SANITARY SEWER MASTER PLAN UPDATE

FINAL | September 2022

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City of Pinole

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Abbreviations

ºF	degrees Fahrenheit
AACE	Association for the Advancement of Cost Engineering
AAF	average annual flow
ADU	accessory dwelling unit
ADWF	average dry weather flow
BWF	base wastewater flow
Carollo	Carollo Engineers
CCTV	closed-circuit television
CIMIS	California Irrigation Management Information System
CIP	capital improvement plan
City	City of Pinole
d/D ratio	depth of flow, d, to pipe diameter, D, ratio
DOF	Department of Finance
DU	dwelling unit
DWF	dry weather flow
ENR CCI	Engineering News Record Construction Cost Index
EPA	Environmental Protection Agency
ETo	evapotranspiration
fps	feet per second
General Plan	General Plan Update
GIS	geographic information system
gpd	gallons per day
gpd/ac	gallons per day per acre
gpd/ADU	gallons per day per accessory dwelling unit
gpd/DU	gallons per day per dwelling unit
gpd/idm	gallons per day per inch of diameter per mile
gpm	gallons per minute
GWI	groundwater infiltration
HGL	hydraulic grade line
I-80	Interstate 80
I/I	infiltration and inflow
IDM	inch of diameter per mile
Master Plan	Sanitary Sewer Master Plan Update
MF	multi-family
mgd	million gallons per day
NOAA	National Oceanic and Atmospheric Administration

NPDES	National Pollutant Discharge Elimination System
PWWF	peak wet weather flow
RDII	rainfall-derived inflow/infiltration
ROW	right of way
SF	single family
SSO	sanitary sewer overflow
SWMM	storm water management model
V&A	V&A Consulting Engineers, Inc.
WaPUG	wastewater planning users group
WCW	West County Wastewater
WPCP	water pollution control plant
WWF	wet weather flow

Chapter 1

INTRODUCTION

This Chapter presents a brief summary of the City of Pinole's (City's) need for the Sanitary Sewer Master Plan Update (Master Plan), the objectives of the Master Plan, and a description and organization of the seven chapters that cover the wastewater collection system.

1.1 Background

The City is located in the San Francisco Bay Area, along the San Pablo Bay in Western Contra Costa County, California. The City is located near the beginning of State Route 4, which begins just north of the City (Figure 1.1). The City limits comprise approximately 5.5 square miles, and consists of varied topography ranging from steep terrain to ocean basin.

The City, which was incorporated in 1903, provides sewer and storm drainage service to its customers. The City provides sewer service to most of its the residential, commercial, and industrial customers. West County Wastewater (WCW) provides sewer service to a small portion of the City.

1.2 Wastewater Collection System Overview

The City provides wastewater services to approximately 19,000 residents, industrial and commercial users. The wastewater collection system includes approximately 50 miles of active gravity sewer lines, ranging from 6 to 30 inches in diameter, two lift stations, and associated force mains. Wastewater generated in the sewer service area is conveyed to the Pinole-Hercules Water Pollution Control Plant (WPCP). WPCP provides wastewater treatment to the cities of Pinole and Hercules. The WPCP is owned and operated by the City.

1.3 Study Purpose, Scope, and Authorization

The purpose of this Master Plan is to gain an understanding of the system's performance and to provide a planning document for the City's wastewater collection system. Overall, the Master Plan will assist the City in their approach to optimize their collection system operations, maximize the use of existing pipelines, and focus spending in key areas in need of improvement.

Carollo Engineers (Carollo) was contracted to prepare the Master Plan. The Master Plan scope of services include the following main tasks:

- Task 1 – Project Management and Quality Control.
- Task 2 – Data Collection and Review.
- Task 3 – Existing and Future Wastewater Flow Analysis.
- Task 4 – Hydraulic Analysis.
- Task 5 – Pipeline Risk Assessment.
- Task 6 – Prepare Capital Improvement Program.
- Task 7 – Sanitary Sewer System Master Plan Report.

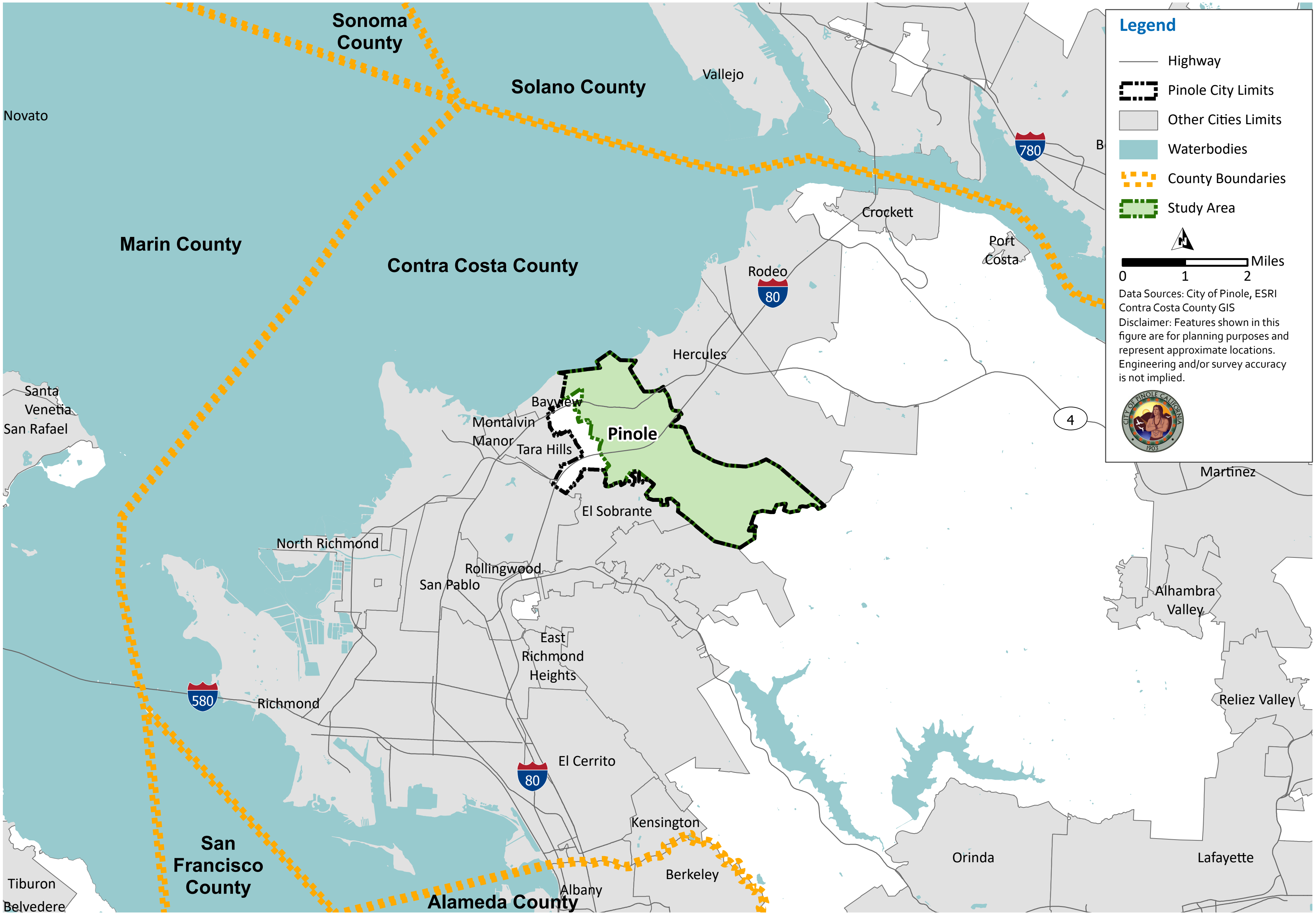

Figure 1.1 Regional Location Map

1.4 Report Organization

The Master Plan report contains seven chapters, followed by appendices that provide supporting documentation for the information presented in the report. The chapters are briefly described below:

- **Chapter 1 - Introduction.** This chapter presents the project background, goals, and organization of this Master Plan.
- **Chapter 2 - Land Use and Population.** This chapter presents a description of the study area, defines the planning horizon for this study, and summarizes the zoning classifications and future development of the study area.
- **Chapter 3 - Planning and Evaluation Criteria.** This chapter presents the planning criteria and methodologies for the analysis used to evaluate the City's existing sanitary sewer collection system and associated facilities, which are utilized to identify existing system deficiencies and to size future improvements.
- **Chapter 4 - Wastewater Flows.** This chapter summarizes the existing and projected wastewater flows for the City's collection system.
- **Chapter 5 - Sanitary Sewer Collection System Facilities and Hydraulic Model.** This chapter describes the development and calibration of the City's collection system hydraulic model. A description of the City's wastewater collection system, the existing hydraulic model, and an outline of the steps used to update the model are provided. A detailed summary of the hydraulic model calibration steps, standards, and results for both dry weather flow (DWF) and wet weather flow (WWF) conditions is also provided.
- **Chapter 6 - Capacity Evaluation and Proposed Improvements.** This chapter discusses the hydraulic evaluation of the wastewater collection system and the proposed projects that correct capacity deficiencies and serve future users.
- **Chapter 7 - Capital Improvement Plan.** This chapter presents the CIP, a summary of the capital costs, and a basic assessment of the possible financial impacts of the proposed CIP.

1.5 Acknowledgements

We would like to thank the following City staff for their assistance and oversight of this project:

- Sanjay Mishra; Public Works Director/City Engineer.
- Misha Kaur; Capital Improvement and Environmental Program Manager.
- Tamara Miller; former Development Services Director/City Engineer.

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

- Tim Loper, P.E.; Project Manager.
- Ryan Orgill, P.E.; Project Engineer.
- Danielle Orgill, P.E.; Hydraulic Modeling Lead.
- Kyle Pierce; Staff Engineer.
- Joaquin Ramirez, P.E.; Quality Management.
- Kevin Christensen; GIS/Graphics.
- Sabrina Bruce; Document Processing.

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Chapter 2

STUDY AREA

This chapter outlines the planning area for the wastewater collection system, defines land use classifications and describes the planned development within the City's service area. A summary of historical population trends and population projections are also presented in this chapter.

2.1 Study Area

The study area generally consists of the City limits, however a small portion of the City is served by WCW and is excluded from the study area for the purposes of this Master Plan. Figure 2.1 shows the study area.

2.2 Planning Horizon

This Master Plan is intended to serve as a guiding document for the planning and implementation of system improvements to accommodate future growth through build out of the City's General Plan Update (General Plan).

2.3 Climate and Topography

Table 2.1 summarizes the study area's climate. As shown, the City's climate is characterized by long, warm, arid summers and short, cold, wet winters. January is the wettest month with an average 4.76-inches of precipitation and July is the driest month with an average of 0.04-inches of precipitation. The average annual precipitation is approximately 23.14-inches, with approximately 90-percent of the average annual precipitation occurring between November and April. The City's elevation ranges from sea level in the northern portion of the City to approximately 780-feet above sea level in the south eastern portion of the City. Figure 2.2 shows a map of the study area topography.

Table 2.1 Study Area Climate

Month	Average Temperature(1) (degrees Fahrenheit [°F])		Monthly Average ETo(2)	Average Total Precipitation(1)
	Minimum	Maximum	(inches)	(inches)
January	42.6	57.5	1.66	4.76
February	45.4	61.5	2.06	3.83
March	46.8	63.8	3.17	3.31
April	48.8	66.5	4.31	1.67
May	51.7	69.0	4.82	0.53
June	54.4	71.1	5.48	0.21
July	55.3	70.4	5.25	0.04
August	56.1	71	4.38	0.07
September	56.3	74.1	4.06	0.21

Month	Average Temperature[1] (degrees Fahrenheit [ºF])		Monthly Average ETo[2]	Average Total Precipitation[1]
	Minimum	Maximum	(inches)	(inches)
October	53.3	72.2	3.34	1.27
November	48.0	64.6	1.89	2.89
December	43.3	58.1	1.49	4.36
Average or Total	**50.2**	**66.6**	**41.91**	**23.14**

Notes:
(1) Source: Western Regional Climate Center Richmond, California (047414). Represents monthly average from December 1950 to June 2016.
(2) Source: California Irrigation Management Information System (CIMIS) Station 213 El Cerrito. Represents monthly average evapotranspiration (ETo) from September 2013 to April 2021.

2.4 Land Use

Land use information is an integral component in determining the wastewater generation within a given service area. The type of land use in an area will affect the volume and character of the wastewater generation. Adequately estimating wastewater flow from various land use types is important in sizing and maintaining effective system facilities.

An important tool for determining land use projections is the City's General Plan, which was updated in November 2010. The land use assumptions provided in the General Plan were used for the purpose of this Master Plan. The Master Plan identifies three general land use categories, residential, commercial and public and other uses. These categories are split down even further into specific land use categories:

- Residential:
  - Low Density Residential.
  - Suburban Residential.
  - Medium Density Residential.
  - High Density Residential.
- Commercial:
  - Regional Commercial.
- Public and Other Uses:
  - Mixed Use Sub-Area.
  - Old Town Service Area.
  - Service Sub-Area:
    - San Pablo Avenue.
    - Pinole Valley Road.
    - Appian Way.
  - Rural.
  - Open Space.
  - Parks and Recreation.
  - Public/ Quasi-Public/ Institutional.
  - San Pablo Bay Conservation Area.
  - Transportation.

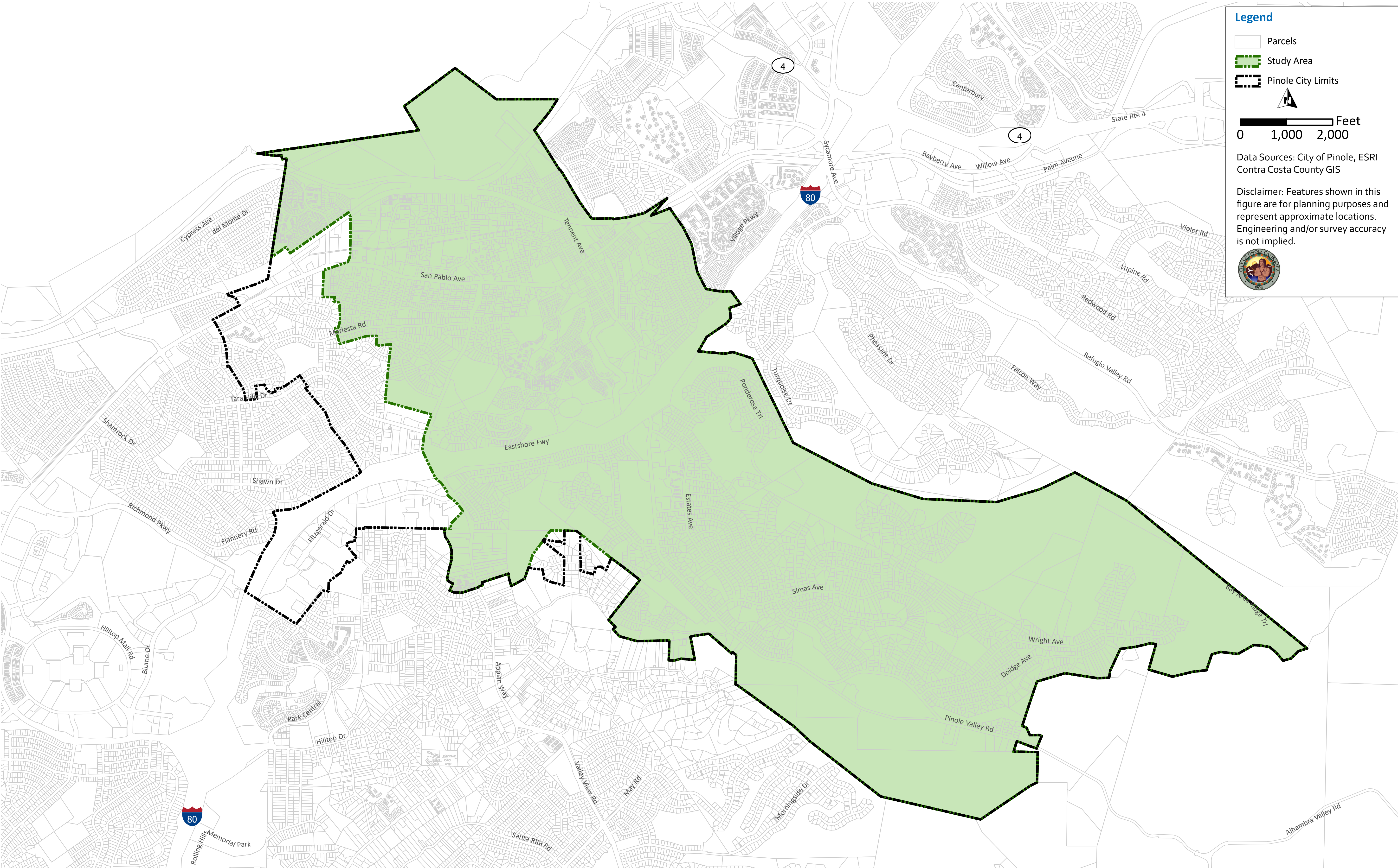


Figure 2.1 Study Area

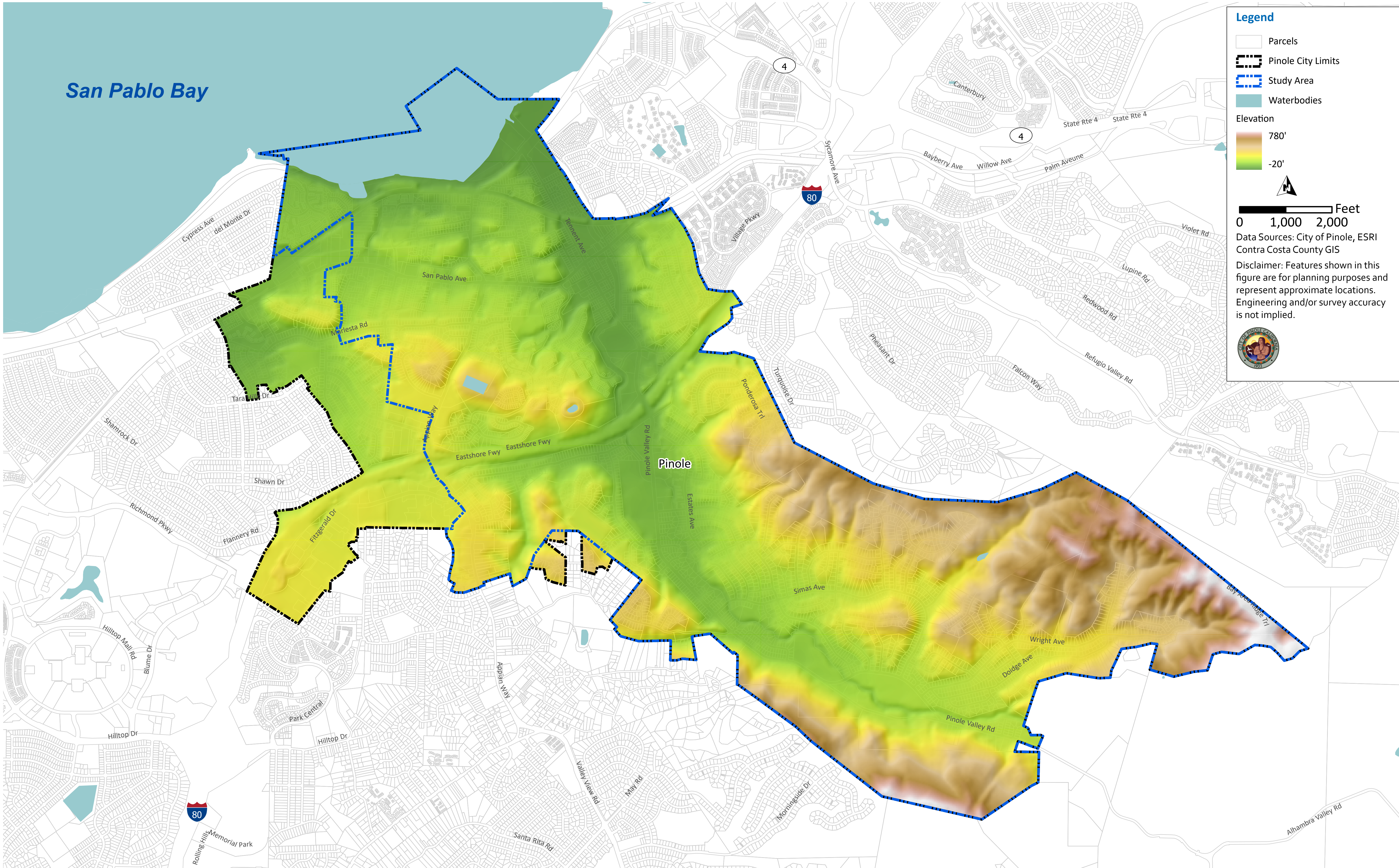


Figure 2.2 Study Area Topography

The City currently has limited open space/land available for future development. However, there is the potential for redevelopment/densification in certain areas of the City. The City's General Plan identifies three primary areas of growth:

- San Pablo Avenue Corridor.
- Appian Way Corridor.
- Pinole Valley Road Corridor.

Figure 2.2 shows the existing land use within the City limits, based on the 2010 General Plan. Table 2.2 provides a summary, by land use, of the amount of developed and developable land within the City limits.

Table 2.2 General Plan Land Use

Land Use Designation	Acres
Residential	
Low Density Residential	48.7
Residential Sub-Area	24.8
Suburban Residential	1,097.0
Medium Density Residential	97.4
High Density Residential	19.3
Residential Subtotal	***1,287***
Commercial	
Regional Commercial	67.8
Commercial Subtotal	***67.8***
Public and Other Uses	
Mixed Use Sub-Area	183.5
Old Town Sub-Area	31.9
Service Sub-Area	40.2
Open Space	392.8
Parks and recreation	295.7
Rural	409.6
Public/Quasi-Public/Institutional	101.4
San Pablo Bay Conservation Area	232.0
Transportation	545.0
Public and Other Uses Subtotal	***2,232***
Total	***3,587***

Notes:
(1) Source: City of Pinole General Plan (2010).

As shown in Table 2.2, out of the total 3,587 acres within the City limits. There are approximately 83 acres (2 percent) of developable land within the City limits. Of the 83 developable acres, 59.6 acres (72 percent) are classified as residential, 5.2 acres (6 percent) are classified as mixed-use, and the remaining 18.3 acres (22 percent) are classified as service sub-area, which is broken down into three primary categories: San Pablo Avenue, Appian Way and Pinole Valley

Road. The service sub-area is part of the Three Corridors Specific Plan Area Land Uses that includes three land use types: Mixed-Use, Old Town, and Service Sub-Area.

2.4.1 Planned Developments

The City provided Carollo with a list of planned developments which includes a variety of mixed-use and service sub-area land use types. These developments are assumed to be fully developed by the buildout planning horizon. The number of units and area of each planned development is summarized in Table 2.3, while the location of each development is shown on Figure 2.3. The planned developments are expected to result in roughly 395 new residential units and approximately 1.7 acres of commercial development.

Table 2.3 Planned Developments

Development Name	Existing Land Use	Study Area	Residential Units[2]	Residential (ac)	Commercial (ac)
811 San Pablo Avenue	Mixed-Use Sub Area	San Pablo Avenue Corridor	33	0.61	
Vista Woods Apartments 1230 San Pablo Avenue	Mixed-Use Sub Area	San Pablo Avenue Corridor	179	2.02	
The BCRE Project 2801 Pinole Valley Road	Service Sub Area	Pinole Valley Road Corridor	29	1.74	1.74
Appian Way 2151 Appian Way	Service Sub Area	Appian Way Corridor	154	3.38	
Total	-	-	**395**	**7.75**	**1.74**

Notes:
(1) Source: City of Pinole.
(2) All residential units are multifamily.

2.4.2 Additional Infill and Redevelopment

The City currently has little open space/land available for future development. However, there is the potential for redevelopment/densification in certain areas of the City. Figure 2.4 shows the areas that are vacant/underdeveloped.

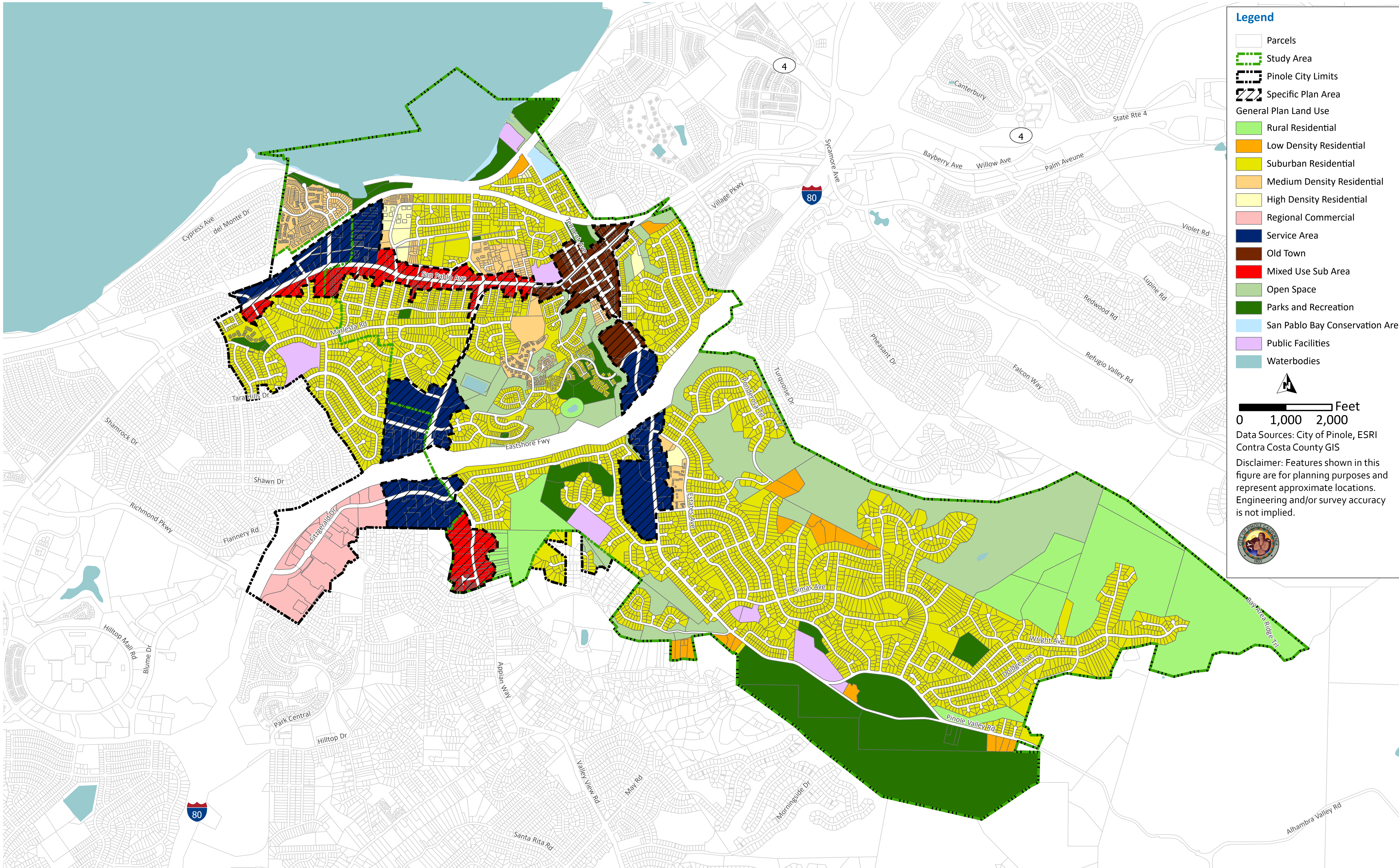


Figure 2.3 General Plan Land Use

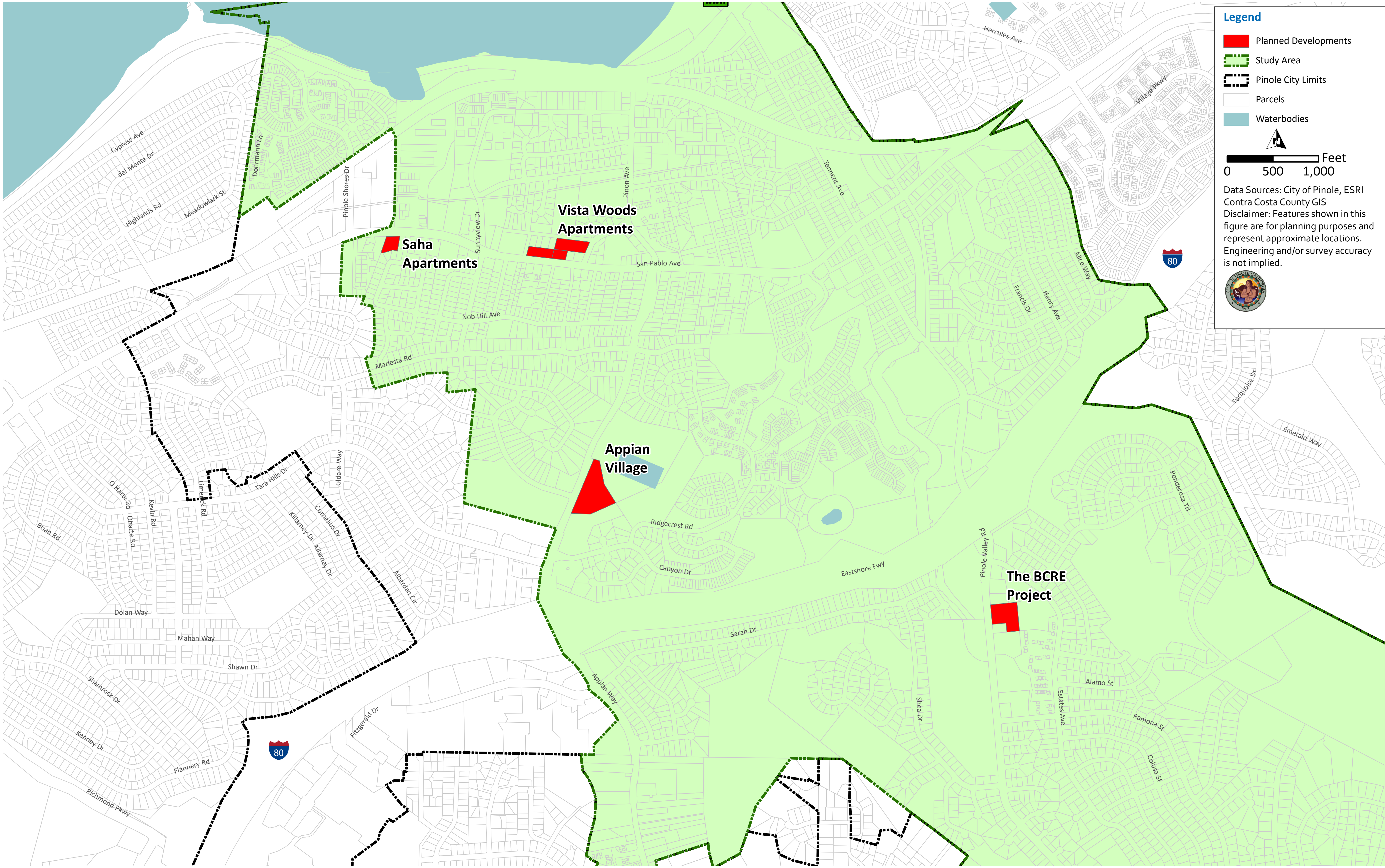


Figure 2.4 Planned Developments

2.4.3 Accessory Dwelling Units

Accessory dwelling units (ADUs) are also expected to contribute wastewater flows in the future. The City provided Carollo with the historic number of ADU applications received from 2018-2021. For the years 2022-2030, a 10-percent increase was assumed. The ADU projection summary is shown in Table 2.4. Development of ADUs beyond 2030 is not included in this Master Plan. It is recommended that the City continue to monitor the number of applications and actual development of ADUs to better estimate the number of future ADUs.

Table 2.4 ADU Projection Summary

Year	ADU's/year[1]
2018	3
2019	4
2020	7
2021	12
2022	13
2023	14
2024	15
2025	17
2026	19
2027	21
2028	23
2029	25
Total	**201**

Notes:
(1) Number of ADU's for 2018-2021 based on number of applications received. Assumed 10 percent increase in ADU applications for 2022 through 2030.

2.5 Population

This section summarizes historical population trends, existing, and projected population.

2.5.1 Historical and Existing Population

Historical population estimates from the California State Department of Finance (DOF) from years 2001 through 2020 are presented in Table 2.5. As of 2020, the total existing population within the City's boundaries was estimated at 19,505 people. This includes the area that is serviced by WCW. The average growth rate, over the past 20 years is approximately 0.12 percent.

Table 2.5 Historic and Existing Population

Year	Population	Population Connected to Sewer	Net Increase	Growth from Previous Year
2001	19,194	17,300	155	0.81 percent
2002	19,140	17,246	-54	-0.28 percent
2003	19,099	17,205	-41	-0.21 percent
2004	19,044	17,150	-55	-0.29 percent

Year	Population	Population Connected to Sewer	Net Increase	Growth from Previous Year
2005	18,837	16,943	-207	-1.10 percent
2006	18,460	16,566	-377	-2.04 percent
2007	18,291	16,397	-169	-0.92 percent
2008	18,304	16,410	13	0.07 percent
2009	18,335	16,441	31	0.17 percent
2010	18,390	16,496	55	0.30 percent
2011	18,533	16,639	143	0.77 percent
2012	18,693	16,799	160	0.86 percent
2013	18,972	17,078	279	1.47 percent
2014	19,117	17,223	145	0.76 percent
2015	19,271	17,377	154	0.80 percent
2016	19,430	17,536	159	0.82 percent
2017	19,498	17,604	68	0.35 percent
2018	19,546	17,652	48	0.25 percent
2019	19,563	17,669	17	0.09 percent
2020	19,505	17,611	-58	-0.30 percent
20 Year Average Growth				**0.12 percent**

Notes:
(1) Source: California DOF.

2.5.2 Projected Population

The projected population growth is summarized in Table 2.6 and shown in Figure 2.5. As shown in Table 2.6, the City is projected to experience a growth rate in population of about 10-percent by 2030, according to the City's General Plan.

Table 2.6 Projected Population Growth

	City Wide Population	Net Increase	Sewer Service Area Population	Net Increase
2020(1)	19,505	-	17,611	-
2025(2)	21,200	8 percent	18,652	6 percent
2030(2)	21,800	10 percent	19,252	3 percent

Notes:
(2) Source: California DOF.
(3) Source: General Plan 2010.

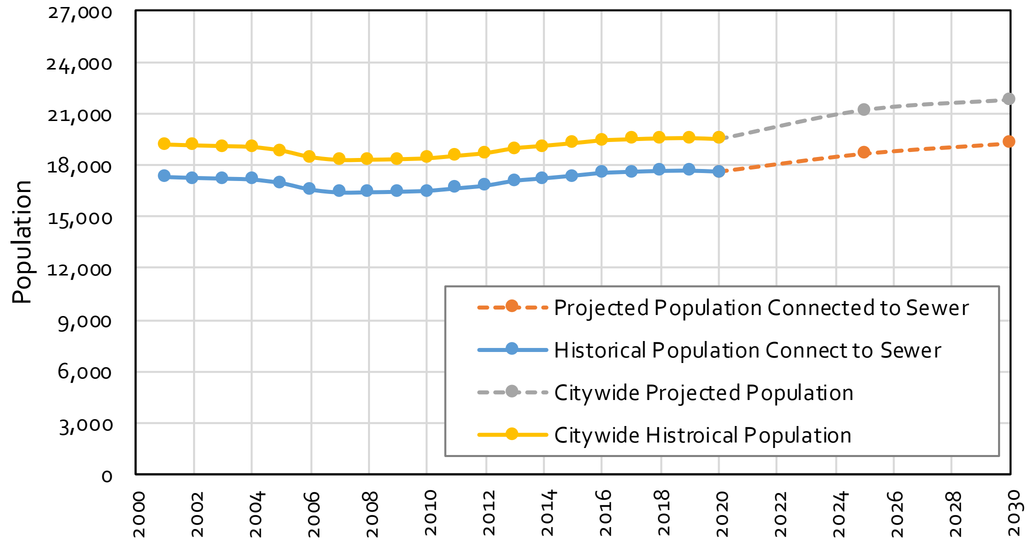


Figure 2.5 Historic and Projected Population for Sewer Service Area

Chapter 3

PLANNING AND EVALUATION CRITERIA

This chapter presents the planning criteria and methodologies for the analysis used to evaluate the City's existing sanitary sewer collection system which are utilized to identify existing system deficiencies, and to size future improvements. The planning criteria address the collection system capacity, acceptable gravity sewer pipe slopes, maximum allowable depth of flow, design velocities, and changes in pipe size.

3.1 Gravity Sewer Criteria

Gravity sewer pipe capacities are dependent on many factors, such as pipe roughness, the chosen maximum allowable depth of flow, pipeline velocity, and slope. The following sections describe the factors that account for the determination of existing and future pipeline capacities in the City's collection system.

3.1.1 Manning's Coefficient (n)

The Manning's coefficient "n" is a friction coefficient that represents resistance to flow, and varies with respect to pipe material and condition, smoothness of joints, root intrusion, and other factors. For sewer pipes, the Manning's coefficient typically ranges between 0.011 and 0.017, with 0.013 being a representative value used for system planning purposes. Due to unknown conditions of existing pipelines, a conservative Manning's "n" factor of 0.013 was initially used for the evaluation of all existing collection system pipelines. Pipe roughness values were adjusted during calibration. The evaluation of all proposed pipelines used a Manning's "n" factor of 0.013.

3.1.2 Peak Flow Depth Criteria

The primary criterion used to identify pipeline capacity deficiencies or to size new sewer improvements is the peak flow depth criteria, which is represented by the d/D ratio (depth of flow, d, to pipe diameter, D, ratio). For example, a minimum d/D of 0.5 means that the maximum allowable flow depth is 50-percent. Based on Carollo's experience, City staff input, and industry standards, the criteria listed in Table 3.1 were used to evaluate existing and proposed sewers.

Table 3.1 Maximum Flow Depth Criteria

Pipe Type	Pipe Diameter (inches)	Maximum Flow Depth Criteria
Existing Sewers	All Pipes	Minimum 3-feet freeboard
New Sewers (Future/Proposed)	Smaller than 15-inch	d/D = 0.50
New Sewers (Future/Proposed)	15-inch and Larger	d/D = 0.75

Maximum flow depth criteria for existing and new sanitary sewers are established based on a number of factors, including the acceptable risk tolerance of the utility, local standards and codes, and other factors.

Using a conservative (low) flow depth criteria when evaluating existing sewers may lead to unnecessary replacement of existing pipelines. Conversely, lenient flow depth criteria could increase the risk of sanitary sewer overflows (SSOs). Ultimately, the maximum allowable flow depth criteria should be established to be as cost-effective as possible while at the same time reducing the risk of SSOs to the greatest extent possible. For the City, existing pipelines where the maximum hydraulic grade line (HGL) reached within 3-ft of the manhole rim. The maximum allowable flow depth for new sewers varies depending on the pipe diameter (0.5 for pipes smaller than 15-inches and 0.75 for pipes 15-inches and larger).

System bottlenecks raise the hydraulic grade line of upstream sewers, leading to backwater conditions. The greater the capacity deficiency, the higher water levels will surcharge upstream of the bottleneck pipeline (or pipelines). The hydraulic model is used to determine "backwater" pipelines in order to specify which specific pipelines are the actual root causes of the capacity deficiency. Capital projects are proposed to provide greater flow capacity for the deficient sewers, which eliminates the backwater conditions that cause surcharging.

3.1.3 Design Velocities and Minimum Slope

To minimize the settlement of sewage solids, it is industry standard to specify a minimum velocity of 2-feet per second (fps). At this velocity, the sewer flow will provide self-cleaning of the pipe. Table 3.2 lists the recommended minimum slopes for new pipes to maintain a minimum velocity of 2-fps.

