the flood depth was not reduced to less than 0.5 feet.

The estimated reduction in flood duration within the Lyons Park neighborhood is summarized within Table 5.3.6 below for Layout 2 of Alternative 3. When comparing these results with Layout 1, it is apparent that the proposed pump station on the east side is more effective than the proposed pump station on the west side at alleviating the flooding problems. For the east service area, peak flood stages are moderately reduced and flood duration is significantly reduced by this pump station layout. This data also suggests that the pump station layout for the west service area is less effective at reducing both peak flood stages and flood duration than the proposed drainage wells under Alternative 2.

Table 5.3.6 – Alternative 3 (Layout 2) Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3 Layout 2	Reduction (%)
IN_2168	3.98	>33	17.6	47
IN_2166	3.88	>33	18.3	45
IN_2163	3.98	26.5	9.3	59
MH_0374	3.44	24.3	2.7	74
IN_2171	3.84	13	5.6	55

The reduction of peak flood stages and flood duration within the Lyons Park neighborhood are minimal for this Alternative 3 – Layout 2. Only areas immediately adjacent to the proposed pump stations receive

any significant benefit under this alternative, which are not considered problem areas for the Lyons Park neighborhood. Alternative 3 (Layout 2) should be eliminated from consideration as a potential solution since other system improvement alternatives considered for this study area provide better flood control benefits.

Layout 3

Under this layout option, the proposed construction includes three new pump stations within the problem areas of Lyons Park in order to increase the discharge capacity of the stormwater system. This additional discharge capacity from the stormwater pump stations will draw down the flooding within the problem areas of the Lyons Park neighborhood quicker than the existing conditions. Under this layout option, the proposed pump stations are located within the west service area at the problem area Node IN_2166, north of SW 12th Street along SW 4th Avenue and within the east service area at the problem area Node MH_0374 on the southeast corner of SW 12th Street and SW 1st Avenue and at the problem area Node IN_2171 on the intersection of SW 14th Street and SW 1st Terrace. An additional pump station was added to this layout option in the west service area in order to reduce the peak stages in both Node MH_0374 and Node IN_2171. The proposed pump station in the east service area is now located in the middle of the problem area at Node IN_2166. Pipe improvements to the existing stormwater system are also proposed to efficiently transmit stormwater runoff to the proposed pump stations in order to maximize the pump operational capacity and to reduce the possibility of creating flooding issues downstream of the proposed pump station. The estimated design and construction costs for this pump station alternative (layout 3) are $3,892,000. The components associated with each pump station are listed below.

West Service Area:

- The proposed wet well will have a total footprint of about 100 square feet and a depth of 8 feet at Node IN_2166.
- A pump discharge capacity will be 35 CFS. Larger discharge capacities were evaluated but this discharge rate was chosen as it is more feasible due to space constraints and no proportional benefit was observed in peak flood stage reduction at the higher discharge rates.
- Install a 24-inch discharge pipe from the pump station to the discharge point at Node IN_2163.
- Replace existing pipe downstream from Node IN_2168 with a 30-inch RCP pipe.
- Replace existing pipe downstream from Node IN_2163 with a 30-inch RCP pipe.
- Replace existing pipe downstream from Node MH_0369 with a 36-inch RCP pipe.
- Replace existing pipe downstream from Node IN_2125 with a 36-inch RCP pipe.

East Service Area:

- The proposed wet well will have a total footprint of about 100 square feet and a depth of 8 feet at Node MH_0374.
- The proposed wet well will have a total footprint of about 100 square feet and a depth of 8 feet at Node IN_2171.
- Install a 30-inch discharge pipe from the pump station at Node MH_0374 to the discharge point at Node MH_0375,
- Install a 30-inch discharge pipe from the pump station at Node IN_2171 to the discharge point at Node In_2176,
- The proposed pump capacity will be 50 CFS at Node MH_0374. Space constraints may limit the applicability of this pump capacity.
- The proposed pump capacity will be 30 CFS at Node IN_2171.
- Replace existing pipe downstream from Node IN_2176 with a 30-inch RCP.
- Replace existing pipe downstream from Node MH_0371 with a 36-inch RCP pipe.
- Replace existing pipe downstream from Node IN_2150 with a 48-inch RCP pipe.

* Replace existing pipe downstream from Node IN_2151 with a 48-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0375 with a 36-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0373 with a 36-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0372 with a 36-inch RCP pipe.
* Replace existing pipe downstream from Node MH_0409 with a 36-inch RCP pipe.
* Install a new 24-inch RCP pipe to interconnect Node IN_2171 and Node IN_2134 to provide additional reduction of peak flood stage in Node IN_2171.

The estimated reduction in peak flood stages for Alternative 3 – Layout 3 is summarized within Table 5.3.8. The reduction of the peak flood stage within the west service area are moderate for the pump station located at Node IN_2166, low for the pump station located at Node IN_2168, and negligible for the pump station located at Node IN_2163. Even though the components around the pump station at Node MH_0374 in the northwest service area are the same as for Layout 2, the peak stages are significantly reduced, which brings the estimated flood depth to 0.53 feet. The reason behind this improvement is that the pump station on Node IN_2171 is self-sufficient and no overland or underground flow interferes with the operation of the pump station at Node MH_0374.

The estimated flood duration within the Lyons Park neighborhood was significantly reduced at all nodes. The estimated flood duration within the Lyon Park neighborhood is summarized within Table 5.3.7 below for both the existing conditions and Layout 3. All critical nodes except for Node IN_2163, which are located downstream from the east service area pump station, are flooded for less than 5 hours.

Table 5.3.7 – Alternative 3 (Layout 3) Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3 Layout 3	Reduction (%)
IN_2168	3.98	>33	4.3	87
IN_2166	3.88	>33	4.0	88
IN_2163	3.98	26.5	14.6	45
MH_0374	3.44	24.3	1.5	94
IN_2171	3.84	13	3.2	76

Table 5.3.8 – Pump Station (Layouts 1/2/3) Peak Flood Stage Summary

Nodes	Existing			Alternative 3 Layout 1		Alternative 3 Layout 2		Alternative 3 Layout 3	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)	Peak Stage (feet)	Peak Reduction (feet)	Peak Stage (feet)	Peak Reduction (feet)
West Service Area – Dual Outfalls south of West McNab Road									
IN_2168*	4.78	3.98	0.80	4.78	0.00	4.74	-0.04	4.69	-0.09
IN_2166*	4.75	3.88	0.87	4.74	-0.01	4.65	-0.10	4.44	-0.31
IN_2163*	4.58	3.98	0.60	4.54	-0.04	4.42	-0.16	4.60	+0.02
MH_0369	4.59	5.36	0.00	4.41	-0.18	4.24	-0.35	4.45	-0.14
IN_2125**	4.55	3.74	0.81	4.00	-0.55	4.00	-0.55	4.00	-0.55
IN_2134**	4.41	4.11	0.30	4.26	-0.15	4.19	-0.22	3.78	-0.63
CS37A_TW	2.31	N/A	N/A	2.30	-0.01	2.32	N/A	2.31	N/A
Northeast Service Area – Outfall west of South Cypress Road									
MH_0379	4.78	4.57	0.21	4.78	0.00	4.70	-0.08	4.67	-0.11
MH_0374*	4.78	3.44	1.34	4.78	0.00	4.22	-0.56	3.97	-0.81
MH_0375	4.65	5.36	0.00	4.65	0.00	4.89	+0.24	4.89	+0.24
MH_0373	5.08	5.00	0.08	5.07	-0.01	4.92	-0.16	4.93	-0.15
MH_0372	4.83	4.66	0.17	4.81	-0.02	4.57	-0.26	4.60	-0.23
MH_0409**	2.40	5.66	0.00	4.69	N/A	3.20	N/A	3.22	N/A
BMCNAB_TW1	2.15	N/A	N/A	2.14	0.00	2.16	N/A	2.15	N/A
Southeast Service Area – Outfall along South Cypress Road									
IN_2171*	4.76	3.84	0.92	4.75	-0.01	4.65	-0.11	4.50	-0.26
IN_2176	4.77	4.97	0.00	4.77	0.00	4.52	-0.25	5.10	+0.33
MH_0371	4.79	4.91	0.00	4.80	0.00	4.55	-0.24	4.75	-0.04
IN_2155	4.62	4.22	0.40	4.61	-0.01	4.60	-0.02	4.60	-0.02
IN_2150	3.82	4.66	0.00	3.81	-0.02	3.80	-0.03	3.05	-0.78
IN_2151	2.69	5.05	0.00	2.69	-0.02	2.69	-0.02	2.80	+0.11
IN_2149**	2.40	4.99	0.00	2.40	-0.01	2.40	-0.01	2.60	+0.20
BCYPR_TW	2.20	N/A	N/A	2.20	-0.02	2.21	-0.01	2.20	N/A

Alternative Comparison

Refer to the Table 5.3.9 below for a comparison of the various system improvement alternatives for the Lyons Park Neighborhood. Please note the peak flood stage and flood reduction results within Table 5.3.9 refers to the average within the critical problem areas of the Lyons Park Neighborhood, which correspond to Node IN_2168, Node IN_2166, Node IN_2163, Node MH_0374, and Node IN_2171 within the stormwater model. Based on our analysis with the stormwater model, multiple system improvement alternatives (Alternative 1, Alternative 2, and Alternative 3-3) can be considered to be a slightly effective option for reducing the peak flood stage and reducing the flood duration within the Lyons Park Neighborhood. Although none of the considered alternatives provide enough additional flood protection to meet the level of service criteria for the public roadways, these alternatives do provide significant benefits which alleviate the flooding problems within the study area.

Table 5.3.9 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.17	18.24	$4,766,000
Alternative 2	0.20	18.16	$3,659,000
Alternative 3-1	0.01	4.86	$3,980,000
Alternative 3-2	0.19	15.26	$5,107,000
Alternative 3-3	0.37	20.44	$3,892,000

Due to various constructability concerns and regulatory limitations, the recommended stormwater improvement project incorporates elements of Alternative 1 which increase the discharge capacity of the existing stormwater system from the Lyons Park neighborhood by upsizing the primary pipe connections to the existing outfalls. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. Due to the residential nature of the Lyons Park neighborhood, the regrading of all existing grass swale areas along the neighborhood roadways will provide additional storage volume for stormwater runoff, which would provide a significant reduction in flooding throughout the neighborhood roadways.

If the City is considering the replacement of the existing sanitary sewer and water distribution infrastructure within the Lyons Park neighborhood, CMA recommends incorporating the recommended stormwater improvement with a potential neighborhood improvement project, which would enhance the cost efficiency of all projects by combining the restoration efforts. A neighborhood improvement project would also provide the opportunity to completely reconstruct the right-of-way cross section of all neighborhood roadways, which would provide the opportunity to elevate low lying roadway sections and to construct interconnected grass swale areas throughout the neighborhood.

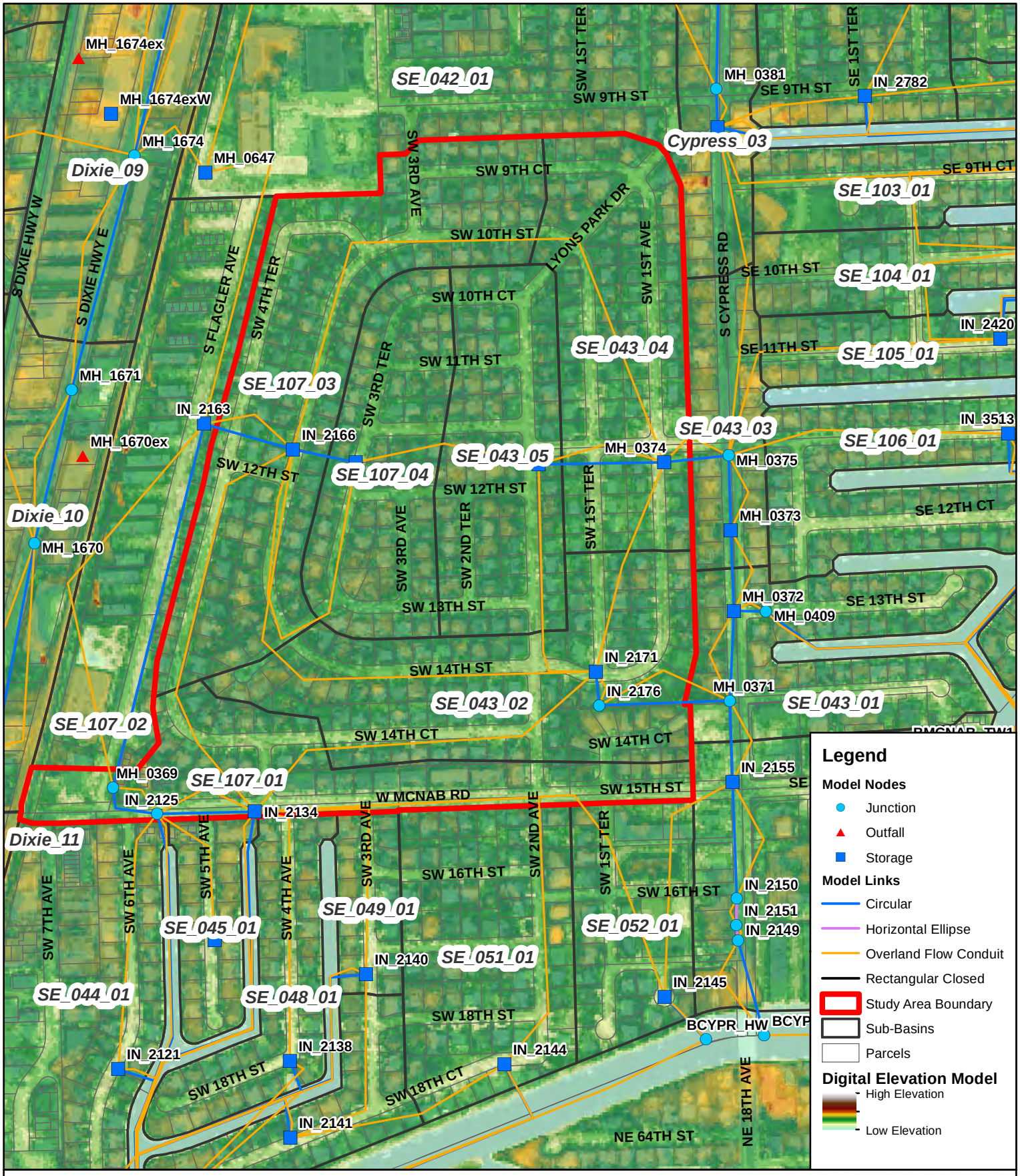


Figure 5-3A
Lyons Park Neighborhood
Existing Topography & Model Schematics

N
W E
S

0 250 500
Feet

Figure 5-3B
Lyons Park Neighborhood
Potential Flooding Depth

N
W E
S

0 250 500
Feet

5.2.4 STUDY AREA 4 – AVONDALE NEIGHBORHOOD

The Avondale Neighborhood was identified as a priority drainage basin in need of stormwater system improvements based on historical flooding problems observed by City staff, flooding complaints from residents or business operators, and the results from the existing conditions stormwater model. The Avondale Neighborhood is bound by I-95 to the west, SW 3rd Street to the south, Dixie Highway to the east and Atlantic Boulevard to the north. The Avondale Neighborhood typically experiences significant flooding throughout the area during heavy rainfall events. Based on the results of the existing conditions stormwater model along with the observations by City staff, the problem area is centered on SW 4th Avenue along with the adjacent intersecting roadways, which is where most of the critical flooding occurs.

The flooding problems within the Avondale Neighborhood are created primarily by the topography within the study area. In general, these roadways which form the perimeter around the study area have at a relatively higher ground surface elevation compared to the majority of the study. This ground surface topography allows stormwater runoff to flow into the study area from these perimeter roadways and also tends to trap any stormwater runoff within the study area. The ground surface elevation within the public right-of-way areas typically range between 3.5 feet and 5.5 feet NAVD, which is relatively low when compared to the groundwater elevation and tailwater elevation at the existing drainage outfalls into the SFWMD G16 Canal. The existing ground surface elevations are displayed within the Existing Topography Map on Figure 5-4A. Each of these factors leads to the past flooding problems which have been observed within the Avondale Neighborhood.

The existing drainage system within the Avondale Neighborhood consists of gravity pipes collecting stormwater runoff from the public right of way areas to eventually discharge to the SFWMD G16 Canal via positive outfalls. The primary system is a network of interconnected drainage pipe ranging from 15-inches to 36-inches in diameter which collects stormwater runoff from the central and western portions of the study area for eventual discharge into the SFWMD G16 Canal via 36-inch outfall pipe. There are also two small independent systems with individual 18-inch outfall into the SFWMD G16 Canal, which serve the northeast portion of the study area. The existing stormwater management system within the study area is displayed within the Existing Topography Map and the Potential Flooding Map enclosed at the end of this section. Although there are existing stormwater facilities within the Avondale Neighborhood, it does not provide an adequate level of service to the right-of-way areas within the study area. In order to alleviate the existing flooding problems within the Avondale Neighborhood, stormwater improvements will need to be implemented to enhance the performance of the existing stormwater management system within the study area.

CMA has reviewed various system improvement alternatives to determine the effectiveness in alleviating the existing flooding problems within the Avondale Neighborhood. Our analysis of the various stormwater improvements was conducted using the existing conditions model. The stormwater model was used to evaluate the performance of the existing stormwater management system within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The existing conditions stormwater model was modified to evaluate the effectiveness of various system improvement alternatives and compare the potential reduction in peak flood stage associated to these system improvements with the existing conditions. Due to the limitations caused by the existing topography within the study area, the proposed system improvement alternatives were analyzed to confirm compliance with a target level of service for public roadway areas during the 5-year, 24-hour design storm event. The target level of service is to reduce the peak flood depth to less than 0.5 feet at the lowest roadway elevation within each sub-basin and to reduce the duration of flooding within roadway areas to less than 5 hours.

Based on the results of the existing conditions stormwater model, the problem area within the Avondale Neighborhood is primarily located within the sub-basin SE_046_03 along SW 4th Avenue. During a storm

event, stormwater runoff typically overflows from adjacent sub-basin SE_046_02 into the problem area, which causes an additional accumulation of stormwater runoff along SW 4th Avenue. During the 5-year, 24-hour storm event, the peak runoff generated by these sub-basins is approximately 120 CFS for SE_046_03 and 55 CFS for SE_046_02. Some of this stormwater runoff is evacuated from the problem area by the existing drainage system into the SFWMD G16 Canal. The remaining stormwater runoff will accumulate within the low lying problem areas along SW 4th Avenue. The goal of the potential system improvement alternatives is to reduce the depth and duration of flooding within the problem areas of the Avondale Neighborhood while meeting regulatory and cost constraints. Prior to our analysis, several potential system improvement alternatives were eliminated from consideration due to the existing constraints within the study area. The following system improvement alternatives were not considered for the Avondale Neighborhood:

- Pipe Size Upgrades – Due to the low lying topography within the Avondale Neighborhood, increasing the diameter of the existing drainage piping would not significantly reduce depth or duration of flooding in the problem area. The lack of sufficient differential head between the ground surface elevation in the problem area and the tailwater elevation within the SFWMD G16 Canal would prevent any significant increase in the discharge capacity via the outfalls, even with an increased pipe diameter. The existing conditions stormwater model was used to conduct a preliminary analysis of potential increases in the pipe diameters of the existing stormwater management system. According to the results of this preliminary analysis, there is not adequate hydraulic head at any point along the existing drainage system where a pipe size upgrade could reduce the flooding depth or duration along SW 4th Avenue.
- Drainage Wells – Due to the lack of brackish groundwater at the bottom of the surficial aquifer in the below the Avondale Neighborhood, the installation of drainage wells was not considered as a potential system improvement alternative for this study area. The boundary of this brackish groundwater within the surficial aquifer is located just east of Dixie Highway. The installation of Class V drainage wells is not allowed outside of this zone by the regulatory agencies. Additionally, the subsurface soil conditions below the study area do not have a high hydraulic conductivity, which would limit the effectiveness of this option.

CMA has summarized our analysis of the various system improvement alternatives under consideration for the Avondale Neighborhood, which is outlined in the following sections:

Alternative 1: Exfiltration Trench

Alternative 1 includes the installation of new exfiltration trench with available right-of-way areas throughout the study area in order to provide additional storage and infiltration capacity to the existing stormwater system. For this potential system improvement alternative, the proposed construction would include the installation of exfiltration trench which extends into areas of the Avondale Neighborhood that are not currently served by the existing system. The goal of this alternative is to intercept stormwater runoff before it reaches that problems area and to provide additional storage and infiltration capacity within the study area. Under Alternative 1, the proposed construction would include approximately 4,900 linear feet of exfiltration trench, which would be aligned along streets in the Avondale Neighborhood, which do not currently have existing drainage infrastructure. In order for an exfiltration trench system to be effective, the existing ground surface should be greater than +5.0 feet NAVD. Exfiltration trench systems located in areas with ground surface elevations below this elevation are not likely to provide tangible benefits since the groundwater table will be located relatively close to the ground surface and would eliminate the storage capacity within the exfiltration trench. Under this system improvement alternative, the proposed exfiltration trenches were aligned in right–of-way areas with existing ground surface elevations which range between +5.0 feet NAVD and +5.5 feet NAVD. For this system

improvement alternative, the amount of proposed exfiltration trench and the average ground surface elevation is summarized per sub-basin within the Table 5.4.1 below.

Table 5.4.1 – Alternative 1 Proposed Exfiltration Trench Summary		
Sub-Basin	**Exfiltration Trench (LF)**	**Mean Ground Surface Elevation (feet NAVD)**
SE_046_02	2,473	+4.92
SE_046_03	1,484	+5.28
SE_047_01	165	+5.17
SE_058_01	796	+5.70
Total	**4,917**	**+5.16**

CMA conducted an analysis with the stormwater model of Alternative 2 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. While analyzing the effectiveness of this system improvement alternative, the following design parameters were assumed within the stormwater model for the proposed exfiltration trench during this evaluation:

- Trench Width: 5 feet
- Trench Height: 5 feet
- Perforated Pipe Diameter: 24-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on the results of our analysis of this system improvement alternative with the stormwater model, the peak flood stages within the Avondale Neighborhood are summarized for both the existing conditions and proposed conditions for comparison purposes within Table 5.4.2 below. As displayed within the table below, the peak flood stages would not be reduced within the Avondale Neighborhood by the implementation of the proposed exfiltration trench under Alternative 1.

Table 5.4.2 – Alternative 1 Peak Flood Stage Summary					
Nodes	**Existing Conditions**			**Alternative 1**	
	Peak Stage (feet, NAVD)	**Ground Elevation (feet, NAVD)** [1]	**Flood Depth (feet)**	**Peak Stage (feet, NAVD)**	**Peak Stage Reduction (feet)**
IN_2330 [2]	5.73	3.80	1.93	5.75	+0.02
IN_0376	5.73	4.84	0.89	5.74	+0.01
MH_0403	5.67	5.84	0.00	5.68	+0.01
IN_0372	5.68	5.86	0.00	5.68	0.0
IN_0370 [3]	5.68	5.31	0.37	5.68	0.0
IN_2346 [3]	5.66	5.79	0.00	5.66	0.0
IN_2344 [3]	5.66	5.05	0.61	5.66	0.0
PCG1610 [4]	5.65	N/A	N/A	5.66	N/A
PCG1609 [4]	5.67	N/A	N/A	5.68	N/A

[1] Reference ground elevation corresponds to adjacent centerline road elevation.
[2] Critical Problem Area Node
[3] Upstream Node from outfall
[4] G16 Canal Model Node

Based on the results of our analysis of this system improvement alternative with the stormwater model, the estimated flood duration within the Avondale Neighborhood is summarized for both the existing conditions and proposed conditions for comparison purposes within Table 5.4.3 below. The reduction in flood duration due to this system improvement alternative is minimal within the problem areas of the Avondale Neighborhood and null for the areas located immediately upstream from the outfalls into the SFWMD G16 Canal.

Table 5.4.3 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_2330 [1]	3.80	21.1	19.8	6
IN_0376	4.84	17.7	17.3	2
IN_0370 [2]	5.31	3.9	3.9	0
IN_2344 [2]	5.05	5.6	5.6	0

[1] Critical Problem Area Node
[2] Upstream Node from Outfall

The estimated reduction of the peak flood stages and flood durations for this system improvement alternative were minimal within the Avondale Neighborhood. The reason for this poor performance in regards to peak flood stage reduction is due to the proposed exfiltration systems being located within relatively high ground with respect to the problem areas along SW 4th Avenue. Although the proposed exfiltration system would intercept some stormwater runoff from these higher areas, stormwater would still accumulate too quickly within the low lying problem areas along SW 4th Avenue for the proposed system to drawdown the flooding. The performance of the proposed exfiltration system is also limited by the relatively low ground surface elevations along the proposed alignments, which leads to limited storage and infiltration capacity of the proposed exfiltration trench. The estimated implementation cost for design and construction of this system improvement alternative would be approximately $2,300,000, which would be a significant investment for minimal results.

Alternative 2: Stormwater Pump Station

For this potential system improvement alternative, the proposed construction would include the construction of a new stormwater pump station located within the problem area of the Avondale Neighborhood, which would discharge stormwater runoff directly into the SFWMD G16 Canal. The proposed pump station would be located at the northwest corner of SW 2nd Court and SW 4th Avenue. The proposed construction would also include the installation of a new transmission main from the new pump station to the outfall location at the SFWMD G16 Canal. This proposed modification to the existing system would help transmit any additional stormwater runoff not handled by the existing system and minimize the risk of flooding within the existing system downstream of these proposed improvements. The purpose of these proposed system modifications reduce the influence of elevated water levels in SFWMD G16 Canal on the performance of the existing stormwater system, which completely relies on gravity discharge. The proposed pump station will ensure that the peak allow discharge rate from the outfalls is met, even during times when the canal level is elevated. The analysis of this system improvement alternative includes the following assumptions within the stormwater model:

- Precast wetwell structure with a footprint of approximately 100 square feet and a depth of 8 feet.
- Axial flow pumps with discharge capacity of approximately 30 CFS.
- 30-inch force main (1,800 linear feet) discharging to the SFWMD G16 Canal aligned along Avondale Drive, SW 2nd Street, and SW 5th Avenue.
- Backflow prevention devices at the existing outfalls to the SFWMD G16 Canal.

- Reversed flow direction within the existing drainage pipes located along Avondale Drive in order to collect runoff and transmit to the wetwell of the stormwater pump station.
- Controlled disconnect between the existing gravity pipe discharging to the SFWMD G16 Canal and the proposed pump station to prevent recycling of stormwater backflow.
- The existing drainage system is left unchanged along SW 5th Avenue, which will continue to discharge into the SFWMD G16 Canal via the 36-inch RCP pipe.
- All existing swale areas along roadways within the study area will be regraded to provide additional storage volume for stormwater runoff.

Due to regulatory limitations, the future discharge rate from study area cannot exceed the existing peak discharge rate into the SFWMD G16 Canal via the existing outfall. In order to meet this regulatory requirement, the proposed pump capacity was assumed to be equivalent to the peak discharge via Link IN_0370_1 during low levels within the canal, which means the discharge rate into the SFWMD G16 Canal will remain constant, even during elevated levels within the canal. The existing conditions stormwater model predicts a peak discharge Link IN_0370 of 30 CFS during low canal levels of -2.0 feet NAVD elevation. The stormwater model was used to estimate for this system alternative with a pump station with discharge capacities of either 30 CFS and 40 CFS discharge capacities. The simulation with a 40 CFS pump capacity was intended to determine if any additional peak stage reductions could be expected by increasing discharge capacity above the expected allowable peak discharge. Based on the results of our analysis of this system improvement alternative with the stormwater model, the peak flood stages within the Avondale Neighborhood are summarized for both the existing conditions and proposed conditions for comparison purposes within Table 5.4.4 below.

Table 5.4.4 – Alternative 2 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 2 (30 CFS)		Alternative 2 (40 CFS)	
	Peak Stage (feet)	Ground Elevation (feet, NAVD) [1]	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2330 [2]	5.73	3.80	1.93	4.47	-1.26	4.48	-1.25
IN_0376	5.73	4.84	0.89	4.85	-0.88	4.85	-0.88
MH_0403	5.67	5.84	0.00	4.8	-0.87	4.83	-0.84
IN_0372	5.68	5.86	0.00	4.94	-0.74	4.97	-0.71
IN_0370 [3]	5.68	5.31	0.37	5.43	-0.25	5.44	-0.24
IN_2346 [3]	5.66	5.79	0.00	5.77	+0.11	5.81	+0.13
IN_2344 [3]	5.66	5.05	0.61	5.77	+0.11	5.81	+0.13
PCG1610 [4]	5.65	N/A	N/A	5.77	N/A	5.81	N/A
PCG1609 [4]	5.67	N/A	N/A	5.79	N/A	5.83	N/A

[1] Reference ground elevation corresponds to adjacent centerline road elevation.
[2] Critical Problem Area Node
[3] Upstream Node from Outfall
[4] G16 Canal Model Node

The peak flood stage within the problem area of the Avondale Neighborhood, which is represented in the stormwater model by Node IN_2330, is significantly reduced by this system improvement alternative. No additional peak stage reduction is predicted by the stormwater model by using a pump station with discharge capacity of 40 CFS. Even though more volume is pumped out of the problem area with this greater pump capacity, a proportional amount of volume is introduced to the problem area by a ‘sink’ type effect of the surrounding topography.

It should be noted that the water level at the SFWMD G16 Canal, which is represented in the stormwater model by Nodes PCG1609 and PCG1610, increases by 0.12 feet at both nodes due to the additional flow from the proposed 30 CFS pump station. The water level rose at the SFWMD G16 Canal by 0.15 feet due to additional flow from the 40 CFS pump station simulation. A higher water level at the SFWMD G16 Canal with respect to the adjacent outfalls and terrain will reduce discharge capacity of the existing gravity outfalls. This reduction in discharge capacity is reflected within the stormwater model with an increase in peak stages at Nodes IN_2344 and IN_2346. The increased stages within the canal indicate that the downstream Water Control Structure G57 is operating at full capacity during the 5-year, 24-hour design storm.

Based on the results of our analysis of this system improvement alternative with the stormwater model, the estimated flood duration within the Avondale Neighborhood is summarized for both the existing conditions and proposed conditions for comparison purposes within Table 5.4.5 below. The flood duration is estimated to be the time from when the flood stage exceeds the respective reference ground elevation (i.e. lowest roadway elevation) until the flood stage falls below this elevation. The 30 CFS pump station option reduces down the flood depth from 1.93 feet to 0.68 feet above the reference ground elevation at Node IN_2330. The reduction in flood duration estimated under this system improvement alternative is estimated to be quite positive with 57% reduction along SW 4Th Avenue and 100% reduction along Avondale Drive. In reference to Node IN_2330, the flood stage is estimated to remain above the reference ground elevation of +3.8 feet for approximately 9 hours during the 5-year, 24-hour event. For most of this timeframe, the flood depth is estimated to be about 0.3 feet or less, with a short peak of about 0.6 feet lasting less than an hour. Therefore, the reduction in flood duration does not show that the established level of service is not met for only a period of less than an hour in this area.

Table 5.4.5 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2 (30 CFS)	Reduction (%)
IN_2330 [1]	3.80	21.1	9.1	57
IN_0376	4.84	17.7	0	100
IN_0370 [2]	5.31	3.9	5	0
IN_2344 [2]	5.05	5.6	5.7	0

[1] Critical Problem Area Node
[2] Upstream Node from Outfall

As discussed previously, an unintended consequence of this system improvement alternative is the increase in peak flood stages within Nodes IN_2346 and IN_2344, which are located adjacent to SFWMD G16 Canal. The discharge capacity of the existing gravity outfalls is being slightly compromised by the higher stages predicted within the SFWMD G16 Canal caused by the additional discharge from proposed pump station. In reference to Node IN_0370, this system improvement alternative reduces its peak stage by 0.24 feet, but it also extends the flooding duration by about one hour due to the increased water level within the canal. Based on our analysis of this system improvement alternative, there would be some significant reductions in the peak flood stage and flood duration throughout the Avondale Neighborhood but there are some additional regulatory considerations which would need to be address during detailed design. The regulatory agencies will limit the peak discharge and the operating conditions from the proposed pump station along with requiring some form of water quality treatment, likely regraded swale areas. The estimated implementation cost for design and construction of this system improvement alternative would be approximately $3,064,000.

Alternative 3: Stormwater Pump Station + Stormwater Retention Area

For this potential system improvement alternative, the proposed construction would include the construction of a new stormwater pump station located within the problem area of the Avondale Neighborhood, which would discharge stormwater runoff directly into a new stormwater retention area(s). The proposed pump station would be located at the northwest corner of SW 2nd Court and SW 4th Avenue. The proposed construction would also include the installation of a new transmission main from the new pump station to the outfall location at the new stormwater retention area. A weir-type control structure would be installed within the stormwater retention area to allow collected stormwater runoff to be connected back to the existing stormwater system for drawdown and overflow purposes.

Under Alternative 3, the proposed stormwater retention area was assumed to encompass a total area of 0.81 acres at a location to be investigated by the City. The average ground surface elevation of the study area is about 5 feet NAVD. A perimeter berm could be be constructed up to 8 feet NAVD and with a 3:1 side slope, the bottom of the pond will be at an elevation of +3.5 feet NAVD, which is 1 foot higher than the seasonal high water table expected at +2.5 feet NAVD. The areas of the storage retention area are assumed to be approximately 0.58 acres at the top of the berm and 0.26 acres at the bottom of the retention area. The water control structure within the stormwater retention area was set with a control elevation of +7.5 feet NAVD, which provides for an additional storage/retention volume of approximately 1.6 acre-feet. This system improvement alternative attenuates the stormwater runoff within the retention in order to delay the discharge through the existing drainage system into the SFWMD G16 Canal until the canals levels are favorable to gravity discharge.

During our analysis of this system improvement alternative with the stormwater model, the pump capacity of the proposed pump discharging into the storage retention area was varied to evaluate its role in the proposed system. Based on the results summarized in Table 5.4.6 below, this system improvement alternative significantly reduces the peak flood stages within the Avondale Neighborhood by over 12 inches from the existing conditions. However, the provided level of service is still unacceptable as the flood depth was estimated to be 0.81 feet above the referenced ground elevation within the roadway areas. A simulation with the stormwater model was also performed to evaluate the effectiveness of using a pump with a higher capacity of 15 CFS in reducing peak flood stages around the problem area. Based on our analysis, an additional increase in pump capacity did not reduce peak flood stages further within the Avondale Neighborhood.

Table 5.4.6 – Alternative 3 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 3 (10 CFS)		Alternative 3 (15 CFS)	
	Peak Stage (feet)	Ground Elevation (feet, NAVD) [1]	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2330 [2]	5.73	3.80	1.93	4.61	-1.12	4.61	-1.12
IN_0376	5.73	4.84	0.89	4.86	-0.87	4.86	-0.87
MH_0403	5.67	5.84	0.00	4.91	-0.76	4.91	-0.76
IN_0372	5.68	5.86	0.00	5.02	-0.66	5.02	-0.66
IN_0370 [3]	5.68	5.31	0.37	5.32	-0.36	5.32	-0.36
IN_2346 [3]	5.66	5.79	0.00	5.64	-0.02	5.64	-0.02
IN_2344 [3]	5.66	5.05	0.61	5.64	-0.02	5.64	-0.02
PCG1610 [4]	5.65	N/A	N/A	5.64	-0.01	5.64	-0.01
PCG1609 [4]	5.67	N/A	N/A	5.66	-0.01	5.66	-0.01

[1] Reference ground elevation corresponds to adjacent centerline road elevation.

[2] Critical Problem Area Node
[3] Upstream Node from Outfall
[4] G16 Canal Model Node

Based on the results of our analysis of this system improvement alternative with the stormwater model, the estimated flood duration within the Avondale Neighborhood is summarized for both the existing conditions and proposed conditions for comparison purposes within Table 5.4.7 below. Under this system improvement alternative, the stormwater model predicts a 36% reduction in flood duration within the problem areas of the Avondale Neighborhood at Node IN_2330. Although the flood duration is longer under Alternative 3 than under Alternative 2 according to our analysis, this alternative does not increase water levels within the canal, which prevent negative flooding impacts in the areas adjacent to the canal (Node IN_2344 and Node IN_2346).

Table 5.4.7 – Alternative 3 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3 (10 CFS)	Reduction (%)
IN_2330 [1]	3.80	21.1	13.5	36
IN_0376	4.84	17.7	0	100
IN_0370 [2]	5.31	3.9	0.3	92
IN_2344 [2]	5.05	5.6	4.9	13

[1] Critical Problem Area Node
[2] Upstream Node from Outfall

Based on our analysis of this system improvement alternative, there would be some significant reductions in the peak flood stage and flood duration throughout the Avondale Neighborhood but there are some additional considerations which would need to be address during detailed design. The property costs for the proposed retention area along with opposition from local residents could eliminate Alternative 3 from consideration. The estimated implementation cost for design and construction of this system improvement alternative would be approximately $3,237,000.

Alternative 4: Composite (Alternative 2 + Alternative 3)

As previously noted above, there are some disadvantages to both Alternative 2 and Alternative 3, which could negatively impact the flooding problem within isolated portions of the Avondale Neighborhood. Under this system improvement alternative, the components of the proposed improvement under Alternative 2 and Alternative 3 will be merged for the purpose of maximizing the advantages of each alternative. For this potential system improvement alternative, the proposed construction would include the following elements previously defined within Alternative 2 and Alternative 3:

- Precast wetwell structure with a footprint of approximately 100 square feet and a depth of 8 feet.
- Axial flow pumps with discharge capacity of 30 CFS to the SFWMD G16 Canal.
- Axial flow pumps with discharge capacity of 10 CFS to the new retention area.
- 30-inch force main discharging to the SFWMD G16 Canal.
- 18-inch force main discharging to the new stormwater retention area.
- New stormwater retention area.
- Weir-type control structure at the new stormwater retention area.
- Backflow prevention devices at the existing outfalls to the SFWMD G16 Canal.
- Reversed flow direction within the existing drainage pipes located along Avondale Drive in order to collect runoff and transmit to the wetwell of the stormwater pump station.

- Controlled disconnect between the existing gravity pipe discharging to the SFWMD G16 Canal and the proposed pump station to prevent recycling of stormwater backflow.
- The existing drainage system is left unchanged along SW 5th Avenue.

Based on the results of our analysis of this system improvement alternative with the stormwater model, the peak flood stages within the Avondale Neighborhood are summarized for both the existing conditions and proposed conditions for comparison purposes within Table 5.4.8 below. These peak flood stage results indicate that Alternative 4 does not provide additional reductions flooding from either Alternative 2 or Alternative 3. Alternative 4 does not provide better level of service to the study area than either Alternative 2 or Alternative 3. The performance of Alternative 4 was limited due to the lack of a system control which would isolate portions of the existing drainage system from the proposed pump station during pump operations, which leads to the recycling of stormwater which backflows back into the existing drainage system.

Table 5.4.8 – Alternative 4 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 4	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD) [1]	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_2330 [2]	5.73	3.80	1.93	4.38	-1.35
IN_0376	5.73	4.84	0.89	4.86	-0.87
MH_0403	5.67	5.84	0.00	4.87	-0.80
IN_0372	5.68	5.86	0.00	5.01	-0.67
IN_0370 [3]	5.68	5.31	0.37	5.33	-0.35
IN_2346 [3]	5.66	5.79	0.00	5.71	+0.05
IN_2344 [3]	5.66	5.05	0.61	5.72	+0.06
PCG1610 [4]	5.65	N/A	N/A	5.71	N/A
PCG1609 [4]	5.67	N/A	N/A	5.73	N/A

[1] Reference ground elevation corresponds to adjacent centerline road elevation.
[2] Critical Problem Area Node
[3] Upstream Node from Outfall
[4] G16 Canal Model Node

Table 5.4.9 – Alternative 4 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 4	Reduction (%)
IN_2330 [1]	3.80	21.1	11.0	48
IN_0376	4.84	17.7	0.5	97
IN_0370 [2]	5.31	3.9	2.0	49
IN_2344 [2]	5.05	5.6	5.6	0

[1] Critical Problem Area Node
[2] Upstream Node from Outfall

Based on our analysis of this system improvement alternative, there would be some significant reductions in the peak flood stage and flood duration throughout the Avondale Neighborhood but these benefits are equivalent to Alternative 2. The estimated implementation cost for design and construction of this system improvement alternative would be approximately $4,514,000. Alternative 4 should be eliminated from consideration since the additional construction cost results does not result in any additional benefits beyond Alternative 2.

Alternative Comparison

Refer to the Table 5.4.10 below for a comparison of the various system improvement alternatives for the Avondale Neighborhood. Please note the peak flood stage and flood reduction information within Table 5.4.10 refers to Node IN_2330 in the stormwater model, which corresponds to the critical problem area within the Avondale Neighborhood. Based on our analysis of the various system improvement alternatives with the stormwater model, Alternative 2 with the stormwater pumping to the SFWMD G16 Canal is the most effective option for reducing the peak flood stage and reducing the flood duration within the Avondale Neighborhood. Although Alternative 2 does not provide enough additional flood protection to meet the level of service criteria for the public roadways, this alternative does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.4.10 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0	1.3	$2,300,000
Alternative 2	1.25	12.0	$3,064,000
Alternative 3	1.12	7.6	$3,237,000
Alternative 4	1.35	10.1	$4,514,000

This project includes discharge into the G-16 which is currently listed as an impaired water by the Florida Department of Environmental Protection and therefore a level of water quality improvement is critical to the implementation. Due to the relatively low existing elevations within the area and regulatory limitations, the recommended Phase 1 option for a stormwater improvement project for this study area incorporates Alternative 2, along with elements of Alternative 4, which increase the discharge capacity of the existing stormwater system from the Avondale neighborhood. The recommended stormwater improvements include the construction of a new stormwater pump station, which pumps into potential dry retention area(s) throughout the study area, which allow for the overflow into the SFWMD G16 Canal. These areas are currently conceptual in location to provide opportunities for the City to review existing land prices or to utilize existing City owned property. One such option would be to include a provision for overflow storage during significant events in Avondale Park, providing temporary relief to minimize flood stages prior to discharge into the G-16 canal. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

If the City wishes to build on Phase 1 improvements and implement comprehensive basin-wide stormwater improvements which would meet level of service throughout all City right-of-way areas within this study area, the only option would be the construction of a new centralized stormwater system which serves the entire study area. This new stormwater system for this study area would need flow into a new dry retention area(s) to provide significantly more storage capacity for stormwater runoff. Based on our preliminary calculations, the new dry retention area would need to provide at least 7 acre-feet in additional storage capacity in order to meet level of service criteria within City right of way areas. Depending on the configuration of the proposed dry retention area, the City would need between 5 to 8 acres of available land area. Based on our review of the study area, there is not adequate City owned or vacant land available for use for a dry retention area. In order to implement this option, the City would need to purchase property within the study area, demolish any existing structures and construct a new dry retention area. Since the bulk of the existing stormwater system flows toward the SFWMD G16 Canal, the ideal location of the dry retention area would be along the canal alignment, which would allow overflow into the canal. Due to the very low ground surface elevation within this study area, the proposed dry retention area(s) would need to be slightly elevated to provide adequate storage capacity. The

installation of a new pump station would be required in order to transmit stormwater runoff to the new elevated dry retention area(s). Due to extensive obstacles to option, more detailed investigation is recommended if the City wishes to pursue this option.

