City of El Centro

Master Plans

SEWER MASTER PLAN

FINAL
March 2008

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City of El Centro

SEWER MASTER PLAN

INTRODUCTION

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Chapter 1

INTRODUCTION

1.1 PROJECT AUTHORIZATION AND SCOPE

This Sewer Master Plan is prepared in accordance with the agreement between the City of El Centro (City) and Carollo Engineers, P.C. (Carollo) dated October 12, 2006. It addresses the following tasks:

- Establishment of sewer system design and planning criteria,
- Evaluation of the existing sewer collection system using computer hydraulic modeling techniques,
- Determination of existing system deficiencies and recommended improvements to correct these deficiencies,
- Recommendations of improvements needed to serve anticipated growth within the Sphere of Influence, and
- Development of a Capital Improvements Program with a planning horizon of Year 2015.

1.2 BACKGROUND AND PURPOSE

The City owns and operates its sewer collection system, lift stations and wastewater treatment plant (WWTP). Previous master planning efforts include the 2001 Master Plan and a Water and Wastewater Master Plan Amendment by Nolte dated April 2004. The Master Plan presented here should aid the City in the planning, development and financing of sewer system facilities to provide reliable and enhanced service for existing customers and to serve anticipated growth. The Master Plan considers existing conditions as well as future Build Out conditions presented in the City's General Plan. Where available, specific development plans have been considered. Build Out includes expansion of the City limits within the existing Sphere of Influence (SOI).

1.3 ACKNOWLEDGEMENTS

Carollo Engineers wishes to acknowledge and thank all the City's staff for their support and assistance in completing this project. Special thanks go to Terry Hagen (Director of Public Works/City Enginer), Randy Hines (WWTP Supervisor), Paul Steward (Water Treatment Division Supervisor) and Carl Fowler (Maintenance Supervisor).

1.4 PROJECT STAFF

The following Carollo Engineers staff members were principally involved in this project:

Dennis Wood - Partner-in-Charge/Technical Review

Donnell Wilcox - Project Manager

Inge Wiersema - Project Engineer

Beth Winton - Staff Engineer

Marci Burt - Staff Engineer

Jeff Weishaar - Staff Engineer

Debra Dunn - GIS/Graphics

1.5 REPORT ORGANIZATION

This Sewer Master Plan contains eight chapters. The chapters are briefly described below.

Chapter 1 - Introduction. This chapter presents the objectives of the study. A list of abbreviations is also provided to assist the reader in understanding the information presented.

Chapter 2 - Wastewater Flows and Design Criteria. This chapter references Chapter 2 of the Water Master Plan that discusses Study Area, Land Use and Population to provide the development and justification for the modeled collection system flows. To calibrate the sewer model, unit flows for each land use were determined based on metered flows at nine locations throughout the City and the total flow at the WWTP. The capacity of the City's sanitary sewer system was evaluated based on the analysis and design criteria defined in this chapter. Historical flows at the WWTP were reviewed and analyzed to determine daily and monthly variations. The developed criteria address the sewer system capacity, acceptable gravity pipe slopes, acceptable depths of flow within pipes and daily and hourly peaking factors.

Chapter 3 - Existing Facilities. This chapter presents an overview of the City's collection system including lift stations and wastewater treatment facilities. Recommended improvements to the WWTP are also presented.

Chapter 4 - Regulatory Analysis. This chapter summarizes state and federal standards applicable to the City's outfall discharge, reviews the City's compliance history and assesses compliance trends, identifies compliance parameters and identifies potential strategies for achieving compliance. It also reviews potential future regulatory changes and emerging discharge issues that may affect future City wastewater operations.

Chapter 5 - Sewer System Evaluation and Proposed Improvements. This chapter presents the results of the capacity evaluation of the sewer system. The chapter also presents improvements to mitigate existing system deficiencies and for servicing future growth. These improvements are recommended based on the system's technical requirements, cost effectiveness and operational reliability.

Chapter 6 - Capital Improvements Program. This chapter presents the recommended Capital Improvements Program (CIP) for the City's sewer system. The program is based on the evaluation of the City's sewer system and on the recommended projects described in the previous chapters. The CIP has been prepared to assist the City in planning and constructing the sewer system improvements through the Year 2015.

1.6 ABBREVIATIONS AND DEFINITIONS

Abbreviation	Description
°F	Degrees Fahrenheit
ADF	Average Daily Flow
ADWF	Average Dry Weather Flow
BO	Build Out
BTU	British Thermal Unit
BTU/hr	British Thermal Units per hour
BWF	Base wastewater flow
Carollo	Carollo Engineers
cfm	Cubic feet per minute
cfs	Cubic feet per second
Ci	Civic
CIP	Capital Improvements Program
City	City of El Centro
CMOM	Capacity, Management, Operation and Maintenance Program
County	County of Imperial
DC	Downtown Commercial
d/D Ratio	Ratio of depth to flow to pipe diameter
du/ac	Dwelling units per gross acre
ENR	Engineering News Record
ENR CCI	Engineering News Records Construction Cost Index
fps	Feet per second
ft	Feet
GC	General Commercial
GI	General Industrial
GIS	Geographical Information Systems

gpd/ac	Gallons per day per acre
gpm	Gallons per minute
HC	Heavy Commercial
HDR	High-Medium Density Residential
hp	Horse power
H_2O Map Sewer	Hydraulic Wastewater Collection System Computer Modeling
I/I	Inflow and Infiltration
IND	Industrial
LF	Linear foot
LDR	Low Density Residential
MDR	Medium Density Residential
MG	Million gallons
mgd	Million gallons per day
msl	Mean sea level
NE	Northeastern
NW	Northwestern
PDWF	Peak Dry Weather Flow
PF	Public Facility
PI	Planned Industrial
PWWF	Peak Wet Weather Flow
ROW	Right-Of-Way
RR	Rural Residential
RTP	Regional Transportation Plan
SCAG	Southern California Association of Governments
SE	Southeastern
SOI	Sphere of Influence
TC	Tourist Commercial
WDR	Waste Discharge Requirements
WWTP	Wastewater Treatment Plant
SSMP	Sewer System Management Plan

WASTEWATER FLOWS AND DESIGN CRITERIA

2.1 STUDY AREA, LAND USE AND POPULATION

Chapter 2 in the the accompanying Water Master Plan presents a discussion of this project's planning area characteristics, land use classifications and population projections. The information was used to provide the basis for the wastewater loadings presented in this chapter. The planning area is shown on Figure 2.1. This figure also shows the collection system including pipes eight inches and larger, the locations of the lift stations and the location of the WWTP.

2.2 FLOW MONITORING PROGRAM

The process of determining wastewater loads to apply to the collection system hydraulic model includes applying measured flows to the land use categories and population data. Flows include those measured at the WWTP influent and flows from representative sewers measured over a given time period in a flow-monitoring program.

To aid in establishing average daily flows for the various land uses, a flow-monitoring program was implemented. Teledyne Isco monitored sewer flows at nine sites from Saturday, March 17, 2007 through Friday, March 30, 2007. These sites are shown in Figure 2.2. The report on compact disc and the Flow Monitoring Summary are provided in Appendix A at the end of this Master Plan. The Flow Monitoring Summary provides the Monitoring site locations and measured and calculated hydraulic data including pipe characteristics and average and peak depth and velocity.

2.3 FLOW FACTOR DEVELOPMENT

The flow monitoring results were used in conjunction with the calculated acreages of the various land uses presented in the 2004 General Plan to determine an average flow per acre. The land use areas were taken from the GIS land use layer provided by Nobel Systems. Weekday data for each monitoring site was averaged for each hour of each day. The methodology presented below outlines the steps for developing the wastewater flow factors on a basis of gallons per acre per day (gpd/ac). These values, when multiplied by the total areas in acres of their corresponding land uses, allowed comparison with the total influent flow at the WWTP.

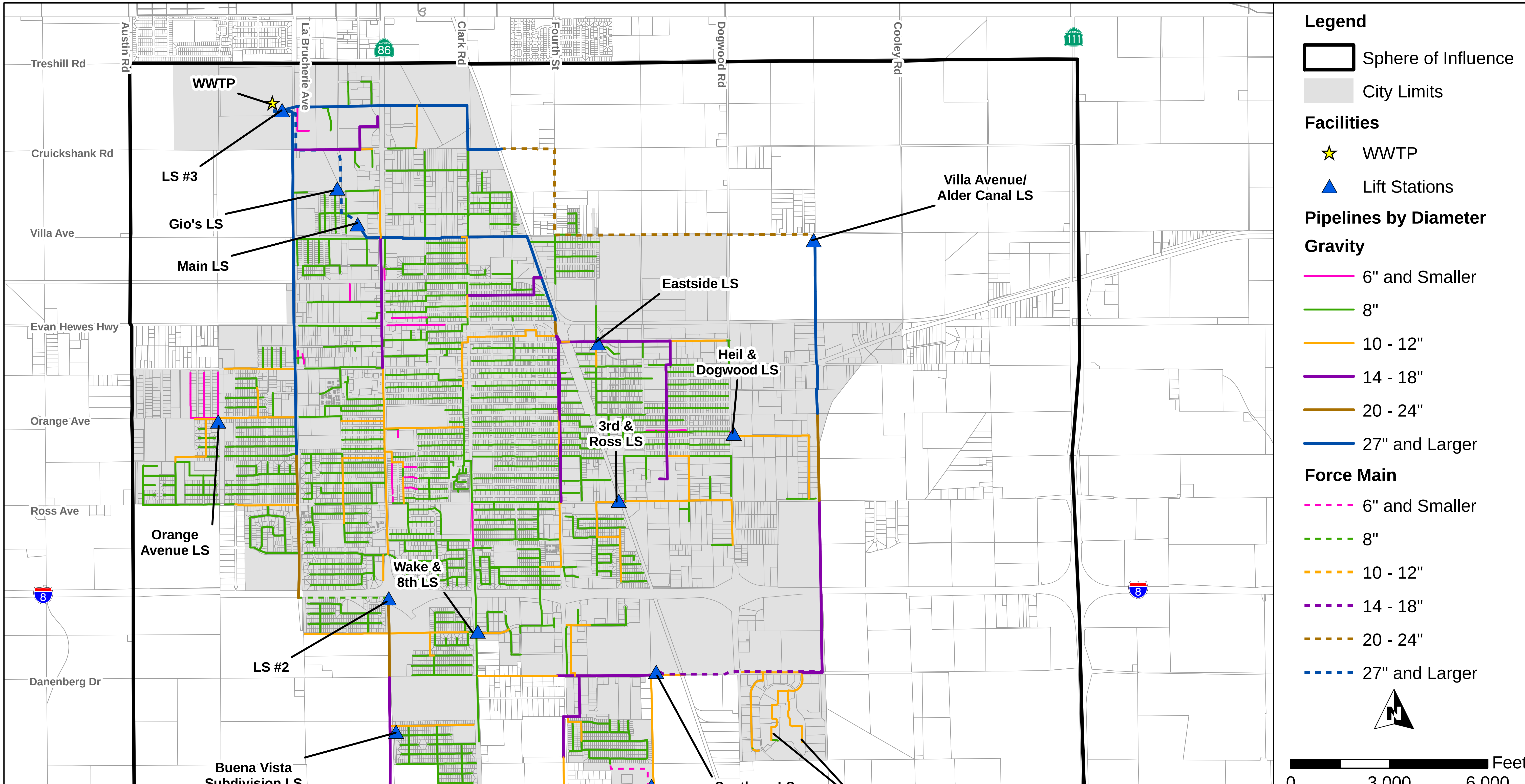


FIGURE 2.1
EXISTING WASTEWATER COLLECTION SYSTEM
SEWER MASTER PLAN
CITY OF EL CENTRO

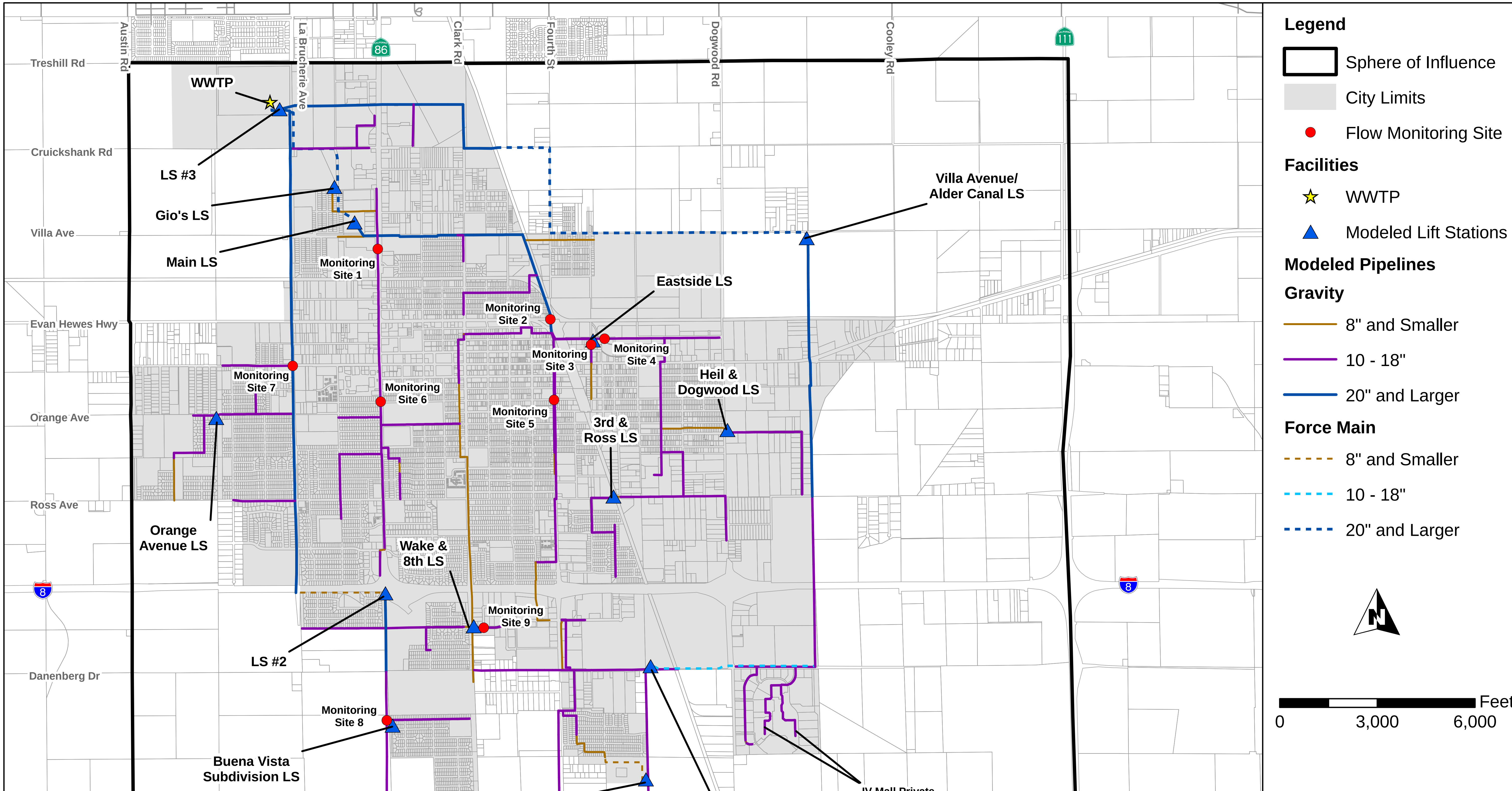


FIGURE 2.2
FLOW MONITORING SITES
SEWER MASTER PLAN
CITY OF EL CENTRO

2.3.1 Methodology

The methodology for applying wastewater loads included the following steps:

- Select a monitoring site comprised of primarily residential flows,
- Determine land uses and areas from GIS that are tributary to this monitoring site,
- Quantify the number of dwellings within each land use from an aerial photo,
- Calculate population using factors presented in Chapter 2 of the Water Master Plan,
- Adjust residential flow factors to allow total calculated flow to match the metered flow,
- Apply residential factors to other metered sites that include non-residential areas and adjust non-residential flow factors to allow the total flow to match the metered flow within reason,
- Compare the calculated flow based on the flow factors applied to the total land areas with the reported influent flow at the WWTP, and
- Adjust non-residential flow factors until the total flow matches the WWTP flow within reason.

2.3.2 Flow Monitoring Site Selection

Monitoring site 7 was selected for residential flow factor development, and monitoring site 4 was selected for commercial and industrial flow factors. These sites are shown on Figure 2.2.

2.3.2.1 Monitoring Site 7

Monitoring site 7 was located on La Brucherie Avenue between Olive and Main Streets on a 27-inch diameter trunk. The tributary area to site 7 included approximately 410 acres, which included 78 percent Low Density Residential (LDR), 10 percent Rural Residential (RR), 10 percent Public, and 2 percent General Commercial (GC). Monitoring site 8 was upstream, but these flows were subtracted from the total flows reported at site 7. Southwest High School is located within the area and has a student body population of 2,186 students, which was obtained from the Central Union School District.

The flow from Southwest High School was removed from the total flow before estimation of the residential flow factors for the RR and LDR land uses. Metcalf and Eddy's ***Wastewater Engineering, Treatment and Reuse*** reports a general flow factor of 15 gallons/day/student. Thus, approximately 32,800 gallons per day (GPD) was removed from the total metered flow. The General Commercial area was assumed to be zero for this analysis.

The RR and LDR land uses comprised the remainder of the flow. The RR land use area within the monitoring site 7 tributary area was approximately 40 acres while that for the LDR land use was approximately 320 acres. Thus, the LDR flow factor determined in the analysis would be the basis for the other residential flow factors. Although there were no set percentage goals for maintaining the relationships among the various residential flow factors, it was assumed that the flow factors would increase as the land use density increased. Also, the flow factors were compared to the water flow factors and were maintained in the range of 35 percent to 60 percent of the water factors.

2.3.2.2 Monitoring Site 4

Commercial and industrial flow factors were estimated using the flow data from monitoring site 4, which was located on Commercial Avenue between 2nd and 3rd Streets. The land use designations were a mix of residential, public commercial and industrial areas with a total tributary area of 650 acres. The General Industrial (GI) and General Commercial (GC) land uses comprised approximately 410 acres and 22 acres, respectively. The Public land use includes two schools, the Desert Oasis High School and Washington Elementary School. Flows from these schools were handled in a similar manner as monitoring site 7. In addition, residential flows were calculated using the flow factors determined from monitoring site 7. The calculated flows for the schools and residential areas were subtracted from the metered flow to determine the flow attributed to the GI and GC land uses. The flow factors for all commercial areas were assumed equal, and the flow factors for all industrial areas were assumed equal.

2.3.2.3 Comparison to Wastewater Treatment Plant Flow

After applying the residential and non-residential flow factors to the various land uses for the total developed area within the City, the calculated flow was approximately 3.9 mgd. The reported WWTP flow was 3.65 mgd for March 2007. The calculated flow is approximately 7 percent higher than the reported WWTP flow. For master planning purposes, this difference is acceptable.

Table 2.1 provides the land uses, developed areas and flow factors used in the hydraulic model. In addition, it provides the total anticipated flow in million gallons per day (mgd) for each land use and the total flow for comparison with the flow measured at the WWTP.

Table 2.1 Land Use Flow Factors Sewer Master Plan City of El Centro			
Land Use	**Flow Factor (gpd/ac)**	**Developed Area (Acres)**	**Calculated Flow (mgd)**
Rural Residential (RR)	350	163	0.06
Low Density Residential (LDR)	1,100	1,425	1.57
Medium Density Residential (MDR)	1,400	192	0.27
High-Medium Residential (HMDR)	1,600	335	0.54
General Commercial (GC)	800	550	0.44
Downtown Commercial (DC)	800	1	0.00
Tourist Commercial (TC)	800	223	0.18
General Industrial (GI)	400	696	0.28
Planned Industrial (PI)	400	115	0.05
Civic	800	46	0.04
Public	650	728	0.47
Total		**4,473**	**3.88** (1)

Notes:
1. The average monthly flow at the WWTP for March 2007 was 3.65 mgd.

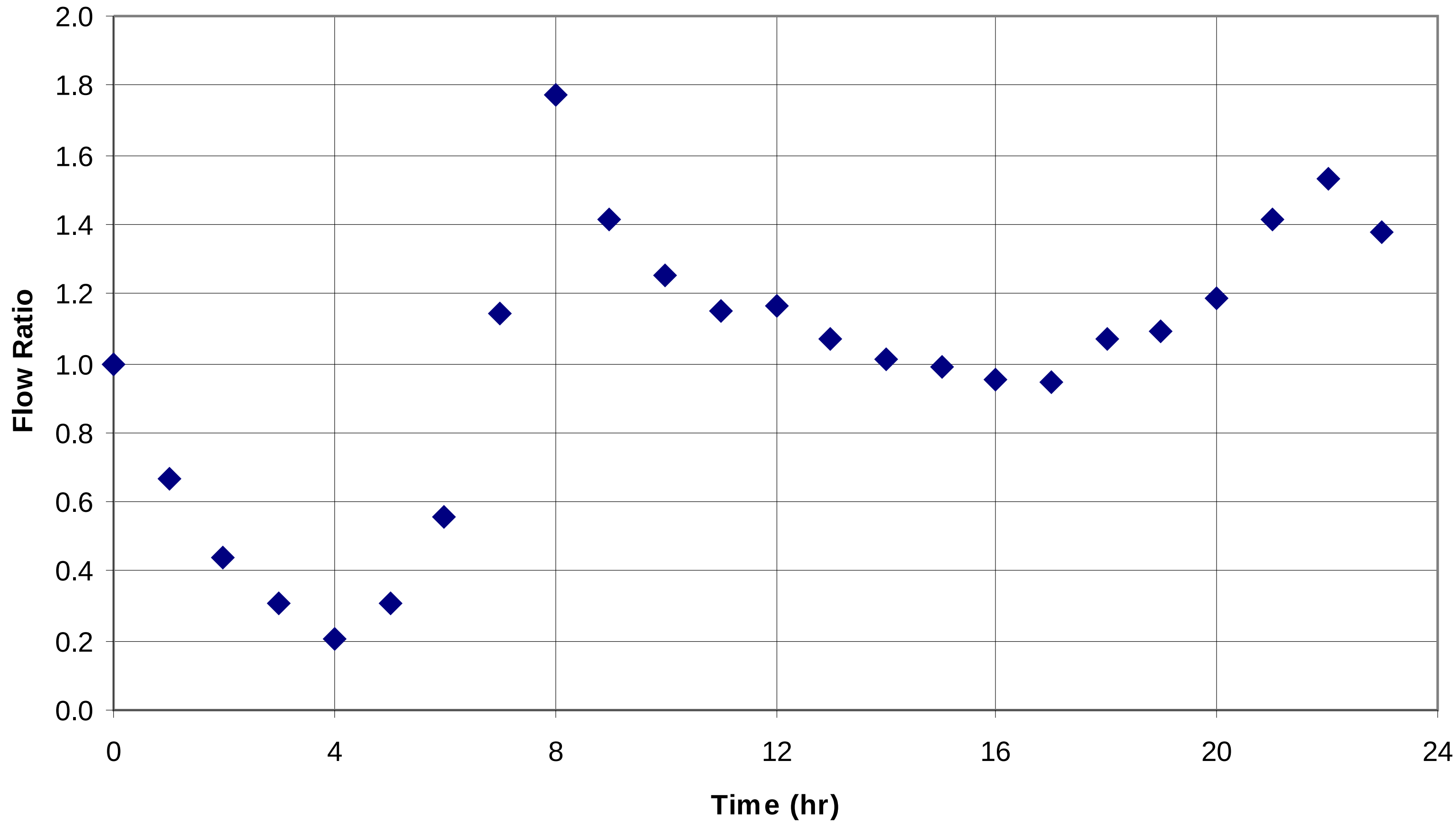
2.3.3 Diurnal Patterns

Residential and commercial loadings are fairly consistent from day-to-day and primarily vary by season. The flows follow a consistent diurnal pattern, with the peak flow typically occurring in the early to mid-morning hours.

The site metering data were used to generate diurnal patterns for residential and non-residential areas. The residential diurnal pattern was created using the metered data from monitoring site 7, a predominantly residential area. The ratio of average hourly flow to average daily flow during the weekdays was used to create the pattern. The pattern differed slightly between the weekdays and weekends because weekend peaks were slightly later in the day and the peaks were not as high. Therefore, weekend flow data was not used.

The non-residential diurnal pattern was created using the data and land use areas from meter 4. The hourly residential flow values were subtracted from the total hourly flows to give a variation based on non-residential flows. The ratio of average hourly flow to average daily flow was then used to create the non-residential diurnal pattern. The residential diurnal pattern is shown in Figure 2.3, and the non-residential diurnal pattern is shown in Figure 2.4.

