WASTEWATER MASTER PLAN
BRAWLEY, CALIFORNIA

(Final Submittal)
APRIL 2013

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Introduction

The City of Brawley (City) owns and operates its sewer system and storm drain system; for the sewer system, this generally includes gravity mains, pump stations, forcemains, and a treatment plant. In addition, a portion of the City contains a combined sewer system that collects stormwater runoff from adjacent inlets. The separate storm drain system includes storm inlets, gravity mains, detention ponds, stormwater pump stations, and forcemains. This Sewer/Storm Drain Master Plan analyzes and evaluates these facilities and their ability to meet current and projected demands, along with the feasibility of separating the combined system. It also addresses capacity and regulatory requirements in order to assure continued reliable sewer service and proper drainage. This Plan is an update to the previous Sewer Master Plan and Capital Improvement Program prepared in 1999.

Scope of Work

The Sewer/Storm Drain Master Plan scope of work includes the following tasks:

1. Sewer System Design Criteria
2. Storm Drain Design Criteria
3. Sewer Demand Projections
4. Sewer System Computer Modeling
5. Wastewater Treatment Plant Capacity
6. Capital Improvement Project Recommendations

Master Plan Development

Portions of this Master Plan have been based on fundamental assumptions established throughout the project. The City and Psomas discussed these assumptions and agreed that they resulted in a reasonable approach to developing the Master Plan. One of these assumptions is to use the Year 2030 as the future buildout planning year.

To help quantify the Master Plan objectives, a minimum acceptable level of service was established to help identify deficiencies in existing facilities, as well as to help determine the need for, and size of, proposed improvements. The established criteria meet typical requirements for sewer agencies throughout Southern California.

Existing Sewer/Combined System Facilities

Currently, the City manages an existing sewer system that includes a 5.9-Million Gallon Per Day (MGD) wastewater treatment plant (WWTP), three (3) sewage pump stations, approximately 2.10 miles of forcemain, 77 miles of gravity sewer pipeline, and approximately 1,440 manholes.

The combined system is located approximately between the borders of River Drive to the north, Best Avenue to the east, Malan Street to the south, and the western-most boundary, which abuts the New River. There are hundreds of inlets throughout this area to collect stormwater runoff and discharge it to the sewer system. The exact number of inlets is unknown.

Existing Separate Storm Drain System Facilities

In addition to the sewer/combined system, the City also manages a separate storm drain system. This system consists of approximately 17 miles of gravity pipeline, hundreds of inlets, 6 to 8 detention basins, three (3) stormwater pump stations, and approximately 300 feet of forcemain. In addition, the City has recently acquired the Bryant Drain from IID and will be undergrounding a portion of it. Most of the older portion of the system discharge to the New River at various locations, while the newer systems (mostly in the areas south of Malan Street) discharge to individual detention basins sized for the 100-year storm. Much of the runoff collected in these basins evaporates and infiltrates into the ground, while a small pump station pumps runoff to the nearest gravity system.

Sewer Demands and Planning Data

Sewer demands represent sewage that enters the distribution system through legal, and sometimes illegal, lateral connections. Sewer demands occur throughout the collection system and vary based on the number and type of consumer in each location. In the past, the City did not meter individual residences. Instead, it charged a flat water/sewer service fee, which led to high water usage. In recent years, the City has installed meters to nearly all customers and as a result, water usage has dropped, which has in turn reduced the amount of sewage generated.

To analyze demands, the historical metered flow at the WWTP was reviewed, along with flow monitoring data and land use. For this report, an average sewage generation of 3.81 MGD was used, based on metered data from the WWTP for a 15-month period between January 2011 and March 2012.

Land use designations were used to calibrate existing demands and project future demands for 2020 and 2030. The current General Plan was used to estimate land use areas, populations, and dwelling units within the current City limits and the existing sphere of influence.

This analysis was complicated by the fact that the current recession has resulted in a number of approved, partially-constructed, and partially-occupied subdivisions that needed to be considered in the existing analysis. For this Plan, it was assumed that only occupied homes/businesses would be included and that the remaining units would be occupied by the 2020 scenario. Projected populations were: 24,953 existing, 42,748 year 2020, and 60,524 year 2030. Existing sewer demand factors were developed for the various land uses, including low density residential, medium density residential, commercial, public facilities, industrial facilities, and light industrial/business park. Sewer demand factors were then developed for each land use to have the total reach the current average daily flow of 3.83 MGD.

Peaking factors were estimated to establish maximum day demands and peak hour demands. For dry weather flows, these factors are 2.32, 2.23, and 2.19 for current, 2020, and 2030 peak dry weather flows.

In addition, the City has been in discussions with the City of Imperial regarding the planned Rancho Los Lagos development located south of the City's sphere of influence. For the purposes of this Plan, it is assumed the City of Imperial will provide sewer services for this development.

Sewer Distribution System Analysis

The model development and analysis for this 2012 Master Plan was completed primarily within the computer modeling software "InfoSewer", with the final model deliverables being exported to EPANet files for the City's use. For this Master Plan, the City elected to leverage GIS by utilizing a hydraulic model that incorporates GIS features into the hydraulic model analyses. The roughness coefficients used for the new hydraulic model were 0.012, 0.011, 0.013, and 0.014 for Ductile Iron Pipe (DIP), Polyvinyl Chloride (PVC), pipe materials that were unknown, and Vitrified Clay (VCP), respectively.

The original models were updated to reflect additional developments and facilities constructed since 1999. Manholes were verified, when practical, and total flows were based on an average daily flow of 3.83 MGD. Inflow and infiltration (I&I) was estimated based on recent rainfall data. I&I flows were allocated evenly to each known storm inlet within the combined system limits and total flows verified.

In master planning, the computer model assists in measuring system performance, analyzing operational improvements, and developing a systematic method of determining the size and timing required for new facilities. The calibrated model can be used to analyze existing sewer systems, future sewer systems, or even specific improvements to the existing sewer system.

The hydraulic computer model was used to simulate the existing and future sewer distribution system in an effort to identify deficiencies that might occur under selected conditions. The following table identifies the model simulations that were conducted for this project.

Table ES-1
Model Simulations

Simulation	Existing	2020	2030	Duration
Average Dry Weather Flow	X	X	X	Steady State
Peak Dry Weather Flow	X	X	X	Steady State
Average Wet Weather Flow	X	X	X	Steady State
Peak Wet Weather Flow	X	X	X	Steady State

For the existing system, based on these runs, there were approximately 100 manholes identified as either nearly full or overflowing during peak dry and wet weather conditions. However, many of these manholes are currently shown within pipelines having a flat or negative slope, which requires field verification.

The future modeling scenarios showed many of the same deficiencies as the existing models. This indicates that these deficiencies should be resolved during the first phases of improvements.

The existing WWTP has a 5.9 MGD ADF capacity and is currently running at approximately 65% capacity. The WWTP does have an equalization basin which allows it to handle peak wet weather flows up to 16 MGD. Based on projected flows and populations, it is anticipated the existing WWTP will meet capacity between 2020 and 2025. Typically, when WWTP flows average over 4.72 MGD (80%), the City should begin planning for expansion of the treatment plant to increase capacity. The current WWTP was designed with expansion anticipated; therefore, the upgrades shouldn't be too complicated.

Capital Improvement Program

A Capital Improvement Program (CIP) will address needed pipeline replacements and operational improvements. These projects will improve flows within the system, provide adequate capacity for future flows, reduce the chance of backup and sewer spills, and improve operation and maintenance.

There are several general recommendations as follows:

CCTV PROGRAM

The City has a CCTV inspection program outlined in the Operation and Maintenance Program of the Sanitary Sewer Master Plan (SSMP). Close attention should be paid to the known problem areas, as described in **Appendix A**, of this document, along with the combined system portion of the City. In conjunction with televising, accurate and consistent assessment is critical for prioritizing improvements. Data obtained from these inspections provides information to evaluate conditions and rank the significance of defects, allowing the City to prioritize pipeline cleaning, repair, and replacement, along with addressing critical issues such as structural integrity, effective I&I reduction, flow capacity, and effectiveness of the Fog, Oils, and Grease (FOG) Control Program.

CLEANING PROGRAM

The City's SSMP identifies a regular cleaning program. The City should continue with a regularly-scheduled cleaning program of all pipelines, with special attention given to those located within the combined system and the commercial part of town with FOG problems. The cleaning program should include manholes, in addition to pipelines, as surcharging results in solids settlement on the bench and walls of manholes.

FATS, OILS, AND GREASE (FOG) CONTROL PROGRAM

The City's SSMP identifies a FOG Control Program. Many of the hotspots and backup locations shown in the SSMP are in the commercial part of town near restaurants. The City has the authority to prohibit discharges to the system by requiring grease traps and/or other devices. If blockages continue, the City should begin to enforce the measures identified in the FOG Control Program.

CONFIRMING EXISTING FLAT OR NEGATIVE SLOPED SEWERS

Psomas has provided **Figure 7-1** to identify all gravity sewers within the City currently shown at a flat or negative slope. This is often due to lack of available information and many times may be incorrect. Psomas recommends that the City create a program to check each of these pipelines over time and update the GIS files accordingly with the correct inverts and slopes.

UPDATING THE CITY'S GIS

Psomas has provided the City with an updated GIS base file. Although the base files are much improved, there are still many areas where assumptions were made for manhole depths, pipe sizes, slopes, and materials. The City should require all staff to keep notes concerning the field conditions of the sewer and storm system on a regular basis and assign one person to be responsible for gathering the notes and updating the GIS base files on a weekly basis. As the sewer system is confirmed, the GIS files should have attribute fields updated so the City can keep track of which areas are field-verified and which are not.

ONGOING MANHOLE REHABILITATION AND REPLACEMENTS

Manholes should be rehabilitated and/or replaced based on the recommendations in Section 2-5. The City should take a systematic approach to this. Any time work is done in the street, whether it is the repair of pavement, sewer, storm, water facilities, etc., there should be a requirement to repair or rehabilitate any necessary manholes within the limits of work. As manholes are upgraded, the City's GIS will need to be updated to include this information.

ONGOING SEPARATION OF COMBINED SYSTEM

The combined system within the City should slowly be separated over a period of years and/or decades with an approach similar to the one described above for manholes. Any time work is performed within the City's right of way, there should be a requirement to separate the storm drain from the sewer whenever practical. There are several storm drain trunk pipelines running north to the New River which can be utilized as discharge points along with the Bryant Drain south of Malan Street. The new storm drain pipelines being installed in N. Rio Vista Drive and River Drive, as described in CIP Projects #2 and #3, also described below, provide a discharge point. The City will have to determine on a case-by-case basis when it is cost effective to separate the system rather than preserving the status quo.

Specific CIP projects and costs are as follows:

Table ES-2
Summary of Wastewater/Stormwater Improvement Costs

ID	Description	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
PHASE 1 CIP					
CIP 1	Upsize Ex. 18, 24, 30" SS Western Trunk, WWTP to Shank Rd., Esmt, Imperial.	24, 30, 36	11,350	$4,985,000	$6,231,250
CIP 2	SS Replacement/SD Installation Imperial Ave., Alamo St., Olive Way, Cemetery, Western Ave., River Dr.	10, 21, 24(sd)	7,630 SS 1,500 SD	$2,510,500	$3,138,125
CIP 3	SS Replacement/SD Installation N. Rio Vista Dr., Sycamore Dr.	12, 18, 21 (SS) 30 (SD)	3,800 SS 3,400 SD	$2,837,700	$3,547,125
CIP 4	Locate & redirect storm connection from ex. SS to ex. SD MH on Cattle Call at El Cerrito	Unknown	Unknown	Unknown	Unknown
CIP 5	SS Replacement A St., 10th St.	18	1,725	$438,150	$547,688
CIP 6	SS Replacement River Dr., 5th St.	8	750	$164,250	$205,313
PHASE 1 SUBTOTALS			**30,155**	**$10,935,600**	**$13,669,501**
PHASE 2 CIP					
CIP 7	SS Replacement Alley, A St.	12	1,200	$285,600	$357,000
CIP 8	SS Replacement Upstream of Lift Station 1	24	670	$242,540	$303,175
CIP 9	SS Replacement Best Rd. from WWTP to River Dr.	30	12,660	$6,595,860	$8,244,825
CIP 10	SS Replacement Cesar Chavez between E and K	10	1,175	$653,400	$816,750
CIP 11	WWTP Expansion by 1.46 MGD	--	--	$6,000,000	$7,500,000
PHASE 2 SUBTOTALS			**15,705**	**$13,777,400**	**$17,221,750**
PHASE 3 CIP					
CIP 12	SS Replacement Best Rd. from Shank to Malan. Malan St. from Best to Avenida de Colimbo	30	6,575	$3,425,575	$4,281,969
CIP 13	SS Replacement West side of Hwy. 86, Cattle Call Dr., El Cerrito Dr.	12	2,350	$559,300	$699,125
PHASE 3 SUBTOTALS			**8,925**	**$3,984,875**	**$4,981,094**
TOTAL CIP IMPROVEMENTS			**54,785**	**$28,697,875**	**$35,872,345 0**

1.0 Introduction

1.1 Background and Purpose

The City of Brawley is located centrally in the broad desert expanse of Imperial County, California. It is approximately 13 miles north of El Centro and 15 miles southeast of the Salton Sea. See **Figure 1-1** for the Vicinity Map. The City was founded in 1902 and became incorporated in 1908. In the early 1900s the population was small, consisting mainly of railroad workers. Since then, the population has grown to over 25,000 persons and now includes year-round agriculture as well as the cattle and feed industries as its biggest economic trades.

Since the City's last Wastewater Master Plan was completed in 1999, changes have occurred in the collection system. After a formal consultant selection process, Psomas was selected to prepare this Waterwater/Stormwater Master Plan to help plan for the City's wastewater/stormwater system improvements and operations by evaluating system deficiencies and capacity needs for existing and future flow conditions.

This report presents the planning methodology, design criteria, and assumptions used to develop the Master Plan, as well as the results of the hydraulic modeling and the recommended Capital Improvement Program (CIP) for the City's collection system.

The Master Plan also includes recommendations for maintenance, repair and rehabilitation of the existing collection system. These recommendations were developed from historical knowledge of the City's system, discussions with City staff, closed-circuit television (CCTV) inspection, flow measurements and condition assessments conducted by Psomas staff on the existing collection system.

1.2 Scope of Work

The Wastewater/Stormwater Master Plan scope of work includes the following tasks:

1. System Design Criteria
    a. Develop maximum capacity criteria for peak wet-weather and dry-weather flow conditions using a ratio of flow depth over pipe diameter (d/D).
    b. Develop criteria for pipeline minimum velocities and slopes.
    c. Based on flow rates from the WTP and incoming flows at the WWTP during summer and winter months, estimate inflow and infiltration rates in areas of combined sewer.

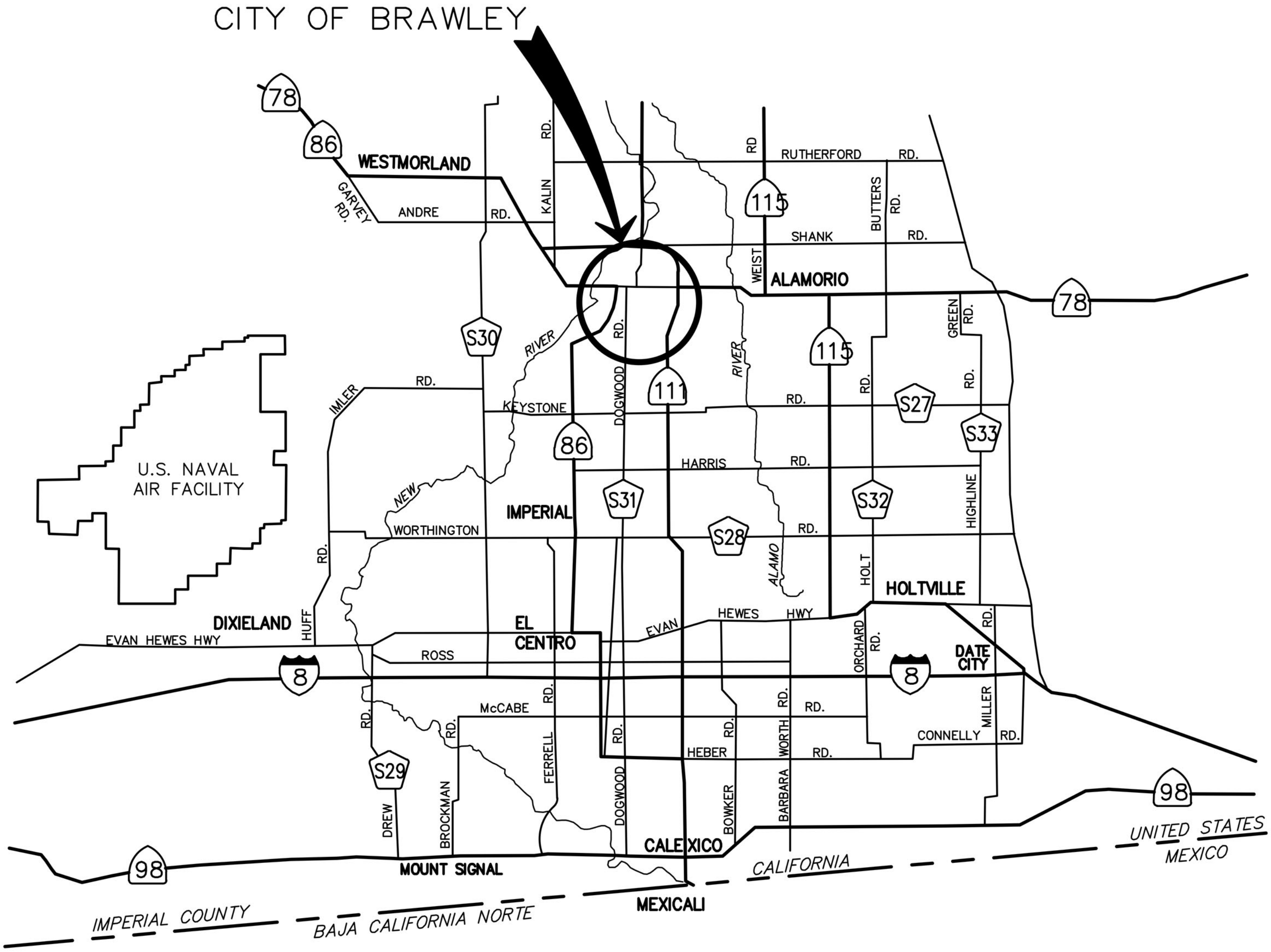

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Figure 1-1
Vicinity Map
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2. Flow Projections
    a. Review the historical water consumption data to calculate the peak dry-weather and wet-weather flows. Develop average per capita flow. The flow rates will be calibrated to reflect flows observed at the wastewater treatment plant (WWTP) and key locations via temporary flow monitoring data collected. Consideration will also be given to possible lower future rates due to the City's continued conservation efforts.

    b. Generate future flow projections based on anticipated land use development and a peaking equation developed from the flow monitoring study.

3. Computer Modeling
    a. Develop base system model including pipes and pump stations.
    b. Generate average pipe slopes based on the manhole invert elevations surveyed, available record information, and field investigations; when no data was available, assumptions were made.
    c. Provide an existing system model based on current conditions.
    d. Calibrate the existing system hydraulic model with the results of temporary flow monitoring data gathered from key locations within the collection system and from Wastewater Treatment Plant (WWTP) flows.
    e. Model the existing system to determine the required upgrades to meet established design criteria.
    f. Model the system using Year 2020 growth projections and determine the required improvements for the future system to meet established design criteria.
    g. Model the system using Year 2030 growth projections and determine the required improvements for the future system to meet established design criteria.
    h. Provide preliminary hydrologic analysis for the existing separate storm drain system and future separate system.

4. Wastewater Treatment Plant Capacity
    a. Evaluate the adequacy of the existing WWTP and projected WWTP capacities to serve the existing and future needs of the City.

5. Capital Improvement Project Recommendations
    a. Develop a schedule of required upgrades to the WWTP to meet Regional Water Quality Control Board (RWQCB) and State Department of Public Health requirements for capacity, reliability, and water quality.
    b. Identify all storm drain connections to the sewer system that are to be disconnected, abandoned, re-directed, and/or protected in place.
    c. Evaluate the results of the sewer model analyses and identify system deficiencies.

d. Develop a Capital Improvement Program for short-term and long-term capital improvements required to meet established design criteria.
e. CIP recommendations to include:

    i. Priority projects
    ii. Phased separation of combined sewer system
    iii. Sewage pump station upgrades, repairs, and replacements
    iv. Recommended future collection system upgrades
    v. Wastewater treatment plant capacities
    vi. Operational improvements
    vii. Cost opinions
    viii. Possible funding sources

1.3 Acknowledgements

Project staff would like to acknowledge the following City of Brawley staff members who provided valuable information and assistance contributing greatly to the successful completion of this project:

- Yazmin Arellano, Public Works Director
- Gordon Gaste, City Planner
- Steven Sullivan, Project Manager
- Guillermo Sillas, Associate Civil Engineer
- Alan Chan, Engineering Technician
- Ruben Mireles, Superintendent of Operations
- Fernando Soto, Water Treatment Facility Supervisor
- David Arvizu, Distribution/Pretreatment Supervisor
- Tony Verdugo, Streets and Utilities Supervisor
- Andrew Escobar, Chief Wastewater Treatment Plant Operator

1.4 Wastewater/Stormwater Master Plan Objectives

This Master Plan has been prepared to provide a reference document for the existing wastewater/stormwater system operations and maintenance and a framework for future system planning. The plan objectives include the following:

1. Develop a comprehensive computer model calibrated to the existing system conditions.
2. Develop performance criteria for both existing and proposed facilities.
3. Using the computer model to conduct analyses of the existing system and identifying current deficiencies in existing system facilities.
4. Identify and evaluate system improvements that will alleviate existing system deficiencies.

5. Incorporate projected flows into the model and identify future system improvements that will be needed to meet the future demands.
6. Perform analyses of the wastewater/stormwater system using the computer model to evaluate operations of the current and future systems.
7. Review and summarize water quality and proposed regulations that may have an impact on stormwater and WWTP discharge to the New River.
8. Develop a capital improvement program and capital cost estimates for wastewater/stormwater system improvements and expansion.
9. Develop a phased project list to prioritize future wastewater system improvement projects, including separation of the stormwater system.
10. Review alternative financing programs for possible funding sources to help fund the recommended improvements

1.5 Study Area

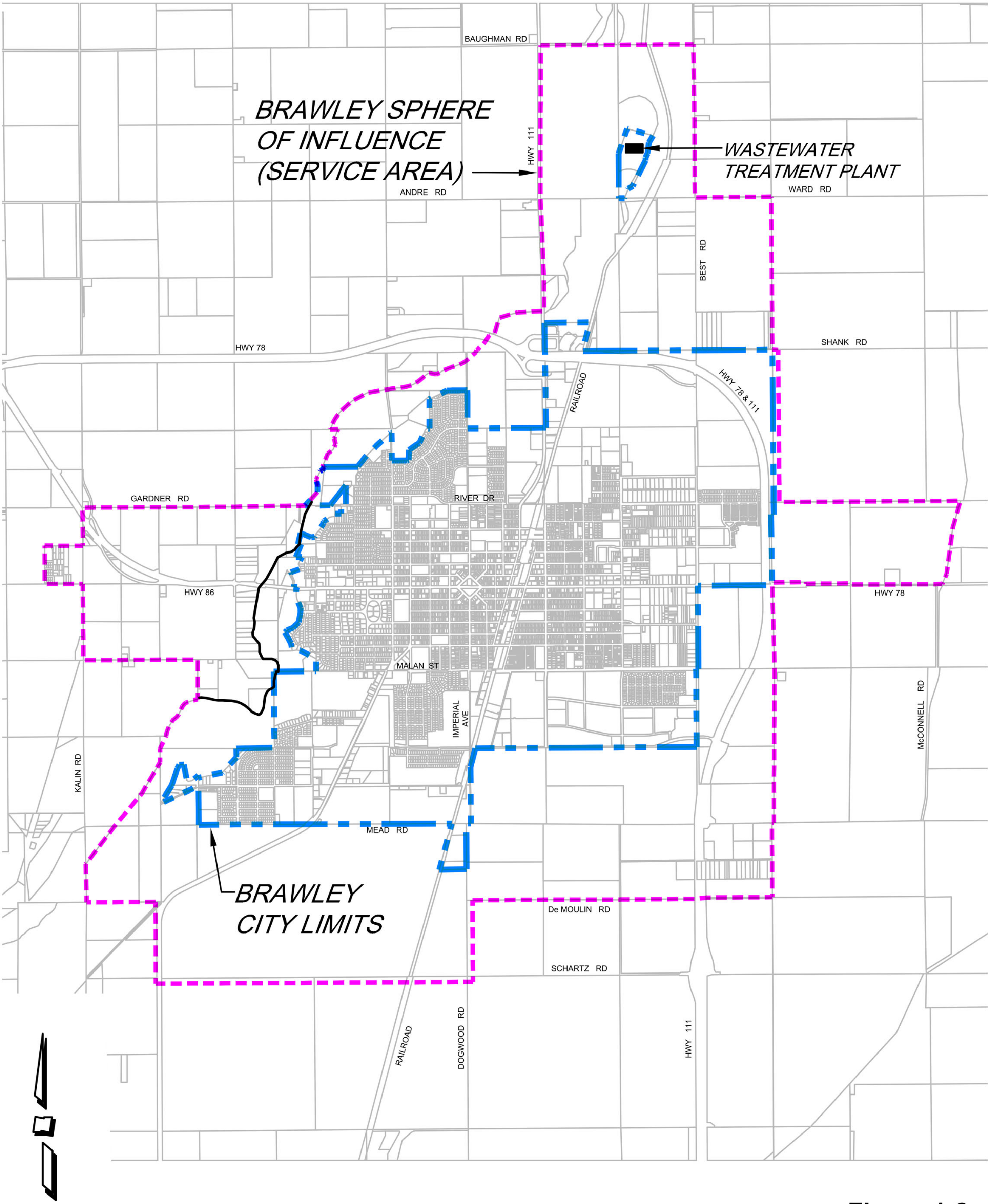
The City of Brawley sewer/storm water system service sphere of influence covers approximately 17 square miles, as shown in **Figure 1-2**. This area includes both the incorporated City of Brawley and unincorporated areas outside the City limits.

1.6 Abbreviations

The following is a list of abbreviations used in this report:

ACP	Asbestos cement pipe
ADWF	Average dry-weather flow
AWWF	Average wet-weather flow
BMP	Best Management Practices
ccf	one hundred cubic feet
CCTV	Closed-circuit television
CDPH	California Department of Public Health
cfs	Cubic feet per second
CIP	Capital Improvement Program
DI	Ductile iron
Dia	Diameter
EDU	Equivalent Dwelling Unit
fps	feet per second
ft	foot or feet
ft-MSL	feet above mean sea level

gpcd	gallons per capita-day
gpd	gallons per day
gpd/ac	gallons per day per acre (volume of water used per acre of land)
gpm	gallons per minute
Hp	Horsepower
IID	Imperial Irrigation District
in	inch or inches
LDR	Low Density Residential
MDR	Medium Density Residential
MG	million gallons
mgd	million gallons per day
MP	Master Plan
PDWF	Peak dry-weather flow
PWWF	Peak wet-weather flow
RWQCB	Regional Water Quality Control Board
SSMP	Sanitary Sewer Management Plan
SW	Stormwater
UFC	Uniform Fire Code
UPC	Uniform Plumbing Code
USGS	United States Geologic Survey
WTP	Water Treatment Plant
WW	Wastewater
WWTP	Wastewater Treatment Plant



SCALE: 1"=4500'

Figure 1-2
City Limits & Sphere of Influence
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1.7 Unit Conversions

This section provides a list of conversion factors that are commonly used to convert values from one unit to another.

- Convert cfs to gpd: Multiply by 646,300
- Convert cfs to gpm: Multiply by 448.8
- Convert cfs to mgd: Multiply by 0.646
- Convert gpd to cfs: Multiply by 0.000001547
- Convert gpd to gpm: Multiply by 0.0006944
- Convert gpd to mgd: Multiply by 0.000001 (or divide by one million)
- Convert gpm to cfs: Multiply by 0.002228
- Convert gpm to gpd: Multiply by 1,440
- Convert gpm to mgd: Multiply by 0.00144
- Convert mgd to cfs: Multiply by 1.547
- Convert mgd to gpd: Multiply by 1,000,000
- Convert mgd to gpm: Multiply by 694.4

2.0 Existing Wastewater Collection System Facilities

2.1 General

The existing wastewater collection system includes sewer main lift stations and forcemains, with a portion of the City served by a combined sewer/storm system. **Figures 2-1** and **2-2** (located in map pockets) show the existing collection system and existing combined system, respectively. The pipelines consist primarily of vitrified clay pipe (VCP) and polyvinyl chloride (PVC) pipe, with a small portion of ductile iron pipe (DIP), reinforced concrete pipe (RCP), and a certain amount that is unknown.

