CITY OF WEST SACRAMENTO

2015 Sewer Master Plan Update

DRAFT REPORT

SEPTEMBER 2017

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DRAFT REPORT | SEPTEMBER 2017

2015 Sewer Master Plan Update

Prepared for

City of West Sacramento

Project No. 040-06-14-25

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CHAPTER 1
Introduction

Chapter 1 presents the background and a brief summary of the 2015 Sewer Master Plan Update study.

1.1 BACKGROUND

The City of West Sacramento (City) has invested greatly in General Plan 2035. The General Plan update will guide how the City grows in the years ahead between now and 2035. The general pattern projected by the Draft General Plan 2035 is continued urbanization driven by infill and refill opportunities in the Bridge District, Washington Neighborhood, Pioneer Bluff Neighborhood, and the Central Business District. Southport is projected to continue its growth, driven by three potential developments in the area.

A critical element of the General Plan 2035 process is ensuring that the City's utilities will have the ability to serve existing and projected future customers. The 2015 Sewer Master Plan Update was undertaken to evaluate both the condition and capacity of the City's sanitary sewer collection system, and to recommend upgrades, improvements and new infrastructure where necessary to provide continuing service to the City's existing and future sewer collection system customers.

1.2 PROJECT DESCRIPTION

The 2015 Sewer Master Plan Update Project consisted of the following general tasks:

- Field Inspection of Collection System: Installation of temporary flow monitors to measure flow; visual inspection of pump stations; visual inspection of selected gravity mains and manholes.
- Capacity Assessment of Collection System: Hydraulic model creation; hydraulic model calibration for dry and wet weather, capacity evaluation of collection system using hydraulic model.
- Risk Assessment: Level of service evaluation; likelihood of failure analysis; consequence of failure analysis; prioritization of collection system by risk.
- Prioritized Capital Improvements Program (CIP): Risk-prioritized list of both condition and capacity improvements to serve existing customers and provide capacity for future growth.
- Financial Analysis: Evaluation of rate structure and connection fee structure required to meet existing levels of service while providing capacity for future growth.

1.3 REPORT ORGANIZATION

The 2015 Master Plan Update contains ten chapters followed by supporting appendices. The chapters are briefly described below:

- **Chapter 1 – Background**. This chapter presents the background and a brief summary of the 2015 Master Plan Update study.

- **Chapter 2 – Existing Sewer System**. This chapter describes the City's existing sewer system. System information was obtained through the review of previous reports, maps, plans, operating records, general plans, Geographic Information System (GIS) data, and other available data.
- **Chapter 3 – Design and Performance Criteria**. This chapter presents the design and performance criteria that were used to evaluate existing capacity in, and to size replacement facilities for, the City's wastewater collection system. Where available, existing City design and performance criteria were used. In other cases, industry standard criteria have been added to those already in place.
- **Chapter 4 – Hydraulic Model Development**. This chapter presents a summary of hydraulic model development and calibration. The computer-based hydraulic model of the City's sewer system, developed using Innovyze® InfoWorks™ CS software, serves as a tool for assessing the flows and capacities of the City's trunk sewers, and for identifying solutions to sewer capacity issues. The hydraulic model is also a tool for performing "what if" scenarios to assess the impacts of future developments, land use changes, and system configuration changes.
- **Chapter 5 – Dry Weather Flow Projections**. This chapter contains a summary overview of the development of existing dry weather flow values from flow monitoring and other data.
- **Chapter 6 – Wet Weather Flow Projections**. This chapter summarizes wet weather flows in the collection system.
- **Chapter 7 – Existing System Capacity Analysis**. This chapter provides an overview of the results of the hydraulic evaluation of the City's collection system under existing conditions. Collection system capacity for gravity mains, lift station/pump stations, and force mains is assessed with respect to the system's performance under the existing peak wet weather flows (PWWF) design flow condition described in Chapter 5 using the criteria described in Chapter 3.
- **Chapter 8 – Future System Capacity Analysis**. This chapter presents the results of the hydraulic evaluation of the City's collection system under 2035 design conditions. Collection system capacity for gravity mains, lift station/pump stations, and force mains is assessed with respect to the system's performance under the future PWWF design flow condition described in Chapter 6 using the criteria described in Chapter 3.
- **Chapter 9 – Asset Management Plan**. This chapter describes the efforts made to assess the condition of the collection system, as well as the resulting Asset Management Plan to manage and improve the condition of the collection system over time.
- **Chapter 10 – Capital Improvement Program**. This chapter presents the recommended Capital Improvement Program for the City's sewer collection system. The project recommendations, configurations, and conceptual costs that are presented in this chapter were described in previous chapters. This chapter summarizes and presents a consolidated list of projects with a recommended priority and implementation schedule.

- **Chapter 11 – Sewer Rate and Connection Fee Update.** This chapter presents the updated sewer rates and sewer connection fees based upon the recommended Capital Improvement Program. The background, approach, and analysis utilized in developing the rate and fee updates are provided in the chapter.

1.4 ACKNOWLEDGEMENTS

The 2015 Sewer Master Plan Update was a collaborative project involving the City and West Yost Associates (West Yost). West Yost acknowledges the following individuals for their collaboration and hard work throughout the course of the project:

- Vin Cay, City Project Manager
- Cameron Beebe, Engineering Manager
- Denix Anbiah, Director of Public Works
- Nitish Sharma, Budget Manager
- Wendy Williams, Senior Civil Engineer
- Mark Collier, Principal Civil Engineer
- Bob Kahrs, Maintenance Superintendent
- Dan Mount, Public Works Operations Manager
- Lyle Waite, Maintenance Superintendent

1.5 ABBREVIATIONS AND DEFINITIONS

To conserve space and to improve the readability of the 2015 Master Plan Update, the following abbreviations are used throughout this document.

2015 UWMP	City of West Sacramento Urban Water Master Plan
ABS	Acrylonitrile Butadiene Styrene
ADWF	Average Dry Weather Flow
AMP	Asset Management Plan
BWF	Base Wastewater Flow
CCI	Construction Cost Index
CCTV	Closed-Circuit Television
CEQA	California Environmental Quality Act
CHP	California Highway Patrol
CIP	Capital Improvement Plan
CIPP	Cured in Place Pipe
City	City of West Sacramento
CMMS	Computerized Maintenance Management System

d/D	Depth to Pipe Diameter Ratio
DIP	Ductile Iron Pipe
ENR	Engineering News Record
ENRCCI	Engineering News Report Construction Cost Index
EPA	Environmental Protection Agency
fps	Feet Per Second
ft/ft	Feet/feet
General Plan	City of West Sacramento General Plan 2015-2035
GIS	Geographic Information Systems
GWI	Groundwater Infiltration
HDD	Horizontal Direction Drilling
HDPE	High-Density Polyethylene
HGL	Hydraulic Grade Line
HVAC	Heating, Ventilation, and Air Conditioning
I/I	Inflow and Infiltration
ICCP	Impressed Correct Cathodic Protection
JDH	JDH Corrosion Consultants, Inc.
LNWI	Lower Northwest Interceptor
LS	Lift Station
MACP	Manhole Assessment Certification Program
NASSCO	National Association of Sewer Service Companies
NOAA	National Oceanographic and Atmospheric Administration
O&M	Operations and Maintenance
PACP	Pipe Assessment Certification Program
PDWF	Peak Dry Weather Flow
PS	Pump Station
PVC	Polyvinylchloride
PWWF	Peak Wet Weather Flows
RCP	Reinforced Concrete Pipe
RDII	Rainfall Dependent Inflow and Infiltration
ROW	Railroad Right-of-Way
RTK	Rainfall/Time/Recession Method
SACOG	Sacramento Area Council of Governments
SCS	Soil Conservation Services
SDR	Standard Dimension Ratio
SSO	Sanitary Sewer Overflow

SUH	Synthetic Unit Hydrograph
V&A	V&A Consulting Engineers
VCP	Vitrified Clay Pipe
West Yost	West Yost Associates
WWTP	Wastewater Treatment Plant

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CHAPTER 2
Existing Sewer System

The purpose of this chapter is to describe the City's existing sewer system. System information was obtained through the review of previous reports, maps, plans, operating records, general plans, GIS data, and other available data. The following sections of this chapter describe the components of the City's existing wastewater collection system:

- Existing Service Area,
- Population Served and Land Use Characteristics, and
- Existing Collection System Facilities.

2.1 EXISTING SERVICE AREA

The City of West Sacramento encompasses a total area of 14,722 acres, or approximately 23 square miles. Of this area, approximately 1.4 square miles are covered by water[1]. The City is situated in Yolo County, California. The City is bounded to the north and east by the City of Sacramento, from which it is separated by the Sacramento River, which is the County line. It is bounded to the west by the unincorporated agricultural land in the Yolo Bypass Wildlife Area and to the south by unincorporated agricultural land. The existing wastewater collection system service area includes all areas within the City's limits, with the exception of a small rural residential area in the southeastern portion of the City that is currently on septic systems, but that will ultimately be served by the collection system. The City and wastewater service area are shown on Figure 2-1.

The City's existing wastewater collection system is comprised of approximately 160 miles of active gravity sewer pipelines with sizes ranging from 4 to 30 inches in diameter, 22 miles of pressure pipelines, 9 pump stations, and 5 lift stations. The City's wastewater is treated at the Sacramento Regional Wastewater Treatment Plant (WWTP), located southeast of the City near Elk Grove, California. The Lower Northwest Interceptor (LNWI), a 120-inch diameter gravity pipeline at the point of the City's connection, conveys all flows from the City's collection system to the WWTP.

2.2 POPULATION SERVED AND LAND USE CHARACTERISTICS

This section describes current and buildout population projections, and associated land use as outlined in the City's General Plan.

2.2.1 Existing Population and Land Use

The City's population is 50,836, based on 2014 population estimates from the California Department of Finance. This population resides within 32,568 households throughout the City, for an average of 2.85 persons per household[2].

[1] Source: 2010 Census Gazetteer Files
[2] Source: 2010 Census

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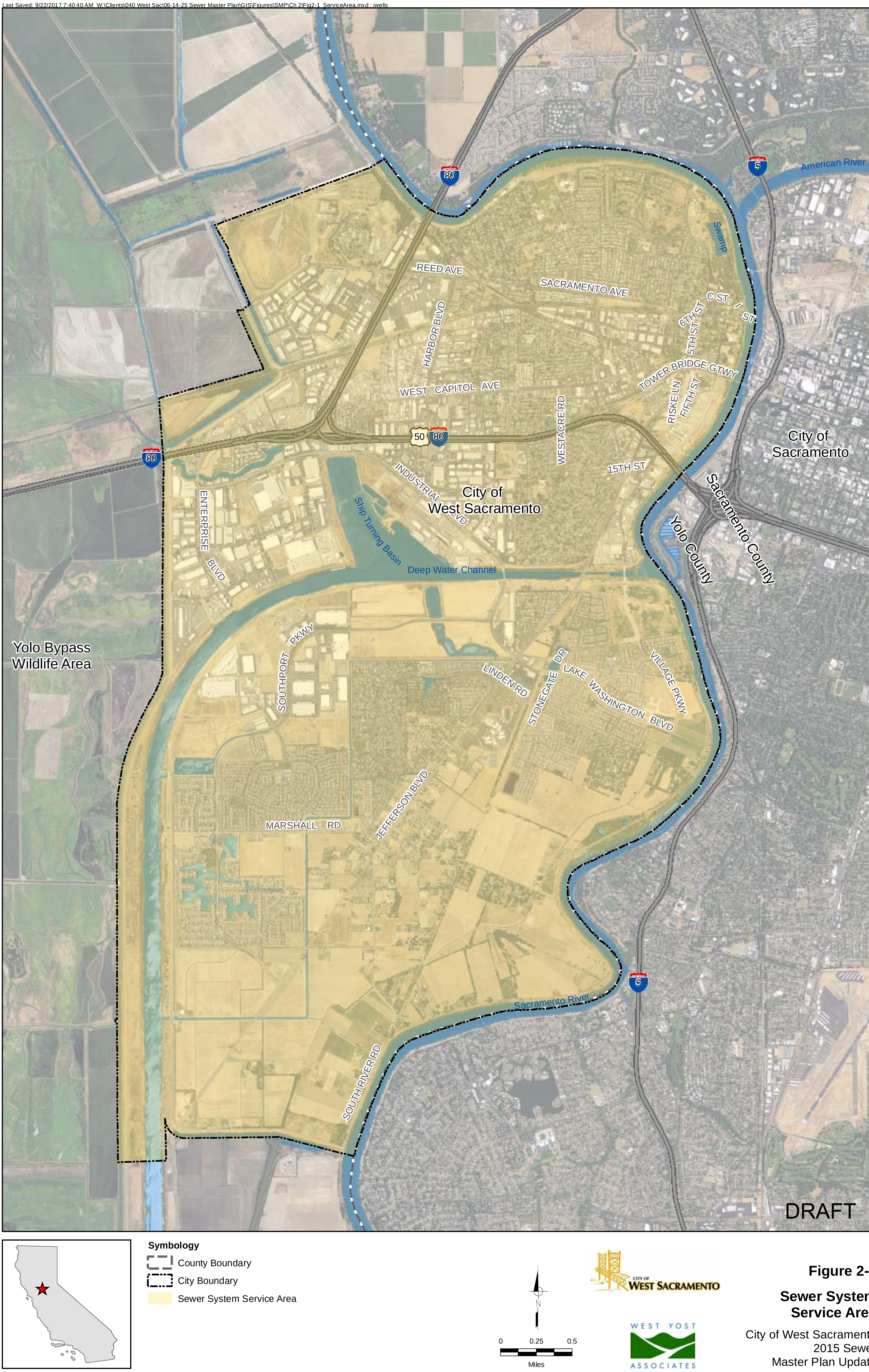

Figure 2-1

Sewer System Service Area

City of West Sacramento 2015 Sewer Master Plan Update

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Land use and zoning within the City is controlled by the *City of West Sacramento General Plan, 2015-2035* (General Plan). The General Plan was very recently updated to account for expected growth and development within the City. Land use and zoning are used to achieve the General Plan goal "to promote the development of a cohesive and aesthetically pleasing urban structure for West Sacramento." The City's service area includes a wide range of mixed land uses. Land use and zoning information presented in the Collection System Master Plan were derived from GIS resources provided by the City.

Figure 2-2 shows existing zoning designations for the City. This information served as the basis for initial flow calculation from the City's service area. Zoning categories are listed in Table 2-1.

Table 2-1. City of West Sacramento Zoning Summary

Zoning Code	Zoning Description	Area, acre	Area, %
C-1	Neighborhood Commercial	73	0.5%
C-2	Community Commercial	207	1.4%
C-3	General Commercial	88	0.6%
CH	Highway Service Commercial	63	0.4%
CW	Commercial-Water Related	20	0.1%
PO	Professional Office	34	0.2%
BP	Business Park	408	2.8%
CBD	Central Business District	117	0.8%
M-1	Light Industrial	504	3.4%
M-2	Heavy Industrial	1,124	7.6%
M-3	Waterfront Industrial	629	4.3%
ML	Limited Industrial	125	0.8%
RE	Rural Estate	447	3.0%
RRA	Rural Residential	632	4.3%
R1-A	Residential One Family - A	688	4.7%
R1-B	Residential One Family - B	1,326	9.0%
R-2	Residential One-Family or Multi-Family	937	6.4%
R-3	Multiple-Family Residential	407	2.8%
R-4	Apartment	3	0.0%
MU	Mixed Use	106	0.7%
WF	Waterfront	730	5.0%
A-1	Agricultural General	1,026	7.0%
PQP	Public Quasi-Public	763	5.2%
RP	Recreation and Parks	462	3.1%
POS	Public Open Space	833	5.7%
None	No Zoning Specified/Waterway	1,514	10.3%
ROW	Right of Way	1,456	9.9%
	Total Area	14,722	100.0%

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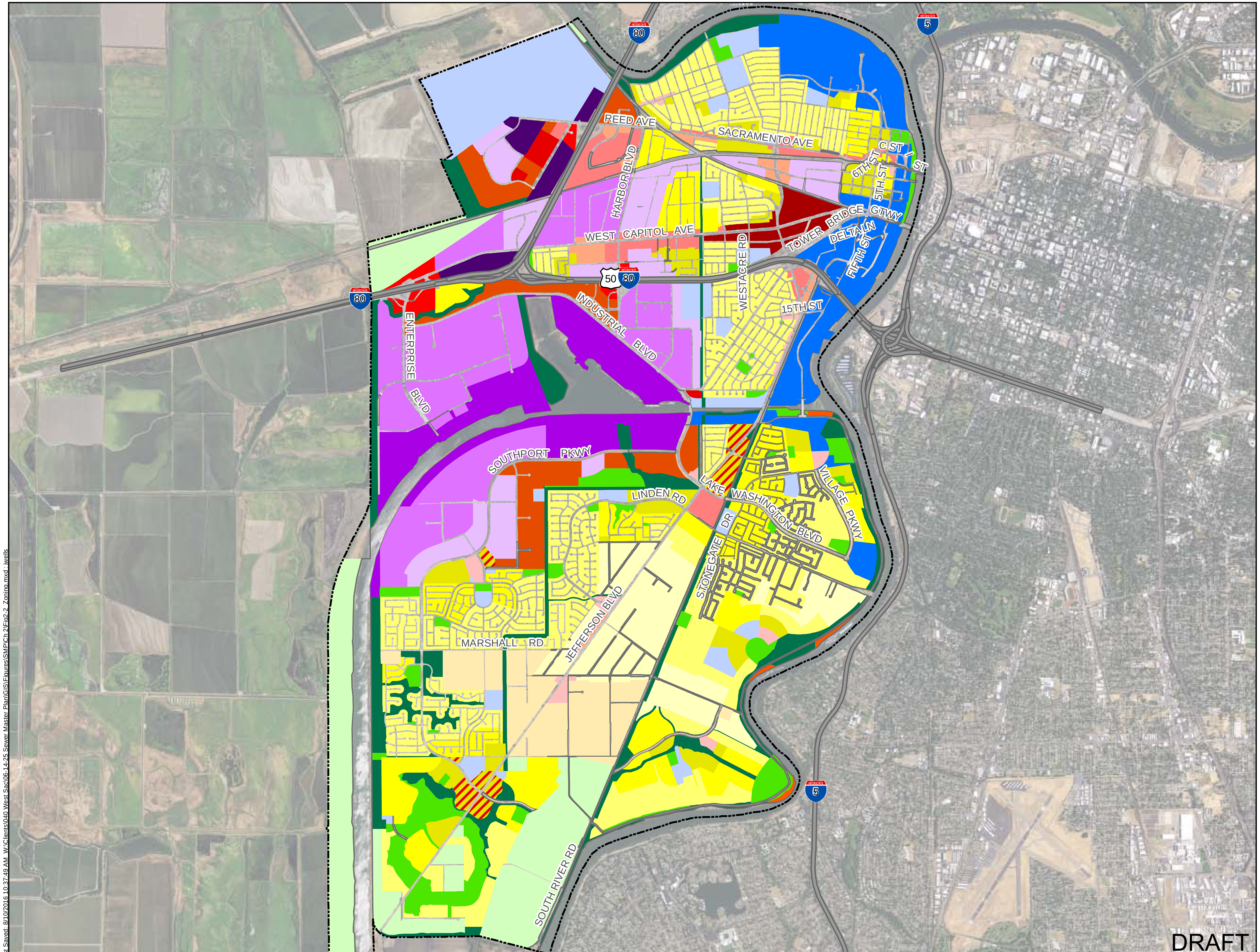

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Symbology

- City Boundary

Zoning Designation

- Neighborhood Commerical
- Community Commercial
- General Commercial
- Highway Service Commercial
- Commercial-Water Related
- Professional Office
- Business Park
- Central Business District
- Light Industrial
- Heavy Industrial
- Waterfront Industrial
- Limited Industrial
- Rural Estate
- Rural Residential
- Residential One Family
- Residential One-Family or Multi-Family
- Multiple Family Residential
- Apartment
- Mixed Use
- Waterfront
- Public Quasi-Public
- Agricultural General
- Recreation and Parks
- Public Open Space
- No Zoning Specified/ Waterway

Source: City Zoning GIS Shapefile

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 2-2

City of West Sacramento Zoning

City of West Sacramento
2015 Sewer
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2.2.2 Buildout Population and Land Use

The *City of West Sacramento Urban Water Management Plan* (2015 UWMP) provides population projections for the City through 2035. These projections are based upon Sacramento Area Council of Government's (SACOG) 2007 Projections. As shown in Table 2-2, population within the City is projected to increase from 50,836 to 87,402 by 2035, an increase of nearly 72 percent.

Table 2-2. Current and Projected Population in the City of West Sacramento[a]

2015	2020	2025	2030	2035
50,836	59,353	66,061	73,529	87,402

[a] Source: City of West Sacramento 2015 Urban Water Management Plan.

Whereas zoning designation describes the current use of a parcel of land, the land use designation describes the ultimate use and extent to which a parcel may be developed. Built-out land use is described by the General Plan Land Use designation, which the City tracks in a GIS database. General Plan Land Use is summarized by acreage in Table 2-3, and is shown on Figure 2-3.

Table 2-3. City of West Sacramento Land Use Summary

Land Use Code	Land Use Description	Area, acre	Area, %
NC	Neighborhood Commercial	73	0.5%
CC	Community Commercial	207	1.4%
GC	General Commercial	88	0.6%
HSC	Highway Service Commercial	63	0.4%
WRC	Water Related Commercial	20	0.1%
O	Office	34	0.2%
BP	Business Park	408	2.8%
CBD	Central Business District	117	0.8%
MCI	Mixed Commercial / Industrial	125	0.8%
LI	Light Industrial	504	3.4%
HI	Heavy Industrial	1,124	7.6%
WRI	Water Related Industrial	629	4.3%
RE	Rural Estates	447	3.0%
RR	Rural Residential	632	4.3%
LR	Low Density Residential	2,013	13.7%
MR	Medium Density Residential	937	6.4%
HR	High Density Residential	407	2.8%
HRR	High Rise Residential	3	0.0%
RMU	River Mixed Use	836	5.7%
PQP	Public / Quasi Public	763	5.2%
AG	Agriculture	1,026	7.0%
RP	Recreation and Parks	462	3.1%
OS	Open Space	833	5.7%
None	No Land Use Specified/Waterway	1,515	10.3%
ROW	Right of Way	1,456	9.9%
	Total Area	14,722	100.0%

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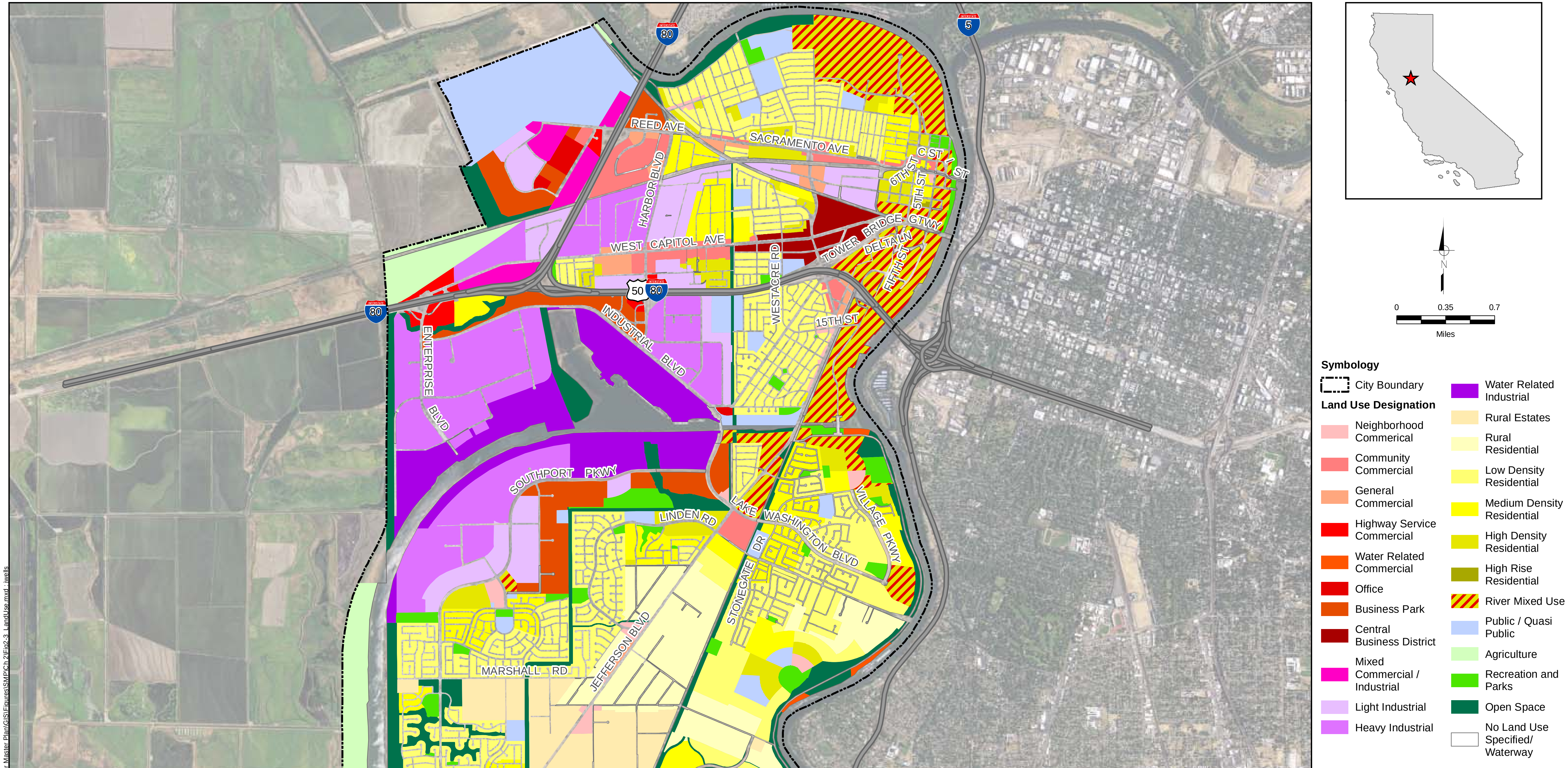

Source: City Land Use GIS Shapefile

Figure 2-3

City of West Sacramento Land Use

City of West Sacramento
2015 Sewer
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2.3 EXISTING COLLECTION SYSTEM FACILITIES

This section describes the facilities that are owned, operated, and maintained by the City. Existing facility information was derived from the City's GIS database, as-built plans, and discussion with City staff. Figure 2-4 shows the collection system facilities, as documented in GIS and updated through review of record drawings. These GIS layers were used to develop the collection system network in the collection system hydraulic model, as described in subsequent chapters.

2.3.1 Gravity Main Characteristics

The City's sewer system is comprised of approximately 160 miles of gravity mains ranging from 4-inch diameter to 30-inch diameter. The gravity main diameters found in the collection system are summarized in Table 2-4, and are presented on Figure 2-5. As shown in the table, 76 percent of the collection system gravity mains are 8-inch diameter or smaller. Because such a high percentage of the collection system is composed of small gravity mains, small gravity mains are serving as trunk sewers for conveyance in some cases. As will be discussed further in subsequent chapters, these small diameter trunk sewers create hydraulic bottlenecks that impact the capacity available for future development in the City.

Table 2-4. Gravity Mains by Diameter

Gravity Main Diameter	Length, linear feet	Length, miles	Percent of System, %
8-inch and smaller	645,511	122.3	76%
10-inch	58,417	11.1	7%
12-inch to 18-inch	116,388	22.0	14%
21-inch to 30-inch	25,820	4.9	3%
Total	846,136	160.3	100%

The gravity mains in the collection system are composed of five different materials. Approximately 34 percent of the system by length is composed of vitrified clay pipe (VCP); 64 percent is polyvinylchloride (PVC); and the remaining 2 percent is either reinforced concrete pipe (RCP), ductile iron pipe (DIP), acrylonitrile butadiene styrene (ABS), or of unknown composition. The gravity main material is summarized in Table 2-5. The location of these materials in the collection system can be seen on Figure 2-6.

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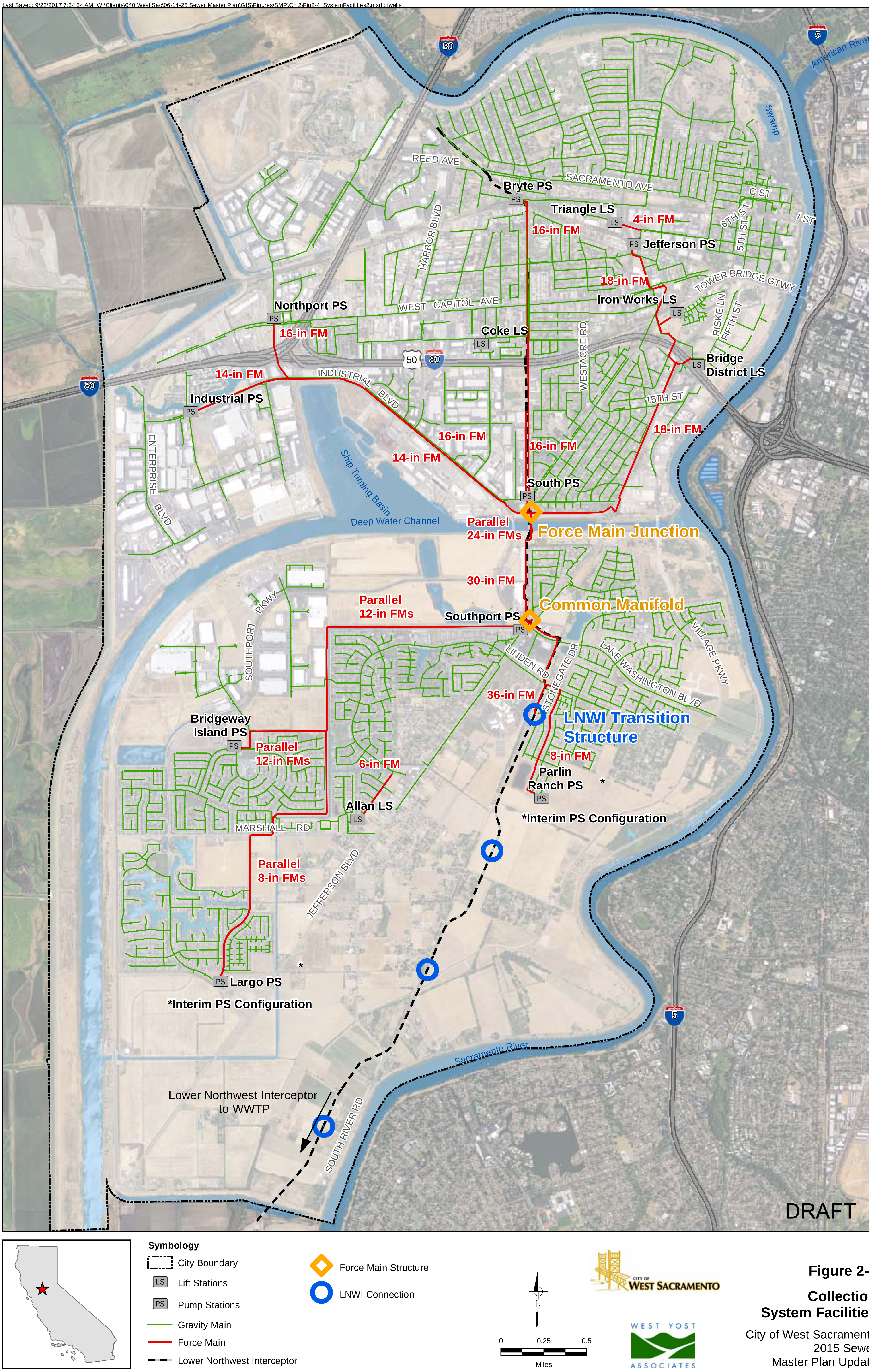

Figure 2-4

Collection System Facilities

City of West Sacramento
2015 Sewer
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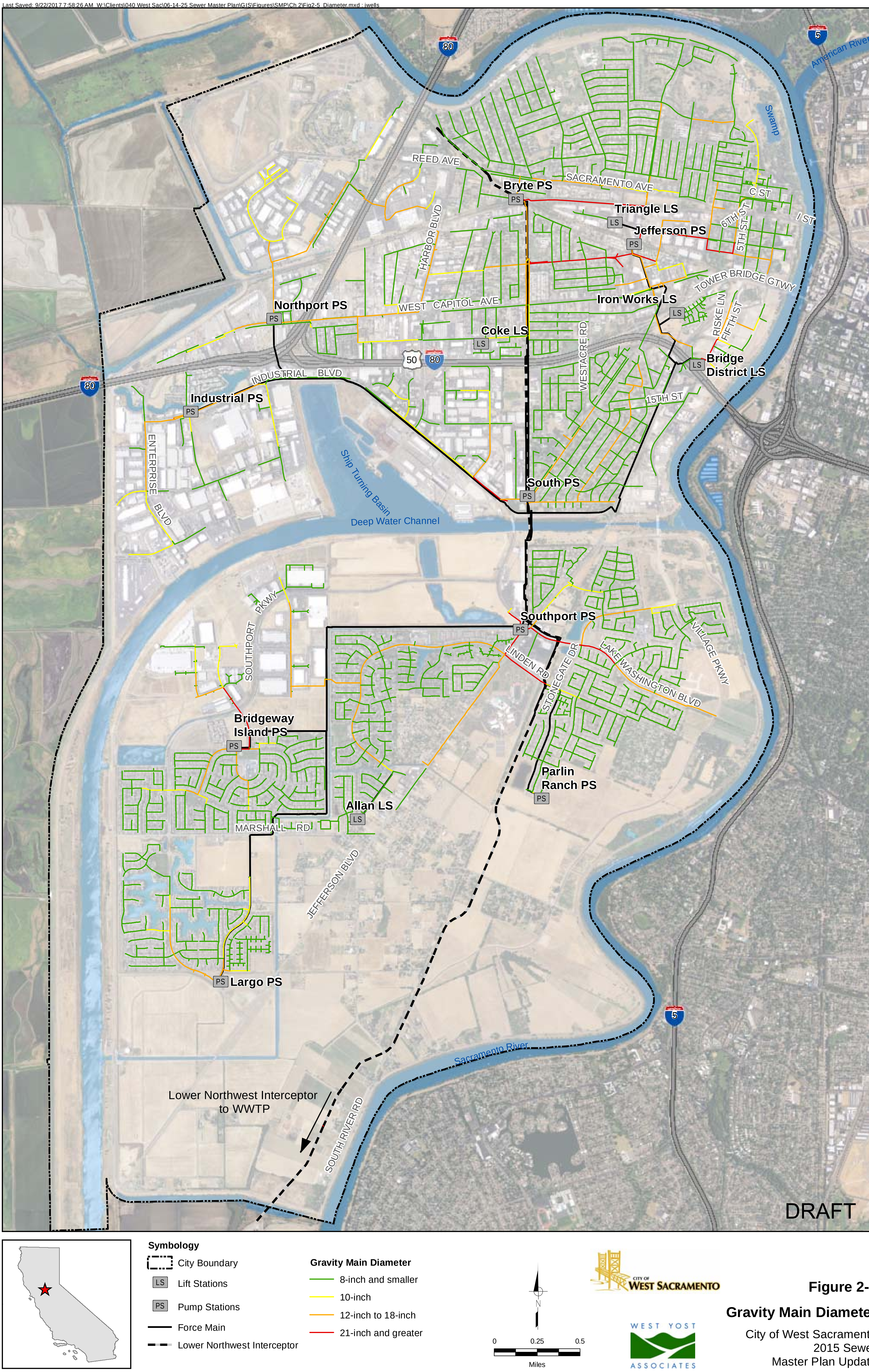

Figure 2-5

Gravity Main Diameter

City of West Sacramento
2015 Sewer
Master Plan Update

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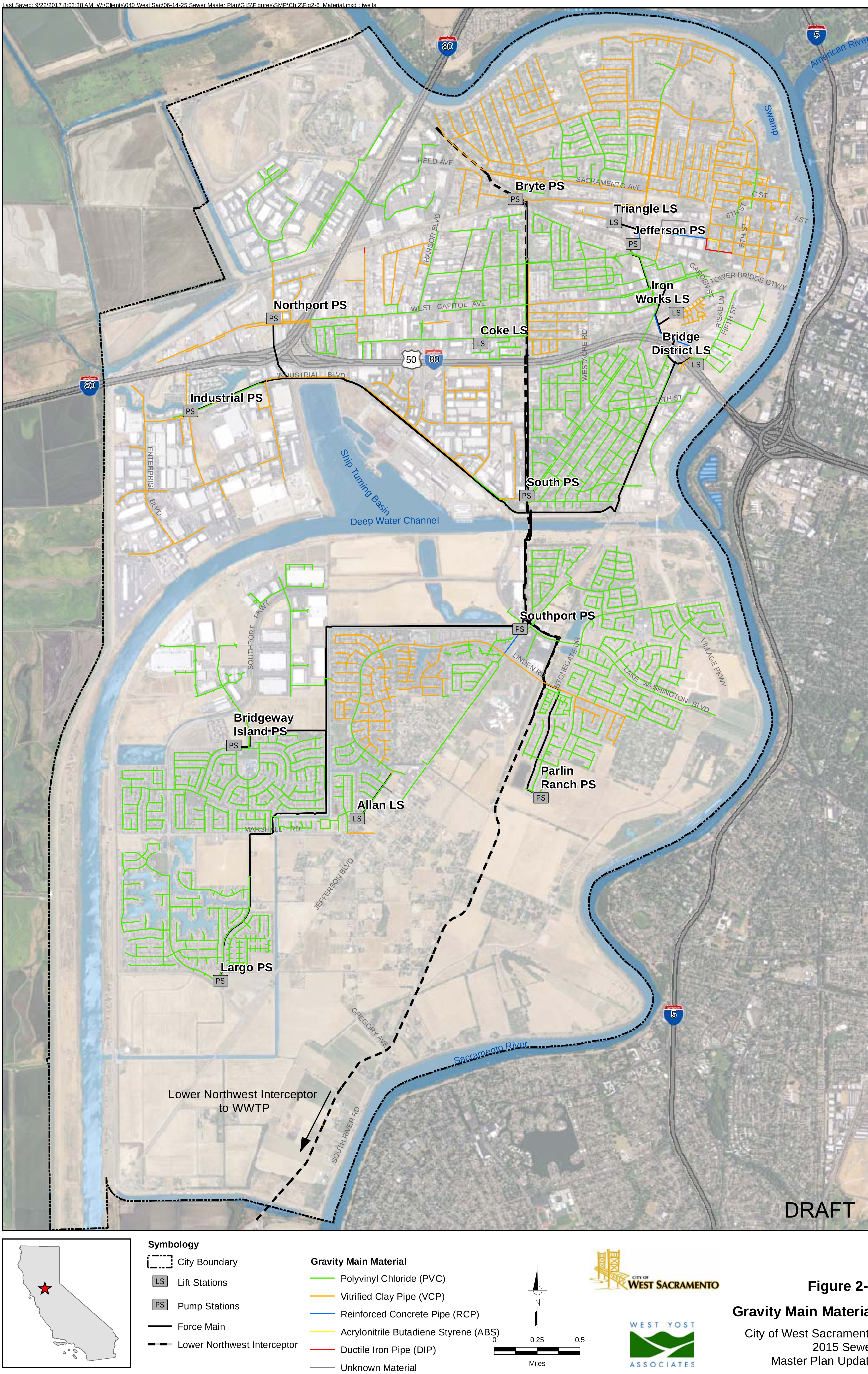

Figure 2-6

Gravity Main Material

City of West Sacramento
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Table 2-5. Gravity Mains by Material			
Pipe Material	Length, linear feet	Length, miles	Percent of System, %
PVC	545,023	103.2	64.4%
VCP	290,038	54.9	34.3%
RCP	5,252	0.99	0.62%
ABS	256	0.048	0.03%
DIP	1,301	0.25	0.15%
Unknown	4,266	0.81	0.50%
Total	846,136	160.2	100%

The installation date of the gravity mains in the City's collection system are known and provided in the City's GIS for the portion of the collection system south of the Deep Water Channel. This portion of the collection system is newer, with all installations after 1970, and the majority after 1990. By contrast, for the older portion of the collection system north of the Deep Water Channel, few installation dates were known and recorded in the GIS. Because an estimate of installation year and gravity main age is important in estimating the condition and risk of failure for gravity mains, installation years were estimated based upon real estate construction ages as provide by the real estate website www.Zillow.com. Where possible, installation dates were confirmed or updated based upon available record drawings for new construction or rehabilitation/repair. The installation years for gravity mains in the collection system (including the estimated years north of the Deep Water Channel described above) are summarized in Table 2-6, and are presented on Figure 2-7.

Table 2-6. Gravity Mains by Installation Year			
Installation Year[a]	Length, linear feet	Length, miles	Percent of System, %
pre-1950s	62,150	11.8	7%
1950s	217,650	41.2	26%
1960s	55,821	10.6	7%
1970s	109,580	20.8	13%
1980s	43,344	8.2	5%
1990s	54,929	10.4	6%
2000s and later	276,145	52.3	33%
Unknown	26,517	5.0	3%
Total	846,136	160.3	100%

[a] Installation years for gravity mains north of the Deep Water Channel were estimated from Zillow home construction dates (www.Zillow.com). Installation years for gravity mains south of the Deep Water Channel were assigned an age from the "Install Date" field in the SSMains shapefile from the City's GIS.

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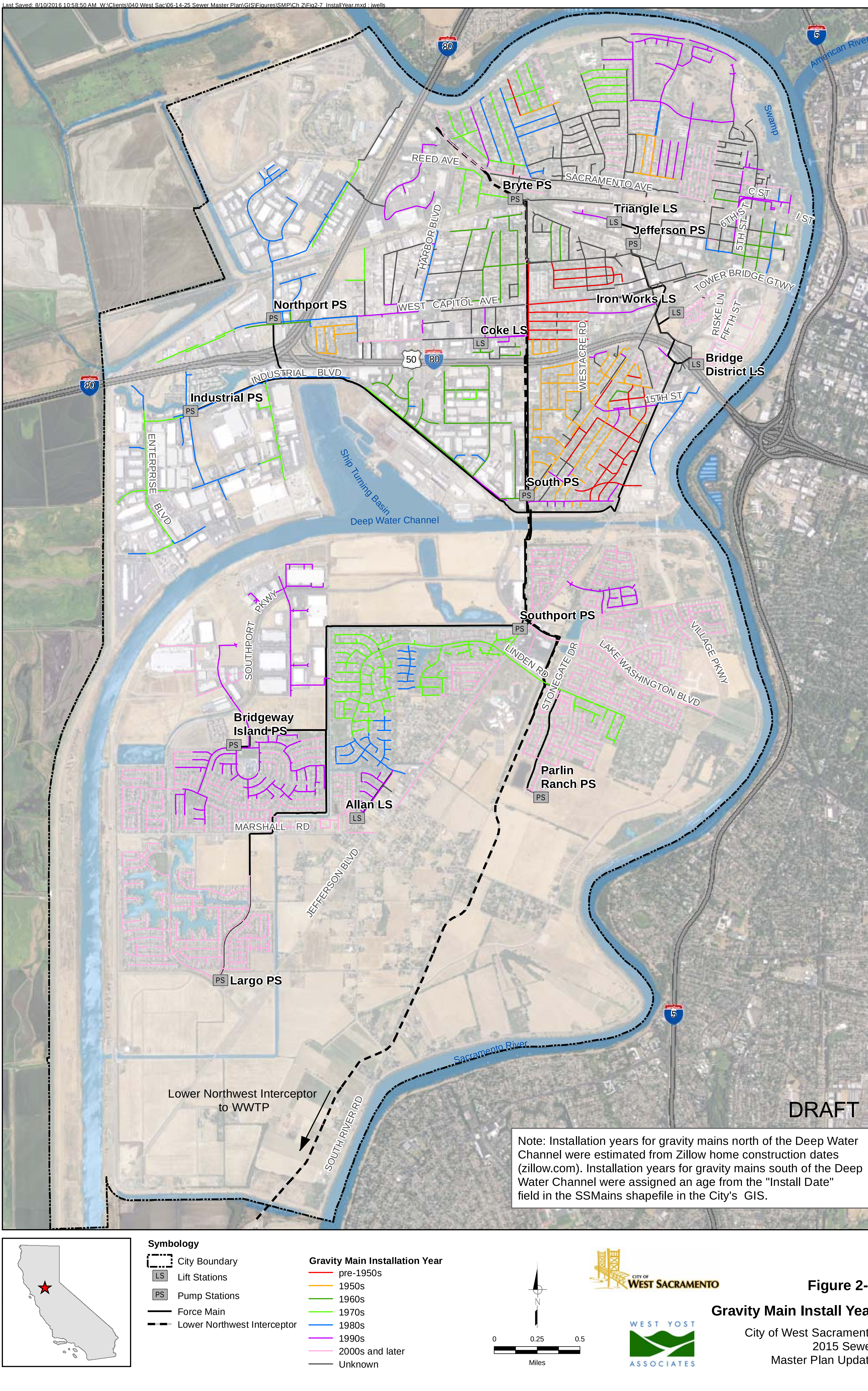

Figure 2-7

Gravity Main Install Year

City of West Sacramento
2015 Sewer
Master Plan Update

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2.3.2 Pump Stations, Lift Stations, and Associated Force Main Characteristics

The City operates nine pump stations, five lift stations, and the associated force mains. The City owns each of these facilities with the exception of Iron Works Lift Station, which is privately owned but City operated and maintained. Table 2-7 lists the City's pump stations and lift stations and summarizes operating characteristics of each. The operating characteristics were derived from City provided data, as well as from inspections of some facilities. Lift stations are considered to be local facilities that serve a local area of the collection system. Pump stations are considered to be larger regional facilities that convey flow to the LNWI connection.