FIGURE 2.3 **Residential Diurnal Pattern**
Sewer Master Plan
City of El Centro



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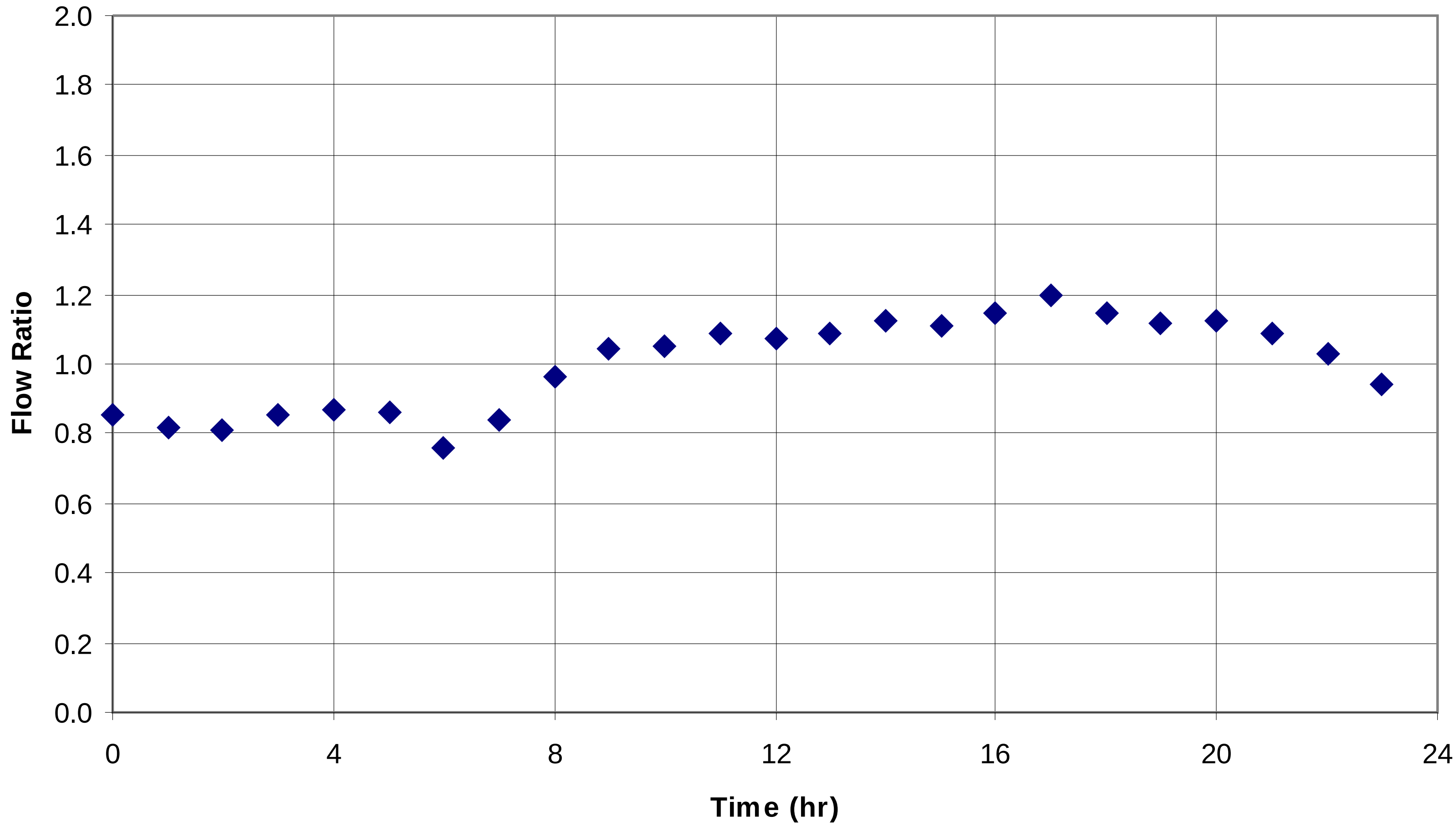
City of El Centro
Shining with Opportunity

FIGURE 2.4 **Non - Residential Diurnal Pattern**
Sewer Master Plan
City of El Centro



2.3.4 Design Flows

The sewer design flow criteria were established based on historical flows as measured at the WWTP and the flow-monitoring program. Flows are categorized as average and peak flows, with peak flows further categorized as peak dry weather and peak wet weather flows. The analyses of the existing collection system and design of future facilities are based on the peak hourly flows.

2.3.5 Average Daily Flow

The average daily flow (ADF) is the average flow collected at the WWTP for an entire year divided by 365 days per year. The flow generated from the residential, commercial and industrial users is termed base wastewater flow (BWF). Additional groundwater or storm flow that may enter sewers through pipe and manhole defects is termed infiltration/inflow (I/I). The City of El Centro experiences groundwater at sewer depths of 8 to 12 feet, but groundwater infiltration during dry weather periods would be difficult to measure separately from BWF. Thus, the combination of flow is referred to as average daily flow (ADF). Any additional flow above the ADF refers to that encountered during a storm event.

2.3.6 Peak Flows

The daily flow is sensitive to daily fluctuations such as storm events, so these are typically categorized as peak dry-weather flow (PDWF) and peak wet-weather flow (PWWF). The PDWF is defined as the maximum hourly flow measured during months when no rainfall was recorded for a given year. The PWWF is the maximum hourly flow including the inflow that would be expected during a storm event.

2.3.7 Wastewater Treatment Plant Flows

The historical monthly flows from January 2001 through December 2006 are shown on Figure 2.5. The influent flows to the WWTP and peaking factors are given in Table 2.2. The peak month factor is the ratio of the peak month flow to the ADF, and the peak day factor is the ratio of the peak day flow to the ADF. The peak month factors for the City are typical of municipal wastewater treatment plants. A peak month design value is used in the design of wastewater treatment plants and is not used in the collection system hydraulic analyses. The peak day factors vary from 1.2 to 1.5. A peak day factor of 1.2 is typical for dry-weather, and is used as the PDWF factor in the hydraulic models.

FIGURE 2.5 **Monthly and Peak Day WWTP Flows**
Sewer Master Plan
City of El Centro

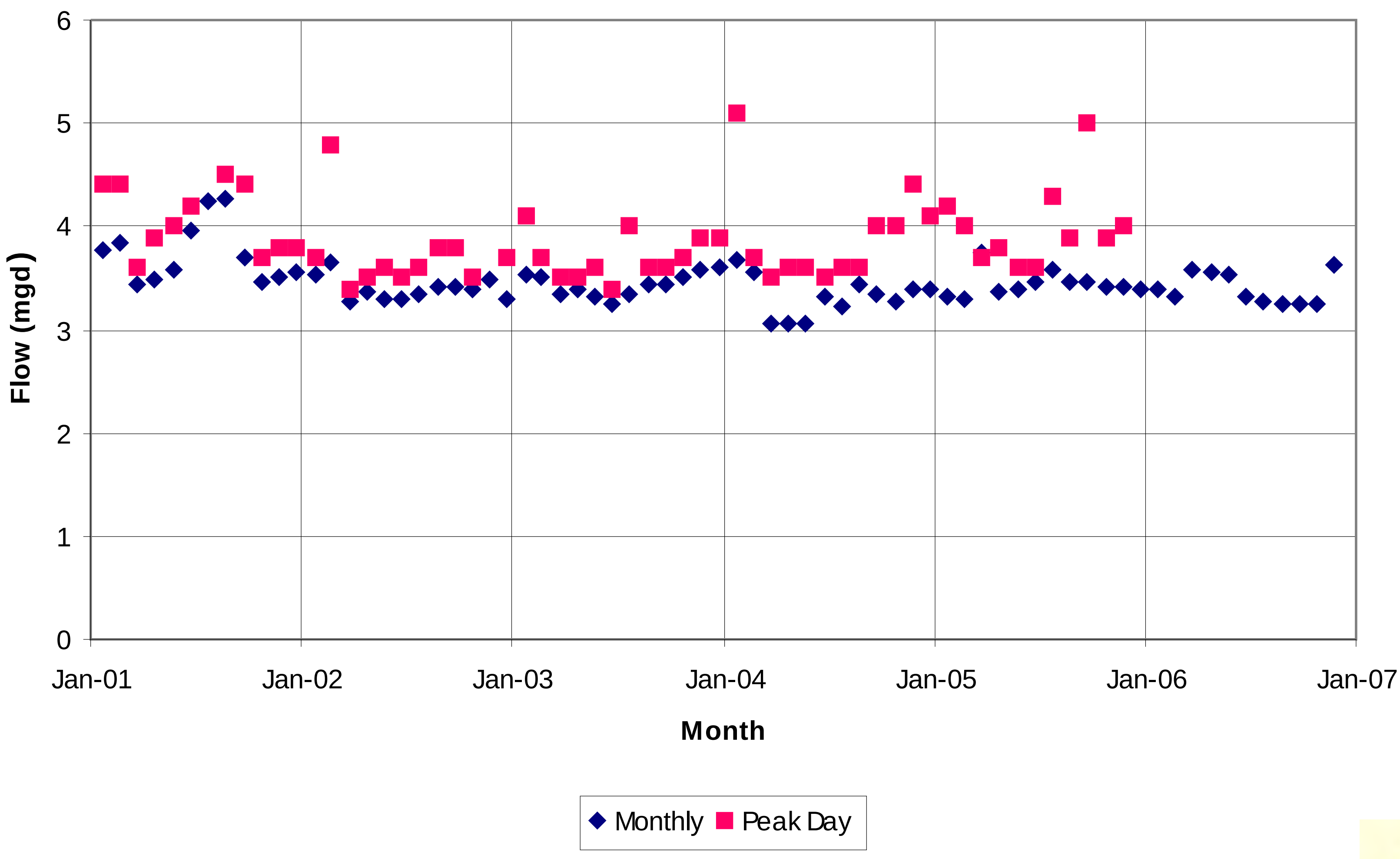


Figure 2.6 shows the monthly rainfall from January 2001 through December 2006. Figure 2.5 indicates that February 2004 has the highest peak day flow recorded in the six-year period. Figure 2.6 indicates a major rainfall event occurred in February 2004. The ratio of the peak day flow for February 2004 to the ADF is 1.5. This value is used as the PWWF factor in the hydraulic models. Table 2.3 provides the peaking factors developed for the City of El Centro. The peak hour factors take into account the maximum value from both the residential and non-residential diurnal patterns.

Table 2.2 Wastewater Treatment Plant Flows
Sewer Master Plan
City of El Centro

Year	Average Daily Flow (ADF)	Peak Month Flow (PMF)	PMF/ADF Factor	Peak Day Flow (PDF)	PDF/ADF Factor
	(mgd)	(mgd)		(mgd)	
2001	3.76	4.27	1.14	4.5	1.2
2002	3.42	3.65	1.07	4.8	1.4
2003	3.42	3.59	1.05	4.1	1.2
2004	3.33	3.68	1.11	5.1	1.5
2005	3.44	3.75	1.09	5	1.4
2006	3.40	3.63	1.07	N/A	

FIGURE 2.6 **Monthly Rainfall January 2001 - December 2006**
Sewer Master Plan
City of El Centro

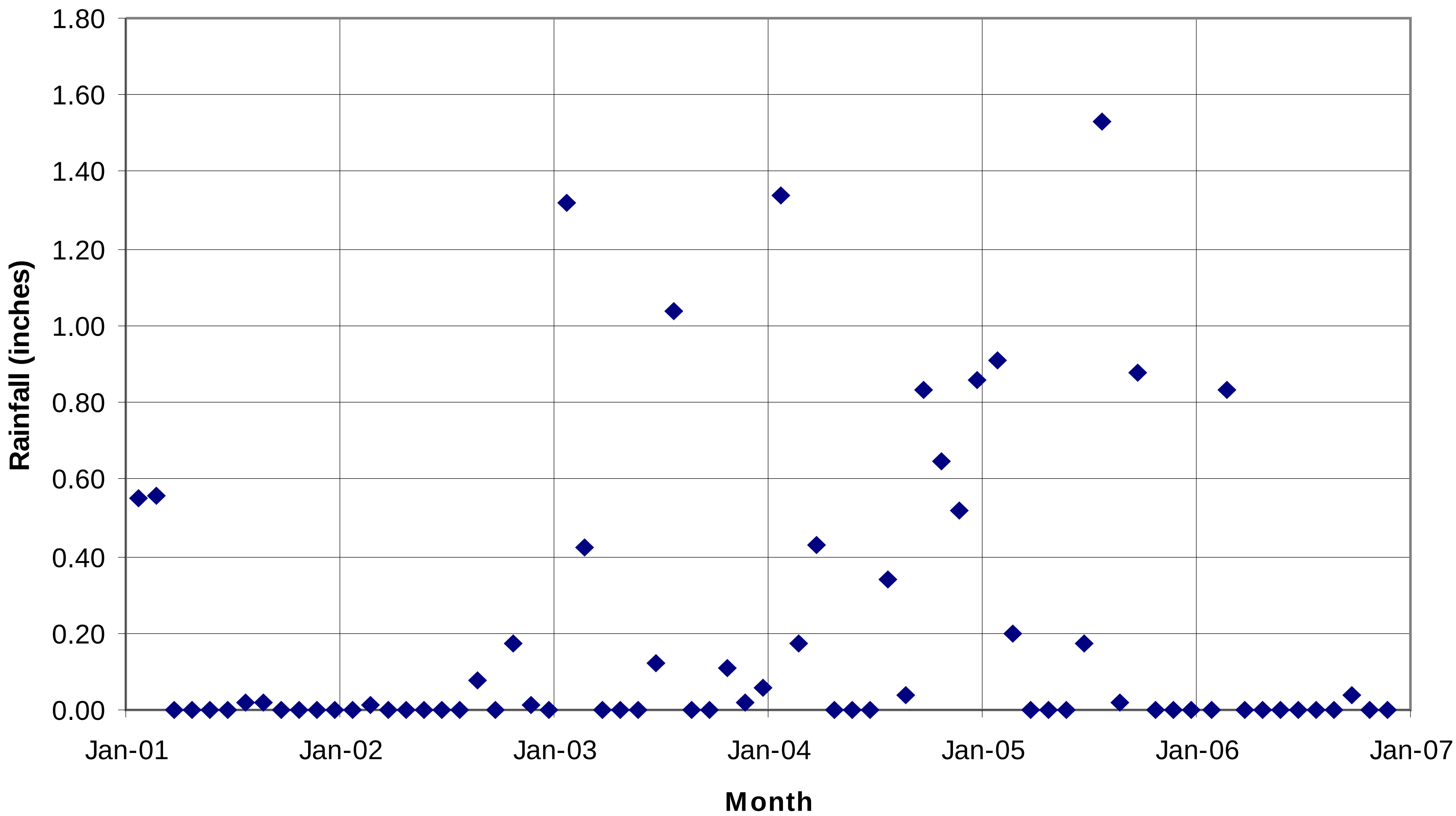


Table 2.3 Peaking Factors Sewer Master Plan City of El Centro	
Condition	**Peaking Factor based on ADF**
Average Daily Flow (ADF)	1.0
Peak Day Dry-Weather Flow (PDWF)	1.2
Peak Day Wet-Weather Flow (PWWF)	1.5
Peak Hour Dry-Weather Flow - Residential (1)	2.1
Peak Hour Dry-Weather Flow - Non-Residential (1)	1.4
Peak Hour Wet-Weather Flow - Residential (1)	2.7
Peak Hour Wet-Weather Flow - Non-Residential (1)	1.8
Notes: 1. Calculated within the hydraulic model based on the peak day factors multiplied by the maximum value from the diurnal patterns.	

2.4 DESIGN CAPACITIES

Capacities of the sanitary sewer collection system of gravity pipes, force mains and lift stations are based on the criteria described below.

2.4.1 Gravity Sewers

Sewer pipe capacities are dependent on many factors. These include roughness of the pipe, maximum allowable depth of flow and limiting velocity and slope. The Continuity Equation and the Manning Equation for steady-state flow are used for gravity sewer hydraulic calculations. The Continuity Equation is:

$$Q = V \times A$$

Where:
Q = peak flow, cubic feet per second (CFS),
V = velocity, feet per second (fps), and
A = cross-sectional area of pipe, square feet (SF).

The Manning Equation is represented as:

$$V = \frac{1.49}{n} \times R^{2/3} \times S^{1/2}$$

Where: V = velocity (fps)
N = Manning's coefficient of friction
R = hydraulic radius (area divided by wetted perimeter), feet, and
S = slope of pipe, feet per feet.

A typical "n" value used for design and analysis is 0.013. This value is used in this master plan.

The d/D ratio, or depth of flow to diameter of pipe, is important not only for analyzing the existing system, but also for designing new pipes for future developments. This value establishes the capacity of the existing or new pipe. Ratios typically range from 0.5 to 1.0 with the lower values used for smaller pipes, which may experience flow peaks greater than planned or may experience blockages. Higher values may be used for larger pipe to prevent premature or unnecessary replacement of existing pipes.

The dry-weather d/D ratio used in the hydraulic models for analysis of existing pipes was 0.75. For wet-weather flows, the ratio was 0.9.

Table 2.4 provides the maximum d/D ratios for new pipes. If the hydraulic model indicated that a pipe has a d/D ratio greater than the value listed, it was considered surcharged, or flowing greater than its intended capacity.

Table 2.4 Maximum d/D Ratio for New Pipes
Sewer Master Plan
City of El Centro

Pipe Size (inches)	Dry Weather (inches/inches)	Wet Weather (inches/inches)
8 - 10	0.5	0.9
12 - 24	0.75	0.9

In order to minimize the settling of sewage solids, standard practice dictates a minimum velocity in the pipeline of two feet per second (fps) when the pipe is flowing half full. At this velocity, the sewer flow will typically provide self-cleaning for the pipe. The velocity of flow when the pipe is half-full approaches the velocity when the pipe is full. This is true because the hydraulic radius, or R from the Manning Equation, is the same for both half full and full flow. Table 2.5 provides the slopes required for various pipe sizes with a Manning's "n" of 0.013 to achieve a velocity of two fps when the pipe is flowing half full. These slopes apply to new pipe only that will serve future developments. The reported slopes are the minimum slopes to achieve the flushing velocities. Although it is possible to lay the larger diameter pipes at the shallow slopes indicated, a greater slope of at least 0.0015 feet/feet should be considered where feasible.

Table 2.5 Minimum Recommended Slopes for New Circular Pipes Sewer Master Plan City of El Centro	
Pipe Size	**Minimum Slope [1]**
(inches)	**(feet/feet)**
8	0.0020 [2]
10	0.0020
12	0.0020
15	0.0015
18	0.0010
21	0.0009
24	0.0008

Notes:
1. To maintain velocity of 2 fps using an “n” value of 0.013 and d/D ratio of 0.5.
2. Slope is less than recommended due to the flat topography surrounding the City.

2.4.2 Lift Stations and Force Mains

Lift stations were evaluated and sized for peak flow with one standby pump. Several lift station pumps operate with variable frequency drives to maintain a wet well level. These include Lift Station No. 3, Main Lift Station and the Alder Canal Villa Avenue Lift Station. The remaining lift stations operate by level control.

The recommended minimum diameter for new force mains is six (6) inches for raw wastewater. The minimum and maximum recommended velocities are 2 fps and 6.5 fps, respectively. The Hazen-Williams formula is commonly used for the design of force mains. It calculates the headloss through a given length of pipe. The roughness coefficient “C” varies by pipe material. The type of construction and age of pipe also influence this coefficient. For all new force mains designed for this Master Plan, a “C” value of 120 is used.

The Hazen-Williams formula is represented as:

$$H = \frac{4.52 \times Q^{1.85}}{C^{1.85} \times D^{4.8655}}$$

Where: H = Headloss in pounds per square inch per foot of pipe,
Q = Flowrate in gallons per minute,
C = Roughness coefficient, and
D = Pipe diameter in inches.

EXISTING FACILITIES

This chapter describes the service area and the wastewater collection system, treatment facilities and treated wastewater disposal. The collection system is described in terms of the individual sewer basins, the major trunk lines, and the lift stations.

3.1 SERVICE AREA

The City of El Centro is located in Imperial County in the southern most region of California. In 2008, the City will celebrate its 100th year. It encompasses over 10.75 square miles and the elevation is below sea level. The climate is temperate in the winter but with summer temperatures that average over 90 degrees Fahrenheit.

This growth results in increased wastewater flows that can affect the overall collection and treatment system. This Master Plan is an important step to assure that the City's facilities have sufficient capacity for the planning period.

3.2 COLLECTION SYSTEM

3.2.1 Sewer Trunks

The City maintains over 125 miles of sewer lines. Gravity pipe sizes range from 4 inches to 36 inches in diameter. Force mains range from 4 inches to 30 inches in diameter. Three sinkholes were reported in 2004 due to collection systems pipe failures on May 19, 2004, May 24, 2004, and October 10, 2004. Three sections of collapsed sewer lines were successfully repaired. Three manholes and approximately 100 feet of sewer lines were replaced. Sewer service was not disrupted during the repairs.

The City plans to replace sewer trunk along Imperial Avenue north of Interstate Highway 8. Capacity in the 12-inch and 15-inch pipes near Wilson Junior High School and the Central Union High School has been reached possibly due to the condition of the pipes. In addition, the City plans to replace the sewer trunk along 8th Street north of Interstate Highway 8. This truck includes 6-inch diameter pipe that should be upsized to match pipe diameters upstream and downstream of this 6-inch segment.

3.2.2 Lift Stations

There are thirteen lift stations within the City. The locations of these stations are shown on Figure 2.1. The capacity of each station is listed in Table 3.1.

Table 3.1 Lift Station Summary
Sewer Master Plan
City of El Centro

Station Name	Atlas Facility ID	Age	Station Firm Capacity [1]	Duty Pumps	Standby Pumps	Force Main Diameter
		(years)	(mgd)	(No.)	(No.)	(inches)
Lift Station No. 3	25-11101	2	25.9	2	1	24
Alder Canal Villa Ave	23-17101	3	5.5	1	1	20
Main Lift Station	23-12101	75	7.3	2	1	30
Eastside Lift Station	22-15101	95	1.4	1	1	10
Lift Station No. 1	14-12101	33	0.6	1	1	6
Lift Station No. 2	18-12101	33	0.6	1	1	8
Wake and Eighth	17-13101	2	0.3	1	1	6
Gios	24-12101	5	0.4	1	1	4
Third and Ross	19-15101	20	0.5	1	1	4
Heil and Dogwood	20-16101	10	0.4	1	1	6
Buena Vista	16-12101	2	0.5	1	1	8
Orange Avenue	20-11101	3	0.4	1	1	4
Countryside South	--- [2]	5	0.1	1	1	6
Southern	--- [2]	New	3.85	2	1	14

Notes:

1. Firm capacity assumes largest pump out-of-service.
2. Lift station not listed in sewer atlas

3.3 WASTEWATER TREATMENT PLANT

3.3.1 Plant Description

The City's WWTP is a conventional primary/secondary plant followed by disinfection by ultraviolet (UV) irradiation. The original plant consisted of primary clarifiers followed by ponds. An activated sludge process replaced the ponds in 1972. The aeration basins have been further modified to replace mechanical turbine aeration with fine bubble aeration. The plant originally was constructed with a chlorine contact basin for disinfection. When the Board imposed disinfection requirements, the chlorination system was removed from operation and UV facilities were added.

The rated capacity of the plant is 8.0 million gallons per day (mgd). This flow is for any 30-day reporting period. The maximum month flow in 2006 was 3.63 mgd. The average annual flow for this same period was 3.40 mgd. The treated wastewater is discharged to Central Main Drain. This drain ultimately discharges into the Salton Sea.

A design is being completed to add a redundant gravity belt thickener, a redundant belt filter press, and a redundant UV channel. The parallel UV channel will allow plant staff to periodically clean the channel and the UV unit. This is important to prevent growth of algae that can affect the performance of the UV disinfection. The design criteria are summarized in Table 3.2.

Solids handling consists of thickening by gravity belt thickeners, anaerobic digestion, digested sludge dewatering by a belt filter press, and further dewater and storage in sludge drying beds.

Table 3.2 Existing WWTP Facilities
Sewer Master Plan
City of El Centro

Description	Units	Criteria
Primary Clarifier	Number	2
Diameter	Feet	80
Surface Area, each	Square Feet	5,027
Sidewater Depth	Feet	10
Aeration Basins	Number	6
Active Volume	Gallons/basins	280,538
Total Active Volume	MG	1.68
Dimensions	Feet x Feet	50 x 50
Water Depth	Feet	15
Anoxic Zone		
Volume	Gallons	280,538
Aerobic Zone		
Volume	Gallons	1,402,690
Diffuser Type		Ceramic
Aeration Basin Blowers	Number	3
Capacity, each	cfm	1250-7500
Horsepower	HP/blower	250
Secondary Clarifiers	Number	4
Diameter	Feet	80

Table 3.2 Existing WWTP Facilities Sewer Master Plan City of El Centro		
Description	**Units**	**Criteria**
Surface Area	Square Feet	5027
Sidewater Depth	Feet	11.5
Return Activated Sludge (RAS) Pumps	Number	2+1 Standby
Capacity	mgd/pump	3.96
Horsepower	Hp/pump	50
Waste Activated Sludge (WAS) Pumps	Number	2
Capacity	gpm/pump	500
Horsepower	HP/pump	10
Gravity Belt Thickener	Number	1
Hydraulic Capacity	gpm	250
Digesters	Number	3
Digester 1:		
Diameter	Feet	60
Sidewall Depth	Feet	28
Digester 2 & 3		
Diameter	Feet	40
Sidewall Depth	Feet	18.5
Belt Press	Number	1
Size	Meter	3
Hydraulic Capacity	gpm	125
Ultraviolet Lamps	Number	32
Output	mW/cm^2	0-300

3.3.2 General Plant Condition

The wastewater treatment facility is well operated and maintained. However, there are issues that are related to age and to original design and construction. Some of these issues should be addressed to prevent further deterioration. The following summarizes the result of a plant inspection performed by Carollo Engineers in February 2007.

3.3.3 Primary Clarifiers

There are two, circular primary clarifiers. Both units were in operation during the inspection, and it was not possible to inspect below the water line. However, it appeared that the coatings are in good condition.

The concrete does not show any signs of corrosion due to hydrogen sulfide. However, they exhibit some concrete cracking. This condition is more related to the poor construction technique as compared to deterioration due to the wastewater or hydrogen sulfide. An example of this cracking is shown on the following photo. The walls for both clarifiers were rebuilt in 1989. Vertical cracks were repaired in 1994. The horizontal cracking shown in the photo is typical of both clarifiers on both the top and bottom of the walls at various locations and are attributable to poor repair procedures. These cracks should be corrected to prevent further advancement. In addition, the patching for the previous repairs should be removed and replaced.

3.3.4 Secondary Treatment

There are six aeration basins in operation, and are operated in series mode. The aeration basins also exhibit considerable concrete cracking along the walkways and original mechanical aerator platforms as shown in following photo. This condition is probably due to original construction and design. As with the primary clarifiers, the concrete walls should be repaired to prevent further deterioration. The aerator platforms should be removed.

With the lack of preliminary treatment, large solids can travel through the primary clarifier to the aeration basins. An example of the solids accumulation is shown on the following photo. These must be removed periodically by hand.

The individual basins require periodic isolation for cleaning and maintenance of the ceramic diffusers. The original gates consisted of aluminum stop plates with aluminum frames. In 1989, the effluent stop gates for each basin were replaced with fiberglass gates, and in 1994, the gates on the northwestern two basins were replaced with stainless steel gates. Many of the gates on the remaining four basins are now inoperable. These gates will need to be replaced with new stainless steel slide gates.

The existing design does not have automatic dissolved oxygen control. This can promote nitrification during the night when flows are reduced allowing dissolved oxygen concentrations to increase. Dissolved oxygen control is recommended to reduce energy costs and to provide better process control.

The plant staff has replaced the ceramic fine bubble diffusers in one of the basins. The aeration system is 12 years old, and the diffusers in the remaining basins should be replaced.

