WASTEWATER MASTER PLAN

October 2019

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COMMITMENT & INTEGRITY DRIVE RESULTS

0455-002
City of Pomona

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

1. INTRODUCTION

The City of Pomona (City) last updated its Wastewater Master Plan in 2005. Since that time, there have been changes in development due to an economic downturn that led to the collapse of the housing market, as well as reductions in water use (and thus wastewater generation) due to increased conservation. Recently, development and redevelopment has begun increasing again within the City, increasing demand on the wastewater system. This Wastewater Collection System Master Plan (Master Plan) evaluates the existing and future system conditions up to year 2040. This planning document identifies system deficiencies and recommends projects to address these deficits. All recommendations are summarized in a Capital Improvement Plan (CIP).

This Master Plan was prepared concurrently with other planning efforts, including the Water Resources Department's Strategic Plan, Integrated Water Supply Plan (IWSP) update, and Potable Water Master Plan, and is intended to be used as a guideline for the improvement of the City's sewer system.

1.1 Objectives and Scope

The primary objective of this Master Plan is to develop a CIP that will provide sewer services that meet the requirements of the City's customers. The scope of work for the Master Plan includes the following tasks that were developed to assist the City in meeting this objective:

- Development of estimated existing and projected sewer flows
- Creation of an accurate and usable hydraulic model
- Evaluation of sewer system performance under various scenarios
- Identification of capital improvement projects
- Development of the Sedaru Smart CIP to dynamically generate a pipeline CIP

1.2 Data Sources

Information presented in this report is obtained from a number of sources that include, but are not limited to:

- Previous Sewer Master Plan (2005)
- Urban Water Management Plan (2015)
- City's GIS data (land use, streets, pipelines, manholes, pump stations)
- Historical water production and billing records (2013-2016)
- General Plan land use data (2014)
- Near-term development information provided by City staff
- Sewer flow monitoring data for a six-week period from January 2017 through February 2017

1.3 Report Outline

The Master Plan is divided into six sections, with **Section 1** serving as the introduction. **Section 2** discusses the study area and land use of the City's service area, while **Section 3** discusses the existing sewer system. **Section 4** describes the sewer system model development and **Section 5** discusses existing and future wastewater flows. **Section 6** describes the sewer system capacity analysis.

1.4 Abbreviations

ADWF	Average Dry Weather Flow
AFY	Acre Feet per Year
BWF	Base Wastewater Flow
Cal Poly	California State Polytechnic University, Pomona
CII	commercial/industrial/institutional
CIP	Capital improvement program
City	City of Pomona
d/D	Ratio of flow depth to pipe diameter
DOF	Department of Finance
DWF	Dry Weather Flow
FY	Fiscal year
GIS	Geographic Information System
gpd	Gallons per day
gpm	Gallons per minute
GSWC	Golden State Water Company
GWI	Groundwater Infiltration
HGL	Hydraulic grade line
I/I	Infiltration and Inflow
IWSP	Integrated Water Supply Plan
LACSD	Los Angeles County Sanitation Districts
mgd	Million Gallons per Day
MH	Manhole
NOAA	National Oceanic and Atmospheric Administration
PDWF	Peak Dry Weather Flow
PS	Pump Station
PWRP	Pomona Water Reclamation Plant
PWWF	Peak Wet Weather Flow
RDI/I	Rainfall Dependent Infiltration and Inflow
ROW	right-of-way
SCAG	Southern California Association of Governments
SFR	Single Family Residential
UWMP	Urban Water Management Plan
VCP	vitrified clay pipe
WDF	water demand factors
WVWD	Walnut Valley Water District
WWF	Wet Weather Flow

2. STUDY AREA AND LAND USE DEVELOPMENTS

2.1 Study Area

The City is located approximately 35 miles east of downtown Los Angeles. **Figure 2-1** shows the City's borders, the service area, and the neighboring cities. The City is bounded on the east by the City of Montclair, on the south by the cities of Chino and Chino Hills, and on the southwest by the City of Diamond Bar. The western boundary is comprised of the cities of Industry, Walnut, and San Dimas. On the northern boundary are the cities of La Verne and Claremont. The study area covers approximately 23 square miles. With a population of approximately 156,500 residents, the City of Pomona is currently the fifth largest city in Los Angeles County.

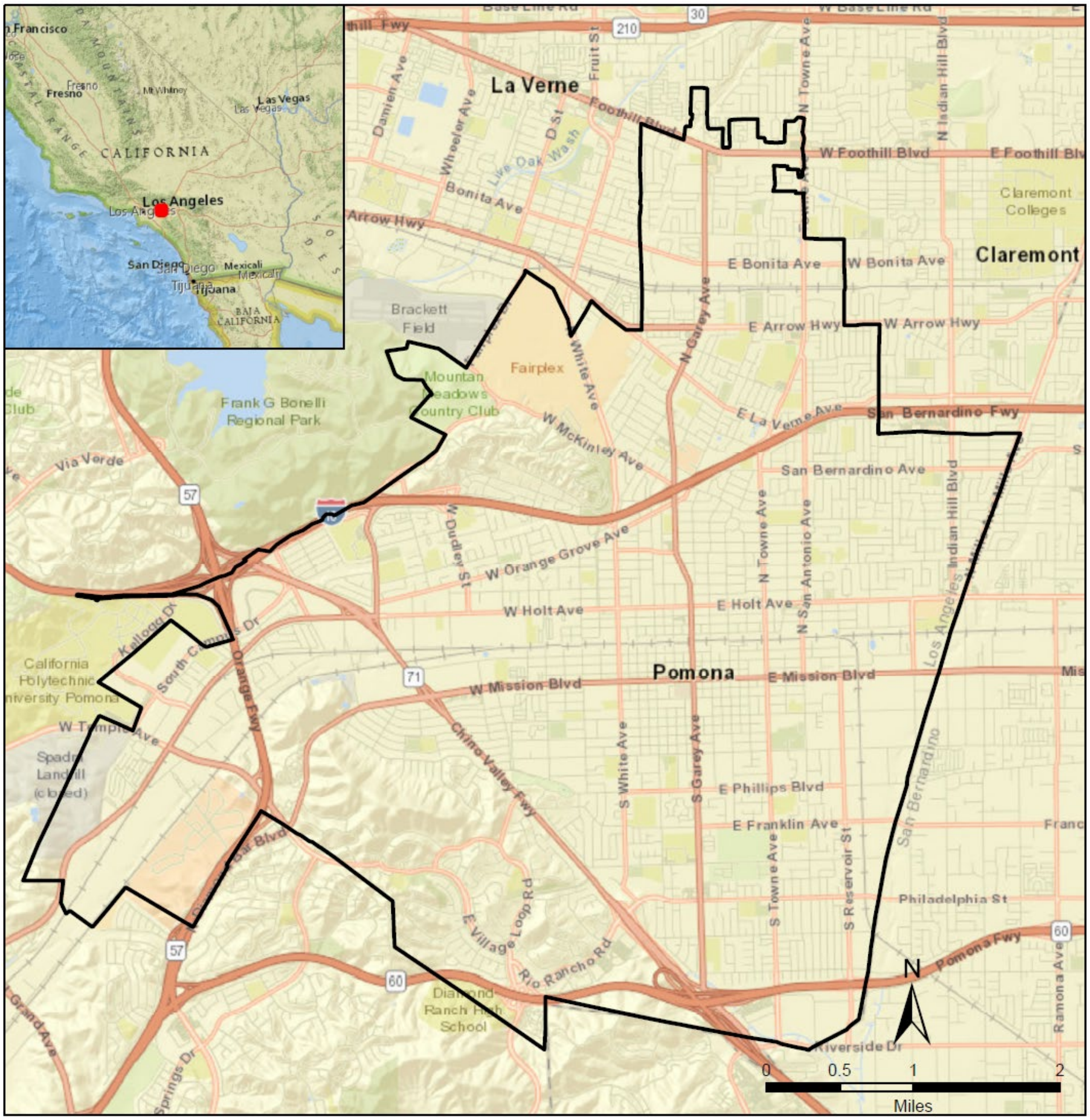
The City was incorporated in January 1888 and became a charter city in March 1911. The City developed as an agricultural base for citrus products in the 1870s and has since developed into a major railway and freeway corridor.

The City's proximity to public transportation facilities has provided convenient access for the City's residents and businesses. Two major east-west freeways pass through the City. The San Bernardino Freeway (Interstate 10) traverses the City's central portion, while the Pomona Freeway (State Route 60) crosses the southern extremity. The Foothill Freeway (Interstate 210) is another major freeway, which runs immediately north of the City. In addition, State Routes 57, 71, and 66 are significant transportation corridors for the City. Union Pacific, Burlington Northern-Santa Fe, Amtrak, and Metrolink provide commercial and passenger/commuter rail services passing through the City.

The sewer service area includes most of the incorporated area within the City limits with the exception of the following areas:

- 20-acre area located south of Foothill Boulevard and west of Towne Avenue which is presently served by the Golden State Water Company (GSWC).
- 20-acre area located north of Foothill Boulevard and west of Garey Avenue which is presently served by SCWC.
- 250-acre area located north of Valley Boulevard and west of Temple Avenue which is served by the Walnut Valley Water District (WVWD).
- 181-acre portion of the Rolling Ridge Estates located south of the Pomona Freeway (State Route 60) and west of State Route 71 in the City of Chino Hills that is outside the City limits

Figure 2-1: Pomona Service Area



2.2 Historical and Projected Population

The City's historical population estimates are based on California Department of Finance (DOF) and United States Census Bureau data, as listed in **Table 2-1**. Future estimates are obtained from the City's 2015 Urban Water Management Plan (UWMP), which are presented in **Table 2-2**. As shown in **Table 2-2**, the population projections provided in the UWMP indicate that the City will reach a population of 167,942 in 2020, and the projected population for 2040 is 213,192. These population increases were used to estimate the demand increases discussed in **Section 3**.

Table 2-1: Historical Population Estimates (2011-2018)

Year	Population	Annual Population Increase (percent)
2011	151,015	--
2012	152,143	0.7%
2013	153,462	0.9%
2014	154,370	0.6%
2015	154,759	0.3%
2016	154,717	0.0%
2017	154,718	0.0%
2018	155,687	0.6%

Source: California Department of Finance, Report E-4 Population Estimates for Cities, Counties and the State, 2011-2018 with 2010 Benchmark

Table 2-2: Projected SCAG 2001 Population Estimates (2020 to 2040)

Year	Population	5-year Increase (percent)
2020	167,942	--
2025	178,264	1.2%
2030	189,219	1.2%
2035	200,848	1.2%
2040	213,192	1.2%

Source: City of Pomona 2015 Urban Water Management Plan / Southern California Association of Governments (SCAG)

2.3 Land Use

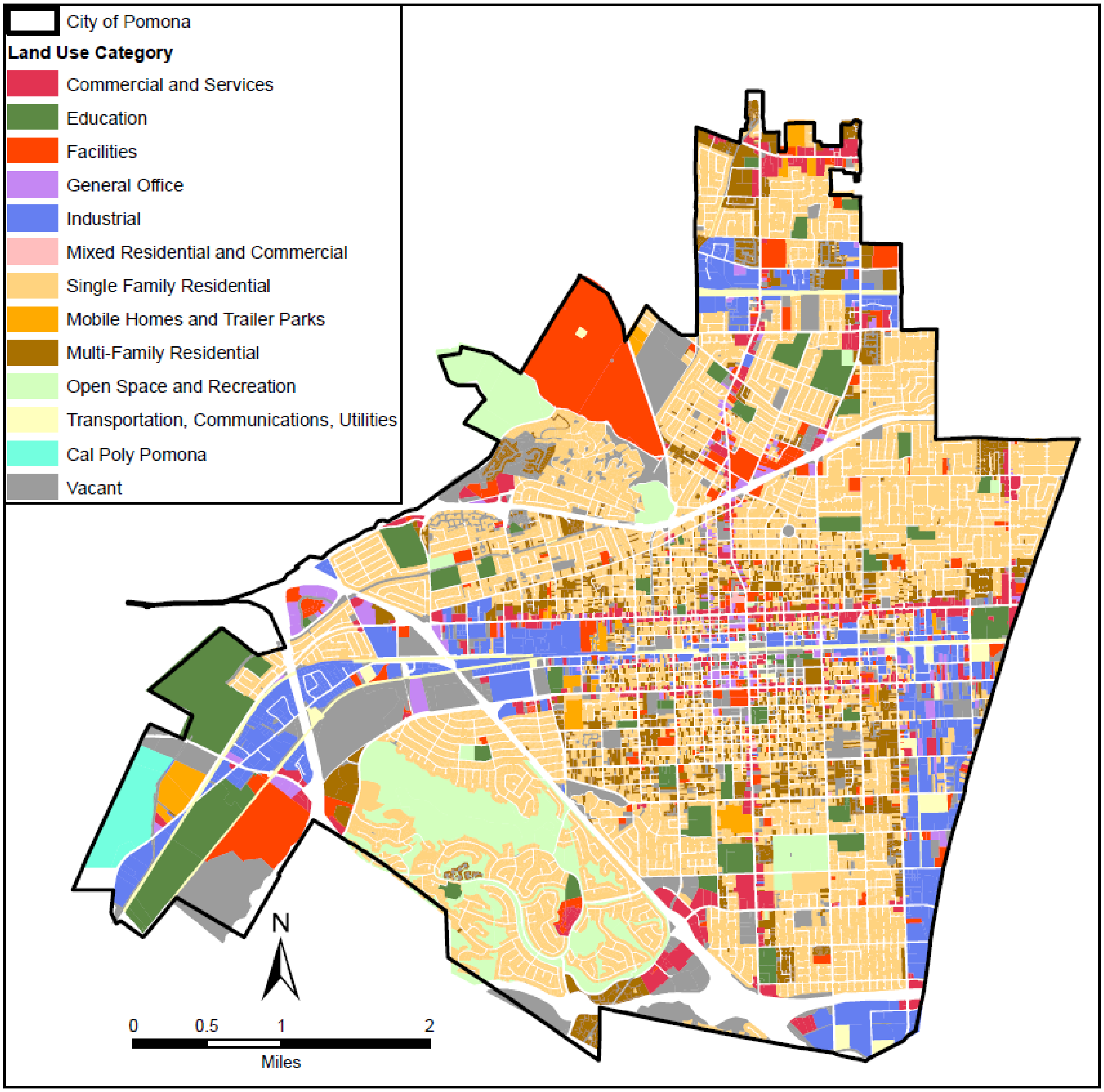
Existing land uses by parcel were included in a GIS file provided by the City and are shown in **Figure 2-2. Table 2-3** lists the approximate net acreage by land use category (streets and roads have been excluded) and the percent of the total net acreage for each land use category. As seen in **Table 2-3**, Single Family Residential (SFR) comprises a larger area (36 percent) of the City than any other land use, and the area of all residential categories comprises about 46 percent of the City. The City is generally considered to be built out; therefore, future development is expected to be comprised of infill of vacant parcels and densification. This is discussed further in **Section 3** as part of the water demand projections.

Table 2-3: Summary of Existing Land Use Distribution

Land Use Category	Area (acres)	Area (square miles)	Area (percent)
Agriculture	1	0.00	<1%
Commercial and Services	531	0.83	5%
Education	846	1.32	7%
Facilities	877	1.37	8%
General Office	172	0.27	2%
Industrial	1,106	1.73	10%
Mixed Residential and Commercial	14	0.02	<1%
Mobile Homes and Trailer Parks	157	0.25	1%
Multi-Family Residential	1,014	1.59	9%
Open Space and Recreation	827	1.29	7%
Single Family Residential	4,172	6.52	36%
Transportation, Communications, and Utilities	224	0.35	2%
Under Construction	1	0.00	<1%
Vacant	1,354	2.12	12%
Water	147	0.23	1%
Total	**11,443**	**17.9**	**100%**

Source: Land Use shapefile provided by the City

Figure 2-2: Existing Land Use



3. EXISTING SEWER SYSTEM

This section summarizes the City's wastewater collection system and describes its facilities and operations.

3.1 Wastewater Service Area

The City owns and operates a sanitary sewer system that serves residents and businesses within the City limits as well as a limited area outside the City limits of approximately 6 acres. **Figure 3-1** shows the City of Pomona service area. The areas outside the City limits include:

- Approximately 303 accounts within the City of Claremont. The City of Claremont connects at two locations to two 8-inch Pomona sewer lines along Lynoak Drive and Towne Park Circle. Both 8-inch sewers flow south and discharge to another 8-inch sewer running east to west along E. Foothill Boulevard.
- Two Towne Avenue properties in the City of Claremont.
- Approximately 11 commercial/industrial properties located in the Mills/Philadelphia Section within the City of Chino on Pomona's southern border. The City has provided sewer service to these 11 properties since their occupation.

For topographical and elevation reasons, certain properties within the City of Pomona cannot be connected to the City's sewer system. The following lists the developed properties that are served by other wastewater utilities.

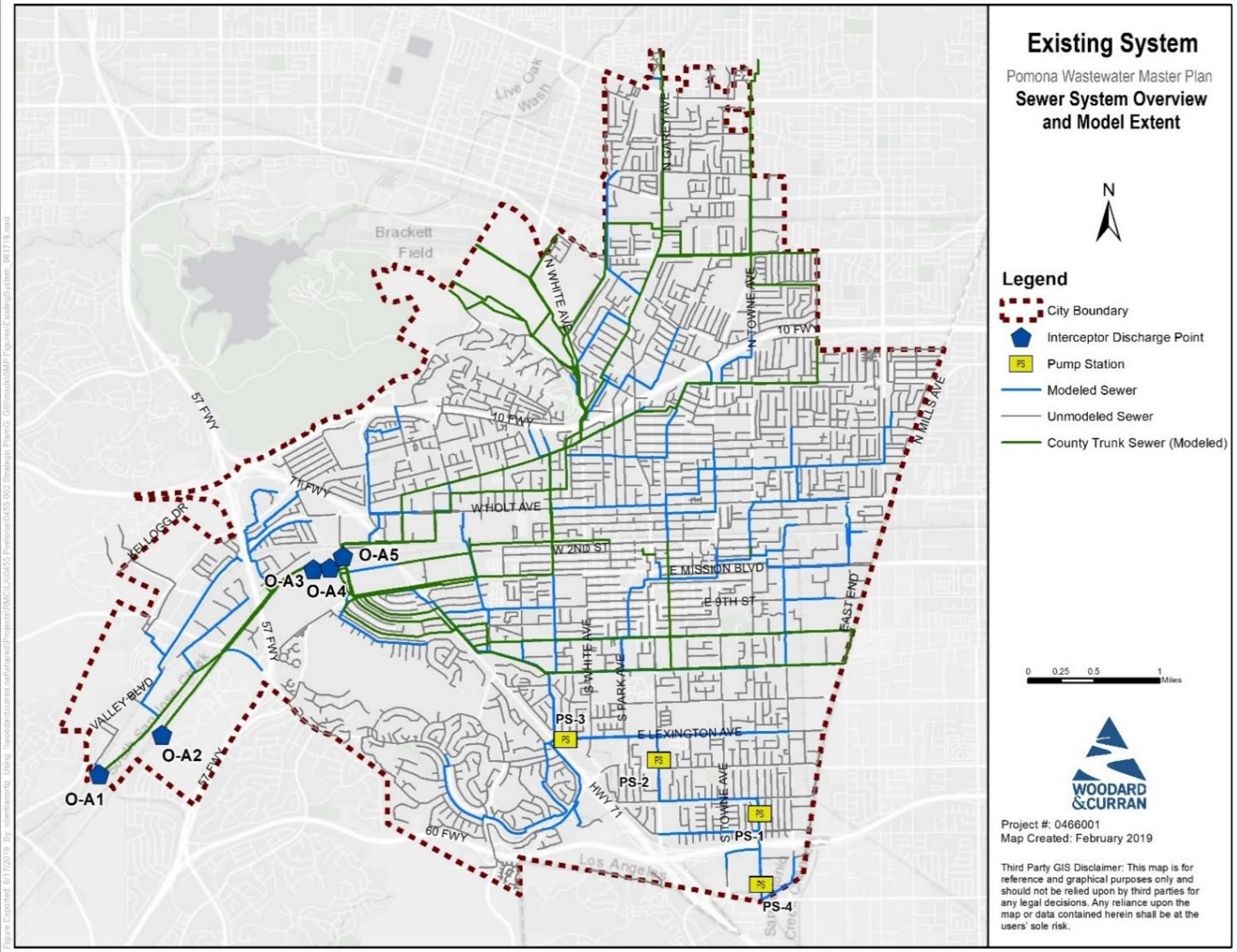
- Rolling Ridge Estates area along Rock Crest Lane and Scenic Ridge Drive. The City of Chino Hills serves these properties and bills the City quarterly for sanitary sewer collection and annually for wastewater treatment.
- Philips Ranch area along Rancho Laguna Drive and W. Temple Avenue. This area discharges into a 10-inch sewer owned and operated by the City of Diamond Bar.