Table 3.2 Minimum Slope for New Pipes

Pipe Diameter (inches)	Maximum d/D Ratio(1)	Recommended Minimum Slope(2) (feet/feet)
8	0.50	0.0034
10	0.50	0.0025
12	0.50	0.0020
15	0.75	0.0012
18	0.75	0.0009
21	0.75	0.0008
24	0.75	0.0006
30	0.75	0.0005
36	0.75	0.0004
42	0.75	0.0003

Notes:
(1) Based on criteria outlined in Table 3.1.
(2) Recommended minimum slope to provide a minimum velocity of 2 fps (based on maximum allowable flow depth).

3.1.4 Changes in Pipe Size

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the 80 percent depth point of both sewers at the same elevation. For planning purposes and designing new pipes, sewer crowns for new/proposed pipelines are typically matched at the manholes.

3.2 Pump Stations and Force Mains

Industry standard practice is to require that sewage pump stations have sufficient capacity to pump peak flows with the largest pump out of service (firm capacity). Force main piping should be sized to provide a minimum velocity between 3-fps and 8-fps. For the determination of headloss, the Hazen-Williams equation is used with a C-factor of 120, which is typical for sewer system master planning purposes.

3.3 Peak Wet Weather Flow Design Storm

Design storms are rainfall events used to analyze the performance of a collection system under extreme wet weather events. The City's design storm was applied to the collection system hydraulic model to determine peak wet weather flows (PWWFs). The first step in the development of the design storm is to define its recurrence interval and rainfall duration. The recurrence interval is based on the probability that a given rainfall event will occur or be exceeded in any given year. For example, a "100-year storm" means there is a 1 in 100 chance that a storm as large as or larger than this event will occur at a specific location in any year.

The design storm for the City of Hercules was based on the Central Contra Costa Sanitary District Master Plan Update (Carollo, 2017). Because both the City of Hercules and Pinole discharge to the WPCP, the design storm used for the City of Hercules was used to be consistent. The design storm used was a 10-year. 24-hour storm event with a total of 24-hour rainfall volume of 3.52-inches. A 10-year, 24-hour design storm is commonly used in California to determine PWWF in collection systems. The distribution of rainfall for the 10-year, 24-hour event was based on the 12/30-31, 2005 historical event and has a peak hourly rainfall intensity of 0.54 inches per hour. The design storm is shown in Figure 3.1

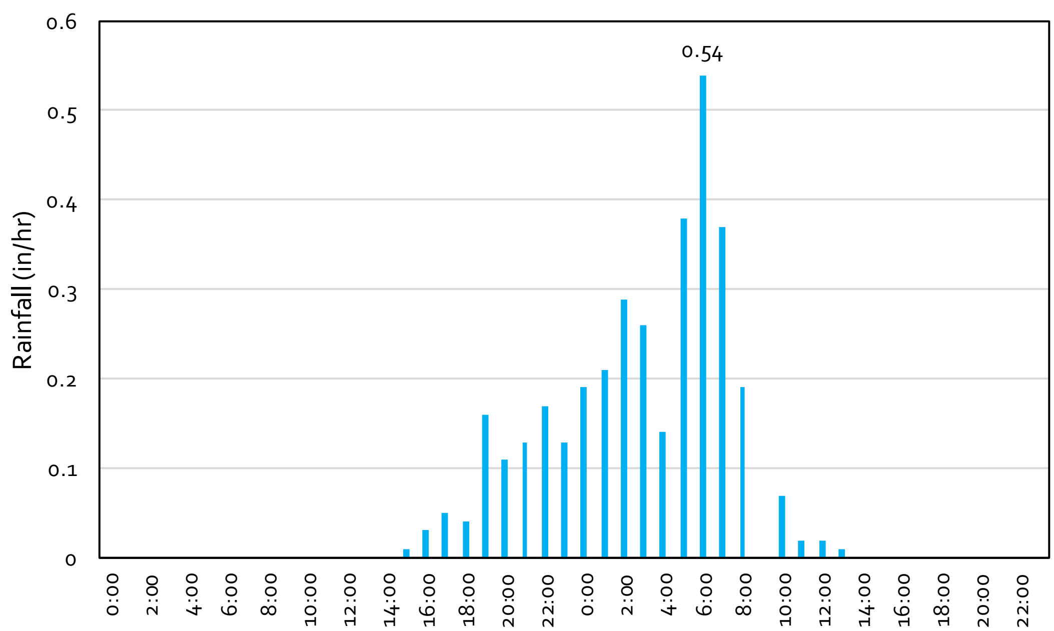


Figure 3.1 10-Year, 24-Hour Design Storm

Chapter 4

WASTEWATER FLOWS

This chapter summarizes the City's historic and projected wastewater flows. Included is a discussion on various flow components present in wastewater and a summary of the flow monitoring data that was used as part of the Master Plan.

4.1 Wastewater Flow Components

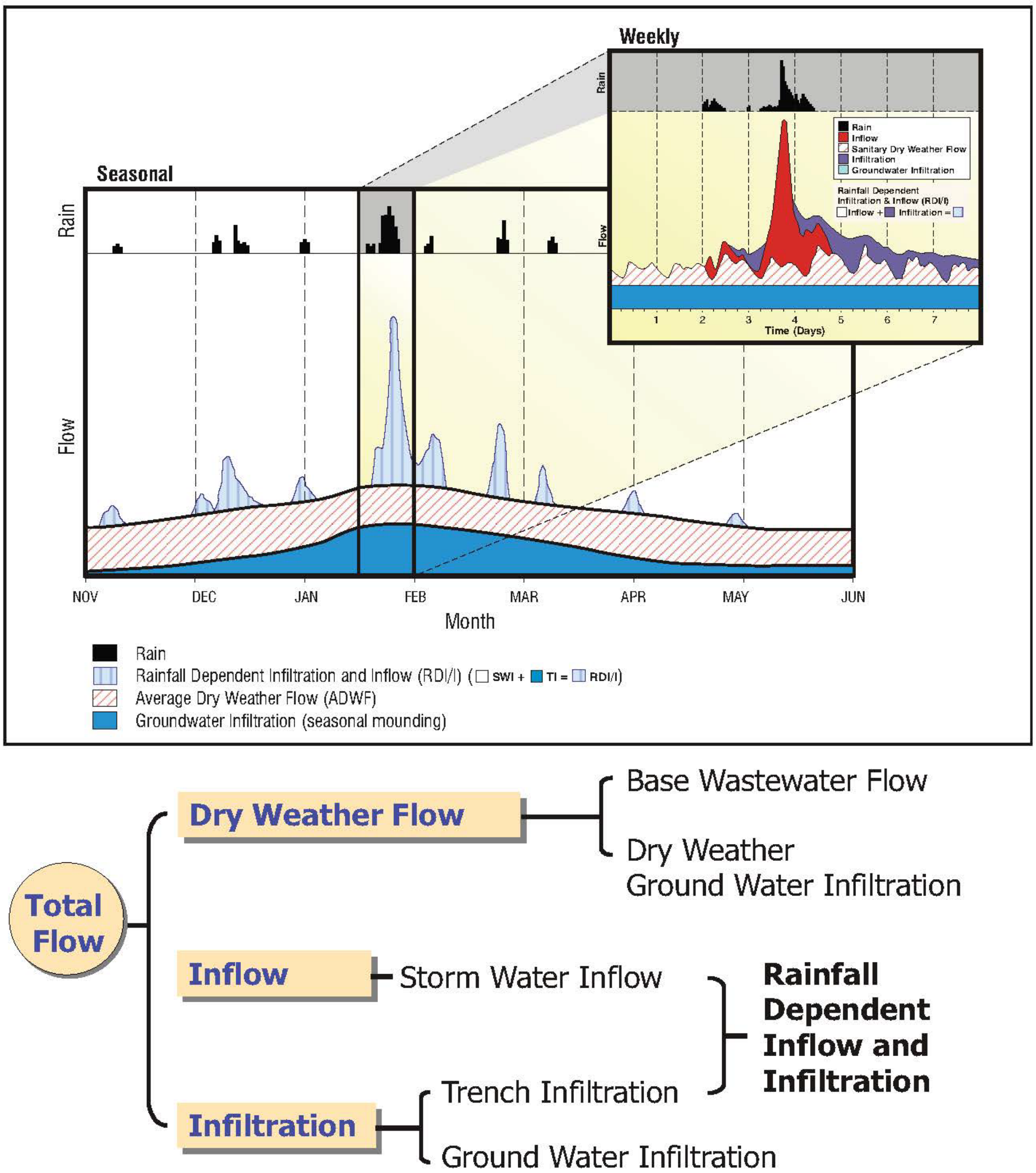
This section describes the terminology used for the hydraulic analysis of the wastewater collection system. Wastewater flows vary according to season and generally consist of DWF and WWF. DWF and WWF both include base wastewater flow (BWF), which is the day-to-day diurnal flow generated by the customers within the City-connected to the sewer system.

Figure 4.1 illustrates the various flow components of wastewater in general, which are described in detail below:

- **BWF.** The BWF is the flow generated by the City's customers independent of wet weather influences. BWF is estimated by measuring flows during dry weather conditions. The flow has a diurnal pattern that varies depending on the type of use. Commercial and industrial patterns, though they vary depending on the type of use, typically have consistently higher flows during business hours and lower flows at night. Furthermore, the diurnal flow pattern experienced during a weekend may vary from the diurnal flow experienced during a weekday.
- **Average Annual Flow (AAF).** The AAF is the average flow that occurs on a daily basis throughout the year, including both periods of dry and wet weather conditions.
- **Average Dry Weather Flow (ADWF).** The ADWF is the average flow that occurs on a daily basis during the dry weather season. The ADWF includes the BWF generated by the City's residential, commercial, and industrial users, plus the dry weather groundwater infiltration (GWI) component.
- **GWI.** GWI is the result of extraneous water entering the sewer system through defects in pipes and manholes. GWI is related to the condition of the sewer pipes, manholes, and groundwater levels. GWI may occur throughout the year, although rates are typically higher in the late winter and early spring. Dry weather GWI (or base infiltration) cannot easily be separated from BWF by flow measurement techniques. Therefore, dry weather GWI is typically grouped with BWF.
- **Rainfall-Derived Inflow/Infiltration (RDII).** Wet weather infiltration and inflow (I/I) causes flows in the collection system to increase. Infiltration is defined as storm water flows that enter the sewer system by percolating through the soil and then through defects in pipelines, manholes, and joints. Examples of infiltration entry points are cracks in pipelines, misaligned joints, and root penetration. Inflow is defined as storm water that enters the sewer system via storm drain cross connections, leaky manhole covers, or cleanouts. Examples of inflow entry points are roof drain and downspout connections, leaky manhole covers, and illegal storm drain connections. Some of the

most common sources of I/I are shown on Figure 4.2. The adverse effects of I/I entering the sewer system is that it increases both the peak flow as well as the total volume, as illustrated on Figure 4.3.

- **PWWF.** PWWF is the highest observed flow that occurs following a design storm event and is typically used for designing sewers, lift stations, and some unit processes in a treatment plant. Therefore, the PWWF and the "Design Flow" are synonymous and will be used interchangeably throughout this report.



Note: This figure is not based on flow data specific to the City or this Master Plan

Figure 4.1 Typical Wastewater Flow Components

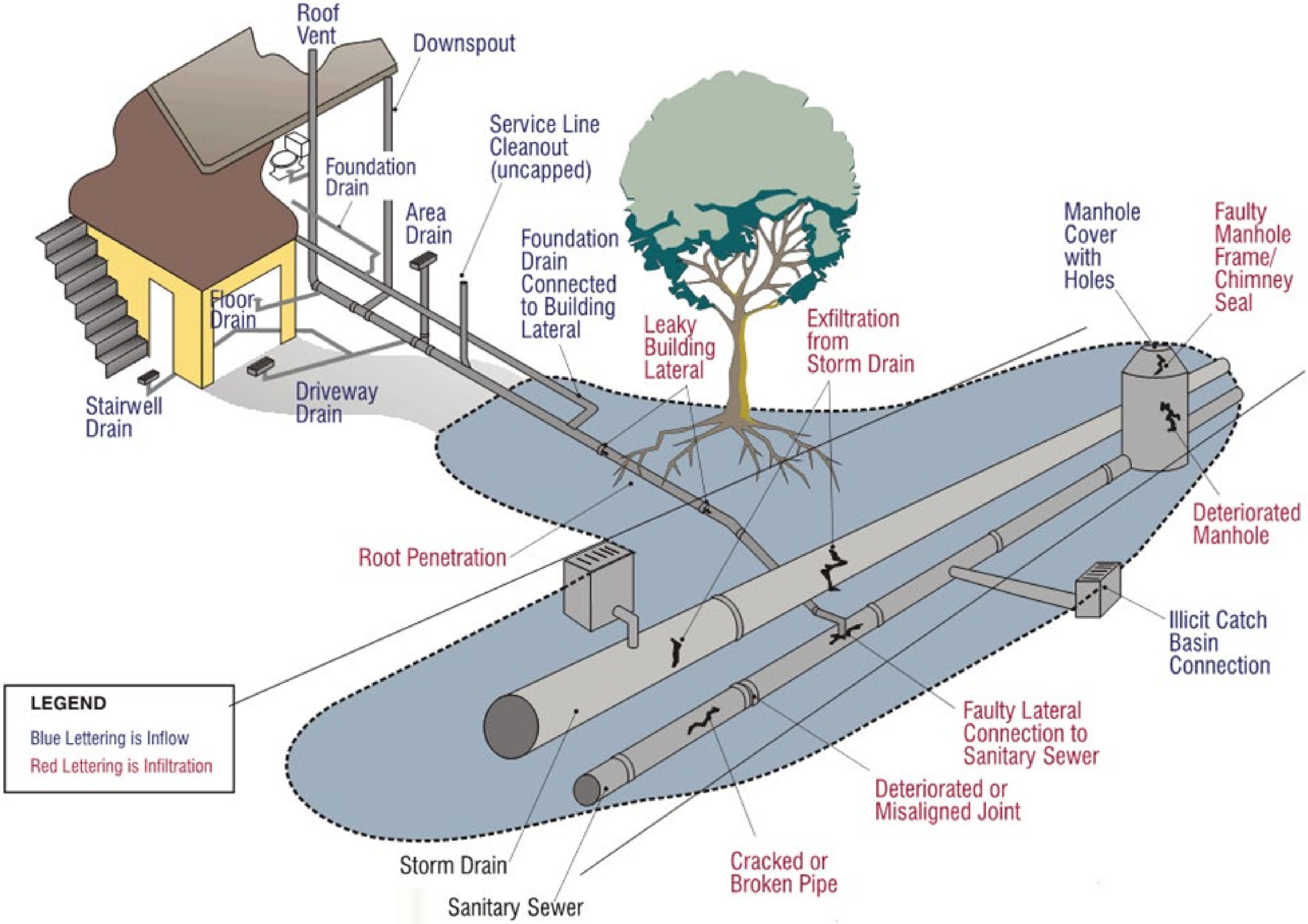


Figure 4.2 Typical Sources of Infiltration and Inflow

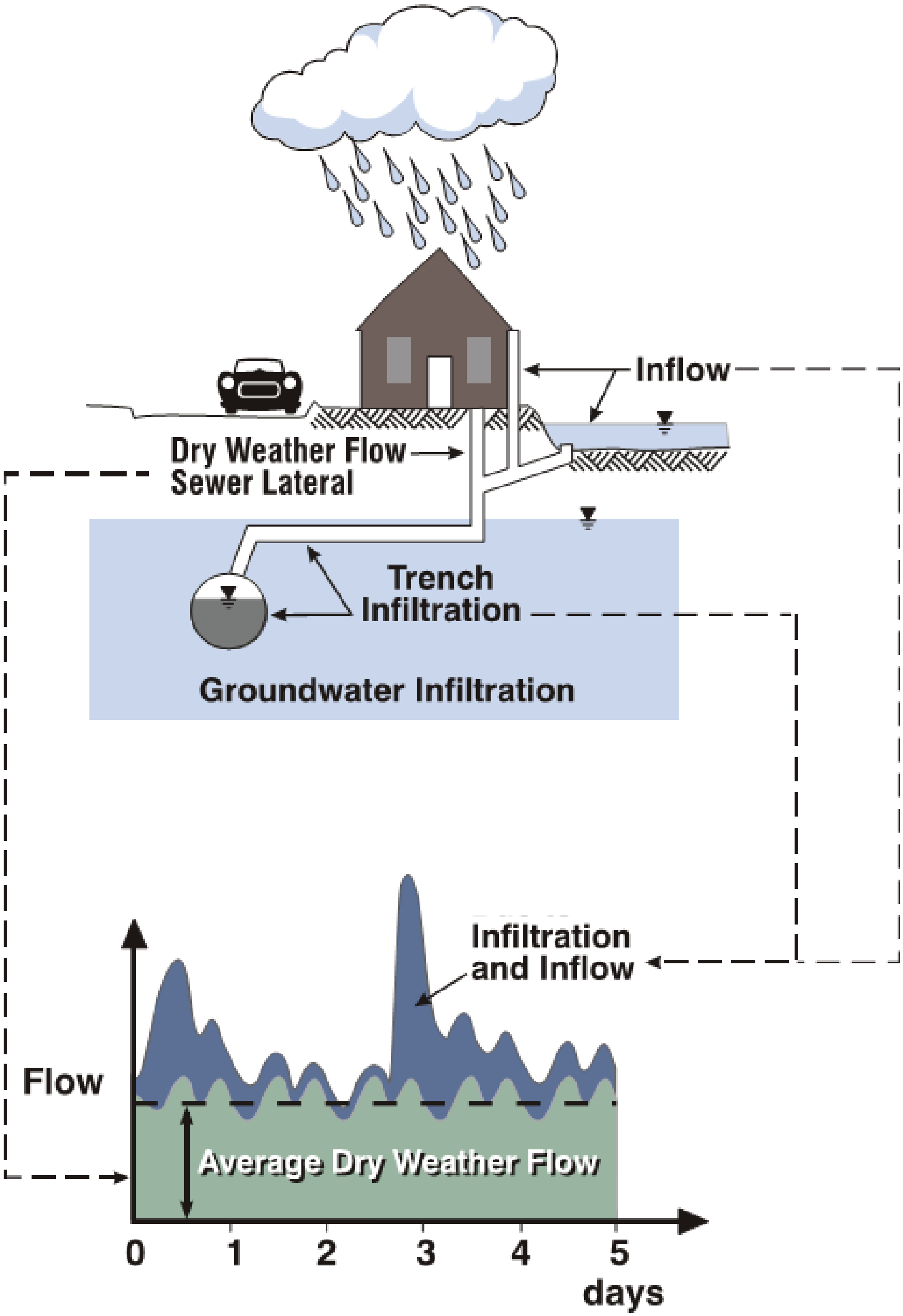


Figure 4.3 Typical Effects of Infiltration and Inflow

4.2 Temporary Flow Monitoring Program

V&A Consulting Engineers, Inc. (V&A) conducted a temporary flow monitoring program between 1/20/2014 and 3/30/2014.Flow monitoring was performed in three phases:

- **Phase 1:** 1/20/2014 through 2/16/2014.
- **Phase 2:** 2/24/2014 through 3/5/2014.
- **Phase 3:** 3/24/2014 through 3/30/2014.

The initial Phase 1 broke the collection system into 16 larger basins generally north of Interstate 80 (I-80), to establish base wastewater flow and identify basins with high rates of I/I. Following the first rain event in Phase 1, some meters were removed and relocated to break up the basins with higher I/I. Phase 3 served to further break down the Phase 2 basins with the highest I/I. Overall, a total of 34 sites were monitored during one or more phases.

The temporary flow monitoring program helped develop design flow criteria and correlate actual collection system flows to the hydraulic model predicted flows. Flow monitoring data was used to calibrate the collection system hydraulic model for dry weather and wet weather flow and to help to identify areas of the system with the highest rates of I/I. The City defined an “Area of Interest” north of I-80 as the goal of the 2014 flow monitoring program.

4.2.1 Flow Monitoring Sites and Tributary Areas

Open-channel flowmeters were installed at 32 gravity sites and volumetric meters sensors were installed at the two lift stations, throughout all three phases. Table 4.1 lists the flow monitoring locations and the sewer diameters where the meters were installed. The 34 flow monitoring locations, as well as the tributary area to each site, are shown on Figure 4.4. Figure 4.5 provides a schematic illustration of the flow monitoring locations for each phase.

Table 4.1 Flow Monitoring Locations

Monitoring Site	Meter Type	Pipe Diameter (in)(2)	Location	Active in Phase 1(1) 1/20/2014-2/16/2014	Active in Phase 2(1) 2/24/2014-3/5/2014	Active in Phase 3(1) 3/24/2014-3/30/2014
Site M1	gravity	15	Pinole valley Road just south of Highway 80	✓	n/a	n/a
Site M2	gravity	30	Tennent Avenue just outside WPCP	✓	✓	n/a
Site M3	LS Logger	n/a	San Pablo Lift Station (San Pablo Avenue west of Sunnyview Drive)	✓	n/a	n/a
Site M3.1	gravity	6	830 Meadows Avenue	n/a	✓	✓
Site M3.1A	gravity	6	Intersection of Meadow Avenue and Betty Avenue	n/a	n/a	✓
Site M3.1B	gravity	6	Intersection of Meadow Avenue and Nob Hill Avenue	n/a	n/a	✓
Site M3.2	gravity	6	830 Meadows Avenue	n/a	✓	n/a
Site M4	LS Logger	n/a	Hazel Lift Station (In easement at west end of Hazel Street)	✓	n/a	n/a
Site M5	gravity	7.25	Appian Way south of San Pablo Avenue	✓	n/a	n/a
Site M5.1	gravity	8	Intersection of Appian Way and Marlesta Road	n/a	✓	n/a
Site M5.2	gravity	8	Intersection of Appian Way and Marlesta Road	n/a	✓	✓
Site M5.2A	gravity	6	1367 Marlesta Road	n/a	n/a	✓
Site M5.3	gravity	6	1171 Marlesta Road	n/a	✓	✓
Site M6	gravity	10	Pinon Avenue north of Bay View Farm Road	✓	n/a	✓
Site M6.0A	gravity	10	Intersection of Roble Avenue and Pinon Avenue	n/a	n/a	✓
Site M6.1	gravity	6	Just west of the intersection of Bay View Farm Road and Pinon Avenue	n/a	✓	n/a
Site M6.2	gravity	8	Intersection of Pinon Avenue and Primrose Lane	n/a	✓	n/a
Site M6.3	gravity	8	Roble Avenue west of Pinon Avenue	n/a	✓	✓
Site M6.3A	gravity	6	Intersection of San Pablo Avenue and 5th Avenue	n/a	n/a	✓
Site M6.3B	gravity	8	Intersection of San Pablo Avenue and Roble Avenue	n/a	n/a	✓
Site M6.4	gravity	8	Intersection of San Pablo Avenue and Rogers Way	n/a	✓	n/a
Site M6.5	gravity	8	747 Sunnyview Drive	n/a	✓	✓
Site M6.5A	gravity	7.75	Intersection of Sunnyview Drive and Patrick Drive	n/a	n/a	✓
Site M6.5B	gravity	7.75	Intersection of Sunnyview Drive and Nob Hill Avenue	n/a	n/a	✓
Site M7	gravity	15	Intersection of Orleans Drive and Zoe Court	✓	n/a	n/a
Site M8	gravity	7.75	Henry Avenue west of Pinole Valley Road	✓	n/a	n/a
Site M9	gravity	6	Intersection of Henry Avenue and Pinole Valley Road	✓	n/a	n/a
Site M10	gravity	8	Intersection of Tennent Avenue and Prune Street	✓	n/a	n/a
Site M11	gravity	10	Intersection of Pinole Valley Road and Rafaela Street	✓	n/a	n/a
Site M12	gravity	8	Intersection of Pinole Valley Road and Rafaela Street	✓	n/a	n/a
Site M13	gravity	6	San Pablo Avenue just west of Quinan Street	✓	n/a	n/a
Site M14	gravity	8	Intersection of Tennent Avenue and Park Street	✓	n/a	n/a
Site M15	gravity	6	Tennent Avenue south of the train tracks west of Fernandez Park	✓	n/a	n/a
Site M16	gravity	11.5	Tennent Avenue north of Orleans Drive	✓	n/a	n/a

Notes:
(1) n/a means it is not active during this phase of the flow monitoring period.
(2) Internal measured diameter.

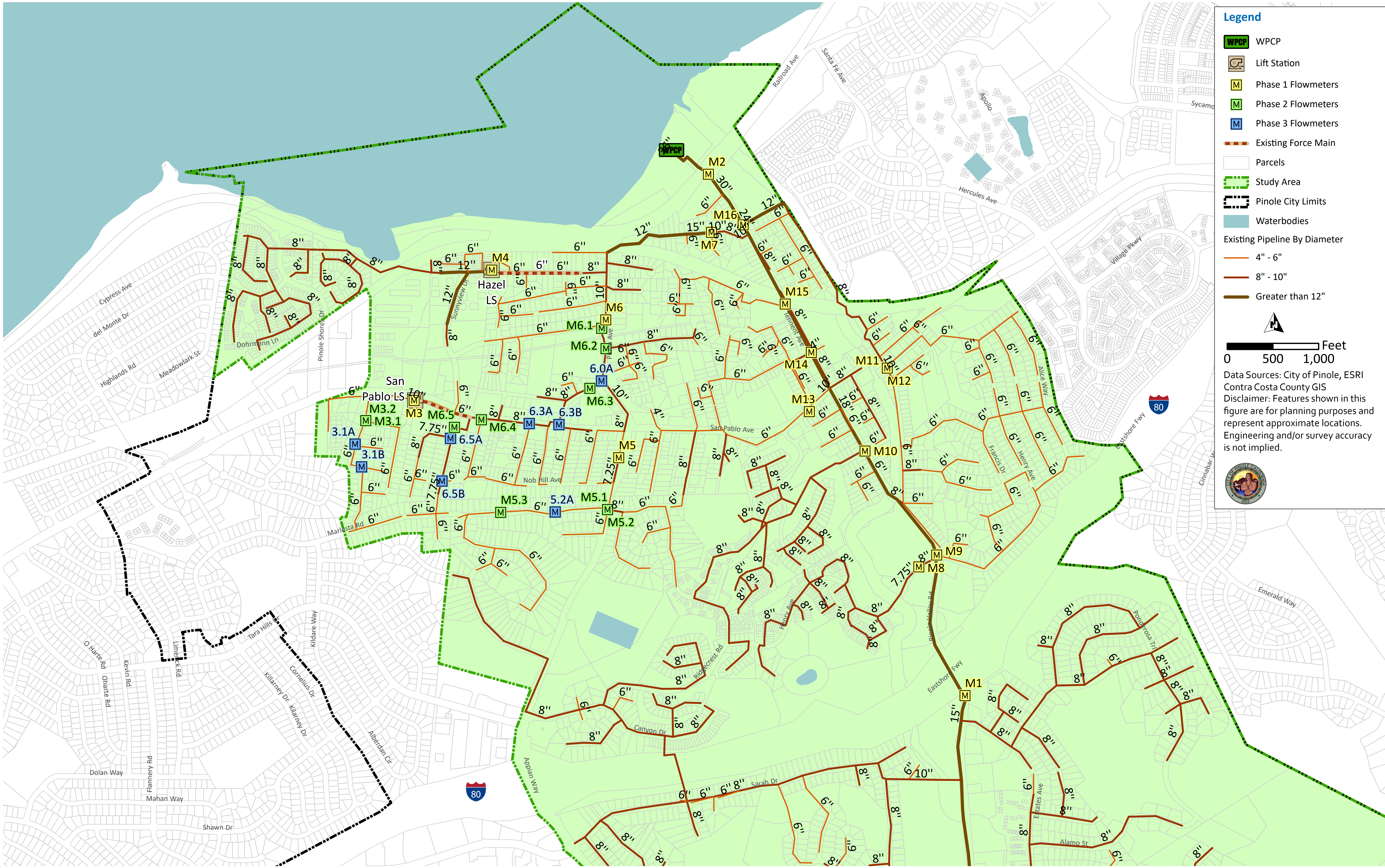


Figure 4.4 Temporary Flow Monitoring Locations

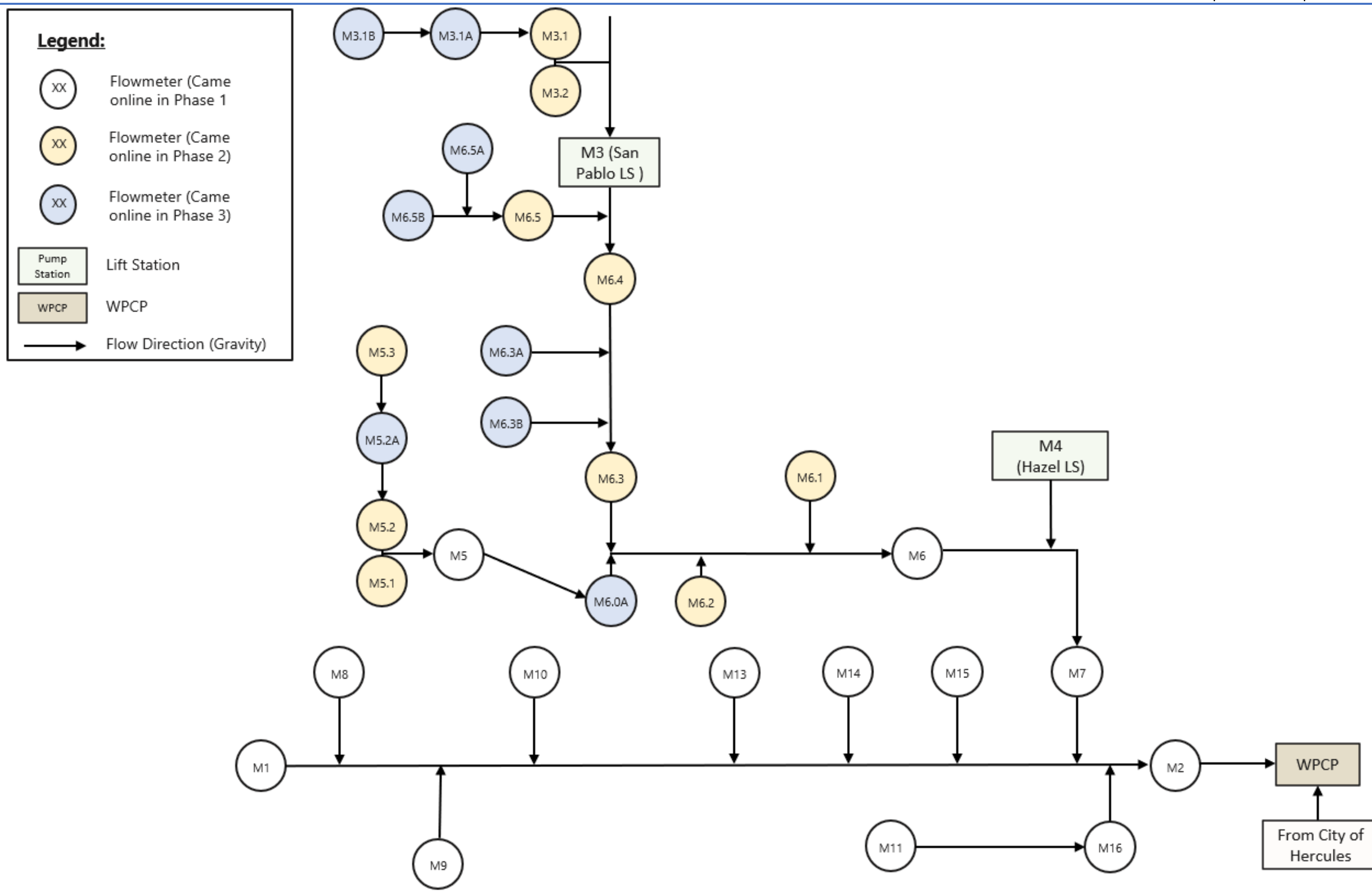


Figure 4.5 Flow Monitoring Schematic

4.2.2 Flowmeter Installation and Flow Calculation

V&A installed a combination of Isco 2150 flowmeters and Hatch Flo-Dar flowmeters for temporary monitoring within the collection system. Isco 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur. Manual level and velocity measurements were taken during the installation of the flowmeters and again when they were removed and compared to simultaneous level and velocity readings from the flowmeters to ensure proper calibration and accuracy. The pipeline diameter was also verified in order to accurately calculate the flow cross-section. The continuous depth and velocity readings were recorded by the flowmeters on 5-minute intervals. The flow at each meter was calculated at 5-minute intervals based on the continuity equation:

$$Q = V \times A$$

where,

Q = Pipeline flow rate, cfs

V = Average velocity, ft/s

A = Cross sectional flow area, ft^2

Finally, the 5-minute flow, velocity, and level data were aggregated into 15-minute increments.

4.2.3 Rain Gauges

V&A collected data from four rain gauges near the City. Rainfall data was collected in 15-minute intervals throughout the flow monitoring period.

4.3 Flow Monitoring Program Results

This section summarizes the results of the flow monitoring program, including dry weather flow, rainfall data, and wet weather flow results. Appendix A includes additional data summaries and other information associated with the temporary flow monitoring program. Results for meter site M7 is presented throughout this chapter as an example.

4.3.1 Dry Weather Flow Data

During the flow monitoring period, flow, depth, and velocity data were collected at each meter at 15-minute intervals. Carollo aggregated the 15-minute data to hourly data for use in the hydraulic model. Characteristic dry weather 24-hour diurnal flow patterns for each site were developed based on the hourly data. This hourly flow data was then used to calibrate the hydraulic model for the observed dry weather flows during the flow monitoring period. For this flow monitoring program, V&A developed two ADWF curves for each site location (weekday and weekend). Hourly patterns were separated this way to better understand how the dry weather flows vary day to day as flows often differ on weekday evenings compared to weekend. V&A used the data from days least affected by rainfall to estimate the weekday and weekend ADWF.

Figure 4.6 illustrates a typical variation of weekday and weekend flows in the City's wastewater collection system, which is based on the data collected from flowmeter site M7. Similar graphics associated with the remaining sites are included in Appendix A. Table 4.2 summarizes the dry weather flows at each meter. As shown on Figure 4.6, flow patterns differ according to the day of the week. Dry weather flow for weekdays experienced an earlier and shorter morning peak and a later evening peak. By contrast, the weekend pattern shows a later and prolonged morning peak, which levels off until the late evening.

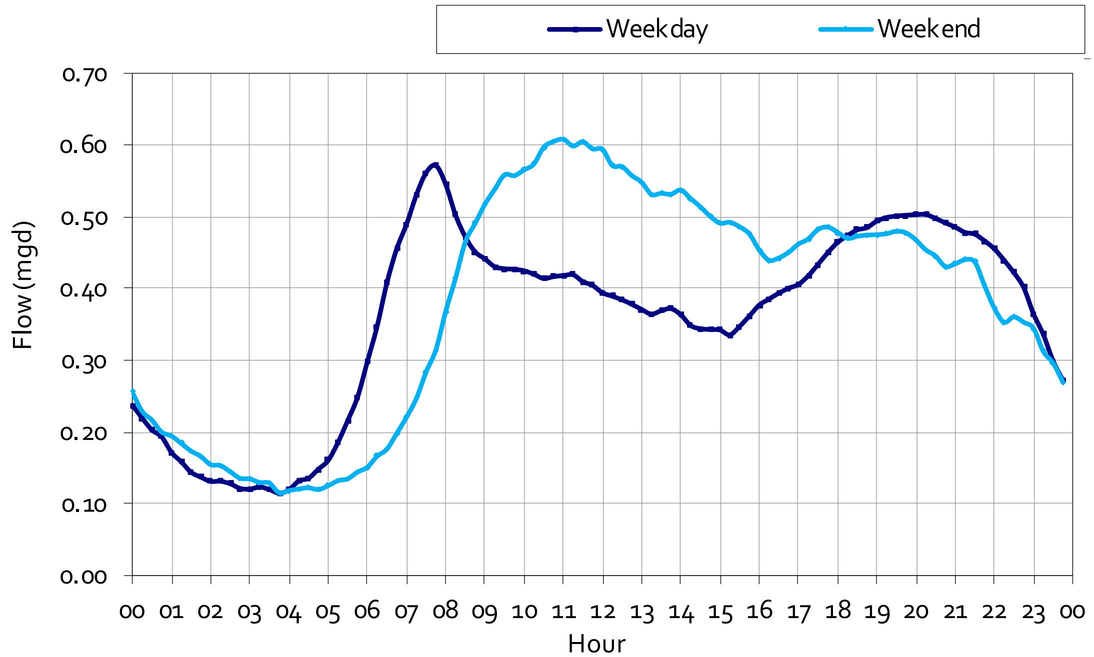


Figure 4.6 Typical Dry Weather Flow Variation (Site M7)

Table 4.2 Dry Weather Flow Summary

Site	ADWF (mgd)		
	Weekday	Weekend	Overall
Phase 1			
Site M1	0.459	0.497	0.47
Site M2	1.11	1.105	1.109
Site M3	0.067	0.068	0.067
Site M4	0.126	0.136	0.129
Site M5	0.019	0.02	0.019
Site M6	0.172	0.189	0.177
Site M7	0.358	0.378	0.364
Site M8	0.007	0.006	0.007
Site M9	0.004	0.005	0.004
Site M10	0.083	0.09	0.085
Site M11	0.018	0.016	0.017
Site M12	0.003	0.003	0.003
Site M13	0.055	0.058	0.056

Site	ADWF (mgd)		
	Weekday	Weekend	Overall
Site M14	0.041	0.044	0.042
Site M15	0.015	0.016	0.015
Site M16	0.083	0.091	0.085
Phase 2			
Site M3.1	0.035	0.025	0.032
Site M3.2	0.011	0.011	0.011
Site M5.1	0.022	0.026	0.023
Site M5.2	0.011	0.013	0.013
Site M5.3	0.022	0.023	0.022
Site M6.1	0.01	0.014	0.011
Site M6.2	0.015	0.018	0.016
Site M6.3	0.171	0.14	0.162
Site M6.4	0.076	0.103	0.084
Site M6.5	0.008	0.011	0.009
Phase 3			
Site M3.1A	0.009	0.01	0.009
Site M3.1B	0.006	0.006	0.006
Site M5.2A	0.009	0.01	0.009
Site M6.0A	0.056	0.061	0.057
Site M6.3A	0.005	0.005	0.005
Site M6.3B	0.003	0.004	0.003
Site M6.5A	0.003	0.003	0.003
Site M6.5B	0.003	0.004	0.003

Notes:
(1) Source: City of Pinole Sanitary Sewer Flow Monitoring and I/I Study, V&A Consulting Engineers, Inc. (2015).
(2) Overall DWF = ((5 x Weekday) +(2 x Weekend))/7.

In addition, V&A provided estimates for the average weekday and weekend levels and velocities at each site, which are used for dry weather calibration.

4.3.2 Rainfall Data

The rainfall data collected by V&A was used to correlate the I/I response observed in the collection system to specific storm recurrence intervals. At least one major rainfall event was captured during each Phase of the flow monitoring program. The rain gauges recorded a total of 6.48 and 11.28-inches of rain during the entire flow program depending on location. The February 2-10, 2014 rainfall event was the most significant event captured and elicited the greatest I/I response throughout the collection system. The rain gauges recorded between 3.00 and 5.47-inches of rain during the February 2-10, 2014 storm event. Table 4.3 summarizes the rainfall amount for each rain gauge for the major storm events for each phase, along with the total rainfall captured during the flow monitoring program.