There are five pump stations north of the Deep Water Channel: Bryte, Jefferson, Northport, Industrial, and South. The force mains from these five pump stations tie into a transition structure north of the Deep Water Channel. Two 24-inch force mains run under the Channel, and then connect to a common manifold near Southport Pump Station. In addition to the five pump stations, there are four lift stations north of the deep water channel: Coke, Triangle, Iron Works, and Bridge District. Coke Lift Station discharges to the gravity system which connects to the Bryte Pump Station. Triangle Lift Station discharges to the gravity side of Jefferson Pump Station. Iron Works and Bridge District Lift Stations discharge into the Jefferson Pump Station force main.

Table 2-7. Wastewater Collection System Pump Station Summary

Pump/Lift Station Name	No. of Pumps	Pump Type	Firm Capacity[a], gpm	Design Total Dynamic Head, feet	Power, hp
Allan Lift Station[b]	2	Smith & Loveless	300	65	15
Bridge District Lift Station[b]	3	Flygt	1,213	130	70
Bridgeway Island Pump Station[b]	3	Smith & Loveless	3,250	140	125
Bryte Pump Station	3	Smith & Loveless	3,400[c]	175	200
Coke Lift Station[b]	2	Smith & Loveless	200	20	3
Industrial Pump Station	3	Smith & Loveless	1,526	118	50
Iron Works Lift Station[b]	2	Sulzer	510	85	10
Jefferson Pump Station	3	Smith & Loveless	3,900	122	100
Largo Pump Station[b]	2	Smith & Loveless	450	200	60
Northport Pump Station[b]	3	Smith & Loveless	4,000	102	75
Parlin Ranch Pump Station[b]	2	Flygt	570	50	12
South Pump Station	3	Smith & Loveless	764	58	15
Southport Pump Station	3	Smith & Loveless	6,459	69	50
Triangle Lift Station[b]	2	Chicago	137	17	3

(a) Firm Capacity is defined as the capacity of the pump/lift station with the largest pump out of service.
(b) Data at these pump/lift stations was not confirmed in site visits. It was pulled from pump station data provided to by the City.
(c) This value assumes all three pumps equally sized. Only one of three pumps has a nameplate.

The four pump stations south of the Deep Water Channel include: Largo, Bridgeway Island, Parlin Ranch, and Southport. Largo Pump Station discharges into the two 8-inch force mains from Bridgeway Island Pump Station, which discharge into a common manifold near Southport Pump Station. Parlin Ranch Pump Station currently discharges to the gravity system which connects to Southport Pump Station; the Parlin Ranch Pump Station will eventually become a lift station that discharges to the gravity section of the LNWI through a future neighborhood system. The Southport Pump Station force main discharges to the common manifold. Allan Lift Station is the only lift station south of the Deep Water Channel. This lift station discharges to the gravity system which connects to the Southport Pump Station.

The Parlin Ranch Pump Station and Largo Pump Station are both in interim conditions with respect to how their force mains are aligned. In the future, the force mains of these two pump stations will be re-routed to a final configuration. The final configuration is discussed more fully in Chapter 8.

All wastewater flows are directed to the common manifold, from which one 36-inch force main runs parallel to the LNWI until it connects to the LNWI's Transition Structure, the interface between the LNWI force mains and the 120-inch gravity sewer south of the intersection of Wigeon Street and Muscovy Road. In total, the collection system contains 21.7 miles of force main primarily composed of DIP and PVC. A schematic of the pump/lift station connections and flow directions can be seen on Figure 2-8.

Last Revised: 08-13-15; w\c\040\06-14-25\wp\mp\081415 ce Figure 2-8

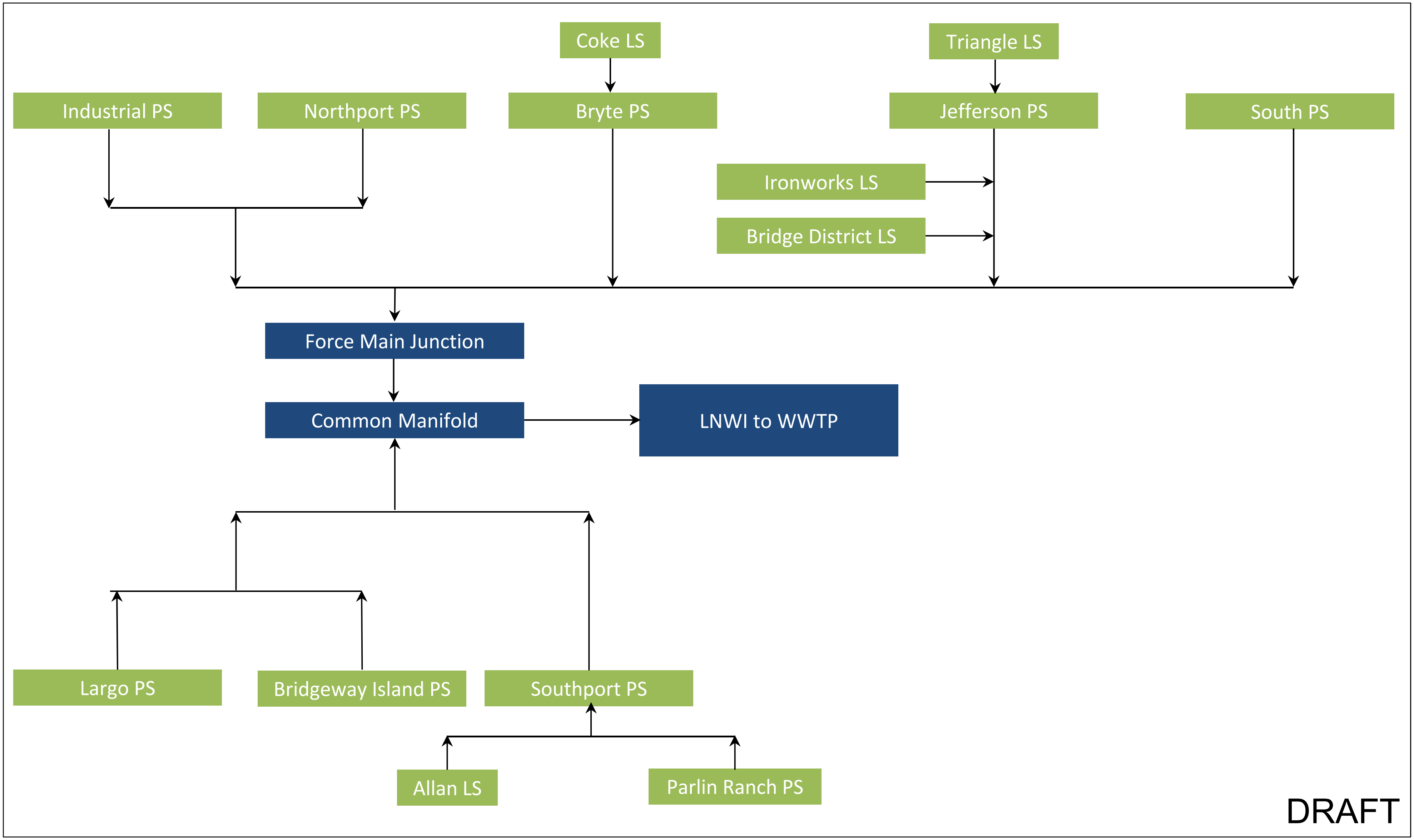

Figure 2-8

Pump/Lift Station Flow Schematic

City of West Sacramento
2015 Sewer Master Plan Update

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2.4 REFERENCES

State of California, Department of Finance. *E-1 Population Estimates for Cities, Counties and the State with Annual Percent Change — January 1, 2013 and 2014*. May 2014.

City of West Sacramento 2015 Urban Water Management Plan (2015 UWMP). Dated October 2016.

City of West Sacramento General Plan.

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CHAPTER 3
Design and Performance Criteria

The purpose of this chapter is to present the design and performance criteria that will be used to evaluate existing capacity in and to size replacement facilities for the City's wastewater collection system. Where available, existing City design and performance criteria will be used; in other cases, industry standard criteria have been added to those already in place. Planning criteria address items such as collection system capacity, gravity sewer slopes, and maximum depth of flow. The elements of this chapter include:

- Existing Sewer System Facility Capacity Criteria,
- New or Replacement Gravity Main Design Criteria,
- New or Replacement Pump Station and Lift Station Design and Operating Criteria,
- New or Replacement Force Main Design and Operating Criteria, and
- Design Storm Criteria.

3.1 EXISTING SEWER SYSTEM FACILITY CAPACITY CRITERIA

Gravity mains must be sized to carry PWWF, and therefore are evaluated for capacity under Design PWWF conditions. The existing City Design Standards, Section 5, contain design criteria that are suitable for smaller diameter collection mains, but which are overly conservative for larger diameter conveyance mains. Although small diameter collection main capacity is not evaluated as part of the 2015 Sewer Master Plan Update, which uses a skeletonized hydraulic model to evaluate the backbone conveyance system, it is important to maintain these criteria so that the criteria can be used to evaluate future development. In order to maintain the City's collection main criteria while still utilizing more appropriate conveyance main criteria, the following design criteria were established for gravity mains in this master plan:

- Collection mains are defined as all gravity mains 6 inches in diameter and smaller, and those 8-inch gravity mains that were not included in the hydraulic model as part of the conveyance system. Collection mains shall be identified as hydraulically deficient when the Design PWWF depth to Diameter (d/D) ratio exceeds 0.70.
- Conveyance mains are defined as all gravity mains 10 inches in diameter and larger, and those 8-inch mains that were included in the hydraulic model. A conveyance main shall be considered hydraulically deficient if the Design PWWF through the main results in a Hydraulic Grade Line (HGL) within three feet of ground surface above the main. The HGL may exceed the crown of the pipe, and manholes may be surcharged, without indicating a deficient conveyance main as long as the HGL remains more than three feet from the surface. Conveyance mains are identified in Chapter 4.
- Pump Stations and Lift Stations will be considered to require capacity improvements if the associated firm capacity (i.e., capacity with the largest pump out of service) is not sufficient to convey the Design PWWF.
- Force mains will be considered to require capacity improvements if maximum velocity exceeds 8 feet per second (fps) during Design PWWF.

3.2 NEW OR REPLACEMENT GRAVITY MAIN DESIGN CRITERIA

Gravity main design criteria, including capacity and design standards, are described below.

3.2.1 Gravity Main Capacity Standards

The Manning Formula [$Q = A\ (1.49/n)\ R^{2/3}\ S^{1/2}$] shall be used to determine gravity main capacity. The roughness coefficient, or Manning's "n" value, used to calculate pipe capacity, shall be equal to 0.013. A Manning's "n" value of 0.013 is somewhat conservative if PVC pipe is used. An "n" value of 0.011 may be more appropriate for PVC. An "n" value of 0.013 is a commonly used value that assumes a buildup of a slime layer in any pipe material after many years of service and is consistent with City standards. By using this value, pipe sizes selected are not restricted to one material type. Manning's "n" values, which are less than 0.013 shall require City Engineer approval and shall only be allowed if accounting for minor losses.

New (parallel relief) or replacement pipelines shall be designed to meet the following criteria. These criteria do not necessarily apply to the rehabilitation and replacement of isolated sections of pipelines within existing alignments:

- Under Peak Dry Weather Flow (PDWF) conditions, velocity at full pipe or half-full pipe conditions shall remain above 2 fps to facilitate self-cleaning;
- Where design velocities for gravity mains exceed 10 fps, polyethylene lined ductile iron pipe conforming to Section 14 of the City' Standard Construction Specifications shall be used. The ductile iron pipe shall be wrapped with an 8-mil polyethylene encasement;
- All gravity mains shall be sized to carry the Design PWWF at a maximum of 70 percent of pipe capacity; and
- The minimum size of any new gravity main shall be 8-inches in diameter to facilitate maintenance.

3.2.2 Gravity Main Design Standards

Gravity main design standards including slope, material, and cover requirements are described below.

3.2.2.1 Minimum Slope

The minimum practical slope for gravity main construction is considered to be 0.0008 feet/feet (ft/ft). Recommended minimum slopes for new gravity construction are summarized in Table 3-1. Construction at the recommended minimum slope allows for self-cleaning velocities of 2 fps at full or half-full conditions.

Table 3-1. Recommended Minimum Gravity Main Slopes

Gravity Main Diameter	Minimum Slope, ft/ft
6-inch	0.0050
8-inch	0.0035
10-inch	0.0025
12-inch	0.0020
15-inch	0.0015
18-inch	0.0012
21-inch	0.0010
24-inch	0.0008
27-inch	0.0008
30-inch	0.0008
33-inch	0.0008
36-inch	0.0008
39-inch	0.0008
42-inch	0.0008

3.2.2.2 Gravity Main Materials

Gravity main material options generally include PVC and RCP. The City no longer allows use of VCP. RCP sewers are subject to deterioration from hydrogen sulfide corrosion if not properly protected. PVC pipe should meet the requirements of ASTM D 3034 for pipe and fittings up to 15 inches in diameter. The recommended standard dimension ratio (SDR) of the PVC pipe up to 15-inch-diameter is either 35 or 26, depending on the depth of bury. SDR 35 is recommended for PVC pipe with up to a 12-foot depth of cover; SDR 26 is recommended for pipe with greater than a 12-foot depth of cover. The SDR is a measure of the thickness of the pipe compared to the pipe diameter and indicates the ability of the pipe to resist forces from static and live loads. Reinforced concrete pipe, ASTM C-76, with a PVC lining for sulfide corrosion resistance is recommended for pipes larger than 24 inches in diameter. Large-diameter PVC pipe (AWWA C-900) could also be considered in such cases. Allowable gravity main materials will be dictated by the most current version of the City Design Standards.

3.2.2.3 Gravity Main Cover and Clearances

The following cover and clearances standards are summarized from the City's current Standard Specifications. Minimum gravity main cover and clearance shall be maintained in the design of sanitary sewers. If certain conditions exist which make it impractical to meet the minimum cover and clearance requirements, the conditions and locations shall be specifically noted above the sewer profile on the plans. Each location not meeting the minimum cover and clearance requirements will require special approval. Any planned condition being specially approved with less than minimum cover will require special pipe, bedding and/or backfill as approved by the City Engineer. The minimum requirements are as follows:

- Gravity mains shall have a minimum depth of 4 feet as measured from the top of the pipe to the finished grade.
- Sewer laterals shall have a minimum depth of 3 feet from the top of the pipe to finished grade.
- Gravity mains shall be located 6 feet south or east of and parallel with the street centerline unless otherwise approved by the City Engineer.
- Alignment of sanitary sewer mains shall be straight between manholes. Whenever it is essential that a curved alignment be used, a minimum radius of 200 feet shall be required, but shall be greater whenever possible. The radius and delta of all curves shall be indicated on the plans adjacent to the curve.
- The deflection in the joint between any two successive pipe sections shall not exceed eighty (80) percent of the maximum deflection as recommended in writing by the pipe manufacturer.
- Minimum horizontal separation between parallel sewer and water mains shall be 10 feet.

3.3 NEW OR REPLACEMENT PUMP STATION AND LIFT STATION DESIGN AND OPERATING CRITERIA

Design and operational criteria for new and replacement pump stations and lift stations are described below.

3.3.1 Pump Station and Lift Station Design Criteria

Pump stations and lift stations will be designed such that the associated firm capacity (i.e., capacity with the largest pump out of service) is sufficient to convey the Design PWWF as predicted by the hydraulic model of the collection system.

3.3.2 Pump Station and Lift Station Operational Criteria

Pump station and lift station wet wells shall be designed with sufficient volume and depth such that the pump station or lift station can be operated using the wet well, and not the tributary collection system, as storage during both PDWF and PWWF conditions. Pumps shall be selected so that the pumps can operate within the pump manufacturer's design curve during both PDWF and PWWF conditions.

3.4 NEW OR REPLACEMENT FORCE MAIN DESIGN AND OPERATING CRITERIA

Force mains are designed such that the maximum velocity does not exceed 8 fps during Design PWWF conditions. Force main material options include PVC, DIP, and high-density polyethylene (HDPE). HPDE should be considered primarily for directional drilling installations. Similar to gravity mains as described above, allowable materials and pipe ratings will be dictated by the most current version of the City Design Standards.

3.5 DESIGN STORM CRITERIA

Design PWWF for capacity analysis of the collection system is generated using a design storm. Design storms are synthetic rainfall events used to evaluate collection system capacity under wet weather flow conditions. A design storm has a specific recurrence interval and rainfall duration.

There are no regulatory requirements concerning the recurrence interval and rainfall duration that the City must use to evaluate the wastewater collection system. A design storm with a 10-year recurrence interval and 24-hour duration (10-year, 24-hour storm) is commonly used to evaluate wastewater collection systems in Northern California, and was used for the evaluation in this Wastewater Collection System Master Plan. A 10-year, 24-hour storm produces 3.40 inches of rainfall in 24 hours, as provided through the National Oceanographic and Atmospheric Administration (NOAA) rainfall atlas. The rainfall during the design storm was distributed over the 24-hour period using the Soil Conservation Services (SCS) Type 1 distribution.

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The computer-based hydraulic model of the City's sewer system, developed using Innovyze® InfoWorks™ CS software, serves as a tool for assessing the flows and capacities of the City's trunk sewers, and for identifying solutions to sewer capacity issues. The hydraulic model is also a tool for performing "what if" scenarios to assess the impacts of future developments, land use changes, and system configuration changes.

The hydraulic model includes the City's main trunk sewers (typically 10-inch diameter and larger) and associated facilities, and is a skeletonized representation of the City's collection system in its configuration and operation. Hydraulic model development usually focuses on trunk sewers only, as the local collector sewers are typically sized to facilitate maintenance, and are usually oversized for flow conveyance. The City's model also includes some smaller diameter sewers as needed to provide system connectivity or to represent available relief sewers (e.g., parallel sewers or basin to basin connections).

This chapter presents a summary of hydraulic model development and calibration. The major sections of this chapter include:

- Model Development,
- Data Validation, and
- Load Allocation.

4.1 MODEL DEVELOPMENT

The hydraulic model transforms information about the physical and operational characteristics of the sewer system into a mathematical model. The model solves a series of differential equations for continuity and momentum (Saint-Venant equations) to simulate various flow conditions for specified sets of flow loads. The modeling results provide information on flows, flow depth, velocity, surcharging, and backwater conditions that are used to analyze system performance and identify system deficiencies. The model is also used to verify the adequacy of recommended or proposed system improvements.

4.1.1 Hydraulic Model Software Selection

There are more than 10 commercially available software packages available for collection system hydraulic modeling. These hydraulic modeling software packages can be evaluated by objective criteria such as price and capability, and by subjective criteria such as ease-of-use and effectiveness of support. For the 2015 Sewer Master Plan Update, discussion with City staff indicated that the following criteria are most important in software selection for the hydraulic model:

- **Fully Dynamic Hydraulic Solution**: The 2015 Sewer Master Plan Update modeling requires fully dynamic hydraulic solutions for two reasons. First, the wet weather hydraulic capacity analysis requires that the Rainfall Dependent Inflow and Infiltration (RDII) from a design storm be routed through the collection system. Such routing includes temporary storage in the collection system due to surcharging and requires a fully dynamic solution. Second, the capacity analysis requires an evaluation of the impact of the Regional conveyance boundary conditions on the

City's collection system, particularly the pump stations. Such an evaluation requires a fully dynamic solution as well as boundary condition data from the Regional conveyance system.

- **Ability to Export Data and Results in GIS Format**: The Collection System Master Plan does not require that the modeling software be fully integrated into GIS software. However, it does require that data and results be exported into GIS for mapping purposes.
- **Convenience of Future Model Updates and Analysis**: Discussion with City staff indicates that the City does not wish to purchase or train on the collection system modeling software used for the Collection System Master Plan. For this reason, features such as cost and ratings such as ease-of-use and ease-of-training will not be significant factors in the requirements. Rather, the ability of the City to procure consultant support for the hydraulic model in the future is of more importance in this selection.

Based upon the criteria listed above and West Yost's experience with all of the commercially available software packages for hydraulic modeling of collection systems, InfoWorks CS was chosen as the software to be used in the Collection System Master Plan. A detailed analysis of the commercially available software packages and the process used for the selection of the software for the Collection System Master Plan can be found in *Appendix A: Technical Memorandum for Hydraulic Model Software Selection*.

4.1.2 Hydraulic Model Elements

The hydraulic model comprises a skeletonized network of nodes (e.g., manholes) and conduits (e.g., pipelines). Several types of nodes and links are used for defining the physical entities within a collection system. The following descriptions provide additional information on elements used in the development of the hydraulic model:

- <u>Node</u>: Nodes represent manholes, split manholes, diversion structures (with no other physical component such as a weir), storage facilities, and outfalls in a collection system. Storage facilities include modeled lift station wet wells. All flows loaded into the model are attached to a node structure. The data required for node structures include elevation data (pipe invert and manhole rim) and manhole diameter.
- <u>Conduit</u>: Conduits represent facilities that convey wastewater from one point in the system to another. Conduits include gravity pipes, force mains, pumps, and weirs. Several different types of pumps and weir structures are available as standard elements. The physical data for gravity pipes and force mains include invert elevation, size, length, and friction factor. The physical data for pumps include type of pump, elevation, head-discharge relationship, and operational parameters such as on/off elevations and sequencing.

- Sewersheds or subcatchments: Subcatchments represent an area that flows tributary to an individual node in the model. Subcatchments usually represent a particular subdivision or grouping of parcels and collection of small diameter sewers (typically 6-inch to 8-inch diameter) that flow into one location along a major trunk sewer. The subcatchment layer serves several purposes, including defining land use, diurnal curves, and dry and wet weather flow inputs. The data required for subcatchments are node connection, land use, flow factors, total and contributing area, diurnal curve profile, rainfall profile, inflow and infiltration parameters, and groundwater parameters.

4.1.3 Model Infrastructure Development

The structural components of the hydraulic model network, (i.e., nodes and conduits) were developed by West Yost from GIS and as-built information provided by the City. The City's GIS is divided into Geospatial and Schematic portions. The Geospatial portion of the GIS primarily encompasses the collection system south of the Deep Water Channel. The Geospatial GIS contains detailed hydraulic attribute (pipeline diameter and invert elevations) and location information for the collection system assets. Because of this detailed and comprehensive data, the Geospatial GIS was imported directly into the hydraulic model. Minor data gaps and inconsistencies were reviewed and corrected using as-builts provided by the City.

The Schematic GIS primarily encompasses the collection system assets north of the Deep Water Channel. The Schematic GIS contains general asset location information, and a limited amount of hydraulic attribute information. The general asset location information includes areas with incomplete manhole data, in which long reaches of pipeline are represented by a single gravity main several thousand feet in length, rather than the actual gravity mains and manholes found in the field. The Schematic GIS was substantially modified to develop the hydraulic model north of the Deep Water Channel. Manholes were added to the system where shown by as-built drawings or aerial photo analysis. Hydraulic attribute data was taken from as-built drawings, where available. Where as-built drawings were not available, invert elevations were assumed using minimum slope criteria. Finally, locations of some collection system assets were adjusted slightly so that the hydraulic model developed from the Schematic GIS. The hydraulic model contains approximately 30 miles of gravity mains and 20 miles of force mains developed from the Geospatial and Schematic GIS files as described above.

All 14 of the City's pump stations and lift stations are included in the hydraulic model. All pump station information was provided by the City through as-built drawings. Additionally, West Yost conducted field investigations for seven of the 14 pump stations within the City in February 2015. Pump operational parameters recorded during these field investigations were used in the model development. The pump station parameters in the model are summarized in Table 4-1. All elevations are taken from an NAD 88 datum. The modeled collection system facilities are presented on Figure 4-1.

Table 4-1. Pump Station Model Parameters

	Wet Well		Pumps				Force Main	
Pump Station	Size	Base Elevation, ft	No. of Pumps	Lead/Lag Pump On Elevation, ft	Lead/Lag Pump Off Elevation, ft	Pump Discharge Rate	Diameter, inches	Type
Allan Lift Station	28.3 ft^2 by 26 ft deep	84	2	89.7 / 90.7	87.0	Variable	6	Single
Bridge District Lift Station	95.0 ft^2 by 20.3 ft deep	1	3	6.7 / 7.7	2.4	Variable	12	Single
Bridgeway Island Pump Station	113.1 ft^2 by 34.48 ft deep	-21.48	3	-14.23 / -13.73	-18.0	Variable	12	Dual
Bryte Pump Station	64.0 ft^2 by 27.53 ft deep	-13.33	3	-6.0/-4.0	-10.0/-10.0	Variable	16	Single
Coke Lift Station	19.6 ft^2 by 18.24 ft deep	-5.74	2	3.0/2.0	0.0/0.0	Variable	8	Single
Industrial Pump Station	28.3 ft^2 by 27.6 ft deep	-15.7	3	-11.0/0.0	-12.0/-12.0	Variable	14	Single
Iron Works Lift Station	19.6 ft^2 by 20.5 ft deep	-3	2	0.5 / 1.5	-0.5	Variable	4	Single
Jefferson Pump Station	185 ft^2 by 21.15 ft deep	-4.75	3	0.0/10.0	-2.0/-1.8	Variable	18	Single
Largo Pump Station	78.5 ft^2 by 37.36 ft deep	-28.5	2	-22.5 / -21.5	-25.5 / -25.0	Variable	8	Dual
Northport Pump Station	28.3 ft^2 by 27.6 ft deep	-15.6	3	-10.0/-9.0	-12.0/-12.0	Variable	16	Single
Parlin Ranch Pump Station	50.3 ft^2 by 27.7 ft deep	-17.7	2	-12.0 / -11.0	-16.3	Variable	8	Single
South Pump Station	518 ft^2 by 25.5 ft deep	-4.1	3	1.0/1.5	0.5/0.5	Variable	20	Single
Southport Pump Station	420 ft^2 by 21.96 ft deep	-19.06	3	-15.0/-14.0	-16.0/-16.0	Variable	16	Single
Triangle Lift Station	20 ft^2 by 15.7 ft deep	-1.5	2	4.0 / 5.0	0.5	Variable	4	Single

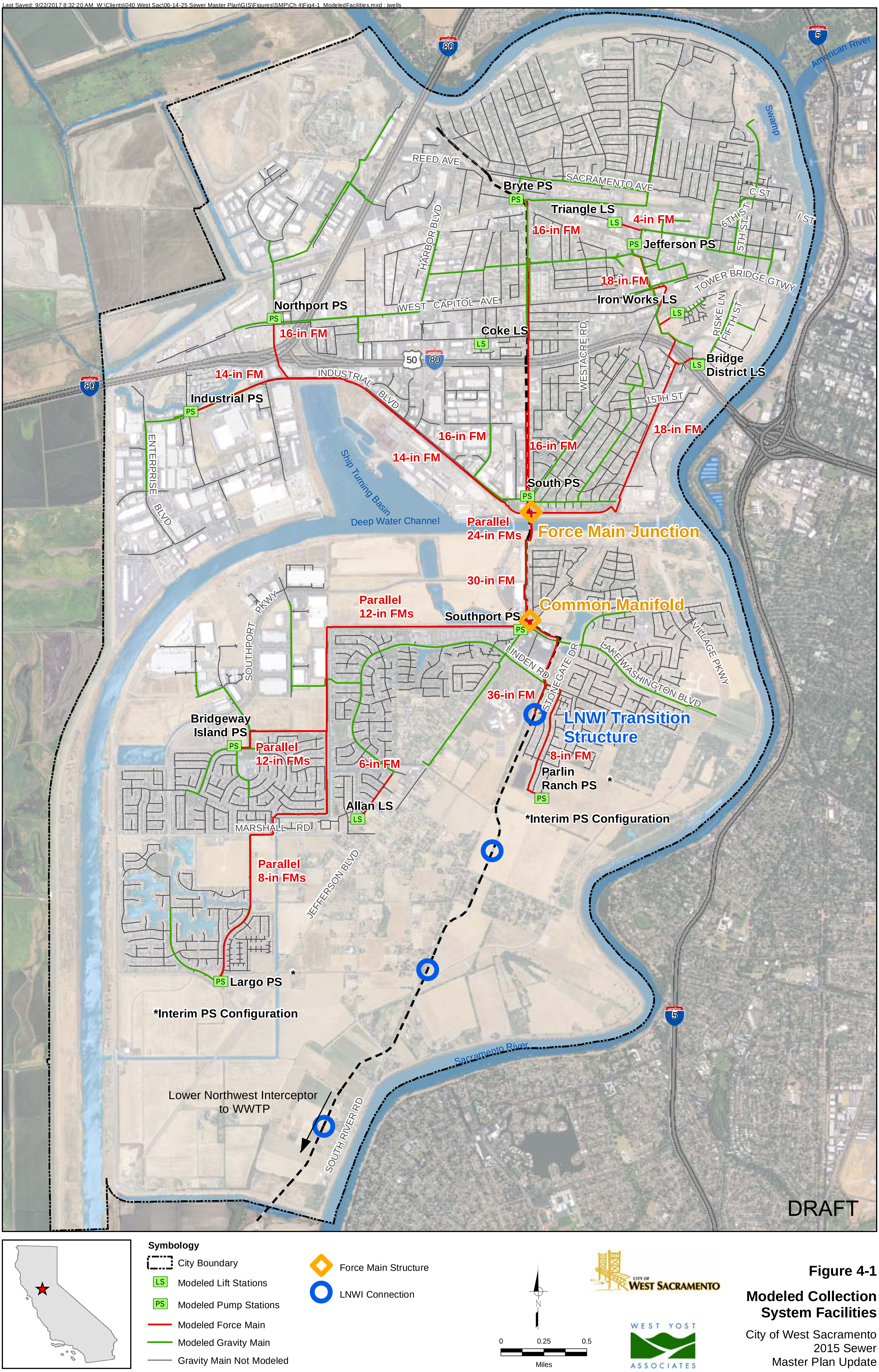

Figure 4-1

Modeled Collection System Facilities

City of West Sacramento
2015 Sewer
Master Plan Update

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4.2 DATA VALIDATION

After the model network was completed, West Yost conducted data validation to confirm that the model comprised a fully-connected network. Data validation included the following steps:

- Ensure each pipe and manhole has a unique identifier;
- Check the modeled network for connectivity, and add smaller pipes as needed to ensure no missing links or manholes in the network;
- Check for missing or inconsistent data such as missing manhole rim or pipe invert elevations, negative pipe slopes, or abrupt elevation changes;
- Identify manholes with more than one outlet pipe, constituting a potential flow split, that require further investigation in the field;
- Populate global parameters such as standard manhole diameters and Manning's "n" coefficient, which is entered as 0.013 for sewer pipelines; and
- Use system flags provided in InfoWorks™ CS to document identified issues and any changes made to the model.

Table 4-2 lists the system flags created by West Yost during model development and verification.

Table 4-2. System Flags

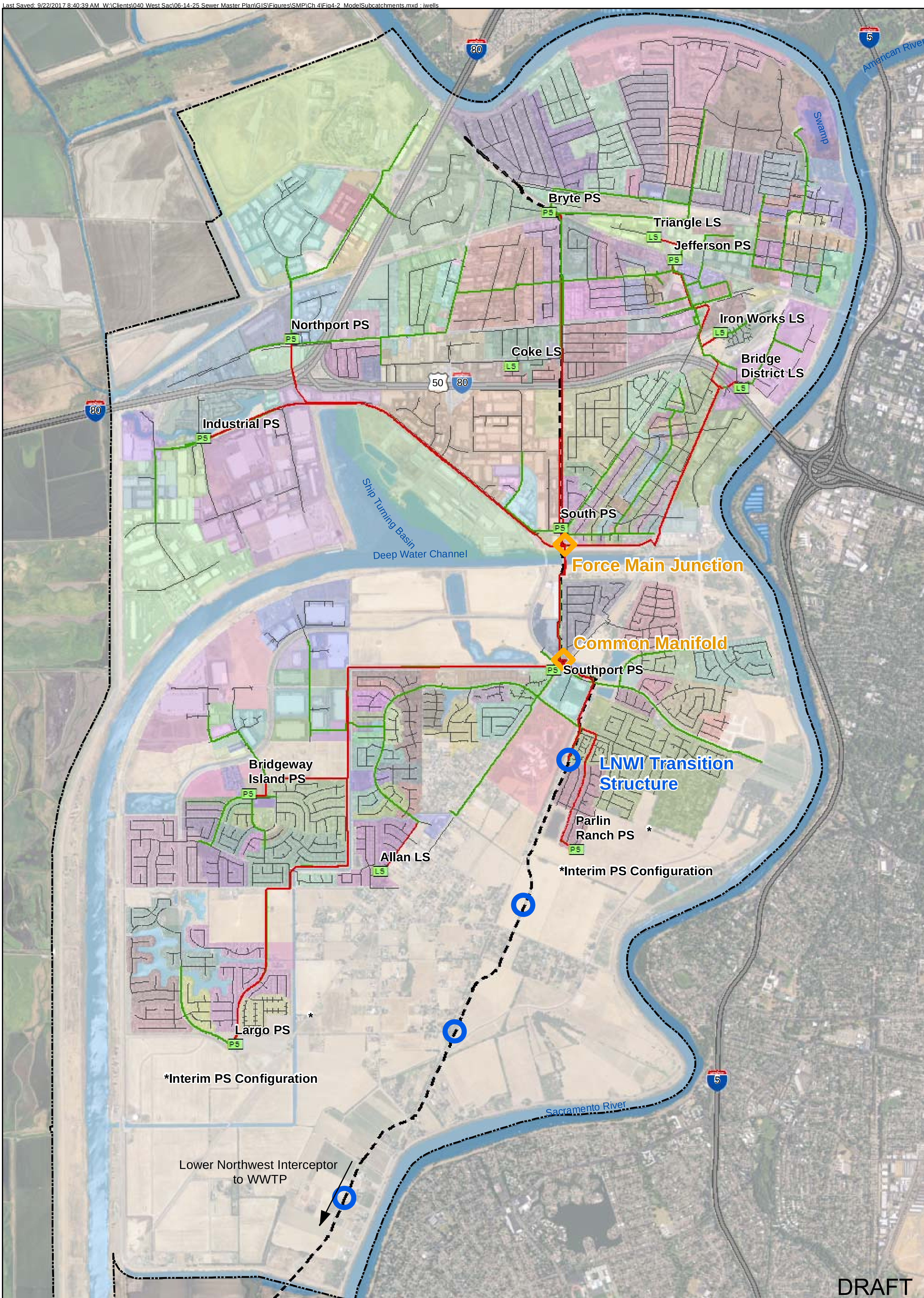
Flag	Source
#A	Asset Data
#D	System Default
#G	GeoSpatial GIS
#S	Schematic GIS
#I	Model Import
#V	CSV Import
AS	Assumed Data
CP	West Yost Capacity Improvement
IG	Inferred Values using the Ground TIN
IN	Other Inferred Data
JH	Suggestion to Assumed Values
PS	Pump Station Assessment Spreadsheet
RS	Relief Sewer Data
W1	West Yost QA/QC Changes

4.3 LOAD ALLOCATION

This section summarizes how sewer loads were calculated and input into the computerized hydraulic model. West Yost delineated 257 existing subcatchments (sewer sheds, or model tributary areas) within the hydraulic model. Subcatchments were based upon parcel GIS data provided by the City, and were generally defined to encompass a single land use or neighborhood that flows to a single model node. Loads were developed and assigned to each subcatchment as will be described in Chapter 5. Wastewater flows for analysis and design of the City's sanitary sewers were divided into two categories. These flow types are discussed further in Chapter 5:

- Base wastewater flow (BWF) includes the average daily dry weather sanitary flow contribution from permitted connections to the collection system; and
- RDII results when flows from wet weather events infiltrate the system, either through defects in existing facilities, or unpermitted connections that convey stormwater into the sewer system.

Wastewater flows were estimated by subcatchment and assigned to the node at the downstream end of the subcatchment. Each subcatchment defines a geographic area where base wastewater flow generated in the area is assigned to a specific node (or manhole) in the model. Figure 4-2 presents an overview of the subcatchments that were included in the hydraulic model. Detailed views of the modeled subcatchments and collection system infrastructure can be found in *Appendix B: Existing Hydraulic Model Subcatchments*. Detailed information on subcatchment flows and hydraulic model calibration are presented for dry weather flows and wet weather flows in Chapter 5 and Chapter 6, respectively.



Symbology

- City Boundary
- Modeled Subcatchment (Various)
- LS Modeled Lift Stations
- PS Modeled Pump Stations
- Modeled Force Main
- Modeled Gravity Main
- Gravity Main Not Modeled
- Lower Northwest Interceptor
- Force Main Structure
- LNWI Connection

0 0.25 0.5
Miles

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 4-2

Existing Model Subcatchment Overview

City of West Sacramento
2015 Sewer
Master Plan Update

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This chapter presents a summary of the development of existing and future dry weather flows. The major sections of this chapter include:

- Existing Dry Weather Flows, and
- Projected Future Dry Weather Flows.

5.1 EXISTING DRY WEATHER FLOWS

The development of existing dry weather flow values from flow monitoring and other data is described below.

5.1.1 Flow Monitoring Program Description

A temporary flow monitoring study was conducted by V&A Consulting Engineers (V&A) to support flow development for and calibration of the hydraulic model of the City's collection system. The description of the flow monitoring program is comprised of the following sections:

- Flow Monitoring Overview,
- Site Selection Criteria,
- Flow Monitor Locations,
- Rain Gage Locations, and
- Flow Monitoring Study Results.

A full description of the flow monitoring program and result can be found in *Appendix C: City of West Sacramento Sewer Flow Monitoring and Inflow/Infiltration Study*.

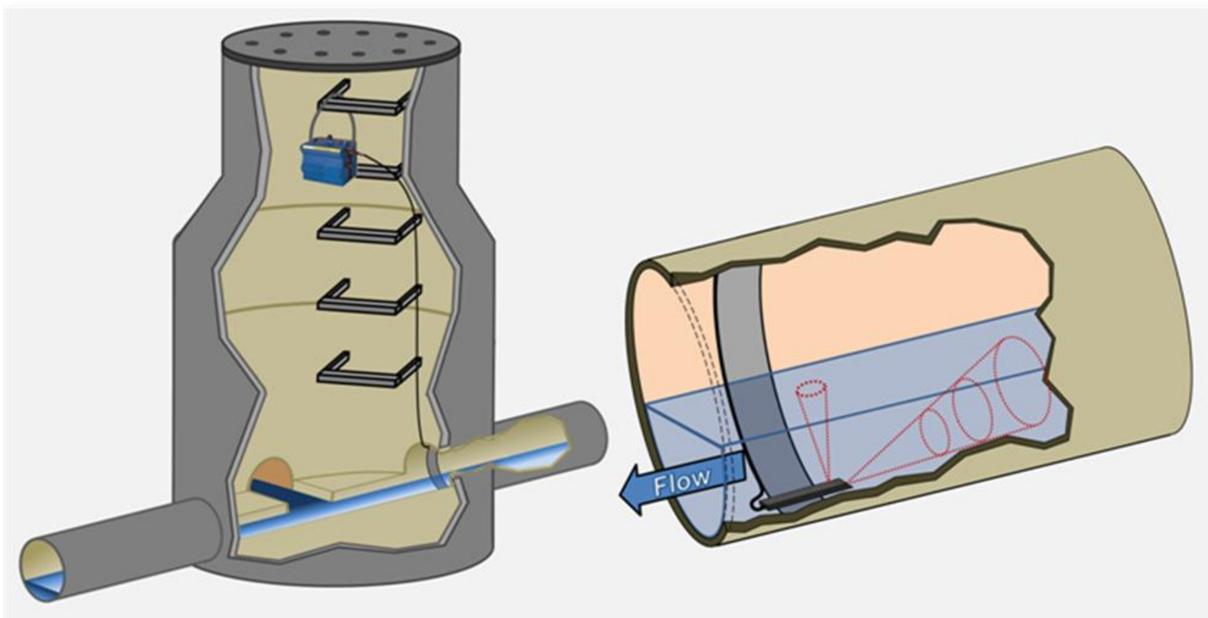
5.1.1.1 Flow Monitoring Overview

During the temporary flow monitoring study, flow monitoring and rainfall monitoring were performed at 11 open-channel flow monitoring sites and two rain gauges, respectively, from February 5, 2015 to April 12, 2015. The flow and precipitation data at these sites were collected at five minute intervals. The main purpose of gathering this data was to establish base sanitary flow rates, quantify groundwater infiltration, and calculate rainfall-dependent inflow and infiltration. This information has been used to calibrate the hydraulic model of the collection system, as described further below.

Teledyne Isco 2150 flow monitors were utilized by V&A for the temporary flow monitoring study. These meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur.

Manual level and velocity measurements were taken during installation of the flow meters and again when they were removed and were compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. The pipe diameter was also verified in order to accurately calculate the flow cross-section. The continuous depth and velocity readings were recorded by the flow meters and downloaded for processing by computer. The typical installation of such a flow monitor can be seen on Figure 5-1.

Figure 5-1. Typical Installation for Flow Monitor with Submerged Sensor



Source: City of West Sacramento Sewer Flow Monitoring and Inflow/Infiltration Study

5.1.1.2 Site Selection Criteria

West Yost staff, City staff, and V&A staff collaborated to determine the appropriate location for flow monitors and rain gauges for the temporary flow monitoring study. The temporary flow monitors were located to:

- Isolate basins with inflow and infiltration (I/I) or previous sewer rehabilitation,
- Meet minimum drainage basin size and flow requirements, and
- Avoid interruption from pump station and lift station on/off cycles.

Rainstorms in the City typically prevail from the west. Therefore, temporary rain gages were located in both the western and eastern portions of the City to capture full rain events to the highest degree possible.

5.1.1.3 Flow Monitor Locations

Using the criteria described above, it was determined that eleven flow monitors were necessary to isolate basins with the collection system and fully characterize their flows. Table 5-1 summarizes the location of these gravity main flow monitors, which are also presented on Figure 5-2. Flow monitors installed in the designated manhole monitored flows in the gravity main discharging into the manhole.

Table 5-1. Flow Monitor Locations

Meter No.	Installation Manhole	Diameter, inches	Location
FM-01	-	16	Intersection of Yolo Street and Mikon Street
FM-02	-	14	Grassy Area on Sacramento Avenue, East of Kegle Drive
FM-03	-	18	Dirt Easement behind 412 Washington Avenue
FM-04	-	24	837 F Street
FM-05	-	12	Behind Big Lots Store at 1270 West Capitol Avenue
FM-06	968	21	Driveway to 2150 Stone Boulevard
FM-07	-	15	In Grass at 2150 Stone Boulevard
FM-08	649	12	Eastern Corner of Jefferson Boulevard and Lake Washington Boulevard
FM-09	643	27	Jefferson Boulevard Between Linden Road and Lake Washington Boulevard
FM-10	725	24	2050 Lake Washington Boulevard
FM-11	561	14	1973 Linden Road

The basins isolated by the temporary flow monitoring locations and the relationship of the flow monitoring locations with regard to the City's lift stations and pump stations are shown schematically on Figure 5-3.