The secondary clarifiers appear in good condition.

3.3.5 Anaerobic Digestion

There are three anaerobic digesters. Except for the age of the facilities, there were no apparent structural or mechanical issues.

The anaerobic digesters need to operate at a temperature of at least 95 degrees Fahrenheit. During the hottest month, the wastewater temperature reaches this value. Sludge heating is required during the other months.

3.3.6 Biosolids Disposal

Primary sludge is pumped directly into the anaerobic digesters for stabilization. Waste activated sludge is concentrated by a gravity belt thickener and then pumped to the digesters. The digested sludge is dewatered by a belt filter press and trucked to on-site drying beds. It is stored for approximately one year to reduce the moisture content to below 10 percent. A private contractor hauls the dried sludge to Arizona. It is then land applied for soil amendment.

3.3.7 Land Outfall

The final effluent is discharged to the Central Main Drain north of the plant. The Central Main Drain conveys the effluent for eight miles to the Alamo River, which then flows 39 miles to the Salton Sea.

3.4 WASTEWATER TREATMENT PLANT PERFORMANCE

The capacity of the El Centro wastewater plant is 8.0 mgd and consistently meets Secondary Treatment standards. Each process will be addressed below with an overview of treatment performance.

3.4.1 Plant Flows

The City's population in 2006 was 41,778. Based on the reported Plant flow of 3.40 mgd and the population for 2006, the per capita wastewater generation rate is approximately 81 gallons per capita per day (GPCD). Although this per capita rate is within the range of rates of other Southern California cities, it is not the basis for determining future flows to the wastewater treatment plant. These flows are based on the land uses and flow factors presented in Chapter 2. Table 5.1 in Chapter 5 presents the future flows used in the hydraulic models based on these values.

The capacity of the current plant will not be adequate for the Build Out time frame. Planning must begin for the next expansion when the monthly flow reaches 6.4 mgd, or 80 percent of the plant's capacity of 8.0 mgd. This will occur well beyond the time frame of this master plan, so facilities for a plant expansion will not be considered here. The characteristics of the influent flow and the impact on the processes will be considered.

3.4.2 Wastewater Characteristics

This section summarizes the wastewater characteristics in terms of the two main loading components, biochemical oxygen demand (BOD) and total suspended solids (TSS). These characteristics will be used in evaluating the future wastewater characteristics. Most of the additional flows are expected to occur in this tributary area. Further, the future industrial discharges are expected to locate in this area. The influent BOD and TSS are reported in Table 3.3.

Table 3.3 Influent BOD and TSS 2001 - 2006
Sewer Master Plan
City of El Centro

Year	2001	2002	2003	2004	2005	2006
BOD (mg/L)						
Annual Average	220	224	214	226	227	243
TSS (mg/L)						
Annual Average	207	197	188	215	216	207

The values in Table 3.3 are average values for a wastewater treatment plant. The monthly trends for influent BOD and TSS are shown on Figures 3.1 and 3.2, respectively.

3.4.3 Primary Clarification

There are two primary clarifiers, each 80 feet in diameter. Primary clarifiers are designed to remove approximately 35 percent of the influent BOD and 65 percent of the influent TSS. The actual Primary Clarifier BOD and TSS influent and effluent values in pounds per day are shown graphically on Figures 3.3 and 3.4 for the years 2001 through 2006. Over the time period analyzed, the actual removal efficiencies averaged 32 percent for BOD and 61 percent for TSS as reported by the City. The average removal rates are within normal parameters.

3.4.4 Secondary Treatment

Six aeration basins are in operation. One basin is used periodically as an anoxic zone by reducing the amount of oxygen flow into that basin. This practice can aid in control of filamentous organisms.

The aeration basins average a detention time of 12 hours with all six basins in service. This is in industry standards for aeration basins. Five basins are aerated while the remaining basin acts as an anoxic zone. The average BOD loading from 2001 through 2006 has been approximately 33 pounds of BOD per 1000 cubic feet (#BOD/kCF) with five basins in service. With one of the aerated basins out-of-service, this value rises to approximately 41 #BOD/kCF.

There are four secondary clarifiers. Currently there are three units in operation. Secondary clarifiers are designed for an overflow rate of 600 to 800 gpd/sf. The historical overflow rate is reported in Figure 3.5. The actual overflow rate is much lower than design limits.

The effluent BOD and TSS values for the period of 2001 to 2006 are shown on Figures 3.6 and 3.7, respectively.

The City has previously violated waste discharge standards for Escherichia coli (E. coli). These violations are discussed further in Chapter 4, Regulatory Requirements. The City's E. coli records from 2001 to 2006 are shown in Figure 3.8.

3.4.5 Digestion

There are three digesters at the WTTP. Digester No. 1 is the largest and newest unit. The digesters should provide at least 20 days of hydraulic detention time to assure that EPA 503b requirements for Class B bio-solids are met.

Digester No. 1 is operated as the primary digester. Primary sludge and thickened waste activated sludge are pumped to this unit. The discharge flows to Digesters 2 and 3, which act as secondary digesters. Digester No. 1 alone averages a detention time of 20-35 days as can be seen in Figure 3.9. The detention time for Digesters Nos. 2 and 3 average an additional time of 6 to 16 days as shown in Figure 3.10. Figure 3.11 provides the combined detention time for all three digesters.

FIGURE 3.1 **Monthly Influent Biochemical Oxygen Demand 2001 - 2006**
Sewer Master Plan
City of El Centro

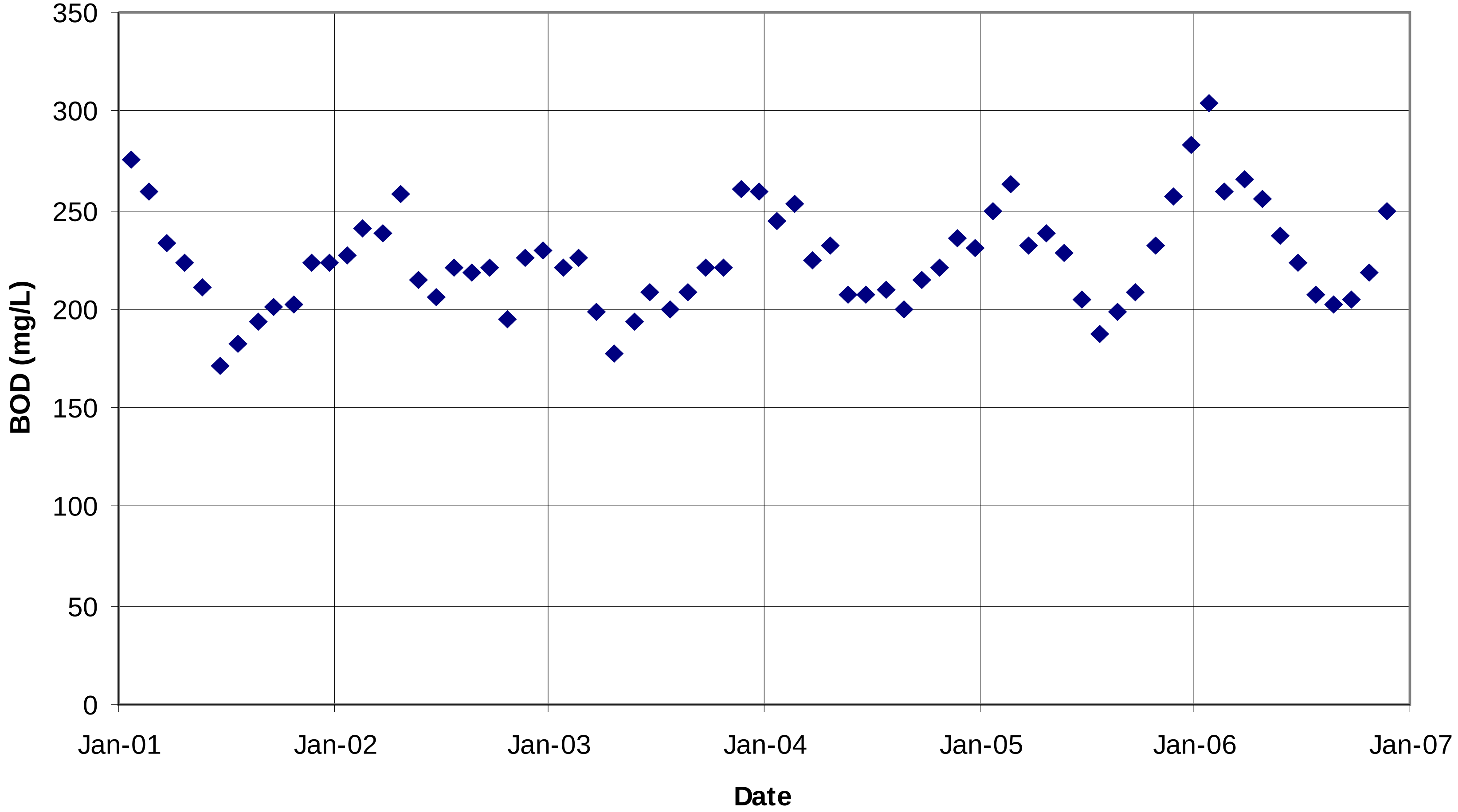


FIGURE 3.2 **Monthly Influent Total Suspended Solids 2001 - 2006**
Sewer Master Plan
City of El Centro

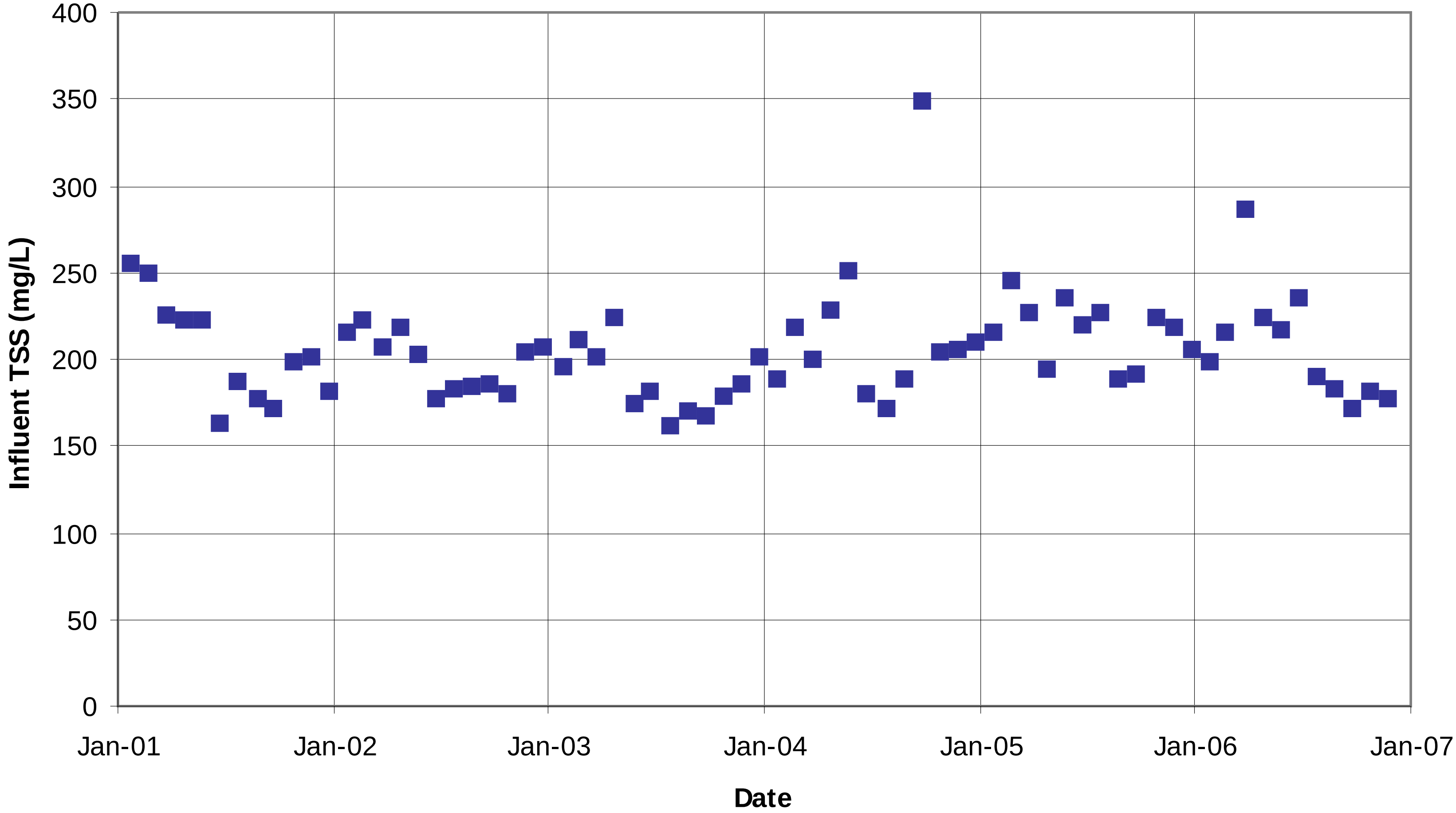


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FIGURE 3.3 **Primary Clarifier Influent and Effluent BOD**
Sewer Master Plan
City of El Centro

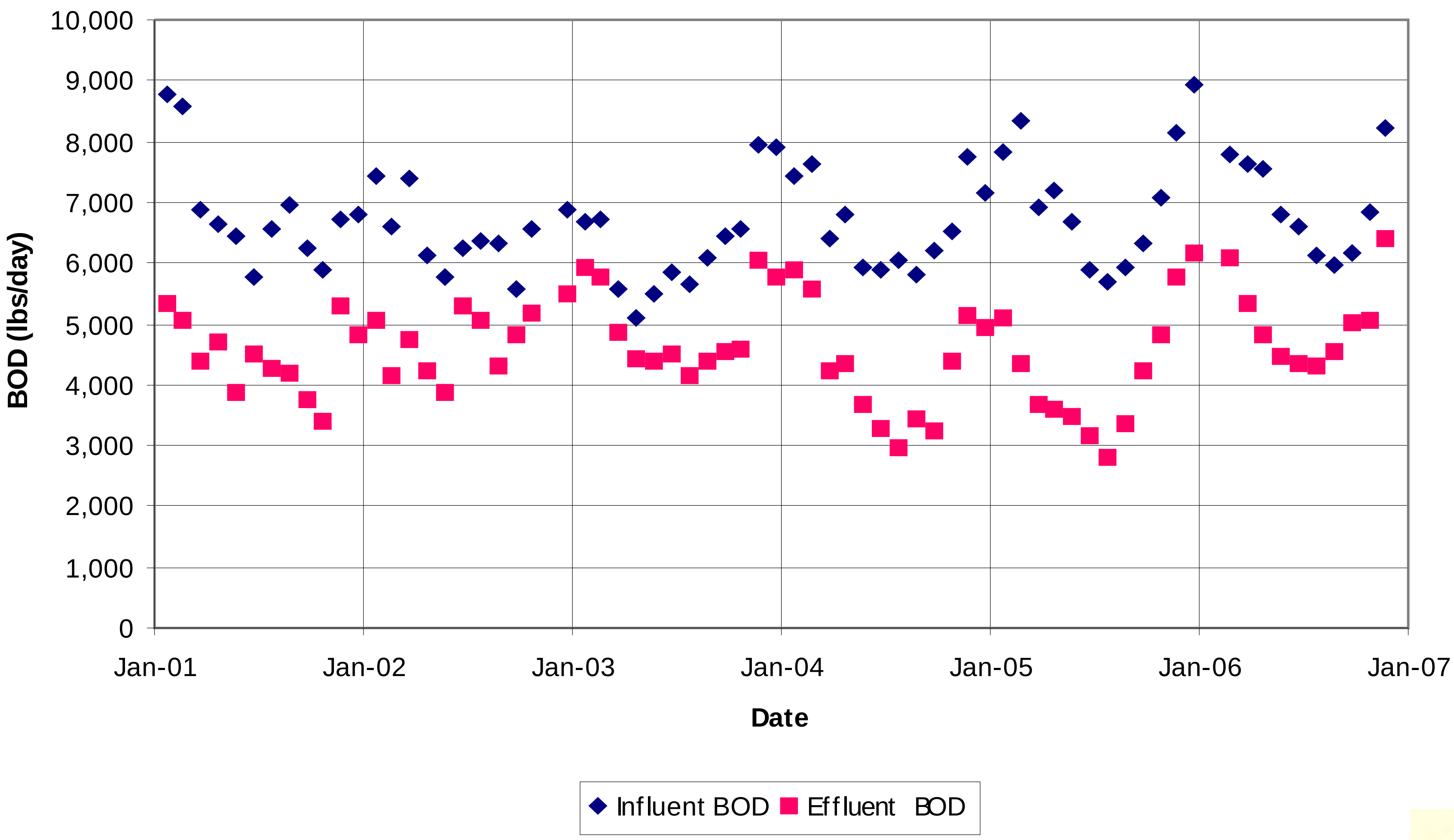

FIGURE 3.4 **Primary Clarifier Influent and Effluent TSS**
Sewer Master Plan
City of El Centro

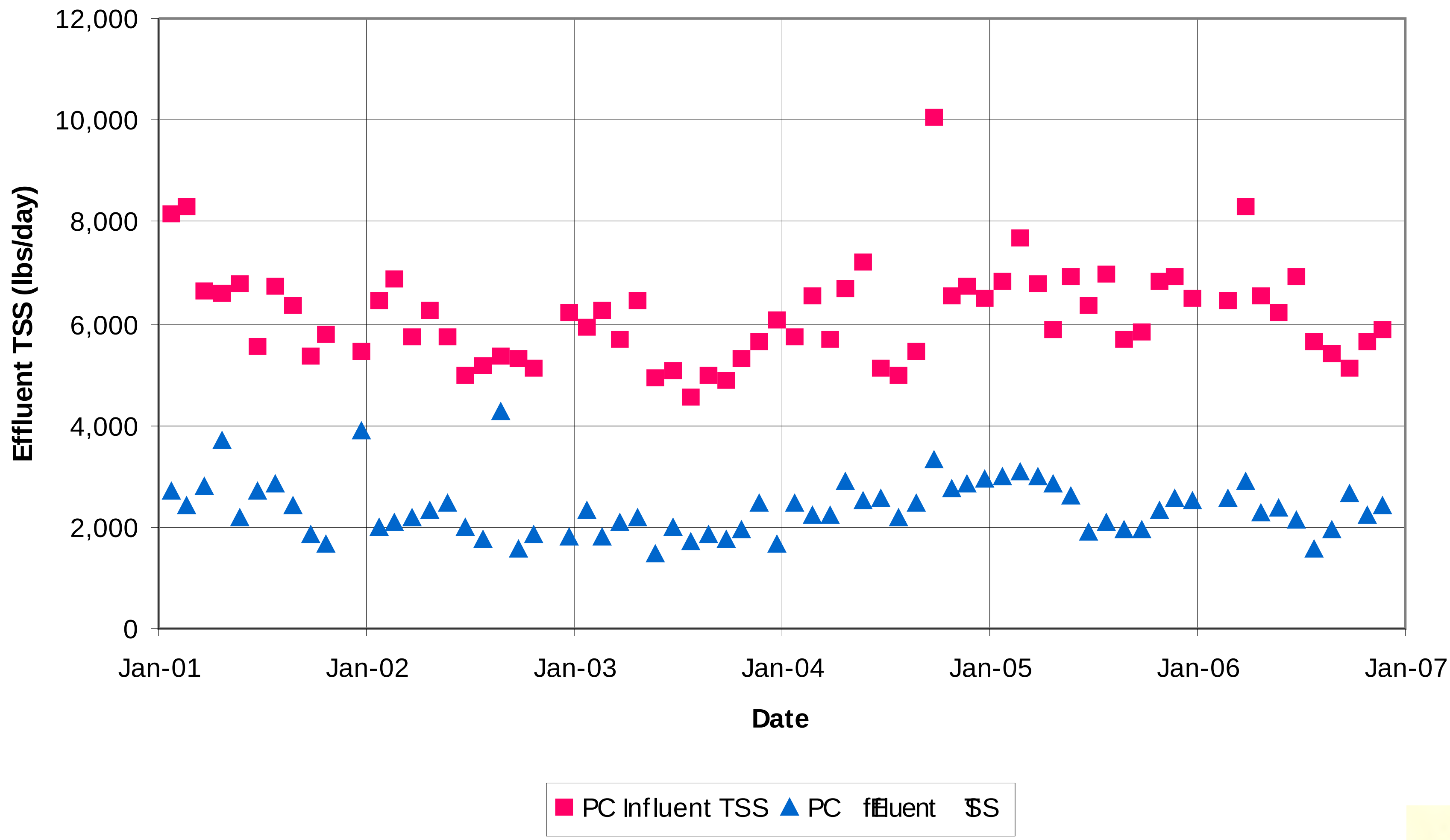

FIGURE 3.5 **Secondary Clarifier Surface Overflow Rate**
Sewer Master Plan
City of El Centro

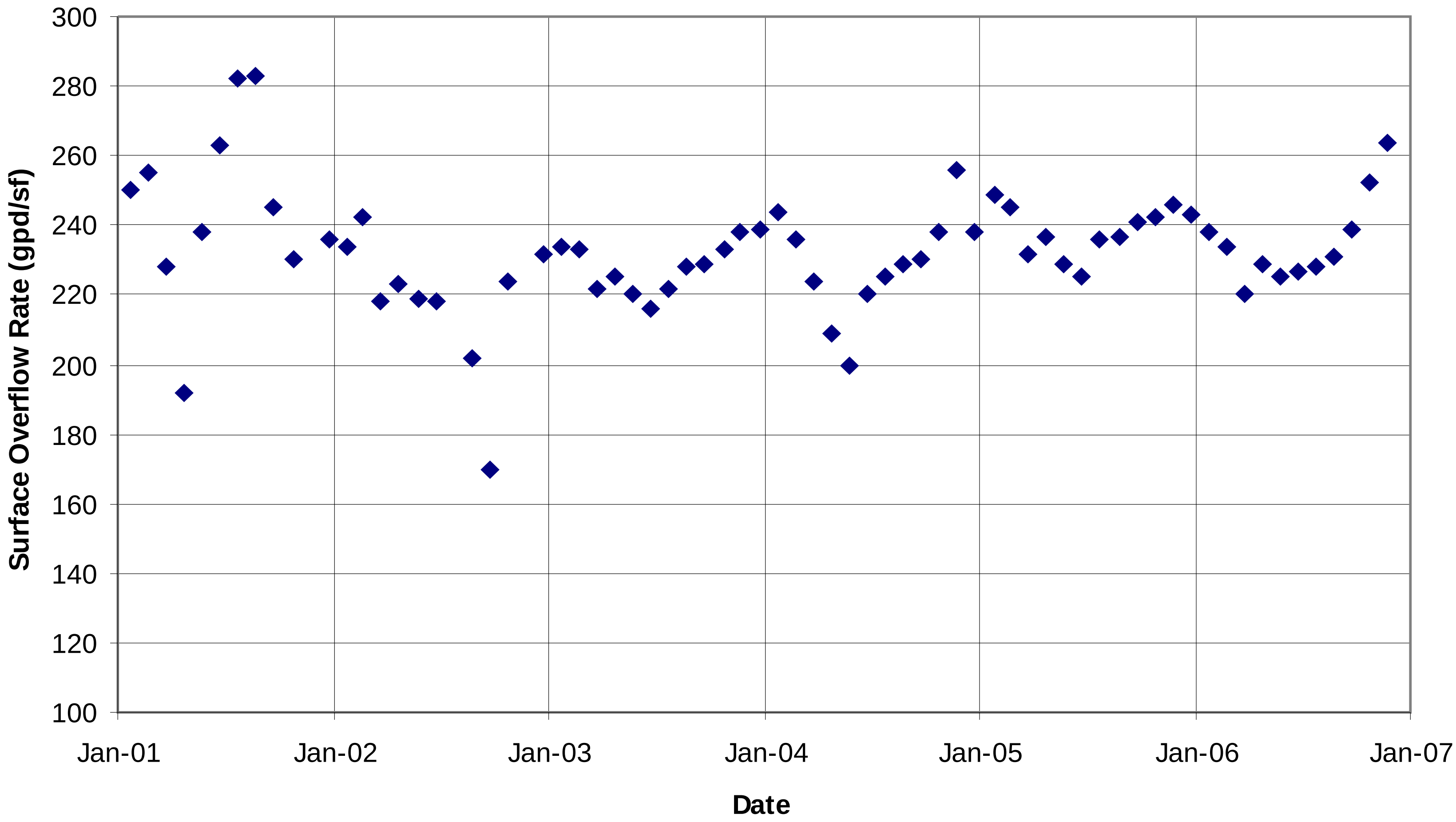

FIGURE 3.6 **Effluent BOD**
Sewer Master Plan
City of El Centro

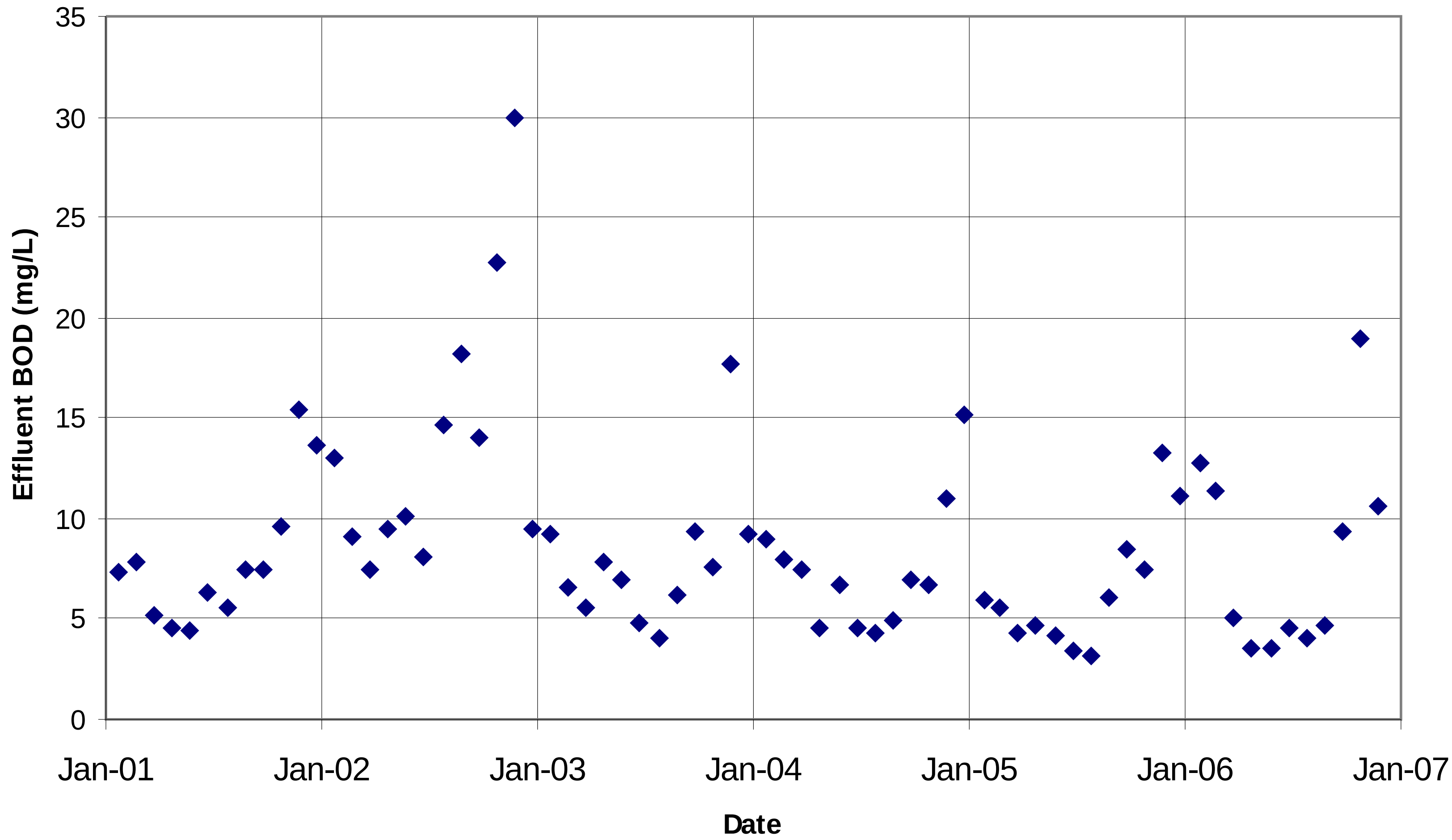

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FIGURE 3.7 **Effluent TSS**
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City of El Centro