The City owns and maintains approximately 75 miles of gravity pipeline, about 2 miles of sewer forcemain, approximately 1,438 manholes, and three (3) sewage lift stations within its service area. Wastewater and stormwater generated within the system flows by gravity to the City's wastewater treatment plant located at 1550 Best Road.

2.2 Wastewater Treatment Pant

2.2.1 Description and Capacity

The City of Brawley owns and operates the WWTP, which is believed to have been originally constructed in the late 1930s and was upgraded in 1980 with the aerated lagoon process. Upgrades made in 1986 included anaerobic sludge digesters which were ultimately taken out of service.

The WWTP is subject to requirements of the California Regional Water Quality Control Board (RWQCB), Colorado River Basin Region, and the National Pollutant Discharge Elimination System (NPDES). By 2007, the WWTP was in violation with the NPDES permit due to more stringent ammonia nitrogen removal requirements. A preliminary design report for plant upgrades was performed in August 2008 and construction was completed in 2012. The current WWTP meets NPDES ammonia nitrogen limits.

The WWTP treats and disposes of an average daily flow of 3.83 MGD of wastewater utilizing the Biolac® activated sludge process. See **Figure 2-4**. This proprietary system provides extended sludge age by mixing the activated sludge with suspended solids in an aerated basin. Bubble diffusers are suspended from aeration chains. Blowers control the air distribution, thereby creating oxic and anoxic zones, thus providing nitrification and nitrogen removal. The effluent from the lagoons is UV-disinfected and discharged to the New River, a tributary to the Salton Sea.

The WWTP has a design capacity of 5.9 MGD, with the ability to easily expand to accommodate future growth. With a current average daily flow of 3.84 MGD, the plant is operating at approximately 65% capacity. The plant has an influent equalization basin capable of storing peak flows in excess of 13 mgd.

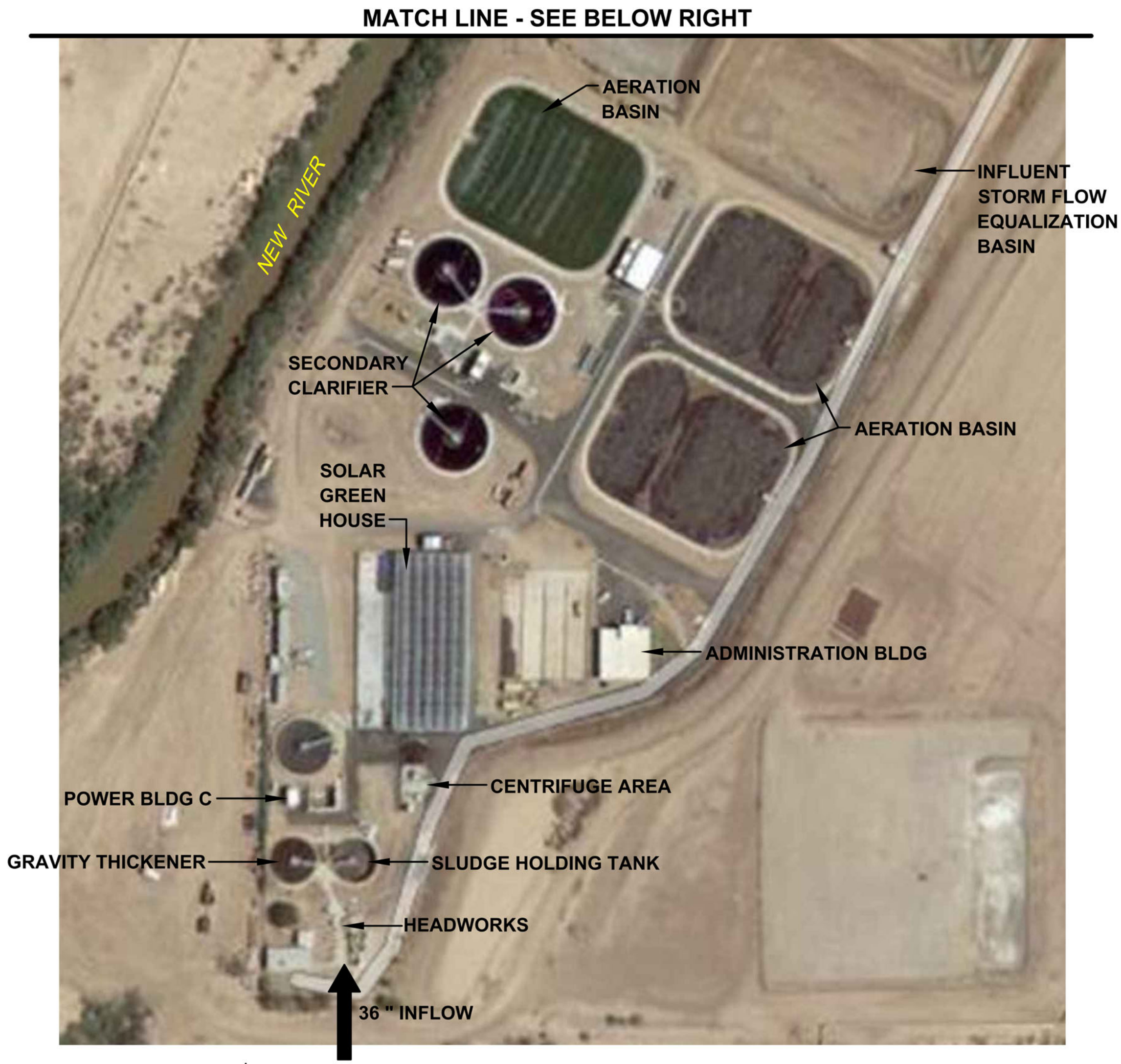

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Figure 2-4
Wastewater Treatment Plant
City of Brawley Integrated Master Plan
April 2013

2.3 Gravity Collection System

As shown in **Figure 2-1A** (located in a map pocket),which also shows the City's sewer drainage basins, the oldest portions of the collection system consist of VCP pipe constructed in the early to mid-20th century and are located throughout the system, but they are predominantly within the combined area of the sewer service area. The newest portions of the sewer system are located south of Malan Street and north of River Drive and consist of mostly PVC. **Table 2-1** provides a summary of the gravity pipes by diameter and material.

Table 2-1

Summary of Gravity Pipe Lengths

Diameter (in)	Length of Pipe (ft) by Pipe Material					Total Length (ft)
	VCP	PVC	DIP	UNK	RCP	
Gravity Mains						
4		280				280
6	3,320	2,649		3,168		9,137
8	93,443	84,809	3,923	67,440		249,615
10	20,816	2,641	636	2,324		26,417
12	13,511	3,049		11,366		27,926
14				528		528
15	4,062	10,809		9,352		24,223
18	7,883	5,348		19,047		32,278
21				751		751
24	3,947			30,731	1,477	36,154
30				200		200
36	240					240
UNKNOWN				425		425
Total Gravity Mains (ft)	**147,222**	**109,585**	**4,559**	**145,332**	**1,477**	**408,174**

It is assumed the majority of pipe material labeled "UNK" is VCP or PVC.

2.3.1 Existing Problem Areas

Over the years, the City has implemented a program to inspect and log deficiencies in the existing sewer system using CCTV. Based on the results of this CCTV inspection program to date, City staff has determined that the older portions of the collection system are in fair to poor condition. Deficiencies have generally been typical of systems of similar age, including root intrusion, cracks, defective lateral connections, and grease build-up. The CCTV data was used to identify some connection locations of the storm drain system into the wastewater collection system. Some of the sewers within the combined system contain pieces of concrete, rocks, bricks, leaves, sticks, and other debris entering the system from adjacent catch basins. Photos 2-4 through 2-6 show some of the results of recent CCTV footage, which represent typical conditions, inside the sewer system.

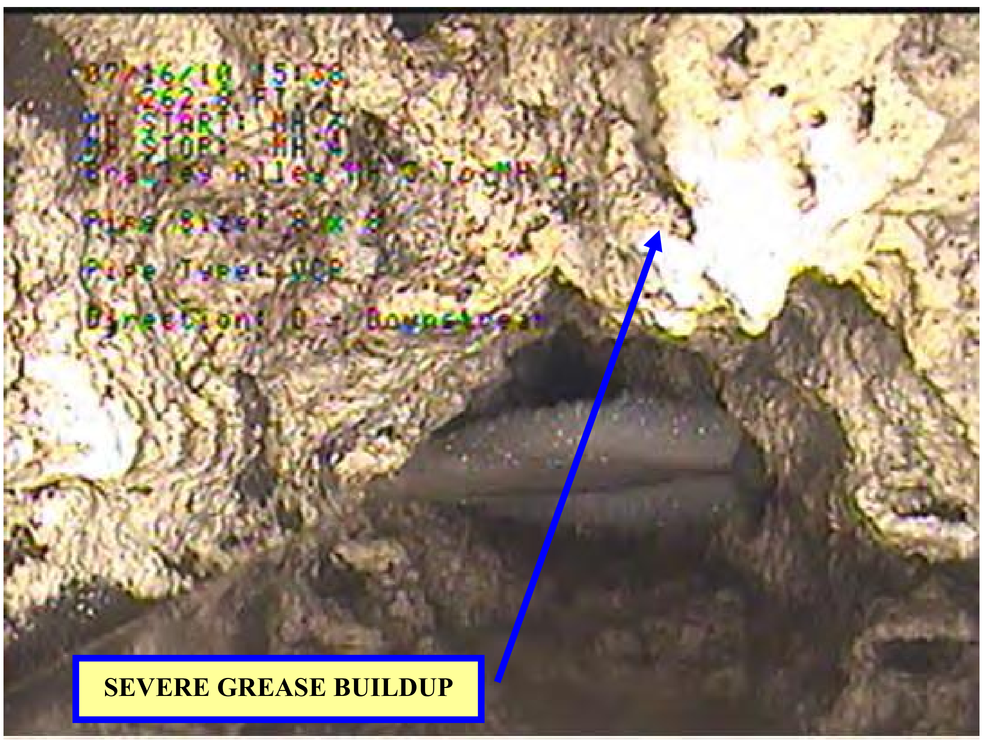


Photo 2-4: Severe grease buildup in sewer main

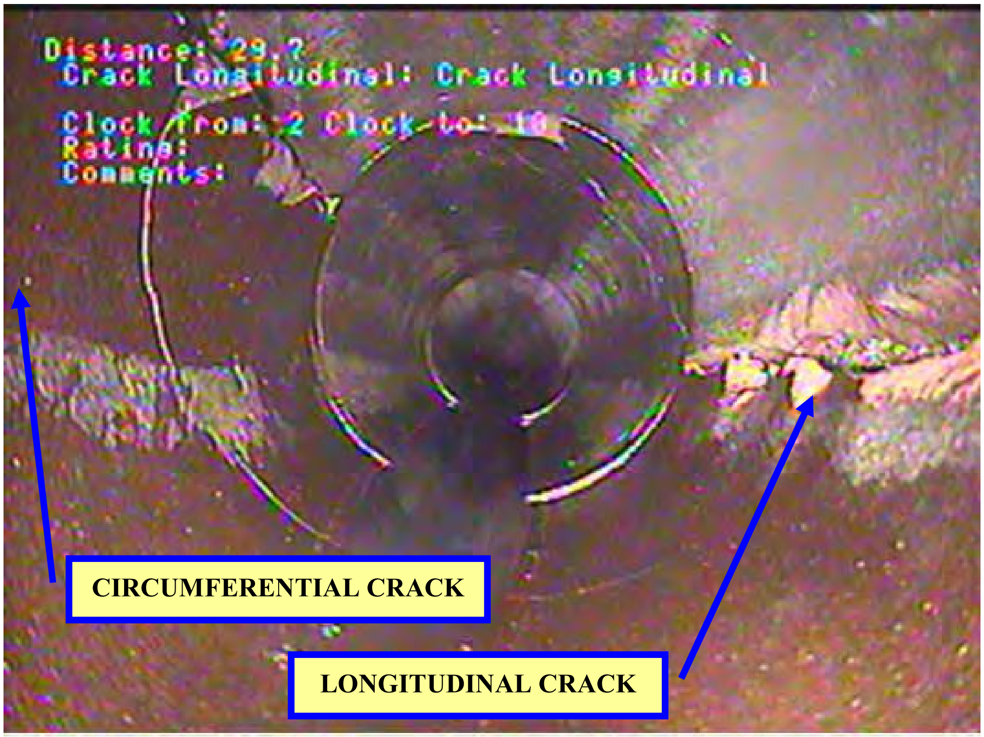


Photo 2-5: Longitudinal and circumferential cracks in sewer main

Photo 2-6: Root buildup in sewer lateral

City maintenance crews schedule cleaning of these problem locations as frequently as monthly to avoid blockages. Due to the flat slopes common to sewers throughout the City and combined systems, these sewers are vulnerable to grease buildup, solids accumulation, and debris entering the sewer from storm inlets. A routine cleaning program will help reduce these problems. In 2008, as part of the City's SSMP, an operation and maintenance program was established for routine preventive maintenance and scheduled cleaning, with more frequent focus on known problem areas. An overflow emergency response plan was also developed within the SSMP that outlines procedures for an appropriate response to all overflows.

The current problem areas with recent blockages are shown in **Table 2-2**.

Table 2-2
Past Sewer Back-Up Locations

Location	Sewer Pipe Size	Details
N. Rio Vista Avenue	15"	Backups within the past few years caused flooding in adjacent houses. Combined system.
Richard Avenue	8"	Recent backup in February 2012 surcharged all upstream sewers. Emergency cleaning required.
E. Main Street, 1500-1600 blk	8"	Commercial Area

The SSMP identifies other locations with sewer back-ups.

2.4 Forcemains

The City currently maintains forcemains from three lift stations. The forcemain material is PVC for Lift Stations #2 and #3, but unknown for Lift Station #1. A condition assessment has not been performed and no historical information exists to determine the condition of the existing forcemains. **Table 2-3** identifies the size and type of forcemains.

Table 2-3
Summary of Forcemain Pipe Lengths

Diameter (in)	Length of Pipe (ft) by Pipe Material					Total Length (ft)
	VCP	PVC	DIP	UNK	RCP	
Forcemains						
6		**3,065**				**3,065**
10				**7,990**		**7,990**
Total Forcemains (ft)		**3,065**		**7,990**		**11,055**

2.5 Manholes

2.5.1 Manhole Inspections and Recommendations

The City of Brawley currently owns and maintains approximately 1,440 manholes. As part of this Master Plan, Psomas performed a condition assessment of one hundred (100) manholes to assess their condition and connectivity within the system. The assessment consisted of performing a visual inspection of the interior of the manhole while noting such items as solids buildup, spalled concrete, root intrusion, wall cracks, offset joints, corroded frames and covers, manhole material, and depth, location, and size of sewer connections, along with other conditions. In addition, Psomas also opened and observed approximately 200 manholes without providing an inspection report to help clarify flow directions, connection points, and other horizontal information. Generally speaking, the majority of the manholes within the combined system are in poor condition, with the majority needing some level of repair ranging from a lining system to complete replacement.

An inspection report was completed on each manhole chosen for inspection as part of this Master Plan. In addition, Psomas developed a new manhole labeling system with the first letter indicating the main sewer drain basin, second letter represents the subbasin. The next 3 numbers represent the manhole # along the trunk sewer and the final 2 numbers represent the manhole # along the branch sewer. This information is shown on a legend included with the Atlas maps. **Table 2-4** shows the results of these reports along with recommendations for improvements, if warranted.

Table 2-4
Existing Manhole Inspection Results

MH ID #	MH Inspection #	Atlas Map & Grid #	Location Description	Overall Condition	Recommended Improvements
EE043.20	1	D14	Ave. Del Valle/Calle Del Sol	Good	None
EE043.22	2	D14	La Valencia Dr./Calle Del Sol	Fair	
EE036.03	3	D14	La Valencia Dr./Calle Del Sol	Good	
EE043.14	4	E15	Kelly Ave./Calle Estrella	Fair	
EE041.00	5	E14	Calle Del Sol/Richard Ave.	Fair	
EE043.00	6	E14	Kelly Ave./Calle Del Sol	Fair	
EE047.12	7	D14	Panno Rd./Ronald St.	Fair	
EE044.00	8	E14	Ronald St./Richard Ave.	Poor	Replace or Total Rehab
EE048.04	9	E14	David St./Ronald St.	Fair	Minor Rehab Needed
WE013.08	10	E12	West K St./Alley	Poor	Replace MH
WE014.16	11	F12	Cattle Call Dr./South Rio Vista Ave.	Poor	Replace MH
WE007.11	12	F12	South El Cerrito Dr./Alley St.	Bad	Replace or Rehab
WE007.16	13	F12	South El Cerrito Dr.	Bad	Replace or Rehab
WE013.15	14	E12	Cattle Call Dr./Alley	Fair	
WE007.02	15	F12	South El Cerrito Dr./Alley	Bad	Replace or Rehab
WE007.00	16	F11	South Rio Vista Ave./West J St.	Poor	Full Rehab Needed
WE012.00	17	E11	West H St./Sycamore Dr.	Poor	Replace or Rehab
CE005.04	18	F11	South 1St St./Alley	Fair	Replace, Reset Frame & Cover
CE002.08	19	G11	South 2nd St./Alley	Poor	Replace
CD010.35	20	G10	North 5th St./Alley	Fair	Replace & Reset Frame Lid
CD010.32	21	G10	North 5th St./Alley	Fair	Replace Stairs
WC055.00	22	F9	Jones St.	Good	
WC054.00	23	F9	Jones St./Flammang Ave.	Good	
WC045.00	24	G8	Jones St.	Fair	

MH ID #	MH Inspection #	Atlas Map & Grid #	Location Description	Overall Condition	Recommended Improvements
WC044.00	25	G8	Jones St.	Good	
CC018.01	26	H11	G St.	Poor	Replace
CB004.00	27	I 9	North Eastern Ave./Jones St.	Poor	Rehab
WE007.31	30	E13	Julia Dr./Orita Dr.	Good	
WE007.30	31	F13	Julia Dr./State HWY 86	Fair	
ED020.00	32	G13	South Imperial Ave.	Good	
ED006.00	33	G13	South Imperial Ave.	Good	
ED007.00	34	G13	South Imperial Ave.	Good	
ED008.00	35	G13	2nd St./Julia Dr.	Good	
ED009.04	36	G12	Santillan Ave./2nd St.	Good	
CE011.00	37	G12	Malan St.	Very Poor	Replace
CE009.00	38	G12	South Imperial Ave./Alley	Very Poor	Replace
CE008.01	39	G12	K St.	Very Poor	Replace
CE007.00	40	G12	K St./Alley	Very Poor	Replace
CF013.00	41	G12	K st/Alley	Poor	Replace
CF012.01	42	G12	South 5th St.	Poor	Replace
CC023.16	43	G12	Malan St./South 5th St.	Poor	Replace
CD029.01	44	G10	North 3rd St.	Fair	
CD028.00	45	G10	A St.	Poor	Replace
CD031.00	46	G10	West C St./Alley	Very Poor	Replace
CD033.00	47	G10	North 2nd St.	Poor	Replace
CD033.01	48	G10	North 2nd St.	Poor	Replace
CD029.03	49	F10	B St./Alley	Poor	Replace
CD010.20	50	G10	E St./Alley	Poor	Replace
CD010.11	51	G10	D St./Alley	Poor	Replace
CE003.00	52	G11	West K St./Alley	Good	
CE005.00	53	G11	H St./Alley	Good	

MH ID #	MH Inspection #	Atlas Map & Grid #	Location Description	Overall Condition	Recommended Improvements
CE005.02	54	G11	South 2^{nd} St./Alley	Poor	Replace
CE002.03	55	G11	South 2^{nd} St.	Fair	
CE002.01	56	G11	South 3rd St./Alley	Poor	Replace
CE008.13	57	F12	South 1st St.	Good	
CE006.14	58	F12	West K St.	Poor	Replace
CE006.09	59	F12	South 1st St.	Poor	Replace
CE006.15	60	F12	West K St./Alley	Poor	Replace
CE006.07	61	G12	South 2^{nd} St.	Poor	Replace
CE008.10	62	G12	South 2^{nd} St./K St.	Poor	Replace
CE008.11	63	G12	South 2^{nd} St.	Poor	Replace
CD010.22	64	G10	North 3rd St./Alley	Poor	Replace
CD010.23	65	G10	North 3rd St./Alley	Fair	
WE020.00	66	E11	Terrace Cir.	Fair	
WE018.00	67	E11	Terrace Cir./Terrace Dr.	Good	
WE017.00	68	E11	Terrace Dr./Alley	Good	
WE016.00	69	E11	Terrace Dr./West H St.	Good	
WE015.00	70	E11	West H St./Los Flores Dr.	Fair	
WE010.05	71	E11	Marjorie Ave./Alley	Fair	
WE009.00	72	E11	West Main St./Alley	Poor	Replace
WE010.00	73	E11	West G St./Alley	Poor	Replace
WE017.02	74	E11	Terrace Dr./Alley	Fair	
WE014.05	75	E12	Russell Rd./Alley	Poor	Replace
WE014.10	76	E12	West K St./Alley	Poor	Replace
WE002.04	77	F11	West H St./South El Cerrito Dr.	Poor	Replace
WE002.14	78	F11	West G St./Andrita Pl.	Poor	Replace
WE002.05	79	F11	West H St./South Rio Vista Ave.	Poor	Replace
WE002.07	80	F11	South Rio Vista Ave./Alley	Poor	Replace

MH ID #	MH Inspection #	Atlas Map & Grid #	Location Description	Overall Condition	Recommended Improvements
WD021.05	81	E10	North Las Flores Dr./West D St.	Fair	
WD021.03	82	E10	West D St./Alley	Poor	Replace
WD012.03	83	E10	West A St.	Fair	
WD004.07	84	F9	North El Cerrito Dr.	Good	
WD004.01	85	F9	North El Cerrito Dr.	Good	
WC033.00	86	F9	River Dr./Western Ave.	Good	
WC033.03	87	G9	River Dr.	Fair	
CE006.01	88	G11	South 3rd St./Alley	Poor	Replace
CC022.04	89	H11	Cesar Chavez/Alley	Poor	Rehab Needed
CC013.07	90	H10	Welcome St.	Fair	Replace Frame/Cover Minor Rehab
CC001.07	91	H9	North Palm Ave.	Poor	Rehab
CB024.06	92	H11	South 11th St./Alley	Poor	Rehab
CB024.03	93	H11	South Palm Ave./Alley	Poor	Rehab
EB037.01	94	I12	16th St.	Good	
CC011.02	95	H10	B St./Alley	Poor	Rehab
CC014.00	96	H10	North 10th St./D St.	Poor	Rehab
CC016.01	97	H10	North 10th St.	Poor	Rehab

Additionally, manhole inspections were completed on existing manholes as part of the North Rio Vista Avenue Sewer Line Rehabilitation Project. **Table 2-5** shows the results of that manhole inspection program along with appropriate recommended improvements.

Table 2-5
Existing Manhole Inspection Results for North Rio Vista Avenue Sewer Line Rehabilitation Project (Completed in 2008)

MH ID #	MH Inspection #	Atlas Map & Grid #	Location Description	Overall Condition	Recommended Improvements
WE003.00	RV-1A	F11	Rio Vista/G St.	Poor	Replace or Rehab
WE002.00	RV-1B	F11	Rio Vista/Alley S. of Main	Poor	Replace or Rehab
WE001.00	RV-1C	F11	Rio Vista/Alley S. of Main	Good	Clean
WD026.00	E-1	F10	Rio Vista /E St.	Good	Clean
WD025.00	E-1A	F10	Rio Vista/E St.	Poor	Replace or Rehab
WD024.00	RV-4	F10	Rio Vista/Alley S. of D St.	Bad	Replace or Rehab
WD023.00	RV-4A	F10	Rio Vista/D St.	Fair	Rehab
WD022.00	RV-5	F10	Rio Vista/Alley S. of C St.	Fair	
WD021.00	RV-6	F10	Rio Vista/Alley S. of C St.	Fair	Replace Frame
WD019.00	RV-7	F10	Rio Vista/Alley N. of C St.	Fair	Replace Frame
WD018.00	RV-8	F10	Rio Vista/Driftwood	Fair	Replace Frame
WD017.00	B-3	F10	Rio Vista/Alley S. of B St.	Fair	Replace Frame
WD015.00	RV-8A	F10	Rio Vista/Alley N. of B St.	Fair	Replace Frame
WD014.00	RV-9	F10	Rio Vista/Alley S. of A St.	Fair	Replace Frame
WD012.00	RV-10B	F10	Rio Vista/Alley N. of A St.	Fair	Replace Frame
WD011.00	RV-11	F10	Rio Vista/Alley S. Magnolia	Fair	Replace Frame
WD010.00	RV-11A	F10	Rio Vista/Magnolia	Fair	Replace Frame
WD009.00	RV-12	F9	Rio Vista/Alley S. of Adler	Poor	Replace or Rehab
WD008.00	RV-13	F9	Rio Vista/Alley S. of River	Fair	Replace Frame
WD007.00	RIV-1	F9	Rio Vista/River (SW)	Poor	Replace or Rehab
WD006.00	RIV-1C	F9	Rio Vista/River (NE)	Poor	Replace or Rehab
WD005.00	RIV-1A	F9	River/W of El Cerrito	Poor	Replace or Rehab
WD004.00	RIV-1B	F9	River/El Cerrito (N)	Fair	

MH ID #	MH Inspection #	Atlas Map & Grid #	Location Description	Overall Condition	Recommended Improvements
WD029.00	RIV-2	F9	River/El Cerrito (S)	Fair	
WD031.00	RIV-2A	F9	River/W. of Western (S)	Fair	
WD030.00	RIV-2B	F9	River/ E. of El Cerrito (S)	Fair	
WD002.00	RIV-3	F9	River/Western	Fair	

2.6 Sewer Lift Stations

The City currently owns and operates three lift stations as shown on **Figure 2-1**. **Table 2-6** gives a brief description of each lift station.

Table 2-6

Sewer Lift Station Capacity & Operation

Facility Name	Location	Pump Number	Pump Type	Backup Power source	Wetwell Volume (gal)	Nameplate Horsepower (hp)	Capacity (gpm)
South Brawley Sewage Lift Station No. 1	Malan Street and Dogwood Road	1	Self Priming, Constant Speed	None	2,500 (Est)	50	800
		2				50	800
Citrus View Sewage Lift Station No. 2	Richard Avenue between Ronald Street and Steven Street	1	Self Priming, Constant Speed	None	380	5	200
		2				5	200
Latigo Ranch Lift Station No. 3	Legion Road at Union Pacific Railroad	1	Submersible, Constant Speed	None	1,820	7.5	320
		2				7.5	320

The Luckey Ranch Pump Station has been installed, but is not in operation. It will become City Pump Station #4. The La Paloma Pump Station is not yet built but will become City Pump Station #5 upon completion.

As of November 2012, Lift Station #2 modifications are out to bid and will be replaced with a packaged, submersible pump station. The station will contain two (2) 5-Hp pumps with single phase power and will be provided with a gas operated, backup generator. As part of this project, a diesel powered generator will be added to the Lift Station #1 site.

2.7 Combined System Facilities

A portion of the City is served by a combined system. This approximate area is surrounded by River Drive to the north, N. Eastern Avenue to the east, Malan Street to the south, and the City limits line to the west. See **Figure 2-2 (**in a map pocket). Stormwater is collected in small inlets (12"x12" up to 18"x24") and discharged to the nearest sewer manhole or, in some cases, via direct connection to the sewer pipeline. **Photo 2-1** shows a typical catch basin as part of the combined system.

Photo 2-1: Existing catch basin as part of combined system

8" VCP is the typical pipe size connecting the stormwater inlet to the sanitary sewer. Many of the manhole connections contain a 90-degree bend directed upward. It is believed this was originally intended to serve as a trap to help prevent odors from escaping the manhole through the catch basin. On occasion, these bends are clogged, thus preventing any stormwater from entering the sewer system. **Photos 2-2** and **2-3** show typical storm connections as part of the combined system.

Photo 2-2: Existing storm connection to sewer manhole

Photo 2-3: Clogged storm connection to sewer manhole

2.8 Non-Combined Storm Drain Facilities

2.8.1 General

As described above, the older portion of the City was designed with a combined sewer system. During rain events, runoff enters the curb inlets, which discharge to the sanitary sewer system where it is mixed with raw sewage and, ultimately, receives treatment at the WWTP. A portion of the City contains a separate storm and sanitary sewer systems, as shown on **Figure 2-3**. These are typically located in areas developed over the last 25 to 30 years. Many of the newer subdivisions south of Malan Street contain on-site detention basins with ultimate discharge points to IID drains.

2.8.2 Topography

The City generally drains from the southwest to the northeast with elevations ranging from -95 to -135 feet below mean sea level. A portion of the City, along the western boundary, ultimately drains to the New River, with a major discharge point located just west of Cattle Call. Another small drainage area and discharge point is located at the intersection of N. Rio Vista Drive and River Drive, where several catch basins collect runoff and discharge to a roadside ditch which ultimately discharges to the New River. The southern portion of the City, in and around Malan Street, discharges to the Bryant Canal, while the eastern portion (east of N. Eastern Ave.) discharges to the drain parallel to Best Road. The northern portion of the City discharges to various roadside ditches that ultimately discharge to the New River.