In addition, various properties are connected to the City's sanitary sewer collection system but are not served by the City's water distribution system. These properties include 15 commercial accounts and 3 residential trailer parks. These receive water from the Walnut Valley Water District. The District provides the City with the water consumption data for sewer billing purposes.

As shown on **Figure 3-1,** the Los Angeles County Sanitation Districts (LACSD) owns, operates, and maintains four pump stations in the southern section of the City. Pump stations 1 and 4 discharge to a 21-inch gravity line at South San Antonio Avenue. The 21-inch then travels west along E. Olive Street which transitions into a 27-inch and then turns North along S. Garey Avenue, eventually discharging into pump station 2. Pump station 2 then pumps flow North along S. Garey Avenue to a 21-inch gravity line. The 21-inch discharges into a 30-inch traveling west along W Lexington Avenue and eventually discharges to pump station 3. Pump station 3 pumps flow North along S. Hamilton Boulevard and discharges into a Los Angeles County Sanitation District (LACSD) 42-inch trunk sewer at W. Philips Boulevard.

The City's wastewater collection system discharges to LACSD interceptors at several locations, as shown in **Figure 3-1**. The City's wastewater is ultimately conveyed to the LACSD Pomona Water Reclamation Plant (PWRP) for treatment and disposal. The PWRP is located at 295 Humane Way near the western edge of the City, just east of State Route 57 and north of the Phillips Ranch area. Wastewater from the neighboring cities of La Verne and Claremont is also treated at PWRP. Because the PWRP lacks capacity, wastewater is sometimes diverted to other LACSD facilities for treatment and disposal. The City is located in Los Angeles County that is served by LACSD. LACSD consists of 26 separate districts, of which the City is located in District No. 21.

Figure 3-1: Existing Collection System



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3.2 City Facilities

The City's wastewater collection system, shown in **Figure 3-1**, includes over 1.8 million feet of sewer pipelines (approximately 343 miles) ranging from 4 to 42 inches in diameter. Most City pipes are 8 inches in diameter, comprising about 75 percent of the total length of pipe. **Table 3-1** itemizes the length of City-owned pipe by diameter as recorded in the City's Geographic Information System (GIS).

Table 3-1: Gravity Sewers by Size

Diameter (in.)	Length (ft.)	Percent
4	3,600	0.2%
6	11,600	0.6%
8	1,357,400	75.0%
9	300	0.0%
10	69,800	3.9%
12	118,700	6.6%
14	25,700	1.4%
15	62,000	3.4%
16	3,000	0.2%
18	17,500	1.0%
20	3,500	0.2%
21	38,100	2.1%
22	4,100	0.2%
24	15,400	0.9%
27	25,200	1.4%
30	4,300	0.2%
33	6,800	0.4%
36	23,200	1.3%
39	6,500	0.4%
42	13,600	0.8%
Total	**1,810,300**	**100%**

Table 3-2 shows the distribution of the City's gravity lines by material. Most of the pipe in the City is vitrified clay pipe (VCP), which accounts for roughly 90 percent of the gravity system.

The City of Pomona was incorporated in 1888, and according to the City's GIS, the oldest recorded sewers were installed in 1900. **Table 3-3** shows the distribution of the City's sewer pipes by age. Approximately 75 percent of the sewers are known to be older than 50 years. **Figure 3-2** illustrates the ages of sewers throughout the City. As can be seen from **Table 3-3** and **Figure 3-2**, most of the City's sewers were built in the 1950s and 1960s. Over half of the sewers (around 53 percent) were built between 1950 and 1969 and are now approximately 50 to 70 years old. There are two confirmed siphon locations within the City, and one that is not confirmed. The siphon locations are as follows:

<u>LACSD</u>

- Southview Pl. & Humane Wy. (G201886, G201887) – FB-1200C, Operations Verified

<u>City</u>

- Valley Blvd. near Thompson Creek, (C251540, C251541) – FB-559, Operations Verified
- Fulton Rd. & Bonita Ave., (K8482, K8483) – FB-459, Not Verified

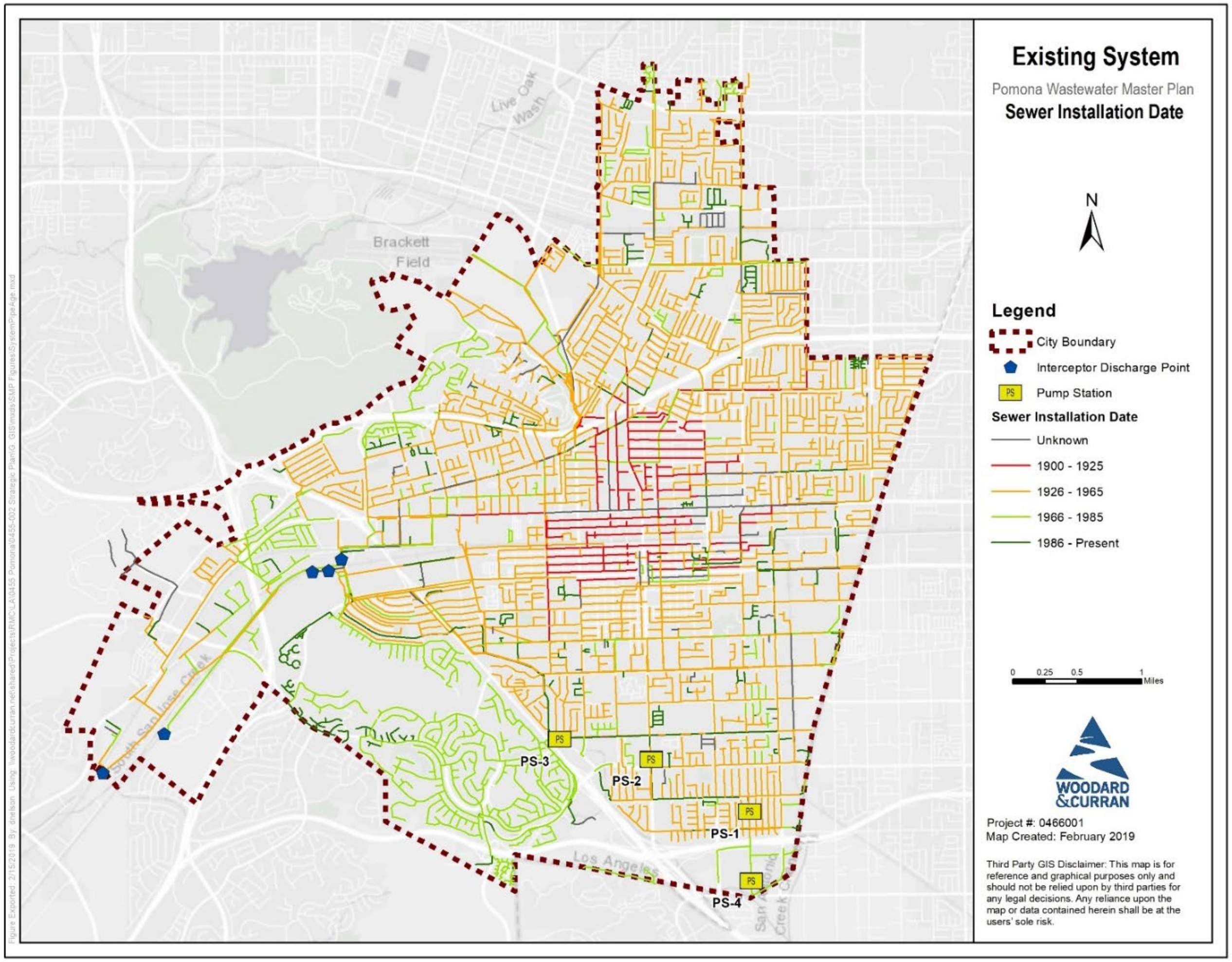
Table 3-2: Gravity Sewers by Material

Material	Length (ft.)	Percent
Acrylonitrile Butadiene Styrene (ABS)	4,700	0.3%
Asbestos Cement (ACP)	33,900	1.9%
Cast Iron (CI)	4,800	0.3%
Ductile Iron (DI)	1,100	0.1%
Reinforced Concrete (NRC – To Confirm)	4,900	0.3%
Polyvinyl Chloride (PVC)	2,700	0.1%
Reinforced Concrete Pipe (RCP)	103,800	5.7%
Steel (STL)	500	0.0%
Vitrified Clay Pipe (VCP)	1,633,500	90.2%
Other/Unknown	19,900	1.1%
Total	**1,810,300**	**100%**

Table 3-3: Estimated Age of Gravity Sewers

Installation Year	Length (ft.)	Percent
1900 - 1909	55,100	3.0%
1910 - 1919	2,700	0.1%
1920 - 1929	159,500	8.8%
1930 - 1939	65,100	3.6%
1940 - 1949	116,200	6.4%
1950 - 1959	624,400	34.5%
1960 - 1969	334,600	18.5%
1970 - 1979	125,300	6.9%
1980 - 1989	172,100	9.5%
1990 - 1999	45,000	2.5%
2000 - 2009	62,400	3.4%
2010 - 2019	2,300	0.1%
Unknown	45,700	2.5%
Total	**1,810,300**	**100%**

Figure 3-2: Age of Collection System



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The City's sewer system is served by four pump stations that are owned, operated, and maintained by the Los Angeles County Sanitation Districts (LACSD) per the 2012 LACSD "Takeover Agreement" (Resolution 2012-162). The pressurized force mains associated with Pumping Plants 1, 2, 3 are owned and maintained by the City. Pumping Plant 4 and the associated replacement redundant force mains are owned, operated, and maintained by the LACSD. As shown on **Figure 3-1**, the pump stations are numbered 1 through 4. **Figure** 3-3 is a flow schematic of the pump stations. As shown on this figure, pump stations 1 and 4 flow into pump station 2. Pump station 2 then re-lifts these flows into pump station 3, which finally discharges flows into a 42-inch LACSD trunk at W. Philips Boulevard. Pump stations 1, 2, and 3 are operated with variable speed pumps. Pump station 4 is operated with constant speed pumps. **Table 3-4** provides more detailed information on each pump station.

Figure 3-3: Pump Station Flow Schematic

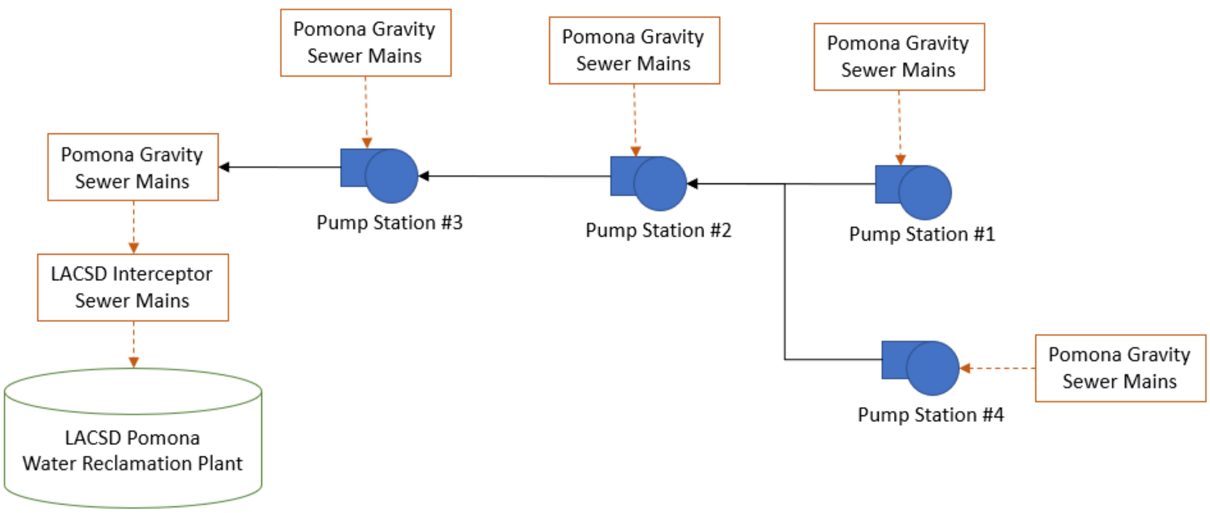


Table 3-4: Pump Station Characteristics

	PS #1	PS #2	PS #3	PS #4
Location	2394 S. San Antonio Ave.	2070 S. Garey Ave.	1026 W. Lexington Ave.	2800 Ficus St.
Year Built or Upgraded	1993 (originally built in 1953)	1995 (originally built in 1953)	2002 (originally built in 1953)	1967
Number of Pumps	2	3	3	3
Pump Type	Variable Speed	Variable Speed	Variable Speed	Constant Speed
Motor Power (hp)	25	58	64	20
Approximate Capacity of One Pump	1,500 gpm	3,000 gpm	4,040 gpm	475 gpm
Approximate Area Served	190 net acres, primarily residential	400 net acres, primarily residential, plus flow from PS #4 and PS #1	1,680 net acres, primarily residential, plus flow from PS #2	150 net acres, primarily industrial
Notes	Equipped with onsite emergency generator.	Replaced station at the intersection of Garey and Philadelphia. Does not have on-site emergency generator. Pump No. 2 assumed to have been installed per the reference document provided by the City (LACSD Lift Station – Set Points.pdf).	Replaced station at 1624½ S. Hamilton. Equipped with onsite emergency generator.	The third (emergency standby) pump has a natural gas engine.

3.3 Los Angeles County Sanitation Districts Facilities

The City is one of 78 cities located in Los Angeles County that is served by LACSD. LACSD consists of 24 separate districts, of which the City is in District No. 21. Wastewater collected by the City's sewer system discharges to LACSD trunk mains at multiple locations. None of the City's connection points is metered to determine the volume of wastewater being transported to the LACSD system. All of Pomona's wastewater is treated and disposed of by LACSD at their PWRP, located at 295 Humane Way near the western edge of the City, just east of State Route 57 and just north of the Phillips Ranch area. Wastewater flow from the neighboring cities of La Verne and Claremont is also treated at the PWRP. However, flow exceeding 15 million gallons per day (mgd) is routinely diverted to the LACSD Joint Water Pollution Control Plant in Carson. It should be noted that LACSD interceptors were not analyzed in this study.

4. SEWER SYSTEM MODEL DEVELOPMENT

An updated hydraulic model of the City's service area and major sewers was the tool used in this study to estimate flows and assess sewer capacities. This section describes the model development process, including selection of the sewers included in the model, the development and validation of the required data for the modeled sewers, and the delineation of sewer sub-catchments (areas tributary to the model system) used to define flow inputs into the model.

4.1 Modeling Software

The City's previous sewer collection system model was developed in 2005 using H_2OMAP Sewer Pro. For this master plan, the model was reconstructed using the most recent GIS data to reflect the City's existing facilities. The updated sewer model developed for this study was built using InfoWorks™ ICM, a standalone GIS-based hydraulic model developed by Innovyze, then converted to InfoSewer for future use by the City. The model provides a robust hydraulic engine which simulates time-varying flows and depths throughout the model network.

4.2 Model Construction

The modeled collection system consists of links and nodes, which represent the major pipes, manholes, pumps, and pump station wet-wells. The service area is divided into sub-catchments, each of which defines the tributary area to a node on the modeled system. Sub-catchment parameters define flows entering the collection system, including sanitary flow, industrial/commercial flow, groundwater infiltration (GWI), and rainfall-dependent inflow and infiltration (RDI/I).

During the initial phase of the study, the model included all pipes derived from the City's GIS. However, after a detailed review of the GIS data and significant effort to clean and validate the data, the project team and the City staff decided to construct a model of the trunk network based on pipe sizes of 10 inches and greater. Areas not served by the trunk network were modeled using selected smaller (8-inch) pipes. The resulting trunk model network includes approximately 28 percent of the system (about 515,000 of 1,865,000 linear feet) and provides sufficient coverage to evaluate and identify capacity issues and develop appropriate capacity driven solutions. The model was developed using the following methodology; additional details are presented in subsequent sections of this report:

- Establish the extent of the model network (pipes 10-inches and larger in diameter and smaller pipes serving relatively large areas or downstream of larger diameter pipes and major flow splits), adjust elevations to a common datum, and validate data (i.e., check for and correct incomplete or erroneous data values).
- Add pump station information and settings.
- Divide the City's sewer service area into sub-catchments and define loading points for those sub-catchments in the model.

Collection System

Using GIS data provided by the City, a trunk model network was developed. The raw GIS data was reviewed and updated to include all sewer inverts and ensure model connectivity. The GIS data was then imported to InfoWorks™ ICM and a model validation was conducted including the following:

- **Connectivity checks –** The modeled networks were checked for connectivity, which includes verifying that correct upstream/downstream manholes were identified for each pipe, with no missing links or nodes in the network. A connected network means that all pipes and manholes will be selected when the network is traced upstream from the model outfalls.
- **Profile review –** Profiles were plotted for each series of pipe segments in the modeled network to visually check for suspect data. Examples of suspect data include negative pipe slopes, abrupt steps up or down in pipe inverts, and pipe diameters that conflict with surrounding pipes. Where appropriate, corrections to suspect data were inferred. Otherwise, verification in the form of as-built drawings or field investigations were

requested from the City, or additional survey was performed. The model was developed using the NAVD88 vertical elevation datum.

- **Special structures –** Flow splits (manholes with more than one outlet pipe) were identified for further verification of outlet pipe elevations and/or the existence of weir overflows or other control structures. Field verification and/or as-built drawings were requested from the City as needed.

Data describing the modeled collection system elements are included in the model database. A summary of the modeled system characteristics is provided in **Table 4-1**.

Table 4-1: Model System Characteristics

Model Characteristic	Value
Number of manholes	1,629
Number of pump stations	4
Number of outlets (free outfalls) to LACSD interceptors	5
Total modeled pipe length	97.5 miles
Range of pipe diameters	8 to 42-inch

Pump Stations

There are four pump stations serving the City's collection system, all of which are owned, operated, and maintained by LACSD. Information about pump on/off levels and sizes, wet well elevations and dimensions, force main invert elevations and lengths, and normal operating flow rates were provided by the LACSD and added manually after the network GIS data was imported to the model. A summary of the modeled pump stations is included in **Table 3-4**.

Sewer Sub-catchments

Sewer sub-catchments, areas contributing flow to the modeled system, were delineated based on City parcels. Sub-catchment parameters define flows entering the collection system, including sanitary flow, industrial/commercial flow, groundwater infiltration (GWI), and rainfall-dependent inflow and infiltration (RDI/I).