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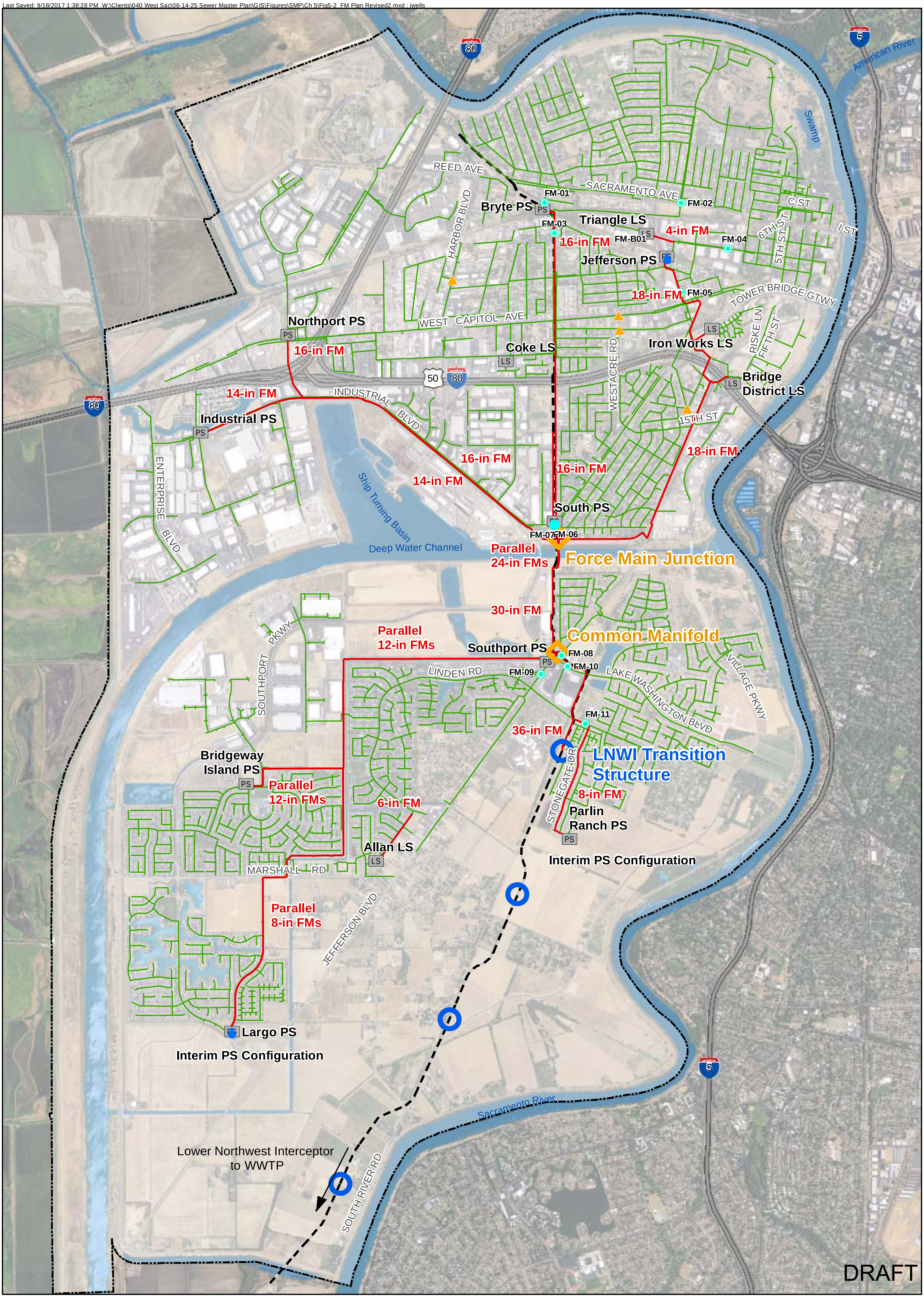

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Gravity Main
- Force Main
- Lower Northwest Interceptor
- Force Main Structure
- LNWI Connection
- Flow Meter
- Rain Gauge
- Plug

0 0.25 0.5
Miles

CITY OF WEST SACRAMENTO

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Figure 5-2

Flow Monitoring Locations

City of West Sacramento
2015 Sewer
Master Plan Update

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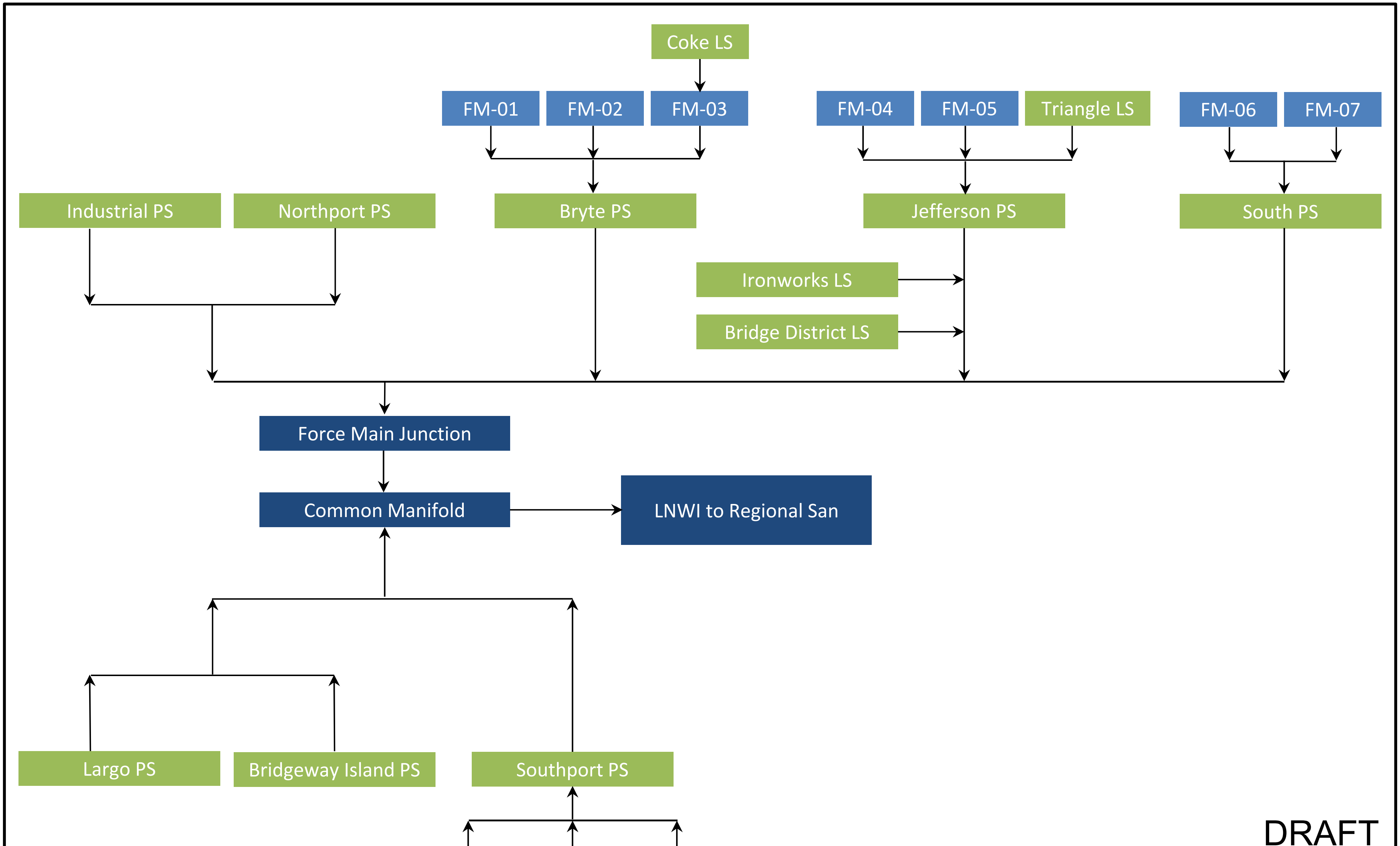

Figure 5-3

City of West Sacramento
2016 Sewer Master Plan

FLOW MONITORING SCHEMATIC

WEST YOST ASSOCIATES
Consulting Engineers

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As part of the temporary flow monitoring study, the City installed temporary plugs at locations where the flow potentially split between two monitoring basins to ensure that the basins remain hydraulically isolated. The following four locations were identified as flow splits by West Yost working with City staff:

- Eleventh Street at Jefferson Boulevard,
- West Capitol Avenue at West Acre Road,
- Merkley Avenue at West Acre Road, and
- Harbor Boulevard between Rice Avenue and West Capitol Avenue.

5.1.1.4 Rain Gage Locations

Two temporary rain gages were installed to accurately quantify rainfall during the flow monitoring period. The rain gauges were tipping buckets with dedicated data loggers. The rain gauge locations were selected to be publicly-owned locations on flat roofs with no tree cover or other obstruction. The rain gauge locations (shown on Figure 5-2) are described in Table 5-2.

Table 5-2. Temporary Rain Gauge Locations

Rain Gauge No.	Location	Purpose/Description
RG-01	Allan Pump Station	Characterize the rainfall in areas in the western part of the City
RG-02	Jefferson Pump Station	Characterize the rainfall in areas in the eastern part of the City

5.1.1.5 Flow Monitoring Study Results

During the temporary flow monitoring period of February 5, 2015 to April 12, 2015, flow monitoring data was captured during both dry weather and wet weather conditions. Wet weather conditions were captured during two rainfall events that took place during the temporary flow monitoring period. The rainfall measured during these events is summarized in Table 5-3.

Table 5-3. Event Rainfall Summary

Event	RG-01 Allan Pump Station, in	RG-02 Jefferson Pump Station, in
Event 1: February 6-8, 2015	2.33	2.23
Event 2: April 7, 2015	0.82	0.83
Total Over Period	3.15	3.06

Event 1 produced more rain and was the more significant event. By historical rainfall return frequency standards, Event 1 was a relatively small precipitation event and is classified with a one year, 12-hour return frequency. Despite the relatively small magnitude of the event, the collection system showed a measurable response to the precipitation; Event 1 was suitable for wet weather flow development and wet weather calibration of the collection system model. All discussion of wet weather system response, I&I, and RDII that follow in this and other chapters are based upon data from Event 1, unless otherwise noted. RDII and wet weather flow response is discussed further in Chapter 6.

Average Dry Weather Flow (ADWF) is generally considered to consist of Base Wastewater Flow (BWF) being generated by collection system users plus Groundwater Infiltration (GWI). Analysis of the dry weather flow monitoring data from the temporary flow monitoring period indicates that GWI values were low, so ADWF is assumed to be consistent with BWF values for the City. ADWF values captured at the 11 flow meter locations, broken down by time of the week, are presented in Table 5-4.

Table 5-4. Baseline Flow Summary

Meter No.	Sediment, in	Monday-Thursday ADWF, mgd	Friday ADWF, mgd	Saturday ADWF, mgd	Sunday ADWF, mgd	Overall ADWF, mgd
FM-01	-	0.566	0.557	0.586	0.582	0.570
FM-02	-	0.115	0.118	0.119	0.126	.0118
FM-03	-	0.514	0.494	0.512	0.527	0.513
FM-04	3.0	0.203	0.209	0.200	0.184	0.201
FM-05	1.0	0.075	0.074	0.070	0.069	0.073
FM-06	-	0.392	0.315	0.142	0.130	0.308
FM-07	0.25	0.271	0.273	0.285	0.291	0.276
FM-08	-	0.085	0.086	0.093	0.091	0.087
FM-09	-	0.955	0.972	1.003	1.010	0.972
FM-10	-	0.152	0.151	0.166	0.172	0.157
FM-11	-	.0214	0.204	0.224	0.234	0.217

Source: City of West Sacramento Sewer Flow Monitoring and Inflow/Infiltration Study

As shown in Table 5-4, the majority of the flow meter locations show weekend flows to be slightly higher than weekday flows. This pattern is typical of areas that are predominantly residential in nature. ADWF is higher and peaks later in the morning on the weekends as residents sleep later and fewer leave for work. However, at FM-04, FM-05, and FM-06, week day flows are slightly higher than weekend flows. Such a pattern is typical of commercial and industrial areas, in which the number of people working or visiting is highest during week day work hours. The day of the week patterns seen in the ADWF is consistent with the residential/non-residential land use profiles seen in the flow meter basins.

5.1.2 Existing Dry Weather Flow Allocation

This section describes the tasks completed in calculation of dry weather flows across the City based upon the flow monitoring data described above.

5.1.2.1 Average Dry Weather Flow Allocation in Study Area

ADWF can be calculated based one or more factors, including residential population, working employee population, water consumption, and land uses. To allocate ADWF across the 2015 Sewer Master Plan Study Area, industry-standard unit flow factors (verified through the flow monitoring data described above) were applied to residential population and employee population to generate ADWF. Residential and employee population were utilized because the City's planning department tracks and projects values for these populations at a very specific, parcel-based level. Because the flow monitoring locations do not fully capture all of the flow generated in the City, lift station and pump station data was used for verification as well.

The City's Planning Department tracks residential population in terms of number of households, and employee population in terms of number of employees. This information is tracked on an individual parcel basis. The number of households and employees was summed from parcels to subcatchments in the hydraulic model. The following equation was used to calculate ADWF for each subcatchment:

*(number of households) * (household unit flow factor) + (number of employees) * (employee unit flow factor) = total subcatchment ADWF*

The unit flow factors were originally estimated to be industry standard values. West Yost refined these unit flow factors by calculating the overall flow generated from each of the basins monitored during the temporary flow monitoring program. The calculated ADWF per basin were then compared with the metered flow data and adjusted until model predicted ADWF was within approximately ten percent of measured data in every metered sewer basin. These flows served as initial base flow inputs to the hydraulic model.

5.1.2.2 Diurnal (24-Hour) Flows

ADWF typically varies throughout the day, with the peak flow generally occurring in the morning and late afternoon periods. West Yost obtained 24-hour diurnal patterns for each monitored basin from the temporary flow monitoring study. A sample diurnal curve is presented in Figure 5-4 for the Site FM-02. A complete set of diurnal curves from all flow monitors is included in *Appendix D: Dry Weather Calibration Plots*. Diurnal flow characteristics were applied to the individual land use Q_{adwf} within each monitored basin to distribute the Q_{adwf} over a 24-hour period. Q_{pdwf}, which represents the Peak Dry Weather Flow (PDWF) is defined as the highest dry weather flow value seen over the 24-hour period.

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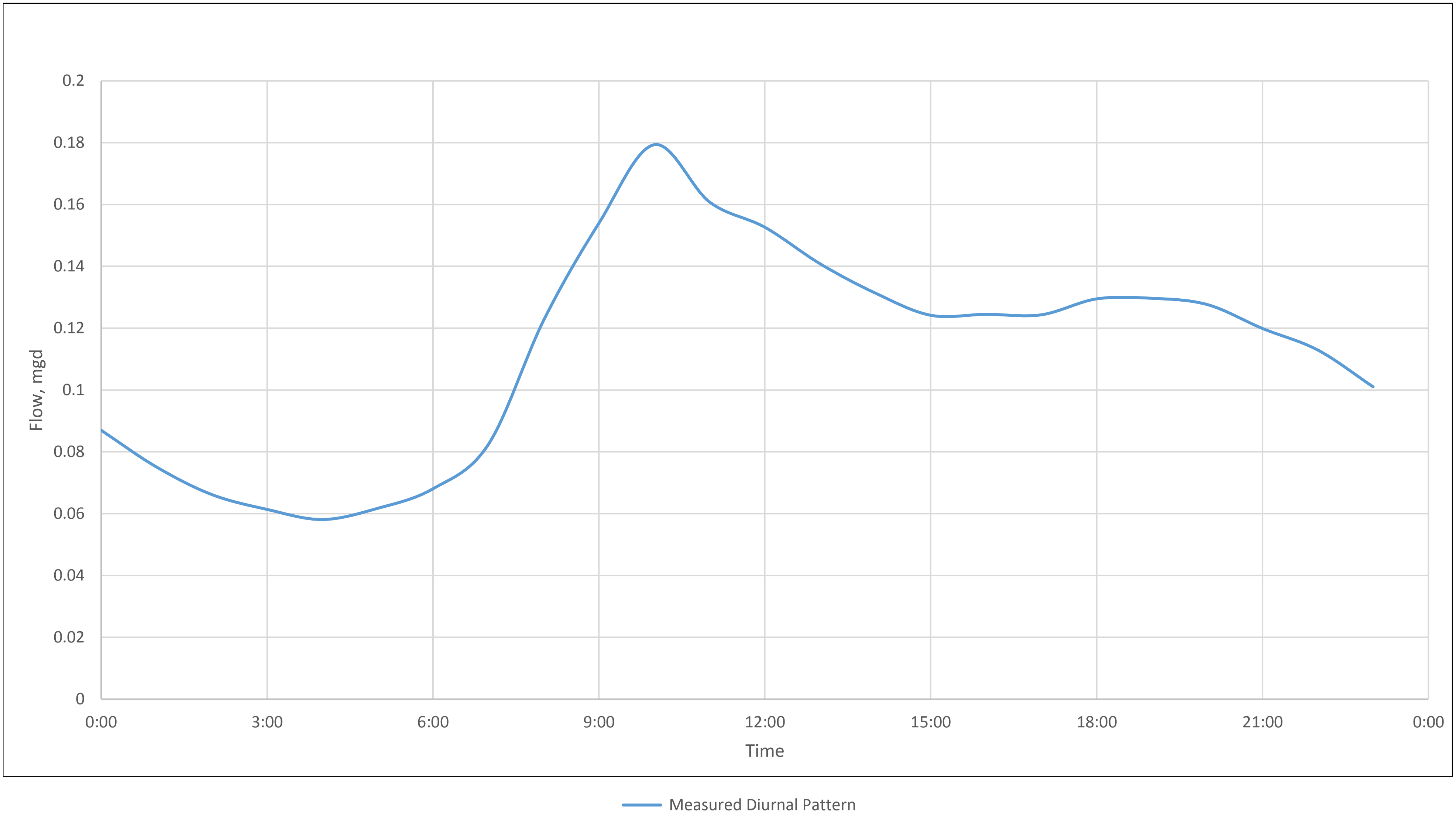

Figure 5-4

Measured Diurnal Pattern FM-02

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5.1.3 Existing Dry Weather Flow Calibration

The unit flow factors, diurnal patterns, and ADWF allocations described above were adjusted until ADWF and PDWF values seen in the hydraulic model matched within 10 percent at each flow monitoring location site. A sample dry weather calibration plot for FM-02 is presented on Figure 5-5. The full set of dry weather calibration plots can be found in *Appendix D: Dry Weather Calibration Plots*. In some cases, proximate lift station and pump station operation causes spikes in the flow reported by the model, but the overall mass balance is preserved.

5.1.4 Calibrated Existing Dry Weather Flow Values

The calibrated unit flow factors developed through the process above are presented in Table 5-5. The presented model values are lower than the design values that have been in use by the City. That existing modeled values are lower than historical design values is typical of wastewater agencies across California at this time. The ongoing drought, which has resulted in both voluntary and mandatory potable water use reductions, has caused a reduction in unit BWF and ADWF factors across the state. Because it is not expected that the lower water usage and resulting lower wastewater generation will persist indefinitely after the drought ends, the City's historical wastewater unit flow factors are used to predict future flows in the 2015 Sewer Master Plan Update.

The City's Planning Department tracks existing and future development across the City by neighborhood. The neighborhoods can be seen on Figure 5-6. Existing wastewater flows calibrated as described above, summarized by neighborhoods north and south of the Main Channel, are presented in Table 5-6.

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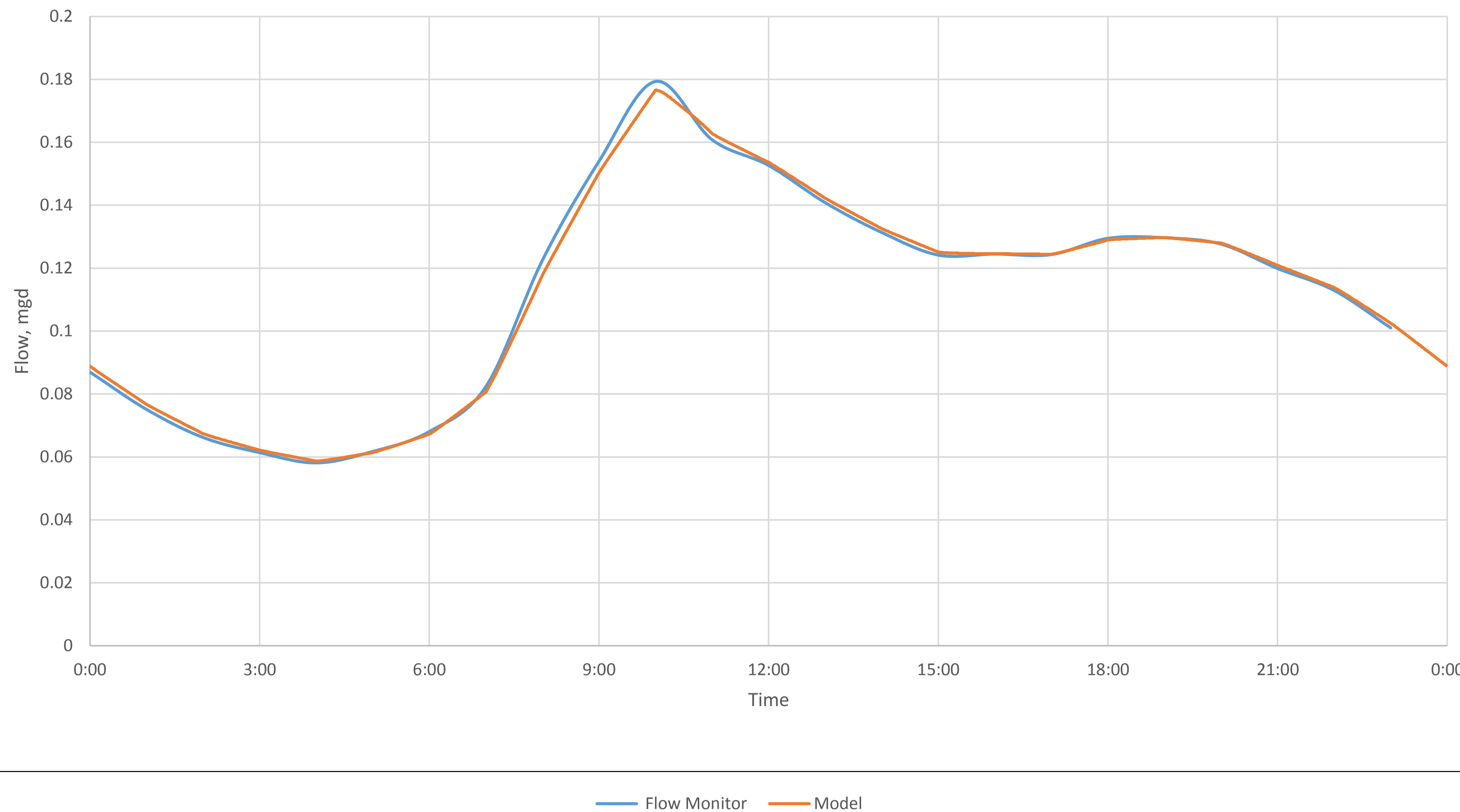

Figure 5-5

Sample Hydraulic Model ADWF Calibration Plot for FM-02

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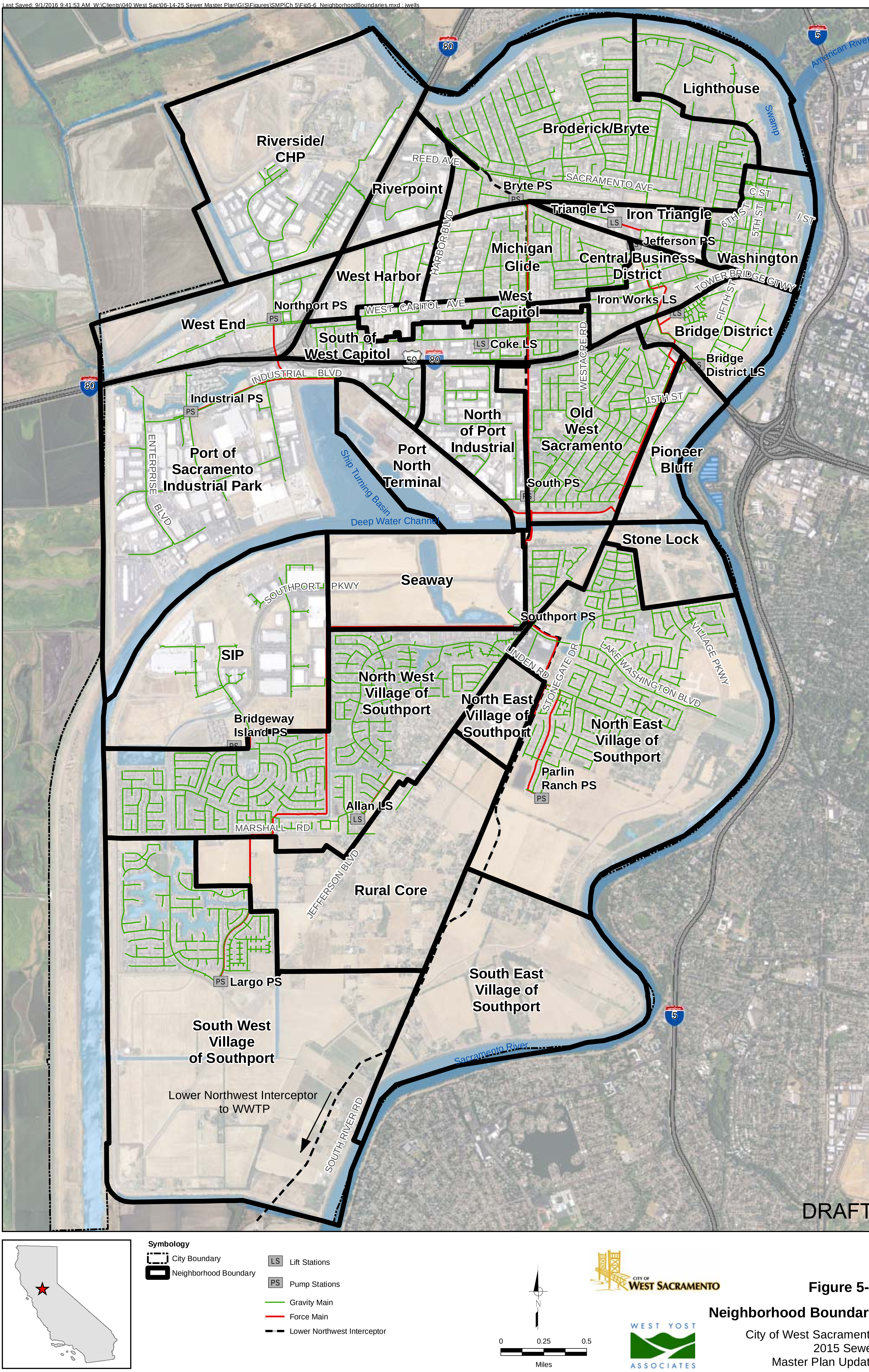

Figure 5-6

Neighborhood Boundary

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Table 5-5. Wastewater Unit Flow Factors

Landuse Code	Landuse	Model Unit Flow, gpd/unit				Design Criteria Unit Flow, gpd/unit	
		Unit	Typical Flow	Max	Min	Unit	Design Flow
AG	Agriculture	acre	0	0	0	*(mixed use, not specified in DC)*	
BP	Business Park	employee	15	15	15	acre	1,500
CBD	Central Business District	acre	50	50	525	*(mixed use, not specified in DC)*	
CC	Community Commercial	employee	15	15	15	acre	1,500
GC	General Commercial	employee	15	15	15	acre	1,500
HI	Heavy Industrial	employee	case-by-case basis	65	100	-	case-by-case basis
HR	High Density Residential	household	110	35	150	household[a]	250
HRR	High Rise Residential	household	100	75	150	household[a]	250
HSC	High Service Commercial	employee	15	15	15	acre	1,500
LI	Light Industrial	employee	15	15	224.5	acre	2,000
LR	Low Density Residential	household	125-150	90	220	household[a]	300
MCI	Mixed Commercial / Industrial	employee	15	15	15	*(mixed use, not specified in DC)*	
MR	Medium Density Residential	household	125	100	195	household[a]	300
NC	Neighborhood Commercial	employee	15	15	15	acre	1,500
O	Office	employee	15	15	15	acre	1,500
OS	Open Space	acre	50	50	50	acre	0
PQP	Public / Quasi Public	student	15	15	15	student	4
RE	Rural Estates	household	100-150	100	220	household[a]	300
RMU	River Mixed Use	acre	150	45	525	*(mixed use, not specified in DC)*	
RP	Recreation and Parks	acre	500	500	500	acre	500
RR	Rural Residential	household	100-150	100	200	household[a]	300
WRC	Water Related Commercial	employee	15	15	15	acre	1,500
WRI	Water Related Industrial	employee	15	15	15	acre	2,000

[a] Residential demands based on 100 gpd/person flow, with 3 persons per single family household and 2.5 persons per multi-family household.

Table 5-6. Summarized Existing ADWF			
Area	Existing Households	Existing Employees	Existing ADWF, mgd
North of Shipping Channel	10,085	22,443	2.93
Southport	7,732	2,765	1.73
Total	17,817	25,208	4.66

5.2 PROJECTED FUTURE ADWF

The projection of future ADWF values from the existing values developed above is described in the following sections.

5.2.1 2035 General Plan Update

The City Planning Department has undertaken a comprehensive update process to create the City's 2035 General Plan Update. The future ADWF projections used for the 2015 Sewer Master Plan Update are based upon the development projections contained in the 2035 General Plan Update. The City Planning Department provided West Yost with 2035 residential and non-residential development projections for individual parcels. These projections are summarized by neighborhood in Table 5-7. As shown in the table, the number of households is expected to increase by approximately 18,500 in the study area by 2035. The number of employees is expected to increase by approximately 28,835 in the same timeframe. The location of residential growth, broken down by neighborhood, can be seen on Figure 5-7. The location of non-residential growth, broken down by neighborhood, can be seen on Figure 5-8.

5.2.2 Specific Development Plans

At the current time, the City has three specific development plans in progress identified by the Planning Department: The Liberty Plan, The Riverpark Plan, and The Yarborough Plan. The location of these three plans can be seen on Figure 5-7 and Figure 5-8. The residential and non-residential growth projected for each of these plans is already contained in the appropriate neighborhood data contained in Table 5-7. The Liberty Plan is projected to contain 1,501 households and 136 employees, the Riverpark Plan is expected to contain 2,722 households, and the Yarborough Plan is expected to contain 2,389 households and 13 employees.

Table 5-7. 2035 General Plan Development Projections Summarized by Neighborhood

	Households			Retail Employment			Non-Retail Employment			Total Employment		
Neighborhood	2012	2035	Growth	2012	2035	Growth	2012	2035	Growth	2012	2035	Growth
North of Deep Water Channel												
Broderick/Bryte	3,778	4,159	381	257	715	458	302	568	266	559	1,283	724
Central Business District	296	1,030	734	769	479	(290)	445	830	385	1,214	1,309	95
Iron Triangle	-	629	629	132	316	183	755	907	153	887	1,223	336
Lighthouse	274	1,159	885	-	136	136	-	90	90	-	226	226
Michigan Glide	1,895	2,037	142	158	224	65	785	891	106	943	1,114	171
North of Port Industrial	-	-	-	314	314	-	1,246	1,246	-	1,561	1,561	-
Old West Sacramento	1,485	1,555	70	204	476	272	338	608	270	542	1,084	542
Pioneer Bluff	-	1,945	1,945	25	372	348	549	3,720	3,172	573	4,093	3,520
Port North Terminal	-	-	-	-	-	-	100	100	-	100	100	-
Port of Sacramento Industrial Park	251	262	11	986	1,711	726	4,232	4,356	124	5,218	6,068	850
Riverpoint	-	-	-	1,575	3,163	1,588	301	1,958	1,657	1,876	5,121	3,245
Riverside/CHP	-	-	-	485	1,232	747	3,104	4,072	968	3,589	5,304	1,715
South of West Capitol	968	1,134	166	376	341	(35)	306	327	21	681	667	(14)
Triangle	122	3,335	3,213	107	1,812	1,705	129	4,987	4,858	236	6,799	6,563
Washington	753	2,743	1,990	386	826	440	2,226	2,776	550	2,612	3,602	990
West Capitol	261	557	296	353	322	(30)	86	56	(31)	439	378	(61)
West End	-	-	-	293	357	64	491	494	2	785	851	66
West Harbor	2	2	-	127	127	-	501	501	-	628	628	-
South of Deep Water Channel												
North East Village of Southport	2,195	4,317	2,122	421	1,325	904	423	651	228	844	1,976	1,131
North West Village of Southport	3,835	4,063	228	80	266	187	158	251	93	238	517	279
Rural Core	266	278	12	55	222	167	48	67	19	103	289	186
Seaway	-		-	316	316	-	3	1,103	1,100	319	1,419	1,100
SIP	217	600	383	97	670	572	1,163	6,281	5,118	1,261	6,950	5,690
South East Village of Southport	37	2,172	2,135	-	-	-	-	-	-	-	-	-
South West Village of Southport	1,182	3,571	2,389	-	13	13	-	44	44	-	57	57
Stone Lock	-	784	784	-	85	85	-	1,338	1,338	-	1,423	1,423
Total	17,817	36,332	18,515	7,516	15,820	8,304	17,692	38,223	20,531	25,208	54,043	28,835

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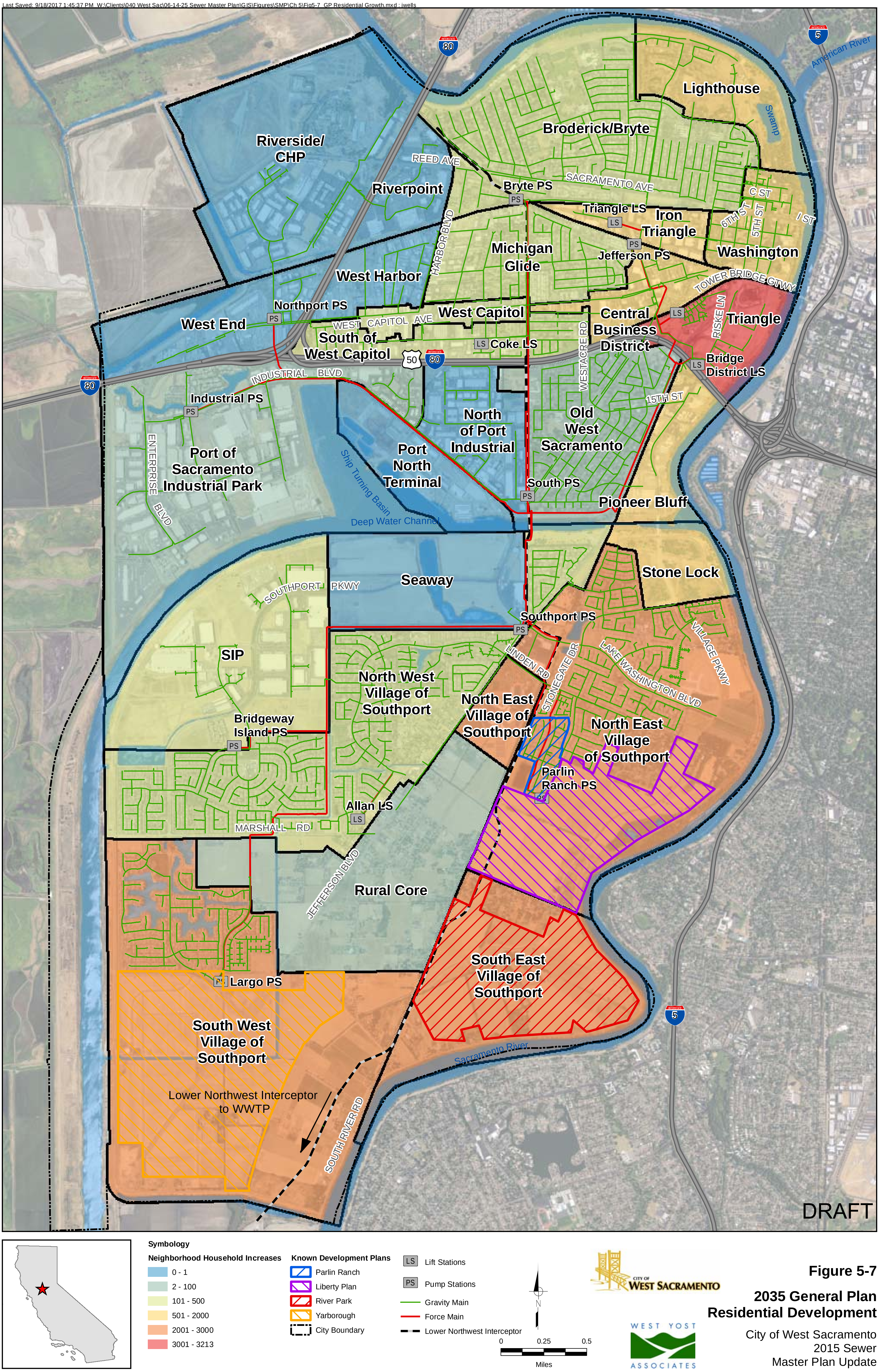

Figure 5-7

2035 General Plan Residential Development

City of West Sacramento
2015 Sewer
Master Plan Update

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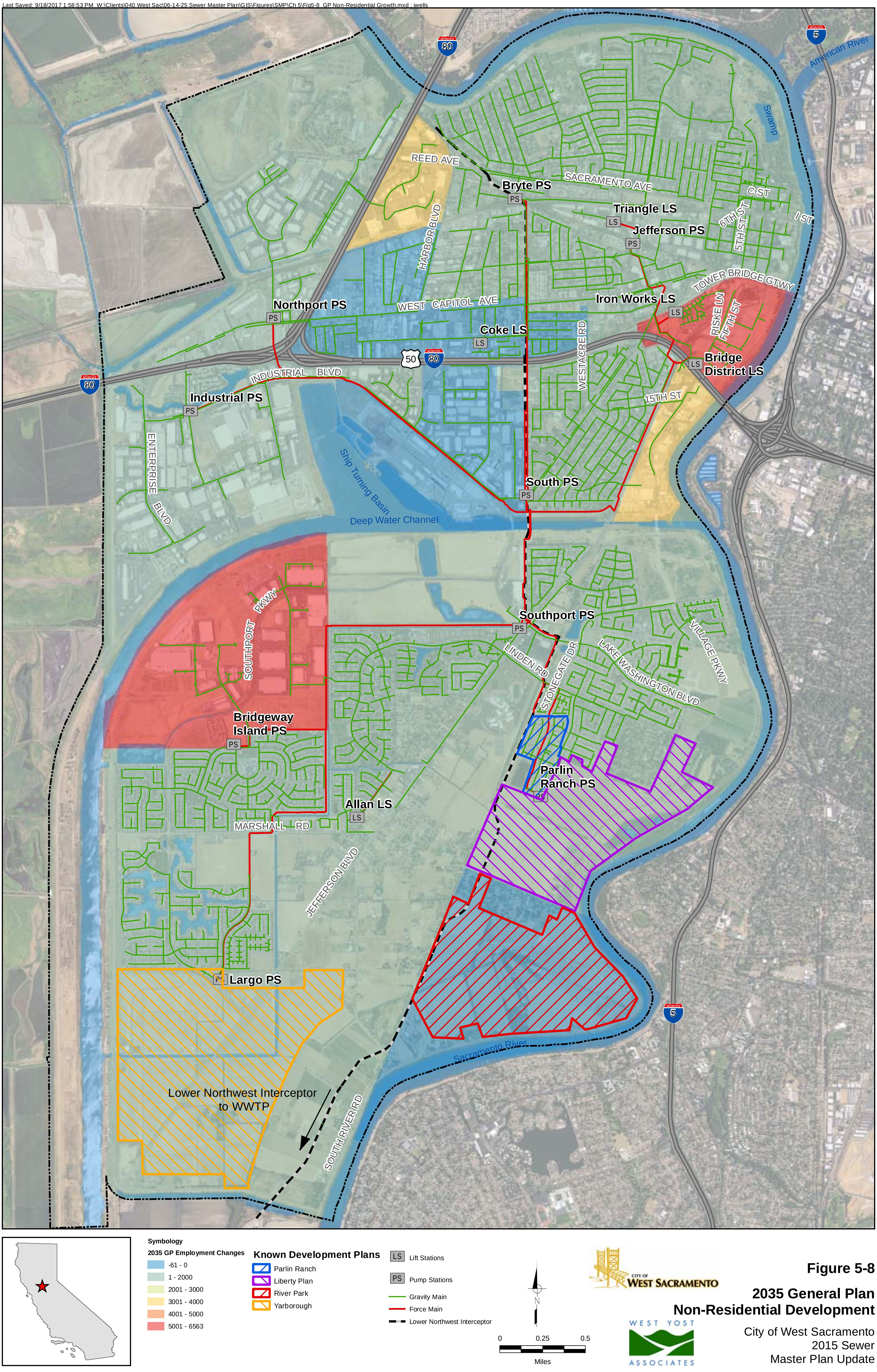

Figure 5-8

2035 General Plan Non-Residential Development

City of West Sacramento
2015 Sewer
Master Plan Update

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5.2.3 Projected 2035 ADWF Values

Using design wastewater flow values of 300 gpd/household and 50 gpd/employee as described above for City design values, future projections of residential and non-residential flow were developed for each parcel, and summarized by neighborhood and subcatchment. The projected 2035 ADWF values can be found in Table 5-8. Total ADWF is projected to grow from 4.66 mgd to 11.56 mgd by 2035.

Table 5-8. Existing and Future (2035) ADWF

Area	Existing Households	Existing Employees	Existing ADWF, mgd	2035 Households	2035 Employees	2035 ADWF, mgd
North of Shipping Channel	10,085	22,443	2.93	20,547	41,412	7.00
Southport	7,732	2,765	1.73	15,785	12,631	4.56
Total	17,817	25,208	4.66	36,332	54,043	11.56

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Chapter 6 summarizes the development, calibration, and projection of wet weather flows in the collection system. The major sections of this chapter include:

- Existing Wet Weather Flows, and
- Projected Future Wet Weather Flows.

6.1 EXISTING WET WEATHER FLOWS

The development of existing wet weather flow values from flow monitoring and other data is described below.

6.1.1 Wet Weather Flows Captured by Flow Monitoring

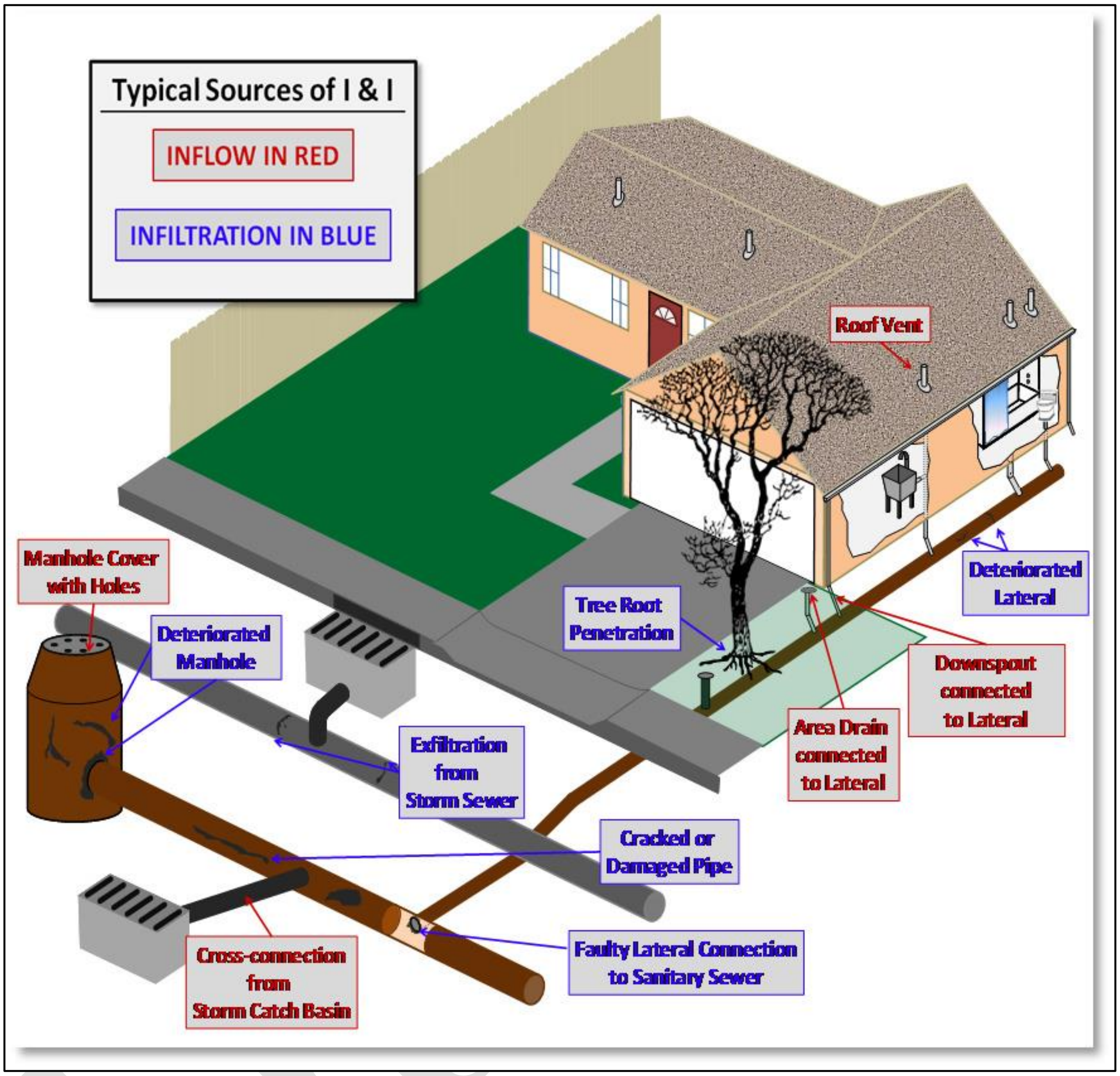
6.1.1.1 Wet Weather Flow Components

As stated in the City of West Sacramento Sewer Flow Monitoring and Inflow/Infiltration Study (Appendix C), I&I consists of storm water and groundwater that enter the sewer system through pipe defects and improper storm drainage connections. In that study, I/I is defined as follows:

- Inflow is defined as water discharged into the sewer system, including private sewer laterals, from direct connections such as downspouts, yard and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins.
- Infiltration is defined as water entering the sanitary sewer system through defects in pipes, pipe joints, and manhole walls, which may include cracks, offset joints, root intrusion points, and broken pipes.

Typical sources of the components of I&I can be seen in Figure 6-1.

Figure 6-1. Typical Sources of Infiltration and Inflow



Source: City of West Sacramento Sewer Flow Monitoring and Inflow/Infiltration Study

Wet weather flow occurs in a collection system when I&I that results from precipitation, typically called RDII, enters the collection system and must be conveyed and treated.