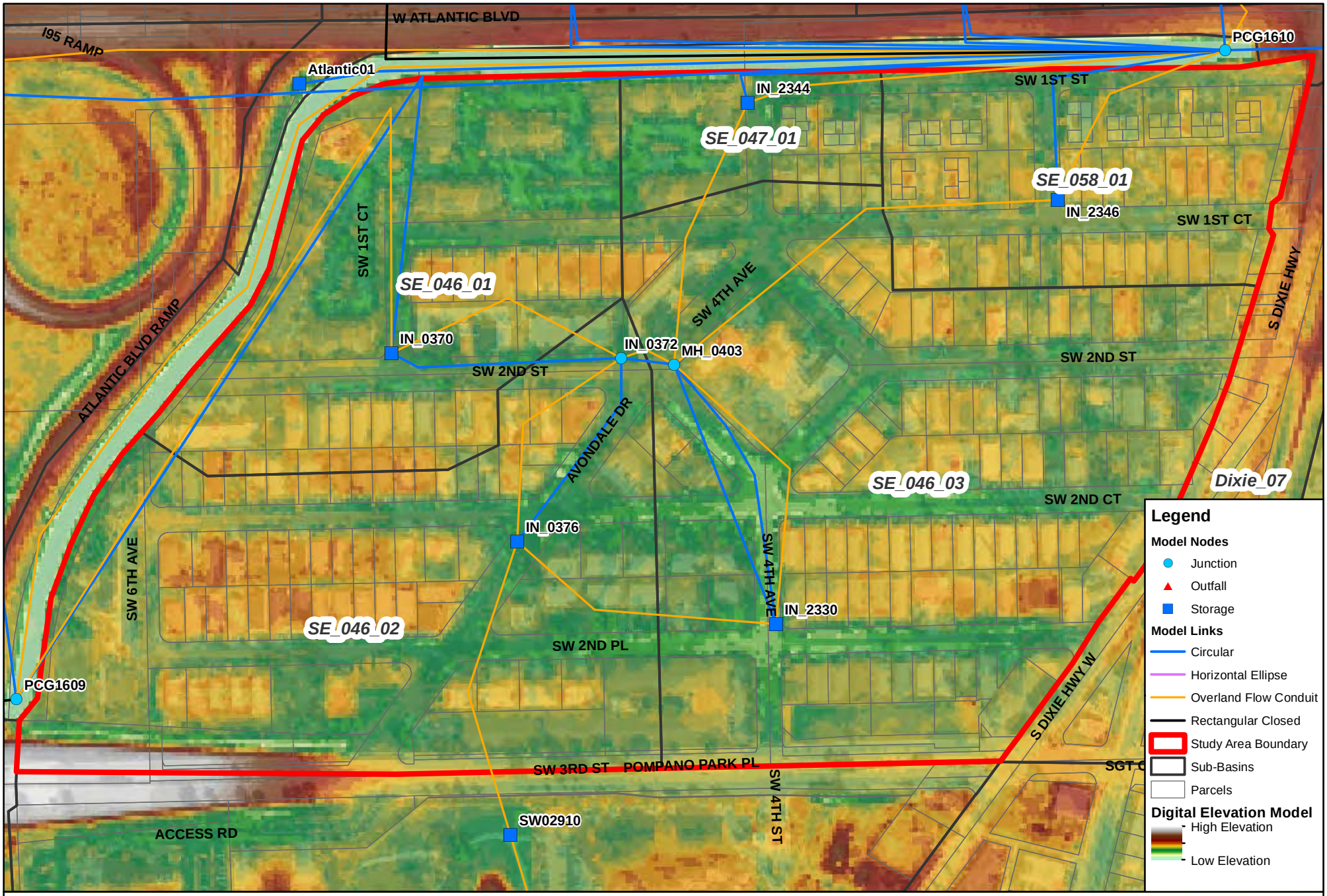


Figure 5-4A
Avondale Neighborhood
Existing Topography & Model Schematics

0 150 300 Feet

N W E S

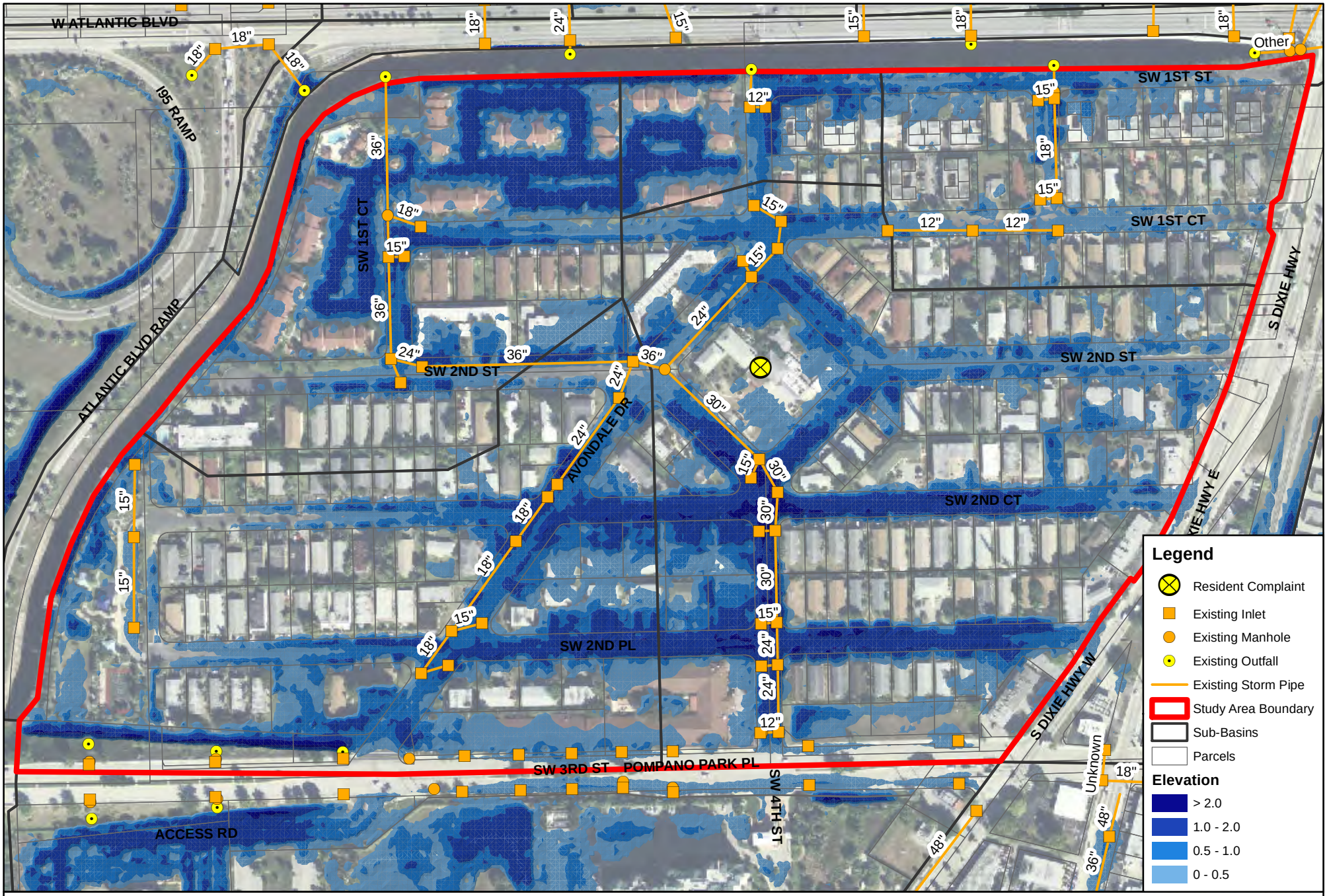


Figure 5-4B
Avondale Neighborhood
Potential Flooding Depth

5.2.5 STUDY AREA 5 – ESQUIRE LAKE NEIGHBORHOOD

The study area for the Esquire Lake Neighborhood is located on the west side of the Powerline Road, south of Martin Luther King Boulevard. This residential neighborhood contains a lake towards the east side, which collects runoff from all local roadways through gravity stormwater pipes ranging from 12 inches to 36 inches. The lake has a weir type control structure that overflows to the system on Powerline Road. The topography of the study area along with the model schematics of the model are displayed within Figure 5-5A at the end of this section. The project area is enclosed by the sub-basins CW_038_04, CW_025_01, CW_026_01, CW_026_02, CW_015_01, CW_016_01, WAT_04 and Pwrlne_04.

The existing conditions stormwater model was used to simulate the performance of the existing stormwater management system in the study area during a 5-year, 24-hour design storm event with a rainfall of 7.8 inches. Based on the results of the stormwater model, the extent of estimated flooding within the Esquire Lake Neighborhood is displayed in Figure 5-5B at the end of this section. As displayed on the map, the expected flooding is more significant along NW 9th Street, NW 23rd Terrace and NW 10th Court, with the remaining roadways in the study area displaying flooding of less than one inch depth. System improvement alternatives investigated for this study area include pipe size upgrades and exfiltration trenches. Drainage wells are not a feasible option since the study area is located too far west.

Alternative 1: Pipe Size Upgrades

The stormwater model was used to conduct several simulations of various proposed pipe size upgrades at specific locations within the existing stormwater system. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. Alternative 1 included the replacement of the existing pipes which discharge into Esquire Lake with larger diameter pipe. Under Alternative 1, the existing 12-inch pipe at NW 23rd Terrace and NW 9th Street will be upsized to a 30-inch RCP pipe and the existing 36-inch pipe will be upsized to a 42-inch, which includes a total pipe replacement of 960 linear feet. The estimated design and construction costs for this pipe size upgrade alternative are approximately $909,000. Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 1 are summarized in Table 5.5.1 below. The model results show the average reduction in peak flood stage is less than 0.5 feet under Alternative 1. At the critical model node within the study area (MH_0104), the flood depth was reduced from 1.26 feet to 0.70 feet under Alternative 1.

Table 5.5.1 – Alternative 1 Peak Flood Stage Summary

	Existing Conditions			Alternative 1	
Nodes	**Peak Stage (feet, NAVD)**	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet, NAVD)**	**Peak Stage Reduction (feet)**
IN_0568	11.12	10.40	0.72	11.11	-0.01
IN_0581	10.91	9.70	1.21	10.42	-0.49
IN_0585	10.24	9.80	0.44	10.26	0.00
IN_0588	10.26	9.80	0.46	10.28	0.00
WAT04	10.24	N/A	N/A	10.27	N/A
MH_0093	11.31	10.40	0.91	11.28	-0.03
MH_0100	11.00	9.90	1.10	10.68	-0.32
MH_0104	10.96	9.70	1.26	10.26	-0.70
MH_1314	13.15	13.00	0.15	13.15	0.00
IN_0597	12.98	12.10	0.88	12.98	0.00

Based on the results of our analysis with the stormwater model, the expected reduction in flood duration throughout the study area under Alternative 1 is summarized in Table 5.5.2 below. The expected reduction in flooding duration within the study area is relatively insignificant peak flood reduction, with a reduction of 16% or less estimated throughout the study area. At the critical model node within the study area (MH_0104), the average flood duration was reduced from 20.1 hours under the existing conditions to 16.9 hours under Alternative 1.

Table 5.5.2 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_0568	10.40	9.75	8.6	12
IN_0581	9.70	20.8	20.4	2
MH_0093	10.40	11.5	10.0	13
MH_0100	9.90	17.7	17.7	0
MH_0104	9.70	20.1	16.9	16

Alternative 2: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within portions of the study area not currently served by the existing stormwater system. The purpose of this system improvement alternative is to intercept stormwater runoff before it reaches the existing outfalls and to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 2, the installation of proposed exfiltration trench was aligned along roadways in the study area with a minimum ground surface elevation greater than +5.0 feet NAVD. The expected construction includes a total of 7,416 linear feet of new exfiltration trench, which is summarized by sub-basin within Table 5.5.3 below. The estimated design and construction costs for this exfiltration trench alternative are approximately $3,008,000.

Table 5.5.3 – Alternative 2 Proposed Exfiltration Trench Summary

Sub-Basin	Exfiltration Trench (LF)	Mean Ground Surface Elevation (feet NAVD)
CW_025_01	510	+10.14
CW_026_01	1,155	+10.16
CW_026_02	3,996	+10.64
CW_038_04	1,755	+11.10
Total	**7,416**	**+10.51**

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. The following design parameters for the proposed exfiltration trench were assumed within the stormwater model during this evaluation:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} cfs/ft^2-ft head

Based on our analysis with the stormwater model, the expected reduction of peak flood stages under Alternative 2 are summarized within Table 5.5.4 below. The model results show estimated reductions of the peak flood stages throughout the study area to be up to 0.44 feet. At the critical model node within the study area (MH_0104), the average flood depth was reduced from 1.26 feet under the existing conditions to 0.82 feet under Alternative 2.

Table 5.5.4 – Alternative 2 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 2	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_0568	11.12	10.4	0.72	11.09	-0.03
IN_0581	10.91	9.7	1.21	10.54	-0.37
IN_0585	10.24	9.8	0.44	9.88	-0.36
IN_0588	10.26	9.8	0.46	9.90	-0.36
WAT04	10.24	N/A	N/A	9.88	-0.36
MH_0093	11.31	10.4	0.91	11.30	-0.01
MH_0100	11.00	9.9	1.1	10.97	-0.03
MH_0104	10.96	9.7	1.26	10.52	-0.44
MH_1314	13.15	13	0.15	13.15	0.00
IN_0597	12.98	12.1	0.88	12.98	0.00

Based on our analysis with the stormwater model, the expected reduction of flood duration under Alternative 2 is summarized within Table 5.5.5 below. Alternative 2 reduces the flood duration within the study area up to 70% from the existing conditions. At the critical model node within the study area (MH_0104), the average flood duration was reduced from 20.1 hours under the existing conditions to 9.6 hours under Alternative 2.

Table 5.5.5 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_0568	10.40	9.75	6.9	29
IN_0581	9.70	20.8	6.2	70
MH_0093	10.40	11.5	12.6	-10
MH_0100	9.90	17.7	12.8	28
MH_0104	9.70	20.1	9.6	52

Alternative Comparison

Refer to Table 5.5.6 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood duration reduction results within Table 5.5.6 refers to Node MH_0104 in the stormwater model, which corresponds to the critical problem area within the study area. Based on our analysis with the stormwater model, both system improvement alternatives provide flood control benefits to the study area. Alternative 1 is slightly more effective than Alternative 2 at

reducing the peak flood stages within the study area. Alternative 2 is significantly more effective than Alternative 1 at reducing the expected flood duration within the study area. Alternative 2 should be implemented for this study area since it provides the best potential flood control benefits. Although Alternative 2 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 2 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.5.6 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.70	3.2	$909,000
Alternative 2	0.44	10.5	$3,008,000

CMA recommends the installation of exfiltration trench in targeted City right-of-ways which will intercept stormwater runoff before it flows into Esquire Lake and will provide additional storage and infiltration capacity for stormwater runoff. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along City roadways throughout the study area to collect stormwater runoff from these areas. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 2 will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

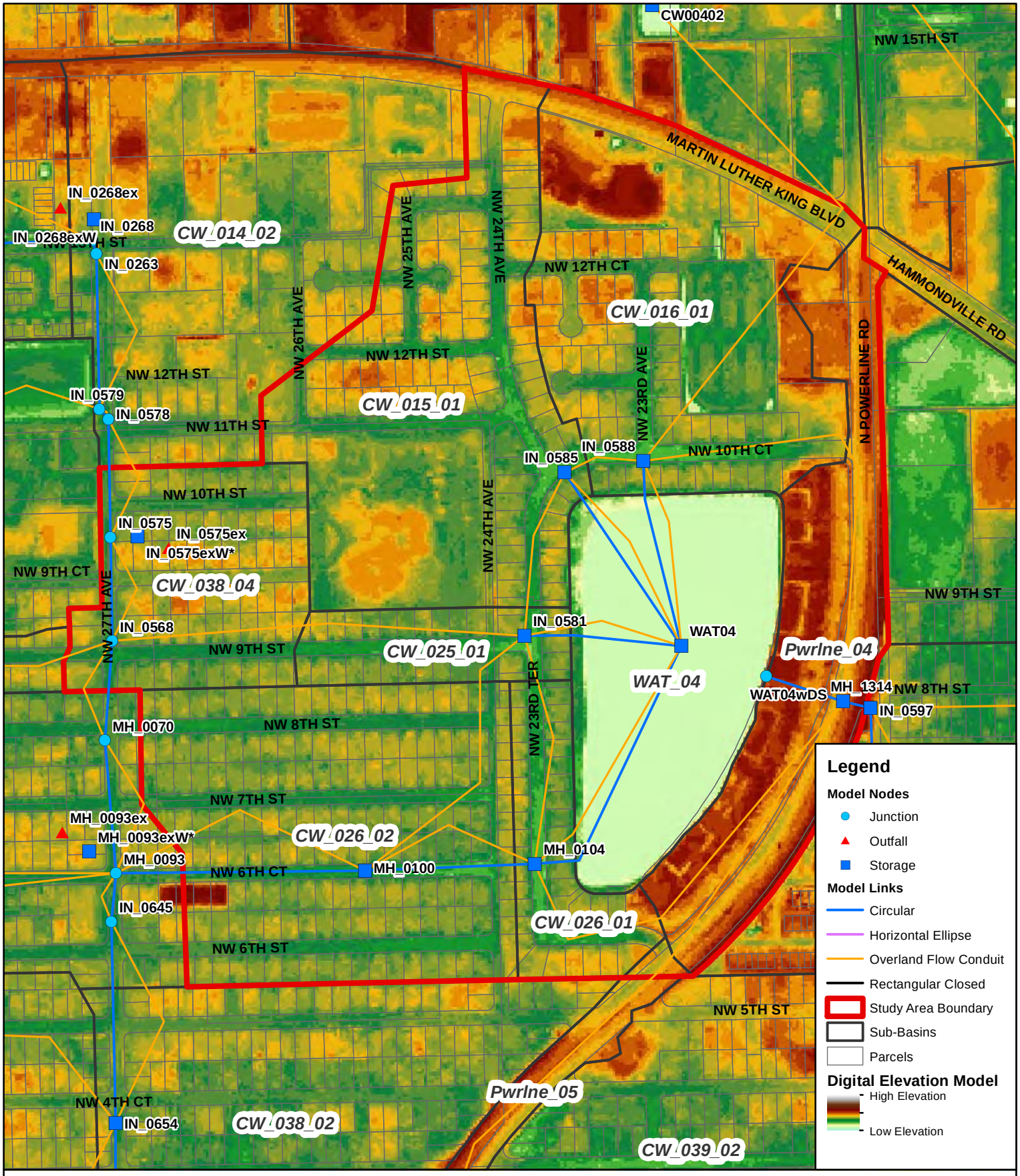


Figure 5-5A
Esquire Lake Neighborhood
Existing Topography & Model Schematics

N
W E
S

0 250 500
Feet

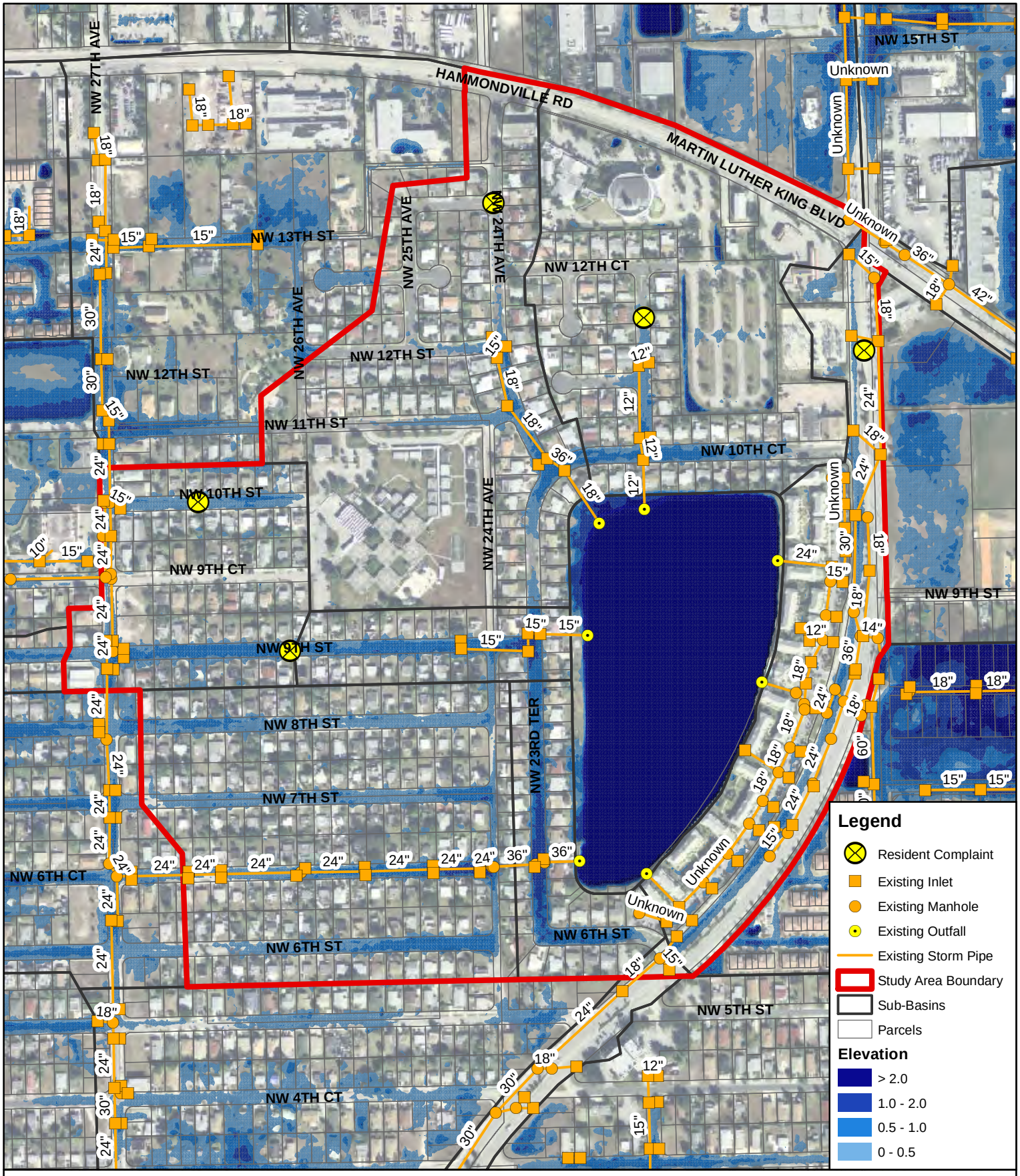


Figure 5-5B
Esquire Lake Neighborhood
Potential Flooding Depth

N
W E
S

0 250 500
Feet

5.2.6 STUDY AREA 6 – GATEWAY DRIVE

The Gateway Drive study area is a commercial and industrial neighborhood bounded by West McNab Road to the south, by Powerline Road to the east, by SW 36th Avenue to the west and by SFWMD C14 Canal to north. Due to the commercial nature of the study area, the public right-of-way areas have a high percentage of impervious ground coverage, which limits the infiltration of stormwater runoff into the ground surface. The public roadways within the study area have a limited existing stormwater system which discharges into a stormwater pond with an overflow connection to the SFWMD C-14 Canal. According to resident complaint information, the potential flooding areas are located in right-of-way areas without existing drainage facilities, which is along SW 29th Avenue, SW 28th Avenue, and SW 27th Avenue.

The existing conditions stormwater model was used to evaluate the performance of the existing stormwater system in the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basin CE_015_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-6A at the end of this section. According to the topography, the private properties within the study area typically have a higher elevation than the adjacent public right-of-way areas, which causes stormwater runoff to collect on these low lying roadways. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-6B at the end of this section. According to the model results, the expected flooding is quite extensive within the study area but only few roadways have a flooding depth of greater than 1 inch.

The stormwater model was used to evaluate the effectiveness of various system improvement alternatives on reducing the flooding problems within the study area. The system improvement alternative for this study area included various configurations of proposed exfiltration trench and new connections with existing stormwater systems. Other system improvement alternatives which increase the discharge capacity of the existing system, such as pump stations or upsized outfall pipes, were not considered due to regulatory restrictions. The water levels in the SFWMD C14 Canal and the stormwater pond are controlled by existing permit restrictions, which limit the peak discharge of stormwater runoff from the study area. The system improvement alternatives which were evaluated with the stormwater model are summarized below.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area not currently served by the existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 5,780 LF of exfiltration trench, which was aligned along right-of-way areas with ground surface elevations greater than +5.0 feet NAVD. The proposed exfiltration trenches were aligned inside the study area along Gateway Drive, SW 26th Avenue, SW 27th Avenue and SW 30th Avenue. The estimated design and construction costs for this exfiltration trench alternative are approximately $2,486,000.

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet

- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.6.1 below. According to the model results, Alternative 1 causes a minor reduction in peak flood stages with a maximum reduction of -0.32 in the study area. At the critical model nodes (Node IN_0002 and Node IN_0003), the average peak flood depth was reduced from 1.11 feet under the existing conditions to 1.10 feet under Alternative 1.

Table 5.6.1 – Alternative 1 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 1	
	Peak Stage (feet)	Ground Elevation (feet)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_0003	5.49	3.8	1.69	5.49	0.00
IN_0002	5.62	5.1	0.52	5.60	-0.02
IN_0020	6.50	4.4	2.1	6.51	+0.01
IN_0008	5.43	4.7	0.73	5.40	-0.03
WCD4_1	5.64	N/A	5.64	5.60	-0.04
WCD4_2	6.01	N/A	6.01	6.03	+0.02
WCD4_3	5.80	N/A	5.8	5.68	-0.12
WCD4_4	5.86	N/A	5.86	5.80	-0.06
WCD4_5	6.36	N/A	6.36	6.04	-0.32
WAT02	5.39	N/A	5.39	5.39	0.00

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.6.2 below. Based on the model results, Alternative 1 has a minimal impact on the estimated flooding duration within the study are with a peak reduction of 6%. At the critical model nodes (Node IN_0002 and Node IN_0003), the average flood duration was reduced from 35.2 hours under the existing conditions to 34.3 hours under Alternative 1.

Table 5.6.2 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_0020	4.40	32.2	32.0	1
IN_0003	3.80	36.5	36.5	0
IN_0002	5.10	34.2	32.1	6

<u>Alternative 2: Exfiltration Trench with Positive Outfall to Canal</u>

Under Alternative 2, the proposed improvements defined under Alternative 1 were modified to include the installation of a new drainage connection between the proposed exfiltration system and the existing drainage canal between SW 30th Avenue and SW 31st Avenue. In addition to the 5,780 linear feet of exfiltration trench in Alternative 1, Alternative 2 includes an additional 200 linear feet of 24-inch pipe to

connect to the drainage canal. The estimated design and construction costs for this exfiltration trench alternative are $2,547,000.

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 2 is summarized within Table 5.6.3 below. According to the model results, Alternative 2 has a comparable reduction in peak flood stages as Alternative 1, which is minimal. At the critical model nodes (Node IN_0002 and Node IN_0003), the average peak flood depth was reduced from 1.11 feet under the existing conditions to 1.08 feet under Alternative 2.

Table 5.6.3 – Alternative 2 Peak Stage Summary

Nodes	Existing Conditions			Alternative 2	
	Peak Stage (feet)	Ground Elevation (feet)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_0003	5.49	3.8	1.69	5.45	-0.04
IN_0002	5.62	5.1	0.52	5.60	-0.02
IN_0020	6.50	4.4	2.1	6.51	+0.01
IN_0008	5.43	4.7	0.73	5.39	-0.04
WCD4_1	5.64	N/A	5.64	5.58	-0.06
WCD4_2	6.01	N/A	6.01	5.99	-0.02
WCD4_3	5.80	N/A	5.8	5.67	-0.13
WCD4_4	5.86	N/A	5.86	5.80	-0.06
WCD4_5	6.36	N/A	6.36	6.04	-0.32
WAT02	5.39	N/A	5.39	5.37	-0.02

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.6.4 below. According to the model results, Alternative 2 provides a slightly better reduction in flood duration than Alternative 1 by approximately 30%. At the critical model nodes (Node IN_0002 and Node IN_0003), the average flood duration was reduced from 35.2 hours under the existing conditions to 22.3 hours under Alternative 2.

Table 5.6.4 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_0020	4.40	32.2	31.6	2
IN_0003	3.80	36.5	22.1	39
IN_0002	5.10	34.2	22.5	34

<u>Alternative 3: Exfiltration Trench – with Positive Outfall to Lake</u>

Under Alternative 3, the proposed improvements defined under Alternative 1 were modified to include the installation of a new drainage connection between the proposed exfiltration system and the existing stormwater pond. In addition to the proposed 5,780 linear feet of exfiltration trench under Alternative 1, Alternative 3 includes an additional 400 linear feet of 24-inch drainage pipe to connect with the stormwater pond. The estimated design and construction costs for this exfiltration trench alternative are approximately $2,423,000. Based on our analysis with the stormwater model, the reduction in peak flood

stage under Alternative 3 is summarized within Table 5.6.5 below. Based on the model results on reducing peak flood stages, Alternative 3 has comparable results as Alternative 1 but is less effective than Alternative 2. At the critical model nodes (Node IN_0002 and Node IN_0003), the average peak flood depth was reduced from 1.11 feet under the existing conditions to 1.08 feet under Alternative 3.

Table 5.6.5 – Alternative 3 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 3	
	Peak Stage (feet)	Ground Elevation (feet)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_0003	5.49	3.8	1.69	5.45	-0.04
IN_0002	5.62	5.1	0.52	5.60	-0.02
IN_0020	6.50	4.4	2.1	6.51	+0.01
IN_0008	5.43	4.7	0.73	5.40	-0.03
WCD4_1	5.64	N/A	5.64	5.50	-0.14
WCD4_2	6.01	N/A	6.01	6.03	+0.02
WCD4_3	5.80	N/A	5.8	5.68	-0.12
WCD4_4	5.86	N/A	5.86	5.81	-0.05
WCD4_5	6.36	N/A	6.36	6.04	-0.32
WAT02	5.39	N/A	5.39	5.38	-0.01

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 3 is summarized within Table 5.6.6 below. Based on the model results on reducing flooding duration within the study area, Alternative 3 is better than Alternative 1 but less effective than Alternative 2. At the critical model nodes (Node IN_0002 and Node IN_0003), the average flood duration was reduced from 35.2 hours under the existing conditions to 27.6 hours under Alternative 3.

Table 5.6.6 – Alternative 3 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
IN_0020	4.40	32.2	32.0	1
IN_0003	3.80	36.5	30.3	17
IN_0002	5.10	34.2	24.8	27

Alternative Comparison

Refer to the Table 5.6.7 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.6.7 refer to the average of the critical problem areas of the study area, which correspond to Node IN_0002 and Node IN_0003 within the stormwater model. Based on our analysis with the stormwater model, all system improvement alternatives provide minimal reductions in the peak flood stages and variable reductions in the expected flood duration within the study area. Alternative 2 should be implemented for this study area since it provides the highest reduction in expected flood duration amongst the system improvement alternatives. Although Alternative 2 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 2 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.6.7 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.01	0.9	$2,486,000
Alternative 2	0.03	12.9	$2,547,000
Alternative 3	0.03	7.6	$2,423,000

CMA recommends the installation of exfiltration trench within the study area which will provide additional storage and infiltration capacity for stormwater runoff. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along Gateway Drive, SW 27th Avenue, SW 29th Avenue, and SW 30th Avenue to collect stormwater runoff from these areas. The proposed improvements will be interconnected with the existing canal via a new control structure to allow for an overflow connection. As feasible, any grass swale areas along these roadways should also be regraded to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 2 will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

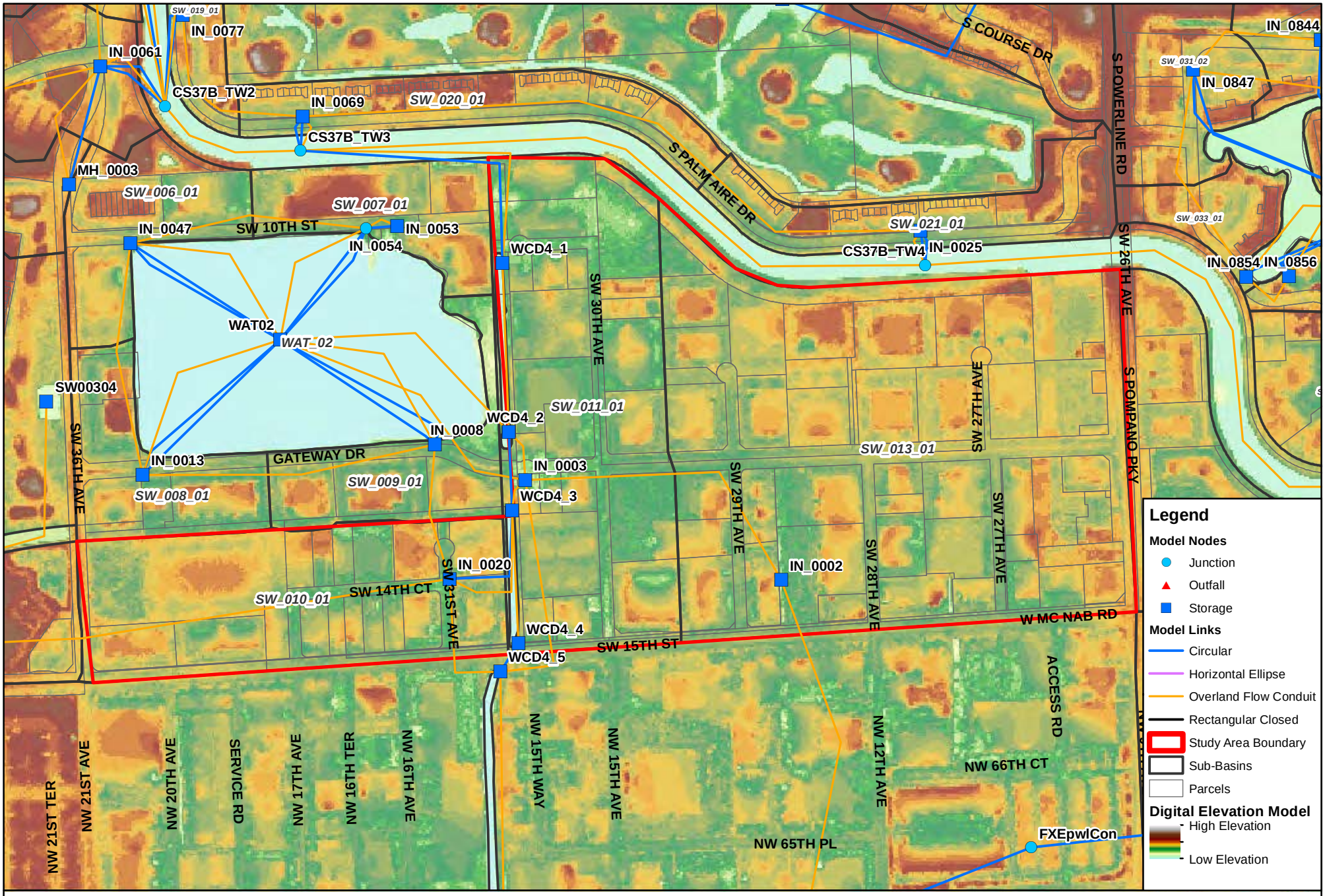


Figure 5-6A
Gateway Drive
Existing Topography & Model Schematics

0 250 500 Feet

N W E S

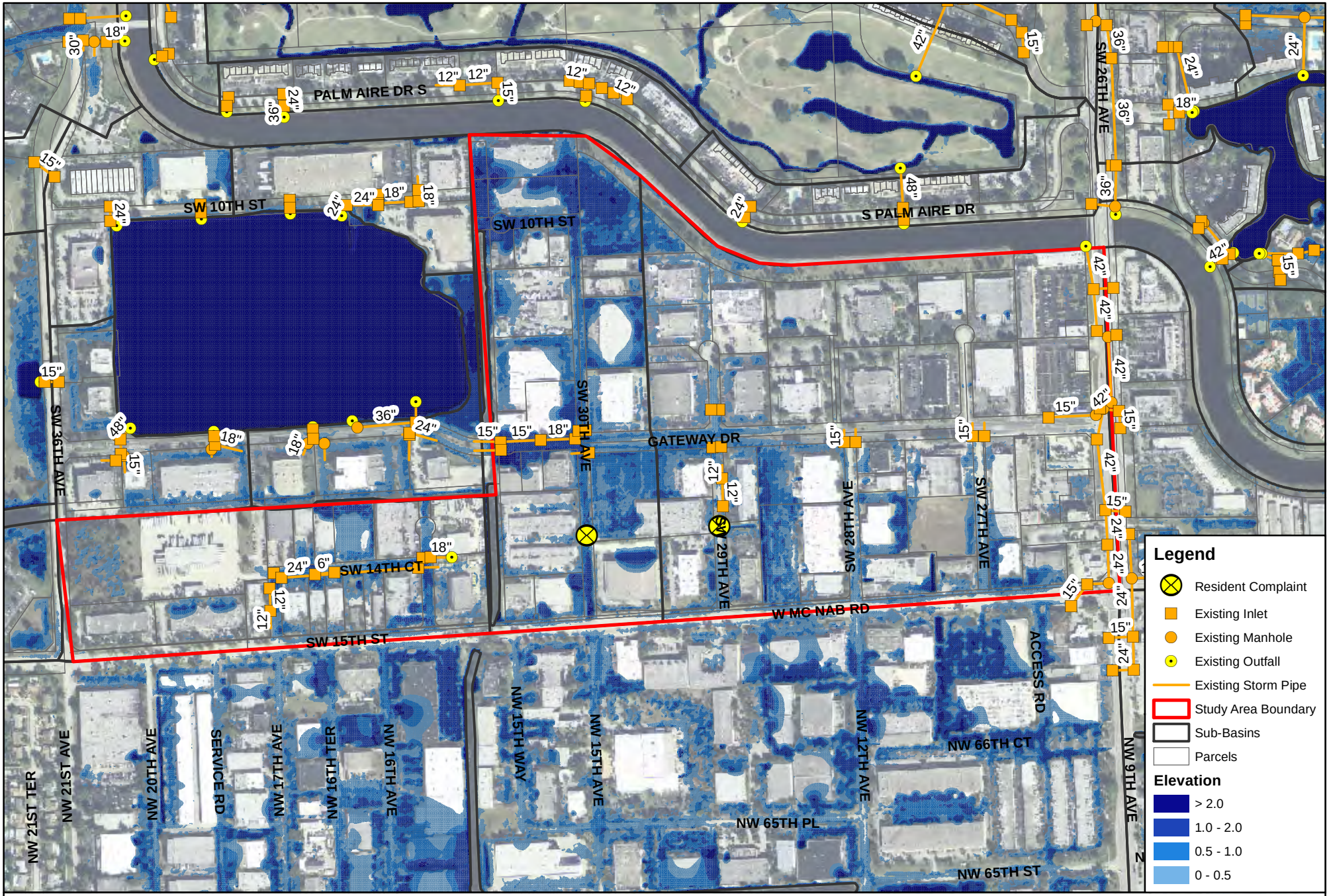


Figure 5-6B
Gateway Drive
Potential Flooding Depth

5.2.7 STUDY AREA 7 – KENDALL LAKE NEIGHBORHOOD

The Kendall Lake Neighborhood is a residential neighborhood bounded by NW 21st Street on the north, by NW 16th Street on the south, NW 5th Way on the west and NW 1st Avenue on the east. The study area consists of all single family developments, which are completely built out. The existing stormwater system is composed of two independent systems. The existing stormwater system in the northeast portion of the study area is a closed exfiltration trench system in the low lying areas. The existing stormwater system in the western portion of the study area includes a drainage pipe network which discharges via three outfalls into Kendall Lake, which does not have an overflow connection and has been observed with a very high water level.

The existing conditions stormwater model was used to evaluate the performance of the existing stormwater system in the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basins CE_007_01, CE_072_01, CE_072_02, CE_087_01, WAT_07 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-7A at the end of this section. According to the topography, the eastern portion of the study area is at a much higher elevation. There is an existing wall separating the east and west sides of this study area, between NW 2nd Terrace and NW 3rd Avenue with an opening in the wall along NW 18th Street. This condition can lead to a concentrated overland flow of stormwater runoff from east to west at this wall opening. According to the stormwater model, a large amount of stormwater runoff is expected to flow into the west side of the study area, which eventually drains into Kendall Lake. Since this lake does not have a control structure for an overflow connection to another drainage system, there could be the danger of overtopping during a heavy storm event. The proposed improvements should focus on reducing stormwater runoff flowing into entering Kendall Lake.

Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-7B at the end of this section. Some public roadways within the study area, such as NW 2nd Terrace and NW 17th Court, can be expected to flood to a depth of greater than 2 inches. Due to the relatively high ground surface elevation of the study area, the system improvement alternative which was investigated for this area consists of expanding the exfiltration trench system. Upsizing the outfall pipes or installing a pump station within the study area was not considered as a feasible alternative since Kendall Lake is already near capacity. The installation of drainage wells is also not allowed at this study area since it is located west of the brackish aquifer boundary.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area not currently served by the existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 4,872 LF of exfiltration trench throughout the western portion of the study area, due to the higher ground elevations. The installation of additional exfiltration systems within the west side of the study area will intercept a portion of the stormwater runoff flowing to the east side of the study area and reduce the flow into Kendall Lake. The general location of the proposed exfiltration trench is summarized by sub-basin within Table 5.7.1 below. The estimated design and construction costs for this exfiltration trench alternative are $1,489,000.

Table 5.7.1 – Alternative 1 Proposed Exfiltration Trench Summary		
Sub-Basin	**Exfiltration Trench (LF)**	**Mean Ground Surface Elevation (feet NAVD)**
CE_072_02	1,700	+14.0
CE_072_04	3,172	+14.7
Total	**4,872**	**+14.35**

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.7.2 below. According to the stormwater model, Alternative 1 results in a reduction in peak flood stages of -0.33 feet. At the critical model node for this study area (Node IN_1592), the peak flood depth is reduced from 1.66 feet under the existing conditions to 1.33 feet under Alternative 1.

Table 5.7.2 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			**Alternative 1**	
Nodes	**Peak Stage (feet, NAVD)**	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet, NAVD)**	**Peak Stage Reduction (feet)**
IN_1599	14.50	14.1	0.4	14.17	-0.33
IN_1592	14.46	12.8	1.66	14.13	-0.33
IN_1578	11.54	10.9	0.64	8.76	-2.78

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.7.3 below. The expected reduction in flood duration under Alternative 1 is significant throughout the study area. At the critical model node for this study area (Node IN_1592), the expected flood duration is reduced from 12.2 hours under the existing conditions to 2.7 hours under Alternative 1. Although the estimated reduction in peak flood stages is not significant under Alternative 1, the reduction in estimated flood duration provides a significant benefit to the study area.