Using the City's complete pipe network, parcels were assigned to a loading manhole using sewer lateral and pipeline GIS data. After the initial parcel assignment, the network was synthesized down to the trunk system and the sub-catchment assignments were traced downstream and reassigned to the first downstream trunk (modeled) manhole. Once loading assignments for the trunk system was complete, parcels tributary to common manholes were combined to create sewer sub-catchments. Model loads, including contributing area for GWI and RDI/I, were consolidated from each of the contributing parcels into the sewer sub-catchments.

5. EXISTING AND FUTURE WASTEWATER FLOWS

This section summarizes the development of the wastewater collection system flows for existing and future conditions.

5.1 Wastewater Flow Components

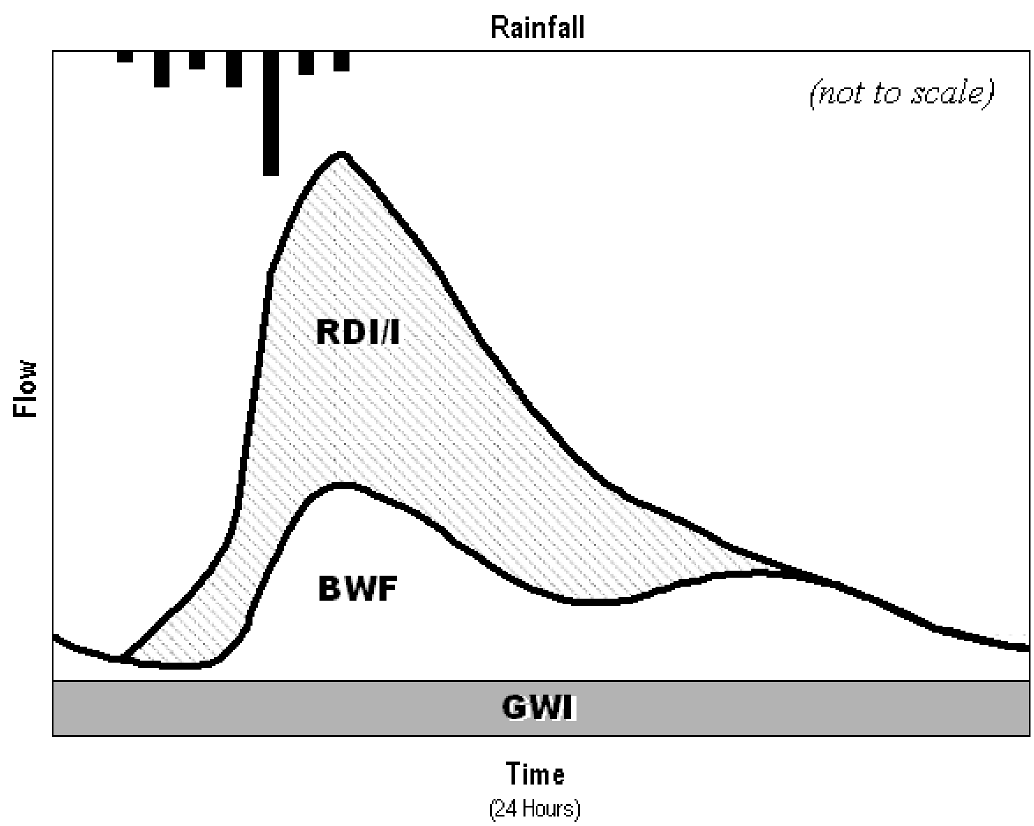
Wastewater flows include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I), as illustrated conceptually in **Figure 5-1**.

BWF represents the sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system. BWF varies throughout the day, but typically follows predictable diurnal patterns depending on the type of land use.

GWI is groundwater that infiltrates into defects in sewer pipes and manholes, particularly in winter and spring in low-lying areas. GWI is typically seasonal in nature and remains relatively constant during specific periods of the year. However, rainfall typically has long-term impacts on GWI rates, as evidenced by measurable increases in GWI after prolonged periods of rainfall.

RDI/I is storm water inflow and infiltration that enter the system in direct response to rainfall events, either through direct connections such as holes in manhole covers or illegally connected roof leaders or area drains, or, more commonly, through defects in sewer pipes, manholes, and service laterals. RDI/I typically results in short term peak flows that recede relatively quickly after the rainfall ends. The magnitude of RDI/I flows are related to the intensity and duration of the rainfall, the relative soil moisture at the time of the rainfall event, and the condition of the sewers.

Figure 5-1: General Wastewater Flow Components



5.2 Dry Weather Flow Development

Current and projected water demands, developed for the City's 2019 Water Master Plan, provided the basis for dry weather sewer flow estimates. The following sections provide a summary of how current and projected sewer flows were developed for the hydraulic model.

Current Flow Development

The City provided meter-billing data for every potable service connection from fiscal year (FY) 2013-2014 through FY 2015-2016 (summarized in **Table 5-1**). This data was georeferenced according to meter location or addresses provided by the City. Four water user classifications are used in the City's billing data. Typically, a portion of the potable water usage is returned to the sewer system with return rates ranging between 80 and 90-percent. For this study, a 90-percent return rate was applied to the water billing data for winter usage to convert from water consumption to sewer flows. The City's major water users were identified based on the average FY2013/2014 to FY 2015/2016 records to determine high demand areas in the service area and were used to identify potential industrial process discharges to the sewer or other uses that may contribute large discharges to the sewer.

Table 5-1: Metered Potable Water Consumption by User Classification

User Classification	FY 2013-14 (AF)	FY 2014-15 (AF)	FY 2015-16 (AF)	Avg. FY 2013-14 to 2015-16 (AFY)	% of Total
Single Family Residential	10,178	9,534	7,613	9,108	53%
Multi-Family Residential	4,033	3,844	3,410	3,762	22%
Commercial	3,357	3,382	2,951	3,230	19%
Irrigation[1]	1,303	1,285	793	1,127	7%
Total Metered Consumption	**18,871**	**18,045**	**14,767**	**17,228**	**100%**

1. Irrigation water usage was not considered for sewer flow estimates.
2. AF – acre feet / AFY – acre feet per year

Projected Flow Development

Projected flows, similar to current flows, were based on projected water demands developed for the Water Master Plan. Water demands for the City of Pomona were projected through the year 2040 to align with the City's 2015 Urban Water Management Plan (UWMP). The 2015 UWMP provided demand projections, developed by City staff, based on population projections prepared by the Southern California Association of Governments (SCAG) and assume that potable water demand in all sectors will increase at the same rate as population, which is estimated at approximately 5.1 percent increase every 5 years. **Table** 5-2 provides the 2015 UWMP projections.

Table 5-2: Demand Projections from the City of Pomona 2015 UWMP (AFY)

	FY 2014/ 2015[1]	FY 2019/ 2020	FY 2024/ 2025	FY 2029/ 2030	FY 2034/ 2035	FY 2039/ 2040	Incremental Demand 2015 to 2040
Single Family	9,607	10,097	10,612	11,153	11,722	12,320	2,713
Multi-Family	3,847	4,043	4,249	4,466	4,694	4,933	1,086
Commercial	5,358	5,632	5,919	6,221	6,538	6,872	1,514
Landscape	1,246	1,310	1,376	1,447	1,520	1,598	352
Total	**20,058**	**21,082**	**22,156**	**23,287**	**24,474**	**25,723**	**5,665**

The process used to geographically distribute the incremental demand for the purposes of hydraulic modeling shown in the above table assumes that demand will increase according to three categories:

1. **Near-term planned developments:** Developments to be constructed by 2020 that have been approved by the City and have demand projections either provided by developers or estimated by the City.
2. **Vacant parcel development:** Developments to occur on currently vacant parcels (infill) where demand can be estimated based on future land use.
3. **Densification:** Increases in demand attributed to densification of population in residential areas and commercial/industrial activities; the process used to estimate and distribute these estimates.

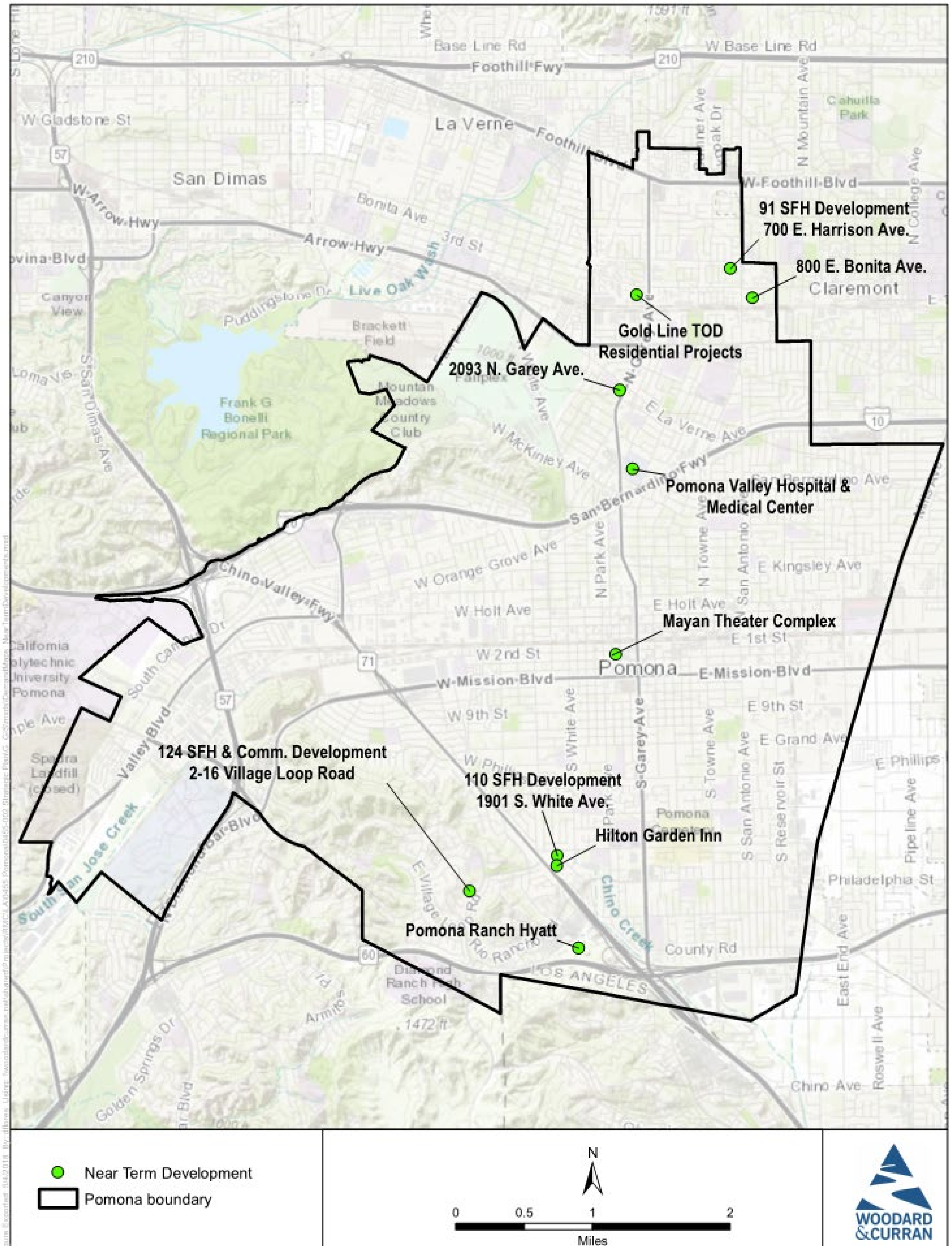
Near-term planned development locations and water demands were provided by City staff and are expected to be completed by 2020. **Table 5-3** provides a summary of the developments, land uses, estimated average day demand, current demand at the site, and incremental demand, while Figure **5-2** shows the locations of each near-term development project. Given that some of these developments are to be constructed on existing sites with previously metered consumption, the average amount billed between FY 2013-2014 and FY 2015-2016 for each site was subtracted from the projected average demand for each project. For those near-term development projects that do not have previously metered consumption onsite, the previously metered consumption is zero in **Table 5-3**.

The incremental demands for near-term developments (1,046 AFY as shown on **Table 5-3**) were subtracted from the 2015 UWMP projections (5,665 AFY as shown on **Table 5-2**) to obtain the remaining incremental demand (4,619 AFY) to be used in the processes described in the following sections.

Table 5-3: Near-Term Development Incremental Water Demands

No.	Project Name	Land Use	Projected Avg. Demand (AFY)	Metered 3-yr Avg. FY2013-14 to FY2015-16 (AFY)	Incremental Demand (AFY)
1	124 SFH & Comm Dev. 2-16 Village Loop Rd	Single Family Residential	46	2	44
2	110 SFH Development 1901 S. White Ave.	Single Family Residential	41	0	41
3	91 SFH 700 E. Harrison Ave.	Single Family Residential	35	0	35
				Single Family Residential Subtotal	**120**
4	2093 N. Garey Ave.	Multi-Family Residential	12	0	12
5	Gold Line TOD Residential Projects	Multi-Family Residential	242	0	242
6	800 E. Bonita Ave.	Multi-Family Residential	53	0	53
				Multi-Family Residential Subtotal	**307**
7	Pomona Ranch Hyatt	Commercial	290	0	290
8	Hilton Garden Inn	Commercial	258	0	258
9	Maya Cinemas	Commercial	47	3	44
10	Pomona Valley Hospital	Commercial	192	165	27
				Commercial Subtotal	**619**
				Total	**1,046**

Figure 5-2: City of Pomona Near-Term Developments



Third Party GIS Disclaimer: This map is for reference and graphical purposes only and should not be relied upon by third parties for any legal decisions. Any reliance upon the map or data contained herein shall be at the users' sole risk. **Data Sources:** City of Pomona, LA LAFCO, ESRI

The estimation of water demand projections for future development of vacant parcels (infill) are based on water demand factors (WDF). WDF are the average daily water use of a given land use type and have been calculated using the City's land use data and geocoded billing records. Given that billing data is only classified as single family residential, multi-family residential, commercial, and irrigation, it was necessary to develop more detailed land use types for each meter by using the land use of the underlying parcel for each meter. WDFs for each land use type was calculated by summing the average FY 2013/2014 to FY 2015/2016 metered use for each land use, then dividing by the total acreage by land use. The WDFs calculated for this demand analysis and the WDFs calculated for the 2005 Pomona WMP are shown in **Table** 5-4. The current WDFs for the City are lower than those used in the 2005 Pomona WMP, which is assumed to be due to a combination of prolonged drought that led to implementation of water conservation mandates as well as increases in the use of irrigation meters (reflected as lower demand volumes for water users such as multi-family residential and C-2 land uses).

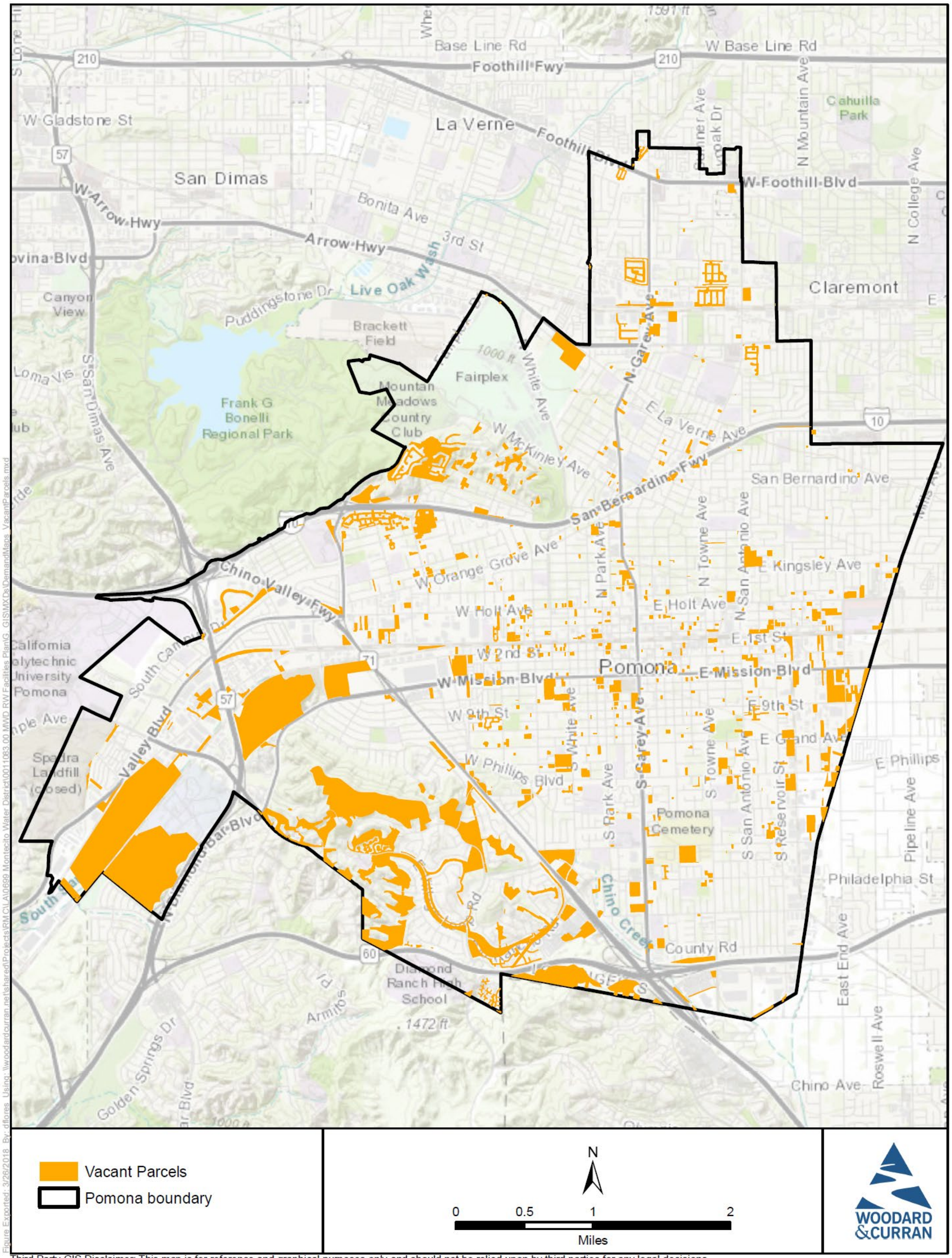
Table 5-4: Current Water Demand Factors Compared to 2005 Pomona WMP

<table>
<tr><th>Land Use</th><th>2005 Pomona WMP WDFs (gpd/acre)</th><th>Current Pomona WDFs, average FY 2013-14 to FY 2015-2016 (gpd/acre)</th></tr>
<tr><td>Single Family Residential</td><td>2,600</td><td>2,100</td></tr>
<tr><td>Medium Density Residential</td><td>6,000</td><td rowspan="2">3,400[1]</td></tr>
<tr><td>High Density Residential</td><td>9,200</td></tr>
<tr><td>Commercial and Services</td><td>2,400</td><td>1,900</td></tr>
<tr><td>Mixed Residential and Commercial</td><td>Not used</td><td>1,700</td></tr>
<tr><td>General Office</td><td>2,600</td><td>1,400</td></tr>
<tr><td>Industrial</td><td>2,000</td><td>900</td></tr>
</table>

1. Medium-density and high-density residential land uses were not included as land uses in the land use data provided by the City Planning Department.