6.1.1.2 Wet Weather Flow Monitoring Study Results

As described in Chapter 5, during the temporary flow monitoring period of February 5, 2015 to April 12, 2015, flow monitoring data was captured during both dry weather and wet weather conditions. RDII was measured at all of the flow meter locations, but in general the RDII rates were higher north of the Deep Water Channel, where the collection system is older and more subject to the defects that result in RDII. Detailed RDII results can be found in the City of West Sacramento Sewer Flow Monitoring and Inflow/Infiltration Study.

6.1.2 Existing Wet Weather Flow Allocation

Extraneous flows can cause significant increases in peak flows in the collection system. Wet weather flows were calculated and input in the City's hydraulic model to replicate measured flow data. PWWF generated in the model is the combination of peak dry weather flow and RDII.

Several broad categories of RDII quantification are used in wastewater master planning, including the following:

- Constant unit rate method – calculates RDII as a fixed constant (e.g., gal/acre) multiplied by measurements of tributary subcatchment characteristics (e.g., area, land use, population, pipe diameter, pipe length, and pipe age);
- R-Value method – calculates RDII as a fixed percentage of rainfall;
- Synthetic unit hydrograph (SUH) method – calculates the RDII hydrograph from a specified "unit" hydrograph shape that relates RDII to unit precipitation volume and duration;
- Probabilistic method – calculates RDII of a given recurrence interval from long-term sewer flow records using probability theory. The method establishes the relationship of peak RDII flow to recurrence interval; and
- Rainfall/sewer flow regression method – calculates peak RDII flows from rainfall data through a relationship between rainfall and RDII flows. This regression, expressed as an equation, is derived from rainfall and flow monitoring data in sewers using multiple linear regression methods and considering dry and wet antecedent conditions.

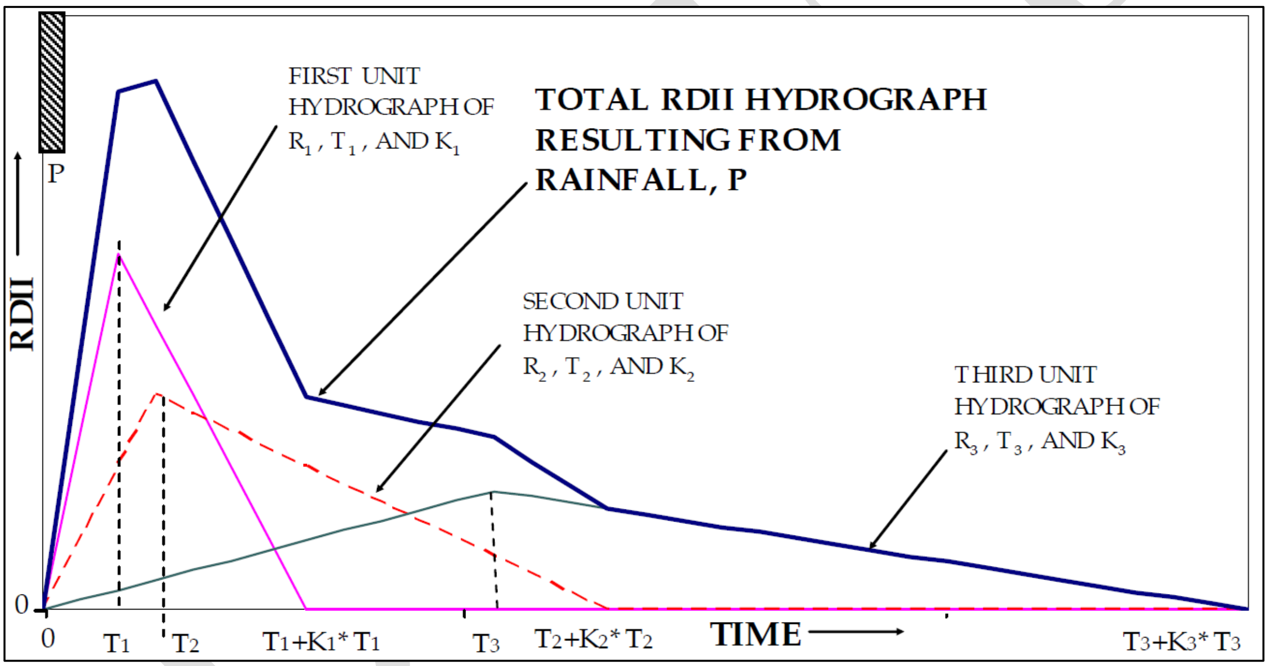
Studies conducted by the Water Environment Research Foundation have concluded that the SUH and rainfall/flow regression methods are the two most accurate methods for predicting peak flows and event volumes for storm events. The rainfall/time/recession (RTK)method, described below, is the most widely used SUH prediction methodology for collection system model development.

West Yost used the RTK method to calculate RDII inputs to the City's hydraulic model. The RTK method generates hydrographs from each subcatchment that represent flows during and immediately after rainfall events caused by seepage of water into the collection system. The RTK method generates a series of three triangular hydrographs that represent short-term, medium-term, and long-term rainfall response. The RTK parameters include:

1. R = the area of the graph representing the portion of rainfall falling on a subcatchment that enters the sewer collection system.
2. T = the time from the onset of rainfall to the peak of the triangle.
3. K = the ratio of the "time to recession" to the "time to peak" of the hydrograph.

Components of the RTK hydrograph are provided courtesy of the United States Environmental Protection Agency (EPA) Office of Research and Development, and are presented in Figure 6-2.

Figure 6-2. Components of RTK Hydrograph



When a wet weather flow simulation is run in the model, the RTK parameters are applied to represent a specific rainfall event. These parameters generate a wet weather flow hydrograph for each subcatchment.

Hourly PWWF values are generated in the model by combining the dry weather flow with flows from the RDII hydrographs, by subcatchment. Typically, the peak wet weather flow will occur shortly after the hourly peak intensity of the rainfall event.

6.1.3 Existing Wet Weather Flow Calibration

Following completion of dry weather calibration as described in Chapter 5, West Yost calibrated the model for wet weather flow conditions. A model that is sufficiently calibrated to wet weather flow should be able to simulate RDII entering the sewer collection system during a rainfall event. Wet weather calibration consisted of the following steps:

- Identifying a wet weather calibration event with heavy rainfall and collection system response (increased flows) from the flow monitoring data.
- Establishing appropriate methodology for I&I generation. The City's model uses the RTK method. Establishing I&I parameters per monitored basin based on collected flow monitoring data from the first calibration event and applying these parameters to the appropriate subcatchment.
- Generating system flows under wet weather loading, which included dry weather plus the I&I component. Comparing metered data with model simulation results, and adjusting RTK parameters if necessary, to maximize agreement for the calibration event.

As described in Chapter 5, wet weather conditions were captured during two rainfall events that took place during the temporary flow monitoring period. Event 1 produced more rain and was the more significant event. By historical rainfall return frequency standards, Event 1 was a relatively small precipitation event and is classified with a one year, 12-hour return frequency. Despite the relatively small magnitude of the event, the collection system showed a measurable response to the precipitation, and Event 1 was suitable for wet weather flow development and wet weather calibration of the collection system model.

The calibrated RTK values developed for the City's collection system are presented in Table 6-1. As can be seen in the table, the City's collection system shows fast response to precipitation (R1 values are larger than R2 and R3 values) and little long-term response (R3 values are small). This type of response most likely results from the fact that the condition of the collection system is fundamentally sound (discussed further in Chapter 9) in combination from the fact that a relatively small storm was captured for flow monitoring purposes during a long-term drought. The antecedent soil conditions were likely not saturated. The model calibration would benefit from further flow monitoring conducted during a wet weather season with more frequent and impactful precipitation. Basin FM-03 shows the highest response to precipitation in the collection system. Figure 6-3 presents a sample of the wet weather flow calibration results from meter FM-02. The remaining calibration graphs are presented in *Appendix E: Wet Weather Calibration Plots*.

Table 6-1. Calibrated RTK Factors for RDII Generation in Hydraulic Model by Basin

Basin	R1, %	R2, %	R3, %	T1, hr	T2, hr	T3, hr	K1	K2	K3
FM-01	0.4	0.0	0.2	1.0	2.0	8.0	2.0	2.0	2.0
FM-02	0.1	0.1	0.0	1.0	3.0	0.0	2.0	1.0	0.0
FM-03	1.0	1.7	0.1	1.0	5.0	12.0	2.0	1.0	1.0
FM-04	0.2	0.5	0.0	0.5	5.0	0.0	1.0	0.5	0.0
FM-05	0.3	.05	0.0	1.0	4.0	0.0	2.0	0.5	0.0
FM-06	0.2	0.0	0.0	1.0	2.0	0.0	2.0	2.0	0.0
FM-07	0.7	0.7	0.1	1.0	3.0	8.0	2.0	1.0	1.5
FM-08	0.1	0.7	0.0	1.0	5.0	0.0	1.0	0.5	0.0
FM-09	0.5	0.7	0.0	1.0	5.0	0.0	1.0	0.5	0.0
FM-10	0.3	0.4	0.0	1.0	5.0	0.0	1.0	0.5	0.0
FM-11	0.4	0.6	0.2	1.0	3.0	12.0	1.0	1.0	1.0

6.1.4 Existing Wet Weather Design Flows

The calibrated RTK values developed as described above were assigned to the collection system, and the design storm described in Chapter 3 was applied to the collection system. The resulting PWWF constitutes the existing design flows for the collection system. The existing design flows are shown in Table 6-2.

Table 6-2. Existing and Projected 2035 ADWF and PWWF Values, mgd

Area	Existing ADWF	Existing Design PWWF	Projected 2035 ADWF	Projected 2035 Design PWWF
North of Channel	2.93	14.00	7.00	20.19
Southport	1.73	8.73	4.56	15.01
Total	4.66	22.73	11.56	35.20

6.2 FUTURE WET WEATHER FLOW PROJECTIONS

In order to develop future wet weather flow projections, the calibrated RDII values developed as described above were applied to existing and future areas in the collection system. Future areas were matched to existing areas that they most closely resemble to determine RDII values. These RDII values were added to the projected future ADWF values developed as described in Chapter 5. The projected 2035 PWWF values are presented in Table 6-2.

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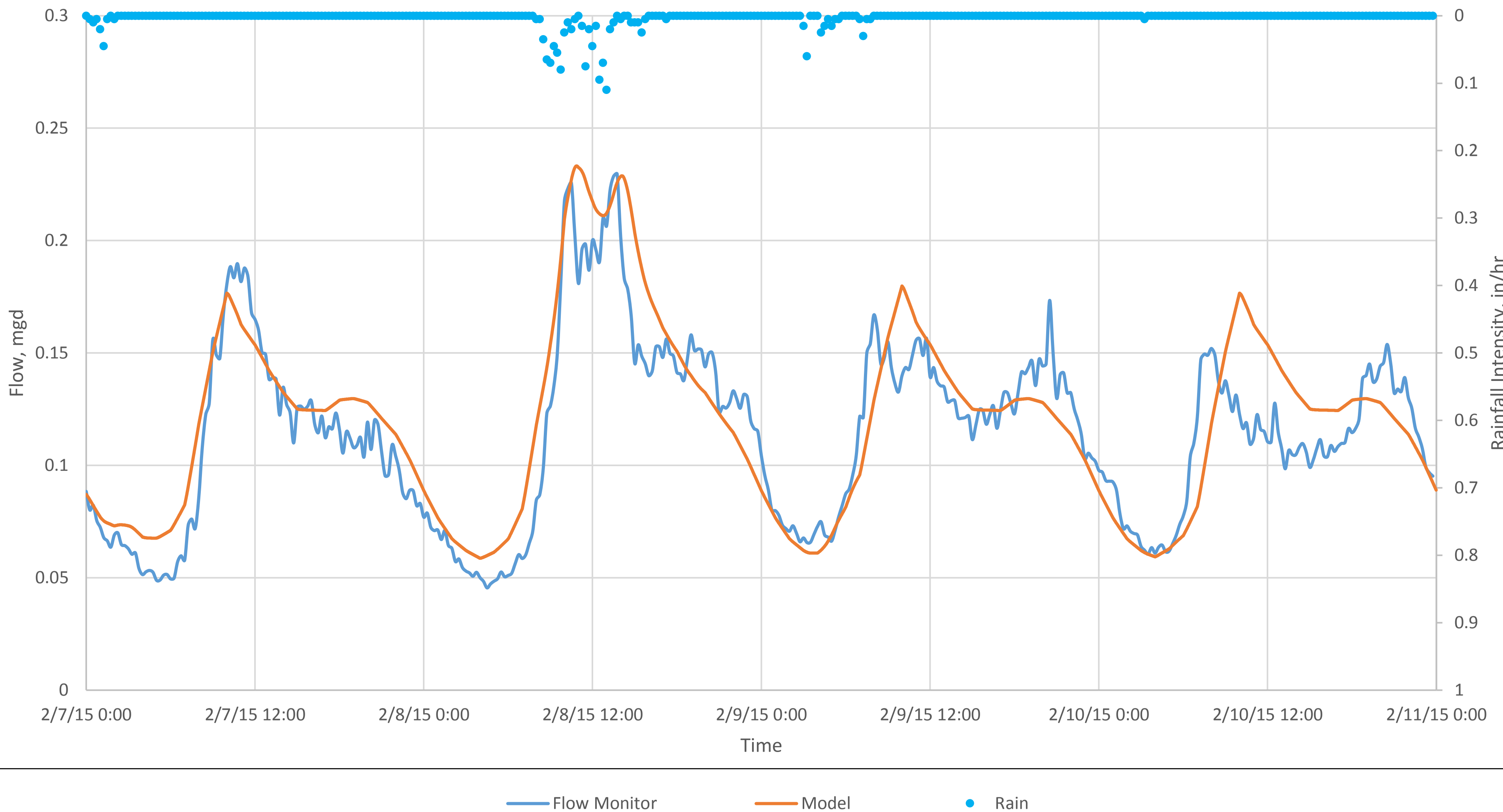

Figure 6-3

Sample Wet Weather Calibration Plot at FM-02

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Chapter 7 presents the results of the hydraulic evaluation of the City's collection system under existing conditions. Collection system capacity for gravity mains, lift station/pump stations, and force mains is assessed with respect to the system's performance under the existing PWWF design flow condition described in Chapter 6 using the criteria described in Chapter 3. The major sections of this chapter include:

- Existing Gravity Main Hydraulic Analysis,
- Existing Lift Station/Pump Station Hydraulic Analysis, and
- Existing Force Main Hydraulic Analysis.

7.1 EXISTING GRAVITY MAIN HYDRAULIC EVALUATION

Existing gravity mains exceed the performance criteria under existing design flows in two locations in the collection system. In general, gravity mains fail to meet the performance criteria due to being undersized or lacking sufficient slope. The gravity mains that fail to meet performance criteria are displayed on Figure 7-1. These gravity main deficiencies are summarized below.

7.1.1 Existing Stillwater Road Capacity Deficiency

Stillwater Road runs parallel to the west of I-80 in the northeast corner of the City. Found in the Riverside/California Highway Patrol (CHP) Neighborhood, Stillwater Road stretches from Reed Avenue in the north to Riverside Parkway in the south. North of Reed Avenue along the same alignment as Stillwater Road, the City owns and maintains a public gravity main in an easement through the CHP Academy which discharges into the gravity main at Reed Avenue and Stillwater Road. The main CHP Academy campus is found to the west of the easement, and the City's George Kristoff Water Treatment Plan is found to the east.

The existing gravity in the CHP easement is 10-inch diameter. When the existing gravity main exits the easement and runs east/west in Reed Avenue for a short stretch before turning south, the existing diameter is reduced to 8-inch diameter. From this point to a point further south in Stillwater Road at which flow from the Riverpoint Neighborhood to the east enters the gravity main and the diameter of the gravity main increases to 12-inch, the existing gravity main is 8-inches in diameter and undersized to carry the design PWWF. There is significant RDII contribution projected in this gravity main because of the large area of the CHP campus served by this gravity main. The project has not been confirmed by direct flow monitoring. It is recommended that the City flow monitor this location to confirm the CIP project.

The detailed project area can be seen on Figure 7-2. The hydraulic profile of this section of gravity main can be seen on Figure 7-3. As shown, the surcharging in the 8-inch gravity main leads to projected overflows upstream under existing PWWF design conditions. The hydraulic model indicates that the City should upsize 1,200 feet of 8-inch gravity main to 12-inch gravity main along this stretch of Stillwater Road. The project to complete this capacity increase is described in more detail in Chapter 10.

7.1.2 Existing Bryte Pump Station Tributary Capacity Deficiency

The Bryte Pump Station is at the eastern end of Citrus Street, just north of the railroad right-of-way (ROW). A large section of the area in the central portion of the City that is north of the Deep Water Channel drains to an interceptor gravity main that then flows north to the Bryte Pump Station. This interceptor runs in an ROW between Maple Street to the west and Poplar Avenue to east; the interceptor tributary to the Bryte Pump Station runs parallel in the ROW to the force main that carries flow south from Bryte Pump Station to the South Pump Station. Also in the same ROW is the LNWI, running from north to south.

The deficiency area can be seen on Figure 7-4. The hydraulic profile of the interceptor gravity main between Michigan Boulevard to the south and Bryte Pump Station to the north is shown on Figure 7-5. As shown, the hydraulic model does not predict overflows under existing PWWF design conditions, but it predicts manhole surcharging to within approximately two feet of the ground elevation, violating the City's design criteria.

The hydraulic model indicates that the City should upsize 2,500 feet of 18-inch gravity main to 24-inch gravity main in this ROW between Michigan Boulevard and the intake of the Bryte Pump Station. This project will include a section of gravity main that runs underneath the railroad ROW just south of the Bryte Pump Station. The project to complete this capacity increase is described in more detail in Chapter 10.

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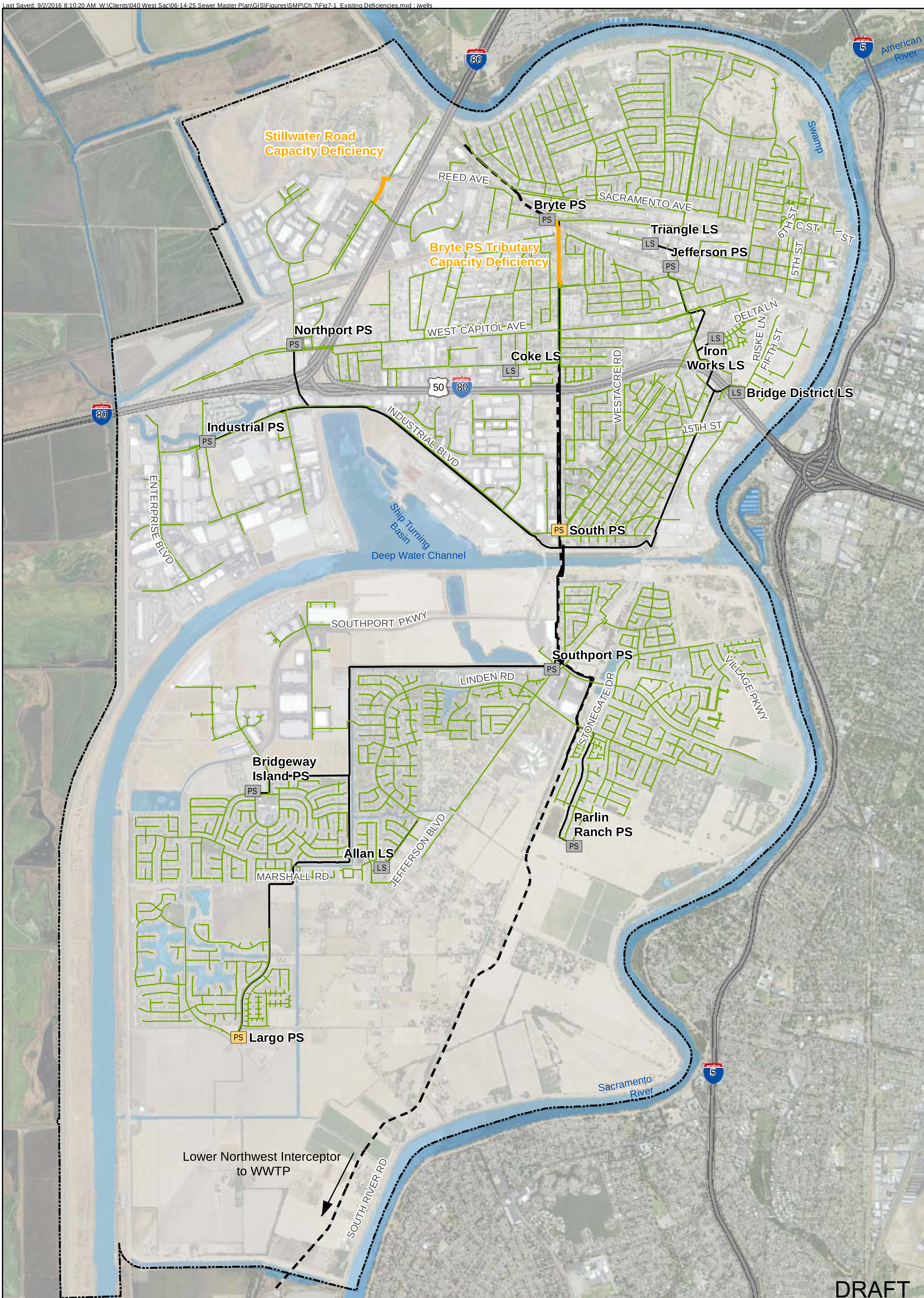

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations with Sufficient Capacity
- PS Pump Station with Capacity Deficiency
- Lower Northwest Interceptor
- Force Main

Gravity Mains

- Sufficient Capacity
- Capacity Deficiency

0 0.25 0.5
Miles

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 7-1

Capacity Deficiencies Existing Design Flow

City of West Sacramento
2015 Sewer
Master Plan Update

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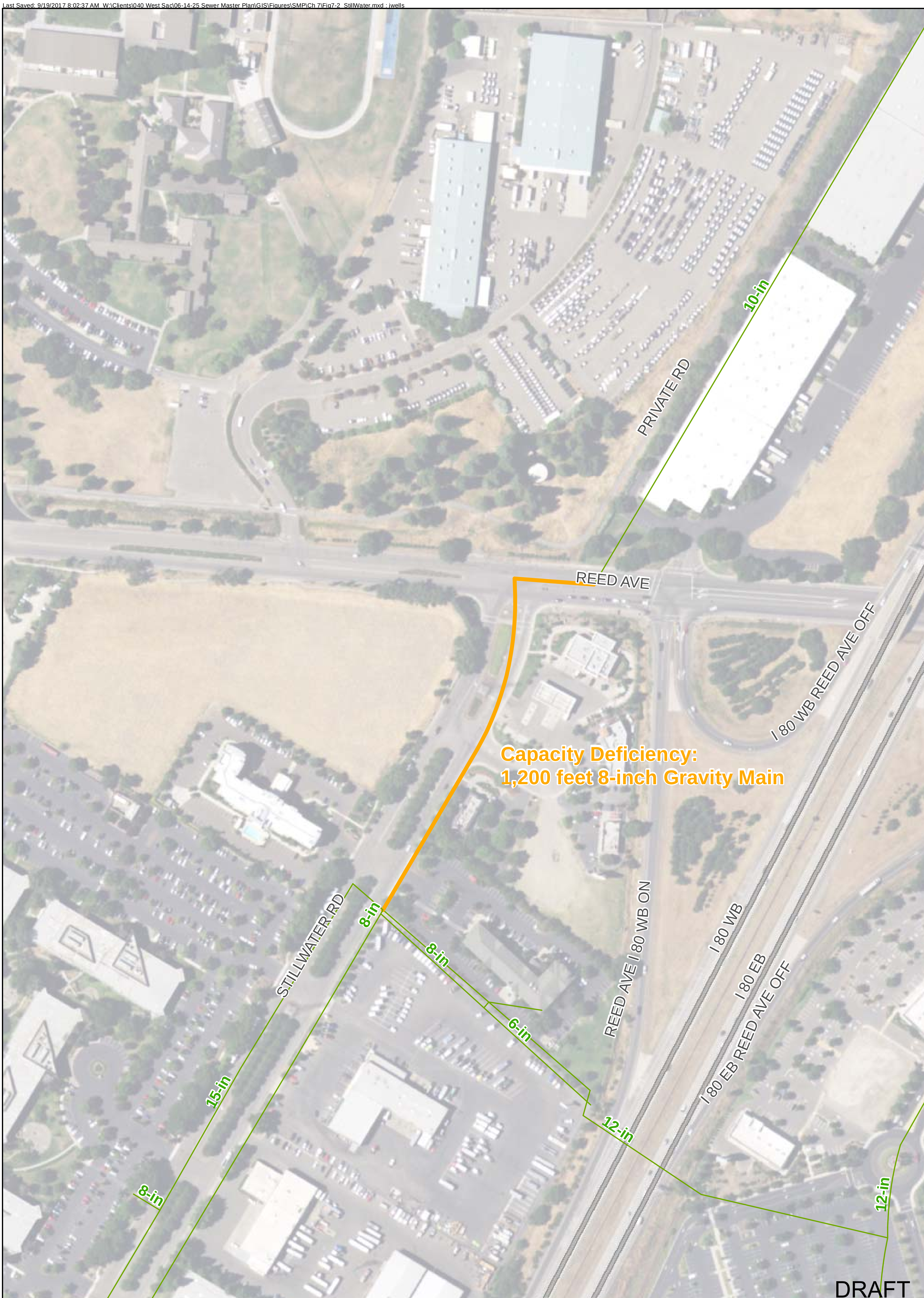

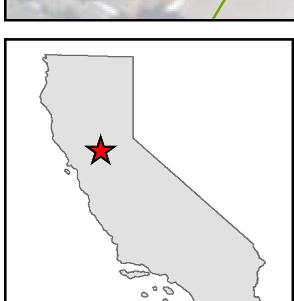

Symbology
- City Boundary
- LS Lift Stations
- PS Pump Stations with Sufficient Capacity
- PS Pump Station with Capacity Deficiency
- Lower Northwest Interceptor
- Force Main

Gravity Mains
- Sufficient Capacity
- Capacity Deficiency

N

0 125 250
Feet

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 7-2

Stillwater Road Capacity Deficiency

City of West Sacramento
2015 Sewer
Master Plan Update

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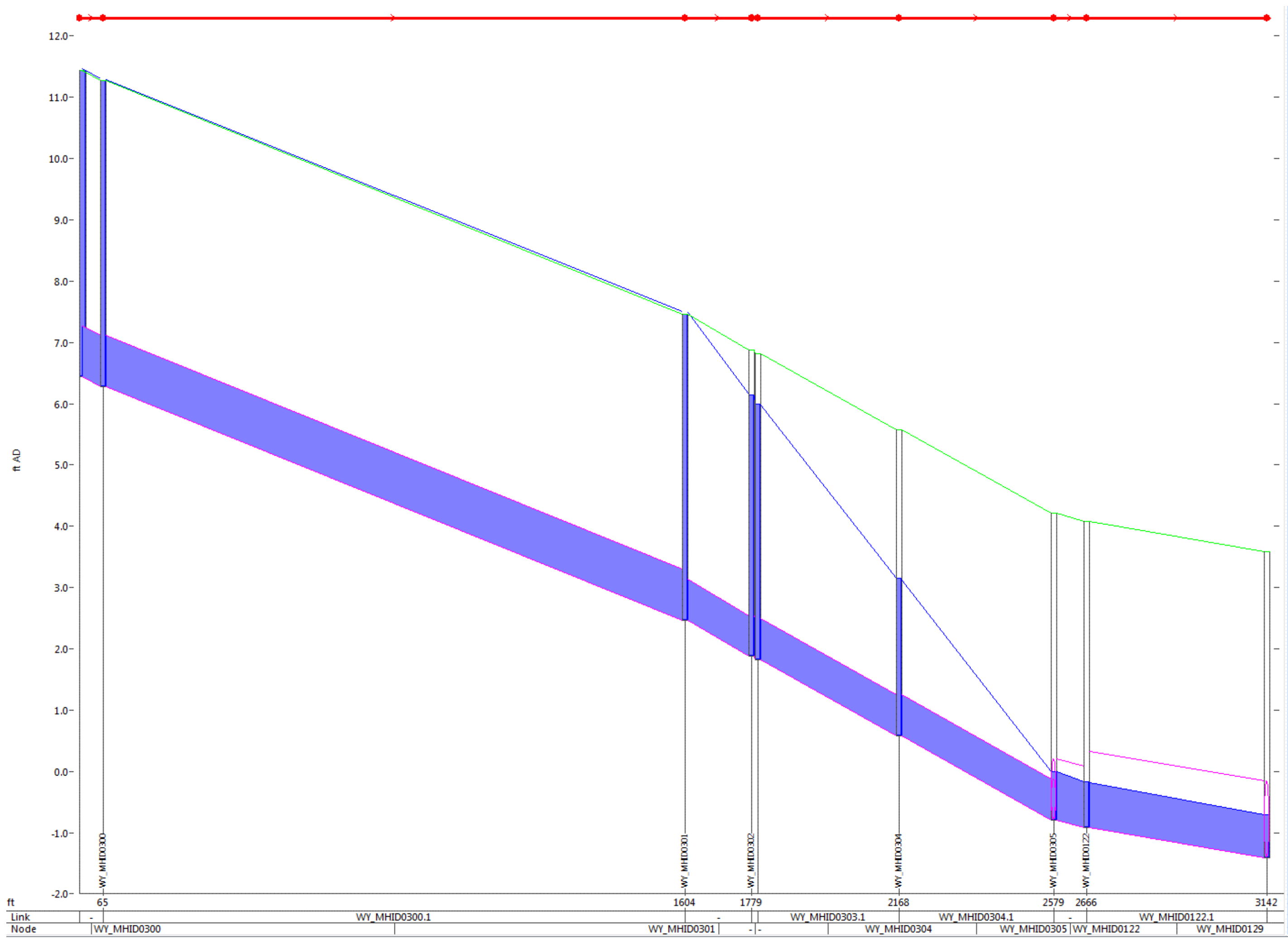

Figure 7-3

Stillwater Road Hydraulic Profile

City of West Sacramento
2015 Sewer Master Plan Update

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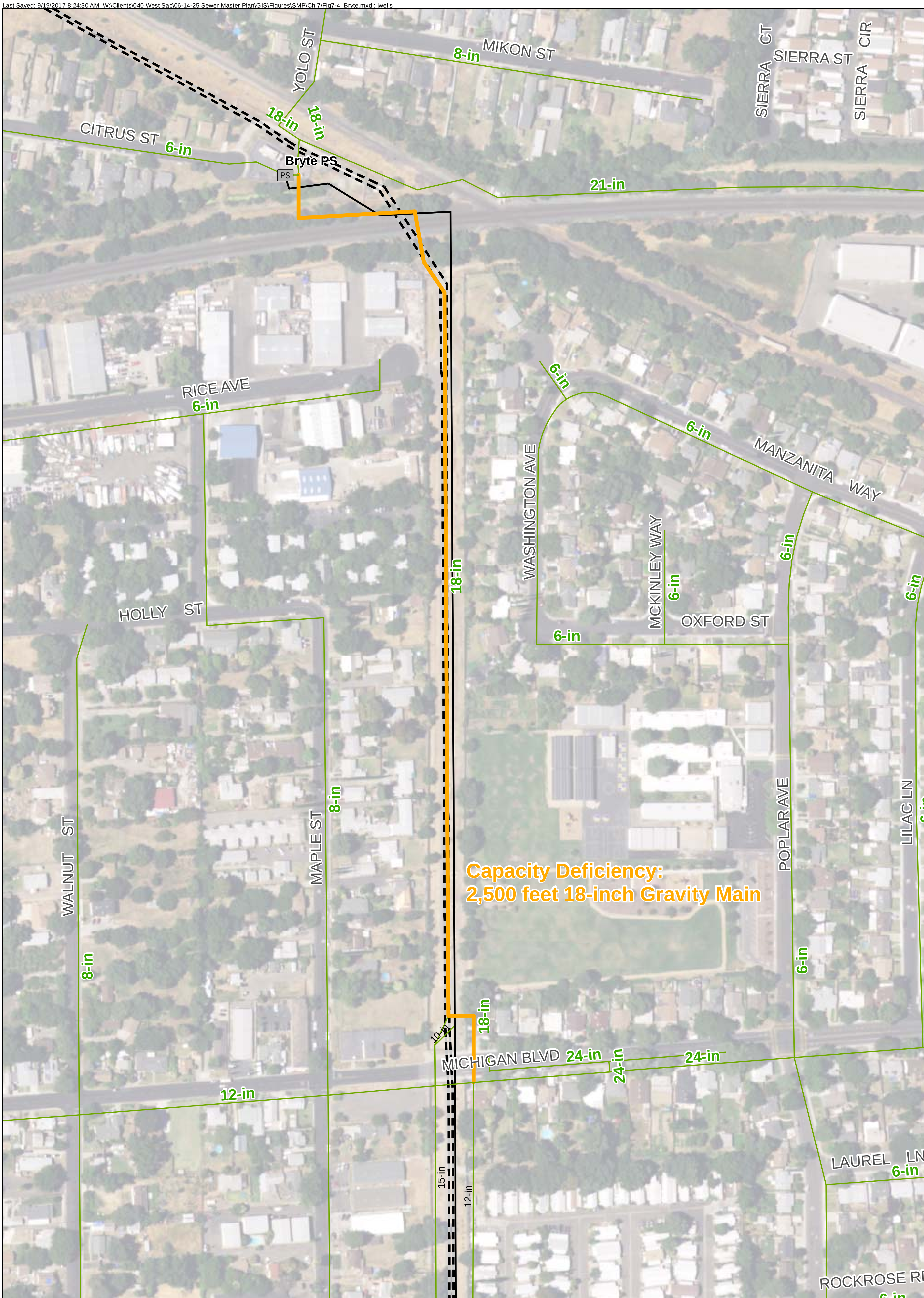

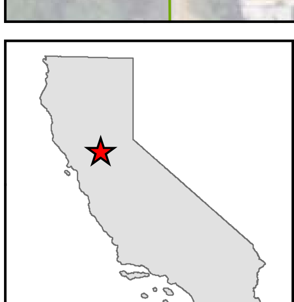

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations with Sufficient Capacity
- PS Pump Station with Capacity Deficiency
- Lower Northwest Interceptor
- Force Main

Gravity Mains

- Sufficient Capacity
- Capacity Deficiency

N

0 125 250
Feet

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

DRAFT

Figure 7-4

Bryte PS Tributary Capacity Deficiency

City of West Sacramento
2015 Sewer
Master Plan Update

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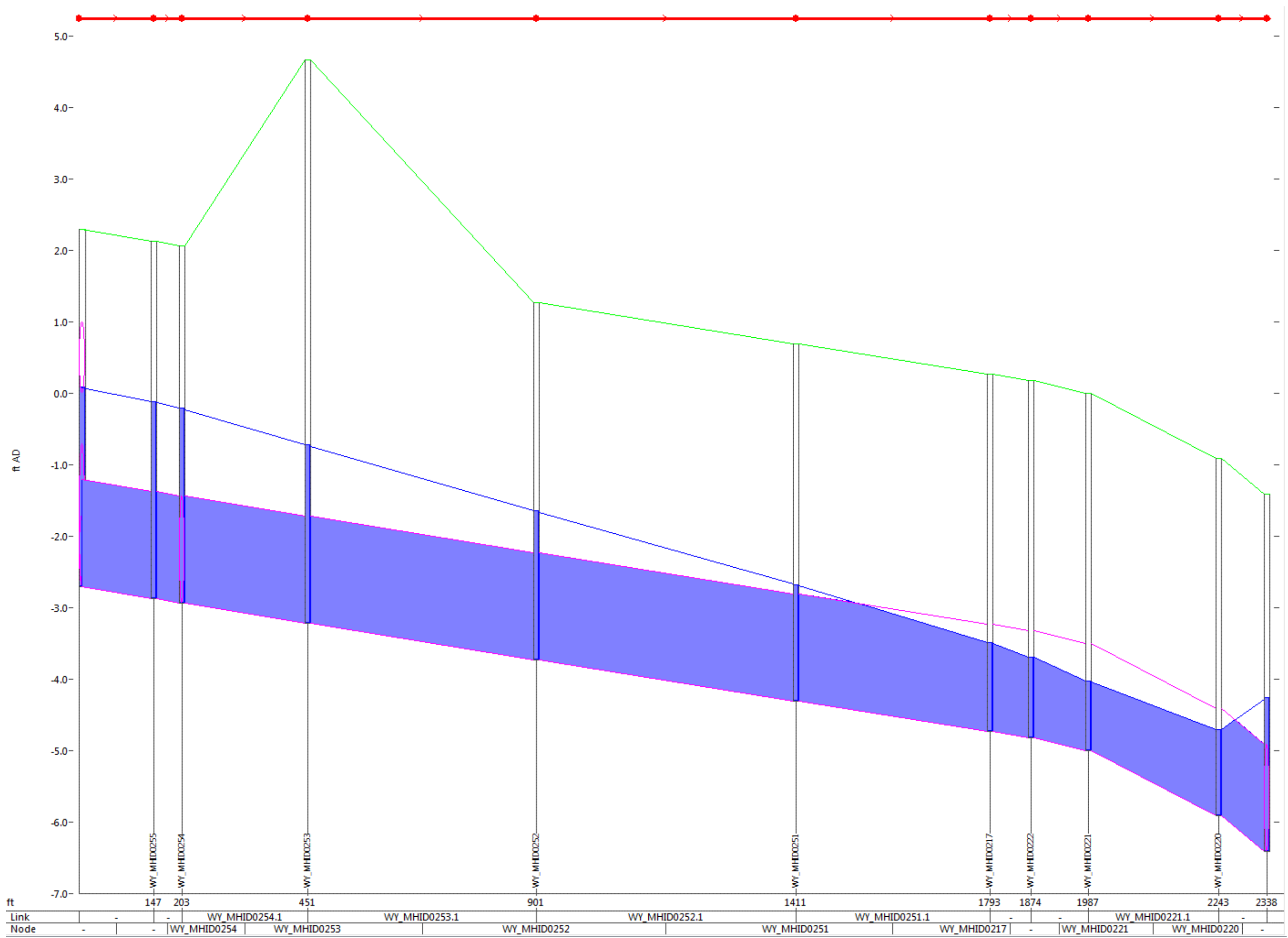

Figure 7-5

Bryte PS Tributary Hydraulic Profile

City of West Sacramento
2015 Sewer Master Plan Update

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7.2 EXISTING LIFT STATION/PUMP STATION HYDRAULIC EVALUATION

As described in Chapter 3, the City's performance standards require that all collection system lift stations/pump stations have sufficient capacity to convey design flows with the largest pump out of service, defined as the "firm capacity" of the lift station/pump station. Each existing lift station/pump station's firm capacity was compared to the existing PWWF design flow conveyed to that station. If the design flow was greater than the lift station/pump station's firm capacity, then the station was considered to have insufficient capacity. The majority of the collection system lift stations/pump stations currently have sufficient firm capacity to convey existing design flows; however, the hydraulic model indicates that there are two stations that lack this capacity under existing conditions. The lift stations that have insufficient firm capacity to convey existing design flows can be seen in Table 7-1. These lift stations are also presented on Figure 7-1. Capacity improvements for these lift stations are proposed in Chapter 10.

Table 7-1. Lift Stations/Pump Stations Not Meeting Performance Criteria Under Existing Conditions

Pump/Lift Station Name	No. of Pumps	Firm Capacity[a], gpm	PWWF Design Flow, gpm
Largo Pump Station	2	450	850
South Pump Station	3	764	1,000

[a] Firm Capacity is defined as the capacity of the pump/lift station with the largest pump out of service.

7.3 EXISTING FORCE MAIN HYDRAULIC EVALUATION

As described in Chapter 3, force main design criteria state that force mains must convey design PWWF at less than 8 fps. There are no force mains that fail to meet the City's performance criteria under existing conditions.

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Chapter 8 presents the results of the hydraulic evaluation of the City's collection system under future design conditions. These future conditions include the development within the City anticipated by the 2035 General Plan Update, as well as expansion of the collection system to serve areas of the City currently utilizing septic service. Collection system capacity for gravity mains, lift station/pump stations, and force mains is assessed with respect to the system's performance under the future PWWF design flow condition described in Chapter 6 using the criteria described in Chapter 3.

8.1 EXISTING COLLECTION SYSTEM CONFIGURATION CAPACITY ANALYSIS

The following sections describe the capacity analysis performed for the existing collection system configuration.

8.1.1 Existing Gravity Main – 2035 Flows Hydraulic Evaluation

With the addition of the flows projected by 2035, existing gravity mains exceed the performance criteria under existing design flows in a number of locations. The specific reason that the gravity main or group of gravity mains fails to meet the performance criteria can vary from being undersized to lacking sufficient slope at a particular location. These specific reasons, and the remedies to address them, are discussed more in Chapter 10. The gravity mains that fail to meet performance criteria under future 2035 conditions are displayed on Figure 8-1. These gravity main deficiencies are summarized below.

8.1.1.1 Hardy Drive Capacity Deficiency

Hardy Drive runs north/south in the northeastern portion of the City. It is found north of Sacramento Avenue (6th Street). Two stretches of gravity main can be found in Hardy Drive. The western stretch is part of the local collector system that serves the Bryte/Broderick Neighborhood. The eastern stretch of gravity main conveys wastewater out of the western portion of the Lighthouse Neighborhood, and to the Bryte Pump Station. As described in Chapter 5, the 2035 General Plan Update projects over 800 households and 200 employees being added to the Lighthouse Neighborhood. A large portion of the wastewater generated by these new developments will be conveyed down the eastern gravity main stretch in Hardy Drive. These gravity mains have sufficient capacity under existing conditions, but are deficient with the 2035 development wastewater added. The deficient gravity mains are found between Lighthouse Drive and Fremont Boulevard. The location of the deficiency can be seen on Figure 8-2. The hydraulic profile of this section of gravity main can be seen on Figure 8-3. As shown, the surcharging in the 6-inch gravity main leads to projected surcharging within two feet of the ground elevation under existing PWWF design conditions. The hydraulic model indicates that the City should upsize 2,400 feet of 6-inch gravity main to 15-inch gravity main along this stretch of Hardy Drive. The project to complete this capacity increase is described in more detail in Chapter 10.

8.1.1.2 Iron Works Lift Station Tributary Capacity Deficiency

The Iron Works Lift Station is found in the eastern section of the City, north of the Deep Water Channel. This lift station is in the Bridge District, formerly called the Triangle Area. The Iron Works Lift Station serves the western portion of the neighborhood, and the new Bridge District Lift Station serves the eastern portion of the neighborhood. The Bridge District is projected to develop heavily in the 2035 General Plan Update, with over 3,000 households and 6,000 employees projected by 2035. Although most of the new wastewater flow generated by this development will be tributary to the Bridge District Lift Station, a portion of it will be tributary to the Iron Works Lift Station. The current gravity main tributary to the Iron Works Lift Station is 8-inches in diameter. This area can be seen on Figure 8-4.

The Iron Works LS and the gravity mains that are tributary to it were designed to be privately owned, serving only the Iron Works development. However, field survey has confirmed that at the time that the Iron Works development was constructed, the development to the northeast of the Iron Works development was connected to the sanitary sewer system tributary to the Iron Works Lift Station. The flow from this development along Delta Lane was previously turned to the north, where it crossed the Tower Bridge Gateway and connected to gravity mains that are tributary to the Jefferson Lift Station.

The connection of the development along Delta Lane to the Iron Works tributary collection system has several implications for the City's collection system. The first is that with the addition of these flows, the gravity main in Chromium Lane and the Iron Works Lift Station itself are no longer private infrastructure. The City will need to look into obtaining ownership of these assets and into obtaining easement rights for maintenance access if flows from Delta Lane continue to flow into the Iron Works tributary collection system.

The second implication is that continued development in the eastern portion of Delta Lane results in the gravity main in Chromium Lane being insufficient for 2035 design flows. The capacity in this gravity main is insufficient for 2035 design PWWF conditions, as shown in the hydraulic profile on Figure 8-5. As shown, the hydraulic model does not predict overflows under existing PWWF design conditions, but it predicts manhole surcharging to within approximately two feet of the ground elevation, violating the City's design criteria. The hydraulic model indicates that the City should upsize 1,000 feet of 8-inch gravity main to 10-inch gravity main upstream of the Iron Works Lift Station. The project to complete this capacity increase is described in more detail in Chapter 10. Alternatively, the City can redirect flow from Delta Lane north across the Tower Bridge Gateway along West Capitol Avenue, and connect to the gravity mains in West Capitol Avenue. These gravity mains and the Jefferson Pump Station to which they are tributary have sufficient capacity to handle these flows.