With the flat topography and minimal inlets, runoff often collects in low points, causing ponding within street rights of way that ultimately evaporates.

2.8.3 Drainage Basins

Due to the uncertainty surrounding the location of the existing system and number of inlets, existing drainage basins have not been identified.

2.8.4 Collection System

The City has a separate stormwater collection system estimated at approximately 17-18 miles, as shown on **Figure 2-3**. The older portions of the system (>15 years old) collect runoff and discharge to the New River. Newer developments (<15 years old), typically have installed retention basins that have been sized for the 100-year storm. Within these retention basins are small pump stations sized to pump at a rate which empties the basin within 72 hours, per Imperial County requirements. The majority of older piping is RCP and, although not inspected as part of this project, is believed to be in good condition. Most of the newer piping is PVC.

2.8.5 Stormwater Lift Stations

The City currently owns and operates three (3) stormwater lift stations, as shown on **Figure 2-3**. **Table 2-7** gives a brief description of each lift station.

Table 2-7
Stormwater Lift Station Capacity & Operation

Facility Name	Location	Pump Number	Pump Type	Backup Power Source	Wetwell Volume (gal)	Nameplate Horsepower (Hp)	Capacity (gpm)
Latigo Ranch SD Lift Station	East end of Legion Road	1	Submersible	None	1,127	7.5	320
		2				7.5	320
Malan Park SW Lift Station	South 1st St at Wildcat Dr.	1	Submersible	None	Unk	Unk	Unk
		2				Unk	Unk
Luckey Ranch SW Lift Station		1	Submersible	None	1,127 (estimated)	Unk	300
		2				Unk	300

2.8.6 Stormwater Forcemains

The City currently maintains forcemains from three (3) lift stations, as described above. The forcemain material is PVC for all lift stations and they are less than 10 years old; therefore, the condition is assumed to be good. **Table 2-8** summarizes the various forcemain sizes and materials.

Table 2-8
Summary of Stormwater Forcemain Pipe Lengths

Diameter (in)	Length of Pipe (ft) by Pipe Material					Total Length (ft)
	VCP	PVC	DIP	UNK	RCP	
Force Mains						
6	--	**150**	--	--	--	**150**
8	--	**172**	--	--	--	**172**
Total Forcemains (ft)	--	**322**	--	--	--	**322**

2.8.7 IID Drains

Portions of the City are served by IID drains which consist of a series of open channels and pipes that convey surface runoff to the New River at several locations. See **Figure 2-3**. The City is surrounded by the Stahl drain along the eastern edge of Hwy 111, and to the south, the Bryant Drain, which runs from Walmart, south of Malan Street, and discharges to the Best Canal parallel with Best Road. The Bryant Drain has recently become the property of the City of Brawley and will be undergrounded as part of future development projects.

3.0 Planning and Design Data

3.1 Existing and Projected Population

Based on the Southern California Association of Governments (SCAG) 2011 Local Profile for the City of Brawley, the City's population was 24,953 people in 2010. (See **Table 3-1**) Based on discussions with the City Planning Department, future year 2030 population projections should be taken from the City's General Plan, which projects that the City will be built out at that time and will have a population of 60,542. As the General Plan does not have projections for the year 2020, a constant rate of increase between 2010 and 2030 was assumed, resulting in a 2020 population of 42,748. Based on this, the year 2020 population projection is 42,748. The average number of people per dwelling unit (population density) was 3.27 in 2010. The population density is projected to drop slightly to 3.24 in 2030, a 1.0 percent change.

Table 3-1
Historical and Projected Population and Housing

	2010(1)	**2020**(2)	**2030**(3)
Population (SCAG)	24,953	42,748	60,542
Total Dwelling Units	7,623	13,059	18,686
Population/DU	3.27	3.27	3.24

(1) Per 2010 Census data for the City of Brawley (for planning purposes it is assumed this includes the City's sphere of influence)

(2) Assuming population will catch up to General Plan 2030 projections at a constant rate (includes sphere of influence)

(3) Per the General Plan 2030 (includes sphere of influence)

3.2 Land Use and Growth

The City of Brawley anticipates new development and continued redevelopment within its sphere of influence. This expansion is expected to increase wastewater flows over the next few years. The City will require accurate wastewater flow projections to adequately prepare for this growth. Population can be used to determine future wastewater flows. However, population alone does not reflect total flows or where flows will occur. Wastewater flows are usually directly proportional to population growth, but they are also related to economic conditions and the type and/or mix of land uses within a community. This is where land use becomes an important parameter, particularly when certain dense land use categories represent a substantial percentage of projected growth.

Actual wastewater flows vary depending on many factors, but land use is one of the primary determining factors for estimating flows. Using land use to estimate wastewater flows is common in master planning because the information is readily available, relatively accurate, and can be used for existing areas as well as future developments.

Therefore, proposed land use was used to determine the location of future development or redevelopment and, consequently, the location of increased wastewater flows. Using projected development was considered a more accurate method of estimating where the future flows will occur within the City's current boundary and sphere of influence.

Due to the downturn in the economy at the end of 2008, some new developments were halted before construction was completed and left uncompleted or vacant. Also, some other planned developments that were previously projected to be constructed by now, have been delayed and/or are being reevaluated. As discussed in the subsequent Existing Land Use section, an analysis of all land use areas within the City's sphere of influence was conducted, so as to identify all areas that have been developed and those still vacant.

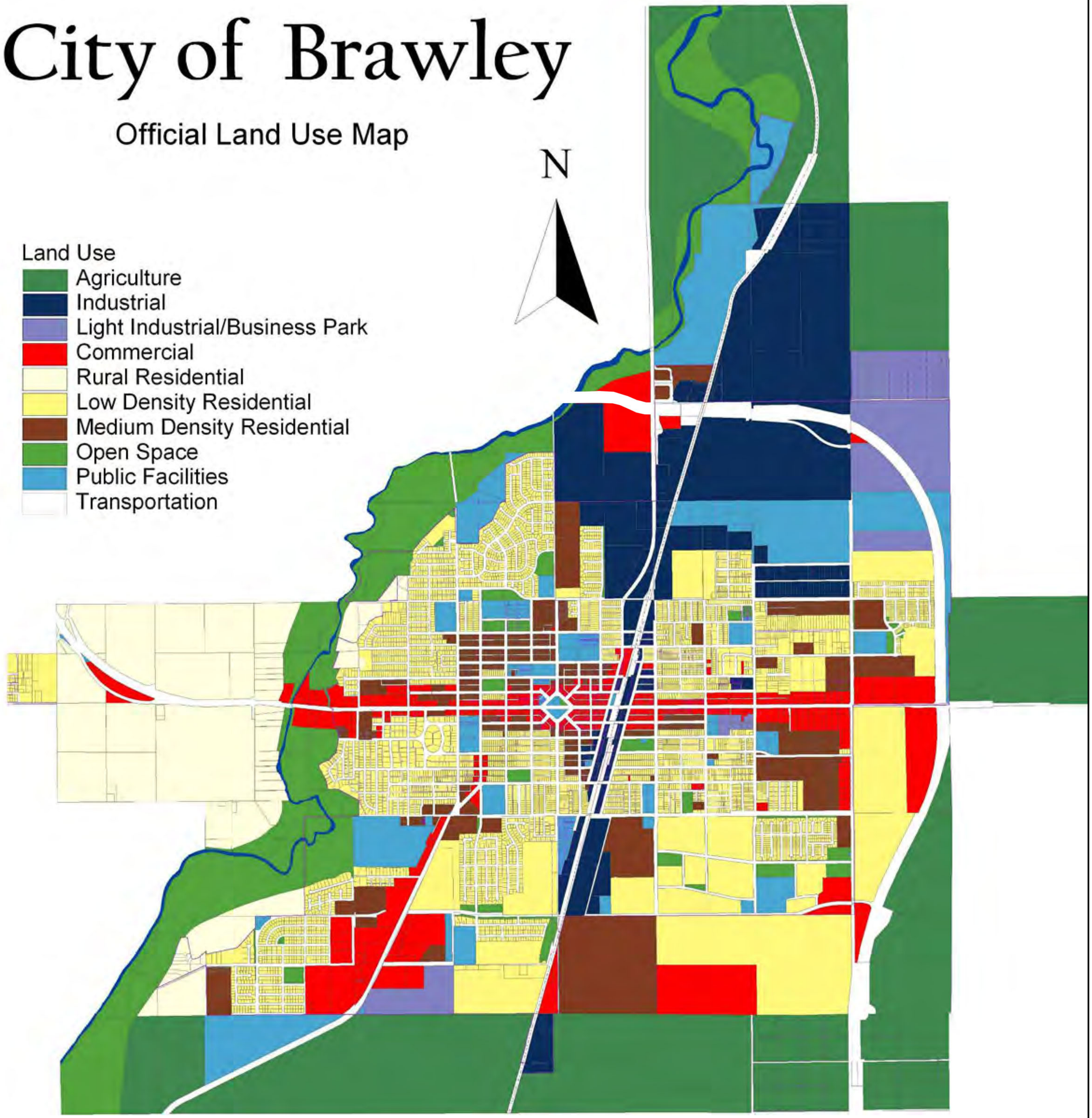
3.2.1 Land Use

Information on land use, including new development and redevelopment zones within the City's sphere of influence, was obtained from the City of Brawley Planning Department. For this Master Plan, the City's current official land use map was used to create the existing land use map in **Figure 3-1**. Based on discussions with the City, this land use map includes all the area within the City's sphere of influence. This area includes both the incorporated City of Brawley, with areas as presented in City's 2030 General Plan, as well as unincorporated areas outside the City limits. The City's land use within the sphere of influence is contained entirely in a geographic information system (GIS) database, from which this land use map was developed. The land use is represented by parcel and can be used for a wide variety of analyses aside from generating wastewater flows. The database consists of parcel information within the City's sphere of influence and each parcel has been assigned a land use classification. Parcels vary in size and range from less than one acre to several hundred acres. Based on discussions with the City's Planning Department, this map represents both existing land use and future land use for the Year 2030. The land use designation categories were used to generate wastewater flows.

For this Master Plan analysis, land uses are divided into four (4) broad categories or designations: residential, commercial, public facilities, and industrial.

Residential land uses are subdivided into categories that generally reflect the density of existing residential development. These categories include single family and multiple family subdivisions. Single family residential development is characterized by those residential neighborhoods or subdivisions with detached housing intended for use by a single family. The majority of the land within the City designated as residential is in this category. Apartments and condominium developments are included in the multiple family residential category.

The commercial land use designation refers to a wide range of retailing, administrative, and service-related activities.



Map per City of Brawley Planning Dept.
Updated April 2011

Figure 3-1
Existing Land Use Map
City of Brawley Integrated Master Plan
April 2013

PSOMAS
3111 Camino Del Rio North, Suite 702
San Diego, CA 92108
(619) 961-2800 (619) 961-2392 fax
www.psomas.com

The public facilities land use designation includes a wide range of public facilities, such as government offices, parks, fire and police stations, religious facilities, hospitals, schools, and medical offices.

Industrial land uses are divided into light industrial/business and industrial. The light industrial designation allows for a range of non-manufacturing uses, such as warehousing and distribution facilities, while industrial refers to industrial activities related to manufacturing and assembly.

3.2.2 Existing Land Use

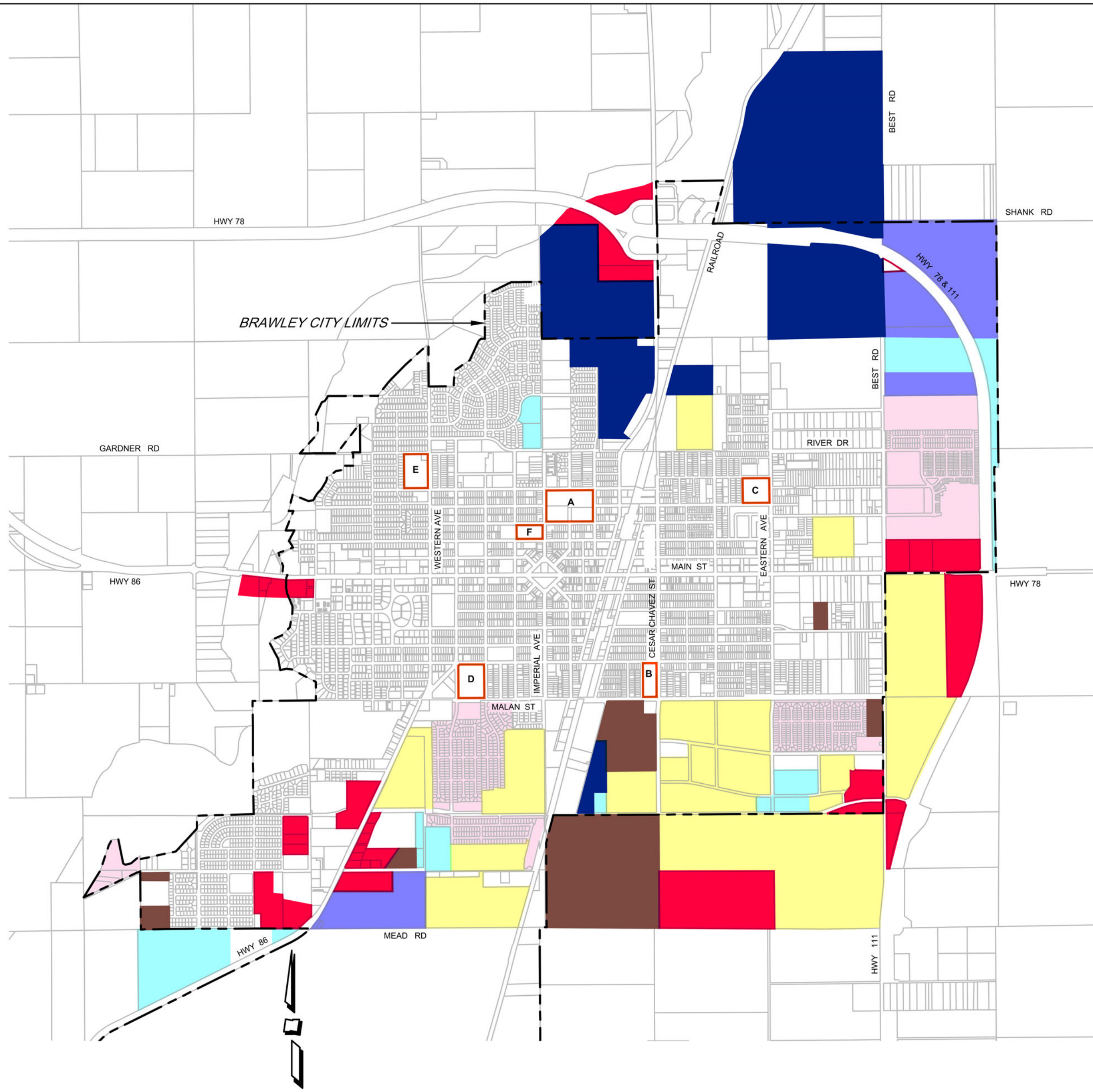
Based on the total calculated areas from the City's GIS database, there are 5,431 acres of land within the City's sphere of influence that were considered in this Master Plan for the purposes of estimating wastewater flows. This acreage of land does not include rural residential, open space, transportation, and agricultural land, as flows from these land use areas were considered negligible. An analysis was performed for all existing land use areas, which took into account new developments under construction that had to be halted or planned developments that were postponed due to the recession and were left undeveloped or vacant. **Figure 3-2** shows the areas that were deemed vacant or 'not developed' by land use category, as a result of this analysis for the existing land use areas. Of the 5,431 acres of land, roughly 2,915 acres were considered existing developed area.

Existing land use in the City's sphere of influence is tabulated in **Table 3-2** by classification. Land uses presented in this Master Plan are solely for the purposes of estimating wastewater flows.

Table 3-2
Existing Land Use

General Plan Land Use Category	Sphere of Influence Total Area (Ac)	Existing Developed Area (Ac)	Vacant Area (Ac)	% Vacant by Land Use	% of Total Vacant Land
Residential					
Low Density (3 to 7 DU/Ac)	1,977	1,127	850	43%	33%
Medium Density (15 DU/Ac)	621	364	257	41%	10%
Residential Subtotal	**2,598**	**1,491**	**1,107**	**43%**	**44%**
Commercial	**704**	**349**	**355**	**50%**	**14%**
Public Facilities	**747**	**551**	**196**	**26%**	**8%**
Industrial					
Industrial	1,073	437	636	59%	25%
Light Industrial/Business	309	87	222	72%	9%
Industrial Subtotal	**1,382**	**524**	**858**	**62%**	**34%**
TOTAL	**5,431**	**2,915**	**2,516**	**46%**	**100%**

As shown in **Table 3-2**, about 46% of the City's sphere of influence, as described, remains vacant land (2,516 acres). Of this vacant land, approximately 7% is zoned commercial and 20% is zoned residential. It is assumed that all of this land will ultimately be developed.



SCALE: 1"=2500'

SCHOOLS:

# ID	SCHOOL NAME	AREA* (ACRE)
A	BRAWLEY HIGH SCH	18
B	MIGUEL HIDALGO EL	6
C	OAKLEY ELEM SCH	8
D	WITTER ELEM SCH	11
E	PHIL SWING ELEM S	10
F	BARBARA WORTH JH	5

* ACREAGE ESTIMATED BASED ON GENERAL PLAN LOT LINES AND ADDRESS METER DATA; NOT TAKEN FROM RECORD INFORMATION.

LEGEND:

SYMBOL	LAND USE / DESCRIPTION
	INDUSTRIAL (VACANT LAND)
	LIGHT INDUSTRIAL (VACANT LAND)
	COMMERCIAL (VACANT LAND)
	LOW DENSITY RESIDENTIAL (VACANT LAND)
	MED DENSITY RESIDENTIAL (VACANT LAND)
	PUBLIC FACILITIES (VACANT LAND)
	BRAWLEY CITY LIMITS
	SCHOOL
	PARTIALLY DEVELOPED AREA

PSOMAS
3111 Camino Del Rio North, Suite 702
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(619) 961-2800 (619) 961-2392 fax
www.psomas.com

Figure 3-2
Vacant Land / School Map
City of Brawley Integrated Master Plan
April 2013

3.2.3 *Projected Land Development*

In designated areas of the City's sphere of influence, proposed projects are being planned for future development, see **Figure 3-3**. The City Planning Department identified project summaries of proposed developments that are in various planning stages. In general, development of these areas can be expected to increase the wastewater flows.

3.2.3.1 Projected 2020 Development

The projected statistics for 2020 development are shown in **Table 3-3**. These areas are based on the assumption that development will increase at a relatively constant rate in order to reach the projected 2030 build out condition.

Table 3-3
Projected 2020 Land Use

General Plan Land Use Category	Sphere of Influence Total Area (Ac)	Projected 2020 Developed Area (Ac)	Vacant Area (Ac)	% Vacant by Land Use	% of Total Vacant Land
Residential					
Low Density (3 to 7 DU/Ac)	1,977	1,535	442	22%	32%
Medium Density (15 DU/Ac)	621	465	156	25%	11%
Residential Subtotal	**2,598**	**2,000**	**598**	**23%**	**43%**
Commercial	**704**	**508**	**196**	**28%**	**14%**
Public Facilities	**747**	**634**	**113**	**15%**	**8%**
Industrial					
Industrial	1,073	737	336	31%	24%
Light Industrial/Business	309	166	143	46%	11%
Industrial Subtotal	**1,382**	**903**	**479**	**35%**	**35%**
TOTAL	**5,431**	**4,045**	**1,386**	**26%**	**100%**

As shown in **Table 3-3**, about 26% of the City's service area, as described, remains as vacant land (1,386 acres). Of this vacant land, approximately 4% is zoned commercial and 11% is zoned residential.

Although there are a multitude of possible scenarios for the proposed 2020 development condition, for simplicity, this Master Plan analysis focuses on one 2020 development scenario. See **Figure 3-4** for the proposed 2020 development land use limits.

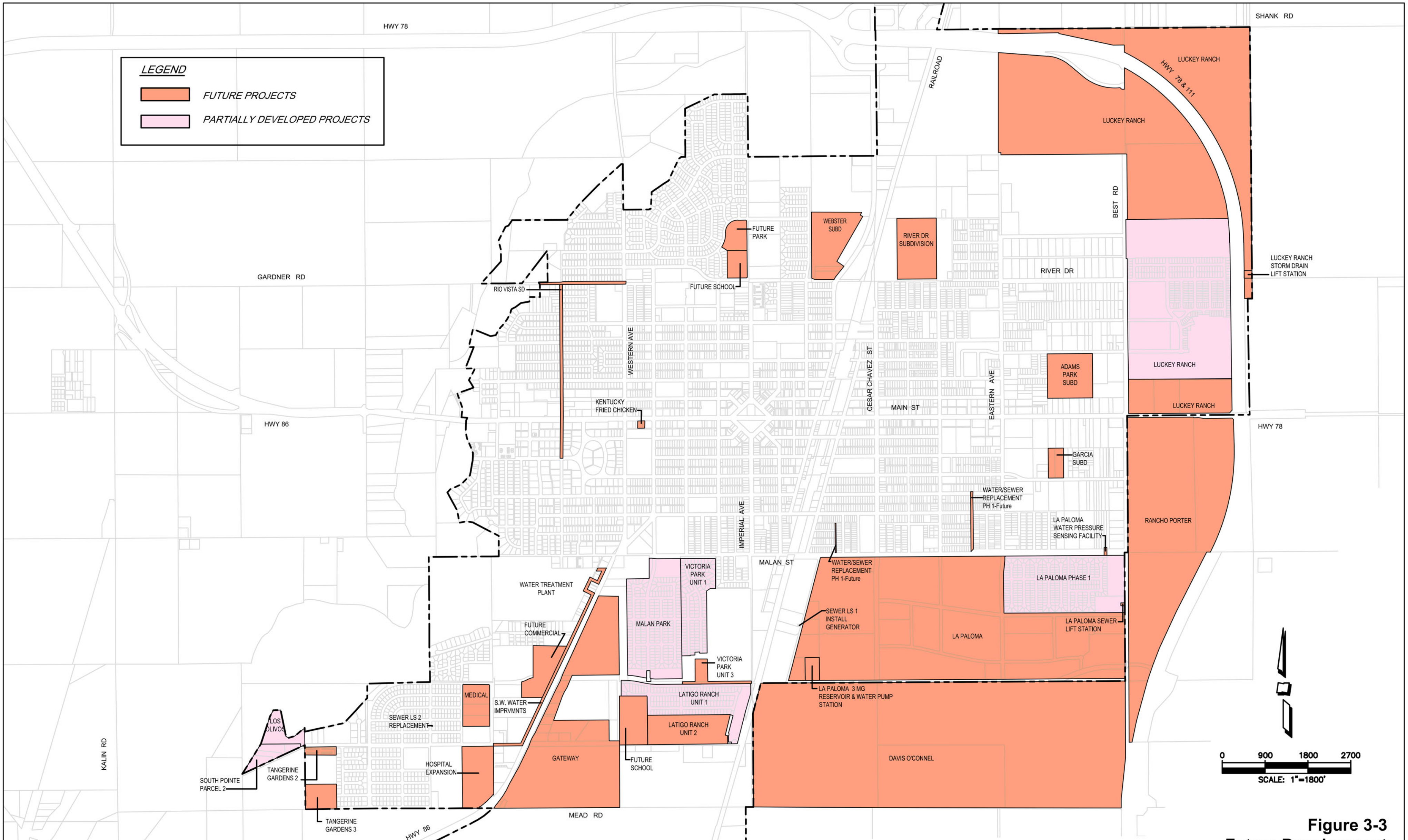

Figure 3-3
Future Development
City of Brawley Integrated Master Plan
April 2013

PSOMAS
3111 Camino Del Rio North, Suite 702
San Diego, CA 92108
(619) 961-2800 (619) 961-2392 fax
www.psomas.com

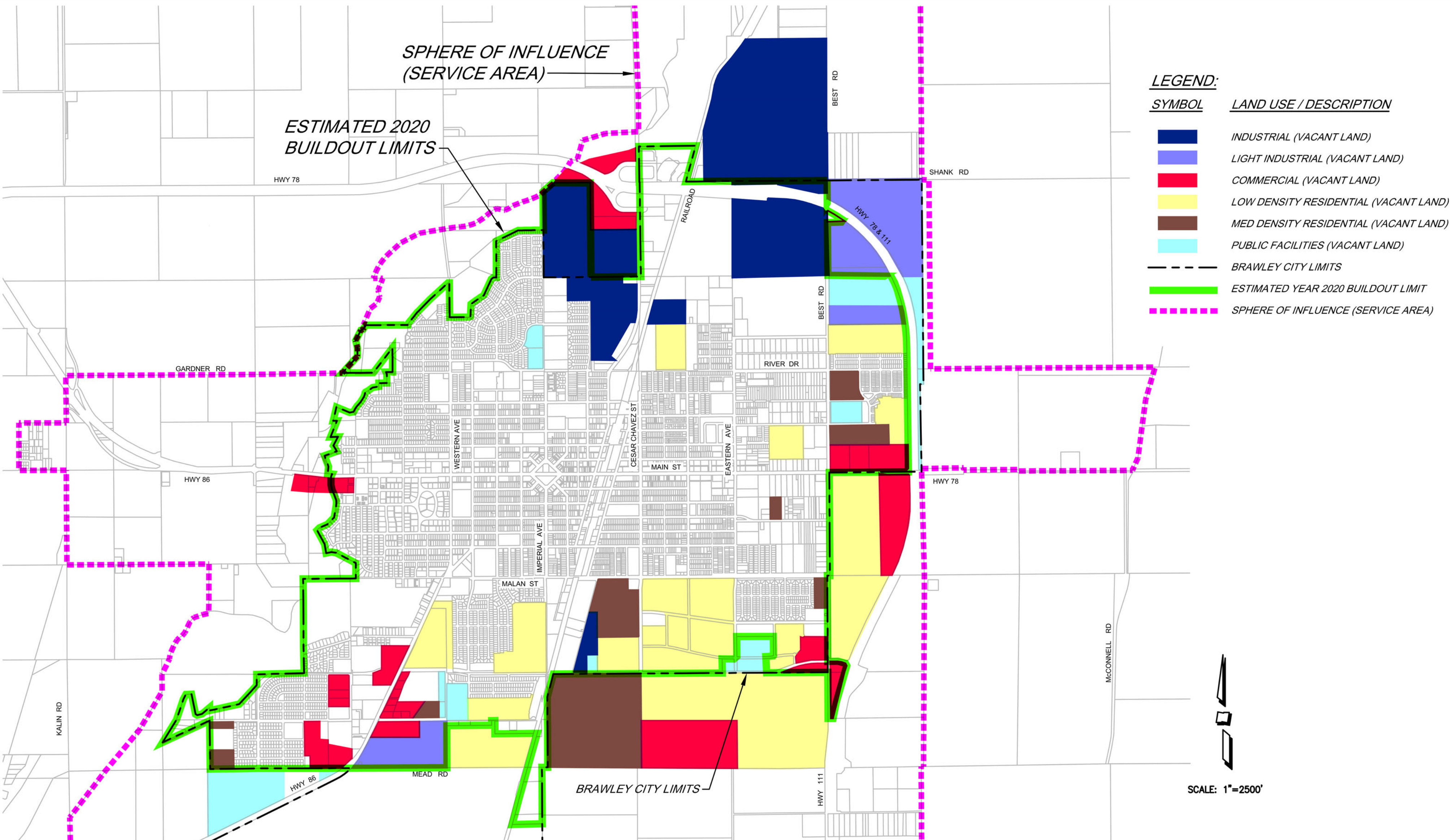

Figure 3-4
2020 Vacant Land by Land Use
City of Brawley Integrated Master Plan
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3.2.3.2 Projected 2030 Development

The projected 2030 development areas are shown in **Table 3-4**. These areas are based on the projected 2030 build out condition.

Table 3-4
Projected 2030 Land Use

General Plan Land Use Category	Sphere of Influence Total Area (Ac)	Projected 2030 Developed Area (Ac)	Vacant Area (Ac)	% Vacant by Land Use	% of Total Vacant Land
Residential					
Low Density (3 to 7 DU/Ac)	1,977	1,977	0	0%	0%
Medium Density (15 DU/Ac)	621	621	0	0%	0%
Residential Subtotal	**2,598**	**2,598**	**0**	**0%**	**0%**
Commercial	**704**	**704**	**0**	**0%**	**0%**
Public Facilities	**747**	**747**	**0**	**0%**	**0%**
Industrial			0	0%	0%
Industrial	1,073	1,073	0	0%	0%
Light Industrial/Business	309	309	0	0%	0%
Industrial Subtotal	**1,382**	**1,382**	**0**	**0%**	**0%**
TOTAL	**5,431**	**5,431**	**0**	**0%**	**0%**

As shown in **Table 3-4**, assuming a build out condition for 2030, 0% of the City's service area remains as vacant land.

