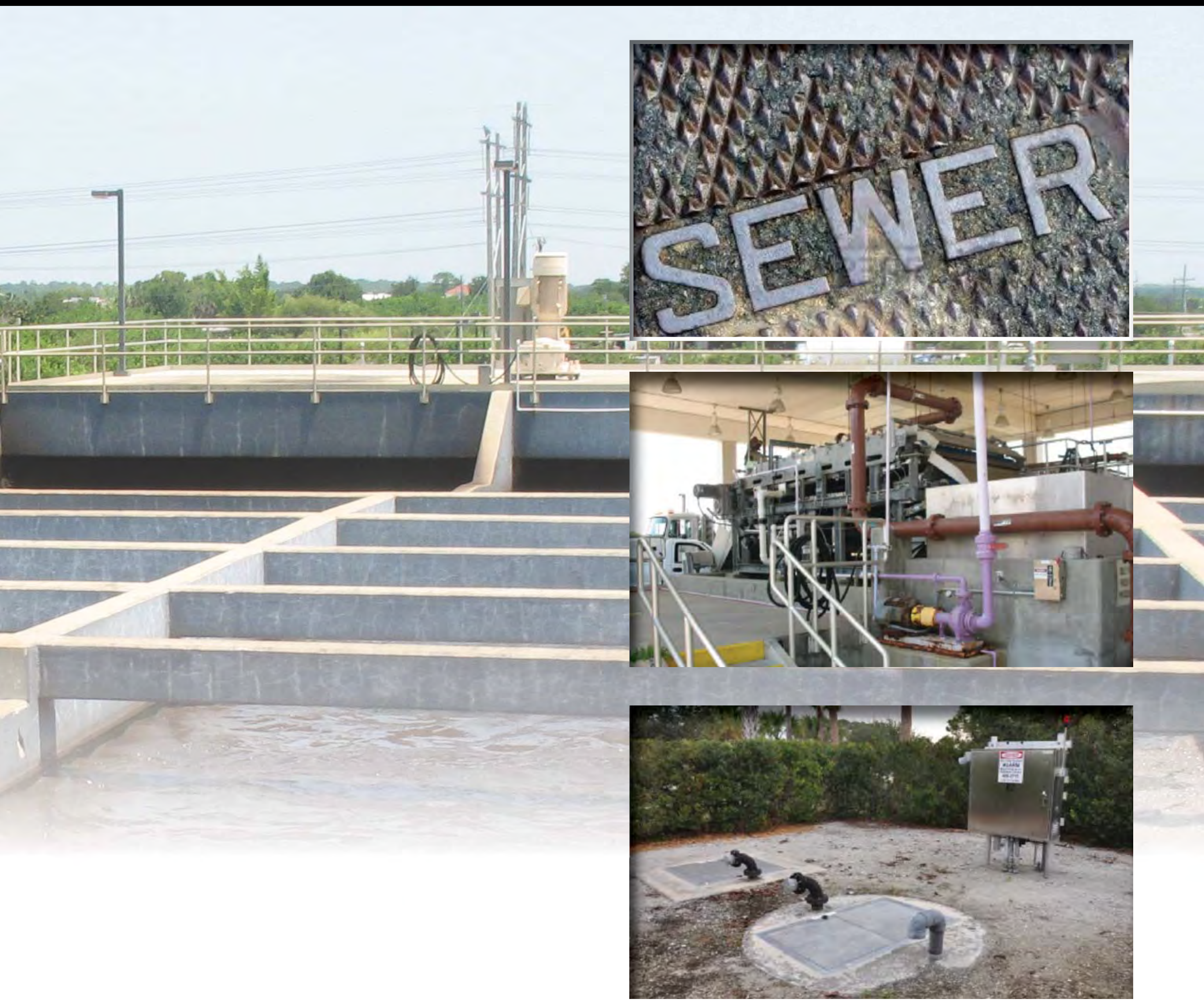
City of Venice
Wastewater Master Plan

August 2012



Wastewater Master Plan

prepared for

City of Venice

August 2012

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City of Venice Wastewater Master Plan

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EXECUTIVE SUMMARY

Introduction

The mission statement for City of Venice is the following: To provide exceptional services through a financially and environmentally sustainable City with engaged citizens. In that pursuit, the City of Venice Utilities Department offers this Wastewater Master Plan as a guide for providing reliable wastewater collection and treatment for the community's current and future needs.

Supporting data was collected from numerous sources including the following: the City of Venice Comprehensive Plan; GIS data; Future land use data; plant operating data; record drawings; pump curves; SCADA data; field investigations; population data; and related engineering reports and studies. Additional sources and references of information are listed in Appendix B.

Wastewater flow projections were developed for the planning years 2015, 2020, 2025, and 2030. The existing and projected flows were used to evaluate the wastewater collection system by performing a hydraulic modeling analysis, and for determining the expansion requirements at the Eastside WRF. The field investigation of the Eastside WRF was used to evaluate the current facility and determine the needs for upgrades, modifications, repairs, and replacement based on the age and condition of the existing equipment. The data was also used to support the wastewater collection system Infiltration and Inflow (I/I) desktop analysis.

Inflow and Infiltration Desktop Analysis

A desktop I/I analysis was conducted in order to get an overview of the impact rain events have on system flows. The daily total rainfall was compared to the average daily flow into the Eastside WRF during 2010. A relationship was defined between spikes in total daily rainfall and the total flow spikes into the Eastside WRF. In order to quantify the amount of I/I into the system, a comparison of the Eastside WRF 2010 DMRs during dry days (less than 0.1-inches per day) vs. wet days (greater than 0.1-inches per day) was made. 545,000 gpd of flow received at the Eastside WRF was identified as a result of I/I from rainfall with 177,000 gpd attributed from Sarasota County and the remaining 369,000 gpd from the City of Venice. Comparing the City's average annual daily I/I flow of 369,000 gpd to the City's average annual daily flow of 2.836 mgd yields an I/I level of service of 13%. This percentage is representative of a system with relatively high inflow and infiltration.

Individual lift stations were evaluated to determine if I/I was impacting their operation and to categorize them based on the extent of the impact. The lift stations were classified into one of four classifications based on the level of I/I observed during rainfall events. The classifications are as follows: no observable I/I (Class IV); light I/I (Class III); moderate I/I (Class II); and high I/I (Class I). **Table 2-2** summarizes the findings of the desktop I/I evaluation.

Level of Service

The City of Venice Comprehensive Plan requires that the LOS be re-evaluated as part of the Wastewater Master Plan. It is recommended that the annual average LOS increase from 123 gpd/ERU to 162 gpd/ERU. The maximum day flow LOS should increase from 244 gpd/ERU to 324 gpd/ERU.

Wastewater Collection System

A hydraulic model was constructed of the collection system backbone pipelines. Backbone pipelines were identified as 8-inch diameter gravity sewers and larger and all sewer mains that provide hydraulic connectivity between sewer basins and lift stations. This information along with the existing lift stations and force mains are the basis for the collection system model (See **Figure 5-3**).

Ten wastewater system scenarios were evaluated for correcting deficiencies and expanding capacity in response to projected growth. These scenarios considered the impact of the average day dry weather and maximum day wet weather wastewater contributions for each planning period. A list of recommended wastewater collection system CIP improvement projects are listed in **Table 7-7**. The improvements consist of upsizing gravity sewers and force mains, upsizing lift station pumps, reducing I/I and lift station monitoring improvements.

Water Reclamation Facility

As illustrated in **Table ES-1** by the cells highlighted in light red, the City is anticipated to exceed their allotted capacity defined in the Interlocal Agreement between the City of Venice and Sarasota County for Wastewater Treatment, by the planning periods of 2015 and 2020. Likewise illustrated in **Table ES-2** by cells highlighted in light purple, the Eastside WRF is anticipated to exceed the FDEP permitted capacity between planning periods of 2025 and 2030. The projections, by which the following analysis and recommended improvements are based upon, are conservative in nature. The wastewater flows to the plant should continue to be monitored in order to verify the actual timing of recommended improvements.

Table ES-1: Projected Wastewater Flows from the City of Venice

Description	2010[1]	Planning Period			
		2015	2020	2025	2030
MTMADF1	2,410,000	2,900,000	3,320,000	3,740,000	4,140,000

1. Flows presented are in gallons per day (gpd).

Table ES-2: Combined Projected Wastewater Flows

Description	2010[1]	Planning Period			
		2015	2020	2025	2030
MTMADF1	3,820,000	4,510,000	5,140,000	5,710,000	6,110,000

1. Flows presented are in gallons per day (gpd).

A list of recommended Eastside WRF CIP improvement projects are listed in **Table 7-8**. The improvements consist of an R&R program, hydraulic study, biosolids management plan, reject and RCW storage tank, disinfection improvements, pretreatment improvements, aeration improvements and internal recycle improvements.

Capital Improvement Program

Table ES-3 through **Table ES-5** summarize the recommended wastewater system improvements that are provided on **Tables 7-7** and **7-8**. CIP project summaries are provided in **Appendix A**.

Table ES-3: Collection System Improvements

Project ID	Project Description	Cost
R-100	Install lower HP pumps in LS 82 to reduce the maximum velocity in the existing 4" FM.	$ 25,000
R-101	Construct 89 LF of 6" HDPE FM to replace existing 4" FM at Royal Palm Rd and Ridgewood Ave.	$ 30,000
R-102	Construct 2,400 LF of 12" PVC FM and 300' of 14" HDPE FM to replace 2,700 LF of 8" existing FM along Albee Farm Rd.	$ 284,000
R-103	Install pumps with greater capacity in LS 42 to prevent wet well from surcharging.	$ 120,000
R-104	Abandon existing FMs and replace with 10" HDPE FM and 12" PVC FM. Revise connections to improve flow routing at Miami Ave W and Nokomis Ave S.	$ 232,000
R-105	Construct 1,100 LF parallel 30" HDPE FM across I-75 to improve system redundancy.	$ 679,000

Table ES-3: Collection System Improvements (Continued)

Project ID	Project Description	Cost
R-116	Construct 2,233 LF of 16" PVC FM and 754 LF of parallel 18" HDPE FM to replace 2,987 LF of cast iron pipe under the intracoastal to improve system redundancy.	$ 607,000
R-200	Construct 12" gravity PVC sewer to replace existing 8" gravity sewer at LS 77.	$ 25,000
R-300	Install pumps with greater capacity in LS 9 to prevent the wet well from surcharging.	$ 120,000
R-301	Install larger impeller and motor in LS 32 to increase lift station flow capacity.	$ 120,000
R-106 R-201	Install telemetry units at all City lift stations with flow meters and pressure transmitter starting with high priority lift stations.	$ 744,000
R-107 R-202	Replace select existing lift station control panels as necessary with newer equipment to support addition of telemetry.	$ 1,240,000
R-108 R-304	Assess the condition of all manholes and cursory inspection of adjacent pipelines with a pole mounted zoom camera using MACP. (Assumed perform every 10 years).	$ 940,000
R-109 R-203 R-305 R-400	CCTV video inspection of identified high priority gravity sewer pipelines using PACP (Assumed 35% of gravity sewer system by 2015, remaining 65% by 2020, and 5% on-going).	$ 1,300,000
R-110 R-204 R-306 R-401	Collection System R&R (Assumed liner rehabilitation on 20% of system by 2017, 2% annually on-going).	$ 8,560,000
R-117 R-209 R-309 R-403	Odor control at master lift stations LS 7, LS 32, and LS 57. Appropriate technology to be determined. Assumed to be vapor phase technology for budgetary purposes.	$ 1,185,000
	Total	**$16,211,000**

Table ES-4: WRF Improvements

Project ID	Project Description	Cost
R-111 R-205 R-307 R-402	Refurbish and Replacement, estimated to be $237,500 annually.	$ 4,275,000
R-112	Hydraulic Study	$ 50,000
R-113	Biosolids Management Plan	$ 75,000
R-114	Reject & RCW Improvements (GST)	$ 2,075,000
R-118	SCADA Master Plan	$ 120,000
R-119	RCW Storage Pond Return and Filtration System	$ 1,800,000
R-206	Disinfection Improvements	$ 415,000
R-207	Pretreatment Improvements	$ 3,395,000
R-208	Aeration Improvements	$ 1,875,000
R-308	Internal Recycle Improvements	$ 1,215,000
	Total	**$ 15,295,000**

TABLE ES-5: Estimated Capital Improvement Costs by Phasing Period

Planning Period	Collection System	WRF	Totals
2012-2015	$ 6,575,000	$ 5,020,000	**$ 11,595,000**
2015-2020	$ 4,180,000	$ 6,810,000	**$ 10,990,000**
2020-2025	$ 3,065,000	$ 2,340,000	**$ 5,405,000**
2025-2030	$ 2,355,000	$ 1,125,000	**$ 3,480,000**
Totals	**$ 16,175,000**	**$ 15,295,000**	**$ 31,470,000**

1.0 INTRODUCTION

1.1 Background

The City of Venice has an approximate area of 16.60 square miles and is bordered by unincorporated Sarasota County on the north, south, and east and the Gulf of Mexico on the west. All wastewater is directed to the northeast through a system of gravity sanitary sewer mains, lift stations and force mains. All wastewater flow is received and treated at the Eastside WRF which operates under FDEP permit No. FL0041441. The Eastside WRF is currently permitted to treat 6.0 mgd based on a 3-MADF. Sarasota County owns 3.0 mgd of the Eastside WRF's capacity and sends flow to the plant on an as needed basis. The interconnection between the County and City is located just upstream of the Eastside WRF entrance road at the intersection of Laurel Road and Knights Trail Road.

1.2 Master Plan Objectives

The City of Venice has established the goal of providing public utility services that meet the needs of the current and future population while protecting the environment and supporting the City's planning framework. The objective of this master plan is to develop a guide for providing reliable collection and treatment of wastewater and supporting the objective to provide reliable and redundant infrastructure. The Wastewater Master Plan will provide the guide for developing a capital improvement program which supports wastewater management needs.

1.3 Historical System Limitations

City staff was interviewed to identify some of the historical system limitations. Large rainfall events have historically increased flow into the collection system. A representative example of a lift station that was adversely affected during a large rainfall event is Lift Station 01 during August 23, 2010 through August 25, 2010. The total pump run time approximately doubled by the third day of the rain event, and it took about 9 days before the total daily pump run time started to approach levels seen before the event. Also, both pumps turned on and ran continuously on August 25, 2010 for almost 4 hours before the inflow decreased, allowing the lift station to resume normal operation. During these types of events, this and other lift stations with similar characteristics are at risk for a SSO. City staff has also reported that the system pressures may exceed 100 psi in the vicinity of Lift Station 7 as a result of several lift stations operating at the same time, usually during a large rain event. Review of the lift station run times during wet and dry days is further reviewed in **Section 2**. An evaluation of SSO events is provided in **Section 2.5**.

Lack of force main system redundancy at two locations was identified as operational limitations. A single 20-inch force main transfers all of the wastewater collected west of I-75, which consists of most of the City's flow, to the Eastside WRF. Should the flow within the pipe be disrupted due to pipe failure or operational needs such as pipe maintenance or cleaning, the City would be unable to send this wastewater flow to the Eastside WRF for treatment. The second location is where two parallel 10-inch force mains cross the Intracoastal Waterway at East Venice Avenue. These force mains transmit all of the wastewater from the island portion of the City's collection system toward the Eastside WRF. The force mains are believed to be beyond their life expectancy because they are cast iron pipe that was installed in 1959. The southernmost 10-inch force main has been inactive for the last 8 to 10 years, eliminating the use of the redundant force main.

1.4 Methodology and Overview

A methodology was developed to complete the master planning process. The first step was data collection which consisted of numerous sources of data. Some of the sources were the City of Venice Comprehensive Plan, GIS data, future land use data, plant operating data, record drawings, pump curves, SCADA data, field investigations, population data, and related engineering reports and studies. A list of references used as part of this master plan is provided in **Appendix B**.

The data was used to determine population and wastewater flow projections for the planning years 2015, 2020, 2025, and 2030. These projections were used to evaluate the wastewater collection system by performing a hydraulic modeling analysis and for determining the expansion requirements at the Eastside WRF. The field investigation of the Eastside WRF was used to evaluate the current facility and determine the needs for upgrades, modifications, repairs, and replacement based on the age and condition of the existing equipment. The data was also used to support a desk-top collection system analysis to quantify current I/I into the system and prioritize sub-areas for detailed field assessment.

Future wastewater collection system and WRF infrastructure requirements were developed based on the findings. Improvement recommendations were completed as part of the master planning process which consisted of capital improvement projects, cost estimates, and the recommended time period in which those projects should be completed and placed into service.

2.0 INFLOW AND INFILTRATION DESKTOP ANALYSIS

2.1 Data and Methodology Overview

The purpose of this desktop I/I analysis is to quantify current I/I into the wastewater system and identify sewer sub-basins for future detailed field assessment. The following data was used to evaluate system I/I:

- Eastside WRF DMRs
- Rainfall data obtained from SWFWMD website
- Existing CCTV video inspections
- Documented SSOs
- Lift station run times

In order to get an overview of the impact rain events have on system flows, the daily total rainfall was compared to the average daily flow into the Eastside WRF during 2010. As shown on **Figure 2-1**, there appears to be a relationship between spikes in total daily rainfall and the total flow spikes into the Eastside WRF during 2010. Rainfall data was obtained from the SWFWMD website. The closest rain gauge stations to the City of Venice were the Laurel Park and Knights Trail stations. The Knights Trail Station is located north of the city limits and east of I-75. Laurel Park is located north of the city and centrally located between the Gulf of Mexico and I-75. Review of the rainfall totals show that the rain events recorded at both stations matched closely. For this analysis, the data from the Laurel Park station was used.

EXECUTIVE SUMMARY

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Level of Service

The City of Venice Comprehensive Plan requires that the LOS be re-evaluated as part of the Wastewater Master Plan. It is recommended that the annual average LOS increase from 123 gpd/ERU to 162 gpd/ERU. The maximum day flow LOS should increase from 244 gpd/ERU to 324 gpd/ERU.

Wastewater Collection System

A hydraulic model was constructed of the collection system backbone pipelines. Backbone pipelines were identified as 8-inch diameter gravity sewers and larger and all sewer mains that provide hydraulic connectivity between sewer basins and lift stations. This information along with the existing lift stations and force mains are the basis for the collection system model (See **Figure 5-3**).

Ten wastewater system scenarios were evaluated for correcting deficiencies and expanding capacity in response to projected growth. These scenarios considered the impact of the average day dry weather and maximum day wet weather wastewater contributions for each planning period. A list of recommended wastewater collection system CIP improvement projects are listed in **Table 7-7**. The improvements consist of upsizing gravity sewers and force mains, upsizing lift station pumps, reducing I/I and lift station monitoring improvements.

Water Reclamation Facility

As illustrated in **Table ES-1** by the cells highlighted in light red, the City is anticipated to exceed their allotted capacity defined in the Interlocal Agreement between the City of Venice and Sarasota County for Wastewater Treatment, by the planning periods of 2015 and 2020. Likewise illustrated in **Table ES-2** by cells highlighted in light purple, the Eastside WRF is anticipated to exceed the FDEP permitted capacity between planning periods of 2025 and 2030. The projections, by which the following analysis and recommended improvements are based upon, are conservative in nature. The wastewater flows to the plant should continue to be monitored in order to verify the actual timing of recommended improvements.

Table ES-1: Projected Wastewater Flows from the City of Venice

Description	2010[1]	Planning Period			
		2015	2020	2025	2030
MTMADF1	2,410,000	2,900,000	3,320,000	3,740,000	4,140,000

1. Flows presented are in gallons per day (gpd).

Table ES-2: Combined Projected Wastewater Flows

Description	2010[1]	Planning Period			
		2015	2020	2025	2030
MTMADF1	3,820,000	4,510,000	5,140,000	5,710,000	6,110,000

1. Flows presented are in gallons per day (gpd).

A list of recommended Eastside WRF CIP improvement projects are listed in **Table 7-8**. The improvements consist of an R&R program, hydraulic study, biosolids management plan, reject and RCW storage tank, disinfection improvements, pretreatment improvements, aeration improvements and internal recycle improvements.

Capital Improvement Program

Table ES-3 through **Table ES-5** summarize the recommended wastewater system improvements that are provided on **Tables 7-7** and **7-8**. CIP project summaries are provided in **Appendix A**.

Table ES-3: Collection System Improvements

Project ID	Project Description	Cost
R-100	Install lower HP pumps in LS 82 to reduce the maximum velocity in the existing 4" FM.	$ 25,000
R-101	Construct 89 LF of 6" HDPE FM to replace existing 4" FM at Royal Palm Rd and Ridgewood Ave.	$ 30,000
R-102	Construct 2,400 LF of 12" PVC FM and 300' of 14" HDPE FM to replace 2,700 LF of 8" existing FM along Albee Farm Rd.	$ 284,000
R-103	Install pumps with greater capacity in LS 42 to prevent wet well from surcharging.	$ 120,000
R-104	Abandon existing FMs and replace with 10" HDPE FM and 12" PVC FM. Revise connections to improve flow routing at Miami Ave W and Nokomis Ave S.	$ 232,000
R-105	Construct 1,100 LF parallel 30" HDPE FM across I-75 to improve system redundancy.	$ 679,000

Table ES-3: Collection System Improvements (Continued)

Project ID	Project Description	Cost
R-116	Construct 2,233 LF of 16” PVC FM and 754 LF of parallel 18” HDPE FM to replace 2,987 LF of cast iron pipe under the intracoastal to improve system redundancy.	$ 607,000
R-200	Construct 12" gravity PVC sewer to replace existing 8" gravity sewer at LS 77.	$ 25,000
R-300	Install pumps with greater capacity in LS 9 to prevent the wet well from surcharging.	$ 120,000
R-301	Install larger impeller and motor in LS 32 to increase lift station flow capacity.	$ 120,000
R-106 R-201	Install telemetry units at all City lift stations with flow meters and pressure transmitter starting with high priority lift stations.	$ 744,000
R-107 R-202	Replace select existing lift station control panels as necessary with newer equipment to support addition of telemetry.	$ 1,240,000
R-108 R-304	Assess the condition of all manholes and cursory inspection of adjacent pipelines with a pole mounted zoom camera using MACP. (Assumed perform every 10 years).	$ 940,000
R-109 R-203 R-305 R-400	CCTV video inspection of identified high priority gravity sewer pipelines using PACP (Assumed 35% of gravity sewer system by 2015, remaining 65% by 2020, and 5% on-going).	$ 1,300,000
R-110 R-204 R-306 R-401	Collection System R&R (Assumed liner rehabilitation on 20% of system by 2017, 2% annually on-going).	$ 8,560,000
R-117 R-209 R-309 R-403	Odor control at master lift stations LS 7, LS 32, and LS 57. Appropriate technology to be determined. Assumed to be vapor phase technology for budgetary purposes.	$ 1,185,000
	Total	**$16,211,000**

Table ES-4: WRF Improvements

Project ID	Project Description	Cost
R-111 R-205 R-307 R-402	Refurbish and Replacement, estimated to be $237,500 annually.	$ 4,275,000
R-112	Hydraulic Study	$ 50,000
R-113	Biosolids Management Plan	$ 75,000
R-114	Reject & RCW Improvements (GST)	$ 2,075,000
R-118	SCADA Master Plan	$ 120,000
R-119	RCW Storage Pond Return and Filtration System	$ 1,800,000
R-206	Disinfection Improvements	$ 415,000
R-207	Pretreatment Improvements	$ 3,395,000
R-208	Aeration Improvements	$ 1,875,000
R-308	Internal Recycle Improvements	$ 1,215,000
	Total	**$ 15,295,000**

TABLE ES-5: Estimated Capital Improvement Costs by Phasing Period

Planning Period	Collection System	WRF	Totals
2012-2015	$ 6,575,000	$ 5,020,000	**$ 11,595,000**
2015-2020	$ 4,180,000	$ 6,810,000	**$ 10,990,000**
2020-2025	$ 3,065,000	$ 2,340,000	**$ 5,405,000**
2025-2030	$ 2,355,000	$ 1,125,000	**$ 3,480,000**
Totals	**$ 16,175,000**	**$ 15,295,000**	**$ 31,470,000**

1.0 INTRODUCTION

1.1 Background

The City of Venice has an approximate area of 16.60 square miles and is bordered by unincorporated Sarasota County on the north, south, and east and the Gulf of Mexico on the west. All wastewater is directed to the northeast through a system of gravity sanitary sewer mains, lift stations and force mains. All wastewater flow is received and treated at the Eastside WRF which operates under FDEP permit No. FL0041441. The Eastside WRF is currently permitted to treat 6.0 mgd based on a 3-MADF. Sarasota County owns 3.0 mgd of the Eastside WRF's capacity and sends flow to the plant on an as needed basis. The interconnection between the County and City is located just upstream of the Eastside WRF entrance road at the intersection of Laurel Road and Knights Trail Road.

1.2 Master Plan Objectives

The City of Venice has established the goal of providing public utility services that meet the needs of the current and future population while protecting the environment and supporting the City's planning framework. The objective of this master plan is to develop a guide for providing reliable collection and treatment of wastewater and supporting the objective to provide reliable and redundant infrastructure. The Wastewater Master Plan will provide the guide for developing a capital improvement program which supports wastewater management needs.

1.3 Historical System Limitations

City staff was interviewed to identify some of the historical system limitations. Large rainfall events have historically increased flow into the collection system. A representative example of a lift station that was adversely affected during a large rainfall event is Lift Station 01 during August 23, 2010 through August 25, 2010. The total pump run time approximately doubled by the third day of the rain event, and it took about 9 days before the total daily pump run time started to approach levels seen before the event. Also, both pumps turned on and ran continuously on August 25, 2010 for almost 4 hours before the inflow decreased, allowing the lift station to resume normal operation. During these types of events, this and other lift stations with similar characteristics are at risk for a SSO. City staff has also reported that the system pressures may exceed 100 psi in the vicinity of Lift Station 7 as a result of several lift stations operating at the same time, usually during a large rain event. Review of the lift station run times during wet and dry days is further reviewed in **Section 2**. An evaluation of SSO events is provided in **Section 2.5**.

Lack of force main system redundancy at two locations was identified as operational limitations. A single 20-inch force main transfers all of the wastewater collected west of I-75, which consists of most of the City's flow, to the Eastside WRF. Should the flow within the pipe be disrupted due to pipe failure or operational needs such as pipe maintenance or cleaning, the City would be unable to send this wastewater flow to the Eastside WRF for treatment. The second location is where two parallel 10-inch force mains cross the Intracoastal Waterway at East Venice Avenue. These force mains transmit all of the wastewater from the island portion of the City's collection system toward the Eastside WRF. The force mains are believed to be beyond their life expectancy because they are cast iron pipe that was installed in 1959. The southernmost 10-inch force main has been inactive for the last 8 to 10 years, eliminating the use of the redundant force main.

1.4 Methodology and Overview

A methodology was developed to complete the master planning process. The first step was data collection which consisted of numerous sources of data. Some of the sources were the City of Venice Comprehensive Plan, GIS data, future land use data, plant operating data, record drawings, pump curves, SCADA data, field investigations, population data, and related engineering reports and studies. A list of references used as part of this master plan is provided in **Appendix B**.

The data was used to determine population and wastewater flow projections for the planning years 2015, 2020, 2025, and 2030. These projections were used to evaluate the wastewater collection system by performing a hydraulic modeling analysis and for determining the expansion requirements at the Eastside WRF. The field investigation of the Eastside WRF was used to evaluate the current facility and determine the needs for upgrades, modifications, repairs, and replacement based on the age and condition of the existing equipment. The data was also used to support a desk-top collection system analysis to quantify current I/I into the system and prioritize sub-areas for detailed field assessment.

Future wastewater collection system and WRF infrastructure requirements were developed based on the findings. Improvement recommendations were completed as part of the master planning process which consisted of capital improvement projects, cost estimates, and the recommended time period in which those projects should be completed and placed into service.

2.0 INFLOW AND INFILTRATION DESKTOP ANALYSIS

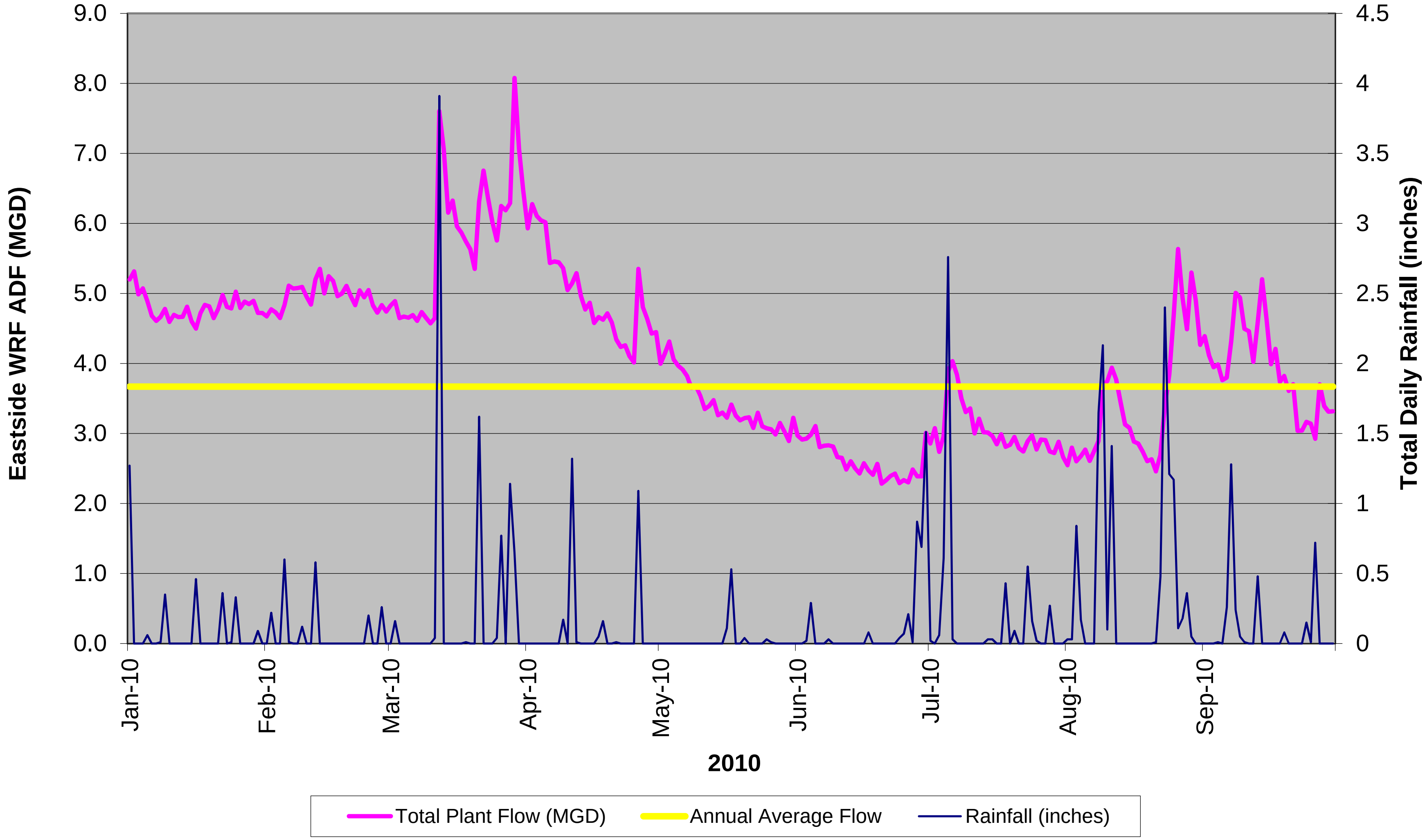
2.1 Data and Methodology Overview

The purpose of this desktop I/I analysis is to quantify current I/I into the wastewater system and identify sewer sub-basins for future detailed field assessment. The following data was used to evaluate system I/I:

- Eastside WRF DMRs
- Rainfall data obtained from SWFWMD website
- Existing CCTV video inspections
- Documented SSOs
- Lift station run times

In order to get an overview of the impact rain events have on system flows, the daily total rainfall was compared to the average daily flow into the Eastside WRF during 2010. As shown on **Figure 2-1**, there appears to be a relationship between spikes in total daily rainfall and the total flow spikes into the Eastside WRF during 2010. Rainfall data was obtained from the SWFWMD website. The closest rain gauge stations to the City of Venice were the Laurel Park and Knights Trail stations. The Knights Trail Station is located north of the city limits and east of I-75. Laurel Park is located north of the city and centrally located between the Gulf of Mexico and I-75. Review of the rainfall totals show that the rain events recorded at both stations matched closely. For this analysis, the data from the Laurel Park station was used.

FIGURE 2-1
Rainfall vs. Eastside WRF Inflow



The months with the most significant amount of rainfall were selected for evaluation of specific lift stations. As shown in **Figure 2-1,** the months during 2010 with the most significant total daily rainfall, with daily total rainfall exceeding 1-inch, were March, April, May, July, August, and September. The large rain events in March and April demonstrate that significant rain events are not limited to the typical wet season.

In order to review the impact that seasonal residents and rainfall have on the flows into the Eastside WRF, a comparison between the wet season and dry season flows was performed. The wet season, generally identified as being between mid-May to mid-October and encompassed by the Atlantic hurricane season, was selected as May 16th through October 17th for the purpose of this analysis. **Table 2-1** summarizes the ratio between the ADF and PDF in the dry season to ratio in the wet season. This ratio also represents the peak day factor. The dry season to wet season ratios are relatively close for each individual year with the exception of 2008. It appears that large rainfall events during the dry season have kept the PDF to ADF ratios relatively close each year. Seasonal residents in the dry season increased the ADF and PDF totals but appear to not affect the ratio.

Table 2-1: Historical Seasonal ADF and PDF

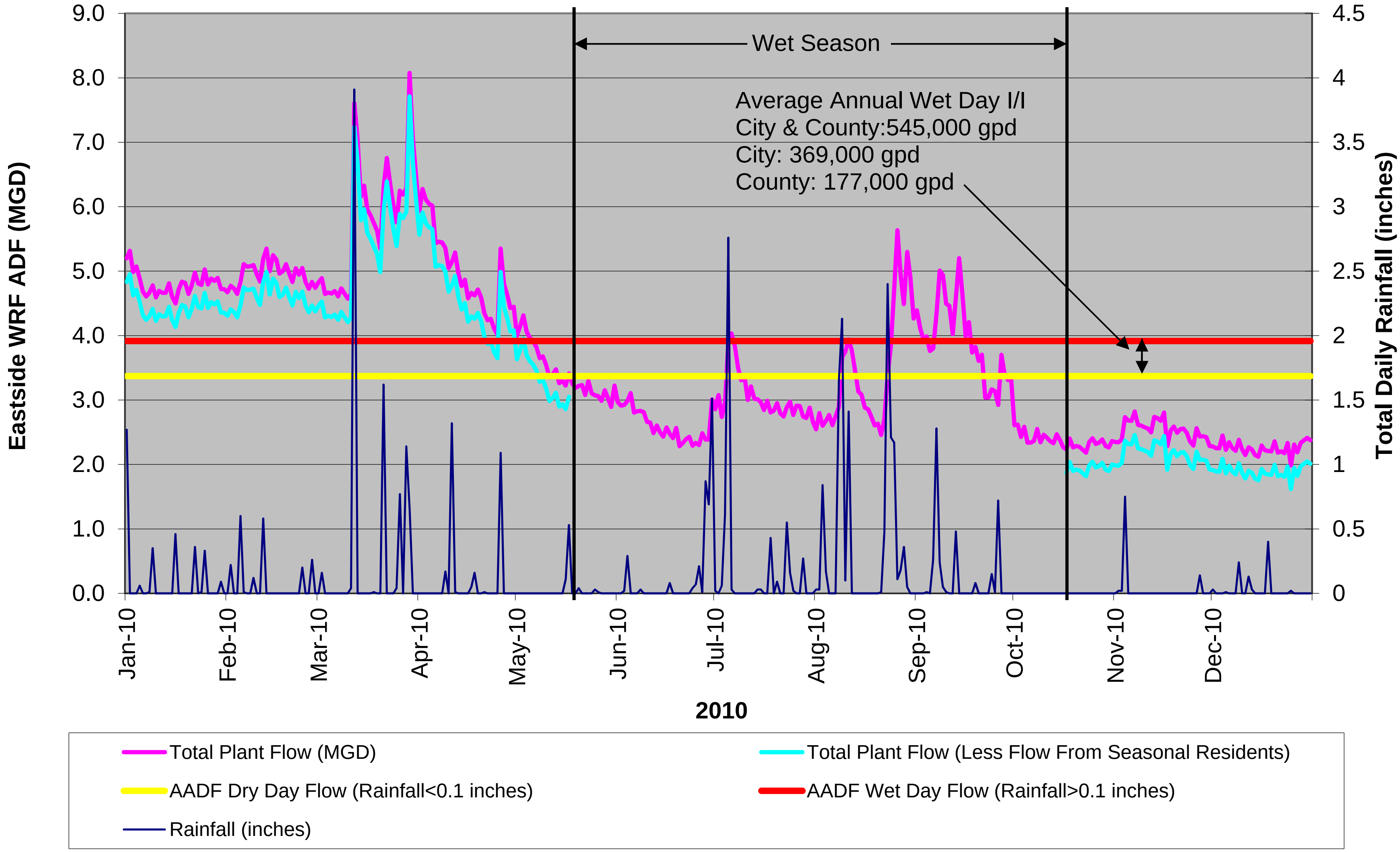
Type	2007 Flow (mgd)	2007 Ratio[1]	2008 Flow (mgd)	2008 Ratio[1]	2009 Flow (mgd)	2009 Ratio[1]	2010 Flow (mgd)	2010 Ratio[1]	2011 Flow (mgd)	2011 Ratio[1]
Dry Season										
ADF	2.61	1.31	2.96	1.39	3.30	1.76	4.03	2.00	2.75	1.46
PDF	3.41		4.13		5.82		8.07		4.02	
Wet Season										
ADF	1.88	1.28	3.09	2.64	2.86	1.73	3.15	1.79	2.64	1.85
PDF	2.42		8.15		4.93		5.63		4.88	

2.2 Estimate of I/I Quantity During Rain Events

In order to quantify the amount of I/I into the system, a comparison of the Eastside WRF 2010 DMRs during dry days (less than 0.1-inches per day) versus wet days (greater than 0.1-inches per day) was made. A minimum total rainfall of 0.1 inches per day was selected as a means to group the more significant rainfall days, those with more likelihood of producing I/I, into the same wet day classification. Before this comparison could be made, the impact of seasonal residents to the City of Venice flows had to be accounted for and removed from this comparison so the flows they produce are not mistaken as system I/I. In order to do this, the wastewater flow from the City's 2010 seasonal population was subtracted from the daily DMR total during the dry season when the seasonal residents are typically in Venice. The flow from seasonal residents was calculated as the seasonal population of 3,645, as is discussed further in **Section 3.1,** multiplied by 100 gpcpd to arrive at 364,500 gpd. The calculations to arrive at 100 gpdpc are presented in **Section 5.2**. Now that the flows from the seasonal population have been subtracted, the increase in flows during wet days was compared to flows during dry days. The average adjusted daily flow into the Eastside WRF during dry days was 3.37 mgd vs. 3.92 mgd during wet days. The difference is 545,000 gpd, which represents the annual average wet day flow into the system as a result of I/I.

Since the City receives some of its flow from Sarasota County, the proportion of I/I from the County was evaluated. As with the City of Venice, the seasonal population for Sarasota County needed to be accounted for so flows from seasonal residents are not mistaken for I/I. Per the Sarasota County Wastewater Management Plan Report dated June 2009, by Greeley and Hansen, the Central and South Counties can potentially send flow to the Eastside WRF. The functional population of Central and South County combined is 70,700. The combined seasonal population for these areas is 18,400, which is 26% of the functional population. The flow from seasonal residents was calculated as 26% of County flow sent to the Eastside WRF. The County's average adjusted daily flow into the Eastside WRF during dry days was 1.012 mgd vs. 1.189 mgd during wet days. The difference is 177,000 gpd which represents the County portion of the total 545,000 gpd. The resulting I/I attributed to the City is 369,000 gpd. As a check if the total I/I is assumed to be proportional to the flows into the Eastside WRF, the City would have 65% or 340,000 gpd of the I/I flow. Comparing the City's average annual daily I/I flow of 369,000 gpd to the City's average annual daily flow of 2.836 mgd yields an I/I LOS of 13%. This means on an average wet day 13% of the flow to the Eastside WRF is from I/I within the City. The-I/I data is graphed on **Figure 2-2**. Please note that it is assumed that the inflow component of I/I accounts for the majority of the increase in flow attributed to rainfall events.

FIGURE 2-2
Average Annual Infow and Infiltration During Rain Events



2.3 Estimate of I/I Quantity During Low Rain Periods

Given that the City of Venice is a coastal community with portions containing aged infrastructure, it can be assumed that a component of the daily influent wastewater flow into the Eastside WRF is attributable to ground water infiltration. However, the City has several dynamic influences on wastewater generation that makes quantifying the infiltration into the system difficult without further data collection and testing. Both the increase in rainfall and the presence of high tide elevations can create seasonal high groundwater conditions and translate into greater infiltration quantities.

Lift station SCADA data from December 2008 through June 2011 provided by the City was utilized to evaluate potential infiltration quantities of the total wastewater flow. The dates of January 4, 2009 and August 17, 2009 were chosen to be typical of low flow conditions for the winter and summer seasons, respectively. These low flow days are representative of the wastewater influent without the influence of rain inflow into the collection system and are plotted in **Figures 2-3** and **2-4**. In both graphs, the trend lines of the scatter plots suggest a system minimum flow of approximately 400 gpm that was unaffected by season or population.

For the purpose of this analysis, the infiltration was assumed to be half of the base flow plotted (or approximately 200 gpm) and to come predominantly from the island portion of the City's collection system (or roughly 1,735 acres). Given these assumptions, the infiltration was calculated to be 165 gpd/acre which is well within the range of 20 to 3,000 gpd/acre given by Wastewater Engineering Treatment, Disposal, and Reuse by Metcalf & Eddy Inc., 3rd Edition © 1991.

Given the data available from the City and the conservative assumptions presented above, a minimum of 15% (280,000 gpd) of the current average yearly flow could be attributed to groundwater infiltration into the collection system.

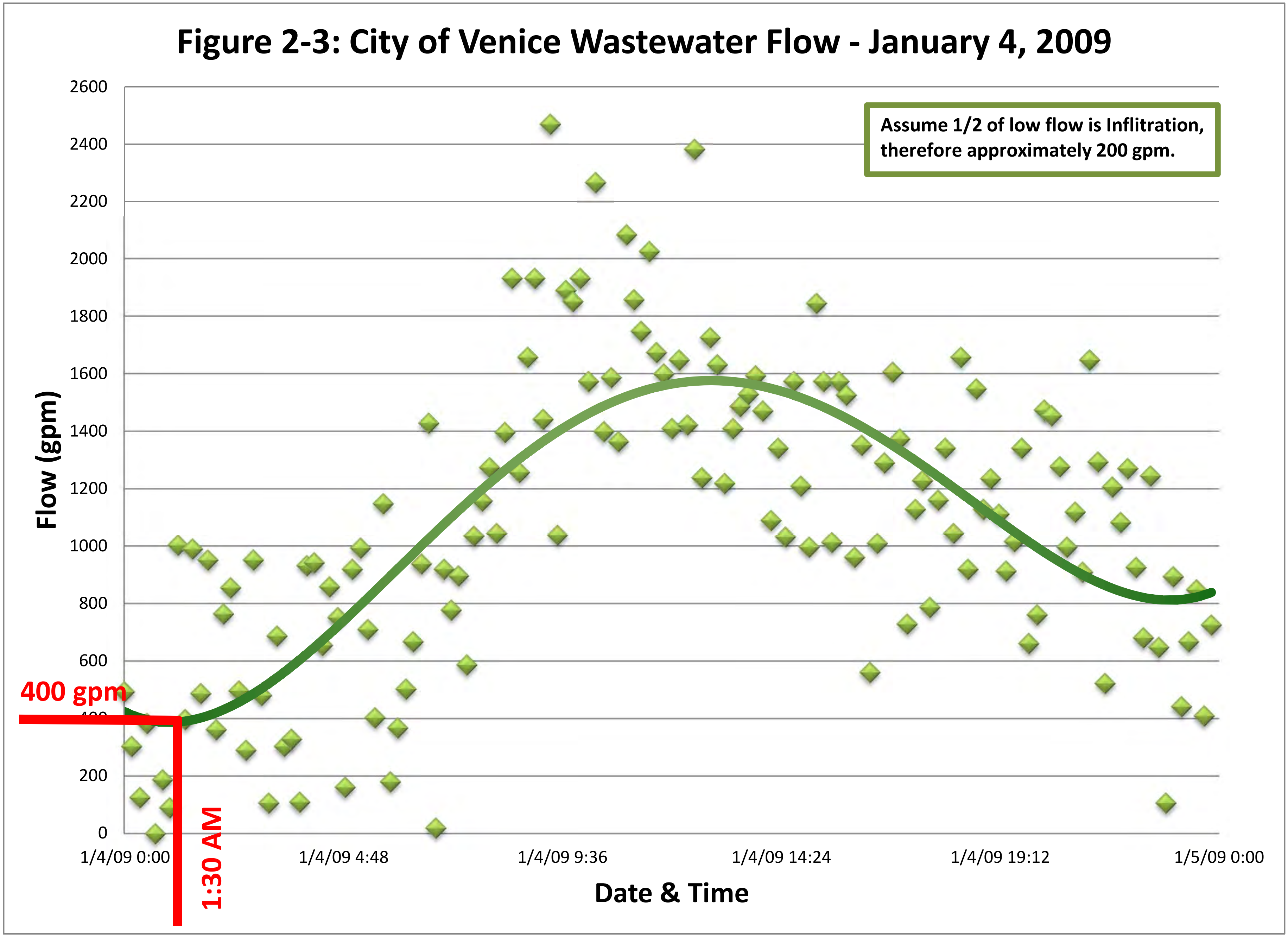


Figure 2-3: City of Venice Wastewater Flow - January 4, 2009
Assume 1/2 of low flow is Inflitration, therefore approximately 200 gpm.
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
Flow (gpm)
400 gpm
1:30 AM
1/4/09 0:00
1/4/09 4:48
1/4/09 9:36
1/4/09 14:24
1/4/09 19:12
1/5/09 0:00
Date & Time

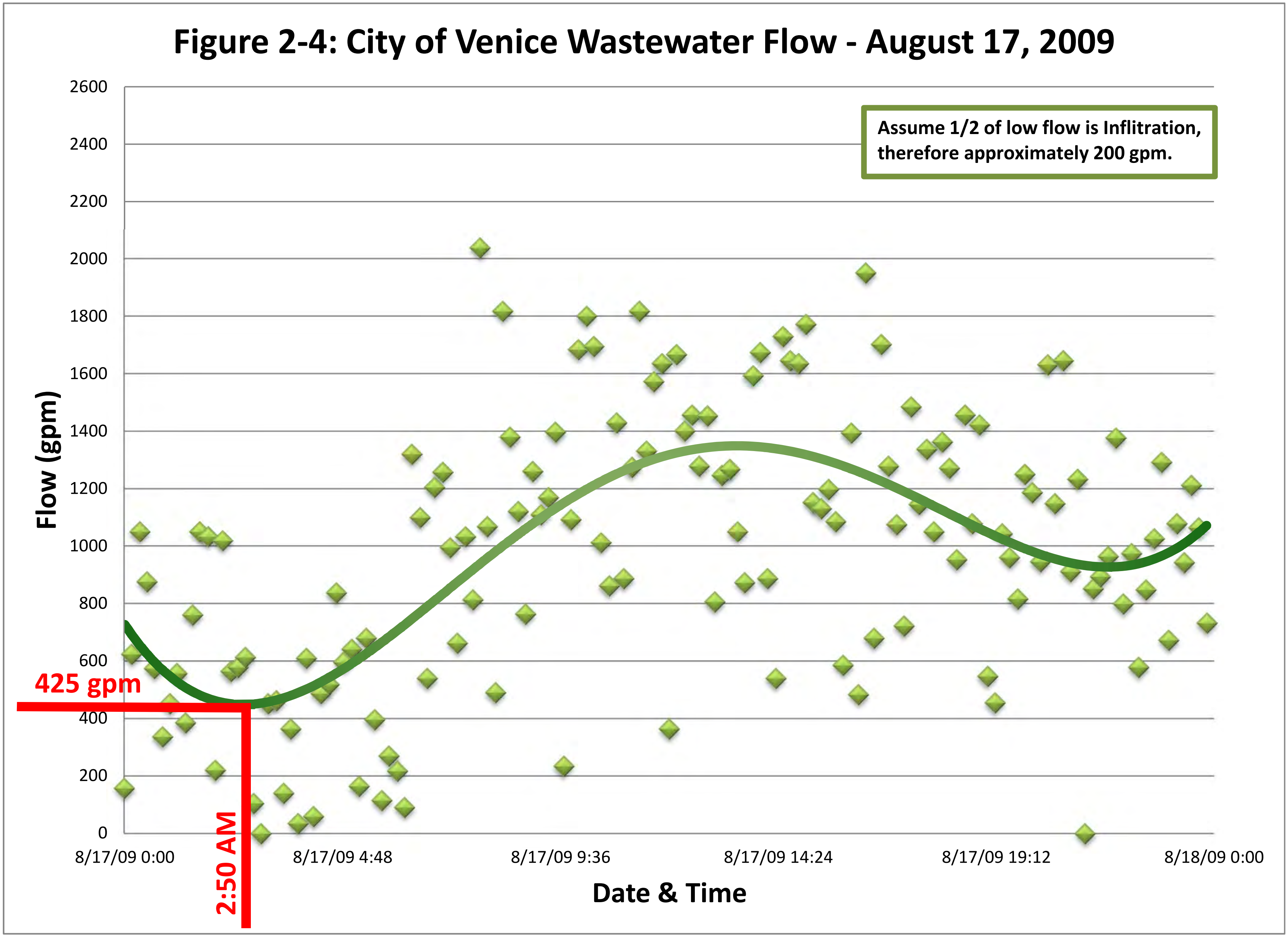


Figure 2-4: City of Venice Wastewater Flow - August 17, 2009
Assume 1/2 of low flow is Inflitration, therefore approximately 200 gpm.
Flow (gpm)
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
425 gpm
2:50 AM
8/17/09 0:00
8/17/09 4:48
8/17/09 9:36
8/17/09 14:24
8/17/09 19:12
8/18/09 0:00
Date & Time

As illustrated by the range given by M&E above, the amount of infiltration varies greatly. In order to identify the lift station sub-basins that have the greatest amount of infiltration, chloride concentrations in the wastewater received at each lift station should be tested. Since the City of Venice is a coastal community, the groundwater is likely brackish and identifiable with elevated chloride levels. Lift station sub-basins with the highest chloride levels should be prioritized for further assessment. Since chlorides are seen as TDS, a reduction in infiltration is anticipated to lower TDS in the wastewater received at the Eastside WRF.