The calculated WDFs were used to estimate the future demands of the vacant parcels by multiplying the appropriate WDF by the parcel acreage, assuming the zoning listed in the vacant parcel shapefile provided by the City's Planning Department is reflective of future land use and that all currently vacant parcels will be developed by 2040. As shown in Figure **5-3**, vacant land is evenly distributed throughout the City as opposed to large, assembled areas suitable for large developments. Per information provided by the City, only vacant parcels zoned for residential, commercial, or industrial use will be developed; additionally, development will not occur on any parcels identified as alleys, ROW, utility corridors, open space, or owned by the City.

Figure 5-3: City of Pomona Vacant Parcels



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The remaining incremental demand after near-term developments and vacant parcel developments is assumed to be attributed to densification of the City's population and therefore can be distributed according to land use and area. The demand associated with densification was distributed based on land use of each parcel and the parcel area to all parcels. **Table 5-5** summarizes the demands associated with each distribution method.

Future sewer flows were derived from the projected water demands distributed across the service area based on vacant land and future development projects. The total incremental demands were converted to sewer flows by applying a 90-percent sewer return rate as used for the existing sewer flows. The resulting incremental sewer flows were added to the existing dry weather sewer flows which were previously calibrated against measured flow data.

Table 5-5: Categories of Demand by Land Use and Method for Geographic Distribution

	Near-Term Development Demands (AFY)	Projected Demand Associated with Vacant Parcel Development (AFY)	Projected Demand Associated with Densification (AFY)	Total Incremental demand between 2015 and 2040 from Table 5-2 (AFY)
Single Family	120	226	2,367	2,713
Multi-Family[1]	307	195	584	1,086
Commercial[2]	619	365	530	1,514
Landscape[3]	0	0	352	352
Total	**1,046**	**786**	**3,833**	**5,665**

1. Multi-Family includes the following land uses: "multi-family residential", "mixed residential and commercial", and "mobile homes and trailer parks".
2. Commercial includes the following land uses: "commercial", "education", "facilities", "general office", "industrial", and "mixed commercial and industrial".
3. Landscape demands were not added to the sewer model as future flows. This demand was provided in the parallel table in the Water Master Plan and included here for consistency.

5.3 Flow Monitoring Program

Flow monitoring data is used to calibrate the hydraulic model by comparing model predictions with observed flow and depth data for dry and wet weather flow conditions. As part of this study, a short-term temporary flow monitoring program was conducted for a 6-week period from January 11, 2017 through February 27, 2017 which included the collection of rainfall data from three rain gauges. Based on recommendations from the City and evaluation of competitive bids, the flow monitoring was conducted by Utility Systems, Science and Software, Inc. (US3) under subcontract to Woodard & Curran. US3 installed Hach Flo-Dar® AV flow meters at ten (10) locations and collected 15-minute data from the flow meters. A detailed flow monitoring report consisting of flow meter installation sheets; flow, velocity, and depth graphs; and tables of weekly statistics (e.g. average, maximum, and minimum flow, velocity, and depth) is provided in **Appendix A**.

The Hach Flo-Dar® AV flow meter uses a combination of radar and ultrasonic sensors to record velocity and depth values, respectively. The meter type utilizes a non-contact sensor mounted above the flow stream which prevents the need for installing velocity and pressure sensors within the flow. In addition, these meters reduce the need for regular maintenance during the flow monitoring period and allows easier access for maintenance and removal without requiring a confined space entry into the manhole. The meter also provide the ability to collect velocity measurements in very shallow flow, a condition in which some types of submerged sensors may not work effectively. The meters use a 'top-down' radar technology to obtain flow velocities from the water surface which in some cases may induce inaccuracies during turbulent flow patterns and floating debris. In addition, the average flow velocity is interpolated by the vendors post-processing software which may lead to inaccuracies with the final flow data. Evidence of the measured flow data is shown in the flow hydrographs presented in **Appendix A**.

Selection of Flow Meter Locations

Meter locations were selected with the intent of isolating upstream basins where possible. Upstream meters that isolate sewer drainage basins provide good information for calibrating model dry weather and wet weather flow inputs. Downstream meters provide verification of the model flows in those sewers, which is affected both by the estimated flow inputs as well as routing of the flow through the system (dictated by flow splits in the system). This flow metering effort did not include any downstream meters which means that a significant portion of the sewer system was not metered and could not be calibrated. **Table 5-6** lists the flow monitoring locations for the 2017 flow monitoring program and includes location (manhole ID), monitored pipe size, size of the contributing area, approximate average dry weather flows, and provides any comments on the metered area or data. **Figure 5-4** shows an overview of the flow meter and rain gauge locations and meter tributary areas. **Figure 5-5** provides a schematic of the combined flow meter sites, showing which meter sites are upstream of other meter sites. An example plot of flow meter and rainfall data (from FM10) is included in **Figure 5-6**.

Table 5-6: Flow Meter and Tributary Area Characteristics

Meter ID	Location (Manhole ID[a])	Pipe Size (in.)	Tributary Area (acre)	Approximate Average DWF (mgd)[b]	Comments
FM1	D21-5042	12	113	0.09	Mostly commercial/industrial area. Weekday diurnal markedly different from weekend diurnal.
FM3	G13-5003	15	498	0.32	
FM5	H20-6514	12	921	0.15	
FM6	J21-6458	12	1,464[c]	0.15	Slight drift up of flow troughs starting on January 30, 2017.
FM7	K24-3364	21	1,352[c]	1.31	Depth of flow during Jan. 22, 2017 storm reached the top of the pipe (21 inches). Suspect depth data as depth recorded maxed out at 21 inches for 2 hours and 15 minutes (from 1/22/2017 17:54 to 1/22/2017 20:09). Pipe surcharged during storm.
FM8	K22-6535	12	437	0.19	
FM9	L22-5292	18	360	0.78	
FM10	K24-3354	24	453	0.32	
FM12	K16-1784	15	692	1.22	
FM13	L11-2146	12	785	0.31	

a. MH ID is the manhole in which the meter was installed. The meter typically measures flow in an inlet pipe to the manhole.

b. Approximate average DWF from dry weather calibration period, January 29-February 4, 2017.

c. There are 854 acres upstream of a flow split that divides flow between FM 5 and FM 6. That acreage double counted in the tributary area listed for FM 5 and FM 6.

Figure 5-4: Flow Meter and Rain Gauge Locations

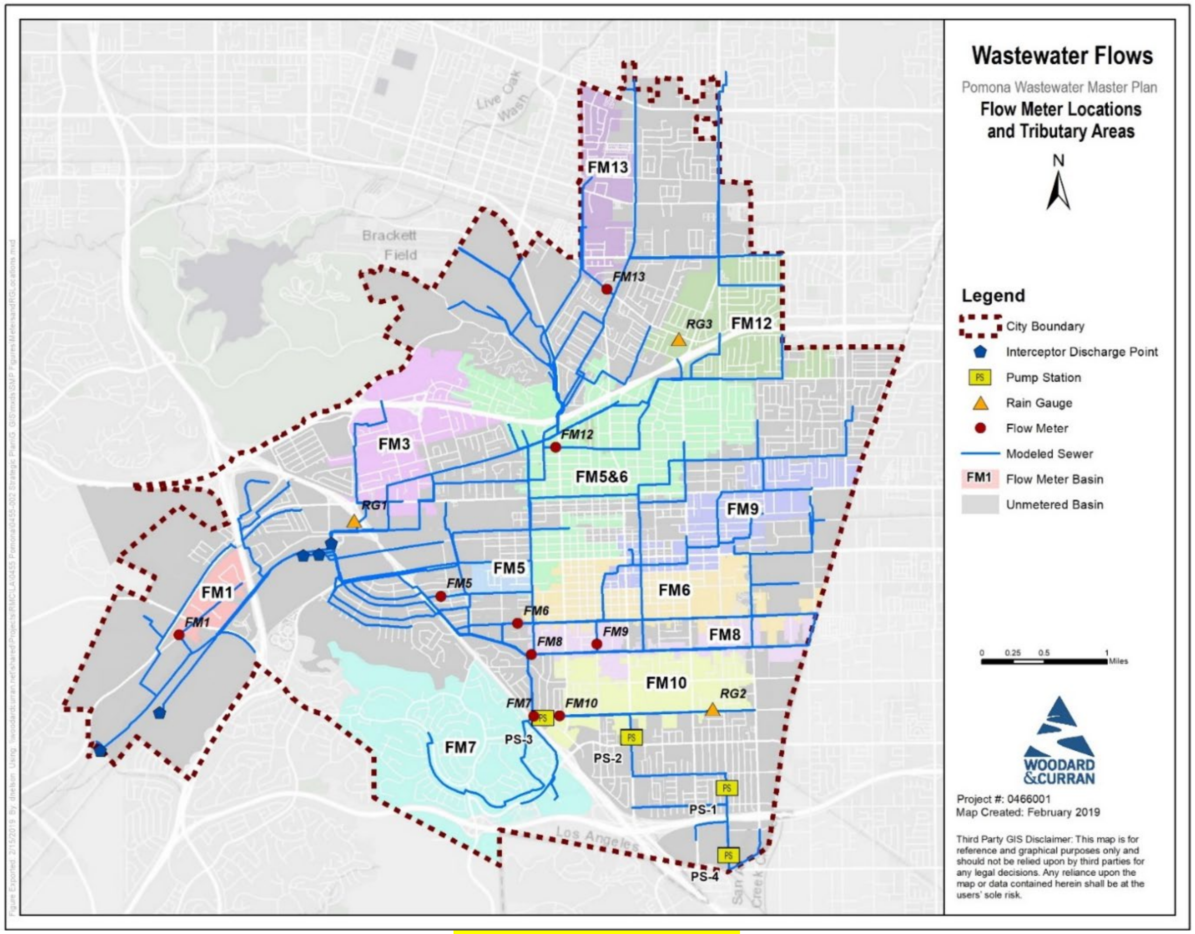

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Figure 5-5: Flow Meter Schematic

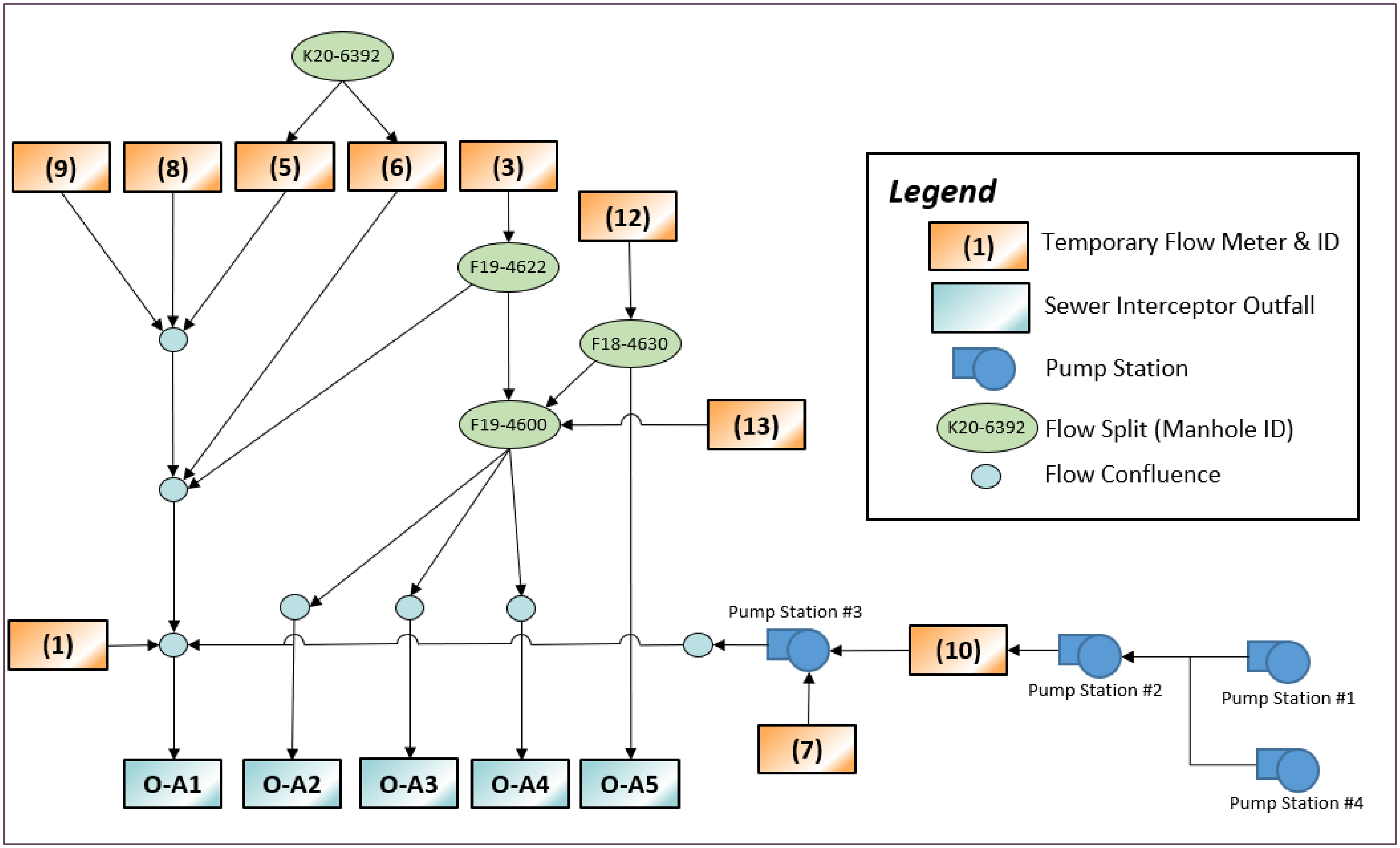
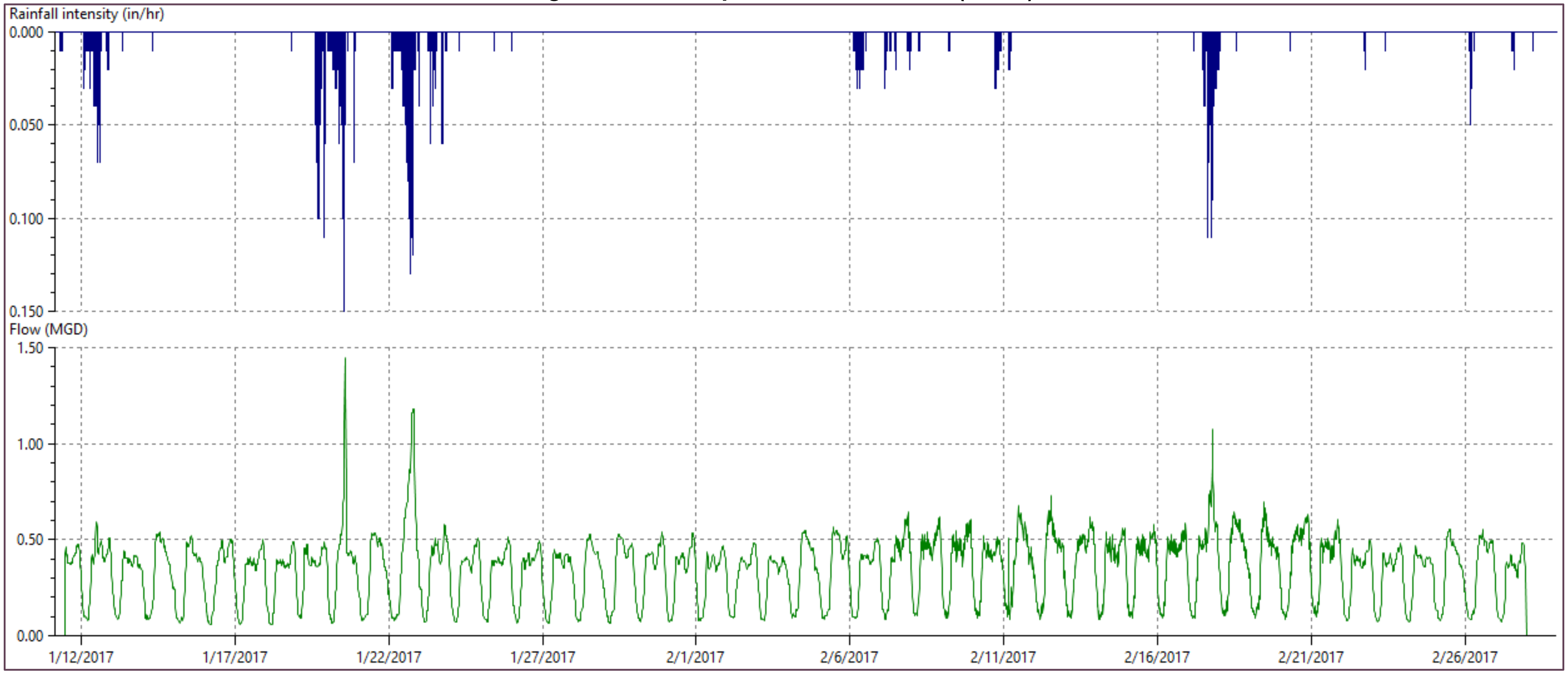


Figure 5-6: Example Flow Meter Data (FM10)



5.4 Model Calibration

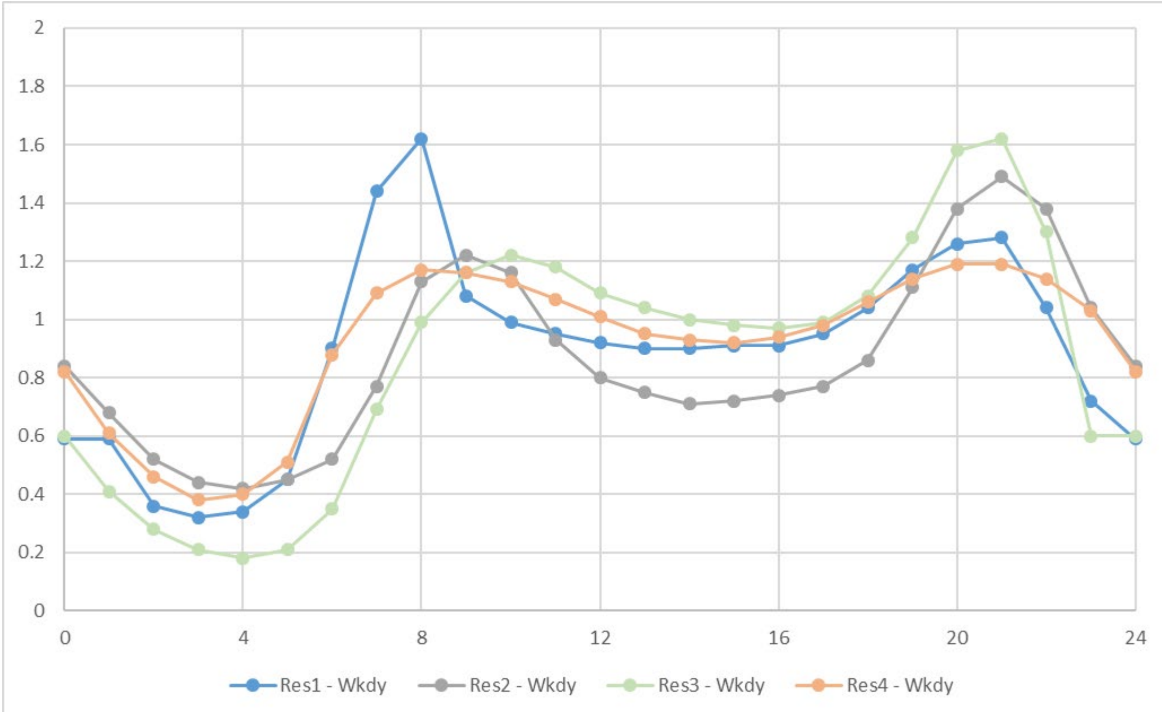
Model calibration is the process of comparing model-computed flows to observed (monitored) flows to verify that the model is accurately simulating flows in the sewer system. The model is calibrated for both dry and wet weather conditions. As described in Section 5.3, a temporary flow monitoring program was conducted in the City of Pomona system during the January through February 2017 wet weather period. The data collected during the flow monitoring program was used for model calibration.