Table 3-5 shows a summary of the grouped land-use types with an existing and projected acreage value for each of the planning years.

Table 3-5
Summary of Existing and Projected Land Use

General Plan Land Use Category	Existing (Ac)	2020 (Ac)	2030 (Ac)
Residential			
Low Density (3 to 7 DU/Ac)	1,127	1,535	1,977
Medium Density (15 DU/Ac)	364	465	621
Residential Subtotal	**1,491**	**2,000**	**2,598**
Commercial	**349**	**508**	**704**
Public Facilities	**551**	**634**	**747**
Industrial			
Industrial	437	737	1,073
Light Industrial/Business	87	166	309
Industrial Subtotal	**524**	**903**	**1,382**
Total Service Area & Sphere of Influence	**2,915**	**4,045**	**5,431**

3.3 Existing Flows

3.3.1 General

This section describes the methodologies used to develop existing unit generation rates for wastewater flows in the City of Brawley.

3.3.2 Existing Flow Generation Rates

The wastewater flow generation factors were the result of calibration against the flow monitoring results, the Wastewater Treatment Plant flows, and from researching cities and wastewater agencies with similar land use classifications. Cities/agencies that were investigated were the Cities of El Centro, La Mesa, Chula Vista, and San Diego. As discussed in the Land Use Section, the existing land use designations within the City's sphere of influence consist of residential, industrial, commercial, and public facilities.

The flow monitoring results were used to determine the wastewater flows generated from the existing low density residential and medium density residential land use areas. The resulting wastewater flow generation factors were estimated to be 1,100 gpd/acre for low density residential and 2,400 gpd/acre for medium density residential. Next, after determining the factors for residential land use, flow meter data from the National Beef Plant identified it as generating the largest wastewater flows in the City's sphere of influence with an average daily flow of 810,240 gpd.

Table 3-10 shows the calibrated generation factors that were used for the existing land use wastewater flow analysis.

To create flow rates for the various land use categories, the following procedure was followed:

Step 1: Set up flow monitors throughout the City. Select one monitoring site comprised entirely of Single Family Residential (SFR) homes.

Step 2: Determine ADF at WWTP.

Step 3: Determine ADF for LDR per acre, per EDU and per capita. Compare with other agencies to confirm these numbers are within a normal range.

Step 4: Estimate an amount for I&I based on recent rainfall data.

Step 5: Determine the flows for the beef plant and schools.

Step 6: Generate sewer demands for remaining land uses.

The resulting wastewater flows represent the system flows for an average day of the year. Seasonal and daily variations in the wastewater flows are accounted for by adjusting the average daily flow using a multiplier to simulate other flow periods. These multipliers, or peaking factors, were developed to calculate the Peak Dry Weather Flow (PDWF) and the Peak Wet Weather Flow (PWWF) using the average daily flow as a basis.

3.3.2.1 Step 1: Flow Monitoring

Downstream Services, Inc. (Downstream), was retained as a subconsultant to conduct a temporary flow monitoring study of the wastewater collection system at five (5) different locations during a three-week period. The sites were monitored during the period of February 2, 2012 through February 22, 2012. The program was designed to capture average, minimum, and peak flows, while hopefully capturing a wet weather event, if possible. Fortunately, approximately 1” of rainfall occurred on February 16, 2012 (recorded at PWS MQCAC1, Holtsville, CA), which resulted in increased flows at all the metered sites.

The purpose of the flow monitoring was to measure and determine the relative flow from different areas of the wastewater collection system. The flow measurements collected established a benchmark for sewer model calibration.

The individual site locations were chosen to collect flow measurements from the major tributary areas within the City and are depicted in **Figure 3-5**. **Table 3-6** provides detailed information on the flow monitoring locations.

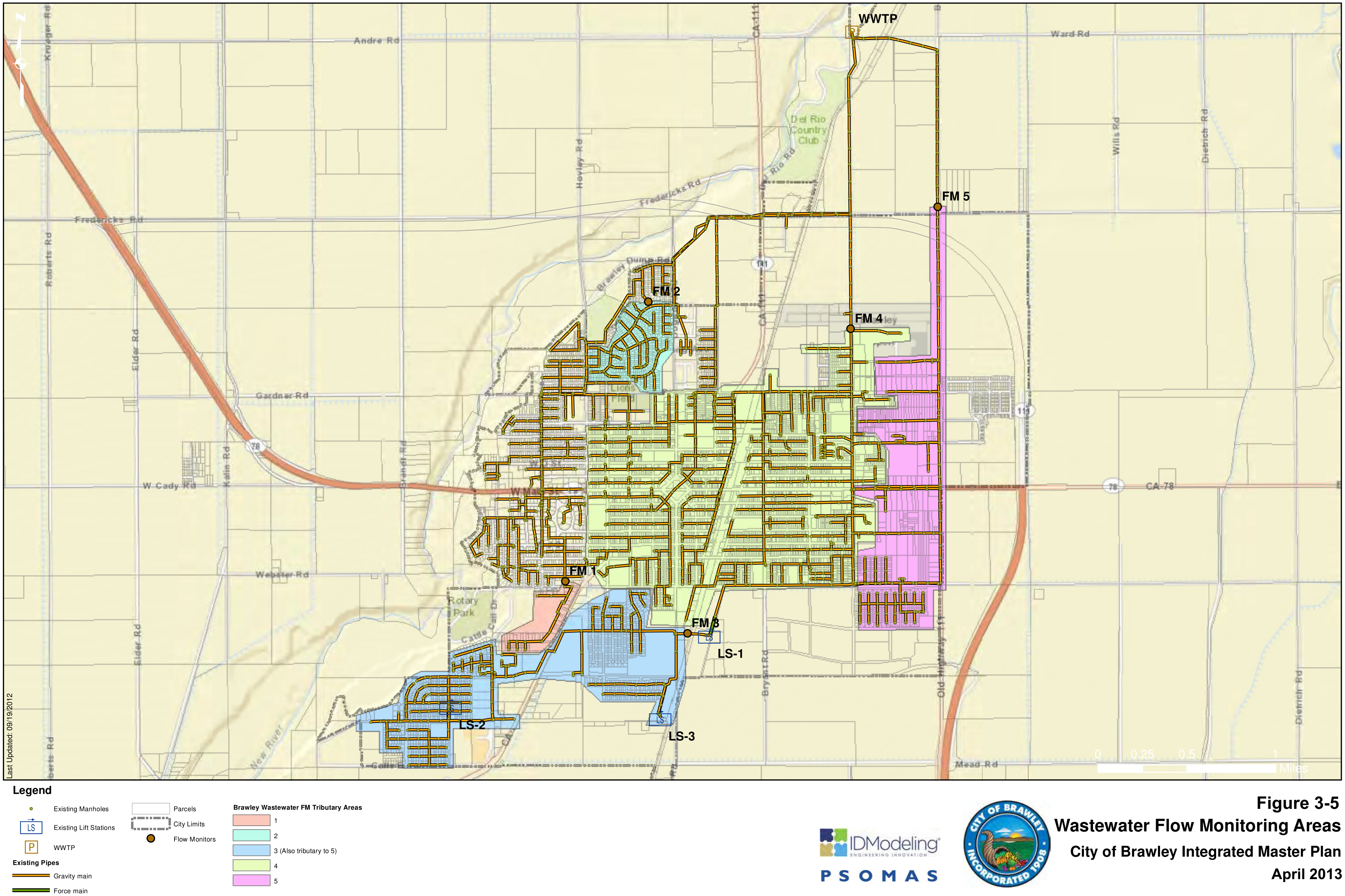

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Figure 3-5

Wastewater Flow Monitoring Areas

City of Brawley Integrated Master Plan

April 2013

Table 3-6
Flow Monitoring Locations

Flow Monitoring Site	Street	MH ID	Pipe Size
Site 1	El Cerrito	WE007.16	8”
Site 2	Chestnut	WC035.00	12”
Site 3	East of Dogwood	ED005.00	15”
Site 4	N. Eastern Ave.	CA026.00	21”
Site 5	Best Rd.	EA022.00	21”

3.3.2.2 Step 2: Determine ADF at WWTP

The City provided average flows for the WWTP for the period between January 2011 through March 2012. For this 15-month period, the ADF was **3.84 MGD**. This number was used to help determine the flow rates for each land use category.

3.3.2.3 Step 3: Determine Existing LDR Sewer Demands

The flow monitoring results for site FM2 were used to determine the base flow rate for all LDR zoning for the City. Some pertinent data for Site 2 are shown in **Table 3-7**.

Table 3-7
Metering Site FM2 Pertinent Information

Flow Monitoring Site	Average Flow (GPM)	# of LDR Dwellings	Gross Acreage
FM2	81	436	105

To determine the LDR sewer demands for acreage and Equivalent Dwelling Units (EDU), the following calculations were performed.

Sewer Demands/Gross Acre
Metered Average Daily Flow (ADF) = 81 GPM = 116,640 GPD
Sewer Demands for LDR by Land Use = 116,640/105 AC = 1,110 GPD/AC
(Use 1,100)

Sewer Demands/EDU
# of EDU’s = 436 dwellings * 1 dwelling/EDU = 436 EDU’s
Sewer Demands for LDR by EDU = 116,640 GPD/436 EDU’s = 268 GPD/EDU

Sewer Demands/Capita
# of people per single family residence = 3.50
(*Table LUE-27 of the City's 2008 General Plan gives an estimated persons per household of 3.24. Psomas used 3.50 per single family residence and 2.90 per multi-family residence*)
Total # of people in metering basin = 3.50 * 436 dwellings = 1,526 people
Sewer Demands for SFR per capita = 116,640 GPD/1,526 = 77 GPD/Person

Table 3-8 summarizes the flow rates for LDR.

Table 3-8
Existing Sewer Demands: LDR

Sewer Demands (GPD/Gross Acre)	Sewer Demands (GPD/EDU)	Sewer Demands (GPD/Person)
1,100	268	77

Calculated demands for this project were compared with those of other agencies.

This compares well with Table 2.1 of the City of El Centro's 2008 Sewer Master Plan which uses a sewer demand of 1,100 gpd/ac for LDR.

Table 3-2 of the City of La Mesa's 2008 Sewer Master Plan uses a single family residential unit generation rate of 270 gpd/DU, while Table 7-1 of the City of Chula Vista's 2005 Sewer Master Plan uses a unit generation rate of 265 gpd/du.

Typically, design standards for sewer agencies in San Diego County assume wastewater flows between 65 and 100 gpcd for residential land use.

These comparisons confirm that our existing sewer demands are within an acceptable range.

3.3.2.4 Step 4: Determine Amount of Inflow & Infiltration

During rain events a certain amount of rainwater enters the sewer system via storm drain inlets in the combined portion of the City and infiltration through cracks and defects within the system. Infiltration was considered to be mostly negligible, since the majority of sewers appear to be above the groundwater table. An attempt could be made to calculate the amount of inflow in the combined system; however, there isn't enough information to accurately do this. Throughout the combined system there are many areas that contain low points with no inlets; therefore, water ponds and ultimately evaporates. In other areas, inlets are present, but the storm drain connection to the sewer may be clogged. Therefore, the most logical method was to compare known rainfall data to WWTP metering data to try and estimate an I&I quantity. To try and estimate this quantity, Psomas reviewed rainfall records for the past 3 years and compared these with flows at the WWTP. Rainfall amounts did not result in consistent increases of flow at the WWTP; therefore, assumptions had to be made. The 1" storm, which corresponds to approximately a 2-year, 24-hour storm, that occurred from February 16, 2012 to February 17, 2012, drained approximately 1,500,000 gallons of stormwater to the sewer system. This number was used to determine the approximate I&I. Based on an average of 3” of rainfall per year for the City of Brawley, an estimated 4,500,000 gallons of stormwater enters the system, which equals approximately 12,329 GPD.

3.3.2.5 Step 5: Determine Flows for Large Users

Flows for the beef plant and schools were calculated separately and used when determining land use flow rates. For the beef plant, the calculated water usage is 16,879 GPD/AC. Psomas estimated approximately 50% of this ends up in the sewer, thus giving a flow rate of **8,440 GPD/AC**. **Table 3-9** shows the flows calculated for each of the schools.

It was determined that the National Beef flows and the existing school site flows should be isolated out of their land use category as part of the calibration process, in order to estimate more accurate wastewater flow generation factors for the remaining industrial, commercial, and public facility land use categories. After isolating out the National Beef Plant and the existing schools, the wastewater flow generation factors were calibrated for the remaining land use categories such that the total summation of existing wastewater flow within the City’s sphere of influence was equal to the total existing metered Wastewater Treatment Plant influent flow of 3.84 MGD.

Table 3-9
Sewer Demands for Brawley Schools

School	Population	Gross Acres	GPD/Student*	GPD	GPDx65%**	GPD/AC
Brawley High	1,750	18	15	26,250	17,063	948
Miguel Hidalgo Elem.	700	6	15	10,500	6,825	1,138
Oakley Elem.	750	8	15	11,250	7,313	914
Witter Elem.	720	11	15	10,800	7,020	638
Phil Swing Elem.	850	10	15	12,750	8,288	829
Barbara Worth Junior High	850	5	15	12,750	8,288	1,658
Totals	**5,620**	**58**	**--**	**84,300**	**54,797**	**945 avg.**

* *This value taken from design information provided by Metcalf & Eddy*

** *65% is based on the assumption that school is open approximately 20 days per month or roughly 65% of the time.*

3.3.2.6 Step 6: Generate Sewer Demands For Remaining Land Uses

Once flow rates for LDR, I&I, beef plant, and schools were determined, the remaining were determined using an iterative process, with an end result that totals the 3.83 MGD average flow at the WWTP.

For the MDR, an iterative process was used. 2,400 gpm is approximately 80% of the 3,000 gpm used for water demands, which is reasonable, given the fact that nearly all the water used by residents of apartments and condos enters the sewer. Very little is used for landscaping. A typical housing complex will have a certain amount of landscaping around the buildings, but in Brawley much of it contains drought tolerant plants or is bare. The 2,400 GPD/AC for MDR results in 71 GPCD and 205 GPD/EDU. These are based on approximately 2.9 persons per dwelling and 11.7 units per gross acre. Each of these numbers is within an acceptable range in comparison with other nearby agencies. 11.7 units per gross acre is based on Table LUE-3 of the 2008 Brawley General Plan, which calls for 13 EDU/Net Acre. The table mentions net acres are calculated based on an assumed 10% reduction of gross acreage. At 13 EDU/Net Acre, this results in approximately 11.7 EDU/Gross Acre.

Table 3-10 gives all other sewer demands by land use.

Table 3-10
Existing Sewer Demand and Factors by Land Use

Land Use Type	Area (ac)*	Sewer Demand Factor (gpd/ac)	ADD (MGD)	ADD (GPD/EDU)	ADD GPCD)
Low Density Residential	1,127	1,100	1.240	268	77
Medium Density Residential	364	2,400	0.874	205	71
Commercial	349	700	0.244		
Public Facilities	493	600	0.296		
Industrial	341	700	0.239		
Light Industrial/Business Park	87	700	0.0609		
Beef Plant	96	8,440	0.810		
Schools	58	950	0.0551		
Inflow & Infiltration			0.0123		
Totals	**2,915**	**--**	**3.831**		

* *The acreages in this table differ from the same Table 3-11 in the Water Master Plan. The reason is because in the Water Master Plan, large users, such as housing projects, apartments complexes, and other establishments were removed from the total acreage and counted separately due to availability of metering data. Since no metering data exists for the sewers of those facilities, the corresponding acreages were left in their respective category. As shown above, the only acreages excluded were the beef plant and schools. Refer to Table 3-2 for calculations of developed land vs. vacant land.*

Sewer demands for commercial, public, and industrial facilities were estimated based on historical data and comparisons with other agencies. For example, the City of El Centro's 2008 Sewer Master Plan uses 800 GPD/AC for commercial, 400 GPD/AC for industrial, and 650 GPD/AC for public facilities.

3.4 Future Sewer Demands

As development and redevelopment continues, cities or agencies see a corresponding increase in population and, therefore, in sewer demands. Population projections for the City show an increase of about 143% percent from existing condition population of 24,953 to the 2030 buildout condition population of 60,542. The majority of the increased sewer demands are projected to come from new development, as opposed to redevelopment.

As a result of the City's metering program, the residential demand factors have dropped over the years since the last master plan was prepared. As the City residential areas are all now metered, it is deemed reasonable that future residential demand factors will likely remain relatively unchanged. For this reason, the low density and medium density residential demand factors used for the future conditions will remain at 1,100 gpd/ac and 2,400 gpd/ac. respectively. However, for planning purposes, increasing the sewer demand factors for commercial, public facilities, industrial, and light industrial/business park land uses was deemed reasonable, due to the numerous future development possibilities for each land use type and the other unknowns at this time.

Table 3-11 summarizes the sewer demand factors used for each planning year by land-use category.

NOTE: ***It is important to note that these sewer demands are for planning purposes only. For individual sites, Psomas recommends the designer determine projected flows by counting fixture units or square footage of buildings and/or number of people occupying the structure.***

Table 3-11
Future Sewer Demand Factors by Planning Year

Land-use Type	Sewer Demand Factor (gpd/ac)		
	2010	2020	2030
Low Density Residential	1,100	1,100	1,100
Medium Density Residential	2,400	2,400	2,400
Commercial	700	1,300	1,300
Public Facilities	600	1,100	1,100
Industrial	700	1,500	1,500
Light Industrial/Business Park	700	1,300	1,300

3.4.1 Future 2030 Sewer Demands and Factors

Table 3-12 shows a comparison of the future sewer demands for the projected 2030 buildout condition by land use category with existing conditions. It is projected that an additional sewer demand of 3.47 MGD will be added to the system by the year 2030. This will increase the total system demand to 7.31 MGD, slightly less than double the existing demand of 3.831 MGD.

Table 3-12
Comparison Existing/Future 2030 Sewer Demands by Land Use

Land Use Type & Existing Users	Existing Conditions			2030 Conditions			Totals	
	Developed Area (ac)	Sewer Demand Factor (gpd/ac)	ADD (MGD)	Additional Developed Area (ac)	Sewer Demand Factor (gpd/ac)	Additional ADD (MGD)	Total Acreage (ac)	Total 2030 ADD (MGD)
Low Density Residential	1,127	1,100	1.24	850	1,100	0.935	1,977	2.18
Medium Density Residential	364	2,400	0.874	257	2,400	0.617	621	1.49
Commercial	349	700	0.244	355	1,300	0.462	704	0.706
Public Facilities*	493	600	0.296	196	1,100	0.216	689	0.512
Industrial	341	700	0.239	636	1,500	0.954	977	1.19
Light Industrial/ Business Park	87	700	0.061	222	1,300	0.289	309	0.35
Beef Plant	96	8,440	0.810	--			96	0.81
Existing Schools	58	950	0.0554	--			58	0.0551
I&I			0.0123					0.0123
Totals	**2,915**		**3.831**	**2,516**		**3.47**	**5,431**	**7.305**

** Future schools are added to public facilities acreage*

3.4.2 Future 2020 Sewer Demands and Factors

Table 3-13 shows a comparison of future sewer demands for the projected 2020 development by land use with existing conditions. As discussed previously, for the purposes of this Master Plan, it is projected that development will occur at a relatively constant rate between now and ultimate buildout in 2030. Therefore, the added 2020 water demands are approximately 1.55 MGD, which is more than half of that projected for 2030.

Table 3-13
Comparison Existing/Future 2020 Sewer Demands by Land Use

Land Use Type & Existing Users	Existing Conditions			2020 Conditions			Totals	
	Developed Area (ac)	Sewer Demand Factor (gpd/ac)	ADD (MGD)	Additional Developed Area (ac)	Sewer Demand Factor (gpd/ac)	Additional ADD (MGD)	Total Acreage (ac)	Total 2020 ADD (MGD)
Low Density Residential	1,127	1,100	1.24	408	1,100	0.440	1,535	1.68
Medium Density Residential	364	2,400	0.874	101	2,400	0.242	465	1.12
Commercial	349	700	0.244	156	1,300	0.203	505	0.447
Public Facilities*	493	600	0.296	83	1,100	0.091	576	0.387
Industrial	341	700	0.239	312	1,500	0.468	653	0.707
Light Industrial/ Business Park	87	700	0.061	79	1,300	0.103	166	0.164
Beef Plant	96	8,440	0.810	--				0.81
Existing Schools	58	950	0.0554	--				0.0554
I&I			0.0123					0.0123
Totals	**2,915**		**3.831**	**1,139**		**1.547**	**3,900**	**5.38**

** Future schools are added to public facilities acreage*

3.4.3 Rancho Los Lagos Future System Water Demands

The City is considering supplying water service to the future Rancho Los Lagos development proposed south of the City's current sphere of influence. Based on data provided in the Rancho Los Lagos development draft EIR, the development will receive water from the City of Brawley and sewer services from the City of Imperial.

3.5 Peaking Factors and Existing and Future Demands

3.5.1 Peaking Factors

Peaking factors were developed in order to determine the sewer demands for conditions other than an average day's water use. Peaking factors account for fluctuations in demands on a daily or hourly basis. For example, during hot summer days, water use is typically higher than on a cold and/or rainy winter day, thus resulting in more sewage generation. Common peaking factors include factors for peak dry weather flow (PDWF) and peak wet weather flow (PWWF) periods. Typically, peaking factors are created by comparing flow monitoring data with metering data at the WWTP. For this project, inconsistent and unreliable metering data was available, while flow monitoring data did not capture the entire City; therefore, a peaking factor developed by the City of Los Angeles was used. This peaking factor has been cross checked with other agencies and is fairly typical for the Southern California region. The peaking factor equation was taken from the City of Los Angeles' Bureau of Engineering Manual, Part F, and is as follows:

$$Q_{PDWF}=2.64(Q_{ADWF})^{0.905}$$

Figure 3-6 is a nomograph showing the relationship between the peaking factor and ADWF.

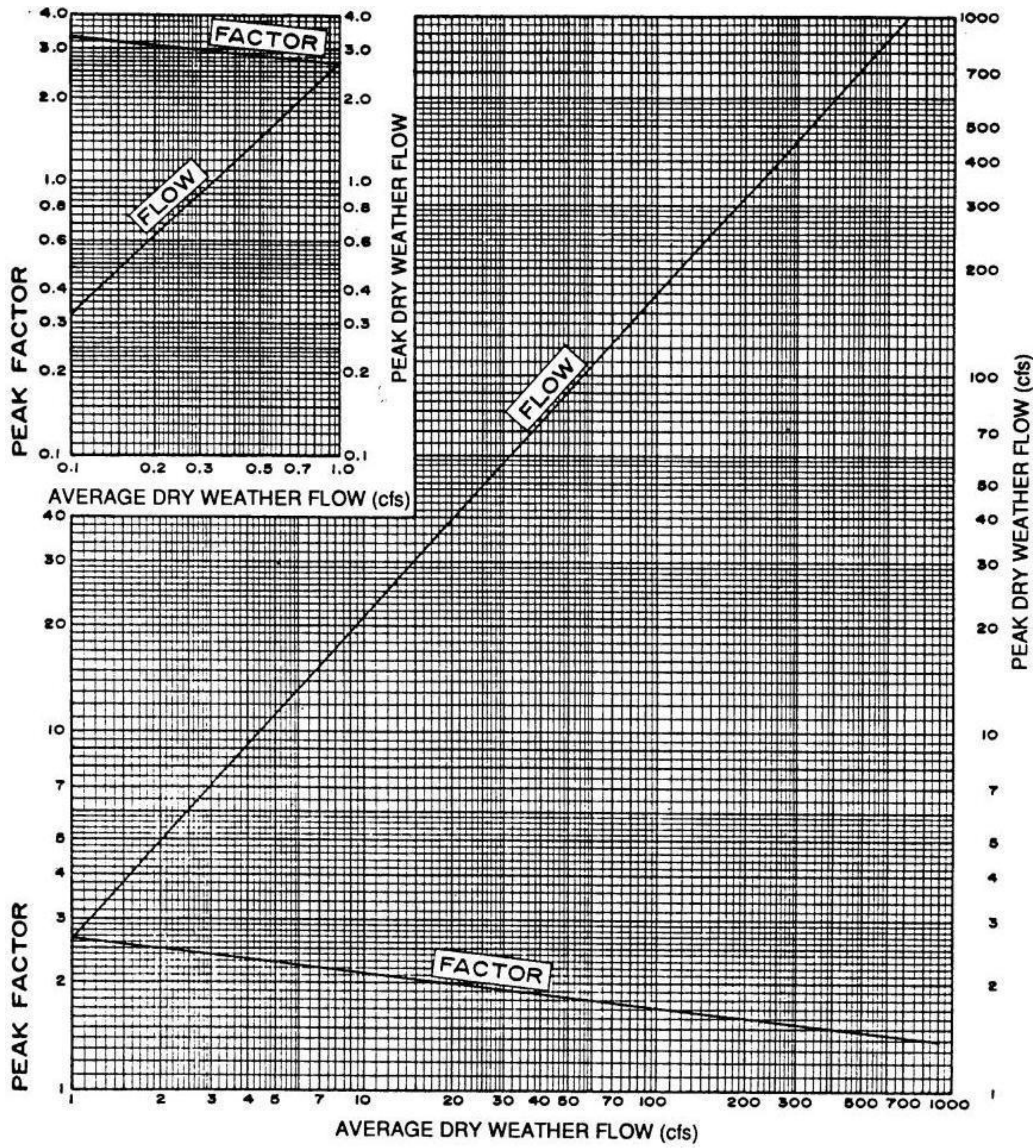


Figure 3-6-ADWF-PDWF Flow Chart

Variations in sewer generation also occur during a 24-hour period. In residential areas, there are often two peak-use periods, in the morning and in the late afternoon.

3.5.2 Existing Average Dry Weather Flow

The ADWF was calculated by averaging the daily metered flows measured at the WWTP between 2010 and early 2012. For the City, the monthly average daily demand was used to establish:

ADWF = 3.831 MGD

3.5.3 Existing Peak Dry Weather Flow

The PDWF represents the ADWF multiplied by the peaking factor. Using the equation described above for the peaking factor, the PDWF was established:

PDWF = 8.90 MGD

3.5.4 Existing Average Wet Weather Flow

Inflow and infiltration (I&I) was estimated as described in Section 3.3.2.4. The Average WWF was calculated by adding the ADWF with I&I, which was estimated at 1.5 million gallons of rain induced runoff introduced to the sewer system for every inch of rain.

AWWF = 5.33 MGD

3.5.5 Existing Peak Wet Weather Flow

PWWF consists of PDWF plus I&I.

PWWF = 10.4 MGD

Flow rates for the years 2020 and 2030 were also computed. **Table 3-14** provides a summary of the existing and future demands.

Table 3-14
Existing and Future Sewer Demands

Year	ADWF (MGD)	PDWF (MGD)	AWWF (MGD)	PWWF (MGD)
2012	3.831	8.90	5.33	10.4
			2012 Peaking Factor	**2.32**
2020	5.38	12.10	6.88	15.13
			2020 Peaking Factor	**2.23**
2030	7.31	15.98	8.81	17.48
			2030 Peaking Factor	**2.19**

3.6 Master Plan Design Criteria

3.6.1 Wastewater System Design Criteria

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

Continuity Equation: $Q = V A$

where:
Q = peak flow, cfs
V = velocity, fps
A = cross-sectional area of pipe, sq. ft.

Manning's Equation: $V = (1.486 R^{2/3} S^{1/2})/n$

where:
V = velocity, fps
n = Manning's coefficient of friction
R = hydraulic radius (area divided by wetted perimeter), ft
S = slope of pipe, ft/ft

3.6.2 Manning's Coefficient (n)

The Manning coefficient 'n' is a friction coefficient and varies depending on the type of material. For example, plastic pipe would have an initial 'n' value of 0.010, while concrete would have an 'n' value of 0.013. There has been much debate about the appropriate 'n' value to use for different piping materials in wastewater systems. To complicate the debate, the slime layer that thrives on the wetted portions of the sanitary piping also contributes to and affects the actual value of ‘n’ along with grease buildup. This study utilized ‘n’ values as shown in **Table 3-15** below based on a conservative value for each pipe material.

Table 3-15
Manning’s Equation “n” Values

Pipe Material	Manning Value
DIP (Ductile Iron)	0.012
PVC (Polyvinyl Chloride)	0.011
Unknown Material	0.013
VCP (Vitrified Clay)	0.014

3.6.3 Design Velocities

In an effort to maintain the suspension of solids in sewers, the minimum design velocity shall be 2 ft/sec during the peak dry-weather flow (PDWF) at the time the pipe is placed into service. At this velocity, the pipe will be self-cleaning. The maximum design velocity shall be limited to 10 ft/sec during full flow or ultimate peak flow conditions.