Table 4.3 Rain Gauge Data

Storm Event	North	East	South	West
February 2 – 10, 2014 (Phase 1)	3.75	5.47	4.76	3.00
February 26 – March 6, 2014 (Phase 2)	2.34	2.48	2.55	1.28
March 26 – April 1, 2014 (Phase 3)	2.45	3.17	3.09	2.46
Season Total (inches)	8.69	11.28	10.55	6.84
February 2/10/2014 Event Classification(2)	< 1 year	4 year, 24 hour	1+ year, 24 hour	< 1 year

Notes:
(1) Source: V&A 2014 Sewer Flow Monitoring and Inflow/Infiltration Study.
(2) Storm event classifications only provide for February 2-10, 2014 event because this was the largest event.

It is important to classify the size of any major storm events captured during the flow monitoring period. National Oceanic and Atmospheric Administration (NOAA) Atlas 14 provides precipitation frequency estimates for the United States based on historical rainfall data and serves as the industry standard for determining total rainfall depth at specified frequencies and durations. The Atlas provides precipitation frequency estimates for 5-minutes through 60-days durations at average recurrence intervals of 1-year through 1,000-year.

The largest storm event classifications for the February 2-10, 2014 storm event is listed in Table 4.4 for each rain gauge site. As shown, the February 2-10, 2014 was classified as a 4-year, 24-hour storm event at the East rain gauge. This storm event was classified as approximately 1-year or less storm event at the other rain gauge sites.

4.3.3 Wet Weather Flow Data

The flow monitoring data was also evaluated to determine how the collection system responds to wet weather events. As mentioned above, the flow monitoring program captured three rainfall events, and all were used for the I/I analysis and model calibration.

Figure 4.7 shows an example of the wet weather response at flowmeter M7 during the February 2-10, 2014 storm event. Additional wet weather monitoring results for all meters can be found in Appendix A. The dashed line is the calculated ADWF (baseline flow) while the black line is the measured flow from the flow monitoring period (real-time flow). The difference between the real-time flow and the baseline flow is essentially I/I. As shown on Figure 4.7 and Appendix A, significant amounts of I/I do enter portions of the collection system during wet weather events. The following section summarizes the results from of V&A's Inflow/Infiltration Analysis as part of the Temporary Flow Monitoring Program.

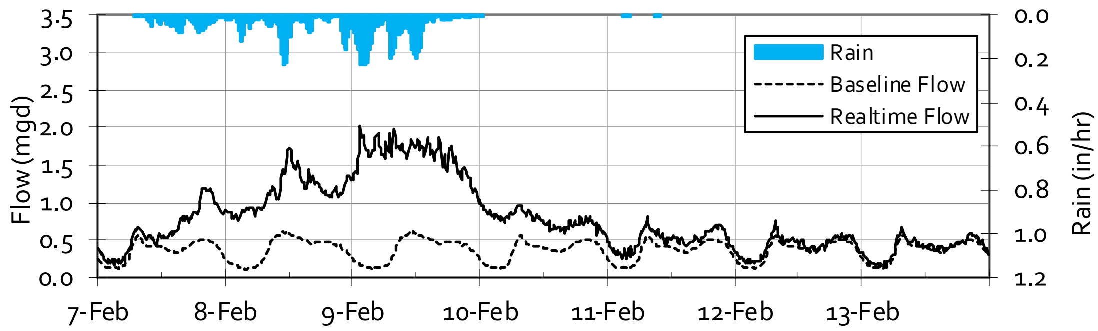


Figure 4.7 Example Wet Weather Flow Response (Site M7)

4.3.4 Inflow and Infiltration Analysis

This section summarizes the findings from V&A's I/I analysis, provided in Appendix A, which was completed for the flowmeters installed as part of the temporary flow monitoring program.

- Site M1 was not included in the analysis because it was used to measure the flow coming into the area of interest.
- Site M2 was not included in the analysis because it was not isolated as a basin because it would have required subtracting flow from 11 other upstream metering sites. This is an issue because when subtracting flows, the inherent error is increased on an additive basis.
- Site M12 was not included because in the analysis because the meter failed after prolonged surcharging during the first storm event.

4.3.4.1 Inflow

As discussed earlier in this chapter, inflow is storm water discharged into the sewer system through direct connections. The corresponding flow rates from these direct connections are closely related to the intensity of the storm. Inflow causes peak flow problems that often dictate downstream pipeline and pump station capacity.

Table 4.4 summarizes the peak measured I/I flows and inflow analysis results. Peak I/I rates for Phase 1 sites were measured for the February 2 – 10, 2014 storm event, Phase 2 sites were measured for the February 26 – March 6, 2014 storm event, and Phase 3 sites were measured for the March 26 – April 1, 2014 storm event. The highest weighted, normalized peak I/I rates are an indicator of high inflow upstream from the flow monitoring basin. Below Summarizes the findings of the inflow analysis:

- In Phase 1, Basins M3, M9 had the highest inflow rankings, respectively. The response for Basin M9 was real and considerable, but the magnitude may not be correct due to metering conditions.
- In Phase 2, Basins M5.2, M3.1, M6.5 had the highest inflow rankings, respectively.
- In Phase 3, Basins M5.2, M6.3A, M3.1 had the highest inflow rankings, respectively.

Table 4.4 Inflow Analysis Summary

Basin	ADWF (mgd)	Acres	Pipe Length (idm)	Measured Peak I/I Rate (mgd)	Measured Peak I/I per idm (gpd/idm)	Measured Peak I/I per acre (gpd/ac)	Measured Peak I/I per ADWF Ratio	Inflow Ranking
Phase 1								
Basin M3	0.067	40	5.09	0.63	123,000	15,700	9.34	1
Basin M4	0.129	587	12.91	0.28	22,000	4,800	2.16	11
Basin M5	0.019	71	10.32	0.64	62,000	9,000	33.50	4
Basin M6	0.09	130	13.01	0.14	11,000	1,100	1.57	13
Basin M7	0.058	41	11.88	0.66	56,000	16,100	11.34	6
Basin M8	0.007	53	6.35	0.02	3,000	400	3.06	12
Basin M9	0.004	28	4.99	0.30	60,000	10,700	67.75	2
Basin M10	0.085	62	11.87	0.26	22,000	4,200	3.05	10
Basin M11	0.017	52	10.48	0.51	49,000	9,800	29.38	7
Basin M13	0.056	17	3.2	0.16	50,000	9,400	2.86	8
Basin M14	0.042	37	6.33	0.30	47,000	8,100	7.17	9
Basin M15	0.015	16	3.65	0.21	58,000	13,100	13.91	5
Basin M16	0.086	12	4.78	0.58	121,000	48,300	6.78	3
Phase 2								
Basin M3.1	0.032	16	2.07	0.55	265,700	34,400	17.3	2
Basin M3.2	0.011	6	1.14	0.06	52,600	10,000	5.7	5
Basin M5.1	0.023	23	3.34	0.12	35,900	5,200	5.2	6
Basin M5.2	0.011	9	1.34	0.23	171,600	25,600	20.9	1
Basin M5.3	0.022	29	3.25	0.08	24,600	2,800	3.6	8
Basin M6.1	0.011	13	2.41	0.10	41,500	7,700	8.9	4
Basin M6.2	0.016	13	3.32	0.03	9,000	2,300	1.9	9
Basin M6.3	0.079	40	2.69	0.04	14,900	1,000	0.5	10
Basin M6.4	0.008	13	1.07	0.03	28,000	2,300	3.7	7
Basin M6.5	0.009	17	3.5	0.11	31,400	6,500	12.6	3
Phase 3								
Basin M3.1	0.0036	3.1	0.69	0.168	243,600	54,200	46.6	3
Basin M3.1A	0.0038	2.4	0.67	0.044	66,000	18,400	11.7	7
Basin M3.1B	0.0057	10.1	1.83	0.087	47,700	8,600	15.2	8
Basin M5.2	0.0025	3.6	0.75	0.220	292,700	61,000	89.3	1
Basin M5.2A	0.0016	5.6	0.69	0	0	0	0	13
Basin M5.3	0.0074	28.9	3.42	0.115	33,500	4,000	15.6	10
Basin M6.0A	0.0459	55.1	10.14	0.703	69,300	12,700	15.3	6
Basin M6.3	0.0146	36.4	5.38	0.283	52,500	7,800	19.4	5
Basin M6.3A	0.0049	7.6	0.97	0.295	304,100	38,800	60.0	2
Basin M6.3B	0.0036	5.9	1.39	0.052	37,600	8,800	14.5	9
Basin M6.5	0.0016	3.4	0.85	0.068	80,600	20,100	41.8	4
Basin M6.5A	0.0029	6.4	1.36	0.028	20,400	4,300	9.4	12
Basin M6.5B	0.0033	7.3	1.32	0.034	25,800	4,700	10.4	11

Abbreviations: gpd - gallons per day; gpd/ac - gallons per day per acre; gpd/idm - gallons per day per inch of diameter per mile; idm - inch of diameter per mile; mgd - million gallons per day.

Notes:

(1) Source: V&A 2014 Sewer Flow Monitoring and Inflow/Infiltration Study (Appendix A).

(2) Ranking of 1 represents most inflow after normalization (compared to other basins).

(3) The inflow ranking is normalized per the following weighting system: 50 percent per IDM, 20 percent per acre, and 30 percent per ADWF.

4.3.4.2 Combined I/I

The combined I/I analysis considers the total volume of I/I over the duration of a storm event. Table 4.5 summarizes the combined I/I results for Phase 1 of the flow monitoring period. Mater basins M11, M16, M15, M5, and M3 had the highest weighted, combined I/I rates, an indicator of high total I/I upstream from the flow monitoring basin.

Table 4.5 Combined I/I Analysis Summary

Basin	ADWF	Total I/I (gallons)	Total I/I per idm	R-Value (per acre)	Total I/I per ADWF	Combined I/I Ranking
Basin M3	0.067	670,000	109,000	19.0 percent	3.07	5
Basin M4	0.129	355,000	8,000	6.9 percent	0.85	11
Basin M5	0.019	570,000	17,000	9.1 percent	9.17	4
Basin M6	0.090	560,000	27,000	4.9 percent	1.90	9
Basin M7	0.058	200,000	5,000	5.5 percent	1.05	10
Basin M8	0.007	1,000	0	0.0 percent	0.05	13
Basin M9	0.004	76,000	5,000	3.1 percent	5.27	7
Basin M10	0.085	291,000	8,000	5.3 percent	1.05	12
Basin M11	0.017	594,000	17,000	12.9 percent	10.51	1
Basin M13	0.056	214,000	21,000	14.2 percent	1.18	8
Basin M14	0.042	412,000	20,000	12.6 percent	3.02	6
Basin M15	0.015	205,000	17,000	14.5 percent	4.17	3
Basin M16	0.086	858,000	55,000	80.9 percent	3.08	2

Notes:
(1) Source: V&A 2014 Sewer Flow Monitoring and Inflow/Infiltration Study (Appendix A).
(2) Ranking of 1 represents most inflow after normalization.

4.4 Design Flows

This section summarizes the historic flows measured at the Hercules-Pinole WPCP and presents the calculation of the design flows used to model the existing and future sewer collection system.

4.4.1 Historical Wastewater Flows

In addition to the flow monitoring program, this project reviewed historical influent flow data at the Hercules-Pinole WPCP from 2013 to 2017 (Data from 2018 to present was not evaluated due to inaccurate flow data) to help establish wastewater flow criteria. Historical flow data is for the City only, it excludes the City of Hercules flow. The City's existing ADWF is 1.10, which was measured in the 2014 flow monitoring program.

Flow data from January 2013 through December 2017 are summarized in Table 4.6. The total annual rainfall was included with the table to help show the trends in the AAF. As expected, the non-drought years generally indicate a higher AAF than in the drought years, which was likely due to water conservation efforts and decreased volumes of I/I entering the collection system. The max day peaking factors for drought years (2013 and 2015) are 1.34 and 2.45 respectively.

The max day peaking factor for non-drought years (2014,2016,2017) are significantly higher and range from 4.02 to 5.66.

Table 4.6 Historical Wastewater Flows

Year	Total Annual Rainfall(3) (inches)	Average Annual Flow (mgd)	ADWF(2)(3) (mgd)	Max Day Flow(3) (mgd)	Max Day: ADWF(3) Peaking Factor
2013	4.0	1.29	1.28	1.72	1.34
2014	16.3	1.34	1.19	5.16	4.34
2015	8.0	1.14	1.10	2.69	2.45
2016	16.4	1.31	1.12	4.49	4.02
2017	20.0	1.60	1.17	6.64	5.66

Notes:
(1) Source: City WPCP Influent flow data.
(2) ADWF is defined as the weighted average flow during the months of June through August.
(3) Total Annual Rainfall Data is from CIMIS Station 170 for Concord, California.
(4) The ADWF and max day flows are from the City only, not the entire WPCP flows which include Hercules flow.

4.4.2 Wastewater Flow Projections

In order to develop wastewater flow projections and allocate future flows to the collection system, relationships between land use and wastewater generation were developed. These relationships, called wastewater flow factors are established based on the average wastewater flow generated (based on flow data collected from the temporary flow monitoring program, discussed in Section 4.3) for each existing land use type. These wastewater flow factors were then compared to the existing population and residential dwelling units to develop a wastewater flow per capita and dwelling unit.

4.4.3 Wastewater Flow Factors

Wastewater flow factors provide a means to estimate flow per acre for each land use category. Wastewater unit flow factors, expressed in gpd/ac, are applied to land use acreage for calculating average day flow generated from a particular land use type. A wastewater flow factor was developed for each of the City's existing land use classifications, based on water billing data and data collected during the 2014 temporary flow monitoring program.

The wastewater flow factors were developed using the following procedure:

1. Water billing data was joined to the parcels and aggregated by billing category (single family [SF] residential, multi-family [MF] residential, government, and non-residential). Winter (January-February) water billing data is usually more representative of average wastewater flows, because less water is typically used for irrigation during winter (wet) months. Water billing data associated with fire hydrants or irrigation were excluded.
2. Each parcel within the Study Area was designated as developed or vacant. The acreage for the developed parcels were added up by land use type and associated water billing category. Open space land use types are assumed to generate negligible wastewater flows and were excluded from this analysis.
3. A return to sewer ratio (the amount of potable water that is returned to the sewer) was applied to each water billing category.

The final wastewater flow factors observed in this Master Plan are summarized in Table 4.7. The wastewater flow factors presented in Table 4.7 represent all customers connected to the City's sewer broken down by billing category.

Table 4.7 Wastewater Flow Factors

Land Use	Area (acres)	Wastewater Flow Factor (gpd/ac)
SF Residential	1,267	770
MF Residential	21	2,890
Government	257	100
Non-Residential	141	1,360

Based on DOF data, there were 17,611 people living within the study area in 2020. Based on the wastewater flow factors presented in Table 4.7, this yields an approximate rate of 58 gpd/person. Based on DOF data, there is 2.81 people/dwelling units (DU). For this Master Plan, SF homes are assumed to have an average of 3 people/DU and MF homes and ADU's are assumed to have an average of 2 people/DU. This yields and approximate rate of 177 gallons per day per dwelling unit (gpd/DU) for SF and 118 gpd/DU for MF residential types. The final flow factors used to estimate the future wastewater flows are presented in Table 4.8. The future flows for residential areas are based on the flow per dwelling unit, whereas the future flows for commercial (non-residential) and government are based on the flow factors listed in Table 4.7, rounded up to the nearest 10 gpd/acre.

Table 4.8 Future Wastewater Unit Flow Rates

Land Use	Units	Factor
SF Residential(1)	gpd/DU	177
MF Residential(2)	gpd/DU	118
Government	gpd/ac	100
Non-Residential	gpd/ac	1,360

Notes:
(1) Assumed 3 people per DU at approximately 58 gpd/person.
(2) Assumed 2 people per DU at approximately 58 gpd/person.

Future ADU's are assumed to be built on existing SF residential lots, however, wastewater flows generated from ADU's are estimated based on the MF wastewater flow factor (118 gallons per day per accessory dwelling unit [gpd/ADU]) which assumes two people per ADU.

4.4.4 Wastewater Flow Projections

Developing an accurate estimate of the future quantity of wastewater generated in the collection system is an important step in maintaining and sizing sewer system facilities, for both existing conditions and future developments.

4.4.4.1 Average Dry Weather Flow

The calibrated ADWF based on the flow monitoring data was not changed for existing parcels currently connected to the City's wastewater collection system. In general, the following assumptions were used for determining the future ADWF for buildout:

- The number of planned residential (SF and MF) units were provided by the City. The unit flow per dwelling unit presented in Table 4.9 were applied to each planned development based on the number of SF and/or MF units proposed.
- Projected wastewater flows for planned commercial developments were based on the acres associated with the planned development and the wastewater flow factors (gpd/acre) presented in Table 4.8.
- The future wastewater flows for the planned developments, vacant infill, and densification were allocated in the model as point loads based on the location of the planned development/parcels, the location of existing sewers, and the topography of the area.
- Projected wastewater flows for ADU's were based on a MF residential unit flow rate of 118 gpd/ADU. Because it is impossible to know where the ADUs will be constructed, the total projected flows for ADUs were evenly distributed throughout the model among the existing SF residential parcels.

Table 4.9 summarizes the existing and projected ADWF for the study area, based on the methodology and assumptions stated above. As shown, the City's wastewater flow is expected to increase by 25.2 percent at buildout (from 1.1 mgd to 1.47 mgd).

Table 4.9 Projected ADWF Summary

Flow Component	Wastewater Flow (mgd)
Existing	1.10
Future	
Planned Developments	0.05
Vacant	0.05
Underdeveloped	0.08
Densification	0.20
ADU's	0.02
Total at Buildout	**1.47**

4.4.4.2 Peak Wet Weather Flow

The PWWF is the highest observed hourly flow that occurs following the design storm event (discussed in Chapter 3). Wet weather I/I, which occurs during and after rainfall events, increases flows in the collection system. The City's collection system was evaluated based on its capacity to convey the PWWF.

The existing PWWF was derived based on the hydraulic modeling results. This was accomplished by applying the 10-year, 24-hour design storm to the hydraulic model, which was calibrated to both dry weather and wet weather conditions. The 10-year, 24-hour design storm volume is approximately 3.52 inches, although may vary slightly depending on the specific location within the study area.

Similar to the existing PWWF, the future (buildout) PWWF was derived by applying a 10-year, 24-hour design storm to the hydraulic model under future conditions. A peak I/I rate of 500 gpd/ac was assumed for future developments and vacant infill. Redevelopment areas and ADU's are not expected to contribute to future I/I.

Table 4.10 presents a summary of the existing and buildout ADWF and PWWFs as well as the PWWF peaking factors. The PWWFs presented in Table 4.10 assume there are no capacity constraints in the collection system. This is essentially the PWWF after the recommended improvements are constructed. Without the recommended improvements, the PWWF at the outfall would be less (due to backwater and SSO's). As shown in Table 4.10, the existing PWWF to ADWF peaking factor decreases from 13.83 to 10.70 for buildout.

Table 4.10 Projected Wastewater Flow Summary

Year	Projected Wastewater Flow (mgd)(1)		
	ADWF(2)	PWWF(3)	PWWF to ADWF Peaking Factor
Existing	1.10	15.21	13.83
Buildout	1.47	15.73	10.70

Notes:
(1) Does not include flows from the City of Hercules.
(2) Model simulated, system-wide peak hourly flow for entire study area.
(3) Modeled average dry weather loads. ADWF loads were adjusted during calibration to closely match all flow meter locations. The resulting final modeled ADWF is slightly higher than the calculated and measured ADWF from the flow monitoring program.

Chapter 5

WASTEWATER COLLECTION SYSTEM FACILITIES AND HYDRAULIC MODEL

This chapter describes the construction and calibration of the City's sewer collection system hydraulic model. It provides a description of the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled collection system, the hydraulic model elements, the model creation process, and the model calibration process.

5.1 Collection System Facilities

The City's collection system consists of gravity sewers, pump stations, and associated force mains that collect and convey wastewater to the Pinole-Hercules WPCP, which is located on Tennent Avenue, south of the San Pablo Bay and north of Highway 80. The City's wastewater collection system consists of approximately 50 miles of gravity sewers, over 1,300 manholes, two pump stations, and approximately 807 linear feet of force mains according to the City's AutoCAD files. Pinole-Hercules WPCP provides wastewater treatment to the Cities of Pinole and Hercules. The WCPC is owned and operated by the City. Figure 5.1 presents the City's existing collection system.

5.1.1 Gravity Collection System

The City's gravity collection system is comprised of roughly 50 miles of gravity pipe up to 36 inches in diameter and over 1,300 manholes. Table 5.1 presents a summary of the existing gravity sewers, by diameter. As shown in Table 5.1, roughly 89 percent of the system is 8-inches in diameter and smaller, with the majority of the system (roughly 54 percent) being 8-inches in diameter.

Table 5.1 Collection System Gravity Pipeline Diameter Summary

Pipe Diameter (inches)	Length[(1)] (miles)	Percent of System (by Length)
4	0.1	0.2
6	17.1	34.6
8	26.7	54.0
10	2.6	5.3
12	0.9	1.9
15	1.0	2.0
18	0.8	1.6
24	0.0	<0.1 percent
30	0.2	0.5
36	<0.01	<0.1 percent
Total	**49.4**	**--**

Notes:
(1) Includes all gravity pipes in the City's Model database within the study area.

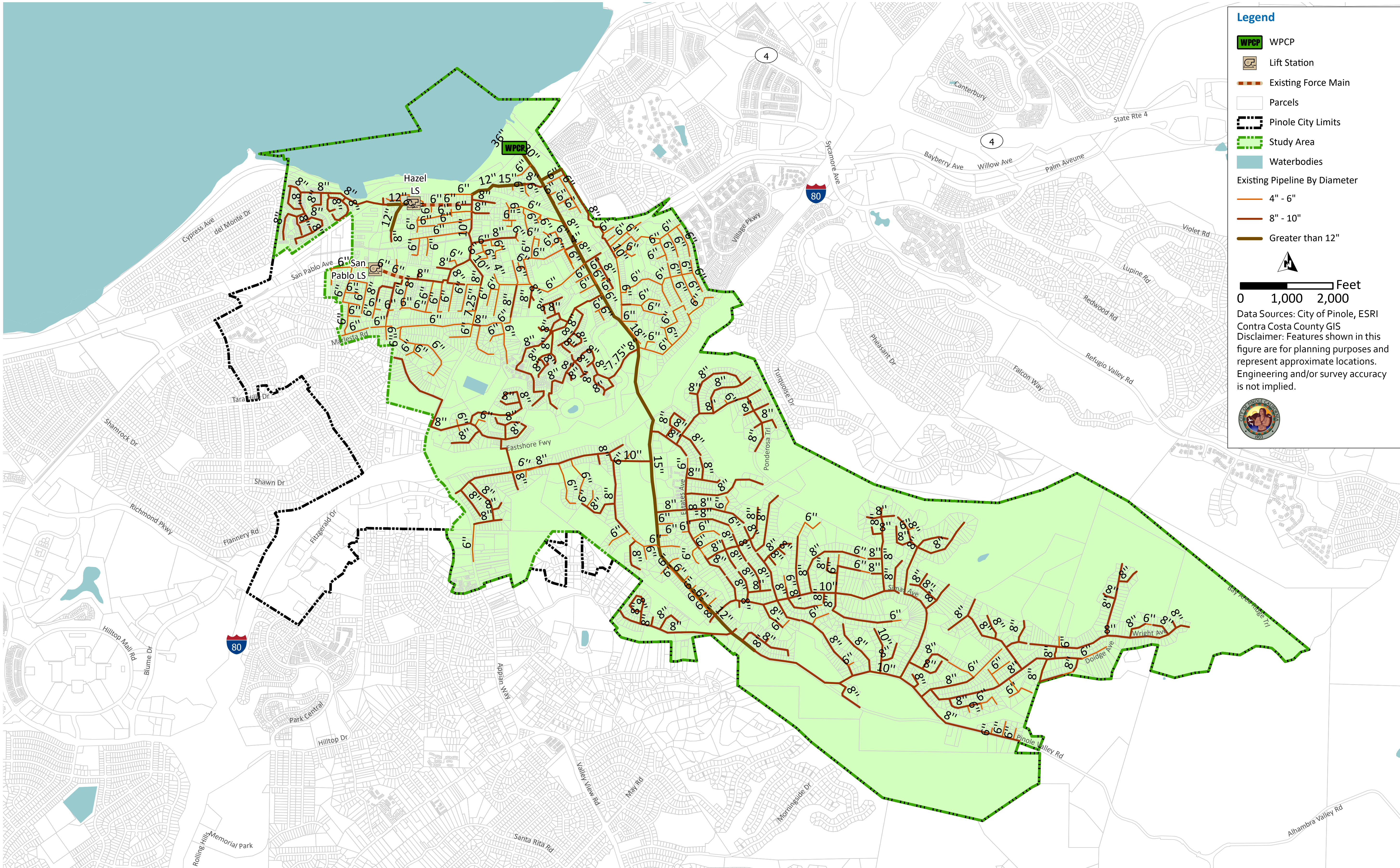


Figure 5.1 Existing Collection System

5.1.2 Pump Stations and Force Mains

The City operates and maintains two wastewater pump stations throughout the City. Figure 5.2 shows the locations of each pump station and the area that it serves. A brief summary of each pump station is presented below:

- **San Pablo:** The San Pablo pump station is located on San Pablo Avenue east of Meadow Avenue and west of Sunnyview Drive. The pump station consists of a 6-foot diameter, 14-feet deep wet well with two 300 gallons per minute (gpm) submersible pumps. The San Pablo Pump Station conveys raw wastewater east via a 6-inch diameter force main approximately 625 feet until it discharges into the 8-inch gravity main on San Pablo Avenue west of Rodgers Way.
- **Hazel:** The Hazel pump station is located between Hazel Street and Sunnyview Drive. The pump station consists of a 8-feet by 4-feet cubical wet well that is 11-feet with two 405 gpm submersible pumps. The Hazel pump station conveys raw wastewater via a 6-inch diameter force main approximately 940 feet until it discharges into a 8-inch diameter sewer main downstream between Alfred Drive and Pinon Avenue.

5.2 Hydraulic Model Development

A sewer collection system model is a simplified representation of the real sewer system. Sewer system models can assess the conveyance capacity for a collection system and can also be used to perform "what if" scenarios to assess the impacts of future developments and land use changes. The City's collection system hydraulic model was constructed using a multi-step process utilizing data from various sources. This section summarizes the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled collection system, the hydraulic elements, and the model calibration process.

5.2.1 Hydraulic Modeling Software

There are several software applications for network analysis with a variety of capabilities and features. The selection of a particular model is generally dependent upon user preference, the requirements of the particular collection system, and the cost associated with the software.

InfoSWMM®, developed by Innovyze (formerly MWH Soft), was selected as the software platform for the development of the City's hydraulic model. The hydraulic modeling engine for InfoSWMM® uses the Environmental Protection Agency's (EPA's) Storm Water Management Model (SWMM), which is widely used throughout the world for planning, analysis, and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems. InfoSWMM® routes flows through the model using the Dynamic Wave method, which solves the complete Saint Venant one dimensional equations of fluid flow.

InfoSWMM® consists of multiple components that work together to bring a graphical approach to the analysis and design of wastewater and stormwater collection systems. The program includes seamless integration with geographic information system (GIS) data.

5.2.2 Elements of the Hydraulic Model

The following provides an overview of the elements of the City's hydraulic wastewater collection system model and the required input parameters associated with each:

- **Junctions:** Sewer manholes, cleanouts, as well as other locations where pipe sizes change or where pipelines intersect are represented by junctions in the hydraulic model. Required inputs for junctions include rim elevation, invert elevation, and surcharge depth (used to represent pressurized systems). Junctions are also used to represent locations where flows are split or diverted between two or more downstream links.
- **Pipes:** Gravity sewers and force mains are represented as pipes in the hydraulic model. Input parameters for pipes include length, friction factor (e.g., Manning's n for gravity mains, Hazen Williams C for force mains), invert elevations, diameter, and whether or not the pipe is a force main.
- **Storage Nodes:** For sewer system modeling, storage nodes typically are used to represent lift station wet wells (although other storage basins would be modeled as storage nodes). Input parameters for storage nodes include bottom elevation, maximum depth, and cross sectional area.
- **Pumps:** Pumps are included in the hydraulic model as links. Input parameters for pumps include pump curves and operational controls.
- **Outfalls:** Outfalls represent areas where flow leaves the system. For sewer system modeling, an outfall typically represents the connection to the influent pump station or the headworks at a wastewater treatment plant. Required input parameters include ground elevation, outfall type (freefall, fixed head, etc.).
- **Inflows:** The following are the two types of wastewater flow sources that can be applied at individual model junctions (and storage nodes):
  - **External.** External inflows can represent any number of sources entering the collection system, such as metered flow data or groundwater infiltration. External inflows are applied to a specific model junction by applying a baseline flow value and a pattern that varies the flow by hour, day, or month of the year. This option was used to simulate future inflow/infiltration.
  - **Dry Weather.** Dry weather inflows simulate base sanitary wastewater flows and represent the average flow. The dry weather flows can be multiplied by up to four patterns that vary the flow by month, day, hour, and day of the week (e.g., weekday or weekend). The dry weather diurnal patterns are adjusted during the dry weather calibration process.

5.2.3 Hydraulic Model Construction

The City's hydraulic model combines information on physical and operational characteristics of the wastewater collection system and performs calculations to solve a series of mathematical equations to simulate flows in pipelines.

The model construction process consisted of six steps, as described below:

- **Step 1:** The City's drawings and GIS shapefiles for the sewer collection system were obtained.

- **Step 2:** The GIS data was reviewed and formatted to allow easy import into the InfoSWMM® modeling platform. The City's collection system alignment and manhole placement were imported into the modeling software.
- **Step 3:** A majority of the physical and operational data for the City's wastewater collection facilities was not available from the City's original base GIS data. Data, such as pipeline inverts, wet well dimensions, pump stations, and other special features, were input manually into the model based on additional sources. In addition, discrepancies with pipeline alignment and junction placement were reviewed and manually input or modified based on City records, field reconnaissance, and engineering judgment.
- **Step 4:** Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the model data was input correctly and that the flow direction and size of the modeled pipelines were logical. Additionally, the modeled lift stations were also checked to verify that they operated correctly.
- **Step 5:** Dry weather wastewater flows were then allocated to the appropriate model junctions. These flows were scaled, as necessary, to match the DWFs recorded during the flow monitoring period.
- **Step 6:** The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run dates, time steps, reporting parameters, output units, and flow routing method. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

5.2.4 Wastewater Load Allocation

Determining the quantity of base wastewater flows generated by a municipality and how they are distributed throughout the collection system is a critical component of the hydraulic modeling process.

Various techniques can be used to assign wastewater flows to individual model junctions, depending on the type of data that is available. Adequate estimates of the volume of wastewater are important in maintaining and sizing sewer system facilities, both for present and future conditions. Baseline wastewater loads were allocated (assigned to specific nodes) in the hydraulic model based on water billing data provided by the City, as well as the flow data from the 2014 temporary flow monitoring program. The following steps outline the wastewater load allocation process:

- **Step 1:** Each parcel within the City's service area was assigned a modeled manhole ID. The water billing data provided by the City was joined to the associated parcel in GIS.
- **Step 2:** Each parcel within the City's sewer service area was then assigned a modeled manhole ID. At the end each parcel has a water consumption amount, land use type, area in acres, and a model junction assigned to it.
- **Step 3:** The water consumption was added for each model manhole ID and allocated in the model.
- **Step 4:** Once the existing wastewater flows were allocated into the model, they were adjusted as needed during model calibration to closely match the dry weather flows recorded during the flow monitoring program.

5.3 Hydraulic Model Calibration

Hydraulic model calibration is a crucial component of the hydraulic modeling effort. Calibrating the model to match data collected during the flow monitoring program ensures the most accurate results possible. The calibration process consists of calibrating to both dry and wet weather conditions. This section summarizes the overall methodology employed to calibrate the City's wastewater collection system hydraulic model and the calibration results, including a detailed description of each of the major components of the model calibration process.

For this project, both dry and wet weather flow monitoring were conducted during three phases along a 2 month period starting in January 2014 and ending in March 2014. DWF calibration ensures an accurate depiction of base wastewater flow generated within the study area. The WWF calibration consists of calibrating the hydraulic model to a specific storm event or events to accurately simulate the peak and volume of I/I into the sewer system. The amount of I/I is essentially the difference between the WWF and DWF components.

5.3.1 Calibration Standards

The hydraulic model was calibrated in accordance with international modeling standards. The Wastewater Planning Users Group (WaPUG), a section of the Chartered Institution of Water and Environmental Management, has established generally agreed upon principles for model verification. The dry weather and wet weather calibration focused on meeting the recommendations on model verification contained in the "Code of Practice for the Hydraulic Modeling of Sewer Systems," published by the WaPUG (WaPUG 2002), as summarized below:

- **Dry Weather Calibration Standards:** Dry weather calibration should be carried out for two dry weather days and the modeled flows and depths should be compared to the field measured flows and depths. Both the modeled and field measured flow hydrographs should closely follow each other in both shape and magnitude. In addition to the shape, the flow hydrographs should also meet the following criteria as a general guide:
  - The timing of flow peaks and troughs should be within 1 hour.
  - The peak flow rate should be within the range of ±10 percent.
  - The volume of flow (or the average rate of flow) should be within the range of ±10 percent. If applicable, care should be taken to exclude periods of missing or inaccurate data.
- **Wet Weather Calibration Standards:** The model simulated flows should be compared to the field measured flows. The flow hydrographs for both events should closely follow each other in both shape and magnitude, until the flow has substantially returned to DWF rates. In addition to the shape, the flow hydrographs should also meet the following criteria as a general guide:
  - The timing of the peaks and troughs should be similar with regard to the duration of the events.
  - The peak flow rates at significant peaks should be in the range of +25 percent to -15 percent and should be generally similar throughout.
  - The volume of flow (or the average flow rate) should be within the range of +20 percent to -10 percent.

5.3.2 Dry Weather Flow Calibration

The DWF calibration process consists of several elements, as outlined below:

- **Divide the system into areas tributary to each flowmeter.** The first step in the calibration process was to divide the City into flowmeter tributary areas, one for each flow monitoring site. A map showing the locations of each flow monitoring site and their associated tributary area are provided in Chapter 4 along with a schematic of the flowmeters.
- **Define flow volumes within each area.** The next step was to define the flow volumes within each area, which was accomplished in the flow allocation step (described in Section 5.2.3).
- **Create diurnal patterns to match the temporal distribution of flow.** A diurnal curve is a pattern of hourly multipliers that are applied to the base flow to simulate the variation in flow that occurs throughout the day. Two diurnal curves were developed for each flow monitoring tributary area, one representing weekday flow and one representing weekend flow. The diurnal patterns were initially developed based on the flow monitoring data and adjusted as part of the calibration process until the model simulated flows matched the field measured flows as closely as possible. Figure 5.2 shows the calibrated weekday and weekend diurnal pattern for the area tributary to Site M5. Additional diurnal patterns were developed for all flowmeter tributaries. These diurnal patterns are found on the DWF calibration sheets that are included in Appendix B.

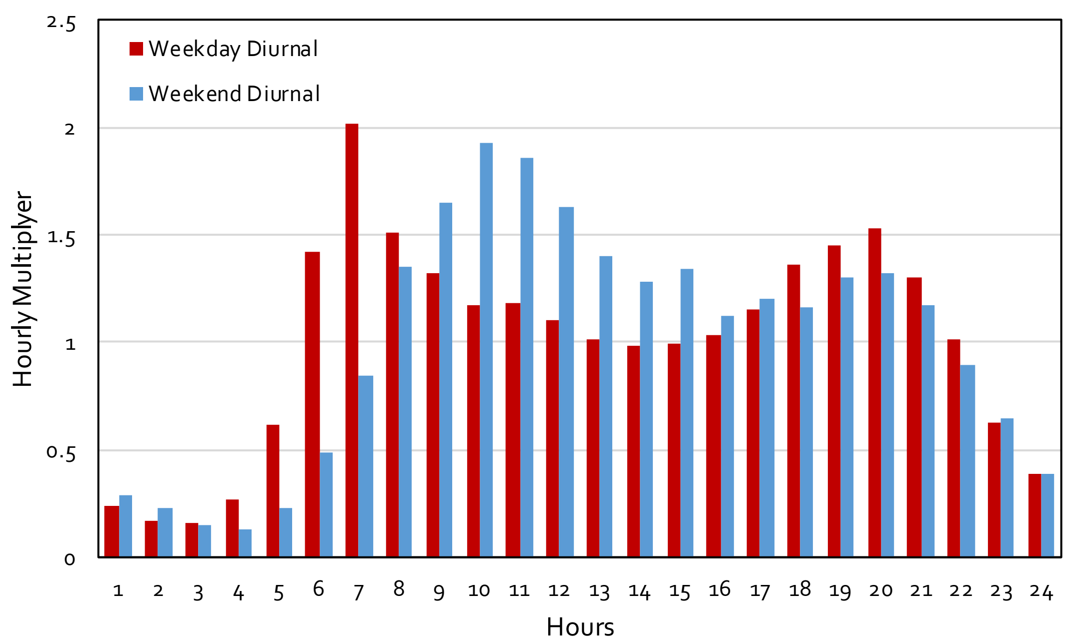


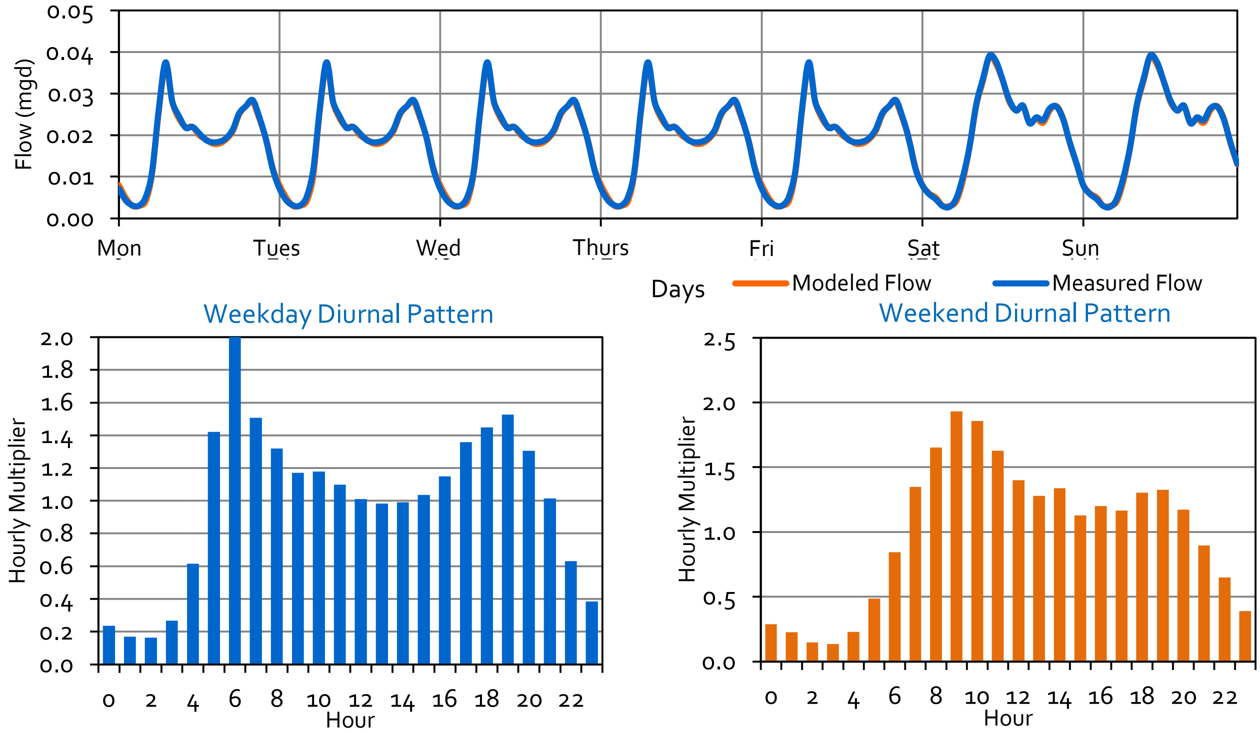
Figure 5.2 Example Weekday and Weekend ADWF Diurnal Patterns (Site M5)

- **Adjust model variables to match field measured velocity and flow depths.** Once the model simulated flows acceptably matched the field measured flows, the model simulated velocity and flow depth were compared to the field measured velocity and flow depth. Adjustments were made to various model parameters until the modeled and measured velocity and depth closely matched one another. The primary varied parameters for this process are pipeline roughness (Manning's n) and sediment buildup in the pipe, although other parameters can also be adjusted as calibration results are generated.