2.4 Lift Station Run Times During Rain Events

Run time data for 60 lift stations recorded in 2010 were provided for analysis. Fifteen of these lift stations were connected to the SCADA system which recorded total daily rainfall. The remaining lift stations have a monthly checklist where pump run times are recorded manually approximately nine times a month.

The total daily rainfall was plotted against the total daily pump run time for each month identified in **Section 2.1** in order to determine if I/I due to rainfall was impacting the operation of the lift station and to what extent (See **Appendix N**). When the pump run time was not recorded daily, as in the case of the monthly lift station checklists, the total run time was divided by the number of days between readings to arrive at the average daily pump run time. The methodology used for the monthly lift station checklist provides a means to evaluate the available data but the classification may not be as accurate when compared to the lift station data obtained by SCADA. Once telemetry is added to lift stations the pump run times should be evaluated further. The lift stations were categorized into one of four classifications based on the level of I/I observed during rainfall events. The classifications are; no observable I/I (Class IV), light I/I (Class III), moderate I/I (Class II), and high I/I (Class I). The difference between Class I and Class IV is somewhat subjective, but generally Class I signifies that the pump station run time more than doubled or two pumps stayed on for an extended period during large or extended rain events. Six lift stations had pump run times that did not exhibit consistent run times; and therefore, impacts from I/I were not identifiable and classified as inconclusive. Several other lift stations exhibited high pump run times that were not coincident with rain events. High pump run times not associated with rain events were disregarded for the purpose of assigning an I/I classification. **Table 2-2** below summarizes the findings of the desktop I/I evaluation.

Table 2-2: Lift Station I/I Classification

Lift Station #	I/I Classification	Average Daily Run Time (minutes)	Maximum Day Run Time (minutes)	Max Run Time Coincides with Rain Event
00	IV	142	679	No
01	I	528	1092	Yes
02	II	310	643	No
03	II	266	503	Yes
04	II	234	438	Yes
05	II	236	465	Yes
06	II	294	428	Yes
07	II	1169	1614	Yes
08	I	396	823	Yes
09	II	187	568	No
10	I	268	714	Yes
11	Inconclusive	201	1377	N/A
12	II	232	383	Yes
13	Inconclusive	101	293	N/A
14	I	21	48	Yes
15	I	62	202	Yes

Table 2-2: Lift Station I/I Classification (Continued)

Lift Station #	I/I Classification	Average Daily Run Time (minutes)	Maximum Day Run Time (minutes)	Max Run Time Coincides with Rain Event
16	I	42	117	No
17	I	90	220	Yes
18	I	70	174	Yes
19	I	176	470	Yes
20	I	90	246	Yes
22	I	44	104	Yes
23	II	207	333	Yes
24	III	42	104	No
25	II	84	155	No
27	I	531	920	Yes
28	II	240	969	No
22	I	44	104	Yes
23	II	207	333	Yes
24	III	42	104	No
25	II	84	155	No
27	I	531	920	Yes
28	II	240	969	No
29	III	126	816	No
30	III	149	882	No
31	I	46	128	Yes
32	I	616	1213	Yes
34	II	481	854	Yes
35	IV	157	615	No
38	III	70	104	No
39	II	184	353	Yes
40	III	138	279	No

Table 2-2: Lift Station I/I Classification (Continued)

Lift Station #	I/I Classification	Average Daily Run Time (minutes)	Maximum Day Run Time (minutes)	Max Run Time Coincides with Rain Event
42	II	110	236	Yes
43	II	126	226	Yes
46	IV	48	102	No
47	III	257	723	No
51	III	91	1045	No
54	III	74	156	No
55	II	52	106	Yes
57	II	481	854	Yes
58	IV	38	71	No
61	III	112	202	No
62	III	234	642	No
63	Inconclusive	133	246	N/A
65	IV	102	159	No
68	III	179	266	Yes
69	IV	81	120	No
70	III	66	84	Yes
71	IV	76	121	No
76	III	120	153	Yes
77	IV	101	168	No
78	Inconclusive	39	85	N/A
81	IV	101	168	No
83	Inconclusive	15	87	N/A
84	Inconclusive	99	910	N/A
86	II	21	30	Yes

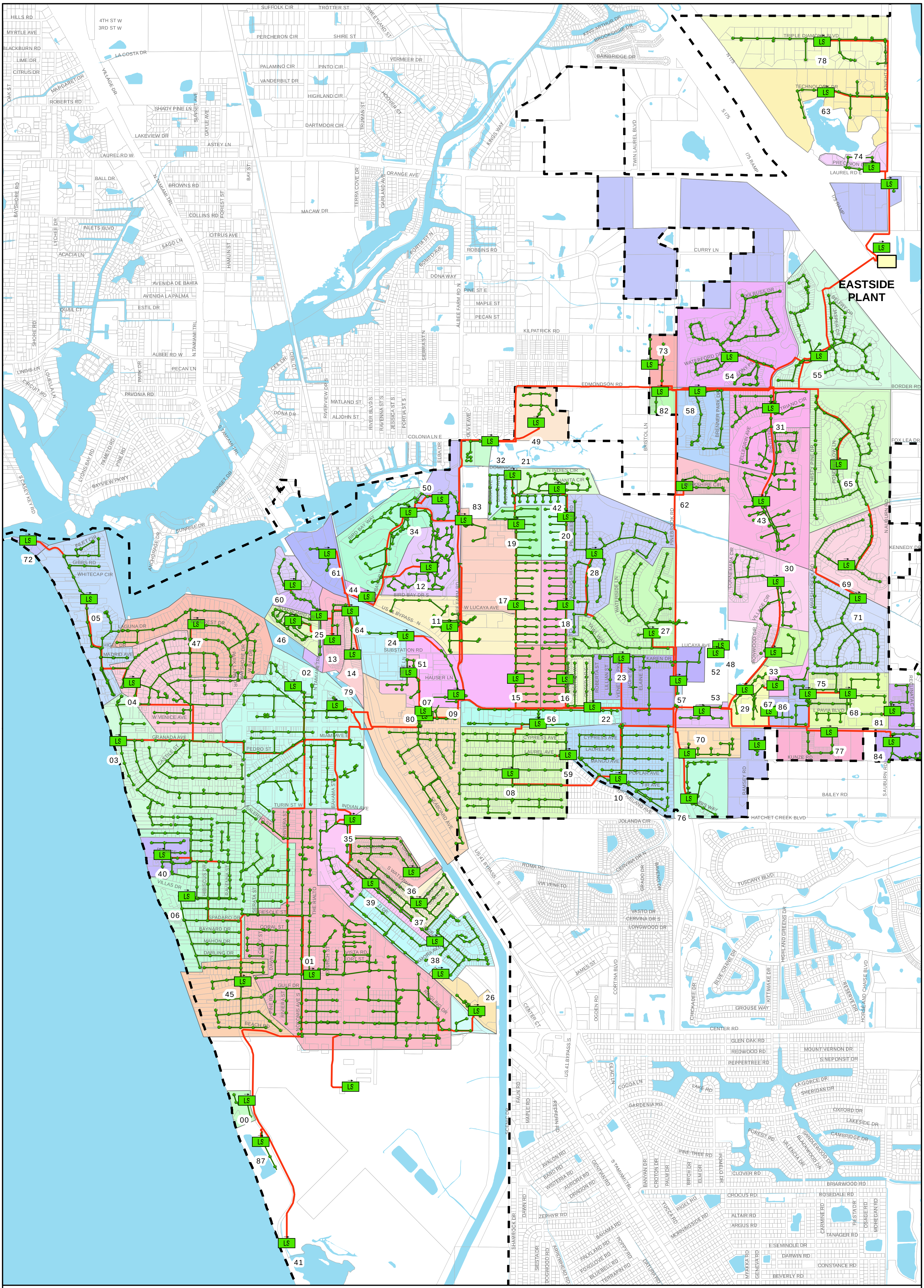
All of the Class I lift stations listed in **Table 2-2** appear to be the most impacted lift stations as a result of I/I. Lift stations that show an immediate increase in run time suggests the effects of inflow are significant. Lift stations which take an extended period for the run times to decrease suggest there is a large amount of infiltration that is accentuated by the increase in groundwater levels. It is recommended that Class I sub-basins, which only receive flow from their sub-basin and not from an upstream list station, are selected first for detailed field assessment since the increase in flow during rain events can be isolated to its gravity collection system. The second priority sub-basins for detailed field assessment are the Class I sub-basins that receive a component of their flow from upstream lift stations. These efforts should reduce the I/I entering the collection system and reduce overall flows to the Eastside WRF. See **Figure 2-5** for the location of the sewer sub-basin associated with each lift station and **Figure 2-6** for the sub-basin I/I classification.

2.5 Historical Sanitary Sewer Overflow Evaluation

The City of Venice provided records of every recorded SSO event from 2004 through May 2011. SSOs that were related to wastewater collection were plotted on a map and color coded based on the year of occurrence (See **Figure 2-7**).

Trends in SSO locations were reviewed to identify areas in the gravity collection system that have numerous and periodic failures. Frequent SSOs may indicate areas with failed piping, grease build-up, pipe blockages and/or inadequate hydraulic capacity.

The locations of SSOs were fairly spread out across the City limits. The most frequent SSOs occurred in the vicinity of Lift Station 7. The SSOs in this area also appear to span across several years as recent as 2010. A majority of the SSOs that occurred in 2008 were a result of a re-occurring gravity sewer blockage in the vicinity of Lift Station 7. The reason for the regular gravity blockages should be investigated and resolved to prevent similar SSOs occurring in this area.



Legend

Lift Station

Force Main

Gravity Sewer

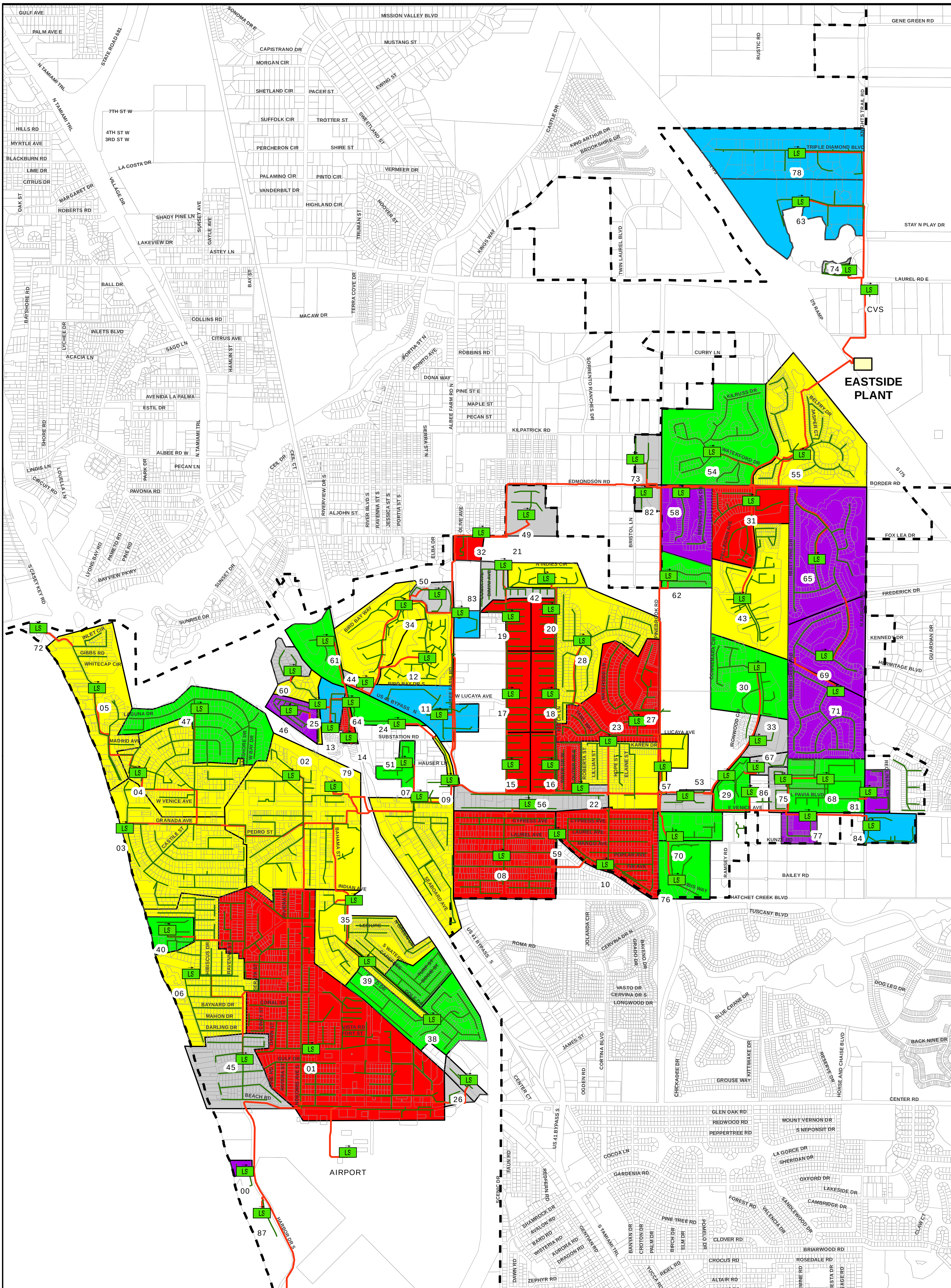
City Limits

McKIM&CREED

VENICE CITY ON THE GULF

Figure 2 - 5
City of Venice, FL
Revised Lift Station Sub-basins

NOTE: Larger copy of map provided with final report.



Legend

I/I Classification

- Not Classified
- I High I/I
- II Moderate I/I
- III Light I/I
- IV None Observed
- Inconclusive

- LS Lift Station
- City Limits

N
W E
S

McKIM&CREED

VENICE CITY ON THE GULF

Figure 2 - 6
City of Venice, FL
Lift Station Subbasins
I/I Classification

NOTE: Larger copy of map provided with final report.

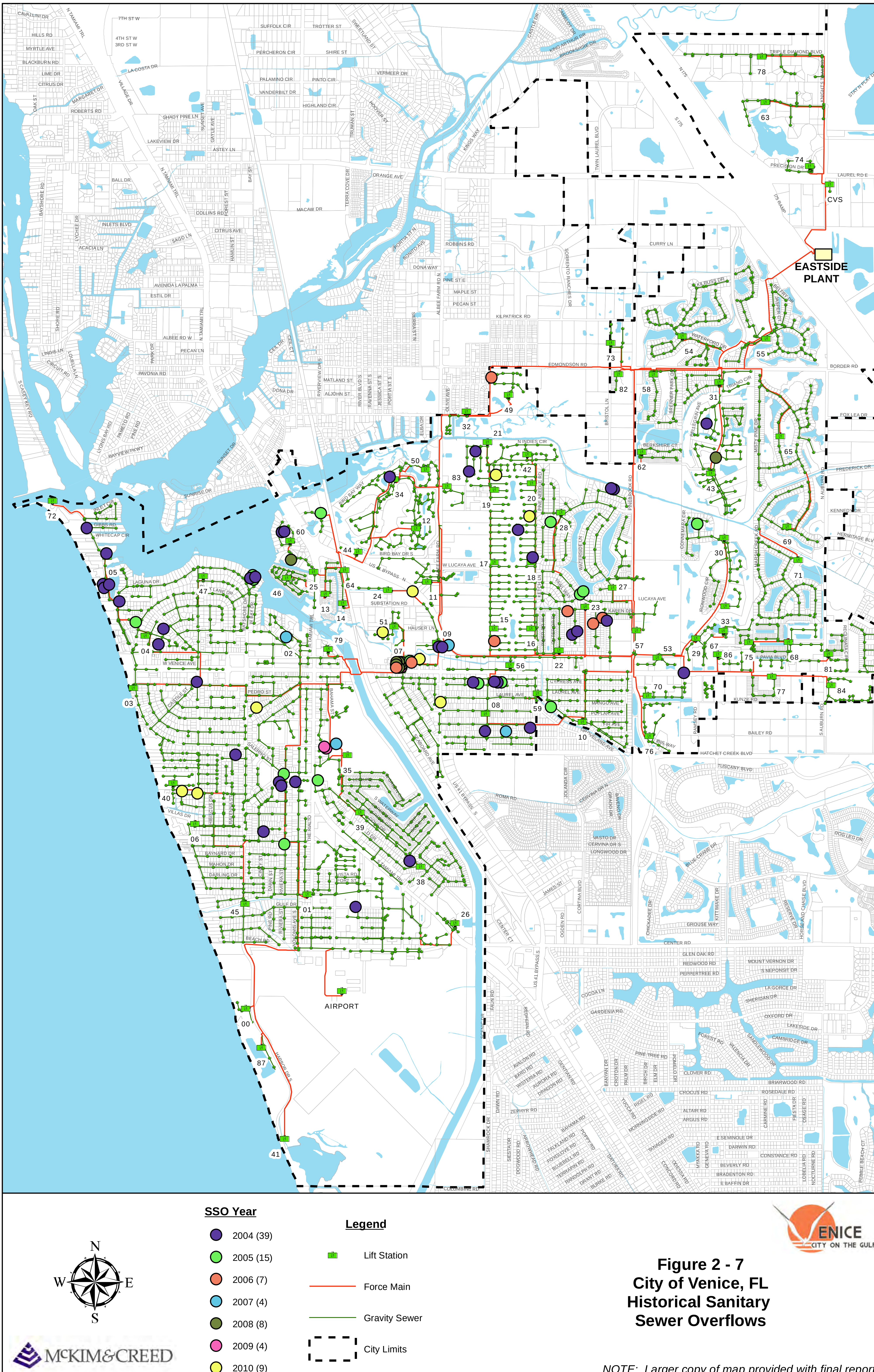


Figure 2 - 7
City of Venice, FL
Historical Sanitary Sewer Overflows

NOTE: Larger copy of map provided with final report.

There were also three relatively recent SSOs in the vicinity of Lift Station 35 along Bahama Street and Indian Avenue. These were due to grease blockages in the gravity sewer. The source of the grease should be investigated including restaurants which may not have grease traps installed.

2.6 CCTV Video Inspection Summary

As shown in **Table 2-2,** the sub-basins serviced by Lift Stations 01, 10, 14-20, 22, 27, 31, and 32 appear to have the most severe I/I of the 52 lift stations reviewed, and Lift Station 7 had the most frequent number of SSOs. Since these sewer sub-basins exhibited the greatest potential for I/I, the CCTV video inspection tapes within these sub-basins were selected and reviewed. **Table 2-3** summarizes the findings of the CCTV video tape reviews.

Table 2-3: CCTV Video Inspection Summary

LS	Tape ID	Pipe Segment	Date	Pipe Condition
00	Sharkey's	MH 06 to MH 05	N/A	New pipe, little to no flow. No apparent infiltration.
00	Sharkey's	MH A05 to MH A04	N/A	New pipe, little to no flow. No apparent infiltration.
01	Cincy Street	N/A	2001	6" clay pipe. At 150' pipe material changes to concrete. At 164' large crack at a joint with roots and steady stream of infiltration. At 196' infiltration coming through joint.
01	Cincy Street Sinkhole	N/A	2009	Short tape. No apparent infiltration.
01	Coral Street Sinkhole	MH 1-11 to MH 1-110	2009	8" clay pipe. No apparent infiltration.
01	Dawn Street	MH-93; Restarted at MH 1-101	N/A	Clay pipe. At 253 some root intrusion. At 277 pile of bubble wrap. At 419' found missing MH. Restarted at MH 1-101, low water levels. No apparent infiltration

Table 2-3: CCTV Video Inspection Summary (Continued)

LS	Tape ID	Pipe Segment	Date	Pipe Condition
01	Rialto	Unknown	2004	6″ pipe of unknown material. Moderate amount of standing water, At 51′ root intrusion and trash blocked camera. No apparent infiltration.
01	Dawn Street	MH-93; Restarted at MH 1-101	N/A	Clay pipe. At 253 some root intrusion. At 277 pile of bubble wrap. At 419′ found missing MH. Restarted at MH 1-101, low water levels. No apparent infiltration
01	Rialto	N/A	2004	6″ pipe of unknown material. Moderate amount of standing water, At 51′ root intrusion and trash blocked camera. No apparent infiltration.
01	Riveria Street	MH 1-98 to MH 1-93, MH 1-108 to MH 1-198, MH 1-108 to MH 1-108A	2008	8″ clay pipe. Dripping at several pipe joints throughout tape. At 174′ crack in pipe. MH 1-108 to MH 1-198 crack at 313′.
01	Venice Hospital	N/A	1997	54″ crack at top of pipe with steady infiltration of water.
07	Seaboard Avenue	MH 7-48 to MH 7-47, MH 7-45 to MH 7-47	N/A	MH 7-48 to 7-47: Pipe dry with build up of grit. No apparent infiltration. MH 7-45 to MH 7-47: Good condition, No apparent infiltration.
10	Area 10 – Cherry Street Behind Mango Drive	MH 10-9 to MH 10-8	2006	Low flow and a large amount of trash in pipe. No apparent infiltration.
10	Area 10 – Terrance Drive and Eastgate Drive	MH 10-52 to MH 10-51	2006	Good condition. At 157′ some cracks and some root intrusion. No apparent infiltration.
10	Cypress Ave Easement	N/A	2002	Good condition. At 135′ large pile of trash blocked camera. No apparent infiltration

Table 2-3: CCTV Video Inspection Summary (Continued)

LS	Tape ID	Pipe Segment	Date	Pipe Condition
14	Hatchett Creek	N/A	2004	Camera submerged. Unable to view pipe.
15	Guadalupe with Sinkhole	N/A	N/A	At 36' a lateral appears to be crushed with excessive root intrusion. No apparent infiltration.
27	Area 27	MH 27-43 to 27-43	2006	Good condition. No apparent infiltration.
27	Lucaya and Sleepy Hallow	N/A	2006	Excessive pipe buildup. Infiltration observed at 162'. At 344' camera block with large piece of debris.
27	Lucaya Sinkhole	N/A	2005	Pipe broken at MH. No apparent infiltration.
27	South Lake Court	MH 27-P1 to 27-P2	2008	Older pipe, fair condition. No apparent infiltration.
31	Triano Subdivision	MH 5 to 7	2003	8" PVC pipe. Minimal water in pipe. No apparent infiltration.
31	Triano Subdivision (Edmonson Road)	MH 1 to 2 MH 7 to 8 MH 2 to 4	2005	8" PVC pipe. Minimal water in pipe. No apparent infiltration.

The results of the CCTV tape review identified two areas where significant infiltration was observed flowing into the gravity collection system. The locations were on the Cincy Street and Venice Hospital tapes, but unfortunately, the associated manholes were not provided to narrow down the location. The Venice Hospital tape is dated 1997 and Cincy Street 2001, so both tapes are over 10 years old. Given the amount of infiltration observed and the age of the tapes, it is recommended a detailed field assessment be performed to identify the observed infiltration location and schedule them for repair.

3.0 POPULATION AND WASTEWATER PROJECTIONS

3.1 Population Projections

Data used to determine the population growth of Sarasota County and correspondingly the City of Venice was obtained from the Bureau of Economic and Business Research, Volume 44, Bulletin 159, June 2011, Projections of Florida Population by County 2010-2040. This document provides three different projections--low, medium and high, and suggests that the medium projection will generally provide the most accurate forecasts, while the low and high provide an indication of uncertainty that surrounds the medium forecast. The population projections provided by this document for Sarasota County are shown in **Table 3-1**. The population of the City of Venice is 5.5% of Sarasota County's population based on 2010 Census data. **Table 3-2** shows the estimated City of Venice population projections. Over the next 20 years, the projection in **Table 3-2** suggests that the population will increase by an annual average of 1.2% for the medium projection rate.

Table 3-1: Sarasota County Population Projections

Year	Projection Rate		
	Low	Medium	High
2015	385,200	400,100	417,300
2020	391,700	424,700	459,900
2025	395,800	448,600	503,700
2030	396,900	470,700	548,100
2035	395,000	490,700	592,500
2040	390,600	509,000	637,300

1. Based on BEBR Projections of Florida Population by County, 2010-2040, FPS Volume 44, Bulletin 159, June 2011

Table 3-2: City of Venice Population Projections

Population	Year				
	2010	2015	2020	2025	2030
Sarasota County Resident Population[1,6]	379,448	400,100	424,700	448,600	470,700
City of Venice Resident Population[2]	20,748	21,877	23,222	24,529	25,738
City of Venice Seasonal Population[3]	3,645	3,843	4,079	4,309	4,521
City of Venice Functional Population[4]	22,570	23,799	25,262	26,684	27,998
Share of County Population[2]	5.5%				
Seasonal Percentage[5]	17.6%				

1. Based on BEBR Medium Range Projections of Florida Population by County, 2010-2040, FPS Volume 44, Bulletin 159, June 2011.
2. Based on 2010 Census Data, which equated to 5.5%; percentage applied to subsequent years.
3. Based on the 2010 Public Supply Annual Report, City of Venice Utilities Department, as prepared for SWFWMD, seasonal percentage applied to subsequent years.
4. Sum of Resident and Seasonal Population (Assumes seasonal population is 6 months of the year).
5. Seasonal Population / Resident Population; percentage applied to subsequent years to determine seasonal projection.
6. Year 2010 based on 2010 Census Data.

Table 3-3: Historical Flow to the Eastside WRF

Year	City of Venice AADF (mgd)	Sarasota County AADF (mgd)[1]	Total AADF (mgd)
2006	1.74	1.10	2.84
2007	1.64	0.67	2.31
2008	1.86	1.16	3.02
2009	1.96	1.16	3.12
2010	2.39	1.28	3.67
2011	1.93	0.78	2.71

1. Per FDEP DMRs and City provided Sarasota County flow summaries.

3.2 Existing Flow

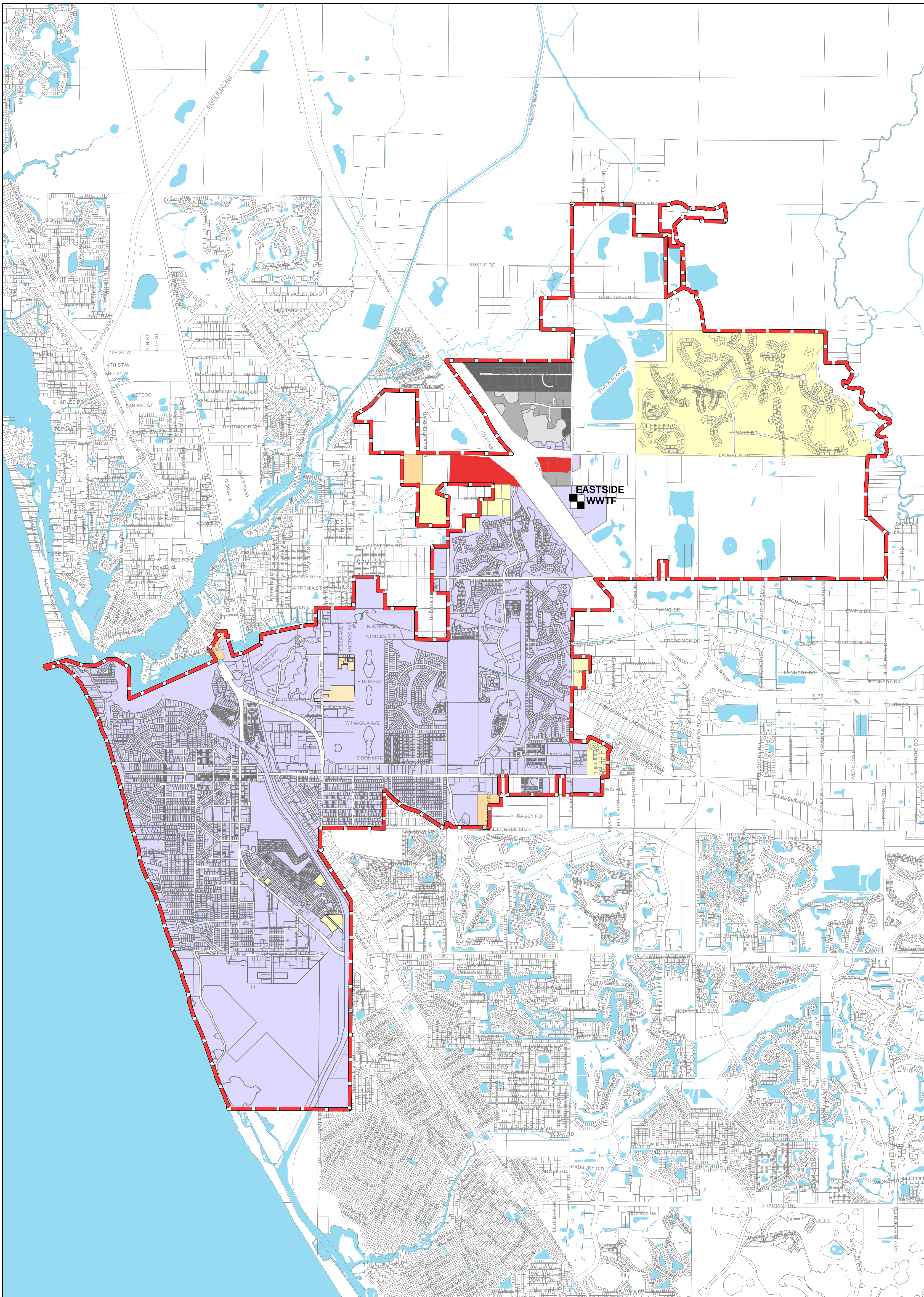
The Eastside WRF had an AADF of 3.67 mgd in 2010. Of this flow, approximately 1.28 mgd was from Sarasota County. The historical flows to the Eastside WRF are summarized in **Table 3-3**.

Several factors were considered to establish the areas within the City of Venice that produce existing wastewater flows. The gross acreage for each land use area, as provided in the WSFWP-10, includes areas outside of the City's existing wastewater collection service area. Sewer basin boundaries were adjusted with direction from City staff to expand and capture serviced land use areas that fall outside of the existing boundary line. The remaining land use areas outside of a sewer basin were subtracted from the total gross areas. Large unincorporated parcels were also subtracted from the gross area. **Figure 3-1** shows the future land use areas subtracted from the 2030 FLUM to arrive at the 2010 service area. Since flow is calculated based on land use, the land use areas not contributing to a sewer basin would overestimate flows to various sewer basins. **Table 3-4** shows the gross acreage of the future land use areas provided in the WSFWP-10 as well as the adjusted gross acreage used in this report. **Figure 3-2** shows the service areas which constitute the revised 2010 gross acreage.

Table 3-4: Gross Acreage Used to Estimate Existing Flow

Future Land Use Designation/Planning Area	WSFWP-10 Gross Acreage[1]	Revised Gross Acreage – 2010 Planning Period
Low Density Residential	2,378.98	1,230.56
Medium Density Residential	951.69	867.28
High Density Residential	33.52	33.55
Mixed Use Residential	0.6	0
Commercial	127.77	45.80
Institutional Professional	87.53	87.53
Industrial [2]	173.78	0
Industrial-Commercial [2]	124.42	0

1. Gross land use areas based on 2030 FLUM.
2. Removed due to overestimation of flow based on land use.



LEGEND

AREAS NOT REMOVED

City Boundary

Future Land Use Areas Removed From 2010 Gross Acreage

LOW DENSITY RESIDENTIAL

MODERATE DENSITY RESIDENTIAL

COMMERCIAL

INDUSTRIAL

INDUSTRIAL- COMMERCIAL

N W E S

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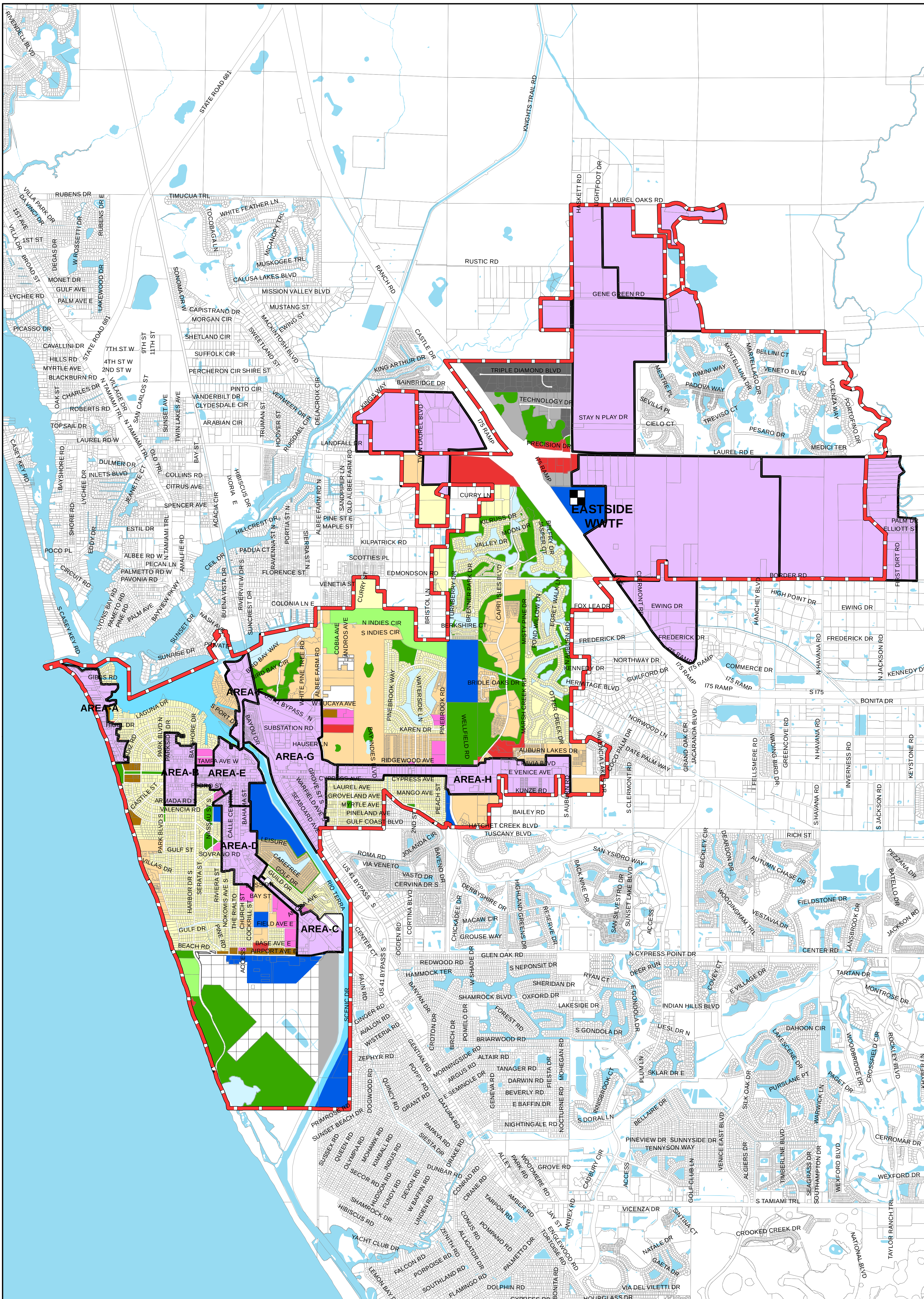
VENICE CITY ON THE GULF

Figure 3 - 1
City of Venice, FL
Wastewater Collection System
Areas Removed from 2010 Service Area



N
W E
S

McKIM&CREED

LEGEND

City Boundary

Future Land Use

- LOW DENSITY RESIDENTIAL
- MODERATE DENSITY RESIDENTIAL
- MEDIUM DENSITY RESIDENTIAL
- INSTITUTIONAL-PROFESSIONAL
- COMMERCIAL
- TRANSITION
- INDUSTRIAL
- INDUSTRIAL- COMMERCIAL
- AIRPORT OPERATIONS
- PUBLIC BUILDINGS & FACILITIES
- RECREATION & OPEN SPACE
- CONSERVATION
- MARINE PARK
- GREENWAY/RIVER BUFFER
- WATERWAYS
- PLANNING AREA

PLANNING AREAS

A - Tarpon Center/Esplanade
B - Heritage Park
C - Southern Gateway
D - Island Professional
E - City Center
F - Northern Seaboard
G - Seaboard
H - Eastern Seaboard

VENICE
CITY ON THE GULF

Figure 3 - 2
City of Venice, FL
Wastewater Collection System
Existing Service Area

Lift Stations 63 and 78, which are entirely in the industrial and/or industrial-commercial land uses, also had their sewer basins subtracted from the total gross areas. Per the 2007 Venice Comprehensive Plan, 100% of the gross acreage for industrial and industrial commercial is allowed for non-residential development. The total area of these sub-basins was therefore used to arrive at the ERU and subsequent flow projections. Review of existing aerial imagery shows that a significant percentage of these sub-basins have large storm water ponds or wetlands. Since these non-developable areas are represented as developable areas, the build-out flow and corresponding 2010 flows are over stated in these areas and are attributed to the overflow conditions simulated in the existing conditions hydraulic model at Lift Stations 63 and 78. In order to better approximate the influent into Lift Stations 63 and 78, water meter records for 2011 were evaluated with the assumption that the amount of water consumed equals the amount of wastewater produced. The revised existing flows into Lift Stations 63 and 78 were significantly lower using this methodology and were assumed to be a better approximation of existing wastewater flows. The sharp decrease in predicted flows from these two sewer sub-basins means that the remaining sewer sub-basins contributed more flow per ERU. The corresponding modification to the level of service is discussed in **Section 3.4**.

The planning areas shown on the 2030 FLUM are a mix of developed and undeveloped areas. South Laurel, Shakett Creek, Knights Trail, and Gene Green planning areas are outside of an existing sewer sub-basin and are relatively undeveloped. These planning areas were therefore excluded from the existing conditions model. The remaining planning areas have existing development and contribute to existing flows. Flows from these planning areas were estimated by gross acreage and assumed to be developed in accordance with the percentages in **Table 3-5**.

Table 3-5: City of Venice Build-out and Growth Percentages

Period	Percent Developed of Build-out	Percent Growth From 2010
2010	33.3%	-
2015	35.1%	5.4%
2020	37.3%	11.9%
2025	39.4%	18.2%
2030	41.3%	24.0%

1. Percent growth from 2010 based BEBR Projections of Florida Population by County, 2010-2040, FPS Volume 44, Bulletin 159, June 2011.

All of the JPA areas shown on the 2030 FLUM were excluded since they do not currently contribute flow. Existing flows were estimated for lift stations without a formally delineated sewer basin and the corresponding land use. The flows from these lift stations were estimated using available information such as the composition of the area and aerial imagery. **Table 3-6** shows the subject lift stations and their estimated flows. More accurate information should be used to update these estimates.

Table 3-6: Estimated Lift Station Flows

Lift Station	Estimated Average Daily Flow (gpm)	Estimated Max Day Flow (gpm)
59	3,024	6,048
35	14,177	28,354
Airport	3,600	7,200
41	1,200	2,520
87	3,600	7,200
Total	**25,601**	**51,322**

Based on the available information, it was estimated that 25,601 gpd of the ADF is from lift stations without a formally delineated sub-basin and corresponding land use. As discussed further in **Section 3.4,** the 2010 average daily flow is 2,074,800 gpd and the maximum day flow is 4,149,600. The AADF from Sarasota County is 1.28 mgd per the flow summaries provided by the City of Venice.

As previously determined, the annual average daily I/I from the City's collection system is 369,000 gpd. This flow was added to the maximum daily flow to arrive at the maximum day wet weather flow. The total flows to the Eastside WRF are summarized in **Table 3-7** below for the 2010 scenario.

Table 3-7: Existing Flow Totals

Source	Average Day Dry Weather (gpd)	Maximum Day Wet Weather (gpd)
City of Venice	2,074,800	4,149,600
Sarasota County	1,280,000	2,560,000
I/I	0	369,000
Total	**3,354,800**	**7,078,600**

3.3 Future Flows

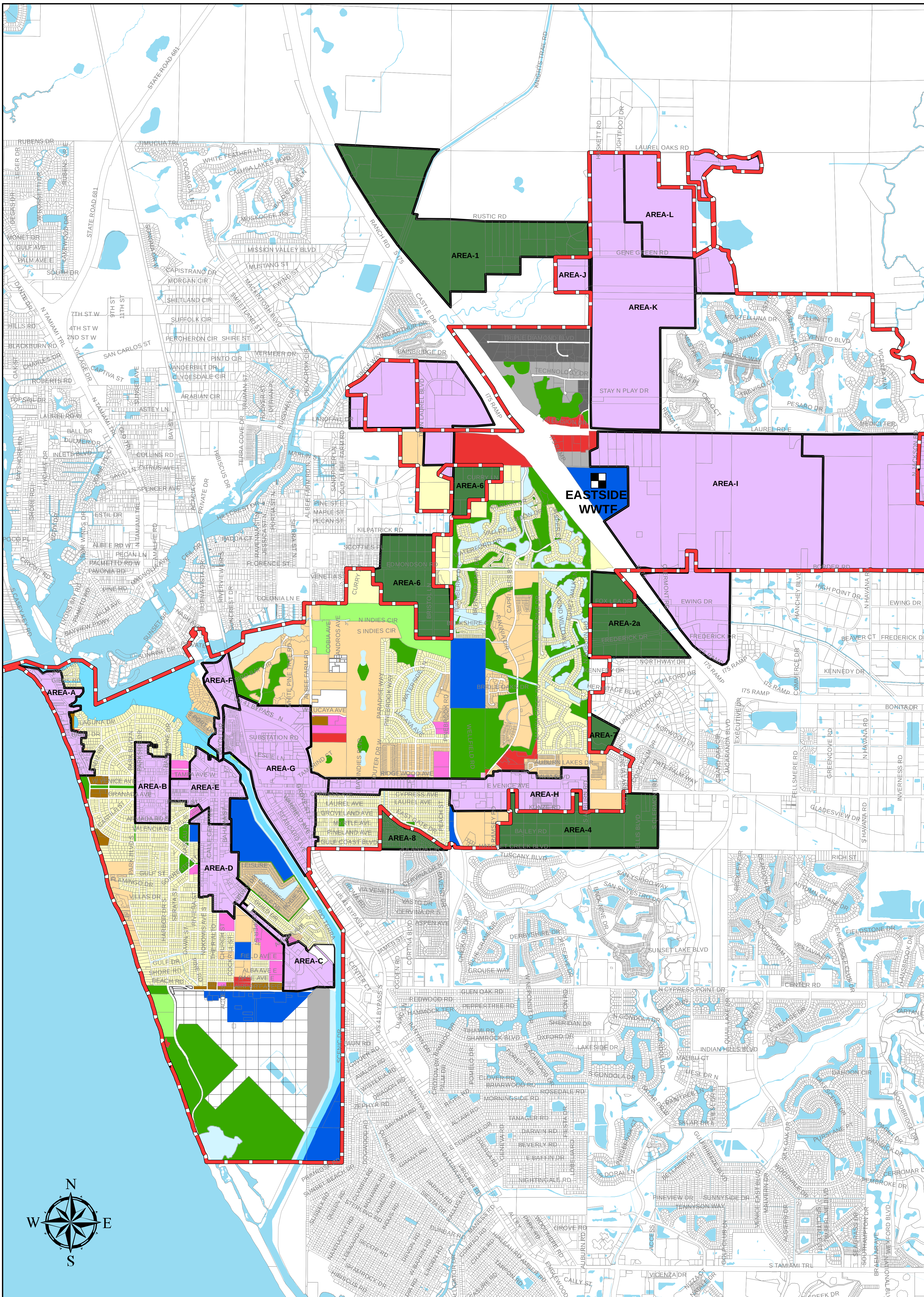
The gross acreage was assumed to increase in the future planning periods starting in 2015. The increase in gross acreage was contributed from several areas. Areas within the FLUM, which were outside of the revised sewer basin boundaries, were classified as future sewer basins and assigned an identification number. Flows from these basins were estimated by gross acreage and assumed to be 41% developed in 2030 and 0% in 2010. The flows for the 2015, 2020, and 2025 planning periods were interpolated. The gross acreages for the future planning periods are provided in **Table 3-8**.