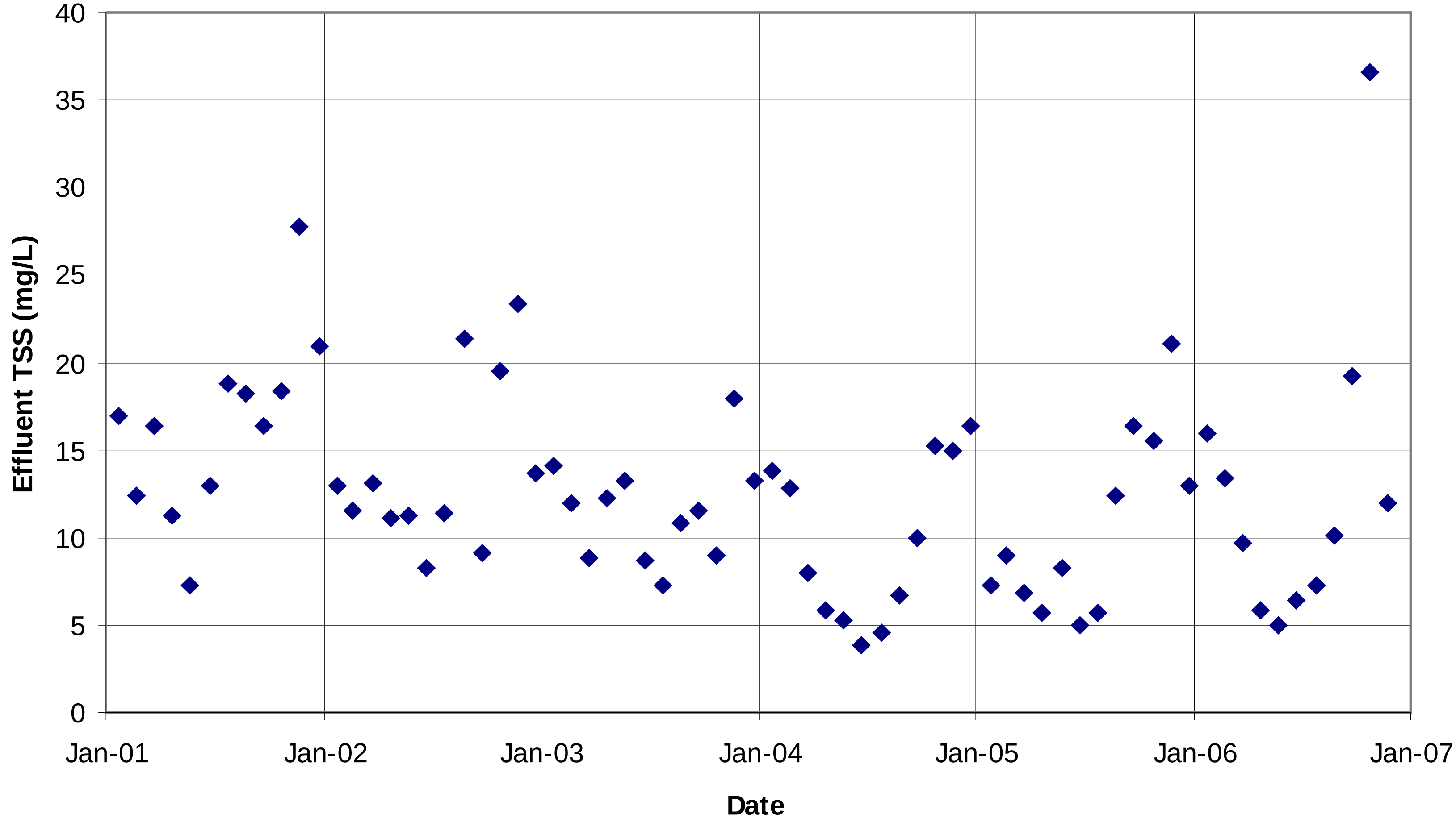

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FIGURE 3.8 **Effluent E. Coli**
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City of El Centro

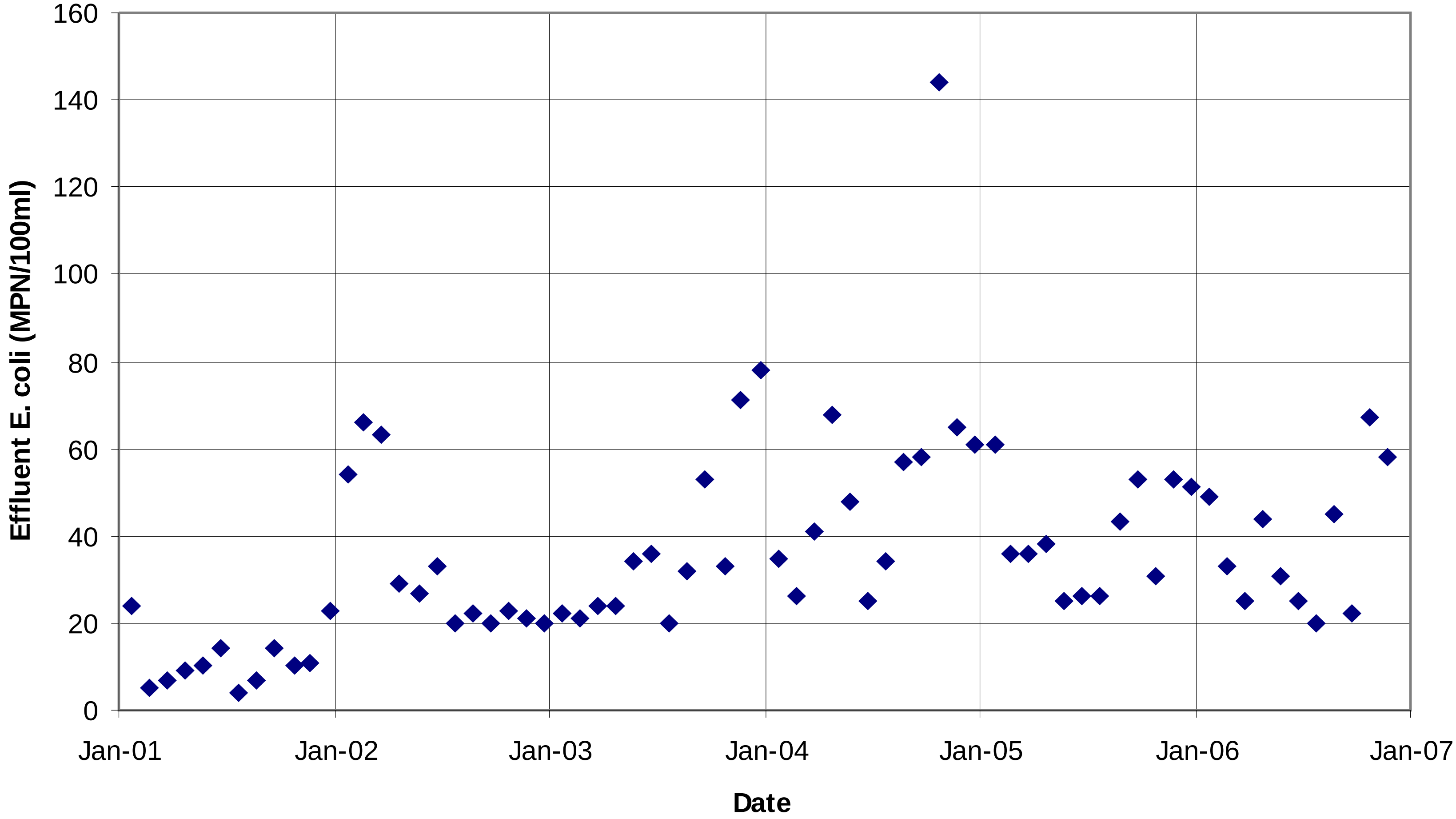

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FIGURE 3.9 **Digester 1 Average Dentention Time**
Sewer Master Plan
City of El Centro

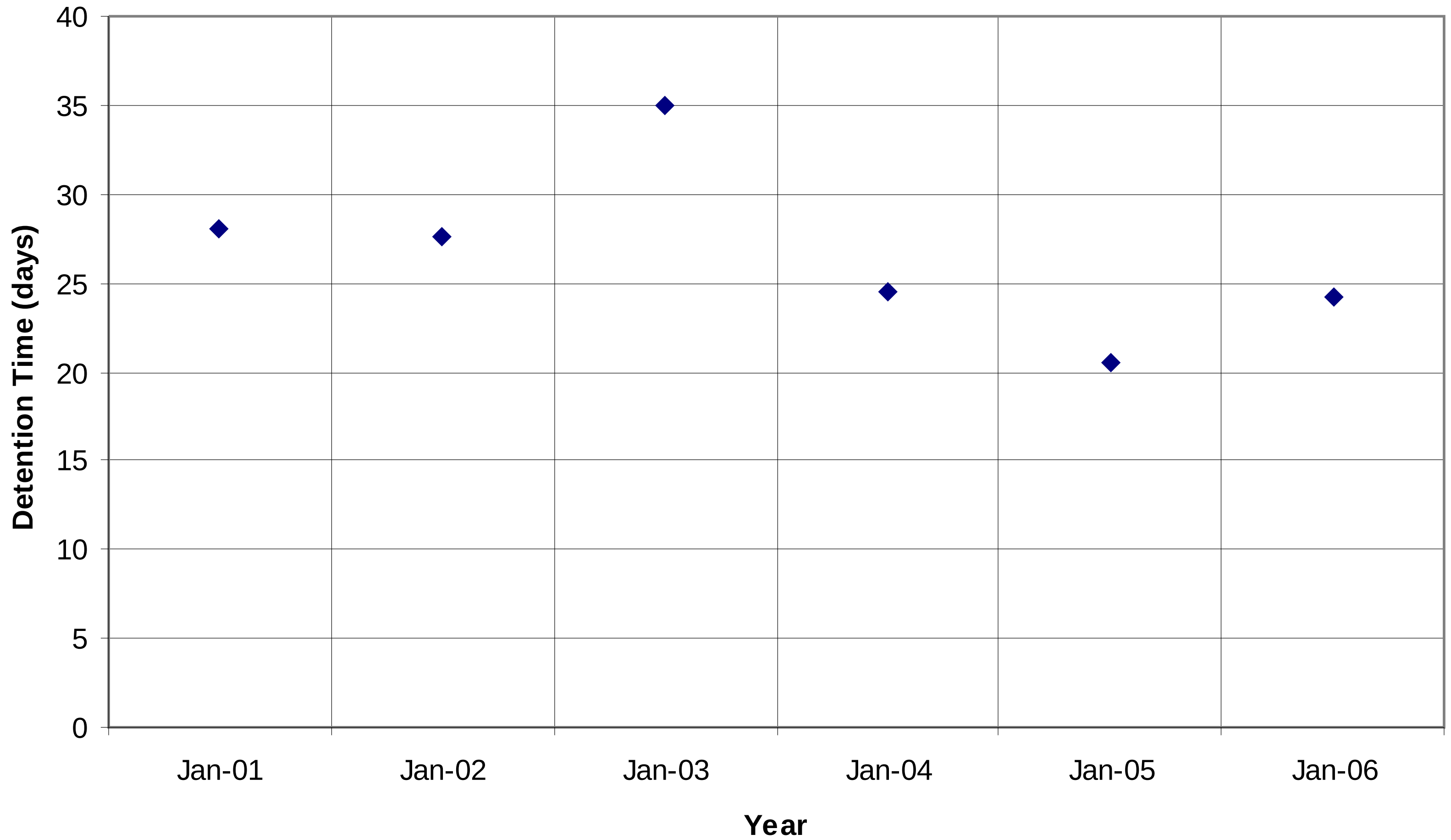

FIGURE 3.10 **Digesters 2 and 3 Average Detention Time**
Sewer Master Plan
City of El Centro

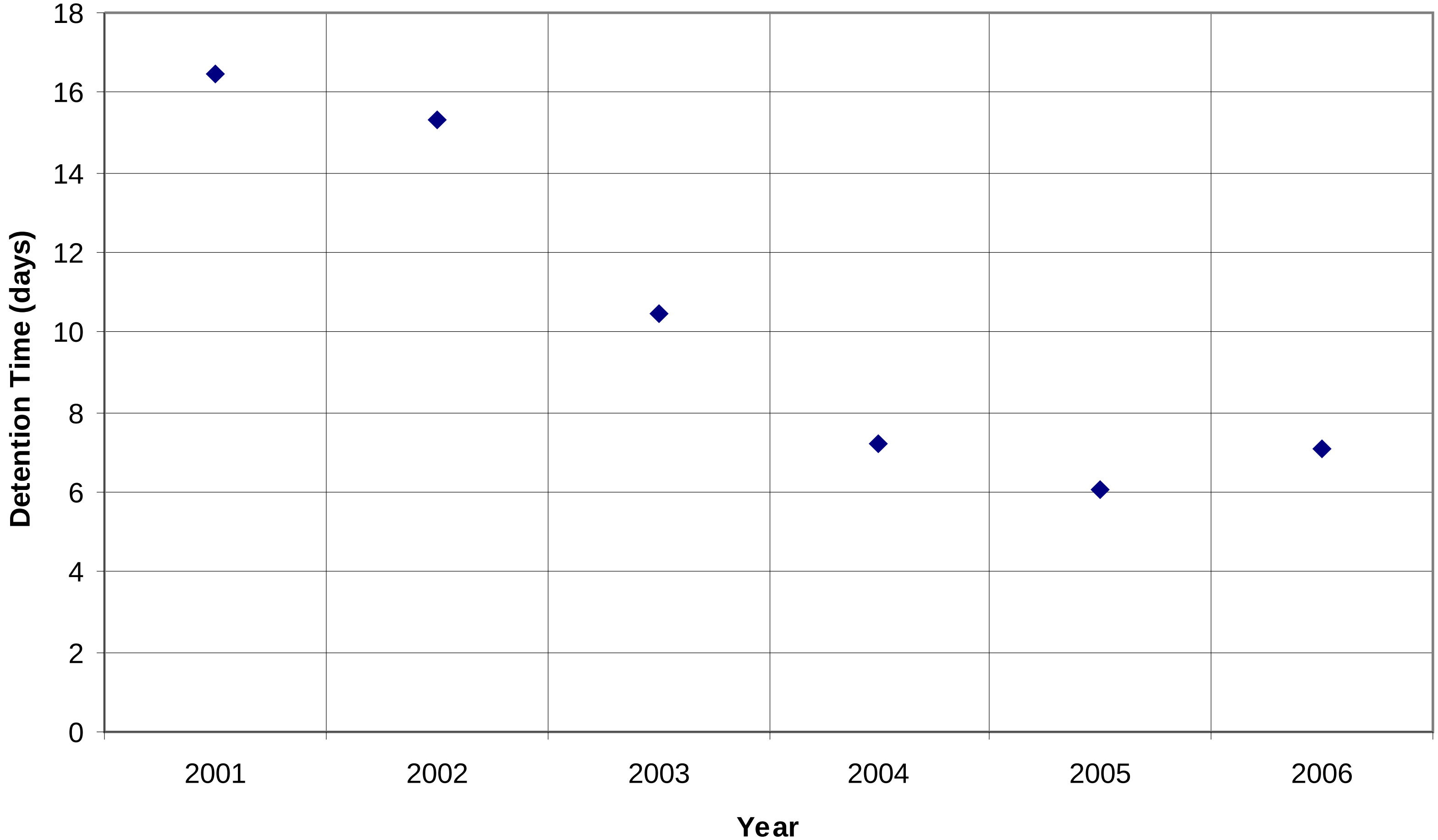

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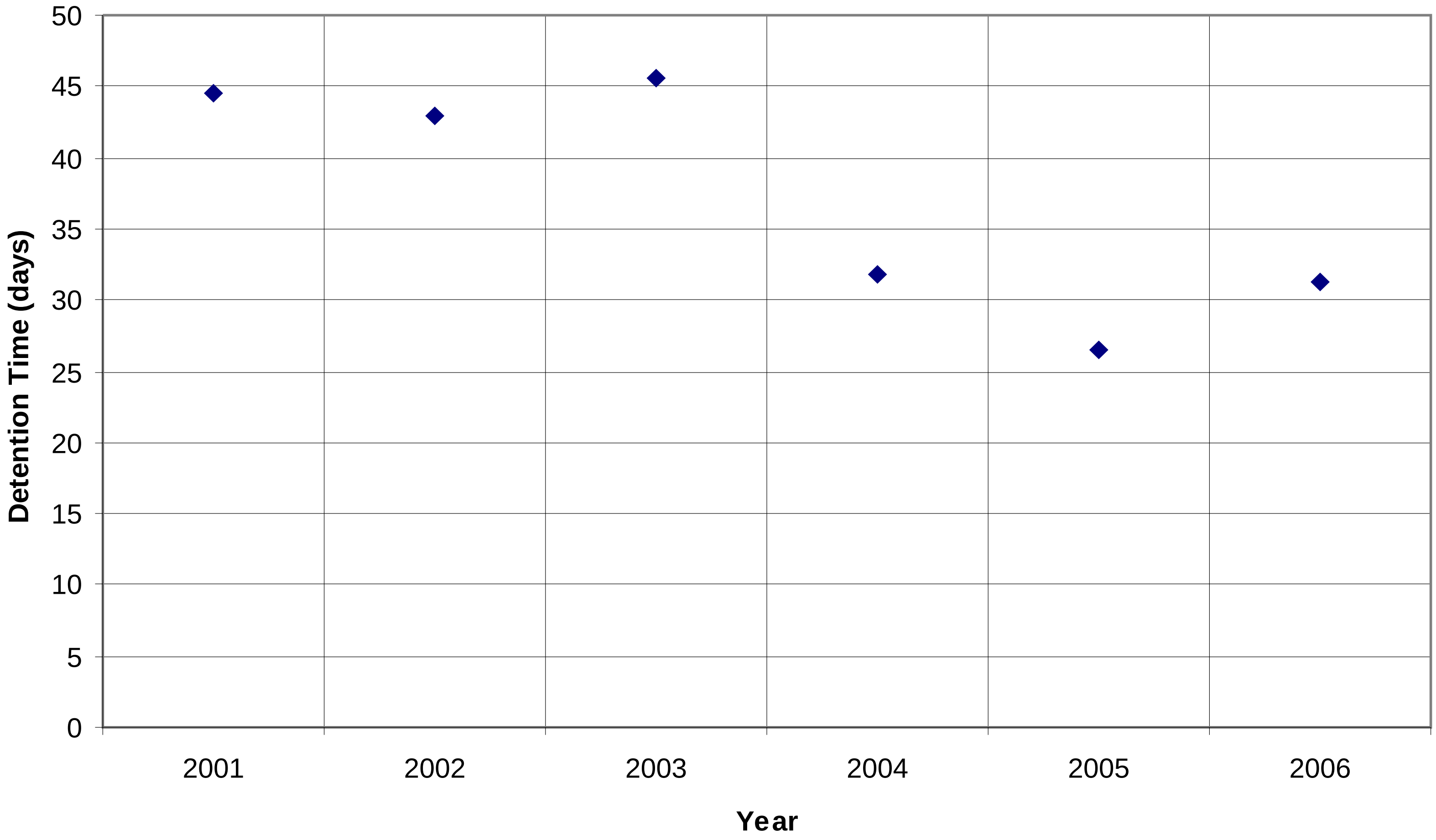
FIGURE 3.11 **Average Detention Time - All Digesters**
Sewer Master Plan
City of El Centro



3.5 RECOMMENDED WASTEWATER TREATMENT PLANT UPGRADES

The WWTP does not have preliminary treatment including screening and grit removal. Screening removes rags, sticks, and other large solids that can carry through the treatment process. Grit can consist of sand, silt, and other small organic and inorganic materials that can settle in the anaerobic digesters. This can lead to increased digester cleaning frequency.

At the WWTP, the lack of screening was evident by the presence of floating materials on the primary clarifiers. Operations personnel reportedly remove this material manually at least once per day. This leads to increased labor costs. As these materials travel through the treatment process, they could result in clogged pumps.

The primary clarifiers have vertical cracks at intervals of approximately five feet. These cracks have been repaired, but the material used in the repair should be removed to allow for better repair.

The aeration basins show considerable spalling on the tops of walls and along the walkways between basins. Repair was attempted approximately 18 years ago, but it is now failing. In general, the top one-inch of all walls and walkways should be removed and replaced. The south walkway between basins 1 and 2 should be removed and replaced. It is also recommended to remove all platforms that previously supported the surface mixers and replace all meter and slide gates on Basins 1 through 4 with new stainless steel gates.

The plant water pump station is in poor structural condition. A new pump station is recommended.

The sludge is heated in one integral boiler/heat exchanger and is rated at 760,000 BTU/hr. The pilot is fueled with propane while the burner is fueled with digester gas. The sludge heater was installed in 1989 and is reaching the end of its useful life. There is no redundancy in case of mechanical breakdown. A new unit is recommended.

REGULATORY ANALYSIS

4.1 INTRODUCTION

As discussed in Chapter 3, the City's WWTP routinely meets effluent limits, but violations have occurred. This section discusses the history of the violations.

The California Regional Water Quality Control Board (RWQCB) of the Colorado River Basin Region regulates the City's outfall discharge through the issuance of a federal National Pollutant Discharge Elimination System (NPDES) permit. NPDES permits have an effective life of five years, but are renewable.

The RWQCB is required to implement applicable federal and state laws, policies, and regulations in establishing effluent limitations for the City's wastewater treatment plants. State and federal water quality standards applicable to the City's discharge (which have been incorporated into the RWQCB's Water Quality Control Plan for the Colorado River Basin) include:

- Federal secondary treatment standards, and
- California effluent and receiving water standards

This chapter summarizes state and federal standards applicable to the City's outfall discharge, and:

Reviews the City's compliance history and assesses compliance trends,

- Identifies compliance parameters and identifies potential strategies for achieving compliance, and
- Reviews potential future regulatory changes and emerging discharge issues that may affect future City wastewater operations.

4.2 FEDERAL SECONDARY TREATMENT STANDARDS

Federal secondary treatment standards are established in Title 40, Section 133.102 of the Code of Federal Regulations (40 CFR 133.102). The federal secondary treatment standards establish numerical effluent limitations for all wastewater discharges to surface waters. Effluent standards are established for total suspended solids (TSS), biochemical oxygen demand (BOD), total dissolved solids (TDS) and pH. At the discretion of the permitting authority (RWQCB), EPA allows the federal secondary treatment BOD requirements to be expresses in terms of either BOD or carbonaceous biochemical oxygen demand (CBOD).

The EPA requires that the RWQCB implement the federal secondary treatment standards in all NPDES discharge permits. BOD and TSS effluent concentrations standards have been implemented in past NPDES permits issued to the City by the RWQCB.

Table 4.1 summarizes federal secondary treatment standards for TSS, BOD/CBOD, TDS, and pH.

Table 4.1 Federal Secondary Treatment Standards
Sewer Master Plan
City of El Centro

	Federal Secondary Treatment Standard [1]			
Parameter	**30-Day Average Concentration**	**7-Day Average Concentration**	**30-Day Average Percent Removal**	**Effluent Samples Complying with Standard 2001-2006**
	(mg/L)	**(mg/L)**	**(%)**	**(%)**
TSS	30	45	85	99.9
BOD	30	45	85	100
TDS	4,000	4,500	85	100
pH	6.0-9.0	6.0-9.0	6.0-9.0	100

1. Federal secondary treatment standard from 40 CFR 131.38.
2. The NPDES permit in effect during 2004-2009 (Order No. 2004-0004) included effluent concentration limits for TSS and BOD, but specified system-wide TSS and BOD percent removal limits using a flow-proportioned calculation. The City's current NPDES permit (Order No. R7-2004-0004) requires that the City comply with the 85 percent TSS and BOD removal requirements.
3. Federal regulations allow the NPDES permitting authority (RWQCB) to apply limits for either 5-day biochemical oxygen demand (BOD) or 5-day carbonaceous biochemical oxygen demand (CBOD). Federal secondary treatment limits for $CBOD_5$ are listed above. Corresponding federal secondary treatment limits for BOD are 30 mg/L (30-day average) and 45 mg/L (7-day average). The RWQCB implemented the above $CBOD_5$ limits in the City's current NPDES permit (Order No. R9-2005-0136).
4. Effluent pH is to be maintained above 6.0 units and below 9.0 units at all times, unless the discharger demonstrates that (1) inorganic chemicals are not added to the waste stream as part of the treatment process, and (2) contributions from industrial sources do not cause the pH to be less than 6.0 or greater than 9.0.