- Manning's roughness coefficients, or n values, have industry accepted ranges based on a number of variables. Roughness coefficients increase over time depending on the construction methods, installation quality, system maintenance, and other environmental factors. There can be certain factors within the City's collection system that can result in roughness coefficients that differ from the typical range. For example, pipeline bellies, joint misalignment, cracks, and debris (e.g., root intrusion, etc.) lead to increased turbulence in a pipe, as well as the apparent Manning's n factor.
- If the model is unable to reasonably match the field measured flow depth and velocity without leaving the acceptable range of manning's roughness coefficients, further investigation is conducted to help determine the cause of the discrepancy. Some issues that could cause such a discrepancy can include errors in the slope or diameter of a pipeline, downstream blockages, pipeline sags, and, in some cases, influences from downstream pump station operations.

Figure 5.3 is an example DWF calibration sheet for flowmeter Site M5. Calibration sheets provide plots and tables that compare model results and the field measured flow, velocity, and level for during the calibration period. Appendix B contains detailed DWF calibrations sheets for all meter locations. As shown in Appendix B, all of the model simulated average flows for weekday and weekend DWF were all within 10 percent.

Overall, the hydraulic model met the established dry weather calibration standards. Some areas require further investigation. Overall, the model accurately simulates DWF, and the sites that did not had little impact on the model's overall accuracy. For these reasons the mode is considered calibrated for DWF conditions.



Model Calibration Summary

Day	Measured Data(1) Avg. Flow (mgd)	Measured Data(1) Peak Flow(2) (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow(2) (mgd)	Percent Error(3) Avg. Flow (%)	Percent Error(3) Peak Flow (%)
Mon.	0.02	0.04	0.02	0.04	-1%	-1%
Tues.	0.02	0.04	0.02	0.04	-1%	-1%
Wed.	0.02	0.04	0.02	0.04	-1%	-1%
Thur.	0.02	0.04	0.02	0.04	-1%	-1%
Fri.	0.02	0.04	0.02	0.04	-1%	-1%
Sat.	0.02	0.04	0.02	0.04	-1%	0%
Sun.	0.02	0.04	0.02	0.04	-1%	0%
Summary						
Weekday	0.019	--	0.018	--	-1%	--
Weekend	0.020	--	0.020	--	-1%	--
ADWF(4)	0.019	--	0.019	--	-1%	--

Notes:

1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Figure 5.3 Example DWF Calibration Sheet (Site M5)

5.3.3 Wet Weather Flow Calibration

The wet weather calibration enables the hydraulic model to accurately simulate I/I entering the collection system during a large storm event. As outlined below, the WWF calibration process consists of several elements:

- **Identify calibration rainfall events.** For this project, the WWF calibration process consists of running model simulations of a historic rainfall event. The goal of any WWF calibration is to capture and characterize a system's response to a significant rainfall event, preferably during wet antecedent moisture conditions. During the temporary flow monitoring program, one major storm event was captured on March 21-22, 2018. This storm event was used for wet weather calibration.
- **Define RDII tributary areas.** For the WWF calibration, RDII flows are superimposed on top of the DWF. The model calculates RDII by assigning "RDII Inflows" to each node in the model. RDII inflows consist of both a unit hydrograph and the total area that is tributary to the model node. The RDII tributary areas were calculated in GIS using the loading polygons. The tributary area provides a means to transform hourly rainfall depth from the rainfall hyetographs into a rainfall volume. The rainfall volume is transformed into actual RDII flows using the unit hydrograph, as described in the next step.
- **Create I/I parameter database and modify to match field measured flows.** The main step in the WWF calibration process involved creating a custom unit hydrograph for the study area using the "RTK Method," which is widely used in collection system master planning. Using the RTK Method, the RDII unit hydrograph is the summation of three separate triangular hydrographs (short term, medium term, and long term), which are each defined by three parameters: R, T, and K. R represents the fraction of rainfall over the sewer basin that enters the collection system; T represents the time to peak of the of the hydrograph; and K represents the ratio of time to recession to the time to peak. Therefore, there are a total of nine separate variables associated with a unit hydrograph. Figure 5.4 shows the shape of an example unit hydrograph.

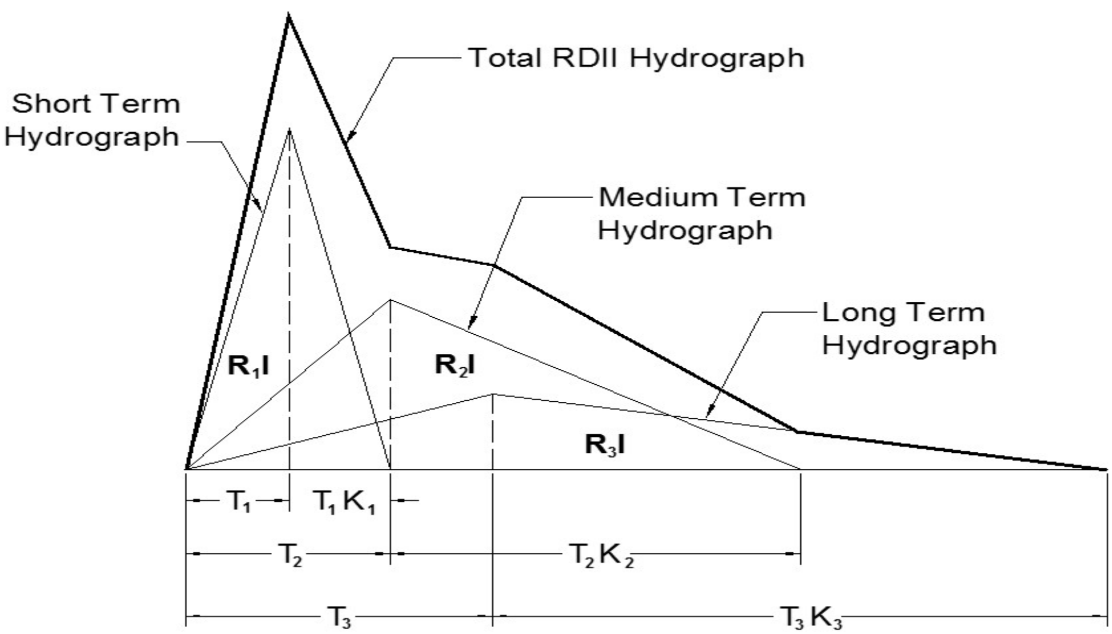


Figure 5.4 Example RDII Unit Hydrograph

The hydrograph utilizes the R-values (percent of rainfall that enters the collection system) calculated for each basin to simulate I/I. The nine variables in each unit hydrograph were initially set based on engineering judgment and then adjusted until the model simulated flows (both peak flows and average flows) matched closely with the field measured flows.

As with the dry weather calibration, the wet weather calibration process compared the measured flow data with the model output. Comparisons were made for average and peak flows as well as the temporal distribution of flow until flows returned to their baseline levels.

Figure 5.5 is an example WWF calibration sheet for flowmeter site M15. The WWF calibration sheets show figures comparing the measured data and model results for flow, velocity, and level in response to rainfall. The WWF calibrations sheets for all sites are provided in Appendix C. There is good correlation between the model-simulated flows and the flows that were measured at each meter location. Overall, the model accurately simulated the effects of wet weather events, and was considered calibrated and ready to use for capacity analysis purposes.

5.3.4 Collection System Hydraulic Model Calibration Summary

In summary, the calibration results indicate the model predicts conditions similar to those observed in the field. Within a few isolated areas of the model, there are some very minor discrepancies, but the overall collection system is very well represented in the model.

Based on the results presented in this chapter, it can be concluded that the model is calibrated to DWF and WWF conditions. The model provides an accurate representation of the City's wastewater collection system to a level suitable for this Master Plan and for the City's future hydraulic modeling needs.

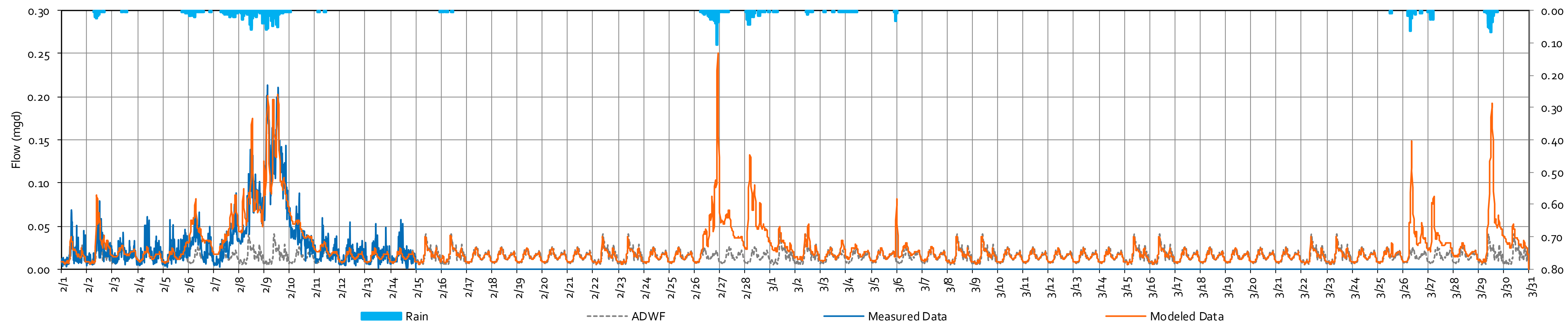


Figure 5.5 Example WWF Calibration Sheet (M15)

Chapter 6

CAPACITY EVALUATION AND PROPOSED IMPROVEMENTS

This chapter discusses the hydraulic evaluation of the sewer collection system and the proposed projects that correct capacity deficiencies and serve future users.

6.1 Capacity Evaluation

Following the dry and wet weather flow calibration, which is summarized in detail in Chapter 5, a capacity analysis of the existing and future collection system was performed. The capacity analysis entailed identifying areas in the sewer system where flow restrictions occur or where pipe capacity is insufficient to convey PWWFs. Sewers that lack sufficient capacity to convey PWWFs create bottlenecks in the collection system that can potentially cause SSOs. The sewer system was evaluated based on planning criteria presented in Chapter 3.

This section discusses the locations of current and projected hydraulic deficiencies resulting from flows exceeding the maximum allowable flow depth criteria.

6.1.1 Existing System

For the existing sewer collection system, the PWWF was routed through the hydraulic model. Pipelines where the maximum HGL reached within 3-feet of the upstream manhole rim, were identified. The existing deficiencies are shown on Figure 6.1. There were some areas that exhibited backwater conditions at PWWF. These are indicated in green on Figure 6.1. Replacing a capacity limited (bottleneck) sewer will allow for higher peak flows to be carried to downstream sewers. In some cases, this increase in peak flow overwhelms the downstream sewers, which creates additional deficiencies. The two lift stations that did not have the firm capacity to convey PWWF. These are indicated in orange on Figure 6.1.

Following the completion of the existing system analysis, improvement projects were identified in order to mitigate existing pipeline capacity deficiencies. The recommended improvement projects are discussed in greater detail in Section 6.2. In accordance with the established planning criteria, new sewer pipelines were sized such that the maximum flow d/D did not exceed the values summarized in Chapter 3.

6.1.2 Future System

The analysis of the future (buildout) system was performed in a manner similar to the existing system analysis. The purpose of the future system evaluation is to verify that the existing system improvements were appropriately sized to convey future peak flows, and to identify the locations of sewers that are adequately sized to convey existing peak flows, but cannot convey future peak flows. The buildout planning period includes the complete buildout of the study area, including full capacity of all planned developments. There were no new deficiencies identified based on the future system evaluation.

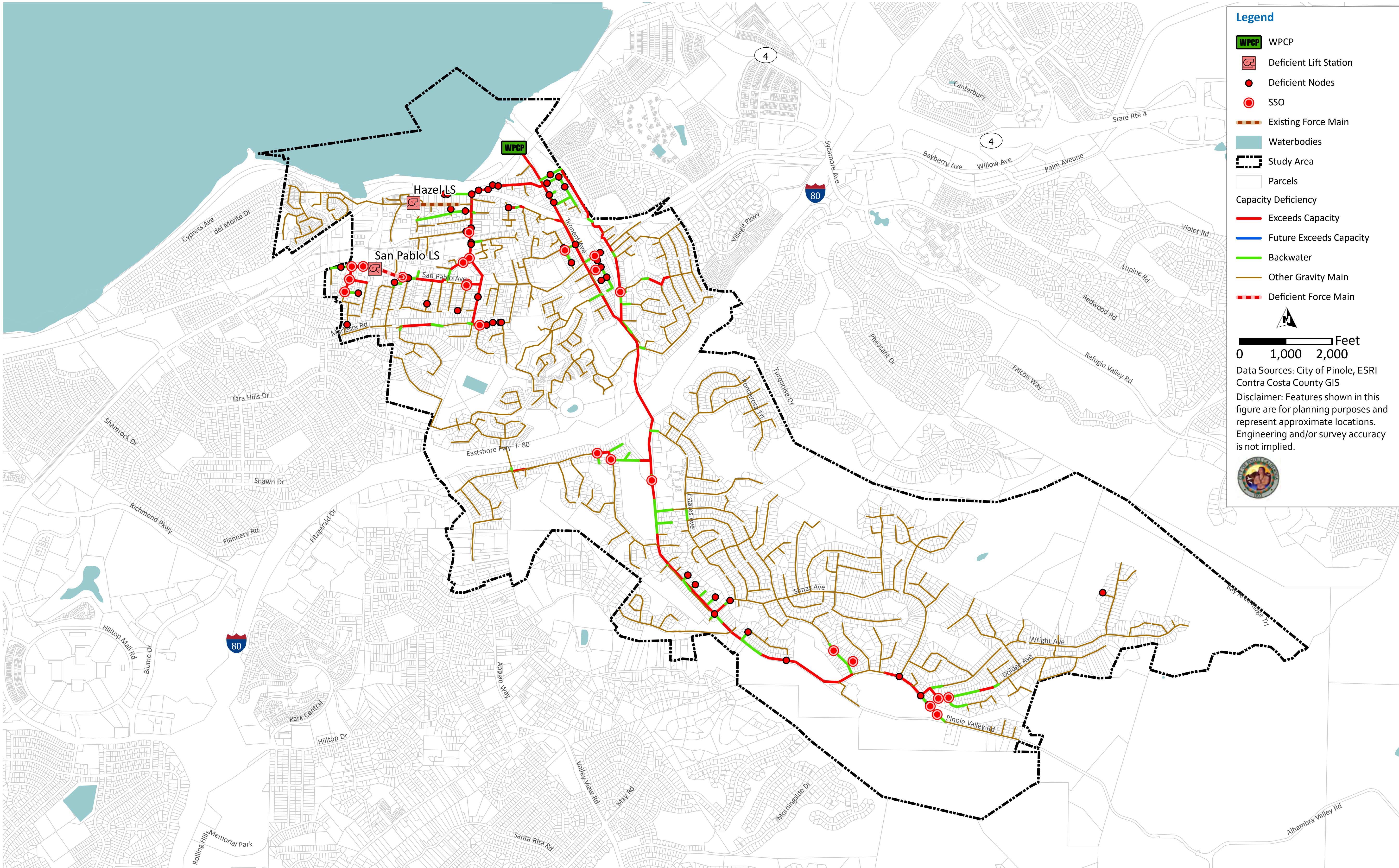


Figure 6.1 Collection System Capacity Deficiencies

6.2 Collection System Improvements

Based on the hydraulic analysis the significant contribution of I/I in the City's collection system has created multiple capacity deficiencies. The improvements to mitigate the capacity deficiencies are all listed in the following sections of this report, however mitigating the deficiencies through pipeline upgrades in the planning horizon of this report are not financially feasible for the City at this time. A programmatic approach to address I/I as well as pipeline condition, while still addressing some of the most pressing capacity constraints is included in this Master Plan in the near term.

6.2.1 Programmatic Recommendations

In addition to developing capacity improvements to mitigate existing and anticipated future deficiencies, the following projects are also recommended:

- **RR-2 (Pipe Rehabilitation and Replacement Program):** This is an annual program to rehabilitate or replace aging pipes or pipes with poor condition. The results of the City's closed-circuit television (CCTV) inspection program should be used to identify the pipes most in need of rehabilitation and replacement. Additionally, a long-term risk assessment should be completed to identify long term rehabilitation and replacement projects and funding needs. It is also recommended that gravity pipes less than 8 inches in diameter be replaced with 8-inch pipe. The length/total cost recommended in the CIP is an estimate. Once the City completes the CCTV inspection and has a better idea of the condition of the collection system, the length/total cost should be adjusted as needed.
- **RR-3 (Inflow Identification Program):** The 2014 flow monitoring program revealed several subbasins within the collection system that exhibited higher rates of inflow. This project includes smoke testing and/or nighttime CCTV and/or field reconnaissance to identify potential sources of inflow. The City should take action if illicit connections to the sewer are found. Figure 6.3 shows the basins that should be targeted first for inflow identification.
- **O-1 (Sewer Master Plan Update):** It is recommended that the City updates their Master Plan every 5 years to re-evaluate the wastewater collection system.
- **O-2 (Flow Monitoring Program):** It is recommended that the City conduct a flow monitoring program every 5 years to aid with the Master Plan (O-1). It is assumed that each program will consist of 15 flow meters for a 1-month period. Flow monitoring should be timed to capture at least one major storm event, preferably following wet ground conditions.

6.2.2 Capacity Improvements

Figure 6.3 illustrates the proposed sewer improvements required to correct existing deficiencies and to serve future users. When an increase to capacity is required, existing sewers can be upgraded or a parallel or relief sewer can be constructed. For the purposes of this study, unless otherwise stated, it was assumed that capacity deficient sewers would be upgraded to a larger diameter. The upgraded pipeline generally followed the same alignment as the existing pipeline.

The proposed existing improvements are sized for buildout conditions. As the City continues to grow, it is recommended that the proposed pipeline diameters be constructed so that the facilities have sufficient capacity for buildout conditions. The proposed pipe diameter represents the ultimate diameter for anticipated buildout conditions.

6.2.3 Existing versus Future Improvements

An existing deficiency is one where the existing facility's capacity is insufficient to meet the planning criteria (e.g., pipeline upgrades required to prevent severe surcharging during the design wet weather event) for existing users. If a project was proposed to exclusively correct an existing deficiency, then existing users would be assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs, however this Master Plan did not identify any projects needed to serve exclusively future growth.

Other recommended improvements could serve future users. In these cases, an existing sewer or pump station may have sufficient capacity to convey current PWWFs, but as growth continues and more users are added to the system, the increased flow results in capacity deficiencies. These projects are classified as future improvements. Future users would be assigned 100 percent of the future project's benefit and 100 percent of the costs.

In some cases, a project is needed to correct an existing capacity deficiency, but is sized to accommodate additional flows from future development. In these cases, the hydraulic modeling results were used to determine the cost breakdown between existing and future users based on the ratio of existing and build out average dry weather flows. More information on the breakdown in cost split between existing and future users and whether a proposed improvement is intended to correct an existing deficiency, to serve a future user, or both, is provided in Chapter 7.

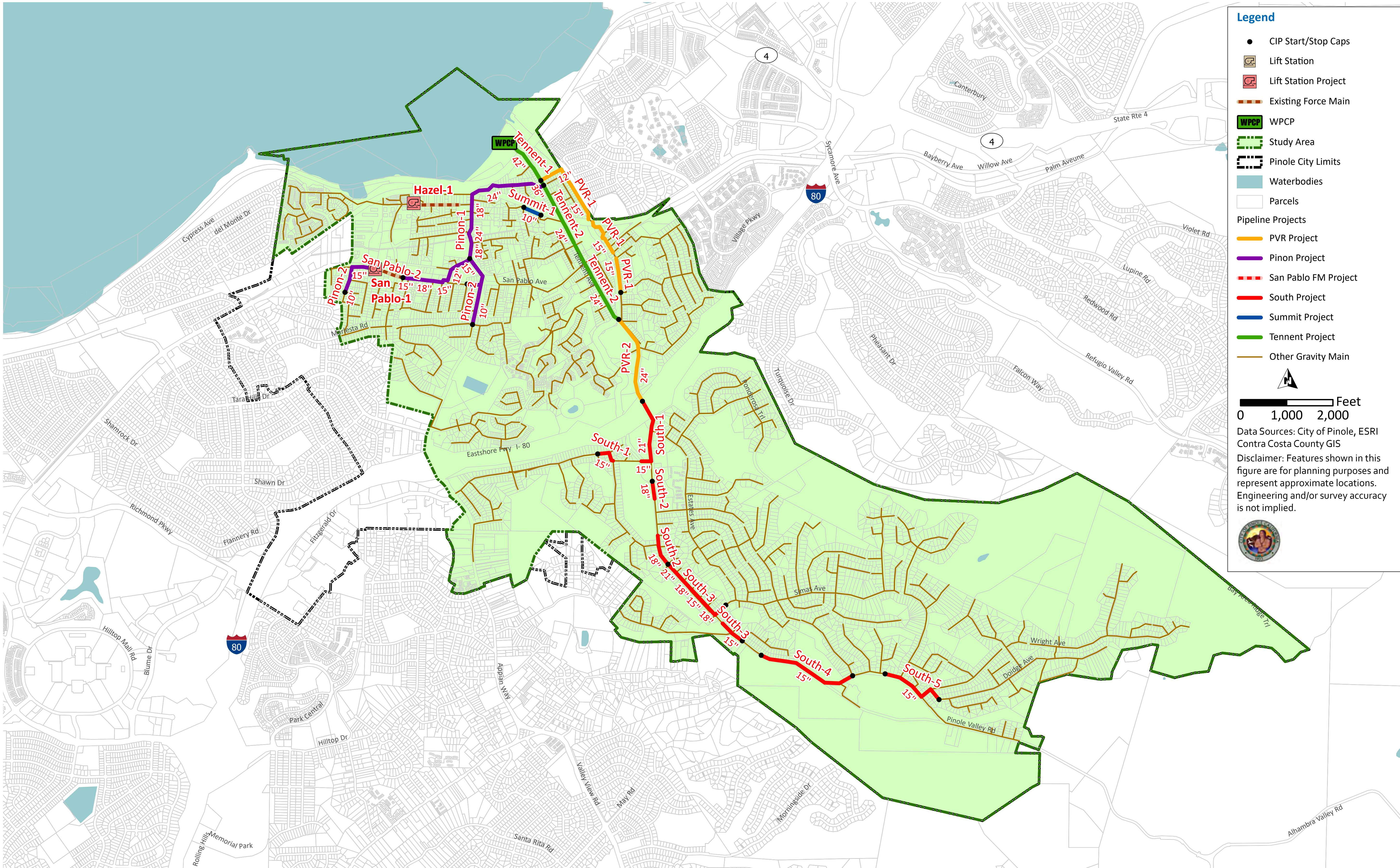


Figure 6.2 CIP Projects

6.2.4 Capacity Improvements

Following the completion of the existing and future system analysis, improvement projects were identified to mitigate pipeline capacity deficiencies while maintaining the maximum flow depth criteria outlined in Chapter 3. The proposed improvements to address existing and future deficiencies are shown on Figure 6.2 and are summarized below.

- **Pinon-1:** This project includes the replacement of approximately 1,050 feet of 8-inch diameter pipeline along San Pablo Avenue, approximately 740 feet of 8-inch diameter pipeline along Roble Avenue, approximately 1,500 feet of 8-inch to 10-inch diameter pipeline along Pinon Avenue, approximately 520 feet of 12-inch diameter pipeline between Pinon Avenue and Orleans Avenue and approximately 1,160 feet of 8-inch to 15-inch diameter pipeline along Orleans Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 12-inch to 24-inch diameter pipeline.
- **Pinon-2:** This project includes the replacement of approximately 820 feet of 6-inch to 10-inch diameter pipeline along San Pablo Avenue, approximately 680 feet of 8-inch to 10-inch diameter pipeline along Pinon Avenue, approximately 890 feet of 6-inch to 8-inch diameter pipeline along Appian Way, approximately 290 feet of 6-inch diameter pipeline along Meadow Avenue, and approximately 290 feet of 6-inch diameter pipeline between Meadow Avenue and San Pablo Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 10-inch to 15-inch diameter pipeline.
- **Tennent-1:** This project includes the replacement of approximately 130 feet of 24-inch diameter pipeline, 1,250 feet of 30-inch diameter pipeline, and approximately 10 feet of 36-inch diameter pipeline along Tennent Avenue and inside of the WPCP. The surcharging of the gravity sewer cause SSO's upstream under PWWF conditions. To mitigate the risk of SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 36-inch to 42-inch diameter pipeline.
- **Tennent-2:** This project includes the replacement of approximately 3,360 feet of 18-inch diameter pipeline along Tennent Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate the risk of SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 24-inch to 36-inch diameter pipeline.
- **PVR-1:** This project includes the replacement of approximately 1,130 feet of 6-inch to 10-inch diameter pipeline along Pinole Valley Road, approximately 1,830 feet of 8-inch diameter pipeline along Pinole Valley Creek, and approximately 530 feet of 12-inch diameter pipeline along Orleans Drive. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 15-inch diameter pipeline.
- **PVR-2:** This project includes the replacement of approximately 1,030 feet of 15-inch diameter pipeline and approximately 970 feet of 18-inch diameter pipeline along Pinole Valley Road. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 24-inch diameter pipeline.

- **South-1:** This project includes the replacement of approximately 1,400 feet of 15-inch diameter pipeline along Pinole Valley Road, approximately 250 feet of 8-inch diameter pipeline along Sarah Drive, approximately 210 feet of 8-inch diameter pipeline along Shea Drive, and approximately 220 feet of 10-inch diameter pipeline between Shea Drive and Pinole Valley Road. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 15-inch to 21-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.
- **South-2:** This project includes the replacement of approximately 1,090 feet of 15-inch diameter pipeline along Pinole Valley Road. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 18-inch to 21-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.
- **South-3:** This project includes the replacement of approximately 320 feet of 8-inch diameter pipeline along Simas Avenue and approximately 1,820 feet of 12-inch to 15-inch diameter pipeline along Pinole Valley Road. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 15-inch to 21-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.
- **South-4:** This project includes the replacement of approximately 2,500 feet of 10-inch to 12-inch diameter pipeline along Pinole Valley Road. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 15-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.
- **South-5:** This project includes the replacement of approximately 980 feet of 8-inch to 10-inch diameter pipeline along Pinole Valley Road, approximately 290 feet of 8-inch diameter pipeline along Doidge Avenue and approximately 260 feet of 8-inch pipeline along Wright Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 10-inch to 15-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.
- **Summit-1:** This project includes the replacement of approximately 410 feet of 6-inch diameter pipeline along Summit Drive. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 10-inch diameter pipeline.

- **Hazel-1:** This project includes the replacement of the existing lift station. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. To mitigate the risk of a SSO occurring during PWWF conditions, it is recommended that the new lift station have a firm pumping capacity of 0.831 mgd.
- **San Pablo-1:** This project includes the replacement of the existing lift station. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. To mitigate the risk of a SSO occurring during PWWF conditions, it is recommended that the new lift station have a firm pumping capacity of 1.38 mgd.
- **San Pablo-2:** The purpose of this project is to mitigate the high velocity (> 8 fps) that the existing forcemain experiences following San Pablo Lift Station under future conditions. It is recommended that an 8-inch diameter forcemain be constructed to replace the existing 6-inch diameter forcemain.

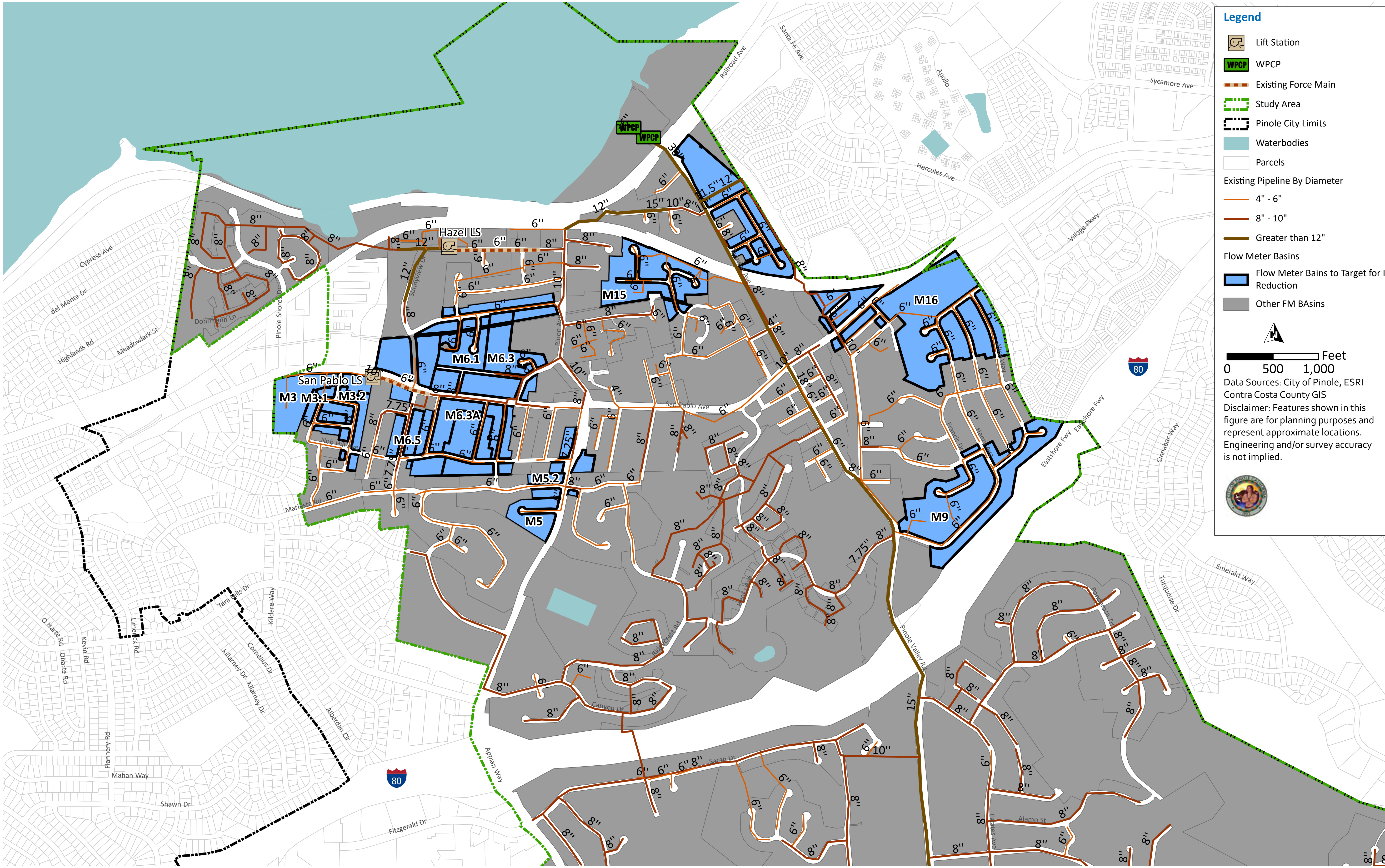


Figure 6.3 Basins for Inflow Identification

Chapter 7

CAPITAL IMPROVEMENT PROGRAM

This chapter presents the City's wastewater collection system CIP and a summary of the capital costs. This chapter is organized to assist the City in making financial decisions. The CIP is based on the evaluation of the City's wastewater collection system, as described in Chapter 6.

7.1 Capital Improvement Project Costs

The capacity upgrades and other system capital improvements set the foundation of the City's wastewater collection system CIP. The cost estimates presented in this study are opinions developed from bid tabulations, cost curves, information obtained from previous studies, and Carollo's experience on other projects. The costs are based on an Engineering News Record Construction Cost Index (ENR CCI) of 14,452 (San Francisco, October 2021).

7.2 Cost Estimating Accuracy

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

The Association for the Advancement of Cost Engineering (AACE) defines an Order of Magnitude Estimate, deemed appropriate for master plan studies as an approximate estimate made without detailed engineering data. It is normally expected that an estimate of this type would be accurate within plus 50 percent to minus 30 percent. The following sections present the assumptions used in developing order of magnitude cost estimates for recommended facilities.

7.3 Construction Unit Costs

The construction costs are representative of wastewater collection system facilities under normal construction conditions and schedules. Costs have been estimated for public works construction.

7.3.1 Gravity Sewer Replacement (Open Cut) Unit Costs

Sewer pipeline improvements range in size from 8 inches to 42 inches in diameter in this study. Unit costs for the construction of pipelines and appurtenances (e.g., manholes) are shown in Table 7.1. The construction cost estimates are based upon these unit costs. The gravity pipeline unit costs are based on an open cut construction method and assume "typical" field conditions with construction in stable soil at a depth ranging between 10 feet to 15 feet. The unit costs were developed based on Carollo's cost database and experience on other projects.

Table 7.1 Gravity Pipeline Unit Costs

Pipe Size (inches)	Replacement Unit Construction Cost(1) ($/linear foot)
8	$220
10	$275
12	$320
15	$405
18	$480
21	$535
24	$620
27	$710
30	$810
33	$870
36	$930
42	$1,045

Notes:
(1) ENR CCI (SF) for October 2021 is 14,452.

7.3.2 Lift Station Unit Costs

Costs associated with the lift station capacity improvements and forcemain, as well as other miscellaneous projects, were compiled based on Carollo's cost database and past experience on similar projects, and are shown in Table 7.2.

Table 7.2 Forcemain Unit Costs

Pipe Size (inches)	Replacement Unit Construction Cost(1) ($/linear foot)
6	$185
8	$185
12	$205

Notes:
(1) ENR CCI (SF) for October 2021 is 14,452.

7.4 Project Costs and Contingency

Project cost estimates are calculated based on elements, such as the project location, size, length, and other factors. Allowances for project contingencies consistent with an "Order of Magnitude" estimate are also included in the project costs prepared as part of this study, as outlined in this section.

7.4.1 Baseline Construction Costs

Baseline Construction Cost is the total estimated construction cost, in dollars, of the proposed improvements for pipelines. Baseline Construction Costs were calculated by multiplying the estimated length by the unit construction cost listed in Table 7.1.

7.4.2 Estimated Construction Cost

Contingency costs must be reviewed on a case-by-case basis because they will vary considerably with each project. Consequently, it is appropriate to allow for uncertainties associated with the preliminary layout of a project. Factors such as unexpected construction conditions, the need for unforeseen mechanical items, and variations in final quantities are a few of the items that can increase project costs for which it is wise to make allowances in preliminary estimates. To assist the City in making financial decisions for these future construction projects, contingency costs will be added to the planning budget as percentages of the baseline construction cost.

Since knowledge about site-specific conditions of each proposed project is limited at the master planning stage, a 30 percent contingency was applied to the Baseline Construction Cost to account for unforeseen events and unknown conditions.

7.4.3 Capital Improvement Cost

Other project construction contingency costs include costs associated with construction management, bid climate, environmental and legal costs, and engineering services. Construction management services covers items such as materials testing, and inspection during construction. Bid climate contingency covers any major fluctuations in the availability and cost of construction materials. Environmental and legal costs cover items such as legal fees, environmental compliance requirements, financing expenses, administrative costs, and interest during construction. Finally, there are engineering services costs associated with new facilities include preliminary investigation and reports, Right of Way (ROW) acquisition, foundation explorations, preparation of drawings and specifications during construction, surveying and staking, sampling of testing material, and start-up services.

The cost of these items can vary, but for the purpose of this study, it is assumed that the other project contingency costs will equal approximately 27.5 percent of the Estimated Construction Cost. Additionally, a bid market contingency of 15 percent is also applied to the estimated construction cost to account for the rapidly changing bid climate.

As shown in the following sample calculation of the Capital Improvement Cost, the total cost of all project construction contingencies (construction, construction management, bid climate, environmental and legal and engineering services) is approximately 185 percent of the Baseline Construction Cost. Note that contingencies were not applied to land acquisition costs. Calculation of the 185 percent is the overall mark-up on the Baseline Construction Cost to arrive at the Capital Improvement Cost. It is not an additional contingency.

Example:

Baseline Construction Cost	**$1,000,000**
Contingency (30 percent)	$300,000
Estimated Construction Cost	**$1,300,000**
Construction Management (10 percent)	$130,000
Bid Climate Contingency (15 percent)	$195,000
Environmental and Legal Costs (7.5 percent)	$97,500
Engineering Services (10 percent)	$130,000
Capital Improvement Cost	**$1,852,500**

7.5 Capital Improvement Program Implementation

As discussed in Chapter 6, the capital projects identified will allow the City to provide reliable service to its customers through buildout. The improvement projects were prioritized based on the following factors:

- Reducing the risk of SSOs in the collection system under PWWF conditions.
- The type and extent of the deficiency.

Based on these factors, each project was assigned an implementation year. The capital improvements were grouped into one of the following phases:

- **Near Term Phase 1 (5 to 7 years):** This phase includes projects that are targeted as the highest priority improvements.
- **Near Term Phase 2 (8 to 12 years):** This phase includes projects that are targeted as high priority improvements that may be mitigated or monitored for several years prior to being implemented.
- **Long Term (13 to 20 years):** This phase generally includes medium priority improvements or projects that eliminate SSO's.

Critical projects were phased in the earlier phases (years) of the 20-year CIP. Less critical projects were phased into later phases of the 20-year CIP.

A summary of the capital projects is presented in Table 7.2. This table identifies the projects, provides a brief description of each project, identifies facility sizes (e.g., pipe diameter and length), and provides capital improvement costs. The columns used in Table 7.2 refer to the following:

- **Project Number:** Assigned project number. This is an alphanumeric number that starts with a project name that corresponds to the associated trunk and continues with a number.
- **Description:** Provides a brief description of the project.
- **Existing Size/Type:** This is the size of the existing pipeline/facility.
- **Proposed Size/Type:** This is the size of the proposed improvement.
- **Proposed Amount:** Estimated length of the proposed improvement (in feet). It should be noted that the length estimates do not account for re-routing the alignments to avoid unknown conditions.
- **CIP Cost Estimate:** This is the total estimated capital cost.
- **Existing and Future User Cost Breakdown:** This shows the cost allocation between existing and future users.
- **CIP Phasing:** This is an estimated improvement project start year.

The projects listed in Table 7.3 are broken down by capacity-related improvements and other projects. Capacity-related improvements were recommended based on the capacity deficiencies described in Chapter 6. Other projects include an annual rehabilitation and replacement program, Master Plan updates (as needed), flow monitoring program (every 5 years), and an inflow identification program. Detailed capital improvement sheets for each project are included in Appendix D.

Project phasing shown in Table 7.3 represents the implementation schedule for the proposed improvements, although funding availability may limit the City's ability to implement the proposed projects according to this schedule.

A summary of the capital projects is presented in Table 7.3. This table identifies the projects, provides a brief description of each project, identifies facility sizes (e.g., pipe diameter and length), and provides capital improvement costs.