Table 3-8: Gross Acreage Used to Estimate Future Flow

Future Land Use Designation/Planning Area	WSFWP-10 Gross Acreage	Gross Acreage – 2015, 2020, 2025, 2030 Planning Period
Low Density Residential	2,378.98	1,317.34
Medium Density Residential	951.69	913.39
High Density Residential	33.52	33.55
Mixed Use Residential	0.6	0
Commercial	127.77	127.42
Institutional Professional	87.53	87.53
Industrial	173.78	14.02
Industrial-Commercial	124.42	0

Planning areas South Laurel, Shakett Creek, Knights Trail, and Gene Green, which were excluded from the 2010 planning period, were included in the future planning periods. Flows from these planning areas were estimated by gross acreage and assumed to be 41% developed in 2030 and 0% in 2010. The remaining planning areas were assumed to be developed in accordance with the build-out percentages in **Table 3-5**.

Not all of the JPA areas will have sewer serviced by the City as shown in **Figure 3-3**. JPA Areas 2B, 3, and 5, will have sewer serviced by Sarasota County according to the Venice Comprehensive Plan. The remaining JPA areas were assumed to grow in accordance with the BEBR medium range projections in accordance with the WSFWP-10 and **Table 3-5**. The projected flows from the City of Venice for the future planning periods are provided in **Table 3-9**.



LEGEND

- City Boundary
- PLANNING AREA
- JPA/ILSBA AREA

Future Land Use

- LOW DENSITY RESIDENTIAL
- MODERATE DENSITY RESIDENTIAL
- MEDIUM DENSITY RESIDENTIAL
- INSTITUTIONAL-PROFESSIONAL
- COMMERCIAL
- TRANSITION
- INDUSTRIAL
- INDUSTRIAL- COMMERCIAL
- AIRPORT OPERATIONS
- PUBLIC BUILDINGS & FACILITIES
- RECREATION & OPEN SPACE
- CONSERVATION
- MARINE PARK
- GREENWAY/RIVER BUFFER
- WATERWAYS

PLANNING AREAS

A - Tarpon Center/Esplanade
B - Heritage Park
C - Southern Gateway
D - Island Professional
E - City Center
F - Northern Seaboard
G - Seaboard
H - Eastern Seaboard

JPA/ILSBA AREAS

1 - Rustic Rd
2a - Auburn Rd to I-75
2b - I-75/Jacaranda Blvd
3 - Border Rd to Myakka River
4 - South Venice Ave
5 - Laurel Rd Mixed Use
6 - Pinebrook Rd
7 - Auburn Rd
8 - Gulf Coast Blvd

VENICE CITY ON THE GULF

Figure 3 - 3
City of Venice, FL
Wastewater Collection System
Future Service Area

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TABLE 3-9: City of Venice Projected Future Flows

Planning Year	Average Daily Flow (gpd)	Maximum Day Flow (gpd)
2015	2,635,199	5,270,399
2020	3,019,173	6,038,346
2025	3,398,252	6,796,504
2030	3,764,686	7,529,373

Future projected flows from Sarasota County to the Eastside WRF were referenced from the Sarasota County Wastewater Management Plan dated June 2009. Several scenarios regarding the amount of flow sent to the Eastside WRF are discussed in the report. For planning purposes, the worst case scenario was selected. **Table 3-10** shows the projected flows outlined in the report.

Table 3-10:Sarasota County Projected Future Flows

Planning Year	Maximum Month Average Daily Flow (MMADF) (mgd)[1]
2006	1.50
2020	2.86
2050	3.10

1. Per the Sarasota County Wastewater Management Plan, June 2009.

The flows provided in the Sarasota County Wastewater Management Plan are MMADF and not AADF. The MMADF peaking factor was necessary in order to determine the AADF for input into the model. The County flow data provided by the City was used to determine the MMADF peaking factor. The County flow data to the Eastside WRF was reviewed from January 2006 through December 2011 and is summarized in **Table 3-11**.

Table 3-11: Sarasota County MMADF Peak Factor

Year	AADF (mgd)	MMADF (mgd)	Peaking Factor (MMADF/AADF)
2006	1.10	2.63	2.39
2007	0.67	1.48	2.20
2008	1.16	1.39	1.20
2009	1.16	1.57	1.35
2010	1.28	2.22	1.73
2011	0.62	1.02	1.52
Average			**1.73**

The flows provided in **Table 3-12** were interpolated to arrive at the MMADF 2015, 2020, 2025 and 2030 planning years. The MMADF peak factor in **Table 3-11** was used to determine the corresponding AADF as shown in **Table 3-12**.

Table 3-12: Projected Sarasota County AADF Flows

Planning Year	Sarasota County MMADF (mgd) [1]	Sarasota County AADF (mgd) [2]
2015	2.54	1.46
2020	2.86	1.65
2025	3.1	1.79
2030	3.1	1.79

1. Per the Sarasota County Wastewater Management Plan, June 2009, Interpolated.
2. Divided by MMADF peak factor of 1.73.

The AADF flows in **Table 3-12** were utilized in the model as a point inflow at node J-152 to represent the interconnect between Sarasota County and the City of Venice. Maximum Day was determined by multiplying the AADF by two.

As previously determined, the 2010 annual average daily I/I from the City's collection system is 369,000 gpd. This flow was added to the 2010 maximum daily flows to arrive at the maximum day wet weather flows. The amount of flow from I/I in the future is a function of the success of the City's I/I reduction program and the increase in the collection systems age. For planning purposes, it was assumed that the increase in I/I as the system ages would be offset by the City's reduction in I/I. The projected I/I flow during the future planning years was therefore assumed to remain at 369,000 gpd. The total flows to the Eastside WRF as inputted in to the model are summarized below in **Table 3-13** for the future planning scenarios.

Table 3-13: Future Planning Flow Totals

Source	Planning Year 2015	
	Average Day Dry Weather (gpd)	Maximum Day Wet Weather (gpd)
City of Venice	2,635,199	5,270,399
Sarasota County	1,460,000	2,920,000
I/I	0	369,000
Total	**4,095,199**	**8,559,399**
Source	**Planning Year 2020**	
	Average Day Dry Weather (gpd)	**Maximum Day Wet Weather (gpd)**
City of Venice	3,019,173	6,038,346
Sarasota County	1,650,000	3,300,000
I/I	0	369,000
Total	**4,669,173**	**9,707,346**

Table 3-13: Future Planning Flow Totals (Continued)

Source	Planning Year 2025	
	Average Day Dry Weather (gpd)	Maximum Day Wet Weather (gpd)
City of Venice	3,398,252	6,796,504
Sarasota County	1,790,000	3,580,000
I/I	0	369,000
Total	**5,188,252**	**10,745,504**
Source	**Planning Year 2030**	
	Average Day Dry Weather (gpd)	**Maximum Day Wet Weather (gpd)**
City of Venice	3,764,686	7,529,373
Sarasota County	1,790,000	3,580,000
I/I	0	369,000
Total	**5,554,686**	**11,478,373**

Projections of future growth should be revised as specific information regarding a particular area's growth is planned. This will allow the projections to best reflect the planned growth of the City and predict the corresponding flows.

3.4 Level of Service

Per the Venice Comprehensive Plan, the current LOS for wastewater is 123 gpd/ERU based on average annual flow and 244 gpd/ERU based on the maximum day flow. The Venice Comprehensive Plan requires that the LOS be re-evaluated as part of this Wastewater Master Plan.

The origin of the current level of service is undefined in the Venice Comprehensive Plan. **Table 3-14** shows the values used to re-evaluate the current LOS.

Table 3-14: Level of Service for Wastewater

Year	Resident Population[1]	Total ERUs[2]	Average Flow (gpd)[4]	Average Demand (gpd/ERU)	Maximum Day Flow (gpd)[3]	Maximum Demand (gpd/ERU)
2010	20,748	12,426	2,011,512	162	4,098,278	324

1. Based on 2010 Census Data.
2. Total Equivalent Residential Units (ERUs) represents the number of connections to the City's wastewater collection system expressed in terms of residential units. ERUs are independent of seasonal population variation (functional versus resident population).
3. Maximum day flow equals average day with 2.0 peaking factor.
4. The total average daily flow is reduced by 37,687 gpd to account for flow from LS 63 and LS 78 basins which were not calculated based on ERUs. Including flow from lift station without sub-basins (25,601 gpd), add 63,288 gpd to arrive at average daily flow of 2,074,800 gpd.
5. Based on BEBR Projections of Florida Population by County, 2010 – 2040, FPS Volume 44, Bulletin 159 June 2011.

The average flow is based on the resident population multiplied by 100 gpdpc as discussed in **Section 5.2.1**. Based on the 2010 Census population of 20,748 and 100 gpdpc, the Average Daily Flow is 2,074,800 gpd in 2010. This flow, for use in determining the revised LOS, was reduced by 24,601 gpd to reflect the amount of flow from lift stations without sub-basins and corresponding land use area. It was also reduced by 37,687 gpd to reflect flow from Lift Stations 63 and 78 which had their flow estimated by water meter data and not by land use area. The adjusted average flow is 2,011,512 gpd for the purpose of determining the revised LOS. The total ERUs decreased from the WSFWP-10 as a result of the reduced gross acreages discussed in **Sections 3.2** and **3.3**. The adjusted average daily flow divided by the number of revised ERUs yields 162 gpd/ERU as shown in **Table 3-14**. The annual average LOS therefore should increase from 123 gpd/ERU to 162 gpd/ERU. The maximum day peaking factor is currently 1.98. In order to verify the maximum day peak factor the MDF/ADF ratio was evaluated from 2007 to 2010. Per **Table 3-15,** the historical average maximum day peaking factor is 2.01. Based on this data a peak factor of 2.0 is appropriate to estimate revised maximum day flow LOS. The revised maximum day flow LOS is therefore 324 gpd/ERU.

TABLE 3-15: Average Maximum Day Peak Factor

Year	ADF	MDF	Ratio
2006	2.84	5.08	1.79
2007	2.31	3.42	1.48
2008	3.02	8.15	2.70
2009	3.12	5.83	1.87
2010	3.67	8.08	2.20
2011	**2.71**	**4.88**	**1.80**
Average[2]	**2.99**	**6.11**	**2.01**

1. Flows based on FDEP DMRs.
2. Average based on 2006 through 2010 since determined prior to the end of 2011 for incorporation in hydraulic model analysis.

4.0 REGULATORY REVIEW AND REQUIREMENTS

4.1 Industrial Pretreatment Program

The current FDEP operating permit does not require an industrial pretreatment program. The FDEP requires an industrial pretreatment program if a wastewater facility receives flow from a significant user which is defined as a service that contributes at least 25,000 gpd on average, or 5% or greater of the plant's organic or hydraulic capacity, or has the potential to disrupt facility operations. This rule applies to facilities that discharge to a surface water body or a reclaimed water system. The goal of the program is to prevent operational disruptions, water quality compliance issues, and protect the public from any pass-through of contaminants via the reclaimed water system. The future land use map provided in the most recent Capacity Analysis Report suggests that some minor land area in the northeast portion of the City may be converted to commercial or industrial land use. Providing that any new future customer meets the above criteria, an industrial pretreatment program may be implemented. It should be noted that although a service may meet some of the significant user conditions, if the facility authority deems the service as having no impact on operations or water quality standards, it is at the discretion of the facility authority to consider the customer not a significant user.

4.2 Water Reclamation Facility Operating Permit

The new WRF permit was issued on December 12, 2011 and has an expiration date of December 11, 2016. The FDEP has implemented a program allowing facilities to apply for extended permit periods (greater than five years). The City does not meet the criteria for the extended schedule due to the surface water outfall and injection well components of the system.

The operating permit includes a 3.0 MGD outfall to Curry Creek, a Class III water body. Permitted surface water discharges may potentially be affected by the TMDL regulations as more stringent water quality parameters may be required. The plan is regulated by FDEP with the goal of reducing contaminant loading to surface water bodies which include contributing flows from wastewater facility outfalls. The program is being implemented in a phased approach where impaired water bodies identified by the FDEP are assessed, TMDL limits set for particular parameters, and affected parties are allowed to comment. The FDEP has developed a map showing priority water bodies that will be impacted by the program. The figure indicates that the closest targeted water body to the City is Gottfried Creek which is located south of Venice, beyond the wastewater service area.

There is no impending rule change prohibiting outfalls; however, stringent water quality requirements and the promotion of beneficial reuse have made surface water outfalls a less attractive disposal option. The most recent Operation and Maintenance Report provided in the operating permit renewal application indicates an exceedance for dichlorobromomethane at nearly twice the Environmental Protection Agency's maximum contaminant level. Sampling for the organic contaminant is only required when the facility utilizes the outfall to the Creek. It was noted in the Report that since this outfall was only utilized once in a five-year period, only one sample was available and trending could not be verified. The City has seven other disposal options that include the reuse system, several golf course ponds, and a connection to Sarasota County; thus use of the Curry Creek outfall is a rare occurrence as demonstrated in the number of sampling events. The City only discharges effluent to the Curry Creek outfall when these other disposal options are exhausted since the water quality requirements are more stringent, requiring limits on phosphorus, nitrogen, and specific disinfection byproduct compounds. Additionally, the permit requires that the Curry Creek outfall meet chlorine residual concentrations for disinfection and maximum chlorine residual concentrations for dechlorination.

The dechlorination step equates to greater chemical consumption, hence, higher operation and maintenance costs.

The current flow and effluent monitoring locations are provided in **Table 4-1** and **4-2** below. Effluent limitations for each discharge are provided in the FDEP Operation Permit located in **Appendix D**.

Table 4-1: Flow Monitoring Locations

Monitoring Location Site Number	Location Description
FLW-01	Influent flow meter
INF-01	At headworks prior to treatment
FLW-02	Prior to discharge at Curry Creek
FLW-03	Prior to discharge to the City of Venice Master Reuse System
FLW-04	Prior to discharge to the Sarasota County Master Reuse System
FLW-05	Prior to discharge to City of Venice RO
FLW-06	Flow meter in Curry Creek
FLW-07	(FLW-03) – (FLW-05 + FLW-04 + FLW-02)

Table 4-2: Effluent Monitoring Locations

Monitoring Location Site Number	Location Description	Description
EFA-01	After disinfection, prior to dechlorination	Effluent
EFB-01	After filtration, prior to disinfection	Turbidity & TSS
EFD-01	Prior to discharge at Curry Creek	Effluent
EFF-001	RO concentrate discharge at D-002 of industrial Wastewater Permit No. FL0035335	Disinfection By-Product
OTH-01	On-site	Rain Gauge

4.3 Operating Protocol

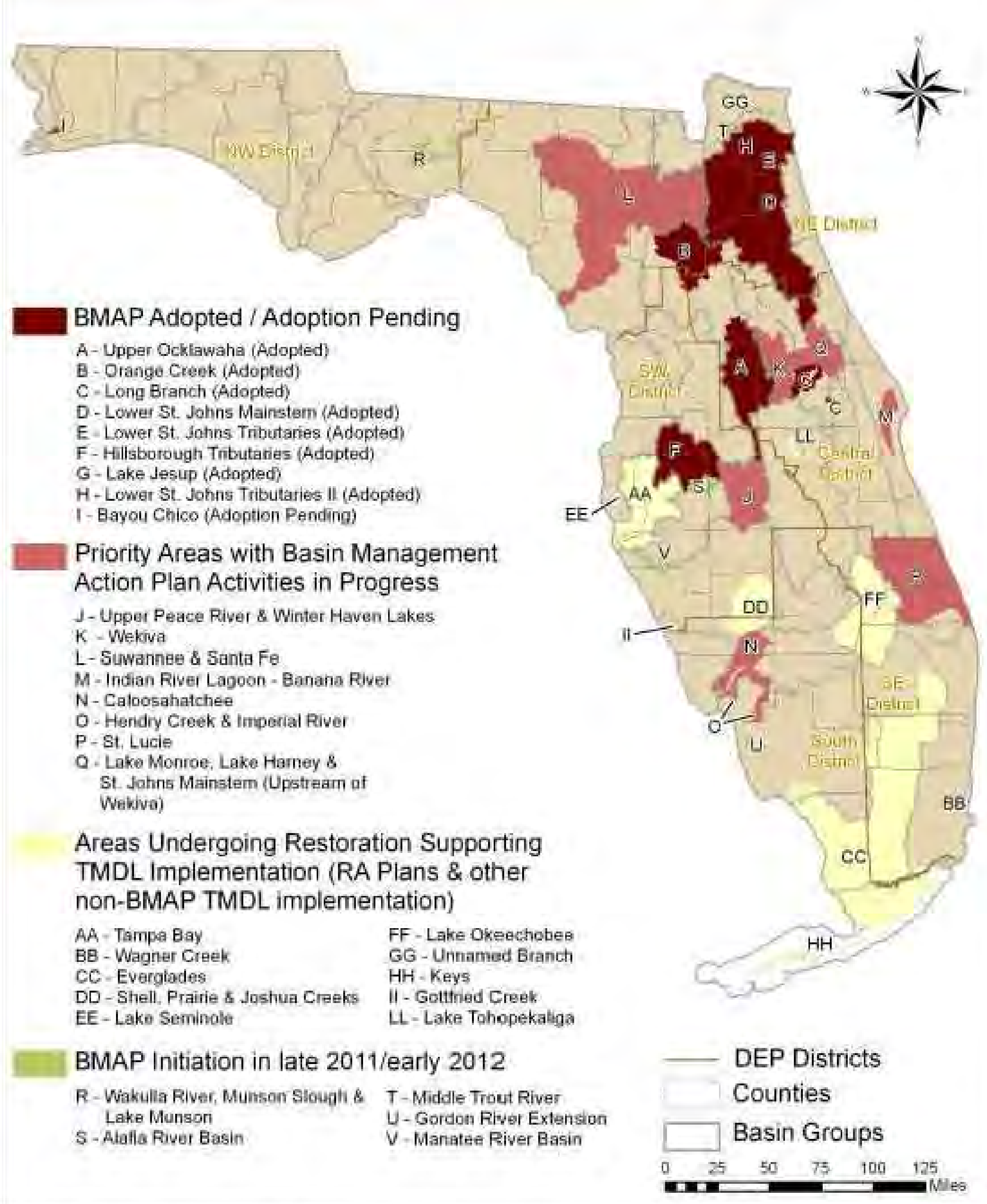
The FDEP requires wastewater facilities that produce reclaimed water for public access develop an operating protocol to ensure reuse water quality standards are met. The FDEP requires that the operating protocol be updated periodically. The most recent update occurred with the submittal of the facility permit renewal and was approved in August 2011. The next update would be required with the next facility operating permit renewal at a minimum and with changes in process or monitoring equipment.

4.4 Regulatory Reporting

Regulatory reporting is expected to remain the same with the exception of the Reuse Reports required by both the FDEP and SWFWMD. It is anticipated that these reports will be consolidated, however a timeline has not been set for this change.

4.5 Future Regulation Evaluation

The TMDL program is ongoing, thus Curry Creek could potentially become a targeted water body for assessment and more stringent water quality requirements in the future. **Figure 4-1** shows the current TMDL Project Implementation Activities.



Florida Department of Environmental Protection
Watershed Planning & Coordination Section (850) 245-8555
WPC Contact: John.Abendroth@dep.state.fl.us
GIS Contact: Ronald.Hughes@dep.state.fl.us

http://www.dep.state.fl.us/water/watersheds/bmap.htm

TMDL Project Implementation Activities

July 2011

MCKIM&CREED
378 Interstate Court
Sarasota, Florida 34240
Phone: (941)379-3404, Fax: (941)379-3530
EB0006691
www.mckimcreed.com

VENICE CITY ON THE GULF

CITY OF VENICE
WASTEWATER
MASTER PLAN

AUGUST
2012

FIGURE
4-1

S:\5883\0001\80-Drawings\Figure 4-1.dwg, 8/6/2012 9:30:24 AM, Brian Naught

5.0 WASTEWATER COLLECTION SYSTEM

5.1 Existing Collection System

5.1.1 Gravity Sewer

All 8-inch and greater gravity sewers identified as part of the backbone system described in **Section 5.2.2** were evaluated to determine if there are any existing hydraulic deficiencies. The criteria used to identify hydraulic deficiencies are discussed in **Section 5.2.5**. The total length of gravity sewer within the City limits that was included in the backbone system is approximately 37,200 LF. **Table 5-1** shows the modeled gravity sewer lengths based on diameter. The evaluation consisted of four different sets of data. First, a hydraulic model was created to simulate existing conditions in the collection system. The model results will indicate if there are capacity issues in the gravity system that need to be addressed. The second tool is the historical records the City has on SSOs. A pattern of SSOs in the same area suggest the gravity pipeline may require further evaluation. The third tool is comparing lift station run times to rainfall data. When there is a high correlation between rainfall and flows into a lift station, the gravity pipelines may have failures which allow a large amount of I/I into them. An immediate increase in lift station run time during a rain event suggests a large amount of inflow may be entering the system. The final tool was reviewing previously performed CCTV inspections using the SSO and lift station run time data to narrow down gravity sewers that warrant the CCTV tape to be reviewed.

Table 5-1: Modeled Gravity Sewer Lengths

Gravity Sewer Diameter (inches)	Length (ft)
6	265
8	24,653
10	8,403
12	119
15	2,741
21	360
24	659
Total	**37,200**

5.1.2 Force Main

The total length of force main within the City limits is approximately 176,658 LF. **Table 5-2** shows the modeled force main lengths based on diameter. When the Island Beach WRF was decommissioned, flows were reversed through some of the force mains to covey wastewater to the Eastside WRF. All force mains in the City's collection system were hydraulically evaluated by simulating them in the hydraulic model. The model was used to identify segments of force main that have excessive flow and pressures or inefficient routing.

Table 5-2: Modeled Force Main Lengths

Force Main Diameter (inches)	Length (ft)
2	4,889
3	1,048
4	23,046
6	53,566
8	31,855
10	25,467
12	10,948
16	11,084
20	14,755
Total	**176,658**

5.1.3 Lift Stations

The City operates 83 lift stations. Twenty-one of these lift stations are monitored by the City's SCADA system which records operational data such as the number of pump starts and pump run time. At this time, none of the lift stations have meters for flow measurement. The pump curves for all lift station pumps were identified by a combination of records from the City and physically pulling the pumps from Lift Stations 01, 02, 06, 31, 34, 45, and 53 prior to being entered into the model. The model will simulate the lift stations and be used to identify pump operational problems such as turning off due to excessive pressure in the system or having undersized pumps relative to the amount of inflow it receives.

5.2 Model Development, Methodology and Evaluation

5.2.1 Developing Model Demands and Flows

As discussed in **Section 3.2,** the flow from the City of Venice in 2010 was 2.39 mgd. This flow rate is approximately 22% higher than the previous year. Per the Updated CAR dated August 16, 2011 prepared by Malcolm Pirnie, the anticipated flow in 2010 was 1.94 mgd. City of Venice flows as high as 2.39 mgd are not predicted in the CAR until the year 2025. The difference between the predicted and actual flow therefore requires further evaluation before entry into the existing conditions model.

Per **Table 5-3,** the flow per capita has generally increased from 2007 to 2010 with the highest value of 115 gpcpd in 2010. Data outlined in the CAR predicted a per capita flow rate of 86 gpdpc. This was based on data from 2007 to 2009 since only data through April 2010 was available for that report. The CAR describes the increase 2010 flows can be attributed to an above average number of seasonal residents combined with an unusually high amount of precipitation during the normally dry winter months. The frequency distribution of rainfall from 1915 to 2010 was evaluated to determine how unusual the amount of rainfall was in 2009 and 2010. Per **Figure 5-1,** the yearly rainfall in 2009 and 2010 was within the range of 45 to 55 inches which occurred 48% of the time between 1915 and 2010. Review of the 2009 rainfall totals for January through March from 1915 to 2010 shows that rainfall during these months were in the bottom 8% of historical totals (See **Figure 5-2**). January through March of 2010 was in the top 16.5% of historical rainfall totals and therefore relatively high. Based on this percentage the large amount of rainfall in early 2010 is anticipated to occur approximately once every 6 years. The large amount of rainfall in late 2009 and early 2010 is therefore relatively frequent and was considered when determining the revised per capita flow.

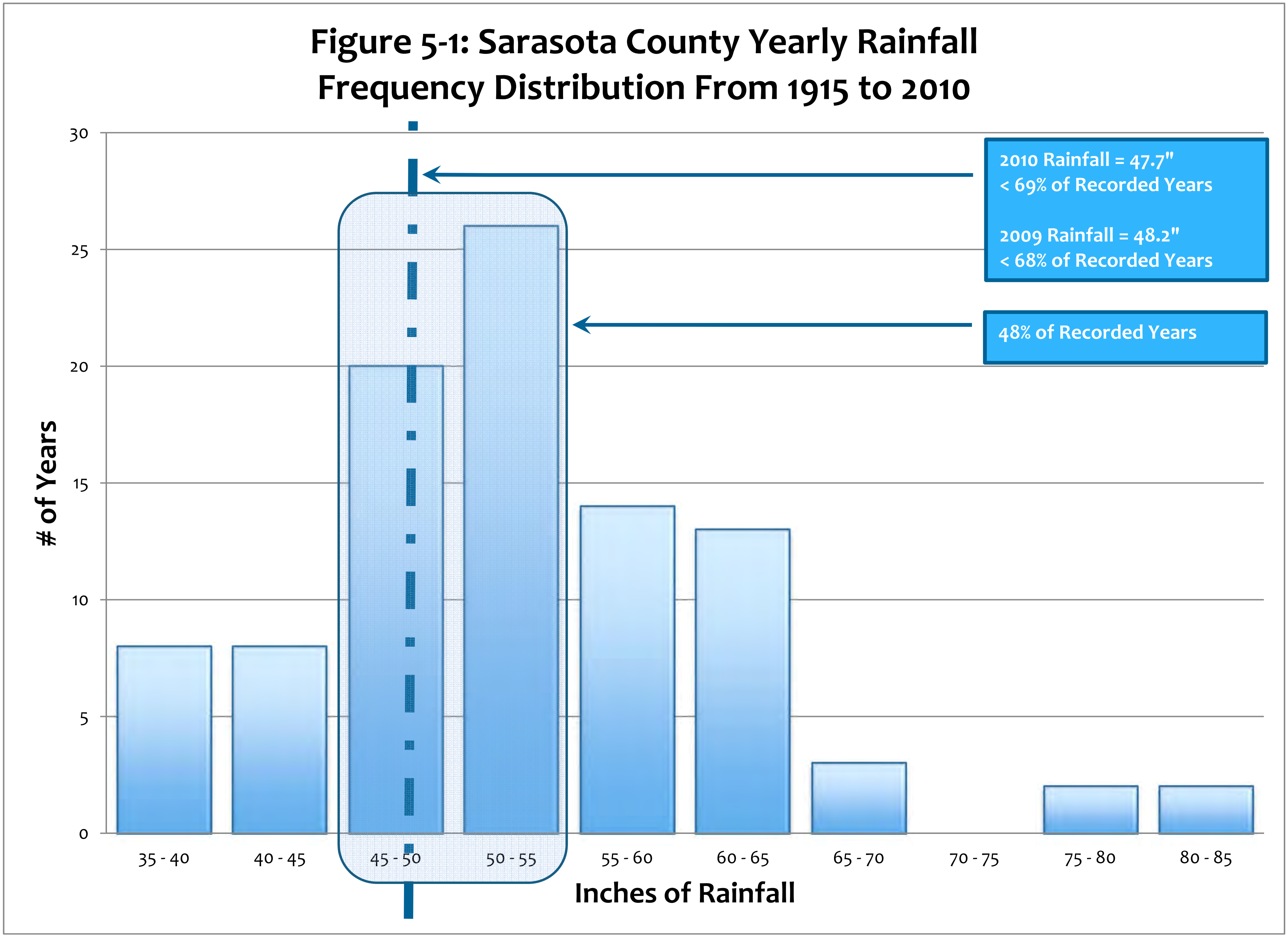


Figure 5-1: Sarasota County Yearly Rainfall
Frequency Distribution From 1915 to 2010
# of Years
30
25
20
15
10
5
0
35 - 40
40 - 45
45 - 50
50 - 55
55 - 60
60 - 65
65 - 70
70 - 75
75 - 80
80 - 85
Inches of Rainfall
2010 Rainfall = 47.7"
< 69% of Recorded Years
2009 Rainfall = 48.2"
< 68% of Recorded Years
48% of Recorded Years

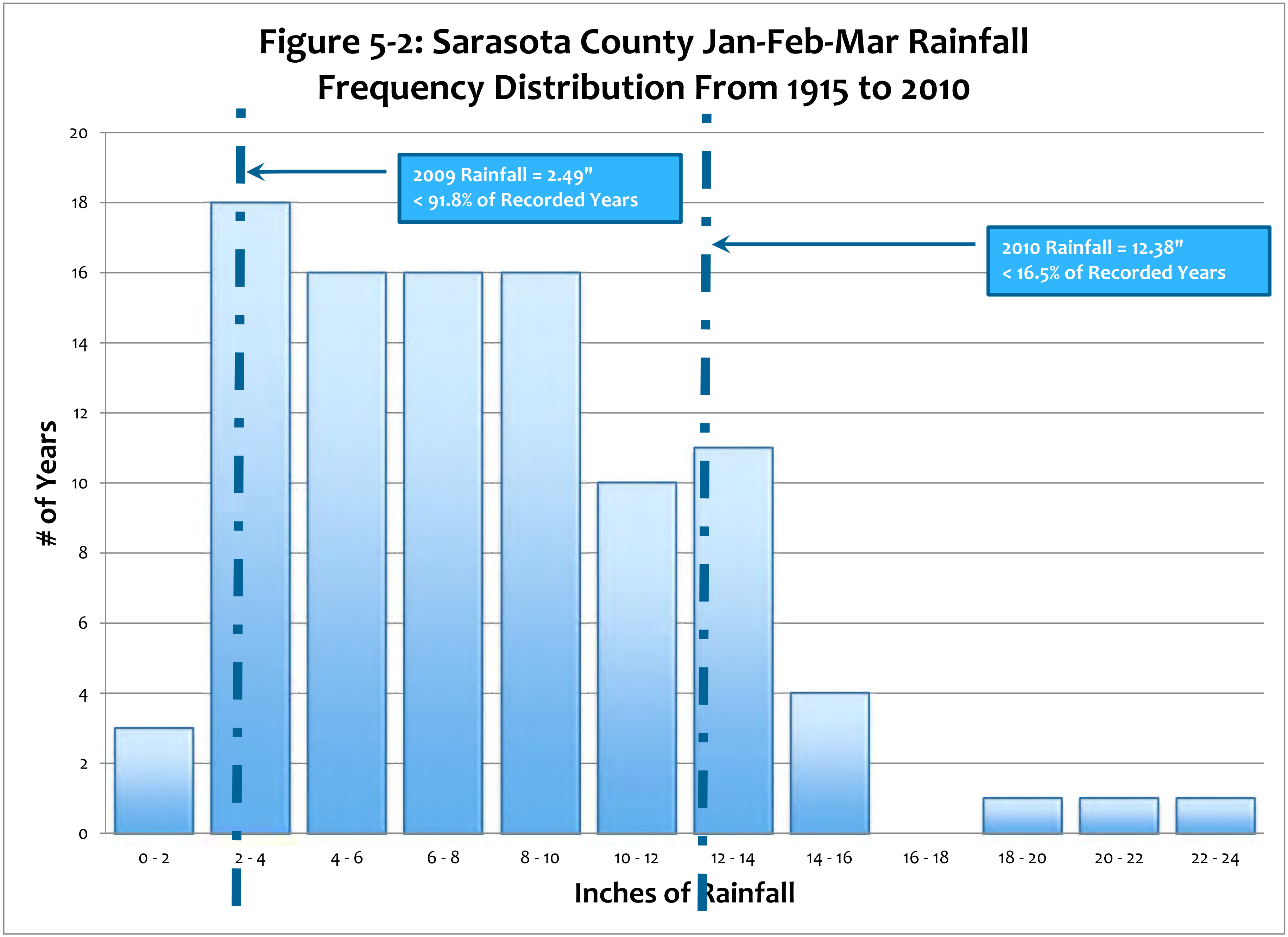


Figure 5-2: Sarasota County Jan-Feb-Mar Rainfall
Frequency Distribution From 1915 to 2010
2009 Rainfall = 2.49"
< 91.8% of Recorded Years
2010 Rainfall = 12.38"
< 16.5% of Recorded Years
# of Years
0
2
4
6
8
10
12
14
16
18
20
0 - 2
2 - 4
4 - 6
6 - 8
8 - 10
10 - 12
12 - 14
14 - 16
16 - 18
18 - 20
20 - 22
22 - 24
Inches of Rainfall

During the five-year period from 2006 to 2010, the average flow is 89 gpcpd. A value of 100 gpcpd was selected to represent existing flows in the model since it reflects a conservative middle ground between 2009 flows and the high flows seen in 2010. The value of 100 gpcpd is also used in the 10-States Standards for wastewater design. Using 100 gpcpd and the City's population of 20,748, the total flow entered into the existing conditions model is 2,074,800 gpd. Since the per capita flow represents AADF divided by the resident population, as shown in **Table 5-3**, the per capita flow of 100 gpdpc includes flow from seasonal residents.

Table 5-3: City of Venice Per Capita Flow

Year[3]	City of Venice AADF (mgd)[1]	City of Venice Resident Population[2]	Per Capita Flow (gpdpc)
2006	1.74	21584	81
2007	1.64	22149	74
2008	1.86	22146	84
2009	1.96	21845	90
2010	2.39	20748	115

1. Flows based FDEP DMRs.
2. Based on BEBR Medium Range Projections of Florida Population by County, 2010-2040, FPS Volume 44, Bulletin 159, June 2011 and 2010 Census Data.
3. 2011-2041 BEBR projection not published with 2011 Venice population data.

5.2.2 Baseline Development

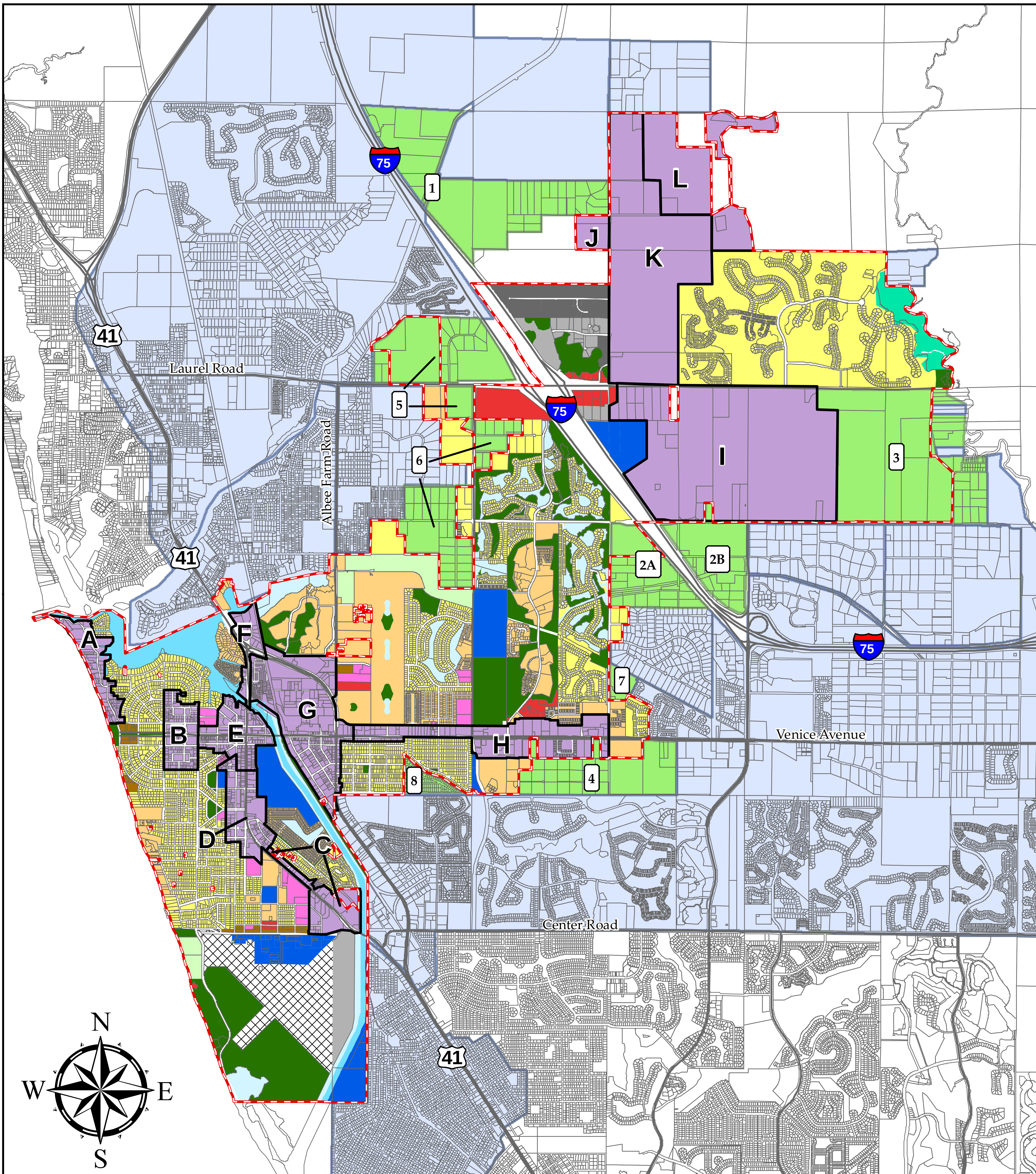
The hydraulic modeling software selected for the Venice collection system model is InfoSWMM software, an Innovyze product. InfoSWMM was recommended because the City currently utilizes Innovyze software for their water distribution system model and the user interface will be similar between the two products. InfoSWMM is the only wastewater collection system modeling software available from Innovyze that is complex enough to solve a combined pressure sewer and gravity sewer pipe network. InfoSWMM is fully integrated with ESRI ArcGIS giving the modeler the ability to use tools available within the GIS for more complex evaluation and reporting options.

Model development started with the City's existing GIS information. The City provided GIS shapefiles for the existing wastewater collection system including lift stations, force mains, manholes and gravity sewer pipelines. This information formed the basis of the collection system model. Paper data included record drawings of lift stations, pump curve data, rainfall data, smoke test data, and pump run time data. The model was constructed to represent the backbone collection system pipelines. Backbone pipelines were identified as 8-inch diameter and larger gravity sewers and all force mains that provide hydraulic connectivity between sewer basins and lift stations. This information along with 83 existing lift stations and associated force main is the basis for the collection system model and is shown in **Figure 5-3**.

Each of the 83 lift stations within the City are represented in the model as a wet well element and several pump elements. The two private lift stations were not modeled. Pump curve data collected from the City and from various manufacturers was digitized and imported into the model. Float elevations were assigned as operational controls in the model to control the pump starts and stops automatically over the duration of the model simulation.

The model is designed to simulate an extended period analysis for 24-hour duration. Two demand scenarios were developed--an average day flow condition, and a peak wet weather flow condition. Both scenarios are based on representative days in 2010 as determined from SCADA data and local rainfall data.

Figure 5 - 3
City of Venice, FL
Wastewater Collection System
Backbone Infrastructure



Boundaries & Features

- CITY OF VENICE LIMITS, 2010
- POTENTIAL VOLUNTARY ANNEXATION AREAS
- POTENTIAL COORDINATION AREAS
- PARCEL BOUNDARIES
- MAJOR ROADS
- PLANNING AREAS

Future Land Use

- LOW DENSITY RESIDENTIAL
- MEDIUM DENSITY RESIDENTIAL
- HIGH DENSITY RESIDENTIAL
- MIXED USE RESIDENTIAL
- COMMERCIAL
- INSTITUTIONAL-PROFESSIONAL
- AIRPORT OPERATIONS
- INDUSTRIAL
- INDUSTRIAL- COMMERCIAL
- GOVERNMENT USE
- RECREATION & OPEN SPACE
- CONSERVATION
- MARINE PARK
- GREENWAY/RIVER BUFFER
- WATERWAYS

0 0.3 0.6 1.2 1.8 2.4 3 Miles

Source: City of Venice Planning & Zoning Department, 2010.
Adopted 10/26/10 | ORD. No. 2010-21 | AMD No. City of Venice 10-1ER

Planning Areas

Specific future land use designations apply to the following planning areas:

A - TARPON CENTER/ESPLANADE*
B - HERITAGE PARK*
C - SOUTHERN GATEWAY
D - ISLAND PROFESSIONAL*
E - CITY CENTER*
F - NORTHERN GATEWAY*
G - SEABOARD
H - EASTERN GATEWAY
I - SOUTH LAUREL*
J - SHAKETT CREEK
K - KNIGHTS TRAIL
L - GENE GREEN

** These areas have been identified as Energy Conservation Areas.*

JPA/ILSBA Areas

The following areas have been designated as Potential Voluntary Annexation Areas under the Joint Planning & Interlocal Service Boundary Agreement between the City of Venice and Sarasota County:

1 - RUSTIC RD
2a - AUBURN RD TO I-75
2b - I-75/JACARANDA BLVD
3 - BORDER RD TO MYAKKA RIVER
4 - SOUTH VENICE AVE
5 - LAUREL RD MIXED USE
6 - PINEBROOK RD
7 - AUBURN RD
8 - GULF COAST BLVD

Figure 5 - 4 **City of Venice 2030 Future Land Use Map**

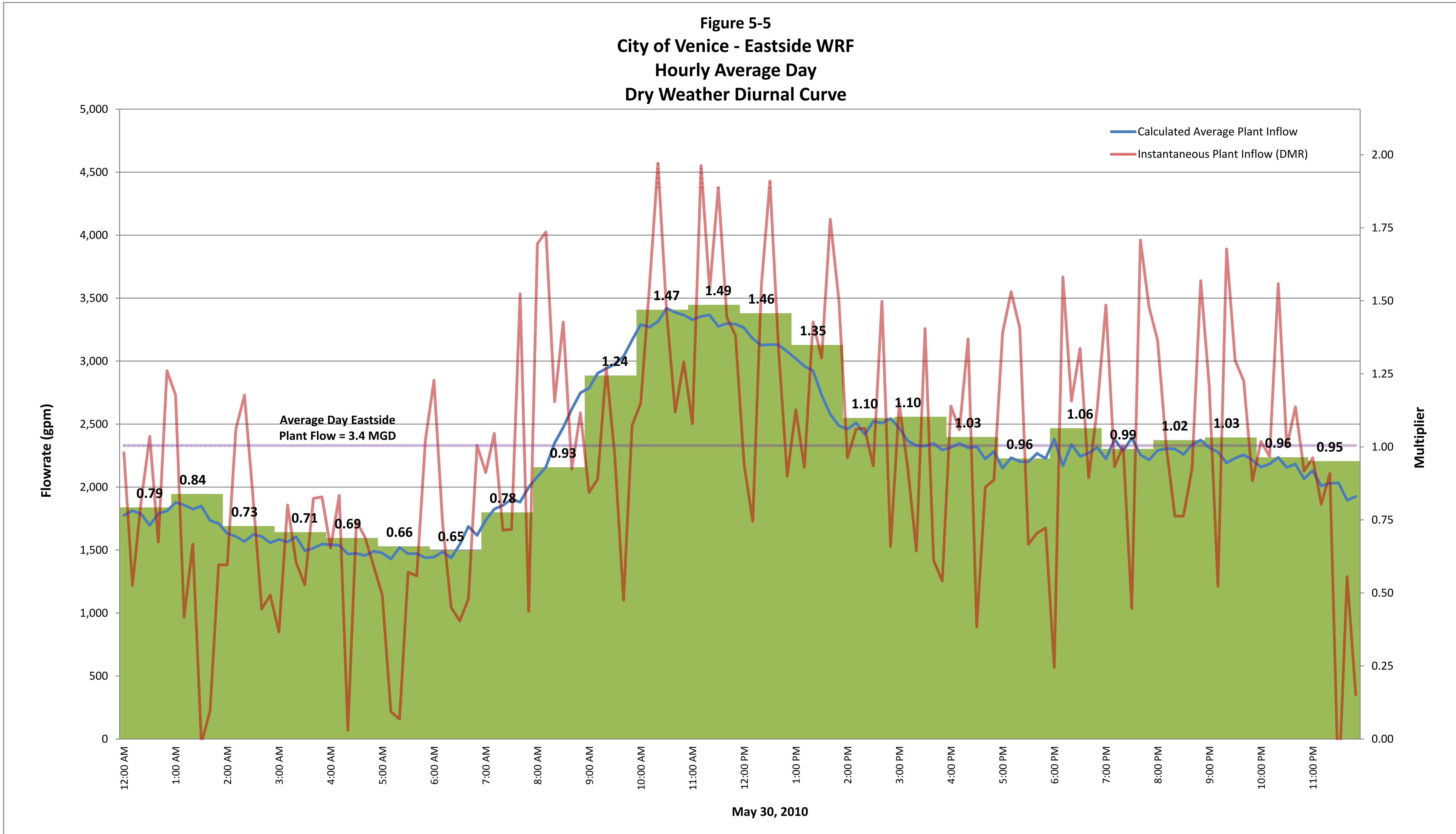
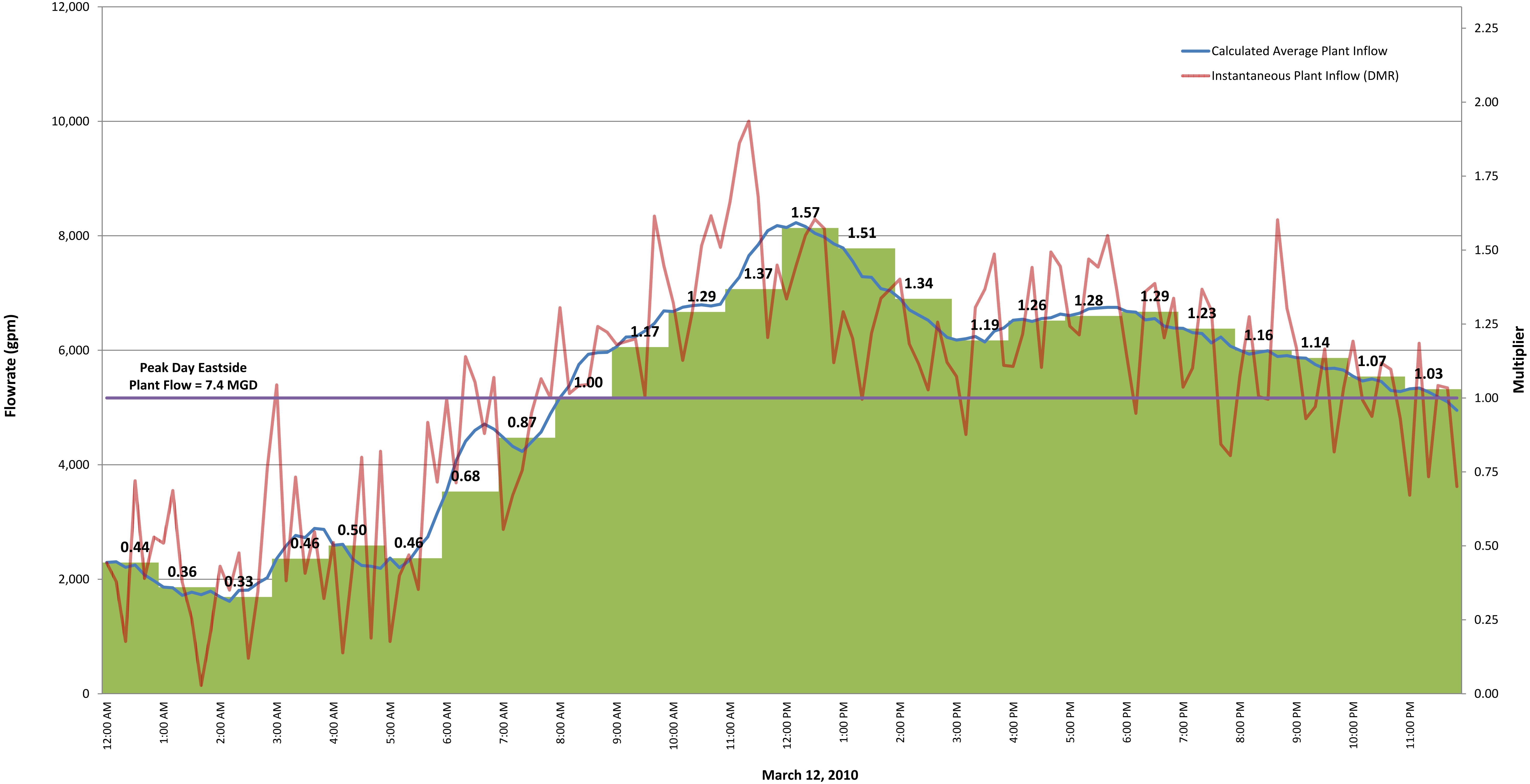


Figure 5-5
City of Venice - Eastside WRF
Hourly Average Day
Dry Weather Diurnal Curve
Calculated Average Plant Inflow
Instantaneous Plant Inflow (DMR)
Average Day Eastside
Plant Flow = 3.4 MGD
Flowrate (gpm)
Multiplier
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
0.79
0.84
0.73
0.71
0.69
0.66
0.65
0.78
0.93
1.24
1.47
1.49
1.46
1.35
1.10
1.10
1.03
0.96
1.06
0.99
1.02
1.03
0.96
0.95
12:00 AM
1:00 AM
2:00 AM
3:00 AM
4:00 AM
5:00 AM
6:00 AM
7:00 AM
8:00 AM
9:00 AM
10:00 AM
11:00 AM
12:00 PM
1:00 PM
2:00 PM
3:00 PM
4:00 PM
5:00 PM
6:00 PM
7:00 PM
8:00 PM
9:00 PM
10:00 PM
11:00 PM
May 30, 2010

Figure 5-6
City of Venice - Eastside WRF
Hourly Maximum Day
Wet Weather Diurnal Curve



The wastewater loading rates in the model are proportionally distributed to each lift station basin based on the City's Future Land Use data as identified in the City's 2030 FLUM (See **Figure 5-4**). For lift station basins that included sections of gravity sewer, the demands were assigned to the manholes within the basin. Lift station basins which do not contain backbone gravity pipelines were assigned flows directly to the wet well. A system-wide diurnal curve was created for both model demand scenarios. The diurnal curves and associated model flow multipliers are based on instantaneous flow data obtained from the Eastside WRF's influent flow meter (See **Figures 5-5** and **5-6**).