4.3 CALIFORNIA RWQCB SECONDARY TREATMENT STANDARDS - ORDER NO. R7-2004-0004

The State Implementation Plan (SIP) provides for the situation where an existing NPDES discharger cannot immediately comply with an effluent limitation derived from the California Toxics Rule (CTR) criterion. The SIP allows for the adoption of interim effluent limits and a schedule to come into compliance with the final limit in such cases. To qualify for interim limits and a compliance schedule, the SIP requires that an existing discharger demonstrate that it is infeasible to achieve immediate compliance with the CTR-based limit.

The RWQCB of the Colorado River Basin is the authority for the City of El Centro for wastewater treatment standards. On March 27, 2001, the RWQCB received the first data set of monitoring results for Priority Pollutants monitoring submitted by the City as required by the CTR. Based on the Reasonable Potential Analysis methodology in the SIP, copper, nickel, and selenium indicated reasonable potential to cause or contribute to an excursion above water quality objectives. The City issued an Infeasibility Report and request for a compliance schedule to the EPA on November 17, 2003

Copper, nickel, and selenium were determined to have reasonable potential to exceed water quality objectives, and final Water Quality Based Effluent Limitations (WQBELs) were required. The governing Water Quality Objective (WQO) for copper is 3.1 µg/L, the saltwater aquatic life criteria contained in the CTR. The copper WQBELs calculated pursuant to SIP procedures were 2.39 µg/L monthly average and 4.80 µg/L daily maximum. In the City's report, they concluded that the WQBELs were infeasible to be able to comply immediately at the plant. The City's previous permit did not contain an effluent limit for copper, and no data was collected to determine the current plant performance on copper removal. Therefore, the EPA established an interim average monthly effluent limit of 8.2 µg/L and an interim maximum daily effluent limit (MDEL) of 8.2 µg/L.

The governing WQO for nickel was 8.2 µg/L, the freshwater aquatic life criteria contained in the CTR. The WQBEL calculated pursuant to SIP procedures were 6.71 µg/L monthly average and 13.5 µg/L daily maximum. The City's previous permit did not contain an effluent limit for nickel, and in the City's report; they stated it was infeasible for them to immediately comply with the WQBELs. Therefore, the EPA established an interim average monthly effluent limit of 7 µg/L and an interim MDEL of 13.5 µg/L. These interim effluent limits were based on the best professional judgment of the Regional Board staff.

The governing WQO for selenium was 5.0 µg/L, the freshwater aquatic life criteria contained in the CTR. The WQBELs calculated pursuant to SIP procedures were 4.09 ug/L monthly average and 8.22 µg/L daily maximum. In the City's report, they stated it was infeasible for them to immediately comply with the WQBELs. Therefore, the EPA established an interim average monthly effluent of 8 µg/L and an interim MDEL of 8.22 ug/L.

The City was not able to consistently comply with the new effluent limitations for copper, nickel, and selenium. Table 4.2 summarizes the copper, nickel, and selenium effluent limits and when the limits would become effective. The 2003 Infeasibility Report by the City stated it was infeasible for them to comply with the WQBELs.

Table 4.2 Copper, Nickel, and Selenium Effluent Limits [1]
Sewer Master Plan
City of El Centro

Constituent	Unit	Date Effluent Limit Becomes Effective	Average Monthly Effluent Limit	Maximum Daily Effluent Limit
Copper (interim)	μg/L	March 10, 2004	8.2	8.2
Copper (final)	μg/L	March 10, 2009	2.39	4.8
Nickel (interim)	μg/L	March 10, 2004	7	13.5
Nickel (final)	μg/L	March 10, 2009	6.71	13.5
Selenium (interim)	μg/L	March 10, 2004	8	8.22
Selenium (final)	μg/L	March 10, 2009	4.09	8.22

Notes:

1. Source: California Regional Water Quality Control Board Civil Liability Order No. R7-2006-0075.

Wastewater effluent discharged to the Central Main Drain must have an Escherichia Coli (E. coli) concentration below a log mean of Most Probable Number (MPN) of 126 MPN per 100 milliliters (based on a minimum of not less than five samples for any 30-day period) and no sample shall exceed 400 MPN per 100 milliliters.

In addition, wastewater discharged to the Central Main Drain has additional limitations outlined in Table 4.3. These limits are calculated based on monitoring results using the CTR and the Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California for water quality based effluent limits:

Table 4.3 Receiving Water Limitations
Sewer Master Plan
City of El Centro

Limitation
Dissolved oxygen shall not fall below 5.0 mg/L. When dissolved oxygen in the receiving water is already below 5.0 mg/L, the discharge shall not cause any further depression.
The presence of oil, grease, floating material or suspended material in amounts that create a nuisance or adversely affect beneficial uses

Table 4.3 Receiving Water Limitations Sewer Master Plan City of El Centro
Limitation
The discharge shall not result in the deposition of pesticides or combination of pesticides to be detected in concentrations that adversely affect beneficial uses.
A significant increase in fungi, slime, or other objectionable growth.
Increase turbidity that results in affecting beneficial uses.
The normal ambient pH to fall below 6.0 or exceed 9.0 units.
Impact the receiving water temperature, resulting in adversely affecting beneficial uses.
Result in the deposition of material that causes a nuisance or adversely affects beneficial uses.
The chemical constituents to exceed concentrations that adversely affect beneficial uses or create nuisances.
Toxic pollutants to be present in the water column, sediments, or biota in concentrations that adversely affect beneficial uses or that produce detrimental physiological responses in human, plant, animal, or aquatic life.

4.4 CALIFORNIA RWQCB SECONDARY TREATMENT STANDARDS - SPECIAL ORDER NO. R7-2007-0069

The CTR and the State Water Resource Control Board (SWRCB) recently updated the criteria for interim and final effluent limits for copper, nickel, and selenium. The SWRCB have specific criteria for fresh waters and specific criteria for salt waters. When the salinity of receiving water is between 1 and 10 parts per thousand, such as is the case for the Central Main Drain, the CTR and SIP provide for the Regional Board to prescribe in a permit the more stringent of the two criteria.

On February 20, 2007, the City conducted a Biological Assessment at the location of the discharge. The areas of observation were approximately 200 meters upstream and 100 meters downstream of the discharge. The object of the Biological Assessment was to determine whether water, plant life, and aquatic life at the discharge location are more typical of a saltwater or a freshwater environment. This assessment determined that the applicable reach of the Central Main Drain is characterized as freshwater; therefore, water quality criteria for the protection of freshwater aquatic life are applicable.

On August 1, 2007 the USEPA issued a tentative approval via Public Notice No. 7-07-48 of the assessment and the application of water quality criteria for the protection of freshwater aquatic life as Tentative Board Order R7-2007-0069. The Regional Board adopted this Order on September 19, 2007. This Order amends Board Order No. R7-2004-0004 to designate the Central Main Drain as a freshwater environment and establish interim and final effluent limits based on CTR and SIP freshwater criteria for selenium only. Both copper and nickel were found not to have a reasonable potential to violate water quality objectives and indicated that WQBELs were not required.

Table 4.4 provides the interim and final effluent limits for selenium.

Table 4.4 Selenium Effluent Limits
Sewer Master Plan
City of El Centro

Constituent	Unit	Date Effluent Limit Becomes Effective	Average Monthly Effluent Limit	Maximum Daily Effluent Limit
Selenium (interim)	µg/L	September 19, 2007	8.0	8.22
Selenium (final)	µg/L	May 18, 2010	4.2	8.1

An additional limitation for discharge to the Central Main Drain was established. The Special Order replaces the numeric effluent limitations for TDS with a narrative effluent limitation and establishes a receiving water limitation for TDS to accurately apply the WQOs of the Basin Plan. The receiving water limitations state that the concentration of TDS shall not exceed an annual average concentration of 4,000 mg/L or an instantaneous maximum concentration of 4,500 mg/L.

4.5 FUTURE COMPLIANCE ISSUES

Compliance with selenium removal limits established in Special Order No. R7-2007-069 represents the most significant new compliance challenge facing the City. The reported maximum effluent concentration of selenium from the WWTP was 27 µg/L. The maximum concentration in the receiving water was 10 µg/L. These are higher than the WQO of 5 µg/L, which exhibits a reasonable potential to cause a violation. The Special Order states that the established WQBELs for selenium prevent adverse impacts on the WARM, MILD and RARE beneficial uses of the Central Main Drain and ensure compliance with the Basin Plan narrative water quality objective for metals.

The interim and final selenium effluent limits presented in Table 4.4 represent total selenium including particulate and dissolved, but the effluent values reported by the City do not distinguish between the two. The fraction of dissolved selenium is very important to the efficiency of the removal process.

Carollo Engineers conducted pilot testing from November 2005 to August 2006 for the City of Davis, CA concerning both conventional and priority pollutant removal. The City's previous Master Plan found that it was unlikely the existing treatment processes could reliably meet anticipated future metals criteria specified by the California Toxics Rule (CTR). Therefore, as part of the Master Plan, the City evaluated several alternatives for discharge, reuse, and treatment to meet these future requirements. These are presented in Appendix C.

4.6 CONCLUSIONS

The City's existing treatment processes have provided beneficial metals removal historically, but cannot meet either existing or future effluent requirements for selenium. The first step to aid the City in developing a selenium removal program should include characterizing the influent and effluent selenium. These characteristics include the soluble fraction as well as speciation to determine the organic fraction. Treatment technologies vary on their ability to remove soluble selenium. These technologies must be identified and characterized. Pilot testing of promising technologies will predict their ability to meet effluent criteria on a full-scale basis. In addition, a potential alternative strategy involves regulatory considerations by negotiating higher receiving water effluent limits.

COLLECTION SYSTEM IMPROVEMENTS

5.1 INTRODUCTION

This chapter outlines the needed improvements to the City's collection system including the existing lift stations. Collection system hydraulic models have been prepared as part of this master plan using H20Map Sewer Suite 8.0 by MWHSoft. Scenarios analyzed include extended period simulations of Present, 2015 and Build Out. Each scenario was analyzed for peak day dry weather (PDWF) and peak day wet weather (PWWF) conditions. The extended period simulations were run over a 48-hour period. Only the results from the second 24-hour period were considered.

5.1.1 Skeletonizing

Skeletonizing is the process by which sewer systems are stripped of pipelines not considered essential for the intended analysis purpose. The purpose of the skeletonizing a system is to develop a model that accurately simulates the hydraulics of the pipelines collecting sanitary sewer flows. Skeletonizing should reduce the complexity of the large model and minimize analysis time while accurately simulating the hydraulics of the pipelines within the collection system. The core pipelines of the sewer system were included in the hydraulic model. These pipes are generally 10-inches in diameter and larger and function to convey the wastewater collected in the City to the WWTP. No pipes smaller than 10-inches that connect larger pipe segments were removed. The skeletonized system is shown on Figure 5.1.

5.2 SUMMARY OF MODELED FLOWS

Table 5.1 provides the dry-weather flows generated by the hydraulic models for the Present, 2015 and Build Out time frames. These flows are based on the land use flow factors and peaking factors presented in Chapter 2 for the expected development for each time frame. The flows presented are the total calculated flows at the WWTP. Although no peak month hydraulic models were conducted, peak month flows are presented based on a peaking factor of 1.07. This is typical for wastewater treatment plants. The City of El Centro's WWTP has experienced peak month flows that range between 1.05 and 1.14 times the average daily flows for the time period of 2001 to 2006. The peak day flow is based on a peaking factor of 1.2, which is the factor used for the peak day hydraulic models.

Table 5.1 Modeled Dry-Weather Flows for Development Time Frames Sewer Master Plan City of El Centro			
Time Frame	**Average Daily Flow (mgd)**	**Peak Month Flow [1] (mgd)**	**Peak Day Flow [2] (mgd)**
Present	3.88	4.15	4.66
2015	4.14	4.43	4.97
Build Out	13.80	14.77	16.56

Notes:
1. Based on a peak month factor of 1.07.
2. Based on a peak day factor of 1.2.

5.3 RECOMMENDED SEWER IMPROVEMENTS

Hydraulic simulations were performed for the Present, 2015 and the Build Out time frames. Each simulation considered the PDWF and the PWWF. Pipes segments that are recommended for upgrade exhibited surcharging based on the critieria provided in Chapter 2. Lift stations were analyzed for high water levels that would surcharge the influent pipe.

5.3.1 Present Condition

Table 5.2 provides the recommended upgrades for the Present time frame for both PDWF and PWWF. Figure 5.1 shows these pipes, and Figures 5.2, 5.3, 5.4 and 5.5 provide a more detailed view of the areas. The recommended upgrade pipe size was chosen to handle Build Out flows.

Table 5.2 Recommended Upgrades - Present Sewer Master Plan City of El Centro								
Pipe Location	**Figure No.**	**H2O Map Manhole No.**	**Atlas Page**	**Upstream Manhole No. (Atlas)**	**Down-stream Manhole No. (Atlas)**	**Pipe Length (feet)**	**Current Pipe Diameter (inches)**	**Required Pipe Diameter (inches)**
Imperial Avenue & Holt Avenue	5.2	20-121055	20-12	110	105	237	12	15
Imperial Avenue & Orange Avenue	5.2	20-121070	20-12	105	146	306	12	15

Table 5.2 Recommended Upgrades - Present
Sewer Master Plan
City of El Centro

Pipe Location	Figure No.	H2O Map Manhole No.	Atlas Page	Upstream Manhole No. (Atlas)	Down-stream Manhole No. (Atlas)	Pipe Length (feet)	Current Pipe Diameter (inches)	Required Pipe Diameter (inches)
Imperial Avenue & Brighton Avenue	5.2	21-121025	21-12	146	139	172	12	15
Imperial Avenue & Olive Street	5.2	21-121026	21-12	139	138	174	12	15
Imperial Avenue & Olive Street	5.2	21-121027	21-12	138	135	160	12	15
Imperial Avenue & State Street	5.2	21-121028	21-12	135	128	381	12	15
Imperial Avenue & Main Street	5.2	21-121029	21-12	128	120	418	12	15
4th Street & Brighton Avenue	5.3	21-141051	21-14	152	145	190	14	18
4th Street & Olive Avenue	5.3	21-141050	21-14	145	143	182	14	18
4th Street & Olive Avenue	5.3	21-141004	21-14	143	136	175	14	18
4th Street & State Street	5.3	21-141005	21-14	136	133	192	14	18
4th Street & State Street	5.3	21-141057	21-14	133	121	187	14	18
3rd Street & State Street	5.3	21-151018	21-15	124	119	189	8	12
3rd Street & Main Street	5.3	21-151008	21-15	119	115	183	8	12
3rd Street & Broadway Avenue	5.3	21-151029	21-15	115	105	386	10	12
3rd Street & Broadway Avenue	5.3	21-151013	21-15	105	101	191	10	12

Table 5.2 Recommended Upgrades - Present
Sewer Master Plan
City of El Centro

Pipe Location	Figure No.	H2O Map Manhole No.	Atlas Page	Upstream Manhole No. (Atlas)	Down-stream Manhole No. (Atlas)	Pipe Length (feet)	Current Pipe Diameter (inches)	Required Pipe Diameter (inches)
3rd Street & Commercial Avenue	5.3	21-151043	21-15	101	118	176	10	12
Adams Avenue & 4th Street	5.3	22-141023	22-14	120	118	112	27	27
Commercial Avenue & 3rd Street	5.3	22-151018	22-15	116	117	295	18	18
8th Street & Ross Avenue	5.4	19-131067	19-13	151	143	194	6	8
8th Street & Yucca Drive	5.4	19-131068	19-13	160	151	500	6	8
8th Street & Tangerine Drive	5.4	19-131069	19-13	177	160	500	6	8
8th Street & Aurora Drive	5.4	18-131062	18-13	101	177	158	6	8
Heil Avenue & Dogwood Road (1)	5.5	20-151017	20-15	122	123	299	8	12
Heil Avenue & Dogwood Road (1)	5.5	20-151015	20-15	120	121	120	8	12
Heil Avenue & Dogwood Road (1)	5.5	20-151016	20-15	121	122	300	8	12
Heil Avenue & Hope Avenue (1)	5.5	20-161006	20-16	101	102	390	8	12
Heil Avenue & Hope Avenue (1)	5.5	20-161009	20-16	102	120	390	8	12
Heil Avenue & Hope Avenue (1)	5.5	20-161023	20-16	100	101	404	8	12

Notes:
1. Upgrading not recommended for this pipe segment. Refer to discussion below.

Figure 5.2 provides a detailed view of the surcharged pipes along Imperial Avenue. The pipes are 12 inches in diameter and have a total length of approximately 1,848 feet. Central Union High School and Wilson Junior High School are located along this segment of pipe. The model results indicate that the surcharged pipe should be upgraded to at least 15-inch diameter pipes. These upgrades will handle the project Build Out flows.

Figure 5.3 shows the area of Commercial Avenue and North 4th Street. Two pipes surcharge because of very shallow slopes. These pipes are called out and are an 18-inch pipe at Adams Avenue and a 27-inch pipe upstream of the Eastside Lift Station along Commercial Avenue. Upgrade is not recommended.

736 feet of 14-inch pipe lay downstream of the surcharged 14-inch pipe segment on North 4th Street near Brighton Avenue. Although not surcharging, the 14-inch pipe is recommended to be upsized to 18 inch to match the diameter of the new upstream pipe.

The 8-inch pipe at 3rd Street and State Street requires an upgrade to 12-inch pipe to alleviate surcharging conditions. It is recommended that 372 feet of 8-inch pipe and 753 feet of 10-inch pipe downstream of this segment be upgraded to 12-inch pipe also.

Figure 5.4 shows approximately 1,352 ft of 6-inch pipe along South 8th Street north of Interstate 8 that is part of the core collection system. Approximately 194 feet of this pipe between Desert Gardens Drive and Ross Avenue surcharges. The remaining 6-inch pipe does not indicate surcharging. It is recommended that the 6-inch pipe segments be upgraded to 8-inch pipe.

Figure 5.5 provides a detailed view of the surcharged gravity pipe that is directly downstream of the Heil & Dogwood Lift Station. The surcharged pipes are 8 inches in diameter with a total length of 1,604 feet. Upgrading the pipe to 12-inch diameter will allow for Build Out flows. An additional 299 feet of 8-inch pipe downstream of the surcharged pipes should be replaced with 12-inch pipe to maintain pipe size. A more cost effective option in lieu of upsizing the 8-inch pipe would include extending the forcemain to the 15-inch pipeline along Fairfield Avenue. This would require pump upgrades to meet the increased head requirement. Extending the forcemain and upsizing the pumps is recommended.

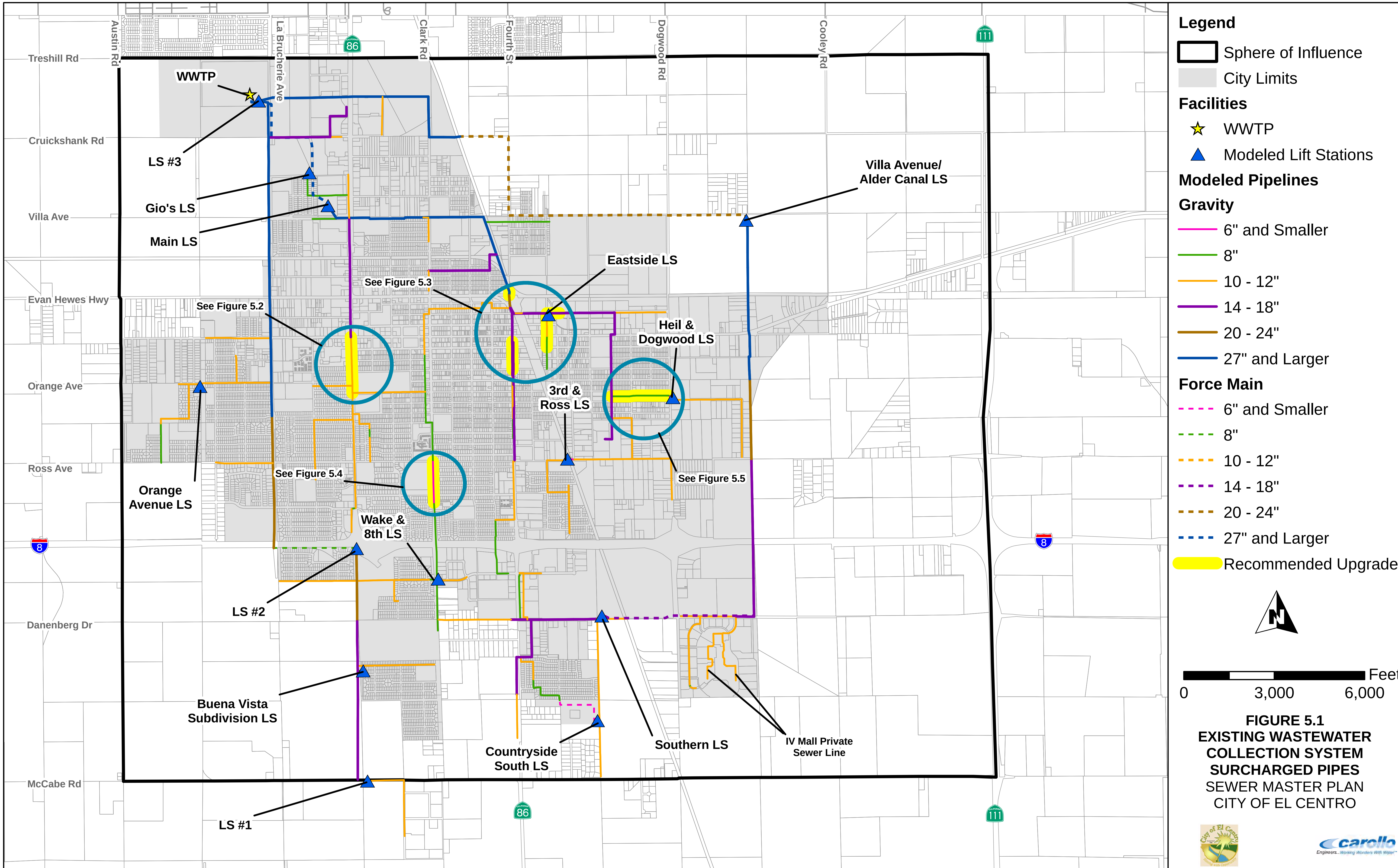


FIGURE 5.1
EXISTING WASTEWATER COLLECTION SYSTEM SURCHARGED PIPES
SEWER MASTER PLAN
CITY OF EL CENTRO

Legend

Manholes

Modeled Pipelines

Gravity

12"

15"