Table 5.7.3 – Alternative 1 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 1**	**Reduction (%)**
IN_1599	14.10	1.6	0	100
IN_1592	12.80	12.2	2.7	78
IN_1578	10.90	28.5	0	100

Alternative 2: Exfiltration Trench and Expanded Lake

The purpose of this system improvement alternative is to provide additional storage in the adjacent existing lake. There is a vacant property owned by the City of Pompano Beach located to the west of the existing lake with an area of approximately 20 acres. This alternative utilizes a portion of this vacant property to excavate a lake that will be interconnected to the existing lake and will be used to reduce the peak stages in the existing lake. The lake expansion alternative will be combined with the proposed exfiltration trench descried in Alternative 1. The estimated design and construction costs for this exfiltration trench and lake expansion alternative are $2,720,000.

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 2 is summarized within Table 5.7.4 below. According to the stormwater model, Alternative 2 results in a reduction in peak flood stages of -0.33 feet. At the critical model node for this study area (Node IN_1592), the peak flood depth is reduced from 1.66 feet under the existing conditions to 1.33 feet under Alternative 2. These peak flood stage results are equivalent to the results from Alternative 1.

Table 5.7.4 – Alternative 2 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 2	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_1599	14.50	14.1	0.4	14.17	-0.33
IN_1592	14.46	12.8	1.66	14.13	-0.33
IN_1578	11.54	10.9	0.64	8.76	-2.78

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.7.5 below. At the critical model node for this study area (Node IN_1592), the expected flood duration is reduced from 12.2 hours under the existing conditions to 2.6 hours under Alternative 2. The estimated reduction under Alternative 2 is relatively insignificant when compared to the exfiltration trench under Alternative 1.

Table 5.7.5 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_1599	14.10	1.6	0	100
IN_1592	12.80	12.2	2.6	79
IN_1578	10.90	28.5	0	100

Alternative Comparison

Refer to the Table 5.7.6 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.7.6 refers to the critical problem node of the study area, Node IN_1592 within the stormwater model. Based on our analysis with the stormwater model, both system improvement alternatives provide similar limited flood control benefits for reducing the peak flood stages and reducing the expected flood duration within the study area. Although neither Alternative provides enough additional flood protection to meet the level of service criteria for public roadways within this study area, it does provide some additional benefits which alleviate the flooding problems within the study area, especially within the problem areas along NW 18th

Street and NW 2nd Terrace. The goal of the alternative is alleviate the overflow possibility of Kendall Lake, which has been observed to reach capacity during heavy rainfall events. Under both alternatives, a portion of the stormwater runoff generated within the study area will be diverted into the proposed exfiltration trench and thereby reduce the total flow into Kendall Lake. The expansion of the existing lake will further reduce the possibility of Kendall Lake overtopping during a heavy storm event, therefore Alternative 2 should be implemented in this study area.

Table 5.7.6 – Alternative Comparison

Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.33	9.5	$1,489,000
Alternative 2	0.33	9.6	$2,720,000

The recommended stormwater improvements for this study area include the installation of new exfiltration trench along City roadways throughout the study area to collect stormwater runoff from these areas and the excavation of a new expanded lake to the west of the existing Kendall Lake, which is connected via a new control structure. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 2 will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

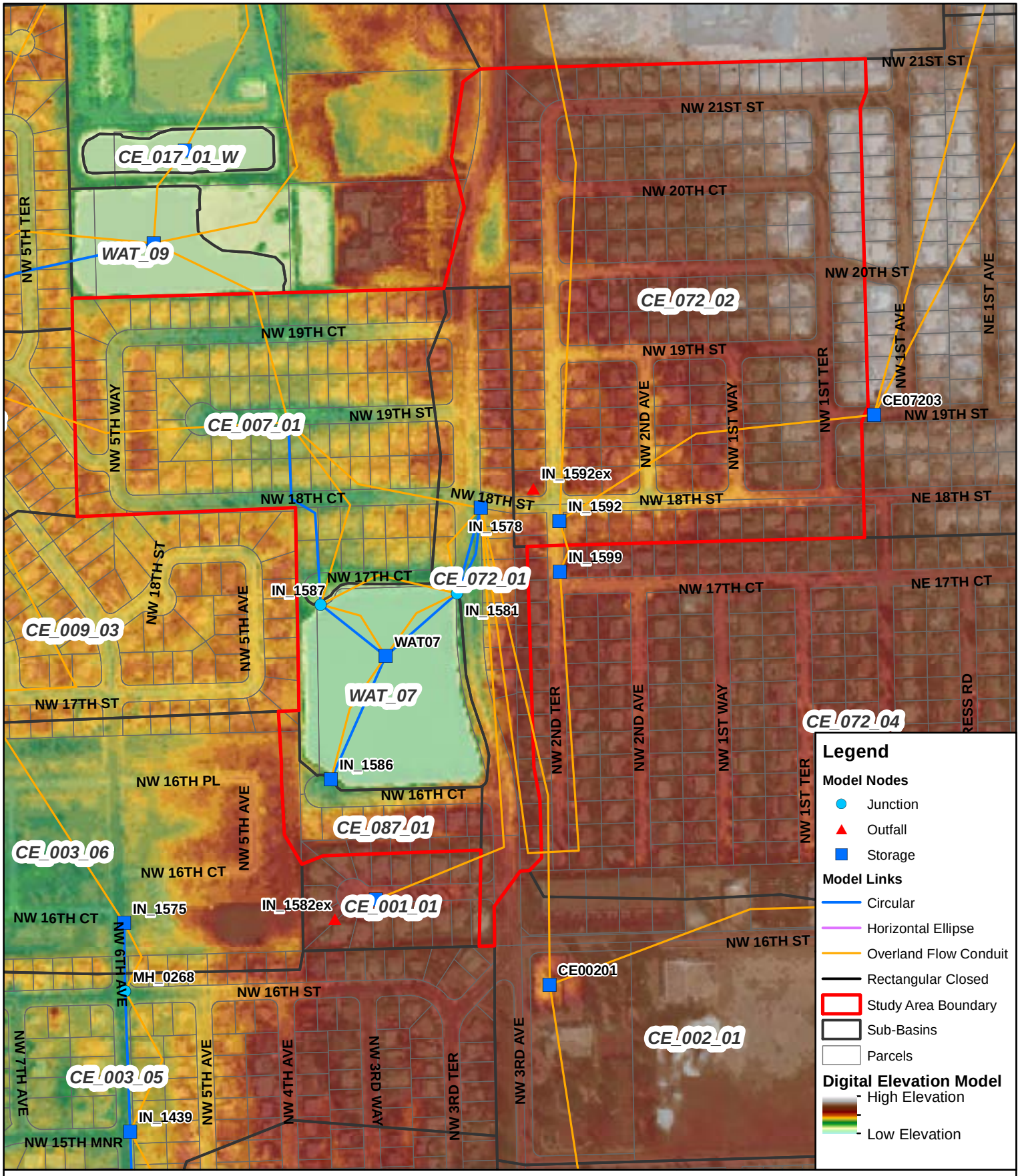


Figure 5-7A
Kendall Lake Neighborhood
Existing Topography & Model Schematics

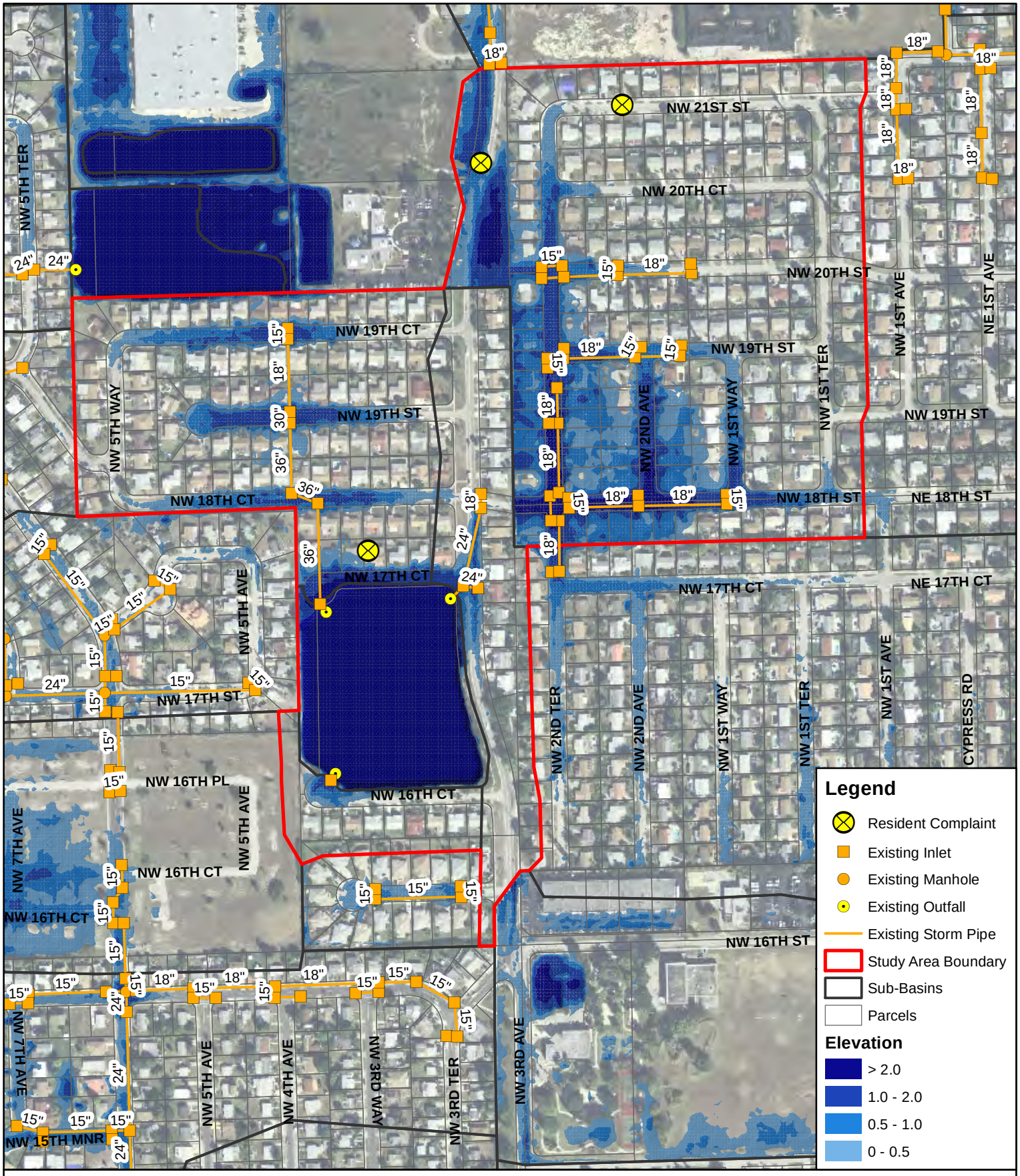


Figure 5-7B
Kendall Lake Neighborhood
Potential Flooding Depth

5.2.8 STUDY AREA 8 – US-1 AND NE 14TH STREET CAUSEWAY AREA

This study area is generally located southeast of the intersection of US Highway 1 and NE 14th Street Causeway. This study area consists chiefly of residential properties along with commercial properties located along US-1 and NE 14th Street. The existing drainage system within the study area includes a few separate systems, such as the FDOT drainage system along US-1 and NE 14th Street Causeway and various independent City systems within the neighborhood. These independent City drainage systems are located in the east side of the study area that discharges via existing outfall pipes into the tidally influenced canal system, which is directly connected to the Intracoastal Waterway. One 15-inch outfall is located towards the east end of the study area along NE 27th Terrace. Another 24-inch is located on the southeast of the study area along NE 12th Street.

The existing conditions stormwater model was used to evaluate the performance of the existing stormwater system during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The topography of the study area along with the model schematics are displayed on Figure 5-8A at the end of this section. According to the topography, NE 26th Avenue is the lowest elevation in the study area, causing any stormwater runoff to travel from the north or south onto this street. The study area is defined by the sub-basins CE_040_03, CE_042_02 and CE_044_01 within the stormwater model. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-8B at the end of this section. A significant portion of the roadways within this study area display flooding problems. The worst flooding problems appear to be located along NE 26th Avenue and NE 23rd Avenue. Several system improvement alternatives were evaluated for this study area, which include pipe size upgrades, pump station and drainage wells to address the existing flooding problems. The installation of exfiltration trench is not a feasible option for this study area due to the low ground surface elevation and was not included in the analysis of potential alternatives.

Alternative 1: Pipe Size Upgrades (Option 1)

The stormwater model was used to conduct several simulations of potential pipe size upgrades to the existing stormwater system. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The estimated design and construction costs for this pipe size upgrades alternative are approximately $1,440,000. The proposed construction under Alternative 1 consisted of the following upgrades:

- Install a new 18-inch pipe to interconnect Node IN_2978 to Node MH_0471.
- Replace an existing 12-inch pipe with an 18-inch pipe from Node MH_0471 to east to outfall.

Based on our analysis with the stormwater model, the estimated reduction in peak flood stages under Alternative 1 is summarized in Table 5.8.1 below. The estimated reduction of peak flood stages in the remainder of the study area was minimal under Alternative 1. At the critical model nodes within the study area (Node IN_2978, Node MH_0471, Node IN_2961, and Node IN_2834), the average peak flood depth was reduced from 1.34 feet under the existing conditions to 1.29 feet under Alternative 1.

Table 5.8.1 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2978	6.51	4.65	1.86	6.38	-0.13
IN_2981	6.28	6.30	0.00	6.16	-0.12
IN_2971	4.17	2.60	1.57	4.17	0.00
IN_2966	2.40	3.50	0.00	2.40	0.00
MH_0471	4.30	3.40	0.90	4.31	+0.01
IN_2961	4.35	3.10	1.25	4.32	-0.03
IN_2834	4.35	3.50	0.85	4.32	-0.03

Based on our analysis with the stormwater model, the estimated reduction in flooding duration under Alternative 1 is summarized in Table 5.8.2 below. According to the model result, the estimated reduction in flooding duration is significant throughout the problem area. Within the problem area at Node MH_0471, the flood duration is estimated to be reduced by 55% from 14.4 hours under the existing conditions to 6.5 hours under Alternative 1.

Table 5.8.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_2978	4.65	6.7	4.7	30
MH_0471	3.40	14.4	6.5	55
IN_2961	3.10	3.6	2.9	19
IN_2834	3.50	3.9	3.9	0

Alternative 2: Pipe Size Upgrades (Option 2)

The stormwater model was used to conduct several simulations of potential pipe size upgrades to the existing stormwater system. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The estimated design and construction costs for this pipe size upgrades alternative are approximately $1,626,000. The proposed construction under Alternative 2 consisted of the following upgrades:

- Install a new 18-inch pipe to interconnect Node IN_2978 to Node MH_0471.
- Replace an existing 24-inch pipe with a 36-inch pipe between Node IN_2971 and Node IN_2981.
- Replace an existing 12-inch pipe with a 15-inch pipe from Node MH_0471 to east.

Based on our analysis with the stormwater model, the estimated reduction in peak flood stages under Alternative 2 is summarized in Table 5.8.3 below. According to the model results, significant reduction in peak flood stage was predicted in the western portion of the study area along NE 14th Street, with a 1.03 foot reduction. The estimated reduction of peak flood stages in the remainder of the study area was minimal under Alternative 2. At the critical model nodes within the study area (Node IN_2978, Node

MH_0471, Node IN_2961, and Node IN_2834), the average peak flood depth was reduced from 1.34 feet under the existing conditions to 1.22 feet under Alternative 2.

Table 5.8.3 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2978	6.51	4.65	1.86	6.19	-0.32
IN_2981	6.28	6.30	0.00	5.25	-1.03
IN_2971	4.17	2.60	1.57	4.32	+0.15
IN_2966	2.40	3.50	0.00	2.45	+0.05
MH_0471	4.30	3.40	0.90	4.32	+0.02
IN_2961	4.35	3.10	1.25	4.34	-0.01
IN_2834	4.35	3.50	0.85	4.34	-0.01

Based on our analysis with the stormwater model, the estimated reduction in flooding duration under Alternative 2 is summarized in Table 5.8.4 below. According to the model result, the estimated reduction in flooding duration is significant at the problem area but shows an increase in flooding duration in other sections of the study areas. Within the problem area at Node MH_0471, the flood duration is estimated to be reduced by 66% from 14.4 hours under the existing conditions to 5.0 hours under Alternative 2.

Table 5.8.4 – Alternative 2 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_2978	4.65	6.7	2.7	59
MH_0471	3.40	14.4	5.0	66
IN_2961	3.10	3.6	4.0	-11
IN_2834	3.50	3.9	4.2	-8

Alternative 3: Pipe Size Upgrades (Option 3)

Alterative 3 includes all proposed improvements defined for Alternative #2 along with additional pipe installation. In addition to the pipe upgrades, the proposed construction will include the installation of additional drainage pipes that will interconnect the problem areas to the existing outfalls in the southern portion of the study area. Alternative 3 will provide additional conveyance capacity to transmit stormwater runoff from the problem area to the existing outfalls on the south side of the study area. Alternative 3 includes the installation of a new 18-inch pipe from Node IN_2978 to Node IN_2834. The estimated design and construction costs for this pipe size upgrades alternative are approximately $2,193,000.

Based on our analysis with the stormwater model, the estimated reduction in peak flood stage under Alternative 3 is summarized in Table 5.8.5 below. According to the model results, the reduction in peak flood stages under Alternative 3 is very similar to Alternative 2. Since the installation of the additional 18-inch pipe under Alternative 3 does not create any additional benefit compared to Alternative 2, it is not a feasible option for implementation. At the critical model nodes within the study area (Node IN_2978,

Node MH_0471, Node IN_2961, and Node IN_2834), the average peak flood depth was reduced from 1.22 feet under the existing conditions to 1.14 feet under Alternative 3.

Table 5.8.5 – Alternative 3 Peak Flood Stage Summary					
	Existing Conditions			Alternative 3	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2978	6.51	4.65	1.86	6.21	-0.30
IN_2981	6.28	6.30	0.00	5.26	-1.02
IN_2971	4.17	2.60	1.57	4.31	+0.14
IN_2966	2.40	3.50	0.00	2.44	+0.04
MH_0471	4.30	3.40	0.90	4.31	+0.01
IN_2961	4.35	3.10	1.25	4.36	+0.01
IN_2834	4.35	3.50	0.85	4.36	+0.01

Based on our analysis with the stormwater model, the estimated reduction in flooding duration under Alternative 3 is summarized in Table 5.8.6 below. According to the model results, Alternative 3 significantly reduces the flooding duration, with a maximum reduction of 56% from the existing conditions at the problem area. At the critical model nodes within the study area (Node IN_2978, Node MH_0471, Node IN_2961, and Node IN_2834), the average flood duration was reduced from 7.2 hours under the existing conditions to 6.3 hours under Alternative 3.

Table 5.8.6 – Alternative 3 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
IN_2978	4.65	6.7	3.0	56
MH_0471	3.40	14.4	14.4	0
IN_2961	3.10	3.6	3.8	-6
IN_2834	3.50	3.9	4.0	-3

Alternative 4: Pump Station

The stormwater model was used to conduct several simulations of a potential pump station within the study area. The purpose of this system improvement alternative is to increase the conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 4 consists of the installation of a pump station adjacent to the existing outfall at Node: IN_2834. The proposed construction also includes new pipe upgrades to adequately convey stormwater runoff to the pump station. The estimated design and construction costs for this pump station alternative are approximately $2,199,000. The components associated to the pump station are listed below.

- Install a new 18-inch discharge pipe from pump station to outfall
- Install flap gate for backflow prevention at the point of discharge
- Wet well with a total footprint of about 150 square feet and depth of 8 feet
- Maximum pump capacity of 33.5 CFS, which is equivalent to the peak discharge of the existing drainage system during low tide conditions

* Install a new 18-inch pipe from Node IN_2978 to Node MH_0471
* Install a new 24-inch pipe from Node MH_0473 to Node IN_2834

Based on our analysis with the stormwater model, the estimated reduction in peak flood stage under Alternative 4 is summarized in Table 5.8.7 below. According to the model results, Alternative 4 reduces the estimated peak flood stage at nodes along NE 14th Street but provide only minimal improvements to the remainder of the study area. Alternative 4 only provides benefits to a localized portion of the study area.

Table 5.8.7 – Alternative 4 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 4	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2978	6.51	4.65	1.86	6.19	-0.32
IN_2981	6.28	6.30	N/A	5.20	-1.08
IN_2971	4.17	2.60	1.57	4.27	+0.10
IN_2966	2.40	3.50	N/A	2.42	N/A
MH_0471	4.30	3.40	0.90	4.30	0.00
IN_2961	4.35	3.10	1.25	4.30	-0.05
IN_2834	4.35	3.50	0.85	4.30	-0.05

Based on our analysis with the stormwater model, the estimated reduction in flooding duration under Alternative 4 is summarized in Table 5.8.8 below. According to the model results, the estimated flood duration is significantly reduced by Alternative 4 in portions of the study area, with a maximum reduction of 68%. At the critical model nodes within the study area (Node IN_2978, Node MH_0471, Node IN_2961, and Node IN_2834), the average flood duration was reduced from 3.8 hours under the existing conditions to 6.3 hours under Alternative 4.

Table 5.8.8 – Alternative 4 Flood Duration Summary

Nodes	Reference Street Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 4	Reduction (%)
IN_2978	4.65	6.7	2.7	60
MH_0471	3.40	14.4	4.6	68
IN_2961	3.10	3.6	3.8	-5
IN_2834	3.50	3.9	3.9	1

Alternative 5: Drainage Wells

The stormwater model was used to conduct several simulations of the installation of proposed drainage wells within problem areas of the study area. The purpose of this system improvement alternative is to intercept stormwater runoff before it reaches the existing outfalls and to provide additional discharge capacity at the problem area to alleviate the existing flooding issues quicker. The expected construction includes a total of three drainage wells along NE 23rd Avenue within the study area. Within the stormwater model, a minimum driving head of 1.5 feet above the SHWT was assumed prior to activating discharge via the proposed drainage wells. Based on the historical information for the area, the discharge rate of each drainage well was assumed to be 450 GPM per foot of head within the stormwater model,

which is approximately equivalent to 1.0 CFS per foot of head. All proposed drainage wells will be interconnected to maintain the same driving head to each drainage well. The estimated design and construction costs for this drainage well alternative are approximately $633,000.

Based on our analysis with the stormwater model, the estimated reduction in peak flood stage under Alternative 5 is summarized in Table 5.8.9 below. According to the model results, Alternative 5 reduces the peak flood stage by a maximum of 0.28 feet within the study area. At the critical model nodes within the study area, Alternative 5 has minimal impact on the peak flood stages within these problem areas.

Table 5.8.9 – Alternative 5 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 5	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2978	6.51	4.65	1.86	6.48	-0.03
IN_2981	6.28	6.30	0.00	6.00	-0.28
IN_2971	4.17	2.60	1.57	4.16	-0.01
IN_2966	2.40	3.50	0.00	2.40	0.00
MH_0471	4.30	3.40	0.90	4.31	+0.01
IN_2961	4.35	3.10	1.25	4.35	0.00
IN_2834	4.35	3.50	0.85	4.35	0.00

Based on our analysis with the stormwater model, the estimated reduction in flooding duration under Alternative 5 is summarized in Table 5.8.10 below. Alternative 5 significantly reduces the flooding duration in the problem area at Node IN_2978 by 64%, but does not have any impact elsewhere within the study area.

Table 5.8.10 – Alternative 5 Flood Duration Reduction

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 5	Reduction (%)
IN_2978	4.65	6.7	2.4	64
MH_0471	3.40	14.4	14.4	0
IN_2961	3.10	3.6	3.6	0
IN_2834	3.50	3.9	3.9	0

Alternative Comparison

Refer to the Table 5.8.11 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.8.11 refers to the average of the critical problem areas of the study area, which correspond to Node IN_2978, Node MH_0471, Node IN_2961, and Node IN_2834 within the stormwater model. Based on our analysis with the stormwater model, all system improvement alternatives provide similar limited flood control benefits for reducing the peak flood stages and reducing the expected flood duration within the study area. Alternative 3 should likely be eliminated from consideration since it provides the least flood control benefits to the study area. Alternative 5 should likely be eliminated from consideration since the flood

control benefits effect only a limited portion of the study area. Alternative 2 should be implemented for this study area instead of Alternative 4 since it provides similar flood control benefits but is significantly more cost effective. Although Alternative 2 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 2 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.8.11 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.05	2.7	$1,440,000
Alternative 2	0.08	3.2	$1,626,000
Alternative 3	0.07	0.9	$2,193,000
Alternative 4	0.11	3.4	$2,199,000
Alternative 5	0.00	1.1	$633,000

The recommended stormwater improvements for this study area include the installation of new drainage along NE 13th Street to interconnect the existing outfalls in the area to the low lying portion of the study area along with the replacement of the existing 12-inch pipe with new 15-inch pipe to increase the transmission capacity to the existing outfall. Due to the low elevation of the study area, the proposed improvements also include the installation of a backflow prevention device at the 15-inch outfall to reduce the negative impact of tidal fluctuation. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff.

For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1, and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 2 will encounter various constructability concerns related to the replacement of outfall pipe within utility easements on private property and regulatory limitations on the peak discharge via the upsized outfall pipes. These items will need to be evaluated in more detail during the design phase of the proposed project.

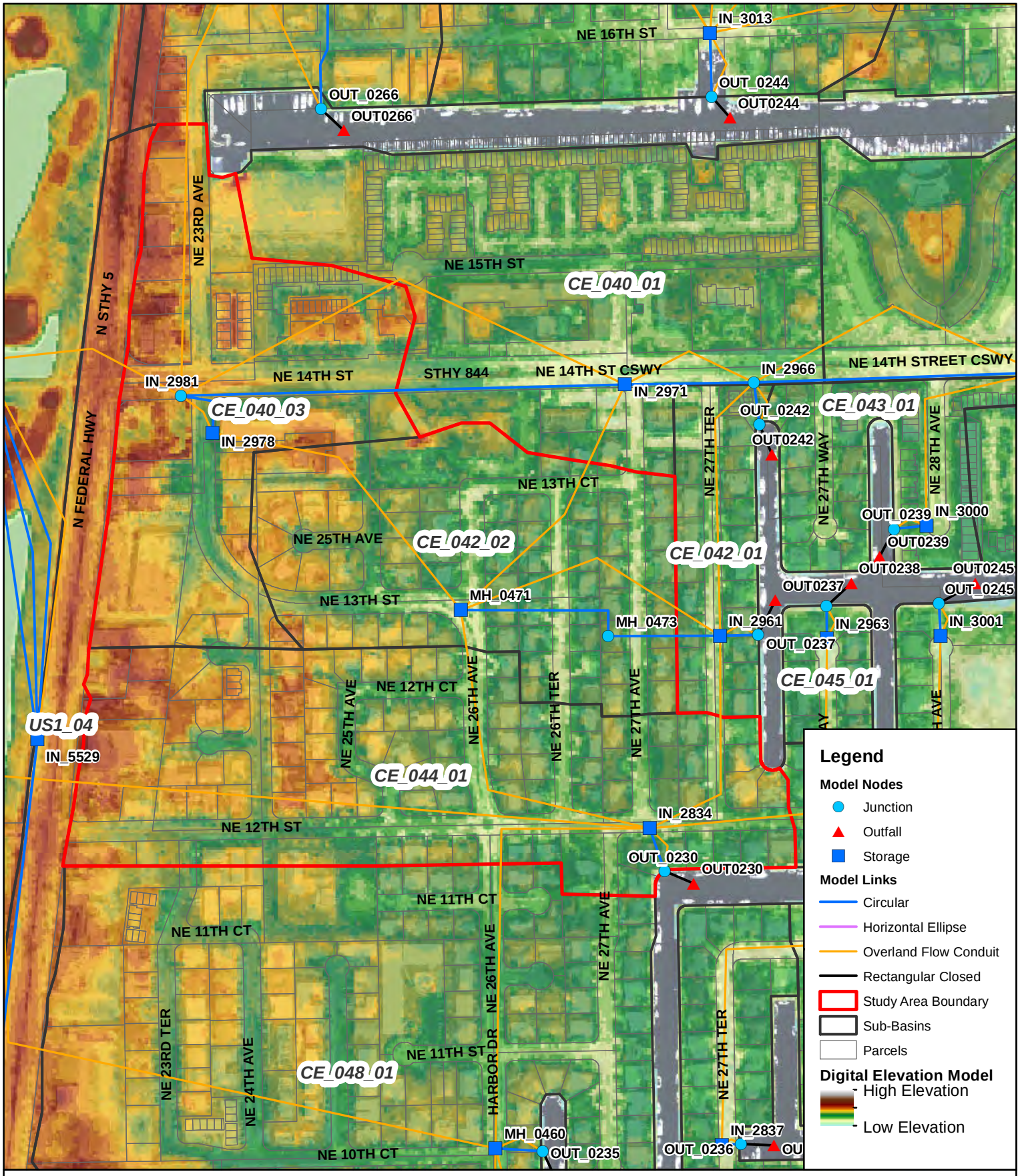


Figure 5-8A
US-1 & NE 14th Street Causeway
Existing Topography & Model Schematics

N
W E
S

0 250 500
Feet

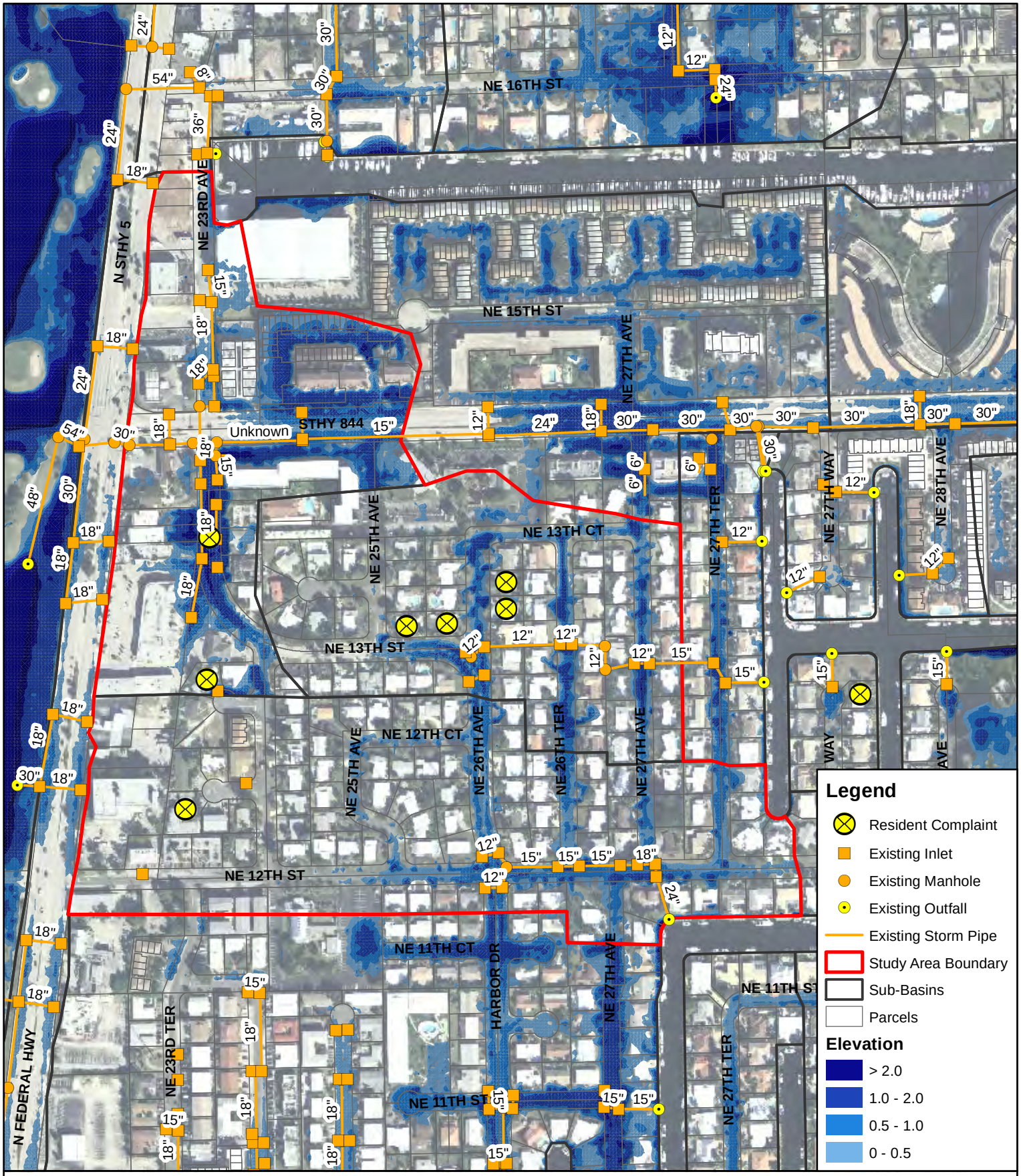


Figure 5-8B
US-1 & NE 14th Street Causeway
Potential Flooding Depth

N
W E
S

0 250 500
Feet

5.2.9 STUDY AREA 9 – NE 4TH STREET AND NE 3RD STREET

This study area includes NE 4th Street and NE 3rd Street to the east of Harbor Drive immediately adjacent to the Intracoastal Waterway. This residential neighborhood includes two separate areas surrounded by the finger canals off the Intracoastal Waterway. The public right-of-way areas within this neighborhood do not have an existing drainage system to address any flooding issues since these roadways are hydraulically isolated from adjacent areas with existing drainage infrastructure, such as Harbor Drive. During rainfall events, stormwater runoff from this neighborhood will collect in right-of-way areas where it can only slowly infiltrate into the ground surface from pervious swale areas adjacent to the roadway.

The existing conditions stormwater model was used to evaluate the potential flooding within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basins CE_058_01 and CE_061_03 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-9A at the end of this section. Based on the results of the existing conditions stormwater model, the extent of potential flooding within the study area is displayed on Figure 5-9B at the end of this section. Both NE 4th Street and NE 3rd Street experience significant flooding depth of greater than 2 inches at the eastern limits near the cul-de-sacs due to the low lying elevations. The stormwater model was used to evaluate effectiveness of various system improvement alternatives, such as exfiltration trench or pump station, in reducing the existing flooding problems. The interconnection of the study area to the existing stormwater system along Harbor Drive was not a feasible option since it actually could worsen the flooding problem when stormwater runoff flows into the study area via the interconnection from higher areas located to the west. The system improvement alternatives which were evaluated with the stormwater model are summarized below.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area not currently served by the existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 5,882 LF of exfiltration trench, which were aligned along right of way areas with ground surface elevations greater than +5.0 feet NAVD. The general location of the proposed exfiltration trench is summarized by sub-basin within Table 5.9.1 below. The proposed exfiltration trenches were aligned inside the study area along NE 3rd Street and NE 4th Street as well as outside the study area along NE 1st Street, NE 2nd Street, NE 23rd Avenue, NE 24th Avenue, NE 25th Avenue and NE 28th Avenue. The estimated design and construction costs for this exfiltration trench alternative are approximately $2,561,000.

Table 5.9.1 – Alternative 1 Proposed Exfiltration Trench Summary

Sub-Basin	Exfiltration Trench (LF)	Mean Ground Surface Elevation (feet NAVD)
CE_058_01	862	+3.75
CE_061_03	775	+4.5
CE_062_01	3,090	+7.0
CE_063_01	721	+4.5
CE_064_01	434	+4.0
Total	**5,882**	**+4.75**

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this

study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.9.2 below. The model results show a slight reduction in peak flood stages throughout the study area under Alternative 1.

Table 5.9.2 – Alternative 1 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 1	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
NE 4th Street					
IN_2904	6.93	6.34	0.59	6.93	0.00
IN_2898	5.44	6.75	N/A	5.41	N/A
IN_2896	5.40	5.30	0.10	5.37	-0.03
CE06103	5.32	3.70	1.62	4.88	-0.44
NE 3rd Street					
MH_0465	5.44	5.40	0.04	5.38	-0.06
CE05801	4.90	3.30	1.60	4.63	-0.27
NE 1st Street					
IN_3581	4.07	3.90	0.17	3.85	-0.22
IN_2935	4.06	4.00	0.06	3.86	-0.20

Based on our analysis with the stormwater model, the reduction of flooding duration within the study area under Alternative 1 is summarized in Table 5.9.3 below. The model results show that Alternative 1 effectively reduces the expected flood duration within the study area from the existing conditions, especially within certain nodes where the flooding has been eliminated.

Table 5.9.3 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_2896	5.30	0.5	0.4	26
CE06103	3.70	45	18.9	58
MH_0465	5.40	0.4	0.0	100
CE05801	3.30	45	37.0	18
IN_3581	3.90	0.4	0.0	100

Alternative 2: Pump Station

The stormwater model was used to conduct several simulations of various proposed pump station within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 2 includes the installation of one pump stations near the existing outfall at Harbor Drive and NE 2nd Street at model Node MH_0465. The estimated design and construction costs for this pump station alternative are approximately $1,089,000. The components associated to the pump station are listed below.

- Install a 18-inch discharge pipe from pump station to outfall location.
- Install a flap gate at the point of discharge for backflow prevention.
- Wet well with a total footprint of about 150 square feet and depth of 8 feet.
- Maximum pump capacity shall be 26.8 CFS, which is equivalent to peak discharge rate from existing drainage system during low tide conditions.

Based on our analysis with the stormwater model, the reduction in peak flood stages under Alternative 2 is summarized within Table 5.9.4 below. The model results do not show any change in peak flood stage under Alternative 2. There are no benefits in peak stage reduction from this pump station alternative.

Table 5.9.4 – Alternative 2 Peak Flood Stage Comparison

Nodes	Existing Conditions			Alternative 2	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
NE 4th Street					
IN_2904	6.93	6.34	0.59	6.93	0.00
IN_2898	5.44	6.75		5.44	0.00
IN_2896	5.40	5.30	0.10	5.40	0.00
CE06103	5.32	3.70	1.62	5.32	0.00
NE 3rd Street					
MH_0465	5.44	5.40	0.04	5.44	0.00
CE05801	4.90	3.30	1.60	4.90	0.00
NE 1st Street					
IN_3581	4.07	3.90	0.17	4.07	0.00
IN_2935	4.06	4.00	0.06	4.06	0.00

Based on our analysis with the stormwater model, the reduction in expected flooding duration is summarized within Table 5.9.5 below. The model results show a limited reduction in flood duration under Alternative 2. The estimated reduction under Alternative 2 is relatively insignificant when compared to the exfiltration trench under Alternative 1.

Table 5.9.5 – Alternative 2 Flood Duration Comparison				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 2**	**Reduction (%)**
IN_2896	5.30	0.5	0.5	0
CE06103	3.70	45	45.0	0
MH_0465	5.40	0.4	0.3	14
CE05801	3.30	45	40	11
IN_3581	3.90	0.4	0.4	5

Alternative Comparison

Refer to Table 5.9.6 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.9.6 refers to averages of the critical problem areas within the study area, which correspond to Node CE06103 and Node CE05801 within the stormwater model. Based on our analysis with the stormwater model, Alternative 1 can be considered to be the most effective option for reducing the peak flood stages and reducing the expected flood duration within the study area. Although Alternative 1 will not completely eliminate the flooding within the study area, it will reduce the duration of flooding within the low lying area. Additional local improvements which provide additional storage volume for stormwater runoff should be considered along NE 3rd Street and NE 4th Street, such as regraded swales or subsurface soil storage.