The flow meter also includes flow to the Eastside WRF from the County. The meter data available from the County's flows is not detailed enough to extract a diurnal specific only to the City's flows, therefore, the diurnal will also encompass the County flow. The County flow is assigned in the model as point inflow located along the influent pipeline into the Eastside WRF.

5.2.3 Assumptions

Where there was an absence of data, assumptions were made for generation of the model. For all pump curves that were not available from the City, curves were selected from the manufacturer's curves that matched the pump table provided by the City. The pump table included pump information such as make, model, impeller size, motor horsepower, and design point and was provided by the City. Since several design points in the table did not fall within the identified pump curve, the pump curve was entered into the model, not the design point.

The model element elevations that were not determined from record drawings or survey data were interpolated using 2-ft. GIS contours. This mostly applies to pressure junctions located along the force main pipelines, but several manhole and wet well elevations were also assumed. The assumed data is denoted in the model with a comment description qualifying the data source.

Gravity sewer and force main pipelines were all imported from the City's GIS data. Pipeline sizes were assigned in the model from the GIS attributes when available. For pipelines missing diameters, the sizes were confirmed from field survey data or verified with City staff. All force main pipelines have been assigned a Hazen-Williams C-factor of 100. The gravity pipelines all have a Manning's N value of 0.01.

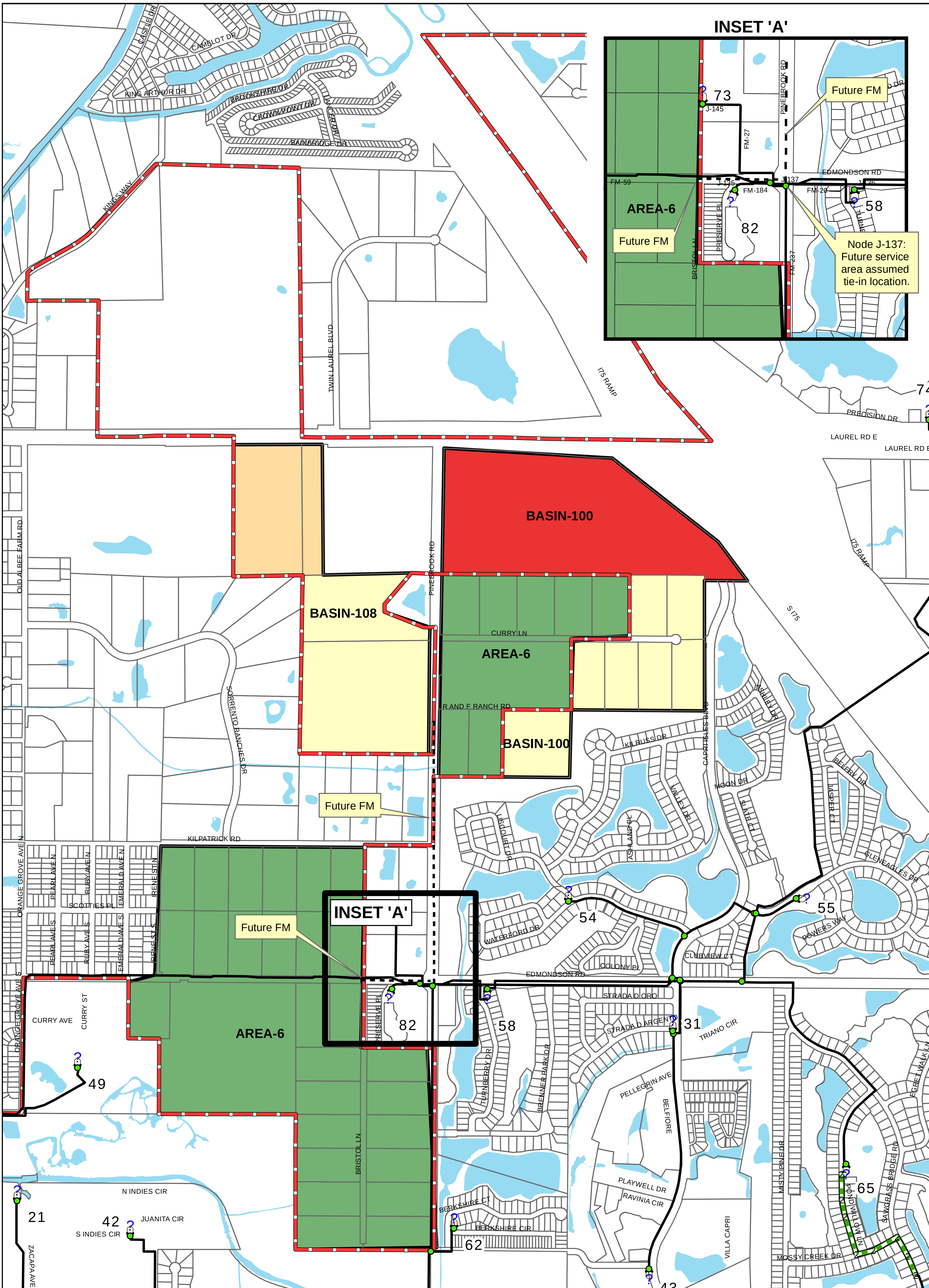
Manhole data and gravity pipe inverts were mostly obtained from survey data. Manhole and pipeline inverts, where unavailable from survey data, were assigned elevations based on a minimum pipe slope criteria.

Most of the lift station data that was unavailable through record drawings was obtained from survey information. Wet well rim elevations were surveyed and then float elevations were measured down from the rim. During survey efforts, wet well levels were mostly above the pump off float making measurements unavailable. The unavailable pump off elevations were assumed to have a depth 1.5 ft.

The primary assumption regarding modeling the future scenarios was determining where the service areas that start producing flow in 2015 will tie into the existing wastewater collection system. Logical groups of planning areas, JPA areas, and new sewer basins were made along with an assumed force main alignment and tie in point. The tie-in point was selected based on existing pipe diameters, flow rate from the new service areas, capacity in the existing force mains, and proximity to the future land areas. The preliminary tie-in locations were selected to represent a cost effective and feasible location but are subject to change based on future analysis and conditions. **Figure 5-7** through **5-11** shows the assumed tie-in locations for the future service areas. **Table 5-4** summarizes the reasoning for the assumed force main alignment and tie-in points.

Table 5-4: Summary of Assumed Force Main Alignments and Tie-ins

Force Main Description	Routing Explanation
Figure 5-7	
Force Main serving Basin 108, Basin 100, and Area 6 along Pinebrook Road	Alignment selected to follow major road and connect to the nearest existing force main.
Force Main serving Area 6	The force main serving Area 6 can tie in anywhere along the FM on Edmondson Road.
Figure 5-8	
Force Main serving Area 8	Force main selected to discharge directly into the wetwell of lift station 10. Tying directly into the force main from lift station 8 may negatively impact those pumps.
Force Main serving Basin 102	Alignment between Aston Drive and stormwater pond was determined after coordinating with City staff on potential alignments.
Figure 5-9	
Force Main serving Area 2a	Alignment selected to follow Border Road until it intersects with the 20-inch force main.
Figure 5-10	
Force Main serving Area 1, L, J, K, I, and Basin 106	Alignment selected to parallel the existing force main along Knights Trail because the existing force main does not have enough available capacity. Tie-in at the 24-inch force main near CVS.



McKIM&CREED

Legend

- COMMERCIAL
- LOW DENSITY RESIDENTIAL
- MODERATE DENSITY RESIDENTIAL
- JPA/ILSBA Areas
- City Boundary

PLANNING AREAS

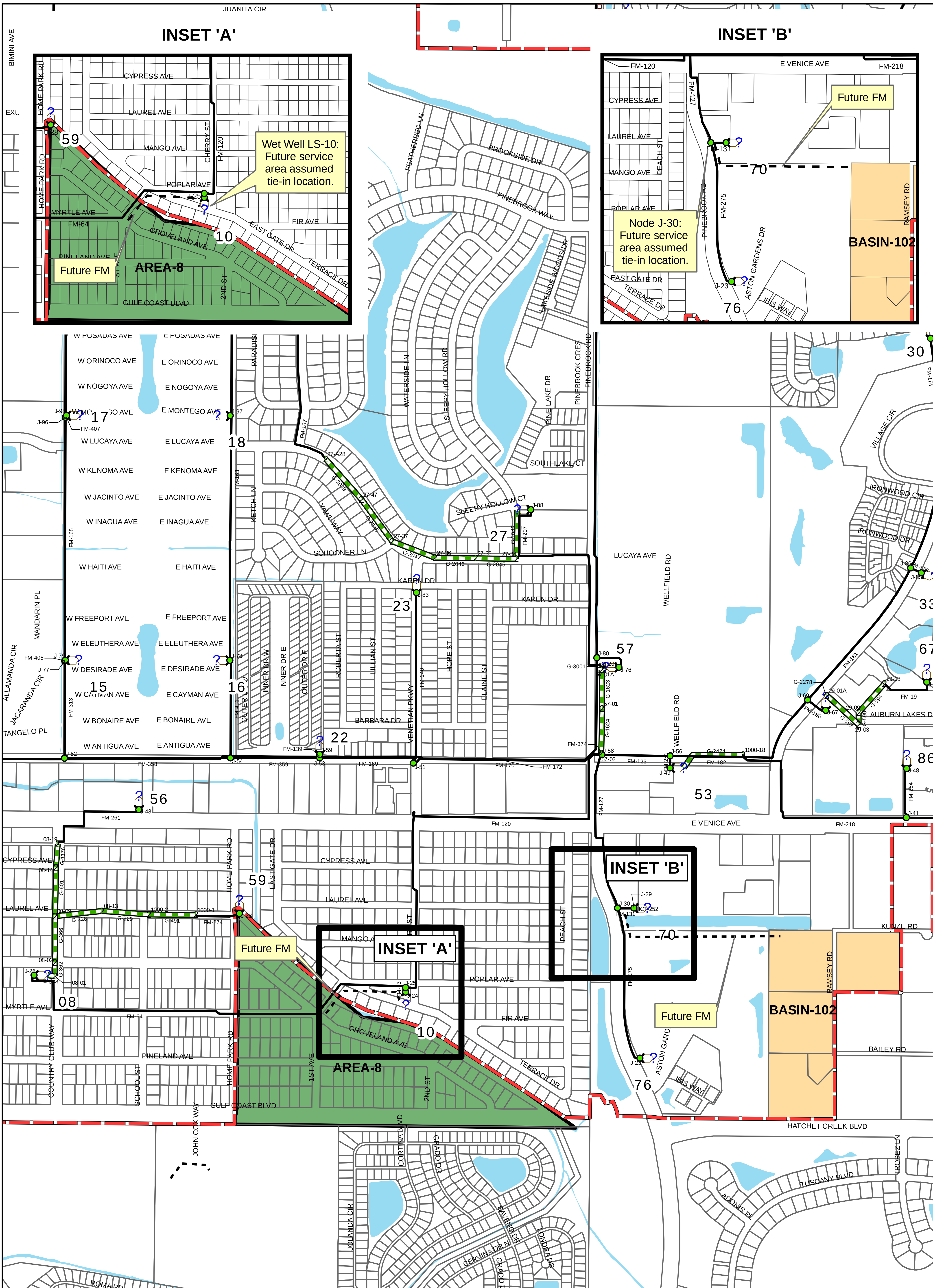
N/A

JPLA/ILSBA AREAS

6 - PINEBROOK RD

VENICE CITY ON THE GULF

Figure 5 - 7
City of Venice, FL
Future Service Areas And Assumed System Tie-in Location



McKIM&CREED

Legend

- MODERATE DENSITY RESIDENTIAL
- JPA/ILSBA Areas
- City Boundary

PLANNING AREAS

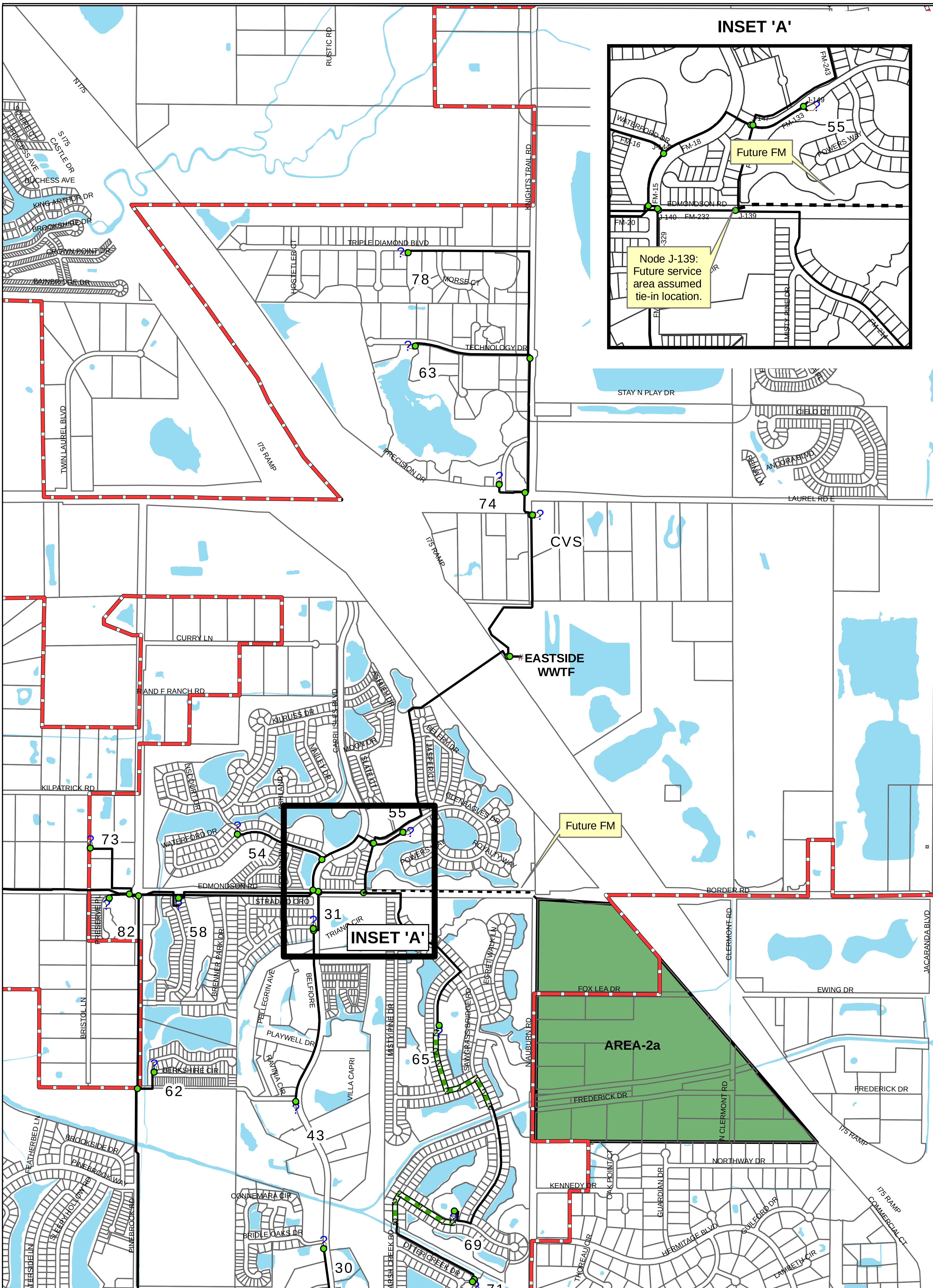
N/A

JPLA/ILSBA AREAS

8 - GULF COAST BLVD

VENICE
CITY ON THE GULF

Figure 5 - 8
City of Venice, FL
Future Service Areas And Assumed System Tie-in Location



Legend

JPA/ILSBA Areas

City Boundary

PLANNING AREAS

N/A

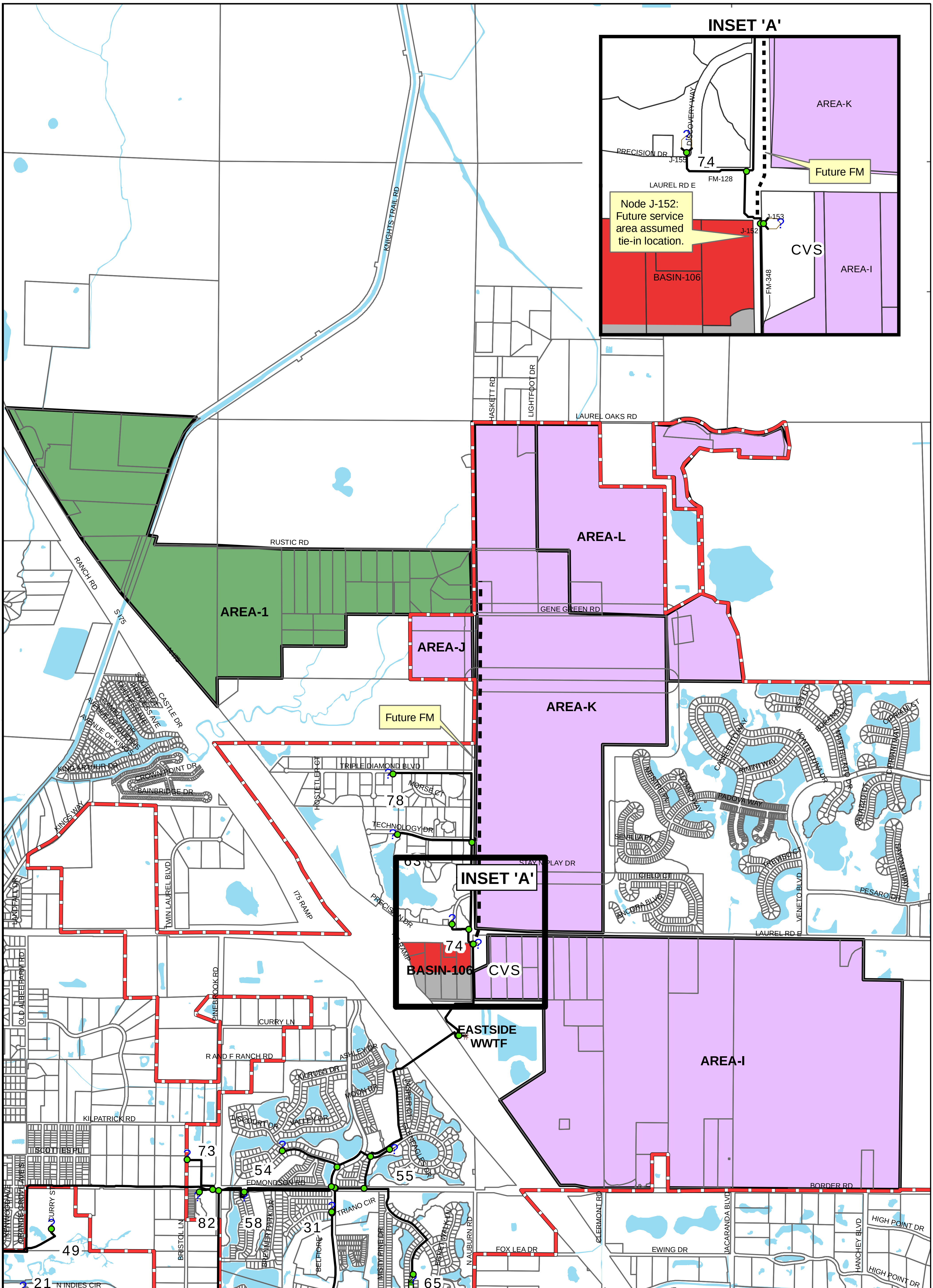
JPLA/ILSBA AREAS

2A - AUBURN RD TO I-75

VENICE CITY ON THE GULF

McKIM&CREED

Figure 5 - 9
City of Venice, FL
Future Service Areas And
Assumed System Tie-in Location



Legend

- INDUSTRIAL
- COMMERCIAL
- JPA/ILSBA Areas
- Planning Areas
- City Boundary

PLANNING AREAS

I - SOUTH LAUREL
J - SHAKETT CREEK
K - KNIGHTS TRAIL
L - GENE GREEN

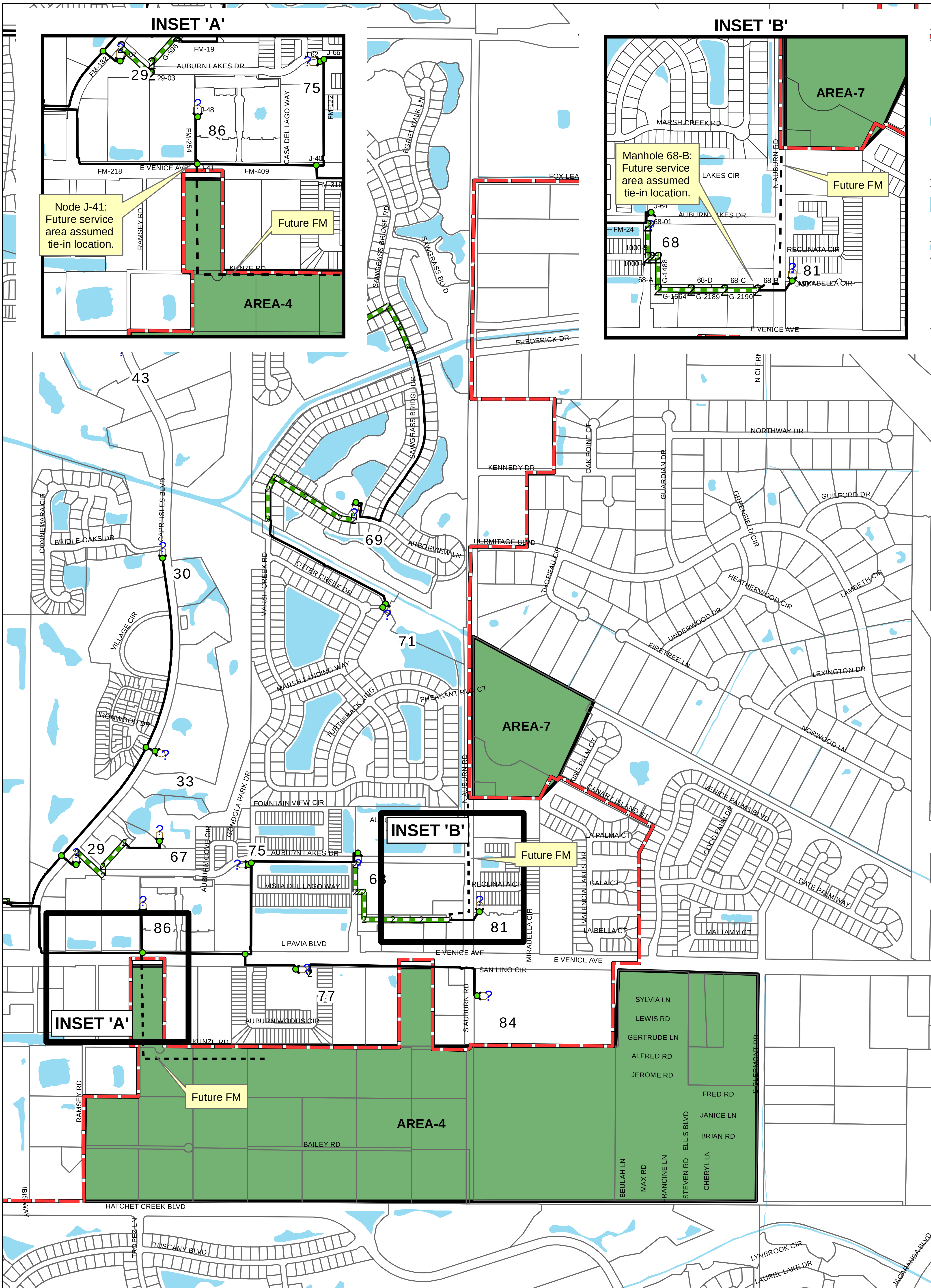
JPLA/ILSBA AREAS

1 - RUSTIC RD

McKIM&CREED

VENICE CITY ON THE GULF

Figure 5 - 10
City of Venice, FL
Future Service Areas And Assumed System Tie-in Location



Legend

JPA/ILSBA Areas

City Boundary

PLANNING AREAS

N/A

JPLA/ILSBA AREAS

4 - SOUTH VENICE AVE

7 - AUBURN RD

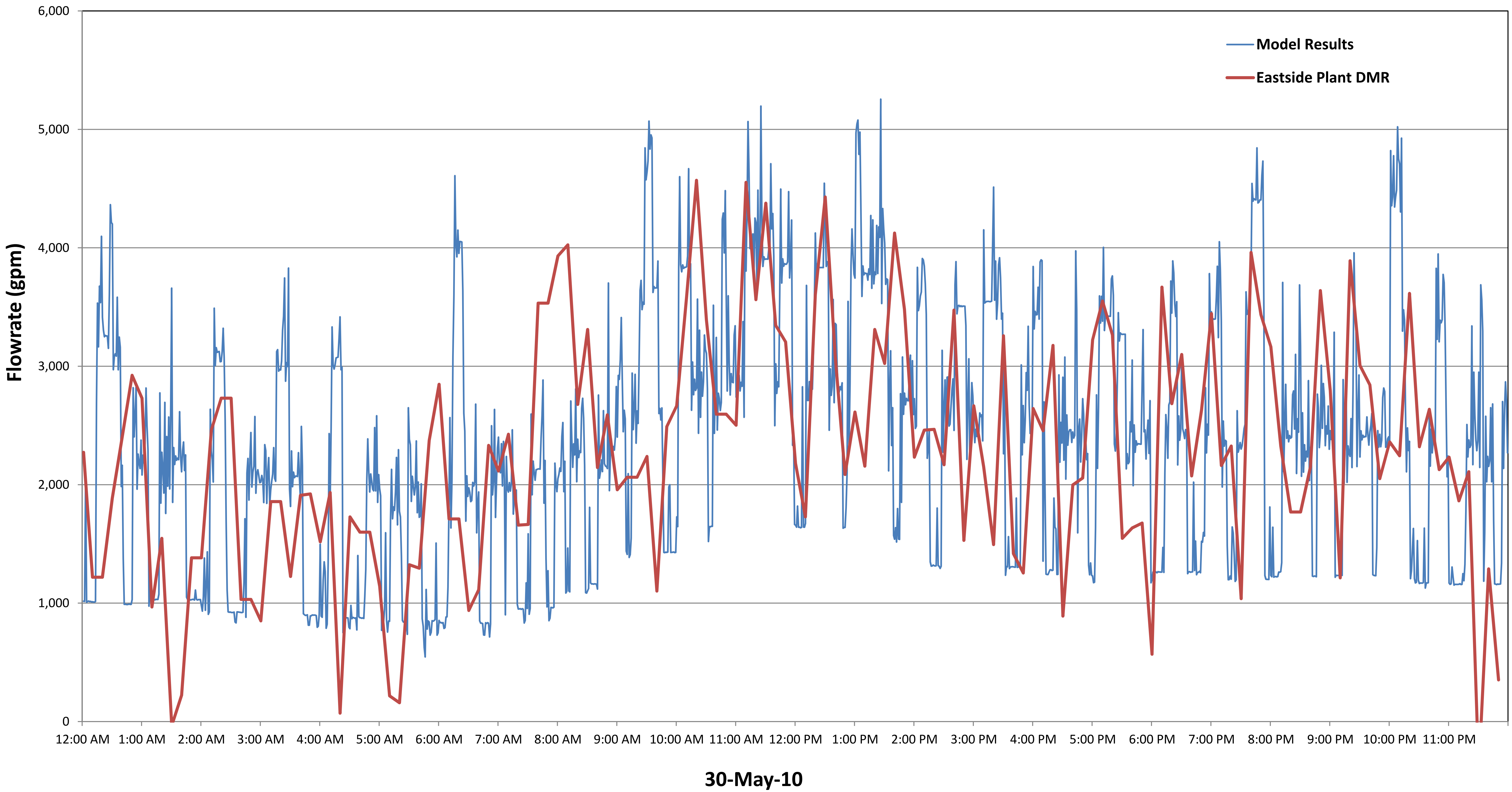
Figure 5 - 11
City of Venice, FL
Future Service Areas And
Assumed System Tie-in Location

5.2.4 Calibration

Model calibration is defined as the adjustment of model variables, such as refining model flows, pipe roughness values, pump operation characteristics, etc., in an effort to achieve a certain level of convergence between model output data and measured field data. The City has a limited amount of field data to assist in the model calibration process. SCADA data from the Eastside WRF was the main source for model comparison. The total inflow trend for the model for a typical average day (2010) condition and a peak wet weather flow (2010) condition were plotted versus the instantaneous flow from the SCADA data (See **Figures 5-12** and **5-13**).

The City has a total of 21 lift stations with telemetry. The SCADA data associated with these lift stations was provided by the City and used for model calibration. The City's three largest Lift Stations, 07, 57 and 32, all have VFD controlled pumps. The control schemes for the VFDs are based on complex calculations performed by the VFD PLC so settings were approximated for modeling purposes. Trend data for average day dry weather and maximum day wet weather helped indicate an approximate pump off elevation, pump on elevations and pump speed. The controls schemes used in the model are as follows:

Figure 5-12
City of Venice - Eastside Plant
Average Day Dry Weather Scenario
Model Results vs Actual Plant Flows



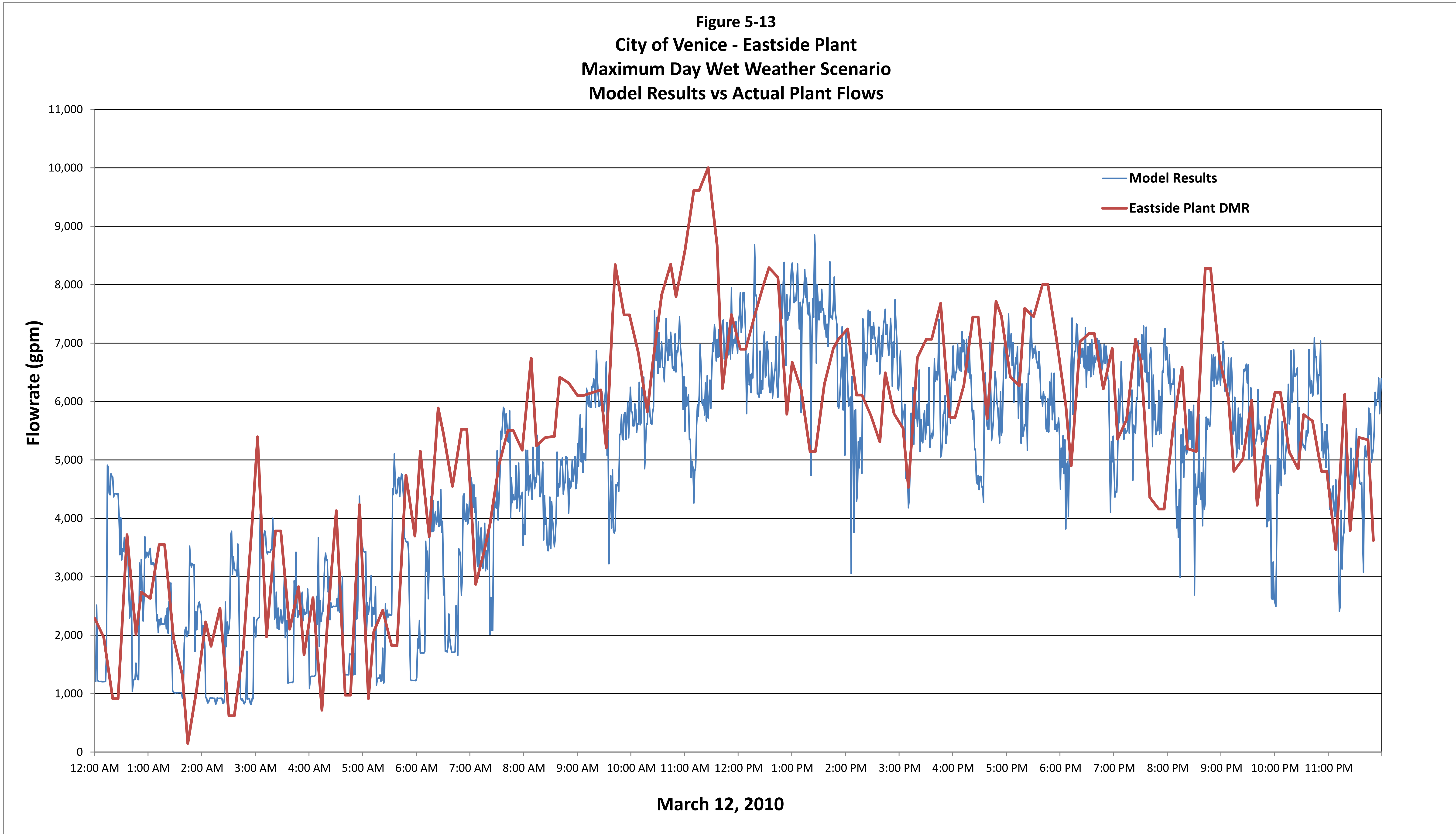


Figure 5-13
City of Venice - Eastside Plant
Maximum Day Wet Weather Scenario
Model Results vs Actual Plant Flows
Flowrate (gpm)
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
Model Results
Eastside Plant DMR
12:00 AM
1:00 AM
2:00 AM
3:00 AM
4:00 AM
5:00 AM
6:00 AM
7:00 AM
8:00 AM
9:00 AM
10:00 AM
11:00 AM
12:00 PM
1:00 PM
2:00 PM
3:00 PM
4:00 PM
5:00 PM
6:00 PM
7:00 PM
8:00 PM
9:00 PM
10:00 PM
11:00 PM
March 12, 2010

LIFT STATION-07

Pump Off @ Wet Well Level = 3.5 ft

Pump On (54%) @ Wet Well Level = 5.0 ft

Pump On (75%) @ Wet Well Level = 5.5 ft

Pump On (100%) @ Wet Well Level = 6.0 ft

LIFT STATION-32

Pump Off @ Wet Well Level = 2.0 ft

Pump On (54%) @ Wet Well Level = 5.75 ft

Pump On (75%) @ Wet Well Level = 6.25 ft

Pump On (100%) @ Wet Well Level = 6.75 ft

LIFT STATION-57

Pump Off @ Wet Well Level = 3.0 ft

Pump On (54%) @ Wet Well Level = 5.75 ft

Pump On (75%) @ Wet Well Level = 6.00 ft

Pump On (100%) @ Wet Well Level = 6.50 ft

5.2.5 Basis of Analysis

The system deficiencies were identified based on several properties. Gravity sewer with velocities less than 2 fps may not receive adequate flushing velocity to prevent the deposition of solids. These pipes should receive priority for periodic cleaning as part of the City's operations and maintenance program. The slopes of gravity sewers, where the manhole inverts are not based on survey data, should be surveyed to verify the pipe slope and corresponding velocity estimate are accurate. The maximum day wet weather scenario was used to evaluate the gravity sewer velocity since it represented the maximum flow in the system during the subject planning period. The maximum depth of flow within the gravity sewer was also evaluated. All gravity pipes with a depth of flow, as predicted by the model, divided by pipe diameter (d/D) equal to 1 was evaluated since it was surcharging. The manholes adjacent to the surcharging gravity sewer were evaluated to see if the water level within the manhole was getting close to the rim elevation and at risk for an overflow.

Force main velocities were evaluated for both the average day dry weather scenario and the maximum day wet weather scenario. Force mains with velocities 6 fps and greater were considered to be at capacity. Force mains with velocities less than 2 fps may not receive adequate flushing velocity to prevent the deposition of solids. These pipes should receive priority for periodic cleaning as part of the City's operations and maintenance program. When the velocity within a force main was in excess of 10 fps the force main was further evaluated in order to determine if an improvement project is needed.

Lift station wetwell water levels were monitored during the average day dry weather scenario and the maximum day wet weather scenario in order to determine if the wetwell was at risk for flooding. Wetwells with high water levels were monitored across multiple planning years to track their water levels over time and determine the potential causes.

Ten scenarios were evaluated. The average day dry weather and maximum day wet weather for each planning period was evaluated. **Table 5-5** outlines the ten scenarios. The model results are provided in **Appendix G** through **Appendix J**.

TABLE 5-5: Model Scenarios

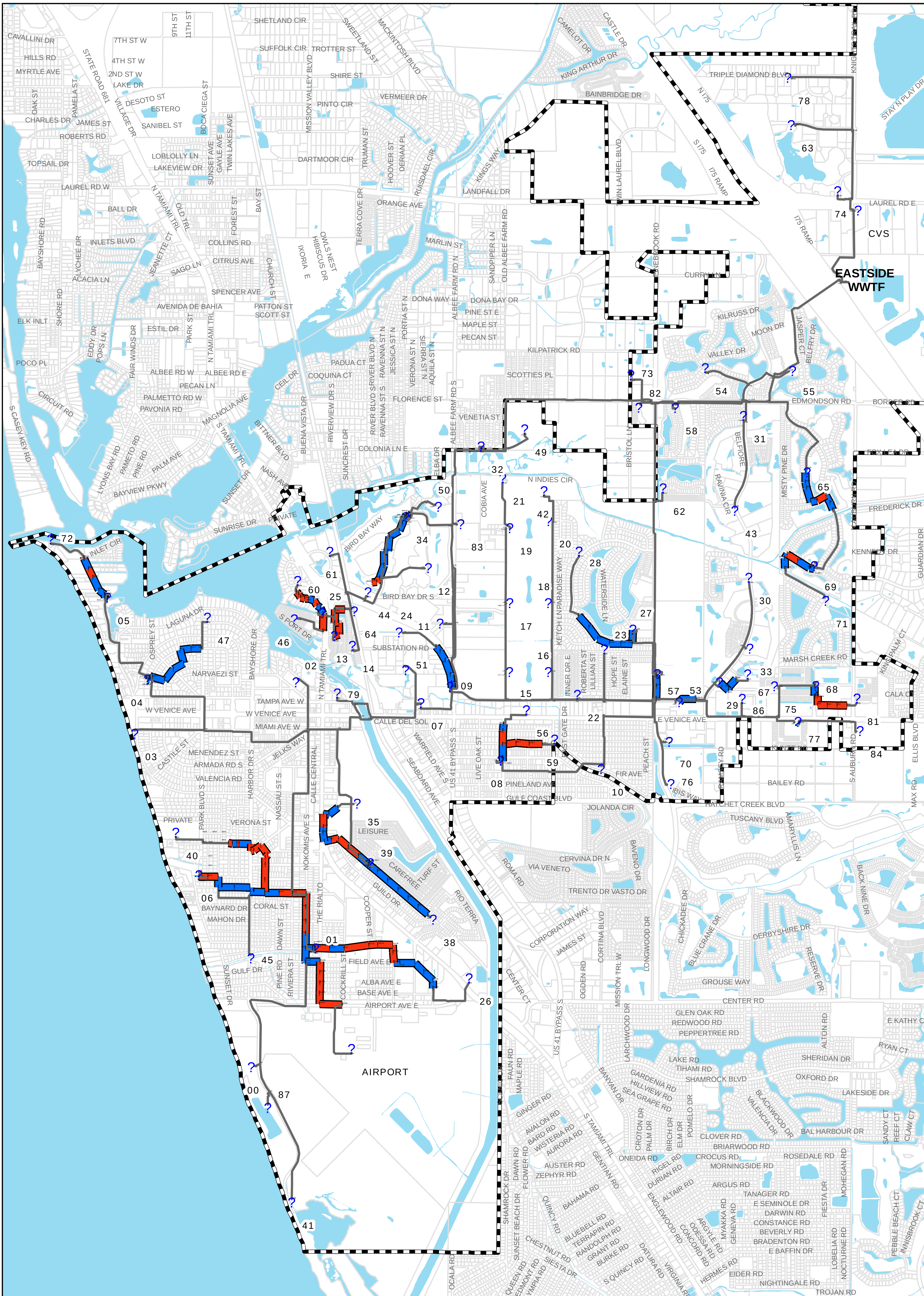
Year	Average Day Dry Weather (Scenario Number)	Maximum Day Wet Weather (Scenario Number)
2010	1	2
2015	3	4
2020	5	6
2025	7	8
2030	9	10

5.3 Existing Conditions: Scenario 1-2 and Results

The results for the existing conditions model showed that in general the collection system is operating well with few deficiencies. Specific deficiencies for the gravity collection system, force mains and lift stations were identified based on the basis of analysis discussed in **Section 5.2.5**.

The gravity collection system has several pipes where the velocity during the maximum day wet weather scenario does not reach 2 fps and therefore may not receive adequate flushing velocity to prevent the deposition of solids. **Figure 5-14** shows the 2010 gravity sewer velocities color coded during the maximum day wet weather scenario. It is important to note this figure is not a complete listing of the City's gravity sewer system and only represents the gravity pipe identified as part of the City's wastewater collection system backbone (See **Figure 5-3**). The gravity sewer along US 41 near the intersection of Albee Farm Road, node 09-06 to the wet well of Lift Station 9, showed the gravity sewer surcharged and overflowing at various manholes along the alignment. This deficiency appears to be caused by elevated water levels within the wet well of Lift Station 9 rather than a lack of capacity within the gravity sewer.

There were several force mains with a flow velocity of 6 fps or greater. These force mains are considered to be at capacity but are not required for upsizing except for FM-251, which is a 4" force main that discharges from Lift Station 53. Flow velocity in this main reaches approximately 16 fps during the average day dry weather and maximum day wet weather scenarios. The length of FM-251 is approximately 89' so excessive headloss was not a major concern. The excessive velocity increases the force main's risk for premature failure. Six force mains were identified as having a velocity less than 2 fps and therefore may not receive adequate flushing velocity to prevent the deposition of solids. **Tables 5-6** through **5-8** summarize the deficiencies regarding velocities in the gravity sewer and force main.