Table 7.3 Capital Improvement Program Summary

City of Pinole
Sewer Master Plan Update
WASTEWATER COLLECTION SYSTEM CAPITAL IMPROVEMENT PLAN SUMMARY

Project Number	Project Description	Existing/Future Deficiency	Deficiency Severity	Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate(1)(2)(3) ($)	Existing User Cost ($)	Future User Cost ($)
Capacity Related Improvements							$ 32,050,000	$ 30,864,000	$ 1,186,000
Gravity Mains				Diameter (in)	Diameter (in)	Length (ft)	$ 26,556,000	$ 25,504,000	$ 1,052,000
Pinon-1	Gravity Main along Pinon Ave, Orleans Ave, Roble Ave, and San Pablo Ave	Existing	Flooding	8-15	12-24	4,970	$ 4,482,000	$ 4,339,000	$ 143,000
Pinon-2	Gravity Main along San Pablo Ave, Pinon Ave, Appian Way, and Meadow Ave	Existing	Flooding	6-10	10-15	2,970	$ 1,866,000	$ 1,858,000	$ 8,000
Tennent-1	Gravity Main along Tennent Ave and at the WWTP	Existing	Flooding	24-36	36-42	1,390	$ 2,664,000	$ 2,582,000	$ 82,000
Tennent-2	Gravity Main along Tennent Ave	Existing	Flooding	18	24-36	3,360	$ 4,239,000	$ 3,895,000	$ 344,000
PVR-1	Gravity Main along Pinole Valley Road, Orleans Drive, and Pinole Creek	Existing	Flooding	6-12	12-15	4,020	$ 3,018,000	$ 2,990,000	$ 28,000
PVR-2	Gravity Main along Pinole Valley Road	Existing	Flooding	15-18	24	2,000	$ 2,298,000	$ 2,122,000	$ 176,000
South-1	Gravity Main along Pinole Valley Road, Sarah Drive, Shea Drive, and between Shea Drive and Pinole Valley Road	Existing	Flooding	8-15	15-24	2,080	$ 1,975,000	$ 1,836,000	$ 139,000
South-2	Gravity Main along Pinole Valley Road	Existing	Within 3-feet of manhole rim	15	18-21	1,090	$ 971,000	$ 949,000	$ 22,000
South-3	Gravity Main along Pinole Valley Road and Simas Avenue	Existing	Within 3-feet of manhole rim	8-15	15-21	2,140	$ 1,807,000	$ 1,766,000	$ 41,000
South-4	Gravity Main along Pinole Valley Road	Existing	Within 3-feet of manhole rim	10-12	15	2,500	$ 1,877,000	$ 1,835,000	$ 42,000
South-5	Gravity Main along Pinole Valley Road, Doidge Avenue and Wright Avenue	Existing	Flooding	8-10	10-15	1,530	$ 1,149,000	$ 1,128,000	$ 21,000
Summit-1	Gravity Main along Summit Drive	Existing	Within 3-feet of manhole rim	6	10	410	$ 210,000	$ 204,000	$ 6,000
Lift Stations				Capacity (mgd)			$ 5,232,000	$ 5,108,000	$ 124,000
Hazel-1	Hazel Lift Station Replacement	Existing	Exceeds firm Capacity	0.432	0.831	N/A	$ 2,153,000	$ 2,144,000	$ 9,000
San Pablo-1	San Pablo Lift Station Replacement	Existing	Exceeds firm Capacity	0.5832	1.38	N/A	$ 3,079,000	$ 2,964,000	$ 115,000
Force Mains				Diameter (in)	Diameter (in)	Length (ft)	$ 262,000	$ 252,000	$ 10,000
San Pablo-2	San Pablo Lift Station Forcemain	Existing	Exceeds maximum	6	8	1,530	$ 262,000	$ 252,000	$ 10,000
Rehabilitation and Replacement Projects							$ 29,000,000	$ 29,000,000	$ -
Gravity Mains				Diameter (in)	Diameter (in)	Length (ft)	$ 29,000,000	$ 29,000,000	$ -
RR-2	Pipe Rehabilitation and Replacement Program	--	--	Varies	Varies	Varies	$ 19,000,000	$ 19,000,000	$ -
RR-3	Inflow Identification Program	--	--	--	--	--	$ 10,000,000	$ 10,000,000	$ -
Other Projects							$ 600,000	$ -	$ 600,000
O-1	Sewer Master Plan Update	--	--	--	--	--	$ 600,000	$ -	$ 600,000
O-2	Flow Monitoring Program	--	--	--	--	--	$ -	$ -	$ -
CIP Total							**$ 61,650,000**	**$ 59,864,000**	**$ 1,786,000**
Annual Cost							**N/A**	**N/A**	**N/A**

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452 (SF).
(2) Estimated Construction Cost includes a 30% contingency of the baseline construction cost.
(3) Total project costs include a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

7.6 Existing Versus Future User Cost Share

A summary of the share of the costs for existing and future users for the proposed capacity-related improvement projects by phase is summarized in Table 7.4. As shown in Table 7.3, the existing user's share of the costs is approximately 96 percent ($30.9 million), and the future user's share of the costs is approximately 4 percent (or $1.2 million) of the proposed improvements. It is anticipated that existing user costs will be paid through existing user fees, while future user costs will be paid through connection fees. The future users cost share is small compared to the existing users share. This is because all of the capacity related improvements are triggered under existing PWWF conditions. If a project was proposed to exclusively correct an existing deficiency, then existing users would be assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs

Table 7.4 CIP Estimate by Reimbursement Category

Reimbursement Category	CIP Cost Estimate ($, Millions)(1)
Existing Users	$59.7
Future Users	$1.8
Total	**$61.7**

Notes:
(1) CIP costs based on assumptions outlined in this Chapter and Table 7.2.

7.7 CIP Summary

A summary of the CIP costs is provided in Table 7.5 and shown graphically on Figure 7.1. As listed in Table 7.5, the total recommended improvements is estimated to be $61.7 million. Near Term Phase 1 (–5 – 7 Years) projects account for approximately 23 percent ($14.35.0 million), Near Term Phase 2 (–8 – 12 Years) projects accounts for approximately 40 percent ($25.36million), and Long-Term (13 – 20 Years) projects account for approximately 37 percent ($23.44 million). The other recommended projects (RR-1 through RR-3 and O-1) is estimated to be $0.6 million.

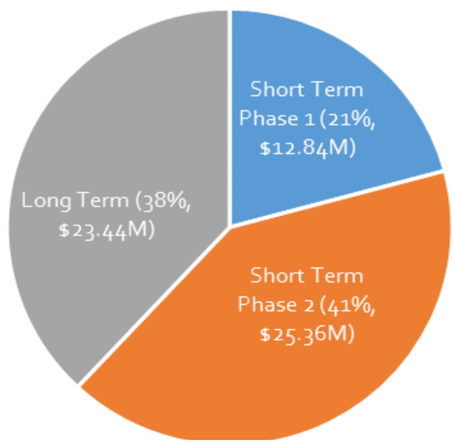


Figure 7.1 CIP by Phase

Table 7.5 CIP Cost Estimate Summary

Improvement Type	CIP Cost Estimate by Phase ($, Millions)(1)(2)(3)(4)			Total ($, Millions)
	Near Term Phase 1 (–5 to 7 Years)	Near Term Phase 2 –8 – 12 Years)	Long Term (–13 to 20 Years)	
Capacity-Related Improvements	$6.35	$17.71	-	**$32.1**
R&R Projects	$6.5	$7.5	$15.0	**$29.0**
Other Projects	-	$0.2	$0.5	**$0.6**
Total	**$12.85**	**$25.34**	**$23.44**	**$61.7**

Notes:
(1) ENR CCI (SF) for October 2021 is 14,452.
(2) Estimated Construction Cost includes a 30 percent contingency of the Baseline Construction Cost.
(3) Total project costs include a 15 percent for bid climate, 10 percent for engineering, 10 percent for construction management, and 7.5 percent for environmental and legal costs. Total Mark-Up is 185 percent of the Baseline Construction Costs.

Appendix A

SEWER INFLOW MONITORING AND INFLOW/INFILTRATION STUDY

CITY OF PINOLE
SEWER FLOW MONITORING & INFLOW/INFILTRATION STUDY

Prepared for: **City of Pinole**
2131 Pear Street
Pinole, CA 94564

Date: August 2015

Prepared by: V&A

V&A Project No. 13-0276

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FIGURES

APPENDICES

Abbreviations, Acronyms, Units of Measure, and Terms and Definitions used in this Report

Table i. Abbreviations

Abbreviation	Term
ADWF	average dry weather flow
C. of	City of…
CO	carbon monoxide
COOP	Cooperative Observer Program
d/D	depth/diameter ratio
gpd	gallons per day
FM	flow monitor
GWI	groundwater infiltration
H_2S	hydrogen sulfide
IDM	inch diameter-miles
I/I	inflow and infiltration
LEL	lower explosive limit
LS	lift station
mgd	million gallons per day
NOAA	National Oceanic and Atmospheric Administration
NWS	National Weather Service
OSHA	U.S. Department of Labor, Occupational Safety and Health Administration
PPE	personal protective equipment
Q	flow rate
RDI	rainfall-dependent infiltration
QA/QC	quality assurance/quality control
RRI	rainfall-responsive infiltration
RG	rain gauge
SSO	sanitary sewer overflow
WEF	Water Environment Federation
WRCC	Western Regional Climate Center

Table ii. Terms and Definitions

Term	Definition
Average dry weather flow (ADWF)	Average flow rate or pattern from days without noticeable inflow or infiltration response. ADWF usage patterns for weekdays and weekends differ and must be computed separately. ADWF can be expressed as a numeric average or as a curve showing the variation in flow over a day. ADWF includes the influence of normal groundwater infiltration (not related to a rain event).
Basin	Sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. Also refers to the ground surface area near and enclosed by pipelines. A basin may refer to the entire collection system upstream from a flow meter or exclude separately monitored basins upstream.
Depth/diameter (*d/D*) ratio	Depth of water in a pipe as a fraction of the pipe's diameter. A measure of fullness of the pipe used in capacity analysis.
Infiltration and inflow	Infiltration and inflow (I/I) rates are calculated by subtracting the ADWF flow curve from the instantaneous flow measurements taken during and after a storm event. Flow in excess of the baseline consists of inflow, rainfall-responsive infiltration, and rainfall-dependent infiltration. **Total I/I** is the total sum in gallons of additional flow attributable to a storm event.
Infiltration, *groundwater*	Groundwater infiltration (**GWI**) is groundwater that enters the collection system through pipe defects. GWI depends on the depth of the groundwater table above the pipelines as well as the percentage of the system that is submerged. The variation of groundwater levels and subsequent groundwater infiltration rates is seasonal by nature. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.
Infiltration, *rainfall-dependent*	Rainfall-dependent infiltration (**RDI**) is similar to groundwater infiltration but occurs as a result of storm water. The storm water percolates into the soil, submerges more of the pipe system, and enters through pipe defects. RDI is the slowest component of storm-related infiltration and inflow, beginning gradually and often lasting 24 hours or longer. The response time depends on the soil permeability and saturation levels.
Infiltration, *rainfall-responsive*	Rainfall-responsive infiltration (**RRI**) is storm water that enters the collection system through pipe defects, but normally in sewers constructed close to the ground surface such as private laterals. RRI is independent of the groundwater table and reaches defective sewers by way of the pipe trench in which the sewer is constructed; particularly if the pipe is placed in impermeable soil and bedded and backfilled with a granular material. In this case, the pipe trench serves as a conduit similar to a French drain, conveying storm drainage to defective joints and other openings in the system.
Inflow	**Inflow** is defined as water discharged into the sewer system, including private sewer laterals, from **direct** connections such as downspouts, yard and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins. Inflow creates a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Overflows are often attributable to high inflow rates.
Normalization	To run an "apples-to-apples" comparison amongst different basins, calculated metrics must be **normalized**. Individual basins will have different runoff areas, pipe lengths and sanitary flows. There are three common methods of normalization. Depending on the information available, one or all methods can be applied to a given project:

Term	Definition
	• Pipe Length: The metric is divided by the length of pipe in the upstream basin expressed in units of inch-diameter-mile (IDM). • Basin Area: The metric is divided by the estimated drainage area of the basin in acres. • ADWF: The metric is divided by the average dry weather sanitary flow (ADWF).
Normalization, *inflow*	The peak I/I flow rate is used to quantify inflow. Although the instantaneous flow monitoring data will typically show an inflow peak, the inflow response is measured from the I/I flow rate (in excess of baseline flow). This removes the effect of sanitary flow variations and measures only the I/I response: • Pipe Length: The peak I/I flow rate is divided by the length of pipe (IDM) in the upstream basin. The result is expressed in gallons per day (gpd) per IDM (gpd/IDM). • Basin Area: The peak I/I flow rate is divided by the geographic area of the upstream basin. The result is expressed in gpd per acre. • ADWF: The peak I/I flow rate is divided by the average dry weather flow (ADWF). This is a ratio and is expressed without units.
Normalization, *GWI*	The estimated GWI rates are compared to acceptable GWI rates, as defined by the Water Environment Federation, and are used to identify basins with high GWI: • Pipe Length: The GWI flow rate is divided by the length of pipe (IDM) in the upstream basin. The result is expressed in gallons per day (gpd) per IDM (gpd/IDM). • Basin Area: The GWI flow rate is divided by the geographic area of the upstream basin. The result is expressed in gpd per acre. • ADWF: The GWI flow rate is divided by the average dry weather flow (ADWF). This is a ratio and is expressed without units.
Normalization, *RDI*	The estimated RDI rates at a period 24 hours or more after the conclusion of a storm event are used to identify basins with high RDI: • Pipe Length: The RDI flow rate is divided by the length of pipe (IDM) in the upstream basin. The result is expressed in gallons per day (gpd) per IDM (gpd/IDM). • Basin Area: The RDI flow rate is divided by the geographic area of the upstream basin. The result is expressed in gpd per acre. • ADWF: The RDI flow rate is divided by the average dry weather flow (ADWF). This is a ratio and is expressed without units.
Normalization, *total I/I*	The estimated totalized I/I in gallons attributable to a particular storm event is used to identify basins with high total I/I. Because this is a totalized value rather than a rate and can be attributable solely to an individual storm event, the volume of the storm event is also taken into consideration. This allows for a comparison not only between basins but also between storm events: • Pipe Length: Total gallons of I/I is divided by the length of pipe (IDM) in the upstream basin and the rainfall total (inches) of the storm event. The result is expressed in gallons per IDM per inch-rain.

Term	Definition
	• Basin Area (R-Value): Total gallons of I/I is divided by total gallons of rainfall water that fell within the acreage of the basin area. This is a ratio and is expressed as a percentage. R-Value is described as "the percentage of rainfall that enters the collection system." Systems with R-Values less than 5%[1] are often considered to be performing well. • ADWF: Total gallons of I/I is divided by the ADWF and the rainfall total of the storm event. The result is expressed in million gallons per MGD of ADWF per inch of rain.
Peaking factor	Ratio of peak measured flow to average dry weather flow. This ratio expresses the degree of fluctuation in flow rate over the monitoring period and is used in capacity analysis.
Surcharge	When the flow level is higher than the crown of the pipe, then the pipeline is said to be in a **surcharged** condition. The pipeline is surcharged when the *d/D* ratio is greater than 1.0.
Weekend/weekday ratio	The ratio of weekend ADWFs to weekday ADWFs. In residential areas, this ratio is typically slightly higher than 1.0. In business districts, depending on the type of service, this ratio can be significantly less than 1.0.

[1] Keefe, P.N. "Test Basins for I/I Reduction and SSO Elimination." 1998 WEF Wet Weather Specialty Conference, Cleveland.

ES EXECUTIVE SUMMARY

Scope and Purpose

V&A Consulting Engineers, Inc. (V&A) has completed sanitary sewer flow monitoring with inflow and infiltration (I/I) analysis within the City of Pinole (City). Flow monitoring was performed over a period of approximately 3.5 months from December 13, 2013 to April 1, 2014 at 16 open-channel flow monitoring sites. The purpose of this study was to identify smaller basins within Area Five and Area Six having the highest rates of I/I.

To the extent possible given the time constraints of a single wet weather season and the unpredictable nature regarding the duration and intensity of storm events, V&A attempted to analyze early season rain events, make a decision on areas with high I/I, and then relocate flow meters within the same wet weather season. Relocations between rainfall events intended to further narrow the search for areas of high I/I with an ultimate goal of identifying possible CIP projects in support of the City's budgeted pipe lining and replacement program.

During the course of Phase 1 of this study, V&A installed 14 flow meters and 2 volumetric meters focused within Area Five and Area Six, dividing these areas into 16 sub-basins. For Phase 2, V&A removed ten Phase 1 meters and redistributed them amongst Basins 3, 5 and 6. A final rain event allowed for a third phase of this study, in which V&A removed seven Phase 1 and 2 meters and reinstalled them within sub-basins 3.1, 5.2, 6.3 and 6.5.

The contents of this deliverable summarize the results of the three aforementioned flow monitoring and I/I phases of this study. Additionally, V&A was asked to analyze and comment on the following items, which are included in this report as additional Appendices:

- **Flow Split between M6.0A and M6:** As a part of Phase 3, the City asked that the flow split between Sites M6.0A and M6.3 be monitored and analyzed during average dry weather and peak wet weather flow conditions.
- **Allocation of Inflow within the Perimeter of the Treatment Plant:** Rain that falls within the perimeter of the treatment plant flows directly into the treatment facility as waste; henceforth, there is an associated cost of treatment. The City asked V&A to determine the volume of inflow that falls within the footprint of the treatment facility.
- **Treatment Plant Influent Meter:** The City has two methods available to measure incoming flows from the City of Pinole. The City requested that V&A analyze the accuracy and determine which method is preferred for flow data reliability. This analysis was important for purposes of billing between the Cities of Pinole and Hercules. V&A dedicated a flow meter to measure the flow into the treatment plant for comparison to the City meter; the results are presented within this deliverable as a Technical Memorandum.

Disclaimer

The following flow monitoring, capacity and I/I results and analyses do not replace a full dynamic hydraulic model. A dynamic model developed by a master planning engineering firm would determine capacity on a node-to-node basis and would be based on pipe slopes of the individual pipe segments within the local collection system. The following data and the interpretation of these data should be used at the discretion of the City Engineer.

Flow Monitoring and Capacity Results

Table ES-1 summarizes the peak recorded flows, levels, *d/D* ratios, and peaking factors per site during the flow monitoring period. Sites that surcharged and sites with peaking factors greater than 10.0 have been shaded in RED. Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the point site locations. Hydraulic conditions in other areas of the collection system will differ.

Table ES-1. Capacity Analysis Summary

Monitoring Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (inches)	Peak Level (inches)	d/D Ratio	Surcharge Level (feet)
Phase 1							
Site M1	0.71	3.50	4.9	15	8.5	0.57	-
Site M2	1.11	7.20	6.5	30	45.5	1.52	1.3
Site M3	0.07	0.71	10.7	n/a	n/a	n/a	-
Site M4	0.13	0.47	3.7	n/a	n/a	n/a	-
Site M5	0.02	0.65	33.9	7.25	10.7	1.53	0.3
Site M6	0.18	1.10	6.2	10	82.3	8.23	6.0
Site M7	0.36	2.02	5.6	15	11.8	0.78	-
Site M8	0.007	0.03	5.0	7.75	1.3	0.17	-
Site M9	0.004	0.30	67.5	6	2.7	0.45	-
Site M10	0.09	0.41	4.8	8	11.4	1.43	0.3
Site M11	0.02	0.53	30.6	10	51.6	5.16	3.5
Site M12[2]	0.003	n/a	n/a	8	51.6	6.45	3.6
Site M13	0.06	0.21	3.8	6	1.7	0.28	-
Site M14	0.04	0.32	7.7	8	9.3	1.16	0.1
Site M15	0.02	0.21	14.1	6	2.6	0.44	-
Site M16	0.09	0.62	7.2	11.5	37.2	3.23	2.1

[2] Site M12 failed during Storm Event 1; the manhole was surcharged for an extended time period, flooding the flow logging computer. Enough data was recovered to establish an average dry weather flow but not enough for a proper capacity and I/I analysis. This site and other sites that comprise the 'Old Henry Road' basin are the subject of a future project.

Monitoring Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (inches)	Peak Level (inches)	d/D Ratio	Surcharge Level (feet)
Phase 2							
Site M3.1	0.03	0.58	18.3	6	3.3	0.41	-
Site M3.2	0.01	0.08	7.3	6	3.2	0.40	-
Site M5.1	0.02	0.15	6.5	8	7.6	0.95	-
Site M5.2	0.01	0.31	27.6	8	6.1	0.77	-
Site M5.3	0.02	0.10	4.5	6	2.1	0.35	-
Site M6.1	0.01	0.12	10.4	6	7.2	1.20	0.1
Site M6.2	0.02	0.04	2.3	8	1.9	0.23	-
Site M6.3	0.16	0.72	4.4	8	5.1	0.64	-
Site M6.4	0.08	0.51	6.1	8	6.2	0.78	-
Site M6.5	0.01	0.12	13.5	8	3.0	0.37	-
Phase 3							
Site M3.1A	0.009	0.15	15.4	6	5.9	0.98	-
Site M3.1B	0.006	0.10	16.7	6	2.0	0.34	-
Site M5.2A	0.009	0.11	12.0	6	3.2	0.53	-
Site M6.0A	0.057	1.09	19.0	10	7.2	0.72	-
Site M6.3A	0.005	0.30	61.5	6	3.2	0.53	-
Site M6.3B	0.004	0.06	16.1	8	1.3	0.16	-
Site M6.5A	0.003	0.03	11.1	7.75	1.3	0.17	-
Site M6.5B	0.003	0.04	11.8	7.75	1.1	0.14	-

The following capacity analysis results are noted:

- **Peaking Factor:** Several sites had peaking factors greater than 10. Larger peaking factors are expected given that the study analyzes basins previously identified as having high I/I rates.
- **d/D Ratio:** Nine of the flow monitoring sites (Sites M2, M5, M6, M10, M11, M12, M14, M16 and M6.1) reached surcharge conditions.
- **Sanitary Sewer Overflow Potential:** Given the level of surcharging seen during Storm Event 1, the manholes at Site 6, Site 11/12 and Site 16 have the potential for a sanitary sewer overflow (SSO) during a larger rainfall event. Site 6 has historical precedence for SSO discharging during large rainfall events.

Inflow and Infiltration Analysis Results

Preface

Per discussions with the City, V&A prioritized I/I evaluations on the comparative analysis of the Peak I/I rate, traditionally associated with inflow. Inflow sources transport rain water *directly* into the sewer system; the corresponding inflow rates are tied closely to the intensity of the storm. This component of RDI/I often causes a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Figure ES-1 illustrates the I/I response curve for Basin 3 during Phase 1 as it related to peak I/I rate.

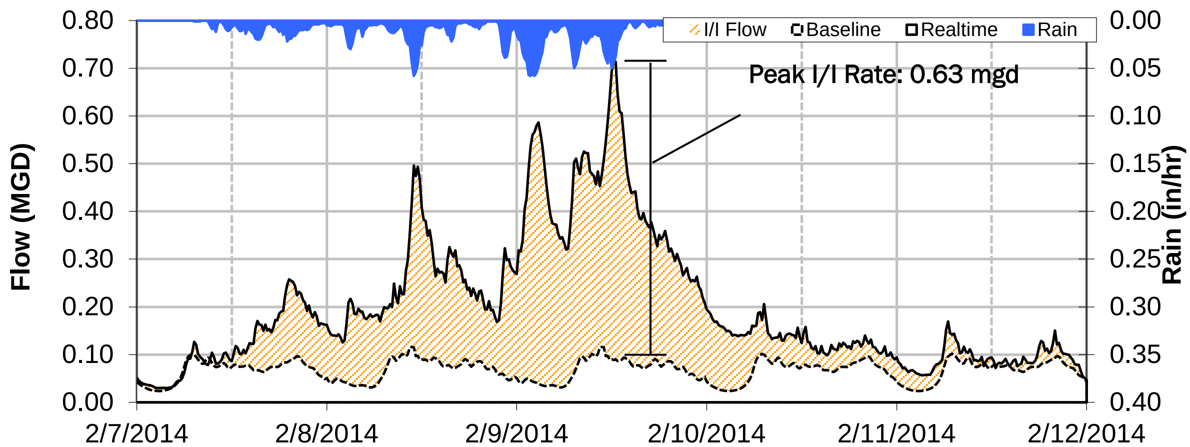


Figure ES-1. I/I Isolation Curve, Basin 3 (Inflow Measurement)

V&A analyzed the I/I isolation curves for all sites and all phases of this study to try to determine the areas of the City collection system that had the highest peak I/I rates.

Phase 1 (Large Basin) Results

The results of the Phase 1 flow monitoring and I/I analyses for the larger basins are shown as follows:

- **Basin 3:** The City cited historical data and field observations regarding known I/I issues within Basin 3.
- **Basin 5:** V&A recommended investigating Basin 5 due to the high peak I/I ratios and high combined I/I totals.
- **Basin 6:** There is a known capacity issue one manhole upstream from the Basin 6 monitoring location.
- **Basin 8:** This is a newer area of the City and this area had minimal I/I contribution.
- **Basin 9:** The flow meter was not in an ideal location for monitoring. The consensus was that the response for Basin 9 was real and considerable but the magnitude may not be correct due to metering conditions.

- **Basin 11:** This location had significant peak I/I rates and combined I/I totals.
- **Basin 12:** The metering manhole for this basin surcharged with evidence that the surcharge was close to an SSO.
- **Basins 15/16:** Both basins were noted for generally high I/I rates and total I/I contribution.
- **Old Henry Road:** The City cited the age of the sewer system along Old Henry Road and noted this correlation to the high I/I rates within Basins 9, 11 and 12.

Final group conclusions were as follows:

- Focus additional phases of monitoring within Basins 3, 5 and 6.
  - The focus of the study moving forward would be to spend the remainder of the 2014 wet weather season identifying smaller high I/I mini-basins within this region for possible future CIP work.
  - Identifying areas of I/I reduction may help to solve two problems within the City: (1) severe flows observed in Basin 3 (San Pablo Pump Station), and (2) capacity issues observed at the manhole at the intersection of Pinon Avenue and Bay View Farm Road.
- Make note of the 'Old Henry Road' Basins as a future candidate for a focused flow monitoring and I/I study.

Figure ES-2 illustrates the Phase 1 inflow rankings for the larger basins.

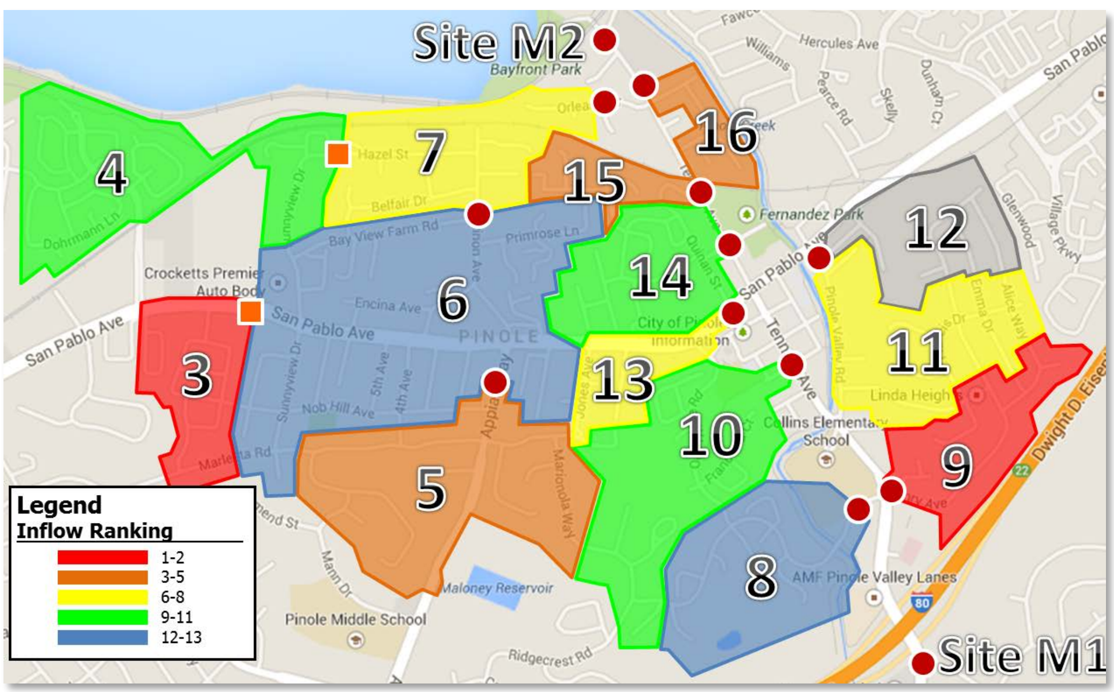


Figure ES-2. Phase 1 Inflow Temperature Map

Phases 2 and 3 Focused Sub-Basin Results

Additional phases of flow monitoring and I/I analysis were focused within Basins 3, 5 and 6. Through focused flow monitoring on a very small basis, V&A was able to find 'hot-spot' locations within Basins 3, 5 and 6 that were contributing a significant percentage of the peak. Figure ES-3 illustrates the hot-spot areas within Basins 3, 5 and 6.

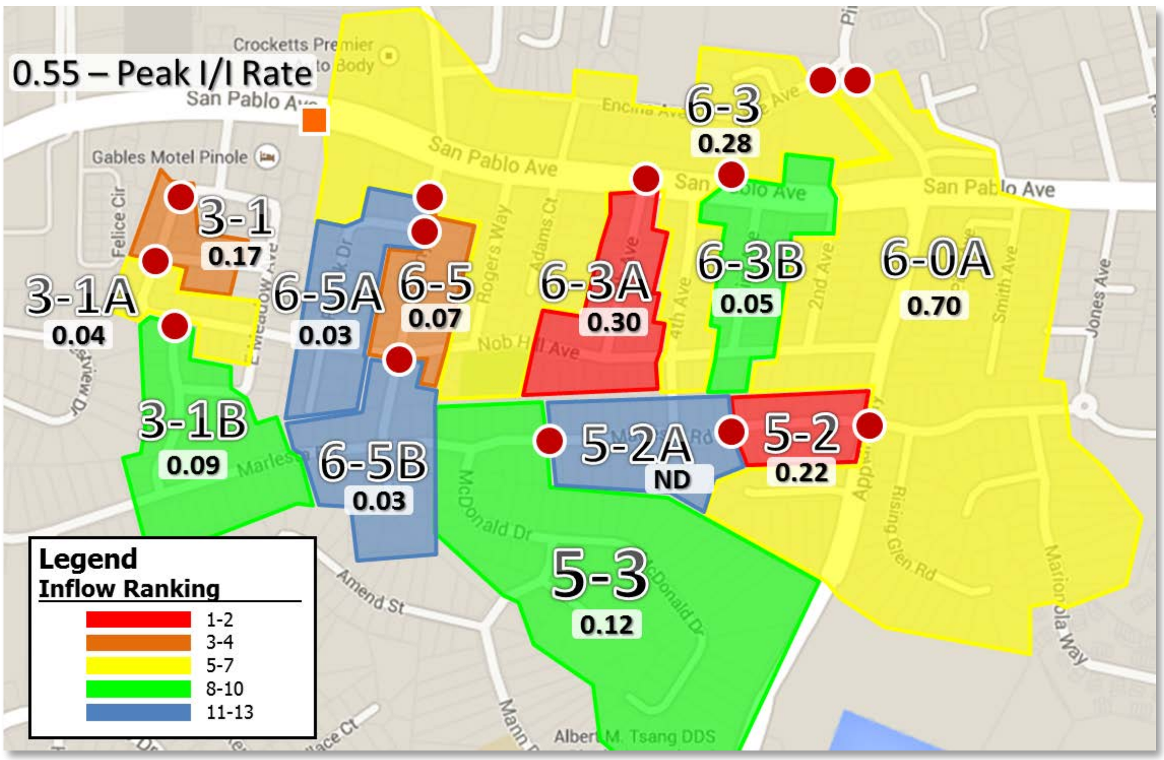


Figure ES-3. Phase 3 Inflow Analysis Temperature Map

Recommendations

V&A advises that future I/I reduction plans consider the following recommendations:

1. **Potential CIP Projects for I/I Mitigation and Reduction**
   a. The City should conduct I/I mitigation and reduction measures in the following mini-basins:
      i. Basin 3-1
      ii. Basin 3-1A
      iii. Basin 5-2
      iv. Basin 6-3A
      v. Basin 6-5
   b. For I/I reduction, V&A recommends rehabilitation of the sewer mains, laterals and side sewers.
      i. The most comprehensive study on the percent of I/I reduction has been conducted by King County, *Initial Infiltration and Inflow Reduction Project Alternatives Analysis Report*. This study confirmed the popular theory that over 50% of infiltration and inflow enters from private lateral connections. The report also makes the following recommendations for I/I mitigation:
         (a) CCTV work is best performed during a rainfall event after groundwater levels have begun to rise, allowing visual confirmation of specific I/I entry points, including determining the source of potential lateral I/I source. A generally consistent deficiency was observed with regards to the joint conditions in the laterals and side sewers.
         (b) Rehabilitation of sewer mains, manholes, laterals and side sewers results in approximately 80% reduction of I/I.
2. **Future I/I Identification – Continued Sub-Basin Flow Monitoring and I/I Analysis:** V&A recommends that the City continue to locate and mitigate potential sources of I/I. Already identified as known contributing sub-basins with high volumes of I/I are Basins 9, 11, 12, 14 and 15. It is possible that a study similar to this study may identify CIP projects that can significantly reduce the overall I/I within the City collection system.
3. **Other I/I Investigation Methods:** Other potential I/I investigation methods include the following:
   a. Smoke testing
   b. Night-time reconnaissance work to (1) investigate and determine direct point sources of inflow, and (2) determine the areas and pipe reaches responsible for high levels of infiltration contribution.
4. **I/I Reduction Cost Effectiveness Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow/infiltration and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional rainfall dependent I/I flow.

1.0 INTRODUCTION

1.1 Scope and Purpose

V&A Consulting Engineers, Inc. (V&A) was retained by the City of Pinole (City) to conduct sanitary sewer flow monitoring and inflow/infiltration (I/I) analysis within the City of Pinole (City). Flow monitoring occurred over a 3.5-month period from December 13, 2013 to April 1, 2014. Two basins within the collection system (Area Five “The Meadows” and Area Six “Old Town”) had already been identified by the City as having high rates of RDI/I. The purpose of this study was to identify smaller basins within Area Five and Area Six having the highest rates of I/I.

To the extent possible given the time constraints of a single wet weather season and the unpredictable nature regarding the duration and intensity of storm events, V&A would attempt to relocate flow meters within the wet weather season. Relocations between rainfall events could further narrow the search for areas of high I/I with an ultimate goal of identifying possible CIP projects in support of the City’s budgeted pipe lining and replacement program. The outline of the strategies for the I/I investigation is shown as follows:

- **Phase 1 – Initial Sub-Basins:** V&A installed 14 flow meters and 2 volumetric meters focused within Area Five and Area Six, dividing these areas into 16 sub-basins.
  - **Mid-Project, Post-Rainfall I/I Analysis:** After the first usable rainfall event, V&A analyzed the flow data for relative I/I contribution and comparison amongst the sub-basins.
  - **Quick Decisions:** V&A conveyed the I/I results to the City, making recommendations for targeted investigation of the highest ranking I/I sub-basins.
  - **Maneuverable Metering:** V&A removed the meters from the sub-basins with lower RDI/I rates and relocated them into the sub-basins with the higher RDI/I rates, dividing the original sub-basins into mini-basins.
- **Phase 2 – Focused Sub-Basins:** The same process from Phase 1 was repeated for Phase 2, focusing the search within the sub-basins with the greatest I/I contribution.
- **Phase 3 – Focused Mini-Basins:** The rain events of the 2013/2014 season were sufficient to conduct a third phase of flow monitoring and I/I analysis.

Additionally, V&A was asked to analyze and comment on the following items:

- **Flow Split between M6.0A and M6:** As a part of Phase 3, the City asked that the flow split between Sites M6.0A and M6.3 be monitored and analyzed during average dry weather and peak wet weather flow conditions.

- **Allocation of Inflow within the Perimeter of the Treatment Plant:** Rain that falls within the perimeter of the treatment plant flows directly into the treatment facility as waste; henceforth, there is an associated cost of treatment. The City asked V&A to determine the volume of inflow that falls within the footprint of the treatment facility.
- **Treatment Plant Influent Meter:** The City has two methods available to measure incoming flows from the City of Pinole. The City requested that V&A analyze the accuracy and determine which method is preferred for flow data reliability. This analysis was important for purposes of billing between the Cities of Pinole and Hercules. V&A dedicated a flow meter to measuring the flow into the treatment plant for comparison to the City meter; the results are presented within this deliverable as a Technical Memorandum.

These additional analyses are included in this report as *Appendix A, Appendix B and Appendix C*.

1.2 Flow Monitoring Sites

Flow monitoring sites are the locations where the flow monitors were placed. Flow monitoring site data may include the flows of one or many drainage basins. Capacity and flow rate information is presented on a site-by-site basis. The flow monitoring sites for the three phases are listed in Table 1-1, Table 1-2 and Table 1-3 and illustrated in Figure 1-1, Figure 1-2 and Figure 1-3. Detailed descriptions of the individual flow monitoring sites are included in *Appendix A*.

Table 1-1. Phase 1 Flow Monitoring Sites

Monitoring Site	Pipe Size (inches)	Location
Site M1	15	Pinole Valley Rd., just south of Highway 80
Site M2	30	Tennant Ave., just outside WWTP
Site M3	n/a	San Pablo Ave., west of Sunnyview Dr.
Site M4	n/a	In easement at west end of Hazel St.
Site M5	7.25	Appian Way, south of San Pablo Ave.
Site M6	10	Pinon Ave., north of Bay View Farm Rd.
Site M7	15	Intersection of Orleans Dr. and Zoe Ct.
Site M8	7.75	Henry Ave., west of Pinole Valley Rd.
Site M9	6	Intersection of Henry Ave. and Pinole Valley Rd.
Site M10	8	Intersection of Tennant Ave. and Prune St.
Site M11	10	Intersection of Pinole Valley Rd. and Rafaela St.
Site M12	8	Intersection of Pinole Valley Rd. and Rafaela St.
Site M13	6	San Pablo Ave. just west of Quinan St.
Site M14	8	Intersection of Tennant Ave. and Park St.
Site M15	6	Tennant Ave., south of train tracks, west of Fernandez Park
Site M16	11.5	Tennant Ave. north of Orleans Dr.

Table 1-2. Phase 2 Flow Monitoring Sites

Monitoring Site	Pipe Size (inches)	Location
Site M2	30	Tennant Ave., just outside WWTP
Site M3	n/a	San Pablo Ave., west of Sunnyview Dr.
Site M4	n/a	In easement at west end of Hazel St.
Site M6	10	Pinon Ave., north of Bay View Farm Rd.
Site M7	15	Intersection of Orleans Dr. and Zoe Ct.
Site M3.1	6	830 Meadows Ave.
Site M3.2	6	830 Meadows Ave.
Site M5.1	8	Intersection of Appian Way and Marlesta Rd.
Site M5.2	8	Intersection of Appian Way and Marlesta Rd.
Site M5.3	6	1171 Marlesta Rd.
Site M6.1	6	Just west of intersection of Bay View Farm Rd. and Pinon Ave.
Site M6.2	8	Intersection of Pinon Ave. and Primrose Ln.
Site M6.3	8	Roble Ave., west of Pinon Ave.
Site M6.4	8	Intersection of San Pablo Ave. and Rogers Way
Site M6.5	8	747 Sunnyview Dr.

Table 1-3. Phase 3 Flow Monitoring Sites

Monitoring Site	Pipe Size (inches)	Location
Site M2	30	Tennant Ave., just outside WWTP
Site M3	n/a	San Pablo Ave., west of Sunnyview Dr.
Site M4	n/a	In easement at west end of Hazel St.
Site M6	10	Pinon Ave., north of Bay View Farm Rd.
Site M3.1	6	830 Meadows Ave.
Site M5.2	8	Intersection of Appian Way and Marlesta Rd.
Site M5.3	6	1171 Marlesta Rd.
Site M6.3	8	Roble Ave., west of Pinon Ave.
Site M6.5	8	747 Sunnyview Dr.
Site M3.1A	6	Intersection of Meadow Ave. and Betty Ave.
Site M3.1B	6	Intersection of Meadow Ave. and Nob Hill Ave.
Site M5.2A	6	1367 Marlesta Rd.
Site M6.0A	10	Intersection of Roble Ave. and Pinon Ave.
Site M6.3A	6	Intersection of San Pablo Ave. and 5th Ave.
Site M6.3B	8	Intersection of San Pablo Ave. and Roble Ave.
Site M6.5A	7.75	Intersection of Sunnyview Dr. and Patrick Dr.
Site M6.5B	7.75	Intersection of Sunnyview Dr. and Nob Hill Ave.