Table 5.9.6 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.36	17.1	$2,561,000
Alternative 2	0.0	2.5	$1,089,000

CMA recommends the installation of exfiltration trench within these right-of-way areas to provide additional infiltration capacity for stormwater runoff. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along NE 3rd Street and NE 4th Street to collect stormwater runoff from these areas. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, the proposed improvements will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

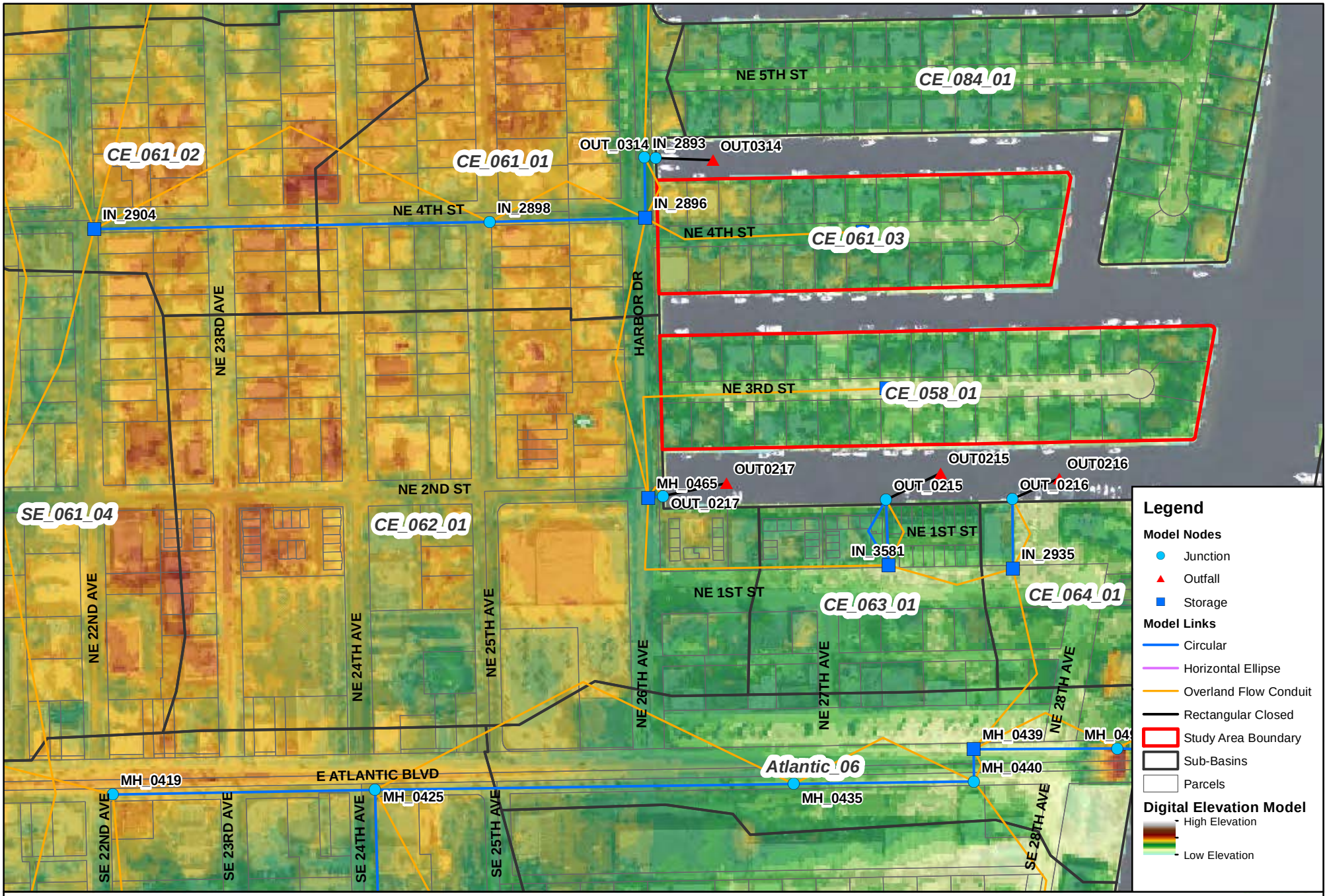


Figure 5-9A
NE 4th Street & NE 3rd Street
Existing Topography & Model Schematics

0 150 300 Feet

N S E W

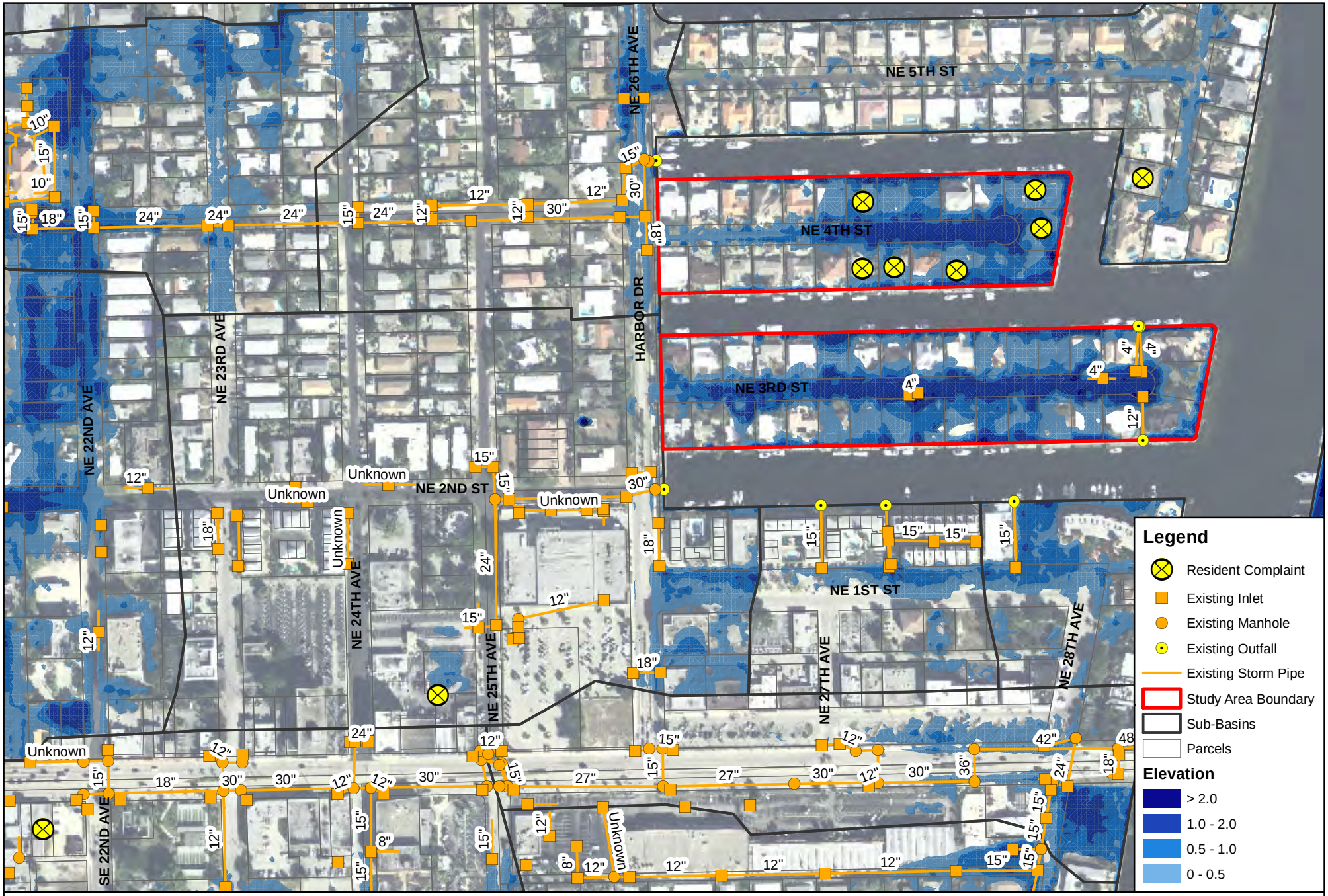


Figure 5-9B
NE 4th Street & NE 3rd Street
Potential Flooding Depth

0 150 300 Feet

N S E W

5.2.10 STUDY AREA 10 – DIXIE HIGHWAY AND WEST MCNAB ROAD

This study area is bounded by Interstate-95 on the west, by SW 9th Street on the north, by Dixie Highway on the east, and by West McNab Road on the south. This study area consists of mixture of residential and commercial properties. A portion of this study area consists of a large development project, which is currently under construction and bounded by SW 13th Court to the south and SW 10th Street to the north. This development project will implement on-site stormwater improvements, which will provide adequate flood control for the property. The remainder of this study area to the south of this development has existing City drainage facilities. There is also an existing FDOT drainage system, which only serves the right of way for Dixie Highway.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basins SE_097_01, SW_029_05, SW_029_06 and SW_029_07 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-10A at the end of this section. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-10B at the end of this section. Many roadways and properties throughout the study area display flooding greater than two inches.

The potential stormwater improvements were investigated within the south portion of the study area along SW 13th Court and SW 14th Court. After discussions with City staff, the existing stormwater system used to be discharge via an outfall into FDOT drainage system along east of Interstate 95. During regrading work within the FDOT right of way, the contractor had plugged the existing outfall pipe, which created additional flooding problems throughout the study area. Since there is extensive existing stormwater infrastructure along both SW 13th Court and SW 14th Court, a system improvement alternative for this study area would include the reconnection of the existing outfall pipe to FDOT system.

Alternative 1: Pipe Connections

The stormwater model was used to conduct a simulation of the installation of a proposed connection of the system within the study area to that of the Interstate 95 system. The purpose of this system improvement alternative is to analyze if a connection will alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a 36-inch Drainage Pipe Connection to the Interstate-95 System. The estimated design and construction costs for this pipe connection alternative are approximately $52,000.

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.10.1 below. According to the stormwater model, Alternative 1 results in a limited reduction in peak flood stages of -0.35 feet at the critical model node (Node SE09701), which corresponds to the problem areas within this study area. The peak flood depth at this location is expected to be reduced from 1.02 feet under the existing conditions to 0.67 feet under Alternative 1.

Table 5.10.1 – Alternative 1 Peak Flood Stage Summary

	Existing Conditions			Alternative 1	
Nodes	**Peak Stage (feet, NAVD)**	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet, NAVD)**	**Peak Stage Reduction (feet)**
SE09701	5.62	4.6	1.02	5.27	-0.35

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.10.2 below. The estimated reduction in flood duration under Alternative 1

is significantly better with a reduction of 88% at the critical model node (Node SE09701), which corresponds to the problem areas within this study area. The peak flood depth at this location is expected to be reduced from over 40 hours under the existing conditions to 4.9 hours feet under Alternative 1.

Table 5.10.2 – Alternative 1 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 1**	**Reduction (%)**
SE09701	4.6	> 40	4.9	88

Alternative Comparison

Alternative 1 provides a limited reduction in peak flood stage within the study area but a significant reduction in the flood duration within the study area. Although Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for public roadways within this study area, it does provide additional benefits which alleviate the flooding problems within the study area and should be implemented. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1, and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design of this potential project, the City will need to negotiate the reactivation of this existing outfall pipe with FDOT. Due to potential capacity limitations of the existing FDOT stormwater system along I-95, the peak discharge via the reconnected outfall pipe may need to be limited by a new control structure or attenuated by regraded swale areas throughout the contributory area.

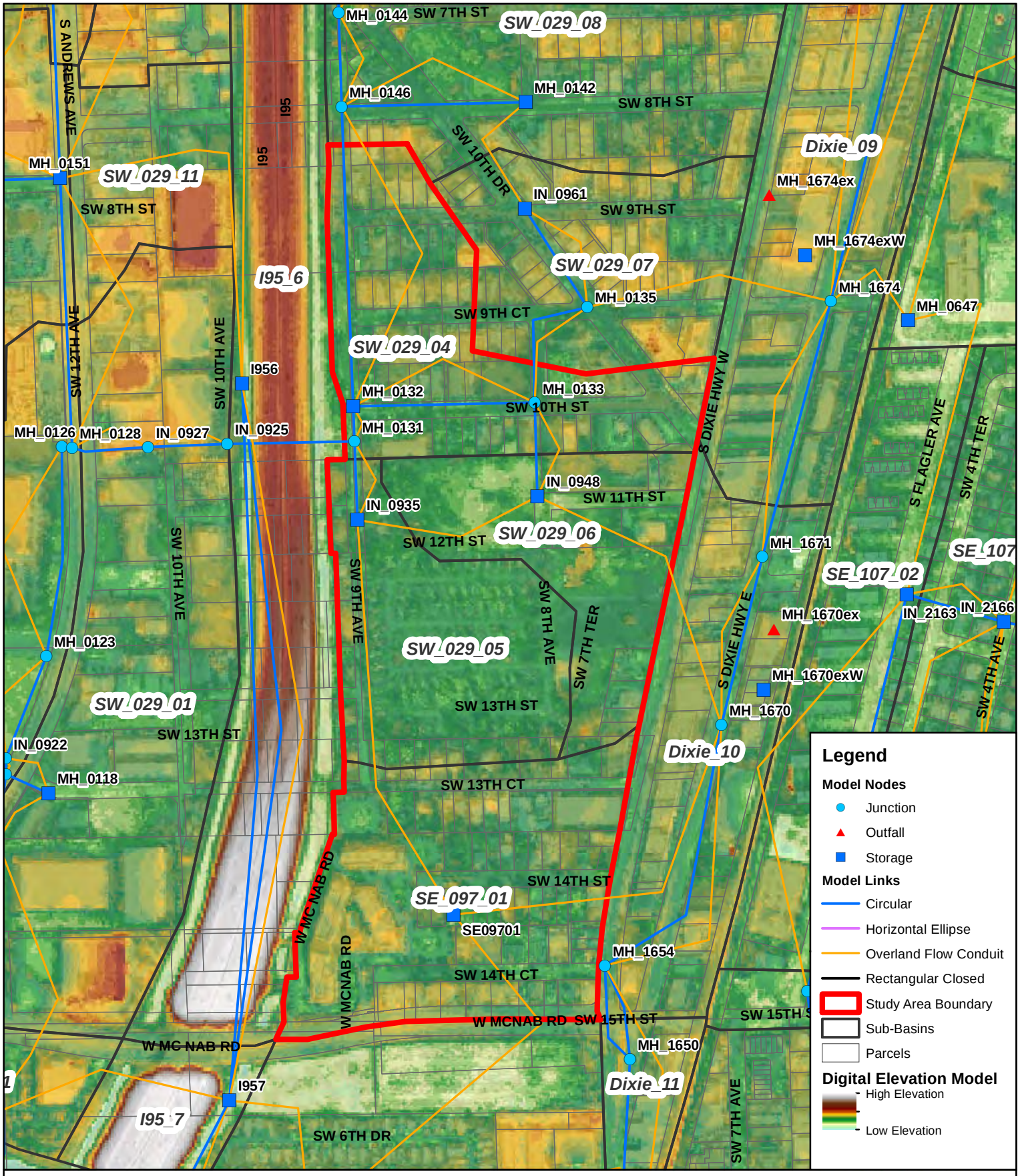


Figure 5-10A
Dixie Highway and McNab Road
Existing Topography & Model Schematics

N
W E
S

0 250 500
Feet

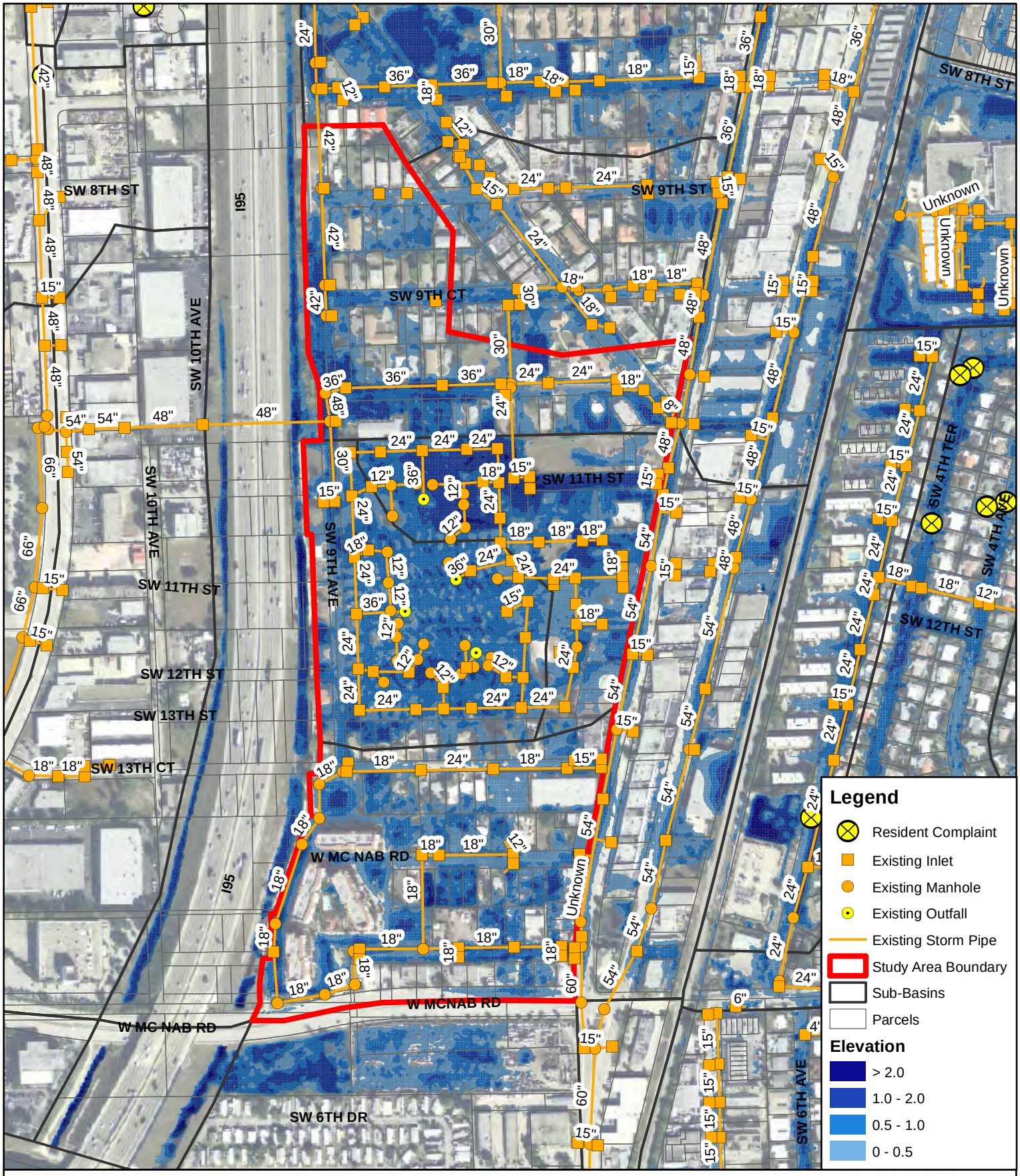


Figure 5-10B
Dixie Highway and McNab Road
Potential Flooding Depth

N
W E
S

0 250 500
Feet

5.2.11 STUDY AREA 11 – BAY DRIVE NEIGHBORHOOD

This study area consists of a residential neighborhood, which is bounded by Robbins Road to the south, by North Riverside Drive to the north, by A1A to the west and Bay Drive to the east. The existing stormwater system within the study area consists of the FDOT system along US A1A and a City system along Bay Drive with an existing outfall discharging directly to the Hillsboro Inlet. The City has received extensive complaints from residents in this area about flooding within the neighborhood roadways.

The existing conditions stormwater model was used to evaluate the potential flooding within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by sub-basins CE_066_01 and CE_068_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-11A at the end of this section. Based on the results of the existing conditions stormwater model, the extent of potential flooding within the study area is displayed on Figure 5-11B at the end of this section. According to the model results, Bay Drive and areas along Beacon Street, Dover Drive and Spring Street displays flooding with depths greater than 2.0 inches. The high flood depths are expected in these areas as the elevations range from 2.0 to 3.0 NAVD.

A system improvement alternative was not modeled for this study area since the typical ground surface elevations are too low to be able to implement any major improvements, such as drainage wells or exfiltration trench, and there is not adequate access to a positive outfall location. Due to the very low ground surface elevations within the study areas, a system improvement alternative to be considered for this study areas would be an extension of the existing stormwater system to ensure the drainage inlets are located within the low lying right-of-way area not currently served by the existing system.

Alternative 1: Pipe Connections

Alternative 1 includes the installation of new drainage pipe and catch basin inlets within low lying portions of Bay Drive, Beacon Street, Dover Road, Spring Street, Leigh Road, and Barton Road. Under Alternative 1, the proposed construction includes a total of 3,460 LF of 15-inch and 18-inch RCP pipe, which will be interconnected with the existing drainage system. The purpose of this system improvement alternative is to alleviate the existing flooding issues at the low elevations areas which are not served by the existing drainage system. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,210,000.

Alternative Comparison

Although Alternative 1 does not provide any reduction in peak flood stage or flood duration within the study area, it will help alleviate nuisance flooding within isolated low lying right-of-way areas that are not currently served by existing drainage infrastructure. Although Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for the study area, it does help alleviate the nuisance flooding within the right-of-way area. CMA recommends the extension of the existing drainage system into right-of-way areas without existing catch basin inlets, which will help draw down the flooding in these areas. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

Figure 5-11A
Bay Drive Neighborhood
Existing Topography & Model Schematics

N
W E
S

0 250 500
Feet

Figure 5-11B
Bay Drive Neighborhood
Potential Flooding Depth

N
W E
S

0 250 500
Feet

5.2.12 STUDY AREA 12 – NORTH RIVERSIDE DRIVE AND NE 14TH STREET CAUSEWAY

This study area is primarily located along North Riverside Drive between NE 14th Street Causeway and NE 8th Street. This neighborhood is a mixture of single family homes, multi-family residential complex and commercial properties. The existing stormwater system within the study area consists of the FDOT system along US A1A and a City system along North Riverside Drive with three existing outfalls discharging directly to the Intracoastal Waterway. The topography of the study area along with the model schematics are displayed on Figure 5-12A after this section. The ground surface elevation along the centerline of North Riverside Drive is as low as 1.3 feet NAVD at some locations. Due to the very low elevation of the study area, the flooding problems within the study area are directly influenced by the tidal fluctuations within the Intracoastal Waterway.

The existing conditions stormwater model was used to simulate the performance of the existing stormwater management system in the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basins CE_041_01, CE_073_01 and CE_075_01 within the stormwater model. Based on topography, these sub-basins receive a significant amount of stormwater runoff from the areas east of US A1A within sub-basins CE_041_02, CE_074_01, and CE_077_01. Based on our analysis with the stormwater model, North Riverside Drive experiences significant flooding of greater than 2 inches throughout the entire length of the study area. Based on the results of the stormwater model, the extent of the estimated flooding within the study area is displayed Figure 5-12B at the end of this section. The system improvement alternatives investigated within this study area include pipe size upgrades and pump stations. Exfiltration trench was not considered as a potential system improvement alternative for this study area due to the very low ground surface elevation which would eliminate the effectiveness of either option.

Alternative 1: Pipe Size Upgrades

The stormwater model was used to conduct several simulations of various proposed pipe size upgrades at specific locations within the existing stormwater system. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. Alternative 1 included the replacement of the existing City outfall pipes which discharge into the Intracoastal Waterway with a larger diameter pipe. Under Alternative 1, the existing 18-inch pipe will be replaced with a 36-inch pipe at NE 12th Street which includes a total pipe replacement of 200 linear feet. The two existing 15-inch outfall pipes at NE 11th Street will remain in place since it is a private system. The estimated design and construction costs for this pipe size upgrade alternative are approximately $636,000.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 1 are summarized in Table 5.12.1 below. Alternative 1 results in a maximum reduction of 0.31 feet in peak flood stage within the study area. Within the critical model node (Node IN_3131), the peak flood depth is reduced from 1.58 feet under the existing conditions to 0.25 feet under Alternative 1. For this alternative to be feasible, additional storage should be provided within sub-basin via regarded swales in the public right-of-way.

Table 5.12.1 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3425	4.05	2.6	1.45	4.02	-0.03
MH_0507	3.96	5.4	0.00	3.90	-0.06
IN_3149	3.52	1.5	2.02	3.31	-0.21
IN_3131	3.51	2.2	1.36	3.29	-0.22
MH_0623	3.50	2.2	1.35	3.19	-0.31

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized in Table 5.12.2 below. According to the stormwater model, Alternative 1 results in a significant reduction in flooding duration with a maximum reduction of 84%. Within the critical model node (Node IN_3131), the flood duration is reduced from 6.7 hours under the existing conditions to 1.1 hours under Alternative 1.

Table 5.12.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_3425	2.60	21.5	21.3	1
IN_3131	2.15	6.7	1.1	84
MH_0623	2.15	8.8	7.6	14

Alternative 2: Pump Station

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 2 includes the installation of one pump station near the existing outfall from North Riverside Drive at model Node: IN_3131. The estimated design and construction costs for this pump station alternative are approximately $1,532,000. The components associated to the pump station are listed below.

* Install a new 18-inch discharge pipe from pump station to outfall into Intracoastal Waterway.
* Install new flap gates at existing outfalls for backflow prevention.
* Wet well with a total footprint of about 150 square feet and depth of 8 feet.
* Maximum pump capacity of 30 CFS, which is equivalent to the peak discharge of the existing drainage system during low tide conditions.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 2 are summarized in Table 5.12.3 below. According to the stormwater model, Alternative 2 results in minimal reductions in the peak flood stage throughout the study area. Within the critical model node (Node IN_3131), the peak flood depth is reduced from 1.36 feet under the existing conditions to 1.30 feet under Alternative 2.

Table 5.12.3 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3425	4.05	2.6	1.45	4.05	0.00
MH_0507	3.96	5.4	0.00	3.95	-0.01
IN_3149	3.52	1.5	2.02	3.46	-0.06
IN_3131	3.51	2.2	1.36	3.45	-0.06
MH_0623	3.50	2.2	1.35	3.44	-0.06

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized in Table 5.12.4 below. According to the stormwater model, the estimated reduction in flooding duration within the study area is relatively limited under Alternative 2. Within the critical model node (Node IN_3131), the flood duration is reduced from 6.7 hours under the existing conditions to 4.8 hours under Alternative 2.

Table 5.12.4 – Alternative 2 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_3425	2.60	21.5	21.5	0
IN_3131	2.15	6.7	4.8	28
MH_0623	2.15	8.8	8.0	9

Alternative 3: Pumped Drainage Well

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 3 includes the installation of one pumped drainage well near the existing outfall from North Riverside Drive at model Node: IN_3131. The estimated design and construction costs for this pump station alternative are approximately $813,000. The components associated to the pumped drainage well are listed below.

- Install a new 18-inch discharge pipe from pump station to outfall into Intracoastal Waterway.
- Install new flap gates at existing outfalls for backflow prevention.
- Install new pumped drainage well, maximum pump capacity of 9 CFS, which is equivalent to the peak discharge of the existing drainage system during low tide conditions.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 3 are summarized in Table 5.12.5 below. According to the stormwater model, Alternative 3 results in minimal reductions in the peak flood stage throughout the study area. Within the critical model node (Node IN_3131), the peak flood depth is reduced from 1.36 feet under the existing conditions to 1.31 feet under Alternative 3.

Table 5.12.5 – Alternative 3 Peak Flood Stage Summary					
	Existing Conditions			Alternative 3	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3425	4.05	2.6	1.45	4.05	0.00
MH_0507	3.96	5.4	0.00	3.95	-0.01
IN_3149	3.52	1.5	2.02	3.47	-0.05
IN_3131	3.51	2.2	1.36	3.46	-0.05
MH_0623	3.50	2.2	1.35	3.45	-0.05

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 3 is summarized in Table 5.12.6 below. According to the stormwater model, the estimated reduction in flooding duration within the study area is relatively limited under Alternative 2. Within the critical model node (Node IN_3131), the flood duration is reduced from 6.7 hours under the existing conditions to 4.5 hours under Alternative 2.

Table 5.12.6 – Alternative 3 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
IN_3425	2.60	21.5	21.5	0
IN_3131	2.15	6.7	4.5	32
MH_0623	2.15	8.8	7.7	13

Alternative 4: Pumped Drainage Well and Pipe Size Upgrades

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 4 includes the installation of one pumped drainage well with the same characteristics from Alternative 3 and the installation of pipe size upgrades with the same characteristics as noted in Alternative 1. The estimated design and construction costs for this alternative are approximately $979,000.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 4 are summarized in Table 5.12.7 below. According to the stormwater model, Alternative 4 results in minimal reductions in the peak flood stage throughout the study area. Within the critical model node (Node IN_3131), the peak flood depth is reduced from 1.36 feet under the existing conditions to 1.04 feet under Alternative 4.

Table 5.12.7 – Alternative 4 Peak Flood Stage Summary					
	Existing Conditions			Alternative 4	
Nodes	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3425	4.05	2.6	1.45	4.01	-0.04
MH_0507	3.96	5.4	0.00	3.89	-0.07
IN_3149	3.52	1.5	2.02	3.20	-0.32
IN_3131	3.51	2.2	1.36	3.19	-0.32
MH_0623	3.50	2.2	1.35	3.12	-0.38

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 4 is summarized in Table 5.12.8 below. According to the stormwater model, the estimated reduction in flooding duration within the study area under Alternative 4 is much greater than the previous alternatives listed. Within the critical model node (Node IN_3131), the flood duration is reduced from 6.7 hours under the existing conditions to 1.0 hours under Alternative4.

Table 5.12.8 – Alternative 4 Percent Flood Duration Reduction				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 4	Reduction (%)
IN_3425	2.60	21.5	21.3	1
IN_3131	2.15	6.7	1.0	85
MH_0623	2.15	8.8	7.6	14

Alternative Comparison

Refer to Table 5.12.9 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.12.9 refer to the critical problem area of the study area, which corresponds to Node IN_3131 within the stormwater model. Based on our analysis with the stormwater model, Alternative 4 provides significantly better flood control benefits to the study area in regards to the reduction of both peak flood stages and expected flood duration than all other alternatives. Alternative 4 should be implemented for this study area since it provides better flood control benefits. Although Alternative 4 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 4 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.12.9 – Alternative Comparison			
Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.22	5.6	$636,000
Alternative 2	0.06	1.9	$1,532,000
Alternative 3	0.05	2.2	$813,000
Alternative 4	0.32	5.7	$976,000

The recommended stormwater improvements for this study area include the installation of a new pumped drainage well and the replacement of one existing outfall pipe with 36-inch diameter pipe. The proposed

upsized outfall pipe is intended to reduce flooding within North Riverside Drive during low tide periods within the Intracoastal Waterway. Due to the extremely low ground surface elevations along North Riverside Drive, the proposed upsized outfall pipe will not assist with the gravity discharge during high tide periods within the Intracoastal Waterway. The installation of the pumped drainage well is intended to reduce flooding within North Riverside Drive during high tide periods within the Intracoastal Waterway. Due to the negative impacts of high tide on the performance of the stormwater system in this study area, the proposed improvements also include the installation of backflow prevention devices at this outfall from North Riverside Drive. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 4 will encounter various constructability concerns related to the replacement of outfall pipe within utility easements on private property and regulatory limitations on the peak discharge via the upsized outfall pipes.

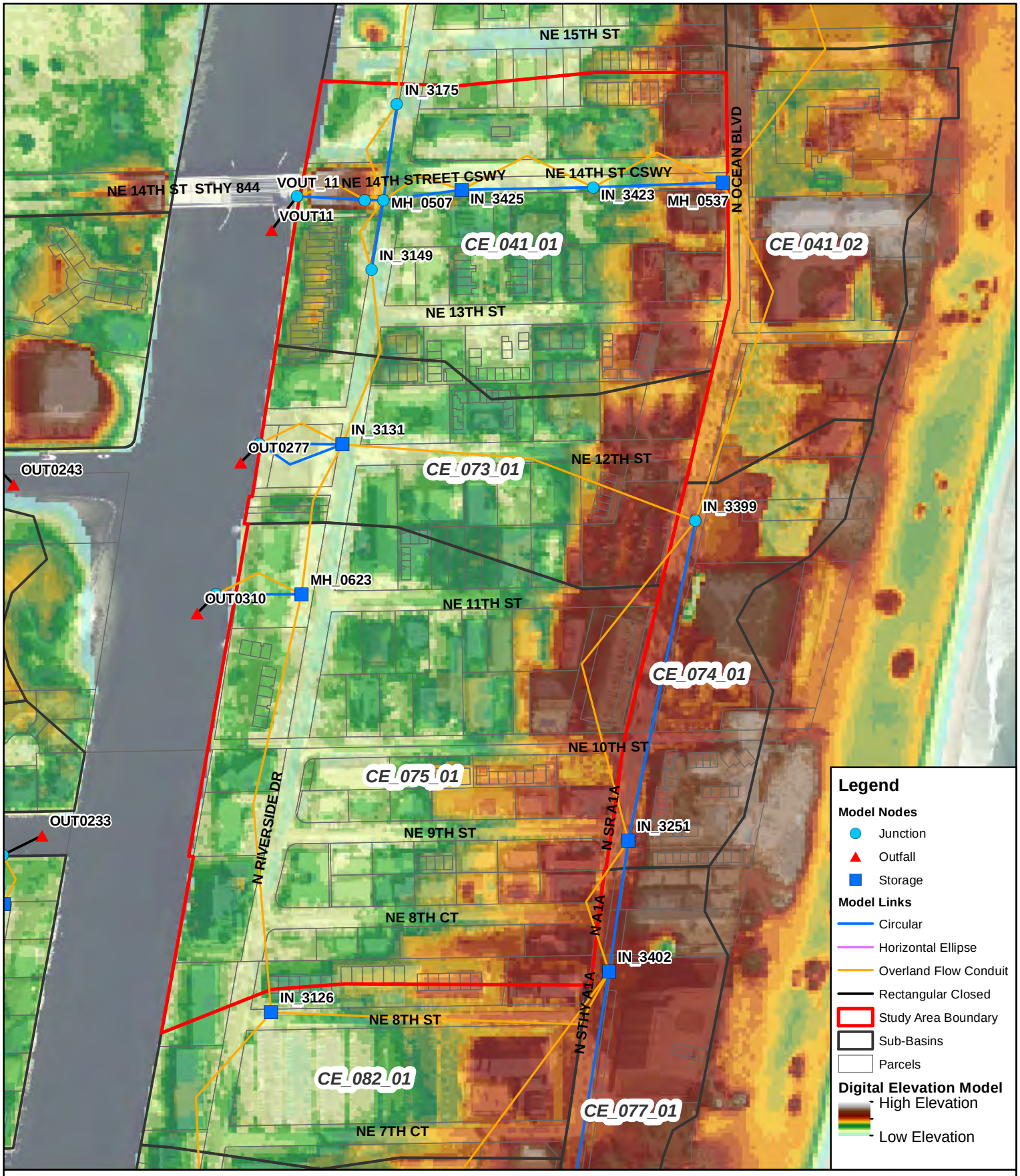


Figure 5-12A
N Riverside Drive & NE 14th Street Causeway
Existing Topography & Model Schematics

N
W E
S

0 200 400
Feet

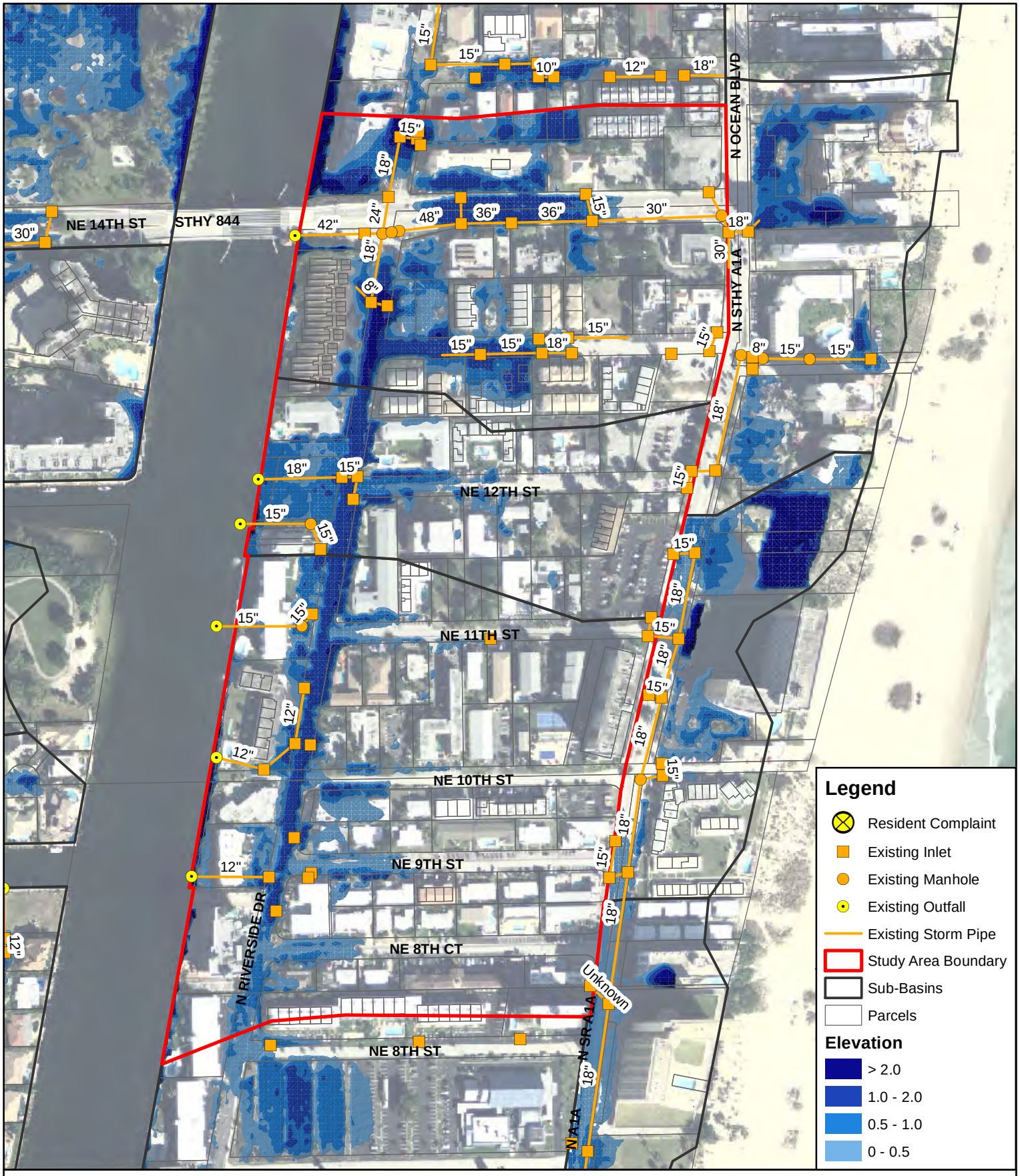


Figure 5-12B
N Riverside Drive & NE 14th Street Causeway
Potential Flooding Depth

N
W E
S

0 200 400
Feet

5.2.13 STUDY AREA 13 – ATLANTIC BOULEVARD AND SOUTH RIVERSIDE DRIVE

This study area is located on the east side of the Intracoastal Waterway chiefly along Riverside Drive. The project area extends along Riverside Drive from the intersection with Atlantic Boulevard on the northern limits to the intersection of SE 10th Street on the southern limits. The existing condition stormwater model was used to evaluate the performance of the existing stormwater system during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. Based on the result of the existing conditions stormwater model, the extent of the expected flooding areas is displayed on Figure 5-13B at the end of this section. Based on our analysis, heavy flooding can be expected at the north side of South Riverside Drive between SE 2nd Street to Atlantic Boulevard, which is very low lying.

The topography of the study area is displayed on Figure 5-13A along with the model schematics. The ground surface elevation along the centerline of South Riverside Drive is as low as 1.3 feet NAVD at some locations. Within the stormwater model, the study area is defined by the Sub-basins SE_024_01, SE_025_01, SE_026_01, SE_041_01, and SE_027_01, which are all bounded to the west by the Intracoastal Waterway. Based on the topography, the study area can be divided in two sub-areas by SE 5th Street. There is no exchange of stormwater runoff between these two sub-areas since they are isolated topographically by a small ridge at this intersection. As shown in the topography map, these sub-basins can be expected receive a significant amount of stormwater runoff from sub-basins CE_081_03, SE_030_01, and SE_007_01, which are located to the east and have higher ground surface elevations. Due to the very low elevation of the study area, the expected flooding is also tidally influenced since backflow from the Intracoastal Waterway can occur via the existing outfall pipes.

The City has received resident complaints on the north Side of South Riverside Drive, just north of SE 2nd Street. Site photographs have been provided which show extensive flooding along South Riverside Drive between SE 2nd Street and Atlantic Boulevard as well throughout the vacant property to the west of Riverside Drive. This documented flooding complaint mimics the model results displayed in Figure 5-13B. Various system improvement alternatives to the existing stormwater system were investigated for this study area, which include upgrading the pipe sizes, installing a pump station that discharges to the Intracoastal Waterway through an existing outfall, and installing a pump station with a stormwater retention area. Please note that the installation of exfiltration trench was not considered as a system improvement alternative since the very low elevation of the study area would limit the effectiveness of these options.

Alternative 1: Pipe Size Upgrades

The proposed improvements under Alternative 1 include upgrading the pipe sizes at specific locations increase the conveyance capacity of the stormwater system, which could increase the discharge rate to alleviate the flooding problems within the study area. For this study area, all existing outfalls pipes were analyzed to receive pipe size upgrades. Alternative 1 includes the removal of existing pipes (1,530 linear feet) with diameters between 12 inches and 21 inches. Under Alternative 1, the proposed pipe sizes include 310 linear feet of 24 inch RCP and 1,220 linear feet of 30 inch RCP. The estimated design and construction costs for Alternative 1 are approximately $1,900,000.

Based on the results of our analysis, the reduction of the peak flood stages under Alternative 1 is displayed in Table 5.13.1 below. The model results show the effectiveness of Alternative 1 at improving flooding conditions at different locations around the study area. The average reduction of peak flood stage is approximately 0.42 feet throughout the study area. However, the flooding at Node IN_3353, which is the location of the major flooding concern at the north end of Riverside Drive, is still expected to flood to depth greater than 1 foot under Alternative 1.

Table 5.13.1 – Alternative 1 Peak Flood Stage Summary					
Nodes	Existing Conditions			Alternative 1	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3353	2.53	1.30	1.23	2.40	-0.13
IN_3350	2.96	2.53	0.43	2.62	-0.34
IN_3349	2.96	1.72	1.24	2.40	-0.56
IN_3345	2.96	2.35	0.61	2.43	-0.53
IN_3343	2.96	2.12	0.84	2.43	-0.53

Based on our analysis with the stormwater model, the expected flooding duration within the study area is summarized within Table 5.13.2 below. The model results show the effectiveness of Alternative 1 at improving flooding conditions along South Riverside Drive throughout this study area, which shows a significant reduction in predicted flooding depth. The flood duration within Node IN_3353, which is the location of the major flooding concern, is expected to be reduced from 2.5 hours under the existing conditions to 0.78 hours under Alternative 1.

Table 5.13.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_3353	1.30	2.5	0.78	69
IN_3349	1.72	3.7	0.18	95
IN_3345	2.35	4.2	0	100
IN_3343	2.12	5	0.51	90

<u>Alternative 2: Pump Station</u>

The proposed construction under Alternative 2 includes the installation of two pump stations adjacent to existing outfalls in order to provide additional hydraulic head on the downstream end of the system to increase the system discharge capacity to alleviate the flooding, especially during high tide conditions. The proposed pump stations are located next to the model Node IN_3353 in the north sub-area and model Node IN_3349 in the south sub-area. The assumed components associated with each pump station are listed below. Additional pipe improvements are proposed to efficiently transmit stormwater runoff to the pump stations. The estimated design and construction costs for this pump station alternative are approximately $2,927,000.The model assumptions in regards to the proposed pump station are as follows:

- The existing gravity outfall pipe discharging to the Intracoastal Waterway is to be replaced by a 24-inch discharge pipe.
- Install a backflow prevention flap gate at the point of discharge into the Intracoastal Waterway.
- A wet well with a total footprint of about 150 square feet and depth of 8 feet.
- A proposed pump capacity shall be approximately 20 CFS, which is equivalent to the peak discharge rate of the existing drainage system during low tide conditions.

Based on our analysis with the stormwater model, the reduction of the peak flood stages under Alternative 2 are summarized within Table 5.13.3 below. The model results show a peak stage reduction of 0.21 feet

adjacent to the proposed pump station for the north sub-area at Node IN_3353, which is the location of the major flooding concern at the north end of Riverside Drive. No other portion of the study area receives any benefit in regards peak stage reduction under Alternative 2.

Table 5.13.3 – Alternative 2 Peak Stage Summary					
Nodes	**Existing Conditions**			**Alternative 2**	
	Peak Stage (feet)	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**
IN_3353	2.53	1.30	1.23	2.32	-0.21
IN_3350	2.96	2.53	0.43	2.96	0.00
IN_3349	2.96	1.72	1.24	2.96	0.00
IN_3345	2.96	2.35	0.61	2.96	0.00
IN_3343	2.96	2.12	0.84	2.97	+0.01

Based on our analysis with the stormwater model, the reduction of flood duration within the study area under Alternative 2 is summarized in Table 5.13.4 below. The estimated reduction in flood duration under Alternative 2 is relatively insignificant when compared to Alternative 1. The flood duration within Node IN_3353, which is the location of the major flooding concern at the north side of Riverside Drive, is expected to be reduced from 2.5 hours under the existing conditions to 1.25 hours under Alternative 1.

Table 5.13.4 – Alternative 2 Flood Duration Summary				
Nodes	**Reference Street Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 2**	**Reduction (%)**
IN_3353	1.30	2.5	1.25	50
IN_3349	1.72	3.7	2.85	23
IN_3345	2.35	4.2	4.05	4
IN_3343	2.12	5	4.90	2

<u>Alternative 3/4: Pump Station and Storage</u>

The proposed construction under Alternative 3 includes the installation of a pump station, which connects to potential stormwater retention area(s) at an undetermined location within the study area. The proposed stormwater retention area was assumed encompasses a total area of 1.0 acres. The proposed retention area was assumed to have a perimeter berm at +8.0 feet NAVD with a 3:1 internal side slope to the bottom elevation of +3.5 feet NAVD. The bottom of the retention area is set to be 1 foot higher than the seasonal high water table, which is expected to be around the site at +2.5 feet NAVD. The areas of the stormwater retention area were estimated to be 0.67 acres at the top of the berm and 0.44 acres at the bottom of the retention area. Overflow from this stormwater retention area would need to be connected back to the existing system through a weir-type control structure. The weir elevation within the control structure was assumed at +7.5 feet NAVD, which would provide a total storage volume of 1.93 acre-feet. The estimated design and construction costs for this pump station and storage alternative are approximately $4,375,000.