Legend

Gravity Sewer Velocity

- Less than 2 ft/sec
- Greater than 2 ft/sec
- Force Main
- City Boundary

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Figure 5 - 14
City of Venice, FL
Maximum Day Wet Weather Scenario 2010
Gravity Sewer Velocities

Table 5-6: Maximum Day Wet Weather 2010
Force Main with Velocity Greater Than 5 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-384	0.33	4	17	655	13.1
FM-251	0.33	4	89	633	16.2
FM-184	0.33	4	303	493	12.6
FM-403	0.67	8	41	1969	12.6
FM-180	0.33	4	161	341	8.7
FM-351	0.83	10	19	2097	8.6
FM-44	0.33	4	104	238	7.2
FM-22	0.33	4	42	262	6.7
FM-391	2.00	24	24	9281	6.6
FM-201	0.67	8	18	993	6.3
FM-123	0.67	8	507	992	6.3
FM-57	0.33	4	125	246	6.3
FM-243	1.67	20	3832	6094	6.2
FM-208	1.33	16	251	3860	6.2
FM-170	1.33	16	7452	3860	6.2
FM-183	0.83	10	134	1498	6.1

Table 5-7: Maximum Day Wet Weather 2010
Force Main with Velocity Less Than 2 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-318	0.33	4	1826	78	2.0
FM-199	0.50	6	2462	169	1.9
FM-128	0.50	6	451	167	1.9
FM-193	0.67	8	972	295	1.9
FM-37	0.50	6	3107	166	1.9
FM-322	0.33	4	1376	72	1.8
FM-80	0.50	6	3072	134	1.5

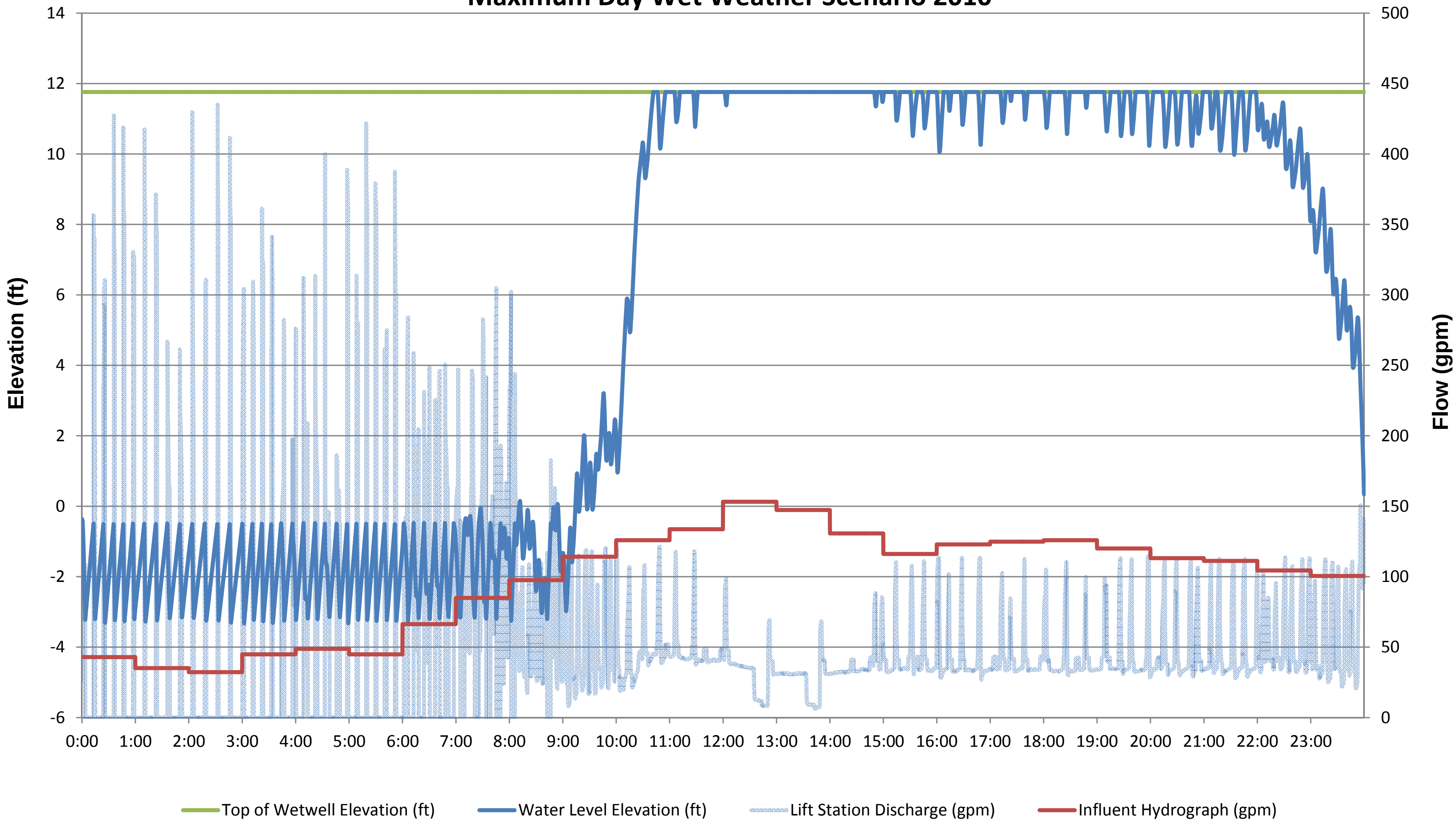
Table 5-8: Maximum Day Wet Weather 2010
Gravity Sewer with Velocity Less Than 2 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6019	98	0.67	8	94	0.39
G-6017	315	0.67	8	88	0.47
G-861	400	0.83	10	42	0.69
G-1189	265	0.67	8	10	0.91
G-1242	128	0.67	8	5	0.92
G-6013	75	1.25	15	76	0.96
G-328	351	0.67	8	36	1.09
G-1243	129	0.83	10	10	1.09
G-1188	352	0.83	10	20	1.11
G-6015	228	0.67	8	81	1.15
G-1353	195	0.67	8	139	1.21
G-1351	282	0.67	8	131	1.31
G-1187	116	0.67	8	29	1.31
G-6021	182	0.83	10	101	1.31
G-6037	272	0.67	8	148	1.32
G-6011	186	0.83	10	69	1.32
G-6023	412	0.83	10	108	1.37
G-491	350	0.67	8	23	1.40
G-1051	284	0.67	8	35	1.41
G-6007	51	0.67	8	63	1.41
G-1067	597	1.25	15	30	1.44
G-1186	316	1.50	18	39	1.44
G-2282	399	0.83	10	145	1.47
G-992	359	1.25	15	274	1.49
G-329	351	0.67	8	28	1.52
G-1054	277	1.75	21	44	1.53
G-858	67	0.83	10	41	1.53
G-995	352	0.67	8	307	1.54
G-6003	103	0.67	8	66	1.62
G-1055	280	0.50	6	55	1.64
G-1765	84	1.25	15	50	1.65

Table 5-8: Maximum Day Wet Weather 2010
Gravity Sewer with Velocity Less Than 2 fps (Continued)

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-12	262	0.67	8	93	1.65
G-185	243	0.67	8	154	1.69
G-590	266	0.67	8	127	1.69
G-1006	333	1.25	15	293	1.69
G-1081	656	0.83	10	160	1.71
G-6025	327	0.67	8	114	1.71
G-971	375	0.67	8	284	1.71
G-1037	284	0.67	8	65	1.74
G-2387	334	0.67	8	76	1.74
G-1177	135	0.67	8	49	1.75
G-2283	240	1.50	18	152	1.81
G-1140	349	1.50	18	108	1.86
G-601	365	1.50	18	76	1.86
G-1764	101	1.50	18	52	1.89
G-4	31	0.67	8	126	1.89
G-1760	161	0.83	10	58	1.90
G-2190	259	0.83	10	69	1.90
G-1488	250	1.50	18	64	1.91
G-3000	383	0.67	8	178	1.91
G-1139	351	0.67	8	102	1.91
G-1762	99	0.67	8	57	1.91
G-970	359	1.50	18	281	1.93
G-1763	116	0.67	8	55	1.94
G-2189	259	0.50	6	62	1.94
G-1155	340	1.50	18	104	1.98
G-1564	259	1.50	18	61	1.99

Figure 5-15
LS 11 Hydrograph
Maximum Day Wet Weather Scenario 2010



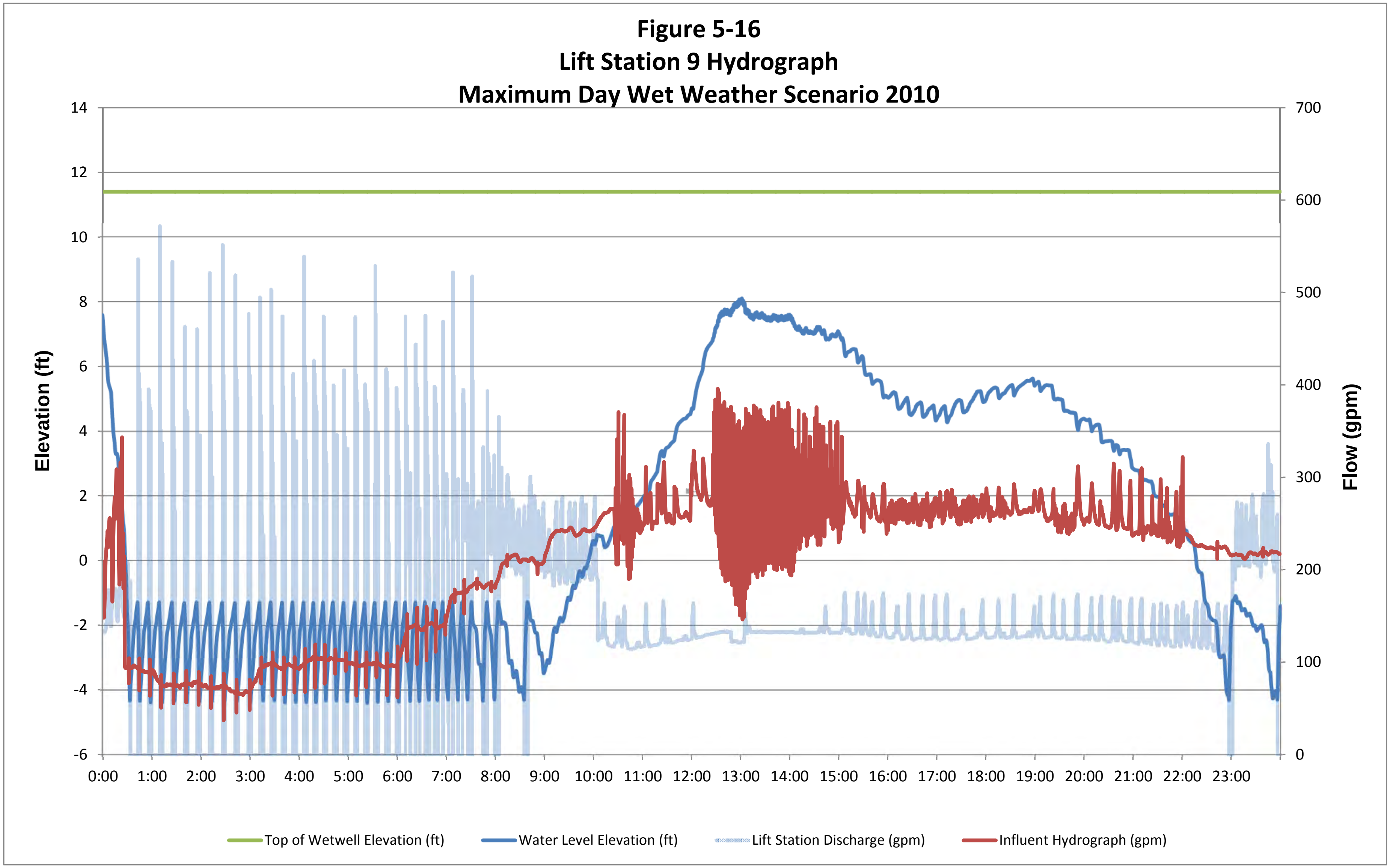


Figure 5-16
Lift Station 9 Hydrograph
Maximum Day Wet Weather Scenario 2010
Elevation (ft)
Flow (gpm)
14
12
10
8
6
4
2
0
-2
-4
-6
700
600
500
400
300
200
100
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
Top of Wetwell Elevation (ft)
Water Level Elevation (ft)
Lift Station Discharge (gpm)
Influent Hydrograph (gpm)

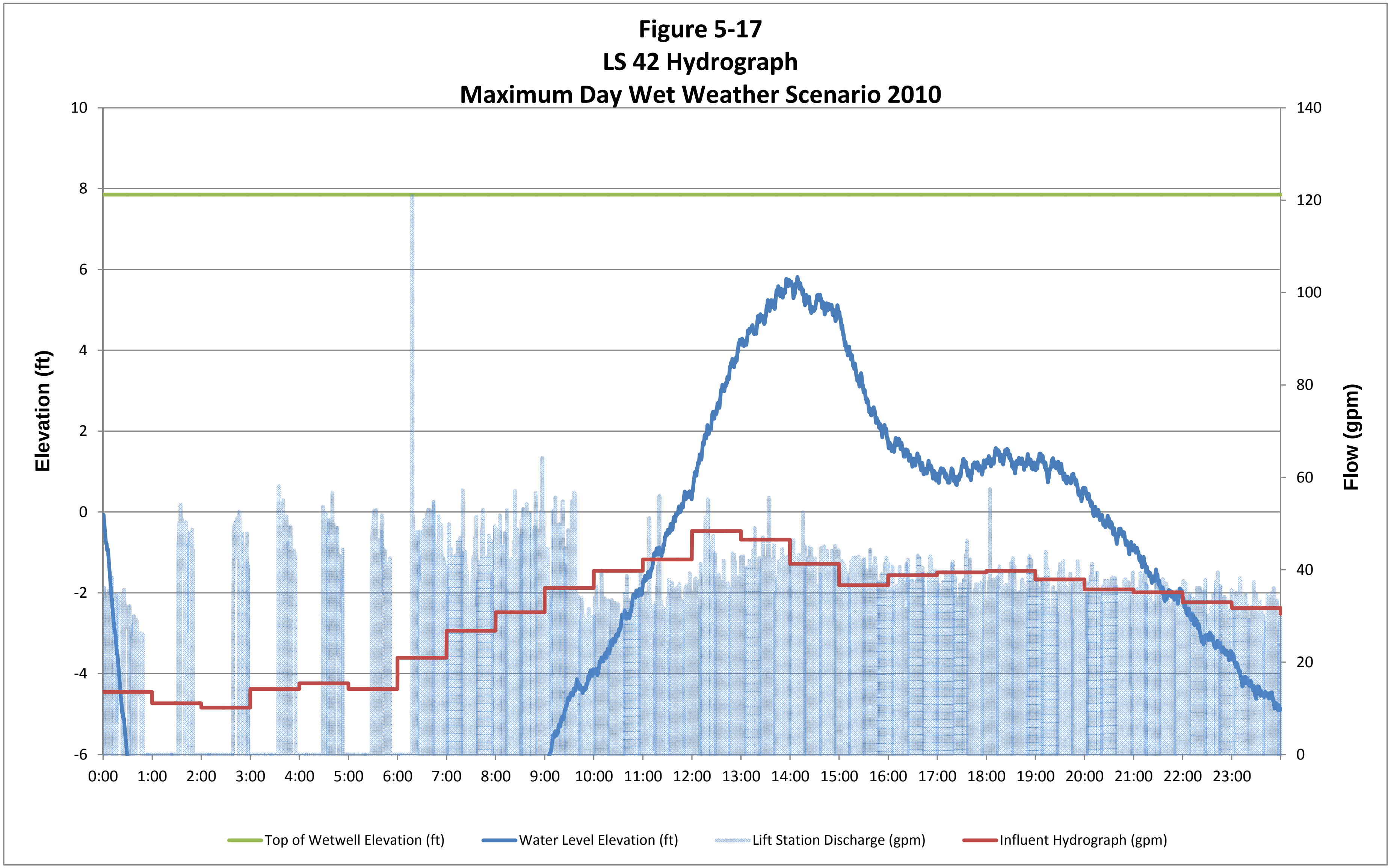


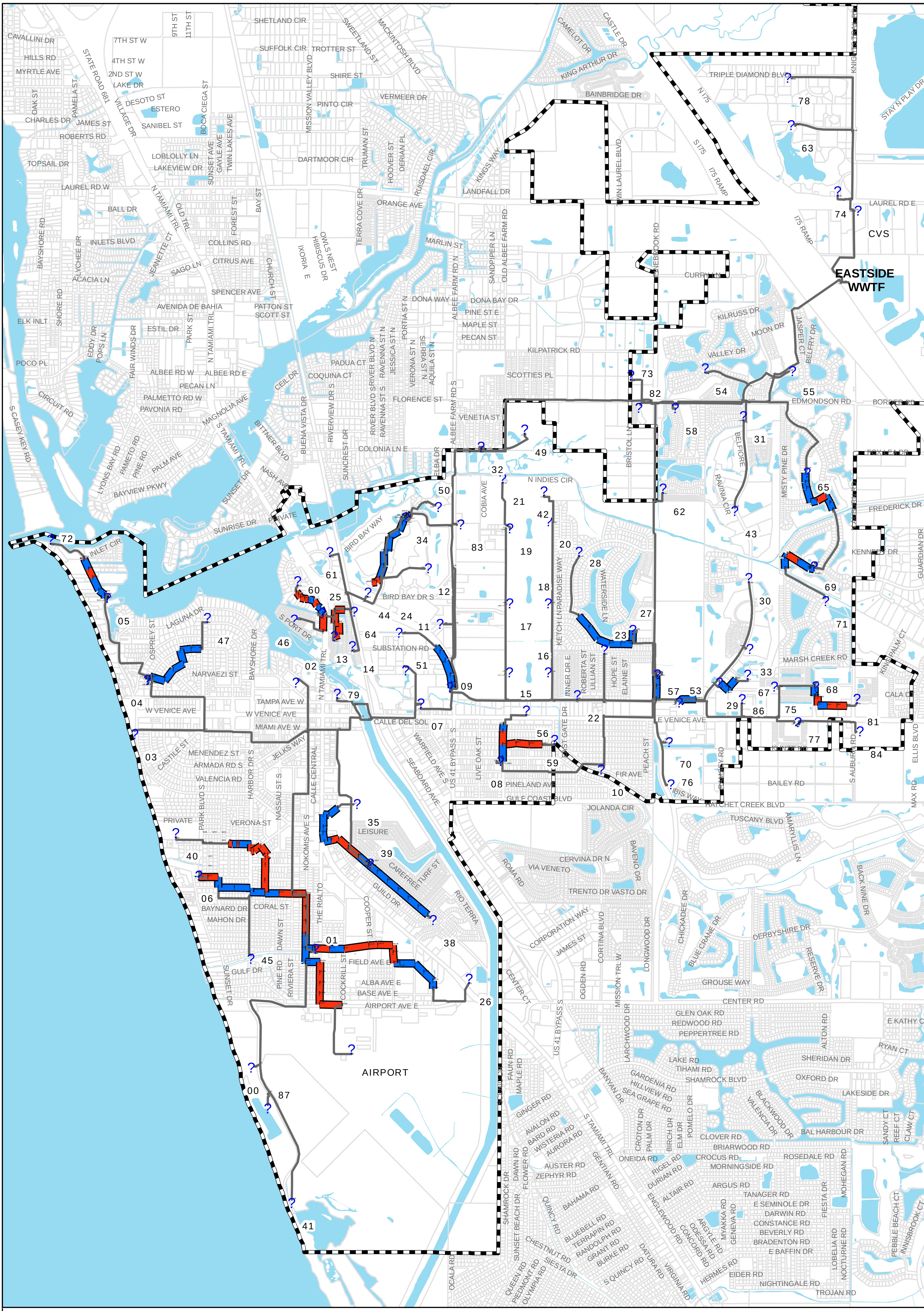
Figure 5-17
LS 42 Hydrograph
Maximum Day Wet Weather Scenario 2010
Elevation (ft)
Flow (gpm)
10
8
6
4
2
0
-2
-4
-6
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
Top of Wetwell Elevation (ft)
Water Level Elevation (ft)
Lift Station Discharge (gpm)
Influent Hydrograph (gpm)

The capacity of Lift Station 11 was adequate during the average day dry weather scenario but was overflowing during the maximum day wet weather scenario. **Figure 5-15** shows the wetwell water level, top of wetwell elevation, influent hydrograph, and lift station discharge over the course of simulated maximum day wet weather scenario. The overflow is a combination of the influent flow increasing per the diurnal curve and the pump capacity decreasing below the influent flow rate as the system pressure increases. Lift Stations 9 and 42 appear to experience high water levels within the wetwell during the maximum day wet weather scenario as shown in **Figures 5-16** and **5-17** respectively. The high water level within Lift Station 9 is believed to be the cause of the surcharging and overflowing conditions simulated on the upstream gravity sewer discussed previously.

5.4 2015 Planning Period: Scenario 3-4 and Results

The results for the 2015 average day dry weather and maximum day wet weather model scenarios identified additional system deficiencies. The deficiencies are a result of the additional flow projected to enter the collection system as discussed in **Section 3.3** and **Table 3-13**.

The gravity collection system has several pipes where the velocity during the maximum day wet weather scenario does not reach 2 fps, and therefore may not receive adequate flushing velocity to prevent the deposition of solids. **Figure 5-18** shows the 2015 gravity sewer velocities color coded during the maximum day wet weather scenario. No gravity sewer segments, beyond the gravity sewer near the intersection of US 41 and Albee Farm Road which was previously discussed in the 2010 scenario, was shown as being deficient.



Legend

Gravity Sewer Velocity

- Less than 2 ft/sec
- Greater than 2 ft/sec
- Force Main
- City Boundary

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Figure 5 - 18
City of Venice, FL
Maximum Day Wet Weather Scenario 2015
Gravity Sewer Velocities

There were several force mains which experienced a velocity of 6 fps or greater. Of these, force mains 184 and 403 had the highest velocities. Force main 184 is a 4" force main that discharges from Lift Station 82 and is approximately 303' long. The velocity within the force main reaches 16 fps during the average day dry weather scenario which puts it at an increased risk for premature failure. Force main 403, located at the intersection of Nokomis Avenue and Miami Avenue, is 8-inches in diameter and approximately 42-feet long. This force main receives the combined flow from an 8-inch force main from the north and a 12-inch force main from the south and ties into a 12-inch force main. The segment of 8-inch force main and the adjacent segments of ACP force main are currently in the design phase to be replaced. The same six force mains, identified as having a velocity less than 2 fps during the 2010 scenario, remained during the 2015 scenario. These force mains may not receive adequate flushing velocity to prevent the deposition of solids. **Tables 5-9** through **5-11** summarize the deficiencies regarding velocities in the gravity sewer and force main.

Lift Station 11 continued to show as overflowing as discussed in Scenario 2010. Lift Station 9 and 42 also continue to experience high water levels within the wet well during the maximum day wet weather scenario.

Table 5-9: Maximum Day Wet Weather 2015
Force Main with Velocity Greater Than 5 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-251	0.33	4	89	629	16.1
FM-384	0.33	4	17	566	14.4
FM-184	0.33	4	303	523	13.3
FM-403	0.67	8	41	1987	12.7
FM-180	0.33	4	161	341	8.7
FM-351	0.83	10	19	1915	7.8
FM-44	0.33	4	104	238	7.2
FM-153	0.83	10	404	1707	7.0
FM-22	0.33	4	42	265	6.8

Table 5-9: Maximum Day Wet Weather 2015
Force Main with Velocity Greater Than 5 fps (Continued)

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-391	2.00	24	24	9543	6.8
FM-123	0.67	8	507	1000	6.4
FM-201	0.67	8	18	991	6.3
FM-243	1.67	20	3832	6068	6.2
FM-401	0.50	6	730	546	6.2
FM-57	0.33	4	125	241	6.2
FM-170	1.33	16	7452	3839	6.1
FM-183	0.83	10	134	1473	6.0

Table 5-10: Maximum Day Wet Weather 2015
Force Main with Velocity Less Than 2 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-199	0.50	6	2462	169	1.9
FM-37	0.50	6	3107	169	1.9
FM-193	0.67	8	972	295	1.9
FM-322	0.33	4	1376	72	1.8
FM-80	0.50	6	3072	141	1.6
FM-128	0.50	6	451	119	1.4

Table 5-11: Maximum Day Wet Weather 2015
Gravity with Velocity Less Than 2 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6019	98	1.50	18	100	0.41
G-6017	315	1.50	18	93	0.48
G-861	400	0.67	8	46	0.73
G-1189	265	0.50	6	10	0.92
G-1242	128	0.67	8	5	0.94
G-6013	75	1.50	18	79	0.97
G-1243	129	0.67	8	10	1.10

Table 5-11: Maximum Day Wet Weather 2015
Gravity with Velocity Less Than 2 fps (Continued)

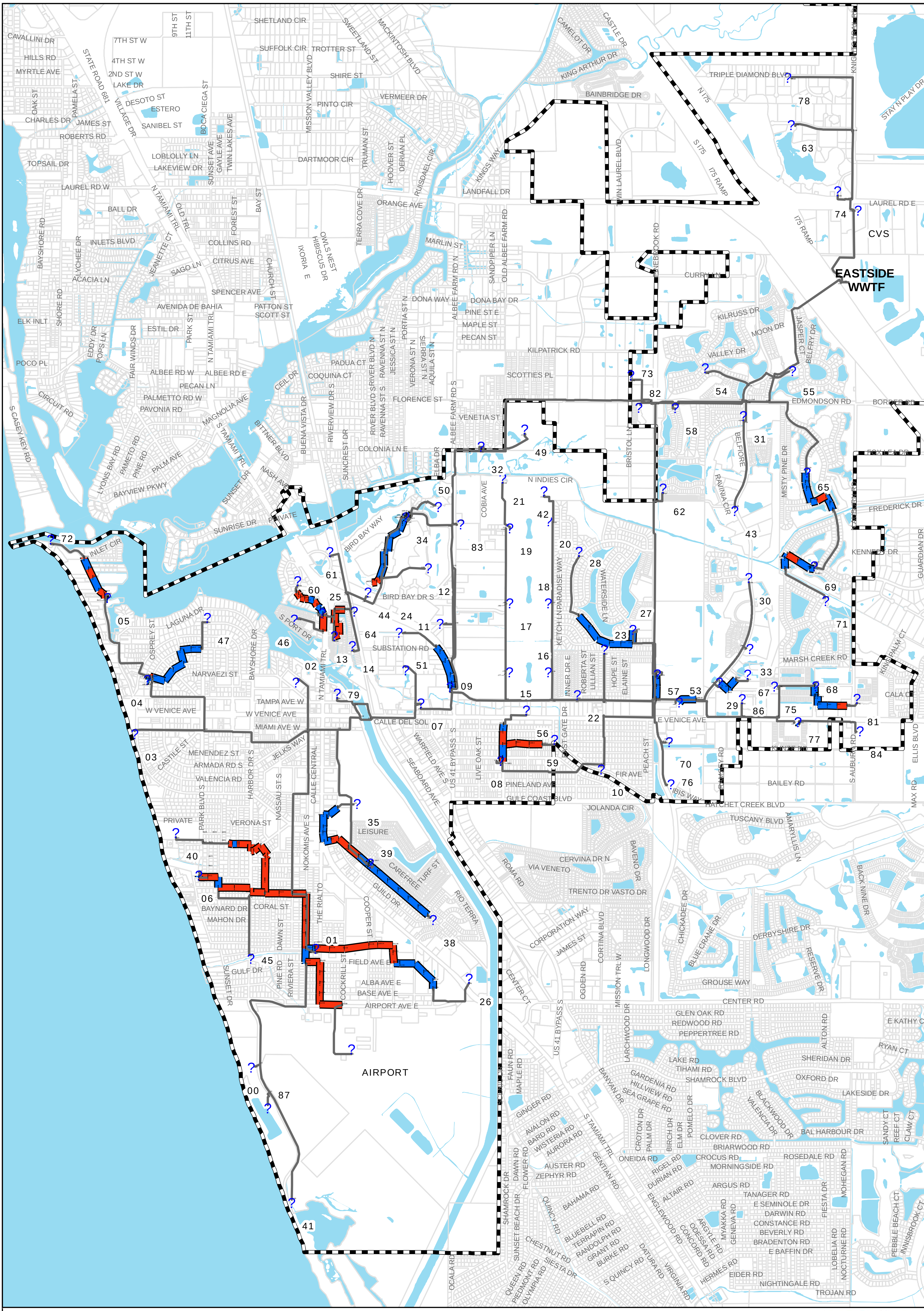
ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-328	351	0.67	8	38	1.10
G-1188	352	0.67	8	21	1.13
G-6015	228	1.50	18	86	1.18
G-1353	195	0.83	10	149	1.24
G-1351	282	0.83	10	140	1.33
G-1187	116	0.67	8	31	1.33
G-6021	182	1.50	18	107	1.34
G-6011	186	1.50	18	72	1.34
G-6037	272	1.50	18	154	1.34
G-6023	412	1.50	18	113	1.39
G-491	350	0.67	8	24	1.42
G-6007	51	0.67	8	66	1.43
G-1067	597	0.67	8	31	1.43
G-1051	284	0.67	8	37	1.43
G-1186	316	0.67	8	41	1.46
G-2282	399	0.83	10	155	1.51
G-992	359	1.25	15	287	1.52
G-329	351	0.67	8	30	1.55
G-1054	277	0.67	8	47	1.55
G-995	352	1.25	15	321	1.57
G-858	67	0.67	8	45	1.58
G-6003	103	0.50	6	68	1.63
G-1765	84	0.67	8	50	1.65
G-1055	280	0.67	8	58	1.66
G-590	266	0.67	8	131	1.70
G-12	262	0.83	10	102	1.71
G-1006	333	1.25	15	309	1.72
G-185	243	1.75	21	161	1.72
G-971	375	1.25	15	299	1.73
G-1081	656	0.83	10	170	1.73
G-6025	327	1.50	18	120	1.74

Table 5-11: Maximum Day Wet Weather 2015
Gravity with Velocity Less Than 2 fps (Continued)

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-1037	284	0.67	8	69	1.76
G-2387	334	0.67	8	79	1.76
G-1177	135	0.67	8	52	1.77
G-2283	240	0.83	10	162	1.84
G-1140	349	0.83	10	112	1.88
G-1764	101	0.67	8	53	1.89
G-4	31	0.83	10	127	1.90
G-601	365	0.67	8	81	1.90
G-1760	161	0.67	8	60	1.91
G-2190	259	0.67	8	71	1.91
G-1762	99	0.67	8	57	1.92
G-1139	351	0.83	10	107	1.93
G-1488	250	0.67	8	68	1.94
G-1763	116	0.67	8	55	1.95
G-970	359	1.25	15	295	1.95
G-3000	383	0.83	10	190	1.95
G-2189	259	0.67	8	65	1.97
G-1131	201	0.83	10	113	1.98

5.5 2020 Planning Period: Scenario 5-6 and Results

The results for the 2020 average day dry weather and maximum day wet weather model scenarios identified additional system deficiencies. The deficiencies are a result of the additional flow projected to enter the collection system as discussed in **Section 3.3** and **Table 3-13**.



Legend

Gravity Sewer Velocity

- Less than 2 ft/sec
- Greater than 2 ft/sec
- Force Main
- City Boundary

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Figure 5 - 19
City of Venice, FL
Maximum Day Wet Weather Scenario 2020
Gravity Sewer Velocities

Table 5-12: Maximum Day Wet Weather 2020
Force Main with Velocity Greater Than 5 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-384	17	0.33	4	2599	12.4
FM-251	89	0.33	4	634	16.2
FM-184	303	0.33	4	513	13.1
FM-403	41	0.67	8	2031	13.0
FM-374	16	0.67	8	1632	12.8
FM-180	161	0.33	4	342	8.7
FM-391	24	2.00	24	10353	7.3
FM-44	104	0.33	4	238	7.2
FM-351	19	0.83	10	1731	7.1
FM-387	381	0.33	4	265	6.8
FM-153	404	0.83	10	1633	6.7
FM-183	134	0.83	10	1584	6.5
FM-123	507	0.67	8	1010	6.4
FM-243	3832	1.67	20	6189	6.3
FM-57	125	0.33	4	246	6.3
FM-63	50	0.5	6	547	6.2
FM-201	18	0.67	8	952	6.1

Table 5-13: Maximum Day Wet Weather 2020
Force Main with Velocity Less Than 2 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-322	1376	0.33	4	76	1.9
FM-193	972	0.67	8	295	1.9
FM-199	2462	0.50	6	163	1.8
FM-128	451	0.50	6	131	1.5
FM-80	3072	0.50	6	120	1.4

Table 5-14: Maximum Day Wet Weather 2020
Gravity with Velocity Less Than 2 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6019	98	1.50	18	61	0.29
G-6017	315	1.50	18	56	0.34
G-861	400	0.67	8	58	0.81
G-6013	75	1.50	18	48	0.87
G-1189	265	0.50	6	11	0.94
G-1242	128	0.67	8	5	0.95
G-328	351	0.67	8	24	0.96
G-6015	228	1.50	18	52	0.98
G-1243	129	0.67	8	11	1.03
G-1353	195	0.83	10	92	1.05
G-6037	272	1.50	18	96	1.08
G-6021	182	1.50	18	65	1.14
G-1188	352	0.67	8	22	1.15
G-6011	186	1.50	18	48	1.16
G-1051	284	0.67	8	20	1.17
G-12	262	0.83	10	39	1.19
G-6023	412	1.50	18	69	1.19
G-491	350	0.67	8	13	1.19
G-2282	399	0.83	10	88	1.22
G-1351	282	0.83	10	94	1.23
G-992	359	1.25	15	180	1.27
G-1054	277	0.67	8	28	1.32
G-1187	116	0.67	8	33	1.35
G-6007	51	0.67	8	52	1.36
G-185	243	1.75	21	101	1.37
G-995	352	1.25	15	210	1.40
G-1081	656	0.83	10	92	1.43
G-1055	280	0.67	8	35	1.44
G-1067	597	0.67	8	18	1.47
G-1186	316	0.67	8	44	1.49
G-6025	327	1.50	18	73	1.50

Table 5-14: Maximum Day Wet Weather 2020 Gravity with Velocity Less Than 2 fps (Continued)

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-329	351	0.67	8	20	1.51
G-1037	284	0.67	8	42	1.51
G-1006	333	1.25	15	202	1.52
G-971	375	1.25	15	194	1.54
G-2283	240	0.83	10	89	1.58
G-3000	383	0.83	10	114	1.60
G-6003	103	0.50	6	69	1.63
G-1765	84	0.67	8	51	1.65
G-1034	392	0.67	8	52	1.67
G-858	67	0.67	8	58	1.69
G-970	359	1.25	15	177	1.70
G-2387	334	0.67	8	76	1.74
G-14	193	0.83	10	62	1.74
G-15	91	0.83	10	74	1.75
G-1077	387	0.83	10	105	1.77
G-590	266	0.67	8	151	1.78
G-4	31	0.83	10	106	1.79
G-6027	392	1.25	15	74	1.79
G-1177	135	0.67	8	55	1.80
G-6029	114	1.25	15	78	1.81
G-1005	389	1.25	15	205	1.83
G-6031	415	1.25	15	83	1.84
G-79	70	0.83	10	93	1.84
G-6033	434	1.25	15	87	1.87
G-996	383	1.25	15	304	1.88
G-6009	324	0.67	8	48	1.88
G-1140	349	0.83	10	117	1.89
G-13	369	0.83	10	49	1.89
G-1764	101	0.67	8	53	1.89
G-362	102	0.83	10	98	1.91
G-1347	161	0.67	8	89	1.92

Table 5-14: Maximum Day Wet Weather 2020
Gravity with Velocity Less Than 2 fps (Continued)

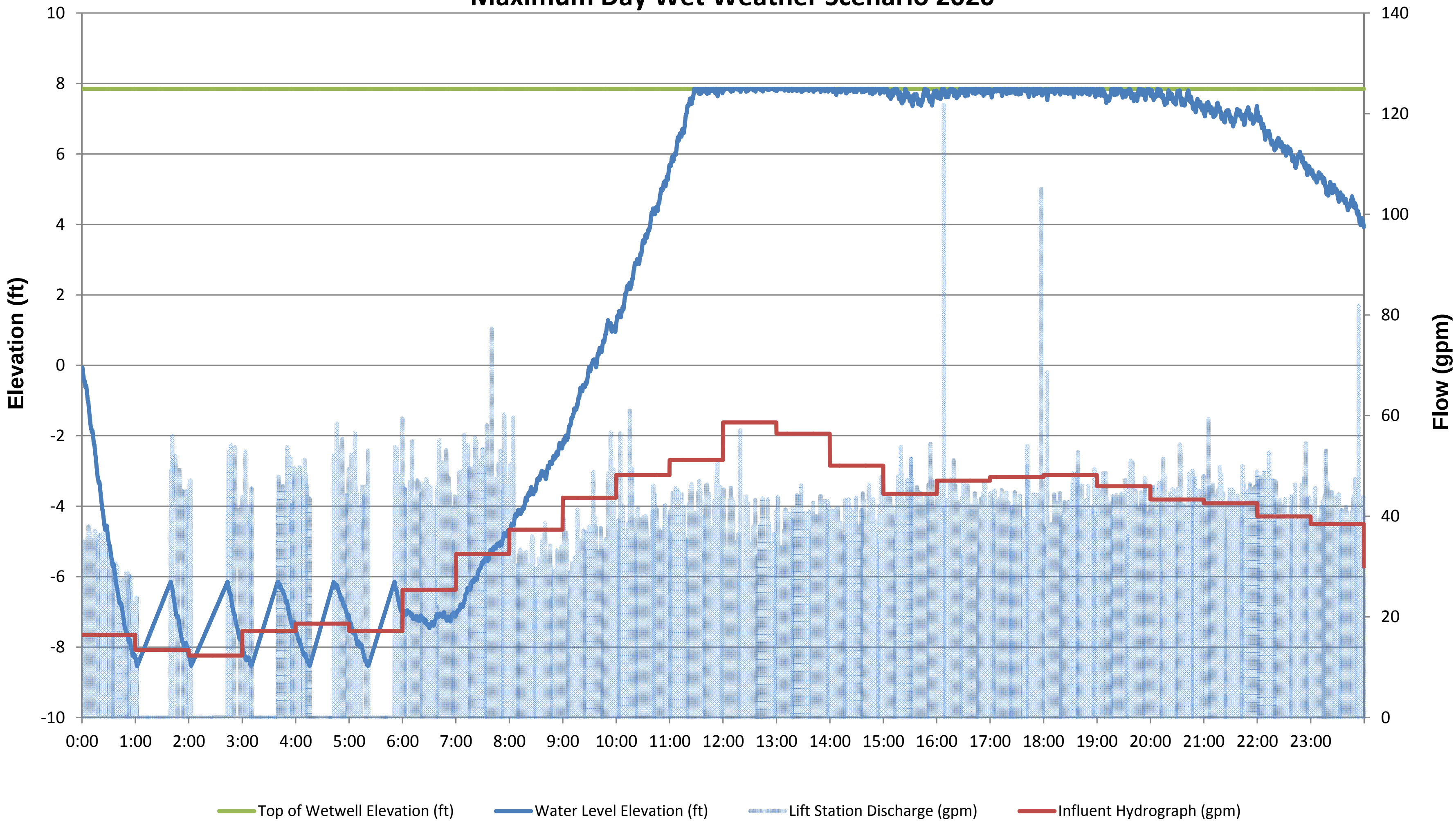
ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-1762	99	0.67	8	58	1.93
G-601	365	0.67	8	87	1.94
G-1760	161	0.67	8	63	1.95
G-1763	116	0.67	8	56	1.95
G-1139	351	0.83	10	112	1.96
G-6035	277	1.25	15	92	1.97
G-78	332	0.83	10	93	1.99
G-2190	259	0.67	8	82	1.99

The gravity collection system has several pipes where the velocity during the maximum day wet weather scenario does not reach 2 fps and therefore may not receive adequate flushing velocity to prevent the deposition of solids. **Figure 5-19** shows the 2020 gravity sewer velocities color coded during the maximum day wet weather scenario.

Only the gravity sewer pipe G-677 was identified as deficient beyond that identified in the 2010 and 2015 scenarios. Pipe G-677 surcharged during the maximum day wet weather scenario. The water elevation within the manhole at node 77-01 rose to approximately 6 feet below the rim elevation and therefore is in need of an improvement.

There were several force mains which experienced a velocity of 6 fps or greater. The majority of the force mains listed remains unchanged from previous scenarios. These force mains are considered to be at capacity. There were no new force main segments, beyond that identified in Scenarios 2010 and 2015, with velocities deemed to put the pipe at an increased risk of premature failure.

Figure 5-20
LS 42 Hydrograph
Maximum Day Wet Weather Scenario 2020



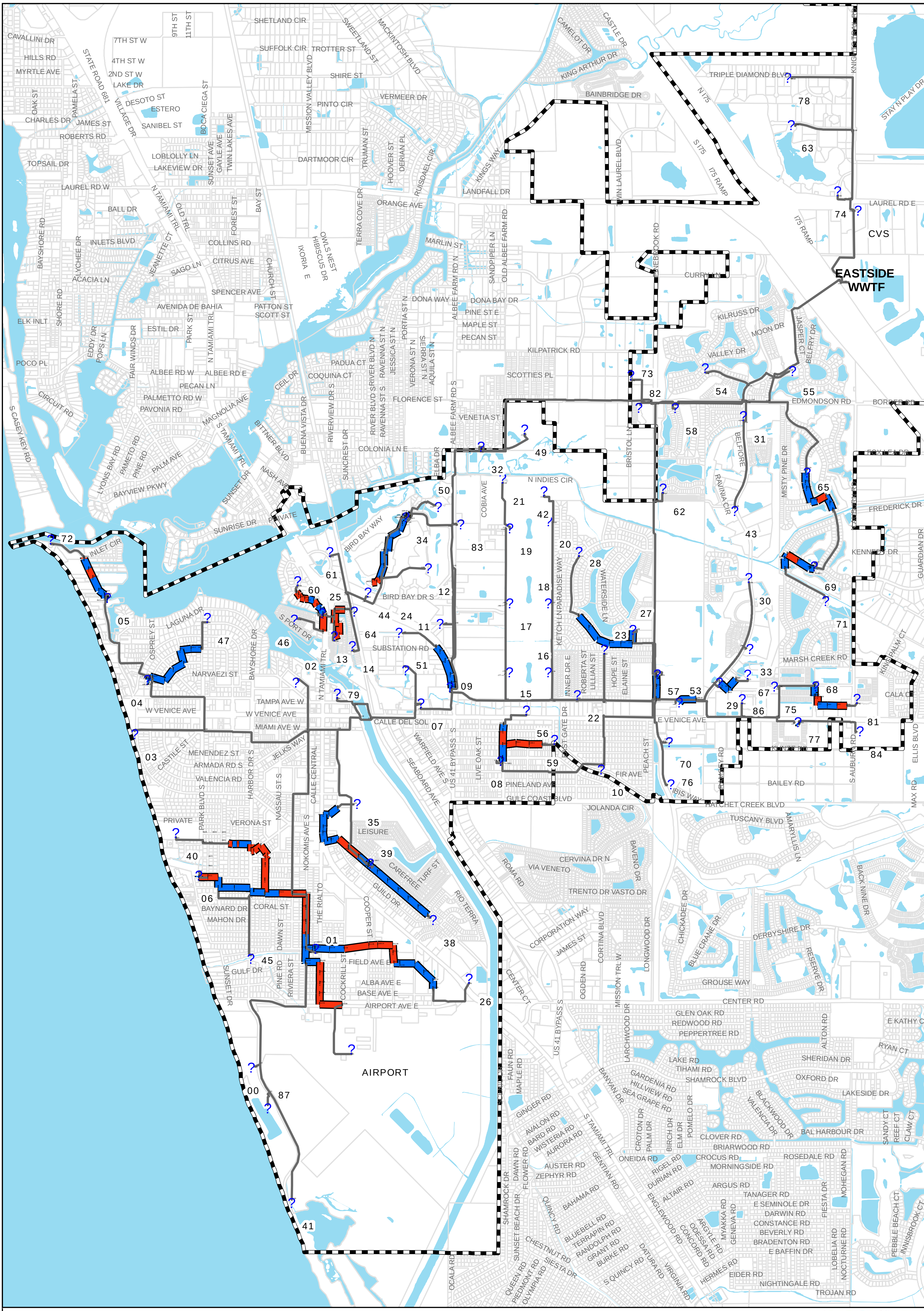
The force mains, identified as having a velocity less than 2 fps during the 2010 and 2015 scenario, remained relatively unchanged during the 2020 scenario. These force mains may not receive adequate flushing velocity to prevent the deposition of solids. **Tables 5-12** through **5-13** summarize the deficiencies regarding velocities in the gravity sewer and force main.

Lift Station 11 continued to show as overflowing as discussed in Scenario 2015. Lift Station 42, which had high water levels in Scenario 2010 and 2015, was overflowing during the maximum day wet weather scenario as shown in **Figure 5-20**. Lift Station 9 continued to experience high water levels within the wet well during the maximum day wet weather scenario but did not overflow.

5.6 2025 Planning Period: Scenario7-8 and Results

The results for the 2025 average day dry weather and maximum day wet weather model scenarios identified additional system deficiencies but none that specifically require an improvement. The deficiencies are a result of the additional flow projected to enter the collection system as discussed in **Section 3.3** and **Table 3-13**.

The gravity collection system has several pipes where the velocity during the maximum day wet weather scenario does not reach 2 fps and therefore may not receive adequate flushing velocity to prevent the deposition of solids. **Figure 5-21** shows the 2025 gravity sewer velocities color coded during the maximum day wet weather scenario.



Legend

Gravity Sewer Velocity

- Less than 2 ft/sec
- Greater than 2 ft/sec
- Force Main
- City Boundary

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VENICE CITY ON THE GULF

Figure 5 - 21
City of Venice, FL
Maximum Day Wet Weather Scenario 2025
Gravity Sewer Velocities

There were several force mains which experienced a velocity of 6 fps or greater. The majority of the force mains listed remains unchanged from previous scenarios. These force mains are considered to be at capacity. There were no new force main segments, beyond that identified in Scenarios 2010, 2015 and 2020 with velocities deemed to put the pipe at an increased risk of premature failure. The force mains, identified as having a velocity less than 2 fps during the 2010, 2015 and 2020 scenarios, remained relatively unchanged. These force mains may not receive adequate flushing velocity to prevent the deposition of solids. **Tables 5-15** through **5-17** summarize the deficiencies regarding velocities in the gravity sewer and force main.

Lift Stations 11 and 42 continued to overflow as discussed in Scenario 2020. Lift Station 9 continued to experience high water levels within the wetwell during the maximum day wet weather scenario but did not overflow.