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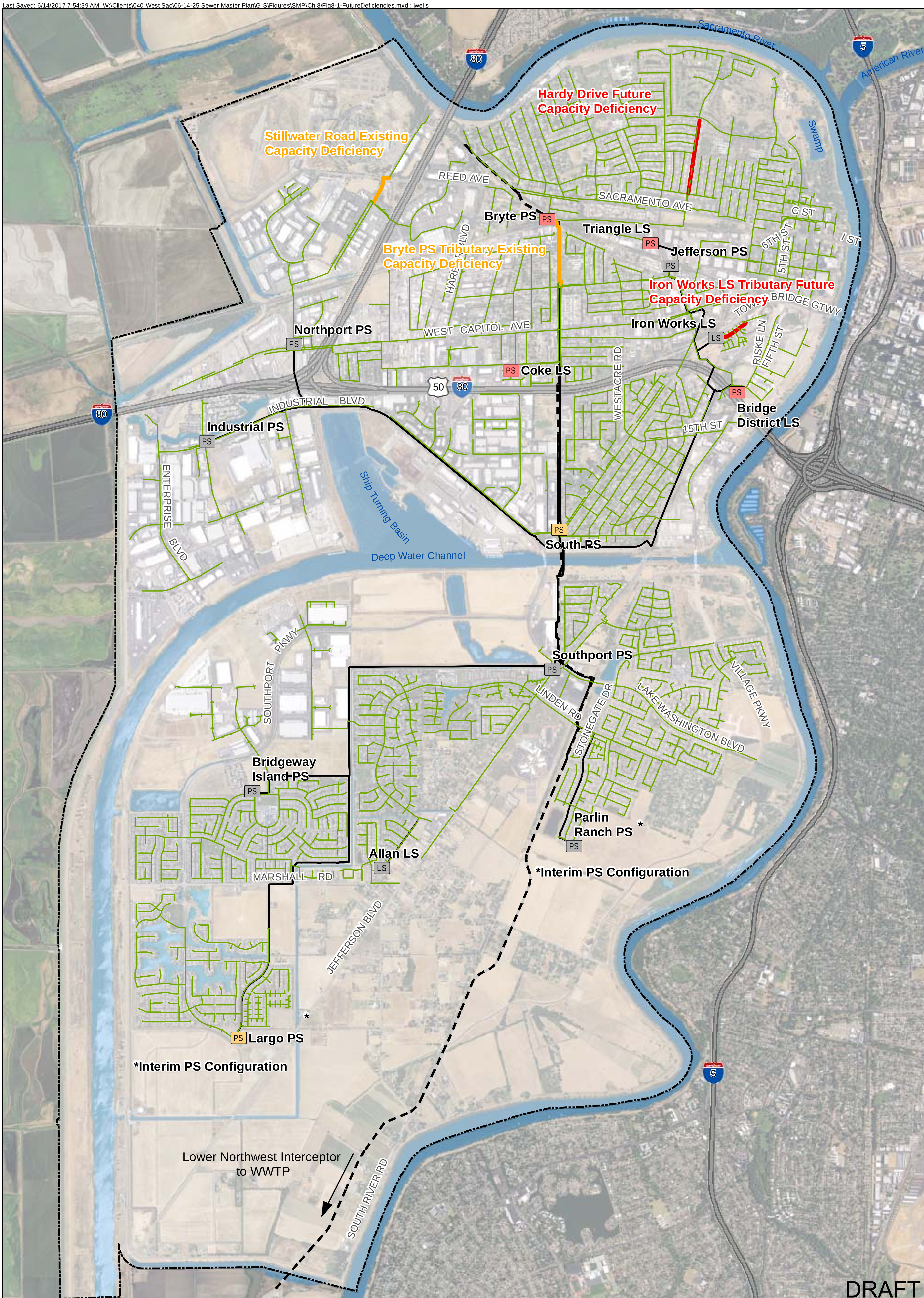

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations with Sufficient Capacity
- PS Pump Stations with Insufficient Capacity - Existing
- PS List Stations with Insufficient Capacity - Future
- Lower Northwest Interceptor
- Force Main

Gravity Mains

- Sufficient Capacity
- Existing Capacity Deficiency
- Future Capacity Deficiency

N

0 0.25 0.5
Miles

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 8-1

Capacity Deficiencies Future Design Flow

City of West Sacramento
2015 Sewer
Master Plan Update

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Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations with Sufficient Capacity
- PS Pump Stations with Insufficient Capacity - Existing
- PS List Stations with Insufficient Capacity - Future
- Lower Northwest Interceptor
- Force Main

Gravity Mains

- Sufficient Capacity
- Existing Capacity Deficiency
- Future Capacity Deficiency

N

0 125 250
Feet

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Figure 8-2

Hardy Drive Deficiency Future Design Flow

City of West Sacramento
2015 Sewer
Master Plan Update

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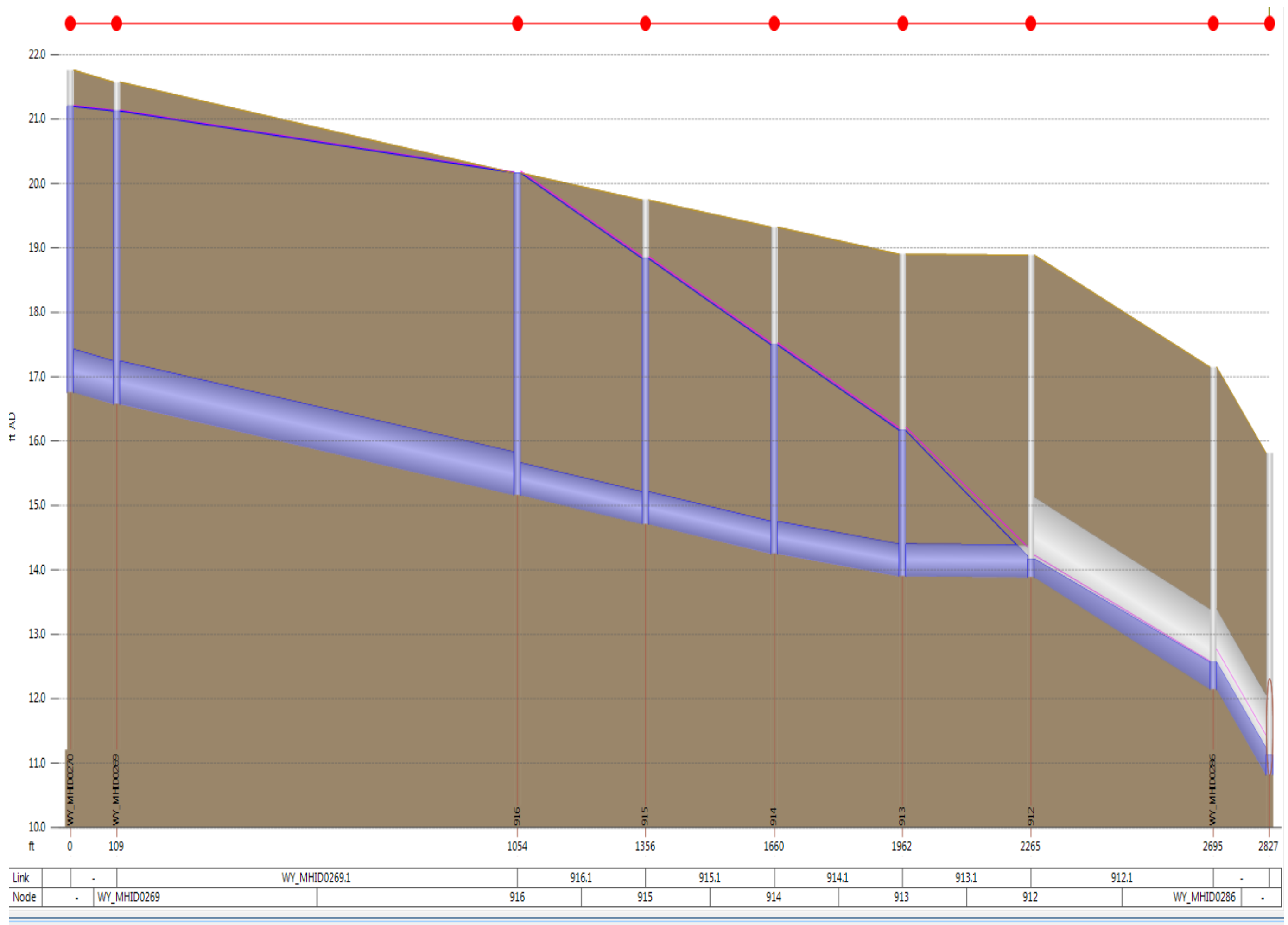

Figure 8-3

Hardy Drive
Hydraulic Profile

City of West Sacramento
2015 Sewer Master Plan Update

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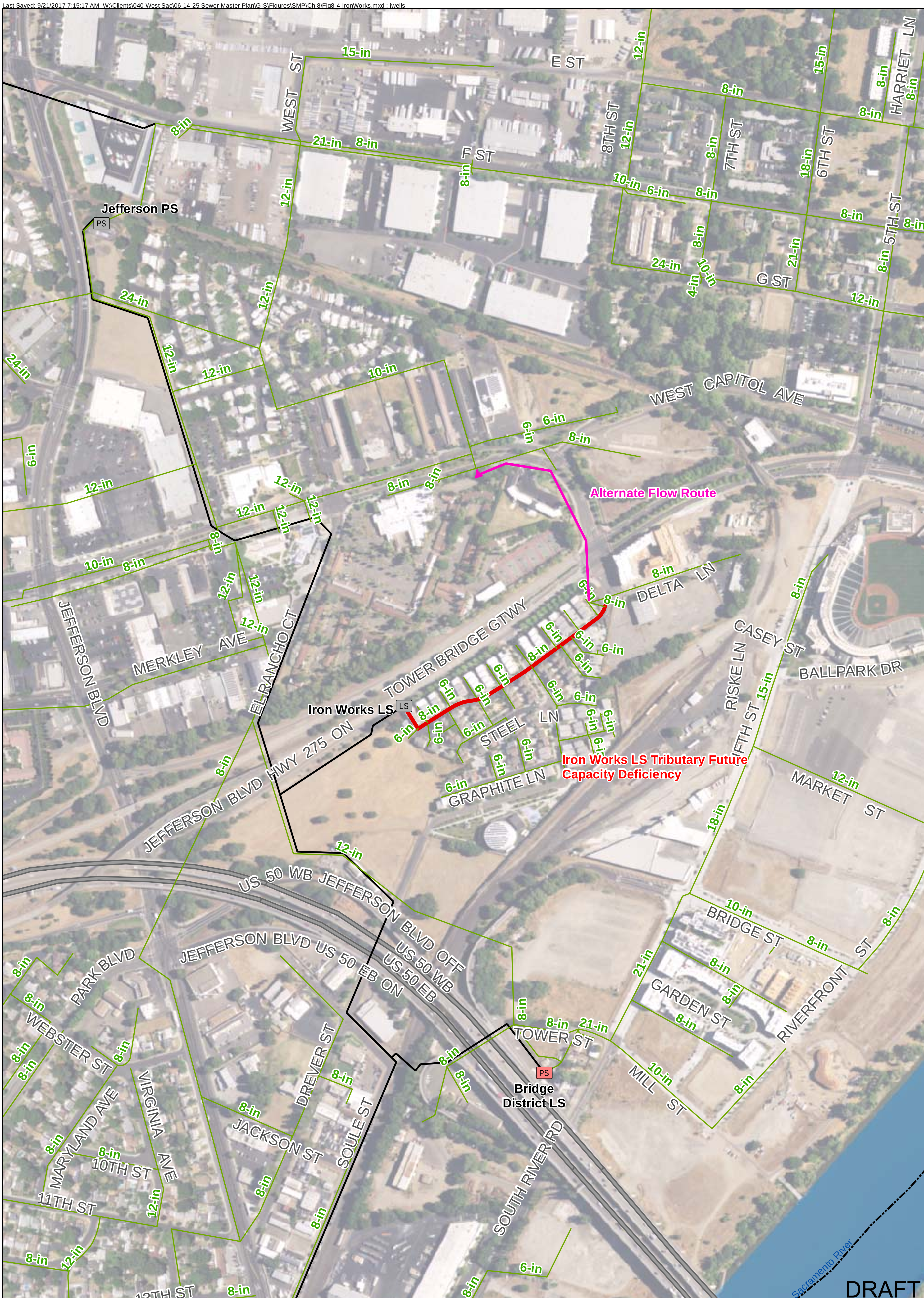

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations with Sufficient Capacity
- PS Pump Stations with Insufficient Capacity - Existing
- PS List Stations with Insufficient Capacity - Future
- Lower Northwest Interceptor
- Force Main

Gravity Mains

- Sufficient Capacity
- Existing Capacity Deficiency
- Future Capacity Deficiency

N

0 250 500

Feet

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Figure 8-4

Iron Works Tributary Deficiency Future Design Flow

City of West Sacramento
2015 Sewer
Master Plan Update

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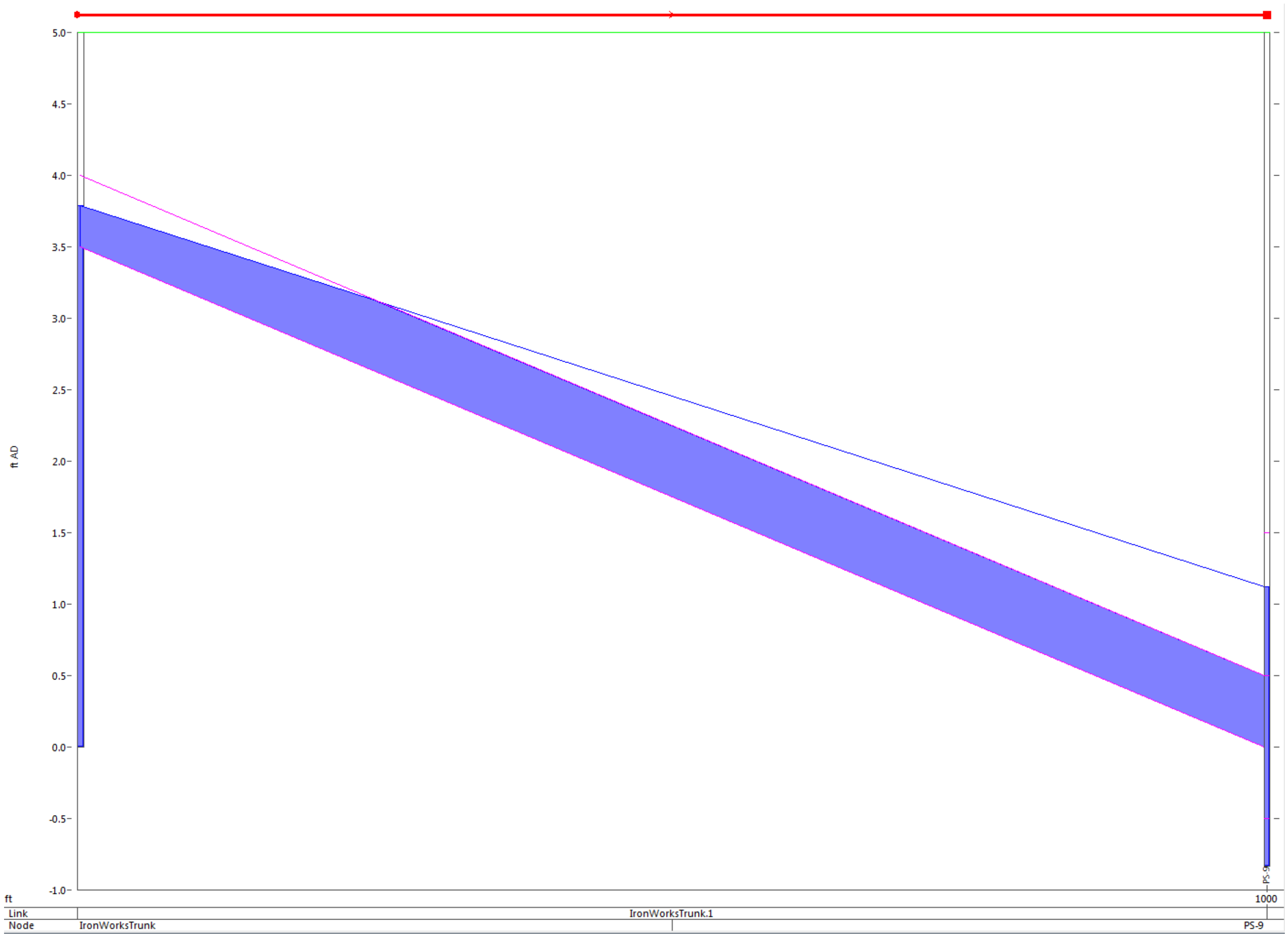

Figure 8-5

Iron Works LS Tributary Hydraulic Profile

City of West Sacramento
2015 Sewer Master Plan Update

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8.1.2 Future 2035 Lift Station/Pump Station Hydraulic Evaluation

As described in Chapter 3, the City's performance standards require that all collection system lift stations/pump stations have sufficient capacity to convey design flows with the largest pump out of service, defined as the "firm capacity" of the lift station/pump station. Each existing lift station/pump station's firm capacity was compared to the existing PWWF design flow conveyed to that station. If the design flow was greater than the lift station/pump station's firm capacity, then the station was considered to have insufficient capacity. The majority of the collection system lift stations/pump stations currently have sufficient firm capacity to convey future 2035 PWWF design flows; however, the hydraulic model indicates that there are four stations that lack this capacity, in addition to the two stations that were identified to have insufficient capacity under existing conditions. The lift stations that have insufficient firm capacity to convey future design flows can be seen in Table 8-1. These lift stations are also presented on Figure 8-1.

Table 8-1. Lift Stations/Pump Stations Not Meeting Performance Criteria Under Future 2035 Conditions

Pump/Lift Station Name	No. of Pumps	Firm Capacity[a], gpm	PWWF Design Flow, gpm
Largo Pump Station	2	450	1,000
South Pump Station	3	764	1,050
Bridge District Lift Station	3	1,213	1,500
Bryte Pump Station	3	3,400	3,800
Triangle Lift Station	2	137	200
Coke Lift Station	2	200	300

[a] Firm Capacity is defined as the capacity of the pump/lift station with the largest pump out of service.

8.1.3 Future 2035 Force Main Hydraulic Evaluation

There are no force mains that fail to meet the City's performance criteria under future design PWWF conditions.

8.2 FUTURE COLLECTION SYSTEM CONFIGURATION CAPACITY ANALYSIS

The previous sections describe the capacity analysis performed using the hydraulic model and the projected future flows on the City's existing collection system configuration. The existing collection system configuration is not expected to change significantly north of the Deep Water Channel. However, south of the Deep Water Channel there is the possibility of significant alteration of the existing collection system configuration. This potential alteration is the result of

- Existing infrastructure that is operating in a planned interim configuration and that will be transitioned to the planned final configuration,
- New development in currently undeveloped areas that will require new infrastructure for wastewater service, and

- A limited number of parcels that are currently on septic service and that will be transitioned to service by the City's collection system.

To evaluate potential future collection system configurations, the area south of the Deep Water Channel was divided into 17 areas. These areas are similar to the 12 areas identified in the 2003 Southport Sanitary Sewer Master Plan, but have been adjusted or split where necessary because of subsequent development. The 17 areas south of the Deep Water Channel can be seen on Figure 8-6. The future and potential future collection system configurations are described below by area. Detailed maps for each area can be seen in *Appendix F: Southport Infrastructure Recommendations*.

8.2.1 Area 1a: Upper Southwest Village and Rural Core

Area 1a is found the southwest portion of the City. The topography is generally low and flat, which necessitated the construction of the Largo Pump Station. The Largo Pump Station currently pumps north all the way to the common manifold near Southport Pump Station. This interim configuration is to be replaced by a more direct connection to the LNWI at Bevan Road. This future force main configuration along Bevan Road is shown on Figure 8-7 and in Appendix F.

The Southwest Village is predicted to have significant residential growth by 2035. The majority of this growth is encapsulated by the Yarborough Development. The northern portion of the Yarborough Development is within Area 1a and will be tributary to the Largo Pump Station. The gravity mains identified in the Yarborough Development preliminary utility plan that will convey flow from this portion of the development to the pump station are shown on Figure 8-7. These gravity mains are not included in the City's CIP because they are assumed to be the responsibility of the Yarborough Development. The remaining majority portion of the Yarborough Development is in Area 1b and will be discussed below.

The Rural Core Neighborhood is predicted to have almost no growth, either residential or non-residential, between now and 2035 as determined by the General Plan. However, there are existing rural residential parcels in this area that are currently developed but served by septic systems. Because some of these parcels less than 2.5 acres, the City may be forced to provide service to them due to changing water quality regulations. Therefore, the potential gravity trunk mains that would be required to serve the Rural Core are shown on Figure 8-7 and in Appendix F in Jefferson Boulevard and Davis Road. These gravity mains are sized assuming that all proximate parcels would be served by the City.

8.2.2 Area 1b: Lower Southwest Village

Area 1b is found in the far southwestern corner of the City. This area is composed entirely of the lower and eastern portion of the Yarborough Development that is not tributary to the Largo Pump Station in Area 1a. As identified by the Yarborough Development preliminary utility plan, flow from this area will be conveyed to a trunk gravity main in Burrows Avenue that will convey the flow to a new connection to the LNWI. This gravity main, and all of the gravity mains within the Yarborough development, are not included in the City's CIP because they are assumed to be the responsibility of the developer.

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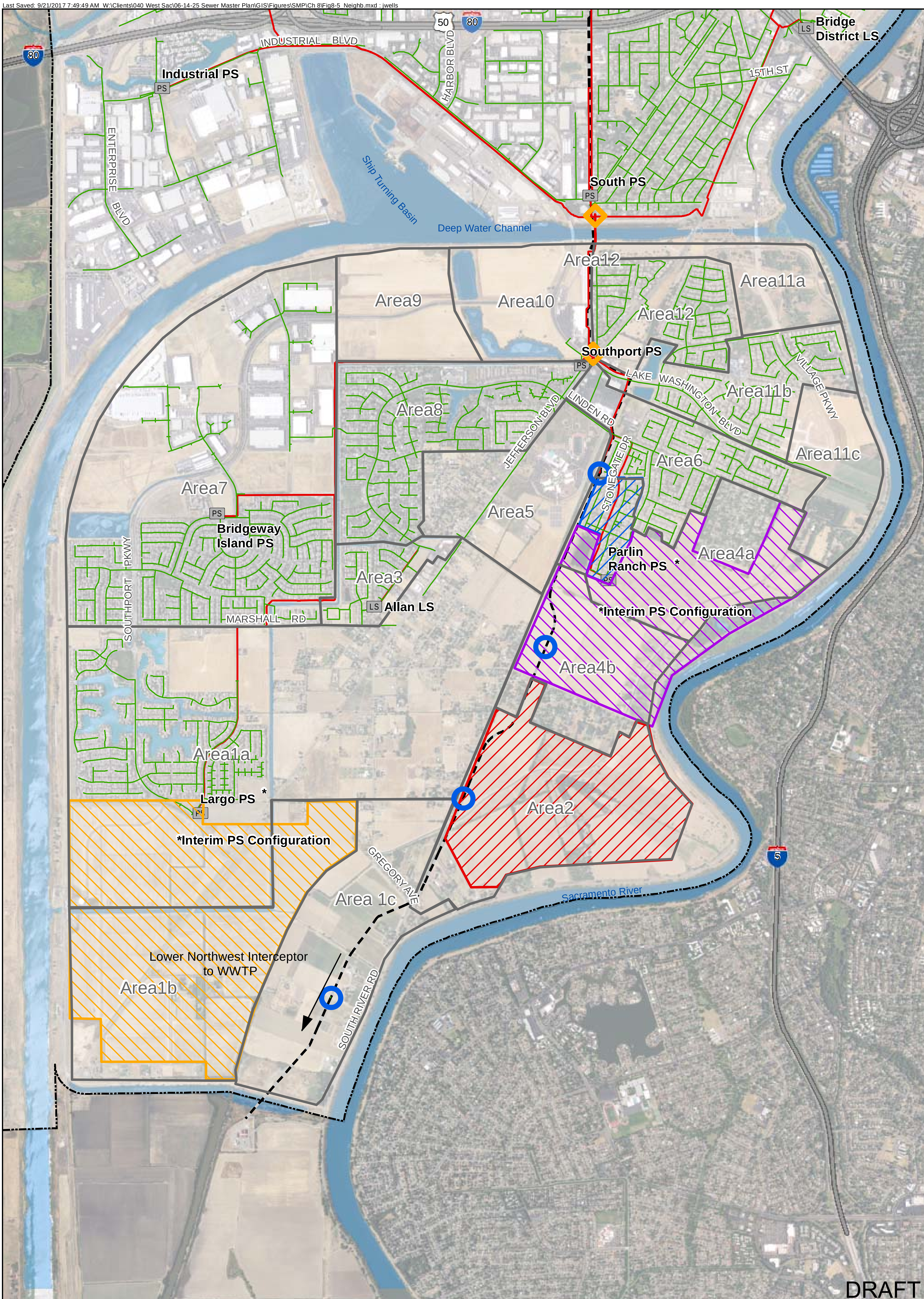

Symbology

- Force Main Structure
- LNWI Connection
- Drainage Area

Development Areas

- Parlin Ranch
- Liberty Plan
- River Park
- Yarborough
- City Boundary

- LS Lift Stations
- PS Pump Stations
- Gravity Main
- Force Main
- Lower Northwest Interceptor

0 0.25 0.5
Miles

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WEST YOST ASSOCIATES

Figure 8-6

Southport Tributary Areas

City of West Sacramento
2015 Sewer
Master Plan Update

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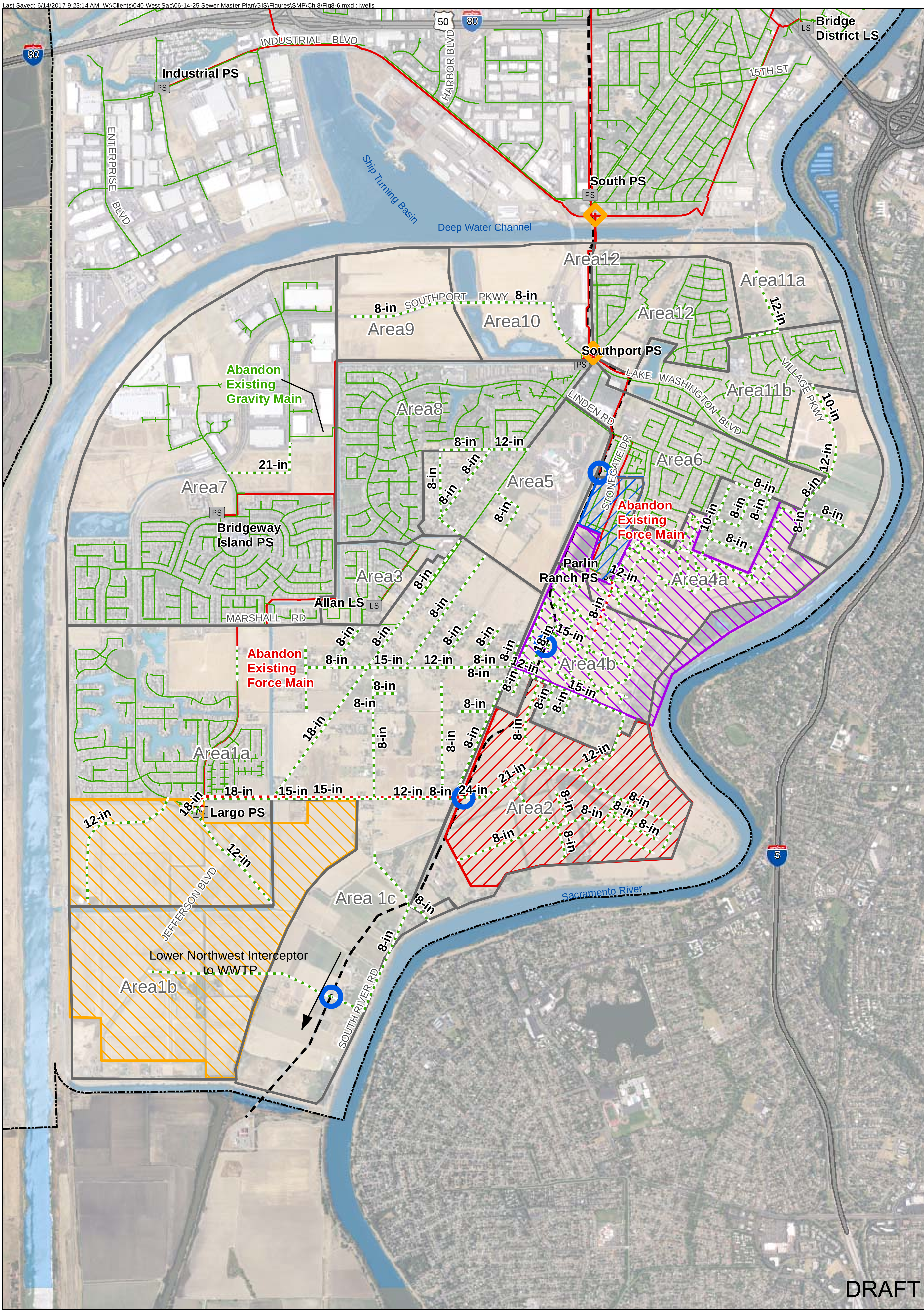

Symbology

- Force Main Structure
- LNWI Connection
- Drainage Area

Development Plans

- Parlin Ranch
- Liberty Plan
- River Park
- Yarborough
- City Boundary

- LS Lift Stations
- PS Pump Stations
- Proposed Gravity Main
- Proposed Force Main
- Gravity Main
- Force Main
- Lower Northwest Interceptor

0 0.25 0.5
Miles

Figure 8-7

Future Collection System Configuration

City of West Sacramento
2015 Sewer
Master Plan Update

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8.2.3 Area 1c: Southeast City

Area 1c is found in the far southeastern corner of the City. There is not expected development in this area by 2035. Although there are no small parcels that would require City service, flow could be connected by gravity to the Burrows Avenue connection of the LNWI.

8.2.4 Area 2: Southeast Village

The Southeast Village can be found in the southeastern corner of the City, east of where Bevan Road intersects the LNWI. This area will be occupied almost entirely by the River Park Development. The River Park Development preliminary utility plan indicates that the development will be served by gravity mains that will convey flow to the new LNWI connection at Bevan Road. The layout of these gravity mains as identified by the preliminary utility plan are shown on Figure 8-7 and in Appendix F. These gravity mains are not included in the City's CIP because they are assumed to be the responsibility of the developer.

8.2.5 Area 3: Allan Lift Station Basin of Northwest Village

The southeast corner of the Northwest Village Neighborhood is hydraulically isolated and served by the Allan Lift Station, which pumps flow to the northeast from this corner to a gravity main in Linden Road. While there are a small number of parcels that remain undeveloped in this area, the collection system infrastructure is adequately sized for future flows in this area. The 2003 Southport Sanitary Sewer Master Plan recommended that the Allan Lift Station be removed from service, and flow from this area be conveyed by gravity to the gravity main recommended in Jefferson Boulevard as part of Area 1a. The City can still accomplish this configuration change assuming that the gravity main in Jefferson Boulevard is constructed deep enough. However, this Master Plan assumes that flow from the Allan Lift Station will continue to go north to Linden Road.

8.2.6 Area 4a: Parlin Ranch Lift Station Basin

Area 4 as identified by the 2003 Southport Sanitary Sewer Master Plan has had boundaries adjusted and has been split into two areas based upon the development plan that has been established for the Liberty Development. Area 4a is the northern portion of the Liberty Development that can be made tributary to the Parlin Lift Station, as well as the current Parlin Ranch development that is tributary to the Parlin Lift Station.

Parlin Lift Station is currently operating in an interim condition, with its force main going north and discharging to a gravity main in Linden Road. The lift station is serving the western portion of Area 4a under the current interim condition. Also, as an interim facility the Parlin Lift Station is currently configured with two pumps. In the recommended final configuration, the Parlin Lift Station will serve all of Area 4a using the gravity main alignments identified in the Liberty Development preliminary utility study. The existing force main will be abandoned, and a new force main will be routed south through the Liberty Development to a gravity main in the southern portion of the Liberty Development (Area 4b). Flow will be conveyed from here to a new connection made under gravity conditions to the LNWI at a connection just north of Davis Road. This layout can be seen on Figure 8-7.

In addition to the force main reconfiguration, the Parlin Lift Station should be updated to a permanent condition with three pumps to provide operational redundancy and reliability. The gravity mains in Area 4a, as well as the Parlin Lift Station reconfiguration, are assumed to be the responsibility of the developer and are not included in the City's CIP.

8.2.7 Area 4b: Southern Liberty and Southeast Village

Area 4b is south of area 4a and is cut by Davis Road. The Liberty Development extends south through the area to Davis Road. The Liberty Development will be served by a gravity sewer system, as identified in the Liberty Development preliminary utility plan. This gravity sewer system will discharge the flow to the new connection to the LNWI near Davis Road. The discharge from the reconfigured Parlin Lift Station force main will enter this gravity system just north of Davis Road. The gravity mains that serve the Liberty Development are assumed to be the responsibility of the developer are not included in the City's CIP.

The portion of Area 4b which is south of Davis Road is in the Southeast Village Neighborhood. Because the Southeast Village is not predicted to have any growth outside of the River Park Development, the portion of Area 4b south of Davis Road is not predicted to need infrastructure for development. However, several parcels in this area contain existing development that is served by septic systems on parcels less than 2.5 acres in area. If the City is required to provide service to such parcels, these parcels can flow by gravity north to Davis Road. The gravity main required for such service is shown on Figure 8-7 and in Appendix F.

8.2.8 Area 5: Central Core

Area 5 is located in the center of the area south of the Deep Water Channel, and encompasses a portion of the Northeast Village, a portion of the Northwest Village, and a portion of the Rural Core neighborhoods. The portion of the Northeast Village included in Area 5 encompasses the River City High School, which will not see significant growth in flows. Neither the Northwest Village nor the Rural Core portions are projected to see significant development either. Therefore, there is projected to be little increase in flows from Area 5 between existing conditions and 2035. The existing gravity main in Jefferson Boulevard has sufficient capacity to serve the area.

There are a number of parcels west of Jefferson Boulevard that have existing development served by septic systems on a parcel less than 2.5 acres in area. If the City is required to provide sewer service to these parcels, it is assumed that it will provide service to all proximate parcels. The gravity mains that would be required to serve these parcels can be seen in Hart Avenue, Allan Avenue, Higgins Road, and Blacker Road on Figure 8-7 and in Appendix F.

8.2.9 Area 6: East Linden Road

Area 6 can be found along Linden Road in the eastern portion of the City, but west of Village Parkway. The area is north of Parlin Ranch and the Liberty Development. There is little development projected for this area between existing conditions and 2035. The existing collection system infrastructure is sufficient for future flows. However, at the eastern edge of Area 6, south of Linden Road, there an existing neighborhood along Bastone Court, Redwood Avenue, Alder Way, Tamarack Road, and Birch Way that does not have sewer service. There are several parcels less than 2.5 acres in area, and the City may be required to provide sewer service to these parcels.

In such a case, the neighborhood generally flows toward Linden Road. As shown on Figure 8-7 and in Appendix F, the gravity main in Linden Avenue can be extended to the east in order to accommodate these parcels.

There is an undeveloped area at the eastern portion of Area 6 that could be served along Linden Road. However, it could also be served by connecting through the Liberty Development. These alternatives are shown in Appendix F.

8.2.10 Area 7: SIP and Northwest Village

Area 7 can be found in the northwest corner of the City area south of the Deep Water Channel. This area contains all of the SIP neighborhood and a portion of the Northwest Village. The SIP neighborhood is expected to see significant non-residential growth by 2035. The collection system infrastructure in Area 7, including the Bridgeway Island Pump Station, is sufficient for future flows. Currently the northeastern corner of Area 7 is served by a gravity main that flows to the east and intersects the gravity main in Linden Road. This configuration is identified as an interim condition. The gravity main required to allow this corner to flow to the Bridgeway Island Pump Station is shown on Figure 8-7 and in Appendix F. This gravity main will represent the final configuration, and the model shows that the infrastructure is sized to handle this configuration.

8.2.11 Area 8: Linden Road in Northwest Village

Area 8 can be found to the northwest of Area 5. There is little development projected in this area, and the collection system infrastructure has sufficient capacity for future flows. This result includes sufficient capacity for the flow entering the gravity main in Linden Avenue from SIP to the west, and flow entering the gravity main from the force main discharging from Allan Lift Station.

8.2.12 Area 9: Western Seaway Neighborhood

Area 9 is found directly south of the Deep Water Channel and encompasses the western portion of the Seaway Neighborhood. The Seaway Neighborhood is expected to experience a small amount of non-residential growth between existing conditions and 2035. There is no infrastructure currently in this area. Gravity mains with diameter of 8-inches are sufficient to carry the small amount of flow projected for the neighborhood. The topography is extremely flat, and small lift stations may be required by individual dischargers depending upon the location of the development.

The proposed gravity mains are shown on Figure 8-7 and in Appendix F. They are not included in the City's CIP because the small amount of development and projected flow indicates that these gravity mains would be the responsibility of the developer.

8.2.13 Area 10: Eastern Seaway Neighborhood

Area 9 is found directly south of the Deep Water Channel and encompasses the eastern portion of the Seaway Neighborhood. The Seaway Neighborhood is expected to experience a small amount of non-residential growth between existing conditions and 2035. There is no infrastructure currently in this area. Gravity mains with diameter of 8-inches are sufficient to carry the small amount of flow projected for the neighborhood. The topography is extremely flat, and small lift stations may be required by individual dischargers depending upon the location of the development.

The proposed gravity mains are shown on Figure 8-7 and in Appendix F. They are not included in the City's CIP because the small amount of development and projected flow indicates that these gravity mains would be the responsibility of a potential developer.

8.2.14 Area 11a: Stone Lock Neighborhood

Area 11 from the 2003 Southport Sanitary Sewer Master Plan has been broken up to account for development that has taken place since that time. Area 11a encompasses the Stone Lock neighborhood, which is projected to see substantial development by 2035, including potential development by the Port. The required gravity mains for this development in Village Parkway and Stonegate Drive are shown on Figure 8-7 and in Appendix F.

8.2.15 Area 11b: Lake Washington Boulevard

Area 11 from the 2003 Southport Sanitary Sewer Master Plan has been broken up to account for development that has taken place since that time. Area 11b encompasses the area along Lake Washington Boulevard. This area has seen significant development since 2003, but is not expected to see significant development in the future. The current infrastructure provides sufficient capacity.

8.2.16 Area 11c: Remaining Newport Estates

Area 11 from the 2003 Southport Sanitary Sewer Master Plan has been broken up to account for development that has taken place since that time. Area 11c encompasses the area at the eastern end of Lake Washington Boulevard. The remaining Newport Estates development is projected to take place in this area. The infrastructure required to serve this development is shown on Figure 8-7 and in Appendix F. The gravity mains are assumed to be responsibility of the developer and are not included in the City's CIP.

8.2.17 Area 12: Northern Jefferson Boulevard

Area 12 is just south of the Deep Water Channel in the eastern portion of the City. Significant development is not expected in this area by 2035, and the current infrastructure has sufficient capacity for future flows.

Previous chapters in the 2015 Sewer Master Plan Update have focused on the hydraulic capacity of the collection system, and the ability of the collection system to convey design flows under existing and projected future conditions. Chapter 9 describes the efforts made to assess the condition of the collection system, as well as the resulting Asset Management Plan to manage and improve the condition of the collection system over time. The major sections of the chapter include:

- Pre-Inspection Risk Assessment for Gravity Mains,
- Pre-Inspection Risk Assessment for Manholes,
- Pre-Inspection Risk Assessment for Lift Stations/Pump Stations,
- Condition Assessment Plan,
- Asset Management Plan Results, and
- Asset Management Plan.

9.1 PRE-INSPECTION RISK ASSESSMENT FOR GRAVITY MAINS

The Asset Management Plan for 2015 Sewer Master Plan Update effort included physical inspection of gravity mains, manholes, and lift stations/pump stations. In order to focus the inspection resources on those collection system assets that are most critical to the City, a pre-inspection risk assessment was performed on the collection system assets.

9.1.1 Gravity Main Approach

As described in this section, a rating system was developed that reflects both the likelihood and consequence of failure, and was then applied to each gravity main in the City's collection system. The risk assessment analyzes the combined the likelihood of failure ratings and the consequence of failure ratings to develop a comprehensive risk rating. For this analysis, a sewer failure is defined as one that could result in a sanitary sewer overflow (SSO). SSOs are violations of state and federal laws and can adversely impact the environment, business activity, and public health. SSOs also cause the City to perform costly emergency repairs, which are disruptive to the community.

In developing a rating system to assess risk, specific system assessment criteria were identified. The development and application of these criteria are based on the following considerations:

- Evaluation information must be readily available for a majority of the gravity mains, such that the assessment criteria are applicable throughout the system.
- The same evaluation information should not be used for more than one criterion, such that no double-counting occurs in the assessment.

9.1.2 Analysis of the Likelihood of Failure for Gravity Mains

This section describes the specific criteria and associated rating factors used in assigning risk associated with the likelihood of failure of a given system asset. Key aspects of this discussion include:

- Definition of Failure,
- Criteria Used to Assess Likelihood of Failure, and
- Rating Factors.

9.1.2.1 Definition of Failure

Failure of a gravity sewer may involve structural failure (primarily from corrosion or cracking/breaking), inadequate hydraulic capacity, or severe maintenance problems related to root intrusions, grease accumulations, and debris. The principal failure modes considered in this analysis are structural failure and maintenance failure.

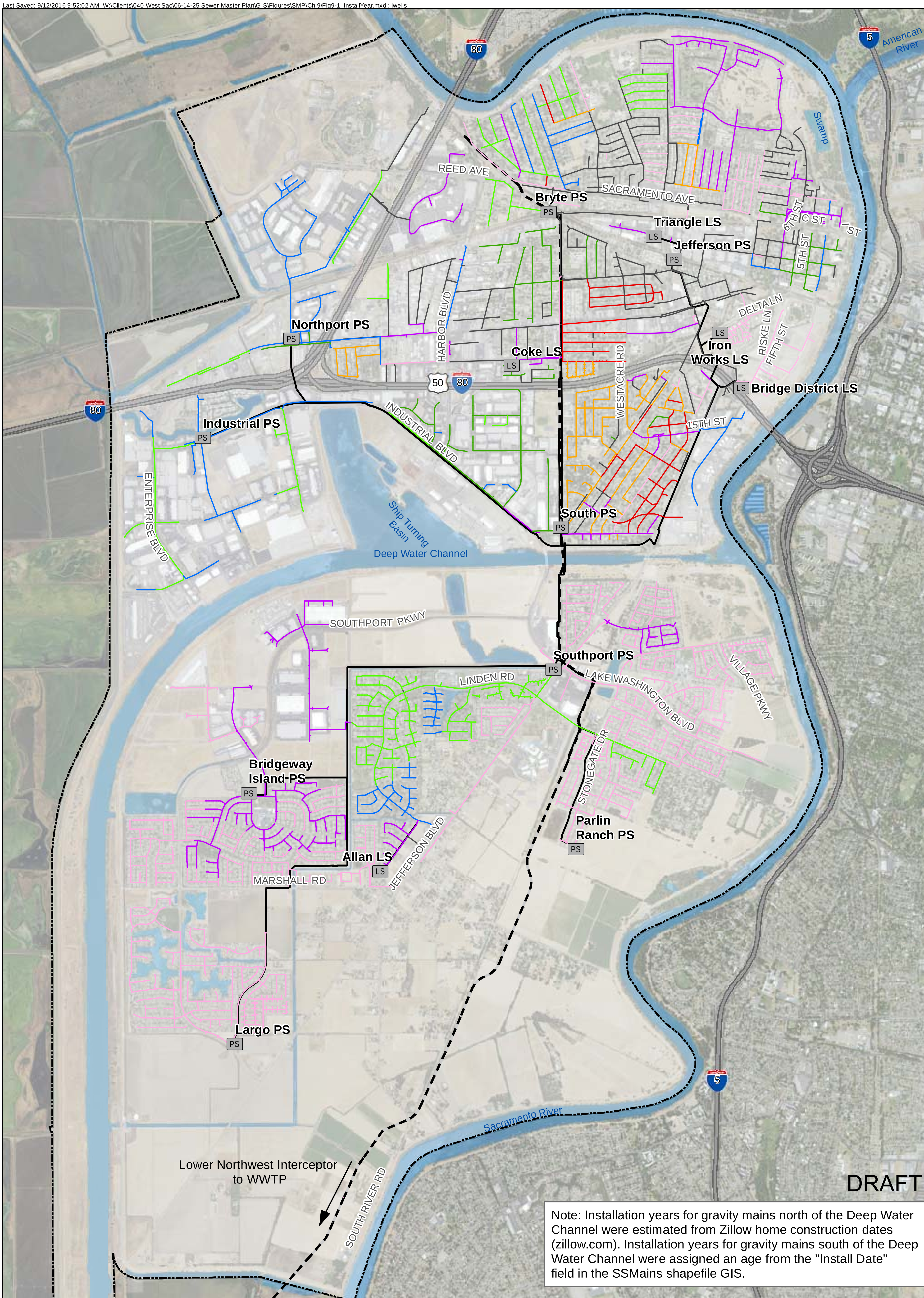
9.1.2.2 Criteria Used to Assess Likelihood of Failure

For each failure mode, one or more factors were considered in determining the likelihood of a failure. These factors are summarized in Table 9-1 and are discussed below.