The stormwater model was used to analyze Alternative 3 with the existing stormwater pipe remaining in place and Alternative 4 with increasing the existing pipe diameters to 30 inch RCP. The reduction of the peak flood stages under Alternative 3/4 are summarized within Table 5.13.5 below. This simulation showed an additional reduction in peak flood stages at Node IN_3353, which is the location of the major

flooding concern at the north end of Riverside Drive. There is no flood reduction benefit in other areas under Alternative 3.

Table 5.13.5 – Alternative 3/4 Peak Flood Stage Summary							
Nodes	**Existing Conditions**			**Alternative 3 (Existing Pipes)**		**Alternative 4 (30-inch RCP)**	
	Peak Stage (feet)	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**
IN_3353	2.53	1.30	1.23	2.31	-0.22	1.88	-0.65
IN_3350	2.96	2.53	0.43	2.96	0.00	2.96	0.00
IN_3349	2.96	1.72	1.24	2.96	0.00	2.96	0.00
IN_3345	2.96	2.35	0.61	2.96	0.00	2.96	0.00
IN_3343	2.96	2.12	0.84	2.96	0.00	2.96	0.00

Based on our analysis with the stormwater model, the reduction in flood duration in the study area is summarized below in Table 5.13.6 for Alternative 3 and in Table 5.13.7 for Alternative 4. The stormwater model predicts a reduction in flood duration of 50% under Alternative 3 and 76% under Alternative 4 at Node IN_3353, which corresponds to the north end of Riverside Drive. There is no reduction in expected flood durations in other areas along Riverside Drive under either Alternative 3 or Alternative 4.

Table 5.13.6 – Alternative 3 (with Existing Pipes) Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 3 (Existing Pipe)**	**Reduction (%)**
IN_3353	1.30	2.5	1.26	50
IN_3349	1.72	3.7	3.7	0
IN_3345	2.35	4.2	4.2	0
IN_3343	2.12	5	5	0

Table 5.13.7 – Alternative 4 (with 30-inch RCP) Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 4 (30-inch RCP)**	**Reduction (%)**
IN_3353	1.30	2.5	0.59	76
IN_3349	1.72	3.7	3.7	0
IN_3345	2.35	4.2	4.2	0
IN_3343	2.12	5	5	0

Alternative 5: Pumped Drainage Well

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 5 includes the installation of three pumped drainage well near the existing

outfalls on South Riverside Drive. The estimated design and construction costs for this pump station alternative are approximately $1,947,000. The components associated to the pumped drainage well are listed below.

* Existing discharge pipe to outfall into Intracoastal Waterway to remain the same size.
* Install new flap gates at existing outfalls for backflow prevention.
* Install new pumped drainage well, maximum pump capacity of 9 CFS, which is equivalent to the peak discharge of the existing drainage system during low tide conditions.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 5 are summarized in Table 5.12.8 below. According to the stormwater model, Alternative 5 results in minimal reductions in the peak flood stage throughout the study area. Within the critical model node, Node IN_3353, the peak flood depth is reduced from 1.23 feet under the existing conditions to 1.11 feet under Alternative 5.

Table 5.13.8 – Alternative 5 Peak Stage Summary

Nodes	Existing Conditions			Alternative 5	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3353	2.53	1.30	1.23	2.41	-0.12
IN_3350	2.96	2.53	0.43	2.86	-0.10
IN_3349	2.96	1.72	1.24	2.85	-0.11
IN_3345	2.96	2.35	0.61	2.87	-0.09
IN_3343	2.96	2.12	0.84	2.88	-0.08

Based on our analysis with the stormwater model, the reduction of flood duration within the study area under Alternative 5 is summarized in Table 5.13.9 below. The estimated reduction in flood duration under Alternative 5 is relatively insignificant when compared to Alternative 1. The flood duration within Node IN_3353, which is the location of the major flooding concern at the north side of Riverside Drive, is expected to be reduced from 2.5 hours under the existing conditions to 1.80 hours under Alternative 5.

Table 5.13.9 – Alternative 5 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 5	Reduction (%)
IN_3353	1.30	2.5	1.8	28
IN_3349	1.72	3.7	1.7	54
IN_3345	2.35	4.2	1.8	57
IN_3343	2.12	5	3	40

Alternative 6: Pumped Drainage Well and Pipe Size Upgrades

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 6 includes the installation of three pumped drainage wells with the same characteristics of Alternative 5 and the installation of upsized outfall pipes with same characteristics as

noted in Alternative 1. The estimated design and construction costs for this alternative are approximately $2,870,000.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 6 are summarized in Table 5.12.10 below. According to the stormwater model, Alternative 6 results in the most significant reductions in the peak flood stage throughout the study area. Within the critical model node, Node IN_3353, the peak flood depth is reduced from 1.23 feet under the existing conditions to 1.10 feet under Alternative 6.

Table 5.13.10 – Alternative 6 Peak Stage Summary

Nodes	Existing Conditions			Alternative 6	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_3353	2.53	1.30	1.23	2.40	-0.13
IN_3350	2.96	2.53	0.43	2.62	-0.34
IN_3349	2.96	1.72	1.24	2.40	-0.56
IN_3345	2.96	2.35	0.61	2.40	-0.56
IN_3343	2.96	2.12	0.84	2.40	-0.56

Based on our analysis with the stormwater model, the reduction of flood duration within the study area under Alternative 6 is summarized in Table 5.13.11 below. The estimated reduction in flood duration under Alternative 6 is the most significant reductions when compared to the previous listed alternatives. The flood duration within Node IN_3353, which is the location of the major flooding concern at the north side of Riverside Drive, is expected to be reduced from 2.5 hours under the existing conditions to 0.60 hours under Alternative 6.

Table 5.13.11 – Alternative 6 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 6	Reduction (%)
IN_3353	1.30	2.5	0.6	76
IN_3349	1.72	3.7	0.14	96
IN_3345	2.35	4.2	0	100
IN_3343	2.12	5	0.4	92

Alternative Comparison

Refer to Table 5.13.12 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.13.12 refer to the critical problem area of the study area, which corresponds to Node IN_3353 within the stormwater model. Based on our analysis with the stormwater model, all system improvement alternatives can be considered to be an effective option for reducing the peak flood stages and reducing the expected flood duration within the study area. However, Alternative 3 and Alternative 4 should likely be eliminated from consideration since using valuable private property in this study area for stormwater retention is not feasible from a cost standpoint. Alternative 1 provides similar flood control benefits as Alternative 2 and is significantly more cost effective, yet does not assist with discharge capacity during high tides. Alternative 6 should be implemented for this study area since it provides flood control throughout the

study area and supplemental discharge capacity during high tide periods within the Intracoastal Waterway. Although Alternative 6 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 6 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.13.12 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.13	1.72	$1,900,000
Alternative 2	0.21	1.25	$2,927,000
Alternative 3	0.22	1.24	$4,375,000
Alternative 4	0.65	1.91	$4,493,000
Alternative 5	0.12	0.90	$1,947,000
Alternative 6	0.13	1.90	$2,870,000

The recommended stormwater improvements for this study area include the replacement of six existing outfall pipes with 24-inch or 30-inch diameter pipe, which will significantly reduce flooding within South Riverside Drive during low tide within the Intracoastal Waterway. Due to the extremely low ground surface elevations along South Riverside Drive, the upsized outfall pipes will not assist with the gravity discharge during high tide within the Intracoastal Waterway. The installation of the pumped drainage wells are intended to reduce flooding within North Riverside Drive during high tide periods within the Intracoastal Waterway. Due to the negative impacts of high tide on the performance of the stormwater system in this study area, the proposed improvements include the installation of backflow prevention devices at all six existing outfalls from South Riverside Drive. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 6 will encounter various constructability concerns related to the replacement of outfall pipe within utility easements on private property and regulatory limitations on the peak discharge via the upsized outfall pipes.

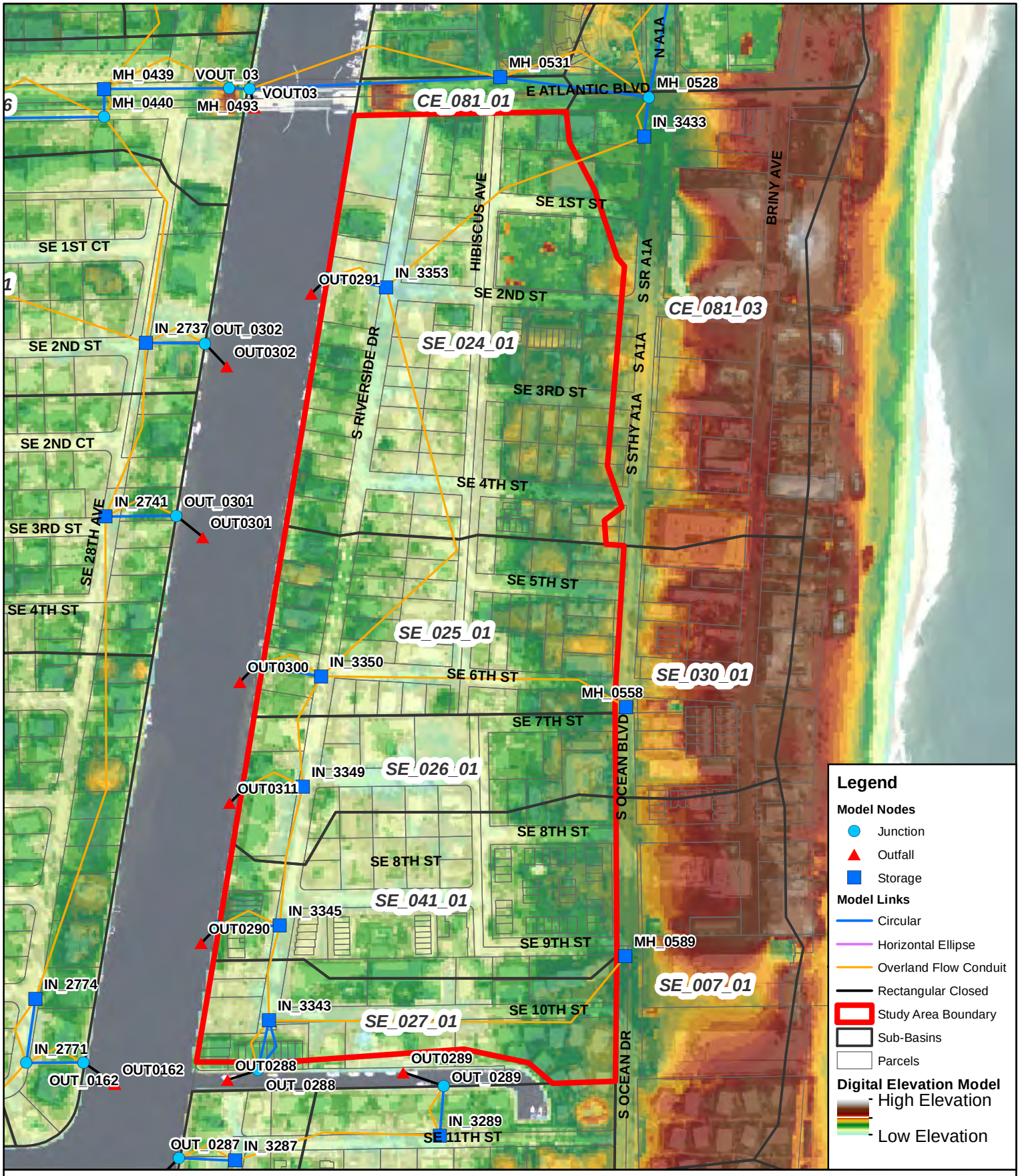


Figure 5-13A
Atlantic Blvd & South Riverside Drive
Existing Topography & Model Schematics

N
W E
S

0 200 400
Feet

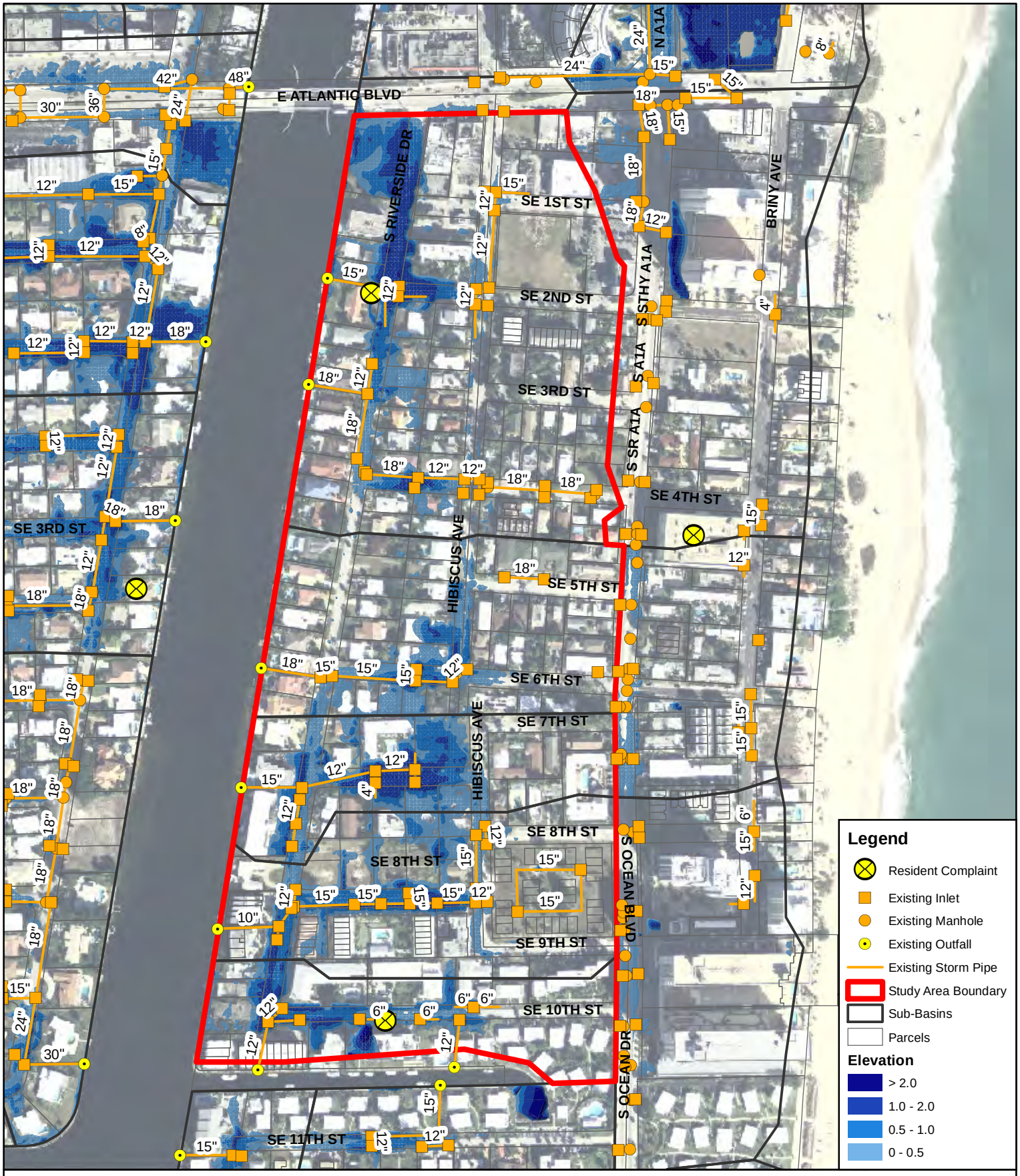


Figure 5-13B
Atlantic Blvd & South Riverside Drive
Potential Flooding Depth

N
W E
S

0 200 400
Feet

5.2.14 STUDY AREA 14 – NE 27TH AVENUE AND NE 16TH STREET

This study area is bounded by US-1 on the west, NE 22nd Street on the north, NE 28th Avenue on the east, and NE 16th Street on the south. This study area consists of primarily single family residential properties with a limited existing drainage system serving the roadways. The existing drainage system within the study area consists of two independent drainage systems that collect stormwater runoff from the public right-of-way and discharges via existing 24-inch outfalls into tidal canals, which are directly connected to the Intracoastal Waterway. An existing outfall is located at the north end along NE 22nd Court while the other existing outfall is located at the south side end of NE 16th Street.

The existing conditions stormwater model was used to evaluate the potential flooding within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basins CE_032_01 and CE_038_01 within the stormwater model. The topography of the study area along with the model schematics are displayed within Figure 5-14A at the end of this section. According to the topography, stormwater runoff can be expected to flow from the north to the south along NE 27th Avenue before collecting in low lying areas in the right-of-way. The lowest elevations within the study area directly correlate to the flooding problems. Based on the results of the existing conditions stormwater model, the extent of potential flooding within the study area is displayed on Figure 5-14B at the end of this section. The significant portions of the public right-of-way areas within this study area display flooding greater than one inch. The worst flooding is expected primarily along NE 27th Avenue and the east section of NE 16th Street and NE 17th Street.

The stormwater model was used to evaluate effectiveness of various system improvement alternatives, such as exfiltration trenches, drainage wells, or pump stations, in reducing the existing flooding problems. The system improvement alternatives which were evaluated with the stormwater model are summarized below.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area not currently served by the existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 8,022 LF of exfiltration trench, which were aligned along right-of-way areas with ground surface elevations greater than +5.0 feet NAVD. The general location of the proposed exfiltration trench is summarized by sub-basin within Table 5.14.1 below. The estimated design and construction costs for this exfiltration trench alternative are approximately $3,390,000.

Table 5.14.1 – Alternative 1 Proposed Exfiltration Trench Summary

Sub-Basin	Exfiltration Trench (LF)	Mean Ground Surface Elevation (feet NAVD)
CE_032_01	5,044	+5.6
CE_036_01	598	+6.1
CE_038_01	2,380	+4.9
Total	**8,022**	**+5.53**

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this

study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.14.2 below. The model results for Alternative 1 show a reduction in peak flood stage of -0.13 feet at the critical model node (Node IN_3013). The peak flood depth is expected to be reduced from 1.79 feet under the existing conditions to 1.66 feet under Alternative 1.

Table 5.14.2 – Alternative 1 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 1	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_3013	4.89	3.1	1.79	4.76	-0.13
IN_3043	4.97	3.3	1.67	4.85	-0.12
IN_3003	2.45	1.5	0.95	2.45	0.00
MH_0484	4.58	5.6	N/A	4.58	N/A

Based on our analysis with the stormwater model, the reduction of flooding duration within the study area under Alternative 1 is summarized in Table 5.14.3 below. The reduction in flood duration under Alternative 1 is moderate with a maximum reduction of 34% from the existing conditions. At the critical model node (Node IN_3013), the flood duration is expected to be reduced from 7.7 hours under the existing conditions to 6.4 hours under Alternative 1.

Table 5.14.3 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_3013	3.10	7.7	6.4	16
IN_3043	3.30	5.0	3.3	34

<u>Alternative 2: Pipe Size Upgrades</u>

The proposed improvements under Alternative 2 include upgrading the pipe size at the outfall to increase the conveyance capacity of the stormwater system, which could increase the discharge rate to alleviate the flooding problems within the study area. Alternative 2 includes replacing the existing 12-inch pipe and 24-inch pipe with approximately 600 linear feet of new 36-inch RCP. The estimated design and construction costs for Alternative 2 are approximately $338,000.

Based on the results of our analysis, the reduction of the peak flood stages under Alternative 2 is displayed in Table 5.14.4 below. Alternative 2 results in a reduction in only the critical model node (Node

IN_3013) with a peak flood depth reduced from 1.79 feet under the existing conditions to 1.30 feet under Alternative 2.

Table 5.14.4 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_3013	4.89	3.1	1.79	4.40	-0.49
IN_3043	4.97	3.3	1.67	4.97	0.00
IN_3003	2.45	1.5	0.95	2.45	0.00
MH_0484	4.58	5.6	N/A	4.58	0.00

Based on our analysis with the stormwater model, the expected flooding duration within the study area is summarized within Table 5.14.5 below. The model results show the effectiveness of Alternative 2 at improving flooding conditions throughout this study area, which shows a significant reduction in predicted flooding depth. The flood duration within Node IN_3013, which is the location of the major flooding concern, is expected to be reduced 7.7 hours under the existing conditions to 2.5 hours under Alternative 2.

Table 5.14.5 – Alternative 2 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_3013	3.10	7.7	2.5	67
IN_3043	3.30	5.0	4.4	12

<u>Alternative 3: Pump Station</u>

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 3 includes the installation of one pump station near the existing outfall at model Node: IN_3013. The estimated design and construction costs for this pump station alternative are approximately $2,198,000. The components associated to the pump station are listed below.

- Install a 24-inch discharge pipe from pump station to outfall location.
- Install a flap gate at the point of discharge for backflow prevention.
- Wet well with a total footprint of about 150 square feet and depth of 8 feet.
- Maximum pump capacity shall be 33 CFS, which is equivalent to peak discharge rate from existing drainage system during low tide conditions.

Based on our analysis with the stormwater model, the reduction in peak flood stages under Alternative 3 is summarized within Table 5.14.6 below. The model results for Alternative 3 show a minimal reduction in peak flood stage at Node IN_3013 adjacent to the proposed pump station. The remainder of the study area receives no benefit from Alternative 3.

Table 5.14.6 – Alternative 3 Peak Flood Stage Summary					
	Existing Conditions			Alternative 3	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_3013	4.89	3.1	1.79	4.81	-0.08
IN_3043	4.97	3.3	1.67	4.97	0.00
IN_3003	2.45	1.5	0.95	2.45	0.00
MH_0484	4.58	5.6	N/A	4.58	N/A

Based on our analysis with the stormwater model, the reduction in expected flooding duration is summarized within Table 5.14.7 below. The model results show a limited reduction in flood duration under Alternative 3. At the critical model node (Node IN_3013), the flood duration is expected to be reduced from 7.7 hours under the existing conditions to 6.5 hours under Alternative 3.

Table 5.14.7 – Alternative 3 Flood Duration Summary				
Nodes	Reference Street Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
IN_3013	3.10	7.7	6.5	15
IN_3043	3.30	5.0	4.5	9

Alternative 4: Drainage Wells

The stormwater model was used to conduct several simulations of the installation of proposed drainage wells within problem areas of the study area. The purpose of this system improvement alternative is to intercept stormwater runoff before it reaches the existing outfalls and to provide additional discharge capacity at the problem area to alleviate the existing flooding issues quicker. The expected construction includes a total of seven drainage wells along NE 16th Street and NE 17th Street within the study area. Within the stormwater model, a minimum driving head of 1.5 feet above the SHWT was assumed prior to activating discharge via the proposed drainage wells. Based on the historical information for the area, the discharge rate of each drainage well was assumed to be 450 GPM per foot of head within the stormwater model, which is approximately equivalent to 1.0 CFS per foot of head. All proposed drainage wells will be interconnected to maintain the same driving head to each drainage well. The estimated design and construction costs for this drainage well alternative are approximately $1,098,000.

Based on our analysis with the stormwater model, the reduction in peak flood stages under Alternative 4 is summarized within Table 5.14.8 below. The model results for Alternative 4 show a reduction in peak flood stage of 0.18 feet at Node IN_3013 adjacent to the proposed pump station. The remainder of the study area receives no benefit from Alternative 4.

Table 5.14.8 – Alternative 4 Peak Flood Stage Summary					
	Existing Conditions			Alternative 4	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_3013	4.89	3.1	1.79	4.71	-0.18
IN_3043	4.97	3.3	1.67	4.97	0.00
IN_3003	2.45	1.5	0.95	2.45	0.00
MH_0484	4.58	5.6	N/A	4.58	N/A

Based on our analysis with the stormwater model, the reduction in expected flooding duration is summarized within Table 5.14.9 below. According to the model results, Alternative 4 shows a limited reduction in flood duration of up to 37% from the existing conditions. Alterative 4 reduces the expected flooding duration to less than 5 hours within the study area. At the critical model node (Node IN_3013), the flood duration is expected to be reduced from 7.7 hours under the existing conditions to 4.8 hours under Alternative 4.

Table 5.14.9 – Alternative 4 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 4	Reduction (%)
IN_3013	3.10	7.7	4.8	37
IN_3043	3.30	5.0	4.5	9

Alternative 5: Exfiltration Trench & Pipe Size Upgrades

This alternative combines the exfiltration trench of alternative 1 and pipe size upgrades of alternative 2. Under Alternative 1, the proposed construction includes a total of 8,022 LF of exfiltration trench, which were aligned along right-of-way areas with ground surface elevations greater than +5.0 feet NAVD. The proposed improvements under Alternative 2 includes replacing the existing 12-inch pipe and 24-inch pipe with approximately 600 linear feet of new 36-inch RCP. The estimated design and construction costs for Alternative 5 are approximately $2,486,000.

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.14.10 below. The model results for Alternative 5 show a reduction in peak flood stage of -0.72 feet at the critical model node (Node IN_3013). The peak flood depth is expected to be reduced from 1.79 feet under the existing conditions to 1.07 feet under Alternative 5.

Table 5.14.10 – Alternative 5 Peak Flood Stage Summary					
	Existing Conditions			Alternative 5	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_3013	4.89	3.1	1.79	4.17	-0.72
IN_3043	4.97	3.3	1.67	4.85	-0.12
IN_3003	2.45	1.5	0.95	2.45	0.00
MH_0484	4.58	5.6	N/A	4.58	N/A

Based on our analysis with the stormwater model, the reduction of flooding duration within the study area under Alternative 5 is summarized in Table 5.14.11 below. At the critical model node (Node IN_3013), the flood duration is expected to be reduced from 7.7 hours under the existing conditions to 2.0 hours under Alternative 5.

Table 5.14.11 – Alternative 5 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 5	Reduction (%)
IN_3013	3.10	7.7	2.0	74
IN_3043	3.30	5.0	2.5	49

Alternative Comparison

Refer to the Table 5.14.12 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood duration results within Table 5.14.12 refer to the critical problem area within the study area, which corresponds to Node IN_3013 within the stormwater model. Based on our analysis with the stormwater model, all system improvement alternatives provide similar flood control benefits to the study area, which are limited. Under all five alternatives, the reduction in peak flood stage ranges from 0.08 feet to 0.49 feet while the reduction in expected flood duration ranges from 1.2 hours to 5.2 hours. Based on the model results, Alternative 5 is slightly more effective than all of the other alternatives at providing additional flood control to the study area. Alternative 5 has the less potential concern that could arise during the detailed design phase which could restrict the complete implementation. Alternative 5 should be implemented for this study area since it provides the best potential flood control benefits. Although Alternative 5 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 5 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.14.12 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.13	1.3	$3,390,000
Alternative 2	0.49	5.2	$338,000
Alternative 3	0.08	1.2	$2,198,000
Alternative 4	0.18	2.9	$1,098,000
Alternative 5	0.72	5.7	$2,486,000

CMA recommends the installation of exfiltration trench within City right-of-ways throughout the study area which provide additional storage and infiltration capacity for stormwater runoff. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along NE 18th Street, NE 19th Street, NE 21st Street, NE 22nd Street and NE 27th Avenue to collect stormwater runoff from these areas. The proposed exfiltration system should be interconnected to existing drainage systems, which will allow drawdown via the existing outfalls. The recommended stormwater improvements also include upsizing the existing 24-inch outfall to a 36-inch outfall. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, the proposed construction will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

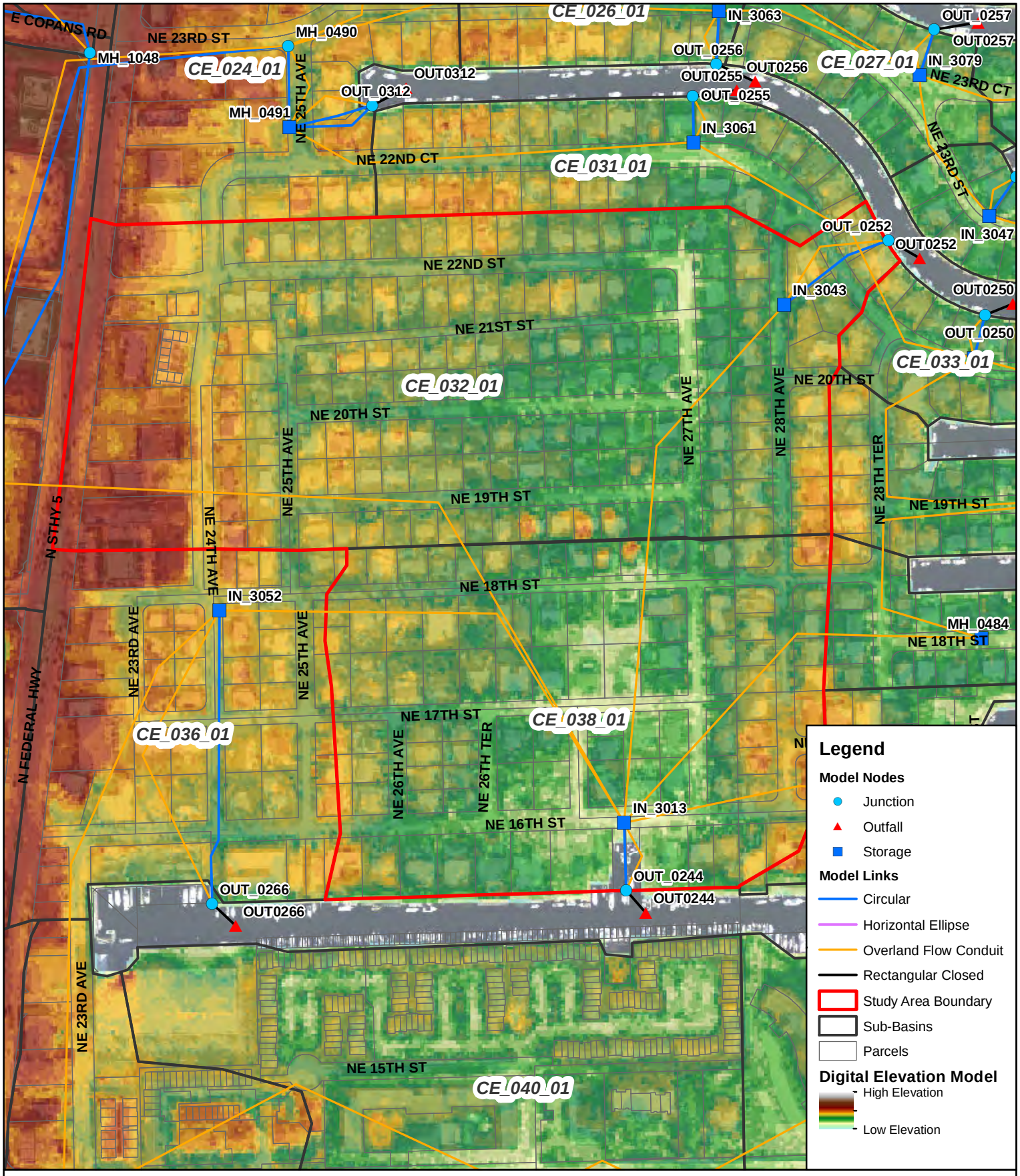


Figure 5-14A
NE 27th Avenue & NE 16th Street
Existing Topography & Model Schematics

N
W E
S

0 200 400
Feet

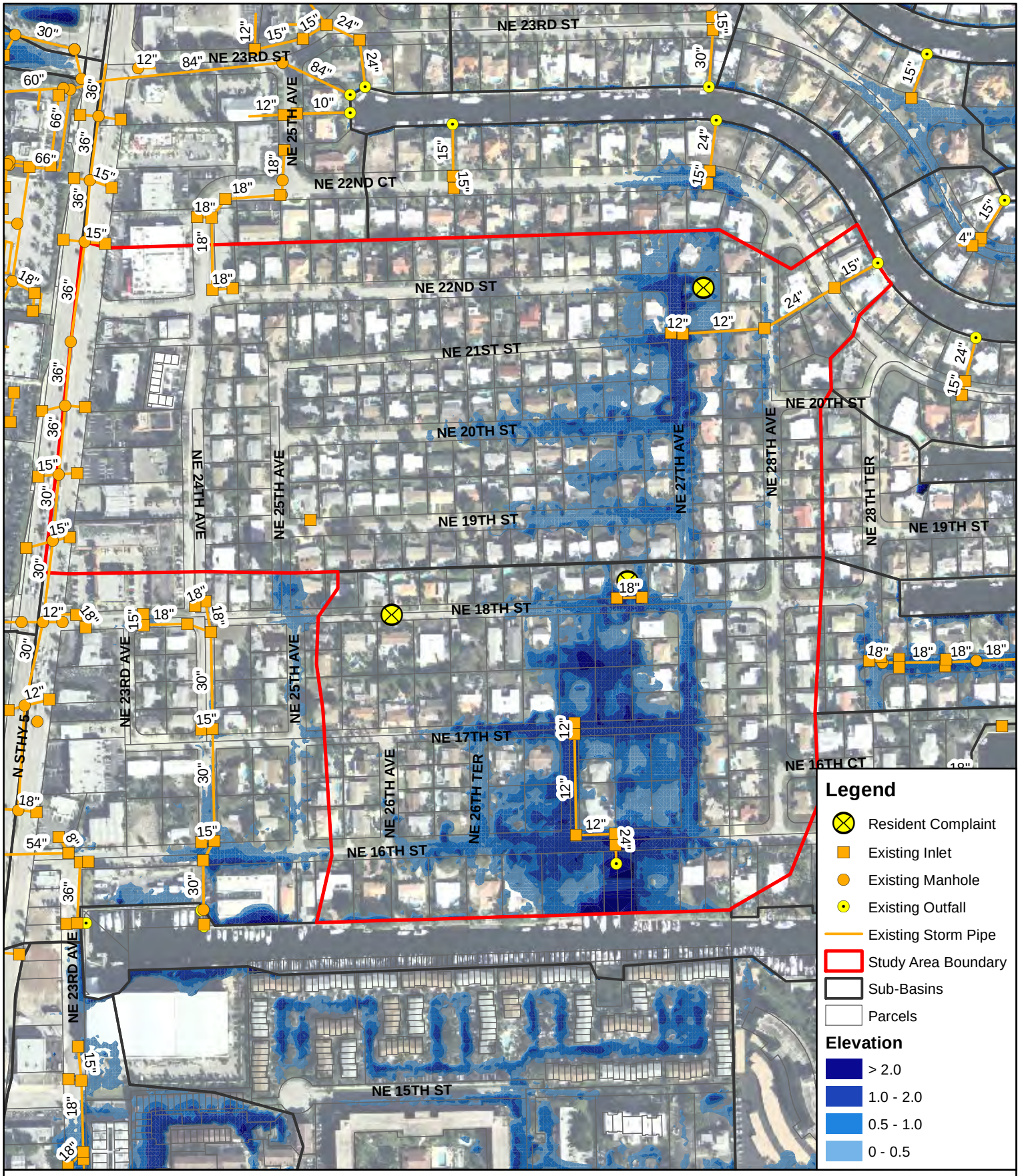


Figure 5-14B
NE 27th Avenue & NE 16th Street
Potential Flooding Depth

N
W E
S

0 200 400
Feet

5.2.15 STUDY AREA 15 – POWERLINE ROAD AND NW 33RD STREET

This study area is bounded by Powerline Road on the west, NW 33rd Court on the north, NW 18th Terrace on the east and NW 31st Street on the south. This study area consists mainly of industrial and commercial properties. The study area has a limited amount of public roadways, which include NW 33rd Court, NW 33rd Street, NW 18th Terrace and NW 32nd Street. These public right-of-way areas have a significant impervious surface coverage, which prevents the infiltration of stormwater runoff into the ground surface after rainfall events. There are no existing City drainage facilities within the study area. There is an existing FDOT drainage system, which only serves the right-of-way for Powerline Road.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basins NW_006_01 and NW_013_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-15A at the end of this section. According to the topography, the south side of the project area is a much lower elevation than the north. Based on the results of the stormwater model, the extent of the expected flooding within the study area is displayed on Figure 5-15B at the end of this section. NW 33rd Street displays flooding greater than two inches towards the west side of the road along with adjacent private properties towards the south of the study area. The only feasible system improvement alternative for this study area includes the installation of exfiltration trench within the public right-of-way areas due to the relatively high elevation, which averages 12.40 feet NAVD. The installation of drainage wells, additional dry retention areas, or a pump station were not feasible options for this study area and were not consider during the analysis of potential alternatives.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area, which is not currently served by an existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 3,330 LF of exfiltration trench t along NW 33rd Court, NW 33rd Street, NW 18th Terrace and NW 32nd Street. The installation of additional exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,339,000.

Table 5.15.1 – Alternative 1 Proposed Exfiltration Trench Summary

Sub-Basin	Exfiltration Trench (LF)	Mean Ground Surface Elevation (feet NAVD)
NW_006_01	1,230	+12.65
NW_013_01	2,100	+12.14
Total	**3,330**	**+12.40**

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.15.2 below. According to the stormwater model, Alternative 1 results in a minimal reduction in peak flood stages of -0.06 feet.

Table 5.15.2 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
NW01301	11.87	11.3	0.57	11.81	-0.06
NW00601	12.63	11.6	1.03	12.57	-0.06

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.15.3 below. The estimated reduction in flood duration under Alternative 1 is slightly better with a reduction of 39% from the existing conditions. Although this alternative does not completely eliminate the flooding within the study area, it is the only feasible option due to the site restrictions.

Table 5.15.3 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
NW01301	11.30	33.1	20.3	39
NW00601	11.60	3.5	2.4	33

Alternative Comparison

Alternative 1 provides minimal reduction in peak flood stage within the study area, but does reduce the flood duration within the study area. Although Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for public roadways within this study area, it does provide some additional benefits which alleviate the flooding problems within the study area. CMA recommends the installation of a new exfiltration trench system along NW 31st Street, NW 32nd Street, NW 33rd Street, and NW 18th Terrace, which will provide additional storage and infiltration capacity for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

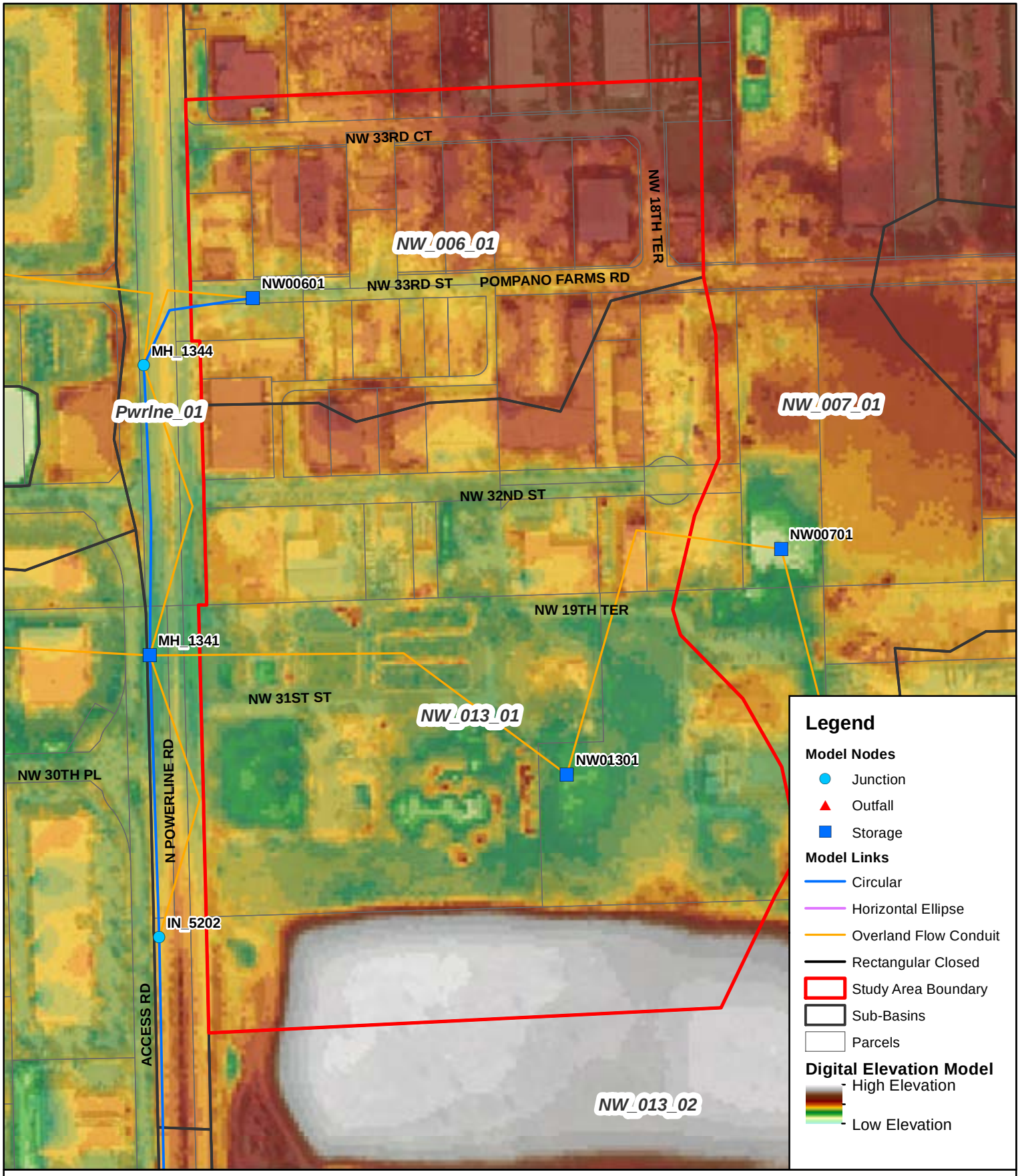


Figure 5-15A
Powerline Road & NW 33rd Street
Existing Topography & Model Schematics

N
W E
S

0 150 300
Feet

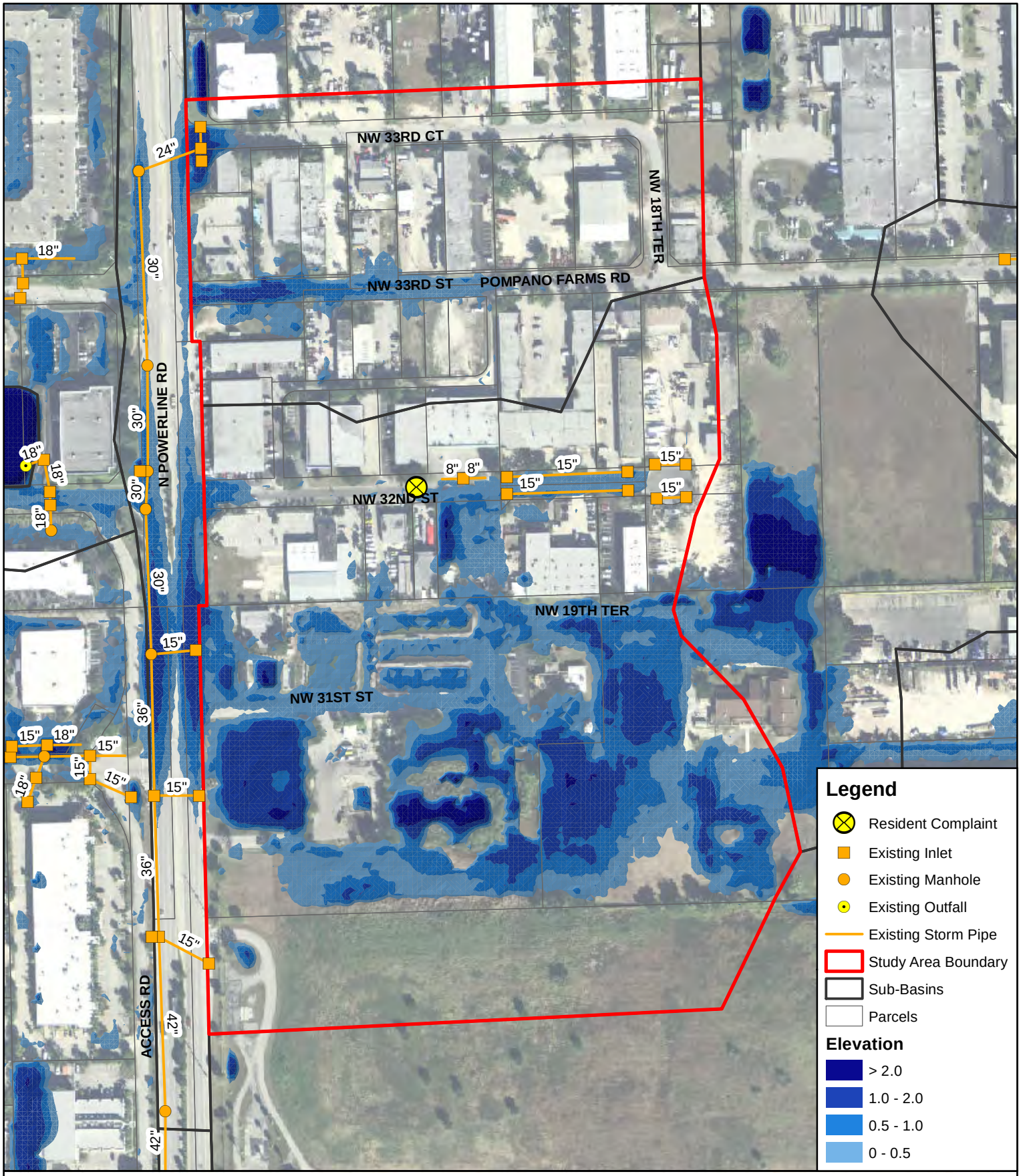


Figure 5-15B
Powerline Road & NW 33rd Street
Potential Flooding Depth

N
W E
S

0 150 300
Feet

5.2.16 STUDY AREA 16 – NW 22ND STREET

This study area is an isolated right-of-way area with heavy flooding problems just south of Copans Road and just west of Powerline Road. This study area mainly consists of industrial and commercial properties, with only one City roadway (NW 22nd Street) with significant impervious ground coverage, which can limit the infiltration of stormwater runoff into the ground surface. The remainder of the study area includes multiple private roadways and driveways which also have flooding problems. The public right-of-way area for NW 22nd Street has an existing stormwater system which consists of exfiltration trench along the south side of the roadway (approximately 870 linear feet of 15-inch and 18-inch pipe).