Surcharged Pipelines

Feet

0 250 500

FIGURE 5.2
EXISTING WASTEWATER COLLECTION SYSTEM
SURCHARGED PIPES - IMPERIAL AVENUE
SEWER MASTER PLAN
CITY OF EL CENTRO

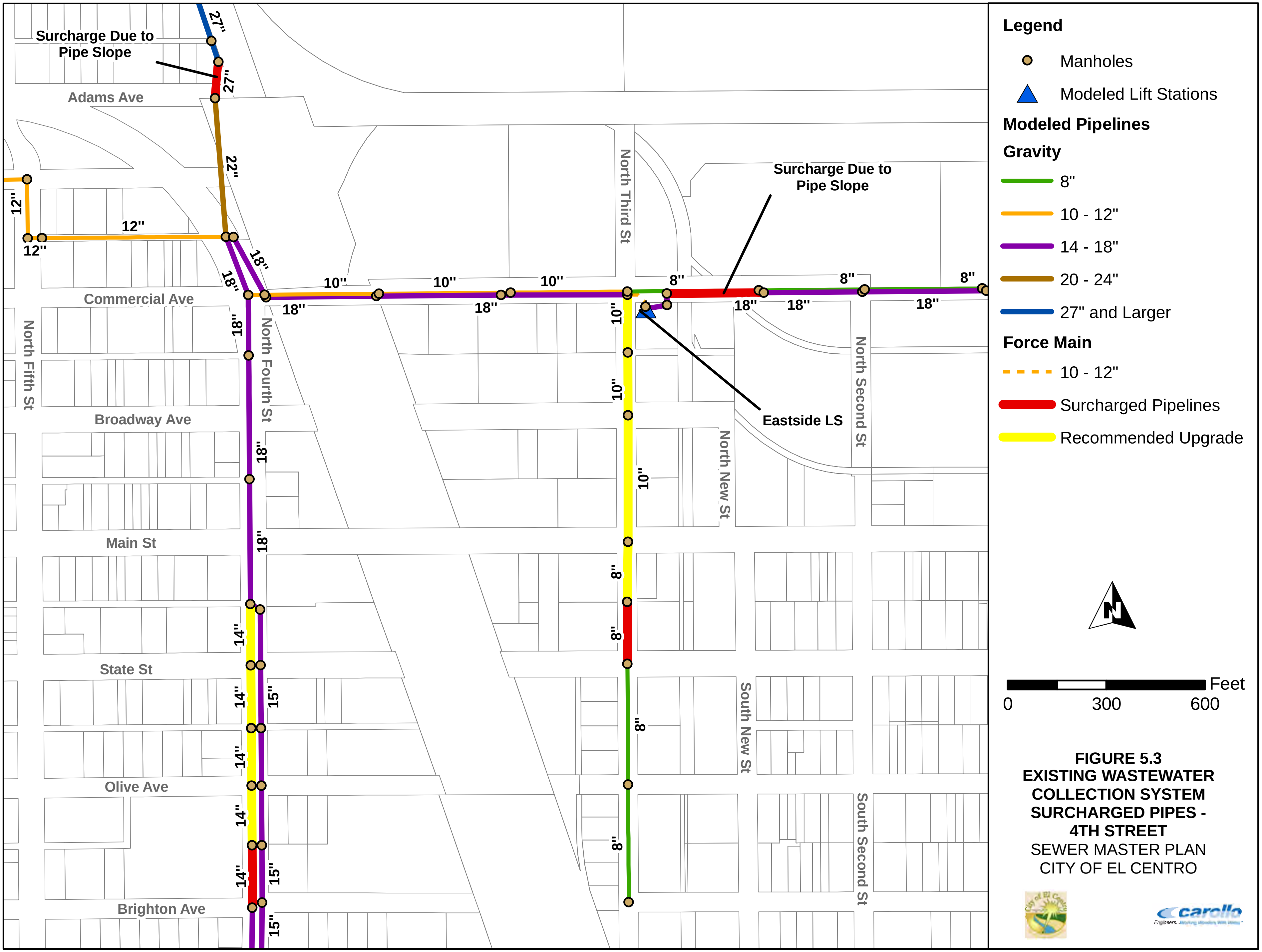


FIGURE 5.3
EXISTING WASTEWATER COLLECTION SYSTEM SURCHARGED PIPES - 4TH STREET
SEWER MASTER PLAN
CITY OF EL CENTRO

FIGURE 5.4
EXISTING WASTEWATER COLLECTION SYSTEM SURCHARGED PIPES - 8TH STREET
SEWER MASTER PLAN
CITY OF EL CENTRO

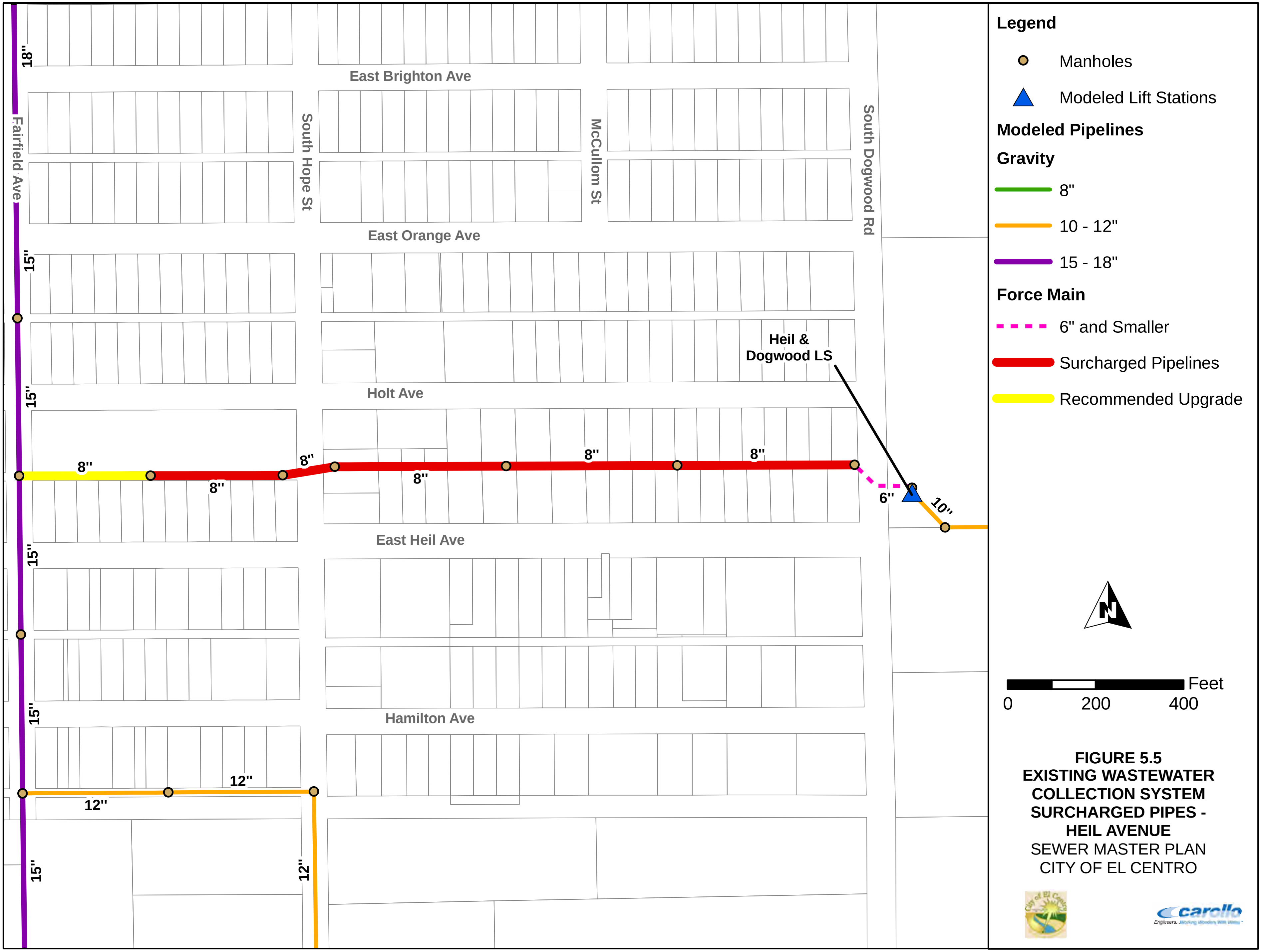


FIGURE 5.5
EXISTING WASTEWATER COLLECTION SYSTEM SURCHARGED PIPES - HEIL AVENUE
SEWER MASTER PLAN
CITY OF EL CENTRO

5.3.2 2015 Condition

No upgrades are recommended for the 2015 time frame. This assumes the upgrades from the Present time frame have been constructed. Figure 5.6 shows new pipelines within the proposed development areas of Lerno and Anderson/Waterford. These developments are discussed in detail in Chapter 2 of the Water Master Plan. These new pipelines and lift station will be paid for by the developers and are not included in the Capital Improvements Program.

5.3.3 Build Out Condition

Table 5.3 provides recommended upgrades for the Build Out time frame. New pipelines are shown to convey wastewater from areas not previously developed but within the SOI. These pipes are shown in the northwest region between the WWTP and Interstate 8, the southwest region and the northeast region. Pipe locations were assumed based on topography. Surcharged pipes within the existing collection system and pipeline additions for Build Out are shown on Figure 5.7. Figure 5.8 provides a more detailed view of the surcharged areas.

In the northwest region, the existing topography of areas west of the WWTP would not allow new pipes to run east into the existing pipeline along La Brucherie Road without incorporating a new lift station. A pipeline running north to the WWTP allows gravity flow. Previous master plans have recommended a similar pipeline, referred to as the Lotus Sewer that would parallel the La Brucherie Road pipeline. This City is evaluating the benefits of installing the Lotus Sewer in lieu of upsizing the La Brucherie Road pipe. Pipelines serving new areas south of the WWTP and in close proximity to La Brucherie Road, however, can connect to the 27-inch and 30-inch pipeline along La Brucherie Road.

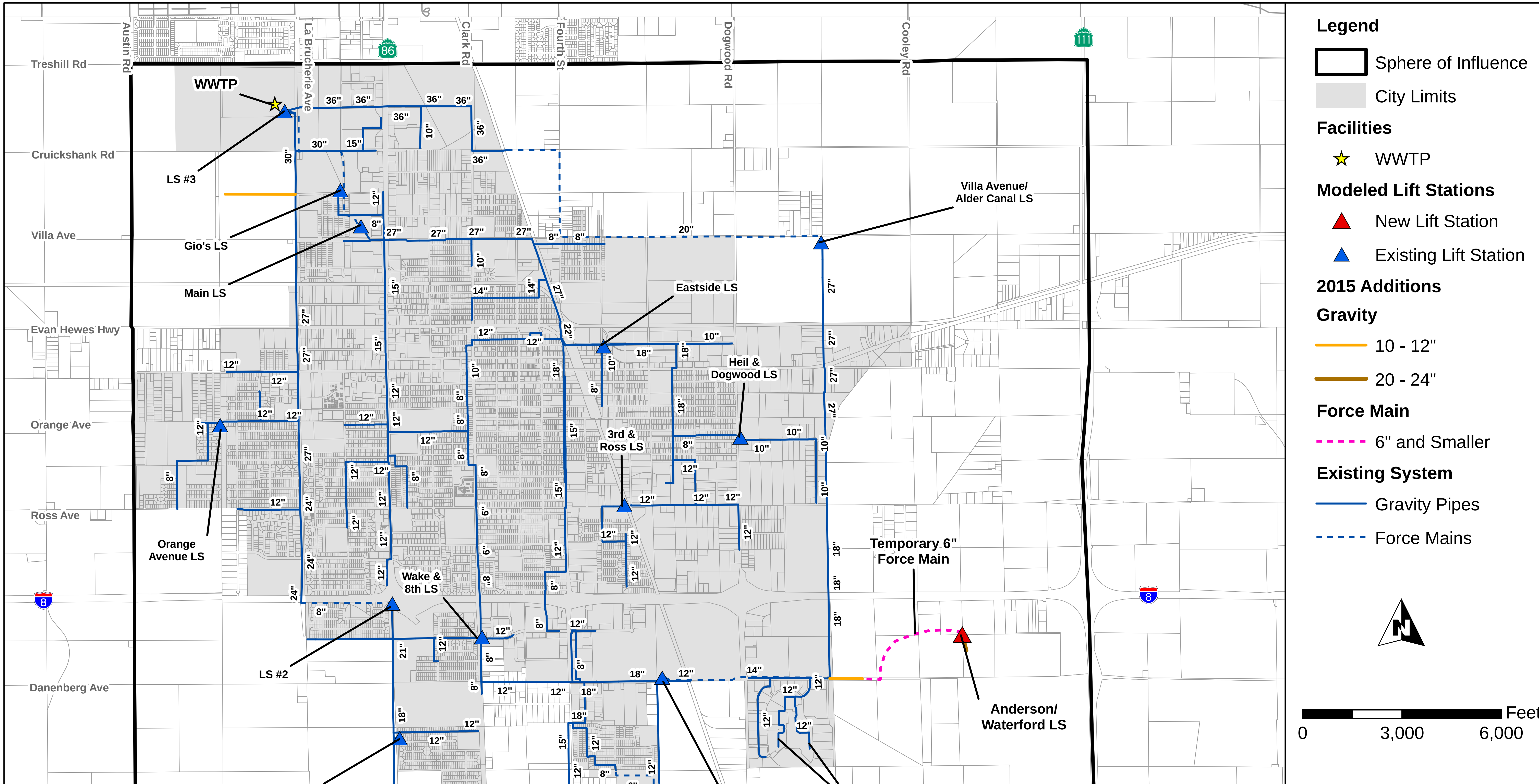


FIGURE 5.6
WASTEWATER COLLECTION SYSTEM 2015 ADDITIONS
SEWER MASTER PLAN
CITY OF EL CENTRO

Table 5.3	Recommended Upgrades - Build Out Sewer Master Plan City of El Centro							
Pipe Location	**Figure No.**	**H2O Map Manhole Number**	**Atlas Page**	**Upstream Manhole No. (Atlas)**	**Down-stream Manhole No. (Atlas)**	**Pipe Length (feet)**	**Current Pipe Diameter (inches)**	**Required Pipe Diameter (inches)**
Plank Drive & Ross Road	5.8	19-101010	19-10	111	106	298	8	12
Plank Drive & Ross Road	5.8	19-101011	19-10	106	101	299	8	12
Plank Drive & Ross Road	5.8	19-101012	19-10	101	115	291	8	12
Plank Drive & Ross Road	5.8	20-101014	20-10	115	113	200	8	12
Plank Drive & Ross Road	5.8	20-101015	20-10	113	109	218	8	12

In the southwest region, the pipeline along Plank Drive was extended south to serve areas north of Interstate Highway 8. Connection to this pipeline will require the existing pipe to be upgraded from 8 inches to 12 inches as shown in Table 5.3. This pipe is detailed in Figure 5.8. In addition, to serve areas south of Interstate Highway 8, a new pipeline connects to the 24-inch pipeline on La Brucherie Road. It ranges in size from 12 inches to 21 inches. The 18-inch pipes utilize the minimum slope of 0.0010 from Table 2.5. This allows gravity lines to extend to the southern most areas of potential development. Many pipes throughout the City with diameters of 14-inches and larger have slopes equal to or less than 0.0010. If the slope is increased, a new lift station would be necessary to serve new development south of Interstate Highway 8.

For the northeast region, pipes up to 24 inches were required to adequately handle the expected Build Out flows. An additional lift station is required to connect to the existing collection system. The lift station was sized similar to the Villa Avenue/Alder Canal Lift Station with an approximate depth of 30 feet. The force main connects to the existing collection system along Cruickshank Drive. The existing 36-inch pipe along Cruickshank Drive has capacity to convey the flows to the WWTP.

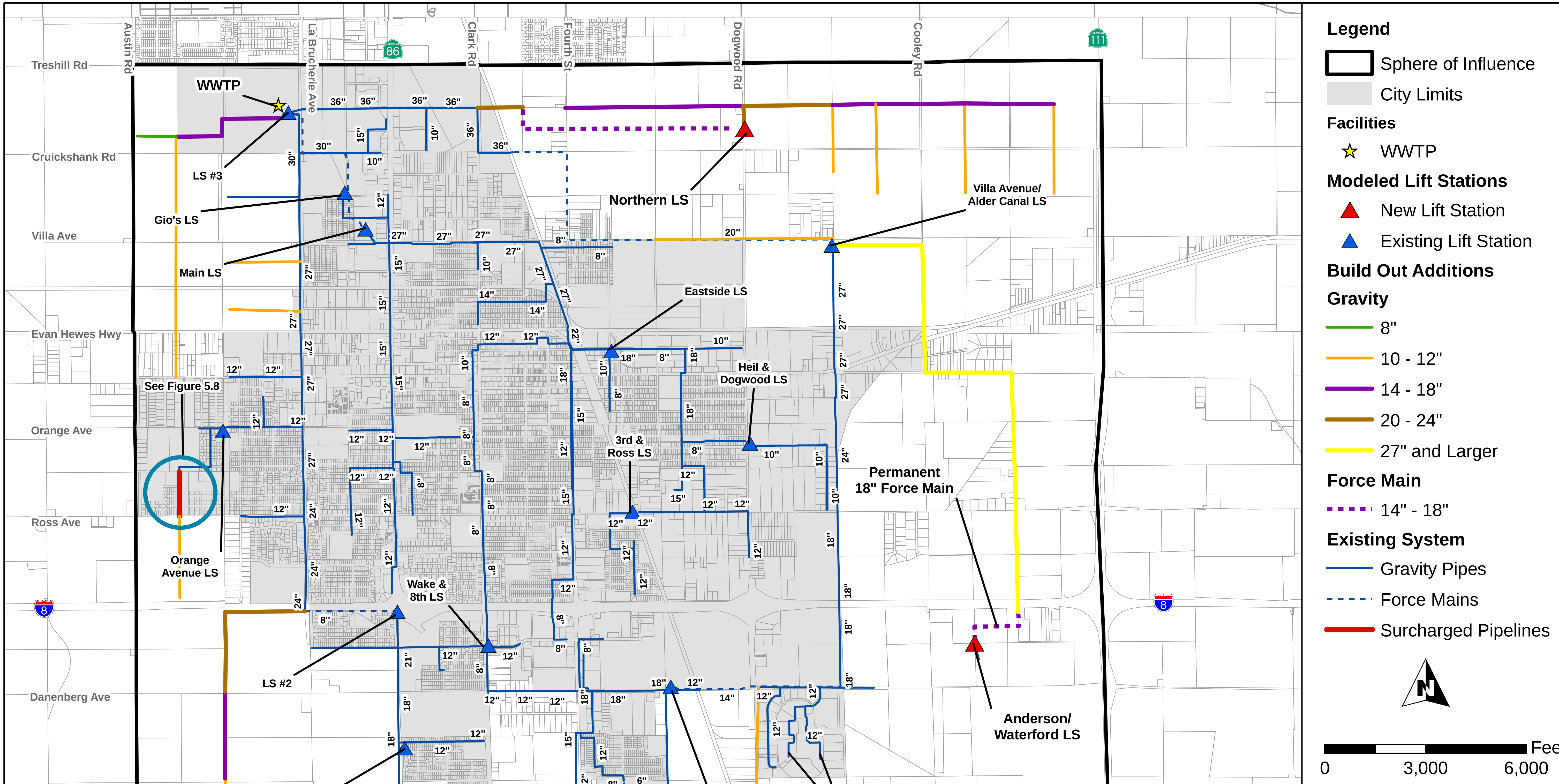
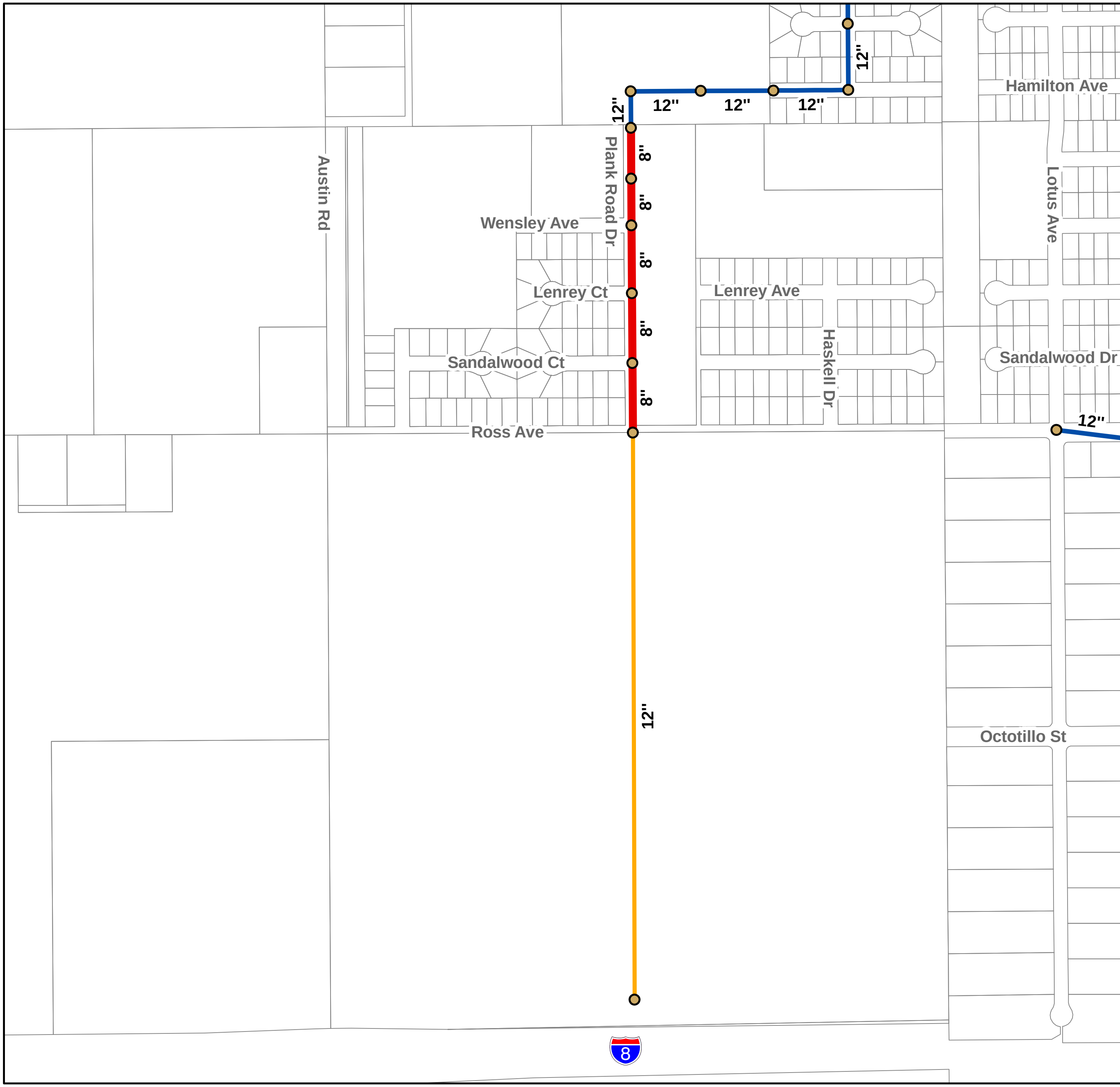


FIGURE 5.7
WASTEWATER COLLECTION SYSTEM BUILD OUT ADDITIONS AND SURCHARGED PIPES
SEWER MASTER PLAN
CITY OF EL CENTRO



Legend

Manholes

Build Out Additions

Gravity

10 - 12"

Modeled Pipelines

Gravity

12"

Surcharged Pipelines

Feet
0 500 1,000

FIGURE 5.8
BUILD OUT WASTEWATER COLLECTION SYSTEM SURCHARGED PIPES - PLANK ROAD DRIVE
SEWER MASTER PLAN
CITY OF EL CENTRO

5.4 LIFT STATION IMPROVEMENTS

The sewer model predicts the peak flow entering the lift stations. The existing pumping capacity is sufficient for the projected peak hour flow for Present and 2015 time frames. The existing pumping capacity is insufficient for several of the lift stations for the projected peak hour flow for the Build Out time frame. Table 5.4 lists the estimated capacity for each lift station. The existing capacities are given for lift stations that do not require upgrades. Capacities for lift stations requiring upgrade are given as the maximum hourly flow for peak wet weather conditions.

Table 5.4 Lift Station Upgrades - Build-Out
Sewer Master Plan
City of El Centro

Lift Station	Location	H2OMap Sewer Model Number	Atlas Page	Atlas Number	Required Pump Capacity (gpm)	Comments
Orange Avenue	Orange Ave. & Haskell Dr.	20-11-750	20-11	750	760	Upgrade Required
Lift Station #2	Imperial Ave. & Interstate 8	18-12-750	18-12	750	1,620	Upgrade Required
Alder Canal / Villa Avenue	Villa Ave. & Alder Canal	23-17-700	23-17	700	6,910	Upgrade Required
Lift Station #3	Wastewater Treatment Plant	25-11-700	25-11	700	14,500	Upgrade Required
Main Lift Station	Villa Ave. & State Highway 86	23-12-700	23-12	700	5,070	Existing Capacity
Eastside Lift Station	Commercial Ave. & 3rd St.	22-15-700	22-15	700	1,000	Existing Capacity
Southern Lift Station	Danenberg Dr. & Farnsworth Rd	SLS9002	---(1)	---(1)	1,338	Existing Capacity
Lift Station #1	McCabe Rd & Imperial Ave.	14-12-700	14-12	700	400	Existing Capacity
Waterford/ Anderson Lift Station	Danenberg Dr. & Pitzer Rd	9000	---(1)	---(1)	3,270	Proposed Capacity (2)
Third and Ross	3rd St. & Ross Avenue	19-15-700	19-15	700	325	Existing Capacity
Wake and Eighth	Wake Ave. & 8th St.	17-13-700	17-13	700	200	Existing Capacity

Table 5.4 Lift Station Upgrades - Build-Out
Sewer Master Plan
City of El Centro

Lift Station	Location	H2OMap Sewer Model Number	Atlas Page	Atlas Number	Required Pump Capacity (gpm)	Comments
Buena Vista	Manuel A. Ortiz Ave. & Imperial Ave.	16-12-700	16-12	700	369	Existing Capacity
Countryside South	Valor Lane & Farnsworth Rd	9006	---(1)	---(1)	100	Existing Capacity
Gios Lift Station	Lincoln St. & Waterman Ave.	SLS9004	24-12	750	250	Existing Capacity
Heil and Dogwood	Heil Ave. & Dogwood Rd	20-16-700	20-16	700	250	Existing Capacity
Northern Lift Station	Dogwood Rd & Cruickshank Rd	9002	---(1)	---(1)	2,810	Build Out Addition

Notes:
1. Lift Station not listed in Sewer Atlas Map.
2. Waterford/Anderson Lift Station Pump Capacity taken from Desert Lakes Water and Sewer Master Plan

Upgrading the lift stations for Build Out will require upgrade and replacement of the pumps and wet wells. Lift Station #2 is to be relocated and upsized as part of an upcoming City project to replace aging pipes along Imperial Avenue. Implementing the Lotus Sewer will result in demolition of the Orange Avenue Lift Station. Lift Station #2 and the Orange Avenue Lift Station will not be included in the CIP.

CAPITAL IMPROVEMENTS PROGRAM

This chapter presents the recommended Capital Improvements Program (CIP) for the City's sewer system. The program is based on the evaluation of the sewer system and the recommended projects described in the previous chapters. The purpose of this chapter is to present the assumptions used in developing order of magnitude cost estimates for recommended facilities. Recommended improvements address current system deficiencies and facilities required to meet the needs of the City through Year 2015.

6.1 COST ESTIMATING CRITERIA

The cost estimates presented were developed from bid tabulations, cost curves, information obtained from previous studies, vendors and Carollo's experience on other projects. The costs estimated for each recommended facility are included in the CIP table developed for this Sewer Master Plan. The table is intended to be used to facilitate revisions to the City's CIP and ultimately to support determination of the user rates and connection impact fees.

6.1.1 Cost Estimating Accuracy

The cost factors are prepared for general master planning purposes and for guidance in project evaluation and implementation. Final costs of a project will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other variable factors such as: preliminary alignments generation, investigation of alternative routings, and detailed utility and topography surveys.

The American Association of Cost Engineers defines three types of cost estimates:

- An Order of Magnitude Estimate for Master Plan Studies. This is an approximate estimate made without detailed engineering data. It is normally expected that an estimate of this type would be accurate within +50 percent to -30 percent.
- A Budget Estimate for Pre-Design Study. A budget estimate is prepared with the use of flow sheets, layouts, and equipment details. It is normally expected that an estimate of this type would be accurate within +30 percent to -15 percent.
- A Definite Estimate (Engineer's Estimate) for time of contract bidding. This estimate is prepared from very defined engineering data. The data includes fairly complete plot plans and elevations, soil data, and a complete set of specs. It is expected that an engineer's estimate would be accurate within +15 to -5 percent.

Costs developed for this study should be considered "order of magnitude" and have an expected accuracy range of +50 percent to -30 percent.

6.1.2 General Cost Estimating Assumptions

Capital cost estimates are opinions developed based on costs obtained from industry manufacturers' bid tabulations, cost curves, previous studies, and Carollo Engineers' (Carollo) experience on similar projects. All estimates have been adjusted to an Engineering News Record (ENR) index for Los Angeles of 9,182 (December 2007). This ENR index is used to adjust construction costs for inflation and current business conditions. The ENR Cost Index is calculated periodically based on various industry factors that adjusts cost and include factors such as inflation for material costs and labor costs.