Table 9-1. Criteria for Likelihood of Failure

Failure Mode	Description	Criteria
Sewer Structural Failure	Pipe cracks or breaks can progress to pipeline collapse. Cracking and breaking increases with time as the pipes approach the end of their useful lives.	Installation date
Sewer Maintenance Failure	The City has prioritized areas based on the probability of maintenance-related blockages and SSOs occurring due to root intrusion, grease accumulation, and sediment accumulation.	Located within a City-defined prioritized maintenance zone
	The City has identified hotspot zones based on a history of service calls and/or system blockages. Reoccurring blockages may indicate structural and/or maintenance problems are present, which may result in an SSO.	Located within a City-defined hotspot zone

Sewer Structural Failure. For this assessment, structural failure was treated as equally likely for both PVC and VCP sewer lines. Both materials are brittle, and cracks or breaks can progress to pipeline collapse. Cracking and breaking increases with time as the pipes approach the end of their useful lives. For pipes with unknown installation years, the highest rating was assigned. The estimated age of each gravity main in the City's collection system is shown on Figure 9-1.



Note: Installation years for gravity mains north of the Deep Water Channel were estimated from Zillow home construction dates (zillow.com). Installation years for gravity mains south of the Deep Water Channel were assigned an age from the "Install Date" field in the SSMains shapefile GIS.

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Force Main
- Lower Northwest Interceptor

Gravity Main Installation Year

- pre-1950s
- 1950s
- 1960s
- 1970s
- 1980s
- 1990s
- 2000s
- Unknown

0 0.25 0.5
Miles

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Figure 9-1

Gravity Main Install Year

City of West Sacramento
2015 Sewer
Master Plan Update

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Sewer Maintenance Failure. Maintenance problems related to root intrusions, grease accumulations, and debris can cause blockages and result in SSOs. The City has identified three zones of varying concern with regard to maintenance. The first of these zones (known as Tier 1) include all areas where known maintenance hotspots exist. The second of these zones (known as Tier 2) encompasses older areas of the City where there are no known hotspots. The third such zone (undesignated) encompasses all other areas of the City. Within the Tier 1 area, the City has identified five prioritized maintenance sub-zones based on the severity and frequency of maintenance activities related to blockages. The Tier 1 and Tier 2 areas are shown on Figure 9-2, and the five hotspot zones within Tier 1 are indicated with color-coding on that same figure.

9.1.2.3 Rating Factors

Likelihood of failure is rated on a 1 to 5 scale with 5 indicating the highest likelihood. Each gravity main was evaluated for each category and factor (where applicable) and an overall likelihood of failure ranking was determined. The factors and their range of potential ratings for each category are summarized in Table 9-2. For a given asset, the various scores indicated in Table 9-2 are additive, such that for a pipeline segment, the aggregated score would range from 2 to 14.

Table 9-2. Likelihood of Failure Rating Factors[(a)]

Category	Factor	Rating (1 being the lowest, 5 being the highest)				
		1	2	3	4	5
Sewer Structural Failure	Installation Year	>1985	1976-1985	1966-1975	1956-1965	≤1955 or Unknown
Sewer Maintenance Failure	Maintenance Frequency	Routine maintenance only	-	Within Tier 2 maintenance zone	-	Within Tier 1 maintenance zone
	Known Hotspots[(b)]	Sub-zone 5	Sub-zone 4	Sub-zone 3	Sub-zone 2	Sub-zone 1

(a) The indicated rating factors are additive.
(b) Areas outside of Tier 1 are assigned a score of zero for this factor.

9.1.3 Analysis of the Consequence of Failure for Gravity Mains

This section describes the specific criteria and associated rating factors used in assigning risk associated with the consequence of failure of a given system asset. Key aspects of this discussion include:

- Criteria Used to Assess Consequence of Failure, and
- Rating Factors.

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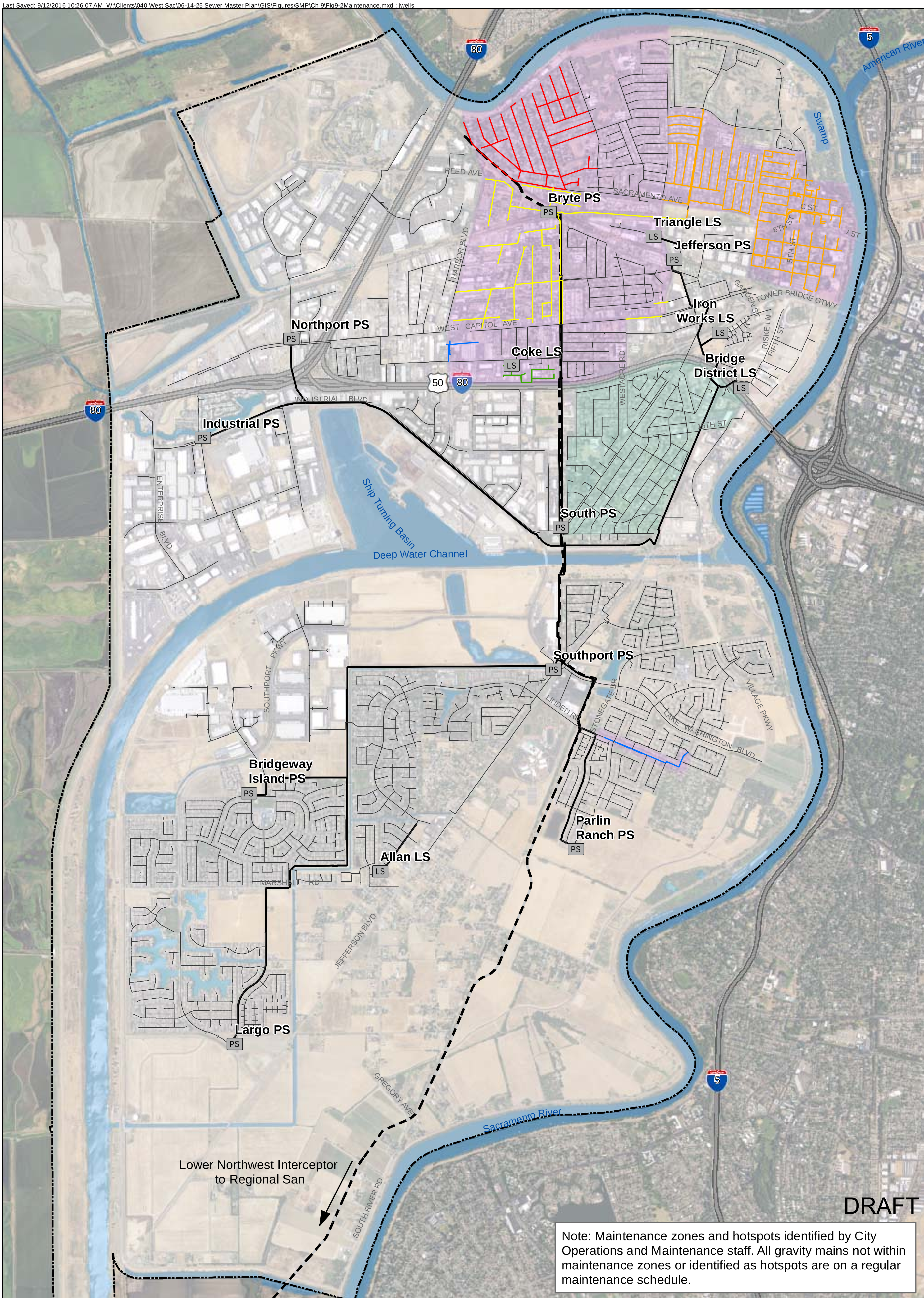

Note: Maintenance zones and hotspots identified by City Operations and Maintenance staff. All gravity mains not within maintenance zones or identified as hotspots are on a regular maintenance schedule.

Symbology
- City Boundary
- LS Lift Stations
- PS Pump Stations
- Lower Northwest Interceptor
- City Force Main

Maintenance Zones
- Tier I Maintenance Zone
- Tier II Maintenance Zone

Known Hotspots
- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Zone 5
- Regular Maintenance

0 0.25 0.5
Miles

Figure 9-2

Maintenance Zones and Hotspots

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9.1.3.1 Criteria Used to Assess Consequence of Failure

The analysis of the consequence of failure is divided into three categories:

- Regulatory Compliance,
- Environmental/Public Health, and
- Community Disruption.

For each category, one or more factors are considered in determining the potential consequence of a failure, as summarized in Table 9-3 and discussed below.

Table 9-3. Criteria for Assessing the Consequence of Failure

Category	Description	Criteria/Factor
Regulatory Compliance	The State Water Resources Control Board requires the City to prevent SSOs and to mitigate SSOs when they occur. The higher the volume of the spill, the more difficult spill mitigation and compliance requirements, and the higher fines become.	Potential volume of SSO (assumed to be proportional to pipe diameter)
Environmental/ Public Health	An SSO will have an increased negative impact on public health and the environment as the proximity to environmentally sensitive areas and public facilities (*e.g.,* schools and parks) increases.	Proximity to environmentally sensitive areas (waterways) or public facilities
Community Disruption	Disruption to the community increases with proximity to arterial streets and highways.	Proximity to arterial streets and highways

Regulatory Compliance. For this analysis, the potential SSO volume was estimated from the diameter of the gravity main. With increasing gravity main diameter, the design flow and the area served by the gravity main generally also increases. Figure 9-3 displays the diameter of gravity mains in the City's collection system.

Environmental/Public Health. SSOs are prohibited by state and federal environmental laws because of their potential adverse impacts on the environment and public health. The environmental and public health consequence of failure analysis is based on two factors: 1) proximity to waterways and 2) proximity to public facilities.

Wastewater that flows into and contaminates streams or rivers directly impacts the environment. The radial distances from gravity sewers to rivers, lakes, and canals were estimated using GIS. As shown on Figure 9-4, gravity mains were divided into three categories:

- Those intersecting a waterway,
- Those within 150 feet of a waterway, and
- Those greater than 150 feet away from a waterway.

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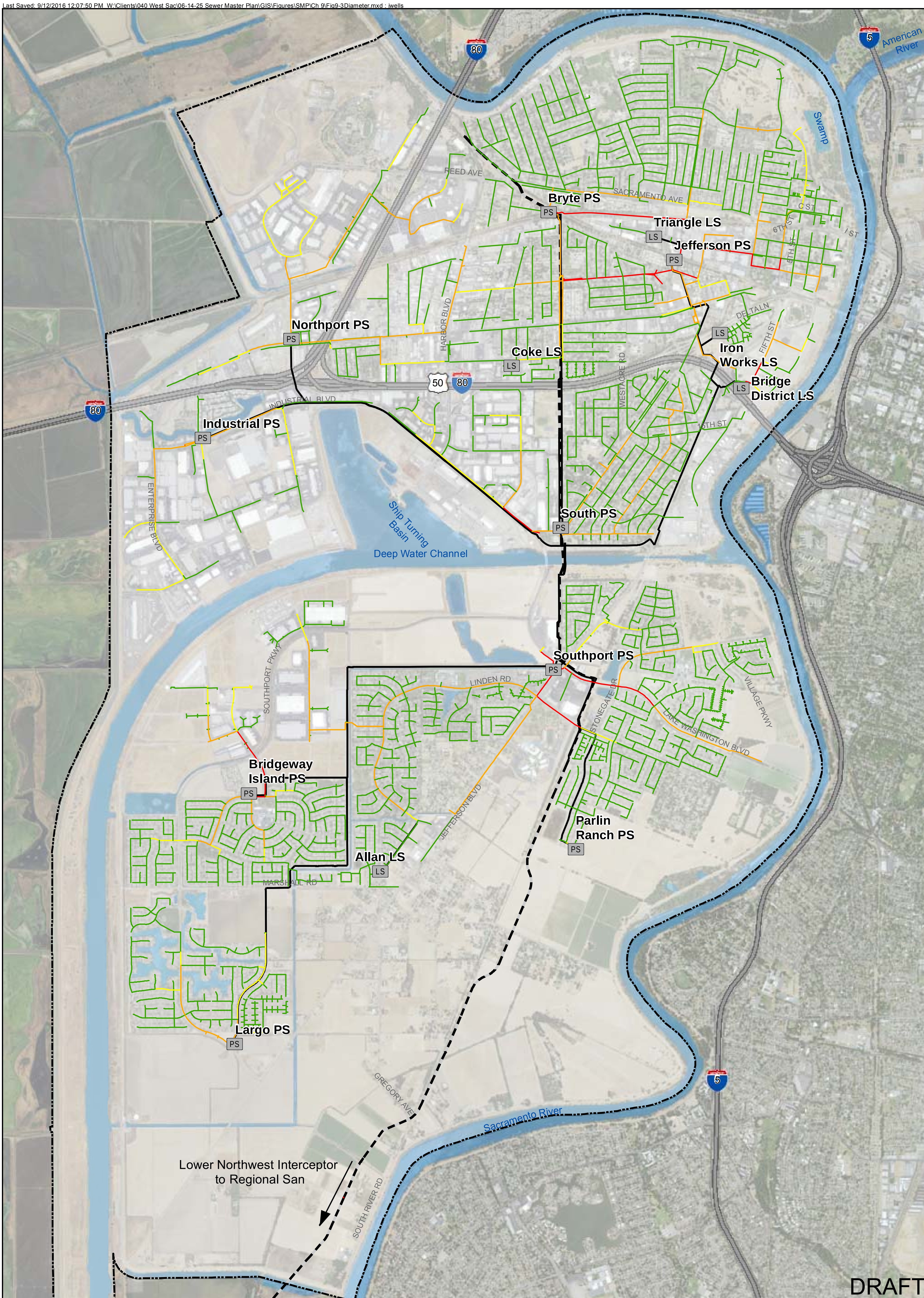

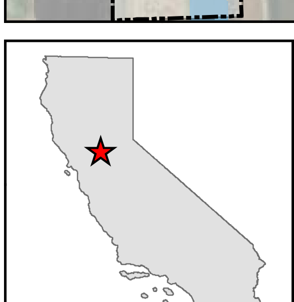

Symbology
- City Boundary
- LS Lift Stations
- PS Pump Stations
- Lower Northwest Interceptor
- City Force Main

Gravity Main Diameter
- ≤ 8-inches
- 10-inches
- 12 to 18-inches
- ≥ 21-inches

0 0.25 0.5
Miles

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Figure 9-3

Gravity Main Diameter

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Human exposure to an SSO poses a public health risk, and the potential for human exposure increases in public facilities such as parks and schools. The radial distances from gravity sewers to parks and schools were also estimated using GIS. As shown on Figure 9-4, gravity mains were divided into four categories:

- Those intersecting a school or park,
- Those within 150 feet of a school or park,
- Those within 500 feet of a school or park, and
- Those greater than 500 feet away from a school or park.

Community Disruption. SSOs impact the community while emergency repairs are being made. Beyond any direct impact associated with sewer service interruption, the principal impact is traffic disruption. GIS was used to analyze gravity main locations within various transportation rights-of-way to identify gravity mains with an increased potential to impact traffic during the SSO event and during any subsequent construction/repairs to gravity mains and associated manholes. As shown on Figure 9-5, gravity mains were divided into four categories:

- Those located in highways or freeway ramps,
- Those located in arterial streets,
- Those located in minor arterial streets, and
- Those located outside all of the higher traffic areas listed above.

9.1.3.2 Rating Factors

The consequence of failure factors described above were assigned numeric ratings of 1 to 5, with 5 indicating the highest or worst consequence. Each gravity main in the City's collection system was evaluated for all of the categories described above, and an overall consequence of failure score was calculated for each gravity main. The numeric ratings for the consequence of failure factors are summarized in Table 9-4. For a given asset, the higher of the two Environmental/Public Health scores is selected and then added to the other two scores. As a result, the aggregated score for any pipeline segment ranges from 3 to 15.

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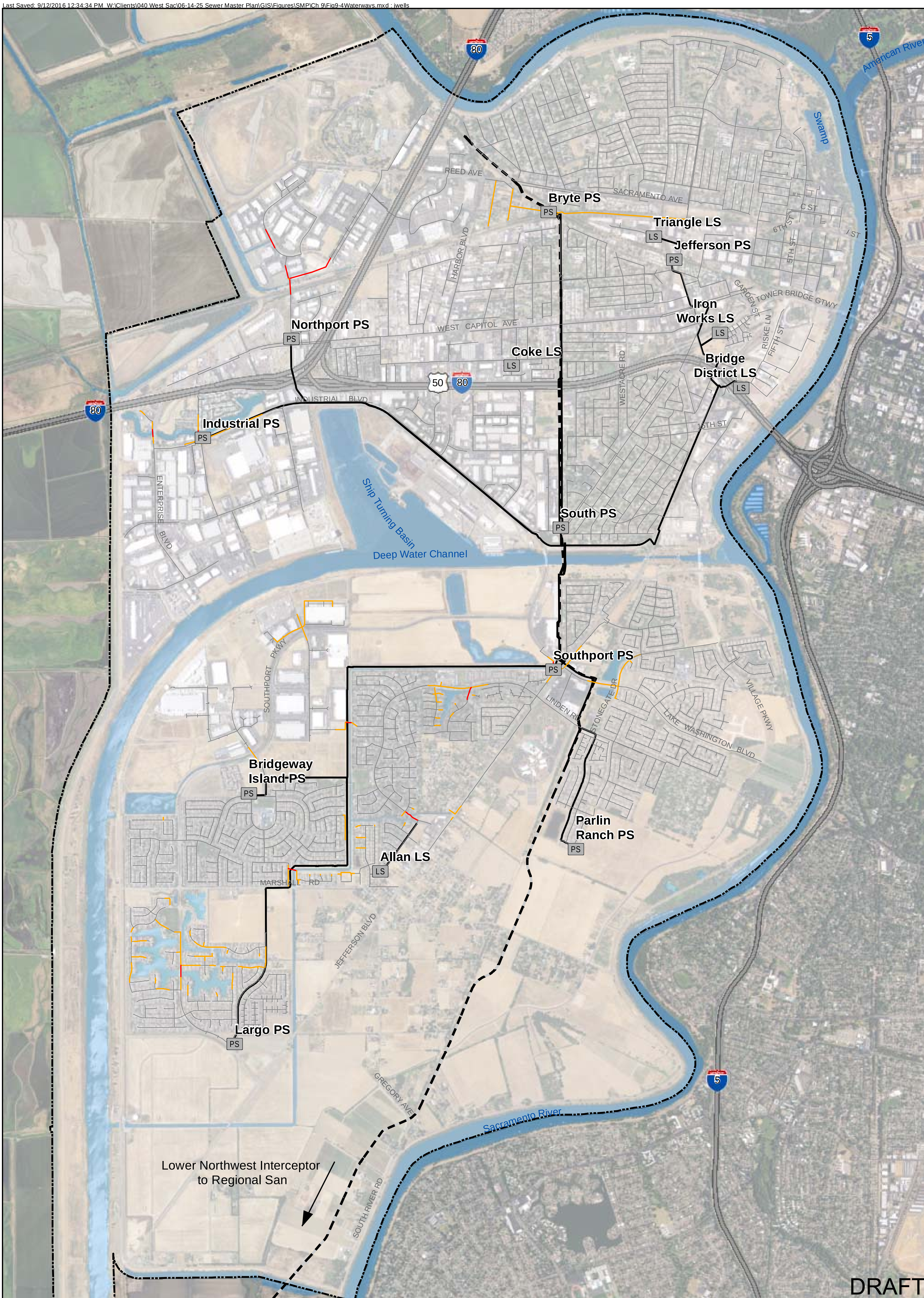

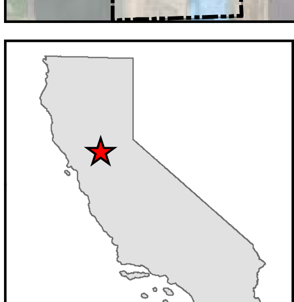

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Lower Northwest Interceptor
- City Force Main

Gravity Main Proximity to Waterway

- Intersecting
- Within 150 feet
- Greater than 150 feet

N

0 0.25 0.5
Miles

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Figure 9-4

Gravity Main Environmental Risk

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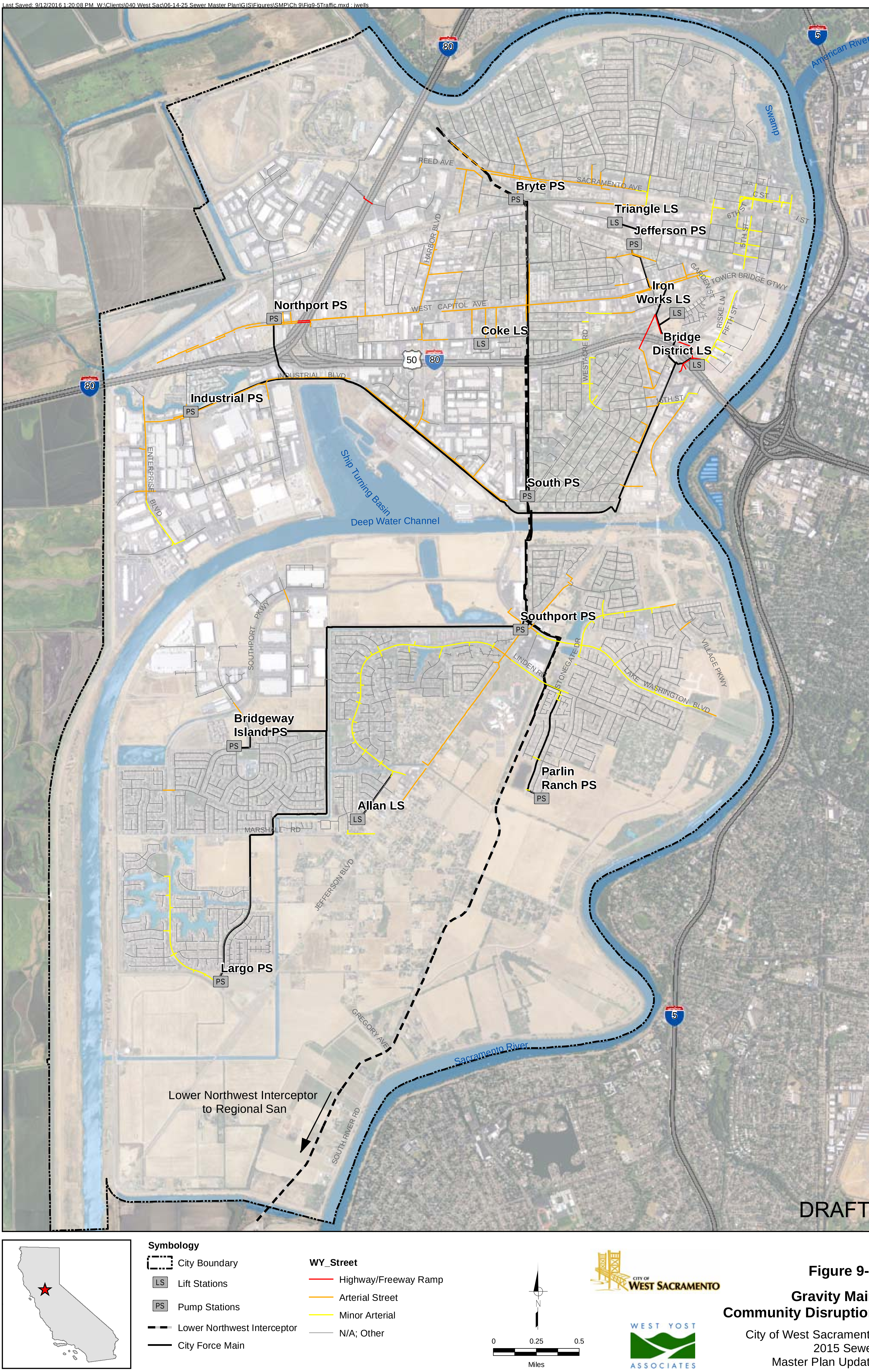

Figure 9-5

Gravity Main Community Disruption

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Table 9-4. Consequence of Failure Rating Factors[a]

Category	Factor	Rating (1 being the lowest, 5 being the highest) 1	2	3	4	5
Regulatory Compliance						
Potential Volume of Spill	Diameter	≤ 8-inches	10-inches	Between 12 and 18 inches	≥ 21-inches	-
Environmental/Public Health						
Environmentally Sensitive Areas	Proximity to Waterways	Other	-	-	Within 150 feet of Waterway	Waterway/ Crossing
Public Exposure	Proximity to public facilities (park & schools)	Other	Within 500 feet	-	Adjacent/ Within 150 feet	Within/ Intersecting
Community Disruption						
Traffic Impacts	Proximity to arterial streets and highways	Other	-	Minor Arterial Street	Arterial Street	Highway/ Freeway Ramp

(a) The higher of the two Environmental/Public Health scores is used. Otherwise the indicated rating factors are additive.

9.1.4 Pre-Inspection Risk Assessment Results for Gravity Mains

A database model was developed to perform the risk assessment calculations. As noted above, for each pipeline, the aggregated score for Likelihood of Failure ranges from 2 to 14, while the aggregated score for Consequence of Failure ranges from 3 to 15. The aggregated scores are then subdivided into five groups (A through E), with A representing scores at the lowest end of the range and E representing scores at the highest end of the range. The model applies a series of algorithms to calculate total consequence and likelihood of failure scores for each gravity main.

By plotting the consequence of failure and the likelihood of failure scores against each other, an overall risk level was assigned to each gravity main. Table 9-5 shows the total miles of gravity sewer that fall into each likelihood and consequence of failure category. Risk levels are prioritized as shown in Table 9-5 into five risk levels: Low Risk, Medium-Low Risk, Medium Risk, Medium-High Risk, and High Risk, each of which is color-coded in the table. These risk levels are assigned to the various cells using best engineering judgment to determine which combinations of score warrant the highest levels of concern versus those that warrant lesser levels of concern.

Table 9-5. Calculated Risk Levels and Associated Gravity Sewer Miles

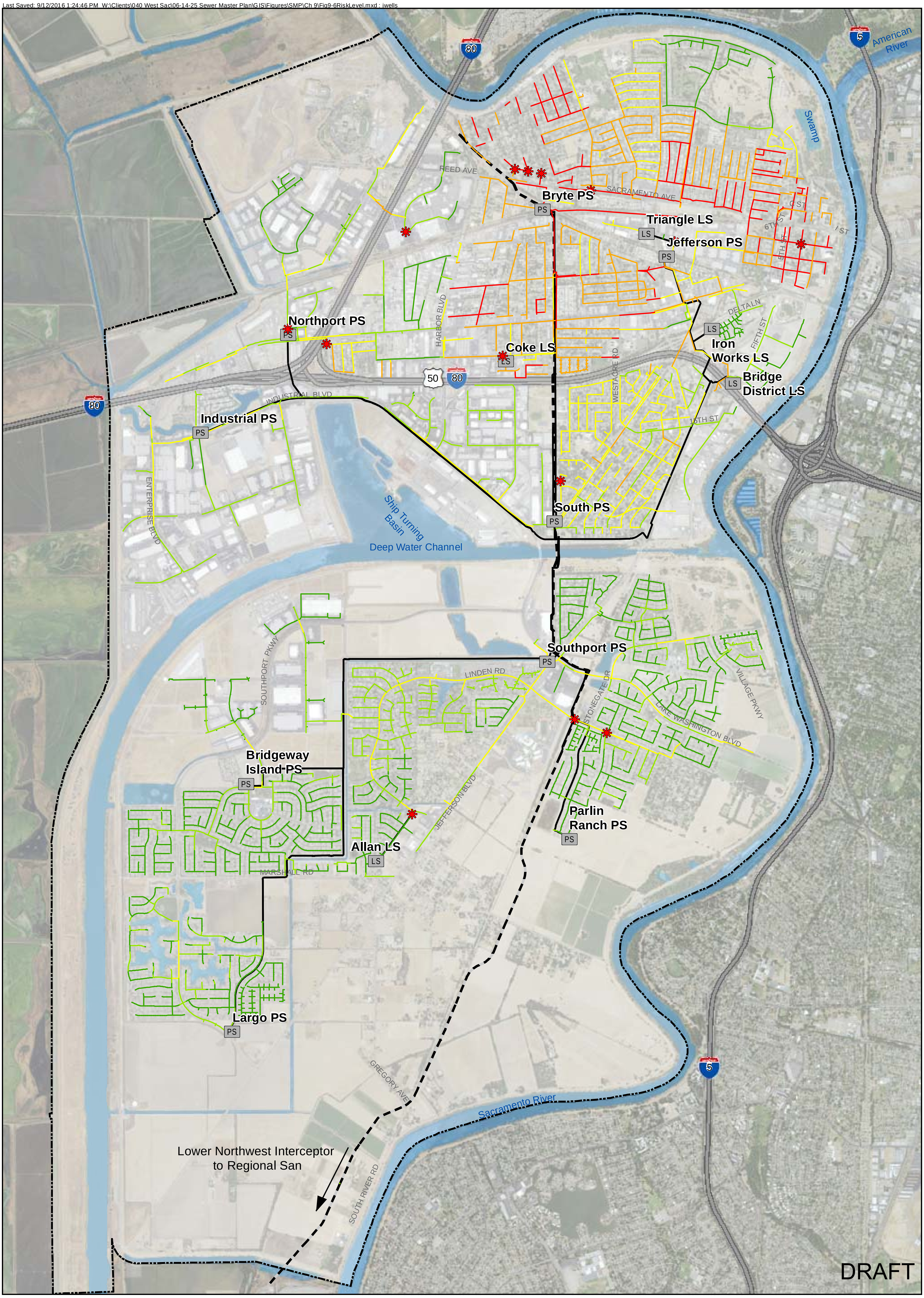
Miles of Gravity Sewer		Likelihood of Failure					
		A	B	C	D	E	Total
Consequence of Failure	A	31.1	9.3	12.1	5.0	8.9	66.4
	B	13.8	3.4	6.9	3.7	2.3	30.1
	C	11.7	3.3	12.5	4.3	6.3	38.2
	D	6.9	3.0	2.5	1.4	1.6	15.4
	E	4.7	1.9	0.7	0.9	0.6	8.9
	Total	68.2	21.0	34.6	15.4	19.7	158.9

Risk Levels: Dark Green = Low, Light Green = Med-Low, Yellow = Medium, Orange = Med-High, Red = High

The risk assessment results shown on Figure 9-6 are also summarized in Table 9-6, which lists the total miles of gravity sewers that fall in each risk level. As shown on Figure 9-6, all of the High Risk gravity mains are located north of the Deep Water Channel. The majority of Low Risk gravity mains are located south of the Deep Water Channel which consists of relatively new pipe. (Approximately 75 percent of the system south of the Deep Water Channel was installed after 1990, whereas the majority of the system north of the Deep Water Channel was installed prior to 1960.) As summarized in Table 9-6, 22.1 miles of the segments modeled are High Risk, which is approximately 14 percent of the gravity collection system.

Table 9-6. Summary of Risk Assessment Results

Risk Level	Miles of Gravity Sewer	% of Total
Low	44.9	28%
Medium-Low	34.7	22%
Medium	41.1	26%
Medium-High	16.2	10%
High	22.1	14%
Total	159	100%



Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Lower Northwest Interceptor
- City Force Main

Gravity Main Pre-Inspection Risk

- High
- Medium-High
- Medium
- Medium-Low
- Low

Manhole Failure Potential

- High

0 0.25 0.5 Miles

Figure 9-6

Gravity Main Pre-Inspection Risk

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9.2 PRE-INSPECTION RISK ASSESSMENT FOR MANHOLES

Manhole condition at selected manholes was assessed during the field investigations. Manholes with a high failure potential, as described in this section, were prioritized for inspection. Sanitary sewer manholes are primarily subject to corrosion-related failures. The structural concrete and concrete mortar in manhole risers, cones, barrels, and bases are subject to hydrogen sulfide-related corrosion. Corrosion is often more severe at manholes associated with inverted siphons and force main discharges. The locations of manholes relative to inverted siphons and force main discharge points were established using the City GIS data. A total of five such manholes were found, four of which are located at force main discharge points and one at a siphon discharge point.

Manholes can also be subject to maintenance-related failures caused by problems such as root intrusion, corroded frames and covers, and misaligned or loose frames and covers. Manholes with recurring maintenance issues were identified by the City and will be prioritized for inspection. As detailed in Table 9-7, fourteen manholes were prioritized for inspection during field investigations.

Table 9-7. Manholes Prioritized for Inspection

Manhole #	Location	Concern
303	3102 Allan Avenue	Forcemain discharge from Allan LS
560	2009 Linden Road	Forcemain discharge from Parlin Ranch PS
FID 9450	975 F Street	Forcemain discharge from Triangle LS
FID 7768	2499 Evergreen Ave	Forcemain discharge from Coke LS
FID 9663	Parking Area East of Ikea Store	Siphon discharge
FID 9624	Hobson Avenue @ Water Street	Known Condition Issue
FID 9623	Hobson Avenue @ Solano Street	Known Condition Issue
FID 9596	Hobson Avenue @ Yolo Street	Known Condition Issue
-	1776 Deerwood Street	Known Condition Issue
541	Linden Road @ Mojave Drive	Grease Accumulation
1172	Palomar Avenue @ Meadowdale Park (20’ inside park)	Root Intrusion
1160	Last manhole before Northport PS	Significant Infiltration
FID 9512	Todhunter Avenue @ Sacramento Avenue	Significant Infiltration
FID 9265	4^{th} Street @ B Street	Hole in MH bottom

Note: Manhole numbers leading with FID are identified through the City’s schematic sanitary sewer point file. Locations were not verified through field surveys.

9.3 PRE-INSPECTION RISK ASSESSMENT FOR LIFT STATIONS/PUMP STATIONS

West Yost collaborated with City staff on a desktop pre-inspection risk assessment for lift stations/pump stations in order to prioritize the inspections to critical assets. During the desktop assessment, it was determined that the electrical and control systems at all pump stations had been updated in 2008, and these systems therefore did not require inspection. In addition, he following priorities were identified during the desktop risk assessment:

- Allan LS, Bryte PS, Industrial PS, Jefferson PS, South PS, and Southport PS were prioritized for inspection.

- Coke LS and Triangle LS were not prioritized for inspection, as the City is already aware of critical limitations of these sites.
- Northport PS was not prioritized for inspection because this station was installed at the same time as Industrial PS with the same design, so City personnel assume that the components at Northport PS are in the same condition as observed at Industrial PS.
- To allow resources to be applied at the stations that the City identified as most in need, inspection was not prioritized for the City's five newest stations: Bridge District LS, Bridgeway Island PS, Ironworks LS, Largo PS, and Parlin Ranch PS. It is assumed that these stations are generally in good condition and performing well.

9.4 CONDITION ASSESSMENT PLAN

The condition assessment plan for gravity mains, manholes, and lift stations/pump stations is described below.

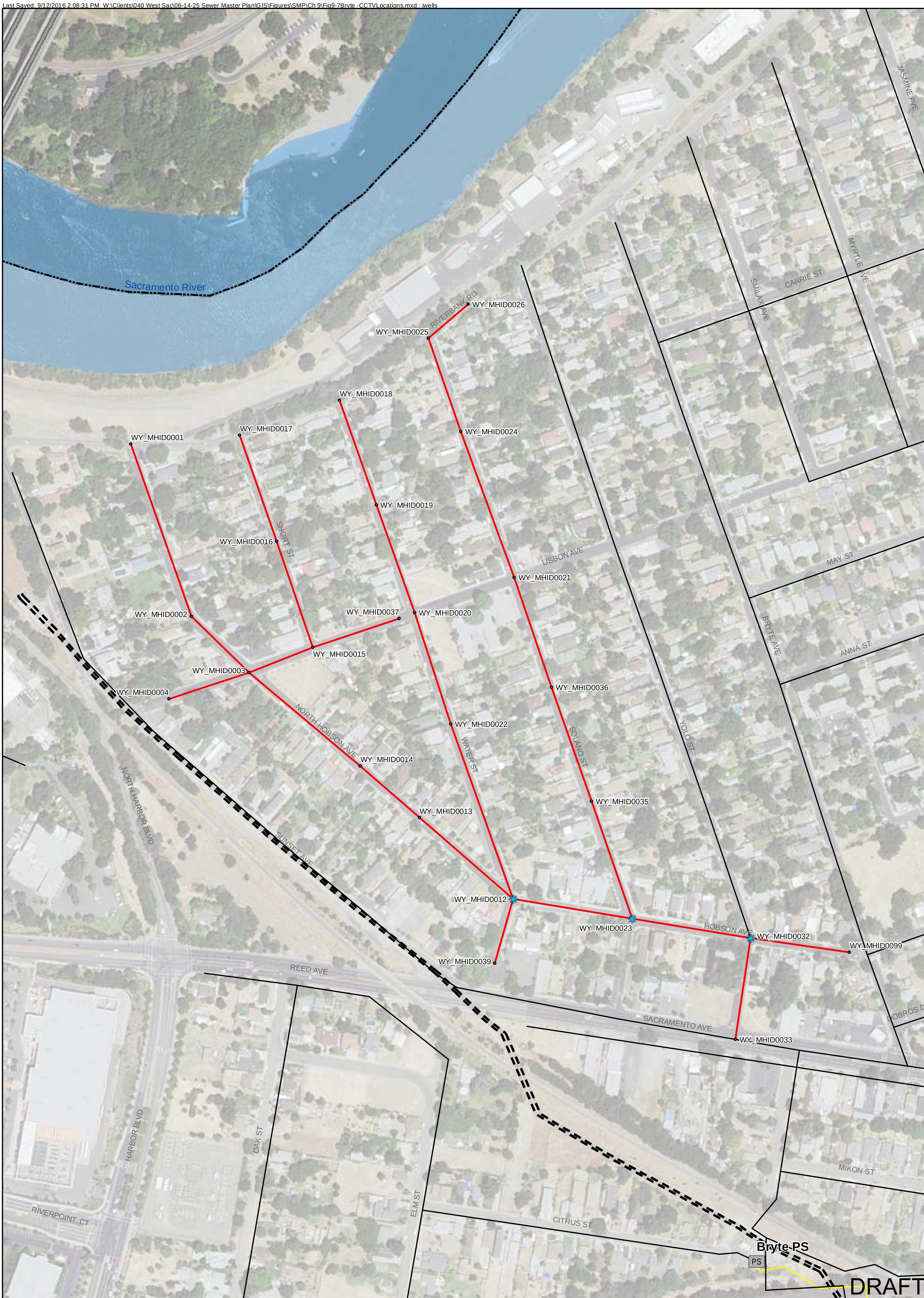
9.4.1 Gravity Main Assessment Plan

The condition assessment plan that resulted from the pre-inspection risk assessment conducted above incorporates assessment of three different prioritized sections of the City's gravity sewer system. These areas were prioritized for inspection through the numerical risk assessment described above, in combination with discussion and collaboration with City staff.

The areas identified for condition assessment were:

- **Bryte Neighborhood**: Representative area of the highest risk portion of the City's collection system.
- **Allan Lift Station**: Significant infiltration issue identified during pump station assessments and by City staff, section chosen to identify source.
- **State Street Area**: Representative area of Old West Sacramento neighborhood, which City staff report is amongst the oldest pipes in the City.

These areas, shown on Figures 9-7, 9-8, and 9-9, respectively, represent a combined 3.1 miles of the City's gravity mains. The condition assessment further included inspection of 56 manholes. All manholes within the gravity sewer condition assessment areas were inspected.



Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Manholes
- Lower Northwest Interceptor
- City Force Main
- City Gravity Main

Condition Assessment Locations

- 2015 Sewer Master Plan CCTV
- High Risk Manholes

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Figure 9-7

Condition Assessment Bryte Neighborhood

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Symbology
City Boundary
LS Lift Stations
PS Pump Stations
Manholes
Lower Northwest Interceptor
City Force Main
City Gravity Main
Condition Assessment Locations
2015 Sewer Master Plan CCTV
High Risk Manholes

N
0 100 200
Feet

CITY OF WEST SACRAMENTO
WEST YOST ASSOCIATES

Figure 9-8

Condition Assessment Allan Lift Station

City of West Sacramento
2015 Sewer
Master Plan Update

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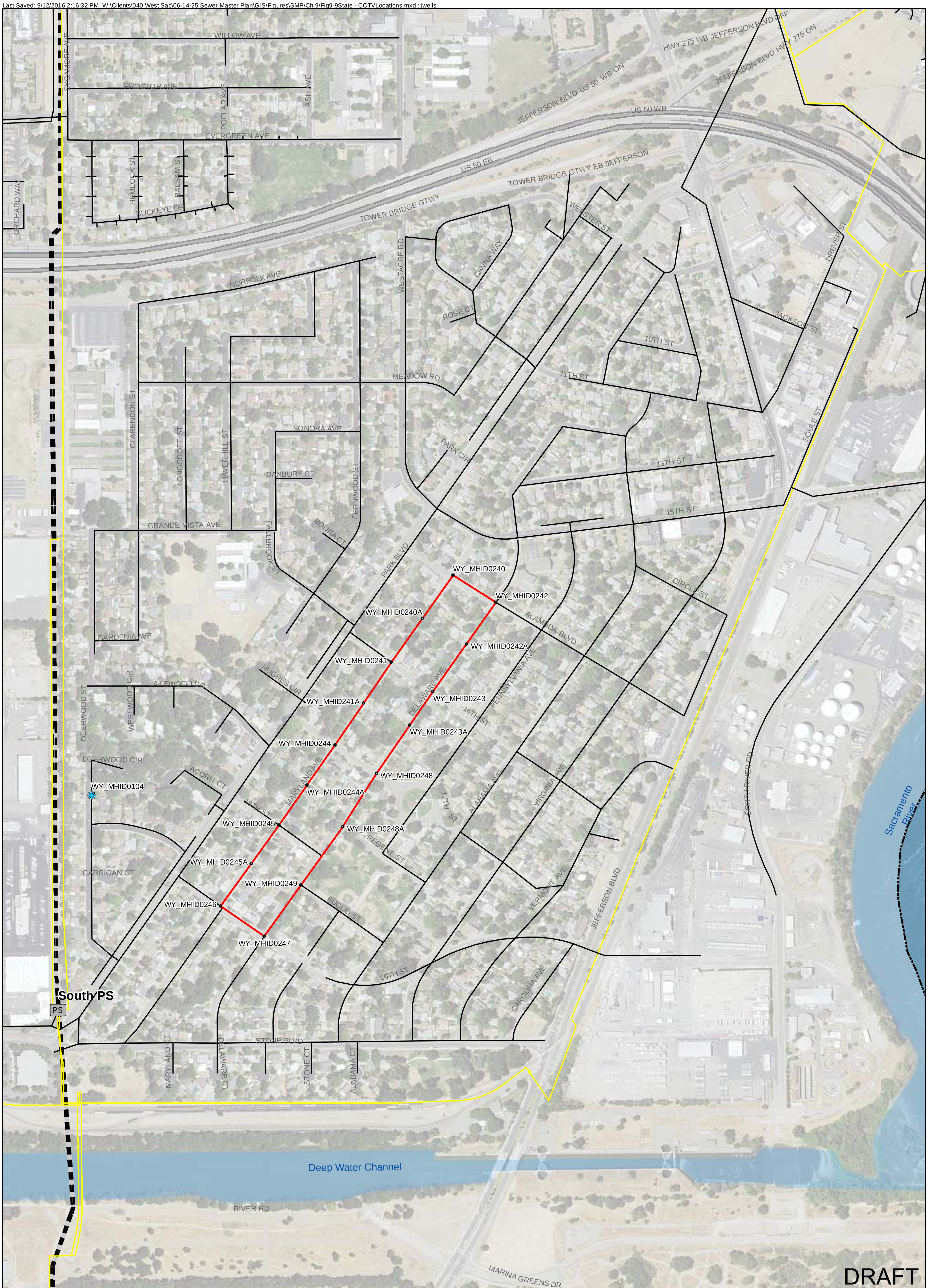

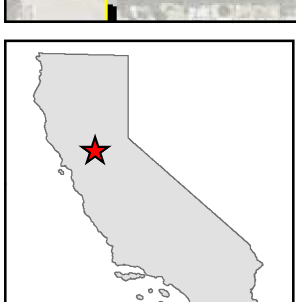

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Manholes
- Lower Northwest Interceptor
- City Force Main
- City Gravity Main

Condition Assessment Locations

- 2015 Sewer Master Plan CCTV
- High Risk Manholes

0 100 200
Feet

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 9-9

Condition Assessment
State Street Neighborhood

City of West Sacramento
2015 Sewer
Master Plan Update

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Data collected during the field inspections is used to assess the current condition of the gravity mains in the City's collection system. It was therefore important to collect high-quality, standardized inspection information. The City selected National Association of Sewer Service Companies (NASSCO) Pipeline Assessment Certification Program (PACP) and Manhole Assessment Certification Program (MACP) as the format for this and all future condition assessment. All closed-circuit television (CCTV) inspections were conducted in accordance with PACP standards, and the data provided to the City is in a NASSCO compatible format. Manhole inspections were conducted following Level 1 MACP protocol.

CCTV inspection is widely used in sewer systems to perform visual condition inspections and is also the primary inspection technology used by the City to assess the condition of its sewer pipelines. It is relatively affordable, straightforward to perform, and particularly useful in providing visual pipeline condition information.

CCTV inspection provides visual condition information, which can be coded according to PACP standards. Typical CCTV systems and software do not provide the ability to obtain physical measurements. Although this technology is widely used, its usefulness is limited to visual inspections above the water surface, the camera system image resolution and lighting, operator experience and training, and environmental factors such as flow depth, velocity, vapor, and splashing. Additionally, defects must be observed by the operator/inspector and the coding of those observations are dependent on the operator's/inspector's interpretation of the defect.