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basin CW_037_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-16A at the end of this section. According to the topography, the lowest elevations within the study area, beside the stormwater retention areas located on private property, are located along NW 22nd Street where stormwater runoff typically collects from the entire study area. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-16B at the end of this section. According to the model results, all public right-of-way areas within the study area display flooding greater than two inches. The system improvement alternatives that were investigated for this study area consist of expanding exfiltration trench system within NW 22nd Street, which are summarized further below. The installation of drainage wells, additional dry retention areas, or a pump station were not feasible options for this study area and were not consider during the analysis of potential alternatives.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct a simulation of the installation of additional exfiltration trench along the north side NW 22nd Street within the study area. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 800 LF of exfiltration trench along NW 22nd Street, which will be interconnected with the existing drainage system. The installation of additional exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $350,000.

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.16.1 on the following page. The model results show minimal to no peak reductions in peak flood stages under Alternative 1. A the critical model node of the study area (Node IN_0159), the peak flood depth is reduced from 0.54 feet under the existing conditions to 0.50 feet under Alternative 1.

Table 5.16.1 – Alternative 1 Peak Flood Stage Summary					
Nodes	**Existing Conditions**			**Alternative 1**	
	Peak Stage (feet, NAVD)	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet, NAVD)**	**Peak Stage Reduction (feet)**
IN_0159	12.24	11.7	0.54	12.20	-0.04
MH_1334	11.60	11.0	0.6	11.60	0.00
MH_1114	12.62	9.9	2.72	12.62	0.00

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.16.2 below. The model results of Alternative 1 show a reduction of flooding duration by 22 percent from the existing conditions. A the critical model node of the study area (Node IN_0159), the flood duration is reduced from 32.9 hours under the existing conditions to 25.7 hours under Alternative 1.

Table 5.16.2 – Alternative 1 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 1**	**Reduction (%)**
IN_0159	11.70	32.9	25.7	22
MH_1334	11.00	7.1	6.3	11
MH_1114	9.90	19.4	19.4	0

<u>Alternative Comparison</u>

Alternative 1 provides a minimal reduction in peak flood stage within the study area, but does reduce the peak flood stages within the study area. Although Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for public roadways within this study area, it does provide some additional benefits which alleviate the flooding problems within the study area. CMA recommends the installation of a new exfiltration trench system along NW 22nd Street which is parallel to the existing drainage piping and will provide additional storage and infiltration capacity for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

Figure 5-16A
NW 22nd Street
Existing Topography & Model Schematics

0 150 300 Feet

N W E S

Figure 5-16B
NW 22nd Street
Potential Flooding Depth

0 150 300 Feet

N W E S

5.2.17 STUDY AREA 17 – SE 28TH AVENUE SOUTH OF ATLANTIC BOULEVARD

This study area is located along SE 28th Avenue between SE 1st Court and SE 4th Street, which is immediately west of the Intracoastal Waterway. The study area is defined by sub-basins SE_009_01, SE_010_01, and SE_009_02 within the existing conditions stormwater model. Sub-basin SE_009_02 is interconnected with the remained of the study area via overland flow, which provides a significant amount of stormwater runoff into the lower lying problem areas next to the Intracoastal Waterway. The existing drainage system in this study area includes by two existing 21-inch RCP conduits connected to Node IN_2737 and Node IN_2741, which discharge directly to the Intracoastal Waterway. According the existing conditions stormwater model, the flood depth within the study area is estimated to be approximately 0.8 feet and 1.0 feet above the lowest roadway elevations within each sub-basin. The existing flooding problems within the study area are created by the following conditions:

- This study area has the lowest lying ground surface elevations located east of US-1 and south of Atlantic Boulevard. The study area is located immediately adjacent to a topographic ridge which causes significant overland flow of stormwater runoff into Sub-basin SE_009_02 along with the remainder of the study area. Based on the simulation of the 5-year, 24-hour design storm event with the existing conditions stormwater model, a peak overland flow of about 83 CFS is estimated to flow into Sub-basin SE_009_02.
- The size of the existing outfall pipe does not provide adequate discharge conveyance to efficiently drain rapidly accumulated stormwater runoff from the adjacent low-lying ground in the study area.
- Due to the direct connection to the Intracoastal Waterway, the performance of the existing stormwater system is tidally influenced, which will limit the discharge capacity of the existing outfalls during high tide conditions.

For the evaluation of this study area with the stormwater model, several potential system improvement alternatives were considered to improve the performance of the existing drainage system, including pipe size upgrades, exfiltration trenches, and drainage wells.

Alternative 1: Pipe Size Upgrades

According to the results of the existing conditions stormwater model results, pipe size improvements of the existing outfalls will provide additional discharge capacity which should alleviate some flooding within the study area. As mentioned previously, the existing conduits discharging to the Intracoastal Waterway are dual 21-inch RCP pipes from each Node IN_2737 and Node IN_2741. Alternative 1 was limited to replacing these two existing outfall pipes with a larger diameter. The estimated design and construction costs for this pipe size upgrade alternative are approximately $756,000.

Based on our analysis with the stormwater model, the estimated reduction of peak flood stages under Alternative 1 is summarized within Table 5.17.1 below. The options of increasing the pipe diameters of both existing outfalls up to 30-inch and 36-inch were analyzed at both locations with the stormwater model. The pipe size upgrade to a 30-inch diameter pipe at the outfalls provides a significant reduction in peak flood stages, which reduces the estimated flood depths within the study area. Under Alternative 1 with 30-pipe, the flooding depths were estimated to be 0.54 feet at Node IN_2737 and 0.69 feet at Node IN_2741. The pipe size upgrades to 36-inch in diameter at the outfalls did provide an additional reduction in peak flood stage, which indicates that there was still enough hydraulic head available between the flooding area and the water level within the Intracoastal Waterway. This 36-inch pipe upgrade alone reduces the estimated flood depth to 0.48 feet at Node IN_2737 and 0.12 feet at Node IN_2741. The hydraulics along the Atlantic Boulevard drainage system also benefits from these pipe upgrades as it does not receive as much of an inflow as it does in the existing conditions model.

Table 5.17.1 – Alternative 1 Peak Flood Stage Summary							
Nodes	**Existing Conditions**			**Alternative 1 (30-inch diameter)**		**Alternative 1 (36-inch diameter)**	
	Peak Stage (feet)	**Ground Elevation (feet, NAVD)[5]**	**Flood Depth (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**
IN_2737 [1]	3.37	2.58	0.79	3.12	-0.25	3.06	-0.31
IN_2741 [1]	3.42	2.45	0.97	3.14	-0.28	2.57	-0.85
IN_2677 [2]	5.11	4.45	0.66	5.11	0.00	5.11	0.00
MH_0440 [3]	3.34	3.32	0.02	2.59	-0.75	2.41	-0.93
IN_2774 [4]	2.97	4.32	0.00	2.97	0.00	2.97	0.00
MH_0439 [3]	2.86	3.40	0.00	2.46	-0.40	2.4	-0.46
MH_0425 [3]	4.17	6.65	0.00	3.66	-0.51	3.53	-0.64

[1] Critical model nodes
[2] Upstream node at sub-basin SE-009_02
[3] Node along Atlantic Boulevard drainage system
[4] Node south of project area along SE 28th Avenue
[5] Reference ground elevations corresponds to adjacent centerline road elevation

Based on our analysis with the stormwater model, the reduction of the estimated flooding duration within the study area under Alternative 1 with the 30-inch pipe upgraded is summarized within Table 5.17.2 below. Within the three critical model nodes (IN_2737, IN_2741 and IN_2677) the average flood duration was reduced from 8.47 hours in the existing condition to 6.43 hours in Alternative 1. Based on these results, Alternative 1 provides a significant reduction in flooding duration throughout the study area.

Table 5.17.2 – Alternative 1 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 1 (36-inch)**	**Reduction (%)**
IN_2737	2.58	4	1.2	70
IN_2741	2.45	3.4	0.1	97
IN_2677	4.45	18	18	0

Please note that localized flooding is exhibited at the intersection of SE 2nd Street and SE 23rd Avenue within Node IN_2677 with an estimated flood depth of 0.66 feet above the lowest roadway elevation. According to our analysis, the flooding lasts about 18 hours at this isolated location, which is a long time relative to adjacent areas. Due to a substantial amount of the stormwater runoff accumulated in the problem area flow in from this Node IN_2677, a potential solution to the flooding at this intersection may also benefit both flood depth and flood duration. Since Alternative 1 does not reduce the flooding within Node IN_2677, the additional alternatives will be analyzed to address the flooding conditions at this isolated problem area.

<u>Alternative 2: Exfiltration Trenches</u>

Under Alternative 2, the proposed construction includes the installation of exfiltration trenches within Sub-basin SE_009_02 in order to minimize the major source of overland flow into the project area. These proposed exfiltration trenches are only proposed along public roadways in the study area with ground surface elevations greater than +5.0 feet NAVD to ensure adequate storage and infiltration capacity. According to the topographic data within the study area, the average ground surface elevation is

approximately +5.5 feet NAVD within the construction area. The estimated design and construction costs for this exfiltration trench alternative are approximately $499,000. Under Alternative 2, the proposed construction includes about 810 linear feet of exfiltration trench. With these parameters, the proposed exfiltration trenches were analyzed with the stormwater model to estimate the effectiveness in reducing the peak flood stage and flooding duration within the study area. Based on our analysis, the estimated reduction of peak flood stages under Alternative 2 are summarized within Table 5.17.3 below. The model results show that by minimizing the amount of overland flow entering this problem area from 83 CFS to 68 CFS has slightly reduced the estimated flooding depth in the problem area. This reduction of flooding depth in the problem area at Node IN_2677 is actually greater than the reduction under Alternative 1, which indicates that significant runoff accumulates at Node IN_2677 from outside the study area.

Table 5.17.3 – Alternative 2 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 2	
	Peak Stage (feet)	Ground Elevation (feet, NAVD)[5]	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2737 [1]	3.37	2.58	0.79	3.29	-0.08
IN_2741 [1]	3.42	2.45	0.97	3.31	-0.11
IN_2677 [2]	5.11	4.45	0.66	5.07	-0.04
MH_0440 [3]	3.34	3.32	0.02	3.23	-0.11
IN_2774 [4]	2.97	4.32	0.00	2.97	0.00
MH_0439 [3]	2.86	3.40	0.00	2.51	-0.35
MH_0425 [3]	4.17	6.65	0.00	3.84	-0.33

[1] Critical model nodes
[2] Upstream node at sub-basin SE-009_02
[3] Node along Atlantic Boulevard drainage system
[4] Node south of project area along SE 28th Avenue
[5] Reference ground elevations corresponds to adjacent centerline road elevation

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.17.4 below. Alternative 2 does not provide any flood reduction within the critical model nodes.

Table 5.17.4 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_2737	2.58	4	4	0
IN_2741	2.45	3.4	3.4	0
IN_2677	4.45	18	17.3	4

Based on the results of our analysis, the proposed exfiltration trenches under Alternative 2 did not reduce the duration of flooding within the study area, even at Node SE_009_02. Since neither the peak flood stage or the flooding duration within the study area is not significantly reduced under Alternative 2, the proposed exfiltration systems is not an effective solution at this study area. The use of these proposed exfiltration systems will have a better impact in reducing flood stages and flooding duration if implemented in conjunction with other improvements in the problem area.

Alternative 3: Drainage Wells

Under Alternative 3, the proposed construction includes the installation of three drainage wells within Sub-basin SE_009_02 in order to minimize the major source of overland flow into the project area. The three drainage wells are proposed in the vicinity of Node IN_2677. The estimated design and construction costs for this drainage well alternative are approximately $381,000. During our analysis with the stormwater model, these proposed drainage wells were assumed to have the following design conditions:

- Minimum separation of at least 150 feet between drainage wells.
- Maintain consistent hydraulic head due to interconnection of all drainage wells.
- Minimum hydraulic head of 1.5 feet to commence gravity inflow.

Based on our analysis with the stormwater model, the estimated reduction in peak flood stages under Alternative 3 is summarized in Table 5.17.5 below. These results indicate that the reduction in peak flood stage under Alternative 3 is slightly better than Alternative 2. The greater benefit of Alternative 3 is experienced within the nodes part of the Atlantic Boulevard drainage system, which is interconnected to Sub-basin SE_009_02.

Table 5.17.5 – Alternative 3 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 3	
	Peak Stage (feet)	Ground Elevation (feet, NAVD) [5]	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
IN_2737 [1]	3.37	2.58	0.79	3.25	-0.12
IN_2741 [1]	3.42	2.45	0.97	3.27	-0.15
IN_2677 [2]	5.11	4.45	0.66	5.05	-0.06
MH_0440 [3]	3.34	3.32	0.02	2.7	-0.64
IN_2774 [4]	2.97	4.32	0.00	2.97	0.00
MH_0439 [3]	2.86	3.40	0.00	2.4	-0.46
MH_0425 [3]	4.17	6.65	0.00	3.6	-0.57

[1] Critical model nodes
[2] Upstream node at sub-basin SE-009_02
[3] Node along Atlantic Boulevard drainage system
[4] Node south of project area along SE 28th Avenue
[5] Reference ground elevations corresponds to adjacent centerline road elevation

Based on our analysis with the stormwater model, the estimated reduction of flooding duration in the study area under Alternative 3 is summarized within Table 5.17.6 below. The proposed drainage wells in Sub-basin SE_009_02 moderately reduce the flooding duration in study area. The flooding duration at Node IN_2677 in Sub-basin SE_009_02 is estimated to be reduced by 72% by reducing from 18 hours under the existing conditions to 5.1 hours under Alternative 3. Although a greater number of proposed drainage wells may have a proportional impact on reducing the flooding within the flooding area, it was not considered since the roadway flooding in Sub-basin SE_009_02 is a localized problem.

Table 5.17.6 – Alternative 3 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 3**	**Reduction (%)**
IN_2737	2.58	4	3.5	13
IN_2741	2.45	3.4	3	12
IN_2677	4.45	18	5.1	72

<u>Alternative 4: Pipe Size Upgrades and Drainage Wells</u>

Alternative 4 merges the proposed improvements from Alternative 2 and Alternative 3 in order to obtain the benefits from each option. Alternative 4 was evaluated since it minimizes overland flow into the study area from adjacent sub-basins along with increasing the discharge capacity via the outfalls. Based on our previous analysis, the proposed drainage wells were found to provide a significant reduction in flood duration and moderately reduction in the amount of runoff flowing from Sub-basin SE_009_02 into the problem area at Sub-basin SE_009_01. Therefore, the proposed pipe size upgrades and drainage wells were incorporated into the stormwater model to estimate the effectiveness of Alternative 4. Based on our analysis with the stormwater model, the estimated reduction in peak flood stages under Alternative 4 are summarized within Table 5.17.7 below. Under Alternative 4, the reduction of peak flood stages is significant for Node IN_2741 where flooding depth is no longer predicted at this node while the flood depth for Node IN_2737 is estimated to be reduced to 0.34 feet from 0.79 feet under the existing conditions. The estimated design and construction costs for this alternative are approximately $1,130,000.

Table 5.17.7 – Alternative 4 Peak Flood Stage Summary					
Nodes	**Existing Conditions**			**Alternative 4**	
	Peak Stage (feet)	**Ground Elevation (feet, NAVD)[5]**	**Flood Depth (feet)**	**Peak Stage (feet)**	**Peak Reduction (feet)**
IN_2737 [1]	3.37	2.58	0.79	2.92	-0.45
IN_2741 [1]	3.42	2.45	0.97	2.40	-1.02
IN_2677 [2]	5.11	4.45	0.66	5.05	-0.06
MH_0440 [3]	3.34	3.32	0.02	2.41	-0.93
IN_2774 [4]	2.97	4.32	0.00	2.97	0.00
MH_0439 [3]	2.86	3.40	0.00	2.40	-0.46
MH_0425 [3]	4.17	6.65	0.00	3.51	-0.66

[1] Critical model nodes
[2] Upstream node at sub-basin SE-009_02
[3] Node along Atlantic Boulevard drainage system
[4] Node south of project area along SE 28th Avenue
[5] Reference ground elevations corresponds to adjacent centerline road elevation

Based on our analysis with the stormwater model, the reduction of flooding duration for Alternative 4 is summarized in Table 5.17.8 below. Under Alternative 4, flooding is not expected at Node IN_2741 and the flood duration in Sub-basin SE_009_02 is significantly reduced to 5.1 hours.

Table 5.17.8 – Alternative 4 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 4**	**Reduction (%)**
IN_2737	2.58	4	1.1	73
IN_2741	2.45	3.4	0	100
IN_2677	4.45	18	5.1	72

Alternative Comparison

Refer to the Table 5.17.9 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.17.9 refers to the average within the critical problem areas of the study area, which correspond to Node IN_2737, Node IN_2741, and Node IN_2677 within the stormwater model. Based on our analysis with the stormwater model, multiple system improvement alternatives (Alternative 1 and Alternative 4) can be considered to be an effective option for reducing the peak flood stages and reducing the expected flood duration within the study area. Alternative 4 is slightly more effective than Alternative 1 at reducing the peak flood stages and flood duration within the study area. Alternative 1 is significantly more cost effective than Alternative 4 due to the limited underground construction required. Neither of these alternatives provides enough additional flood protection to meet the level of service criteria for all public roadways within the study area. Although these alternatives do provide significant benefits which alleviate the flooding problems within the study area, both alternatives have been eliminated from consideration by the City since the two existing 21-inch outfall pipe were recently rehabilitated at a significant cost. The City prefers to not replace these outfall pipes since the service life has been significantly extended by the pipe lining rehabilitation.

Table 5.17.9 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.39	2.03	$756,000
Alternative 2	0.08	0.23	$499,000
Alternative 3	0.11	4.6	$381,000
Alternative 4	0.51	6.4	$1,130,000

In order to provide additional flood relief to this study area, CMA recommends the installation of backflow prevention devices at the two existing 21-inch outfalls to prevent high tides from impacting flood control in low lying areas, such as SE 28th Avenue. All grass swale areas within the study area should also be regraded to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

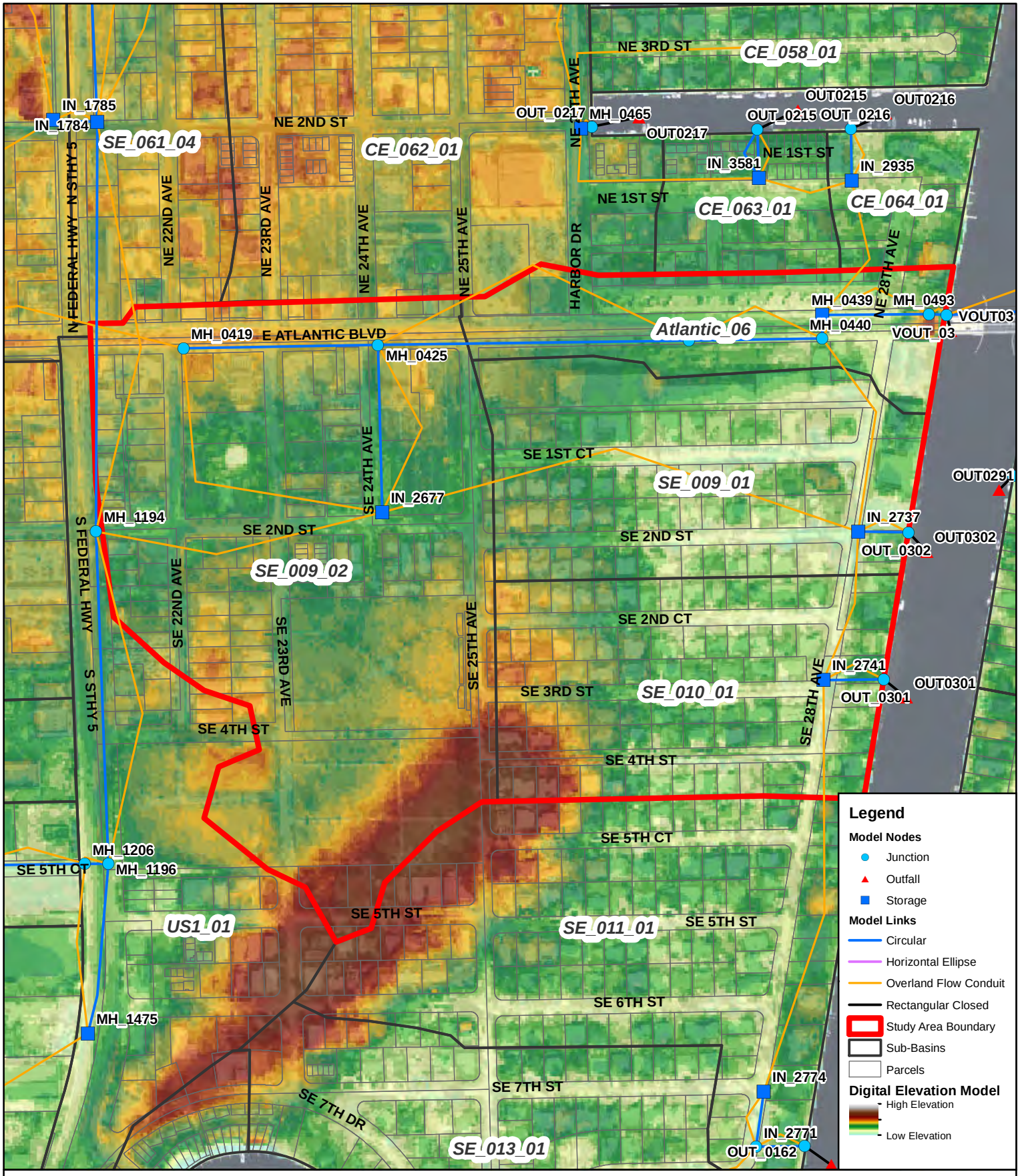


Figure 5-17A
SE 28th Avenue South of Atlantic Blvd
Existing Topography & Model Schematics

N
W E
S

0 200 400
Feet

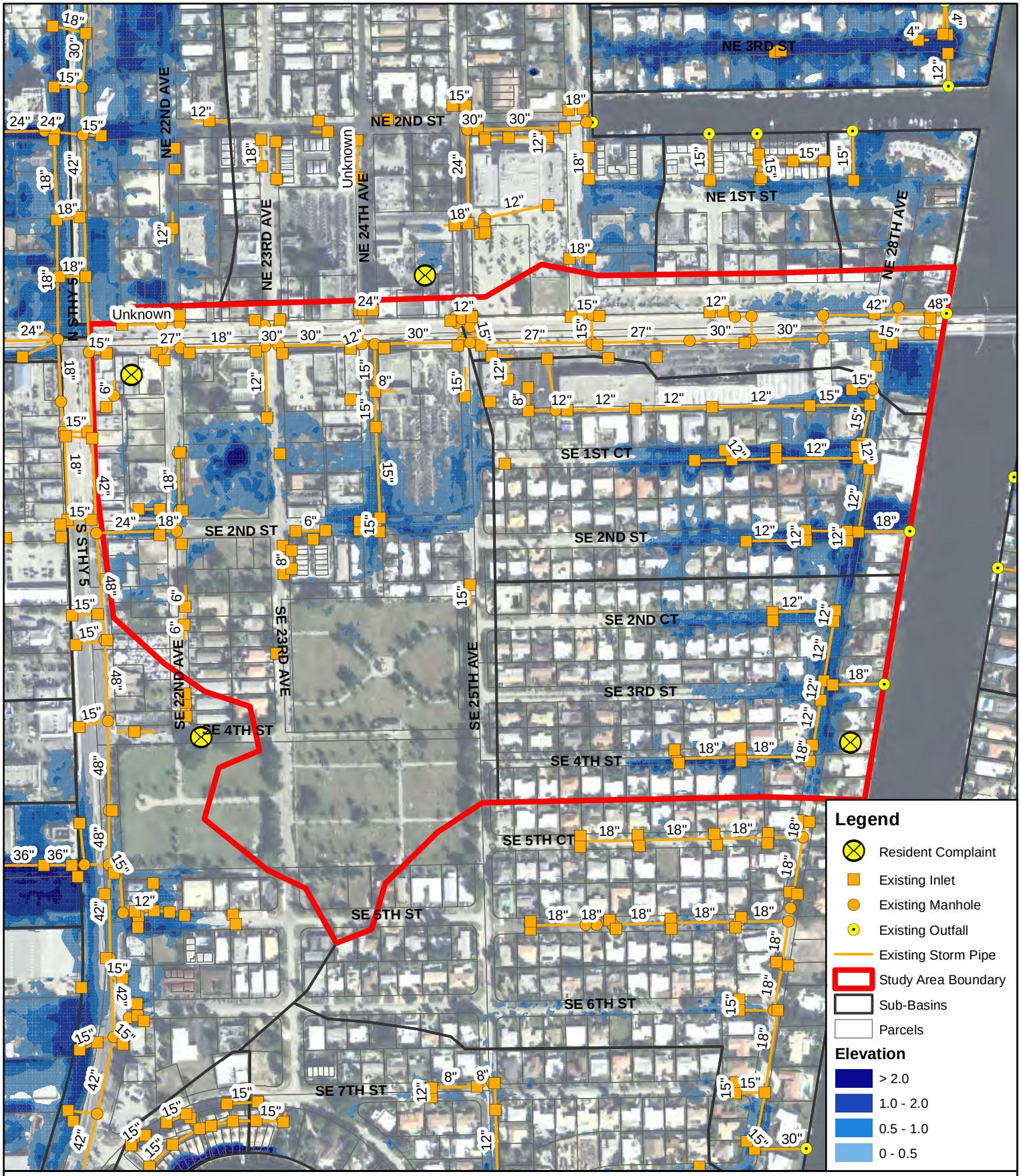


Figure 5-17B
SE 28th Avenue South of Atlantic Blvd
Potential Flooding Depth

N
W E
S

0 200 400
Feet

5.2.18 STUDY AREA 18 – NW 22ND COURT

This study area is an isolated right-of-way area with heavy flooding problems just south of Copans Road and just east of Powerline Road. This study area mainly consists of industrial and commercial properties, with only two City roadways (NW 22nd Court and NW 18th Avenue) with significant impervious ground coverage, which can limit the infiltration of stormwater runoff into the ground surface. The problem area is located along NW 22nd Court between NW 18th Avenue and NW 15th Avenue.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basin CW_042_04 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5.18A at the end of this section. According to the topography, the lowest elevations within the study area are located along NW 22nd Court where stormwater runoff collects from the entire study area. Based on the results of the stormwater model, the extent of the expected flooding within the study area is displayed on Figure 5-18B at the end of this section. According to the model results, all public right-of-way areas within the study area display flooding greater than two inches. The system improvement alternatives that were investigated for this study area consist of upsizing the existing pipes and installing additional exfiltration trench, which are summarized further below.

Alternative 1: Pipe Size Upgrades

The stormwater model was used to conduct several simulations of proposed pipe size upgrades at specific locations within the existing stormwater system. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. Alternative 1 included the replacement of the existing pipes which discharge into the adjacent drainage canal with larger diameter pipe. Under Alternative 1, the existing 18-inch pipe will be replaced with a 24-inch pipe along NW 18th Avenue, which includes a total pipe replacement of 400 linear feet. The estimated design and construction costs for this pipe size upgrades alternative are are approximately $138,000. Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 1 are summarized in Table 5.18.1 below. Alternative 1 results in a minimal reduction in peak flood stage within the study area. At the critical model node of the study area (Node MH_0006), the peak flood depth is reduced from 1.12 feet under the existing conditions to 1.09 feet under Alternative 1.

Table 5.18.1 – Alternative 1 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 1	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_0185	12.43	12.0	0.43	12.40	-0.03
MH_0006	12.42	11.3	1.12	12.39	-0.03
IN_0164	12.42	11.7	0.72	12.38	-0.04
MH_0007	12.20	11.7	0.5	12.17	-0.03
IN_0168	12.81	11.8	1.01	12.81	0.00
IN_0176	12.51	11.7	0.81	12.51	0.00
IN_0172	9.99	11.4	N/A	10.37	N/A
WCD3_03	9.84	N/A	N/A	10.34	N/A

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized in Table 5.18.2 below. According to the model results, the average flood duration within the study area is estimated to be reduced by about 49% from the existing conditions under Alternative 1. At the critical model node of the study area (Node MH_0006), the flood duration is reduced from 21.5 hours under the existing conditions to 18.0 hours under Alternative 1.

Table 5.18.2 – Alternative 1 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
MH_0006	11.30	21.5	18.0	16
IN_0164	11.70	13.8	7.1	49

Alternative 2: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of additional exfiltration trench within the study area in locations not currently served by an existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 2, the proposed construction includes a total of 3,843 LF of exfiltration trench along NW 22nd Court and NW 23rd Street. The installation of additional exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,470,000.

Table 5.18.3 – Alternative 2 Proposed Exfiltration Trench Summary

Sub-Basin	Exfiltration Trench (LF)	Mean Ground Surface Elevation (feet NAVD)
CW_042_05	1,919	+12.0
CW_042_04	1,564	+12.5
CW_043_01	360	+12.5
Total	**3,843**	**+12.33**

CMA conducted an analysis with the stormwater model of Alternative 2 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 2, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.18.4 on the following page. The model results show minimal to no peak reductions in peak flood stages throughout the study area from implementing this alternative. At the critical model node of the study area (Node MH_0006), the peak flood depth is reduced from 1.12 feet under the existing conditions to 1.07 feet under Alternative 2.

Table 5.18.4 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_0185	12.43	12.0	0.43	12.46	+0.03
MH_0006	12.42	11.3	1.12	12.37	-0.05
IN_0164	12.42	11.7	0.72	12.36	-0.06
MH_0007	12.20	11.7	0.5	12.17	-0.03
IN_0168	12.81	11.8	1.01	12.81	0.00
IN_0176	12.51	11.7	0.81	12.51	0.00
IN_0172	9.99	11.4	N/A	10.35	N/A
WCD3_03	9.84	N/A	N/A	10.35	N/A

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.18.5 below. The model results for Alternative 2 show an average reduction in flood duration throughout the study area of approximately 59%, which is significantly better than Alternative 1. At the critical model node of the study area (Node MH_0006), the flood duration is reduced from 21.5 hours under the existing conditions to 8.6 hours under Alternative 2.

Table 5.18.5 – Alternative 2 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
MH_0006	11.30	21.5	8.6	60
IN_0164	11.70	13.8	6.0	57

Alternative Comparison

Refer to the Table 5.18.6 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood duration results within Table 5.18.6 refer to the critical problem area in the study area, which corresponds to Node MH_0006 within the stormwater model. Based on our analysis with the stormwater model, Alternative 2 is slightly more effective at reducing the peak flood stages and expected flood duration within the study area. Alternative 2 should be implemented for this study area since it provides the better potential flood control benefits to the study area. Although Alternative 2 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, it does help alleviate the flooding problems within the study area.

Table 5.18.6 – Alternative Comparison			
Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.03	3.5	$138,000
Alternative 2	0.05	12.9	$1,470,000

CMA recommends the installation of exfiltration trench along NW 22nd Court and NW 23rd Street to provide additional storage and infiltration capacity for stormwater runoff. As feasible, any grass swale

areas within the construction area should also be regraded to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, the proposed construction will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

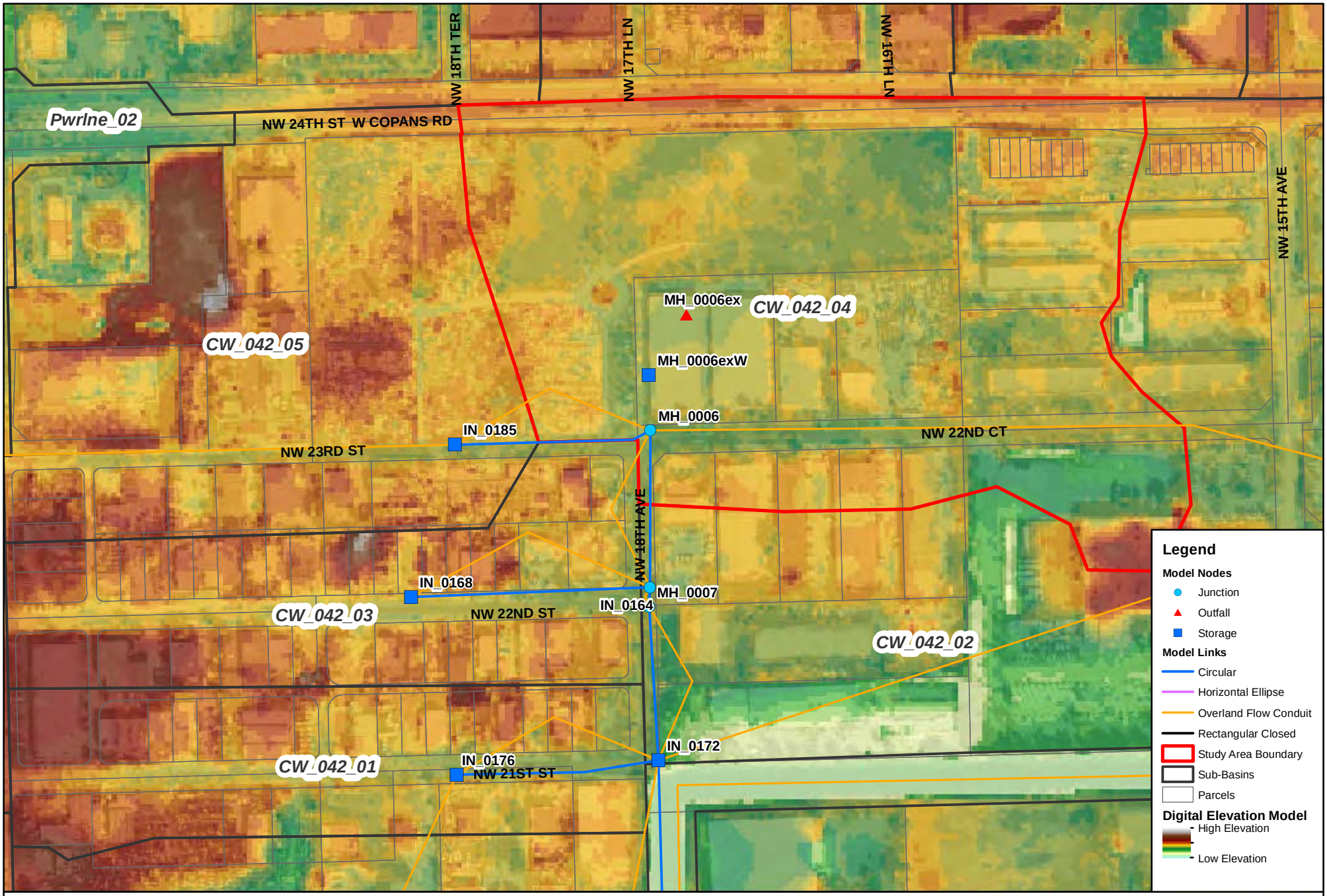


Figure 5-18A
NW 22nd Court
Existing Topography & Model Schematics

0 150 300 Feet

N W E S

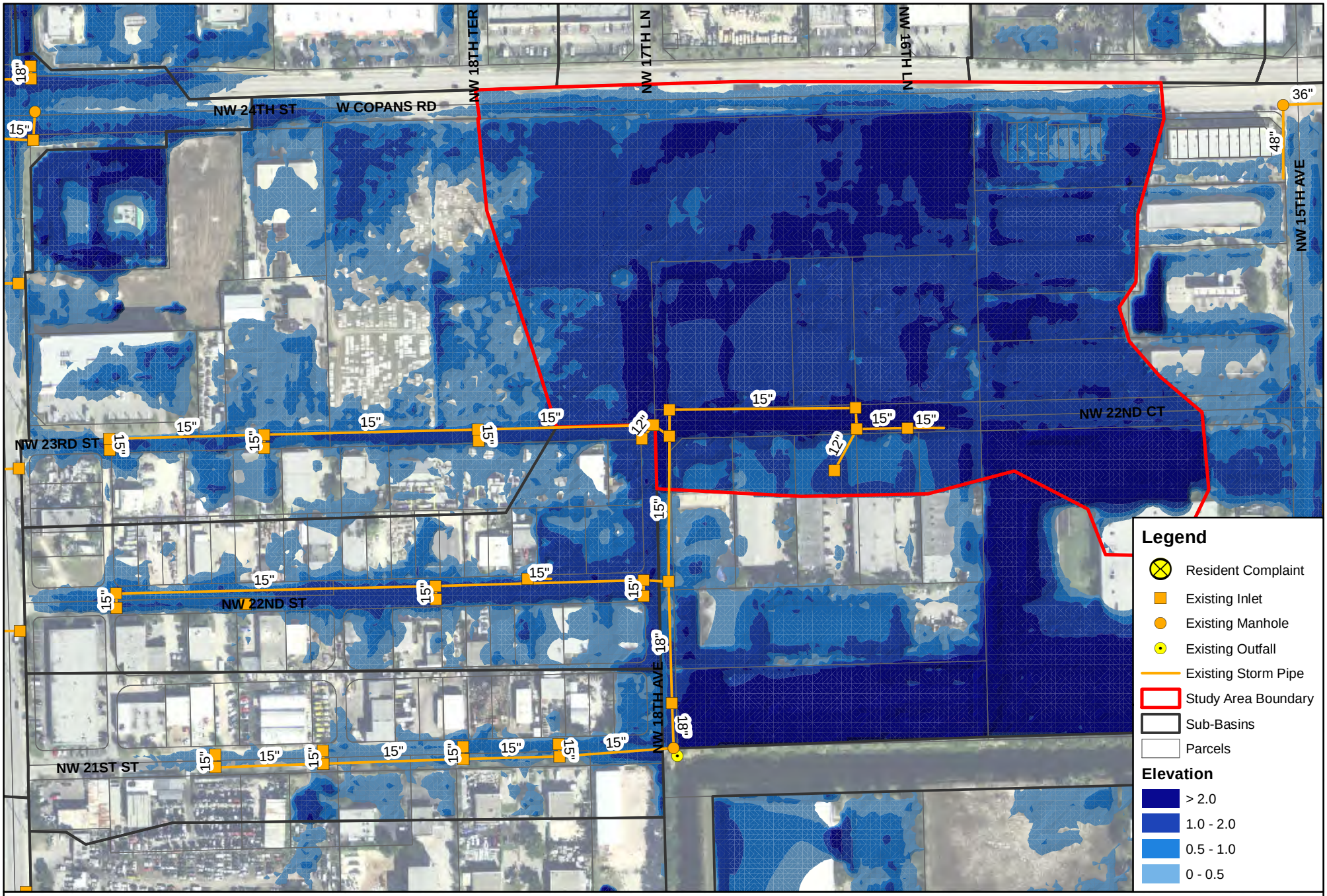


Figure 5-18B
NW 22nd Court
Potential Flooding Depth

0 150 300 Feet

N W E S

5.2.19 STUDY AREA 19 – NE 10TH STREET & DIXIE HIGHWAY

This study area is a single family residential neighborhood bounded by NE 10th Street to the north, by Dixie Highway to the west, by NE 6th Street to the south and by NE 5th Avenue to the east. According to the drainage atlas, there are no existing drainage facilities located within this study area. The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basin CE_015_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-19A at the end of this section. According to the topography, the central portion of the study area is a lower elevation than the perimeter which leads to the collection of stormwater runoff in these low lying areas. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-19B at the end of this section. According to the model results, limited flooding can be expected within low lying areas of NE 9th Street, NE 7th Street, and NE 3rd Avenue.

The only feasible system improvement alternative for this study area includes the installation of exfiltration trench within the public right–of-way areas due to the relatively high elevation, which averages 16.0 feet NAVD. The higher elevation will allow for additional storage and infiltration capacity within the exfiltration system. Drainage wells were not considered since the saltwater intrusion barrier does not extend to this study area. The construction of new dry retention areas were also not analyzed due to the lack of available property within this study area. Due to the lack of existing drainage infrastructure within the study area, upsizing any existing drainage system either within or near the study area was also not an option.

Alternative 1: Exfiltration Trench - Option 1

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area, which are not currently served by an existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 2,685 LF of exfiltration trench within the study area. The installation of additional exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,193,000.

CMA conducted an analysis with the stormwater model of Alternative 2 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.19.1 below. According to the model results, the reduction in peak flood stage is estimated to be 0.19 feet under Alternative 1. At the critical model node (Node CE01501), the peak flood depth is reduced from 1.55 feet under the existing conditions to 1.36 feet under Alternative 1.