Dry Weather Flow Calibration

The dry period from the end of January to early February 2017 was used as the dry weather calibration period for the model. The dry weather calibration process was used to verify base wastewater flow (BWF) loads and diurnal curves, and to quantify GWI (as indicated by monitored flows that were higher than estimated BWF).

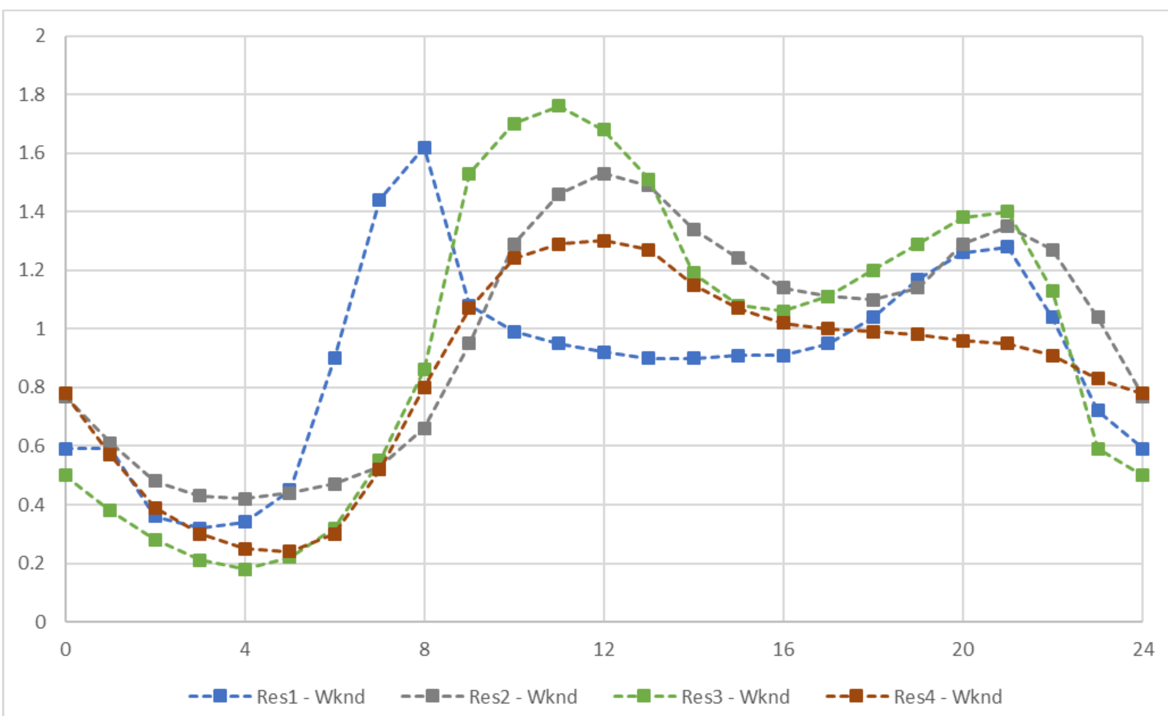
Diurnal profiles were developed based on flow meter data and then adjusted based on calibration results. **Figure 5-7**, **Figure 5-8**, and **Figure 5-9** show the final calibrated diurnal profiles for residential and non-residential flows, respectively. The curves show the flow multiplier (ratio of hourly flow to average daily flow) for each hour of the day. The peaking factor (maximum hourly flow/average daily flow) ranges from 1.2 to 1.8.The diurnal profiles for residential flows include a 90 percent return factor to account for the water demand that does not end up in the sewer (resulting in the average of the diurnal curve equaling 0.9). The different residential diurnal profiles differ in the magnitude and timing of the peak flow, ranging from a high peak occurring in the morning, a somewhat lower morning peak occurring in the morning, and a higher peak occurring in the evening. Residential curve 1 was developed from the previous master plan effort. In addition to fitting the flow data for the FM1 basin, this curve was applied as the default in areas where there was no downstream flow meter. Additional residential profiles were developed and applied based on the observed patterns from the flow meter data. **Figure 5-10** shows the sub-catchments where these different profiles were applied.

Figure 5-7: Calibrated Residential Weekday Diurnal Profiles



Note: These diurnal profiles include a 90% return factor to account for the water demand that does not end up in the sewer.

Figure 5-8: Calibrated Residential Weekend Diurnal Profiles



Note: These diurnal profiles include a 90% return factor to account for the water demand that does not end up in the sewer.

Figure 5-9: Calibrated Non-Residential Diurnal Profiles

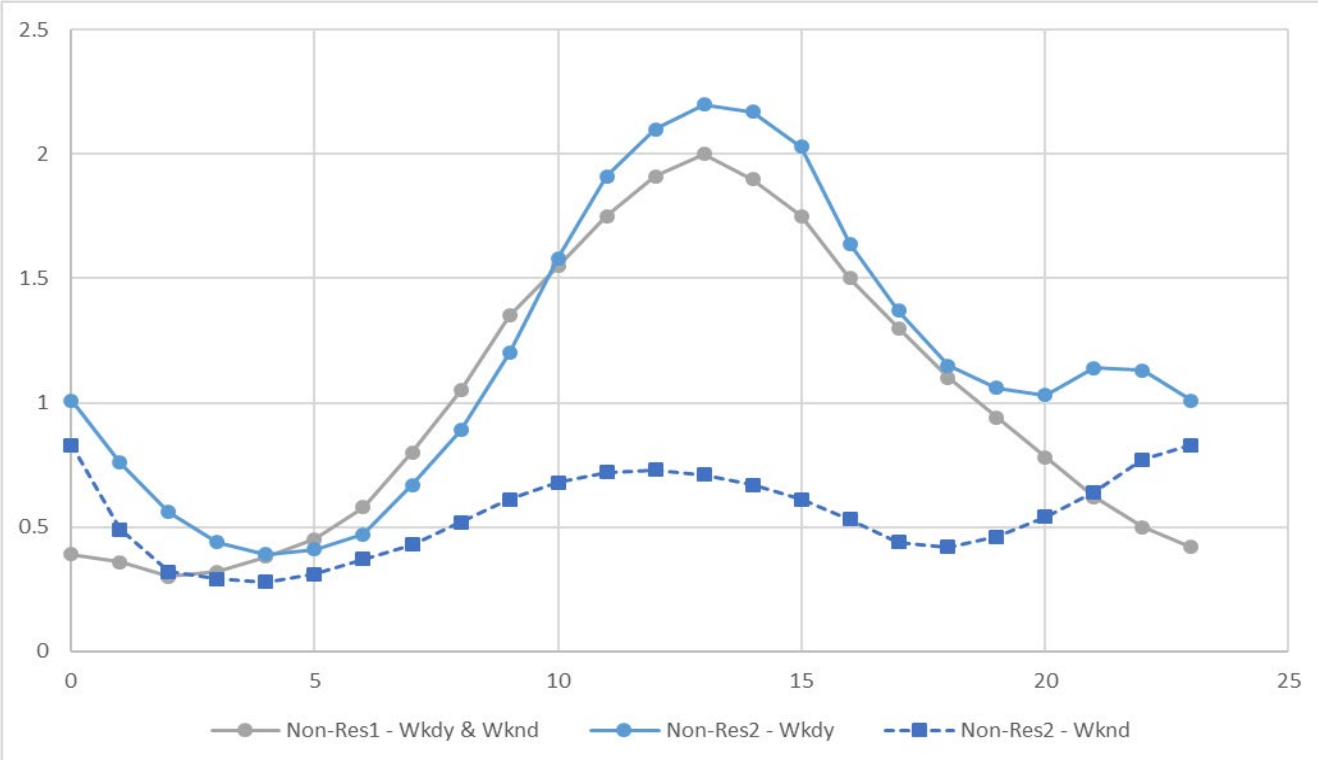
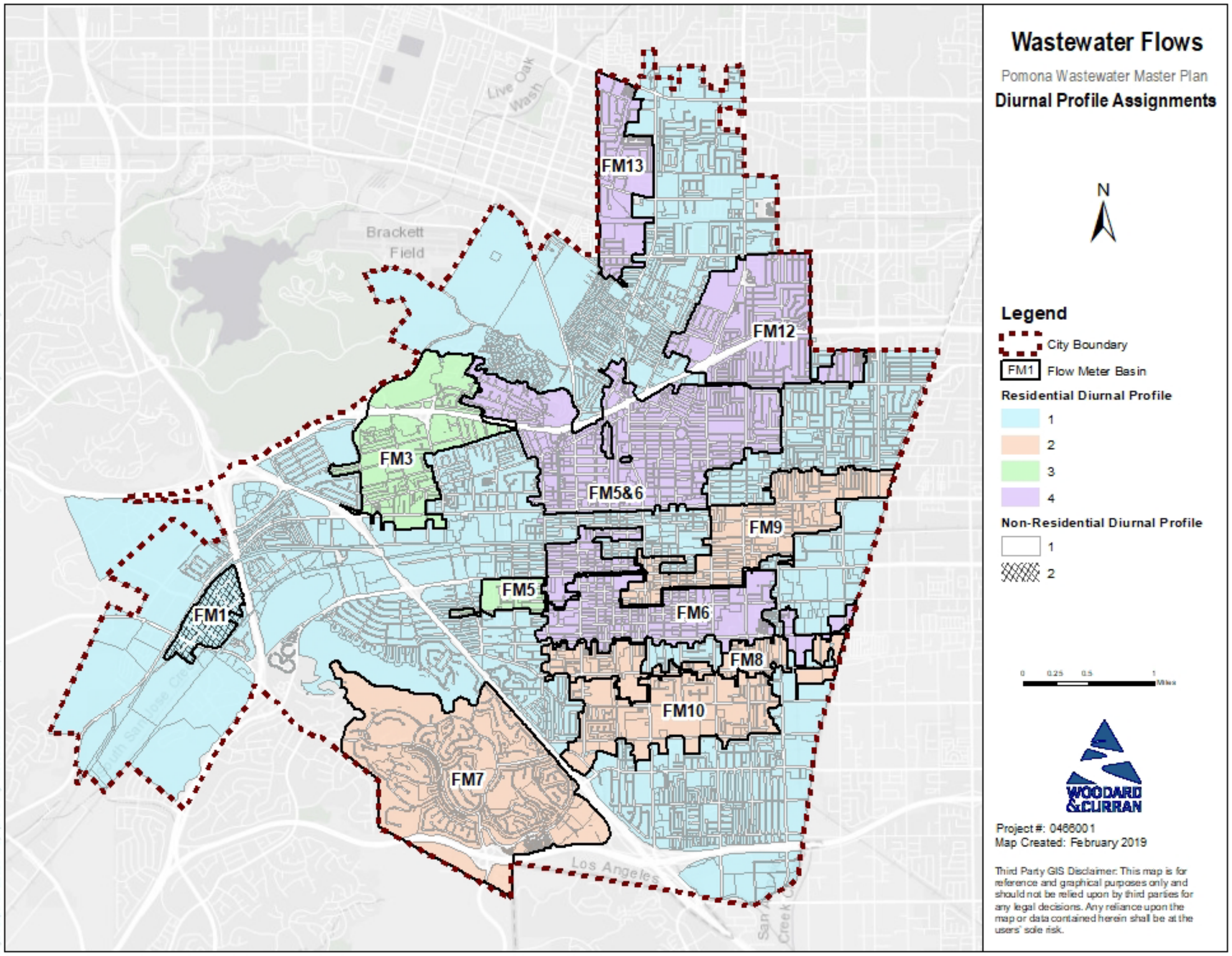


Figure 5-10: Diurnal Profile Assignments



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GWI was added to the model as a constant flow in addition to the BWF. The amount of GWI added was determined during dry weather flow calibration by comparing the modeled base flows to actual observed flows at the flow meter locations. The resulting GWI was expressed as a unit flow rate (e.g., gallons per day per acre [gpd/acre]) based on the sewered portion (called the "contributing area") of the area tributary to the flow meter (called the "meter basin"). This GWI rate was then applied to each sub-catchment's contributing area to generate the GWI contribution from that sub-catchment. The contributing area (i.e., area that potentially contributes GWI and infiltration/inflow [I/I]) for each sub-catchment was determined by subtracting the acreage of vacant land or open space land uses in the sub-catchment from the total sub-catchment area.

Figure 5-11 shows an example plot of model vs. metered flow for a single meter location in a residential area (meter 13). In this graph, the green line represents the monitored (observed) flow, and the red line is the model-simulated flow. The first and last days on this figure are weekend days, and the middle five are weekdays, illustrating how the residential flow pattern changes on the weekend. Dry weather calibration graphs for all meters are included in **Appendix B**. **Table 5-8** summarizes the dry weather loading parameters determined for each flow meter area during calibration. Estimated GWI rates for each flow meter area are indicated on **Figure 5-13**. Most flow meter areas had no GWI. Winter time GWI in remaining areas ranged from 350 to 650 gpd/acre. Flow meter area 7 had significant winter time GWI (650 gpd/acre).

Overall, the model calibration provided a satisfactory comparison between predicted flows and observed flow data for the flow meter sites. Minor differences remain for some sites, likely resulting from potential inaccuracies in flow meter data, insufficient or missing water billing data, or unaccounted for flow splits existing in the un-modeled system.

Wet Weather Flow Calibration

During wet weather calibration, parameters are adjusted to simulate the volume and timing of RDI/I for monitored storm events. Rainfall was assigned to parcels or sub-catchments based on which of three rain gauges the centroid of the sub-catchment was closest to. Through the wet weather calibration process, RDI/I hydrograph parameters were developed for each metered area. For calibration of the City's system, the rainfall period from January 19 to 26 was used to determine RDI/I parameters. This period had two storms: the first storm occurring on January 19 to 20 and the second storm on January 22 to 23, with the second storm generally having the highest peak flows. This is expected, as the antecedent conditions for the January 19 to 20 storm were relatively dry. The soils for the second storm were more saturated and generated a larger response. To calibrate to the observed peak and flow volume, RDI/I parameters were selected to best match the response to the January 22 to 23 storm. A summary of the major rain events observed during the flow monitoring period is provided in **Table 5-7.**

Table 5-7: Observed Rain Events During Flow Monitoring Period

Rain Event	Duration (Hours)	Total Rainfall (in.)	Peak Hour Rainfall (in./hr.)
January 12, 2017	15	1.68	0.30
January 19 -20, 2017	37	2.95	0.40
January 22 - 23, 2017	45	4.10	0.56
February 18, 2017	15	2.00	0.44

Note: The January 22-23 rain event was used for the wet weather calibration.

Table 5-9 summarizes the results of the wet weather calibration in terms of the flow response to the rain event (R values) assigned to each flow meter basin. An example wet weather calibration graph is presented in **Figure 5-12**. **Appendix B** contains copies of wet weather calibration graphs for all of the meters. Overall, most meters had relatively low R values, indicative of a tight system with newer pipes (see **Figure 5-14**). Flow meter areas 5, 7, and 9 did have total R values above 10 percent with the FM 7 area exhibiting the largest wet weather response with a total R of 22.5 percent. Further investigations, such as smoke tests or CCTV, may be appropriate in this area to identify potential sources of I/I (such as unauthorized stormwater discharge or leaking pipes or manholes) and any capacity concerns. The high R values in this area may also be indicative of pipes in poorer condition that may need rehabilitation.

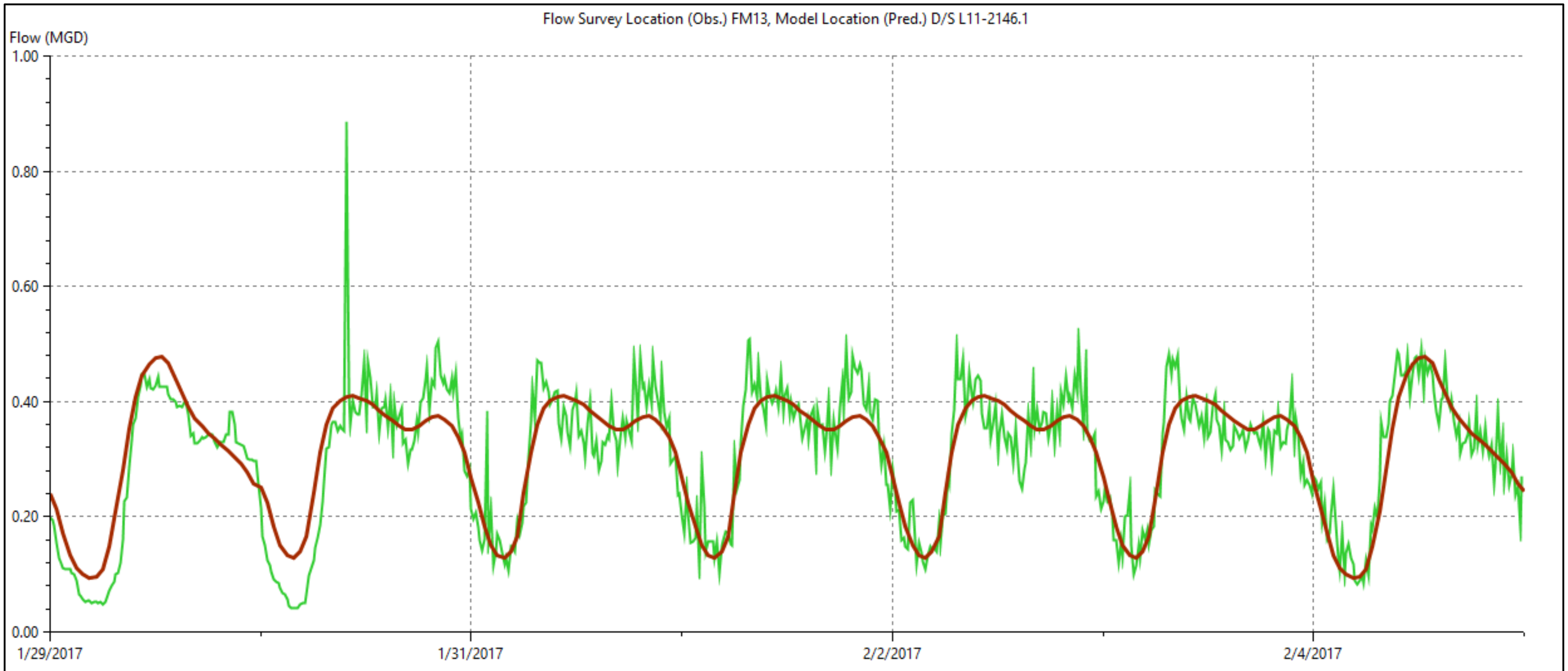
Table 5-8: Dry Weather Flow Loading Parameters

Flow Meter ID	Contributing Area (acres)	ABWF (mgd)	GWI (gpd/acre)	GWI (mgd)	ADWF (mgd)
Non FM Areas	4,160	5.58	35	0.15	5.73
FM1	72	0.09	0	0.00	0.09
FM3	315	0.50	0	0.00	0.50
FM5	49	0.12	0	0.00	0.12
Upstream of flow split to FM 5 and 6	700	1.24	57	0.04	1.28
FM6	346	0.60	0	0.00	0.60
FM7	534	0.73	644	0.34	1.08
FM8	272	0.30	0	0.00	0.30
FM9	244	0.63	0	0.00	0.63
FM10	290	0.41	0	0.00	0.41
FM12	381	0.58	391	0.15	0.73
FM13	376	0.34	0	0.00	0.34
Total	**7,739**	**11.1**	**--**	**0.68**	**11.78**