Table 5-15: Maximum Day Wet Weather 2025
Force Main with Velocity Greater Than 5 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-384	0.33	4	17	3934	12.2
FM-251	0.33	4	89	633	16.2
FM-184	0.33	4	303	519	13.3
FM-403	0.67	8	41	1992	12.7
FM-180	0.33	4	161	343	8.8
FM-351	0.83	10	19	1935	7.9
FM-391	2.00	24	24	11061	7.8
FM-44	0.33	4	104	238	7.2
FM-153	0.83	10	404	1697	6.9
FM-57	0.33	4	125	271	6.9
FM-123	0.67	8	507	1021	6.5

Table 5-15: Maximum Day Wet Weather 2025
Force Main with Velocity Greater Than 5 fps (Continued)

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-243	1.67	20	3832	6253	6.4
FM-201	0.67	8	18	990	6.3
FM-183	0.83	10	134	1539	6.3
FM-170	1.33	16	7452	3836	6.1

Table 5-16: Maximum Day Wet Weather 2025
Force Main with Velocity Less Than 2 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-199	0.50	6	2462	168	1.9
FM-193	0.67	8	972	296	1.9
FM-322	0.33	4	1376	72	1.8
FM-37	0.50	6	3107	160	1.8
FM-128	0.50	6	451	143	1.6
FM-80	0.50	6	3072	126	1.4

Table 5-17: Maximum Day Wet Weather 2025
Gravity with Velocity Less Than 2 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6019	98	1.50	18	112	0.44
G-6017	315	1.50	18	105	0.52
G-861	400	0.67	8	46	0.73
G-1189	265	0.50	6	12	0.95
G-1242	128	0.67	8	6	0.96
G-6013	75	1.50	18	89	1.00
G-1243	129	0.67	8	11	1.10
G-328	351	0.67	8	43	1.13
G-1188	352	0.67	8	23	1.17

Table 5-17: Maximum Day Wet Weather 2025
Gravity with Velocity Less Than 2 fps (Continued)

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6015	228	1.50	18	97	1.23
G-1353	195	0.83	10	164	1.29
G-1351	282	0.83	10	153	1.35
G-1187	116	0.67	8	35	1.38
G-6021	182	1.50	18	120	1.39
G-6011	186	1.50	18	81	1.39
G-6037	272	1.50	18	175	1.42
G-6023	412	1.50	18	128	1.44
G-1067	597	0.67	8	34	1.45
G-491	350	0.67	8	26	1.45
G-6007	51	0.67	8	72	1.47
G-1051	284	0.67	8	41	1.47
G-1186	316	0.67	8	46	1.51
G-2282	399	0.83	10	171	1.57
G-992	359	1.25	15	317	1.58
G-858	67	0.67	8	46	1.58
G-329	351	0.67	8	33	1.59
G-1054	277	0.67	8	53	1.60
G-995	352	1.25	15	356	1.61
G-1765	84	0.67	8	51	1.65
G-6003	103	0.50	6	72	1.66
G-12	262	0.83	10	104	1.71
G-1055	280	0.67	8	65	1.72
G-590	266	0.67	8	143	1.75
G-1006	333	1.25	15	339	1.76
G-971	375	1.25	15	329	1.78
G-2387	334	0.67	8	83	1.79
G-1081	656	0.83	10	190	1.79
G-6025	327	1.50	18	136	1.80
G-1037	284	0.67	8	77	1.82
G-185	243	1.75	21	183	1.83

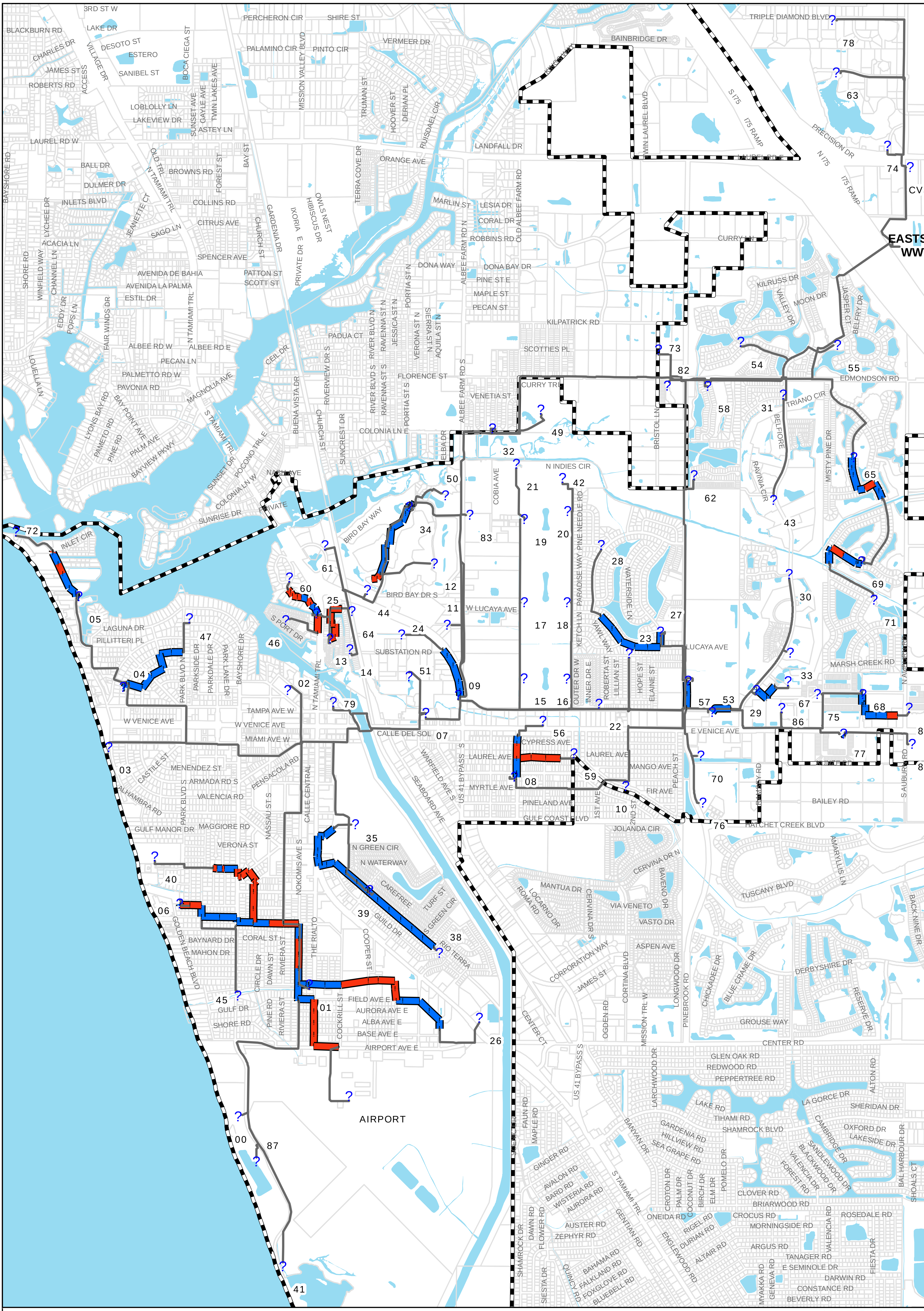
Table 5-17: Maximum Day Wet Weather 2025
Gravity with Velocity Less Than 2 fps (Continued)

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-1177	135	0.67	8	58	1.83
G-1764	101	0.67	8	53	1.90
G-2283	240	0.83	10	181	1.90
G-1140	349	0.83	10	122	1.90
G-4	31	0.83	10	129	1.90
G-1762	99	0.67	8	59	1.93
G-2190	259	0.67	8	74	1.94
G-1760	161	0.67	8	63	1.94
G-601	365	0.67	8	89	1.95
G-1763	116	0.67	8	56	1.96
G-1139	351	0.83	10	118	1.99
G-1488	250	0.67	8	75	1.99

5.7 2030 Planning Period: Scenario 9-10 and Results

The results for the 2030 average day dry weather and maximum day wet weather model scenarios identified additional system deficiencies but none that specifically require an improvement. The deficiencies are a result of the additional flow projected to enter the collection system as discussed in **Section 3.3** and **Table 3-13**.

The gravity collection system has several pipes where the velocity during the maximum day wet weather scenario does not reach 2 fps, and therefore may not receive adequate flushing velocity to prevent the deposition of solids. **Figure 5-22** shows the 2030 gravity sewer velocities color coded during the maximum day wet weather scenario.



Legend

Gravity Sewer Velocity

- Less than 2 ft/sec
- Greater than 2 ft/sec
- Force Main
- City Boundary

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VENICE CITY ON THE GULF

Figure 5 - 22
City of Venice, FL
Maximum Day Wet Weather Scenario 2030
Gravity Sewer Velocities

There were several force mains which experienced a velocity of 6 fps or greater. The majority of the force mains listed remains unchanged from previous scenarios. These force mains are considered to be at capacity. There were no new force main segments, beyond that identified in previous scenarios, with velocities deemed to put the pipe at an increased risk of premature failure. The force mains, identified as

having a velocity less than 2 fps during the previous scenarios, remained relatively unchanged. These force mains may not receive adequate flushing velocity to prevent the deposition of solids. **Tables 5-18** through **5-20** summarize the deficiencies regarding velocities in the gravity sewer and force main.

Lift Stations 11 and 42 continued to overflow as discussed in Scenario 2025. Lift Station 9 continued to experience high water levels within the wetwell during the maximum day wet weather scenario but did not overflow as shown in **Figure 5-23**.

Table 5-18: Maximum Day Wet Weather 2025
Force Main with Velocity Greater Than 5 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-384	0.33	4	17	4685	12.3
FM-251	0.33	4	89	628	16.0
FM-184	0.33	4	303	488	12.4
FM-403	0.67	8	41	1898	12.1
FM-180	0.33	4	161	341	8.7
FM-351	0.83	10	19	2124	8.7
FM-391	2.00	24	24	11404	8.1
FM-44	0.33	4	104	238	7.2
FM-153	0.83	10	404	1680	6.9
FM-243	1.67	20	3832	6328	6.5
FM-170	1.33	16	7452	4006	6.4

Table 5-18: Maximum Day Wet Weather 2025
Force Main with Velocity Greater Than 5 fps (Continued)

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-123	0.67	8	507	1001	6.4
FM-201	0.67	8	18	1000	6.4
FM-57	0.33	4	125	243	6.2
FM-183	0.83	10	134	1511	6.2
FM-208	1.33	16	251	3855	6.2

Table 5-19: Maximum Day Wet Weather 2025
Force Main with Velocity Less Than 2 fps

ID	Diameter (ft)	Diameter (in)	Length (ft)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
FM-199	0.50	6	2462	168	1.9
FM-193	0.67	8	972	298	1.9
FM-37	0.50	6	3107	158	1.8
FM-128	0.50	6	451	155	1.8
FM-80	0.50	6	3072	134	1.5

Table 5-20: Maximum Day Wet Weather 2025
Gravity with Velocity Less Than 2 fps

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6019	98	1.50	18	118	0.46
G-6017	315	1.50	18	110	0.54
G-861	400	0.67	8	48	0.74
G-1189	265	0.50	6	12	0.97
G-1242	128	0.67	8	6	0.97
G-6013	75	1.50	18	94	1.02
G-1243	129	0.67	8	12	1.11
G-328	351	0.67	8	45	1.15
G-1188	352	0.67	8	24	1.18
G-6015	228	1.50	18	102	1.25
G-1353	195	0.83	10	175	1.32
G-1351	282	0.83	10	163	1.37
G-1187	116	0.67	8	36	1.39

Table 5-20: Maximum Day Wet Weather 2025 Gravity with Velocity Less Than 2 fps (Continued)

ID	Length (ft)	Diameter (ft)	Diameter (in)	Maximum Flow (gpm)	Maximum Velocity (ft/s)
G-6021	182	1.50	18	126	1.41
G-6011	186	1.50	18	86	1.41
G-1067	597	0.67	8	34	1.45
G-6037	272	1.50	18	184	1.45
G-6023	412	1.50	18	134	1.46
G-491	350	0.67	8	28	1.47
G-1051	284	0.67	8	42	1.49
G-6007	51	0.67	8	76	1.50
G-1186	316	0.67	8	49	1.53
G-995	352	1.25	15	360	1.59
G-992	359	1.25	15	327	1.60
G-2282	399	0.83	10	182	1.60
G-858	67	0.67	8	48	1.60
G-329	351	0.67	8	35	1.62
G-1054	277	0.67	8	54	1.62
G-1765	84	0.67	8	51	1.66
G-6003	103	0.50	6	76	1.68
G-12	262	0.83	10	111	1.73
G-1055	280	0.67	8	67	1.74
G-1006	333	1.25	15	353	1.78
G-590	266	0.67	8	152	1.78
G-971	375	1.25	15	341	1.79
G-2387	334	0.67	8	84	1.79
G-1081	656	0.83	10	202	1.81
G-6025	327	1.50	18	143	1.83
G-1037	284	0.67	8	80	1.83
G-1177	135	0.67	8	61	1.86
G-185	243	1.75	21	192	1.87
G-1764	101	0.67	8	54	1.90
G-4	31	0.83	10	131	1.91
G-1140	349	0.83	10	127	1.92
G-2283	240	0.83	10	192	1.92
G-1762	99	0.67	8	59	1.94
G-2190	259	0.67	8	75	1.94
G-1760	161	0.67	8	64	1.95
G-1763	116	0.67	8	56	1.96
G-601	365	0.67	8	94	1.98

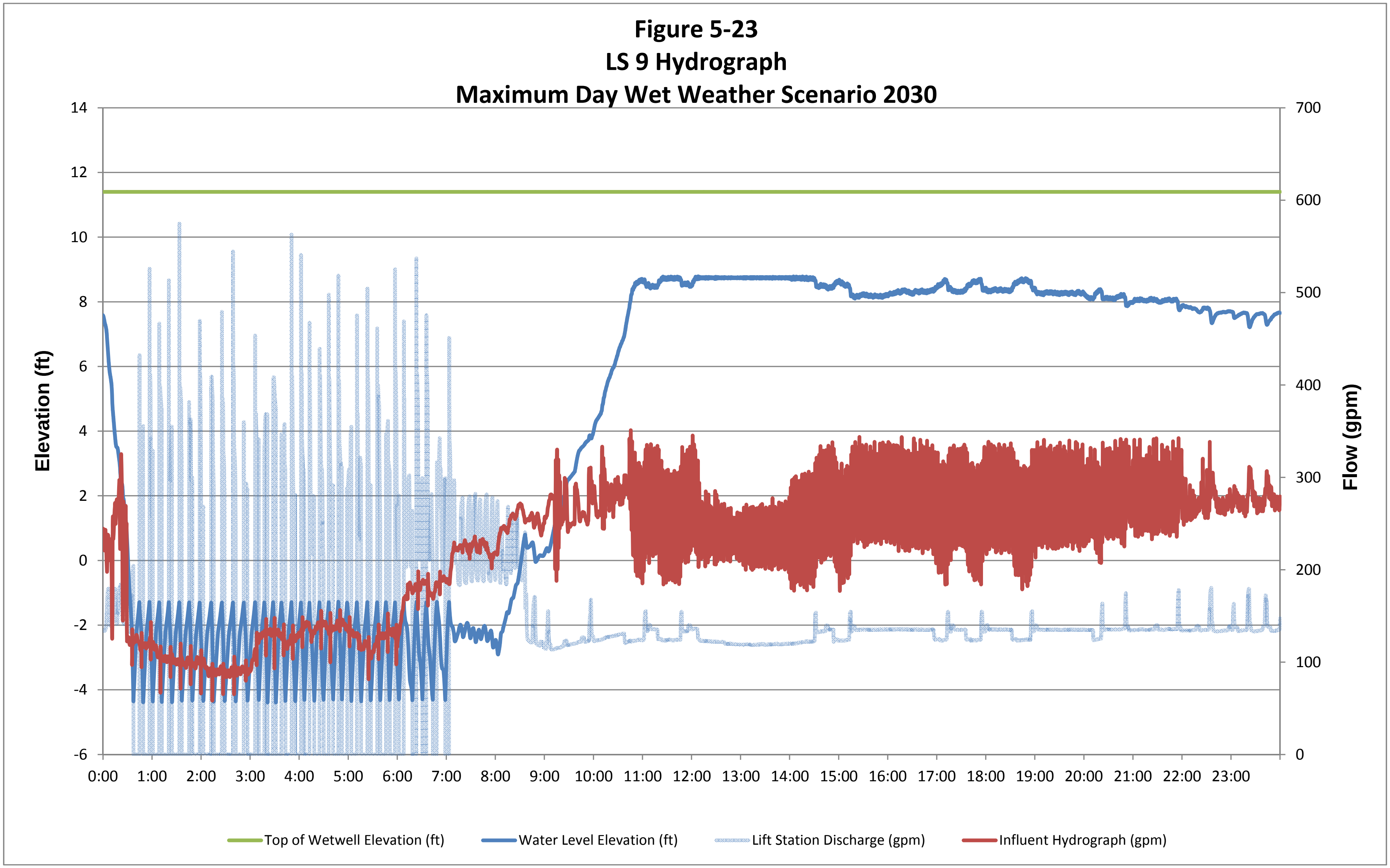
Figure 5-23
LS 9 Hydrograph
Maximum Day Wet Weather Scenario 2030
Elevation (ft)
Flow (gpm)
14
12
10
8
6
4
2
0
-2
-4
-6
700
600
500
400
300
200
100
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
Top of Wetwell Elevation (ft)
Water Level Elevation (ft)
Lift Station Discharge (gpm)
Influent Hydrograph (gpm)

5.8 2015 Planning Period Recommendations

Several improvements are recommended by 2015 in order to resolve some of the deficiencies discussed in **Section 5.4**. Force main 251, which is 4-inch and approximately 89 LF, discharges from LS 53 with simulated velocities up to 16 fps in 2010. It is recommended that it be replaced with a 6-inch force main to reduce the velocity. Force main 184, which is 4-inch and approximately 303 LF, discharges from LS 82 with simulated velocities up to 16 fps in 2015. It appears that the installed 50 HP pumps at LS 82 are oversized given the existing head conditions. It is recommended that the existing pumps be replaced with smaller pumps to reduce the velocity in the force main and improve pumps cycle and runtimes. Force main 403, which is a short segment of 8-inch pipe at the intersection of Miami Avenue and Nokomis Avenue, is currently in the design phase for replacement. The new configuration will eliminate the high velocities in force main 403, replace ACP pipe segments to the north and south, and improve flow routing.

Lift Station 11 was simulated as getting pushed back on it curve causing it to overflow. To resolve this force main 62 should be upsized from 8-inch to 12-inch. Increasing the size of the pumps in Lift Station 11 was evaluated first but it had a negative effect on downstream lift stations such as Lift Stations 83 and 34. It required those lift station pumps to be upsized also which negatively affected Lift Station 57. Upsizing force main 62 to reduce system head may be more cost effective than modifying several lift stations. Upsizing force main 62 also provided a near term solution regarding the simulated surcharging and overflowing of the gravity sewer along US 41 near the intersection of Albee Farm Road. The reduced system pressure allowed Lift Station 9 to pump at a higher flow rate and not surcharge the wet well and upstream gravity sewer. Lift Station 42 wet well was simulated in 2015 as 88% full at its peak. The existing pumps should be upsized to help the pumps keep up with the maximum inflow.

The operating point for the upsized Lift Station 42 pump was simulated at 130 gpm @ 76′ which has adequate capacity in 2015 as shown in **Figure 5-24.** It is recommended that during the design phase for this improvement further analysis be performed, including data collection of system flow rates and pressures, to refine the model and identify the extent of the capital improvements necessary in the future.

5.9 2020 Planning Period Recommendations

The single improvement recommended by 2020 is the gravity sewer pipe G-677, which was simulated as surcharging and with a high water levels in the upstream manhole. It is recommended that the existing 8-inch gravity sewer be replaced with a 12-inch gravity sewer to prevent surcharging.

5.10 2025 Planning Period Recommendations

Two lift station improvements are recommended in 2025. Lift Station 9 should have larger capacity pumps installed to prevent the wet well from surcharging and flooding the upstream manholes. The operating point for the upsized pumps at Lift Station 9 was simulated at 450 gpm @ 93′, which has adequate capacity in 2025 as shown in **Figure 5-25**. The upsized pumps at Lift Station 9 will require the pumps at Lift Station 32 to be upsized to maintain adequate capacity. The operating point for the upsized pump at Lift Station 32 was simulated at 1,000 gpm @187′. It is recommended that during the design phase for this improvement further analysis be performed, including data collection of system flow rates and pressures, to refine the model and identify the extent of the capital improvements necessary in the future.

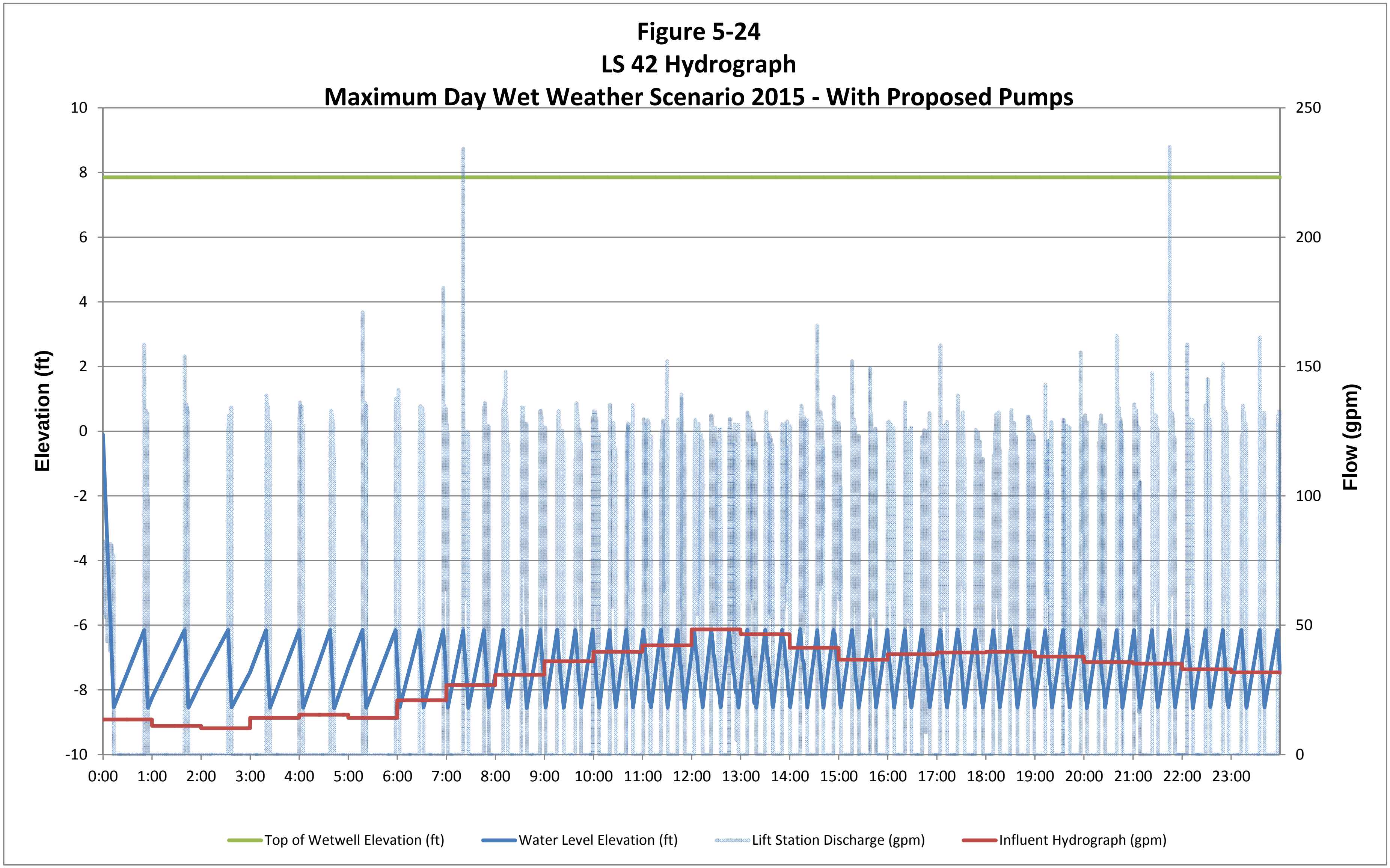

Figure 5-24
LS 42 Hydrograph
Maximum Day Wet Weather Scenario 2015 - With Proposed Pumps
Elevation (ft)
Flow (gpm)
10
8
6
4
2
0
-2
-4
-6
-8
-10
250
200
150
100
50
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
Top of Wetwell Elevation (ft)
Water Level Elevation (ft)
Lift Station Discharge (gpm)
Influent Hydrograph (gpm)

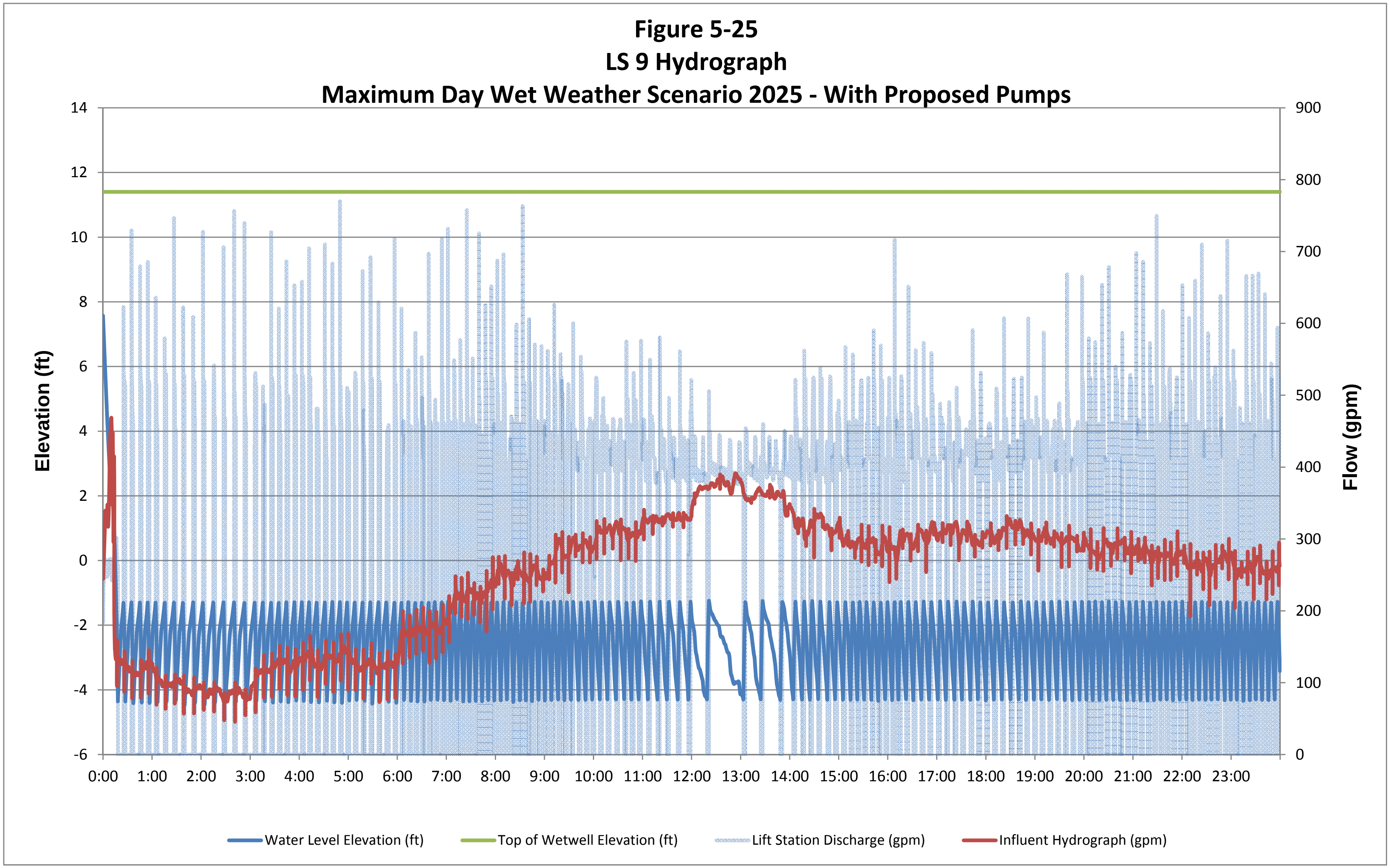

Figure 5-25
LS 9 Hydrograph
Maximum Day Wet Weather Scenario 2025 - With Proposed Pumps
Elevation (ft)
Flow (gpm)
14
12
10
8
6
4
2
0
-2
-4
-6
900
800
700
600
500
400
300
200
100
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
Water Level Elevation (ft)
Top of Wetwell Elevation (ft)
Lift Station Discharge (gpm)
Influent Hydrograph (gpm)

5.11 2030 Planning Period Recommendations

The 2030 average day dry weather and maximum day wet weather scenarios did not identify deficiencies that required improvements beyond what has already been recommended for improvement.

5.12 Lift Station 7 Recommendations

As discussed in **Section 1.3**, City staff reported that the system pressures may exceed 100 psi in the vicinity of Lift Station 7 as a result of several lift stations turning on at the same time; usually during a large rain event. The 2010 maximum day wet weather scenario simulated a maximum pressure of 87 psi in this area, but it was still below the reported pressures. One potential cause of the excessive pressure is a greater number of lift stations pumping downstream of Lift Station 7 at the same time than simulated. This condition was simulated by increasing the inflow to the manifolding lift stations so all the lift stations were on at the same time. The simulated pressure at Lift Station 7 increased to approximately 95 psi. Improvements to reduce I/I to the Class I lift station as identified in **Section 5.14** will help reduce the number of lift stations on at the same time during peak rain events.

The simulated maximum pressure was still below the peak pressures reported by City staff which suggests there may be other factors contributing to the excessive pressure. The force main from Lift Station 7 should be evaluated for locations where air may be trapped and restricting flow.

5.13 System Reliability Recommendations

Based on discussions with the City and review of the collection system, there are two primary areas of interest regarding system redundancy. The first is the 20-inch force main crossing I-75. This force main transmits all wastewater flow west of I-75 to the Eastside WRF. Should this pipe segment fail and/or require maintenance of any kind, a majority of the total wastewater to the Eastside WRF would be cut off. It is recommended that a 24-inch force main be installed parallel to the existing 20-inch force main crossing I-75 by 2015. A 24-inch force main was selected to maintain lower peak velocities in the force main relative to the existing 20-inch force main.

The second primary area is where two parallel 10-inch force mains cross the Intracoastal Waterway at East Venice Avenue and manifolds to a 16-inch force main. The two 10-inch force mains (FM-287 & FM-153) and 16-inch force main (FM-343) are believed to be cast iron pipe and installed in 1959. The southernmost force main has not been in operation for the last 8 to 10 years so only one force main is currently used to transmit flow off the island. A simulation was performed with one of the 10-inch force mains turned off to evaluate the hydraulic capacity of the single 10-inch force main. The maximum velocity in the remaining 10-inch force main was up to 8 fps, which is high for a cast iron pipe of its age. The 10-inch and 16-inch force mains are beyond their life expectancy and should be replaced. Parallel 12-inch and 16-inch force mains, assuming an upsized HDPE pipe for the HDD segments to maintain the effective internal diameters, were evaluated to replace the parallel 10-inch force mains. The maximum velocity with both parallel 12-inch force mains open was 5.5 fps. This velocity is high for designing a new force main so parallel 16-inch force mains are recommended under the Intracoastal Waterway to replace the parallel 10-inch force mains. Review of the pipe velocities shows that only maximum day wet weather scenario does not adequately provide a minimum flushing velocity greater than 2 fps in both force mains. Only one 16-inch force main should be open at any given time to produce acceptable flushing velocities during average conditions. The existing 16-inch cast iron force main should be replaced up to the wet well of Lift Station 7.

Diversion options were investigated in order to determine locations to loop the existing force main piping should a critical segment of force main fail or need to be taken offline for maintenance. Lift station 9 is in close proximity to the force main downstream of Lift Station 7 near the intersection of East Venice Avenue and Venice Bypass. Connecting the force mains associated with these lift station was evaluated to see if flow from Lift Station 7 could be diverted north along Venice Bypass. The results of the simulation showed that Lift Station 7 overwhelms all of the smaller lift stations along this route, even with the VFD at Lift Station 7 at 60% speed, and therefore is not feasible. No other diversion options to improve system reliability were identified.

5.14 I/I Improvement Recommendations

The inflow and infiltration desktop analysis as discussed in **Section 2** should be used to help direct future efforts to reduce the amount of I/I. The lift station I/I classification provided in **Table 2-2** should be referenced with the Class I sub-basins investigated first. Class I sub-basins, which only receive flow from their sub-basin and not from an upstream lift station, should be selected first for detailed field assessment since the increase in flow during rain events can be isolated to its gravity collection system. In order to determine which gravity sewers require immediate attention and which ones are projected to function satisfactory for several years, a CCTV inspection program implementing the NASSCO PACP and MACP is recommended. PACP and MACP will enable the City to create a comprehensive database using a standardized method to evaluate gravity sewer pipe for use in planning and renovating the gravity collection system in a cost effective way. It is recommended that the Class I sub-basins be the first areas classified under PACP and MACP in an effort to eventually classify the entire collection system.

Since the Class I basins are fairly large, it will be necessary to identify issues that are potentially the most likely to cause I/I and SSOs. A three step method to perform this evaluation is called pipeline and manhole triage. The triage approach employs high resolution, pole mounted zoom cameras to quickly assess and certify all accessible manholes, collect relative system information, and perform cursory pipeline inspections to identify high priority areas. Once those areas are identified, CCTV is used to closely inspect, evaluate, and identify defects.

The City of Venice is a coastal community so the groundwater is likely brackish which can be identified by elevated chloride levels. To help better identify areas of high infiltration, it is recommended that the City test lift stations for high TSS which is an indicator of high chlorides. Basins with high chlorides should be inspected first along with other Class I sub-basins.

Based on the immediate increase in several lift station run times during a rain event, it is evident that inflow is a major contributing factor. It is recommended the City continue to smoke test the wastewater collection system in order to identify major sources of inflow.

5.15 Lift Station Monitoring Recommendations

It is recommended that the City continue efforts to add telemetry to their lift stations. Telemetry will allow for a quicker response from operators when an alarm is triggered and improve operational monitoring. To support operational monitoring at the lift station, the installation of a flow meter at each lift station is recommended. Telemetry and flow meters will provide the City with the necessary information to troubleshoot operational problems such as worn out or clogged pumps, pumps not operating at their desired operating points due to system conditions, and excessive flow entering the wetwell due to I/I. As an alternative, flow can be approximated on lift stations with Data Flow SCADA systems installed by initiating a program to calculate flow rate based on the physical parameters of the lift station. This alternative is less expensive and recommended to be implemented by 2015. Flow meter installations can then be limited to locations where additional accuracy is needed. The data collected from telemetry will allow the City's wastewater hydraulic model to be updated and maintained for future system evaluations.

6.0 WATER RECLAMATION FACILITY

6.1 Existing Facility

The City of Venice owns and operates the Eastside WRF located at 3510 East Laurel Road in Venice, FL which treats domestic wastewater received from the City and Sarasota County. The original facility was put into service in July of 1992 and expanded in 2001 to its current capacity of 6.0 mgd on a maximum 3-month rolling average. A general site plan of the facility is shown in **Figure 6-1**.

The expanded facility consists of preliminary treatment followed by dual five-stage Bardenpho process trains, four clarifiers, three dual media automatic backwash traveling bridge filters, and three chlorine contact chambers fitted with a sodium hypochlorite system and the option to provide aeration in the event surface water discharge is necessary. Sludge is processed by four aerated holding tanks and dewatered using two belt filter presses prior to being transported by contract haulers for stabilization and final disposal. A process flow schematic is provided in **Figure 6-2**.

Reclaimed water is stored in either a 3 million gallon above-ground concrete storage tank or a 35-million gallon on-site lined storage pond. The City has the option to filter and disinfect the water stored in the pond prior to sending it into the reclaimed water distribution system. For disposal, the City has three permitted reuse locations and five permitted surface water locations. Substandard effluent is diverted to a 6 million gallon clay-lined reject pond where it can be returned to the headworks via gravity.

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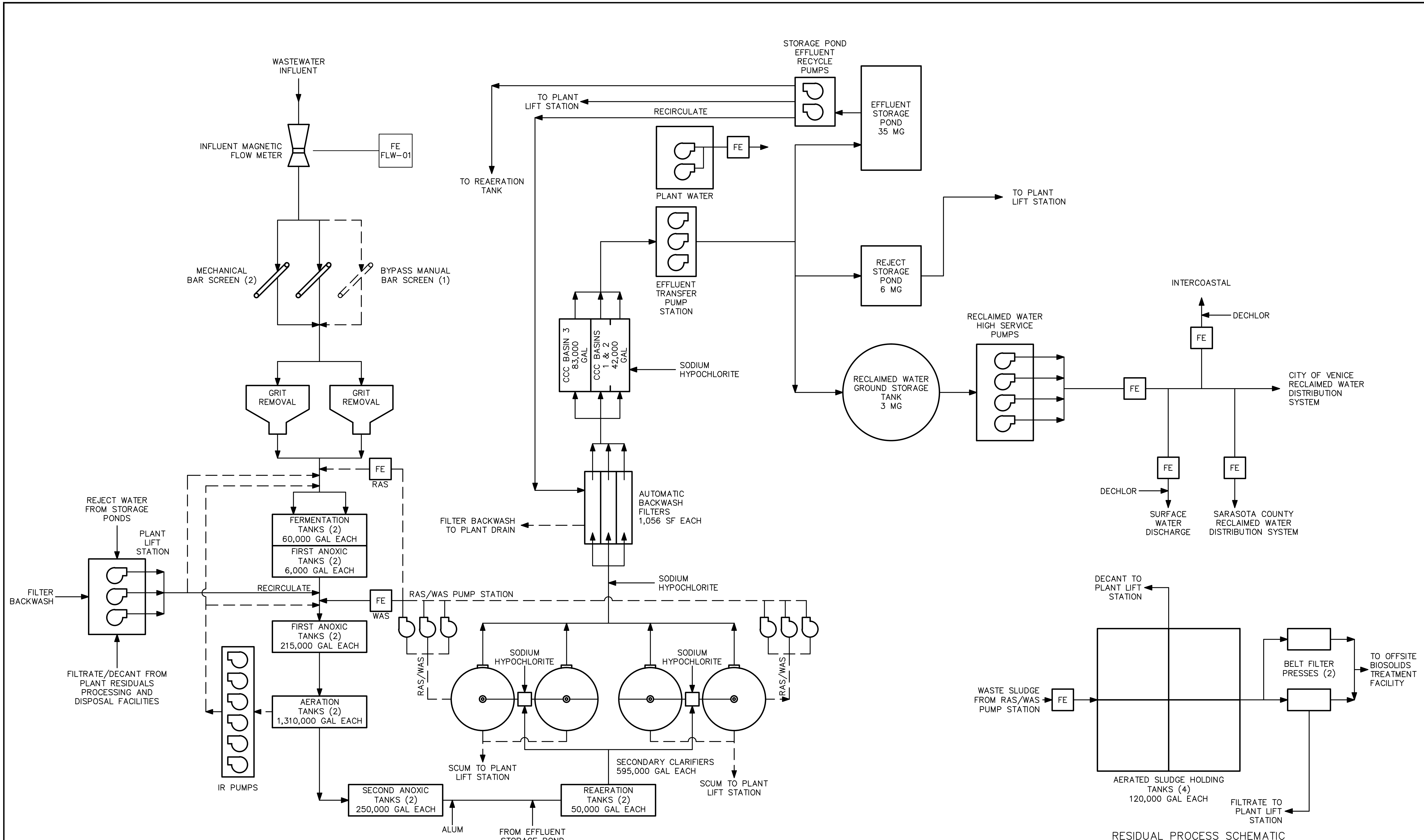
VENICE CITY ON THE GULF

CITY OF VENICE
WASTEWATER MASTER PLAN

EASTSIDE WRF GENERAL SITE PLAN

AUGUST 2012

FIGURE 6-1



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WASTEWATER MASTER PLAN

AUGUST 2012

EASTSIDE WRF PROCESS FLOW SCHEMATIC

FIGURE 6-2

S:\5883\0001\80-Drawings\Figure 6-2.dwg, 8/6/2012 9:31:09 AM, Brian Naught

The City recently submitted a permit renewal application and the supporting CAR and O&M Performance Report prepared by Malcolm Pirnie, Inc. to the FDEP for processing. The renewed permit is provided as **Appendix D** and is due to expire on December 11, 2016. The supporting documents concluded that the WRF will be operating at 70 percent and 82 percent of its permitted AADF and 3-MADF respectfully in the year 2030. Further, the permitted flow capacity will not be exceeded within the next 20 years based on the current flow projections. It should be noted that the CAR considers only historic flows from Sarasota County and not the required 3.0 mgd of capacity to be reserved for Sarasota County. Further, the CAR indicates that plant flows from December 2009 through March 2010 increased significantly from historical flows (April of 2010 reached a 3-MADF of 5.24 mgd or 87% of permit capacity) and attributes this to an above average seasonal population increase combined with an unusual amount of precipitation. From May 2010 until May 2011 the 3-MADF receded to a high of 3.45 mgd or 58% of permitted capacity during the months of July, August, and September.

Table 6-1 summarizes the influent and effluent design parameters published in the Preliminary Design Report prepared by Boyle Engineering Corp. dated Nov. 1996. **Table 6-2** summarizes the current influent and effluent loadings from April 2009 through April 2010 as established in the facility's most recent CAR. Influent sampling for TN and TP is not performed by the City and is not required by the facility's operating permit. Effluent sampling of these nutrients under previous permits was only required if reclaimed water is discharged at the Curry Creek location, however the current operating permit requires the effluent be tested once per month for reporting purpose only. Over the past 5 years, this has only occurred for 11 days during a period in March and April of 2010.

Table 6-1: Design Parameters

Parameter	Influent		Effluent	
	Conc. (mg/l)	Loading (ppd)	Conc. (mg/l)	Loading (ppd)
$cBOD_5$	210	10,508	5 (max)	250(max)
TSS	231	11,559	5(max)	250(max)
TN	38	1,902	3	150
TP	8	400	1	50

Table 6-2: Current Parameters

Parameter	Influent (mg/l)			Effluent (mg/l)		
	Annual Ave	3-Month Max	Max Month*	Annual Ave	3-Month Max	Max Month*
$cBOD_5$	165	181	180	1.99	1.99	2.00
TSS	210	236	264	0.62	0.66	0.67

* Occurred 3/2010.

The City submitted a Rerate Assessment to the FDEP in 2011 indicating that the facility's biological process has the capacity to treat 8 MGD at a 3-MADF. As such, the City requested that the Department increase the discharge rate of R001 by an additional 2 mgd to further supply the City's reclaimed water needs as additional flow is treated. It is our understanding that this request was not granted due to the insufficient capacity of the chlorine contact basins and the ability to meet water quality requirements in the surface water outfall. It should be noted that if this condition was corrected, there may be other limiting factors.

The following is a summary of the existing WRF components and deficiencies based on review of existing as-built information, historical flows and waste strength characteristics, maintenance records, O&M performance reports, capacity analysis reports and available manufacturer's recommendation for maintenance. Typical life expectancies for wastewater equipment have been published in the Water Environment Federation Manual of Practice 8 *Design of Municipal Wastewater Treatment Plants* and the University of Michigan's *Center for Sustainable Systems* to be 15 to 20 years. For the purpose of this report equipment located in the more hostile environments such as the preliminary treatment, RAS/WAS, internal recycle, plant drain pump station and sludge processing we have estimated a life expectancy of 15-years. All other equipment has been estimated a life expectancy of 20 years.

Scheduled equipment maintenance and repairs at the facility are tracked through a computer program called AMMS. Upon implementation, the City imported the manufacturer's recommended maintenance schedule for each piece of equipment into the software's data base. As scheduled maintenance is due, the software generates work orders which are carried out by City staff. Subsequently, data is re-entered into the program indicating the work done and the date completed. Historic logs can be generated and reviewed for each piece of equipment stored within its data base. A review of the AMMS and a summary of closed work orders was conducted on major pieces of equipment from each process area and found that preventative maintenance was occurring and being documented.

6.1.1 Capacity Analysis

The original facility was put into service in 1991 and expanded in 2001 to its current capacity of 6.0 mgd on a maximum 3-month rolling average daily flow. **Table 6-3**, **Table 6-4,** and **Table 6-5** summarize projected wastewater flows to the facility in 5-year planning periods from 2015 to 2030.

Table 6-3: Projected Wastewater Flows from the City of Venice[1, 8]

Description	2010[2]	Planning Period			
		2015	2020	2025	2030
AADF[3]	2,190,000	2,640,000	3,020,000	3,400,000	3,760,000
MTMADF[4]	2,410,000	2,900,000	3,320,000	3,740,000	4,140,000
MMADF[5]	2,850,000	3,430,000	3,930,000	4,420,000	4,890,000
PDF[6]	4,750,000	5,650,000	6,410,000	7,170,000	7,890,000
PHF[7]	4,820,000	5,810,000	6,640,000	7,480,000	8,270,000

Table 6-4: Projected Wastewater Flows from Sarasota County[1, 6, 9]

Description	2010[2]	Planning Period			
		2015	2020	2025	2030
AADF[3]	1,280,000	1,460,000	1,650,000	1,790,000	1,790,000
MTMADF[4]	1,410,000	1,610,000	1,820,000	1,970,000	1,970,000
MMADF[5]	1,660,000	1,900,000	2,150,000	2,330,000	2,330,000
PDF[6]	2,560,000	2,920,000	3,300,000	3,580,000	3,580,000
PHF[7]	2,820,000	3,210,000	3,630,000	3,940,000	3,940,000

Table 6-5: Combined Projected Wastewater Flows to the Eastside WRF[7]

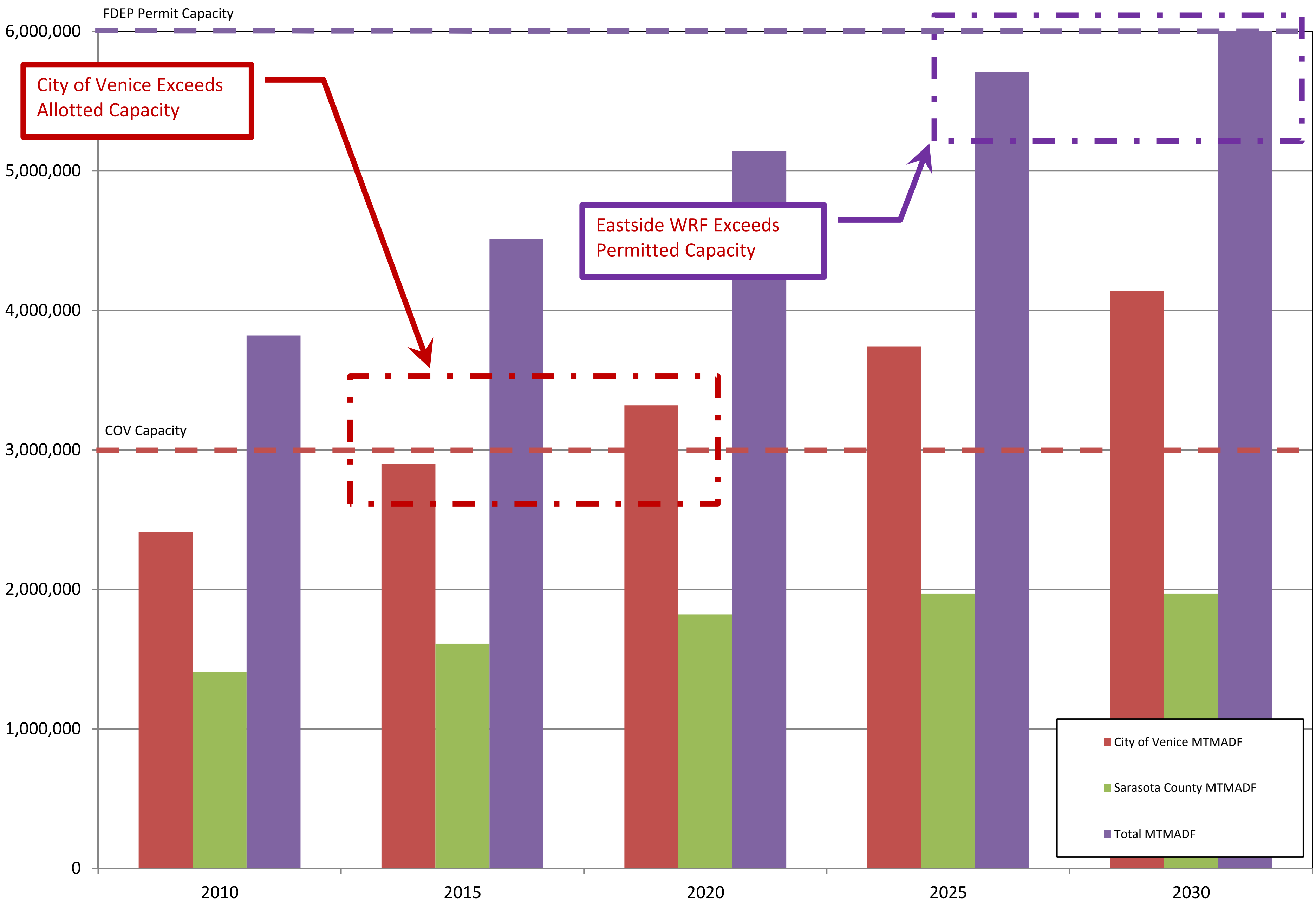
Description	2010[1]	Planning Period[7]			
		2015	2020	2025	2030
AADF[2]	3,470,000	4,100,000	4,670,000	5,190,000	5,550,000
MTMADF[3]	3,820,000	4,510,000	5,140,000	5,710,000	6,110,000
MMADF[4]	4,510,000	5,330,000	6,080,000	6,750,000	7,220,000
PDF[5]	7,310,000	8,570,000	9,710,000	10,750,000	11,470,000
PHF[6]	7,640,000	9,020,000	10,270,000	11,420,000	12,210,000

1. 2010 wastewater flows based on actually data obtained from the City of Venice.
2. Annual Average Daily Flow (AADF).
3. Maximum Three-Month Average Daily Flow (MTMADF), MTMADF = AADF x 1.1.
4. Maximum Month Average Daily Flow (MMADF), MMADF = AADF x 1.3.
5. Peak Daily Flow (PDF), PDF = AADF x 2.0 + I&I, Inflow & Infiltration (I&I) estimated at 369,000 gallons per day.
6. Peak Hourly Flow (PHF), PHF = AADF x 2.2, based on Recommended Standards for Wastewater Facilities (10 State Standards), 2007 Edition, Page 10-6.
7. Flows presented are in gallons per day (gpd).