The cost estimates include the following components:

- Estimated construction cost including 30-percent contingency,
- 15-percent engineering and construction management, and
- 10-percent for the City's legal and administration costs.

6.1.3 Collection System Unit Costs

Table 6.1 provides unit costs for pipe sizes ranging from 8 inches through 36 inches. These costs include manholes. Lift station and land acquisition costs are also included separately. The lift station cost applies to stations with capacities less than approximately 4 mgd.

Table 6.1 Unit Construction Costs – Sewer System Improvements
Sewer Master Plan
City of El Centro

Description	Estimated Cost	
Pipelines	**Construction Cost (1)**	**Capital Cost (2)**
Diameter	**Unit Cost ($/lineal ft)**	**Unit Cost ($/lineal ft)**
8 inches	$ 218	$ 276
10 inches	$ 256	$ 324
12 inches	$ 263	$ 333
15 inches	$ 291	$ 368
18 inches	$ 316	$ 400
21 inches	$ 380	$ 481
24 inches	$ 418	$ 529
27 inches	$ 534	$ 676
30 inches	$ 642	$ 812
33 inches	$ 705	$ 892
36 inches	$ 770	$ 975
Lift Stations		

Table 6.1	Unit Construction Costs – Sewer System Improvements Sewer Master Plan City of El Centro
Description	**Estimated Cost**
Capacity (hp)	**Unit Cost ($/mgd)**
All	$ 450,000
Land Acquisition	
Area (acres)	**Unit Cost ($/acre)**
All	$ 200,000

Notes:

1. Includes 30-percent construction contingency.
2. Includes additional 15-percent for engineering, construction management and 10-percent for legal and administrative costs.

6.2 CAPITAL IMPROVEMENTS PROGRAM

6.2.1 Collection System Improvement Costs - Present

Table 6.2 provides the project costs for each pipe segment identified in Chapter 5. The table includes surcharged pipes and the connected pipes that are to be upgraded to maintain downstream pipe size.

Table 6.2 Recommended Upgrades - Present
Sewer Master Plan
City of El Centro

Pipe Location	H2OMap Sewer Model Number	Atlas Page	Pipe Length	Upgraded Pipe Size	Unit Cost	Total Cost
			(feet)	(inches)	($)	($)
Imperial Avenue & Holt Avenue	20-121055	20-12	237	15	$ 368	$ 87,200
Imperial Avenue & Orange Avenue	20-121070	20-12	306	15	$ 368	$ 112,600
Imperial Avenue & Brighton Avenue	21-121025	21-12	172	15	$ 368	$ 63,300
Imperial Avenue & Olive Street	21-121026	21-12	174	15	$ 368	$ 64,000
Imperial Avenue & Olive Street	21-121027	21-12	160	15	$ 368	$ 58,900

Table 6.2	Recommended Upgrades - Present Sewer Master Plan City of El Centro					
Pipe Location	**H2OMap Sewer Model Number**	**Atlas Page**	**Pipe Length**	**Upgraded Pipe Size**	**Unit Cost**	**Total Cost**
Imperial Avenue & State Street	21-121028	21-12	381	15	$ 368	$ 140,200
Imperial Avenue & Main Street	21-121029	21-12	418	15	$ 368	$ 153,800
Heil Avenue & Dogwood Road	20-151017	20-15	299	12	$ 333	$ 99,600
Heil Avenue & Dogwood Road	20-151015	20-15	120	12	$ 333	$ 40,000
Heil Avenue & Dogwood Road	20-151016	20-15	300	12	$ 333	$ 99,900
Heil Avenue & Hope Avenue	20-161006	20-16	390	12	$ 333	$ 129,900
Heil Avenue & Hope Avenue	20-161009	20-16	390	12	$ 333	$ 129,900
Heil Avenue & Hope Avenue	20-161023	20-16	404	12	$ 333	$ 134,500
4th Street & Brighton Avenue	21-141051	21-14	190	18	$ 400	$ 76,000
4th Street & Olive Avenue	21-141050	21-14	182	18	$ 400	$ 72,800
4th Street & Olive Avenue	21-141004	21-14	175	18	$ 400	$ 70,000
4th Street & State Street	21-141005	21-14	192	18	$ 400	$ 76,800
4th Street & State Street	21-141057	21-14	187	18	$ 400	$ 74,800
3rd Street & State Avenue	21-151018	21-15	189	12	$ 333	$ 62,900
3rd Street & Main Street	21-151008	21-15	183	12	$ 333	$ 61,000
3rd Street & Broadway Avenue	21-151029	21-15	386	12	$ 333	$ 128,500
3rd Street & Broadway Avenue	21-151013	21-15	191	12	$ 333	$ 63,600

Table 6.2	**Recommended Upgrades - Present** **Sewer Master Plan** **City of El Centro**					
Pipe Location	**H2OMap Sewer Model Number**	**Atlas Page**	**Pipe Length**	**Upgraded Pipe Size**	**Unit Cost**	**Total Cost**
3rd Street & Commercial Avenue	21-151043	21-15	176	12	$ 333	$ 58,600
8th Street & Ross Avenue	19-131067	19-13	194	8	$ 276	$ 53,600
8th Street & Yucca Drive	19-131068	19-13	500	8	$ 276	$ 138,000
8th Street & Tangerine Drive	19-131069	19-13	500	8	$ 276	$ 138,000
8th Street & Aurora Drive	18-131062	18-13	158	8	$ 276	$ 43,600
Total:						**$2,432,000**

As discussed in Chapter 5 and shown on Figure 5.5, upgrading approximately 1,900 feet of 8-inch gravity sewer downstream of the Heil and Dogwood Lift Station to the 15-inch sewer on Fairfield Avenue between Holt and East Heil Avenues would cost approximately $633,700. This is based on the unit cost for 12-inch pipe including contingency, engineering and construction management and legal and administrative fees provided in Table 6.1. Extending the force main would cost approximately $366,000 as provided in Table 6.3. Thus, it is recommended to extend the force main.

Table 6.3	6-Inch Forcemain Extension - Heil and Dogwood Lift Station Sewer Master Plan City of El Centro			
Description	**Length (feet)**	**Construction Unit Cost ($/linear foot)[1]**	**Construction Cost**	**Capital Cost [2]**
C900 PVC Pipeline from South Dogwood Road to Fairfield Avenue between Holt and East Heil Avenues	1,900	$ 126	$ 239,400	$ 302,800
Lift Station Pump Upgrades	-		$ 50,000	$ 63,000
Total:				**$ 365,800**

Notes:

1. Includes 30-percent construction contingency.
2. Includes additional 15-percent for engineering, construction management and 10-percent for legal and administrative costs.

6.2.2 Collection System Improvement Costs - Build Out

Table 6.4 provides the project costs for Build Out improvements to pipe segments identified in Chapter 5.

Table 6.4 Recommended Upgrades - Build Out
Sewer Master Plan
City of El Centro

Pipe Location	H2OMap Sewer Model Number	Atlas Page	Pipe Length	Pipe Size	Unit Cost	Total Cost
			(feet)	(inches)	($)	($)
Plank Drive & Ross Avenue	19-101010	19-10	298	12	$ 333	$ 99,200
Plank Drive & Ross Avenue	19-101011	19-10	299	12	$ 333	$ 99,600
Plank Drive & Ross Avenue	19-101012	19-10	291	12	$ 333	$ 96,900

Table 6.4	Recommended Upgrades - Build Out Sewer Master Plan City of El Centro					
Pipe Location	**H2OMap Sewer Model Number**	**Atlas Page**	**Pipe Length**	**Pipe Size**	**Unit Cost**	**Total Cost**
Plank Drive & Ross Avenue	20-101014	20-10	200	12	$ 333	$ 66,600
Plank Drive & Ross Avenue	20-101015	20-10	218	12	$ 333	$ 72,600
Total:						**$ 434,900**

Table 6.5 provides the project costs for Build Out improvements to lift stations as identified in Chapter 5.

Table 6.5	Recommended Lift Station Upgrades - Build Out Sewer Master Plan City of El Centro					
Lift Station	**H2OMap Sewer Model Number**	**Atlas Page**	**Duty Pumps**	**Existing Capacity**	**Required Capacity**	**Total Cost (1)**
			(#)	**(gpm / pump)**	**(total gpm)**	**($)**
Alder Canal / Villa Avenue	23-17-700	23-17	2	1,908	6,910	$4,478,000
Lift Station #3	25-11-700	25-11	2	3,000	14,500	$9,396,000
Northern Lift Station	9002	---	---	---	2,810	$1,821,000
Total:						**$15,695,000**
Notes:						
1. Total cost for lift station upgrades are based on $450,000 per mgd of pump capacity. Cost does not include land acquisition costs.						

6.2.3 WWTP Improvement Costs

Table 6.6 provides the CIP improvements identified for the WWTP.

Table 6.6 Recommended Wastewater Treatment Plant Upgrades
Sewer Master Plan
City of El Centro

Upgrade	Quantity	Unit	Unit Cost	Total Cost
Preliminary Treatment (Headworks and Grit Removal)	1	LS	$ 5,000,000	$ 5,000,000
Primary Clarifier Concrete Crack Repair (2 Clarifiers)	350 (1)	LF	$ 75	$ 26,000
Aeration Basin Aerator Platform Demolition (Basins 1 - 6)	6	Ea	$ 20,000	$ 120,000
Aeration Basin Ceramic Disk Replacement - (Basins 1 - 4 with 663 disks)	4	Ea	$ 5,300 (2)	$ 21,000
Aeration Basin Ceramic Disk Replacement - (Basins 5 - 6 with 810 disks)	2	Ea	$ 6,500 (2)	$ 13,000
Aeration Basin Stainless Steel Gate Replacement	38	Ea	$ 20,000 (3)	$ 760,000
Aeration Basin Concrete Spall Repair	2,000	SF	$ 100	$ 200,000
Aeration Basin South Walkway Replacement	1	LS	$ 18,000	$ 18,000
Boiler/Heat Exchanger	1	LS	$ 200,000 (4)	$ 205,000 (5)
Plant Water Pump Station	1	LS	$ 450,000	$ 450,000

Notes:

1. Assumes a 3.5-foot vertical crack every 5 feet around each clarifier.
2. Assumes disks installed by City personnel.
3. Includes $15,000 material cost and $5,000 labor for existing gate removal and disposal, concrete rehabilitation and new gate installation.
4. 760,000 BTU/hr Integral boiler/heat exchanger cost provided by JDV Equipment Corporation.
5. Includes labor cost for installation by City personnel.

TYLEDYNE FLOW MONITORING DATA AND SUMMARY

CITY OF EL CENTRO
WATER, SEWER AND STORM DRAINAGE MASTER PLAN

1.0 Project Summary

1.1 Overview

Carollo Engineers contracted with Teledyne Isco, Inc. to provide wastewater flow monitoring at nine (09) locations within the City of El Centro, CA. The scope of work was for a fourteen (14) day flow study. The actual flow monitoring project included installation, operation and maintenance over twenty (20) consecutive days. The monitoring period began in March 2007 and was completed by April 2007. This study was in support of the Water, Sewer and Storm Drainage Master Plan. The result of these efforts is described in this report.

To verify meter operation, Isco conducts meter verifications independent of meter equipment measurements. Manual depths of flow (DOF) measurements are performed using a ruled tape. Velocity measurements are taken using a handheld point-velocity measurement probe. Depth measurement is used for direct comparison or to verify an offset value, if the sensor is not mounted on the invert of the pipe. The velocity measurement is used as a guide to verify proper operation of the velocity sensor. Handheld velocity measurements cannot be directly compared to meter measurements due to spatial and temporal differences between the two measurements.

Isco Field Application Specialists performed flow meter site visits approximately three (3) times following initial installation to confirm satisfactory equipment operation and to collect depth, velocity and flow measurements.

Every installed flow meter on this project met or exceeded engineering performance specifications. High quality flow data was collected at each of the nine (09) locations.

The data contained in this report is for the period of 17 March 2007 through 30 March 2007.

Table 1.1 – This Flow Monitoring Summary Table contains a summary of important flow data and a numerical and narrative description of each meter location. Additional site detail is contained in the attached site sheets.

Table 1.1 - Flow Monitoring Summary

Manhole# TI Site	Location	Pipe Diam (in)	Avg Depth (in)	Peak Depth (in)	Max d/D Peak Depth / Diam.	Avg Vel (fps)	Peak Vel (fps)	Avg Daily Flow (mgd / gpm*)	Peak Flow (mgd / gpm*)	Peaking Factor Peak Flow / Avg Flow
07020301	N Imperial Avenue.	15.00	6.98	9.48	63.2%	2.90	3.32	1.07	1.64	1.54
07020302	4th Street & Adams Avenue	22.00	4.97	6.12	27.8%	5.90	6.25	1.72	2.40	1.40
07020303	Commercial Avenue & 3rd	10.00	4.43	6.55	65.5%	0.66	1.27	0.09	0.29	3.19
07020304	Commercial Avenue	18.00	10.31	12.62	70.1%	1.17	1.48	0.58	0.99	1.71
07020305	4th Street & Brighton Avenue	14.00	4.88	6.44	46.0%	1.87	2.36	0.41	0.68	1.65
07020306	Imperial & Brighton Avenues	12.00	5.04	11.81	98.4%	2.58	3.00	0.53	1.02	1.91
07020307	La Brucherie Avenue	27.00	5.57	7.54	27.9%	1.47	1.79	0.59	1.04	1.76
07020308	Imperial & Manuel Ortiz Avenues	18.00	5.03	6.31	35.1%	1.00	1.44	0.28	0.49	1.74
07020309	Wake & Merrill Center Drive	12.00	0.75	1.64	13.7%	0.46	1.25	**4.71***	**36.98***	7.85

CITY OF EL CENTRO
WATER, SEWER AND STORM DRAINAGE MASTER PLAN

2.0 Equipment Selection

Isco is a manufacturer of flow meter equipment. The flow meters produced by Isco use various depth measurement and velocity measurement technologies. Each of the technologies will provide data of high quality when properly applied to specific environmental, hydraulic and physical conditions. Flow meter equipment provided by Isco is suitable for open channel wastewater flow metering.

Isco provided portable, battery-powered equipment at each of the flow meter locations. Each flow meter was programmed to record the measured flow depth and velocity at 5-minute intervals.

The actual meter technology used at each location was based upon an evaluation of site conditions during flow meter installation. The evaluation includes but is not limited to pipe size, water quality, expected minimum depth, expected maximum depth, expected minimum velocity, expected maximum velocity, silt levels, presence of debris, and surcharge evidence. Isco used its considerable global experience in complex flow metering systems to determine the best type of meter technology to install based on the specific environmental, hydraulic and physical conditions observed at the time of equipment installation.

For the El Centro Master Plan project, the 2150 Area Velocity (AV) flow meter was primarily used. The 2150 employs continuous wave Doppler velocity measurement technology and a differential pressure depth measurement sensor housed in a single probe. Continuous wave Doppler

(CWD) velocity measurement technology is best suited for pipes with shallow water depths less than 40 inches, velocity ranging from 0.8 fps to 5.0 fps and near uniform flow conditions. The 2150 obtains data of high quality when properly applied in a wide range of flow conditions.

3.0 Data Presentation

This section of the report provides a detailed description of the flow meter station information and flow data provided for each meter location. Flow meter station information and flow meter data graphs and tabular data are provided for each meter location at the end of this report.

3.1 Field Investigation Reports

The Field Investigation Report consists of the Temporary Flow Monitoring (TFM) Site Information Form. The TFM Site Information Form provides an illustration of the physical location of each flow monitoring station. Pertinent information relative to site access, safety, instrumentation, additional notes, and hydraulic conditions are listed. Manhole Condition and Site Calibration is recorded and kept on file for reference.

3.2 Site Summary Sheet

The Site Summary Sheet is provided for a quick overview of the flow monitoring results at each site. It contains the average, minimum, and maximum values for depth of flow, average velocity, and flow rate over the duration of the monitoring period and a flow data hydrograph.

3.3 Hydrograph (Times Series) Data Presentation

A graphical time series presentation of flow Depth (inches), Average Velocity (feet/second), and Flow Rate (mgd or gpm) is provided for each site. Graphs are created using 15-minute averages of the measured data. The stacked axis allows easy visual identification of system performance.

3.4 Scattergraphs

Scattergraphs, or X-Y plots of observed average velocities and flow rates versus observed depths, are provided for each site. These plots provide a graphical representation of hydraulic conditions at the sites. These graphs are particularly useful for showing a site's hydraulic reaction to conditions such as backwater and surcharge.

3.5 Tabular Data Presentation

Tabular presentations of Flow Rate (mgd or gpm), Average Velocity (ft/s), and Depth (inches) are provided as a function of time of day and date. Per contract requirements, the flow monitors recorded data at all monitoring locations at 5-minute intervals. The tabular information in this report provides hourly averages of these 5-minute recordings. For example, all flow rate, or discharge, measurements recorded from 00:00 through 00:59 for a given day are reported as an hourly average on the row "0-1" of the tabular report.

Hourly-averages of the measured data are provided for each day of monitoring as well as the average, hourly minimum and maximum and instantaneous minimum and maximum values.

At the bottom of each day's column of hourly average data are summary statistics for that day, as follows:

- The *"Average"* is the average of all instantaneous readings recorded during that day.

- The *"Maximum Hourly Average"* is the maximum hourly average shown in the hours 0 through 24.
- The *"Minimum Hourly Average"* is the minimum hourly average shown in the hours 0 through 24.
- The *"Instantaneous Maximum"* is the greatest single reading data value obtained during the day.
- The *"Instantaneous Minimum"* is the smallest single reading data value obtained during the day.

3.6 Electronic Data Presentation

Electronic data are provided on the accompanying CD. Flow rate, depth and average velocity data in 15-minute increments are provided in a CSV format. The data are identified by the file name, which consists of the Teledyne Isco's contract number (070203), site number (01) and the CSV extension (ex. 07020301.CSV).

4.0 Results and Findings

4.1 Flow Data Reduction

An evaluation of flow data, as recorded by the flow monitors, was performed by an Isco data analyst. A detailed reporting of flows for each location follows. Overall, the meters operated well during the metering period, and the results are within the expected abilities of open channel flow metering and equipment.

Because of particularly low flows at Site #09, the data for Flow Rate is presented in gallons per minute (GPM) instead of the standard million gallons per day (MGD). Using gallons per minute (GPM) will provide you with a comprehensible actual flow quantity for this low flow site.

4.2 Flow Data Observations

Site #01 a 15” influent installation with no silt. Data indicates a typical diurnal pattern and equivalent scattergraph. Depths ranged from a minimum of 4.62” to a maximum of 9.48”. The average depth was 6.98”. The peak d/D ratio (peak depth / pipe diameter) was 63.2%. Velocity ranged from a minimum of 2.27 fps to a maximum of 3.32 fps. The average velocity was 2.90 fps. Flow rates ranged from a minimum of 0.48 mgd to a peak of 1.64 mgd. The average flow rate for the fourteen (14) days was 1.07 mgd.

Site #02 a 22” influent installation with no silt. Data indicates turbulent flow conditions due to the high rate of velocity as evidenced by the average velocity being nearly 6.0 fps. Depths ranged from a minimum of

3.16” to a maximum of 6.12”. The average depth was 4.97”. The peak d/D ratio (peak depth / pipe diameter) was 27.8%. Velocity ranged from a minimum of 5.51 fps to a maximum of 6.25 fps. The average velocity was 5.90 fps. Flow rates ranged from a minimum of 0.85 mgd to a peak of 2.40 mgd. The average flow rate for the fourteen (14) days was 1.72 mgd.

Site #03 a 10” effluent installation with 1.0” of silt noted at time of equipment installation. Data reflects the low flow conditions complicated by the presence of silt. Depths ranged from a minimum of 3.47” to a maximum of 6.55”. The average depth was 4.43”. The peak d/D ratio (peak depth / pipe diameter) was 65.5%. Velocity ranged from a minimum of 0.34 fps to a maximum of 1.27 fps. The average velocity was 0.66 fps. Flow rates ranged from a minimum of 0.03 mgd to a peak of 0.29 mgd. The average flow rate for the fourteen (14) days was 0.09 mgd.

Site #04 a 15” effluent installation with 4.0” of silt noted at time of equipment installation. Data reflects a deep slow flow complicated by a significant silt bed; these conditions create non-uniform flow. The diurnal pattern and scattergraph are consistent with this problematical flow environment. Depths ranged from a minimum of 7.38” to a maximum of 12.62”. The average depth was 10.31”. The peak d/D ratio (peak depth / pipe diameter) was 70.1%. Velocity ranged from a minimum of 0.73 fps to a maximum of 1.48 fps. The average velocity was 1.17 fps. Flow rates ranged from a minimum of 0.19 mgd to a peak of 0.99 mgd. The average flow rate for the fourteen (14) days was 0.58 mgd.

CITY OF EL CENTRO
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Site #05 a 14" influent installation with no silt. Data indicates a typical diurnal pattern and equivalent scattergraph. Depths ranged from a minimum of 3.40" to a maximum of 6.44". The average depth was 4.88". The peak d/D ratio (peak depth / pipe diameter) was 46.0%. Velocity ranged from a minimum of 1.29 fps to a maximum of 2.36 fps. The average velocity was 1.87 fps. Flow rates ranged from a minimum of 0.17 mgd to a peak of 0.68 mgd. The average flow rate for the fourteen (14) days was 0.41 mgd.

Site #06 a 12" effluent installation with no silt. Data indicates a typical diurnal pattern and equivalent scattergraph consistent with recurring high flow events. Depths ranged from a minimum of 2.96" to a maximum of 11.81". The average depth was 5.04". The peak d/D ratio (peak depth / pipe diameter) was 98.4%. Velocity ranged from a minimum of 1.74 fps to a maximum of 3.00 fps. The average velocity was 2.58 fps. Flow rates ranged from a minimum of 0.18 mgd to a peak of 1.02 mgd. The average flow rate for the fourteen (14) days was 0.53 mgd.

Site #07 a 27" influent installation with no silt. Data indicates a typical diurnal pattern and equivalent scattergraph. Depths ranged from a minimum of 2.97" to a maximum of 7.54". The average depth was 5.57". The peak d/D ratio (peak depth / pipe diameter) was 27.9%. Velocity ranged from a minimum of 0.95 fps to a maximum of 1.79 fps. The average velocity was 1.47 fps. Flow rates ranged from a minimum of 0.15 mgd to a peak of 1.04 mgd. The average flow rate for the fourteen (14) days was 0.59 mgd.

Site #08 an 18” effluent installation with both silt and debris noted at time of installation. Both silt and debris were confined to the bench region. Data indicates a heavy pumping influence throughout the monitoring period. Depths ranged from a minimum of 3.26” to a maximum of 6.31”. The average depth was 5.03”. The peak d/D ratio (peak depth / pipe diameter) was 35.1%. Velocity ranged from a minimum of 0.34 fps to a maximum of 1.44 fps. The average velocity was 1.00 fps. Flow rates ranged from a minimum of 0.05 mgd to a peak of 0.49 mgd. The average flow rate for the fourteen (14) days was 0.28 mgd.

Site #09 a 12” effluent installation with no silt. Data indicates a diurnal pattern and scattergraph consistent with low depth / low velocity flow conditions. For example, during periods of minimum flow, depth was less than 0.27 inch and velocity at 0.17 fps. The extreme low flow conditions are reflected in the data throughout the monitoring period. Depths ranged from a minimum of 0.27” to a maximum of 1.64”. The average depth was 0.75”. The peak d/D ratio (peak depth / pipe diameter) was 13.7%. Velocity ranged from a minimum of 0.17 fps to a maximum of 1.25 fps. The average velocity was 0.46 fps. Flow rates ranged from a minimum of 0.57 gpm to a peak of 36.98 gpm. The average flow rate for the fourteen (14) days was 4.71 gpm.

FLOW FACTOR DEVELOPMENT METHODOLOGY

Teledyne Isco monitored sewer flows at nine sites from Saturday, March 17, 2007 through Friday, March 30, 2007. Results of the study were provided to Carollo Engineers on April 20, 2007. The Flow Monitoring Summary provided in the report gives the meter site locations and measured and calculated hydraulic data. These results were used in conjunction with the calculated acreages of the various land uses presented in the 2004 General Plan to determine an average flow per acre. The land use areas were taken from the GIS land use layer provided by Nobel Systems. Weekday data for each meter site was averaged for each hour of each day. Hybrid flow and diurnal curves were developed by averaging the hourly data for each day. The methodology presented below outlines the steps for developing wastewater flow factors for the various land uses, which when multiplied by the areas for each land use, allowed comparison with the metered data and the total influent flow at the wastewater treatment plant.

1.1 METHODOLOGY

The initial methodology for estimating wastewater flow factors included the following steps:

- Select a meter site comprised of primarily residential flows,
- Determine land uses and areas from GIS that are tributary to this meter site,
- Quantify the number of dwellings within each land use from an aerial photo,
- Calculate population using factors from the Study Area, Land Use and Population Technical Memorandum,
- Adjust residential flow factors to allow total calculated flow to match the metered flow within reason while maintaining general relationships among the various factors,
- Apply residential factors to other metered sites that include non-residential areas and adjust non-residential flow factors to allow the total flow to match the metered flow within reason,
- Compare the calculated flow based on the flow factors applied to the total land areas with the reported influent flow at the wastewater treatment plant, and
- Adjust non-residential flow factors until the total flow is within five percent of the plant flow.

1.2 METER SITE 7

To implement the above methodology, meter site 7 was chosen for step 1 since it was comprised of primarily residential flows, which included 78 percent Low Density Residential (LDR), 10 percent Rural Residential (RR), 10 percent Public, and 2 percent General Commercial (GC). The total tributary area is approximately 410 acres. Meter site 8 was upstream of site 7, but the flows from site 8 were subtracted from the flows from site 7. The

Public land use area is Southwest High School with a student body population of 2,186 students, which was obtained from the Central Union School District. Meter site 7 was located on La Brusherie Avenue between Olive and Main Streets on a 27-inch diameter trunk.

The flow from Southwest High School was removed from the total flow before estimation of the residential flow factors for the RR and LDR land uses. Metcalf and Eddy's ***Wastewater Engineering, Treatment and Reuse*** reports a general flow factor of 15 gallons/day/student. Thus, approximately 32,800 gallons per day (gpd) was removed from the total metered flow. The General Commercial area was assumed to be zero for this analysis.