Table 5-9: Wet Weather Calibration Parameters

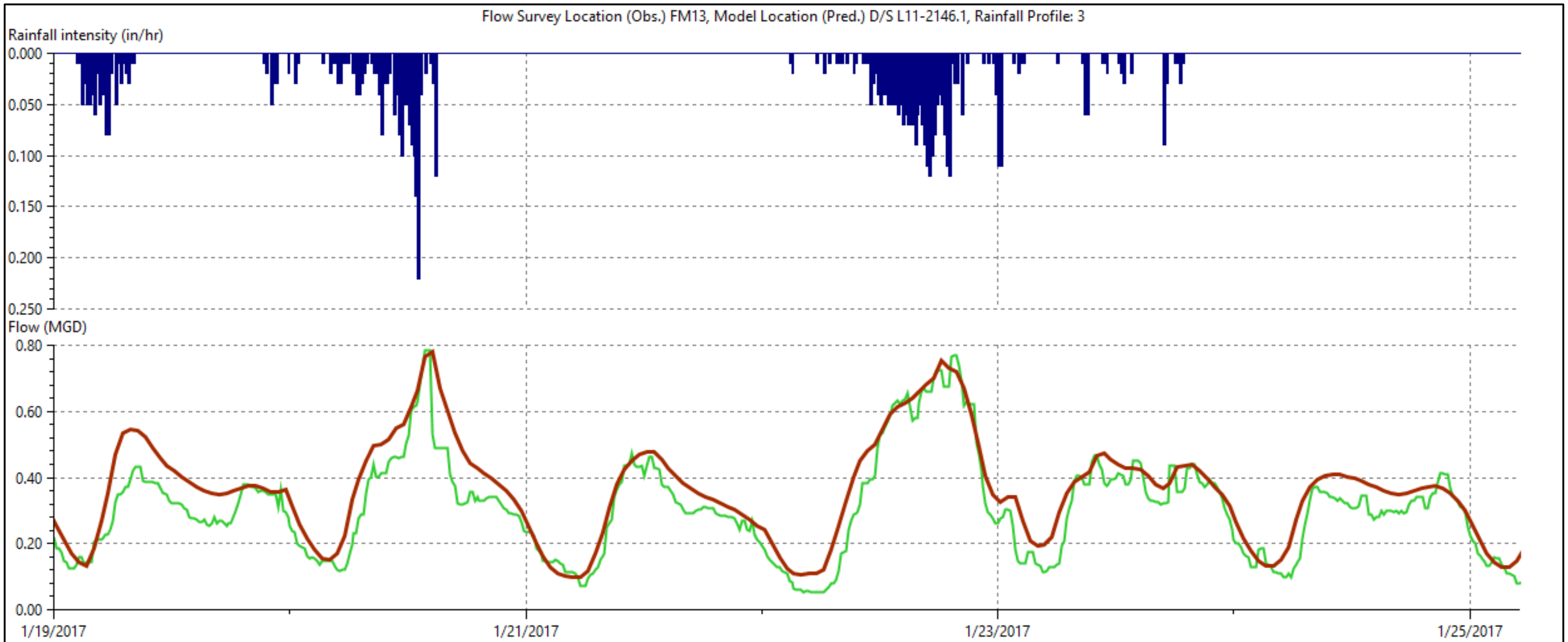
Flow Meter ID	R1 RDI/I Volume (%)	R2 RDI/I Volume (%)	R3 RDI/I Volume (%)	Rtot RDI/I Volume (%)
Non FM Areas	1.0%	0.1%	0.6%	1.7%
FM1	1.0%	0.1%	0.1%	1.2%
FM3	4.5%	0.5%	0.2%	5.2%
FM5	9.0%	1.0%	0.2%	10.2%
Upstream of flow split to FM 5 and 6	3.0%	0.5%	0.5%	4.0%
FM6	3.0%	0.5%	0.5%	4.0%
FM7	3.5%	8.0%	11.0%	22.5%
FM8	9.0%	0.3%	0.1%	9.4%
FM9	11.0%	0.1%	0.1%	11.2%
FM10	2.5%	2.5%	0.1%	5.1%
FM12	1.2%	0.5%	1.0%	2.7%
FM13	1.0%	1.0%	0.1%	2.1%

Figure 5-11: Example Dry Weather Flow Model Calibration Graph (Flow Meter 13)



	Flow		
	Min (MGD)	Max (MGD)	Volume (US Mgal)
Observed	0.043	0.865	2.140
DWF_existing_2018_test020718>DWF_w-GWI	0.094	0.478	2.178

Figure 5-12: Example Wet Weather Flow Model Calibration Graph (Flow Meter 13)



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.502	0.220	0.010			
Observed				0.052	0.786	1.910
...ff Gauge WWF (1/19-1/27)				0.097	0.781	2.235

Figure 5-13: Calibrated GWI Rates by Flow Meter Area

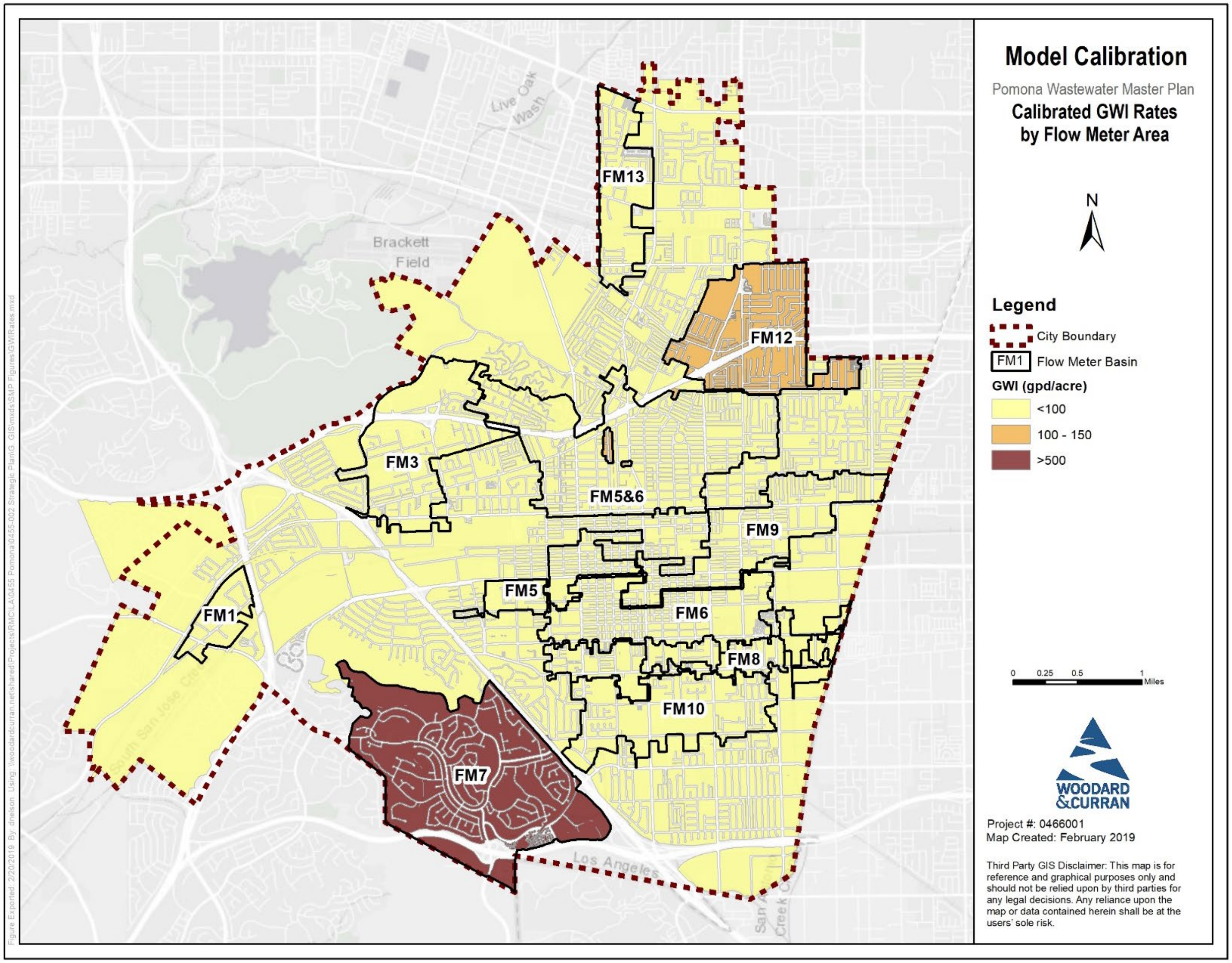


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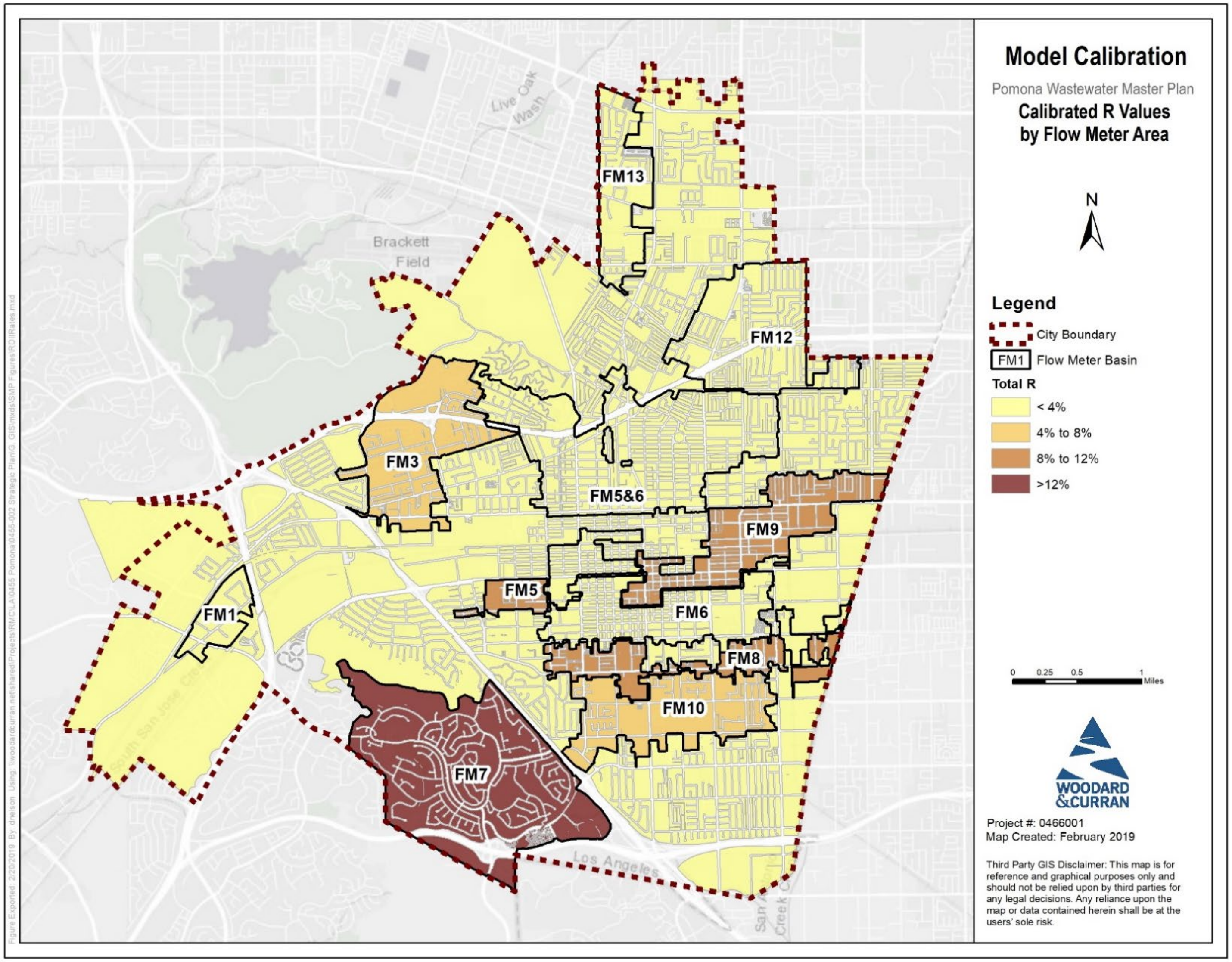
Figure 5-14: Calibrated Total R by Flow Meter Area



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6. SEWER SYSTEM CAPACITY ANALYSIS

The capacity analysis of the system and potential need for capacity improvements were evaluated using the calibrated hydraulic model.

6.1 Performance Criteria

The calibrated model was run for existing and future conditions to identify areas of the system that fail to meet specified performance criteria under both peak dry weather flow (PDWF) and design storm peak wet weather flow (PWWF). The performance criteria define the hydraulic conditions that prompt the need to upgrade a sewer pipeline to convey the projected future peak flows. Performance criteria were presented and confirmed during a project meeting (held remotely) and are summarized below.

Dry Weather Flow Criteria

PDWF in buildout condition should not cause the hydraulic grade line (HGL) in an existing main to rise:

- Above the crown of the pipe.

Wet Weather Flow Criteria

PWWF in a buildout condition should not cause the HGL in an existing main to rise:

- More than 2 feet above the crown of the existing pipe, or
- Within 5 feet of the lowest manhole rim elevation.

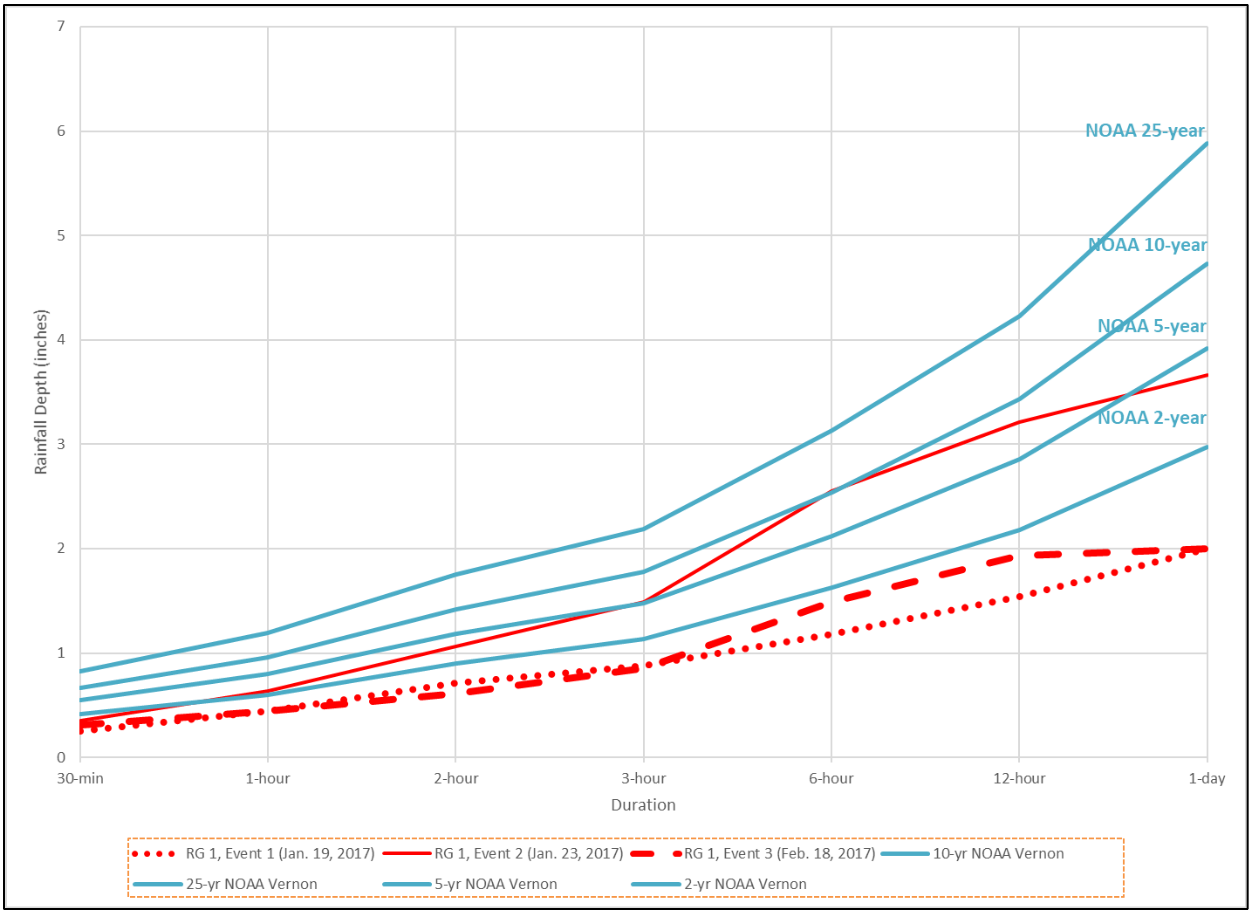
Design Storm Selection

The use of wet weather design events as the basis for sewer capacity evaluation is a well-accepted practice. The approach is to first calibrate a hydraulic model of the system to match wet weather flows from observed storm(s), and then apply the calibrated model to a design rainfall event to identify capacity deficiencies and size improvement projects. The design event may be synthesized from rainfall statistics or may be an actual historical rainfall event of appropriate duration and intensity. There is no regulatory standard for design return periods for wastewater collection systems; however, most California agencies that have adopted a specific return period have selected return periods of 5 or 10 years.

The rainfall data from the three rain gauges installed during the flow monitoring period was reviewed and compared to the National Oceanic and Atmospheric Administration (NOAA) depth-duration-frequency curves to quantify the return periods of the monitored storms. **Figure 6-1** shows the comparison between the NOAA 2-, 5-, 10-, and 25-year events, with the January rainfall events recorded at rain gauge 1. The NOAA data shown on this figure reflects depth-duration data for the rain gauge 1 location (2205 Vernon Avenue). The graph shows the January 23 event recorded at rain gauge 1 corresponds to the NOAA 10-year/6-hr rainfall depth and is between the 5- and 10-year/12-hour rainfall depth. After review of this data, the 12-hour rain event on January 23, 2018, was used as the design storm. **Figure 6-2** shows the temporal distribution of the January 23, 2018, rainfall.

The spatial variation of rainfall over the City was analyzed by obtaining NOAA data at rain gauge 2 and 3 locations (925 E. Lexington Avenue and 520 E. Laverne Avenue, respectively). **Figure 6-3** shows the comparison between the January 23 event recorded at rain Gauge 1 and the NOAA depth-duration data at the three rain gauge locations. As shown on this figure, the spatial variation of rainfall depth-duration does not vary by much. At E. Lexington Avenue, the January 23 event recorded at rain gauge 1 is slightly above NOAA's 10-year/6-hour rainfall depth and closer to the 10-year/12-hour rainfall depth. As a result, the same January 23 storm was applied throughout the City's service area.

Figure 6-1: Rainfall Depth-Duration Comparison Between Rain Gauge 1 and NOAA Data[a]



[a] Data obtained from the NOAA Atlas 14 Point Precipitation Frequency Estimate website.