Table 5.19.1 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
CE01501	16.75	15.2	1.55	16.56	-0.19
Outside project area					
MH_0205	12.77	14.66	N/A	12.73	-0.04
MH_0204	11.75	14.86	N/A	11.73	-0.02
MH_0208	9.01	12.98	N/A	9.00	-0.01

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.19.2 below. According to the model results, the reduction in flooding duration under Alternative 2 was estimated to by 84% less than the existing condition. At the critical model node (Node CE01501), the flood duration reduced from 39.5 hours under the existing conditions to 6.2 hours under Alternative 1.

Table 5.19.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
CE01501	15.20	39.5	6.2	84

<u>Alternative 2: Exfiltration Trench - Option 2</u>

In an effort to increase the level of service provided under Alternative 1, Alternative 2 included an interconnection of the proposed exfiltration system with existing drainage systems located nearby. Alternative 2 includes the construction of 2,010 linear feet of exfiltration trench in addition to the 2,685 linear feet of exfiltration trench defined under Alternative 1. Alternative 2 will provide a connection to the existing stormwater system on NE 2nd Street. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,359,000.

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 2 is summarized within Table 5.19.3 below. According to the model results, the reduction in peak flood stage is slightly greater than Alternative 1 with reduction of 0.22 feet at the critical model node (Node CE01501).

Table 5.19.3 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
CE01501	16.75	15.2	1.55	16.53	-0.22
Outside project area					
MH_0205	12.77	14.66	N/A	12.74	-0.03
MH_0204	11.75	14.86	N/A	11.73	-0.02
MH_0208	9.01	12.98	N/A	9.00	-0.01

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.19.4 below. Based on the model results, the reduction in flood duration under Alternative 2 was estimated to be 88% less than the existing conditions, which is slightly better than Alternative 1. At the critical model node (Node CE01501), the flood duration reduced from 39.5 hours under the existing conditions to 4.9 hours under Alternative 2.

Table 5.19.4 – Alternative 2 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
CE01501	15.20	39.5	4.9	88

Alternative Comparison

Refer to Table 5.19.5 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.19.5 refer to the critical problem area of the study area, which corresponds to Node CE_015_01 within the stormwater model. Based on our analysis with the stormwater model, both system improvement alternatives can be considered to an effective option for reducing the peak flood stages and reducing the expected flood duration within the study area. Alternative 1 should be implemented for this study area since it provides similar flood control benefits as Alternative 2 but is more cost effective. Although Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 1 does provide significant benefits which alleviate the flooding problems within the study area.

Table 5.19.5 – Alternative Comparison

Alternative	Peak Flood Stage Reduction (feet)	Flood Duration Reduction (hours)	Implementation Costs ($)
Alternative 1	0.19	33.3	$1,193,000
Alternative 2	0.22	34.6	$1,359,000

CMA recommends the installation of exfiltration trench within City right-of-ways throughout the study area which provide additional storage and infiltration capacity for stormwater runoff. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along NE 3rd Avenue, NE 7th Street, NE 8th Street and NE 9th Street to collect stormwater runoff from these areas. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, the proposed construction will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

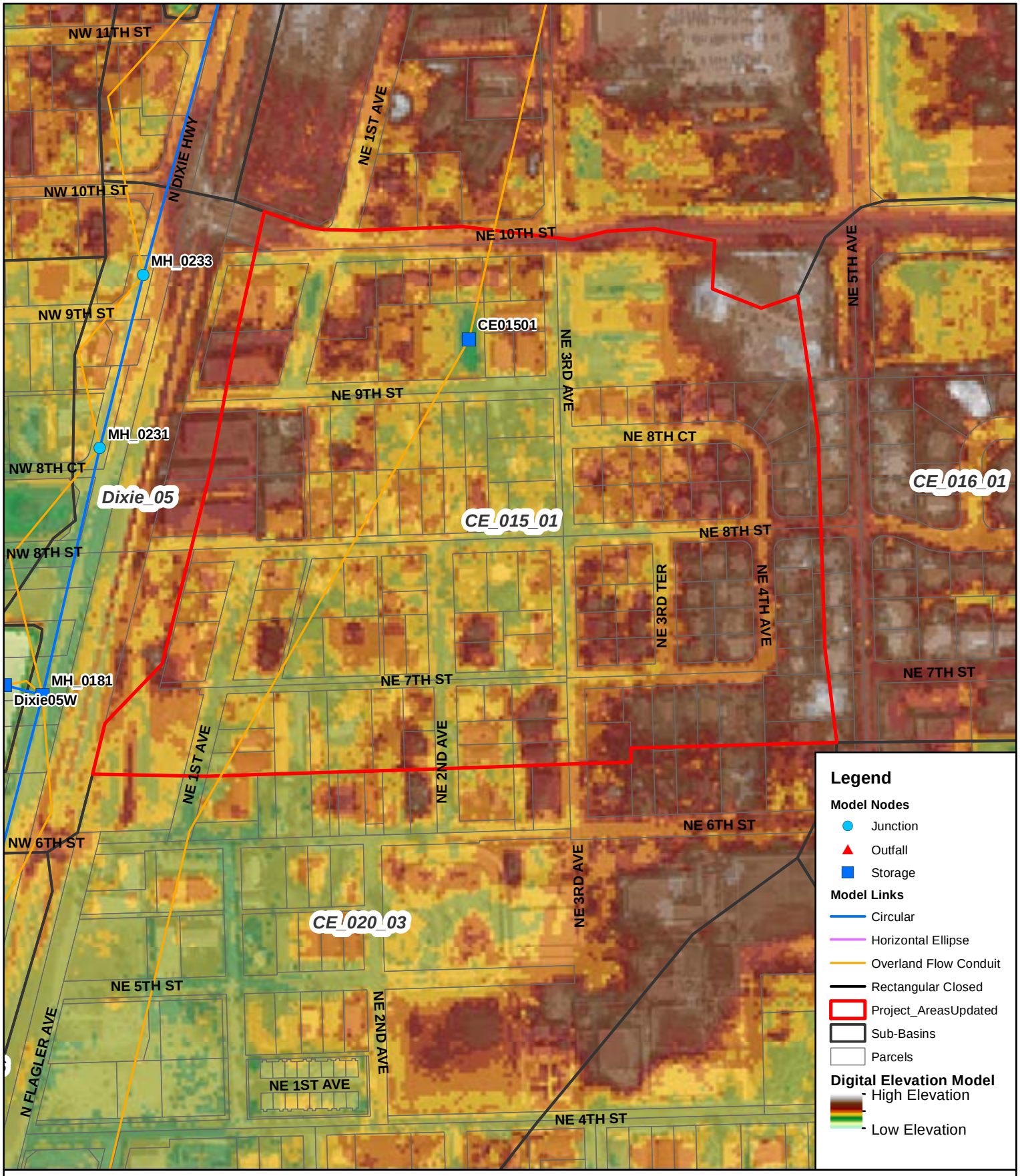


Figure 5-19A
NE 10th Street & Dixie Highway
Existing Topography & Model Schematics

N
W E
S

0 200 400
Feet

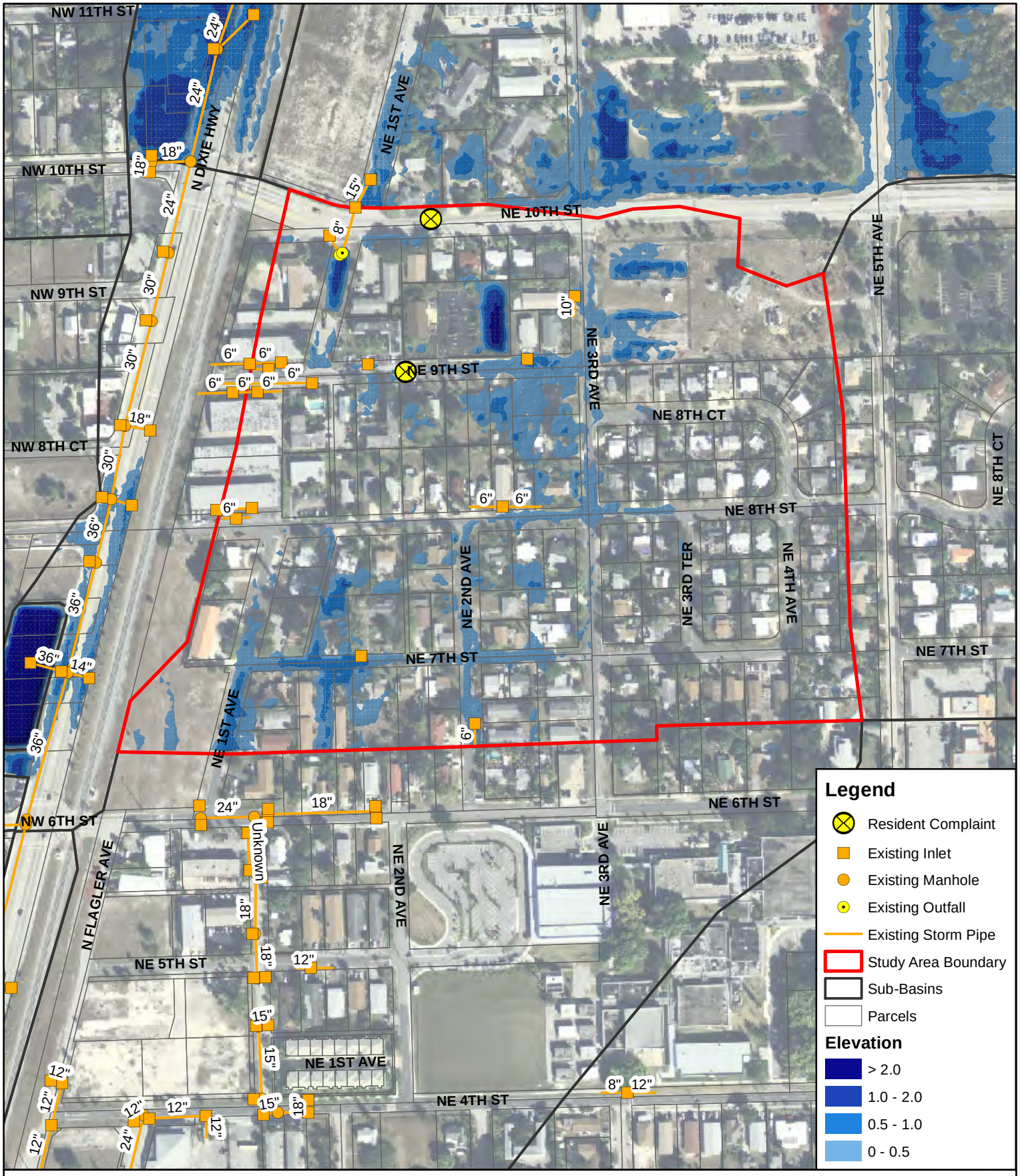


Figure 5-19B
NE 10th Street & Dixie Highway
Potential Flooding Depth

N
W E
S

0 200 400
Feet

5.2.20 STUDY AREA 20 – US-1 AND SE 15TH STREET

This study area is a residential neighborhood located between US-1 and the Intracoastal Waterway along SE 13th Street, SE 13th Court, SE 14th Street and SE 15th Street. The ground surface elevations within this study area are very low, which creates some the flooding problems. The existing stormwater system includes recently installed exfiltration system, approximately 7,000 linear feet, without a positive outfall into the Intracoastal Waterway. The storage and infiltration capacity of the existing exfiltration system is limited due to the low elevations within these right-of-way areas. However, there are some roadways in the study area, such as SE 23rd Avenue, SE 24th Avenue and SE 24th Terrace, which are without existing drainage facilities.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The topography of the study area along with the model schematics are displayed on Figure 5.20A at the end of this section. The study area is defined by the sub-basin SE_023_01 within the stormwater model. According to the topography, most roadways within the study area are consistently low. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5.20B at the end of this section. According to the results of the stormwater model, the estimated flooding depth is greater than 1 inch throughout the entire study area.

The only feasible system improvement alternative for this study area is expanding the exfiltration trench system within the public right-of-way areas which are not currently served by drainage infrastructure. The installation of a new positive outfall is not possible due to regulatory restrictions. The installation of the drainage well would not have an impact to the low ground surface elevation in relation to the water table depth, which would limit the discharge capacity. The system improvement alternative for this study area is summarized below.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of the proposed exfiltration trench within the study area, which is not currently served by an existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 1,730 LF of exfiltration trench along SE 23rd Avenue, SE 24th Avenue, and SE 24th Terrace. The installation of additional exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $1,889,000. CMA conducted an analysis with the stormwater model of Alternative 2 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.20.1 below. The results show a negligible reduction in peak flood stages from implementing this alternative. Based on the model results, Alternative 1 does not provide any significant reduction in peak flood stage. At the critical model node (Node SE02301), the peak flood depth is reduced from 0.71 feet under the existing conditions to 0.70 feet under Alternative 1.

Table 5.20.1 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
SE02301	4.21	3.5	0.71	4.20	-0.01

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.20.2 below. Alternative 1 does not reduce the expected flood duration within the study area at the critical model node (Node SE02301).

Table 5.20.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Street Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
SE02301	3.50	22.0	22.0	0

Alternative Comparison

Based on our analysis with the stormwater model, Alternative 1 provides basically no flood control benefit to the study area as a whole in regards to the reduction of both peak flood stage or flood duration. Alternative 1 will not provide any flood control benefit to the entire study area. Additional system improvement alternatives, such as drainage wells, pump stations, pipe size upgrades, and retention areas, were not feasible for this study area due to the low ground surface elevations and the lack of existing positive outfalls or existing drainage system to interconnect with. Due to these site conditions within the study areas, a system improvement alternative to be considered for this study areas would be the implementation of additional local improvements which provide additional storage volume for stormwater runoff, such as regraded swales or subsurface soil storage along the public roadways within the study area. These additional local improvements will only provide limited flood control benefits to the entire study area but will reduce the extent of localized flooding within the roadway areas.

CMA recommends the installation of exfiltration trench in targeted City right-of-ways which address isolated flooding problem locations within the study area without existing drainage infrastructure. The recommended stormwater improvements for this study area include the installation of new exfiltration trench along SE 23rd Avenue, SE 24th Avenue and SE 24th Terrace to address localized flooding problems in this area. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 2 will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed.

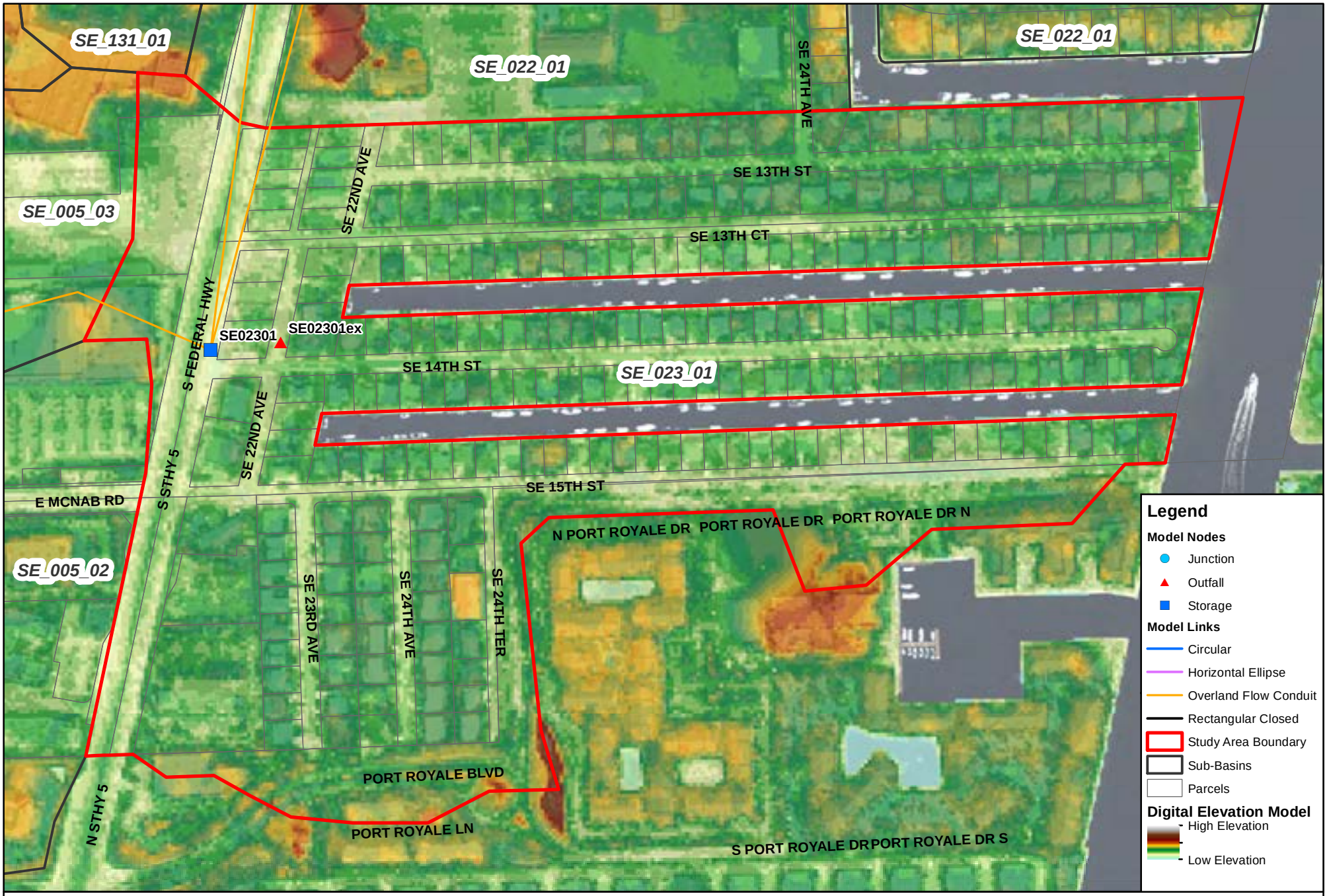


Figure 5-20A
US-1 & SE 15th Street
Existing Topography & Model Schematics

0 150 300 Feet

N W E S

Figure 5-20B
US-1 & SE 15th Street
Potential Flooding Depth

0 150 300 Feet

N W E S

5.2.21 STUDY AREA 21 – SE 9TH STREET

This study area is along SE 9th Street to the east of SE 22nd Avenue, which is surrounded by the Intracoastal Waterway. This residential neighborhood has a closed exfiltration trench system of 1,500 linear feet of 18-inch without a positive outfall at the eastern end of the right–of-way. There is also an isolated inlet structure at the western end of the right-of-way that discharges via a 15-inch outfall pipe into the Intracoastal Waterway. Due to a small ridge midway along SE 9th Street, these two stormwater systems are hydraulically isolated from each other, which can lead to flooding during heavy rainfall events. The performance of the existing outfall is also limited by tidal influences due to the very low elevation of the study area.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The topography of the study area along with the model schematics are displayed on Figure 5-21A at the end of this section. The project area is defined by the sub-basins SE_019_01 and SE_019_02 within the stormwater model. According to the topography, the entire right-of-way of SE 9th Street is very low with elevation ranging between +2.8 feet NAVD and +3.9 feet NAVD. Based on the results of the stormwater model, the extent of the expected flooding within the study area is displayed on Figure 5-21B at the end of this section. SE 9th Street experiences significant flooding of greater than 1 inch towards the east and west side of the study area in the low lying area of the right-of-way. The system improvement alternatives investigated within this study area include new pipe connections, additional exfiltration trench and a pump station.

Alternative 1: New Pipe Connections

Due to the isolation of the two separate systems in this study area, the purpose of Alternative 1 is to interconnect the existing stormwater system to equalize the flooding within the study area and to provide a more consistent discharge capacity throughout the study area. Under Alternative 1, the proposed construction includes the installation of new 18-inch pipe to connect the existing exfiltration trench system to the east with the existing 15-inch outfall to the west. The proposed construction also includes the installation of 18-inch pipe along SE 22nd Avenue to interconnect the existing outfalls. Alternative 1 includes the installation of approximately 2,400 linear feet of 18-inch RCP. The estimated design and construction costs for this new gravity pipe alternative are approximately $1,133,000. Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.21.1 below. According to the model results, peak flood stages actually rise slightly throughout the study area under Alternative 1 since the interconnections allow stormwater runoff to flow from higher areas to the west, except for the eastern end of SE 9th Street which has a slight reduction in peak flood stage at the critical model node (Node SE0902) of -0.04 feet.

Table 5.21.1 – Alternative 1 Peak Flood Stage Summary

	Existing Conditions			Alternative 1	
Nodes	**Peak Stage (feet, NAVD)**	**Ground Elevation (feet, NAVD)**	**Flood Depth (feet)**	**Peak Stage (feet, NAVD)**	**Peak Stage Reduction (feet)**
SE01902	4.17	3.2	1.02	4.13	-0.04
IN_2713	3.15	2.2	0.95	3.41	+0.26
IN_2706	3.09	3.0	0.09	3.40	+0.31
IN_2711	2.40	2.9	N/A	3.33	+0.93
IN_2640	4.20	2.8	1.4	4.10	-0.10
IN_2644	3.98	3.9	0.08	3.98	0.00

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.21.2 below. According to the model results, the expected flooding duration is reduced under Alternative 1 in the eastern end of SE 9th Street but has inconsistent impacts in other portions of the study area. At the critical model node (Node SE01902), the flood duration is reduced from over 40 hours to 17.3 hours under Alternative 1.

Table 5.21.2 – Alternative 1 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 1**	**Reduction (%)**
SE01902	3.15	> 40	17.3	-62
IN_2713	2.20	12.6	15.3	+21
IN_2706	3.00	0.25	5.9	+2240
IN_2640	2.80	31.7	27.9	-12
IN_2644	3.90	0.25	0.6	-120

Alternative 2: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area, which is not currently served by an existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 2, the proposed construction includes a total of 5,035 LF of exfiltration trench along roadways with ground surface elevations greater than +5.0 feet NAVD. SE 9th Street has lower ground surface elevations which cause additional exfiltration trench to be ineffective. The proposed exfiltration trench under Alternative 2 is located outside of the study area along SE 10th Street, SE 10th Court, SE 11th Street and SE 12th Street. The installation of these additional exfiltration systems will help intercept stormwater runoff before it reaches SE 9th Street and help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $2,063,000.

Table 5.21.3 – Alternative 2 Proposed Exfiltration Trench Summary		
Sub-Basin	**Exfiltration Trench (LF)**	**Mean Ground Surface Elevation (feet, NAVD)**
SE_021_01	1,195	+4.49
SE_022_01	3,840	+5.00
Total	**5,035**	**+4.75**

CMA conducted an analysis with the stormwater model of Alternative 2 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 2, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 2 is summarized within Table 5.21.4 below. Based on the model results, Alternative 2 results in a minimal

reduction in peak flood stages throughout the study area, except for the eastern end of SE 9th Street where there is no impact. At the critical model node (Node SE01902), the peak flood depth is unchanged from 1.02 feet under Alternative 2.

Table 5.21.4 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
SE01902	4.17	3.2	1.02	4.17	0.00
IN_2713	3.15	2.2	0.95	3.13	-0.02
IN_2706	3.09	3.0	0.09	3.05	-0.04
IN_2711	2.40	2.9	N/A	2.40	N/A
IN_2640	4.20	2.8	1.4	4.14	-0.06
IN_2644	3.98	3.9	0.08	3.81	-0.17

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.21.5 below. Although the expected flooding duration is significantly reduced under Alternative 2 throughout most of the study area, there is no impact within the eastern portion of SE 9th Street. At the critical model node (Node SE01902), the flood duration is unchanged from over 40 hours under Alternative 2.

Table 5.21.5 – Alternative 2 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
SE01902	3.15	> 40	>40	0
IN_2713	2.20	12.6	10.5	17
IN_2706	3.00	0.25	0	100
IN_2640	2.80	31.7	21.3	33
IN_2644	3.90	0.25	0.0	100

Alternative 3: Pump Station

The stormwater model was used to conduct several simulations of various proposed pump stations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. The proposed construction under Alternative 3 includes the installation of one pump station at the western end of SE 9th Street. Alternative 3 includes the installation of a new discharge pipe from the pump station to the west along SE 21st Terrace into the outfall. The estimated design and construction costs for this pump station alternative are approximately $1,515,000. The components associated to the pump station are listed below.

- Install a new 18-inch discharge pipe from pump station to the existing outfall.
- Install new flap gates at existing outfalls for backflow prevention.
- Wet well with a total footprint of about 150 square feet and depth of 8 feet.
- Maximum pump capacity of xx CFS, which is equivalent to the peak discharge of the existing drainage system during low tide conditions.

* Connection to existing exfiltration system to the east

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 3 are summarized in Table 5.21.6 below. According to the stormwater model, Alternative 3 results in reductions in the peak flood stage in the western portions of the study area but has minimal impact on the eastern end of SE 9th Street. At the critical model node (Node SE01902), the peak flood depth is reduced from 1.02 feet under the existing conditions to 0.87 feet under Alternative 3.

Table 5.21.6 – Alternative 3 Peak Flood Stage Summary

Nodes	Existing Conditions			Alternative 3	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD) [1]	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
SE01902	4.17	3.2	1.02	4.02	-0.15
IN_2713	3.15	2.2	0.95	2.77	-0.38
IN_2706	3.09	3.0	0.09	2.40	-0.69
IN_2711	2.40	2.9	N/A	3.19	N/A
IN_2640	4.20	2.8	1.4	4.20	0.00
IN_2644	3.98	3.9	0.08	3.98	0.00

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 3 is summarized in Table 5.21.7 below. According to the stormwater model, Alternative 3 results in minimal reduction in flooding duration, which is limited to western areas of the study area. There is no impact on the eastern end of SE 9th Street. At the critical model node (Node SE01902), the flood duration is unchanged from over 40 hours under Alternative 3.

Table 5.21.7 – Alternative 3 Flood Duration Summary

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
SE01902	3.15	> 40	>40	0
IN_2713	2.20	12.6	12.1	4
IN_2706	3.00	0.25	0	100
IN_2640	2.80	31.7	31.6	0
IN_2644	3.90	0.25	0.25	0

<u>Alternative Comparison</u>

Refer to the Table 5.21.8 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood duration results within Table 5.21.8 refer to the critical problem areas in the study area, which corresponds to Node SE01902 within the stormwater model. Based on our analysis with the stormwater model, all system improvement alternatives provide limited flood control benefits to the study area. Alternative 1 is slightly more effective at reducing the expected flood duration within the study area and is the only option which addresses the flooding within the eastern portion of SE 9th Street. Although Alternative 1 will not completely eliminate the flooding along SE 9th Street, it will reduce the duration of flooding along SE 9th Street. Additional local

improvements which provide additional storage volume for stormwater runoff should be considered along SE 9th Street, such as regraded swales or subsurface soil storage.

Table 5.21.8 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.04	22.7	$1,133,000
Alternative 2	0.00	0	$2,063,000
Alternative 3	0.15	0	$1,515,000

CMA recommends the installation of drainage pipe which will interconnect the existing closed exfiltration system on the eastern end of SE 9th Street with the existing 15-inch outfall at the western end of SE 9th Street. The proposed construction will allow the existing outfall to draw down the flooding within the lower eastern portion of SE 9th Street, which is hydraulically isolated under the existing conditions. Due to the very low ground surface elevations along SE 9th Street, backflow prevention devices will also need to be installed on the existing outfall to reduce the impact of high tide on flooding. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, the proposed construction will encounter various constructability concerns related to potential utility conflicts with other underground utilities within the public right-of-way area, which could reduce the extent of the exfiltration trench installed. These items will need to be evaluated in more detail during the design phase of the proposed project.

Figure 5-21A
SE 9th Street
Existing Topography & Model Schematics

0 150 300 Feet

N W E S

Figure 5-21B
SE 9th Street
Potential Flooding Depth

0 150 300 Feet

N S E W

5.2.22 STUDY AREA 22 – NW 16TH LANE

This study area is an isolated right-of-way area with heavy flooding problem just north of Copans Road between Powerline Road and Andrews Avenue. This study area mainly consists of industrial and commercial properties with only one City roadway (NW 16th Lane) with significant impervious ground coverage, which can limit the infiltration of stormwater runoff into the ground surface. As shown in the City Stormwater Atlas, there is existing drainage facilities located along NW 16th Lane according to City staff, which does not provide adequate flood protection to the right of way areas based on past observations during rainfall events. The topography of the study area along with the model schematics are displayed on Figure 5-22A at the end of this section. According to the topography, the lowest elevations within the study area are located along NW 16th Lane where stormwater runoff collects from the entire study area.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The study area is defined by the sub-basin NW_017_01 within the stormwater model. Based on the results of the stormwater model, the extent of the expected flooding within the study area is displayed on Figure 5-22B at the end of this section. According to the model results, the majority of NW 16th Lane displays flooding greater than two inches. Due to the lower ground surface elevations relative to the surrounding areas, NW 16th Lane has collects stormwater runoff which flows from surrounding private property.

The system improvement alternatives that were investigated for this study area consist of replacing the existing drainage infrastructure along NW 16th Lane with new additional exfiltration trench, which are summarized further below. The installation of a new closed exfiltration trench along NW 16th Lane should be effective at alleviating the flooding problems due to the high elevation in the project area of approximately 11.2 feet NAVD. The installation of drainage wells, new dry retention areas, stormwater pumps stations, or upsized outfall pipes were not feasible options for this study area are were not considered during the analysis of alternatives.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct a simulation of the installation of new exfiltration trench along NW 16th Lane which is not currently served by an existing stormwater system. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 910 LF of exfiltration trench along NW 16th Lane. The installation of new exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $436,000.

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.22.1 below. The model results show minimal to no reductions in peak flood stages under Alternative 1.

Table 5.22.1 – Alternative 1 Peak Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet)	Ground Elevation (feet)	Flood Depth (feet)	Peak Stage (feet)	Peak Reduction (feet)
NW01701	12.07	11.2	0.87	12.12	+0.05
NW01701W	11.43	N/A	N/A	11.47	-0.05

Based on our analysis with the stormwater model, the reduction in flood duration under Alternative 1 is summarized within Table 5.22.2 below. The model results show minimal to no reductions in flood duration under Alternative 1.

Table 5.22.2 – Alternative 1 Flood Duration Reduction				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
NW01701	11.36	> 40	> 40	0

<u>Alternative Comparison</u>

Although Alternative 1 does not provide any reduction in peak flood stage or flood duration within the study area, it will reduce extent of the existing flooding within the isolated right-of-way area of NW 16th Lane. Although Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for NW 16th Lane, it does help alleviate the nuisance flooding within the right-of-way area. According to the City Stormwater Atlas, there is existing drainage infrastructure located along NW 16th Lane, which is not performing adequately based on past observations by City staff. Prior to any drainage improvements along NW 16th Lane, the City should conduct a video inspection of all existing drainage pipe and structures to confirm whether any additional maintenance could alleviate the flooding issues. Based on the assumption that additional maintenance and/or repairs would not improve the flooding issues, CMA recommends the replacement the existing drainage infrastructure within the public right of way for NW 16th Lane with new additional exfiltration trench, which will provide additional storage and infiltration capacity for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2.

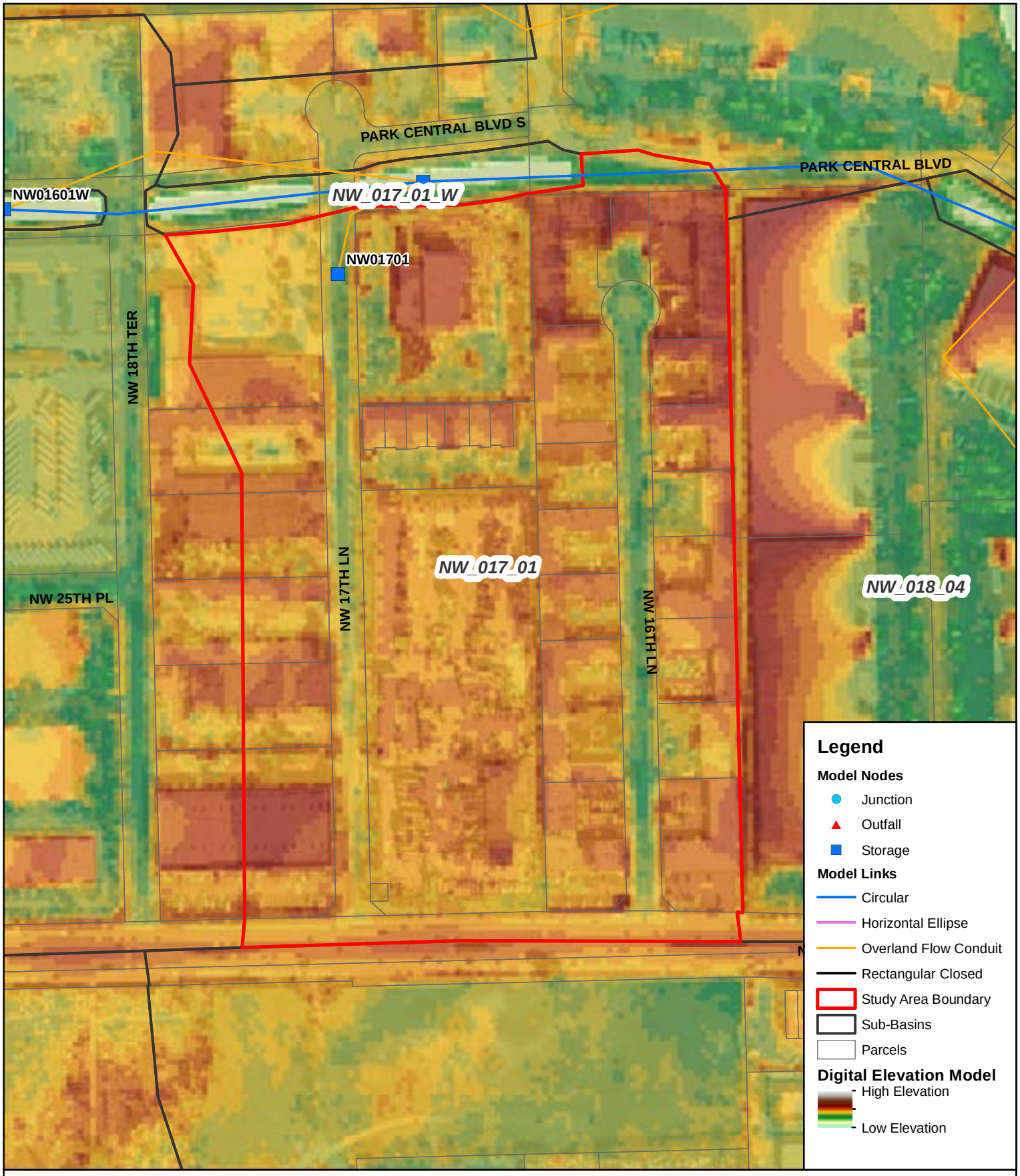


Figure 5-22A
NW 16th Lane
Existing Topography & Model Schematics

N
W E
S

0 150 300
Feet

Figure 5-22B
NW 16th Lane
Potential Flooding Depth

N
W E
S

0 150 300
Feet

5.2.23 STUDY AREA 23 – NORTHEAST MARTIN LUTHER KING BOULEVARD AND POWERLINE ROAD

This study area consists of a mixture of industrial and commercial properties, which is bounded by Martin Luther King Boulevard to the south, Powerline Road to the west, NW 16th Street to the north and NW 18th Avenue to the east. The City right of way areas within this study area is basically limited to NW 15th Street and NW 16th Street. The majority of the study area is private property. The existing City stormwater system within the study area is limited to two independent closed exfiltration systems along NW 16th Street and along NW 15th Street, which also include dry retention area within the right of way. There is also an existing stormwater system along North Powerline Road, which is not owned or maintained by the City of Pompano Beach.

The existing conditions stormwater model was used to evaluate the potential flooding within the study area during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. The topography of the study area along with the model schematics are displayed on Figure 5-23A at the end of this section. The study area is defined by the sub-basin CW_009_01 within the stormwater model. Based on the results of the existing conditions stormwater model, the extent of potential flooding within the study area is displayed on Figure 5-23B at the end of this section. According to the model results, NW 15th Street displays flooding depths between 0.5 to 1.0 feet. The high flood depths are expected in swale areas throughout the study area, which is the intention of the swale area. There is some expected flooding at the intersection of Powerline Road and NW 15th Street. As displayed on the flooding map, the worst flooding within this study area is located primarily within private property areas, which indicate inadequate stormwater facilities within these private properties.

A system improvement alternative is not recommended for this study area since flooding problems within the public right-of-way areas appear to be localized based on our additional investigation. Due to space constraints within the right of way areas of NW 15th Street and NW 16th Street, the existing stormwater systems could not be modified to provide any additional flood protection to the study area. The bulk of the flooding problems appear to occur on various private properties within the study area and along Powerline Road, which is not the responsibility of the City of Pompano Beach. Any proposed stormwater improvement by the City within this study area should be limited to minor extension of the existing drainage system to address specific localized flooding that may occur. These minor improvements would not result in any reduction in peak flood stage or flood duration throughout the study area. The City should conduct some additional inspections of the existing City-owned infrastructure within this study area to ensure system maintenance is not needed to help alleviate any flooding within the study area.

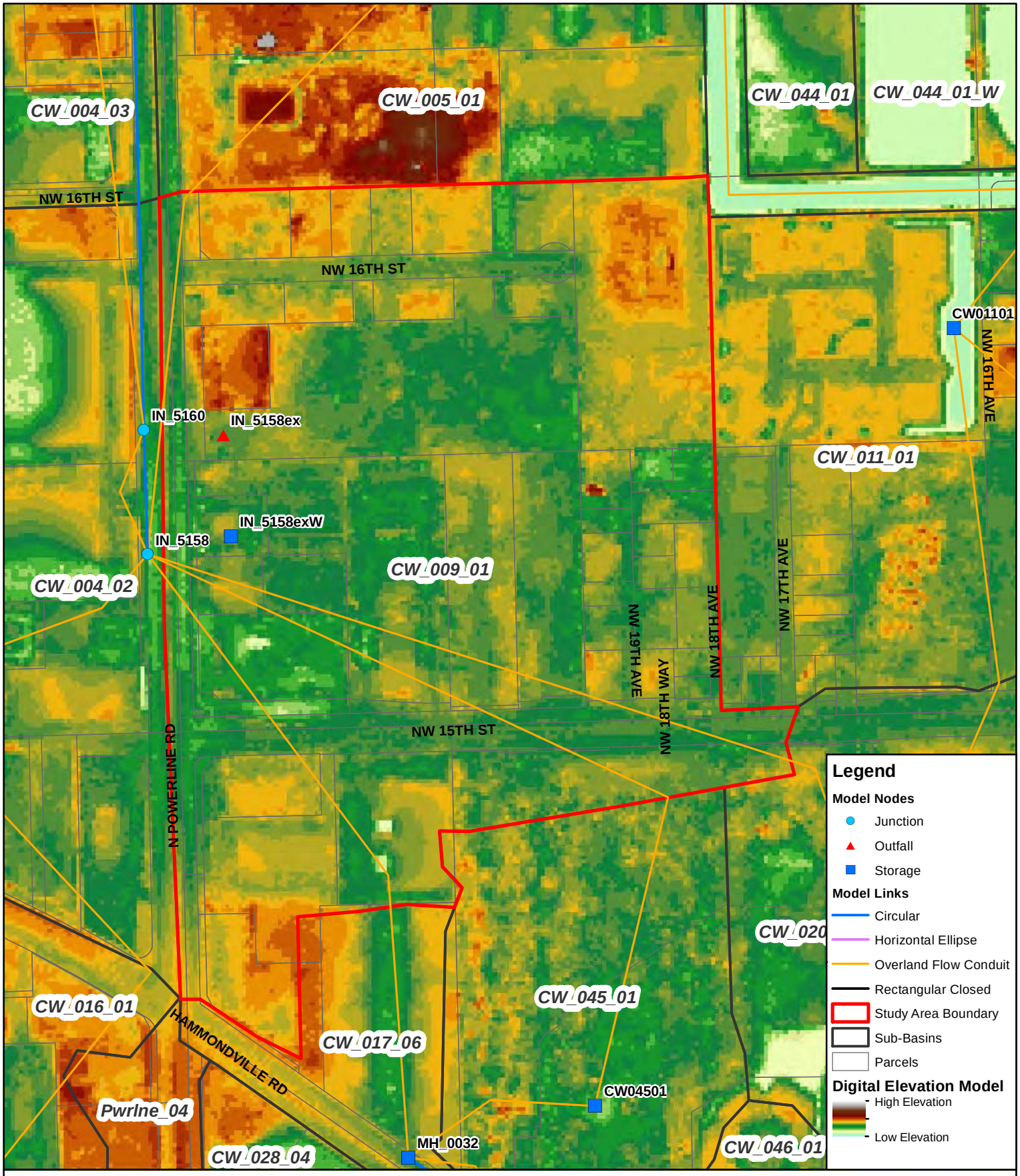


Figure 5-23A
Northeast MLK Boulevard and Powerline Road
Existing Topography & Model Schematics

N
W E
S

0 200 400
Feet

Figure 5-23B
Northeast MLK Boulevard and Powerline Road
Potential Flooding Depth

N
W E
S

0 200 400
Feet

5.2.24 STUDY AREA 24 – NW 7TH TERRACE

This study area is an isolated section of right-of-way along NW 7th Terrace and NW 7th Lane immediately east of I-95. This study area mainly consists of single family residential properties. The public right-of-way area for NW 7th Terrace and NW 7th Lane has an existing stormwater system which ranges from 15 inch to 24 inch pipe and discharges into a stormwater retention pond at north side of the study area. The study area is defined by the sub-basin CE_083_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-24A at the end of this section. According to the topography, the north side of the study area has a much lower elevation than the south side.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. Based on the results of the stormwater model, the extents of the expected flooding within the study area is displayed on Figure 5-24B at the end of this section. According to the model results, both NW 7th Terrace and NW 7th Lane display flooding towards the north side of the study area. The installation of drainage wells and stormwater pumps stations were not feasible options for this study area are were not considered during the analysis of alternatives. The system improvement alternatives that were investigated for this study area consist of expanding exfiltration trench system along NW 7th Terrace and NW 7th Lane and expanding the existing retention area, which are summarized further below.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct a simulation of the installation of additional exfiltration trench along NW 7th Terrace and NW 7th Lane within the study area. The purpose of this system improvement alternative is to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 2,165 LF of exfiltration trench, which will be interconnected with the existing drainage system. The installation of additional exfiltration systems within the study area will help draw down any flooding after rainfall events. The estimated design and construction costs for this exfiltration trench alternative are approximately $828,000.

CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.24.1 below. The model results show an average reduction in peak flood stage of 0.13 feet under Alternative 1. Node IN_1417 is the critical node for this study area which corresponds to the public right of way areas. The peak flood depth is reduced at Node IN_1417 from 1.03 feet under the existing conditions to 0.85 feet under Alternative 1, which is still relatively significant.

Table 5.24.1 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_1417	10.83	9.8	1.03	10.65	-0.18
IN_1419	10.61	9.8	0.81	10.49	-0.12

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.24.2 below. The model results for Alternative 1 show that the expected flood duration within the study area is reduced by about 53% from 7.3 hours under the existing conditions to 3.4 hours under the proposed improvements.