The Eastside WRF operates under FDEP Permit Number FL0041441 with a permitted capacity of 6.0 mgd MTMADF as a Type I advanced wastewater treatment (dual train 5-stage Bardenpho process) domestic wastewater treatment plant. Further, the existing permitted capacity is apportioned so that the City has 3.0 mgd MTMADF and the County has 3.0 mgd MTMADF per the interlocal agreement, Contract No. C89-457 Amendment No. 1 entered into October 10th, 2000. This agreement allows the City the opportunity to utilize the County's unused capacity at a negotiated wastewater processing fee or to purchase capacity outright from the other party at a negotiated price. Similarly, the interlocal agreement provides reciprocating mechanisms for the County. At present, the City has not entered into negotiations with the County for either of these capacity mechanisms available through the interlocal agreement.

As illustrated in **Table 6-3** by the cells highlighted in light red, the City is anticipated to exceed the capacity defined in the interlocal agreement between the planning periods of 2015 and 2020. Likewise illustrated in **Table 6-5** by cells highlighted in light purple, the Eastside WRF is anticipated to exceed the FDEP permitted capacity between planning periods of 2025 and 2030. **Figure 6-3** summarizes the projected wastewater flows at the Eastside WRF with the relationship to the capacity meridians defined by the FDEP operating permit and the interlocal agreement.

Figure 6-3: Eastside WRF MTMADF Projections



These projections contrast with the updated CAR prepared May 2010, which indicates that the City will remain under the allotted 3.0 mgd MTMADF and the Eastside WRF will remain under the permitted 6.0 mgd MTMADF in the planning periods from 2015 to 2030. In brief, the methodology utilized in preparation of the CAR utilizes generation rate based on historical flows from 2007 to 2009 and applies that rate to the BEBR population projections for the Eastside WRF service area to predict future flows in the planning periods. This approach is typical for the preparation of CAR projections, however the growth for some non-residential land uses projected by the City of Venice Planning and Zoning Department in the WSFWP-10 and the City of Venice Comprehensive Plan are not encompassed in this methodology, but are considered in the projections utilized in this report. As such, the projections by which the following analysis and improvements are based upon are conservative in nature. The flows to the plant should continue to be monitored in order to verify the necessity of any recommendations.

6.1.2 Water Quality Characteristics

The current facility configuration consists of preliminary treatment followed by dual 5-Stage Bardenpho process trains, four clarifiers, three dual media automatic backwash traveling bridge filters, and three chlorine contact chambers fitted with a sodium hypochlorite system and the option to provide aeration in the event surface water discharge is necessary. Sludge is processed by four aerated holding tanks and dewatered using two belt filter presses prior to being transported by contract haulers for stabilization and final disposal. The facility has eight permitted effluent disposal sites as outlined in **Table 6-6**.

Table 6-6: Eastside WRF Permitted Effluent Disposal Sites

Permit Designation	Location	Permitted Capacity
D-001	Curry Creek	3.0 mgd AADF
D-002	Capri Isle Golf Course North, Stormwater Storage Lake System	Record Only
D-003	Capri Isle Golf Course South, Stormwater Storage Lake System	Record Only
D-004	Bird Bay Golf Course, Stormwater Storage Lake System	Record Only
D-005	Island Beach, Stormwater Storage Lake System	Record Only
R-001	City of Venice, Master Reuse System	3.0 mgd AADF
R-002	Sarasota County, South Master Reuse System	2.5 mgd AADF
R-003	Venice Reverse Osmosis Concentrate Disposal System	1.0 mgd AADF

The Eastside WRF treatment unit processes are limited by the most stringent requirements outlined by the current FDEP operating permit. As illustrated in **Table 6-7**, the disposal to Curry Creek (D-001) and to the Venice Reverse Osmosis Concentrate Disposal System (R-003) currently impose the process limitations at the Eastside WRF. As reported in the most recent CAR, the facility currently meets or exceeds the effluent requirements outlined in the FDEP operating permit. The evaluation of the existing conditions at the facility as presented in **Section 6.0** confirmed the individual processes at the facility presently have adequate capacity to meet the effluent criteria up to and including the permitted treatment capacity of 6.0 MTMADF.

Table 6-7: Process Water Quality Influent and Effluent Parameters

Parameter[4]	Influent Loading		Effluent Loading			
	mg/l[1]	ppd[1]	mg/l[2]	ppd[2]	mg/l[3]	ppd[3]
$cBOD_5$	210	10,508	5 (max)	250 (max)	20 (max)	1000 (max)
TSS	231	11,559	5 (max)	250 (max)	5 (max)	250 (max)
TN	35	1,902	3	150	Report	Report
TP	7	400	1	50	Report	Report

1. Design parameters presented for mass balance in the record drawings of the Eastside Wastewater Treatment Plant Expansion, January 2001.
2. Effluent parameters presented in the FDEP Permit Number FL0041441-011-DW1P/NR (expires 12-11-16) for disposal designations D-001 (page 3) and R-003 (page 13).
3. Effluent parameters presented in the FDEP Permit Number FL0041441-011-DW1P/NR (expires 12-11-16) for disposal designations R-001 (page 9) and R-002 (page 11).
4. $cBOD_5$ = 5 day carbonaceous biological oxygen demand, TSS = total suspended solids, TN = total nitrogen, TP = total phosphorus, mg/l = milligram per liter and ppd = pounds per day.

The City may elect to remove disposal sites D-001 and R-003 from the FDEP operating permit in the future should additional disposal capacity become available at the less restrictive locations. As illustrated in **Table 6-7**, this would ease the restrictions on the effluent quality parameters required by the unit processes at the facility. This may extend the permitted capacity of unit processes at the facility. However, for the purposes of this report, the more stringent effluent quality parameters are assumed to remain in place for the planning periods.

6.1.3 Preliminary Treatment

The original preliminary treatment installed in 1990 consisted of a mechanical bar screen, a manual bar screen and a grit removal system (grit chamber, grit classifier and two grit pumps). During the 2001 expansion, a second mechanical bar screen, grit chamber, grit classifier, and new influent 24-inch magmeter were added, thus doubling the preliminary treatment capacity to 12 mgd. A 30-inch by-pass pipeline was also constructed during the expansion to allow the required work at the preliminary treatment to be completed. This was left in place for the City's future use if required.

The mechanical equipment has been listed to be in a "Good" general condition in the latest O&M Report prepared earlier this year. The preliminary equipment was mentioned as an area of identified concerns; plant staff has indicated that the screenings and grit removal are insufficient. Further, the current FDEP Field Evaluation Form and Checklist indicated that some rags get through the screens and into the treatment plant. It should be noted that the original equipment installation is over 20 years old and has exceeded the estimated life expectancy. The original bar screen did get rehabilitated since the expansion occurred. The grit chambers are a free vortex technology with relatively low efficiency performance history in Florida's beach communities where the geology is dominated by very fine soils and sugar sands. A system inventory of the major preliminary treatment equipment is summarized in **Table 6-8.**

Table 6-8: Preliminary Treatment Major Equipment Summary

Unit	Qty	Mfg	Model	Capacity Ea.(mgd)	Year	Life Exp (years)
Mech. Bar Screen 1	1	Parkson	Aqua Guard AG-MN-A	12	1990	15
Grit Chamber 1	1	EIMCO	550 Jeta Grit Trap	12	1990	15
Grit Classifier 1	1	Wemco	1000C Wemclone		1990	15
Grit Pump	1	Wemco	3x3 C		1990	15
Grit Pump	1	Fairbanks	N/A		2001	15
Mech. Bar Screen 2	1	Parkson	Aqua Guard	12	2001	15
Grit Chamber 2	1	EIMCO	550 Jeta Grit Trap	12	2001	15
Grit Classifier 2	1	Wemco	1000C Wemclone		2001	15

1. Life expectancy presented in this report is given in years.

The original bar screen has been rehabilitated since the expansion occurred. Known improvements scheduled at the preliminary treatment include the rehabilitation of the two mechanical bars screens including material upgrades and the replacement of the 6mm bar screens with smaller 3 mm screens. Also, odor control may be installed on the influent channel. Odor sampling and analysis, along with an evaluation of control technologies, is currently being conducted.

6.1.4 Biological Process

The original biological process consisted of two Carrousels™ each having aeration, second anoxic and reaeraton within. This process was modified in 2001 to a five-stage nutrient removal process that includes four fermentation/first anoxic basins, two first anoxic basins, two aeration basins, two second anoxic basins and two reaeration basins.

The 2001 expansion modifications included:

- Removal of the existing second and reaeration processes from the Carrousels™
- Extending the Carrousels™ by 37.5-ft and adding two new surface aerators
- Alka-Pro System
- The addition of two more internal recycle (IR) pumps and piping modifications allowing for dedicated pumps to either the existing or new set of 1st Anoxic Basins complete with a stand-by pump for either
- The addition of four new Fermentaton/1st Anoxic Basins and four submerged mixers
- The addition of two new 2nd Anoxic Basins with submersible mixers
- The addition of coarse bubble diffusers in the reaeation process (existing centrifugal blowers were utilized)
- Capability to add alum to reaeration process for additional phosphorus removal

The design criteria of the biological basins are summarized in **Table 6-9.**

Table 6-9: Biological Process Design Criteria

Description	Units	Value
Fermentation / 1st Anoxic Basins		
Number of Basins	-	4
Surface Area (each)	sf	484
Volume (each)	gal	63,000
Sidewater Depth	ft	17.4
Width x Length	ft x ft	22 x 22
Detention Time	hr	1
1st Anoxic Basins (original basins)		
Number of Basins	-	2
Volume (each)	gal	215,000
Sidewater Depth	ft	14.9
Width x Length	ft x ft	44 x 44
Detention Time	hrs	1.72
Aeration Basins		
Number of Basins	-	2
Volume (each)	mg	1.31
Sidewater Depth (each)	ft	14.8
Width x Length	ft x ft	57 x 218
Detention Time	hrs	10
Freeboard at Aerators	ft	5.2
MLSS Conc.	mg/l	3,500-4,000
MLVSS Conc.	mg/l	2,450-2,800
Solids Retention Time	days	12.4

Table 6-9: Biological Process Design Criteria (Continued)

Description	Units	Value
Volumetric Loading	Lbs. BOD/1,000cf/day	31
Microorganisms Ratio	Lbs. BOD/lb MLVSS	0.18
Oxygen Requirement	lbs/hr	823
Oxygen Transfer Rate	lbs/hr/hp	2.9
WAS, total at ADF	lbs TSS/day	10,500
Effluent Weir Length	ft	15
IR Ratio	-	4-5
2nd Anoxic Basins		
Number of Basins	-	2
Volume (each)	gal	250,000
Sidewater Depth	ft	14.0
Width x Length	ft x ft	45.5 x 52.5
Total Detention Time	hrs	2
Reaeration Basins		
Number of Basins	-	2
Volume (each)	gal	50,000
Sidewater Depth	ft	12.4
Width x Length	ft x ft	10.5 x 52.5
Total Detention Time	hrs	0.4
Air Flow	SCFM	350

A system inventory of the major biological equipment is summarized in **Table 6-10**.

Table 6-10: Biological Process Major Equipment Summary

Unit	Qty	Mfg	Model	Flow (gpm)	Head (ft)	Year	Life Exp (years)
1st Anoxic Submerged Turbine Mixers 1 - 2	2	EIMCO	DR121K	-	-	1990	20
1st Anoxic Submerged Turbine Mixers 5 - 8	4	EIMCO	FZAM 148	-	-	2002	20
Surface Aerators 1 - 2	2	EIMCO	XSBL400	-	-	1990	20
Surface Aerators 3 -4	2	EIMCO	XSBN400	-	-	2002	20
2nd Anoxic Submerged Turbine Mixers	2	EIMCO	FZAZ 148	-	-	2002	20
Reaeration Blowers	2	Lamson	407-0-7-5000-AB	-	-	1990	20
IR Pumps 1 - 4	4	Lawrence	14" BPO	6,000	10	1990	15
IR Pumps 5 - 6	2	Hayward Gordon	XCS16c	6,000	10	2002	15

The mechanical equipment has been listed to be in a "Good" general condition in the latest O&M Report. However, it should be noted that Submerged Turbine Mixers 1 & 2, Surface Aerators 1 & 2 and IR Pumps 1-4 have been in service for over 20 years. Additionally, the City has indicated that the IR Pumps have been inefficient and are obsolete. The original pump manufacturer (Lawrence Pump and Engine Co.) is no longer in business and spare parts are not available. The

Alka-Pro system installed during the 2001 expansion to control DO levels never worked properly and its use has been discontinued.

Known improvements include the possibility of replacing the surface aerators with fine bubble diffusers with DO probes and control. The existing grit system would need to be upgraded for these improvements.

6.1.5 Secondary Clarification

Clarifiers 3 & 4, equal in dimensions to existing Clarifiers 1 & 2, were added during the 2002 expansion along with a new clarifier splitter box consisting of three chambers; influent, effluent, and sludge well. Similar to the existing clarifiers, they are fed from the center well and have bottom scraping, sludge plow type sludge removal mechanisms. Hydraulic pressure moves the sludge from the collection sump to the sludge well and is controlled by telescoping valves. Scum is collected by a rotating scrapper truss with a scum deflector that directs scum to a scum box.

Clarifiers 1 & 2 are equipped with pneumatic scum ejectors and associated air receivers and compressors. Clarifiers 3 & 4 were provided with positive displacement double-disk pumps. Collected scum is sent to the plant drain pump station.

The design criteria for the secondary clarifiers are summarized in **Table 6-11.**

Table 6-11: Secondary Clarification Design Criteria

Description	Units	Value
Number of Clarifiers	-	4
Type of Clarifiers	-	circular
Diameter	ft	85
Surface Area, each Clarifier	sq. ft	5,675
Sidewater Depth	ft	14
Freeboard	ft	2
Design Flow, each Clarifier, ADF	mgd	1.5
Hydraulic Overflow Rate, four units in service @ ADF	gpd/sq.ft.	264
Hydraulic Overflow Rate, three units in service @ ADF	gpd/sq.ft.	352
Detention Time at Design Flow, four units in service	hrs	9.5
Detention Time, three units in service @ADF	hrs	7.1
Solids Loading, each clarifier @ADF	lbs/sq.ft./hour	0.39
Clarifier Weir Length	ft	267
Weir Loading Rate @ADF	gpd/ft	5,618
Scum Pumping Capacity Head	- gpm ft	4 75 25
Sludge Telescoping Valves	-	4

Table 6-12 provides a summary of the major equipment associated with the secondary clarification process.

Table 6-12: Secondary Clarification Major Equipment Summary

Unit	Qty	Mfg	Model	Flow (gpm)	Head (ft)	Year	Life Exp (years)
Center Drive 1 - 2	2	EIMCO	C3	-	-	1990	20
Center Drive 3 - 4	2	EIMCO	C3	-	-	2002	20
Scum Ejector 1 - 2	2	James Eq. & Man. Co.	LBC	75	25	1990	20
Scum Pumps 3 - 4	2	Penn Valley Pump Co.	DD4	75	25	2002	20

The existing facility has a total of six RAS/WAS pumps, three for each pair of clarifiers that return RAS to the fermentation basins and/or second stage of the first anoxic basins. As required, WAS is bled off through a motor-operated plug valve and sent to the sludge holding tanks. The pumps are controlled by VFDs. The RAS and WAS flow rates are monitored by magnetic flow meters. **Table 6-13** provides a summary of the major RAS/WAS equipment.

Table 6-13: RAS/WAS Major Equipment Summary

Equipment	Qty	Mfg	Model	Flow (gpm)	Head (ft)	Year	Life Exp (years)
RAS/WAS Pumps 1 - 3	3	Aurora Pumps	611A	1,200	34	1990	15
RAS/WAS Pumps 4 - 6	3	Hayward Gordon	XCS8D	1,200	34	2002	15

The clarification and RAS/WAS mechanical equipment has been listed to be in a "Good" general condition in the latest O&M Report. There are no known improvements currently proposed for the secondary clarification process and RAS/WAS pumping facilities.

6.1.6 Filtration

The existing facility utilizes three traveling bridge filters to remove suspended solids from the clarified wastewater. Individual cells within each filter can be backwashed based on turbidity, headloss or timed sequence, allowing the remaining filter to stay on-line. The backwash is sent to the plant drain pump station, and the filtered effluent discharges into the chlorine contact basins. Traveling Bridge Filters 1 & 2 were installed with the original facility in 1990. Traveling Bridge Filter 3 was installed during the 2002 expansion. The filtration design criteria are provided in **Table 6-14**.

Table 6-14: Filtration Design Criteria

Description	Units	Value
Number of Tanks	each	3
Surface Area per Tank	sq.ft.	1,056
Loading Rate at ADF	gpm/sq.ft.	1.32
Loading Rate at PHF	gpm/sq.ft.	2.64
Design Headloss	inches	18
Filter Media Depth, Dual Media	inches	12 – sand 12 – anthracite
Backwash Pumps Capacity Motor	- gpm hp	3 350 5
Washwater Pumps Capacity Motor	- gpm hp	3 350 5

The major filtration equipment is summarized in **Table 6-15**.

Table 6-15: Filtration Major Equipment Summary

Unit	Qty	Manufacturer	Model	Year	Life Exp (years)
Traveling Bridge 1 - 2	2	Aqua-Aerobic Systems, Inc.	ABF-1666	1990	20
Traveling Bridge 3	1	Aqua-Aerobic Systems, Inc.	ABF-1666	2002	20

The filtration mechanical equipment has been listed to be in a "Good" general condition in the latest O&M Report. There are no known improvements currently proposed for the filtration process.

6.1.7 Chlorination

The chlorination process consists of three chlorine contact basins that receive filtered effluent from the traveling bridge filters. A third basin was added during the 2002 expansion. The basin's volume is just less than half the total volume of all three basins thus meeting Class I Reliability which requires at least 50% of the volume must be in service with the largest basin out of service. Rapid mixer/chemical inductors are provided at the head of each basin to mix the chlorine with the wastewater. Each chlorine contact basin is also fitted with coarse bubble diffusers fed by constant speed blowers which oxygenate the water in the event of surface water discharge. Flow is measured over each basin's effluent weir.

In 2002, the facility switched from chlorine gas to sodium hypochlorite. Peristaltic metering pumps with variable speed drives were installed to feed sodium hypochlorite to the clarifiers, filters and chlorine contact basins. The feed rate to the chlorine contact basins is flow paced. The design criteria for the chlorination process are provided in **Table 6-16**.

Table 6-16: Chlorination Design Criteria

Description	Units	Value
Design Dosage Rate	mg/1	10
Design Dosage Rate at Average Flow	lbs/day	500
Design Dosage Rate at Peak Flow	lbs/day	1,000
Design Chlorine Residual	mg/l	1.25
Influent Fecal Coliform	#/100 ml	<1,000
Number of Basins	-	3
Contact Time, @ ADF, Max Month	min.	40
Contact Time @ PHF	min.	20
Original Basins:		
Number of Basins		2
Design Flow, each basin	mgd	1.5
Volume, each basin	gal.	42,000
Channel Width	ft	8
Channel Depth	ft	6.97
New Basin:		
Design Flow	mgd	3.0
Volume	gal.	83,000
Channel Width	ft	8
Channel Depth	ft	6.97
Sodium Hypochlorite		
Tanks		3
Capacity (each)	gal	5,500
Chlorination System (design-peak)	ppd	500-1,000

The major equipment for the chlorination process is summarized in **Table 6-17**.

Table 6-17: Chlorination Major Equipment Summary

Unit	Qty	Manufacturer	Model	Year	Life Exp (years)
Blowers (275 scfm at 4 psi)	2	Lamson	407-0-7-4000-AB	1990	20
Peristaltic Metering Pumps	6	Watson-Marlow	SP/10	2002	20
Peristaltic Metering Pump	3	Watson-Marlow	SP/15	2002	20
Rapid Mixers 1 - 2	2	ABS	RW4042A 35/8CR	2002	20
Rapid Mixer 3	1	ABS	RW6533A 90/12CR	2002	20

The chlorination mechanical equipment has been listed to be in a "Good" general condition in the latest O&M Report. There are no known improvements currently proposed for the chlorination process.

6.1.8 Effluent Storage and Pumping

The chlorine contact basins discharge into the effluent wet well which contains the plant water pumps and the effluent transfer pumps. The plant water pumps operate on pressure and provide process water throughout the facility. The effluent transfer pumps operate on wet well level and, through valving options, can discharge to the reclaimed water storage tank, reclaimed water storage pond, or the reject storage pond.

The reclaimed water storage pond has return pumps that can recycle reclaimed water to either the reaeration tanks or to the plant drain pump station. The reject storage pond can be manually drained to the plant drain lift station for reprocessing. The reclaimed water storage tank supplies the reclaimed water pump station which can discharge into the three reuse or five surface water discharge locations summarized in **Table 6-18**.

Table 6-18: Discharge Summary

Discharge	Location	Permitted Capacity
D-001	Curry Creek	3.0 mgd AADF
D-002	Capri Isle Golf Course North, Stormwater Storage Lake System	Record Only
D-003	Capri Isle Golf Course South, Stormwater Storage Lake System	Record Only
D-004	Bird Bay Golf Course, Stormwater Storage Lake System	Record Only
D-005	Island Beach (Lake Venice Golf Club) Stormwater Storage Lake System	Record Only
R-001	City of Venice Master Reuse System	3.0 mgd AADF
R-002	Sarasota County South Master Reuse System	2.5 mgd AADF
R-003	City of Venice RO Concentrate Disposal System	1.0 mgd AADF

The effluent storage and the major pumping components are summarized in **Tables 6-19** and **6-20**.

The effluent pumping mechanical equipment has been listed to be in a "Good" general condition in the latest O&M Report. It should be noted that the City is not able to take the 3 MG ground storage tank out of service without taking the complete reuse system off-line.

Table 6-19: Effluent Storage

Unit	Qty	Capacity	Year	Life Exp (years)
RCW GST	1	3 mg	1990	50
RCW Lined Pond	1	35 mg	1990	50
Reject Lined Pond	1	6 mg	2001	50

Table 6-20: Effluent Pumping Major Equipment Summary

Unit	Qty	Mfg	Model	Flow (gpm)	Head (ft)	Year	Life Exp (years)
RCW Pumps	4	Ingersoll-Dresser	8LR14 A	2,780	150	2002	20
Plant Water Pumps[1]	2	Layne & Bowler		1,200	54	1990	20
Effluent Transfer Pumps	3	Ingersoll-Dresser	18 ENH-1	1,200	54	2002	20
Storage Pond Recycle Pumps	2	Hydromatic	S4L	500		2002	20

1. Since installation, one pump has been rebuilt and one pump has been replaced 6-7 years ago.

6.1.9 Solids Handling

The facility's original solids handling consisted of two sludge holding tanks with mechanical mixers and decanters, a lime feed system, and two sludge transfer pumps. WAS consisting of 0.8% solids was decanted to 1.0% and stabilized with lime before being pumped into tanker trucks for land application.

With the 2002 expansion, four additional tanks with course bubble diffusers and decanters were added along with two dual function belt filter presses (thickening and dewatering capability), three sludge feed pumps, two thickened sludge transfer pumps and two polymer feed systems. Currently, the City does not use the lime feed system or the original sludge holding tanks.

After decanting, the sludge is further thickened using the belt filter press. Polymer is added to condition the sludge. Subsequently, the sludge can then be returned to any of the sludge holding tanks via the thickened sludge transfer pumps for further processing or conveyed into a parked transfer truck. The City currently has Appalachian Material Service Inc. under contract through January 31, 2012 to haul and dispose of the biosolids. This contract is currently being extended for an additional 2 years. The solids handling design criteria and equipment are summarized in **Tables 6-21** and **6-22**.

Table 6-21: Solids Handling Design Criteria

Description	Units	Value
Sludge Production Rate (Annual Avg)	# / day	10,500
Sludge Concentrations WAS Decanted Thickened Dewatered	- % % % %	 0.8 1.0 3.0 – 5.0 15.0 – 17.0
Number of Tanks	-	6
Tank Type	-	Rectangular, Concrete
Total Tank Volume	-	460,000
Total Storage Time WAS Decanted Thickened (at 4.0% solids)	- days days days	 3.4 4.2 17.5
Sludge Air Blowers	-	4
Sludge Feed Pumps	-	3
Thickened Sludge Transfer Pumps		2
Spray Water Booster Pumps		2
Polymer Feed Pumps		2
Number of Belt Filters	---	2
Belt Width	meters	2
Belt Filter Design Feed Rate	gpm	200
Batch Run Frequency, per unit	hrs/day days/week	8 5

Table 6-22: Sludge Handling Mechanical Equipment

Unit	Qty	Mfg	Model	Flow (gpm)	Head (ft)	Year	Life Exp (years)
Thicken Sludge Transfer Pumps 1-2	2	Watson-Marlow	SP/80			2002	15
Sludge Feed Pumps 1-3	3	Alfa Laval	DRM 6/600	367	60	2002	15
Sludge Feed Pumps 5-6	2	Alfa Laval	DRM 6/600	400	47	2002	15
Submersible Mixers 1-2	2	Unknown	Unknown			1990	15
Blowers	4	Sutorbilt	GAFLDPA			2002	20
Belt Filter Press	2	Ashbrook Corp.	KP1127	200	-	2002	15
Wash Pumps	2	Flowserve	3*2*8 D814			2002	20
Polymer Feed	2	US Filter-Stranco	SP60101551			2002	20

The solids handling mechanical equipment have been listed to be in a "Good" general condition in the latest O&M Report. The sludge holding tanks and the headworks have been identified as locations to receive odor control.

6.1.10 Ancillary Equipment

The following are descriptions of ancillary equipment utilized at the facility.

Plant Drain Pump Station

The facility's sanitary sewer system collects sewage from the various on-site buildings and acts as a drain for the process tanks and pumps. The sewer system discharges to the plant drain pump station which returns it to the influent pipe after the preliminary treatment process. The existing plant drain pump station was constructed as part of the original facility in 1990 and consists of three submersible pumps rated for 650 gpm at 42-ft of head. The 15 hp pumps are manufactured by Hydromatic Model No. S4M1500M4-4.

Sulfur Dioxide Feed System

When discharging to surface waters, sulfur dioxide gas (SO_2) is used to dechlorinate the effluent at the end of the chlorination process. The dechlorination system consists of several 150 pound cylinders stored in an enclosure, two 500-ppd sulfonators, and two 100-ppd rotameters which were installed as part of the original facility.

Lime Feed System

For sludge stabilization, the facility previously utilized lime stored in a silo between the sludge holding tanks. The lime feed system consisted of storage silo, lime slaker, slurry tank and mixers, and slurry pumps. The system was installed as part of the original facility, but is no longer in use and most of the components have been removed.

Alum Feed System

The facility has the option to use alum to chemically remove phosphorus from the wastewater by injecting it into the reaeration process and again prior to the clarification process. The alum feed system was installed as part of the original facility and consisted of one 3,000 gallon storage tank and two feed pumps. The alum feed pumps are manufactured by Wallace and Tiernan, Model No. 44-114, rated for 80 gpd at 75 psi. A second 3,000 gallon tank was installed during the 2002 expansion.

Back-up Generator

The facility is equipped with a 1,500 kW back-up generator complete with critical grade exhaust silencer, automatic transfer switch, Pneumercator monitoring system, and 7,500 gallons of diesel fuel storage. The generator can provide an alternate source of power to operate the facility in the event of a power loss.

6.2 Expansion Requirements

As illustrated in **Figure 6-3**, the flow from the City's wastewater collection system, not including contribution from Sarasota County, is anticipated to exceed the 3.0 MTMADF capacity allotted to the City through the interlocal agreement between the planning periods of 2015 and 2020. Although Sarasota County is not projected to exceed their allotted 3.0 MTMADF capacity in the planning periods from 2015 to 2030, it is assumed that there is no available capacity remaining at the Eastside WRF. Therefore, the City must expand the capacity at the Eastside WRF by 1.14 mgd MTMADF or approximately 19% of the facilities total permitted capacity. In consideration of the more conservative flow projections utilized in this report, the City may choose to discuss with Sarasota County the use of their additional capacity at the facility via the mechanisms available in the interlocal agreement.

The City had the existing activated sludge biological process at the Eastside WRF evaluated for a potential capacity increase utilizing the existing basins with associated equipment in the fermentation, first anoxic, aeration, second anoxic and re-aeration reactors. This evaluation was considered only for those areas associated with the activated sludge process and did not extend to the hydraulic profile, pretreatment, secondary clarification, tertiary filtration, disinfection, reject storage, residuals management, RCW storage or RCW distribution. The evaluation was conducted in BioWin, a wastewater treatment process modeling software developed by Envirosim and a well-established tool in the wastewater industry. The results of this study are published in the Rerate Assessment Report dated February 2011 (Rerate Study) in which the modeling demonstrates that as much as 8.0 MTMADF can be processed with existing biological process infrastructure at the facility. Therefore, expansion for this report will be to a

FDEP permitted capacity of 8.0 mgd MTMADF or an increase of 2.0 mgd MTMADF. This additional capacity will meet the needs of the City beyond the 2030 planning period.

6.2.1 Hydraulic Profile Analysis

A hydraulic study should be conducted for the entire facility to confirm no flow restrictions exist in the gravity or pressure systems at a permitted capacity of 8.0 mgd MTMADF. The gravity hydraulic grade line of the facility is critical to maintaining the proper freeboard between the high water levels and the tops of structures, typically 12 inches or greater. The freeboard between high water levels and weir crests, typically 6 inches or greater, is also important to the performance of the facility. In particular, the freeboard in the effluent launder in the secondary clarifiers is critical, as a submerged effluent weir in the clarifier can cause eddies and currents that can carry high concentrations of solids from the clarification to the filtration, blinding that filter unit operation and creating a reject event. The pressure systems must be checked to confirm velocities and pressures do not exceed the design intent or good engineering practice. The hydraulics for all unit processes should be evaluated for Class I Reliability as outlined in 1974 by the USEPA publication MCD-05, Design Criteria For Mechanical, Electric, And Fluid System And Component Reliability and adopted by the FDEP under FAC Rule 17-610.462(l). **Table 6-23** presents a summary of design flows.

TABLE 6-23: Hydraulic Profile Analysis Parameters

Description	Flow	Unit	Basis
MTMADF	8.0	MGD	Rerate Study
PHF	17.6	MGD	10 State Standards
Return Activated Sludge (RAS)[1]	8.0	MGD	100%
Internal Recycle Rate (IR)[2]	40.0	MGD	500%

1. Wastewater Engineering Treatment, Disposal, and Reuse, Metcalf and Eddy, Inc, Third Edition, 1991 (M&E)
2. USEPA Manual, Nitrogen Control, September 1993

6.2.2 Pretreatment Facility Modifications

The pretreatment facility, or headworks, consists of screening and grit removal. As reflected in the Existing TM and presented in **Table 6-24**, the existing mechanical bar screen and grit classifiers appear to have adequate capacity to handle the increase in permitted flow.

Table 6-24: Summary of Pretreatment Facility Unit Operations

Unit	Capacity (mgd, each)	Year	Life Exp (years)	Capacity Available	Capacity Required
Mechanical Bar Screen #1[1]	12	1990	15	24 MGD	17.6 MGD
Mechanical Bar Screen #2[1]	12	2001	15		
Grit Chamber #1	12	1990	15	24 MGD	17.6 MGD
Grit Chamber #2	12	2001	15		
Grit Classifier #1	N/A	1990	15	N/A	N/A
Grit Classifier #2	N/A	2001	15	N/A	N/A
Grit Pump	N/A	1990	15	N/A	N/A
Grit Pump	N/A	2012	15	N/A	N/A

1. The City of Venice had the mechanical screens refurbished and rake screens replaced with openings reduced from 6 mm to 3 mm in 2011.

The headworks structure has an anticipated 50-year lifespan of which 28 years remain. The headworks structure is typically subjected to corrosive gases, hydrogen sulfide (H_2S), released from the influent flow at areas of turbulence. For this reason, the lifespan of the headworks structure can be greatly reduced when H_2S is allowed to pool in areas of stagnate air flow creating the increased corrosion potential. However, the channels of the existing headworks structure at the Eastside WRF are not enclosed, and H_2S is less likely to pool in areas with free air flow. The structure, from a visual perspective, appears to be in relatively good condition at this time. On the basis of age, condition and capacity, the headworks does not need modification to increase capacity.

The City recently conducted a study to evaluate odor at the Eastside WRF. The results are presented in Eastside Water Reclamation Facility Odor Control Study (Odor Study) dated December 2011. The Odor Study identified the headworks as the main source of odors at the facility, quantified at 98% of the objectionable odor source at the facility. Given the headworks is an elevated structure, the raw influent channels are uncovered, and the close proximity of the structure to I-75 directly adjacent to the facility, it was recommended to cover the channels and to install a biofilter able to remove 99% of the H_2S from the air stream at an estimated cost of $220,000.

As noted in **Table 6-24**, the effective screen size for the mechanical screens has been changed from 6 mm to 3 mm in order to capture a greater quantity of solids entering the facility from the collection system. Mechanical screens operate on hydraulic headloss differential from the upstream to the downstream water levels. Solids are captured on the upstream face of the screen, known as matting, which creates greater headloss and causes the upstream water level to rise, eventually activating a cleaning cycle at a preset height differential to remove the mat. With the effective screen size reduced to 3 mm, the matting will occur more quickly and the mechanical screen is expected to experience more frequent cleaning cycles. This phenomenon should be evaluated during the recommended hydraulic study as the increased headlosses from greater flows, coupled with the hastening of the cleaning cycles, may impact the existing mechanical screens ability to maintain effective capacity or the ability to maintain acceptable freeboard distances.

Grit consists of high density inorganic solids such as sand, gravel, cinders, eggshells, bone chips, seeds, coffee grounds, etc. with a specific gravity between 2.00 to 2.65 and has an effective size greater than 50 μ, 300 mesh. Facilities can receive as much as 5 cubic feet per million gallons of grit in the raw influent flow. The existing grit removal system is based on a traditional forced vortex principal that targets 150 μ or greater particle size for grit removal. This type of grit removal technology is not classified as high efficiency.

The City of Venice is a coastal community in Florida that is susceptible to receiving very fine sands, particularly during rain events that tend to scour gravity collection systems of deposits left from I&I. The grit received at the facility can be expected to have a size distribution where about 30% to 40% of the quantity received will be smaller than 150 μ as this is typical for these types of communities. Further, this type of technology has difficulty removing grit encapsulated with grease. This occurrence is known as light grit phenomenon where the effective specific gravity of a particle is reduced well below 2.00 and the grit "floats" through the existing grit removal system. The existing grit technology may be removing as little as 20% to 30% of the incoming grit quantity from the raw influent wastewater.

Removing grit from the influent flow is an important component of most wastewater processing facilities. Grit abrades equipment at the facility and reduces the life expectancy of mechanical devices like pumps and increases their maintenance. Grit also deposits in areas of the facility with low velocities, typically the biological reactors, reducing the volume available to process nutrients at the facility. As the City is considering replacing the existing mechanical surface aerators, grit also greatly impacts the operation of diffused air systems by slowly covering diffuser heads, reducing air flow, and effectively starving areas of the biological reactors from process air.

Several high efficiency grit removal technologies exist at this time such as the Teacup, the Grit King and the Headcell. See the **Appendix M** for product information. These technologies typically target removal of 95% solids at a particles size of 100 μ or larger. Both the Teacup and the Grit King are free vortex technologies that subject the grit particles to shear forces and reduce the light grit phenomena, but require a great deal of headloss to achieve their benefits. The Headcell has much less restrictive hydraulic requirements, but may be inappropriate if the facility receives large grease loadings. Unfortunately, there are no high efficiency technologies that can be retrofitted within the existing headworks structure.

Table 6-25: High Efficiency Grit Improvement Alternative 1

Equipment / Discipline	Estimated Cost
Demolition	$ 15,000
Sitework	$ 10,000
Structural	$ 350,000
Mechanical	$ 50,000
Equipment	$ 525,000
Electrical (15%)	$ 140,000
I&C (10%)	$ 95,000
Subtotal	$ 1,185,000
Engineering & Construction Administration (20%)	$ 235,000
Contingency (30%)	$ 355,000
Total	$ 1,775,000

Table 6-25 presents the conceptual estimate to retrofit the existing headworks with a high efficiency grit removal system. This estimate is based on the Headcell equipment as this technology is the only option that can be installed with the facility's existing hydraulic profile without additional pumping. The existing influent channels and screening facilities are reused. The existing grit chambers are bypassed and a new grit structure must be constructed. The estimate does not include odor control or coatings.

Table 6-26: New Headworks Alternative 2

Equipment / Discipline	Estimated Cost
Demolition	$ 25,000
Sitework	$ 15,000
Structural	$ 550,000
Mechanical	$ 300,000
Equipment[1]	$ 780,000
Coatings	$ 60,000
Electrical (15%)	$ 360,000
I&C (10%)	$ 175,000
Subtotal	$ 2,265,000
Engineering & Construction Administration (20%)	$ 450,000
Contingency (30%)	$ 680,000
Total	$ 3,395,000

1. Includes odor control and reuse of mechanical screens as they were recently refurbished.

Although the existing pretreatment facility appears to meet the needs for a capacity increase, the recommendation is to replace the headworks facility (see **Table 6-26**) for the following reasons:

1. Upcoming installation of odor control modifications to the headworks represents a substantial capital investment improvement ($220,000) in a unit operation that may require substantial modifications for future considerations.

2. Unknown structural condition of the headworks may require further unanticipated modifications which may only be evident once the odor control is installed and high corrosive environment is more prevalent.

3. Existing structure and equipment may be inadequate for capacity increase pending the hydraulic analysis.

4. Existing grit equipment is inadequate to protect a diffused air system from grit depositing and would require significant capital investment to be replaced. $1,775,000 as a standalone project.

5. Components of the existing grit equipment are nearing the expected life span and will require significant investment to refurbish and/or replace.

6. Existing grit equipment may be inadequate given the coastal community influent wastewater characteristics anticipated at the facility. Replacement of the grit equipment should reduce maintenance at the facility and extend the life of downstream mechanical equipment.

6.2.3 Biological Process – Aeration Modifications

As presented in the Rerate Study, the existing biological process, consisting of fermentation, first anoxic, aeration, second anoxic and re-aeration reactors, does not require modification to increase capacity. However, the City is considering changing from the surface mechanical aerator system to a diffused air system. The existing surface mechanical aerator #1 and #2 have been in service for the 20 year life span expected, see **Table 6-27**. The existing surface mechanical aerator #3 and #4 have been in service for approximately half of the 20 year life span expected, see **Table 6-27**.

Table 6-27: Summary of Aeration Unit Operations

Unit	Manufacturer	Year	Life Exp (years)	Drive Type	Equipment Size (each)
Surface Aerators #1 & #2	EIMCO	1990	20	Constant	150 hp
Surface Aerators #3 & #4	EIMCO	2002	20	VFD	150 hp
IR Pumps #1 through #4	Lawrence	1990	20	VFD	6,000 gpm
IR Pumps #5 & #6	Hayward Gordon	2002	20	VFD	6,000 gpm

Given the age of the existing aeration systems and the anticipated improvement costs to refurbish or replace the existing surface mechanical aerators (see **Table 6-29**) in the near future, the replacement of the existing aeration system with diffused air may present a more attractive option for the City at this time.

Diffused air is also a more efficient method of providing dissolved oxygen (DO) into the biological process. The estimated connected power for the blowers required for diffused air will be approximately the same as the existing mechanical surface aerators (~600 hp, see appendices). However, with DO probes and VFDs, the diffused air system will provide greater energy efficiency in the control of DO provided as well as a higher oxygen transfer rate typical of diffused air systems. Some of the energy efficiency would be lost due to the need for submerged mechanical mixers required to keep velocities and flow patterns of the oxidation ditches that currently make up the aeration reactors. The basins could be modified as pass through plug flow reactors with no internal circulation; however, the estimated cost may be greater than that presented in

Table 6-28. The City has entered into the agreement with the County to analyze the current biological process at the Eastside WRF.

Table 6-28: Aeration System Improvements Alternative 1

Equipment / Discipline	Estimated Cost
Diffuser System	$400,000
Blowers	$400,000
Mechanical Mixers	$50,000
Piping	$150,000
Electrical (15%)	$150,000
I&C (10%)	$100,000
Subtotal	$1,250,000
Engineering & Construction Administration (20%)	$250,000
Contingency (30%)	$375,000
Total	$1,875,000

Table 6-29: Aeration System Improvements Alternative 2

Equipment / Discipline	Estimated Cost
Replace Surface Mechanical Aerator #1 & #2 w/ VFD	$400,000
Refurbish Surface Mechanical Aerator #3 & #4	$365,000
Electrical (10%)	$75,000
I&C (10%)	$75,000
Subtotal	$915,000
Engineering & Construction Administration (20%)	$180,000
Contingency (30%)	$275,000
Total	$1,370,000

The City has averaged approximately $425,000 per year in operating costs from 2007 to 2010, providing power to the facility to process wastewater. The aeration system at the facility represents nearly half of the power cost. Preliminary estimates demonstrate a potential 15% power reduction to the aeration power requirements utilizing diffused air. That is a cost savings of more than $30,000 per year which would pay for the estimated cost difference within 15 years. Given the volatility of power costs and the increasing flows at the facility, the payback period for the estimated cost differential may be shortened. Further analysis should be performed to more closely estimate the power differential and cost benefit; however the installation of diffused air as presented in Alternative 1 is recommended.

Table 6-30: Internal Recycle Pump Improvements

Equipment / Discipline	Estimated Cost
IR Pumps #1 through #4 Replacement	$485,000
IR Pumps #5 and #6 Replacement or Refurbishment	$220,000
Electrical (10%)	$70,000
I&C (5%)	$35,000
Subtotal	$810,000
Engineering & Construction Administration (20%)	$160,000
Contingency (30%)	$245,000
Total	$1,215,000

The internal recycle pumps installed in the original plant construction are at the end of their 20-year life expectancy. This encompasses IR Pump #1 though #4. The original manufacturer of these pumps, Lawrence Pump and Engine Company, is no longer in business and any maintenance requiring parts must be custom fabricated to keep them in service. The City has expressed the desire to replace these pumps, therefore it is recommended that at the same time IR Pump #5 and #6, manufactured by Hayward Gordon, are in need of refurbishing these pumps be replaced as well. (See **Table 6-30**).

6.2.4 Disinfection and Effluent Transfer Expansion

The secondary clarifiers and tertiary filters have enough processing ability available for the anticipated increase in permitted capacity including peak hour and Class I Reliability design flows. See appendices for summary spreadsheet.

Table 6-31: Hydraulic Retention Time Analysis of Existing CCC

Description	MTMADF (MGD)[1]			Units
	6.0	7.0	8.0	
MTMADF HRT[2]	43.3	37.1	32.5	Minutes
PHF HRT[3]	19.7	16.9	14.8	Minutes
Class I Reliability HRT[4]	19.7	16.9	14.8	Minutes
CT Value[5]	29.5	25.3	22.1	N/A
Volume Required[6]	0	0	47,664	Gallons

1. Total CCC Volume = 164,000 gallons with 3 basins
2. MTMADF Hydraulic Retention Time (HRT) minimum 30 minutes, 10 State Standards.
3. PHF HRT minimum 15 minutes, 10 States Standard & FAC 62-600.440(5)(b). PHF = MTMADF x 2.2, 10 State Standard.
4. Class I Reliability HRT minimum 15 minutes, 10 States Standard & FAC 62-600.440(5)(b). Class I Reliability criteria defined in MCD-05.
5. Target CL Residual = 1.25 mg/l per Operating Protocol approved as a part of FDEP Permit Number FL00414412-011-DW1P/NR (expires 12-11-16). CT = CL residual x PHF HRT > 25, FAC 62-600.440(5)(c)(1). Target CL residual will need to be increased to 1.5 mg/l in the future to maintain CT = 25.
6. Additional volume required to meet CT requirement outlined in FAC 62-600.440(5)(c)(1).

The City requested an increase in permitted capacity from 6.0 MGD MTMADF to 8.0 MGD MTMADF predicated upon the results of the Rerate Study; however, FDEP indicated that the current disinfection volumes are inadequate to meet the CT value of 25 at the increased flow requested. The analysis presented in **Table**

6-31 confirms the FDEP assertion. Although the existing permitted capacity is available at the facility, the capacity available is dependent upon the targeted CL residual as dictated by the disposal point. This allows a marginal ability for the plant to deal with quality effluent variation.

The City is currently considering additional capacity for the facility's RCW system, either by expanding their RCW system with the addition of new users, or increasing the quantity to be transferred to the Sarasota County RCW system through the existing interlocal agreement, Contract 89-457 Amendment No. 2. In the case of gaining additional capacity in the RCW system for the facility, the City may remove the facility's existing surface water discharges and approach FDEP for a variance of the current chlorine residual requirements which would reduce or obviate the need for additional contact volume.

Table 6-32: Summary of Effluent Transfer Pumping

Unit	Mfg	Year	Life Exp (years)	Qty	Capacity/Pump
Effluent Transfer Pumps	Ingersoll-Dresser	2002	20	3	4,200 gpm (6 MGD)

The existing plant effluent transfer pumps have adequate pumping capacity to move the increased permitted capacity and the anticipated peak hourly flow (16.0 MGD) (See **Table 6-32)**. To meet Class I Reliability, an additional pump of similar size must be added for pumping redundancy (See **Table 6-33**). The existing yard effluent distribution network consists of 20" piping. The PHF will result in velocities at approximately 10 fps. Typically, yard piping within a facility is fully restrained and should be capable of handling these anticipated velocities; however, further evaluation should be conducted to confirm proper restraint of the existing piping.

Table 6-33: Disinfection and Effluent Transfer Estimate

Equipment / Discipline	Estimated Cost
Sitework	$ 10,000
Structural	$ 105,000
Effluent Transfer Pump	$ 35,000
Mechanical	$45,000
Electrical (15%)	$ 50,000
I&C (10%)	$ 30,000
Subtotal	$ 275,000
Engineering & Construction Administration (20%)	$ 55,000
Contingency (30%)	$ 85,000
Total	$ 415,000

Expanding the chlorine contact volume and installing an additional pump is recommended to develop additional capacity at the Eastside WRF.