9.4.2 Manhole Assessment Plan

All manholes within the gravity sewer condition assessment areas were inspected. In addition, manholes with a high failure potential (shown on Figure 9-10) were included. Manholes were inspected using NASSCO MACP condition assessment protocol. The inspection team followed a Level 1 inspection protocol, and therefore, did not require confined space entry. Level 1 inspections involve gathering basic condition assessment information from the top of the manhole to evaluate general condition. Pictures of the interior of the manhole were taken with a camera mounted on a pole to document manhole condition.

9.4.3 Lift Station/Pump Station Assessment Plan

Allan LS, Bryte PS, Industrial PS, Jefferson PS, South PS, and Southport PS were prioritized for inspection over two days of inspections. The inspection was conducted using standardized inspection forms based upon the information that was preliminarily developed for the desktop pre-inspection assessment.

9.5 ASSET MANAGEMENT PLAN RESULTS

This Asset Management Plan (AMP) begins with a condition assessment of the City's wastewater pump stations and gravity mains. The AMP then uses a risk-based system for prioritizing the most urgent improvements and defines the triggers that indicate when each asset is ready for each type of maintenance or improvement action.

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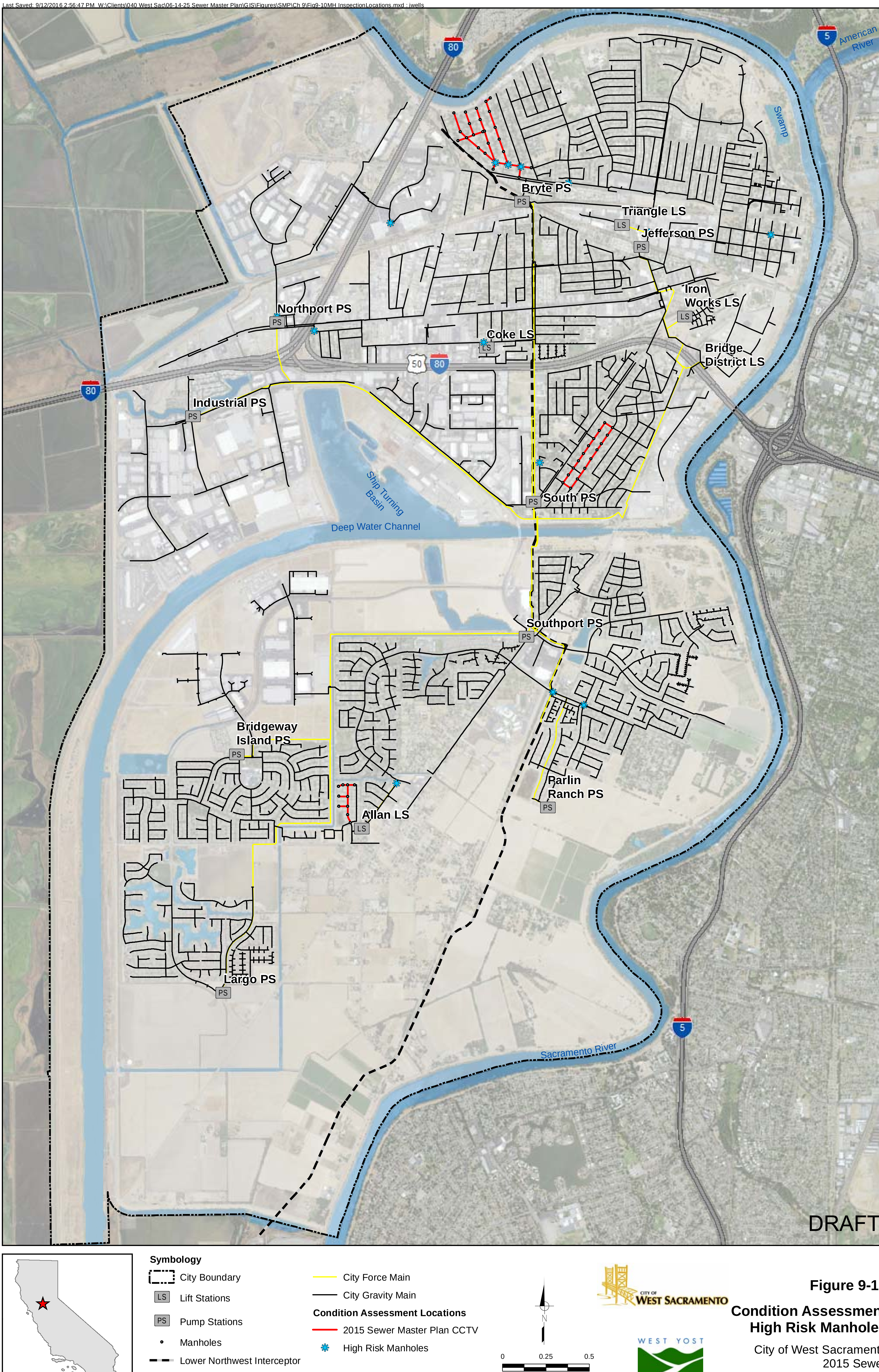

Figure 9-10

Condition Assessment High Risk Manholes

City of West Sacramento
2015 Sewer
Master Plan Update

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9.5.1 Collection System Levels of Service

This section defines the levels of service used as a factor to prepare the AMP.

West Yost worked with the City to define the levels of service in Table 9-8. These levels of service allow the City to focus its efforts and resources, communicate service expectations and choices, and evaluate risk levels.

Table 9-8. Collection System Levels of Service

Maximized Efficiency and Useful Life	Regulatory Compliance
• Continue to proactively maintain the gravity mains and pump stations • Proactively replace infrastructure	• Eliminate SSOs caused by collection system failures

The City wishes to minimize unscheduled repairs. Mechanical maintenance activities are currently scheduled by City staff, and unscheduled mechanical repairs are infrequent. Instrumentation and control maintenance activities are typically unscheduled emergency repairs.

9.5.2 Collection System Asset Inventory

This section summarizes the City's existing asset inventory and describes the process used to expand and improve the City's inventory of individual assets in the wastewater collection system.

Pump station asset information and work order history is currently managed by the City using the PMC2000 Computerized Maintenance Management System (CMMS). The PMC2000 Inventory includes a unique identifier for each component in an alpha-numeric format (e.g., HVAC-S18-001). This identifier includes the asset class (HVAC) and station number (S18), followed a unique number string. The City provided a list of the IDs of assets currently included in CMMS. West Yost added assets identified during site visits, applying the same alpha-numeric format to identify each asset.

The City is in the process of completing a robust inventory of the collection system linear assets and maintenance holes. This GIS-based registry includes asset information such as material, diameter, length, installation date, and a unique numerical identifier. This inventory currently only includes linear assets and maintenance holes in the portion of the City south of the Deep Water Channel; however, assets in the northern portion of the City are being added to the registry as projects occur north of the Deep Water Channel. For those assets not included in the City's current registry, West Yost populated necessary asset information such as diameter, material, and installation year from as-built plans and historic system maps. All assets were assigned a unique numeric identifier.

9.5.3 Condition Assessment Results

The following section describes the results of the condition assessment that was prioritized and planned as described above.

9.5.3.1 Gravity Mains

The gravity sewer condition assessment included assessment of three different prioritized sections of the City's gravity sewer system, as described above:

- Bryte Neighborhood
- Allan Lift Station
- State Street Area

In total, a combined 3.1 miles of the City's gravity sewer were inspected via CCTV from August 31st to November 13th, 2015. The condition assessment also included inspection of approximately 56 manholes, including manuals within the gravity sewer condition assessment area and manholes with a high failure potential, as identified in the pre-inspection risk assessment.

Additionally, the City had previously conducted 5 miles of gravity sewer inspections via CCTV from June 20th to July 25th, 2014 within the Washington District. Similar to the State Street Area the Washington District represents some of the older original areas of the City; some of the gravity mains in the Washington District have already been rehabilitated or replaced.

CCTV inspection was conducted according to PACP standards. The PACP scoring system provides standardized procedures for assessing pipeline defects observed during CCTV inspections. Each defect was assigned a score based on its severity: 1 represents low severity, 5 represents high severity. The observations were broken into two categories: structural and operations and maintenance (O&M) defects. A Quick Score was calculated separately for the maintenance and structural defects as a shorthand way of expressing the number of occurrences for the two highest severity grades in a specific pipe segment. Table 9-9 shows the number of gravity main segments in each structural quick rating category.

Table 9-9. Gravity Main Structural Inspection Results

Inspection Area	Structural Quick Rating, gravity main segments						Total
	<1,100	1,100-2,000	2,100-3,000	3,100-4,000	4,100-5,000	>5,100	
Bryte Neighborhood	20	2	2	0	0	0	24
Allan Lift Station	9	0	0	0	0	0	9
State Street Area	4	0	0	10	0	0	14
Washington District	29	1	8	8	1	2	49
Total	62	3	10	18	1	2	96

The inspections showed that the system is in good condition structurally, with far fewer high severity defects than would be expected for its age. However, the maintenance defects were significantly worse than expected, showing a system that is in need of a routine maintenance program. In fact, 12 percent of the inspections had to be abandoned due to maintenance defects that blocked the CCTV crawler from progressing. Table 9-10 details the number of gravity main segments in each maintenance quick rating category.

Table 9-10. Gravity Main Maintenance Inspection Results

Inspection Area	Maintenance Quick Rating, gravity main segments						Total
	<1,100	1,100-2,000	2,100-3,000	3,100-4,000	4,100-5,000	>5,100	
Bryte Neighborhood	4	0	6	8	5	1	24
Allan Lift Station	9	0	0	0	0	0	9
State Street Area	6	0	3	5	0	0	14
Washington District	35	1	3	0	9	1	49
Total	54	1	12	13	14	2	96

9.5.3.2 Manholes

The manhole inspections found the manholes to be in typically good condition from a structural point of view. The worst condition was found at the manholes that served as discharge points for force mains from the Allan LS and Parlin Ranch PS. The full manhole inspection report can be found in *Appendix G: Manhole Inspection Report*.

9.5.3.3 Lift Stations/Pump Stations

West Yost conducted site visits at six stations on February 18, 2015 and February 19, 2015. The site visits were conducted by a condition assessment team that included City maintenance personnel and a Civil Engineer from West Yost. The assessment team examined the civil, structural, and mechanical systems. The assessment team did not evaluate the electrical and controls systems as the City replaced these systems at all stations in 2008. The assessment team observed the performance and the external condition of the facilities and equipment, and rated the condition and performance of each component using the scale described in Table 9-11. The assessment team did not conduct confined space entries or destructive testing.

Table 9-11. Condition and Performance Ranking Index		
Score	Condition Ranking Index	Performance Ranking Index
1	Excellent	Component functioning as Intended
2	Slight visible degradation	In Service, but higher than expected O&M Costs
3	Visible degradation	In service, but function is impaired
4	Integrity of component moderately compromised	In service, but function is highly impaired
5	Integrity of component severely compromised	Component is not functioning as intended

The observed conditions were also compared to the standard useful life expectancies for the various components. For example, a valve vault might have a standard useful life of 50 years. However, a well-maintained valve vault that is 50 years old might have a remaining life of over 10 years. The standard useful life estimates used in this assessment are listed in Table 9-12. The results of the condition assessment for each of the inspected pump stations are noted in detail on the Pump Station Inspection Forms located in *Appendix H: Pump Station Inspection Reports*.

In June of 2015, after completion of the condition assessments for the lift stations/pump stations inspections described above, City O&M staff identified the possibility that the Triangle Lift Station was in danger of immanent collapse and failure. O&M staff were concerned that that the exterior shell wall of the lift station was corroding, losing structural integrity, and in danger of collapse under the soil load on the shell wall. Staff members expressed hesitance in entering the lift station for regular maintenance.

West Yost secured the services of JDH Corrosion Consultants, Inc. (JDH) to inspect the shell wall of the Triangle Lift Station and determine if corrosion was impacting the thickness and structural integrity of the wall. JDH identified pinhole leaks in the shell wall. These leaks were determined to have been caused by corrosion taking place before the lift station's current impressed current cathodic protection (ICCP) system was implemented. The ICCP was determined to be functioning and protecting the shell wall effectively. The shell wall thickness was measured by JDH to be 0.25 inches, which was confirmed by the manufacturer to be the original installed thickness for the shell wall.

Using the measured shell wall thickness and a soil depth of 10 feet, West Yost determined that the shell wall was not in danger of immanent collapse. Although the Triangle Lift station requires prioritization for replacement, it was determined that the lift station does not require emergency intervention, and O&M staff are not endangered by threat of collapse when entering the station. The full corrosion assessment performed for the Triangle Lift Station can be found in *Appendix I: Triangle Lift Station Corrosion Evaluation*.

Table 9-12. Industry Standard Life Estimates

Asset Class	Asset Subclass	Industry Standard Life (years)
Access	Access - Asphalt	20
	Access - Concrete	40
	Access - Gravel	10
Building	Roll-up Door	50
	Roof	50
	Structure	50
Air Compressor	Air Compressor	20
Drainage	Drainage	50
General Electrical	Convenience Receptacles	20
	Lighting Panel	25
	Motor Control Center	20
	PLC	20
	Surge Protector	20
	System Service	40
	Other (General Electrical)	20
Personnel Elevator	Elevator	15
Fencing	Fencing - Chain Link	40
	Fencing - CMU	40
Fire Protection Equipment	Automatic Sprinkler	30
	Fire Extinguisher	15
Gates	Gates	40
Generator	Adaptor for Portable Unit	20
	On-site Generator	25
Hoists	Hoists	20
Access Hatch	Access Hatch	25
Heating, Ventilation, and Cooling Systems	Air Conditioner	30
	Other (HVAC)	30
Field Instruments	Bearing Temp Sensor	15
	Field Instruments (pH, chlorine, turbidity, etc.)	15
	Flow Element	15
	Level Element	15
	Level Sight Glass	30
	Pressure Element	15
	Transmitter	15
	Vibration Sensor	15
Lighting	Lighting - Exterior	20
	Lighting - Interior	20
Motor	Motor	30
Odor Control Unit	Blower	30
	Odor Control - Activated Carbon	10
	Odor Control - Biofilter	10
Piping	Pump Discharge Piping	50
	Pump Suction Piping	50
	Yard Piping	50
Pump	Pump	30
	Sump Pump	30
SCADA/Telemetry	SCADA/Telemetry	20
Seals	Seal Water System	30
	Seals	30
Security	Alarm Systems	15
	Locks	15
Surge & Transient Control	Surge & Transient Control	30
System Dry/Wet Well	Dry Well	50
	Wet Well	50
Valves	Air Valve	30
	Check Valve	25
	D/S Isolation Valve	30
	Pressure Relief Valve	25
	Pump Control Valve	25
	Sluice Gate	30
	Solenoid Control	20
	U/S Isolation Valve	30
Valve Vault	Valve Vault	50
Water Service	Water Service	50

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9.5.4 Final Risk Assessment

The risk assessment evaluated the likelihood and consequence of a sewer failure, incorporating the inspection results discussed above. For this analysis, a sewer failure is considered to be a failure that could result in a sanitary sewer overflow (SSO). SSOs are violations of state and federal laws and can adversely impact the environment and public health. SSOs can also cause the City to perform costly emergency repairs which are disruptive to the community.

The likelihood of failure assesses the probability that a failure will occur. The consequence of failure considers the impact an asset's failure may have on ability to convey wastewater. This section summarizes the analysis that used available information to assign a rating for both likelihood and consequence of failure to each individual gravity sewer main and each pump station component and facility.

9.5.4.1 Final Gravity Main Risk Results

The pre-inspection risk model described above was updated with the CCTV inspection results. Risk levels are prioritized into five levels: Low Risk, Medium-Low Risk, Medium Risk, Medium-High Risk, and High Risk, each of which is color-coded in Table 9-13. These risk levels are assigned to the various cells using best engineering judgment to determine which combinations of score warrant the highest levels of concern versus those that warrant lesser levels of concern.

Table 9-13. Calculated Risk Levels and Associated Gravity Main Miles

Miles of Gravity Sewer		Likelihood of Failure					
		A	B	C	D	E	Total
Consequence of Failure	A	31.1	9.3	12.1	5.0	8.9	66.4
	B	13.8	3.4	6.9	3.7	2.3	30.1
	C	11.7	3.3	12.5	4.3	6.3	38.2
	D	6.9	3.0	2.5	1.4	1.6	15.4
	E	4.7	1.9	0.7	0.9	0.6	8.9
	Total	68.2	21.0	34.6	15.4	19.7	158.9

Risk Levels: Dark Green = Low, Light Green = Med-Low, Yellow = Medium, Orange = Med-High, Red = High

The risk assessment results are summarized in Table 9-14, which lists the total miles of gravity sewers that fall in each risk level.

Table 9-14. Summary of Gravity Main Risk Assessment Results

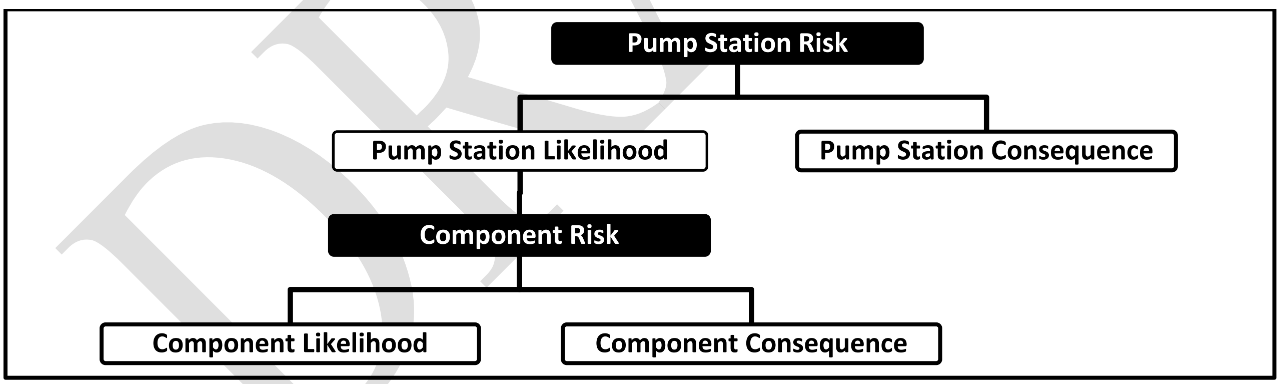
Risk Level	Miles of Gravity Sewer	% of Total
Low	44.9	28%
Medium-Low	34.7	22%
Medium	41.1	26%
Medium-High	16.2	10%
High	22.1	14%
Total	158.9	100%

9.5.4.2 Final Lift Station/Pump Station Risk Results

This section summarizes the analysis which used available information to assign a risk level for each pump station component and the pump station facility.

The risk for each pump station component is based on the likelihood and consequence of the component's failure. The aggregate of the risk levels of the components within a given pump station is used to determine the likelihood of a pump station failure, which is then combined with the consequence of failure rating of each pump station to determine a comprehensive risk (or criticality) rating for each pump station as a whole. For this analysis, a failure is defined by the pump station's inability to meet demand flows. The analysis is summarized in Figure 9-11.

Figure 9-11. Lift Station/Pump Station Risk Assessment Methodology



The component risk levels will contribute to the overall criticality level for each pump station as a whole in the risk analysis.

The likelihood of failure analysis considers the probability that a failure will occur in a given component. The analysis for likelihood of failure includes the two primary failure modes: physical mortality and diminished level of service. For each failure mode, one or more factors are considered in determining the potential likelihood of a failure, as discussed below.

Physical Mortality Failure. Older assets are more likely to fail than newer ones due to the age of materials and wear from repeated use. The percent of useful life remaining was determined by comparing the number of remaining years estimated during the field assessments to the industry standard lifetime for each asset. Similarly, assets with visible degradation are more likely to fail. While condition and age are often dependent, newer components may be in poor condition due to environmental conditions and older components may be in good condition due to regular preventative maintenance.

Level of Service Failure. Impaired function of assets can cause higher O&M costs or reduced ability of the pump station to meet system demands. An asset's performance may affect the level of service provided by the pump station, depending on the asset's role in day-to-day operations. Assets that require frequent maintenance lead to increased costs of operation.

Likelihood of failure is rated on a one to five scale with five indicating the highest likelihood. Each component is evaluated for each failure mode factor and an overall likelihood of failure ranking is determined. The factors and their range of potential ratings for each category are summarized in Table 9-15. For a given asset, for the various scores indicated in Table 9-15 the maximum rating for each failure mode is additive, such that for a pump station component, the aggregated score would range from 2 to 10.

The consequence of failure considers the impacts a component failure may have on operating the pump station as a whole and maintaining service reliability. For each category, one or more consequences are considered in determining the potential consequence of a failure, as discussed below.

Decreased Operating Ability. Operating ability considers the functionality of the pump station if a component fails. Component failure will have a varying degree of impact on the ability of the station to pump wastewater depending on the role of the component and the configuration of the pump station. Component failure may lead to a lack of redundancy, reduced efficiency, or decreased ability/inability to convey wastewater. In some, the condition could not be determined because the component was inaccessible (e.g., buried pipe). If the installation year was unknown for the component, the physical mortality failure could not be determined.

Decreased Service Reliability. Reliability of service decreases as the time and/or resources required to repair or replace a component increases. An easy repair or replacement is defined as taking one person no more than one day to complete the task. A difficult repair or replacement would take more than one person and/or more than one day to complete. If the repair or replacement requires the pump station to be taken offline, even for a short amount of time, this is an even greater service impact. If the component is obsolete, it is assumed that a partial redesign or programming of the controls would need to occur.

The consequences of failure described above were translated in numeric rankings of one to five, with five indicating the highest or worst consequence. Each component at each of the City's pump stations was evaluated for the consequence factor described above, and an overall consequence of failure score was calculated for each component. The factors and their range of potential ratings for each consequence are summarized in Table 9-16. For a given asset, the various scores indicated in Table 9-16 are additive for each category, such that for a pump station component, the aggregated score would range from 1 to 10.

Table 9-15. Likelihood of Component Failure Rating Factors

Factor	Rating (1 being the lowest, 5 being the highest					Rating Logic
	1	2	3	4	5	
Physical Mortality Failure						
Condition Rating	(1) Excellent	(2) Slight Visible Degradation	(3) Visible Degradation	(4) Integrity Moderately Compromised	(5) Integrity Severely Compromised	Condition Rating, if available; Otherwise RUL
Percent Useful Life Remaining	> 70%	40 to 70%	10 to 40%	5 to 10%	< 5%	
Level of Service Failure						
Performance Rating	(1) Functioning as Intended	(2) In Service, but Higher than Expected O&M Costs	(3) In Service, but Function is Impaired	(4) In Service, but Function is Highly Impaired	(5) Not Functioning as Intended	Single rating

Table 9-16. Consequence of Component Failure Rating Factors

Factor	Rating (1 being the lowest, 5 being the highest)					Rating Logic
	1	2	3	4	5	
Operating Ability						
Functionality of Pump Station (PS)	PS Operates Normally Without	Lack of Redundancy/ Potential Reduced Efficiency	Reduced Security	-	PS Cannot Operate Without	Single rating
Service Reliability						
Repair/Replacement Difficulty	Easy	Difficult	-	Offline	Obsolete	Single rating

A database model was developed to perform the risk assessment calculations. The model applies a series of algorithms to calculate total consequence and likelihood of failure scores for each component.

By plotting the consequence of failure and the likelihood of failure scores against each other, an overall risk level was assigned to each component. Table 9-17 shows the total number of components that fall into each likelihood and consequence of failure category. Risk levels are prioritized into five risk levels: Low Risk, Medium-Low Risk, Medium Risk, Medium-High Risk, and High Risk, each of which is color-coded in Table 9-17. These risk levels are assigned to the various cells using best engineering judgment to determine which combinations of score warrant the highest levels of concern versus those that warrant lesser levels of concern.

Table 9-17. Pump Station Component Risk Levels

Number of Components		Consequence of Failure					
		A	B	C	D	E	Total
Likelihood of Failure	A	21	5	1	3	0	30
	B	29	17	2	3	0	51
	C	7	6	0	1	0	14
	D	62	36	4	4	0	106
	E	37	5	1	4	0	47
	Total	156	69	8	15	0	248

Risk Level: **Red** = High, **Orange** = Med-High, **Yellow** = Medium, **Light Green** = Med-Low, **Dark Green** = Low

The risk assessment results are summarized in Table 9-18, which lists the total number of pump station components that fall in each risk level.

Table 9-18. Summary of Pump Station Component Risk Assessment Results

Risk Level	No. of Components	% of Total
Low	21	8%
Medium-Low	34	14%
Medium	27	11%
Medium-High	115	46%
High	51	21%
Total	248	100%

The individual component analysis described above drives the risk analysis at the individual lift station/pump station level. The likelihood of failure analysis considers the probability that a failure will occur in a given pump station. The analysis for likelihood of failure includes the two primary failure modes: 1) component failure, and 2) power failure. For each failure mode, one or more factors are considered in determining the potential likelihood of a failure, as discussed below.

Component Failure. Since the risk assessment for each component within each pump station considers the likelihood that a failure will occur and its overall effect on the pump station as a whole, the likelihood of a pump station failure increases as the risk level of the components within it increase. The risk level of lift station/pump station assets are summarized on Figure 9-12.

Only assets at the following stations were inspected: Allan LS, Bryte PS, Industrial PS, Jefferson PS, South PS, and Southport PS. Northport PS was not inspected because it was installed at the same time with the same design as Industrial PS; the City expects that the likelihood of component risk is equal to what was determined through site visits of Industrial PS. For these stations, a statistical method whereby the risk levels of the components in each pump station could be compared to the risk levels of components in the other stations was developed. The medium risk level of each pump station (shown as a black dot on Figure 9-12) considers the percentage of assets worse than average, measured by the number of high risk, medium-high risk, and half of the medium risk components. The Risk level of each pump station components was compared to the risk level of the total of all components in this evaluation.

Risk Levels of Assets, Percent of Total. To preserve resources, only facilities with unknown condition and performance were inspected. City staff was able to provide anecdotal information to determine the likelihood of component failure for the stations that were not inspected. Bridge District LS, Bridgeway Island PS, Ironworks LS, Largo PS, and Parlin Ranch PS are newer stations and the City expects the components to be at low risk of failure. Coke LS and Triangle LS were not initially inspected because the operators have indicated that the condition of the assets at these stations is poor. Triangle LS was inspected by corrosion engineers, as described above in this chapter, because immanent failure of the shell wall was feared. It was determined that although the Triangle LS should be prioritized for replacement, structural failure due to corrosion was not immanent.

An additional measure of the potential likelihood of component failure is the frequency of work orders developed for the station. A pump station with regular unscheduled work orders is more likely to fail than a station that only has preventative maintenance work orders.

Likelihood of failure is rated on a one to five scale with five indicating the highest likelihood, with the level of deviation scored as shown in Table 9-19. For a given asset, the various scores indicated in Table 9-19 are additive for each failure mode, such that for a pump station component, the aggregated score would range from 2 to 10.

The consequence of failure considers the impact a pump station's failure may have on level of service provided by the City's wastewater collection system. A pump station failure may have multiple consequences including regulatory non-compliance, environmental/public health impact, and/or community disruption. For each consequence, one or more factors are considered in determining the potential consequence of a failure, as discussed below.

Regulatory Noncompliance. The State Water Resources Control Board requires the City to prevent SSOs and to mitigate SSOs when they occur. The higher the volume of the spill, the more difficult spill mitigation and compliance requirements, and the higher fines become. The potential spill volume from each pump station is determined from the modeled peak wet weather flows.

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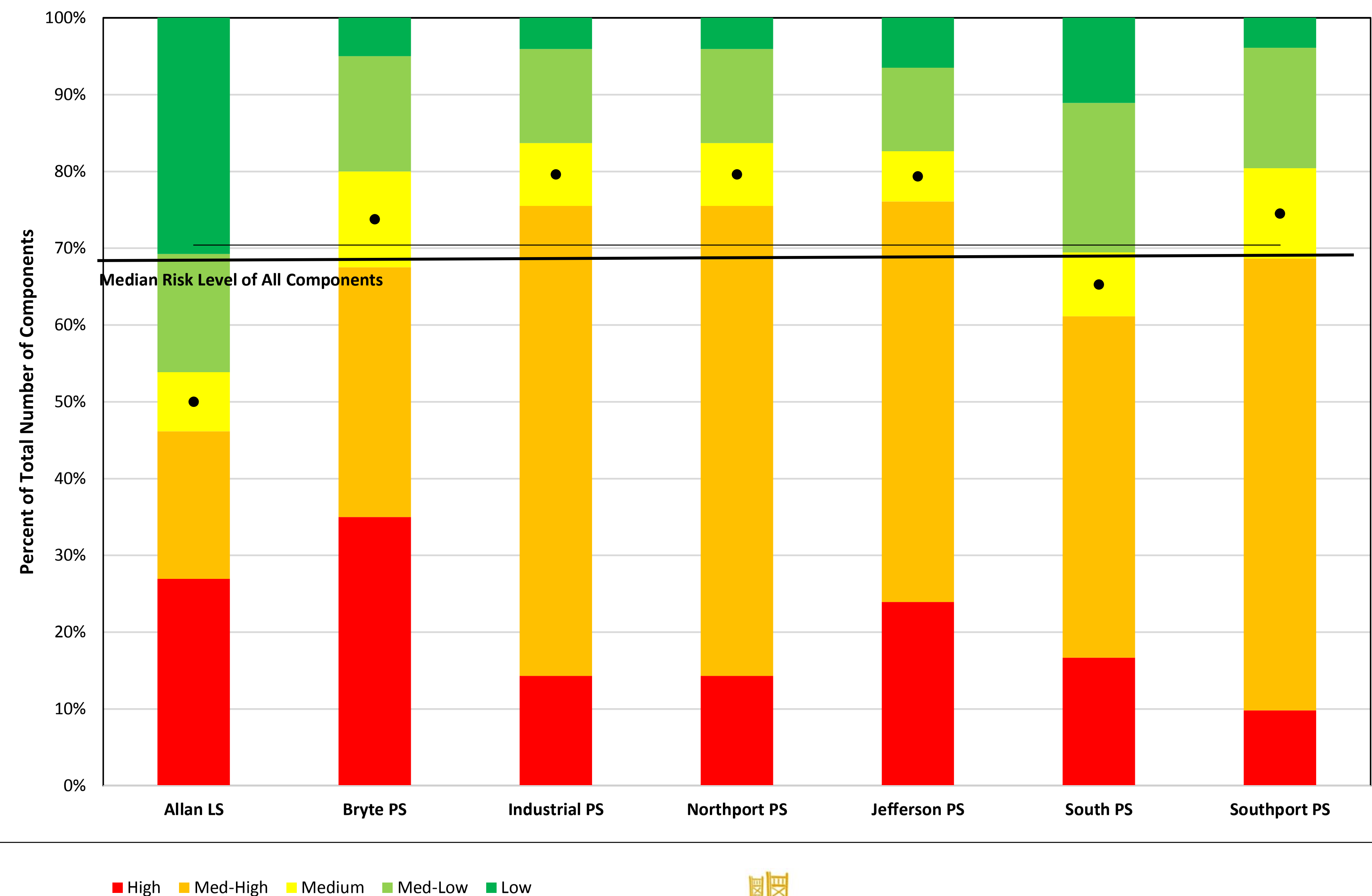

Figure 9-12

Risk Levels of PumpStation Assets, Percent of Total

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Table 9-19. Pump Station Likelihood of Failure Rating Factors

Factor	Rating (1 being the lowest, 5 being the highest)[a]					Rating Logic
	1	2	3	4	5	
Component Failure						
Condition of Critical Components	(1) Excellent	(2) Slight Visible Degradation	(3) Visible Degradation	(4) Integrity Moderately Compromised	(5) Integrity Severely Compromised	Condition Assessment if available; otherwise, Installation Year
Installation Year	2000s	1990s	1980s	1970s	pre-1970s	
Level of Service Failure						
Design Errors	No Identified Design Errors	Pump TDH ≥ 30% design error	Pump TDH ≥ 100% design error	Pump TDH ≥ 150% design error	Full Replacement Necessary	Single Rating
Capacity Failure						
Capacity Assessment through Hydraulic Model	No Identified Capacity Issues	-	Capacity Deficiency at Build-Out	-	Existing Capacity Deficiency	Single Rating

Table 9-20. Pump Station Consequence of Failure Rating Factors

Factor	Rating (1 being the lowest, 5 being the highest)					Rating Logic
	1	2	3	4	5	
Regulatory Noncompliance						
Potential Volume of Spill (PWWF, gpm)	< 100	100 – 400	400 – 1,000	1,000 – 3,000	> 3,000	Single Rating
Environmental/Public Health Impact						
Proximity to Waterways	Other	-	-	Within 150’ of Waterway	Waterway Crossing	Highest of two rating factors
Proximity to Public Facilities (Parks & Schools)	Other	Within 500’	-	Adjacent/ Within 150’	Within/ Intersecting	
Community Disruption						
Traffic Impacts	Other	-	Minor Arterial Street	Arterial Street	Highway/ Freeway Ramp	Single Rating

Environmental/ Public Health Impact. An SSO will have an increased negative impact on public health and the environment as the proximity to environmentally sensitive areas (e.g., waterways) and public facilities (e.g., schools and parks) increases. Using GIS, an intersection of the pump station location with streams or lakes identified stations located in close proximity to waterways. The distance from a pump station to a public facility, was estimated using GIS data of park and school locations. The consequence of failure increases as the proximity decreases, thus differing degrees of risk were assigned to stations within 150-feet and stations intersecting waterways and public facilities.

Community Disruption. Beyond any direct impact associated with sewer service interruption, the principal impact of SSOs is traffic disruption. Unexpected work in streets to contain and mitigate a spill caused by an SSO and any potential pipeline repairs causes a community disruption. Disruption to the community increases with proximity to arterial streets and highways.

The rating factors for consequence of failure are determined by the location of service interruption and the severity of the disruption. The greater of the consequence of failure ratings for the two categories is used for the AMP analysis. Table 9-20 presents a summary of the basis for consequence of failure.

A database model was developed to perform the risk assessment calculations. The model applies a series of algorithms to calculate total consequence and likelihood of failure scores for each facility. Risk levels increase as likelihood and consequence increase, generally depicted in Table 9-21 with green indicating lowest risk and red indicating highest risk.

Table 9-21. Pump Station Facility Risk Levels

Facility		Consequence Score				
		A	B	C	D	E
Likelihood Score	A	Ironworks LS				
	B		Parlin Ranch LS Allan LS		Bridgeway Island PS	Southport PS
	C			Jefferson PS Northport PS	Bryte PS Bridge District LS Industrial PS	
	D			South PS Largo PS		
	E		Triangle LS Coke LS			

The facility risk from high to low is as shown in Table 9-22.

Table 9-22. Summary of Pump Station Facility Risk Assessment Results

Risk Ranking	Facility Name
High	Triangle LS
High	Coke LS
Medium High	Industrial PS
Medium High	South PS
Medium High	Bridge District PS
Medium High	Bryte PS
Medium High	Southport PS
Medium High	Largo PS
Medium	Northport PS
Medium	Bridgeway Island PS
Medium	Jefferson PS
Medium Low	Allan LS
Medium Low	Parlin Ranch PS
Low	Iron Works LS

9.6 ASSET MANAGEMENT PLAN

This section provides the recommended Asset Management Plan that allows the City to 1) maximize efficiency and useful life, and 2) increase reliability and customer satisfaction.

9.6.1 Preventative Maintenance Program

This section describes the City's existing pump station preventative maintenance program, and presents an optimized program based on the risk levels above.

9.6.1.1 Existing Preventative Maintenance Program

Gravity Mains. The City performs routine cleaning with hydrojet of the prioritized maintenance zones. The maintenance department has plans to expand preventative maintenance to other areas of the system as resources become available.

Pump Stations. The City maintains a regular preventative maintenance schedule for assets at each of their pump stations, as summarized in Table 9-23. Preventative maintenance schedules are currently being set as defects are found or problems occur.

Force Mains. Force mains are currently maintained as problems are identified. The maintenance department has plans to expand preventative maintenance to other areas of the system as resources become available.

Table 9-23. Preventative Maintenance Schedule

Component Type	PM Frequency					Component Type	PM Frequency				
	Monthly	Quarterly	Bi-Annually	Annually	No PM		Monthly	Quarterly	Bi-Annually	Annually	No PM
Access Hatch				✓		Lighting	✓	✓			
Adjustable Frequency Drive			✓			Lighting Panel		✓			
Air Conditioning		✓		✓		Load Bank				✓	
Air Low Alarm				✓		Low Air Pressure - Bubbler System				✓	
Air Relief Valve	✓					Low Level Alarm Switch				✓	
Analytical Instrument	✓			✓		Low-Low Float Level Switch		✓			
Automatic Transfer Switch		✓				Main Switchboard				✓	
Bathroom					✓	Manual Transfer Switch		✓			
Bio Cube Blower					✓	Manway Lift	✓			✓	
Bio Cube Control Panel		✓				Meter Electric					✓
Bubbler & Transfer Panel		✓				Motor				✓	
Building			✓			Motor Control Center	✓			✓	
Cathodic Protection	✓			✓		Motor Control Panel		✓			
Cathodic Protection Panel	✓			✓		Odor Control Unit	✓				
Control Cabinet for Bio Cube					✓	Plc Control Panel		✓		✓	
Crane				✓		Power Distribution Panel		✓			
Dehumidifier				✓		Pressure Transmitter				✓	
Emergency Power Generator	✓		✓	✓		Pump Sewage		✓		✓	
Fan		✓				Recorder, Chart					✓
Flooded Alarm Switch				✓		Seal Water Filter					✓
Flow Transmitter				✓		Soft Start		✓			
Harmonic Conditioner				✓		Tank				✓	
Heater				✓		Telephone					✓
Heating, Ventilation, and Cooling	✓			✓		Temperature Switch				✓	
High-High Float Level Switch		✓				Uninterruptable Power Supply				✓	
Intrusion Alarm				✓		Valve		✓		✓	
Level Transmitter				✓		Variable Frequency Drive		✓		✓	
Lift, Maintenance			✓			Well		✓	✓	✓	

9.6.1.2 Optimized Preventative Maintenance Program

Gravity Mains. A systematic and proactive cleaning program for the gravity mains in the collection system is recommended. Given the maintenance defects identified in the CCTV inspections, a five-year cleaning and inspection cycle is recommended initially to establish a baseline for the collection system, with subsequent cleaning frequencies to be determined from the data gathered during this initial cycle. City operations and maintenance staff do not have the resources to complete this "catch up" cleaning and inspection in addition to their regular duties in the collection system and elsewhere. Therefore, a project consisting of contract cleaning and inspection of the collection system over five years has been developed and recommended as part of the CIP in Chapter 10.

Pump Stations. There remains an opportunity to optimize maintenance schedules using the newly-available results of the risk assessment. Table 9-24 is a prioritized list of recommendations for more frequent maintenance observations at each pump station. This list is prioritized according to the risk assessment described above.

Table 9-24. Lift Station/Pump Station Optimized Inspection Improvements

Name	Concerns	Recommended Improvement
Allan LS	• Wet well coating is deteriorating	• Inspect wet well and coating more frequently until re-coating project is implemented.
Bryte PS	• Wet well coating is deteriorating. • Dry pit is leaking.	• Inspect wet well and coating more frequently until re-coating project is implemented. • Inspect dry pit more frequently until leaks are repaired.
Industrial PS	• Check valves are in poor condition.	• Inspect check valves more frequently until replacement.
Jefferson PS	• Inlet sluice gates are deteriorating. • Component in dry pit are leaking.	• Inspect sluice gates more frequently until re-coating project is implemented. • Inspect dry pit more frequently until leaks are repaired.
South PS	• Dry pit steel structure is corroding.	• Monitor condition of the dry pit more frequently.
Southport PS	• Dry pit riser is corroding.	• Inspect dry pit more frequently with focus on corrosion.

Force Mains. Force mains were not inspected as part of this master plan, and have not been inspected by the City. A prioritized and focused force main inspection program and preventative maintenance program should be developed in the next five years. City staff has indicated that the preventative maintenance program should initially focus on identification of and regular maintenance for the air relief valves and blow off valves along the force main alignment.

9.6.2 Rehabilitation/Replacement Program

This section describes the City's existing program for rehabilitating and replacing collection system infrastructure, and presents an optimized program based on the risk levels determined above.

9.6.2.1 Existing Rehabilitation/Replacement Program

The City is currently rehabilitating or replacing collection system infrastructure as problems are discovered, including both gravity mains and pump station components. City staff wishes to transition to a more proactive and less reactive rehabilitation/replacement program.

9.6.2.2 Optimized Rehabilitation/Replacement Program

Gravity Mains. Three rehabilitation/replacement projects were identified from the CCTV inspections described above. These projects are detailed in Chapter 10, and they will correct specific defects identified during the inspections. Although the CCTV inspections performed for this master plan update indicate that the gravity mains are in generally good condition, the inspections did find areas of structural deterioration and defects. In particular, it was found that the City has areas of deteriorating condition in the gravity mains particularly in areas of old RCP material. Because the collection system has not been systematically cleaned or inspected recently, it is expected that further areas in need of rehabilitation/replacement will be discovered as the systematic cleaning and inspection program described above is implemented.

Therefore, four years of generic or "placeholder" rehabilitation/repair projects are identified in Chapter 10 for subsequent years based upon high priority gravity mains identified in the risk assessment, and upon priorities that will be identified as part of the recommended cleaning and inspection. As these areas of deterioration are identified, either through the cleaning and inspection activities identified above, or as the conditions become critical and are brought to attention of City staff, the budget for this CIP item should be utilized for rehabilitation or repair of these areas. The recent blockage and sinkhole on Fernwood Street caused by deterioration of the gravity main, as well as recent surcharges around the collection system are examples of improvements that will be funded by these placeholder projects. As cleaning and inspection progress, more areas will be discovered and rehabilitated/replaced as part of normal procedures, and before they become critical.

Pump Stations. Pump station components that require rehabilitation/replacement in the near- or long-term were identified in the inspections as described above. The City expressed interest in grouping improvements by pump station (rather than by type of improvement such as SCADA upgrades that would be completed at all pump stations under one contract) because it provides the City the most operational flexibility during implementation. Specific improvements to the pump stations and pump station components are identified in Chapter 10.

When the City's WWTP was abandoned and the City's sewer flows were routed to the LNWI for conveyance to treatment, the pumping head of the City's pump stations was altered. The 2003 Collection System Master Plan identified that the individual pumps in the pump stations should be altered to better match the new required head conditions. Review of the pump and motor nameplates conducted as part of the pump station condition assessment indicates that in many cases, the recommended alterations were not made. The hydraulic model confirms that the pump stations are often running off of their pump curves because the pumps have excess head generation capability compared to the requirements seen in the system. Therefore, the pump station rehabilitation/replacement program identified in Chapter 10 includes replacement of pumps to better match head requirements.

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CHAPTER 10
Capital Improvement Program

Chapter 10 presents the recommended CIP for the City's sewer collection system. The project recommendations, configurations, and conceptual costs that are presented in this chapter were summarized previously in Chapters 8, Capacity Assessment, and Chapter 9, Asset Management Program. This chapter summarizes and presents a consolidated list of projects by proposed priority and implementation schedule.

The recommended CIP identifies the improvements at a master planning level, and does not constitute conceptual or preliminary design of these improvements. Subsequent alignment studies and preliminary designs are recommended to finalize pipeline configuration, pump station needs, and to determine the final sizes, locations, and details of the proposed improvements.

The capital improvement program describes a combination of pipeline, pump station, and storage improvements to address SSOs that are predicted to result from the design storm event. The proposed combination of projects presents a solution that appears viable and practical, based on the information that was known as of the date of the Wastewater Collection System Master Plan. Additional information that is gained through preliminary design activities (permitting, easement acquisition, environmental documentation, etc.) and additional evaluation of the capacity of the City's system is expected to lead to changes in the final project descriptions, costs, and the implementation timeline, and may also result in changes to the types of projects implemented.

The proposed projects have not been subject to the California Environmental Quality Act (CEQA) process, although a programmatic CEQA process is part of the master planning process. Also, the City's concurrent, ongoing efforts to reduce I&I will result in a reduced need for the planned capacity improvements. Therefore, the proposed capital improvement program is an evolving planning tool that will be refined throughout the implementation of the CIP.

This chapter is organized as follows:

- Basis for Capital Improvement Costs,
- Basis for Capital Improvement Program Development, and
- Proposed CIP.

10.1 BASIS FOR CAPITAL IMPROVEMENT COSTS

The following sections describe the methods and associated costs evaluated for completing rehabilitation, repair, and replacement projects in the City's collection system for both capacity enhancement and condition repair.

10.1.1 Pipeline Rehabilitation, Repair, and Replacement Methods and Conceptual Costs

The following rehabilitation, repair, and replacement methods are potential options for the City's pipeline projects: open cut construction, pipe bursting, pipe reaming, and tunneling. For projects that require the installation of a new relief sewer to address wet weather flows, in-situ methods for the existing pipe, such as the use of cured-in-place pipe, may be considered in conjunction with construction of the new relief sewer pipeline. Specific to the City's projects, factors that determine the most cost-effective rehabilitation method include geological and physical setting, existing pipeline material and condition, and available construction access.