3.6.4 Minimum Slopes

To maintain the velocities presented above, minimum slopes are required for various pipe sizes. These slopes will apply to new developments. Although these slopes will provide 2 ft/sec when flowing half full, every attempt should be made to increase slopes when possible. Larger pipes can be placed at flatter slopes and still achieve 2 ft/sec; however, due to difficult of construction, are not recommended. See **Table 3-16** for recommended pipe slopes.

Table 3-16
Minimum Recommended Pipeline Slopes

Pipe Size (IN)	Minimum Slope [1] (%)
8	0.34
10	0.26
12	0.20
15	0.15
18	0.15
21	0.15
24	0.15
30	0.15
36	0.15

[1] Based on a velocity of 2 ft/sec with "n" value of 0.013 and d/D of 0.50. For pipes >15" diameter, slopes shown above provide >2 ft/sec. Slopes <0.15% are not recommended due to difficulty of construction and to allow a margin of error.

3.6.5 Flow Depth Criteria

The capacity criteria for gravity sewers are typically evaluated by a ratio of flow depth over pipe diameter (d/D). Sewers for this analysis shall be sized so the d/D ratios (specified below) are not exceeded while flowing under the peak dry-weather flow (PDWF) conditions. Table 3-17 gives d/D requirements.

Table 3-17
d/D Requirements

Pipe Size (IN)	PDWF (IN/IN)	PWWF (IN/IN)
≤ 12	0.5	0.9
>12	0.75	0.9

3.7 Stormwater System Design Criteria

Stormwater pipe capacities are dependent upon many factors. These include the roughness of the pipe, the maximum allowable Hydraulic Grade Line (HGL), limiting velocity and slope. The Continuity Equation and the Manning's Equation for steady state flow are used for gravity pipe hydraulic calculations as described in section 3.6.1:

3.7.1 Manning's Coefficient (n)

The Manning coefficient 'n' is described in more detail in section 3.6.2. For the storm drain system, this study will utilize an 'n' value of 0.013 for RCP and 0.011 for PVC.

3.7.2 Design Velocities

In an effort to maintain the suspension of solids in stormwater pipelines, the minimum design velocity shall be 2.5 ft/sec during the design storm at the time the pipe is placed into service.

3.7.3 Design Storm Criteria

A design storm is a representation of precipitation events that reflect conditions of a given area for design of infrastructure. Accepting a set of design storm criteria provides guidelines and consistency in sizing storm drain facility improvements.

Storms are classified by intensity, duration, and recurrence interval. Recurrence intervals may be represented as a 10-year or 50-year storm, meaning, statistically, a storm of a given duration and intensity can be expected to occur once every 10 or 50 years. For instance, a 10-year storm is less severe than a 50-year storm.

The following engineering standards were used to determine system deficiencies and needed improvements.

- 10-Year Storm: Pipes flow full
- 50-Year Storm: HGL 6" below rim elevation of lowest drainage structure in system.
- 100-Year Storm: Generally any stormwater runoff may overflow the existing pipeline capacities, but must be retained within the street rights-of-way

3.8 Conservation Efforts

Due to the City's water metering program and conservation efforts, water consumption has shown a slight decrease. The City's wastewater flows decrease as a direct result of the decrease in water usage. Since 2003, the City has installed meters on all single family homes, most multi-residential lots, and on larger commercial/industrial services. However, in most cases, the commercial services are not currently metered. Typically, water usage decreases with water meter installation and conservation efforts and, as a result, sewer flows decrease as well.

4.0 Hydraulic Computer Model Development

4.1 Introduction

This section describes the model development of the City's wastewater system that includes both the combined and separated portions of the collection system. This was completed as part of the 2012 Water Master Plan update (Project). The existing and future system model was developed from available GIS, field data, and information provided by City staff. The Innovyze InfoSewer modeling software was used to construct the model and conduct the analyses for the master plan. The InfoSewer model was exported to EPA SWMM format for the Project model deliverable.

This section describes the development of the existing system wastewater model, dry and wet weather calibration, and development of the future system wastewater model.

4.2 Existing System Wastewater Model Development

The following sections describe the process of creating the GIS combined sewer and stormwater geodatabase for model input, and developing the facility, flow loading, and operational information in the existing system model. A description of the future system model development is provided in Section 4.11.

4.2.1 System GIS Creation

Creation of a GIS geodatabase (Brawley.gdb) that includes the existing collection network data and associated attributes was the first step in development of the sewer model. Entering the collection system data within a geodatabase allowed for the use of GIS data review and connectivity tools prior to entering the data in the wastewater modeling software. The geodatabase includes both the combined and separated portions of the wastewater collection system. The primary sources for the data in the GIS geodatabase were AutoCAD Map files and shapefiles. The data sources for the CAD files were as-built and field collected information. However, there still remained some missing data fields and records due to a lack of available information and this is discussed in greater detail in the following sections.

There was some stormwater system data available in the AutoCAD file, but it was incomplete, and insufficient for development of a stormwater model. The AutoCAD file was imported into an empty geodatabase and separate feature classes were created for each network feature type.

Each type of network feature in the GIS and the corresponding model element type and fields modeled are shown in **Table 4-1**.

Table 4-1
GIS to Model Field Mapping

GIS Field Name	Model Field Name / Type	Model Fields Imported from GIS	Source Data
Manholes	Manholes	Rim Elevation	Base mapping files
Catch Basins	Manholes (Description: Catch Basin)	Rim Elevation	Base mapping files
Fittings	Manholes (Description: Fitting)	Rim Elevation	Base mapping files
Cleanouts	Manholes (Description: Cleanout)	Rim Elevation	Base mapping files
Lift Stations	Wet Wells	Floor (Base) EL High Water Level	As-builts and data provided by the City
Gravity Mains	Pipes (Type: Gravity)	Invert Elevations	GIS manhole inverts
Lateral Lines	Pipes (Type: Gravity)	Invert Elevations	GIS manhole inverts
Force Mains	Pipes (Type: Force Main)	Invert Elevations	GIS manhole inverts

An attribute was added within the Geodatabase that identified features as 'modeled' or 'not modeled' in order to filter which features would be exported to the hydraulic model. Established connectivity was the primary criteria required for modeled features. **Figure 4-1** shows the features in the GIS that were not modeled. Below is a summary of the number of elements that were exported to the hydraulic model for each feature class.

- **Manholes** – Orphan manholes (not connected via a gravity main, forcemain or lateral line) were not modeled (1,402 of 1,438 manholes modeled)
- **Catch Basins** – Orphan catch basins were not modeled (96 of 375 catch basins modeled)
- **Fittings** – Orphan fittings and fittings at the end of laterals were not modeled (63 of 89 fittings modeled)
- **Cleanouts** – Cleanouts were not modeled, except where necessary to maintain connectivity (7 of 29 cleanouts modeled)
- **Other Points** – Other Points were not modeled as they were orphans or the inlet and outlet of a single drainage pipe (0 of 3 other points modeled)
- **Lift Stations** – Private lift stations were not modeled (5 of 7 lift stations modeled)

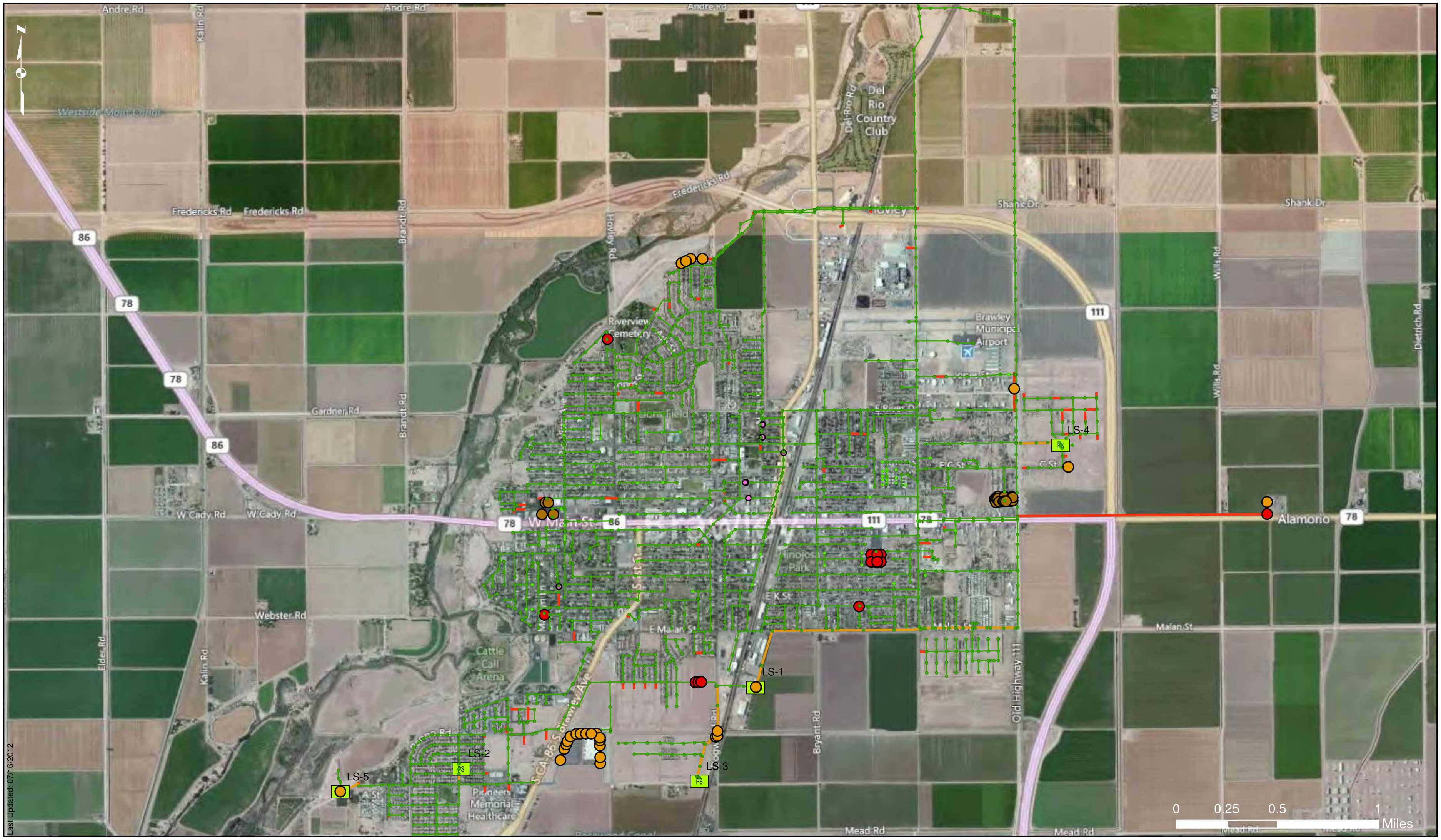
- **Gravity Mains** – Orphan gravity mains (not connected via a manhole, fitting, or cleanout) were not modeled (1,454 of 1,595 modeled)
- **Force Mains** – Orphan force mains (not connected to a manhole or cleanout) were not modeled (19 of 20 force mains modeled)
- **Lateral Lines** – Orphan lateral lines (not connected via a manhole, catch basin or fitting) and lateral lines with fittings at the end were not modeled (105 of 120 laterals modeled)

Missing data that is required for a running model was populated using other data sources that included a local contour layer (X-TOPO-BMP.dwg), USGS National Elevation Dataset (NED), and data. Missing upstream/downstream pipe diameters were assumed, based on the diameters of adjacent pipes. Missing invert elevations were interpolated using the nearest upstream and downstream invert elevation data available.

In addition to the assumptions made for missing data, there were several changes made to the existing data because of what appeared to be invalid manhole depths. Tracked within the GIS are the features where missing data was assumed or populated from another source, as well as original information from the AutoCAD file that was edited.

The following additional tasks were completed to prepare the geodatabase for development of the hydraulic model:

- Populated unique Facility IDs for all modeled network features.
- Established pipe connectivity with upstream and downstream nodes.
- Verified and corrected pipe connectivity and direction throughout the system.
- Inserted fittings and manholes where pipes were not split.
- Assigned system attribute (wastewater or combined) to all features based on combined system boundary.
- Assigned conditions attribute (existing or future) to all features.



Legend

Gravity Mains	Fittings	LiftStations
Included in Model	Included in Model	Forcemains
Not in Model	Not in Model	

Cleanouts	Manholes
Included in Model	Included in Model
Not in Model	Not in Model

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Figure 4-1

Wastewater GIS Features Not Included in Hydraulic Model

City of Brawley Integrated Master Plan

April 2013

4.3 System Facilities

The City's wastewater model was developed for both existing and future conditions. Existing system facilities were added to the model from the GIS geodatabase that had been created to include facilities and attribute data necessary for the hydraulic model. There were manholes that were surveyed as part of the 1999 Master Plan project as well as during this project, which were added to the GIS. However, information for all manholes and other facilities within the existing wastewater system is not available. It is estimated that approximately 90 to 95 percent of the system is being modeled.

While recent projects within the City's service area are constructed as separate wastewater and stormwater facilities, the oldest areas of the City's existing system are combined sewer and stormwater. The existing system model was developed with both the combined and separated wastewater facilities. As described previously, there was insufficient information for the stormwater system facilities from which to develop a model. The existing pipe lengths by diameter and material that are modeled are summarized in **Table 4-2** for gravity mains and in **Table 4-3** for forcemains.

Table 4-2
Summary of Gravity Pipe Lengths by Diameter & Material

Pipe Diameter (in)	Length (ft) by Material					Total Length by Diameter (ft)
	PVC	DIP	VCP	RCP	Unknown	
4	280	NA	NA	NA	NA	280
6	2,649	NA	3,320	NA	3,168	9,137
8	84,809	3,923	93,443	NA	67,440	249,615
10	2,641	636	20,816	NA	2,324	26,417
12	3,049	NA	13,511	NA	11,366	27,926
14	NA	NA	NA	NA	528	528
15	10,809	NA	4,062	NA	9,352	24,223
18	5,348	NA	7,883	NA	19,047	32,278
21	NA	NA	NA	NA	751	751
24	NA	NA	3,947	1,477	30,731	36,154
30	NA	NA	NA	NA	200	200
36	NA	NA	240	NA	NA	240
unknown	NA	NA	NA	NA	425	425
Total Pipe Length by Material	**109,585**	**4,559**	**147,222**	**1,477**	**145,332**	**408,174**

Table 4-3
Summary of Forcemain Pipe Lengths by Diameter & Material

Pipe Diameter (in)	Length (ft) by Material		Total Length by Diameter (ft)
	PVC	Unknown	
6	3,065	NA	3,065
10	NA	7,990	7,990
Total Pipe Length by Material	**3,065**	**7,990**	**11,055**

Other facilities included in the model are the lift stations and the system outlet. There are three (3) existing lift stations that are modeled, which are summarized in **Table 4-4**. The future lift station data is provided in Section 4.0. All the flows conveyed by the existing collection system travel generally in a northern direction to the City's wastewater treatment plant (WWTP), which is located at 5015 Best Rd. The most downstream model manhole is defined as the system's outlet (or outfall) representative of this location.

Table 4-4
Summary of Existing Sewer Lift Stations

Lift Station / Pump	Variable or Constant Speed	Ground Elevation (ft)	Wet Well Bottom Elevation (ft)	Design Flow (gpm)
South Brawley (9th Street) Sewer Lift Station No. 1 (Existing)				
Pump No. 1	Constant	-105.5	-126.0	800
Pump No. 2	Constant	-105.5	-126.0	800
Citrus View Lift Station No. 2 (Existing)				
Pump No. 1	Constant	-98.8	-113.8	200
Pump No. 2	Constant	-98.8	-113.8	200
Latigo Ranch Lift Station No. 3 (Existing)				
Pump No. 1	Constant	-108.0	-134.5	320
Pump No. 2	Constant	-108.0	-134.5	320

4.4 Model Construction

The model was built from the GIS geodatabase using Innovyze InfoSewer 7.5, Update No. 4, modeling software. The existing system model includes the combined sewer and stormwater system. As part of the model construction process, data fields from the geodatabase were mapped to specific model attribute fields.

Once these attributes had been imported from GIS, a number of standard attributes were assigned to the model elements. These attributes are required by InfoSewer, but were not part of the data imported from the GIS. These model-specific attributes are summarized below in **Table 4-5**.

Table 4-5
Model-Specific Attributes not Imported from GIS

Model Field Name	Description of Model Field	Model Element
TYPE	Default type of Manhole = Manhole, Manually assign chambers and outlet manhole types (see below)	Manholes
DIAMETER	Set all to 4 ft	Manholes
TYPE	Set all to Type: 0 (Cylindrical)	Wet Wells
MIN_LEVEL	Set all to 1 ft	Wet Wells
MAX_LEVEL	Set all to Inlet Elevation - 1 ft	Wet Wells
INIT_LEVEL	Set all to 4 ft	Wet Wells
DIAMETER	Calculated from capacity and height	Wet Wells
TYPE	Set to fixed capacity	Pumps
ID	Set to P_[Lift Station Number]	Pumps
LENGTH	Auto-calculated pipe length	Pipes
COEFF	Gravity (Manning's numbers – see Table 3)	Pipes (Type: Gravity)
COEFF	Force mains set to C-factor of 100	Pipes (Type: Force main)
TYPE	Set to gravity or forcemain	Pipes
GIS_SOURCE	Set to name of GIS feature class	All Elements
IN_MODEL	Set to Yes (Y)	All Elements
IN_GIS	Set to Yes (Y)	All Elements

Lift stations were created by importing the lift station feature from GIS as the model wet well. Next, pumps were added at each lift station, starting at the wet well and ending at the first downstream manhole. The manhole type for these downstream manholes was set to 'Chamber', as required by InfoSewer. Two parallel pumps were created at each lift station using the attributes listed in **Table 4-4**.

The manhole Type of the most downstream manhole (CA001_00) in the collection system was defined as "2: Outlet", to designate this as the system outlet.

Once the GIS topology and attributes had been imported to InfoSewer and the lift stations had been developed, as described above, a number of connectivity checks were performed using InfoSewer native tools. These connectivity checks included:

- Orphan Nodes – Finds manholes not connected to a pipe
- Orphan Pipes – Finds pipes with no upstream or downstream manhole specified
- Trace Upstream Network – Starting at the outlet, traces all upstream pipes to confirm the flow direction of each pipe
- Nodes in Close Proximity – Identifies potential duplicate manholes
- Pipe Split Candidates – Identifies pipes that are near to, but do not split a pipe, to help confirm if a pipe should be split
- Crossing/Intersecting Pipes – Identifies pipes that cross but do not split at the crossing, to help confirm that pipes at a crossing should not be split
- Parallel Pipe – Identifies pipes with the same upstream/downstream manholes, to determine if there is a duplicate pipe that should be deleted.

All connectivity issues identified during this step were resolved prior to proceeding with the next model development steps.

Manning's Equation 'n' values were assigned to all gravity pipes as described below in **Table 3-15**.

A Hazen-Williams C-factor of 100 was assigned to all force main pipes.

4.5 Node Elevations and Coordinate System

The elevations in the CAD base mapping file and in other sources are not real-world elevations, but were adjusted by adding 1,000 ft to make them positive, since all the elevations within the system are negative. The GIS and model node elevations were converted to real-world elevations by subtracting 1,000 ft. These elevations are consistent with the elevations in the water distribution system hydraulic model.

The coordinate system used in the GIS geodatabase and the model is NAD 1983 State Plan Calibration VI FIPS 0406 Feet. This is the same coordinate system used in the water system model.

4.6 Flow Loading

Wastewater flow loading to the sewer model was separated into dry and wet weather flows. The dry weather flow was further separated into sanitary sewer flows and dry weather (base) infiltration. The wet weather flow is comprised of the dry weather flow and rainfall induced inflow and infiltration (I&I). The Joint WEF Manual of Practice FD2 – ASCE Manual and Report on Engineering Practice No. 62 defines infiltration as water entering a sewer system from the ground through defective pipes, pipe joints, damaged lateral connections or manhole walls. Inflow is extraneous storm water that enters a sanitary sewer system through roof leaders, cleanouts, foundation drains sump pumps and cellar, yard and area drains. Development and allocation to the model of the dry and wet weather flows is described in the following sections.

4.6.1 Existing Dry Weather Flows

Dry weather flows are divided into two components: dry weather infiltration (base infiltration) and domestic sewer flows. The dry weather infiltration was developed by analyzing the flow monitor data, calculating 75 percent of the lowest dry weather flow, and applying that calculated value evenly across the upstream manholes. The assumed dry weather infiltration as a percent of the minimum dry weather flow is an empirical number based on other similar wastewater studies and similar approaches used in the energy industry.

The dry weather domestic flows for the model were developed from land use based domestic sewage flow factors and parcel area throughout the majority of the collection system. Specific sewer flows for certain customers that used an alternative flow estimate method were applied as point loads at the actual location of those customers. These are described in greater detail later in this section.

The City of Brawley land use parcel shapefile was used as the starting point for the average dry weather loading. This is the same land use parcel shapefile that was used for allocating demand to the water distribution model. It should be noted that this shapefile was modified to remove non-developed parcels prior to developing existing system flows for the sewer model. The dry weather domestic sewer flow rates used for initial model loading under existing development conditions are listed below in **Table 4-6**.

The dry weather sewer flow for each developed parcel was calculated by multiplying the parcel area by the appropriate loading rate based on the land use of the parcel. The sewer flows by parcel were then spatially joined in GIS from the centroid of the parcel to the nearest model manhole. The dry weather sanitary loads were adjusted as part of the calibration process, which is discussed in Section 4.9.

Table 4-6
Existing Dry Weather Sewer Flow Rates

Land Use	Model Loading Rate (gpad)
Low Density Residential	1,100
Medium Density Residential	2,400
Commercial	700
Public Facility	600
Industrial	700
Light Industrial	700

The next step was to assign special point loads to the model for large users and schools. These included the water treatment plant, the National Beef Packing Company, and the six (6) largest schools and are summarized in **Table 4-7**. These point loads were added to the Load4 field in the manhole table of the model.

Table 4-7
Special Existing System Dry Weather Sanitary Flow Loads

Description	Model Manhole ID	Sewer Loading (gpd)
Water Treatment Plant		60
National Beef Packing Company	WA004_00	16,379
Brawley High School	CD025_10	17,063
Miguel Hidalgo Elementary	CC023_01	6,825
Oakley Elementary	CB010_00	7,313
Witter Elementary	CE008_11	7,020
Phil Swing Elementary	FIT-8	8,288
Barbara Worth Junior High	CD010_11	8,288

The following peaking equation was used to develop the peak dry weather flow scenario. The peaking equation was only applied to the sanitary sewer loads for each customer (not the base infiltration).

$$Q_{pdwf} = 2.64 (Q_{adwf})^{0.905}$$

4.6.2 Future Dry Weather Flows

The same methodology was used to develop the future dry weather flows as was used for the existing dry weather flows. The base infiltration, which was applied to the Load2 field in the model, remained the same for the future flow scenarios. The sanitary sewer flows were calculated by multiplying the parcel area by the future land use unit flows, which are summarized in **Table 4-8**. The centroid of all parcels that are located within the existing collection system were associated with the nearest existing manhole using a GIS process of spatial association. The sanitary flows for the future undeveloped parcels that are outside of the existing system service are boundary were assigned to the nearest existing system manhole.

Table 4-8
Future Dry Weather Sewer Flow Rates

Land Use	Model Loading Rate (gpad)
Low Density Residential	1,100
Medium Density Residential	2,400
Commercial	1,300
Public Facility	1,100
Industrial	1,500
Light Industrial	1,300

Specific point loads for the existing and future schools, water treatment plant, and National Beef plant were applied at specific customer locations similar to the existing system point loads. These special point loads for the future system are summarized in **Table 4-9**. The existing system point loads for existing schools, the National Beef plant, and the water treatment plant are the same as for the existing system flow loading. Additional schools for the future system are included and are the same for years 2020 and 2030, with the exception of the future junior high school, which is only included in the year 2030 collection system.

Table 4-9
Special Future System Dry Weather Sanitary Flow Loads

Description	Model Manhole ID	Sewer Loading (gpd)
Water Treatment Plant	WE007_22	60
National Beef Packing Company	CA024_00	16,379
Brawley High School	CD025_10	17,063
Miguel Hidalgo Elementary	CC023_01	6,825
Oakley Elementary	CB010_00	7,313
Witter Elementary	CE008_11	7,020
Phil Swing Elementary	FIT-8	8,288
Barbara Worth Junior High	CD010_11	8,288
1 (Elementary School)	WC044_29	6,908
2 (Elementary School)	EB012_00	12,793
3 (Elementary School)	ED024_07	15,169
4 (Elementary School)	10	6,809
5 (Junior High School)	10	16,918

4.6.3 Existing and Future Wet Weather Flows

The I&I component of the wet weather flow loading was developed based on an estimated total sewer system load of 1.5 mgd for a typical 1 inch rainfall event within a 24-hour time period. This estimate of I&I was developed based on a review of the WWTP flow data. The rainfall induced I&I loading was distributed throughout the collection system in the following manner:

- Combined System – 85 percent (1.275 mgd) of the wet weather load was applied to the combined system.
    - There are a total of 375 catch basins identified in the geodatabase. Further, there are 21 manholes located upstream of Flow Monitor 1 (FM1), which showed a large wet weather event response during the flow monitoring period, even though there were no catch basins identified within this contributing area. The nearest manholes to the 375 catch basins and the manholes upstream of Flow Monitor 1 were used as the flow loading points for the combined system.

- The wet weather flow loading for the combined area of 1.275 mgd was distributed evenly across the 375 catch basins and FM1 manholes. The sewer loading from the catch basins was allocated to the nearest associated manholes. In locations where catch basin laterals were known, the catch basin load was applied to its corresponding manhole. In locations with no catch basin laterals defined, load from each catch basin was applied to the nearest combined system manhole.

- Sanitary System – the remaining 15 percent of the wet weather I&I load (0.225 mgd) was evenly applied to the manholes in the separated wastewater collection system.

All I&I wet weather loads were allocated to the Manhole Attributes field, Load3 to keep the wet weather load separate from the dry weather domestic load and base infiltration. Once the model had been loaded for wet weather conditions, the loading in the model database was checked to confirm that the total wet weather I&I load equaled 1.5 mgd. The same wet weather induced I&I applied for the existing system wet weather scenario was used for all of the future system wet weather scenarios.

4.7 Scenario and Dataset Development

The InfoSewer model was developed with multiple scenarios with several unique datasets. Different scenarios were used to differentiate between dry and wet weather flows, average, and peak flows, as well as existing and future development conditions. These scenarios and their associated data sets are listed below in **Table 4-10**.

Table 4-10
Sewer Model Scenarios

Scenario Name	Scenario Description	Unique Data Set
EX_ADWF	Existing Average Dry Weather Development	Manhole Set (ADWF)
EX_PDWF	Existing Peak Dry Weather Development	Manhole Set (PDWF)
WWF_CALIB	Existing Average Wet Weather Development	Manhole Set (WWF)
EX_PWWF	Existing Peak Wet Weather Development	Manhole Set (PWWF)
2020_ADWF	Year 2020 Average Dry Weather Development	Manhole Set (2020_ADWF)
2020_PDWF	Year 2020 Peak Dry Weather Development	Manhole Set (2020_PDWF)
2020_PWWF	Year 2020 Peak Wet Weather Development	Manhole Set (2020_PWWF)
2030_ADWF	Year 2030 Average Dry Weather Development	Manhole Set (2030_ADWF)

Scenario Name	Scenario Description	Unique Data Set
2030_PDWF	Year 2030 Peak Dry Weather Development	Manhole Set (2030_PDWF)
2030_PWWF	Year 2030 Peak Wet Weather Development	Manhole Set (2030_PWWF)
2030_CIP	Recommended improvements sized for build-out future development conditions	Manhole Set (2030_PWWF) Pipe Set (CIP_PIPES)

Notes
(1) Scenarios shown in bold are the InfoSewer scenarios exported to EPA-SWMM.

The year 2020 and 2030 system includes future pipes that are not included in the existing system, and are located within the Luckey Ranch area along the eastern portion of the system.