Figure 1-1. Phase 1 Flow Monitoring Site Map

Figure 1-2. Phase 2 Flow Monitoring Site Map

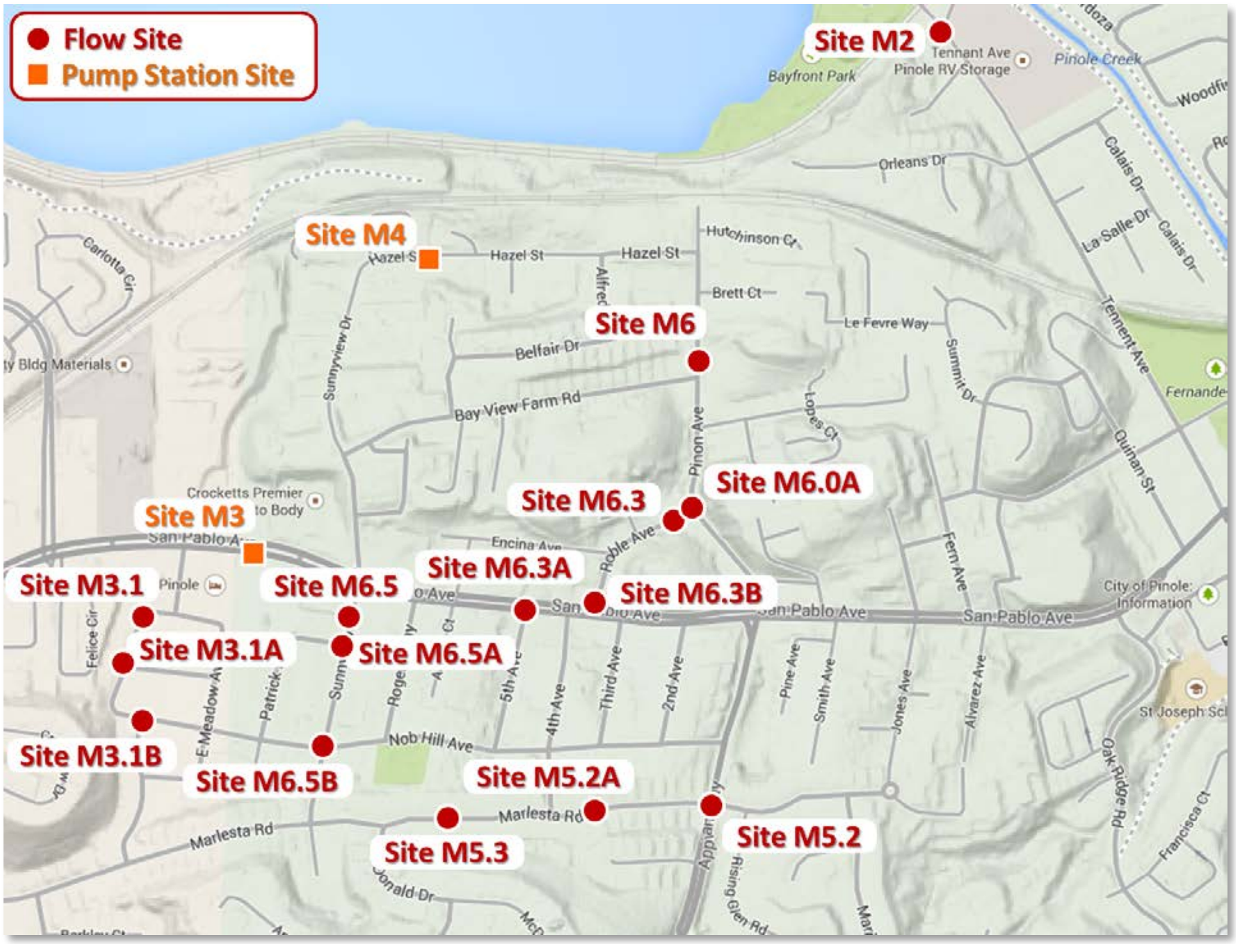


Figure 1-3. Phase 3 Flow Monitoring Site Map

1.3 Flow Monitoring Basins

Flow monitoring basins are localized areas of a sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. The basin refers to the ground surface area near and enclosed by the pipelines. A basin may refer to the entire collection system upstream from a flow meter or may exclude separately monitored basins upstream.

To isolate a flow monitoring basin, an addition or subtraction of flows may be required[3]. Site M1 was not used as a basin because it served to measure the flow coming into the area of interest. Site M2 was not isolated as a basin because it would have required subtracting the flow from 11 other sites.

I/I analysis in this report will be conducted on a basin-by-basin basis. Table 1-4 lists the basins and sub-basins that were isolated[4] and utilized for I/I analysis and Figure 1-4, Figure 1-5 and Figure 1-6 illustrate the basins utilized for I/I analysis.

Table 1-4. List of Flow Monitoring Basins

Sub-Basin	Area (acres)	Pipe Length (IDM)	Basin Flow Calculation
Phase 1			
Basin M3	40	5.09	$Q_{M3(Basin)} = Q_{M3(Site)}$
Basin M4	58	12.91	$Q_{M4(Basin)} = Q_{M4(Site)}$
Basin M5	71	10.32	$Q_{M5(Basin)} = Q_{M5(Site)}$
Basin M6	130	13.01	$Q_{M6(Basin)} = Q_{M6(Site)} - Q_{M5(Site)} - Q_{M3(Site)}$
Basin M7	41	11.88	$Q_{M7(Basin)} = Q_{M7(Site)} - Q_{M6(Site)} - Q_{M4(Site)}$
Basin M8	53	6.35	$Q_{M8(Basin)} = Q_{M8(Site)}$
Basin M9	28	4.99	$Q_{M9(Basin)} = Q_{M9(Site)}$
Basin M10	62	11.87	$Q_{M10(Basin)} = Q_{M10(Site)}$
Basin M11	52	10.48	$Q_{M11(Basin)} = Q_{M11(Site)}$
Basin M12	31	4.47	$Q_{M12(Basin)} = Q_{M12(Site)}$
Basin M13	17	3.20	$Q_{M13(Basin)} = Q_{M13(Site)}$
Basin M14	37	6.33	$Q_{M14(Basin)} = Q_{M14(Site)}$
Basin M15	16	3.65	$Q_{M15(Basin)} = Q_{M15(Site)}$
Basin M16	12	4.78	$Q_{M16(Basin)} = Q_{M16(Site)}$

[3] There is error inherent in flow monitoring. Adding and subtracting flows increases error on an additive basis. For example, if Site A has error ±10% and Site B has error ±10%, then the resulting flow when subtracting Site A from Site B would be ±20%.
[4] There may be locations with cross-connections between trunk sewers or overflow bypass sewers to help equalize basins and prevent sanitary sewer overflows during peak rain events. However, unless the inter-basin connections are plugged, the behavior of flows may not be known with certainty. The basin isolation equations shown are per the best of V&A's knowledge.

Sub-Basin	Area (acres)	Pipe Length (IDM)	Basin Flow Calculation
Phase 2			
Basin M3.1	16	2.07	$Q_{M3.1(Basin)} = Q_{M3.1(Site)}$
Basin M3.2	6	1.14	$Q_{M3.2(Basin)} = Q_{M3.2(Site)}$
Basin M5.1	23	3.34	$Q_{M5.1(Basin)} = Q_{M5.1(Site)} - Q_{M5.3(Site)}$
Basin M5.2	9	1.34	$Q_{M5.2(Basin)} = Q_{M5.2(Site)}$
Basin M5.3	29	3.25	$Q_{M5.3(Basin)} = Q_{M5.3(Site)}$
Basin M6.1	13	2.41	$Q_{M6.1(Basin)} = Q_{M6.1(Site)}$
Basin M6.2	13	3.32	$Q_{M6.2(Basin)} = Q_{M6.2(Site)}$
Basin M6.3	40	2.69	$Q_{M6.3(Basin)} = Q_{M6.3(Site)} - Q_{M6.4(Site)}$
Basin M6.4	13	1.07	$Q_{M6.4(Basin)} = Q_{M6.4(Site)} - Q_{M6.5(Site)}$
Basin M6.5	17	3.50	$Q_{M6.5(Basin)} = Q_{M6.5(Site)}$
Phase 3			
Basin M3.1	3.1	0.69	$Q_{M3.1(Basin)} = Q_{M3.1(Site)} - Q_{M3.1A(Site)}$
Basin M3.1A	2.4	0.67	$Q_{M3.1A(Basin)} = Q_{M3.1A(Site)} - Q_{M3.1B(Site)}$
Basin M3.1B	10.1	1.83	$Q_{M3.1B(Basin)} = Q_{M3.1B(Site)}$
Basin M5.2	3.6	0.75	$Q_{M5.2(Basin)} = Q_{M5.2(Site)} - Q_{M5.2A(Site)}$
Basin M5.2A	5.6	0.69	$Q_{M5.2A(Basin)} = Q_{M5.2A(Site)} - Q_{M5.3(Site)}$
Basin M5.3	28.9	3.42	$Q_{M5.3(Basin)} = Q_{M5.3(Site)}$
Basin M6.0A	55.1	10.14	$Q_{M6.0A(Basin)} = Q_{M6.0A(Site)} - Q_{M5.2(Site)}$
Basin M6.3	36.4	5.38	$Q_{M6.3(Basin)} = Q_{M6.3(Site)} - Q_{M6.3A(Site)} - Q_{M6.3B(Site)}$
Basin M6.3A	7.6	0.97	$Q_{M6.3A(Basin)} = Q_{M6.3A(Site)}$
Basin M6.3B	5.9	1.39	$Q_{M6.3B(Basin)} = Q_{M6.3B(Site)}$
Basin M6.5	3.4	0.85	$Q_{M6.5(Basin)} = Q_{M6.5(Site)} - Q_{M6.5A(Site)} - Q_{M6.5B(Site)}$
Basin M6.5A	6.4	1.36	$Q_{M6.5ABasin)} = Q_{M6.5A(Site)}$
Basin M6.5B	7.3	1.32	$Q_{M6.5B(Basin)} = Q_{M6.5B(Site)}$

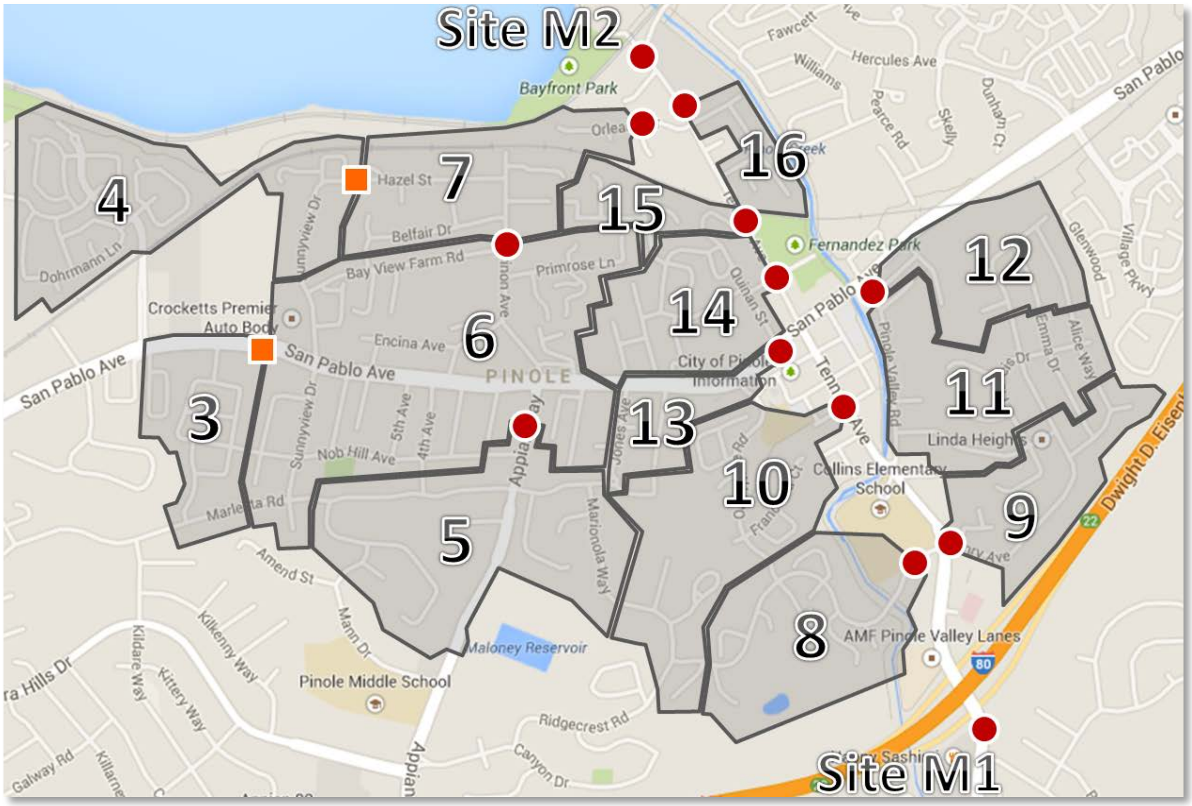


Figure 1-4. Phase 1 Flow Monitoring Basins Map

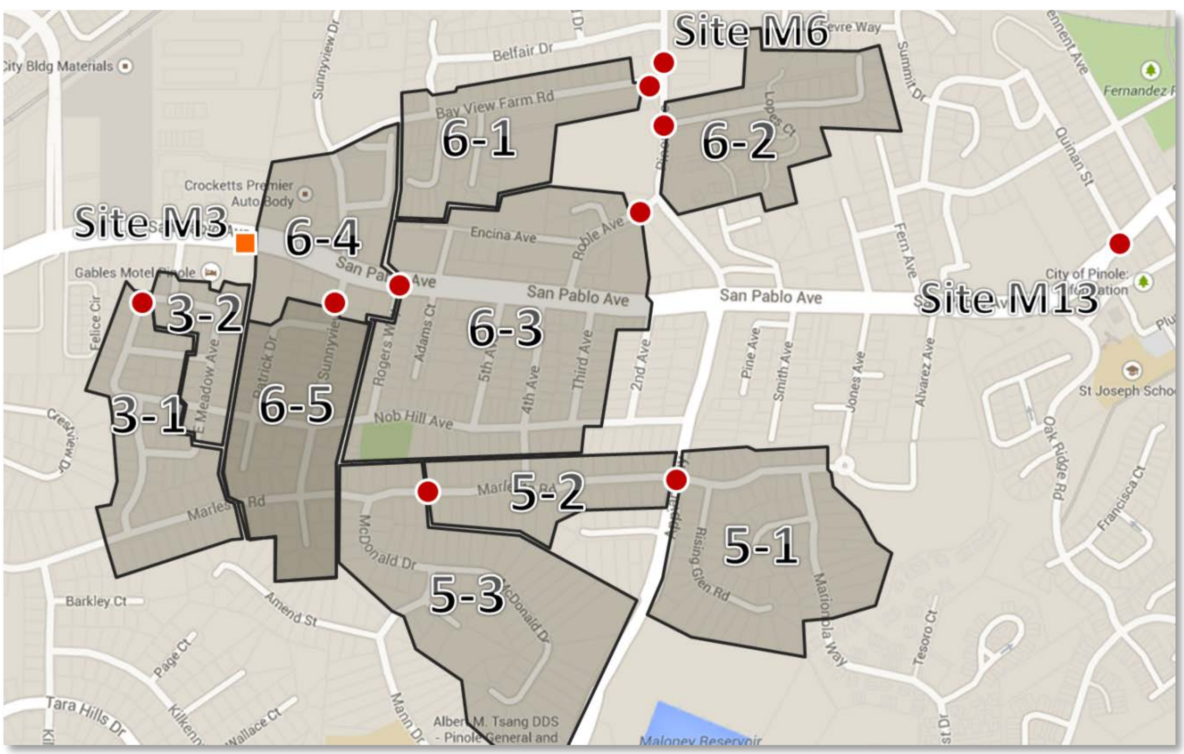


Figure 1-5. Phase 2 Flow Monitoring Basins Map

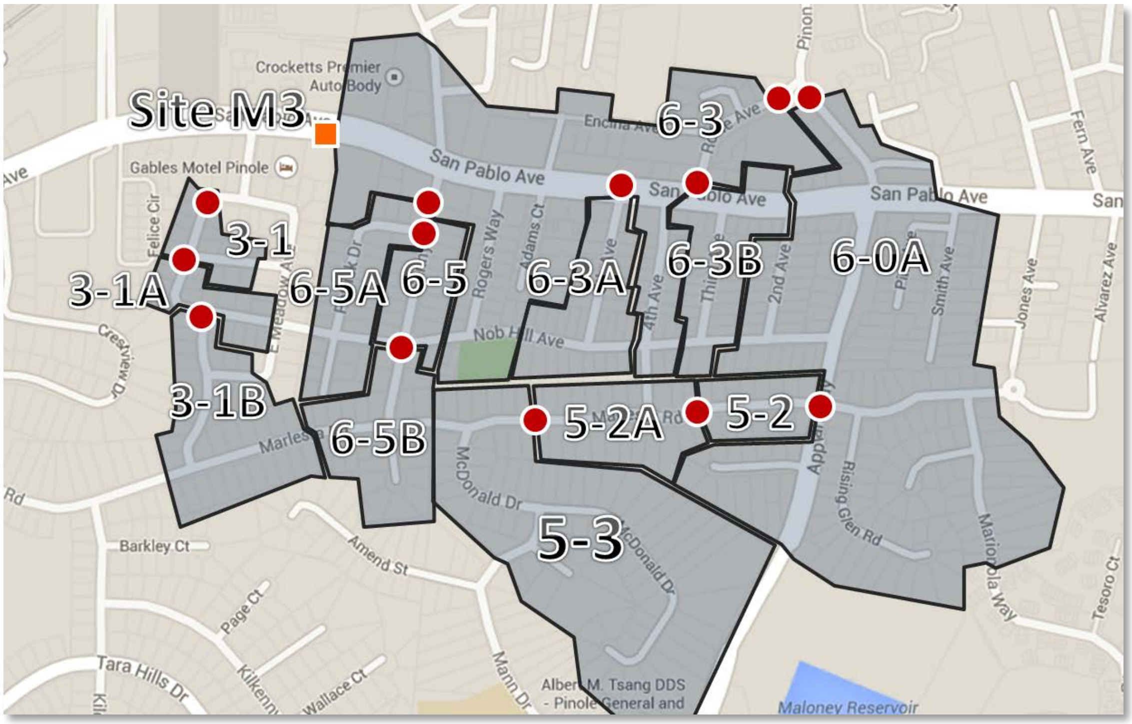


Figure 1-6. Phase 3 Flow Monitoring Basins Map

2.0 METHODS AND PROCEDURES

2.1 Confined Space Entry

A confined space entry (Photo 2-1) is defined as any space large enough and so configured that a person can bodily enter and perform assigned work, has limited or restricted means for entry or exit, and is not designed for continuous employee occupancy. In general, the atmosphere must be constantly monitored for sufficient levels of oxygen (19.5% to 23.5%), and the absence of hydrogen sulfide (H_2S) gas, carbon monoxide (CO) gas, and lower explosive limit (LEL) levels. A typical confined space entry crew has members with U.S. Department of Labor, Occupational Safety & Health Administration-defined (OSHA-defined) responsibilities of "entrant," "attendant," and "supervisor." The entrant is the individual performing the work. He or she is equipped with the appropriate level of personal protective equipment (PPE) needed to perform the job safely, including a personal four-gas monitor (Photo 2-2). If it is not possible to maintain line-of-sight with the entrant, then more entrants are required until line-of-sight can be maintained. The attendant is responsible for maintaining contact with the entrants to monitor the atmosphere using another four-gas monitor and maintaining records of all entrants. The supervisor is responsible for developing the safe work plan prior to entering.

Photo 2-1. Confined Space Entry

Photo 2-2. Typical Personal Four-Gas Monitor

2.2 Flow Meter Installation

A combination of Isco 2150 area-velocity meters and Hach Flo-Dar meters were installed by V&A in the sewer lines in Table 1-1, Table 1-2 and Table 1-3. Continuous depth and velocity readings were recorded by the flow meters on 5-minute intervals.

Isco 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. Figure 2-1 shows a typical installation for a flow meter with a submerged sensor.

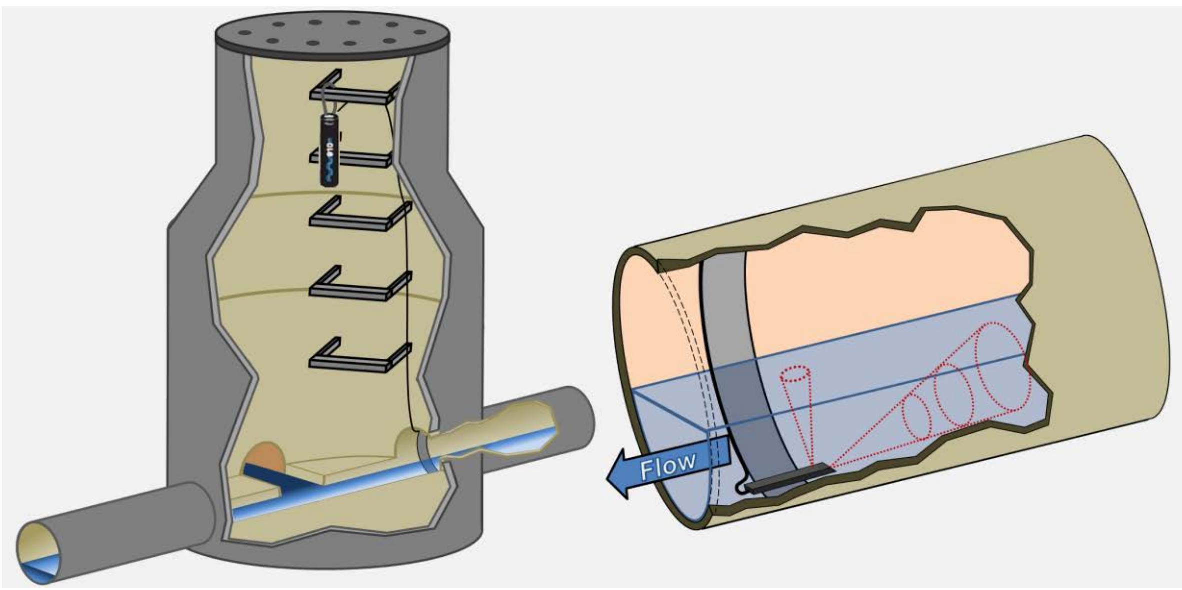


Figure 2-1. Typical Installation for Flow Meter with Submerged Sensor

The pipe diameter was verified in order to accurately calculate the flow cross-section. In-situ manual (hand) level and velocity measurements were taken and compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. This determination of the level offset is required because of variations in individual flow meters, position of the sensor upon installation, thickness of the mounting band and other factors.

During the in-situ calibrations, the technician/engineer reports the actual depth of the flow to the invert of the pipe (d_A) while the flow meter reports the depth of water to the sensor (d_S). The difference between these is the offset. These sets of measurements are taken at least three times during installation and removal of the flow meters and during mid-project calibrations. The various sets of measurements are used to track the data quality. During site visits, observations of sediment are noted. If sediment is present, several depth of sediment readings (S) are taken and the type of sediment encountered (sandy, rocky, pebbly) is noted.

Most area-velocity meters employ a forward-looking ultrasonic Doppler sensor that does not record velocity if covered by sediment. To mitigate this, the sensor may be offset to a position where sediment is less likely to affect the sensor. It is important to take multiple sediment readings in multiple locations; sediment tends to settle in waves, which affects the accuracy of the sediment measurement if not accounted for.

Figure 2-2 illustrates a sensor offset for sediment, and the level measurements recorded during an in situ calibration. Figure 2-3 illustrates sediment wave pattern settlement commonly observed in sewer lines.

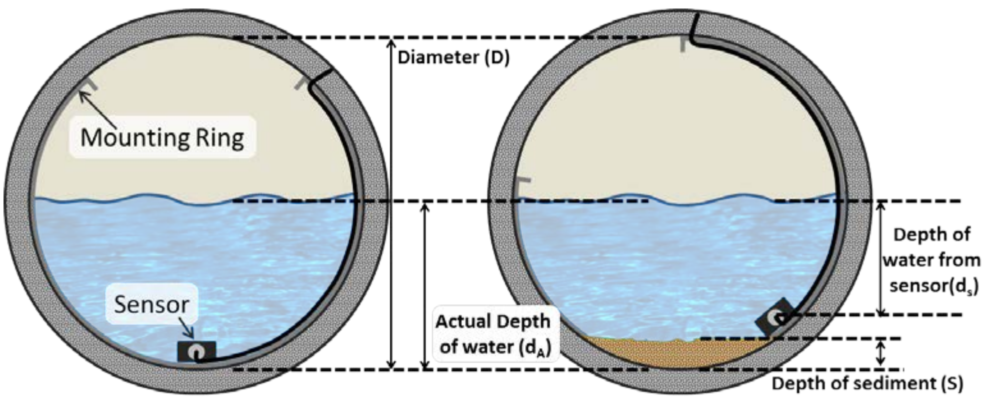


Figure 2-2. Sensor Offset due to Sediment

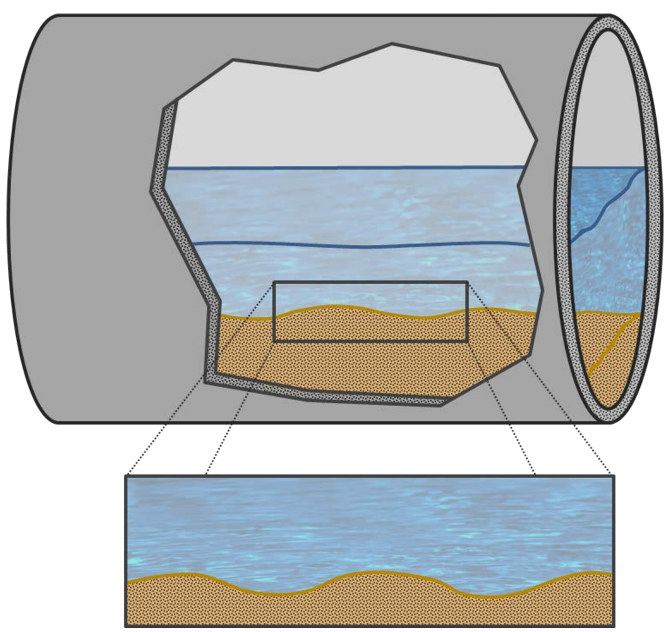


Figure 2-3. The Settling of Sediment in Pipelines

A Flo-Dar flow meter is a non-contact flow meter that uses radar to measure velocity and a down-looking ultrasonic sensor to measure depth. Figure 2-4 illustrates a typical Flo-Dar installation.

Figure 2-4. Typical Flo-Dar Flow Meter Installation

2.3 Flow Calculation

Data retrieved from each flow meter was placed into a spreadsheet program for analysis. Data analysis includes data comparison to field calibration measurements, as well as necessary geometric adjustments as required for sediment (sediment reduces the pipe's wetted cross-sectional area available to carry flow). Area-velocity flow metering uses the continuity equation,

$$Q = v \cdot A = v \cdot (A_T - A_S)$$

where Q : volume flow rate
v: average velocity as determined by the ultrasonic sensor
A: cross-sectional area available to carry flow
A_T: total cross-sectional area with both wastewater and sediment
A_S: cross-sectional area of sediment.

For circular pipe,

$$A_T = \left[\frac{D^2}{4}\cos^{-1}\left(1 - \frac{2d_W}{D}\right)\right] - \left[\left(\frac{D}{2} - d_W\right)\left(\frac{D}{2}\right)\sin\left(\cos^{-1}\left(1 - \frac{2d_W}{D}\right)\right)\right]$$

$$A_S = \left[\frac{D^2}{4}\cos^{-1}\left(1 - \frac{2d_S}{D}\right)\right] - \left[\left(\frac{D}{2} - d_S\right)\left(\frac{D}{2}\right)\sin\left(\cos^{-1}\left(1 - \frac{2d_S}{D}\right)\right)\right]$$

where d_W: distance between wastewater surface level and pipe invert
d_S: depth of sediment
D: pipe diameter

2.4 Average Dry Weather Flow Determination

Weekday and weekend flow patterns differ and must be separated when determining average dry weather flows. Days least affected by rainfall were used to estimate weekend and weekday average flows. The overall average dry weather flow (ADWF) was calculated per the following equation:

$$ADWF = \left(ADWF_{Mon-Fri} \times \frac{5}{7} \right) + \left(ADWF_{Sat-Sun} \times \frac{2}{7} \right)$$

Figure 2-5 illustrates the varying flow patterns within a typical dry week.

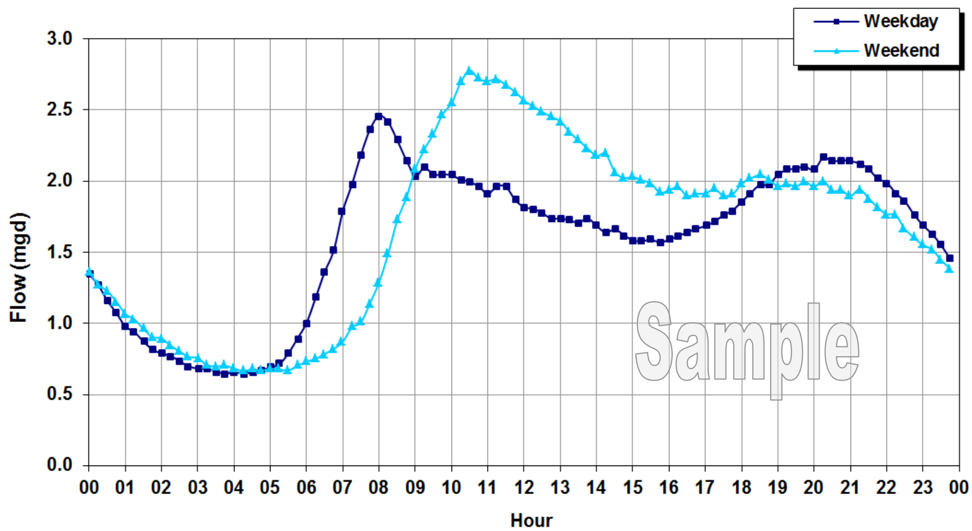


Figure 2-5. Sample ADWF Diurnal Flow Patterns

2.5 Pipeline Capacity Analysis Methods

Peak measured flows and peak flow depths are important factors to consider when evaluating the capacity of the collection system. The peak flows and flow levels reported are from the peak measurements taken across the entirety of the flow monitoring period and may or may not correspond to a simultaneous event for all sites.

The following capacity analysis terms are defined as follows:

- **Peaking Factor:** Peaking factor is defined as the peak measured flow divided by the ADWF. A peaking factor threshold value of 3.0 is commonly used for sanitary sewer design of new pipe; however, it is noted that this value is variable and subject to attenuation and the size of the upstream collector area. The City should follow its own standards and criteria when examining peaking factors.
- **d/D Ratio:** The d/D ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). Standards for d/D ratio vary agency to agency, but typically range between $d/D \leq 0.5$ and $d/D \leq 0.75$. The d/D ratio for each site was computed based on the maximum depth of flow for the flow monitoring study.

2.6 Inflow/Infiltration Analysis Methods

Inflow and infiltration (I/I) consists of stormwater and groundwater that enter the sewer system through pipe defects and improper storm drainage connections and is defined as follows.

2.6.1 Definition and Typical Sources

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

Figure 2-6 illustrates the possible sources and components of I/I.

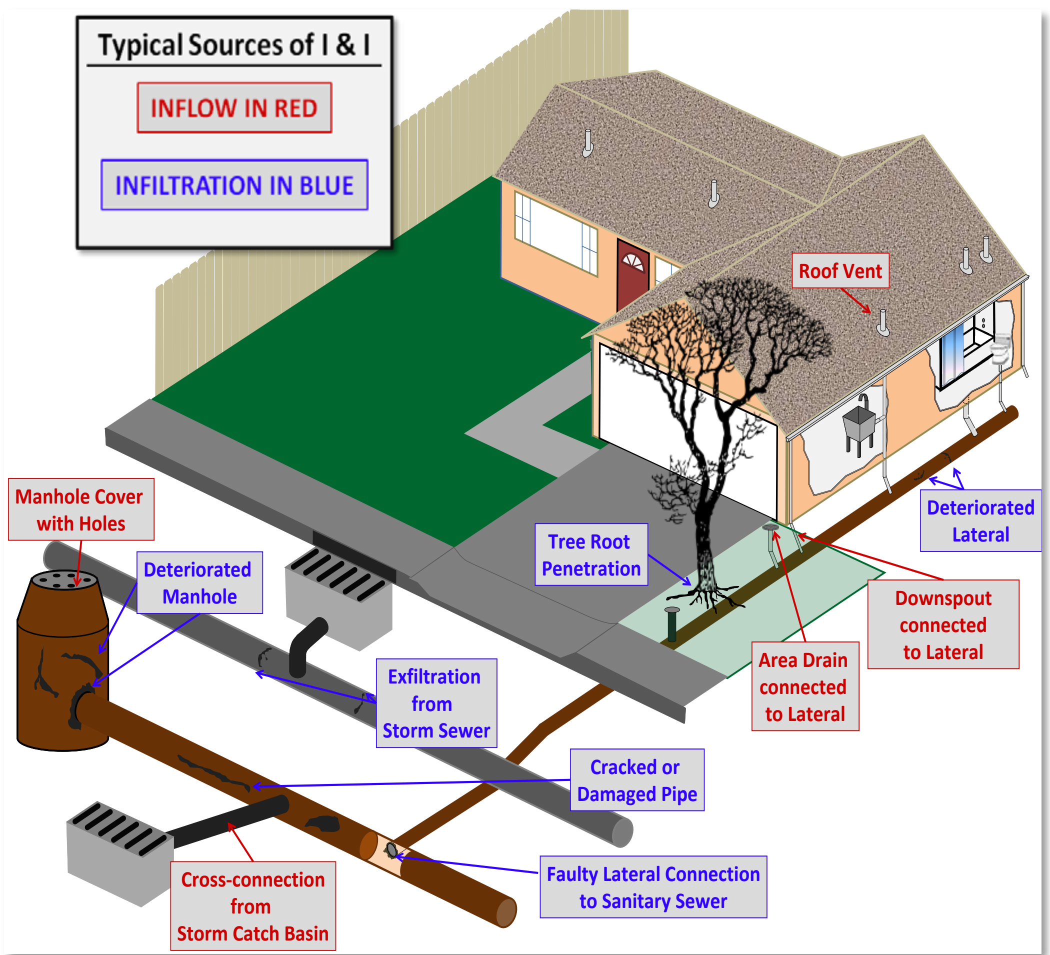


Figure 2-6. Typical Sources of Infiltration and Inflow

2.6.2 Infiltration Components

Infiltration can be further subdivided into the following components:

- **Rainfall-Dependent Infiltration:** This component occurs as a result of storm water and enters the sewer system through pipe defects, as with GWI. The storm water first percolates directly into the soil and then migrates to an infiltration point. Typically, the time of concentration for rainfall-dependent infiltration (RDI) may be 24 hours or longer, but this depends on the soil permeability and saturation levels.
- **Groundwater Infiltration:** Groundwater infiltration (GWI) depends on the depth of the groundwater table above the pipelines as well as the percentage of the system submerged. The variation of groundwater levels and subsequent GWI rates are seasonal by nature. On a day-to-day basis, GWI rates are relatively steady and will not fluctuate greatly.
- **Rainfall-Responsive Infiltration** is storm water that enters the collection system indirectly through pipe defects, but normally in sewers constructed close to the ground surface such as private laterals. Rainfall-responsive infiltration (RRI) is independent of the groundwater table and reaches defective sewers via the pipe trench in which the sewer is constructed, particularly if the pipe is placed in impermeable soil and bedded and backfilled with a granular material. In this case, the pipe trench serves as a conduit similar to a French drain, conveying storm drainage to defective joints and other openings in the system. This type of infiltration can have a quick response and graphically can look very similar to inflow.

2.6.3 Impact and Cost of Source Detection and Removal

- **Inflow:**
  - **Impact:** This component of I/I creates a peak flow problem in the sewer system and may dictate the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Because the response and magnitude of inflow is tied closely to the intensity of the storm event, short-term peak flows may result in surcharging and overflows within a collection system. Severe inflow can result in sewage dilution, which may upset the biological process (secondary treatment) at the treatment facility.
  - **Cost of Source Identification and Removal:** Inflow locations are usually less difficult to find and less expensive to correct than infiltration sources. These sources include direct and indirect cross-connections with storm drainage systems, roof downspouts, and various types of surface drains. Generally, the costs to identify and remove sources of inflow are low compared to potential benefits to public health and safety or the costs of building new facilities to convey and treat the resulting peak flows.
- **Infiltration:**
  - **Impact:** Infiltration typically creates long-term annual volumetric problems. The major impact is the cost of pumping and treating the additional volume of water, and of paying for treatment (for municipalities that are billed strictly on flow volume).
  - **Cost of Source Detection and Removal:** Infiltration sources are usually harder to find and more expensive to correct than inflow sources. Infiltration sources include defects in deteriorated sewer pipes or manholes that may be widespread throughout a sanitary sewer system.

2.6.4 Graphical Identification of Inflow and Infiltration

Inflow is usually recognized graphically by large-magnitude, short-duration spikes immediately following a rain event. Infiltration is often recognized graphically by a gradual increase in flow after a wet-weather event. The increased flow typically sustains for a period after rainfall has stopped and then gradually drops off as soils become less saturated and as groundwater levels recede to normal levels. Real-time flows were plotted against average dry weather flow (ADWF) to analyze the I/I response to rainfall events. Figure 2-7 illustrates a sample of how this analysis is conducted and some of the measurements that are used to distinguish I/I. Similar graphs were generated for the individual flow monitoring sites and can be found in *Appendix A*.

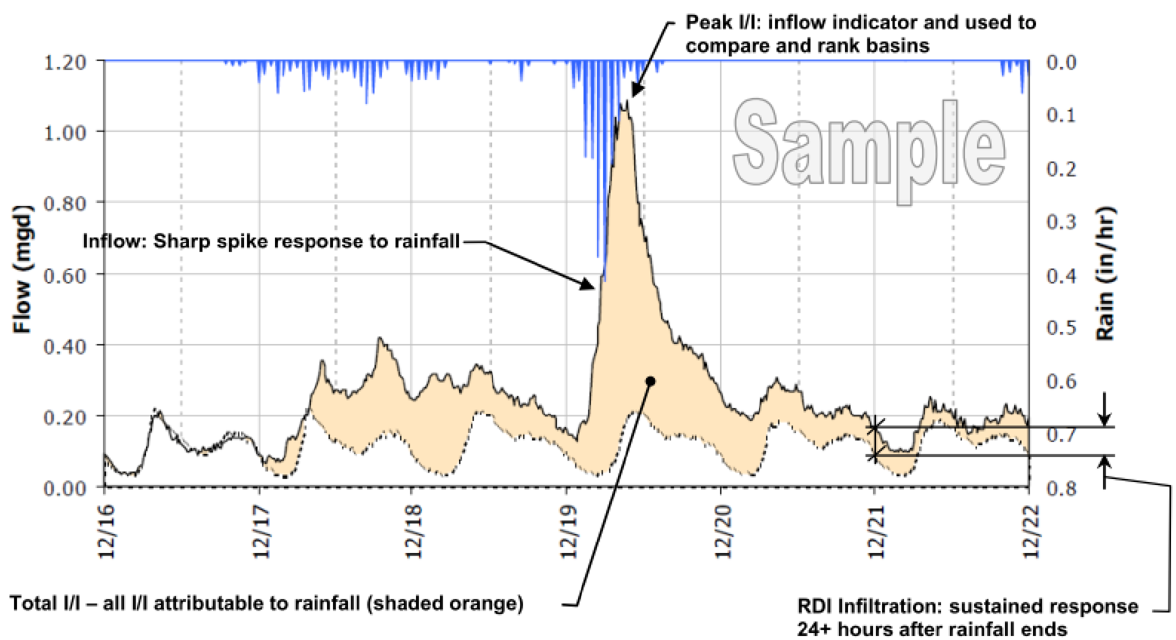


Figure 2-7. Sample Infiltration and Inflow Isolation Graph

2.6.5 Analysis Methods

After differentiating I/I flows from ADWF flows, various calculations can be made to determine which I/I component (inflow or infiltration) is more prevalent at a particular site and to compare the relative magnitudes of the I/I components between drainage basins and between storm events. Inflow or infiltration components are typically normalized in up to three ways:

- **per-IDM:** Inflow or infiltration rates are divided by length of pipe within the drainage basin, expressed in units of inch-diameter-mile (IDM) (miles of pipeline multiplied by the diameter of the pipeline in inches). Final units are gallons per day (gpd) per IDM.
- **per-ACRE:** Inflow or infiltration rates are divided by the acreage of the drainage basin. Final units are gallons per day (gpd) per ACRE.
- **per-ADWF:** Inflow or infiltration rates are divided by the ADWF that was measured and established within the drainage basin. This is a ratio. The number is unitless, but can be thought of in the same light as a Peaking Factor.