6.2.5 Reject Storage Expansion

Per FAC Rule 62-640.464(3), the Eastside WRF must have one day of reject storage. The facility currently has a 6 million gallon clay-lined reject storage pond to store effluent not meeting permitted effluent limits. M&C conducted a brief volume analysis of the reject pond (see **Table 6-34**) based on aerial photographs available and information provided in the Eastside Wastewater Treatment Plant Expansion record drawings dated 2002. The existing control structure used to regulate flow into and out of the pond has a top of structure elevation at 20.67 and the over flow throat is set at elevation 20.0. It does not appear that the pond can be filled to an elevation of 21.0 as required by the volume analysis to achieve the 6.0 MG of reject storage.

Likewise, the top of bank for the reject pond is set at elevation 22.0 which provides 1 foot of freeboard. As the Eastside WRF is a coastal community, the 24- hour 100-year rainfall event can produce between 10 to 11 inches of rain volume as presented in the SWFWMD ERP Information Manual, Part D, Project Design Aids, Page D-11. The existing freeboard appears to be inadequate for the design capacity.

Table 6-34: Reject Pond Volume Analysis

Elevation	Surface Area	Volume (ft^3)	Volume (MG)
22.0[1]	157,270	957,400	7.2
21.0	151,410	803,060	6.0
20.0	145,560	654,580	4.9
19.0	139,700	511,950	3.8
18.0	133,840	375,180	2.8
17.0	127,990	244,260	1.8
16.0	122,130	119,200	0.8
15.0[2]	116,280	N/A	N/A

1. Elevation 22.0 is the top of bank (TOB).
2. Elevation 15.0 is the inner toe of slope (TOS).

There does not appear to be a spillway provided to allow the reject pond to overflow into the existing RCW storage pond. Although contaminating the 35 MG RCW storage pond would require retreatment of the entire volume through the plant, a spillway would provide a safety measure to protect the existing pond berm from failure and to contain a potential reject discharge event. Given the adjacent proximity the reject pond has to I-75, the pond is categorized as a Significant Hazard Class as presented in the USDA publication TR-60, Earth

Dams and Reservoirs, July 2005. A spillway should be considered given the limited freeboard available.

The existing reject pond can be expanded to provide the additional volume required to increase the permitted capacity as well as provide additional freeboard for safety. However, the expansion of the reject pond would require modification of the existing RCW storage pond and would cause a reduction in RCW storage available. The City is currently considering the construction of a second 3 MG pre-stressed RCW GST. If a new GST were placed in service, a protocol can be put into place where the new GST tank is to be drained during the early period of a reject event by pumping the RCW into the RCW system or transferring the RCW to the existing 35 MG RCW storage pond. Once emptied, the tank would be available for reject storage. Until the tank is emptied, the reject can be sent to the existing reject storage pond. A second RCW GST would provide additional RCW storage during normal operation and provide operational flexibility should a GST need to be out of service for maintenance. Therefore, a second 3 MG pre-stressed GST is recommended (See **Table 6-35**).

Table 6-35: Reject and RCW Storage

Equipment / Discipline	Estimated Cost
Sitework	$ 25,000
Pre-Stressed Ground Storage Tank	$ 1,000,000
Mechanical / Yard Piping	$100,000
Electrical (15%)	$ 150,000
I&C (10%)	$ 110,000
Subtotal	$ 1,385,000
Engineering & Construction Administration (20%)	$ 275,000
Contingency (30%)	$ 415,000
Total	$ 2,075,000

6.2.6 RCW Storage Pond Return and Filtration System

The City operates a 35-MG lined Part III RCW storage pond onsite at the Eastside WRF. The pond serves to attenuate the difference between RCW produced at the Eastside WRF and seasonal variations of RCW demand by system users. This is in contrast to the 3-MG GST located onsite that serves to store RCW water during the off-peak demand and peak diurnal production period for use during peak irrigation demands in the system. During seasonal periods of low customer demand for RCW in the system, the pond can be filled with the excess water to be stored and used when demands become higher than the RCW production available at the facility.

Under existing operations, the effluent transfer station can pump water to the RCW storage pond via an existing 20" main. Assuming 8 fps as the typical engineering design limit for a main, the existing main can shed approximately 9 MGD (6,500 gpm) of RCW excess to the storage. This appears to be adequate for facility needs under normal operating conditions. However, it should be noted that this will not handle the complete plant flow during diurnal peaks at the current permitted capacity and it will be necessary to allocate flows to the RCW system and storage pond accordingly during this periods of time.

The existing Storage Pond Recycle Pump Station consists of a duplex station with each pump having 500 gpm of capacity. The flow is returned to the head of the tertiary sand filters or to the biological process via a network of 8" and 10" mains. The system can return approximately 1.5 MGD (1,000 gpm) to the plant to augment RCW flow to meet system demand.

The existing RCW storage pond system presents several challenges to the operation of the facility. Firstly being an open top method of storage, the pond allows for direct sunlight to come in contact with the RCW water. Given the nutrients present in the RCW coupled with the addition of nutrients deposited into the pond by rain and wildlife, the top 18" of water depth is an ideal environment to grow green and blue-green algae that range in size from 2 µm to 100 µm in size. Depending on the length of time that the RCW is stored, the quantity of nutrients present in the water and the amount of sunlight, the algal content of RCW returned from the storage pond can be significant. Filtering algae is not the ideal application for traditional sand filters. Heavy algae loads to the filter can blind the filter more quickly and force frequent backwashing cycles. Algae can also bind with the sand to cause a phenomenon known as “mud-balling” that can be difficult to remove with typical backwash cycles and that can reduce the effective capacity of the filter. During heavy algae loadings, the facility is must return the RCW from the storage pond at reduced rates to offset impacts to the filtration system.

Secondly, the additional flow being sent to the filters must also be reprocessed through the chlorine contact basin. As previously discussed, the existing chlorine contact basin has a limited processing capacity that can be negatively impacted by additional flow reducing the contact time. Although the circumstances by which RCW would be returned under high diurnal flows is unusual, this remains a process limitation. Another impact to the chlorine contact basin is a higher consumption of chlorine for disinfection purposes. The chlorine residual must remain within the parameters outlined by the facility’s permit. This permit residual may be higher than the chlorine needed to polish the RCW returned from the pond.

Thirdly, the current configuration returns water having already met Part III standards back into the compliance zone. That water must meet all parameters of the permit, not just the fore mentioned chlorine residual, and may trigger reject events if the water quality parameters are not met.

To address these operational issues and to improve the availability of the stored RCW to meet system demands, an independent pond filtration system is recommended to filter the RCW returned from the pond. The pond filtration system should have 2.5 MGD of average (5 MGD peak) filtration capacity, should be a gravity disc configuration with a stainless steel media and should be sized for 25 µm screening size. In addition, the existing Storage Pond Recycle Pump Station should be upgraded to a duplex station with each pump having a capacity of 2.5 mgd (1750 gpm) and a new 16” RCW main should be installed to return water to the effluent transfer station wet well. These parameters are based from a study performed for Manatee County by McKim & Creed where it was found that the gravity disc filter targeting 25 µm with stainless steel media performed well for this application, unlike cloth media, and provided adequate protection for micro-irrigation type systems. The City should investigate pond algae constituents and available filter technologies during the design phase for optimal filter selection.

Table 6-36: Pond Filtration Improvements

Equipment / Discipline	Estimated Cost
Sitework	$20,000
Structural	$40,000
Mechanical	$200,000
Equipment	$700,000
Electrical (15%)	$145,000
I&C (10%)	$95,000
Subtotal	$1,200,000
Engineering & Construction Administration (20%)	$240,000
Contingency (30%)	$360,000
Total	$1,800,000

6.2.7 Auxiliary Systems

The following auxiliary systems were considered during the evaluations to upgrade or expansion of the plant: electrical, instrumentation, HVAC, and plant service water. There does not appear to be any deficiencies apparent or expressed by plant staff in these systems that will need to be addressed directly and immediately. However, these systems should be considered in the normal rehabilitation and restoration activities at the facilities. Any deficiencies not readily apparent can be addressed at that time.

The electrical system and backup generator appear to be adequate for the recommended improvements. The recommended improvements to pretreatment, aeration, internal recycle, disinfection, and RCW storage appear to be power neutral in that the anticipated electrical loads required for the improvements will be nearly the same as current electrical loads associated with existing equipment. The lake filtration power requirements are relatively low and should not overtax the existing system.

The plant staff did express the need for a redundant SCADA PLC. This is not a regulatory requirement of the facility, however given the potential for lightning strikes and power outages that are common in Florida, the addition of a redundant SCADA PLC is a prudent suggestions and is recommended. A separate CIP item has not been included. This should be addressed under the R&R activities when funds are available.

6.3 Rehabilitation and Restoration

The initial facility construction was completed in 1991 and was a "green field" project. The original facility was a 3 MGD MTMADF advanced wastewater treatment plant that included pretreatment, 4-Stage Bardenpho biological process, clarification, filtration, disinfection, reclaimed water distribution, reject storage, and sludge storage. The facility was expanded in 2002 to a 6 MGD

MTMADF advanced wastewater treatment plant that included upgrading: the biological process to a 5-Stage Bardenpho biological process; the capacity of the pretreatment, clarification, filtration, disinfection, reclaimed distribution and reject storage; and biosolids with additional storage, thickening and dewatering.

With few exceptions, the equipment at the facility has remained constant from initial construction to the expansion and from the expansion to the present. This creates the unique circumstance of having two large blocks of equipment nearing the end of their expected lifespans simultaneously. The equipment installed under the initial construction is at the end of their expected lifespan and the equipment installed under the expansion will be at the end of their expected lifespan in about 10 years. **Table 6-37** shows the equipment to be refurbished and/or replaced and the estimated cost. See **Appendix K** for complete table.

Table 6-37: Refurbish and Replacement Priority Table

Rank	Equipment Description[4]	Year[1]	Cost to Replace[2]	Lifespan		R&R Cost[3]
				Expected	Remains	
1	Clarifier Drive #1	1991	$ 202,000	20	-1	$ 183,000
2	Clarifier Drive #2	1991	$ 202,000	20	-1	$ 183,000
3	Redundant SCADA PLC	N/A	N/A	10	N/A	$ 100,000
4	RAS / WAS Pump #1	1991	$ 27,000	15	-6	$ 25,000
5	RAS / WAS Pump #2	1991	$ 27,000	15	-6	$ 25,000
6	RAS / WAS Pump #3	1991	$ 27,000	15	-6	$ 25,000
7	Scum Ejector #1	1991	$ 54,000	15	-6	$ 49,000
8	Scum Ejector #2	1991	$ 54,000	15	-6	$ 49,000
9	ABW Traveling Bridge #1	1991	$ 202,000	20	-1	$ 183,000
10	ABW Traveling Bridge #2	1991	$ 202,000	20	-1	$ 183,000
11	Submerged Turbine Mixer #1	1991	$ 27,000	20	-1	$ 25,000
12	Submerged Turbine Mixer #2	1991	$ 27,000	20	-1	$ 25,000
13	Reaeration Blower #1	1991	$ 40,000	20	-1	$ 36,000
14	Reaeration Blowers #2	1991	$ 40,000	20	-1	$ 36,000
15	Submersible Mixer #1	1991	$ 27,000	15	-6	$ 25,000
16	Submersible Mixer #2	1991	$ 27,000	15	-6	$ 25,000
17	Plant Drain Pump #1	1991	$ 13,000	20	-1	$ 12,000
18	Plant Drain Pump #2	1991	$ 13,000	20	-1	$ 12,000
19	Plant Drain Pump #3	1991	$ 13,000	20	-1	$ 12,000
20	CCC Blower #1	1991	$ 54,000	20	-1	$ 49,000
21	CCC Blower #2	1991	$ 54,000	20	-1	$ 49,000
22	Belt Filter Press #1	2002	$ 336,000	15	5	$ 306,000
23	Belt Filter Press #2	2002	$ 336,000	15	5	$ 306,000

Table 6-37: Refurbish and Replacement Priority Table (Continued)

Rank	Equipment Description[4]	Year[1]	Cost to Replace[2]	Lifespan		R&R Cost[3]
				Expected	Remains	
24	Polymer Feed Pump #1	2002	$ 13,000	20	10	$ 12,000
25	Polymer Feed Pump #2	2002	$ 13,000	20	10	$ 12,000
26	Sludge Blower #1	2002	$ 20,000	20	10	$ 18,000
27	Sludge Blower #2	2002	$ 20,000	20	10	$ 18,000
28	Sludge Blower #3	2002	$ 20,000	20	10	$ 18,000
29	Sludge Blower #4	2002	$ 20,000	20	10	$ 18,000
30	Sludge Feed Pump #1	2002	$ 20,000	15	5	$ 18,000
31	Sludge Feed Pump #2	2002	$ 20,000	15	5	$ 18,000
32	Sludge Feed Pump #3	2002	$ 20,000	15	5	$ 18,000
33	Sludge Feed Pump #4	2002	$ 20,000	15	5	$ 18,000
34	Sludge Feed Pump #5	2002	$ 20,000	15	5	$ 18,000
35	Sludge Feed Pump #6	2002	$ 20,000	15	5	$ 18,000
36	Thickened Sludge Transfer Pump #1	2002	$ 34,000	15	5	$ 31,000
37	Thickened Sludge Transfer Pump #2	2002	$ 34,000	15	5	$ 31,000
38	Wash Pump #1	2002	$ 13,000	20	10	$ 12,000
39	Wash Pump #2	2002	$ 13,000	20	10	$ 12,000
40	Clarifier Drive #3	2002	$ 202,000	20	10	$ 183,000
41	Clarifier Drive #4	2002	$ 202,000	20	10	$ 183,000
42	RAS / WAS Pump #4	2002	$ 20,000	15	5	$ 18,000
43	RAS / WAS Pump #5	2002	$ 20,000	15	5	$ 18,000
44	RAS / WAS Pump #6	2002	$ 20,000	15	5	$ 18,000
45	Scum Pump #3	2002	$ 54,000	15	5	$ 49,000
46	Scum Pump #4	2002	$ 54,000	15	5	$ 49,000
47	High Service RCW Pump #1	2002	$ 134,000	20	10	$ 122,000

Table 6-37: Refurbish and Replacement Priority Table (Continued)

Rank	Equipment Description[4]	Year[1]	Cost to Replace[2]	Lifespan		R&R Cost[3]
				Expected	Remains	
48	High Service RCW Pump #2	2002	$134,000	20	10	$ 122,000
49	High Service RCW Pump #3	2002	$134,000	20	10	$ 122,000
50	High Service RCW Pump #4	2002	$134,000	20	10	$ 122,000
51	Effluent Transfer Pump #1	2002	$34,000	20	10	$ 31,000
52	Effluent Transfer Pump #2	2002	$34,000	20	10	$ 31,000
53	Effluent Transfer Pump #3	2002	$34,000	20	10	$ 31,000
54	Submerged Turbine Mixer #3	2002	$27,000	20	10	$ 25,000
55	Submerged Turbine Mixer #4	2002	$27,000	20	10	$ 25,000
56	Submerged Turbine Mixer #5	2002	$27,000	20	10	$ 25,000
57	Submerged Turbine Mixer #6	2002	$27,000	20	10	$ 25,000
58	Submerged Turbine Mixer #1	2002	$27,000	20	10	$ 25,000
59	Submerged Turbine Mixer #2	2002	$27,000	20	10	$ 25,000
60	1500 KW Generator	2002	$605,000	20	10	$ 551,000
61	Peristaltic Pump #1	2002	$7,000	20	10	$ 7,000
62	Peristaltic Pump #2	2002	$7,000	20	10	$ 7,000
63	Peristaltic Pump #3	2002	$7,000	20	10	$ 7,000
64	Peristaltic Pump #4	2002	$7,000	20	10	$ 7,000
65	Peristaltic Pump #5	2002	$7,000	20	10	$ 7,000
66	Peristaltic Pump #6	2002	$7,000	20	10	$ 7,000
67	Peristaltic Pump #7	2002	$7,000	20	10	$ 7,000
68	Peristaltic Pump #8	2002	$7,000	20	10	$ 7,000
69	Peristaltic Pump #9	2002	$7,000	20	10	$ 7,000
70	ABW Traveling Bridge #3	2002	$202,000	20	10	$ 183,000

Table 6-37: Refurbish and Replacement Priority Table (Continued)

Rank	Equipment Description[4]	Year[1]	Cost to Replace[2]	Lifespan		R&R Cost[3]
				Expected	Remains	
71	Plant Water Pump #1	2005	$34,000	20	13	$ 31,000
72	Plant Water Pump #2	2005	$34,000	20	13	$ 31,000
73	Storage Pond Recycle Pump #1	2002	$34,000	20	10	$ 31,000
74	Storage Pond Recycle Pump #1	2002	$34,000	20	10	$ 31,000

1.Year of installation, refurbishing or replacement.
2.Replacement cost based on original equipment cost inflated 3% per year since installation.
3.R&R cost based on 70% of replacement cost + 30% contingency
4. Excludes equipment to be replaced as identified in the expansion requirements.

The methodology used to develop the rankings was based on age of the equipment, priorities expressed by City Staff and process area. The total value of improvements represented is approximately $4.375 million for an estimated annual renewal budget of $225,000 per year.

6.4 Biosolids Management and Regional Coordination

Both the EPA and FDEP regulate three biosolids disposal alternatives – land application, incineration or surface disposal. For Class B biosolids, land application requirements include both specific levels of pathogen reduction and VAR as well as a pollutant level ceiling. Land application for Class B also requires specific permitting requirements including setback distances and the recently added prohibition on odor. Land application regulatory requirements for Class A biosolids include site restrictions but no time limitation as with Class B.

At a minimum, disposal of biosolids at a landfill require some level of dewatering. Landfilled biosolids must pass a free-liquid test known as a "paint filter test" for disposal. A paint filter test is different than measuring for percent solids. It is a measurement of free-liquid within the material and is determined by placing the biosolids in a paint filter. If no liquids drop out after five minutes, the biosolids cake passes the test. Other landfill considerations include the transportation cost, tipping fees, and the landfill operators' potential challenges in moving the product onsite. Lastly, a minor FDEP permit modification is required if a site is not permitted to receive biosolids.

From a processing standpoint, incineration disposal focuses on meeting maximum metal concentration levels and some level of dewatering. Incineration disposal faces many of the same handling concerns as landfill disposal. The biosolids cake must be manageable in order to place in the correct area. Incineration also includes transportation cost, onsite processing cost, and the onsite operators' acceptance of the material. Lastly, permitting biosolids incineration through FDEP is more involved than landfill permitting due to emission requirements and may require a significant capital investment of air emissions monitoring equipment.

Surface disposal of biosolids can range from Class B to Class AA. Class B requires a significant reduction of pathogens and secondary pollutant limit standards as well as VAR treatment. In addition to the treatment requirements, there are setback restrictions to the application site. Based on this level of treatment, public access is limited for one year to Class B disposal sites and the regulations require monitoring and reporting of the cumulative pollutant loading. Class B sites are generally restricted public access areas such as agricultural sites, forests, roadway shoulders, and medians. Class A requires a higher level of pathogenic reduction than Class B but the same secondary pollutant limit and VAR standard. Public access is not limited, but there are site restrictions and required monitoring of the cumulative pollutant loading. Class A sites are generally in unrestricted public access areas such as playgrounds, parks, golf courses, lawns, and hospital grounds. Class AA biosolids are treated to the highest regulated levels for pathogen reduction, pollutant limits, and VAR; and this enables development as a fertilizer for distribution to the public. A higher level of dewatering is necessary in order to efficiently transport publically distributed Class A biosolids, and this requires a high capital cost and/or operating cost facility.

The City of Venice currently holds biosolids in aerated sludge holding tanks. The biosolids are thickened mechanically via gravity belt thickeners and by settling via decanting. The thickened is conveyed to a truck to be hauled off site for disposal. The City currently has Appalachian Material Service Inc. under contract through January of 2014 to haul and dispose of the biosolids.

M&C conducted a survey of local municipalities in the immediate area that presented likely partners to collaborate on a regional solution to biosolids management. The following is a summary of area municipalities' biosolids operations.

A. Bradenton

In the City of Bradenton, biosolids are processed at the WRF and the facility can produce Class B through Class AA Biosolids. Bradenton's biosolids are land applied at agricultural facilities in nearby Hardy County. Due to the high cost of processing, Bradenton is in discussions with Manatee County to use the County's dryer for treatment.

B. Charlotte County

Charlotte County currently transports liquid wastewater sludge from three water reclamation facilities to the East Port WRF for storage, decanting, and dewatering using a belt press. Charlotte County disposes of the biosolids at the Charlotte County landfill. Charlotte County is currently evaluating processing and disposing alternates.

C. Englewood Water District

The Englewood Water District currently dewaters sludge at the wastewater treatment plant using centrifuges. The Class B biosolids are hauled away by contract for landfill disposal.

D. Manatee County

Manatee County constructed a biosolids dryer system in 2009. The County dryer has a capacity of 20 dry ton per day at 18% solids and currently processes 9 dry tons per day leaving an excess capacity of 11 dry tons per day. Manatee County is currently in discussions with the City of Bradenton, the City of Sarasota and Sarasota County about accepting other municipalities' biosolids at the dryer facility. A sample contract for this type arrangement is currently under review by the Manatee County attorney. The potential exists for Manatee County to accept additional biosolids from other communities depending on capacity availability.

E. City of North Port

The City of North Port currently uses a Sarasota County contract with Synagro for sludge dewatering and landfill disposal. North Port is in the process of preparing specifications in order to add dewatering facilities at the City WRF. North Port will continue to contract transportation operations for biosolids disposal.

F. City of Sarasota

The City of Sarasota currently operates an in-vessel composting system at its wastewater treatment plant using woodchips and/or sawdust to create a soil amendment. Due to increasing costs of operation and the potential for odor generation, the City of Sarasota is in discussions with Manatee County to use their dryer for disposal.

G. Sarasota County

Sarasota County currently processes all biosolids through a contract with Synagro for dewatering and landfill disposal of all biosolids. The contract is in the second year of a three-year contract with two one-year renewal options, and there is no plan to change this approach. It is expected that Sarasota County will advertise a similar contract prior to expiration of the current one.

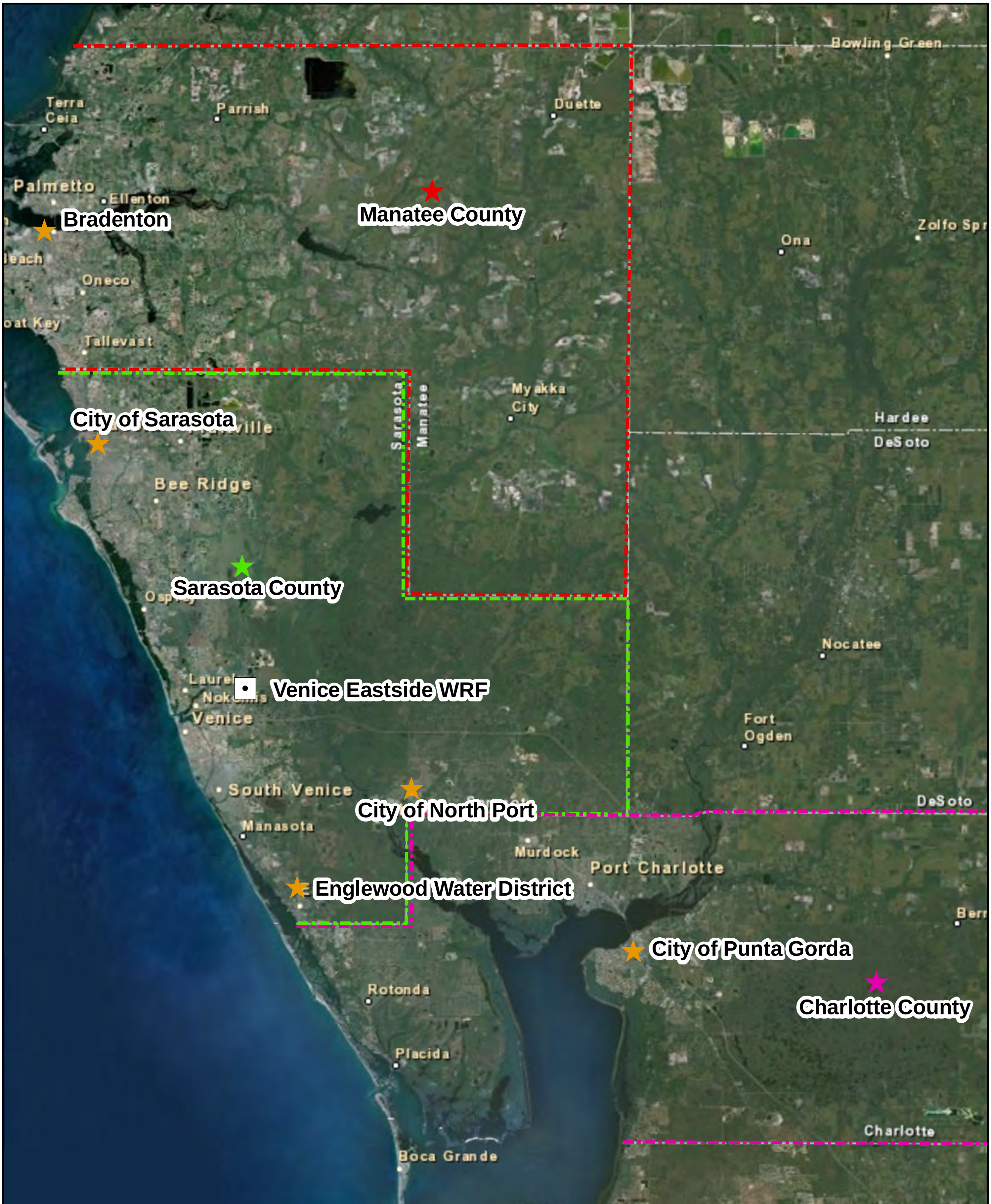
H. City of Punta Gorda

The City of Punta Gorda currently land applies biosolids on non-public access land at their WRF facility. The City uses aerobic digestion in order to stabilize and produce a Class B biosolids.

The City does not appear to need to expand the current onsite biosolids facilities. However, FDEP has implemented revisions to the biosolids rules, Chapter 62-640, on August 29, 2010, requiring all biosolids land application sites to be permitted by December 31, 2012. The revision is a significant change to the rule and includes:

- An engineer (or certified nutrient planner) must prepare a nutrient management plan for the life of the permit.
- An initial step is to develop and evaluate the site phosphorous index to determine whether site loading can be nitrogen based.
- Requires soil sampling for metals initially, and ongoing soil fertility sampling.
- Some changes are specific to lime-stabilized biosolids.

These revisions may result in a reduction in available disposal sites due to permitting costs driving landowners out of the market. Likewise, the volatility in fuel costs may create uncertainty in trucking costs which may make hauling sludge great distance more and more prohibitive. It is recommended that the City conduct further investigations into a biosolids management plan to deal with the long term needs of the facility.



Legend

Coordination Municipality

Water Reclamation Facility

Charlotte County Boundary

Manatee County Boundary

Sarasota County Boundary

Figure 6-4
Biosolids Regional Coordination

6.5 Water Reclamation Facility Recommendations

Table 6-38 summarizes the recommended improvements at the Eastside WRF. A more levelized approach is presented in **Figure 6-5** where the engineering costs are encumbered in the year prior to construction of the recommended improvements. The construction period for the pretreatment improvements was assumed to take 18 months given the complexities of keeping the plant fully operational while having to bypass certain key elements of the plant operation. These improvements are illustrated in **Figure 6-6**. **Section 7** summarizes these capital improvements based on planning year.

Table 6-38: Recommended Capital Improvements

Description	Years Implemented	Cost
Refurbish and Replacement Program	2013 to 2030	$225,000 / Annually
Hydraulic Study	2013	$ 50,000
Biosolids Management Plan	2013	$ 75,000
Reject and RCW Storage Improvements	2013 to 2014	$ 2,075,000
RCW Storage Pond Return and Filtration System	2014 to 2015	$ 1,800,000
Disinfection Improvements	2015 to 2016	$ 415,000
Pretreatment Improvements	2016 to 2018	$ 3,395,000
Aeration Improvements	2019 to 2020	$ 1,875,000
Internal Recycle Improvements	2021 to 2022	$ 1,215,000
	Total	$ 15,295,000

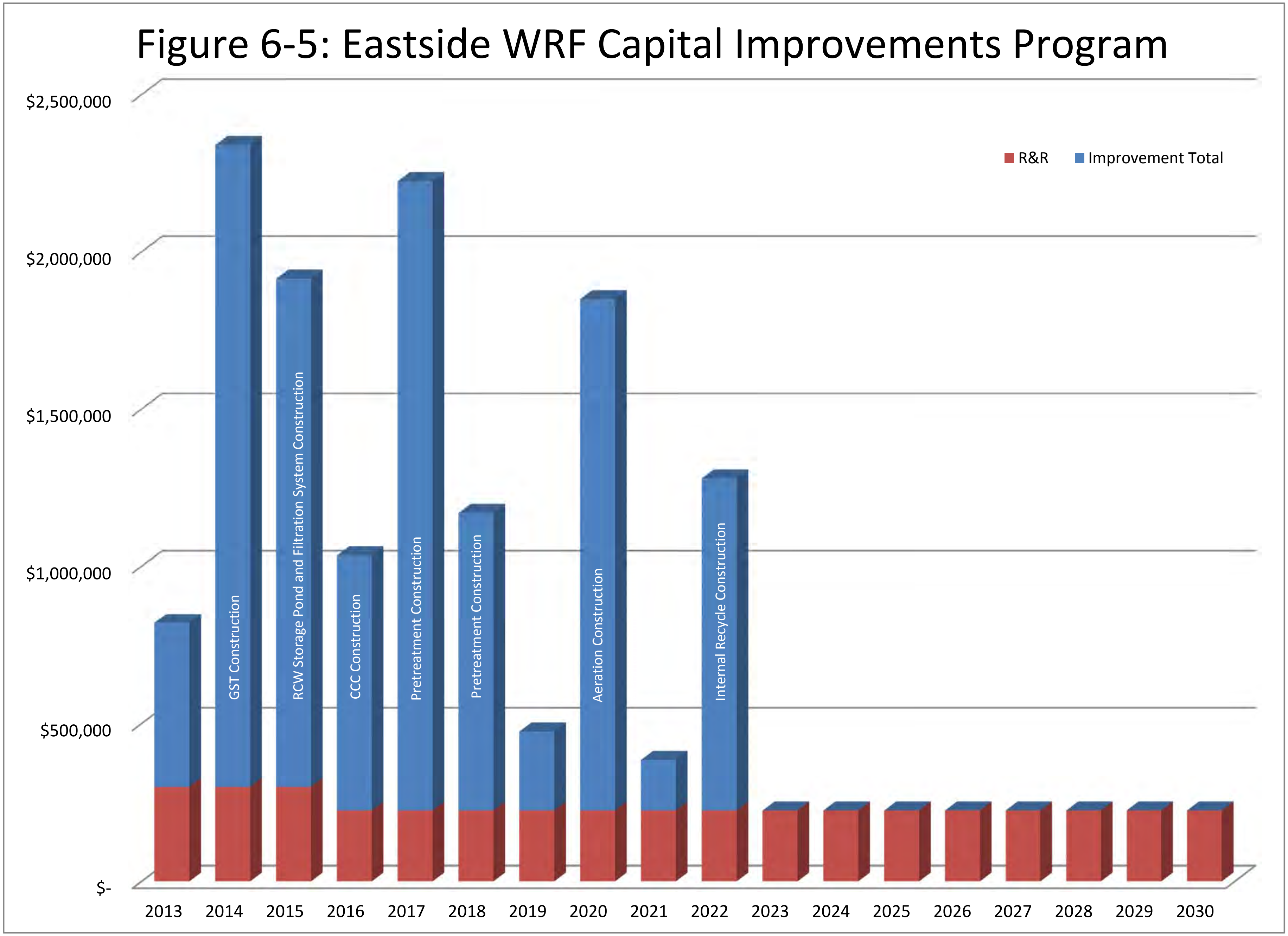


Figure 6-5: Eastside WRF Capital Improvements Program
$2,500,000
$2,000,000
$1,500,000
$1,000,000
$500,000
$-
R&R
Improvement Total
GST Construction
RCW Storage Pond and Filtration System Construction
CCC Construction
Pretreatment Construction
Pretreatment Construction
Aeration Construction
Internal Recycle Construction
2013
2014
2015
2016
2017
2018
2019
2020
2021
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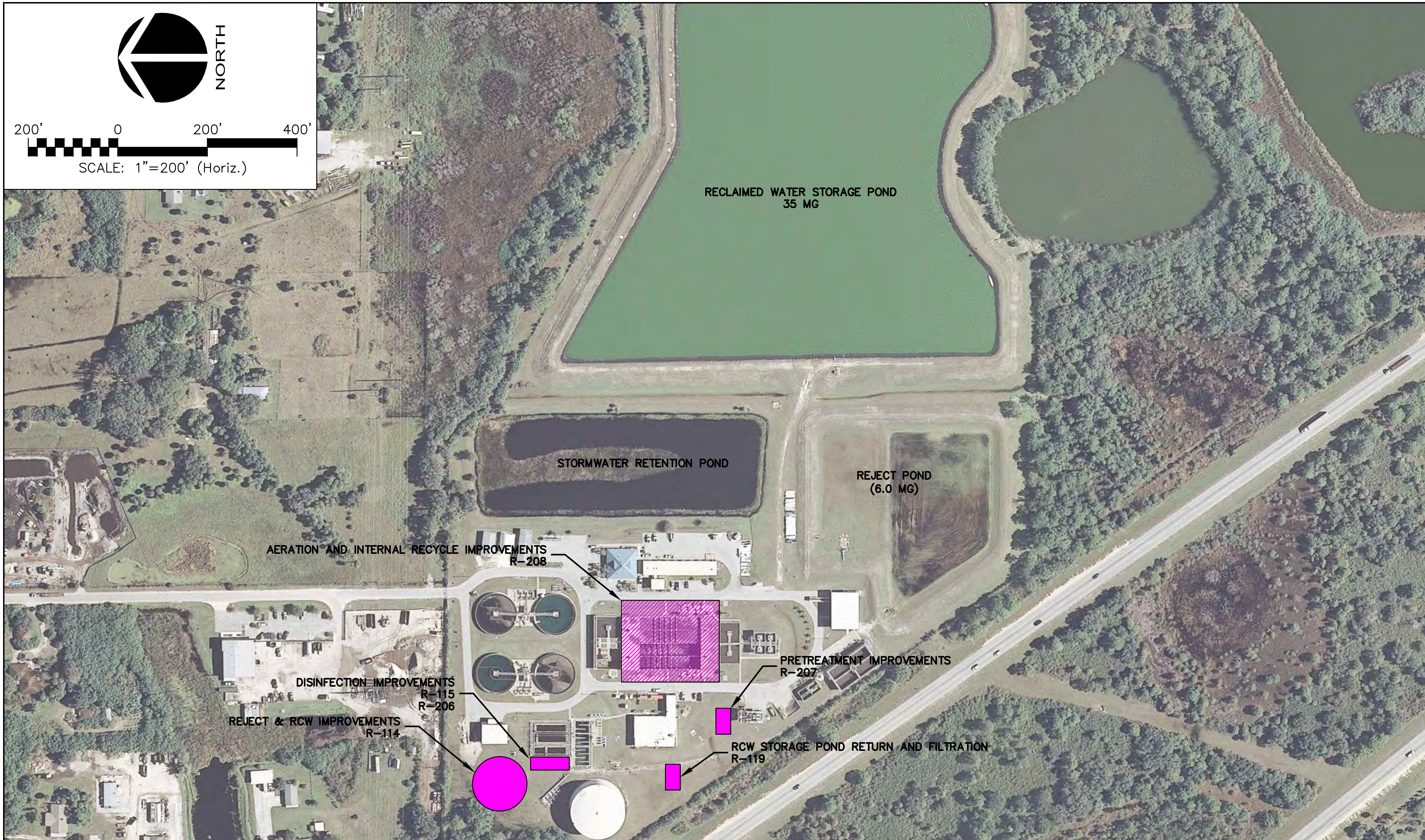

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VENICE
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CITY OF VENICE
WASTEWATER MASTER PLAN

EASTSIDE WRF RECOMMENDED
IMPROVEMENTS

AUGUST 2012

FIGURE 6-6

S:\5883\0001\80-Drawings\Figure 6-6.dwg, 8/6/2012 9:31:32 AM, Brian Naught

7.0 CAPITAL IMPROVEMENT PROGRAM

7.1 Cost Factors

The cost factors used to estimate the wastewater collection system improvements project costs are presented in **Tables 7-1** through **7-5**. All of the cost factors include design, administrative, construction, and contingency. **Table 7-6** shows the percentages of construction cost used to estimate the other project costs. The pipe cost estimates were derived from the contractor bids from 2009 through 2011. The cost for pump station capital improvements require further analysis be performed, including data collection of system flow rates and pressures, to refine the model and identify the extent of the capital improvements. Capital costs to resolve lift station deficiencies may vary substantially since a larger horsepower pump may require extensive modifications to the electrical system. All three lift stations identified for improvement will require a pump that has a higher horsepower rating than currently installed. A budgetary project cost estimate of $120,000 per lift station was therefore assumed.

Table 7-1: Pipe - Project Cost Estimates

Pipe Size & Material	Installed Cost (2012-$/LF)"
6" PVC	54
24" PVC	252

Table 7-2: Directionally Drilled Pipe - Project Cost Estimates

Directional Drill Pipe Size	Equivalent Pipe Size	Installed Cost (2012 - $/LF)
6″ HDPE	4″ PVC	$90
8″ HDPE	6″ PVC	$135
10″ HDPE	8″ PVC	$180
12″ HDPE	10″ PVC	$216
14″ HDPE	12″ PVC	$225
16″ HDPE	14″ PVC	$235
18″ HDPE	16″ PVC	$378
20″ HDPE	18″ PVC	$422
24″ HDPE	20″ PVC	$468
30″ HDPE	24″ PVC	$617

TABLE 7-3: Pipe Liner - Project Cost Estimates

Pipe Size	Installed Cost (2012-$/LF)
6″	$40
8″	$40
10″	$48
12″	$56
14″	$64
16″	$64
18″	$72
21″	$104
24″	$120

Table 7-4: PACP - Project Cost Estimates

Pipe Size	Inspection and Evaluation Cost (2012-$/LF)
Varies	$1.80

Table 7-5:MACP - Project Cost Estimates

Manhole Size/Depth	Inspection and Evaluation Cost (2012-$/MH)
Varies	$180

Table 7-6: Pipeline Project Costs
Assumed Percentage of Construction Cost

Item	Percent of Construction
Engineering	20%
City Administrative	10%
Valves and Fittings	20%
Contingency	30%

7.2 Project Costs and Recommended Phasing Implementation

Tables 7-7 and 7-8 summarize the cost of each recommended wastewater improvement project discussed in **Sections 5** and **6**. The improvements listed have been assigned a project identification "R" number. The first digit of the "R" number corresponds to the phase it is recommended. The number one (1) is associated with planning year 2015, two (2) with planning year 2020, three (3) with planning year 2025, and four (4) with planning year 2030.

Table 7-7: Recommended Wastewater Collection System Improvements

Project ID	Project Description	2015	2020	2025	2030
R-100	Install lower horsepower pumps in LS 82 to reduce the maximum velocity in the existing 4" FM.	$ 25,000			
R-101	Construct 89 LF of 6" HDPE FM to replace existing 4" FM at Royal Palm Rd and Ridgewood Ave.	$ 30,000			
R-102	Construct 2,400 LF of 12" PVC FM and 300' of 14" HDPE FM to replace 2,700 LF of 8" existing FM along Albee Farm Rd.	$ 284,000			
R-103	Install pumps with greater capacity in LS 42 to prevent wet well from surcharging.	$ 120,000			
R-104	Replace existing 8" FM with 10" HDPE and 12" PVC FMs. Revise connections to improve flow routing.	$ 232,000			
R-105	Construct 1,100 LF parallel 30" HDPE FM under I-75 to improve system redundancy.	$ 679,000			
R-116	Construct 2,233 LF of 16" PVC FM and 754 LF of parallel 18" HDPE FM to replace 2,987 LF of cast iron pipe to improve system redundancy.	$ 607,000			
R-200	Construct 12" gravity PVC sewer to replace existing 8" gravity sewer at LS 77.		$ 25,000		
R-300	Install pumps with greater capacity in LS 9 to prevent the wet well from surcharging and flooding the upstream manholes.			$ 120,000	

Table 7-7: Recommended Wastewater Collection System Improvements (Continued)

Project ID	Project Description	2015	2020	2025	2030
R-301	Install larger impeller and motor in LS 32 to increase lift station flow capacity.			$ 120,000	
R-106 R-201	Install telemetry units at all City lift stations with flow meters and pressure transmitter starting with high priority lift stations.	$ 384,000	$ 360,000		
R-107 R-202	Replace select existing lift station control panels as necessary with newer equipment to support the addition of telemetry.	$ 640,000	$ 600,000		
R-108 R-304	Assess the condition of all manholes and cursory inspection of adjacent pipelines with a pole mounted zoom camera using MACP. (Assumed perform every 10 years)	$ 470,000		$ 470,000	
R-109 R-203 R-305 R-400	CCTV video inspection of identified high priority gravity sewer pipelines using PACP (Assumed 35% of gravity sewer system by 2015, remaining 65% by 2020, 5% on-going).	$ 300,000	$ 560,000	$ 220,000	$ 220,000
R-110 R-204 R-306 R-401	Collection System R&R (Assumed liner rehabilitation on 20% of system by 2017, 2% annually on-going)	$2,480,000	$2,360,000	$1,860,000	$1,860,000
R-117 R-209 R-309 R-403	Odor control at master lift stations 7, 32, and 57. Appropriate technology to be determined. Assumed to be vapor phase technology for budgetary purposes.	$ 360,000	$ 275,000	$ 275,000	$ 275,000
	Total	**$6,575,000**	**$4,180,000**	**$3,065,000**	**$2,355,000**

Table 7-8: Recommended Water Reclamation Facility Improvements

Project ID	Project Description	2015	2020	2025	2030
R-111 R-205 R-307 R-402	R&R	$ 900,000	$ 1,125,000	$ 1,125,000	$1,125,000
R-112	Hydraulic Study	$ 50,000			
R-113	Biosolids Management Plan	$ 75,000			
R-114	Reject & RCW Improvements (GST)	$2,075,000			
R-118	SCADA Master Plan	$ 120,000			
R-119	RCW Storage Pond Return and Filtration System	$1,800,000			
R-206	Disinfection Improvements		$ 415,000		
R-207	Pretreatment Improvements		$ 3,395,000		
R-208	Aeration Improvements		$ 1,875,000		
R-308	Internal Recycle Improvements			$ 1,215,000	
	Total	**$5,020,000**	**$ 6,810,000**	**$ 2,340,000**	**$1,125,000**

7.3 Cost Summary

The following **Table 7-9** summarizes the recommended wastewater system improvements.

Table 7-9: Estimated Capital Improvement Costs by Phasing Period

Type of Improvement	2012-2015	2015-2020	2020-2025	2025-2030	Totals
Collection System	$ 6,575,000	$ 4,180,000	$ 3,065,000	$ 2,355,000	**$16,175,000**
WRF	$ 5,020,000	$ 6,810,000	$ 2,340,000	$ 1,125,000	**$15,295,000**
Totals	**$11,595,000**	**$10,990,000**	**$ 5,405,000**	**$ 3,480,000**	**$31,470,000**

8.0 OTHER CONSIDERATIONS

8.1 SCADA Master Plan

As the City's SCADA system grows it is important that the City has a road-map for the system's evolution in the form of a SCADA System Master Plan. This master plan will document the overall vision of the City's SCADA system and provide guidance and direction for future system enhancement and growth.

8.2 Master Lift Station Odor Control

The City currently has 3 master lift stations conveying flow directly to the Eastside WRF. Because of the quantity of wastewater, agitation, and time for wastewater to reach these master lift stations, odor generation is high at these locations. City staff has indicated these master lift stations are a priority for the addition of odor control. There are two types of technologies for odor caused by wastewater, vapor phase technologies and liquid phase technologies. The vapor phase technologies remove odor from the air. These technologies include chemical scrubbers, adsorbers/dry scrubbers, and biological filters. The liquid phase technologies use chemical and biochemical technologies to inhibit the generation of odor. These technologies include bioxide, iron salts, hydrogen peroxide, and sodium chlorite.

The City is using the liquid phase technology, AlkAQUIT at lift station 57 and conducting a pilot study of the AlkAQUIT system at lift station 7. The AlkAQUIT solution uses a combination of an alkaline enhancing component and dissolved sulfide inhibitor to control odor and corrosion within the wastewater system. It is recommended that both liquid and vapor phase technologies for odor control at the master lift stations be further investigated. The best technology will be dependent on the odor concentrations, site constraints, maintenance requirements and capital and operational costs. Generalized cost for the incorporation of liquid or vapor odor control is summarized in **Table 7-7** and **Appendix A**.

8.3 Asset Management Plan Study

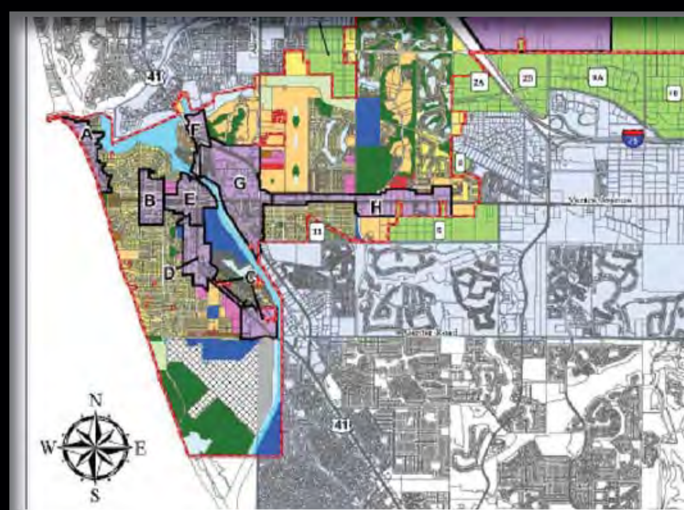
An Asset Management Plan (AMP) is a tactical plan for managing the City's infrastructure assets. The AMP should combine multi-disciplinary management techniques (including technical & financial) over the life cycle of the City's assets in the most cost effective manner to provide a specific level of service.

The City currently maintains and utilizes several databases to manage assets at the plant, the lift stations, and the collection system. These include GIS, Excel spreadsheets, accounting software and maintenance software. The City would benefit from a centralized approach to maintaining these sources of information.

As the City implements many of the recommendations in this report, the collection and management of data will become more and more critical to identify areas to focus for collection system improvements as well as keeping the efforts of the City's wastewater master planning up to date.

An AMP Study will assist the City in identifying the most effective method to centralize and share information as well as identifying areas of concern from an informational point of view. The City may also begin the effort of collecting information that is considered insufficient as a part of this effort.

City of Venice
Wastewater Master Plan
August 2012



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