10.1.1.1 Open Cut Construction

Description: Open cut or open trench construction, also known as cut and cover, has historically been the most widely used approach for sewer pipe replacements. A trench is excavated that is approximately 18 inches to two feet wider than the replacement pipe, and six to 12 inches deeper than the bottom of pipe. A new pipe is installed, backfill material placed and compacted, and pavement and surface facilities restored. Often, the new pipe is installed in a different location than the original pipe, and the original pipe abandoned in place. In this case, sewer flow continues through the original pipe, and a planned shutdown is scheduled during the "tie-in," when the new pipe is connected to the existing pipe. Alternatively, the existing pipe is removed to allow replacement of the new pipe in the same location. The existing flow is bypassed through a temporary pumped system during construction operations.

Advantages and Limitations: Historically, open cut construction has been more cost effective than trenchless technologies, and consequently, more widely used for pipe replacement. Open cut construction is appropriate in most soil conditions, and could be beneficial in locations where significant utility crossings are present, depending on the depths of existing utilities. An open trench can be adjusted in the field to avoid existing underground obstructions, or to otherwise relocate the new pipe. This method enables installation of a larger diameter pipeline where capacity issues are present, or improved materials when available or needed.

One limitation to open cut construction is in shoring and dewatering. Shoring of the trench walls is required for personnel safety and an engineered shoring system is required when a trench is greater than five feet in depth, in accordance with California Labor Code Section 6705. Excavation below the groundwater table, or in soils that permit infiltration of groundwater into the open trench necessitate aggressive dewatering methods. The added cost of these requirements can decrease the economic viability of open cut construction in specific situations. For pipeline installations in new alignments, a geotechnical investigation is recommended during the design phase to determine shoring requirements and whether groundwater is anticipated during construction.

Open cut construction is also difficult where construction access is limited, or on steep hillsides. Open cut construction also impacts surface features and traffic, may introduce safety concerns in highly used or highly traveled locations, and creates temporary noise and dust impacts. Historically, CalTrans has required trenchless construction methods to be used for the installation of new pipelines within this roadway.

Probable Unit Costs: The unit cost of open cut construction varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, open cut pipeline installation costs range from $10 to $14 per inch diameter per foot of pipe installed.

These pipeline installation costs include excavation, shoring, pipe installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and for planning purposes, are considered equal to fifty percent of the cost of pipeline installation.

For the City's projects, the following unit costs (rounded to the dollar) were applied:

Normal construction conditions:	$18 per inch diameter per foot of pipe
Difficult construction access:	$20
Construction with high groundwater:	$25

10.1.1.2 Pipe Bursting

Description: Pipe bursting is a trenchless construction method by which existing pipe is replaced with the same size or typically one size larger pipe in the same location. Pipe bursting is most effective in replacing pipes that are less than 24-inches in diameter and are at least 4 feet deep. This method is the most cost effective when there are few lateral connections, when the old pipe is structurally deteriorated or is easily fractured (e.g., vitrified clay pipe), and when additional capacity is needed and trenchless methods are desired or required.

A conical pipe bursting head is conveyed through the pipe, exerting outward forces that fracture the existing pipe and displace fragments outward into the soil. The head is driven by pneumatic pressure, hydraulic expansion, or static pull; the head is connected to and pulls in the new pipe. The pipe bursting head is inserted and also retrieved through new access pits that are located at approximately 400 to 500 foot intervals.

The optimal pull length is dependent upon the size of the host pipe, the degree of upsize required, and the type of soil in the surrounding subsurface. Additional pits, typically two feet wide by two feet long, are required at each service lateral connection and at crossing utilities. Pipes suitable for pipe bursting are those made of brittle materials, such as vitrified clay. Special bursting heads with cutting elements are required for more ductile pipe materials such as steel, PVC and ductile iron. Typically, the replacement pipe material will be HDPE or fused PVC. Construction using PVC requires longer pit lengths than with HDPE because PVC requires a longer bending radius.

Advantages and Limitations: Pipe bursting is quickly gaining popularity as a replacement methodology for small diameter sewers. If HDPE pipe is used, a relatively small pit (as compared to open trench) is required for entry of the pipe bursting head, which can be extracted through an existing manhole. Pipe bursting replaces the existing pipe by up to two diameter sizes without significant open trenching, and therefore reduces surface impacts. The unit cost of pipe bursting is decreasing, and often comparable to open cut methods.

Existing conditions must be considered carefully when specifying pipe bursting. Flowing soils such as sand, highly incompressible soils such as rock, installations below the groundwater table, sensitive utilities located within two to three pipe diameters of the pipe to be burst, historical point repairs that are not conducive to bursting such as steel couplings, or significant sags or pipe collapses will limit the success of pipe bursting operations. Pipe bursting may also create ground vibrations and outward ground displacements adjacent to the pipe alignment; these displacements are exacerbated in shallow installations or when the pipe is significantly upsized. When the existing pipe is shallow, this ground displacement may be controlled by saw cutting pavement over the pipe in advance of the bursting operation. This approach localizes surface heave and provides for more simplified trench patch repair.

Pipe bursting is performed between pits spaced 400-500 feet apart. A manhole can be used in lieu of the receiving pit. During the pipe bursting process, the rehabilitated pipe segment must be taken out of service by rerouting or bypassing sewer flows. Laterals are reconnected through external pits after the pipe bursting activities are completed.

Probable Unit Costs: The unit cost of pipe bursting varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, pipe bursting costs range from $15 to $20 per inch diameter per foot of pipe installed. These pipeline installation costs include excavation and shoring of pits, pipe bursting and installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances/ lateral restoration, which are estimated as separate item, and considered equal to the cost of pipeline installation.

The City's projects generally require an increase in pipe diameter that is greater than recommended for pipe bursting. For the City's projects, the more conservative cost for open cut construction was used for all pipelines that are not anticipated to require installation using tunneling methods.

10.1.1.3 Cured in Place Pipe (CIPP)

Description: CIPP is a trenchless repair method that installs a resin-saturated felt liner into the host pipe through existing manholes. The liner is made of interwoven polyester and may be fiber-reinforced for additional strength. Commonly manufactured resins include unsaturated polyester, vinyl ester, and epoxy, each having distinct chemical resistance to domestic wastewater. The CIPP liner is installed by inversion using water or pressurized air; after the liner is in place, the resin-impregnated tube is cured using hot water, steam, or high-intensity UV light, creating a seamless pipe that fits tightly against the host pipe wall. Laterals are then connected to the mainline pipe using a remote-controlled cutting device.

Advantages and Limitations: CIPP is a viable rehabilitation technology in 6-inch or larger gravity sewers where the existing pipe has sufficient capacity. Because laterals are connected from inside the lined pipe, little or no trenching is required. Therefore, CIPP may be a preferred alternative in pipelines where trenching would be cost prohibitive. The CIPP method can be used to address structural problems such as cracks and structurally deficient segments, as well as root intrusions because the liner forms itself generally to the shape of the host pipe, and can span gaps caused by roots up to one inch in diameter. Larger gaps and alignment deficiencies such as offset joints and sags would require a point repair prior to lining.

The flexibility of the resin tube allows installation through existing bends, further minimizing the need for excavation. The liner is resistant to chemical attack, eliminates groundwater from entering the sewer, and retards further corrosion and erosion of the pipeline.

The thickness of CIPP liner typically ranges from ½ inch to 1 inch and therefore, the final inside diameter is approximately 1 to 2 inches less than the inside diameter of the existing pipe. The liner typically has less flow friction compared to the host pipe, so the reduction in diameter does not result in a reduction in hydraulic capacity, particularly for pipe above eight inches in diameter.

CIPP installation requires bypass pumping and groundwater dewatering, if in a high groundwater area. Installation length is generally limited to approximately 800 feet due to curing limitations. Therefore, if manholes are located further apart than 800 feet, intermediate trenched access locations are required. Another challenge associated with using CIPP is the procurement, treatment, and/or disposal of water used during the curing process; during the curing process of any resin system, volatile organic compounds are released and must be closely monitored.

CIPP is a viable alternative to pipeline replacement when pipeline replacement options are cost-prohibitive, and when existing pipe diameter can be reduced without compromising system performance. CIPP is not recommended when pipeline slopes or other constraints limit the use of hydroflushing as a cleaning method.

Probable Unit Costs: The cost of CIPP varies significantly depending on site access, pipeline configuration, liner specifications, curing method, ease of disposal of curing water, and bidding climate. However, for conceptual estimating purposes, CIPP installation costs range from $10 to $15 per inch diameter per foot of liner installed in normal conditions. These costs do not include mobilization, trenching if needed, special disposal costs, lateral connections, or traffic control, which are estimated as a separate item, and considered equal to the cost of CIPP pipeline installation.

For the Wastewater Collection System Master Plan, it is assumed that all of the City's projects will require the installation of new, larger pipe to address capacity constraints. However, during preliminary design, the opportunity to provide smaller, parallel relief sewers in conjunction with repair of the existing pipe using CIPP liner should be considered.

10.1.1.4 Pipe Reaming

Description: Pipe reaming is very similar to pipe bursting in that an existing pipe is drilled out and a new pipe of equal or greater diameter inserted in its place. Because pipe reaming does not displace the broken pieces of the old pipe into the soil, this method is better suited to pipe rehabilitation where nearby pipes or utilities might be impacted by the displaced soil.

Pipe reaming employs a directional drill which pulverizes and grinds up the existing pipe while a new pipe is inserted behind it. The old pipe is accessed by an insertion trench, and the drill head is pulled through the pipe by a drill line which runs from an insertion trench where the pipe is accessed to the next manhole. The broken pipe is carried away through the old pipe by drill fluid and collected at the downstream manhole.

Pipe reaming can be used to remove brittle pipes such as those composed of vitrified clay, PVC, asbestos concrete, or ductile iron. Fused PVC or HDPE are typically used for the replacement pipe. Pipe reaming has been effective at replacing sections of sewer over 1000 feet in length or more with little soil disruption.

Advantages and Limitations: Like other trenchless technologies, pipe reaming is advantageous when trying to minimize the impact of construction on traffic and business. When using pipe reaming as a rehabilitation technology, adequate space must be available for the insertion pit and the heavy machinery necessary for directional drilling and handling of the solids generated by the drilling process. Pipe reaming can become very expensive if there are a large number of laterals that must be reconnected to the replaced pipe.

Probable Unit Costs: Similar to pipe bursting, the unit cost of pipe reaming varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, pipe reaming costs range from $18 to $22 per inch diameter per foot of pipe installed. These pipeline installation costs include excavation and shoring of pits, pipe reaming and installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and considered equal to the cost of pipeline installation. As discussed under pipe bursting, above, it was assumed that pipelines would be installed using open cut methods unless tunneling is required.

10.1.1.5 Tunneling

Description: Where open cut construction is not feasible, practical, or cost effective, trenchless methods can be used to install the sewer pipe. Commonly used trenchless methods include jack-and-bore above the water table, micro tunneling below the water table, and horizontal direction drilling (HDD). These methods involve pre-drilling the pipeline alignment and then installing new pipe through the opening. When installed below Caltrans or railroad right of ways, an additional casing may be required by the governing jurisdiction.

Advantages and Limitations: Tunneling presents similar advantages to pipe bursting and pipe reaming related to minimized surface impacts when compared to open cut construction. Pipe size increase is not limited with tunneling methods and longer lengths of pipe can be replaced through a single bore.

Tunneling requires precise location of existing utilities and is not always appropriate where the new pipeline must maintain a precise slope or avoid numerous underground facilities. Additionally, tunneling requires an understanding of the materials to be tunneled through.

Tunneling requires experienced equipment operators that are skilled with the location and guidance of the necessary equipment. Tunneling is assumed to be required along and across Caltrans and railroad rights-of-way.

Probable Unit Costs: The unit cost of tunneling varies depending on site conditions and construction access limitations. However, in areas without significant utility or traffic issues, tunneling costs are generally 1.5 to 2 times the cost of open cut construction, or from $27 to $36 per inch diameter per foot of pipeline installed depending on the difficulty of material to tunnel through. These pipeline installation costs include excavation and shoring of pits, drilling, pipe installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and considered as fifty percent of the cost of pipeline installation.

10.1.2 Pump Station Upgrade Methods and Conceptual Costs

A condition assessment was conducted for seven of the City's 14 pump stations. The pump station included in the condition assessment have improvements required in addition to the capacity improvements. Additionally, pump station that have conditions that of structural reliability of worker safety will be replaced.

Pump station capacity and reliability improvements are identified as individual elements of work within the pump station with a cost associated with it. The cost is based on a vendor quote for the equipment and a factor for installation The factor is 167 percent of the vendor quote and is made up of the following elements.

- Labor: 30%
- Overhead and profit 15%
- Tax 8.5%
- Shipping: 3%

Pump station replacement construction cost estimates use pre-established West Yost costs curves for wastewater pump stations, and compared these cost curves with the costs curves presented in Shank's "Pumping Station Design." Although the West Yost curves do not differentiate between wet-pit/dry pit and submersible stations, the curves in "Pumping Station Design" provide separate curves for these configurations.

The pump station reliable capacity (the capacity of the station with the largest pump in reserve) is the key value to input to the curves. From the capacity value, a line is drawn to where capacity intersects the cost curve lines. Two lines are provided to reflect difficult construction conditions and comparatively easy construction conditions.

The cost curves return cost values linked to an Engineering News Report Construction Cost Index (ENRCCI) for the "20-Cities Average." This returned cost is then adjusted to better reflect the current value of money and the construction market in the greater Sacramento area. This adjustment is a ratio of the current ENRCCI to the ENRCCI used for the curve. Finally, a 30 percent contingency was added to define the range of the pump station costs based on this planning level of accuracy.

10.2 BASIS FOR CAPITAL IMPROVEMENT PROGRAM DEVELOPMENT

The CIP was developed from the risk assessment performed on the condition of the collection system, the hydraulic analysis performed on the collection system using the hydraulic model, and discussion of system requirements with City staff.

10.3 PROPOSED CIP

The proposed CIP is detailed below, broken into collection system components.

10.3.1 Cleaning/Inspection CIP

A five-year cleaning/inspection program was identified that will complete cleaning and inspection of the collection system within this timeframe. The total capital cost is approximately $2.8M. The cleaning program can be seen on Figure 10-1, and the details can be seen in Table 10-1.

The cleaning and inspection CIP should include the formation of a condition assessment plan to inspect and assess the force mains in the City's collection system.

10.3.2 Backyard Main CIP

During the development of the gravity main risk assessment discussed in Chapter 9, discussions with City staff indicated that backyard mains running through private property were a particular risk for the City. Two backyard main replacement projects have been identified through GIS analysis and discussion with city staff. The projects total $1.3M in capital costs. The projects can be seen on Figure 10-2 and in Table 10-1.

10.3.3 Gravity Main Rehabilitation/Repair CIP

Three rehabilitation/repair projects were identified from CCTV inspections. Additionally, four years of rehabilitation/repair projects are identified for subsequent years based upon high priority gravity mains identified in the risk assessment, and upon priorities that will be identified as part of the Cleaning/Inspection CIP. As discussed in Chapter 9, it is known that the City has areas of deteriorating condition in the gravity mains, particularly in areas of old RCP material. As these areas are identified, either through the Cleaning/Inspection CIP activities identified above, or as the conditions become critical and are brought to attention of City staff, the budget for this CIP item should be utilized for rehabilitation or repair of these areas. The recent blockage and sinkhole on Fernwood Street caused by deterioration of the gravity main, as well as recent surcharges around the collection system are examples of improvements that will be funded by this CIP item. As the Cleaning/Inspection CIP progresses, more areas will be discovered and rehabilitated/ repaired as part of normal procedures, and before they become critical.

Technology such as smart covers and similar monitoring equipment could be used to track performance and further prioritize the collection system for rehabilitation. The total capital cost is $11.9M. The projects can be seen on Figure 10-3 and in Table 10-1.

10.3.4 Gravity Main Hydraulic Improvement CIP

Four projects, two for existing conditions and two for ultimate conditions, were identified using the hydraulic model. These projects improve the capacity in existing gravity mains handle existing flows and flows predicted for General Plan 2035. The projects can be seen on Figure 10-4. The details and costs, which total $4.8M, can be seen in Table 10-1.

In addition to the improvements required above, there is the possibility of the City providing service to parcels in the Southport Area of the City that are currently served by septic tank. The infrastructure required for this service, if all potential parcels were converted septic service to City service, is discussed in Chapter 8. Because of the uncertainty in whether these conversions will take place, the uncertainty in timing of these conversions, and the uncertainty in phasing of potential conversions, the infrastructure requirements for these conversions have not been added to the CIP.

10.3.5 Pump Station Improvements CIP

CIP projects for the City's pump stations combine both capacity improvements as discussed in Chapter 7 and Chapter 8, and condition improvements as discussed in Chapter 9. The improvement timeframe varies greatly for these projects depending upon the priority of the improvements. The improvement projects can be seen in Table 10-1.

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Table10-1. Recommended Wastewater Collection System Capital Improvement Plan[a]							
Improvement Priority	Improvement Timeframe	CIP ID	Type	Reason for Improvement	Improvement Description	Construction Cost[b]	Capital Cost[c]
Cleaning/Inspection CIP							
Year 1	0-5 years	Clean_1	Regional Cleaning and Inspection	CCTV Inspection found that gravity mains identified as high risk during the pre-inspection categorization generally require maintenance to remove FOG and debris, followed by inspection to identify structural condition	32 miles of comprehensive cleaning, followed immediately by PACP gravity main inspection.	$342,000	$578,000
Year 2	0-5 years	Clean_2	Regional Cleaning and Inspection	CCTV Inspection found that gravity mains identified as high risk during the pre-inspection categorization generally require maintenance to remove FOG and debris, followed by inspection to identify structural condition	37 miles of comprehensive cleaning, followed immediately by PACP gravity main inspection.	$386,000	$652,000
Year 3	0-5 years	Clean_3	Regional Cleaning and Inspection	CCTV Inspection found that gravity mains identified as high risk during the pre-inspection categorization generally require maintenance to remove FOG and debris, followed by inspection to identify structural condition	32 miles of comprehensive cleaning, followed immediately by PACP gravity main inspection.	$340,000	$575,000
Year 4	0-5 years	Clean_4	Regional Cleaning and Inspection	CCTV Inspection found that gravity mains identified as high risk during the pre-inspection categorization generally require maintenance to remove FOG and debris, followed by inspection to identify structural condition	39 miles of comprehensive cleaning, followed immediately by PACP gravity main inspection.	$410,000	$693,000
Year 5	0-5 years	Clean_5	Regional Cleaning and Inspection	CCTV Inspection found that gravity mains identified as high risk during the pre-inspection categorization generally require maintenance to remove FOG and debris, followed by inspection to identify structural condition	21 miles of comprehensive cleaning, followed immediately by PACP gravity main inspection.	$216,000	$365,000
Existing	5-10 years	FM_Inspection	Inspection and Condition Assessment of Forcemains	Determine inspection protocols and implement inspection of aging force mains.	Inspect 5,000 linear feet of force main within the City.	$250,000	$422,500
					Subtotal Cleaning/Inspection Improvements	$1,944,000	$3,285,500
Backyard Main CIP							
Ultimate	5-10 years	Backyard_1	Move Backyard Mains	The City has identified backyard mains as a maintenance challenge.	Move 3,500 feet of 8-inch diameter gravity main to the public right of way near 8th Street and James Street.	$672,000	$1,136,000
Ultimate	5-10 years	Backyard_2	Move Backyard Mains	The City has identified backyard mains as a maintenance challenge.	Move 600 feet of 8-inch diameter gravity main to the public right of way near Evergreen Ave and Walnut Ave.	$115,000	$194,000
					Subtotal Backyard Main Improvements	$787,000	$1,330,000

Table10-1. Recommended Wastewater Collection System Capital Improvement Plan[a]							
Improvement Priority	Improvement Timeframe	CIP ID	Type	Reason for Improvement	Improvement Description	Construction Cost[b]	Capital Cost[c]
Gravity System Rehabilitation/Repair CIP							
Existing	5-10 years	Rehab_1	Rehabilitation and/or repair of existing gravity mains.	CCTV Inspection determined that the structural condition of the gravity mains requires rehabilitation or repair to meet level of service requirements.	Replace 2,700 feet of 8-inch diameter gravity main with 8-inch diameter gravity main in the vicinity of Alabama St.	$389,000	$657,000
Existing	5-10 years	Rehab_2	Rehabilitation and/or repair of existing gravity mains.	CCTV Inspection determined that the structural condition of the gravity mains requires rehabilitation or repair to meet level of service requirements.	Replace 1,600 feet of 6-inch diameter gravity main with 8-inch diameter gravity main in the vicinity of Hobson St.	$230,000	$389,000
Existing	0-5 years	Rehab_3	Rehabilitation and/or repair of existing gravity mains.	CCTV Inspection determined that the structural condition of the gravity mains requires rehabilitation or repair to meet level of service requirements.	Replace 5,000 feet of 8-inch and 12-inch gravity main with 8-inch and 12-inch gravity main in the vicinity of C St.	$720,000	$1,217,000
Existing	0-5 years	Rehab_4	Rehabilitation and repair of existing deteriorated manholes	Manhole inspection has determined that low pH value in force main discharges manholes is resulting in deteriorating manhole conditions at manholes in Allan Avenue and Linden Avenue.	Coat two manholes with calcium aluminate in order to prevent future corrosion.	$20,000	$33,000
Future	10-15 years	Rehab_4	Rehabilitation and/or repair of existing gravity mains.	Rehabilitation and/or repair of gravity mains determined to be high priority during Risk Assessment, or identified as high priority during future CCTV inspection.	Identify and rehabilitate/replace high priority gravity main as follow up to CCTV/Inspection Program.	$1,420,000	$2,400,000
Future	10-15 years	Rehab_5	Rehabilitation and/or repair of existing gravity mains.	Rehabilitation and/or repair of gravity mains determined to be high priority during Risk Assessment, or identified as high priority during future CCTV inspection.	Identify and rehabilitate/replace high priority gravity main as follow up to CCTV/Inspection Program.	$1,420,000	$2,400,000
Future	15-20 years	Rehab_6	Rehabilitation and/or repair of existing gravity mains.	Rehabilitation and/or repair of gravity mains determined to be high priority during Risk Assessment, or identified as high priority during future CCTV inspection.	Identify and rehabilitate/replace high priority gravity main as follow up to CCTV/Inspection Program.	$1,420,000	$2,400,000
Future	15-20 years	Rehab_7	Rehabilitation and/or repair of existing gravity mains.	Rehabilitation and/or repair of gravity mains determined to be high priority during Risk Assessment, or identified as high priority during future CCTV inspection.	Identify and rehabilitate/replace high priority gravity main as follow up to CCTV/Inspection Program.	$1,420,000	$2,400,000
					Subtotal Gravity Main Replacement/Repair CIP	$7,039,000	$11,896,000
Gravity Main Capacity Improvements CIP							
Existing	5-10 years	GM_1	Upsized Gravity Main	Gravity Main Capacity Required Under Existing Conditions	Upsize 1,200 feet of 8-inch gravity main to 12-inch gravity main along Stillwater Road	$260,000	$439,000
Existing	5-10 years	GM_2	Upsized Gravity Main	Gravity Main Capacity Required Under Existing Conditions	Upsize 2,500 feet of 18-inch gravity main to 24-inch gravity main along upstream of Bryte Pump Station. Cost includes portion of project that must go under railroad tracks near Bryte Pump Station.	$1,728,000	$2,921,000
Ultimate	15-20 years	GM_3	Upsized Gravity Main	Gravity Main Capacity Required Under Ultimate Conditions	Upsize 2,400 feet of 6-inch gravity main to 15-inch gravity main in Hardy Drive	$648,000	$1,096,000
Ultimate	15-20 years	GM_4	Upsized Gravity Main	Gravity Main Capacity Required Under Ultimate Conditions	Upsize 1,000 feet of 8-inch gravity main to 10-inch gravity main upstream of Iron Works Lift Station with development. Alternatively, development flow can be routed away from this gravity main.	$180,000	$304,000
					Subtotal Gravity Main Capacity CIP	$2,816,000	$4,760,000
WWTP Decommissioning CIP							
Ultimate	15-20 years	WWTP_1	Full Decommissioning of Unused WWTP	Decommission old WWTP, which is unused and abandoned, but still in place.	Full decommissioning and removal of unused and abandoned WWTP.	$8,213,000	$13,880,000
					Subtotal WWTP Decommissioning	$8,213,000	$13,880,000

Table10-1. Recommended Wastewater Collection System Capital Improvement Plan[a]

Improvement Priority	Improvement Timeframe	CIP ID	Type	Reason for Improvement	Improvement Description	Construction Cost[b]	Capital Cost[c]
Pump Station Improvements CIP							
Existing	0-5 years	PS_1	Complete Replacement of Lift Station	Lift Station condition requires replacement.	Coke Lift Station: Replace 200 gpm lift station with 300 gpm lift station.	$1,245,000	$2,105,000
Existing	5-10 years	PS_2	Complete Replacement of Lift Station	Lift Station condition requires replacement.	Triangle Lift Station: Replace 137 gpm lift station with 200 gpm lift station.	$1,182,000	$1,998,000
Existing	15-20 years	PS_3	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Allan Lift Station: Rehabilitate lift station. Evaluate replacing pumps with approximately 40 feet less Total Dynamic Head. Re-coat wet well.	$413,000	$698,000
Existing/Ultimate	10-15 years	PS_4	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Bridge District Lift Station: Rehabilitate lift station. Increase capacity to 1,500 gpm for 2035 projected flows. Verify that pumps will require approximately 80 feet less Total Dynamic Head as Total Dynamic Head is reduced at other pump stations.	$649,000	$1,097,000
Existing	15-20 years	PS_5	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Bridgeway Island Pump Station: Rehabilitate pump station. Confirm pump replacement with approximately 80 feet less Total Dynamic Head.	$980,000	$1,657,000
Existing/Ultimate	10-15 years	PS_6	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Bryte Pump Station: Rehabilitate pump station. Replace pump with approximately 80 feet less Total Dynamic Head and increase capacity to 3,800 gpm for future conditions. Remedy wet well coating problems and dry pit leaking.	$1,080,000	$1,826,000
Existing	10-15 years	PS_7	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Industrial Pump Station: Rehabilitate lift station. Replace pumps with approximately 60 feet less Total Dynamic Head. Replace check valves.	$654,000	$1,106,000
Existing	15-20 years	PS_8	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Jefferson Pump Station: Rehabilitate pump station. Replace pumps with approximately 60 feet less Total Dynamic Head. Re-coat sluice gates and repair dry pit leaking.	$1,098,000	$1,856,000
Existing/Ultimate	10-15 years	PS_9	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Largo Pump Station: Rehabilitate pump station. Replace pumps with approximately 60 feet less Total Dynamic Head and increase capacity to 1,050 gpm for future conditions. Transition to ultimate force main alignment.	$562,000	$950,000
Existing	15-20 years	PS_10	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Northpoint Pump Station: Rehabilitate pump station. Replace pumps with approximately 30 feet less Total Dynamic Head.	$1,116,000	$1,887,000
Existing	15-20 years	PS_11	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Parlin Lift Station: Add new pump and adjust capacity for Liberty Development planned flows. Add force main to the south and abandon existing force main to the north.	$467,000	$790,000
Existing/Ultimate	10-15 years	PS_12	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	South Pump Station: Rehabilitate pump station. Replace pumps with approximately 60 feet less Total Dynamic Head and increase capacity to 1,050 gpm for future conditions. Monitor dry pit condition and replace if necessary.	$562,000	$950,000
Existing	10-15 years	PS_13	Pump Replacement at Lift Station	Lift Station requires pump replacement for capacity/pump head enhancements.	Southport Pump Station: Rehabilitate pump station. Replace pumps with approximately 30 feet less Total Dynamic Head.	$1,362,000	$2,302,000
					Subtotal Pump Station Improvements	$11,370,000	$19,222,000
					Total Improvement Costs	$32,169,000	$54,373,500

[a] All costs shown are based on costs presented in Chapter 10 and are rounded to the nearest thousand. Costs are presented in November 2015 dollars and are based on an ENR CCI of 9846 (20-City Average).

[b] Construction cost includes a 30% construction Contingency.

[c] Capital cost includes an additional 30% implementation multiplier, in addition to the construction cost, to include all other implementation factors.

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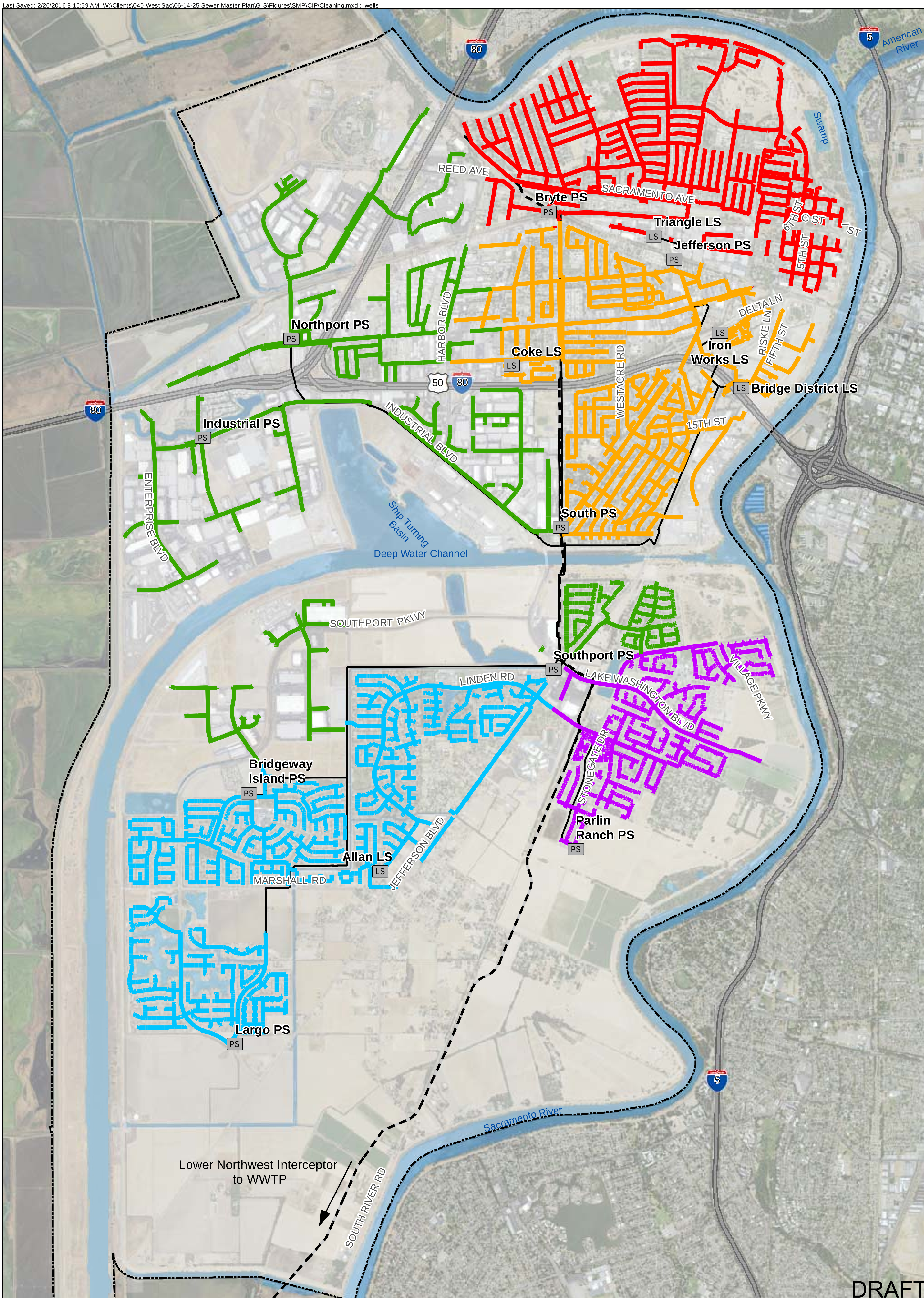

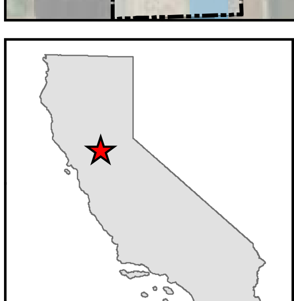

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Force Main
- Lower Northwest Interceptor

Cleaning/Inspection Year

- 1
- 2
- 3
- 4
- 5

N

0 0.25 0.5
Miles

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 10-1

Cleaning/Inspection CIP

City of West Sacramento
Wastewater Collection System
Master Plan

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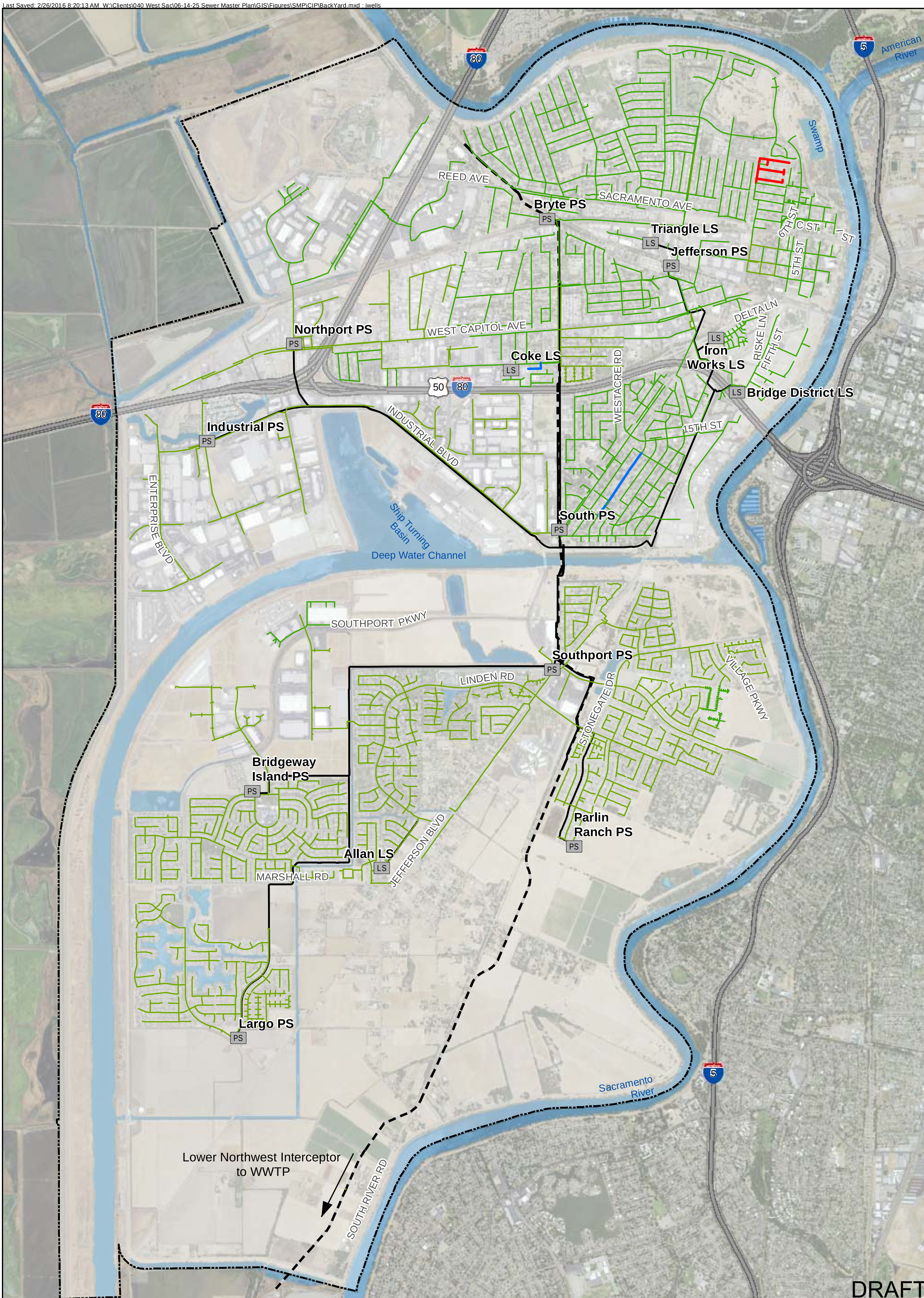

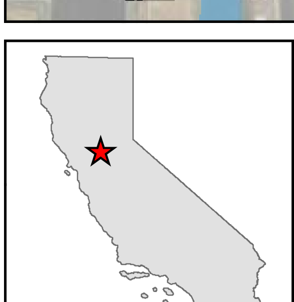

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Lower Northwest Interceptor

Backyard Main CIP Projects

- Gravity Main
- Project 1
- Project 2

0 0.25 0.5
Miles

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 10-2

Backyard Main Removal CIP

City of West Sacramento
Wastewater Collection System
Master Plan

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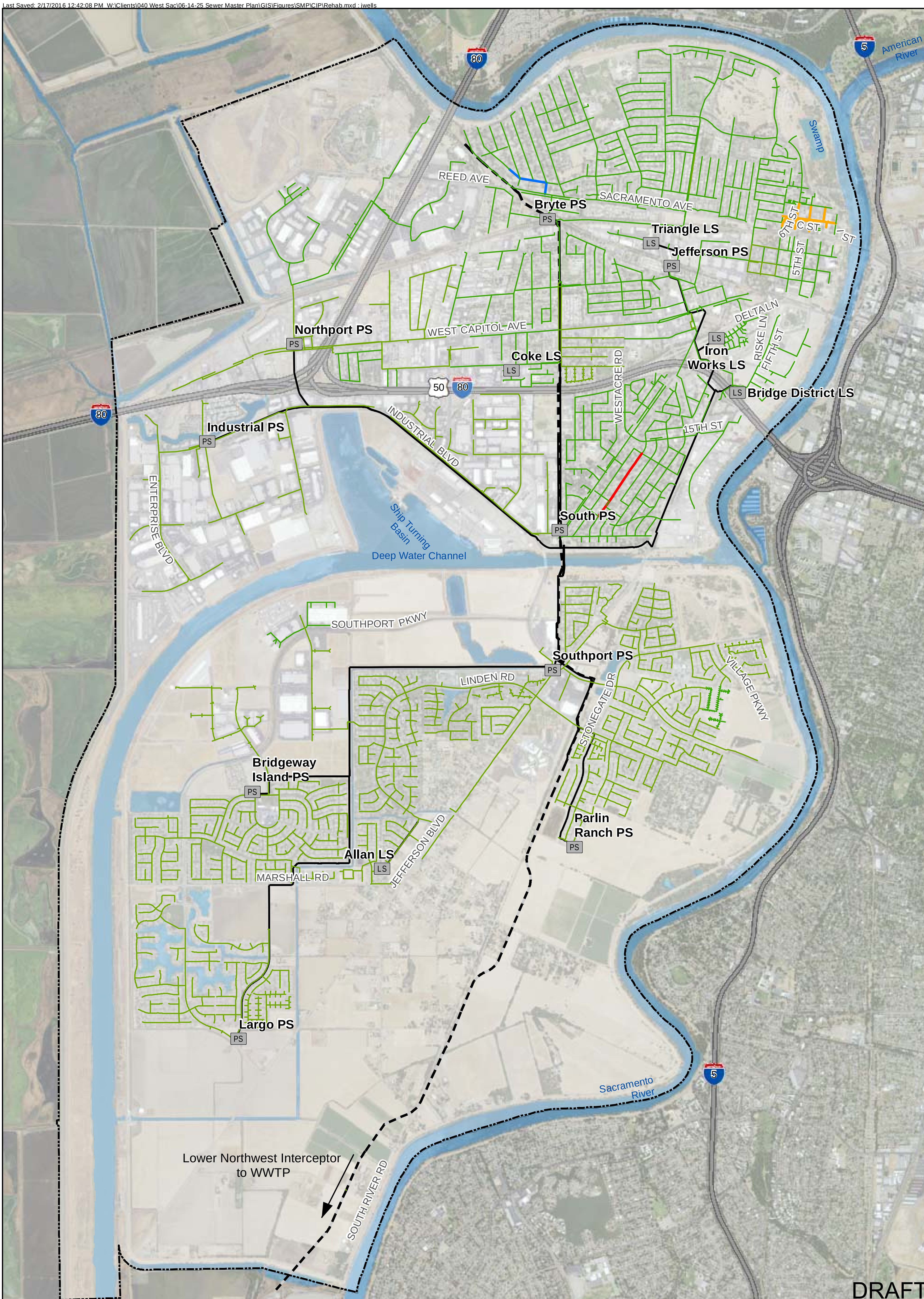

Symbology

- City Boundary
- LS Lift Stations
- PS Pump Stations
- Lower Northwest Interceptor

Rehabilitation/Repair CIP

- Gravity Main
- Project 1
- Project 2
- Project 3

N

0 0.25 0.5

Miles

CITY OF WEST SACRAMENTO

WEST YOST ASSOCIATES

Figure 10-3

Rehab/Repair CIP

City of West Sacramento
Wastewater Collection System
Master Plan

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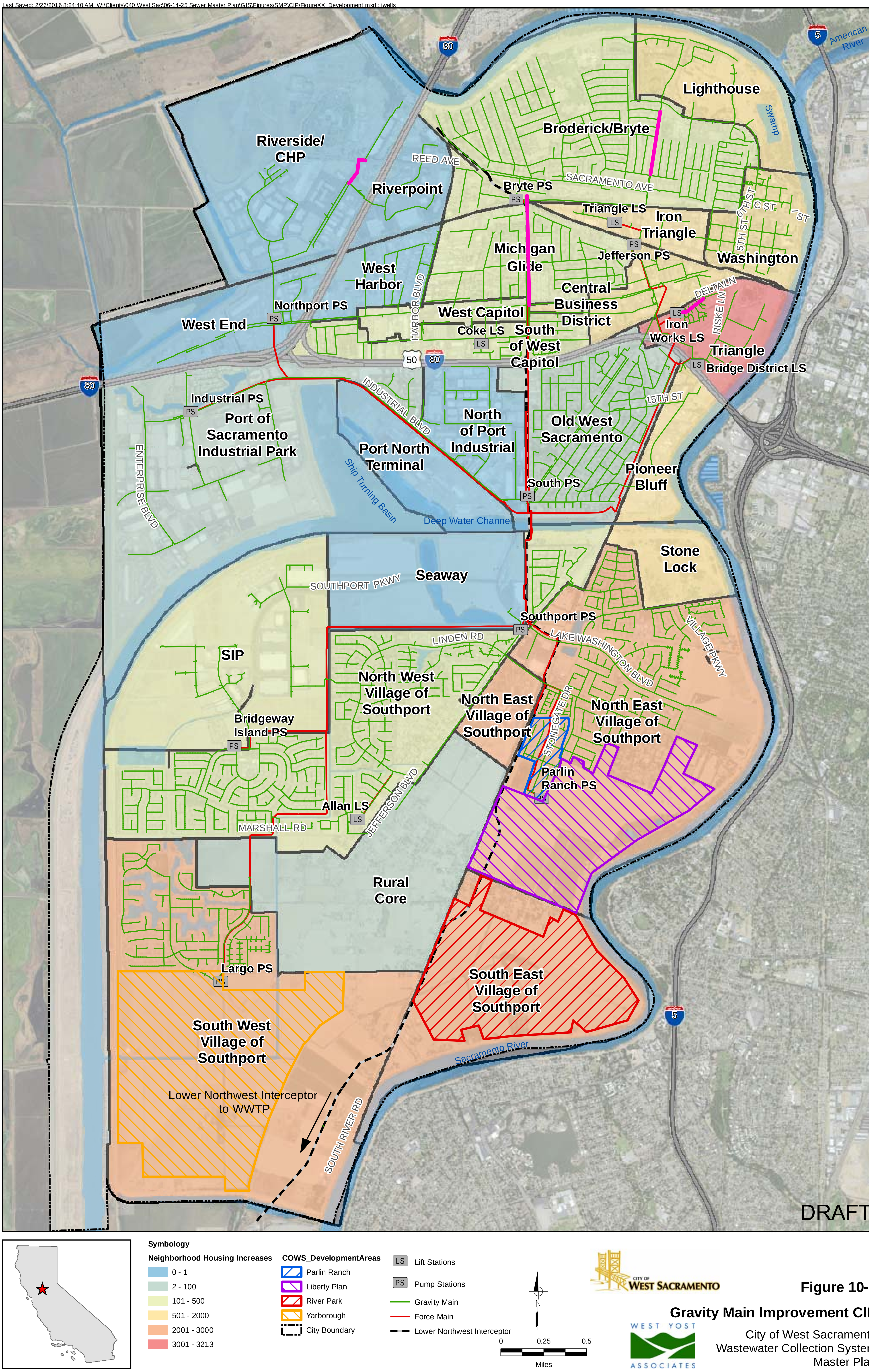

Figure 10-4

Gravity Main Improvement CIP

City of West Sacramento
Wastewater Collection System
Master Plan

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CHAPTER 11
Sewer Rate and Connection Fee Update

Chapter 11 will be submitted at a later date.

11.1 METHODOLOGY

11.2 EVALUATION

11.3 RESULTS

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WEST YOST ASSOCIATES