4.8 Model Calibration

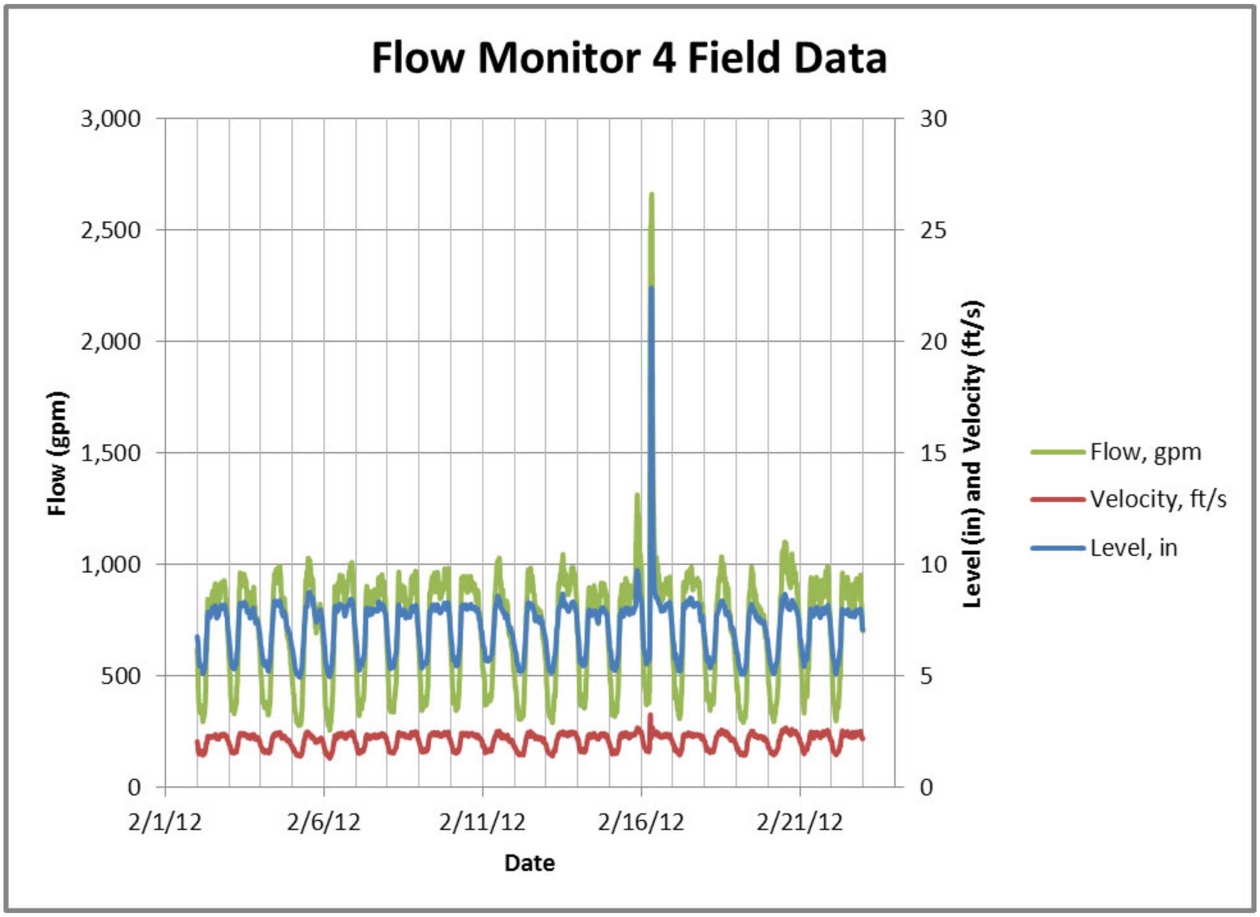
Calibration of the sewer hydraulic model was completed for steady state with existing wastewater flow conditions. Calibration of the hydraulic model compares the model results to flow monitoring and wastewater treatment plant data to ensure the model is representing the actual system with a reliable degree of accuracy. Calibration was first completed for dry weather flows and then for rainfall induced inflow and infiltration (I&I).

Five (5) locations were selected for installation of temporary flow monitors. The locations of the five (5) flow monitors and the contributing service area for each are shown in **Figure 3-5**. Flow, depth, and velocity data were gathered between February 2, 2012 and February 22, 2012 at each of these locations. The total contributing flow from these five locations represents approximately half of the total flow recorded at the WWTP.

4.9 Dry Weather Calibration

The data from the temporary flow monitors was analyzed to identify a typical dry weather period to use as the basis for dry weather calibration. **Figure 4-2,** below, presents an example of a graph prepared from the flow monitor data. This graph was used to verify that there were no obvious gaps or other anomalies in the data, and to select the typical dry weather period.

Figure 4-2 – Example Graph Prepared from Temporary Flow Monitor Data



Based on the graph above, the system shows a clear response to a rainfall event on February 16, 2012. In order to ensure a time period when there was no rainfall influences, an average dry weather flow was calculated using data from February 2nd through February 14th for each temporary flow monitor. Although monitoring data was not available prior to February 2nd, the nearest rainfall gauge data indicates no rainfall events for at least two weeks prior to this time period. These average dry weather flows were then established as the dry weather calibration target for the model flows at each of the five temporary flow monitor locations.

The dry weather flows developed and loaded to the model as described in Section 2.5.1, were compared to the average dry weather flows calculated from the flow monitoring data. Adjustments were made to the model sanitary loading of each area tributary to the flow monitors for closer consistency with the field data. These adjustments were made by uniformly scaling the load at all manholes within each area. **Table 4-11** presents the results of dry weather steady state calibration.

The contributing flow from the monitored areas is approximately half of the total flow recorded at the WWTP. The monitored and unmonitored areas also consist of both combined and separated portions of the wastewater system.

Table 4-11
Dry Weather Calibration Results and Total Dry Weather Flows

Flow Monitoring Location	Dry Weather Flow Field Data (gpm)	Dry Weather Flow Model Results (gpm)	Percent Difference
1	35	39	10%
2	80	76	-5%
3	182	191	5%
4	722	815	13%
5	289	298	3%
Unmonitored Area	NA[2]	1,081	NA
WWTP [1] (Total Dry Weather Flow)	2,500	2,494	0%

Notes
(1) Average dry weather flow for the inlet to the wastewater treatment plant was determined based on historical WWTP records provided by the City.
(2) Average dry weather flow at the treatment plant shown in Table 7 does not correspond to the days for which the flow monitoring data within the collection system was collected.

Flows recorded at Flow Monitoring Location 1 are the smallest flows of any of the locations where field data was collected. Therefore, differences between the monitored flows recorded and the model flows will result in a larger percent difference while the actual flows are relatively close.

The contributing area to flow monitor 4 is the largest area of any of the five contributing areas to the flow monitors. There is a relatively good confidence in the land use based sanitary loads and since the model load is somewhat more conservative than the average field flow, no further adjustments were made.

4.10 Wet Weather Calibration

Similar to the dry weather calibration, flow data from the field flow monitoring as well as treatment plant data was used for the wet weather calibration. During the flow monitoring conducted at the beginning of 2012, two rainfall events occurred within a 24-hour period from February 15th to the 16th. It was considered a typical storm for Brawley of 1 inch rainfall intensity for that entire time period, based on review of available historical data. Historical flow data recorded at the inlet of the wastewater treatment plant was also reviewed and found that an average daily flow of 1.5 mgd is typical for wet weather inflow and infiltration for a 24-hour, 1 inch rainfall event.

The response time of inflow and infiltration can vary significantly throughout a service area, both in terms of when it occurs relative to the rainfall event, as well as in the duration of the response. Other factors, such as variations in rainfall intensity across the service area and different characteristics of the drainage basins and facilities and potential errors or inaccuracies of the monitoring equipment, can also have an impact on the flow recorded at each flow monitoring location. For these reasons, it is ideal to have multiple rainfall events from which flow data can be reviewed. However, due to the limitations of the available data, the rainfall on February 15th and 16th was the only rainfall data used for the wet weather calibration. There were two rainfall events within a 24-hour period during these two days, but the time period when the I&I response was recorded varied at each location. A total volume at each flow monitoring location within the 24-hour period was used to determine the relative percent distribution of the total flow recorded at all five flow monitoring sites. The relative percent of the total monitored wet weather flow was then used to adjust the I&I assigned within each of the contributing flow areas.

Table 4-12 summarizes the calibrated rainfall induced I&I loaded to the model, which together with the dry weather flow, is the total wet weather flow. Also included in the table is the dry weather to wet weather peaking factor for flows recorded at each of the flow monitoring locations, as well as within the unmonitored area. As seen in **Table 4-12**, there is a significant difference in the peaking factors for different areas within the City's collection system. These differences cannot be attributed solely to stormwater entering the collection system, since some areas with high peaking factors include portions of the separated system. As mentioned previously, the flows are based on a limited amount of flow data and there may be inaccuracies in the flow recording equipment. However, the high peaking factors may identify areas with higher levels of I&I and therefore would be a higher priority for rehabilitation.

Table 4-12
Wet Weather Calibrated Flows & Peaking Factors

Contributing Flow Area	I&I Loading to the Model[1] (gpm)	Average Dry to Wet Weather Peaking Factor	Percent of the Total I&I
Flow Monitor 1	121	5.40	12%
Flow Monitor 2	7.87	1.09	0.76%
Flow Monitor 3	47.4	1.21	4.6%
Flow Monitor 4	524	2.35	50%
Flow Monitor 5	44.9	1.34	4.3%
Unmonitored Area	292	0.27	28%
WWTP (Total I&I)	1,038		

Note
(1) The wet weather loading presented in this table is only the rainfall induced inflow and infiltration portion of the wet weather flow and does not include the sanitary sewer flows or base infiltration.

4.11 Future System Wastewater Model Development

As described previously in this technical memorandum and summarized in **Table 4-10**, there were seven (7) future scenarios developed based on different future flows for dry and wet weather conditions. The flows for future developments were described in Sections 2.5.2 and 2.5.3. With the exception of a portion of the Luckey Ranch development, which has already been constructed, all future developments were modeled by applying the future flow loading to the nearest existing model node. It is assumed that as these developments get to the planning and design phase, refined flows based on the level and type of development will be used to size the required connecting trunk pipelines.

The three (3) existing system lift stations were modeled with no changes for the future development conditions. Two (2) additional future lift stations in Luckey Ranch and in La Paloma were modeled for the future development scenarios. There are some existing and planned private future lift stations serving small developments that were not included in the wastewater model. **Table 4-13** presents the data for the existing and future lift stations that are modeled.

Table 4-13
Summary of Sewer Lift Stations Modeled for the Future System

Lift Station / Pump	Variable or Constant Speed	Ground Elevation (ft)	Floor Elevation (ft)	Design Flow (gpm)
South Brawley (9th Street) Sewer Lift Station No. 1 (Existing)				
Pump No. 1	Constant	-105.5	-126.0	800
Pump No. 2	Constant	-105.5	-126.0	800
Citrus View Lift Station No. 2 (Existing)				
Pump No. 1	Constant	-98.8	-113.8	200
Pump No. 2	Constant	-98.8	-113.8	200
Latigo Ranch Lift Station No. 3 (Existing)				
Pump No. 1	Constant	-108.0	-134.5	320
Pump No. 2	Constant	-108.0	-134.5	320
Luckey Ranch Lift Station No. 4 (Future)				
Pump No. 1	Constant	-130.5	-152.5	750
Pump No. 2	Constant	-130.5	-152.5	750
La Paloma Lift Station No. 5 (Future)				
Pump No. 1	Constant	-90.3	-110.9	350
Pump No. 2	Constant	-90.3	-110.9	350

An additional discussion of the future system analyses is included Section 5.0.

5.0 Collection System Analysis and Recommended Improvements

5.1 Introduction

This technical memorandum describes the analysis results of the City's wastewater system that includes both the combined and separated portions of the collection system. This was completed as part of the 2012 Water Master Plan update (Project). The existing and future system model was developed from available GIS, field data, and information provided by City staff.

Section 5.2 describes the existing system analysis results and Section 5.3 describes the future system analysis results. Recommended improvements that were analyzed with the wastewater model are described in Section 5.4.

5.2 Existing System Analysis Results

The following section describes the analysis results for the existing system. The peak dry weather and peak wet weather scenarios were used for the analyses to identify deficiencies within the existing collection system. The criteria used to identify deficiencies within the system were pipes that have a d/D equal to 1.00 and the level of water in manholes where surcharging occurred.

5.2.1 Existing Dry Weather Results

As described in chapter 4, the average dry weather flows were calculated based on land-use based unit flows and parcel area. These were applied to the nearest manhole and system-wide calibration was completed using the field flow monitoring data. The peak dry weather flows were developed using the peaking equation presented in chapter 4.

Based on the deficiency criteria there are 67 pipes identified with a d/D equal to 1.00 and 115 manholes within two feet of the ground surface. Four of the manholes are at the ground surface (overflowing) and all four are near N. Rio Vista Drive and River Drive where there is a significant amount of surcharging due to downstream capacity issues to the north. **See Appendix A** for detailed pipe profile and other information at the locations where deficiencies have been identified for both peak dry and wet weather analyses.

5.2.2 Existing Wet Weather Results

The existing peak wet weather scenario combined the peak dry weather flow with the calibrated inflow and infiltration. There are a total of 94 pipes with a d/D equal to 1.00, 41 manholes within two feet of the ground surface, and 17 that have flow at the rim of the manhole. **Figure 5-1** shows the pipes within the collection system with a d/D equal to 1.00 and **Figure 5-2** shows the manholes that are within two feet of the ground surface or overflowing. **Appendix A** provides detailed pipe profile and other information at the locations where deficiencies have been identified.

The majority of the pipes with a d/D equal to 1.00 (89 percent) have a negative or flat slope, which is the primary reason, or at least a contributing factor, for these pipes flowing full in the model. As shown in **Figure 5-1** these pipes are located throughout the collection system. The pipes identified on **Figure 5-1** do not include pipes experiencing backwater condition due to a downstream capacity issue. The location showing the most segments of pipe with d/D equal to 1.00 are the major trunks and interceptors along the western part of the system flowing towards the treatment plant.

The primary location showing manholes with the HGL at the rim of the manhole or within two feet of the rim are in the northwest portion of the system near N. Rio Vista Avenue and River Road.

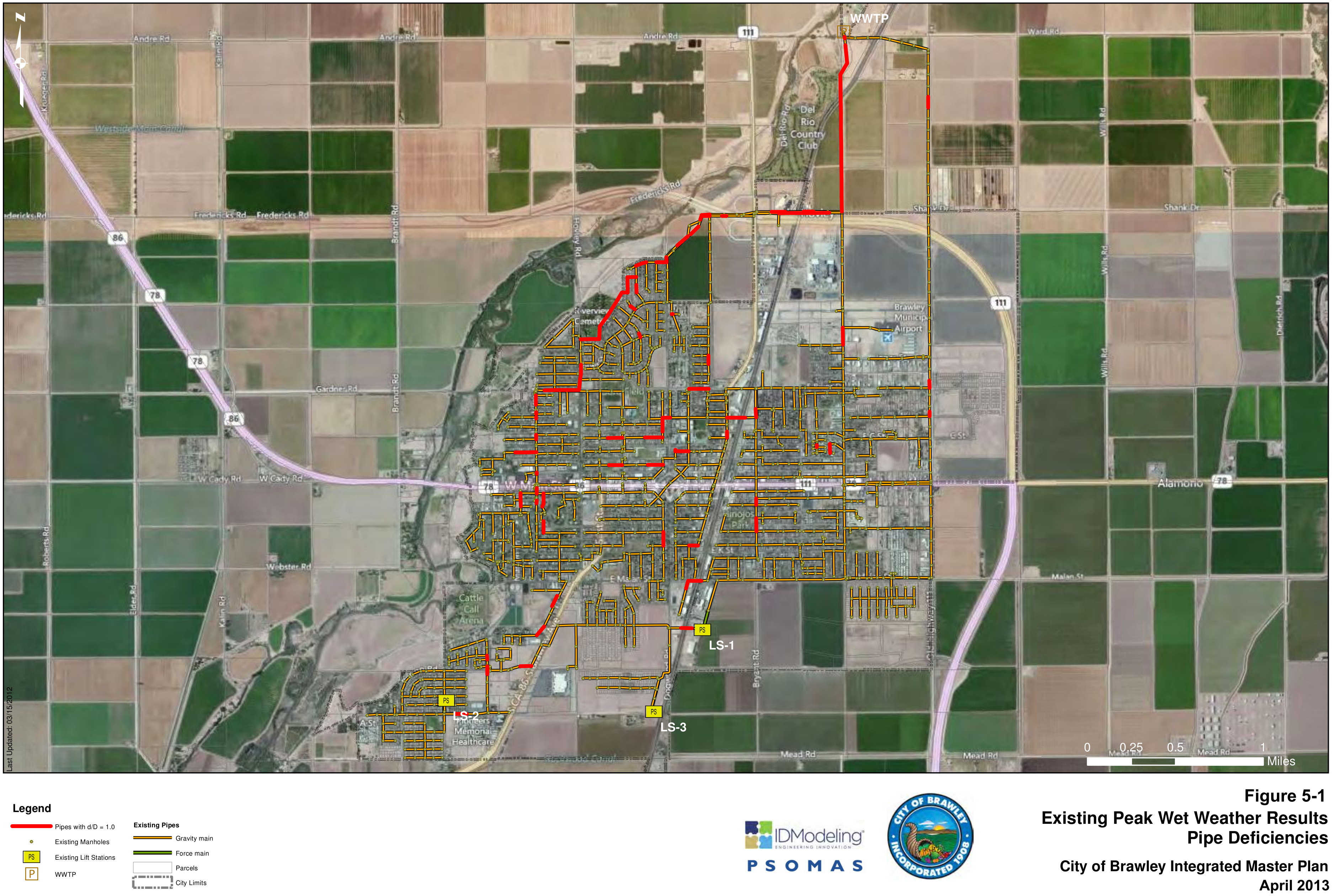
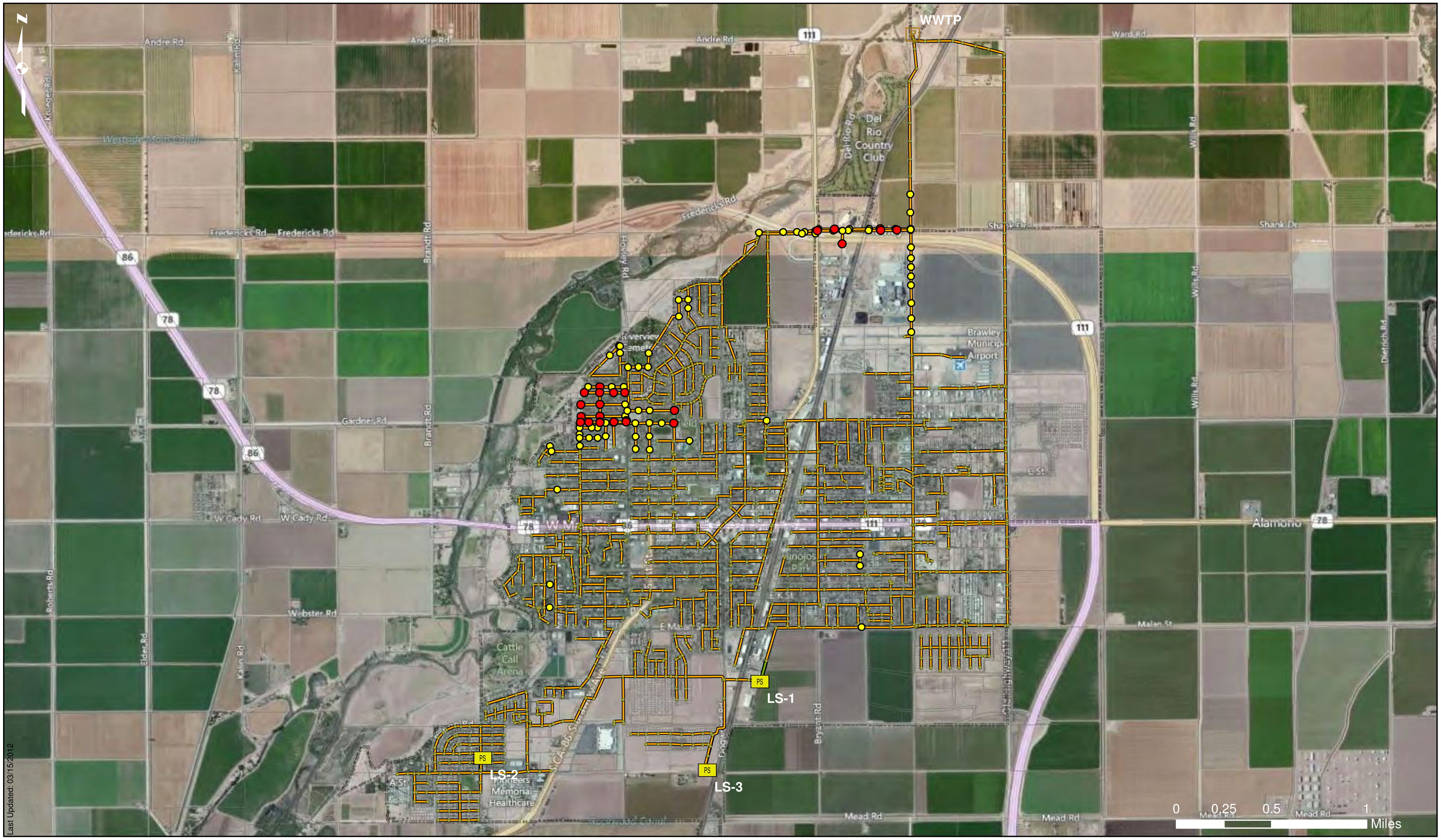


Figure 5-1
Existing Peak Wet Weather Results
Pipe Deficiencies

City of Brawley Integrated Master Plan
April 2013



Legend

- Manholes overflowing
- Manholes within 2 ft of ground surface
- Existing Manholes
- Existing Lift Stations
- WWTP

Existing Pipes

- Gravity main
- Force main
- Parcels
- City Limits

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Figure 5-2

Existing Peak Wet Weather Results
Manholes Nearly Full or Overflowing

City of Brawley Integrated Master Plan
April 2013

5.3 Future System Analysis Results

Model scenarios were analyzed for peak wet weather flows for year 2020 and year 2030 in order to identify potential deficiencies based on the projected type and level of development. As described in the model development section, the only future area that included modeled pipes and manholes is the Luckey Ranch development, since the piping has already been constructed. The other future development parcels that have no existing infrastructure associated with them were modeled by assigning the sanitary sewer loads to the nearest existing manhole to each parcel. The two future lift stations, Luckey Ranch and La Paloma, were both included in the future model scenarios.

The same criteria used to identify deficiencies for the existing system were also-used for the year 2020 and year 2030 analyses. Pipes with d/D equal to 1.00, manholes that are full (flow reaching the rim of the manhole and therefore likely an overflow situation), and manholes that have flow within two feet of the manhole rim are all highlighted as locations with potential system deficiencies.

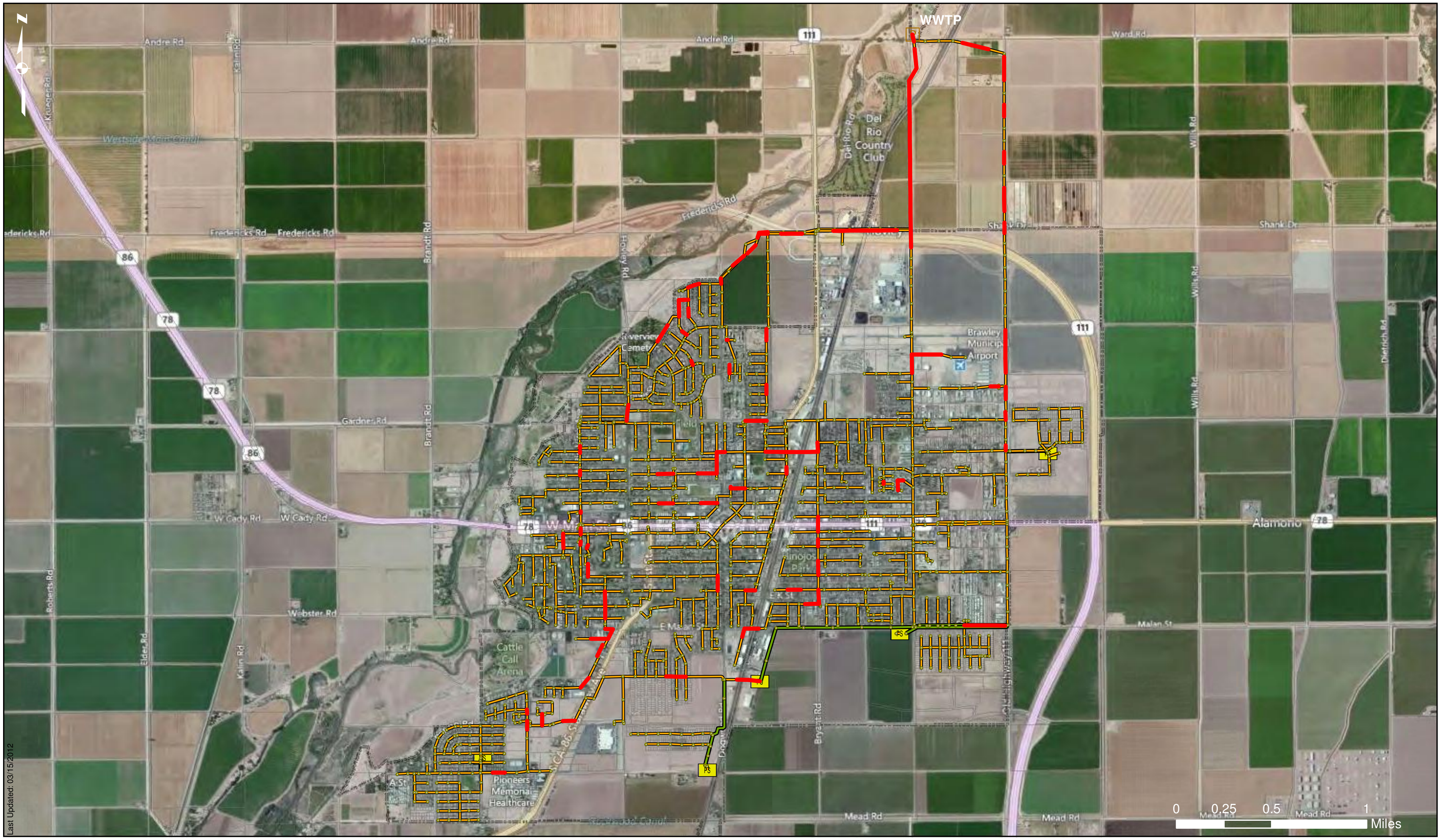
5.3.1 Year 2020 Peak Wet Weather Results

The year 2020 peak wet weather model scenario identified 122 pipes with a d/D equal to 1.00, 35 percent of which has a slope of zero or is negative. There are four manholes that are identified as full and 41 manholes with flow that is within two feet of the rim of the manhole. It is important to note that the number of manholes that are full are less for the year 2020 (and year 2030 scenarios) than for the existing peak wet weather scenario. And, although there is the same number of manholes for existing conditions and year 2020, conditions that have flow within two feet of the rim of the manholes, they are different manholes for the existing and future conditions. **Figures 5-3** and **5-4** show the deficient pipes and manholes, respectively.

The existing conditions analysis showed more deficiencies than the future conditions near Rio Vista Avenue and River Road because of higher flows throughout that area and through the far western trunk of the collection system for the existing conditions. Both the existing and future scenarios used the same methodology for initial sanitary flow allocation based on land use flow factors and the land use of each parcel. Adjustments were made to the initial flow loading for the existing scenario as part of the calibration process, so that model flows were more consistent with the field-monitored flows and total flows at the treatment plant. Flow factors for non-residential customers used for the future scenarios were higher than those used for the existing flow scenarios.

The field recorded flows within Flow Monitoring Area #1 were much greater than anticipated. This area is supposed to be part of the separated system, yet the flow monitoring data showed a strong response to rainfall events.

Additionally, the dry weather flows recorded in the field were much higher than the initial flow estimates in the model. This could be due to higher sewer flows from a particular customer or group of customers, but the City may also want to investigate potential undocumented sources of flow that are resulting in higher than expected dry and wet weather flows from the western part of the collection system. Additional flow monitoring for a longer period of time and at a greater number of locations throughout the western portion of the collection system would also be very helpful in better characterizing the flows in this area.