Figure 6-2: Design Storm (10-year, 12-hour)

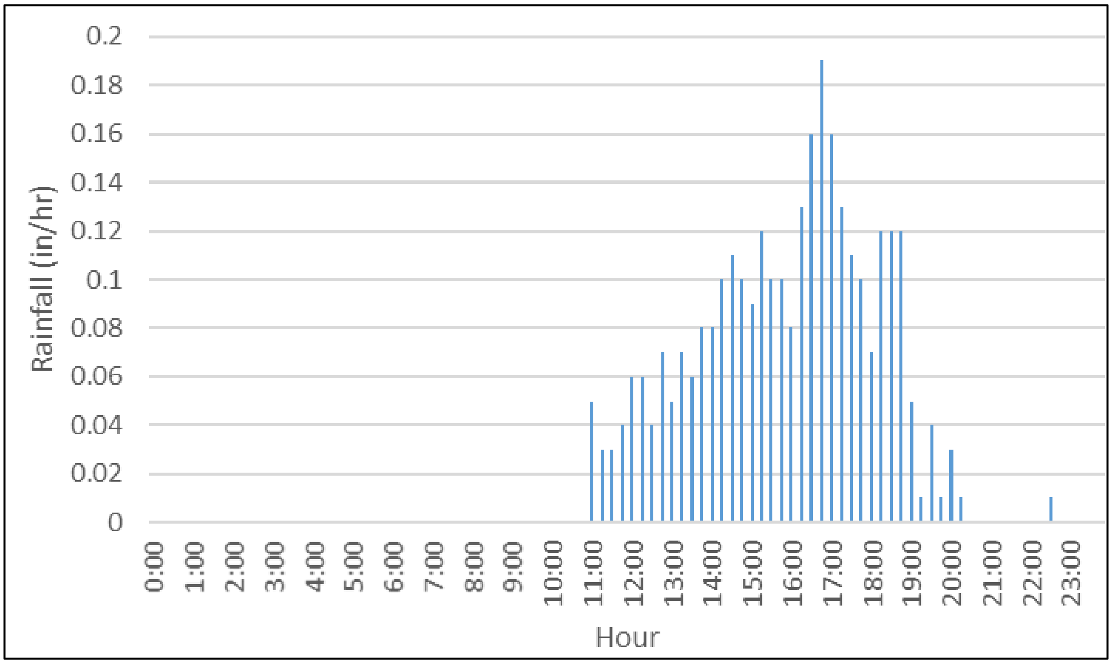
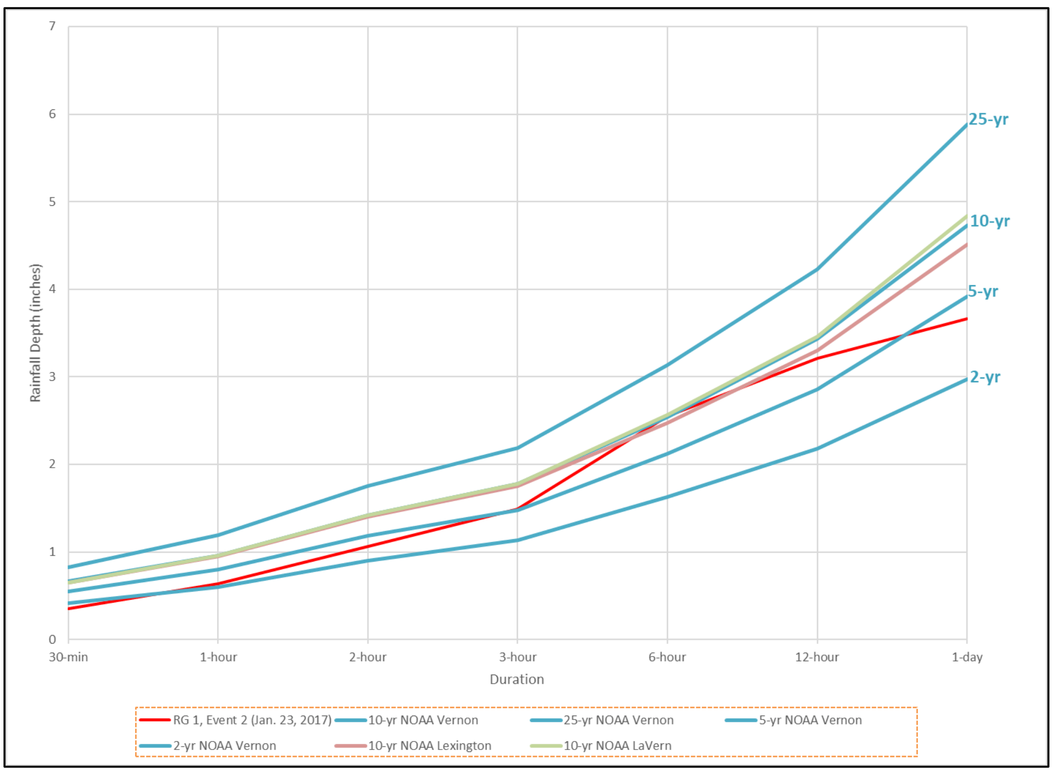


Figure 6-3: Rainfall Depth-Duration Comparison Between Rain Gauge 1 and NOAA Data at All Rain Gauge Locations



[a] Data obtained from the NOAA Atlas 14 Point Precipitation Frequency Estimate website.

The timing of the design storm also affects the resulting peak wastewater flows. If the design storm is timed to cause peak RDI/I at roughly the same time as peak BWF ("peak-on-peak"), the total peak wet weather flow will be higher than if the peak RDI/I generated by the design storm occurs at the time of the average or minimum BWF. Timing the storm to produce peak-on-peak results is generally thought to create a wastewater flow return period that is greater than the return period of the design rainfall event itself (e.g., the peak flow during a 10-year storm event occurring at the same time as peak BWF would occur less often than a 10-year storm occurring at any other time during the day). In order to avoid an overly conservative condition, the design storm timing was unchanged from the actual storm event (with peak RDI/I flows coinciding with slightly above average BWF).

6.2 Sewer Capacity Evaluation

The capacity of the modeled sewer system was evaluated using the criteria identified in the previous section to determine potential capacity deficiencies. Capacity was evaluated under existing and future land use. Results from the existing and future loading scenario are presented in the following sections.

The capacity evaluation illustrates the performance of the existing system, including where surcharging or overflows may occur assuming no changes or upgrades are made to the pipes or to the existing flow splits. Predicted surcharging or overflows do not necessarily mean that pipes are capacity deficient at that location, as flows can back up due to downstream capacity limitations and cause surcharging or potential overflows at upstream locations due to backwater. Additionally, the results reflect an "unrelieved" system, meaning that peak flows are dampened out in the pipes that are under heavy surcharge or reduced due to overflows. This means that as upstream deficiencies are relieved through capacity projects, the peak flows reaching downstream pipes will increase, potentially causing additional surcharging or overflows.

Dry Weather Flow Capacity Analysis

Under dry weather, existing loading conditions there are small sections of pipeline that are just at or above their capacity (d/D equal to or above 1). Though the City's criteria states that pipes should not surcharge under dry weather conditions, the predicted flow and depth in these pipes are so close to the capacity that they should not be considered deficient under existing loading conditions.

Figure 6-4 illustrates the maximum depth to Diameter (d/D) ratio results for the existing PDWF.

Under dry weather, future loading conditions, there are some areas of the system that exhibit capacity deficiencies per the City's criteria. Segments of sewer along Grand Avenue, Berkeley Avenue, Elaine Street, and Lexington Avenue all predict capacity deficiencies (the mains surcharge). **Figure 6-5** illustrates the d/D ratio results for the 2040 PDWF, with capacity deficient pipes shown in red. Results of the dry weather flow evaluation were used to inform the wet weather flow assessment.

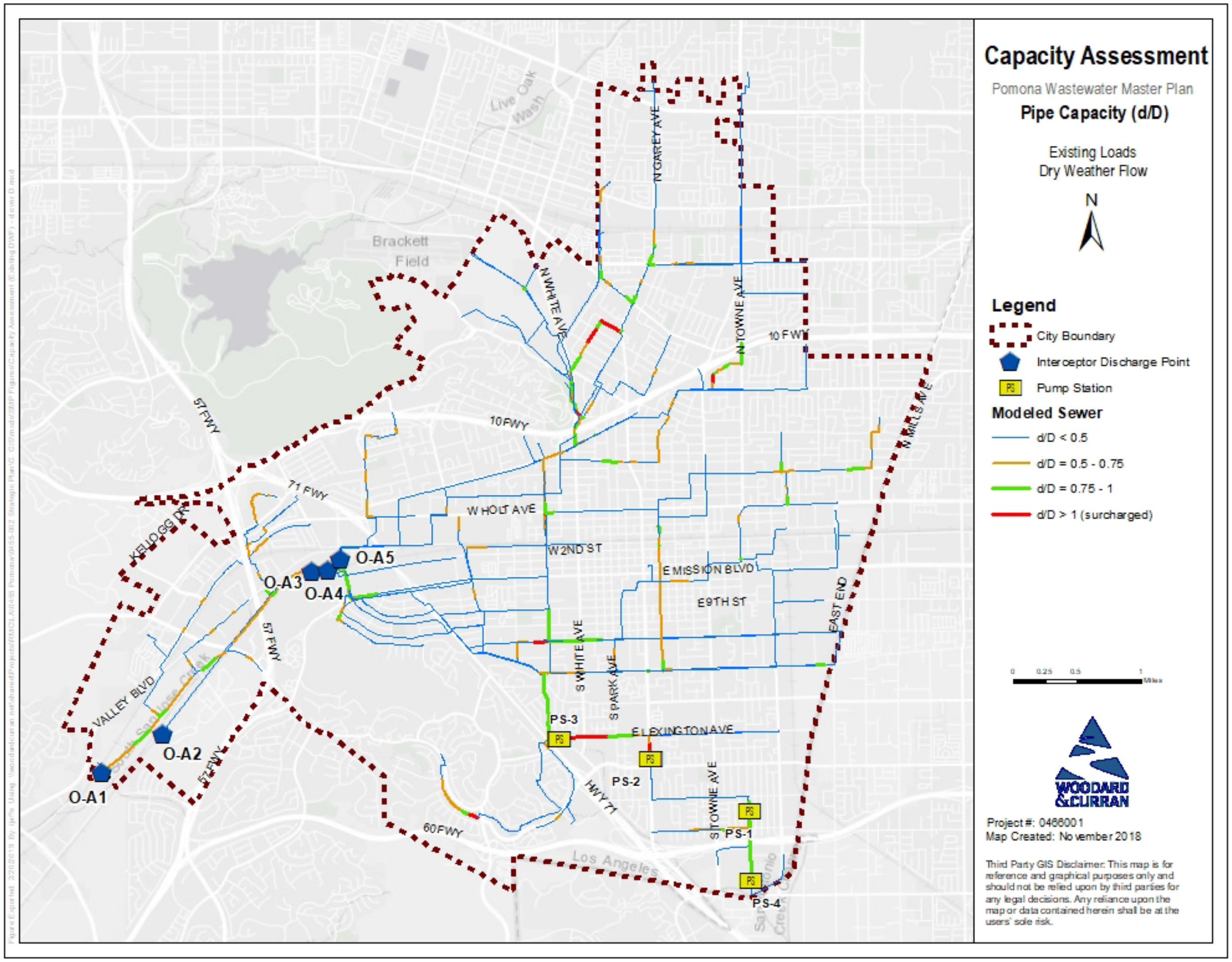
Wet Weather Flow Capacity Analysis

In addition to those segments of pipe triggered as part of the dry weather flow analysis, there are segments of pipe that violate the capacity criteria under PWWF conditions. The existing WWF capacity analysis utilized the RDI/I parameters developed during the calibration process. It was assumed that any parcels developed in the future would not contribute to I/I so the future contributing area was set equal to the existing contributing area.

Under PWWF, existing loading conditions, segments of pipe along Village Loop Road, Rio Rancho Road, Rainbow Ridge Road, Philips Boulevard, Butterfield Road, West Ninth Street and East Sixth Street exhibit surcharging violating wet weather capacity performance criteria, with overflows predicted along Village Loop Road, Rio Rancho Road and Rainbow Ridge Road. **Figure 6-6** illustrates the d/D ratio results for the existing peak wet weather flow, and **Figure 6-7** shows the surcharge and predicted overflow locations under existing loading conditions.

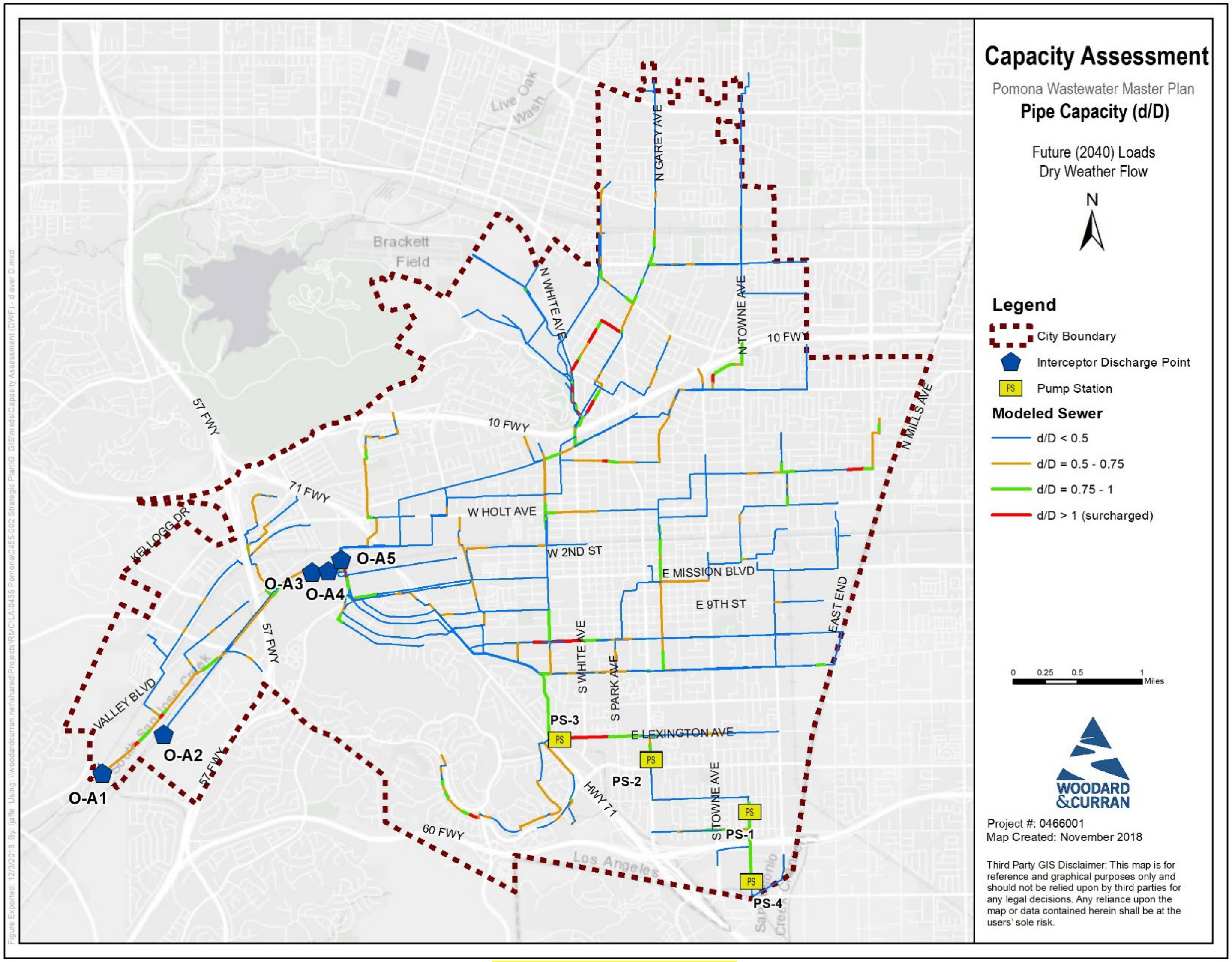
In addition to those segments exhibiting surcharging under existing conditions, sections of sewer along Casa Vista Boulevard, Hamilton Boulevard, Gordon Street and E 6th Street trigger wet weather criteria violations under 2040 flows, with overflows predicted at Casa Vista Boulevard and Hamilton Boulevard. **Figure 6-8** presents the d/D ratio results for the future (2040) wet weather flow, and **Figure 6-9** shows the surcharge and predicted overflow locations under future loading conditions.

Figure 6-4: Existing PDWF Depth to Diameter (d/D) Ratio



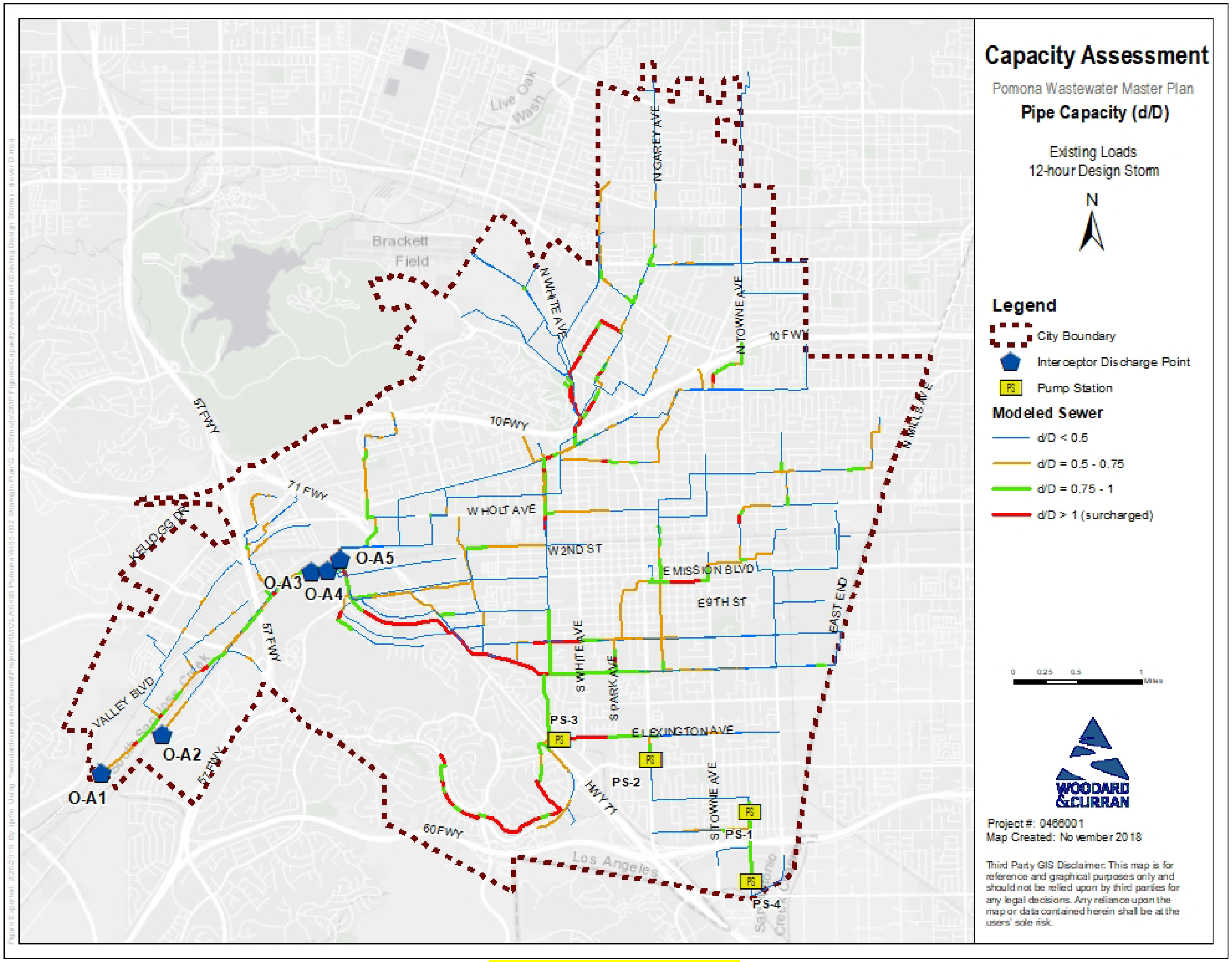
(Placeholder for PDF map)