The infiltration and inflow indicators were normalized by all three methods for this report per the following weighting system:

I/I Analysis Method	Weight
per-IDM	50%
per-ACRE	20%
per-ADWF	30%

The per-IDM method was given the highest weight because capital improvement projects concerning rehabilitation or replacement of sanitary sewer pipelines are most commonly bid based on the length of pipe. Note: inflow is subject to the effects of attenuation, explained in the following section.

2.7 Flow Attenuation

Flow attenuation in a sewer collection system is the natural process of the reduction of the peak flow rate through redistribution of the same volume of flow over a longer period of time. This occurs as a result of friction (resistance), internal storage and diffusion along the sewer pipes. Fluids are constantly working towards equilibrium. For example, a volume of fluid poured into a static vessel with no outside turbulence will eventually stabilize to a static state, with a smooth fluid surface without peaks and valleys. Attenuation within a sanitary sewer collection system is based upon this concept. A flow profile with a strong peak will tend to stabilize towards equilibrium, as shown in Figure 2-8.

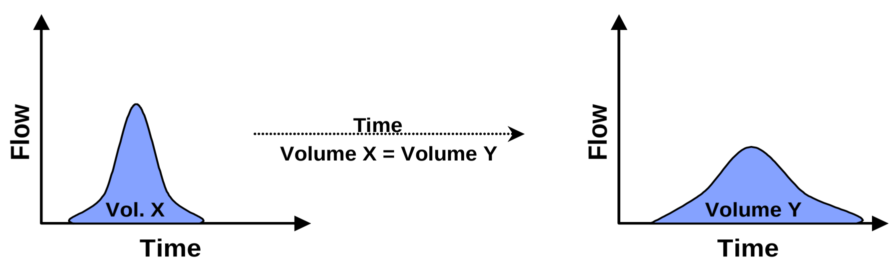


Figure 2-8. Attenuation Illustration

Within a sanitary sewer collection system, each individual basin will have a specific flow profile. As the flows from the basins combine within the trunk sewer lines, the peaks from each basin will (a) not necessarily coincide at the same time, and (b) due to the length and time of travel through the trunk sewers, peak flows will attenuate prior to reaching the treatment facility. The sum of the peak flows of the individual basins within a collection system will usually be greater than the peak flows observed at the treatment facility.

3.0 RAINFALL RESULTS

3.1 Rainfall Data

There were sufficient rainfall events over the flow monitoring period that could be used to conduct multiple sets of infiltration and inflow analysis, allowing for relocation of flow meters and isolation of multiple drainage basins. V&A utilized rain data publically available through the National Weather Service (NWS) Cooperative Observer Program (COOP). While V&A performed QA/QC analysis to ensure, to the extent possible, the quality of the rainfall data, it is noted that V&A had no direct control over those gauges.

Table 3-1 shows the precipitation for the notable rainfall events measured from the four rain gauges. Figure 3-1 illustrates the rain events over the monitoring period (average of all rain gauges shown). Figure 3-2 shows the rainfall accumulation during the monitoring period, as well as the historical average rainfall[5] in Pinole at the approximate study centroid during this project duration.

The cumulative precipitation at the four rain gauges ranged from 52% to 85% of the historical precipitation for the time period shown.

Table 3-1. Rainfall Events Summary

Rainfall	RG North (inches)	RG East (inches)	RG South (inches)	RG West (inches)
Event 1: February 2, 2014 – February 10, 2014	3.75	5.47	4.76	3.00
Event 2: February 26, 2014 – March 6, 2014	2.34	2.48	2.55	1.28
Event 3: March 26, 2014 – April 1, 2014	2.45	3.17	3.09	2.46
Total over Monitoring Period:	*8.69*	*11.28*	*10.55*	*6.84*

[5] Historical data taken from the WRCC (Station 45378 in Martinez, CA and Station 47414 in Richmond, CA): http://www.wrcc.dri.edu/summary/climsmnca.html

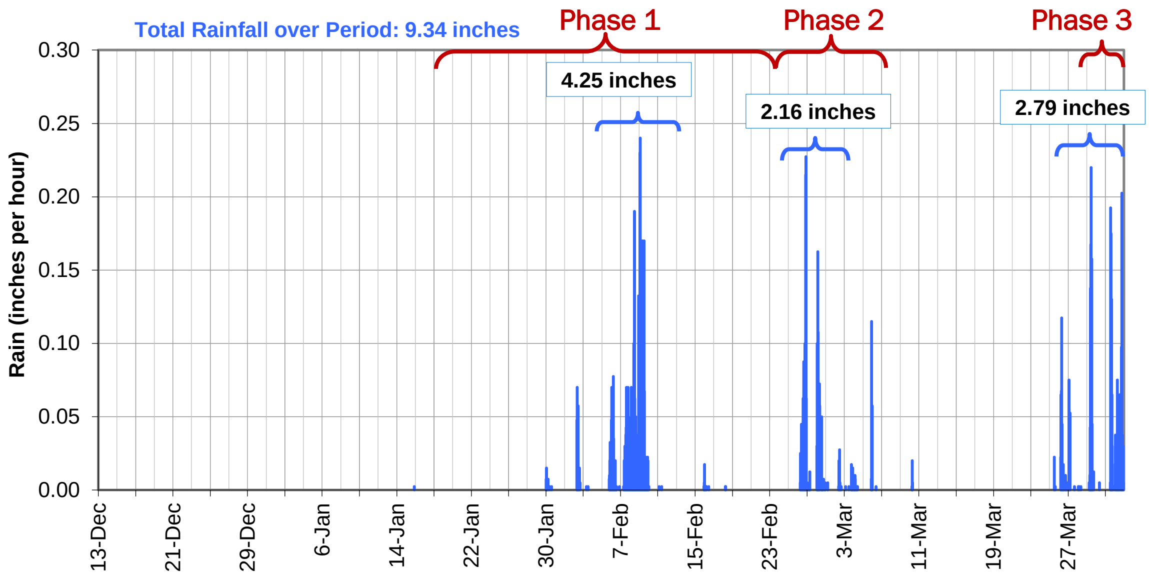


Figure 3-1. Rainfall Distribution over Flow Monitoring Period (Avg. of Four Rain Gauges)

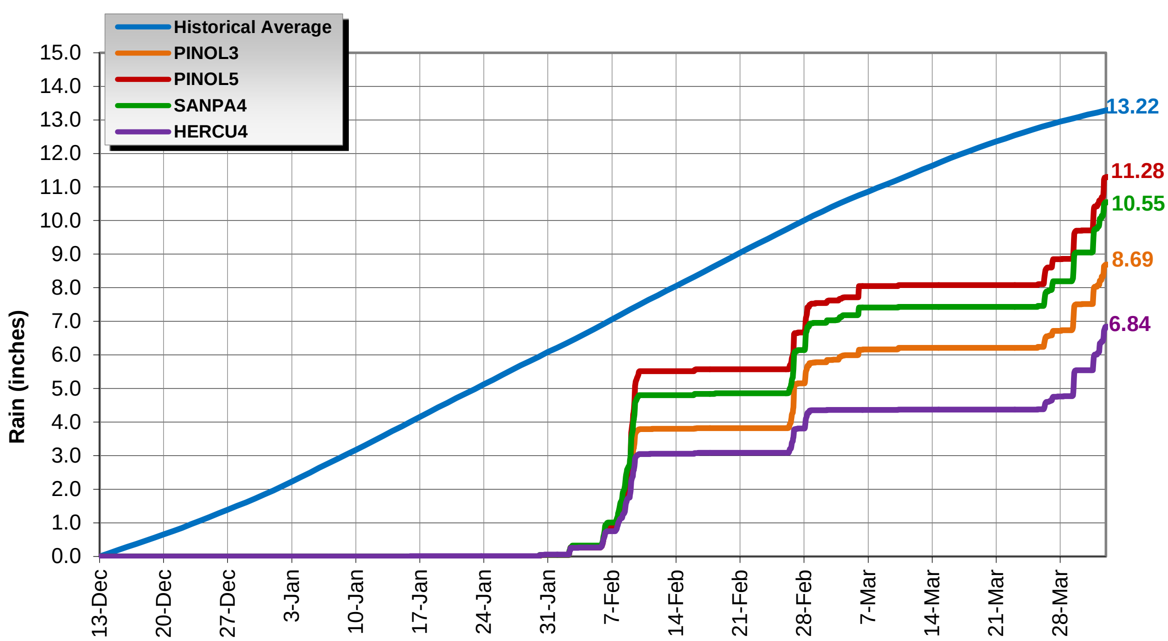


Figure 3-2. Rainfall Accumulation Plot

3.2 Regional Rainfall Event Classification

It is important to classify the relative size of a major storm event that occurs over the course of a flow-monitoring period[6]. Rainfall events are classified by intensity and duration. Based on historical data, frequency contour maps for storm events of given intensity and duration have been developed by the National Oceanic and Atmospheric Administration (NOAA) for Northern California (Figure 3-3).

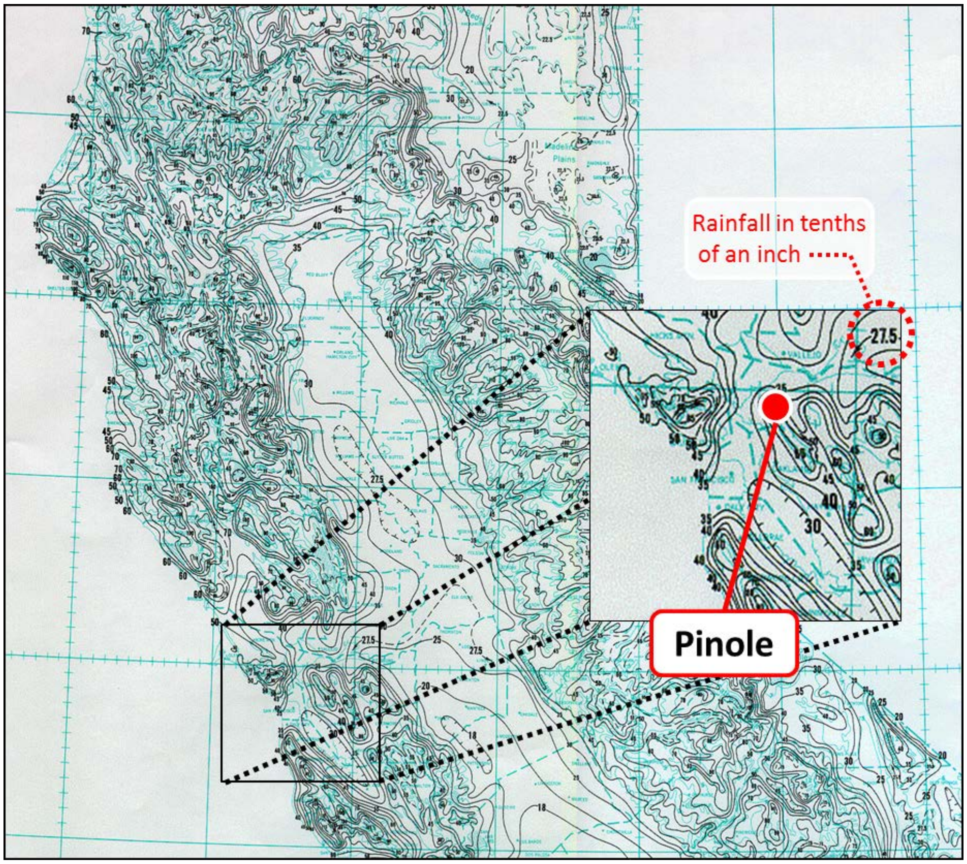


Figure 3-3. NOAA Northern California Rainfall Frequency Map, Isopluvials of 10-year 24-hour precipitation in inches

For example, the NOAA *Rainfall Frequency Atlas*[7] classifies a 10-year, 24-hour storm event at the PINOL5 rain gauge location as 3.75 inches. This means that in any given year, at this specific location, there is a 10% chance that 3.75 inches of rain will fall in any 24-hour period.

From the NOAA frequency maps, for a specific latitude and longitude, the rainfall densities for period durations ranging from 1 hour to 24 hours are known for rain events ranging from 1-year to 100-year intensities. These were plotted to develop a rain event frequency map specific to each rainfall monitoring site. Superimposing the peak-measured densities for the rainfall events on the rain event

[6] Sanitary sewers are often designed to withstand I/I contribution to sanitary flows for specific-sized "design" storm events.
[7] NOAA Western U.S. Precipitation Frequency Maps Atlas 14, 1973: http://www.wrcc.dri.edu/pcpnfreq.html

frequency plot determines the classification of the rainfall event. Figure 3-4 depicts the classification curves for the rainfall events for the SANPA4 rain gauge. Table 3-2 lists the intensity of the various storm events for each rain gauge.

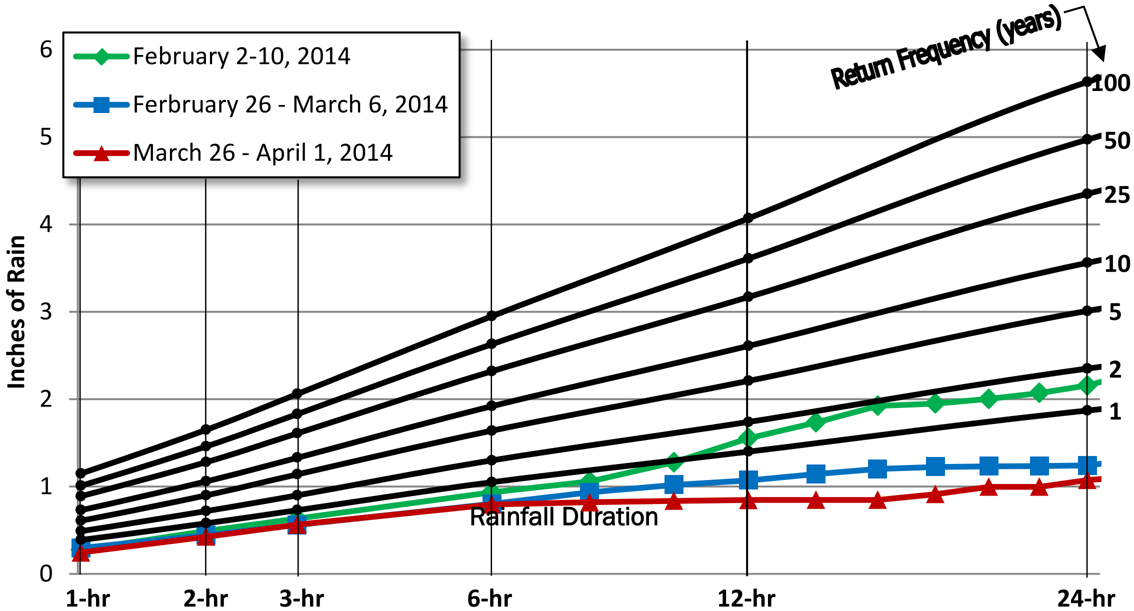


Figure 3-4. Storm Event Classification at SANPA4

Table 3-2. Rainfall Frequency Return Summary

Rainfall Event	PINOL3 (in)	PINOL5 (in)	SANPA4 (in)	HERCU4 (in)
Event 1: February 2 – February 10, 2014	< 1 year	4 year, 24 hour	1+ year, 24 hour	< 1 year
Event 2: February 26 – March 6, 2014	< 1 year	< 1 year	< 1 year	< 1 year
Event 3: March 26 – April 1, 2014	< 1 year	< 1 year	< 1 year	< 1 year

3.3 Rainfall Summary

- Though the total rainfall for the entirety of the 2013/2014 rainfall season was generally low, Storm Event 1 provided sufficient rainfall to conduct a solid I/I analysis of the flow monitoring data gathered for Phase 1.
- Event 1 was classified as greater than a 1-year, 24-hour event for some rain gauges in the region. Event 1 also increased the soil saturation levels so that future rainfall events (Events 2 and 3) were effective for I/I analysis of the flow monitoring data collected for Phase 2 and Phase 3.

4.0 FLOW MONITORING RESULTS

4.1 Average Dry Weather Flows

ADWF flows were established during dry days within the flow monitoring period when RDI had the least impact on the flow rates. Table 4-1 summarizes the ADWF flow data measured during this study. Figure 4-1, Figure 4-2 and Figure 4-3 show flow schematic diagrams of the ADWF and flow levels for the flow monitoring sites, all phases.

Table 4-1. Average Dry Weather Flow Summary

Monitoring Site	Weekday ADWF (mgd)	Weekend ADWF (mgd)	Overall ADWF (mgd)
Phase 1			
Site M1	0.70	0.76	0.71
Site M2	1.11	1.10	1.11
Site M3	0.07	0.07	0.07
Site M4	0.13	0.14	0.13
Site M5	0.02	0.02	0.02
Site M6	0.17	0.19	0.18
Site M7	0.36	0.38	0.36
Site M8	0.007	0.006	0.007
Site M9	0.004	0.005	0.004
Site M10	0.08	0.09	0.09
Site M11	0.02	0.02	0.02
Site M12	0.003	0.003	0.003
Site M13	0.06	0.06	0.06
Site M14	0.04	0.04	0.04
Site M15	0.01	0.02	0.02
Site M16	0.08	0.09	0.09
Phase 2			
Site M3.1	0.035	0.025	0.032
Site M3.2	0.011	0.011	0.011
Site M5.1	0.022	0.026	0.023
Site M5.2	0.011	0.013	0.011
Site M5.3	0.022	0.023	0.022
Site M6.1	0.010	0.014	0.011

Monitoring Site	Weekday ADWF (mgd)	Weekend ADWF (mgd)	Overall ADWF (mgd)
Site M6.2	0.015	0.018	0.016
Site M6.3	0.171	0.140	0.163
Site M6.4	0.076	0.103	0.084
Site M6.5	0.008	0.011	0.009
Phase 3			
Site M3.1A	0.009	0.010	0.009
Site M3.1B	0.006	0.006	0.006
Site M5.2A	0.009	0.010	0.009
Site M6.0A	0.056	0.061	0.057
Site M6.3A	0.005	0.005	0.005
Site M6.3B	0.003	0.004	0.004
Site M6.5A	0.003	0.003	0.003
Site M6.5B	0.003	0.004	0.003

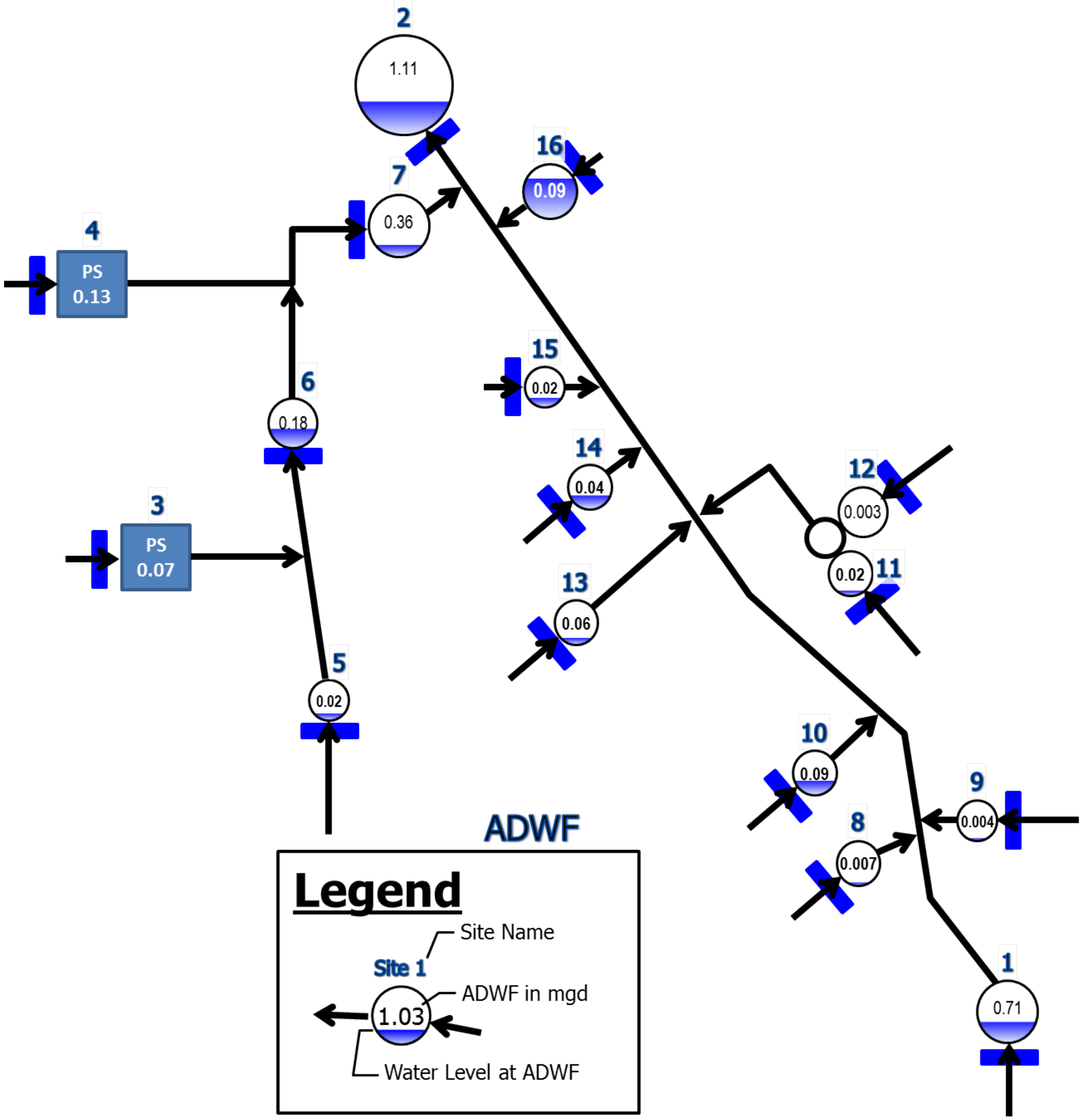


Figure 4-1. Average Dry Weather Flow Schematic (Phase 1)

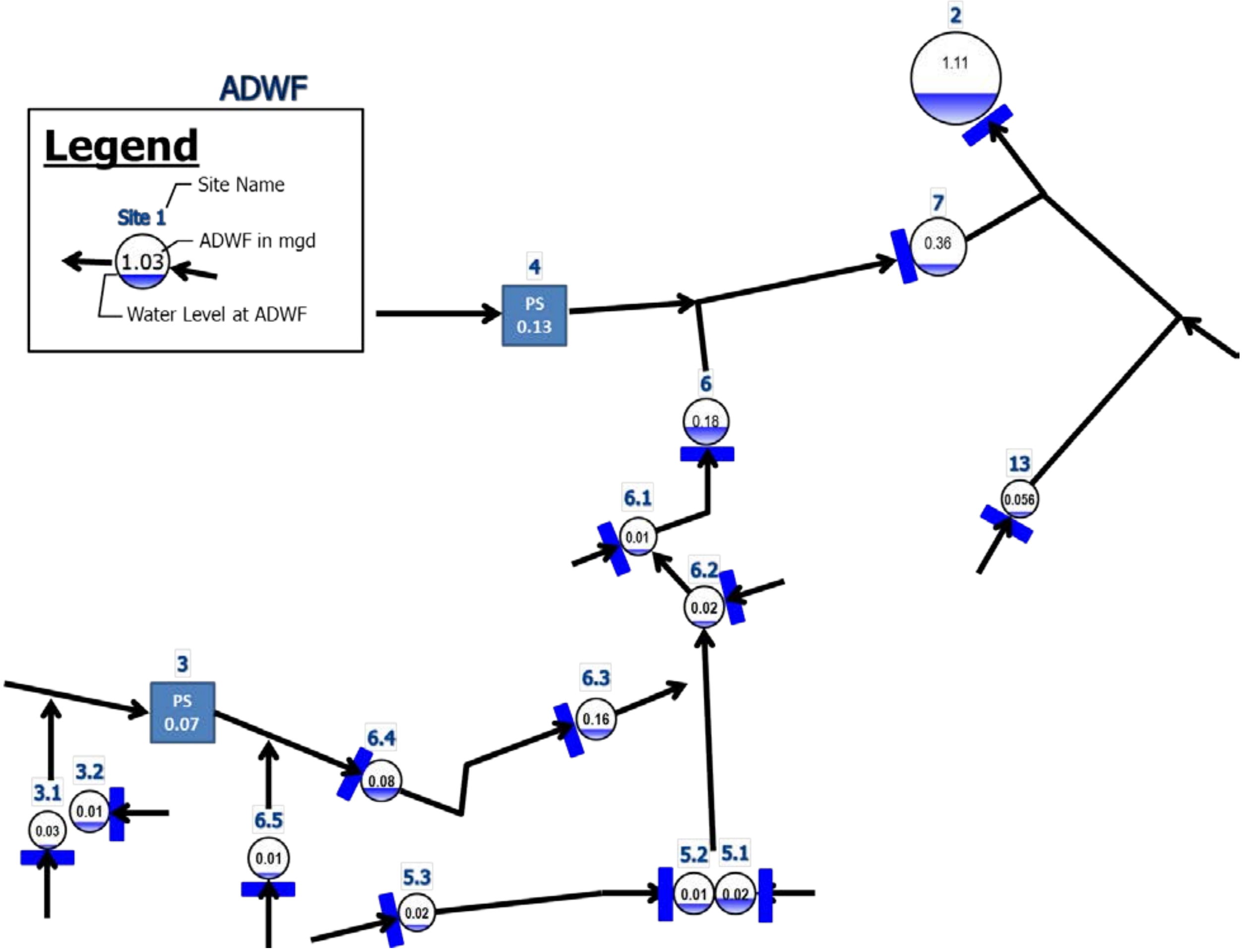


Figure 4-2. Average Dry Weather Flow Schematic (Phase 2)

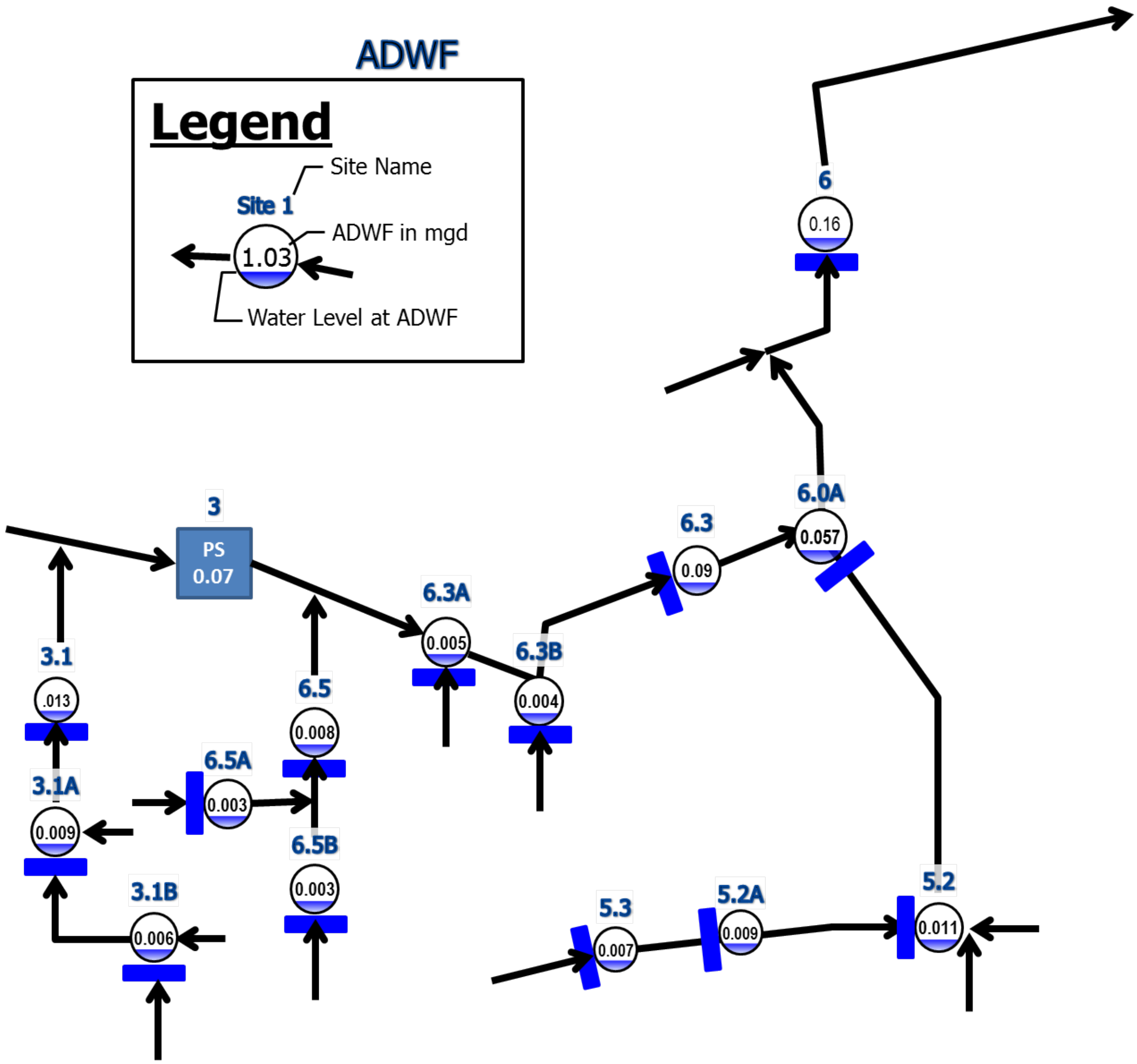


Figure 4-3. Average Dry Weather Flow Schematic (Phase 3)

4.2 Peak Measured Flows and Pipeline Capacity Analysis

Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the point site locations. Hydraulic conditions in other areas of the collection system will differ. Due to the relocation of flow meters and different flow data capture periods, the peak flows and peak levels should not necessarily be directly compared to each other. The period of meter installation are coded per the following scheme:

Scheme	Install Period
A	1/17/2014 – 2/25/2014
B	1/17/2014 – 3/28/2014
C	1/17/2014 – 4/1/2014
D	2/25/2014 – 3/28/2014
E	2/25/2014 – 4/1/2014
F	3/28/2014 – 4/1/2014

Table 4-2 summarizes the peak recorded flows, levels, *d/D* ratios, and peaking factors per site during the flow monitoring period. Sites that surcharged and sites with peaking factors greater than 10.0 have been shaded in RED. Figure 4-4, Figure 4-5 and Figure 4-6 show bar graphs of the capacity results for all phases. Figure 4-7, Figure 4-8 and Figure 4-9 show schematic diagrams of the peak measured flows with peak flow levels for all phases of the flow monitoring.

Table 4-2. Capacity Analysis Summary

Monitoring Site	Install Period Scheme	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (inches)	Peak Level (inches)	d/D Ratio	Surcharge Level (feet)
Phase 1								
Site M1	A	0.71	3.50	4.9	15	8.5	0.57	-
Site M2	C	1.11	7.20	6.5	30	45.5	1.52	1.3
Site M3	C	0.07	0.71	10.7	n/a	n/a	n/a	-
Site M4	C	0.13	0.47	3.7	n/a	n/a	n/a	-
Site M5	A	0.02	0.65	33.9	7.25	10.7	1.53	0.3
Site M6	C	0.18	1.10	6.2	10	82.3	8.23	6.0
Site M7	B	0.36	2.02	5.6	15	11.8	0.78	-
Site M8	A	0.007	0.03	5.0	7.75	1.3	0.17	-
Site M9	A	0.004	0.30	67.5	6	2.7	0.45	-
Site M10	A	0.09	0.41	4.8	8	11.4	1.43	0.3
Site M11	A	0.02	0.53	30.6	10	51.6	5.16	3.5
Site M12[8]	A	0.003	n/a	n/a	8	51.6	6.45	3.6

[8] Site M12 failed during Storm Event 1; the manhole was surcharged for an extended time period, flooding the flow logging computer. Enough data was recovered to establish an average dry weather flow but not enough for a proper capacity and I/I analysis. This site and other sites that comprise the 'Old Henry Road' basin are the subject of a future project.

Monitoring Site	Install Period Scheme	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (inches)	Peak Level (inches)	d/D Ratio	Surcharge Level (feet)
Site M13	B	0.06	0.21	3.8	6	1.7	0.28	-
Site M14	A	0.04	0.32	7.7	8	9.3	1.16	0.1
Site M15	A	0.02	0.21	14.1	6	2.6	0.44	-
Site M16	A	0.09	0.62	7.2	11.5	37.2	3.23	2.1
Phase 2								
Site M3.1	E	0.03	0.58	18.3	6	3.3	0.41	-
Site M3.2	D	0.01	0.08	7.3	6	3.2	0.40	-
Site M5.1	D	0.02	0.15	6.5	8	7.6	0.95	-
Site M5.2	E	0.01	0.31	27.6	8	6.1	0.77	-
Site M5.3	E	0.02	0.10	4.5	6	2.1	0.35	-
Site M6.1	D	0.01	0.12	10.4	6	7.2	1.20	0.1
Site M6.2	D	0.02	0.04	2.3	8	1.9	0.23	-
Site M6.3	E	0.16	0.72	4.4	8	5.1	0.64	-
Site M6.4	D	0.08	0.51	6.1	8	6.2	0.78	-
Site M6.5	E	0.01	0.12	13.5	8	3.0	0.37	-
Phase 3								
Site M3.1A	F	0.009	0.15	15.4	6	5.9	0.98	-
Site M3.1B	F	0.006	0.10	16.7	6	2.0	0.34	-
Site M5.2A	F	0.009	0.11	12.0	6	3.2	0.53	-
Site M6.0A	F	0.057	1.09	19.0	10	7.2	0.72	-
Site M6.3A	F	0.005	0.30	61.5	6	3.2	0.53	-
Site M6.3B	F	0.004	0.06	16.1	8	1.3	0.16	-
Site M6.5A	F	0.003	0.03	11.1	7.75	1.3	0.17	-
Site M6.5B	F	0.003	0.04	11.8	7.75	1.1	0.14	-

The following capacity analysis results are noted:

- **Peaking Factor:** Several sites had peaking factors greater than 10. Larger peaking factors are expected given that the study analyzes basins previously identified as having I/I rates.
- **d/D Ratio:** Nine of the flow monitoring sites (Sites M2, M5, M6, M10, M11, M12, M14, M16 and M6.1) reached surcharge conditions.
- **Sanitary Sewer Overflow Potential:** Given the level of surcharging seen during Storm Event 1, the manholes at Site 6, Site 11/12 and Site 16 have the potential for a sanitary sewer overflow (SSO) during a larger rainfall event. Site 6 has historical precedence for SSO discharging during large rainfall events.
- These capacity results do not replace a full hydraulic model which would determine capacity on a node-to-node basis and would be based on pipe slopes of the individual pipe segments within the local collection system.