The RR and LDR land uses comprised the remainder of the flow. The RR land use area within the meter site 7 tributary area was approximately 40 acres while that for the LDR land use was approximately 320 acres. Thus, the LDR flow factor determined in the analysis would be the basis for the other residential flow factors. Although there were no set percentage goals for maintaining the relationships among the various residential flow factors, it was assumed that the flow factors would increase as the land use density increased. Also, the flow factors were compared to the water flow factors and were maintained in the range of 35 percent to 60 percent of the water factors.

1.3 METER SITE 4

Commercial and industrial flow factors were estimated using data from meter site 4, which was located on Commercial Avenue between 2nd and 3rd Streets. The land use designations were a mix of residential, public commercial and industrial areas with a total tributary area of 650 acres. The General Industrial (GI) and General Commercial (GC) land uses comprised approximately 410 acres and 22 acres, respectively. The Public land use includes two schools, the Desert Oasis High School and Washington Elementary School. Flow factors established from meter site 7 were applied to the land areas for these schools. In addition, residential flows were calculated using the flow factors determined from meter site 7. The calculated flows for the schools and residential areas were subtracted from the metered flow to determine the flow attributed to the GI and GC land uses. The flow factors for all commercial areas were assumed equal, and the flow factors for all industrial areas were assumed equal.

1.4 CITY-WIDE FLOW VERSUS WASTEWATER TREATMENT PLANT FLOW

After applying the residential and non-residential flow factors to the various land uses for the total developed area within the City, the calculated flow was higher than the reported flow at the Plant. The calculated flow was approximately 4.5 million gallons per day (mgd). The reported Plant flow was 3.65 mgd. Based on the reported Plant flow and the current population, the per capita wastewater generation rate is approximately 88 gallons per capita per day (gpcd). Based on 4.5 mgd, the rate is approximately 107 gpcd. Given that the Plant flow meter is calibrated regularly, no reasons exist to believe any other factors may be causing erroneous readings and that 88 gpcd is a more reasonable generation rate, it was concluded that the flow rates from meter site 4 were incorrect. This could be a result of an improperly calibrated meter or use of incorrect hydraulic parameters such as pipe slope when calculating flow rates. Carollo is currently determining how Teledyne ISCO calculated the flow rates using the measured depth and velocity information.

The residential flow factors developed from meter site 7 data are reasonable based on previous master planning projects. Thus, the non-residential components determined from site 4 were adjusted downward to allow the total City flow to match the Plant flow. Table 1 lists the final flow factors.

Table 1 Land Use Flow Factors
El Centro Sewer Master Plan
City of El Centro

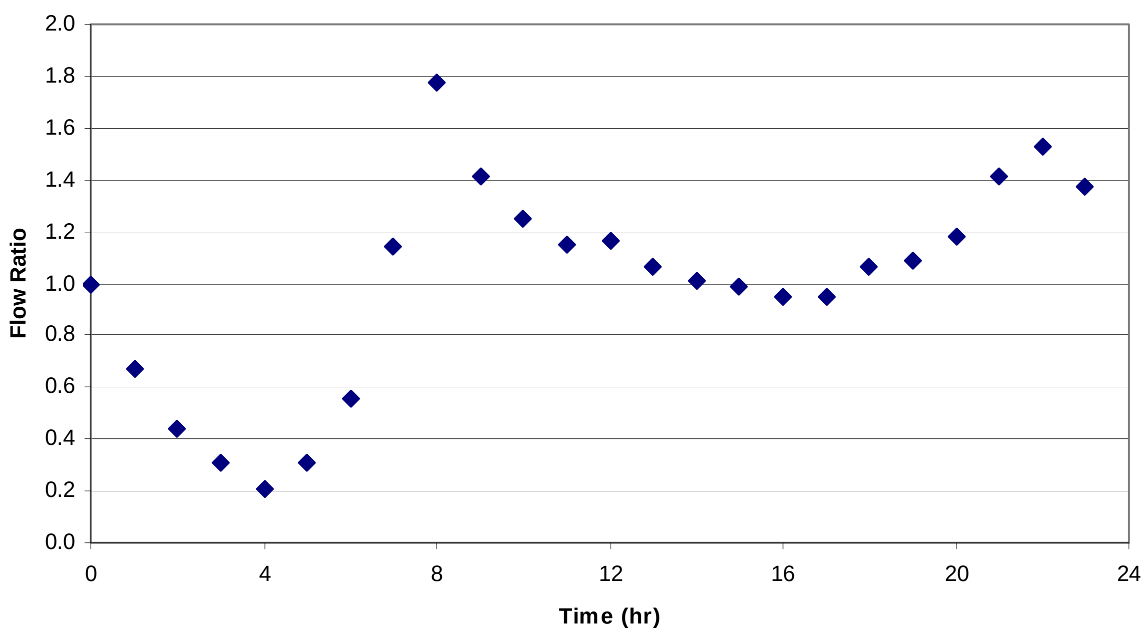
Land Use	Flow Factor (gpd/acre)	Developed Area (acre)	Calculated Flow (mgd)
Rural Residential	350	162	0.06
Low Density Residential	1,100	1,452	1.60
Medium Density Residential	1,400	191	0.27
High-Medium Residential	1,600	334	0.53
General Commercial	800	480	0.38
Downtown Commercial	800	1	0.00
Tourist Commercial	800	223	0.18
General Industrial	400	695	0.28
Planned Industrial	400	115	0.05
Civic	800	54	0.04
Public	650	652	0.42
Total:		**4,359**	**3.81**

Table 1	Land Use Flow Factors El Centro Sewer Master Plan City of El Centro		
Land Use	**Flow Factor (gpd/acre)**	**Developed Area (acre)**	**Calculated Flow (mgd)**
Note: The average monthly flow for March, 2007 was measured as 3.65 million gallons per day (mgd).			

1.5 DAILY FLOW VARIATION - DIURNAL CURVES

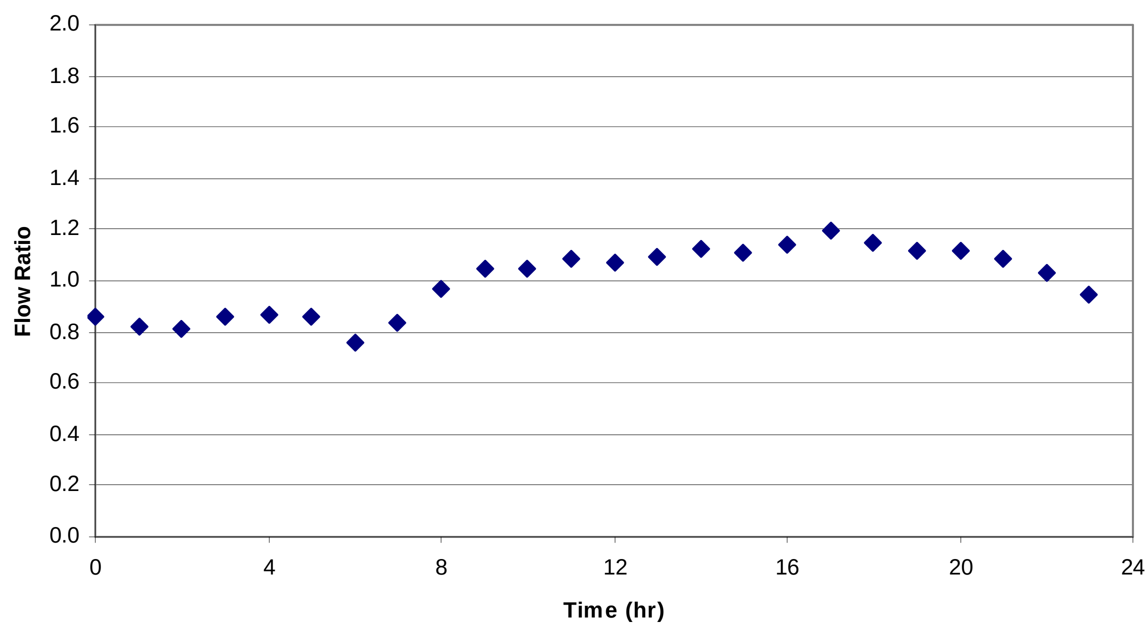
Flow factors and metering data were used to generate diurnal curves for residential and non-residential areas. The residential diurnal curve was created using the metered data from meter site 7, a predominantly residential area. The ratio of average hourly flow to average daily flow during the weekdays was used to create the curve. The curves differed slightly between the weekdays and weekends; therefore, weekend flow data was not used. The residential diurnal curve is shown in Figure 1.

FIGURE 1
Residential Diurnal Curve



The non-residential diurnal curve was created using the data and land use areas from meter site 4. The hourly residential flow values were subtracted from the total hourly flows to give a variation based on non-residential flows. The ratio of average hourly flow to average daily flow was then used to create the non-residential diurnal curve. The non-residential diurnal curve is shown in Figure 2.

FIGURE 2
Non-Residential Diurnal Curve



The diurnal curves were then applied to the calculated flow for each metered area. The results are shown in Figures 3 through 8. Only six of the nine metered sites are shown. The tributary area for site 5 was unclear because the meter was located on one of the parallel pipes along 4th Street. These two parallel pipes interconnect. Meter site 8 included flows outside the Sphere of Influence south of the City, and the flow from meter site 9 was very low. Site 9 was located at Wake and Merrill Center Drive in the south part of the City, and the average flow was less than 10,000 gpd. Teledyne Isco reported the instantaneous flows for site 9 in gallons per minute instead of million gallons per day for the other sites.

The calculated flows and the metered flows compare well for sites 2, 3, 6 and 7. Site 2 calculated flow shows a dip during mid-afternoon, but the peak flow in the late morning is maintained. Sites 1 and 4 do not compare well. Site 4 was discussed above. The non-residential flow factors were adjusted downward to allow the total City flow to match the Plant flow. Thus, the overall curve adjusted downward.

Site 1 was downstream of site 6, and the metered data from site 1 indicates that flow doubled over that for site 6 for a relatively small area. It is not clear why this is the case.

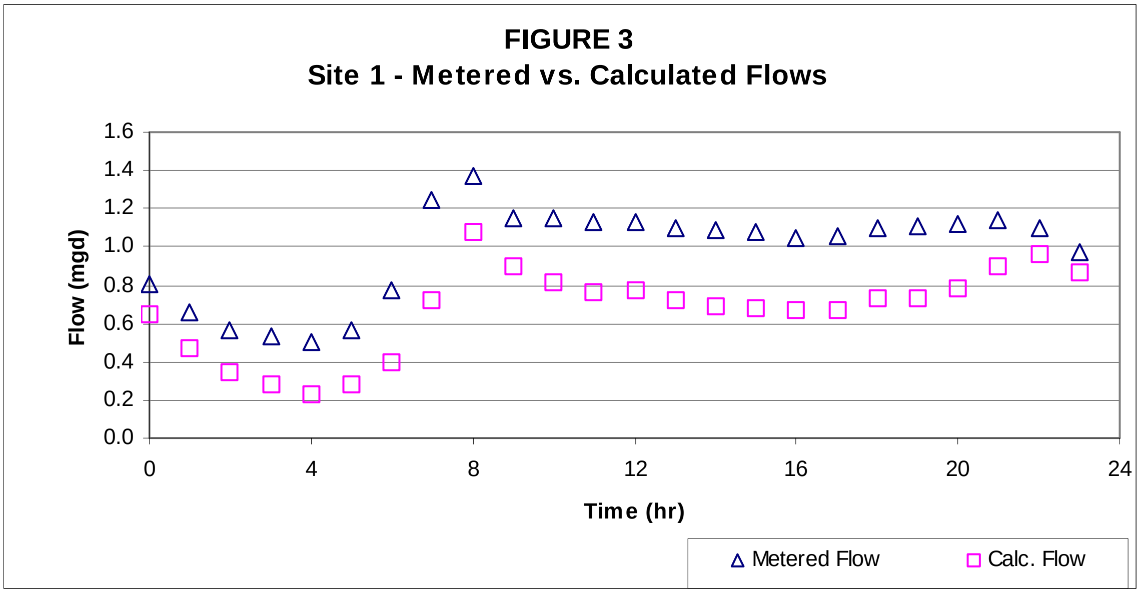


FIGURE 3
Site 1 - Metered vs. Calculated Flows
Flow (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
4
8
12
16
20
24
Time (hr)
Metered Flow
Calc. Flow

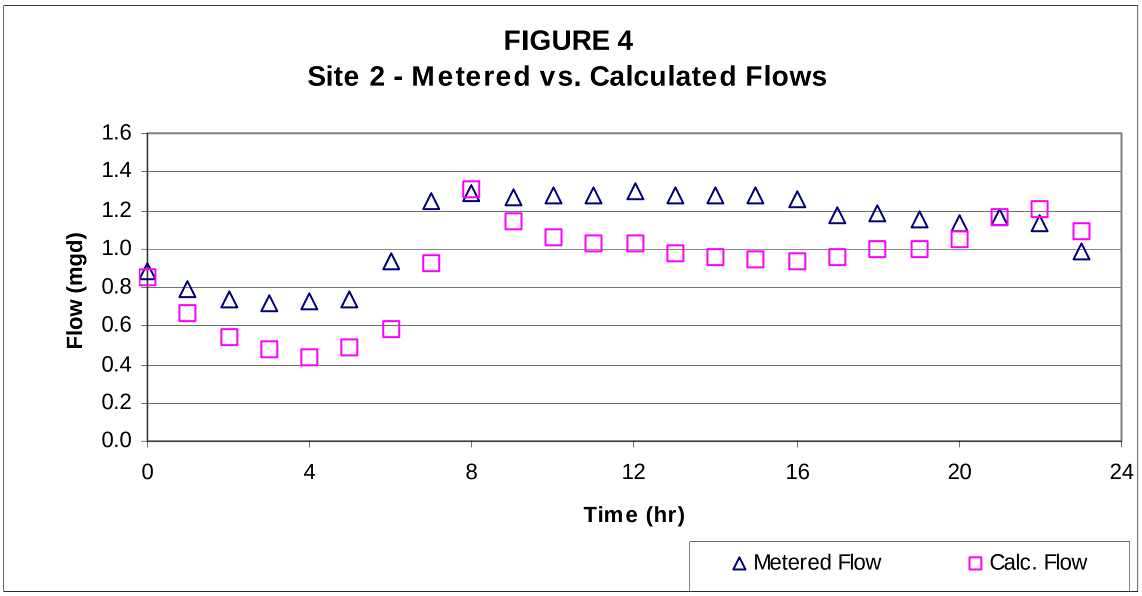


FIGURE 4
Site 2 - Metered vs. Calculated Flows
Flow (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
4
8
12
16
20
24
Time (hr)
Metered Flow
Calc. Flow

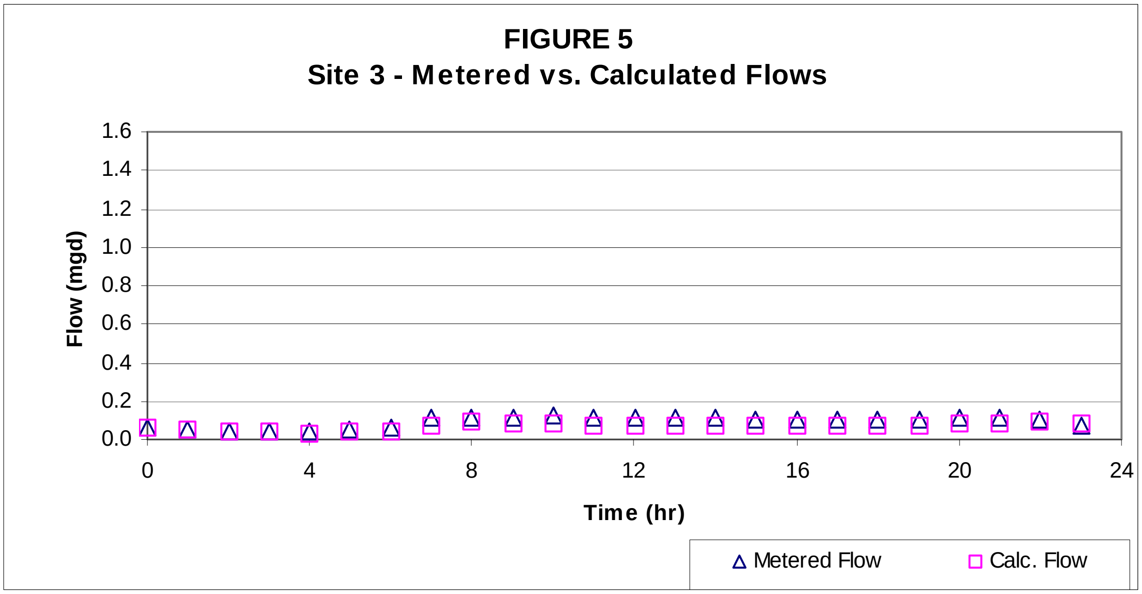


FIGURE 5
Site 3 - Metered vs. Calculated Flows
Flow (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
4
8
12
16
20
24
Time (hr)
Metered Flow
Calc. Flow

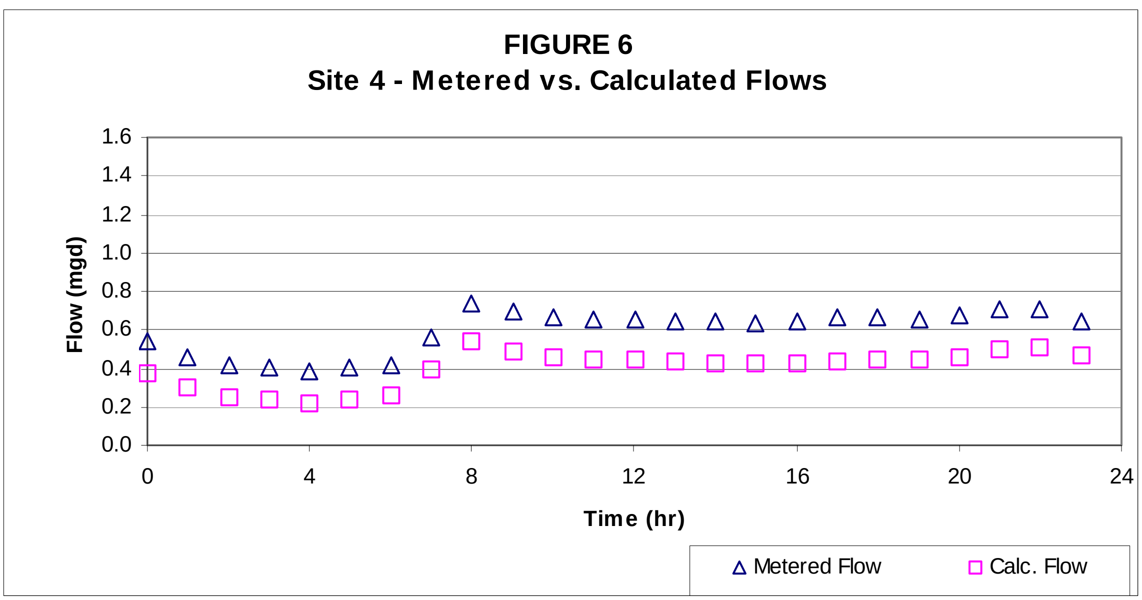


FIGURE 6
Site 4 - Metered vs. Calculated Flows
Flow (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
4
8
12
16
20
24
Time (hr)
Metered Flow
Calc. Flow

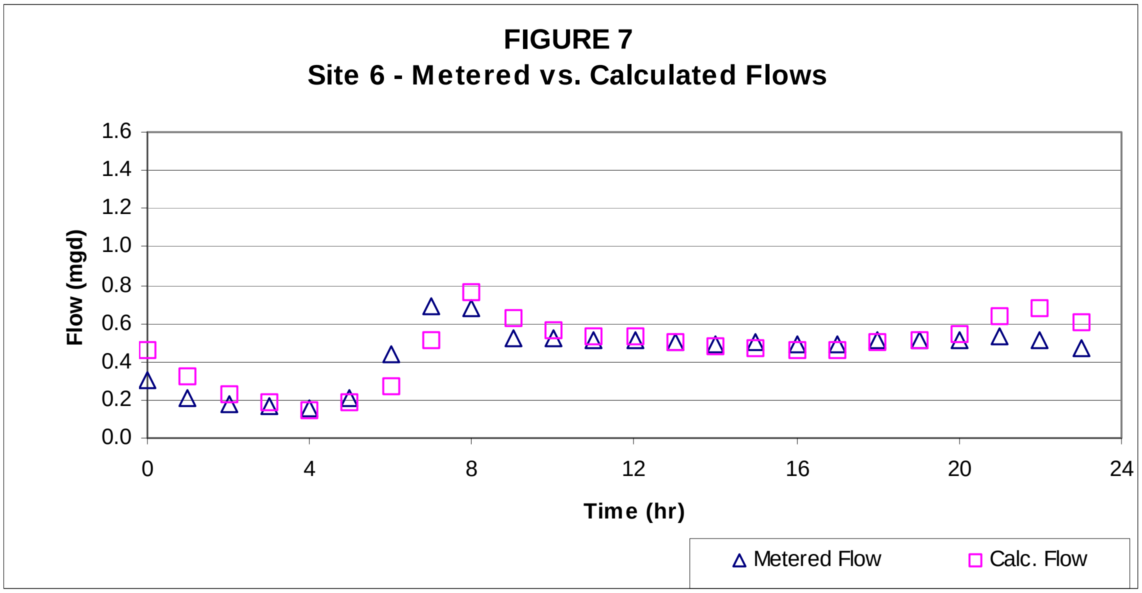


FIGURE 7
Site 6 - Metered vs. Calculated Flows
Flow (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
4
8
12
16
20
24
Time (hr)
Metered Flow
Calc. Flow

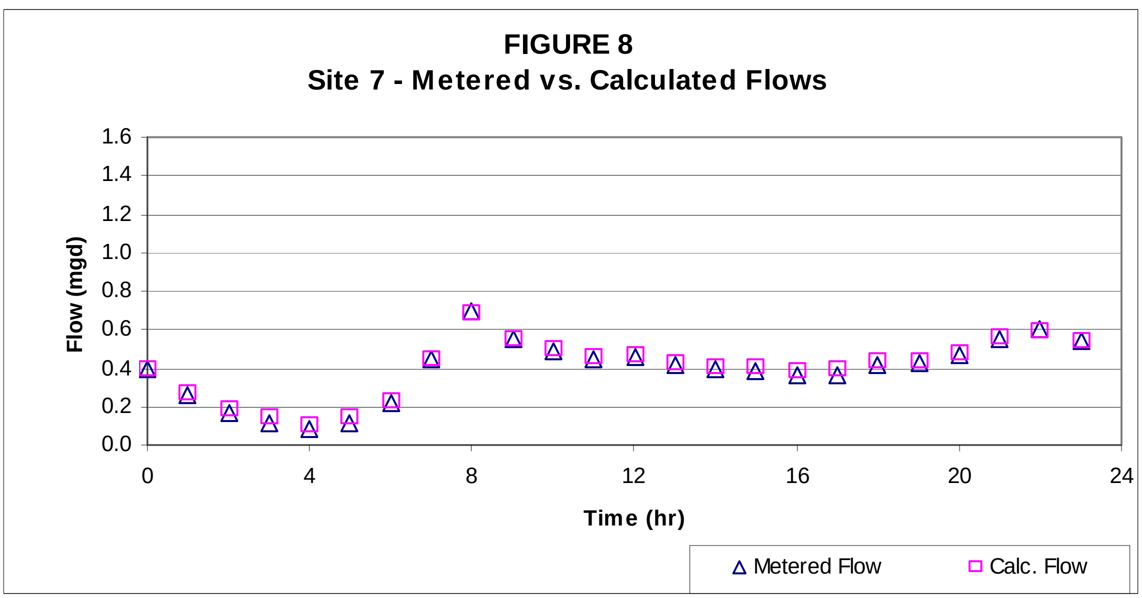


FIGURE 8
Site 7 - Metered vs. Calculated Flows
Flow (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
4
8
12
16
20
24
Time (hr)
Metered Flow
Calc. Flow

PILOT TESTING FOR CONVENTIONAL AND PRIORITY POLLUTANT REMOVAL - CITY OF DAVIS, CA

Based on the Master Plan recommendations, a conventional secondary activated sludge process followed by either tertiary filtration or membranes was considered. Specifically for metals removal, the pilot testing investigated the ability of an activated sludge process followed by tertiary filtration, specifically in a membrane biological reactor (MBR) configuration, to remove metals to anticipated levels dictated by CTR requirements. The MBR process requires fine screens downstream of the primary treatment process to remove fibrous materials not previously removed. The facilities include two basins: a plug-flow activated sludge reactor with aeration and a tank to house the membranes. The membranes would be polymeric filtration media with a pore size from 0.04 to 0.4 microns. The effluent would be pumped to UV disinfection facilities before discharge.

Based on characterization of the primary effluent entering the MBR system, the dissolved selenium fraction was on average 99 percent of the total. Selenium speciation showed that the MBR influent was predominantly organic, or more likely to bond with some organic constituent. The MBR process alone was not expected to remove dissolved metals, especially those bound to organic compounds to any significant degree, but the team investigated several operational strategies in an attempt to increase total metals removal. These investigations specifically targeted selenium reduction through the MBR process.

The pilot testing involved several phases. Each phase varied the solids retention time (SRT) and chemical addition. Two phases included no chemical addition with an SRT of 10 and 30 days, respectively. Three other phases included the addition of ferric chloride ($FeCl_3$) at either 20 mg/L or 40 mg/L at SRTs of either 10, 20 or 30 days. Coagulation with ferric chloride had negligible effect on selenium removal. The 30-day SRT did show some improvement, though, but none of the tests reduced the selenium levels to meet the effluent criteria.

In other testing, neither oxidation of organic selenium with chlorine or anaerobic treatment prior to coagulation and filtration was observed to have a beneficial impact on selenium reduction. The pilot testing concluded that the MBR process produced a high quality effluent with respect to conventional pollutants and offered benefits for downstream UV disinfection. In addition, due to high suspended solids removal, particulate metals were generally removed below the CTR requirements. However, the MBR process was not able to remove dissolved metals to any degree. Although not tested, other potential metals removal strategies were compared based on risk, cost and benefits. Two non-site specific processes included ion exchange and reverse osmosis. These would act as an effluent polishing step to an activated sludge/filtration train. The reverse osmosis system was determined to be the most likely to remove selenium, but it has a high cost with installing and operating the system and includes the challenge of brine disposal.