Legend

- Pipes with d/D = 1.0
- Manholes
- Lift Stations
- WWTP

Pipes

- Gravity main
- Force main
- Parcels
- City Limits

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Figure 5-3

Year 2020 Peak Wet Weather Results

Pipe Deficiencies

City of Brawley Integrated Master Plan

April 2013

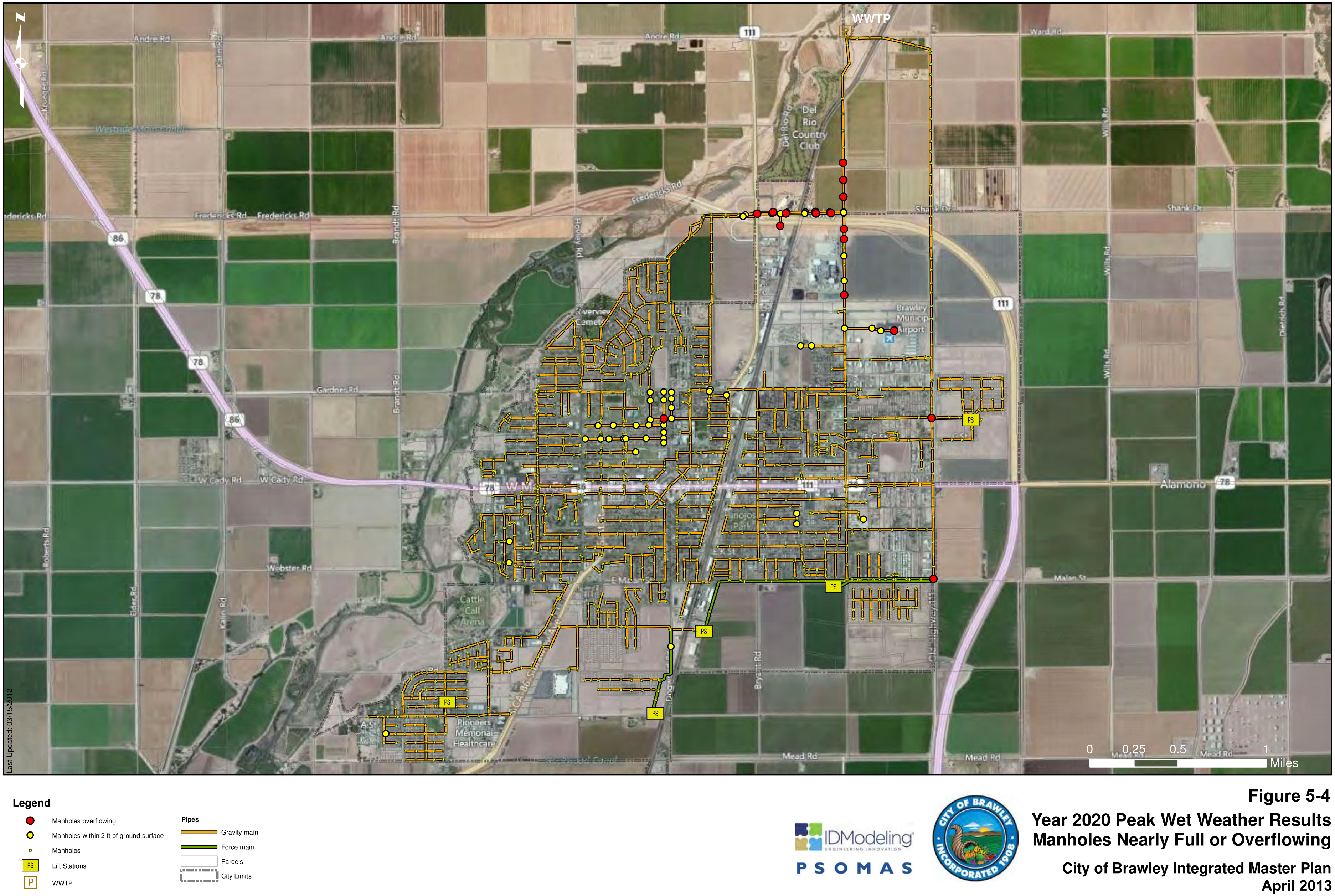

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Figure 5-4

Year 2020 Peak Wet Weather Results
Manholes Nearly Full or Overflowing

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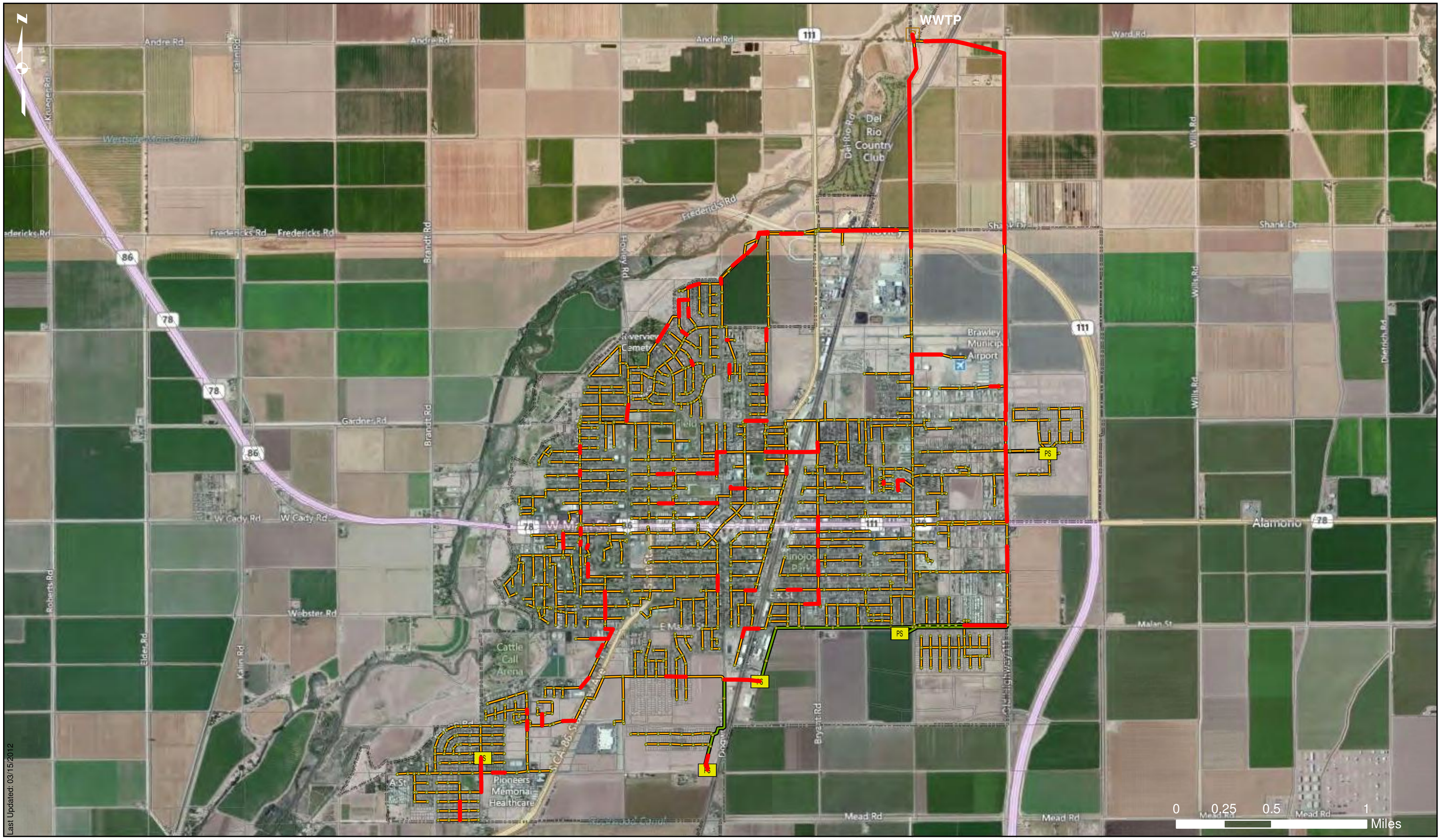
5.3.2 *Year 2030 Peak Wet Weather Results*

The year 2030 results were very similar to the year 2020 results. The conditions for year 2030 assumes sewer flows based on a fully built-out service area. The same areas identified as problem areas for year 2020 conditions were also identified as deficiency areas for year 2030, with several additional locations. There were 165 pipes identified with a d/D equal to 1.00, 26 percent of which have a slope of zero or are negative. These pipes are shown in **Figure 5-5**.

There are 48 manholes that are identified as full for build-out conditions and 154 manholes that have flows within 2 feet of the manholes. There are a much greater number of manholes with flow rising near or to the ground surface for year 2030 conditions compared to year 2020. These are mostly located along the eastern and southern parts of the collection system and are shown in **Figure 5-6**.



Legend

Pipes with d/D = 1.0
Manholes
PS Lift Stations
P WWTP

Pipes

Gravity main
Force main
Parcels
City Limits

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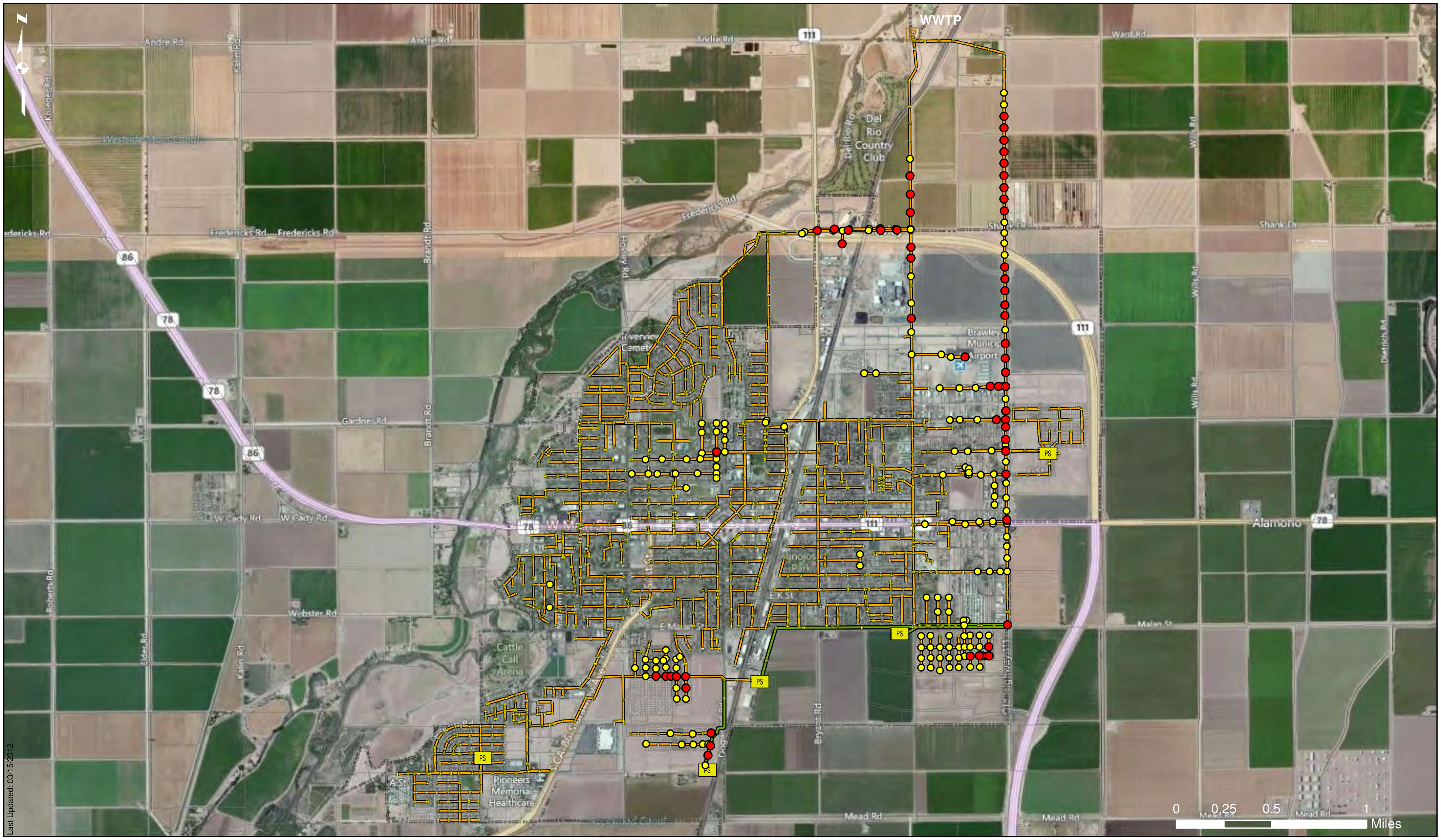
Figure 5-5
Year 2030 Peak Wet Weather Results
Pipe Deficiencies

City of Brawley Integrated Master Plan
April 2013



Legend

- Manholes overflowing
- Manholes within 2 ft of ground surface
- Manholes
- PS Lift Stations
- P WWTP

Pipes

- Gravity main
- Force main
- Parcels
- City Limits

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Figure 5-6

Year 2030 Peak Wet Weather Results
Manholes Nearly Full or Overflowing

City of Brawley Integrated Master Plan
April 2013

6.0 Wastewater/Stormwater Quality

6.1 Wastewater Quality

6.1.1 Regional Water Quality Control Board Regulations

The City of Brawley falls under the jurisdiction of the California RWQCB, Colorado River Basin. The existing WWTP discharges to the New River, which ultimately drains to the Salton Sea. Beginning February 1, 2007, the City of Brawley was in violation of their NPDES Permit which had established stringent removal requirements of ammonia nitrogen. At the time, the City was operating an aerated lagoon process, which had received several upgrade attempts over the years, but was still not meeting the effluent requirements set forth in the permit. The City hired a consultant in 2008 to design an upgraded WWTP that would meet the ammonia nitrogen requirements.

6.1.2 Existing

The influent wastewater characteristics that were used for design of the current WWTP are shown in **Table 6-1**.

Table 6-1
Existing Influent Wastewater Characteristics

Parameter	Average	Monthly Peak
Biochemical Oxygen Demand BOD_5 (mg/l)	137	175
Total Suspended Solids TSS (mg/l)	148	190
Total Kjeldahl Nitrogen TKN-N (mg/l)	45	55
Organic Nitrogen Org.-N (mg/l)	15	18
Ammonia Nitrogen NH_3-N (mg/l)	30	37

The existing WWTP contains a Biolac® process, which is an activated sludge process that uses extended retention of biological solids. The submerged aeration creates a longer sludge age that reduces BOD and ammonia nitrogen levels.

6.1.3 Year 2030

By the year 2030, RWQCB requirements are expected to be much more stringent, particularly in regards to ammonia nitrogen. Prior to any WWTP upgrades, the City will need to check updated requirements to make sure they will be met with any upgrades.

6.2 Stormwater Quality

6.2.1 Regional Water Quality Control Board Regulations

The Federal Clean Water Act, Section 402, requires operators of Municipal Separate Storm Sewer Systems (MS4) in urbanized areas with populations >100,000 people to obtain NPDES Permits for stormwater discharges, along with any construction activity disturbing more than 5 acres. Phase 2 of this program expanded coverage to include small operators of MS4s in urbanized areas as delineated by the Bureau of the Census and construction activity disturbing more than 1 acre, but less than 5 acres. The City of Brawley falls under the Phase 2 category and as such, submitted a proposed Stormwater Management Plan (SWMP) to the RWQCB per the NPDES Permit application. An updated SWMP was provided in 2008.

As of November 2012, the City is preparing permit requirements that will govern all future stormwater discharges within the City limits. These requirements will set limits on the quantity and quality of stormwater discharge both pre and post construction.

6.2.2 Best Management Practices

The City's separate stormwater system carries runoff from developed areas and discharges to a receiving stream; usually the New River. Newer developments have constructed retention basins which retain a certain amount of runoff for evaporation. This runoff carries pollutants that gather on streets, parking lots, roof tops, and open space. Pollutants can range anywhere from trash, such as plastic bottles and paper, to fecal coliform bacteria from animal waste, detergents from car washing activities, oil, grease, and other fluids from leaking automobiles, fertilizers, and pesticides.

Six minimum controls with examples of appropriate BMPs are as follows:

1. Public Education and Outreach – Distribute brochures, flyers or bill inserts to educate homeowners and business operators about the problems associated with stormwater runoff and the steps they can take to reduce pollutants in stormwater discharges.
2. Public Participation/Involvement – Provide notice of stormwater management plan development and hold meetings at which citizens and business operators are encouraged to communicate ideas. Include citizen and business representatives in a Citizen's Advisory Group.
3. Illicit Discharge Detection and Elimination – Inventory and map the stormwater system and test for the possible cross connections of sanitary wastewater to the stormwater conveyance system. Modify system to eliminate illicit discharges.

4. Construction Site Runoff Control – Require the implementation of erosion and sediment controls and other waste. Review site plans and perform periodic inspections. Establish penalties for non-compliance.
5. Post-Construction Runoff Control – Require the consideration and implementation of post-construction stormwater controls for any new construction. This might include on-site detention, pollutant reduction or both.
6. Pollution Prevention/Good Housekeeping – Train maintenance staff to employ pollution prevention techniques and to maintain and operate public facilities to ensure the most efficient pollutant reduction. Materials handling, fleet vehicle maintenance, and application of chemicals in public areas, such as parks and roadways, should be managed to reduce impact on stormwater quality.

6.2.3 Treatment Options

1. **Porous Pavement**: Porous pavements are being used more often, particularly in low traffic areas, such as parking lots and maintenance access roads. Both concrete and asphalt can be provided as porous, thereby allowing all surface runoff to infiltrate directly through to the subgrade. These pavements are very effective at reducing site runoff. They require a certain amount of maintenance by vacuuming on occasion to remove silt buildup in the voids. Porous pavements cost more in upfront costs, but can significantly reduce the size and quantity of the storm drain collection system, while simultaneously reducing the amount of pollutant loading entering the receiving stream.
2. **On-site Retention/Detention**: Many of the newer subdivisions have installed retention/detention basins designed to collect the runoff from a particular storm and either discharge at a controlled rate to a nearby storm drain system or allow for percolation and evaporation. These basins require very little maintenance, but can consume large swaths of land.
3. **Hydrodynamic Separators**: These are advanced stormwater treatment systems which utilize screens and hydrodynamic separation to capture pollutants. They typically consist of a concrete box with filtration screen, sediment chambers, and oil skimmers. Water enters the box at one end with trash and debris being trapped in the screen. Solids settle in the sediment chambers, while oil and grease are trapped in the skimmer. Some manufacturers claim pollutant removals of 99% for oil and grease, 71% for turbidity, and 87% for total suspended solids (TSS). These systems are often used in large parking lots for commercial or industrial areas. Routine maintenance is required to keep the trash and pollutants removed.
4. **Inlet Filters**: There are numerous types of filtering systems installed on storm drain inlet structures. Water flows to the inlet, through the filtering system, where solids are trapped, allowing treated stormwater to enter the storm collection system. Routine maintenance is required to clean the filters.

5. **Bioswales**: These are grass-lined swales that provide a certain amount of pollutant removal by allowing runoff to filter through the vegetation lining and soil to an underdrain system. The treated water is discharged to the nearest stormwater collection system.

Refer to the City's SWMP for more information on BMPs.

7.0 Capital Improvement Program

7.1 Information Regarding CIP Projects

The data used for the sewer and storm drain analysis was provided from a variety of sources, including a GIS base file provided by the City, construction drawings, some field survey, site investigations, previous work, and other miscellaneous sources. The accuracy of the GIS base file was poor, as was, much of the other available information. There are many parts of the sewer/storm base file that had to be assumed or estimated due to no available information. As a result, many of the sewer pipeline diameters, rims, inverts, and slopes are nothing more than an educated guess. Therefore, it is important that the City staff investigate each project with a field survey, prior to implementing, to confirm existing conditions shown match existing data shown in the CIP projects.

7.2 General Cost Assumptions

Cost estimates developed for this Master Plan are based on November 2012 dollars. Total project costs include estimates for construction, engineering and technical services, legal, administration, construction management, and contingency. Estimated construction costs are based on historical bids for similar project for the City, the 2012 RSMeans Sitework and Landscape Cost Data and recent bids. The estimated costs of engineering and technical services were assumed to be 15 percent and legal, administration, and construction management were assumed to be 10 percent of the estimated construction cost. A contingency of 30 percent of the estimated construction cost was also included in the total project cost estimates.

The estimates contained herein are planning level cost estimates based on current perceptions of conditions at the project locations. These estimates reflect professional opinion of costs at this time and are subject to change as the project design matures.

The project costs for sewer main pipelines were estimated using a unit cost per foot of pipe and have been broken into two (2) categories. **Table 7-1** represents costs for replacing existing sewers within existing streets. **Table 7-2** represents costs for installing new sewers for new developments, which excludes cost to remove existing sewers and pavement repairs. All unit costs assume PVC for pipe sizes up to 24" diameter and RCP for pipe sizes >24", an average depth of 8' with manholes spaced a minimum of 300 feet apart, along with lateral connections every 75', and were assumed to include the material and installation, as well as engineering, legal, administration, construction management, and contingency. The cost of acquisition of land or easements is not included in the pipeline cost estimates.

Table 7-1
Unit Construction Costs – Existing Sewer System Replacements

Pipelines (Diameter)	Construction[1] Unit Cost ($/lineal ft)	Capital[2] Unit Cost ($/lineal ft)
8 inches	$219	$274
10 inches	$232	$290
12 inches	$238	$298
15 inches	$245	$306
18 inches	$254	$318
21 inches	$297	$371
24 inches	$362	$454
30 inches	$521	$651
36 inches	$586	$733

[1] *Includes 30% construction contingency*
[2] *Includes 15% for engineering and construction management and 10% for legal and administrative costs*

Table 7-2
Unit Construction Costs – New Sewer System Replacements

Pipelines (Diameter)	Construction[1] Unit Cost ($/lineal ft)	Capital[2] Unit Cost ($/lineal ft)
8 inches	$147	$184
10 inches	$161	$201
12 inches	$170	$213
15 inches	$178	$223
18 inches	$186	$233
21 inches	$214	$268
24 inches	$264	$330
30 inches	$407	$509
36 inches	$472	$590

[1] *Includes 30% construction contingency*
[2] *Includes 15% for engineering and construction management and 10% for legal and administrative costs*

7.3 Information about the Combined System

The City has been operating a combined system for decades. Generally speaking, this system has worked well with minimal problems. Based on field work and a previous study, the following paragraphs describe what we have determined is occurring.

Inlets are spaced randomly throughout the combined system but do not cover the entire drainage area. There are many low areas and sumps with no inlet; therefore, ponding of storm runoff is occurring in these locations where it ultimately evaporates. Some of the inlets that exist are partially or fully clogged thereby causing more ponding. As a result, not as much storm runoff enters the collection system as would happen if the inlets were located more efficiently and kept open. As long as this doesn't cause major flooding, leading to customer complaints, it is a good thing.

Although storm runoff does not appear to be causing major problems to the system hydraulically, there is still concern with debris, such as rocks, sticks, leaves and trash entering the sewer from the storm inlets. This debris can reduce capacity within the pipes and has led to sewage backups, particularly along N. Rio Vista Drive. In addition, the debris requires additional cleaning of the system.

That being said, we do not feel it is cost-effective to convert the entire combined system to a separate system; however, a separate storm drain system along N. Rio Vista Drive and River Drive is recommended, since this area is known to have flooded in the past.

In addition, we recommend slowly converting much of the system to separate storm drain system over a period of years or decades. Each sewer pipeline within the combined system will ultimately need replacing. During this construction, instead of simply reconnecting storm inlets back to the sewer, we recommend installing a new storm drain pipeline to the nearest drain. This may not be practical in the heart of the City, but around the edges of the combined system (River Drive, Best Road, Malan Street) it is reasonable. For example, in the southeast corner near Malan Street, much of this combined system could be separated and discharged to the Bryant Drain to the south. As part of a separate project, the Bryant Drain was analyzed and made a recommendation for sizing the drain. See Appendix I for this study.

7.4 Recommended Improvement Projects

Several sewer/storm system improvements were identified based on the existing and future system analyses described previously. The highest priority projects are those associated with existing system deficiencies. All projects are sized to meet the ultimate demand and development conditions of year 2030.

Recommended improvements described here are based on the model results, but also include other improvements identified due to issues reported in the field by operations and maintenance staff. A priority level was assigned for each of the locations identified as a problem area. There are three priority levels:

- Phase 1: Typically, the highest priority projects because of the potential for overflows in the system.
- Phase 2: Medium priority projects that include locations where an overflow was not identified by the model analyses, but where there is a significant rise in the HGL such that the flows are within two feet of the ground surface and will potentially overflow as development within the City increase.
- Phase 3: The lowest priority projects include locations where there are negative or zero sloped pipes. Pipe inverts at these locations should be verified. If a negative or flat sloped pipe exists, these pipes should be replaced at a positive slope when it is necessary to replace them due to age, corrosion, or other operational or maintenance issues. This collection of projects also considers future developments and consists of upsizing to meet the increased demands.

7.4.1 CCTV Program

The City has a CCTV program outlined in the Operation and Maintenance Program of the SSMP. Close attention should be paid to the known problem areas as described in Appendix A of this document along with the combined system portion of the City. In conjunction with televising, accurate and consistent assessment is critical for prioritizing improvements. Data obtained from these inspections provides information to evaluate conditions and rank the significance of defects. **Table 7-3** provides a potential rating system that could be used for condition assessments.

Table 7-3
Condition Assessment Criteria and Priority Matrix

Defect	Assessment Rating				
	0	1 to 2	3, 4 or 5	6 or 7	8
Cracks	None	Minimal	Medium	High	Immediate
Roots	Minimal	10% to 35%	40%-60%	60%-80%	>80%
Offset Joints	Minimal	½”	1”	1” to1.5”	>1.5”
Grease Buildup	None	Minimal	1” thick	1” to 1.5” thick	>1.5” thick
Debris Buildup	None	Minimal	Minor blockage	Major blockage	Full blockage
Infiltration	Minimal	<1 gpm	1-2 gpm	2-3 gpm	3 gpm
Flow	Minimal	2/5 or less full	2/5-3/5 full	½ full	¾ full

By using a coding system similar to the one above, the City can prioritize pipeline cleaning, repairs and replacements along with addressing critical issues such as structural integrity, effective I&I reduction, flow capacity and effectiveness of the Fats, Oils, and Grease (FOG) Control Program.

7.4.2 Cleaning Program

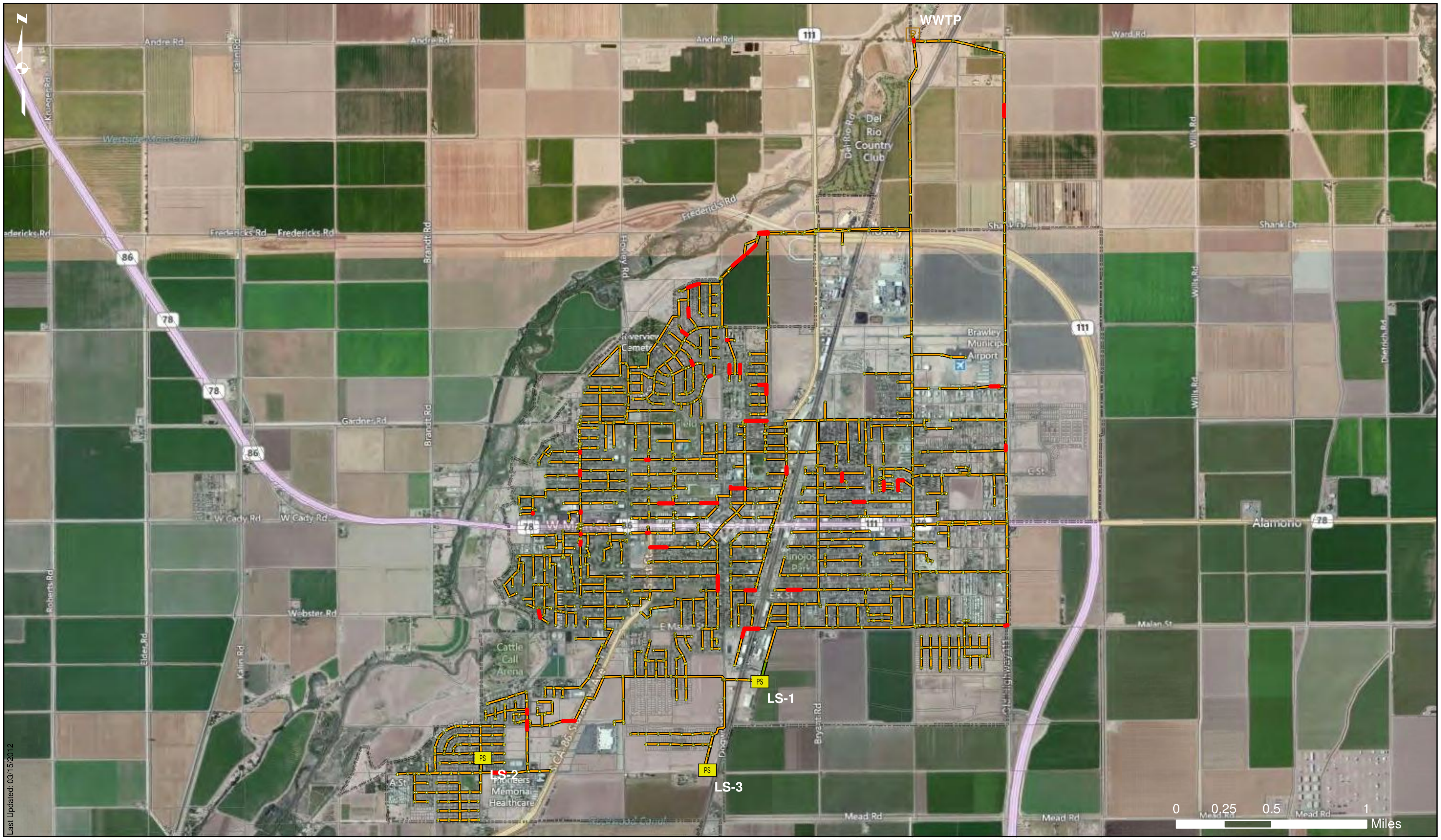
The City's SSMP identifies a regular cleaning program. The City should continue with a regularly scheduled cleaning program of all pipelines, with special attention given to those located within the combined system and the commercial part of town with FOG problems. The cleaning program should include manholes in addition to pipelines, as surcharging results in solids settlement on the bench and walls of manholes.