Figure 6-5: 2040 PDWF Depth to Diameter (d/D) Ratio



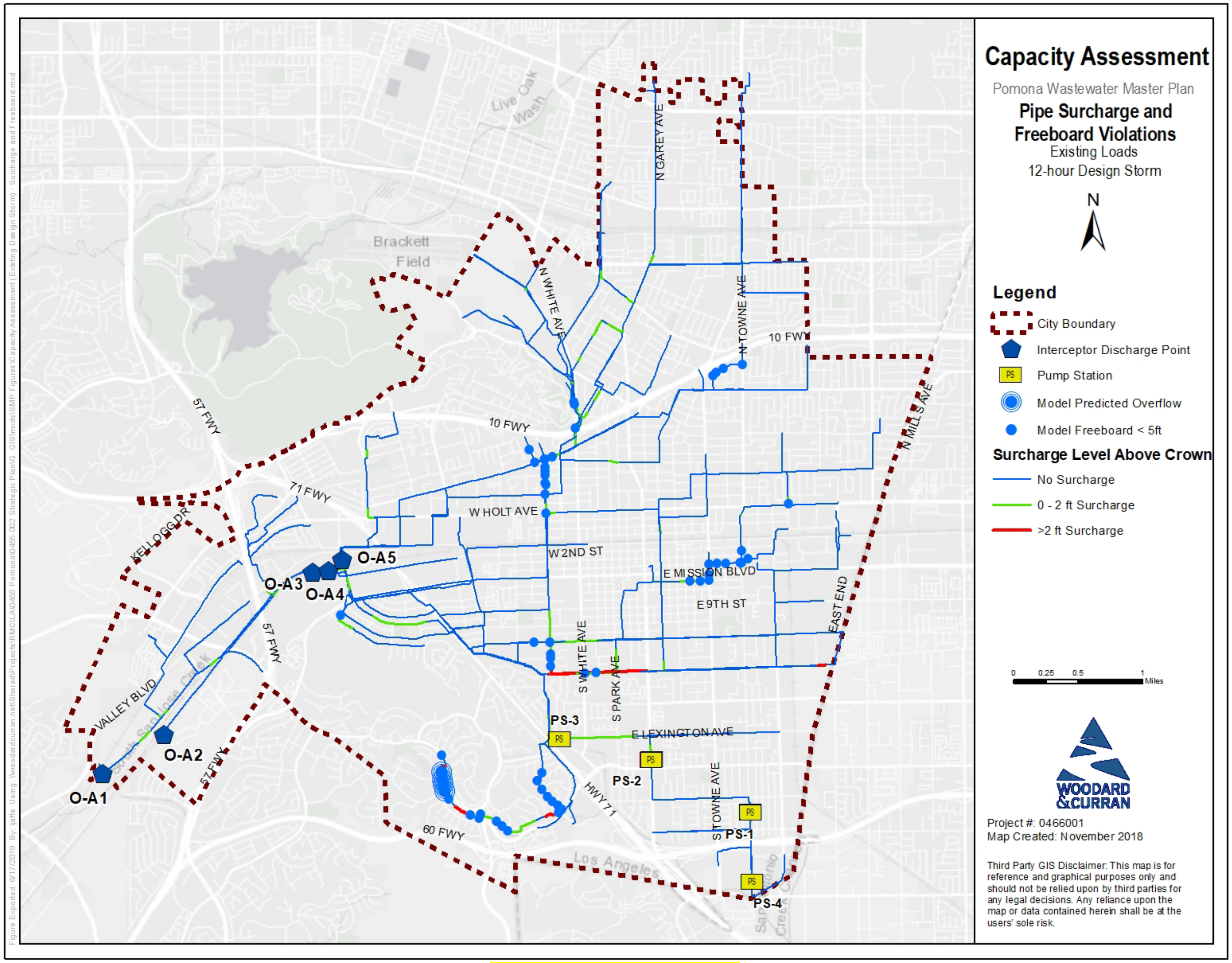
(Placeholder for PDF map)

Figure 6-6: Existing PWWF Depth to Diameter (d/D) Ratio



(Placeholder for PDF map)

Figure 6-7: Existing PWWF Surcharge and Freeboard Violations



(Placeholder for PDF map)

Figure 6-8: 2040 PWWF Depth to Diameter (d/D) Ratio

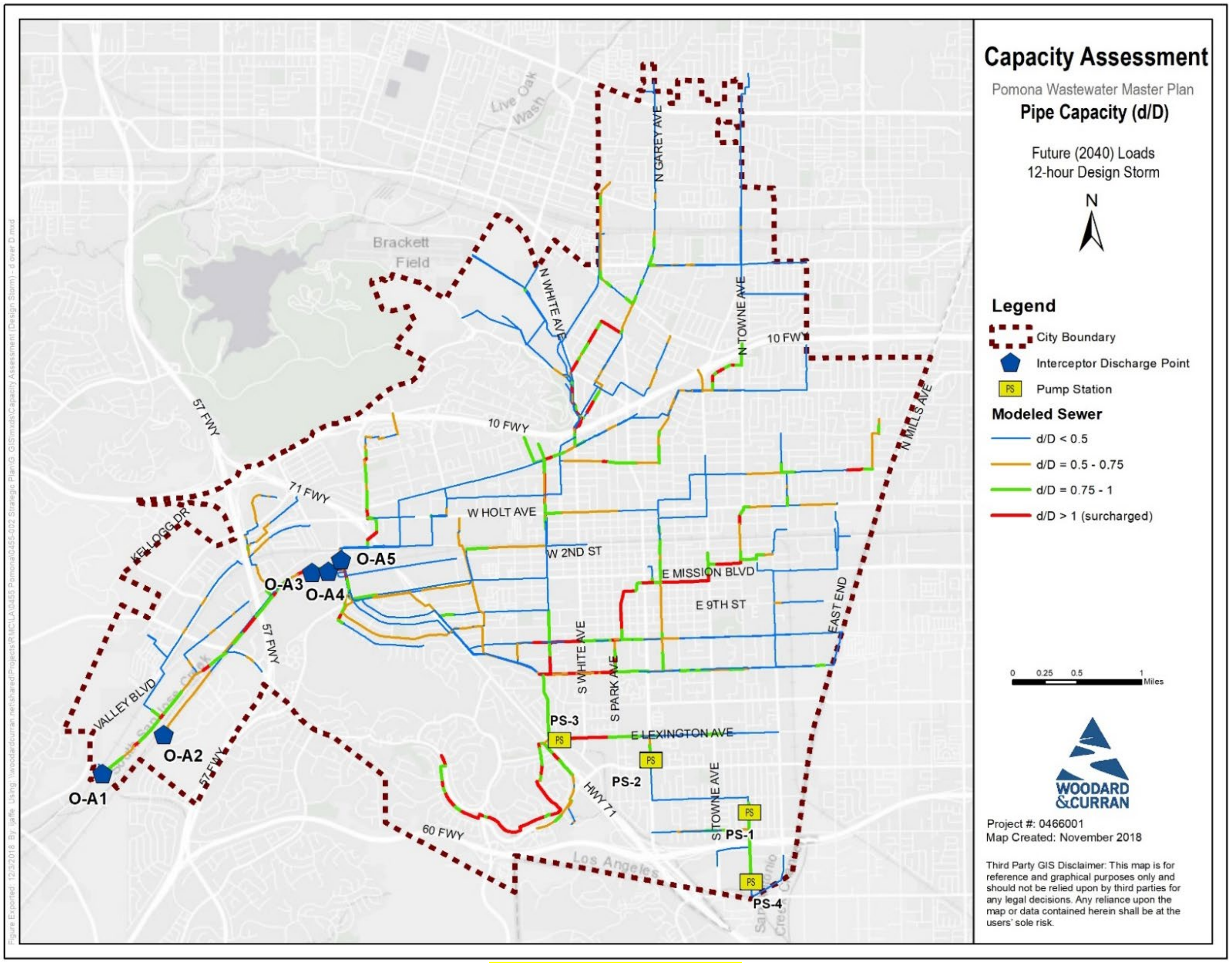


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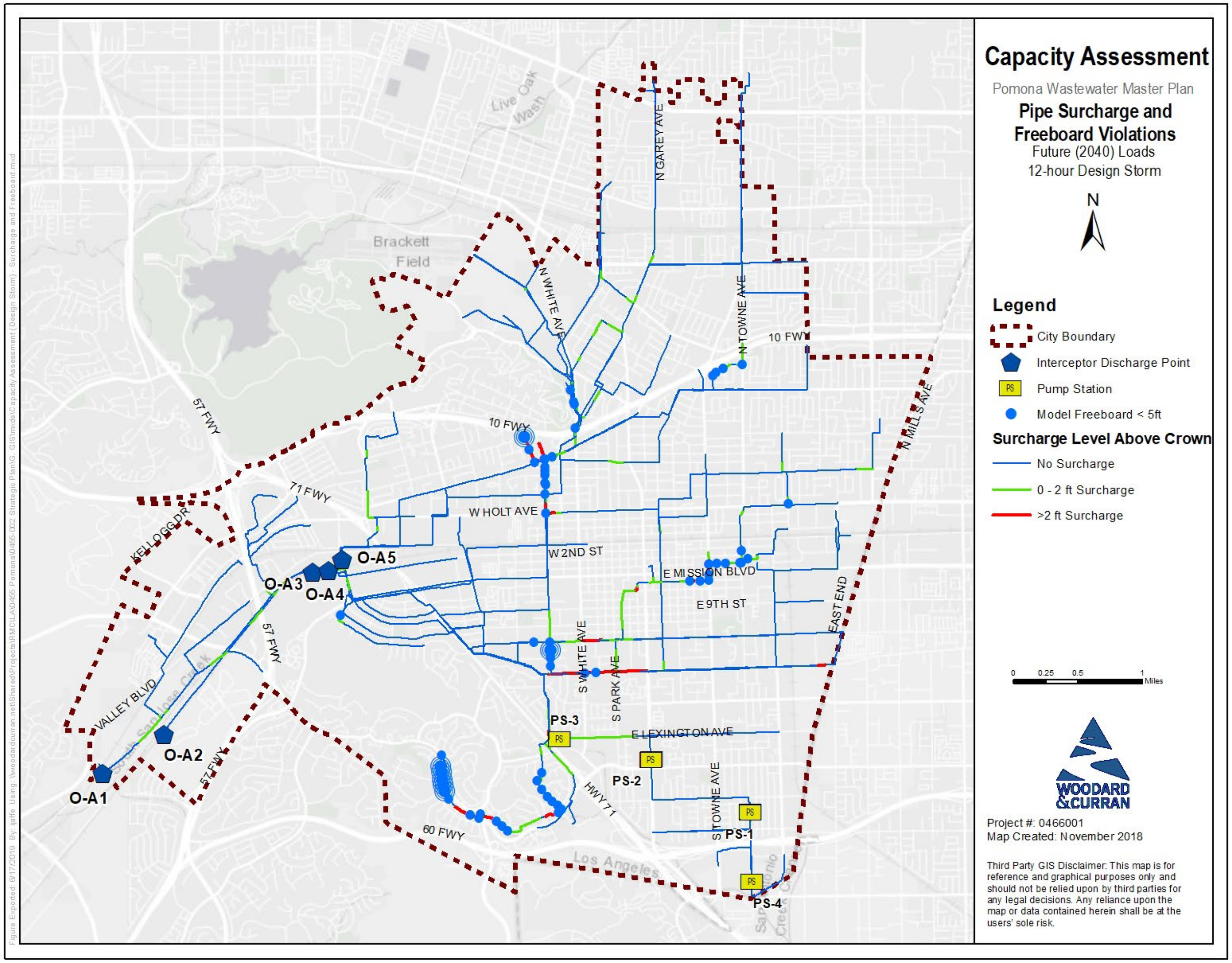
Figure 6-9: 2040 WWF Surcharge and Freeboard Violations



(Placeholder for PDF map)

Capacity Analysis Summary

Table 6-1 summarizes all the identified capacity deficiencies based on the performance criteria discussed in **Section 6.1**. The deficiencies listed are the pipes that would need some type of capacity relief, either to increase their capacity (e.g., upsize pipe to larger diameter) or to reduce the flow (e.g., divert flow away from pipe). These deficiencies are illustrated in **Figure 6-4** through **Figure 6-9**. **Appendix C** includes a plan and profile view of each deficiency. Information on the pump stations follows in **Section 6.3**.

In addition to those pipe segments flagged for a capacity improvement project, the following areas are recommended for additional monitoring and/or investigation:

- The flow split at MH L12-2155 (in Garey Avenue) should be investigated and adjusted as needed to direct the majority of flow south to Alameda Street.
- Review the rim elevations upstream of MH 14-2260 (San Bernardino Avenue and Shirley Place) to confirm pipe depth.
- The pipeline from S Humane Way to outfall O-A1 (along the railroad) should be investigated to confirm the pipe diameters.

Table 6-1: Capacity Deficiency Summary

Location	Diameter (in)	Length (ft)	Trigger Loading Condition	CIP Project	Comments
Berkeley Ave	8	3,300	Existing (PWWF)	None	City staff should confirm that the majority of flow at the MH L12-2156 flow split is directed south to Alameda Street.
Kingsley Ave	8, 10	1,050	Future (PDWF)	None	There is minimal surcharge, no project recommended.
Grand Ave	12, 14	1,990	Existing (PWWF)	None	This deficiency is partially caused by very flat pipe. While triggered due to surcharging, no project is recommended as there is sufficient freeboard
Elaine St	8	290	Future (PDWF)	None	This deficiency is caused by very flat pipe, though there is minimal surcharge. It is recommended that the pipe depth be confirmed by City staff.
Lexington Ave	21	360	Future (PDWF)	None	This deficiency is caused by very flat pipe feeding into the PS 3 wet well. The surcharge is minimal so no project is recommended.
Casa Vista and Hamilton Blvd.	8	3,350	Future (PWWF)	1	Future PWWF exceeds pipeline capacity.
Village Loop Rd, Rio Rancho Rd, Rainbow Ridge Rd	8, 10, 12, 15, 18	10,840	Existing (PWWF)	2A, 2B, 2C	This deficiency is partially caused by very flat pipe. The wet weather flow should be confirmed for this area (upstream of MH J24-6243).
Hamilton Rd	8	980	Future (PWWF)	3	Inverts should be confirmed along this stretch of pipe and the flow split at MH K21-6093 should be investigated.
Philips Blvd, Butterfield Rd and W Ninth St	21	9,370	Existing (PWWF)	4A, 4B	Existing PWWF exceeds pipeline capacity.
E Sixth St	15	1,570	Existing (PWWF)	5, 6	Existing PWWF exceeds pipeline capacity.
Gordon St to E Sixth St	15	2,880	Future (PWWF)	5, 6	Future PWWF exceeds pipeline capacity.

Review of CIP Projects from 2005 Master Plan

The 2005 Sewer Master Plan identified four pipeline capital improvement projects to eliminate capacity deficiencies (summarized in **Table 6-2**). The City has since implemented projects 1 through 3. As part of the capacity evaluation for this master plan, the locations of those four projects were reviewed. There are no capacity concerns at any of the four sites under wet or dry weather conditions.

Table 6-2: 2005 CIP Projects

Project	Location	Description	Implemented	Existing Capacity Concerns
1	Phillips Blvd, from Rebecca St to west of Hamilton Blvd	Upsize 2,130 feet of 10 to 12-inch pipe to 15-inch pipe	Yes	None
2	Kingsley Ave, from Washington Ave to Towne Ave	Upsize 3,120 feet of 12-inch pipe to 15-inch pipe	Yes	None
3	Between Holt Ave at Fairplex Dr and Mount Vernon Ave at Bellevue Ave	Upsize 1,600 feet of 12-inch pipe to 15-inch pipe	Yes	None
4	Between 2nd St and Mission Blvd, west of Oak Ave and east of the 71 Freeway	Upsize 1,500 feet of 10-inch pipe to 15-inch pipe	No	None

6.3 Pumping Station Capacity Evaluation

The LACSD owns and maintains four pump stations within the City. As part of this Master Plan, the pump station capacities were evaluated based on modeled existing and future inflows. For reference, a schematic of the flow through the pump stations is shown in **Figure 6-10**.

As summarized in **Table 6-3**, the capacity evaluation of the pump stations indicates that all four pump stations have adequate capacity to handle peak flows through 2040. That said, the estimated peak flow at PS# 2 under 2040 conditions is nearing the capacity of the pump station, so the operation of this station should be monitored and additional evaluation is recommended as the City continues to develop toward buildout. Additionally, flows at PS# 3 should be monitored in association with the review of wet weather flows upstream of MH J24-6243 as there is predicted surcharging directly upstream of the pump station.

Figure 6-10: Pump Station Flow Schematic

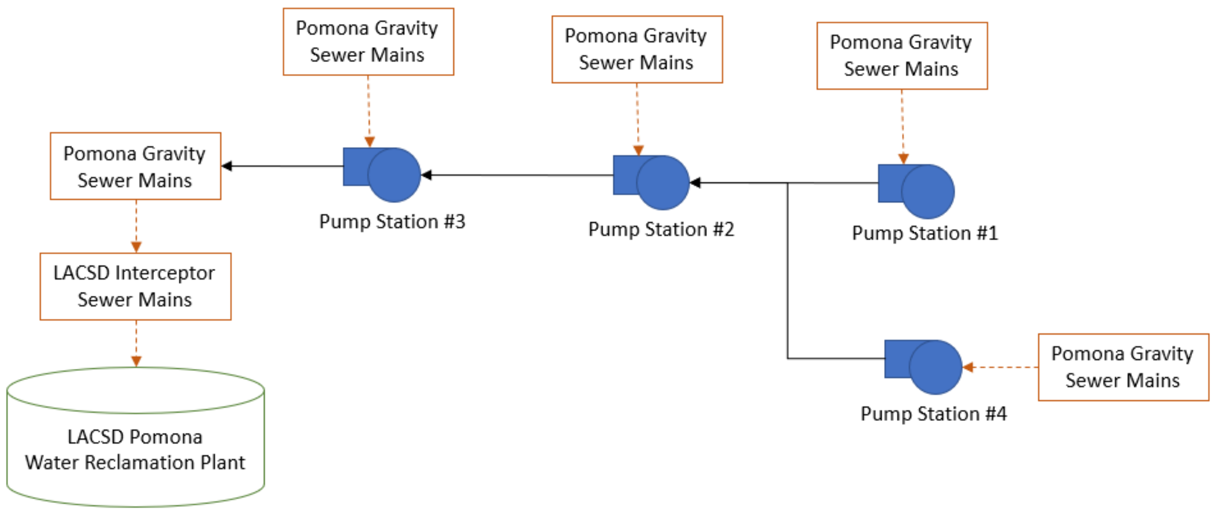


Table 6-3: Pump Station Capacity Analysis

	PS #1	PS #2	PS #3	PS #4
Number of Pumps	2	2[a]	3	3
Pump Type	Variable Speed	Variable Speed	Variable Speed	Fixed Speed
Pump Station Firm Capacity (gpm)	1,500	6,000	8,080	950
Existing Peak Wet Weather Flow (gpm)	910	3,750	980	420
2040 Peak Wet Weather Flow (gpm)	1,100	4,170	4,040	430

[a] Pump 2 assumed to have been installed per the reference document provided by the City (LACSD Lift Station – Set Points.pdf).

APPENDIX A: FLOW MONITORING DATA

APPENDIX B: CALIBRATION PLOTS

APPENDIX C: CAPACITY DEFICIENCY PLANS AND PROFILES

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