Table 5.24.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_1417	9.80	7.3	3.4	53
IN_1419	9.80	7.2	3.4	53

Alternative 2: Expand Retention Area

The stormwater model was used to conduct a simulation of expanding the existing retention area to the north of the study area. The purpose of this system improvement alternative is to provide additional storage to alleviate the existing flooding issues quicker. Under Alternative 2, the proposed construction includes a total of 0.34 AC of additional retention area, which is interconnected with the existing drainage system. The estimated design and construction costs for this retention alternative are approximately $93,000.

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 2 is summarized within Table 5.24.3 below. Node IN_1417 is the critical node for this study area which corresponds to the public right of way areas. The peak flood depth is reduced at Node IN_1417 from 1.03 feet under the existing conditions to 0.99 feet under Alternative 2.

Table 5.24.3 – Alternative 2 Peak Flood Stage Summary					
	Existing Conditions			Alternative 2	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_1417	10.83	9.8	1.03	10.79	-0.04
IN_1419	10.61	9.8	0.81	10.59	-0.02

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized within Table 5.24.4 below. The model results for Alternative 2 show that the expected flood duration within the study area is not changed.

Table 5.24.4 – Alternative 2 Flood Duration Summary				
Nodes	**Reference Roadway Elevation (feet, NAVD)**	**Flood Duration (hours)**		
		Existing Conditions	**Alternative 2**	**Reduction (%)**
IN_1417	9.80	7.3	7.3	0
IN_1419	9.80	7.2	7.2	0

Alternative Comparison

Alternative 1 provides a limited reduction in peak flood stage within the study area, but does significantly reduce the flood duration within the study area. Alternative 1 does not provide enough additional flood protection to meet the level of service criteria for public roadways within this study area. Due to the presence of existing stormwater pipe along with other underground utilities within the right-of-way, the installation of parallel exfiltration trench within the right of way would be difficult to implement due to likely conflicts with other underground utilities. Under Alternative 2, the expansion of the existing retention area will provide some additional benefits which will alleviate the existing flooding problems within the study area. Alternative 2 should be implemented to provide additional flood control benefits to the right of way areas of NW 7 Terrace and NW 7th Lane.

Table 5.24.5 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.18	3.9	$828,000
Alternative 2	0.14	0	$93,000

CMA recommends the expansion of the existing retention area in the northwest corner of the study area, which will provide additional storage capacity for stormwater runoff. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1, and a preliminary cost estimate, which is enclosed within Appendix A-2.

Figure 5-24A
NW 7th Terrace
Existing Topography & Model Schematics

N
W E
S

0 100 200
Feet

Figure 5-24B
NW 7th Terrace
Potential Flooding Depth

N
W E
S

0 100 200
Feet

5.2.25 STUDY AREA 25 – SE 15TH AVENUE

This study area is a single family residential neighborhood located immediately south of East Atlantic Boulevard along SE 15th Street. The existing drainage system within the study consists of a small pipe network that collects stormwater runoff along SE 15th Avenue between SE 2nd Street and SE 3rd Street and discharges via an existing 18-inch outfall pipe between SE 14th Avenue and SE 15th Avenue into a tidal canal. The study area is defined by the sub-basin SE_060_01 within the stormwater model. The topography of the study area along with the model schematics are displayed on Figure 5-25A at the end of this section. According to the topography, stormwater runoff typically flows south along SE 15th Avenue from a higher elevation toward SE 3rd Street, which has the lowest elevation in the study area.

The existing conditions stormwater model was used to evaluate the flooding under the existing conditions during a 5-year, 24-hour design storm event with 7.8 inches of rainfall. Based on the results of the stormwater model, the extent of the expected flooding within the study area is displayed on Figure 5-25B at the end of this section. According to the results of the stormwater model, the flooding can be expected to occur at the southern half of study area where the ground surface elevations are the lowest. The roadways on the southern portion of the study area, specifically along SE 3rd Street, display an expected flooding depth between 0.5 and 1.0 feet.

Alternatives analyzed for this project area included exfiltration trenches and a pump station. Drainage wells were not analyzed due to the elevations and minimum head available within the low lying problem areas along SE 3rd Street. The construction of new dry retention areas were also not analyzed due to the lack of available property within this study area. According to the results of the existing condition model, both the flood depth and duration currently meet the level of service in large portions of the study area, especially within the northern half. Therefore, some of the drainage concerns may be due to issues with the existing system, where they may be damaged or clogged, causing additional flooding this is not shown in the existing conditions model.

Alternative 1: Exfiltration Trench

The stormwater model was used to conduct several simulations of the installation of proposed exfiltration trench within the study area, which are not currently served by an existing stormwater system on the north side where the ground elevation is greater than +5.0 feet NAVD. The purpose of this system improvement alternative is to intercept stormwater runoff before it flows south toward the problem areas and to provide additional storage and infiltration capacity to alleviate the existing flooding issues quicker. Under Alternative 1, the proposed construction includes a total of 1,120 LF of exfiltration trench along SE 1st Street and SE 15th Avenue. The estimated design and construction costs for this exfiltration trench alternative are approximately $556,000. CMA conducted an analysis with the stormwater model of Alternative 1 to estimate the maximum potential reduction in peak flood stage and flood duration due to an exfiltration trench system within this study area. During the evaluation of Alternative 1, the design parameters for the proposed exfiltration trench are assumed within the stormwater model to be as follows:

- Trench Width: 4 feet
- Trench Height: 4 feet
- Perforated pipe diameter: 18-inch
- Hydraulic conductivity: 9.8×10^{-4} CFS/ft^2-ft head

Based on our analysis with the stormwater model, the reduction in peak flood stage under Alternative 1 is summarized within Table 5.25.1 below. Under Alternative 1, the model results show a minimal change in peak flood stage with a peak reduction of -0.07 feet from the existing conditions. The critical model node

is Node IN_2482 where the peak flood depth was reduced from 0.34 feet under the existing conditions to 0.31 feet under Alternative 1.

Table 5.25.1 – Alternative 1 Peak Flood Stage Summary					
	Existing Conditions			Alternative 1	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_2482	2.94	2.6	0.34	2.91	-0.03
IN_2488	2.95	2.6	0.35	2.92	-0.03
IN_2489	2.89	2.6	0.29	2.82	-0.07
IN_2471	2.88	2.3	0.58	2.86	-0.02
IN_2485	2.95	2.2	0.75	2.94	-0.01
IN_2648	3.43	3.3	0.13	3.43	0.00
IN_2715	4.32	3.3	1.02	4.32	0.00

Based on our analysis with the stormwater model, the reduction in flooding duration under Alternative 1 is summarized within Table 5.25.2 below. Under Alternative 1, the model results show a moderate reduction in expected flooding duration within the study with a maximum reduction of 14% from the existing conditions. At the critical model node (Node IN_2482), the flood duration was reduced from 1.8 hours under the existing conditions to 1.5 hours under Alternative 1.

Table 5.25.2 – Alternative 1 Flood Duration Summary				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 1	Reduction (%)
IN_2482	2.60	1.8	1.5	14
IN_2485	2.20	3.6	3.6	1
IN_2488	2.60	1.5	1.3	13
IN_2489	2.60	1.2	1.0	14

Alternative 2: Pump Station

The stormwater model was used to conduct several simulations of various proposed pump station configurations within the study area. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker, especially within the low lying areas along SE 3rd Street. The proposed construction under Alternative 2 includes the installation of one pump station near the existing outfall from SE 3rd Street at model Node IN_2482. The estimated design and construction costs for this pump station alternative are approximately $1,295,000. The components associated to the pump station are listed below.

- Install a new 30-inch discharge pipe from pump station to outfall into Intracoastal Waterway.
- Install new flap gates at existing outfalls for backflow prevention.
- Wet well with a total footprint of about 150 square feet and depth of 8 feet.
- Maximum pump capacity of 36 CFS, which is equivalent to the peak discharge of the existing drainage system during low tide conditions.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 2 are summarized in Table 5.25.3 below. Under Alternative 2, the model results show a maximum reduction of 0.39 feet in peak flood stage from the existing conditions. The critical model node is Node IN_2482 where the peak flood depth was reduced from 0.34 feet under the existing conditions to 0.25 feet under Alternative 2.

Table 5.25.3 – Alternative 2 Peak Flood Stage Comparison

Nodes	Existing Conditions			Alternative 2	
	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_2482	2.94	2.6	0.34	2.85	-0.09
IN_2488	2.95	2.6	0.35	2.88	-0.07
IN_2489	2.89	2.6	0.29	2.50	-0.39
IN_2471	2.88	2.3	0.58	3.03	+0.15
IN_2485	2.95	2.2	0.75	2.91	-0.04
IN_2648	3.43	3.3	0.13	3.43	0.00
IN_2715	4.32	3.3	1.02	4.32	0.00

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 2 is summarized in Table 5.25.4 below. According to model results of Alternative 2, the flood duration is estimated to be significantly reduced within the study area, with a maximum reduction of 100% from the existing conditions. At the critical model node (Node IN_2482), the flood duration was reduced from 1.8 hours under the existing conditions to 0.5 hours under Alternative 2.

Table 5.25.4 – Alternative 2 Flood Duration Comparison

Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 2	Reduction (%)
IN_2482	2.60	1.8	0.5	71
IN_2485	2.20	3.6	1.3	64
IN_2488	2.60	1.5	0.4	72
IN_2489	2.60	1.2	0.0	100

Alternative 3: Pipe Size Upgrades

The stormwater model was used to conduct several simulations of proposed pipe size upgrades at specific locations within the existing stormwater system. The purpose of this system improvement alternative is to increase conveyance capacity of the stormwater management system to alleviate the existing flooding issues quicker. Alternative 3 includes the replacement of the existing outfall pipes from SE 3rd Street, which discharge into the adjacent drainage canal with larger diameter pipe. Under Alternative 3, the existing 12-inch and 15-inch pipe will all be replaced with 24-inch pipe, which includes a total pipe replacement of 480 linear feet. The estimated design and construction costs for this pipe size upgrades alternative are approximately $400,000.

Based on the results of our analysis with the stormwater model, the reduction in peak flood stages under Alternative 3 are summarized in Table 5.25.5 below. Alternative 3 results in a minimal reduction in peak

flood stage within the study area. At the critical model node of the study area (Node IN_2482), the peak flood depth is reduced from 2.94 feet under the existing conditions to 2.85 feet under Alternative 3.

Table 5.25.5 – Alternative 3 Peak Flood Stage Comparison					
	Existing Conditions			Alternative 3	
Nodes	Peak Stage (feet, NAVD)	Ground Elevation (feet, NAVD)	Flood Depth (feet)	Peak Stage (feet, NAVD)	Peak Stage Reduction (feet)
IN_2482	2.94	2.6	0.34	2.85	-0.09
IN_2488	2.95	2.6	0.35	2.87	-0.08
IN_2489	2.89	2.6	0.29	2.54	-0.35
IN_2471	2.88	2.3	0.58	2.40	-0.48
IN_2485	2.95	2.2	0.75	2.59	-0.36
IN_2648	3.43	3.3	0.13	3.37	-0.06
IN_2715	4.32	3.3	1.02	4.32	0.00

Based on the results of our analysis with the stormwater model, the reduction in flooding duration under Alternative 3 is summarized in Table 5.25.6 below. According to the model results, the average flood duration within the study area is estimated to be reduced by about 83% from the existing conditions under Alternative 3. At the critical model node of the study area (Node IN_2482), the flood duration is reduced from 1.8 hours under the existing conditions to 0.4 hours under Alternative 3.

Table 5.25.6 – Alternative 3 Flood Duration Comparison				
Nodes	Reference Roadway Elevation (feet, NAVD)	Flood Duration (hours)		
		Existing Conditions	Alternative 3	Reduction (%)
IN_2482	2.60	1.8	0.4	77
IN_2485	2.20	3.6	0.7	81
IN_2488	2.60	1.5	0.4	73
IN_2489	2.60	1.2	0.0	100

Alternative Comparison

Refer to Table 5.25.7 below for a comparison of the various system improvement alternatives for this study area. Please note the peak flood stage and flood reduction results within Table 5.25.7 refer to the critical problem area of the study area, which corresponds to Node IN_2482 within the stormwater model. Based on our analysis with the stormwater model, none of the system improvement alternatives provide significant flood control benefits to the study area. Alternative 3 should be investigated for implementation for this study area since it reduces the peak flood stage and flood duration more than the other alternatives. Although Alternative 3 does not provide enough additional flood protection to meet the level of service criteria for all public roadways within the study area, Alternative 3 does provide benefits which alleviate the flooding problems within the study area.

Table 5.25.7 – Alternative Comparison			
Alternative	**Peak Flood Stage Reduction (feet)**	**Flood Duration Reduction (hours)**	**Implementation Costs ($)**
Alternative 1	0.03	0.3	$556,000
Alternative 2	0.09	1.3	$1,295,000
Alternative 3	0.09	1.4	$405,000

CMA recommends the replacement of existing outfall pipes with larger diameter pipe, which will increase the discharge capacity into the adjacent canal during heavy rainfall events. The recommended stormwater improvements for this study area include the replacement of the existing outfall pipes from SE 3rd Street, which discharge into the adjacent drainage canal with larger diameter pipe. The swale areas should also be regraded throughout the study area to provide additional storage volume for stormwater runoff for flooding attenuation and water quality treatment purposes. For the recommended stormwater improvements for this study area, CMA has prepared a conceptual layout, which is enclosed within Appendix A-1 and a preliminary cost estimate, which is enclosed within Appendix A-2. During the detailed design phase, Alternative 3 will encounter some regulatory obstacles to the upsizing the existing outfall pipes related to minimum water quality requirements. In order to gain regulatory approval for the larger outfall pipes, the recommended improvements will need to include regraded swales and/or limited exfiltration trench to provide the minimum water quality treatment volume for the study area. These items will need to be evaluated in more detail during the design phase of the proposed project.

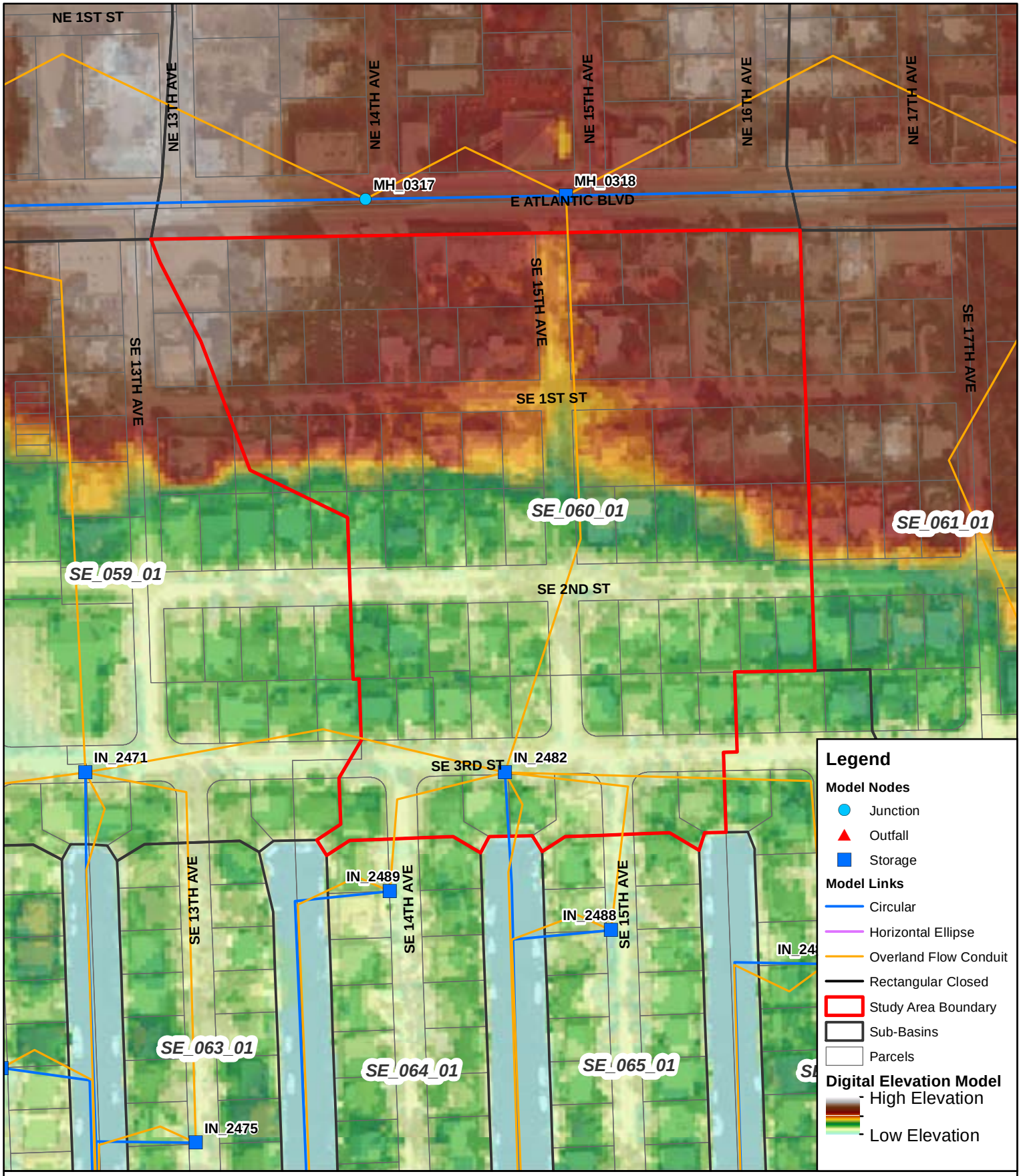


Figure 5-25A
SE 15th Avenue
Existing Topography & Model Schematics

N
W E
S

0 150 300
Feet

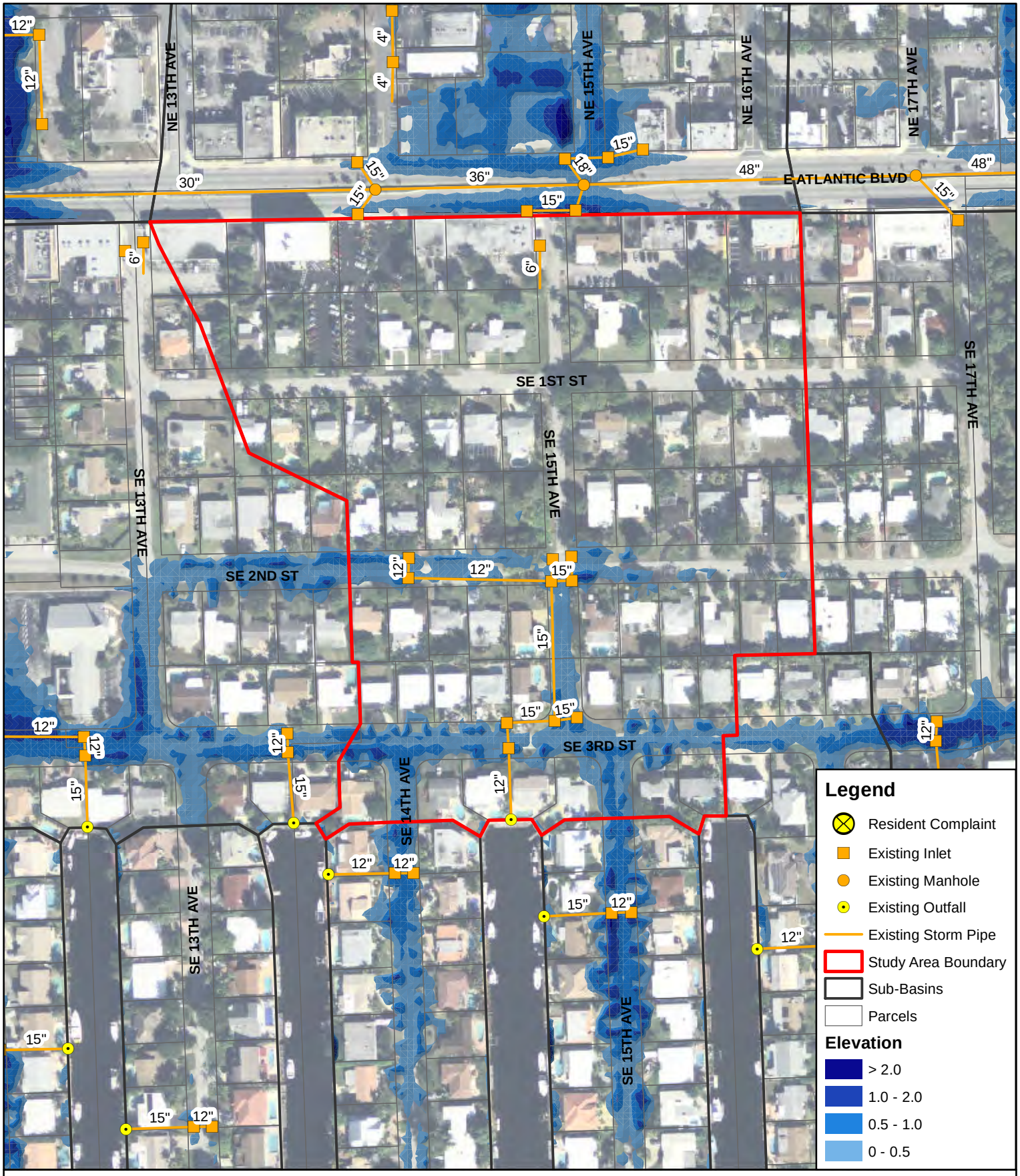


Figure 5-25B
SE 15th Avenue
Potential Flooding Depth

N
W E
S

0 150 300
Feet

SECTION 6 – CONCLUSION

6.1 RECOMMENDATIONS

CMA prepared this Stormwater Master Plan for the entire limits for the City of Pompano Beach. The purpose of this Stormwater Master Plan was to identify any deficiencies in the existing stormwater management system and to recommend system improvements to alleviate flooding issues throughout the City. Within the Stormwater Master Plan, CMA provided recommendations for improvements to the system that will reduce the ponding currently encountered within right-of-way areas during or after rainfall events. The recommendations include regional alternatives and local alternatives to address flooding issues within the City, which are defined below.

6.1.1 RECOMMENDED STORMWATER CAPITAL IMPROVEMENT PROJECTS (CIP)

Based on our analysis with the stormwater model, CMA has developed the recommended system improvement alternatives for each study area within the City of Pompano Beach. The recommended system improvement alternative for each study is defined in detail within Section 5 of this report. CMA has prepared a conceptual layout for the recommended system improvements within each study area along with a preliminary cost estimate for the implementation of each recommended system improvement. The conceptual layouts of the recommended system improvements within each study area are enclosed within Appendix A-1 of this report. The preliminary cost estimates of the recommended system improvements within each study area are enclosed within Appendix A-2 of this report. The estimated implementation costs for these recommended system improvements are summarized for each priority study area within the Table 6.1 – Recommended Stormwater CIP Cost Summary on the following page. The recommended system improvements are listed in the order of the study area prioritization, as determined by the basin prioritization formula.

Table 6.1 – Recommended Stormwater CIP Cost Summary		
No.	**Study Area**	**Cost**
1	Pompano Park Place & Andrews Avenue*	$396,000
2	Northwest CRA – TOC Area**	$1,982,000
3	Lyons Park Neighborhood	$2,434,000
4	Avondale Neighborhood	$2,488,000
5	Esquire Lake Neighborhood	$1,656,000
6	Gateway Drive	$3,524,000
7	Kendall Lake Neighborhood	$2,720,000
8	US-1 & NE 14th Street Causeway	$909,000
9	NE 4th Street & NE 3rd Street	$942,000
10	Dixie Highway & McNab Road	$52,000
11	Bay Drive Neighborhood	$1,210,000
12	North Riverside Drive & NE 14th Street Causeway	$980,000
13	Atlantic Boulevard & South Riverside Drive	$2,873,000
14	NE 27th Avenue & NE 16th Street	$2,572,000
15	Powerline Road & NW 33rd Street	$1,480,000
16	NW 22nd Street	$349,000
17	SE 28th Avenue South of Atlantic Boulevard	$585,000
18	NW 22nd Court	$1,012,000
19	NE 10th Street & Dixie Highway	$1,348,000
20	US-1 & SE 15th Street	$1,185,000
21	SE 9th Street	$377,000
22	NW 16th Lane	$436,000
23	NE MLK Boulevard & Powerline Road***	$0
24	NW 7th Terrace	$93,000
25	SE 15th Avenue	$394,000
	TOTAL	**$31,997,000**

Stormwater CIP Cost Notes

* Study Area #1 (Pompano Park Place & Andrews Avenue) – As defined in more detail within Section 5.2.1, the estimated costs for this study area are for recommended system improvements which are to alleviate the localized flooding problems in City right of way areas without existing drainage infrastructure. Any comprehensive basin-wide stormwater improvements which would meet level of service criteria throughout this study area are not currently feasible due to the lack of available property for stormwater storage.

** Study Area #2 (Northwest CRA – TOC Area) – As defined in more detail within Section 5.2.2, the estimated costs for this study area include stormwater improvements that would serve the existing conditions of the TOC area. However, CMA recommends a master stormwater management system which would serve the entire TOC Area under future development conditions defined by the CRA. Since these recommended improvements would not be necessary under the existing conditions and would be funded by the CRA, the future conditions estimated cost is not included in the total Stormwater CIP cost.

*** Study Area #23 (NE MLK Boulevard & Powerline Road) – As defined in more detail within Section 5.2.23, system improvements are not recommended for this study area since the bulk of the flooding problems are located on various private properties within the study area and along Powerline Road, which is not the responsibility of the City of Pompano Beach.

6.1.2 SWALE RESTORATION PROGRAM

Grass swale areas within the right-of-way are intended to collect stormwater runoff from adjacent roadways, to provide water quality treatment by filtering pollutants and sediments carried by the stormwater runoff, and to control flooding by providing stormwater storage volume and allow stormwater to infiltrate into the ground surface. The majority of residential properties within the City have grass swale areas located within the right-of-way areas adjacent to the roadways. These grass swales areas were intended to be the primary component of the stormwater management system. Although limited existing underground drainage facilities are typically found within these residential neighborhoods, the grass swale areas were supposed to provide storage capacity for stormwater runoff before it reaches the underground drainage facilities for eventual disposal via the existing outfalls.

Based on our observations of the existing conditions, the grass swale areas are typically overgrown and overbuilt throughout the City's residential neighborhoods, which causes ponding within roadway areas during rainfall events. This can happen if the swales were not originally constructed properly, if the resident modifies the swale by adding landscape features, or if sediment builds up within the swale over time. Typically, the center of the grass swale areas should be 6 inches deeper than the edge of pavement elevation. Since the existing swale areas within the City residential areas are not properly graded to provide adequate storage for stormwater runoff and to allow for infiltration into the ground surface, the stormwater management system does not meet level of service criteria for the roadways. The overbuilt swales can cause stormwater ponding within roadway areas in various ways. The overbuilt swale will block stormwater runoff from flowing off the roadway areas, which can create ponding within the travel lanes. The overbuilt swale can prevent stormwater runoff from reaching drainage inlets by trapping it in low lying areas, such as driveway approaches. The overbuilt swale can also cause stormwater runoff to collect initially over impervious surfaces, which prevents any infiltration into the ground surface.

The regrading of the grass swales will increase the depth which provides additional storage volume for stormwater runoff. Since the regraded grass swale areas will now be at a lower elevation than the adjacent roadway, stormwater runoff will flow from the roadway and accumulate in the swale areas to infiltrate into the ground surface. Based on the soil conditions within the City, the stormwater runoff from these residential roadways can be expected to infiltrate into the ground surface within 6 hours if the swale area is properly graded. Properly graded grass swale areas will reduce the flooding of roadway areas by providing additional storage capacity and reducing the total discharge from the existing outfalls by allowing for infiltration into the ground surface. Grass swale areas are the most cost effective method of reducing flooding within the roadway areas of residential neighborhoods when compared to exfiltration trench, which is the other option within South Florida. For comparison purposes, the advantages and disadvantages of grass swale areas and exfiltration trench are listed within Table 6.2 below.

Table 6.2 – Effectiveness Comparison (Swale Area vs. Exfiltration Trench)

	Grass Swale Areas	Exfiltration Trench
Advantages	• Limited capital costs • Additional green space • Limited maintenance requirements • Significant pollutant load reduction • Attenuation of stormwater flooding • Recharge groundwater	• Preservation of land for other uses • Easily incorporated for retrofit • Typical in South Florida region • Limited pollutant load reduction • Attenuation of stormwater flooding • Recharge groundwater
Disadvantages	• Standing water during wet periods • Objections from some residents • Potential alteration by property owners	• Limited longevity • High maintenance requirements • Potential clogging due to sediments
Requirements	• Surface soils with high permeability • Adequate open space	• Subsurface soils with high permeability • Deep water table

The City should implement an annual swale program to systematically regrade grass swale areas within the residential neighborhoods to address localized ponding issues by providing additional storage volume for stormwater runoff adjacent to the roadway. The Swale Restoration Program will need to rehabilitate swales throughout entire neighborhoods since restoration of isolated swales will not be sufficient to resolve flooding issues. In order for the Swale Restoration Program to be effective, all non-essential items will need to be removed from the public right-of-way to allow the existing swales to be excavated up to 6 inches deeper than the edge of pavement elevation. Upon completion of the grading operation, the swale area should be immediately re-sodded. Since it will not be feasible to implement the Swale Restoration Program across the entire City immediately, the City should plan on implementing the program on a basin by basin basis.

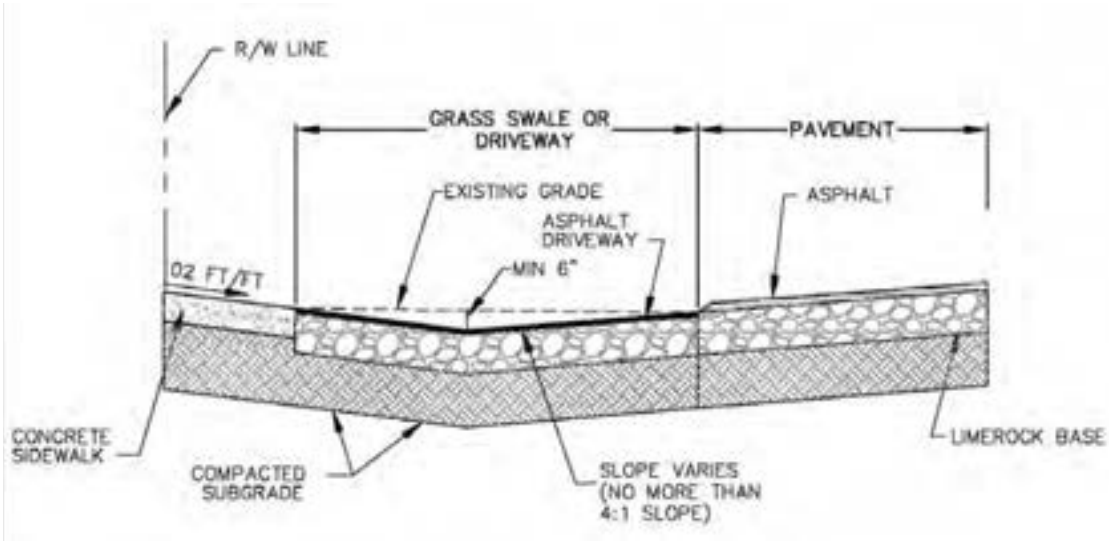


Figure 6-1 Typical Swale Restoration Detail

The City should expect some opposition to the Swale Restoration Program from residents since most view the grass swale areas within the right-of-way to be their private property, which is not the case. Prior to any implementation of the Swale Restoration Program, the City should develop a Community Outreach Program on the importance of the swale restoration program. The purpose of the Community Outreach Program is to provide educational information to residents on the objectives of the Swale Restoration Program along with the flood control and environmental benefits. The City should include an annual budget for a Swale Restoration Program, which would improve the performance of the stormwater management system on an incremental basis as existing swale areas are regraded neighborhood by neighborhood over the years. The City should also consider various implementation strategies for the Swale Restoration Program. Some other municipalities within South Florida implement their Swale Restoration Program using the following options:

- Voluntary: City will allow residents to volunteer for the Swale Restoration Program. City shall begin swale restoration along a roadway once sufficient percentage of residents along the roadway volunteer for Swale Restoration Program.
- Mandatory: City will provide residents an adequate amount of time to remove private landscaping and other items from right-of-way before beginning the restoration of the swale areas
- Mandatory: City will be responsible for the relocation of private landscaping and other items from the right-of-way to private property before beginning the restoration of the swale areas.

6.1.3 OPERATIONS AND MAINTENANCE

The existing stormwater management system requires a significant operations and maintenance effort by the City to ensure that it continues to operate at full capacity while complying with all regulatory requirements. The operations and maintenance of the stormwater management system is an on-going effort, which includes the routine inspection of drainage structures, the routine cleaning of drainage structures and piping, the repair of any damaged drainage structures, the replacement of any collapsed piping, the removal of any sediments and debris from the system, and annual compliance with regulatory programs. An adequate operations and maintenance program is necessary to limit flooding problems throughout the City during rainfall events.

The routine maintenance of drainage piping and structures on an annual basis is necessary to remove any sediment from the stormwater management system. Typically, sediments can accumulate within the drainage structures and drainage piping over time if the drainage system does not undergo routine cleaning. The accumulation of sediments within the drainage piping can significantly reduce the transmission capacity of the pipe, limit the discharge capacity from system outfalls, and slow the percolation rate via exfiltration trenches, which will slow the draining of stormwater runoff from the right-of-way areas during a rainfall event. The accumulation of sediments within the stormwater management system can also negatively impact the water quality within receiving water bodies when pollutant loading is transmitted within the stormwater runoff to the outfalls. The routine maintenance of outfalls from the stormwater management system into surface water bodies is also necessary on an annual basis. In coastal areas such as the City of Pompano Beach, many of the existing outfalls will have an accumulation of sediment and/or barnacles, which can negatively impact the performance of the existing stormwater management system. A partial blockage of the outfall pipe can significantly reduce the peak discharge capacity via the outfall, which will slow the draining of stormwater runoff from the right-of-way areas during a heavy rainfall event.

The routine inspection of drainage structures on an annual basis is necessary to identify any locations within the stormwater management system in need of system maintenance. According to the requirements of the NPDES Permit, all components of the stormwater management system must be inspected on a routine basis to ensure that it is operating appropriately. The new requirements of the NPDES Permit specify that the City must now inspect 10% of all drainage structures within the stormwater management system each year. The City must also prepare and submit an annual NPDES compliance report to FDEP for the structural controls and components of the stormwater management system, which is summarized in Section 3.4 for this report and defined in detail within digital Appendix B-13. CMA has also developed Standard Operating Procedures (SOPs) which have been developed to provide guidance on the frequency of inspections and maintenance activities related to the stormwater management system, which are enclosed within digital Appendix B-14 for future use. The City must continue to track and report all routine inspection activities in order to meet the requirements of the NPDES Permit.

The City's Stormwater Management Utility provides funding for the operations and maintenance program for the existing stormwater management system. The primary role of the Stormwater Management Utility is to clean drainage structures and drainage pipe, to repair damaged drainage structures, to replace collapsed drainage piping, to remove collected debris and sediments from the system, and to inspect drainage structures on a routine basis. This effort is completed by staff members of the Stormwater Management Utility, which currently consist of 8 positions. The Stormwater Management Utility staff consists of 1 Utilities Stormwater Supervisor position, 1 Utilities Maintenance Foreman position, 2 Utilities System Operator III positions, 2 Utilities System Operator II positions, and 2 Utilities System Operator I positions. The Stormwater Management Utility is also responsible for equipment, which include vactor trucks for system maintenance and service trucks for staff members. The Stormwater Management Utility had a total budget for operations and maintenance activities of $2,148,688 in FY2010, $2,397,993 in FY2011 and $2,137,594 in FY2012, which do not include any capital

improvement costs. The adopted budget for the Stormwater Management Utility is $2,764,784 for FY2013. The City Stormwater Management Utility tracks their performance measures on an annual basis which is outlined within Table 6.3 below.

Table 6.3 – Stormwater Management Utility Performance Measures			
Performance Measures	**FY2011**	**FY2012**	**FY2013**
Structure Cleaning (EA)	1,291	1,700	1,700
Pipe Cleaning (LF)	48,016	41,500	49,500
Structure Repairs (EA)	81	100	100
Pipe Replacement (LF)	260	200	200
Debris Removal (TN)	170	120	170
Inspections (EA)	1,503	1,650	1,650

Due to previous and planned improvements to the existing stormwater management system along with additional regulatory requirements, the City will need to expand its stormwater operations and maintenance program to ensure the stormwater management system continues to operate at full capacity in the future. Although the stormwater management system has expanded and the regulatory compliance issues have increased over the years, the manpower and equipment dedicated to the stormwater operations and maintenance program has remained relatively unchanged over the years. With the stormwater improvements proposed within this Stormwater Master Plan along with the additional regulatory compliance requirements, the City should budget for any additional manpower and equipment necessary for this additional maintenance of an expanded stormwater management system.

CMA has assembled a breakdown to display the recent expansion and the planned future expansion of the City's stormwater management system. The City has recently implemented multiple stormwater improvement projects since the previous Stormwater Master Plan in 1999, which has expanded the extent of the stormwater management system. The City has also recently annexed previously unincorporated areas of Broward County, which include the Highlands, Collier Manor, Cresthaven, and Leisureville neighborhoods, into the northeast portion of the City. During the annexation process, Broward County completed extensive drainage improvements throughout these neighborhoods. By annexing these neighborhoods, the extent of the City's stormwater management system was significantly expanded with additional drainage structures and pipes. As outlined within this Stormwater Master Plan, the recommended stormwater improvements will expand the stormwater management system with additional drainage structures and pipe. The expanded stormwater management system will require additional labor and equipment to ensure the additional drainage structures and piping continues to be properly maintained. The recent and planned expansion of the stormwater management system is summarized within Table 6.4 below, which tabulates the additional drainage structure and additional drainage piping added to the system.

Table 6.4 – Stormwater Management System Expansion Summary					
Stormwater Component	**1999 Stormwater System**	**Stormwater Improvements (since 1999)**	**North Annexation Area**	**Recommended Stormwater Improvements**	**Future Stormwater System**
Structures (EA)	3,655	801	1,954	427	6,837
Pipe (LF)	315,977	53,251	162,073	76,896	608,197

The City's stormwater management system has expanded significantly since 1999 while additional stormwater improvements are recommended in the future to address flooding issues within the City. The number of drainage structures has increased by 2,755 structures or 75% since 1999. The length of drainage piping has increased by 215,324 linear feet or 68% since 1999. Based on the recommended

improvements within this Stormwater Master Plan, the stormwater management system will expand by an additional 427 drainage structures and 76,896 linear feet of drainage piping, which will need to be maintained by the City in the future. In order to properly maintain the expanded stormwater management system, the recommended staffing increases are displayed within proposed organization chart for the Stormwater Management Utility in Figure 6.2 below, which displays the existing positions in blue and the proposed additional positions in green.

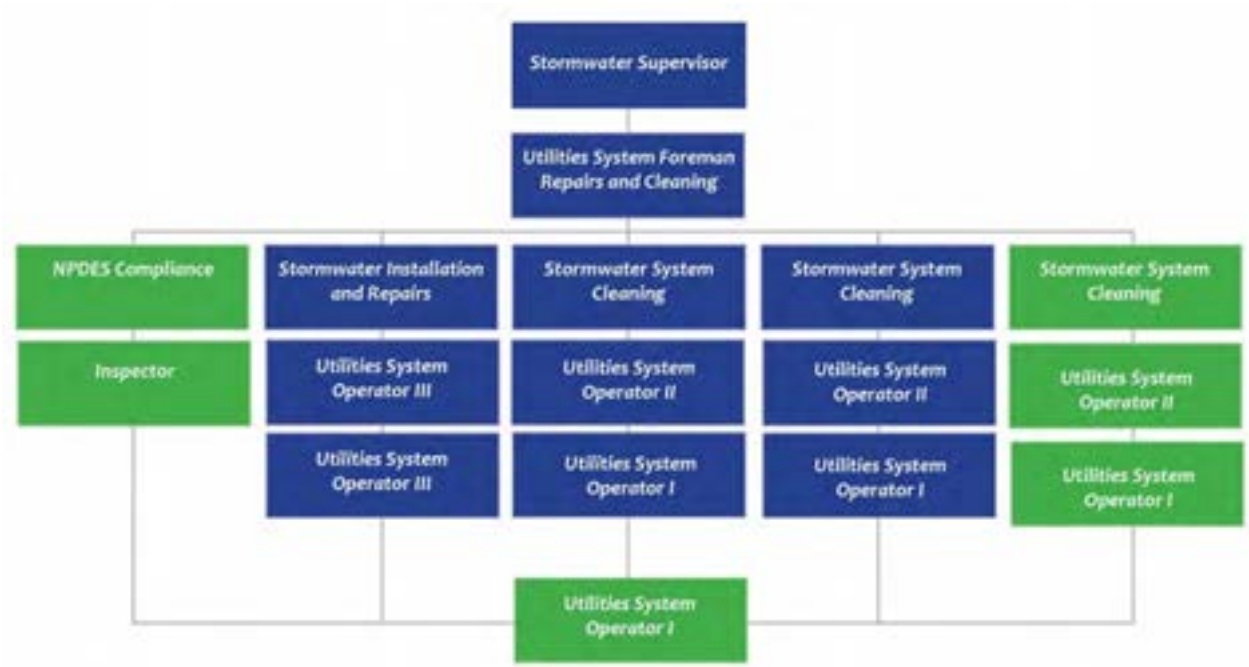


Figure 6-2 Stormwater Utility Proposed Organizational Chart

The estimated annual and capital costs for the recommended staffing increases and equipment purchases for the Stormwater Management Utility are summarized within Table 6.5 below.

Table 6.5 – Stormwater Utility O&M Increases

Recommended Additions	**Estimated Annual Costs**	**Estimated Capital Costs**
Hire Utility System Operator II Position (Additional Stormwater System Cleaning Crew)	$70,000	N/A
Hire Utility System Operator I Position (Additional Stormwater System Cleaning Crew)	$55,000	N/A
Hire Utility System Operator I Position (Miscellaneous)	$55,000	N/A
Hire Engineering Inspector Position (NPDES Program Compliance)	$70,000	N/A
Purchase/Lease Service Truck (Additional Stormwater System Cleaning Crew)	N/A	$35,000
Purchase/Lease Vactor Truck (Dedicated to Stormwater System Maintenance)	N/A	$390,000
Total	**$250,000**	**$425,000**

6.1.4 FUNDING

The City of Pompano Beach needs to develop a plan to fund the proposed stormwater CIP improvements recommended within this Stormwater Master Plan along with the additional manpower and equipment needed to adequately operate and maintain the stormwater management system. The City will need to increase the Stormwater Utility Fee in order to accomplish the following goals defined within this Stormwater Master Plan.

- Recommended Stormwater CIP Improvement Projects
- Additional Stormwater Utility Staff for O&M Purposes
- Additional Stormwater Utility Equipment for O&M purposes