7.4.3 Fats, Oils, and Grease (FOG) Control Program

The City's SSMP identifies a FOG Control Program. Many of the hotspots and backup locations shown in the SSMP are in the commercial part of town near restaurants. The City has the authority to prohibit discharges to the system by requiring grease traps and/or other devices. If blockages continue, the City should begin to enforce the measures identified in the FOG Control Program.

7.4.4 Confirming Existing Flat or Negative Slope Sewers

Psomas has provided **Figure 7-1**, which identifies all gravity sewers within the City currently shown at a flat or negative slope. This is often due to lack of available information and many times may be incorrect. Psomas recommends the City create a program to check each of these pipelines over time and update the GIS files accordingly with the correct inverts and slopes.



Legend

- Zero & Negative Sloped Pipes
- Existing Manholes
- Existing Lift Stations
- WWTP

Existing Pipes

- Gravity main
- Force main
- Parcels
- City Limits

Figure 7-1
Pipes with Negative or Zero Slope

City of Brawley Integrated Master Plan
April 2013

7.4.5 Updating the City's GIS

Psomas has provided the City with an updated GIS base file. Although the base files are much improved, there are still many areas where assumptions were made for manhole depth, pipe size, slope, and material. The City should require all staff to keep notes concerning the field conditions of the sewer and storm system on a regular basis and assign one person to be responsible for gathering the notes and updating the GIS base files on a weekly basis. As the sewer system is confirmed, the GIS files should have attribute fields updated in order for the City to keep track of which areas are field verified and which are not.

7.4.6 Ongoing Manhole Rehabilitation and Replacement

Manholes should be rehabilitated and/or replaced based on the recommendations in Section 2-5, above. The City should take a systematic approach to this. Any time there is work done in the street, whether it is the repair of pavement, sewer, storm and water facilities, there should be a requirement to repair or rehabilitate any necessary manholes within the limits of work. As manholes are upgraded, the City's GIS will need to be updated to include this information.

7.4.7 Ongoing Separation of Combined System

The combined system within the City should slowly be separated over a period of years and/or decades with an approach similar to the one described in Section 7.3.5 for manholes. Any time work is performed within the City's right of way, there should be a requirement to separate the storm drain from the sewer whenever practical. There are several storm drain trunk pipelines running north to the New River which can be utilized as discharge points along with the Bryant Drain south of Malan Street. The new storm drain pipelines being installed in N. Rio Vista Drive and River Drive, as described in CIP Projects #2 and #3, below, also provide a discharge point. The City will have to determine on a case-by-case basis when it is cost effective to separate the system versus leaving status quo.

7.4.8 Phase 1 (2013-2018)

CIP PROJECT #1

SANITARY SEWER REPLACEMENT
Upstream of WWTP (Western Trunk Sewer) from WWTP to Shank Road, Easement, and N. Imperial Avenue

The pipes have a d/D >1 during existing peak dry and wet weather conditions with the HGL several feet below ground surface; however, during future analyses, this segment of the system has insufficient capacity. In addition, during existing conditions, the HGL in this pipeline is causing some of the problems along Western Avenue, River Drive, and N. Rio Vista Avenue. Several of the pipes are at a negative slope; thus causing HGL rises in upstream segments.

Recommended Improvement: Upsize pipes as shown in **Table 7-4**.

NOTE: This project should be field surveyed, CCTV'd and provided with flow monitoring for several months prior to implementation.

Table 7-4
CIP #1: Sanitary Sewer Replacement, WTTP to Shank Road, Easement, and N. Imperial Avenue

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
CA001.00 CA002.00	I4	30	36	150	$87,900	$109,875
CA002.00 CA016.00	I4, I5, I6, I7	24	30	5,300	$2,761,300	$3,451,625
CA016.00 WC010.01	I7, H7, G7	18	24	5,900	$2,135,800	$2,669,750
		TOTALS		**11,350**	**$4,985,000**	**$6,231,250**

CIP PROJECT #2

SANITARY SEWER REPLACEMENT, STORM DRAIN INSTALLATION
N. Imperial Avenue, Alamo Street, Olive Way, Cemetery, Western Avenue, River Drive

SEGMENT 1: Sewer on N. Imperial Avenue, Alamo Street, Olive Way
There is a flow split at manhole WC010 01 on N. Imperial Avenue, where the flow discharges through two 18-inch trunk lines. Upstream of this location, the single 18-inch trunk line does not provide sufficient capacity for the peak dry and wet weather flows and results in surcharging upstream. Additionally, pipes GM-199 and GM-520 on Olive Way and Mesquite Avenue reduce to 15 inches, which is an additional capacity restriction. Improvements downstream to increase capacity, reduces surcharging through this area, although the HGL still rises above the crown of the pipe during peak wet weather conditions. In addition, several pipe segments have flat or negative slopes.

Recommended Improvement: Increase all pipes to 21" diameter as shown in **Table 7-5**.

SEGMENT 2:

- **Sewer at Cemetery, Western Avenue, River Drive**
- **Storm Drain along River Drive**

These pipes have a d/D of 1.00 during existing peak wet weather conditions, but all pipes are flowing full and there is surcharging along this portion of the system due to downstream capacity deficiencies. With peak dry weather flows pipes GM-250 and GM-519, located in the cemetery have a d/D equal to 1.00 and there is surcharging of several feet along this entire segment of the collection system. Improvements downstream reduces the surcharging through this area, although the HGL still rises above the crown of the pipe with existing peak wet weather flows.

The surcharging is made worse by the decrease in pipe diameter on Western Avenue from 18” to 12” between MH’s WD001.00 and WC032.00.

Recommended Improvement:

- Increase all sewer pipes to a 21” diameter through cemetery.
- Confirm sewer pipe sizes on Western Avenue and upsize to 21” minimum.
- Confirm sewer pipe sizes on River Drive and upsize to 21” minimum.
- Install a new 24” storm drain pipe on River Drive, with discharge to the New River. Disconnect all combined storm inlets to sewer and connect to new storm drain.
- Re-direct sewer from WD032.00 to WD002.00 via new MH WD031A.00 constructed in Western Avenue/River Drive intersection. Abandon sewer from new MH to MH WD031.00.

NOTE: The exact location of the pipeline in the cemetery should be field verified vertically and horizontally. The location shown on the atlas maps is taken from the City-provided GIS; however, there are questions as to its accuracy. The size of pipes on Western Avenue and River Drive should be confirmed and anything less than 18” diameter should be upsized to 18” diameter.

Table 7-5
CIP #2: Sewer Replacement and Storm Drain Installation

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
WC010.01 WC023.00	G7, G8	15, 18	21	2,120	$629,640	$787,050
WC023.00 WC029.00	F8	18	21	2,325	$497,550	$621,938
WC029.00 WD006.00	F8, F9	12, 15, 18	21	2,770	$822,690	$1,028,363
		SUBTOTALS SS REPLACEMENTS		**7,215**	**$1,949,880**	**$2,437,351**
NEW SEWER INSTALLATION						
New MH WD031a.00 WD002.00	F9	--	10	35	$8,120	$10,150
New MH WD031a.00 WD031.00	F9	10	Abandon	380	$9,500	$11,875
		SUBTOTAL NEW SEWER INSTALLATION			**$17,620**	**$22,025**
NEW STORM DRAIN SYSTEM						
--	F9	--	24	1,500	$543,000	$678,750
		SUBTOTAL NEW STORM DRAIN SYSTEM		**1,500**	**$543,000**	**$678,750**
		TOTALS			**$2,510,500**	**$3,138,126**

CIP PROJECT #3
SANITARY SEWER REPLACEMENT, STORM DRAIN INSTALLATION
N. Rio Vista Drive, Sycamore Drive

SEGMENT 1: Sewer on N. Rio Vista Drive from River Drive to West G Street
There is surcharging along N. Rio Vista Drive, with overflows occurring at several manholes along the alignment during dry and wet weather flows. There is a decrease in pipe size to a 12" diameter pipe near West B. Street which contributes to the backups. This area has a history of backups.

Recommended Improvement: Increase all pipes to 18" and 21" diameter per **Table 7-6**.

SEGMENT 2: Storm Drain on N. Rio Vista Drive from River Drive to West G Street

This storm drain system will serve as a trunk main to allow for future connections. The City should have a phased program to separate the combined system into storm and sewer. This pipeline will be placed at a depth such that approximately 25% to 30% of the existing combined system can be separated with new storm connections leading to it. Redirect existing storm connections from the existing sanitary sewer to the new storm drain pipeline

Recommended Improvement:

- Install a new 24" and 30" pipeline.
- Re-direct existing storm drain connections from the sewer to this new storm drain pipeline.
- See **Table 7-6**.

SEGMENT 3: Sewer on Sycamore Drive from North of H Street to North of G Street

The sewer pipe segment between MH WE011.00 and MH WE009.00 (Atlas Map page E11) is surcharging during existing peak wet and dry weather flows.

Recommended Improvement: Upsize the pipes to 12" diameter per **Table 7-6**.

Table 7-6
CIP #3: Sewer Replacement and Storm Drain Installation, N. Rio Vista Drive, Sycamore Drive

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
WD006.00 WD026.00	F9, F10	12, 15	21	2,500	$742,500	$928,125
WD026.00 WE003.00	F10,F11	15	18	900	$228,600	$285,750
WE011.00 WE009.00	E11	8	12	400	$95,200	$119,000
		SUBTOTALS SS REPLACEMENTS		**3,800**	**$1,066,300**	**$1,332,875**
NEW STORM DRAIN SYSTEM						
--	F9, F10, F11	--	30	3,400	$1,771,400	$2,214,250
		SUBTOTAL NEW STORM DRAIN SYSTEM		**3,400**	**$1,771,400**	**$2,214,250**
		TOTALS			**$2,837,700**	**$3,547,125**

CIP PROJECT #4

RE-DIRECT STORM CONNECTION FROM EXISTING SEWER TO EXISTING STORM DRAIN, S. El Cerrito Drive

Flow Metering Site (FM01) consisted of installing a flow meter in MH WE007.16 (Atlas Map page F12) on S. El Cerrito Drive, just north of Cattle Call Drive, for approximately 20 days in February 2012. During this time a 1" rain storm occurred from February 16 to February 17. Flow monitoring indicated significant inflow during this event, with a peaking factor of 8.21 recorded. There is a direct storm drain connection(s) upstream of this manhole; however, the location(s) is unknown.

Recommended Improvement: It is recommended the City locate the source(s) of this inflow either by CCTV or smoke testing and redirect to the existing storm drain pipeline located in Cattle Call Drive. See **Table 7-7.**

Table 7-7
CIP #4: Re-direct Storm Connection in S. El Cerrito Drive

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost
PRICE UNKNOWN. CITY TO LOCATE SOURCE OF INFLOW AND RE-DIRECT TO STORM DRAIN ON CATTLE CALL DR.					

CIP PROJECT #5
SANITARY SEWER REPLACEMENT, A Street, 10th Street

NOTE: There appears to be a pipe size reduction from 18" to 12" along A Street from MH CD 017.00 to MH CC010.00 (Atlas Map H10). There also appears to be a negative sloped pipe from MH CC010.00 to MH CC009.00 (Atlas Maps H9, H10). The City shall confirm these prior to construction.

Recommended Improvement: If the information noted above is confirmed, the pipeline should be upsized from MH CD 017.00 to MH CC008.00 (Atlas Maps H9, H10) to an 18" diameter. The pipe segment between MH CC010.00 to MH CC009.00 should be installed at a positive slope. See **Table 7-8**.

Table 7-8
CIP #5: Sewer Replacement, A Street, 10th Street

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
CD017.00 CC008.00	H9, H10	12, 15	18	1,725	$438,150	$547,688
		TOTALS		**1,725**	**$438,150**	**$547,688**

CIP PROJECT #6
SANITARY SEWER REPLACEMENT, River Drive, 5th Street

The pipes along River Drive from approximately 7th Street to 5th Street are currently shown at a negative slope.

Recommended Improvement: The manhole inverts and pipe diameters need to be field verified. If they are, in fact, at a negative slope, it is recommended the entire segment be replaced with an 8" pipeline at a positive slope from MH CD025.08 to MH CD025.04 (Atlas Map G9). See **Table 7-9**.

Table 7-9
CIP #6: Sewer Replacement, River Drive, 5th Street

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
CD025.08 CD025.04	G9	8, 15	8	750	$164,250	$205,313
		TOTALS		**750**	**$164,250**	**$205,313**

7.4.9 Phase 2 (2019-2024)

CIP PROJECT #7
SANITARY SEWER REPLACEMENT, Alley west of N. Imperial, A Street

The pipeline segment between MH CD030.00 and MH CD025.00 (Atlas Map G10) surcharges under peak wet weather conditions, with a portion of the alignment surcharging during peak dry weather flows.

Recommended Improvement: Confirm the pipe sizes. If they are as shown on the atlas maps, they should be upsized from an 8" to 12" diameter. See **Table 7-10**.

Table 7-10
CIP #7: Sewer Replacement, Alley, A Street

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
CD030.00 CD025.00	G10	8	12	1,200	$285,600	$357,000
		TOTALS		**1,200**	**$285,600**	**$357,000**

CIP PROJECT #8
SANITARY SEWER REPLACEMENT, Upstream of Pump Station #1

This pipeline is surcharging during existing peak wet weather conditions. There also appears to be a decrease in pipe diameter from 24" to 15". This occurs between MH ED005.00 and MH ED002.00 (Atlas Map G13)

Recommended Improvement: Confirm the pipe sizes. If they are as shown on the Atlas Maps, they should be upsized from a 15" to 24" diameter. See **Table 7-11**.

Table 7-11
CIP #8: Sewer Replacement Upstream of PS1

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
ED005.00 ED 002.00	G13	15	24	670	$242,540	$303,175
		TOTALS		**670**	**$242,540**	**$303,175**

CIP PROJECT #9

SANITARY SEWER REPLACEMENT, Best Road between WWTP and Shank Drive to River Drive

Portions of this pipeline are surcharging during existing peak wet weather conditions. The majority of the pipeline surcharges during 2020 flows and the entire segment surcharges during 2030 flows. This occurs between MH CA002.00 (Atlas Map I4) and MH EB005.00 (Atlas Map J9)

Recommended Improvement: Upsize all pipe segments to 30" diameter. Improve slopes where possible. See **Table 7-12**.

Table 7-12
CIP #9: Sewer Replacement Best Road, from WWTP to River Drive

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
CA002.00 EB005.00	I4, J4, J5, J6, J7, J8, J9	24	30	12,660	$6,595,860	$8,244,825
		TOTALS		**12,660**	**$6,595,860**	**$8,244,825**

CIP PROJECT #10

SANITARY SEWER REPLACEMENT, Cesar Chavez between E Street and K Street

Portions of this pipeline are surcharging during existing peak wet weather conditions. This occurs between MH CC015.00 (Atlas Map H10) and MH CC023.01 (Atlas Map H12). Existing pipe sizes are 8" and 10", with a large number of tributary sewers.

Recommended Improvement: Confirm pipe sizes and slopes in the field, upsize all pipe segments to 10" and 12" diameter, and improve slopes where possible. See **Table 7-13**.

Table 7-13
CIP #10: Sewer Replacement, Cesar Chavez between E Street and K Street

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
CC015.00 CC020.00	H10, H11	10	12	1,600	$380,800	$476,000
CC020.00 CC023.00	H11, H12	8	10	1,175	$272,600	$340,750
		TOTALS		**2,775**	**$653,400**	**$816,750**

CIP PROJECT #11

WWTP Expansion

Planning for the expansion of the WWTP should begin when average flows approach 5.9 MGD. Expansion could include engineered improvements to the clarifiers, expansion of existing aeration basins, upsizing pumps or adding clarifiers, basins, and pumps. Projected flows estimate the ADF to the WWTP to be 5.38 by 2020 and 7.31 by 2030. Although this project is being shown in Phase 2, the City should closely monitor the influent meter to the WWTP and begin planning for expansion when the ADF reaches approximately 85% of existing capacity. Prior to upsizing, the City should update the General Plan to estimate future growth to 50 years out. The estimated future growth should be used to estimate the future demands on the WWTP. For now, the 2030 flows will be used.

Recommended Improvement: Upsize the WWTP from 5.9 MGD to 7.31 MGD. See **Table 7-14**.

Table 7-14
CIP #11: WWTP Expansion

Existing Size (MGD)	Proposed Size (MGD)	Increase (MGD)	Construction Cost	Capital Cost
5.9	7.31	1.41	$6,000,000**	$7,500,000

** *Unit price taken from PDR for existing WWTP construction. PDR used $3.90/gallon. The price above uses approximately $4.25/gallon.*

7.4.10 Phase 3 (2025-2030)

CIP PROJECT #12

SANITARY SEWER REPLACEMENT, Best Road from River Drive to Malan Street, Malan Street from Best Road to Avenida De Colimbo

Portions of this pipeline surcharge during 2020 flows and the entire segment surcharges during 2030 flows. This occurs between MH EB005.00 (Atlas Map J9) and MH EB031.00 (Atlas Map I12).

Recommended Improvement: Confirm pipe sizes, upsize all pipe segments to 30" diameter and improve slopes, where possible. See **Table 7-15**.

Table 7-15
CIP #12: Sewer Replacement, Best Road and Malan Street

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
SEWER REPLACEMENTS						
EB005.00 EB031.00	J9, J10, J11, J12, I12	8, 24	30	6,575	$3,425,575	$4,281,969
		TOTALS		**6,575**	**$3,425,575**	**$4,281,969**

CIP PROJECT #13

SANITARY SEWER REPLACEMENT: West side of Hwy. 86, Cattle Call Drive, S. El Cerrito Drive

NOTE: This project should only be pursued once full development in the area from approximately Julia Drive to Cattle Call Drive along the western portion of Hwy. 86 has begun.

Based on projected 2030 flows, the sewer along S. El Cerrito is surcharging during peak and dry weather flows. This segment extends from MH WE007.01 (Atlas Map page F11) to MH WE007.26 (Atlas Map page F12).

Recommended Improvement: Upsize the sewer to a 12" diameter. See **Table 7-16**.

Table 7-16
CIP #13: Sanitary Sewer Replacement, West Side of Hwy 86, Cattle Call Drive, S. El Cerrito Drive

From MH to MH	Atlas Map Page(s)	Existing Size (in)	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
WE007.01 WE007.26	F11, F12	8, 10	12	2,350	$559,300	$699,125
		TOTALS		**2,350**	**$559,300**	**$699,125**

7.5 Summary of Recommended Sewer System Improvements

The recommended sewer system improvements and associated costs are presented in **Table 7-17**. Existing sewer pipeline replacements total approximately 10 miles. Existing storm drain installations include approximately 1 mile. The existing system improvements also include a 1.41 MGD increase to the WWTP capacity.

Table 7-17
Summary of Wastewater/Stormwater Improvement Costs

ID	Description	Proposed Size (in)	Length (ft)	Construction Cost	Capital Cost
PHASE 1 CIP					
CIP 1	Upsize Ex. 18, 24, 30" SS Western Trunk, WWTP to Shank Rd., Esmt, Imperial	24, 30, 36	11,350	$4,985,000	$6,231,250
CIP 2	SS Replacement/SD Installation Imperial Ave, Alamo St., Olive Way, Cemetery, Western Ave., River Dr.	10, 21, 24(sd)	7,630 SS 1,500 SD	$2,510,500	$3,138,125
CIP 3	SS Replacement/SD Installation N. Rio Vista Dr., Sycamore Dr.	12, 18, 21 (SS) 30 (SD)	3,800 SS 3,400 SD	$2,837,700	$3,547,125
CIP 4	Locate & redirect storm connection from ex. SS to ex. SD MH on Cattle Call at El Cerrito	Unknown	Unknown	Unknown	Unknown
CIP 5	SS Replacement A St., 10th St.	18	1,725	$438,150	$547,688
CIP 6	SS Replacement River Dr., 5th St,	8	750	$164,250	$205,313
PHASE 1 SUBTOTALS			**30,155**	**$10,935,600**	**$13,669,500**
PHASE 2 CIP					
CIP 7	SS Replacement Alley, A St.	12	1,200	$285,600	$357,000
CIP 8	SS Replacement Upstream of Lift Station 1	24	670	$242,540	$303,175
CIP 9	SS Replacement Best Rd. from WWTP to River Dr.	30	12,660	$6,595,860	$8,244,825
CIP 10	SS Replacement Cesar Chavez between E and K	10	1,175	$653,400	$816,750
CIP 11	WWTP Expansion by 1.46 MGD	--	--	$6,000,000	$7,500,000
PHASE 2 SUBTOTALS			**17,305**	**$13,777,400**	**$17,221,750**
PHASE 3 CIP					
CIP 12	SS Replacement Best Rd. from Shank to Malan. Malan St. from Best to Avenida de Colimbo	30	6,575	$3,425,575	$4,281,969
CIP 13	SS Replacement West side of Hwy. 86, Cattle Call Dr., El Cerrito Dr.	12	2,350	$559,300	$699,125
PHASE 3 SUBTOTALS			**8,925**	**$3,984,875**	**$4,981,094**
TOTAL CIP IMPROVEMENTS			**56,385**	**$28,697,875**	**$35,872,344**

8.0 Funding Sources

This chapter describes financing alternatives for proposed sewer projects. Funding sources include Federal, State, and local financing programs. Revenue sources include ad valorem taxes, special districts, and developer-imposed impact fees. Funding sources are explored that are not dependent on user charge revenue. The sources of funds for new capital projects are described, but this chapter does not address the amount of funds the City could raise or the repayment impacts. Most of these sources of funding are summarized in **Table 8-1**.

Federal Programs

BECC

The Border Environment Cooperation Commission (BECC), in conjunction with North American Development Bank (NADB), is a source of funding that the City of Brawley has used in the past to fund water and wastewater projects. These two organizations were created in 1994 by the Governments of the United States and Mexico under a side-agreement to the North American Free Trade Agreement (NAFTA). The goal of BECC/NADB is to help improve the environmental conditions of the Mexico–United States border region in order to advance the well-being of residents in both nations. BECC focuses on the technical, environmental, and social aspects of project development, while NADB concentrates on project financing and oversight for project implementation.

ARRA

The American Recovery and Reinvestment Act (ARRA) of 2009 has provided funds for a number of different programs. The United States Department of Agriculture (USDA) program contains a sewer category titled "*Water and Waste Disposal Loan and Grant*". Eligible applicants include cities. The purpose of this category is to develop and repair water, sewer, storm drainage and solid waste systems. The USDA either makes a below market, 40-year fixed loan or grant to agencies that have eligible projects. Typical funding is between $1 million and $3 million. All loans must be fully secured by revenues or assessments. Grants focus on the neediest applicants, depending on population, income, and health factors. Application is open year round and the City should contact the USDA Rural development local office for more information.

State Programs

CDBG

Community Development Block Grants (CDBG) are administered by the State. The primary statutory objective of the CDBG program is to develop viable communities by providing decent housing and a suitable living environment and by expanding economic opportunities, principally for persons of low- and moderate-income. The State must ensure that at least 70 percent of its CDBG grant funds are used for activities that benefit low- and moderate-income persons over a one-, two-, or three-year time period selected by the State. This general objective is achieved by granting “maximum feasible priority” to activities which benefit low- and moderate-income families or aid in the prevention or elimination of slums or blight. Under unique circumstances, States may also use their funds to meet urgent community development needs. A need is considered urgent if it poses a serious and immediate threat to the health or welfare of the community and has arisen in the past 18 months.

Local governments have the responsibility to consider local needs, prepare grant applications for submission to the State, and carry out the funded community development activities. Local governments must comply with Federal and State requirements.

Local Financing Programs

General Obligation Bonds

General obligation bonds are debt instruments that are backed by the full faith and credit of the issuing municipality. They are generally repaid by ad valorem property taxes and are typically used to fund projects that serve the entire community and are for projects that do not provide direct sources of revenues such as user charges. They must be approved by two-thirds (2/3) of the jurisdiction’s voters.

Districts

Most of the commonly used sources of debt for public facilities involve special districts. The interest rates on these sources of debt are not subsidized, as are some of the State and Federal loans, and will vary with market conditions and the time of the sale. For the last several years, these rates have been in the range of 5 to 6.5 percent. Several special districts are described below.

Assessment Districts

Assessment Districts formed under the conventional statutes (Improvement Act of 1911, Municipal Improvement Act of 1913, and Improvement Bond Act of 1915) provide some of the less costly development money available because of the real estate security. Assessment Districts do not require an election vote, but a mailed ballot vote. Votes are tabulated at a protest hearing and if more than 50 percent of the property owners vote against the formation of the district (weighted by assessment amount) the proceedings must be halted. Assessment Districts are initiated by petition of the property owners in the proposed district or by action of the City Council.

Mello-Roos Districts

The Mello-Roos Community Facilities Act authorizes cities, counties, and special districts to form “community facilities districts” to finance the construction, improvement or purchase of public facilities that benefit a clearly defined service area. Two or more government agencies may form a community facilities district through a joint financing agreement. All government agencies with jurisdiction in the proposed district boundary must agree to the formation of the district.

The community facilities district may issue bonds, if approved by two-thirds of the voters within the district. Bonds are repaid through special tax assessments. The assessment may not be strictly proportionate to property value. Unlike special assessment districts, the tax does not have to be based directly on benefit derived from the public facilities, although it may be. Taxes have been based on acreage, street frontage, or the square footage of buildings.

Infrastructure Financing Districts

Infrastructure financing districts are formed in proceedings similar to the formation of a Mello-Roos Community Facilities District and, once formed, can use the property tax increment resulting from new development within the district to finance capital facilities. The act to establish these districts became law in 1991. All projects must have community-wide significance, the district must have the consent of affected taxing entities, the district cannot overlap a redevelopment project area, and a two-thirds vote is required to create the district. Typically, utility facilities such as water, sewer, and storm drain improvements do not increase property tax revenues, so this may not be an appropriate vehicle for these improvements.

Developer-Imposed Programs

Developer-imposed programs can be used to fund improvements. One approach is for the developer to agree to build the improvements as part of the development. Another approach involves revenues from developer-impact fees. This method typically involves pay-as-you-go, where impact fees would be collected in a special fund until enough money has accumulated to begin construction. The size of the construction outlay may make pay-as-you-go a difficult approach or, at a minimum, require project phasing.

A third approach would be to issue a revenue bond to obtain upfront construction funding based on the pledge of future impact fees. However, this is unconventional and would require a large reserve fund and may also require a guarantor other than the City. Other options may prove to be more effective.

Revenue Sources

Ad Valorem Taxes

To issue general obligation bonds, a two-thirds majority vote to incur the debt and its repayment is required. This repayment is in the form of ad valorem taxes. The amount of general obligation bonds that can be issued is dependent on the other general obligations outstanding and the total assessed valuation of the City.

Assessment Districts (1911/1913 with 1915 Bond Act and Mello-Roos)

The formation of an assessment district creates its own direct revenue source. The project costs are spread to property owners based on an allocation of costs in proportion to the property's benefit or on a tax formula based on benefit. The costs and benefits received are used to create an equation that spreads cost equitably among the benefiting properties. This allocation becomes a lien on the property if the assessment is not paid.

Infrastructure Financing District

This district generates revenue on a tax increment basis. Tax increment revenues are calculated as follows: the property taxes collected from properties within the boundaries at the time the district is formed are the frozen base, and the additional amount collected above this amount is the tax increment. Revenue generation will depend on the amount of increased property values resulting from the planned improvements. Tax increment revenue also tends to lag a few years after the improvements are put in place.

Impact Fees

For new development, revenues can be generated by imposing impact fees. The magnitude of these fees is dependent on the costs attributed to new development and the City's philosophy on collecting these fees. The impact fee calculation will be regulated by Section 66000 of the California Government Code, which governs impact fees from not being more than the costs that can be attributed to each new user.

Table 8-1
Comparison of Financing Alternatives

Name	Type of Financing	Amount Available	Terms	Revenue Sources	Voter Approval Required?	Minimum time to Implement	Likelihood of Obtaining Financing	Other Comments
Pay-As-You-Go	Cash	Depends on Level of Charges	None	Impact Fees	No	-	This method will not generate sufficient funding for many improvements	
Revenue Bond	City Obligation	Depends on revenue stream	10-30 years	Impact Fees	No	6-12 months	Low, Unconventional	Needs Guarantor other than City
General Obligation Bond	City Obligation	Dependent on other GO bonds and total assessed valuation	20-30 years	ad valorem tax	Yes, 2/3	24 months	Low (voter approval)	
Infrastructure Financing District	District obligation	Depends on tax increment available	Unknown	Tax Increment	Yes, 2/3 within District	24 months	Low (voter approval), not yet used in California	Cannot overlap redevelopment area
Conventional Assessment District (1911, 1913 – 1915 IBA)	District obligation	Depends on level of assessment	10-30 years	Property Assessments	Property Owner Protest Vote/ Hearing	6 months	OK, should have diversity of ownership	
Mello-Roos Community Facilities District	District obligation	Depends on level of assessment	10-30 years	Property Assessments	Property Owner Election, 2/3 Vote	12 months	Depends on stability of revenue	Timing depends on election