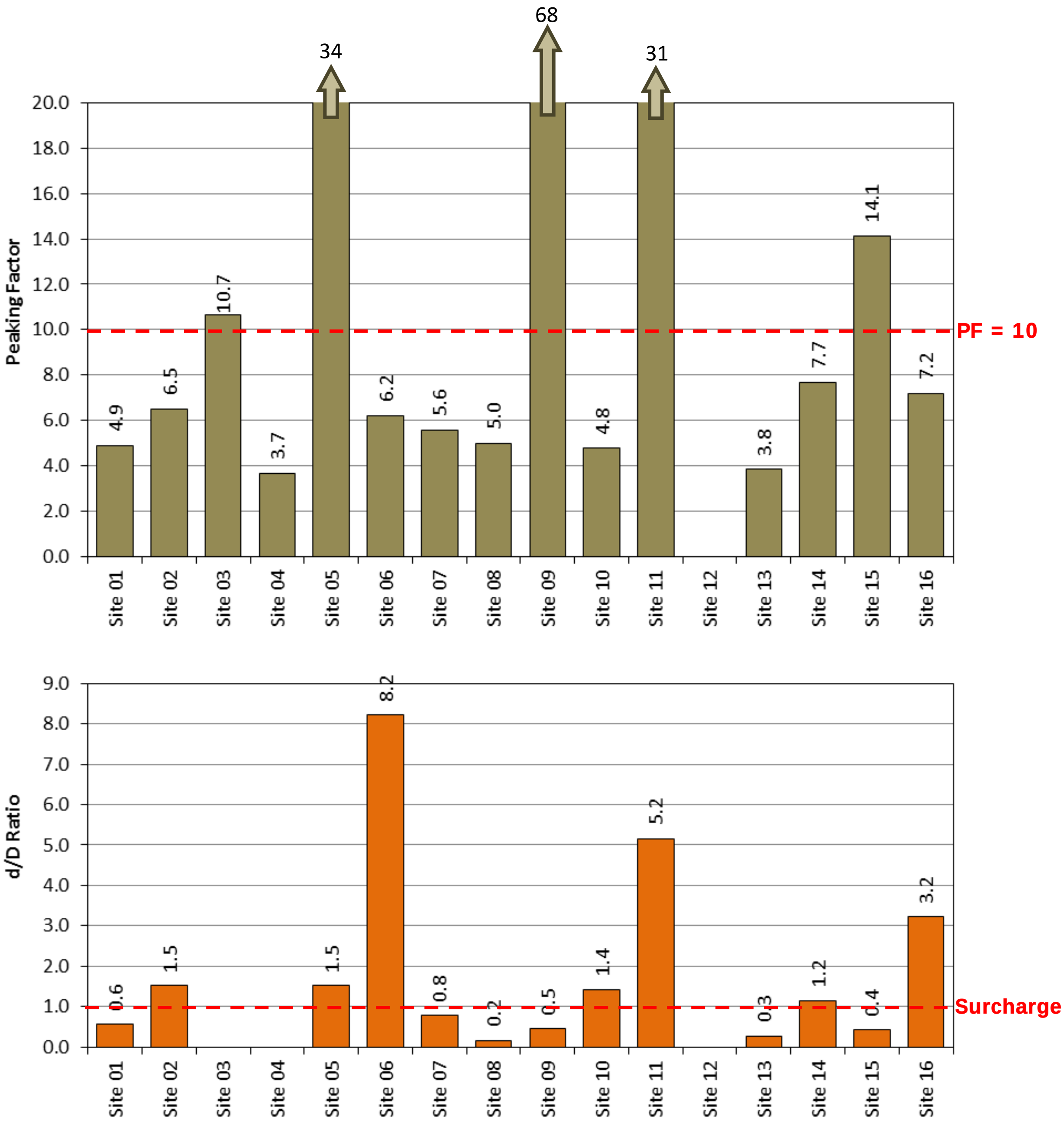
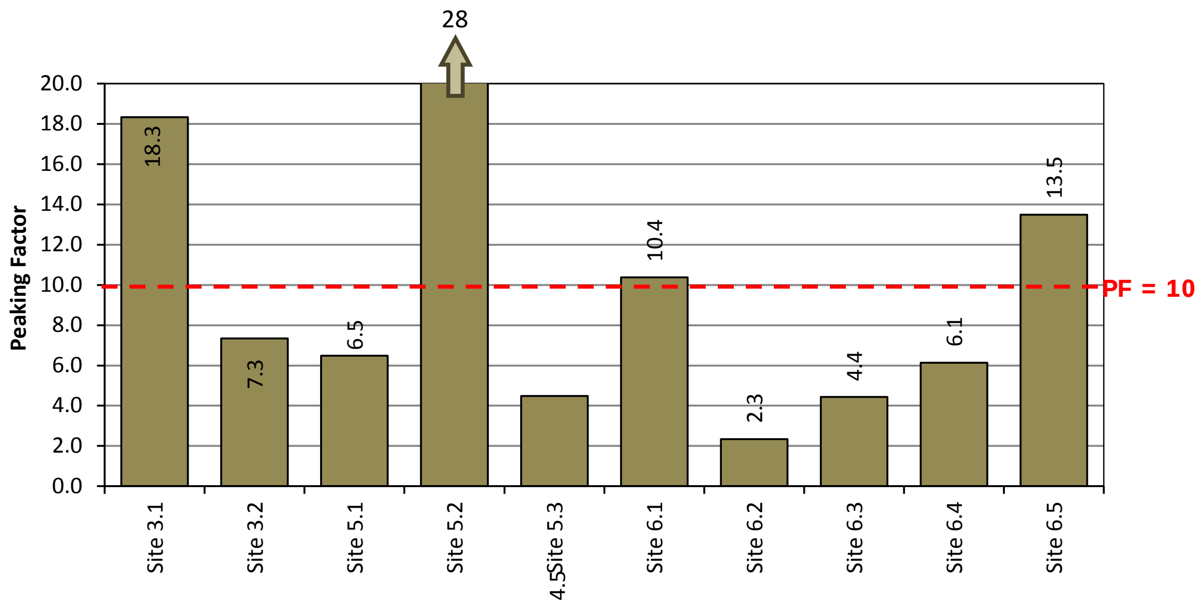


Figure 4-4. Phase 1 Capacity Summary: Peaking Factors and d/D Ratios



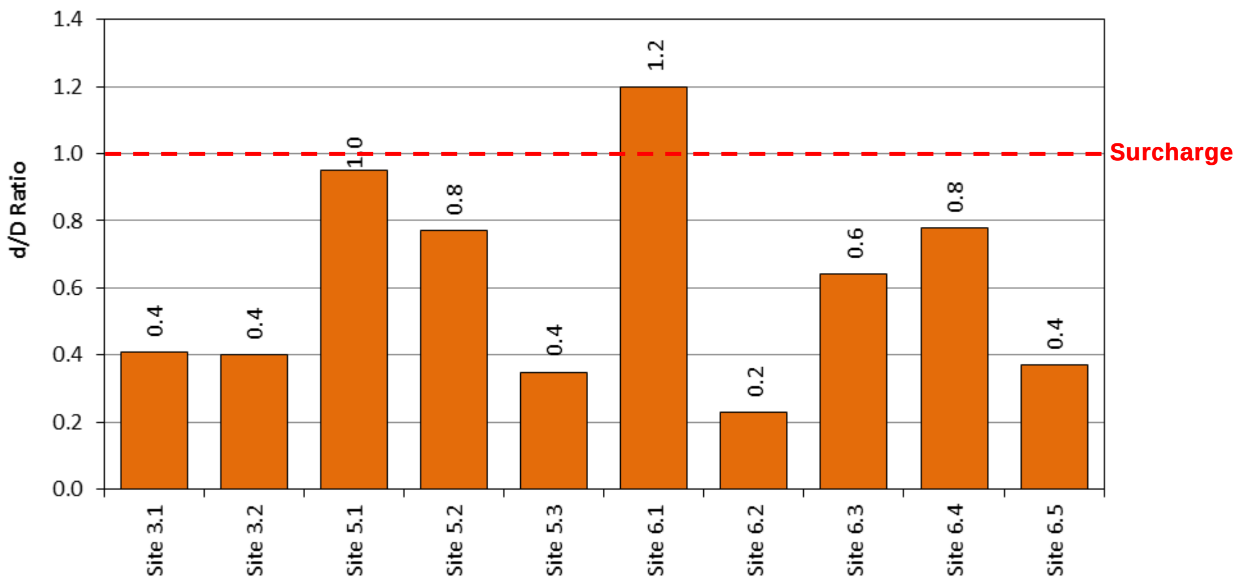
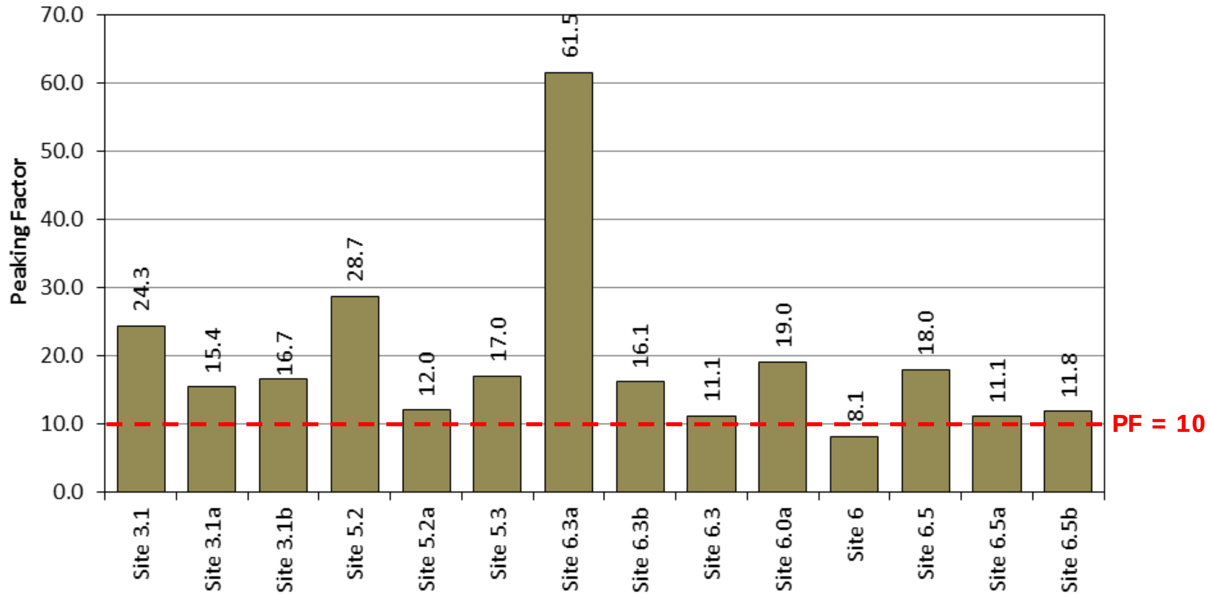


Figure 4-5. Phase 2 Capacity Summary: Peaking Factors and d/D Ratios



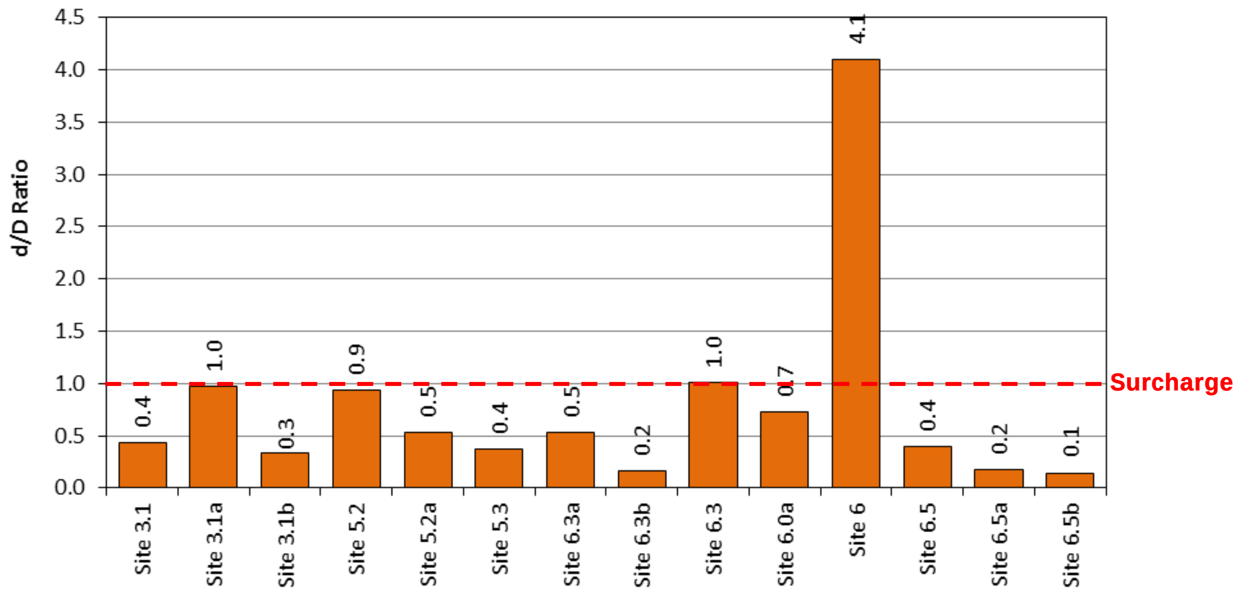


Figure 4-6. Phase 3 Capacity Summary: Peaking Factors and d/D Ratios

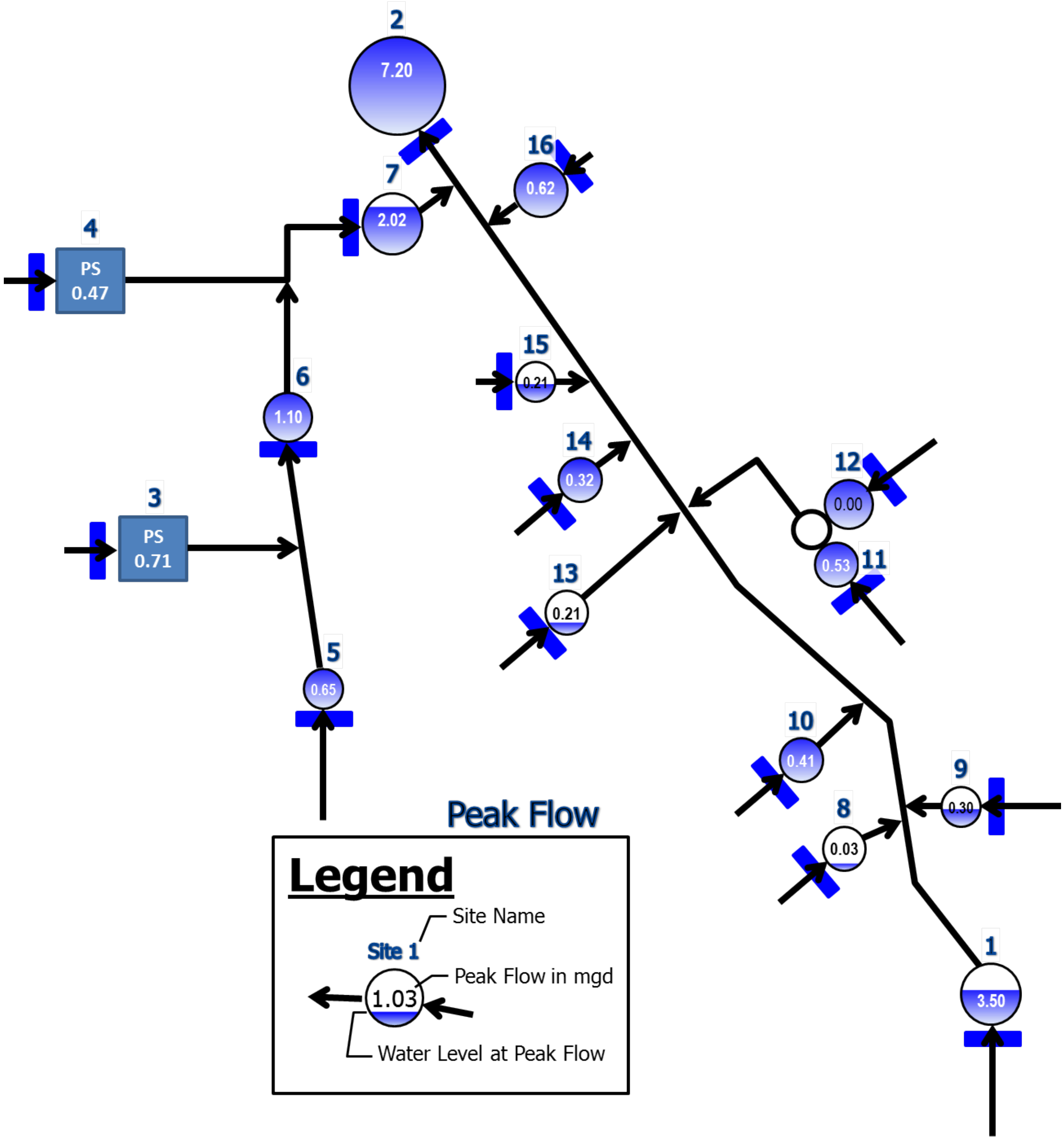


Figure 4-7. Peak Measured Flow Schematic (Phase 1)

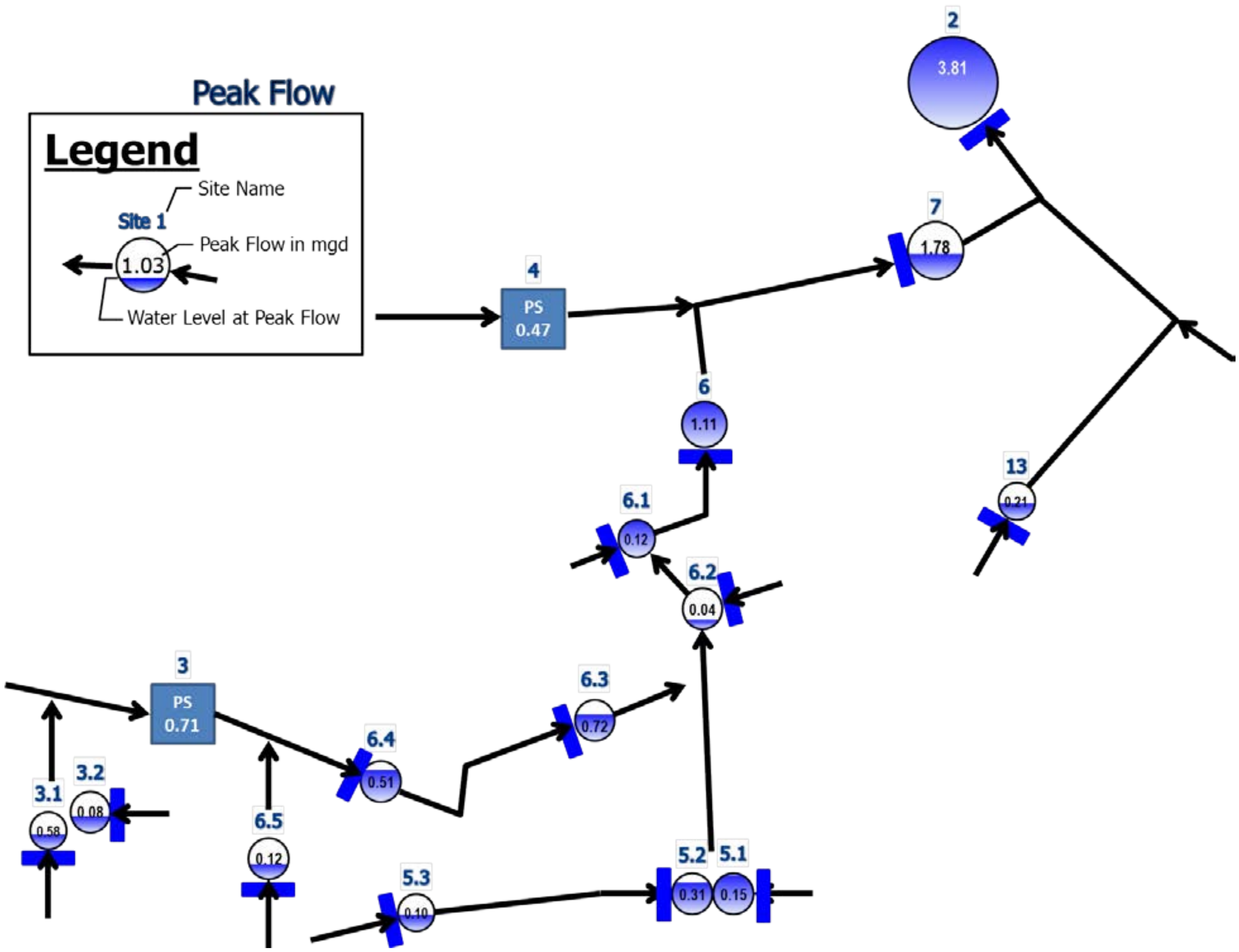


Figure 4-8. Peak Measured Flow Schematic (Phase 2)

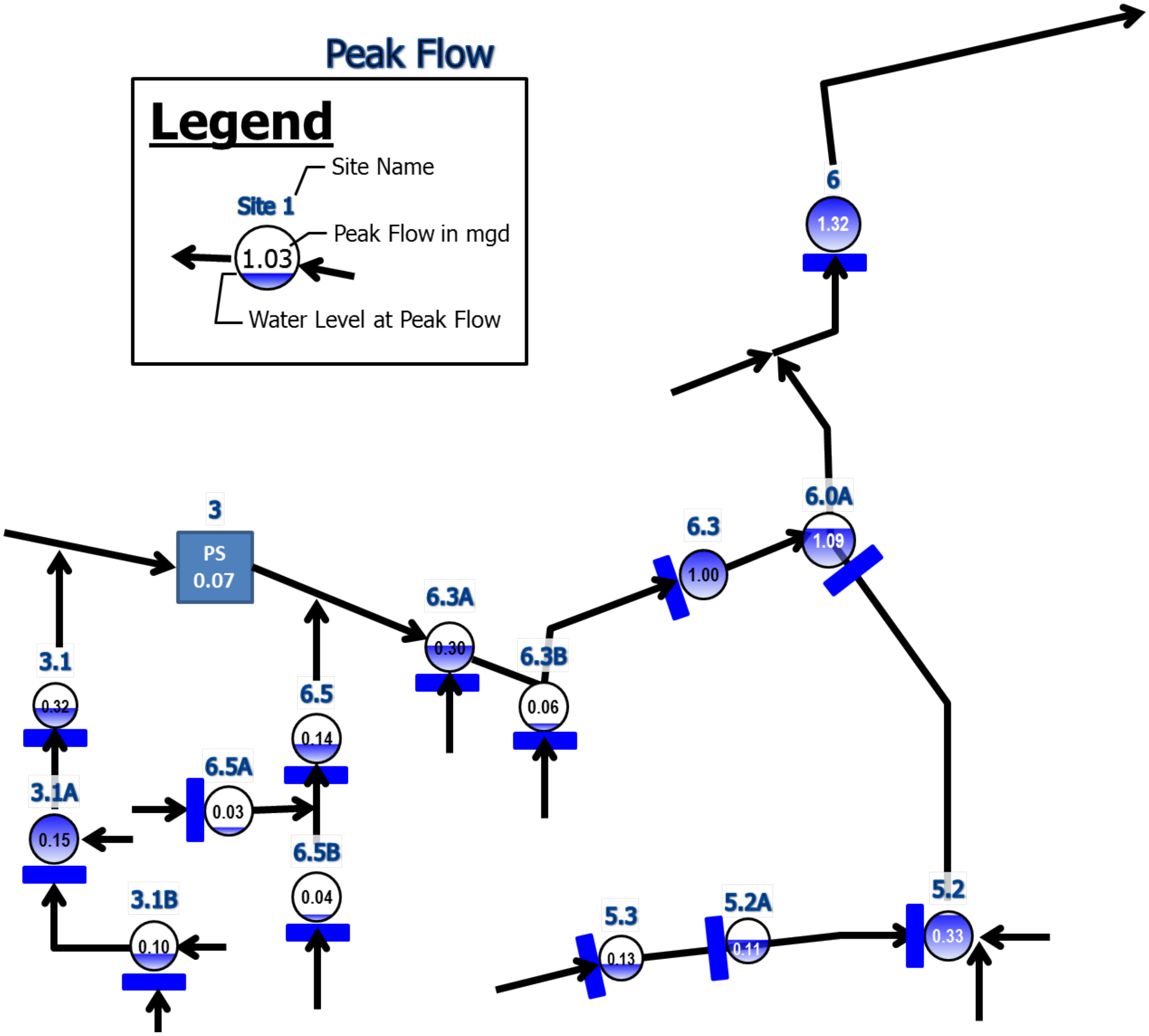


Figure 4-9. Peak Measured Flow Schematic (Phase 3)

5.0 PHASE 1 I/I RESULTS

5.1 Preface

The original 16 flow monitoring basins were illustrated in Figure 1-4. The following items are noted regarding the early-season analysis:

- Results are presented on a basin-by-basin basis (not a site-by-site basis).
- Results are for the original 15 basins only (Basin 12 excluded).
- Results presented to the City during the early-season were considered preliminary and based upon the information known at the time of presentation.
- Results presented in the following pages of analyses have been updated from the initial presentation to reflect final data and results; values may be different from the initial presentation; however, the conclusions and recommendations are the same.

5.2 Inflow Analysis

Inflow sources transport rain water *directly* into the sewer system; the corresponding inflow rates are tied closely to the intensity of the storm. This component of RDI/I often causes a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows.

Figure 5-1 illustrates the I/I response curve for Basin 3 and Storm Event 1 as it relates to peak I/I rate.

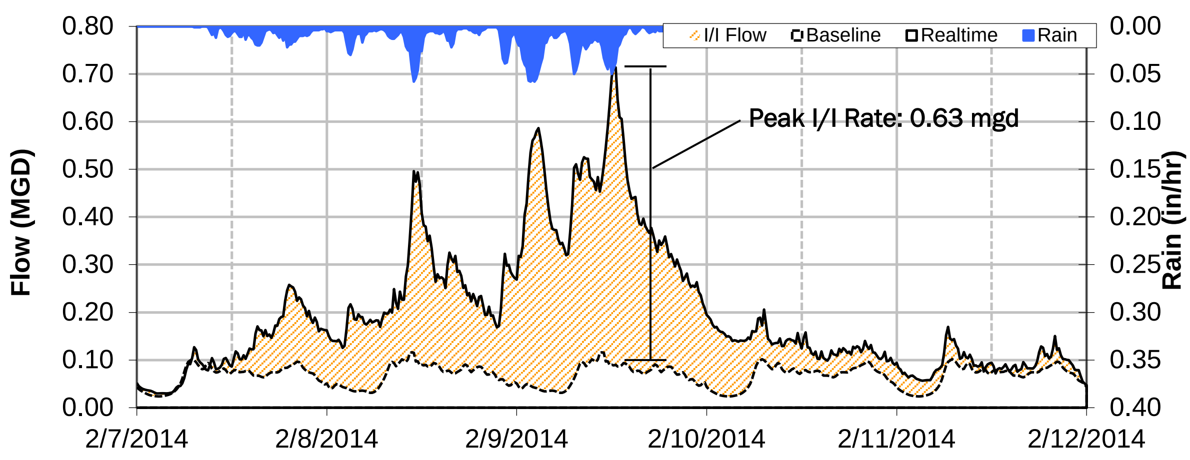


Figure 5-1. I/I Isolation Curve, Basin 3 (Inflow Measurement)

Table 5-1 summarizes the peak measured I/I flows and inflow analysis results for the Phase 1 basins for Storm Event 1 (refer to the *Methods* section for more information on inflow analysis methods). The peak I/I rate was normalized by three different methods: length of pipe (IDM), basin area (acres) and sewerage contribution (ADWF). Basins that ranked in the top 5 have been color coded red. Figure 5-2 illustrates a temperature map summary of the inflow analysis results per basin.

Table 5-1. Inflow Analysis Summary, Large Basins

Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Peak I/I per IDM (gpd/IDM)	Peak I/I per ACRE (gpd/AC)	Peak I/I per ADWF Ratio	Inflow Ranking
Basin 3	0.067	0.63	123,000	15,700	9.34	1
Basin 4	0.129	0.28	22,000	4,800	2.16	11
Basin 5	0.019	0.64	62,000	9,000	33.50	4
Basin 6	0.090	0.14	11,000	1,100	1.57	13
Basin 7	0.058	0.66	56,000	16,100	11.34	6
Basin 8	0.007	0.02	3,000	400	3.06	12
Basin 9	0.004	0.30	60,000	10,700	67.75	2
Basin 10	0.085	0.26	22,000	4,200	3.05	10
Basin 11	0.017	0.51	49,000	9,800	29.38	7
Basin 13	0.056	0.16	50,000	9,400	2.86	8
Basin 14	0.042	0.30	47,000	8,100	7.17	9
Basin 15	0.015	0.21	58,000	13,100	13.91	5
Basin 16	0.086	0.58	121,000	48,300	6.78	3

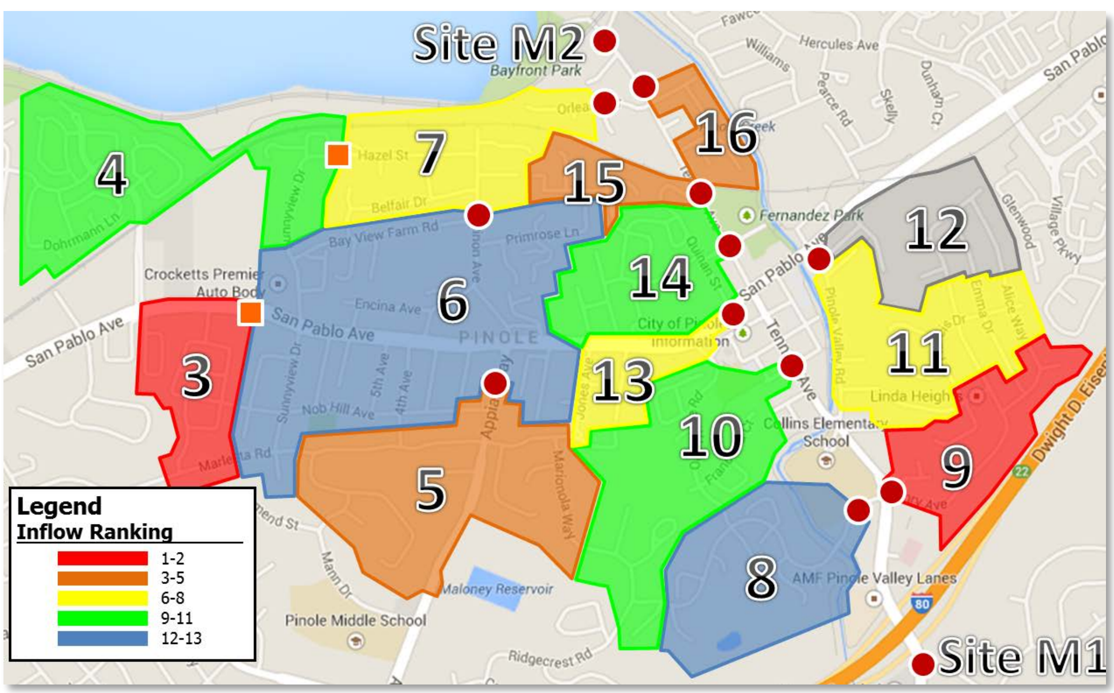


Figure 5-2. Phase 1 Inflow Temperature Map

5.3 Combined I/I Analysis

Combined I/I analysis considers the totalized volume (in gallons) of both inflow and rainfall-dependent infiltration over the course of a storm event. For example, the total volume of infiltration and inflow into Basin 3 for the February 7 - 12, 2014 storm event calculated out to 670,000 gallons (hatched area below in Figure 5-3).

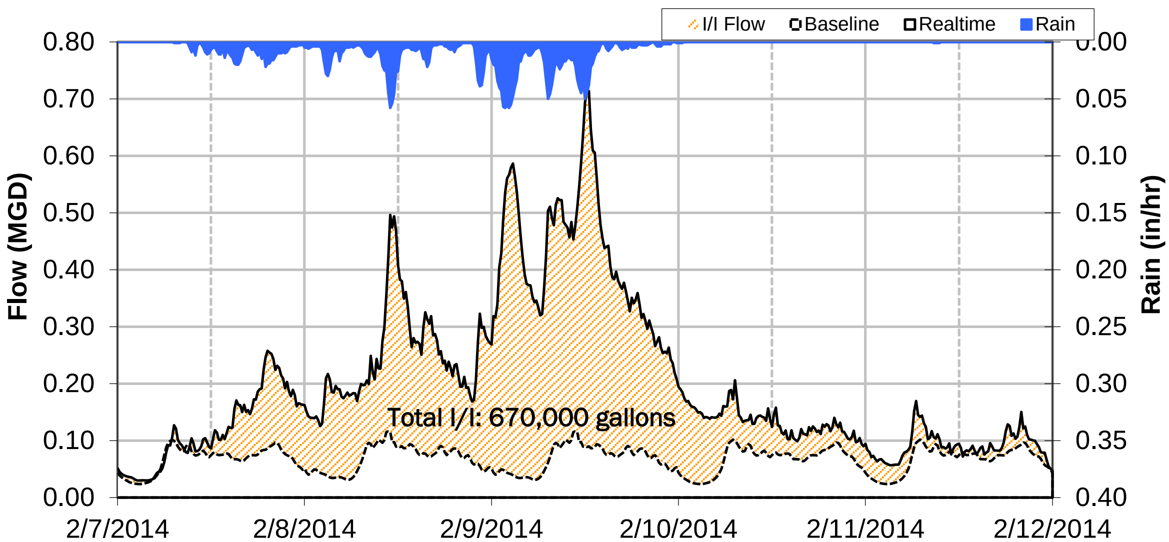


Figure 5-3. I/I Isolation Curve, Basin 3 (Combined I/I Measurement)

Table 5-2 summarizes the combined I/I analysis results for the large basins. Figure 5-4 illustrates a temperature map of the basin rankings for combined I/I.

Table 5-2. Combined I/I Analysis Summary, Large Basins

Basin	ADWF (mgd)	Total I/I (gallons)	Total I/I per IDM	R-Value (per Acre)	Total I/I per ADWF	Combined I/I Ranking
Basin 3	0.067	670,000	109,000	19.0%	3.07	5
Basin 4	0.129	355,000	8,000	6.9%	0.85	11
Basin 5	0.019	570,000	17,000	9.1%	9.17	4
Basin 6	0.090	560,000	27,000	4.9%	1.90	9
Basin 7	0.058	200,000	5,000	5.5%	1.05	10
Basin 8	0.007	1,000	0	0.0%	0.05	13
Basin 9	0.004	76,000	5,000	3.1%	5.27	7
Basin 10	0.085	291,000	8,000	5.3%	1.05	12
Basin 11	0.017	594,000	17,000	12.9%	10.51	1
Basin 13	0.056	214,000	21,000	14.2%	1.18	8
Basin 14	0.042	412,000	20,000	12.6%	3.02	6
Basin 15	0.015	205,000	17,000	14.5%	4.17	3
Basin 16	0.086	858,000	55,000	80.9%	3.08	2

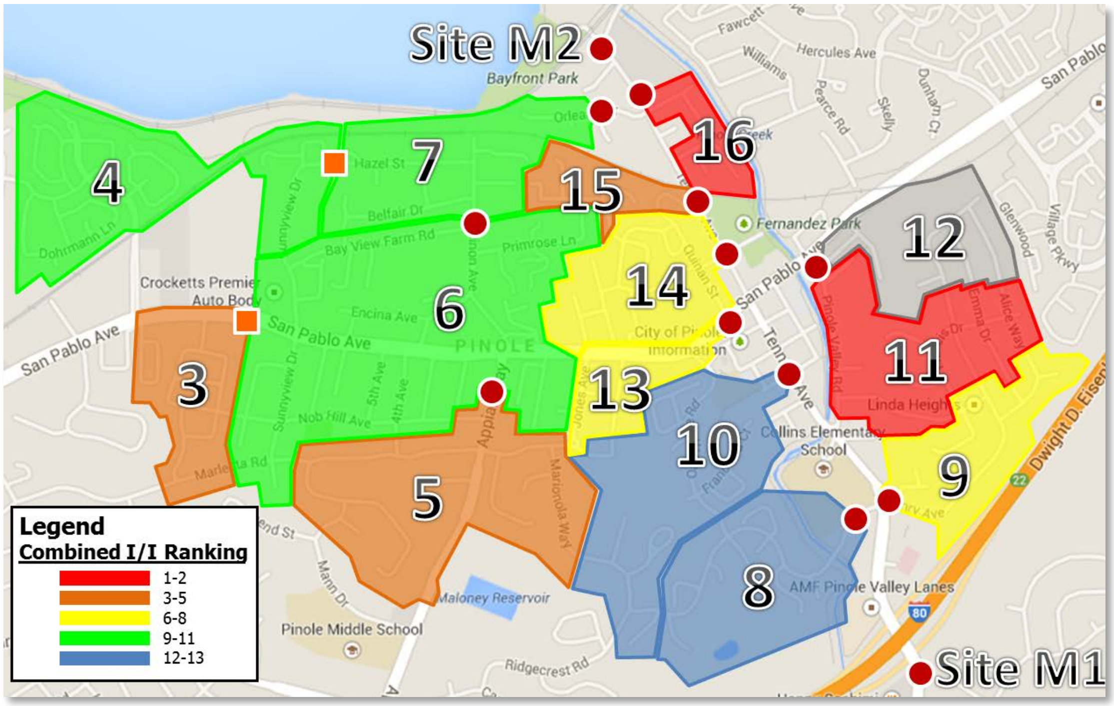


Figure 5-4. Phase 1 Combined I/I Temperature Map

5.4 Phase 1 Summary

On February 20, 2014, V&A met with the City to discuss the Phase 1 findings. At the time of the discussion, peak I/I to ADWF ratios and total combined I/I contribution were main topics of discussion, presented in the format of I/I response curves per each monitoring site (similar to Figure 5-1). The following bullet items highlight important topics of discussion between V&A and the City:

- **Basin 3:** The City cited historical data and field observations regarding known I/I issues within Basin 3.
- **Basin 5:** V&A recommended investigating Basin 5 due to the high peak I/I ratios and high combined I/I totals.
- **Basin 6:** There is a known capacity issue one manhole upstream from the Basin 6 monitoring location.
- **Basin 8:** This is a newer area of the City and this area had minimal I/I contribution.
- **Basin 9:** The flow meter was not in an ideal location for monitoring. The consensus was that the response for Basin 9 was real and considerable but the magnitude may not be correct due to metering conditions.
- **Basin 11:** This location had significant peak I/I rates and combined I/I totals.

- **Basin 12:** The metering manhole for this basin surcharged with evidence that the surcharge was close to an SSO.
- **Basins 15/16:** Both basins were noted for generally high I/I rates and total I/I contribution.
- **Old Henry Road:** The City cited the age of the sewer system along Old Henry Road and noted this correlation to the high I/I rates within Basins 9, 11 and 12.

5.4.1 Conclusions

Final group conclusions were as follows:

- Focus the Phase 2 monitoring within Basins 3, 5 and 6.
  - The focus of the study moving forward would be to spend the remainder of the 2014 wet weather season identifying smaller high I/I mini-basins within this region for possible future CIP work.
  - Identifying areas of I/I reduction may help to solve two problems within the City: (1) severe flows observed in Basin 3 (San Pablo Pump Station), and (2) capacity issues observed at the manhole at the intersection of Pinon Avenue and Bay View Farm Road.
- Make note of the 'Old Henry Road' Basins as future candidates for a focused flow monitoring and I/I study.

5.4.2 Actions

Future actions for Phase 2 included the following:

- Remove ten flow meters from the following sites: M1, M5, M8 – M16.
- Utilize the ten available flow meters to further analyze and gather data on the high I/I basins as follows:
  - Basin 3: Sub-divide into two basins (3.1 and 3.2) and perform Phase 2 flow monitoring.
  - Basin 5: Sub-divide into three basins (5.1, 5.2 and 5.3) and perform Phase 2 flow monitoring.
  - Basin 6: Sub-divide into five basins (6.1, 6.2, 6.3, 6.4 and 6.5) and perform Phase 2 flow monitoring.

6.0 PHASE 2 I/I RESULTS

6.1 Preface

The Phase 2 flow monitoring occurred from February 26, 2014 through March 6, 2014. The ten flow monitoring sub-basins designated for the Phase 2 analysis are illustrated in Figure 1-5. The following items are noted regarding the early-season analysis:

- Results are presented on a basin-by-basin basis (not a site-by-site basis).
- Results are for the ten Phase 2 basins only.
- Results presented to the City during Phase 2 were considered preliminary and based upon the information known at the time of presentation.
- Results presented in the following pages of analyses have been updated from initial presentation to reflect final data and results. The values presented in this report will differ from the initial presentation; however, the conclusions and recommendations are the same.

6.2 I/I Summary of Results

V&A performed flow monitoring and I/I analysis similar to Phase 1. Figure 6-1 shows the I/I response curve for Basin 3.1.

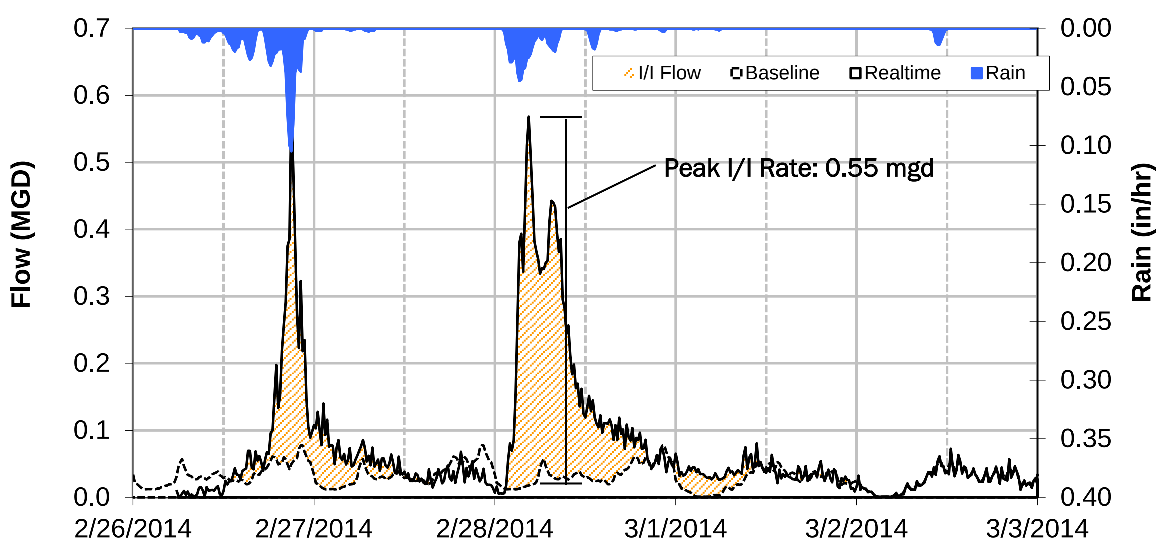


Figure 6-1. Peak I/I Rate, Basin 3.1 (Event 2)

Table 6-1 summarizes the peak measured I/I flows and inflow analysis results for the Phase 2 basins for Storm Event 2. Basins that ranked in the top 5 have been color coded red. Figure 6-2 illustrates a temperature map summary of the inflow analysis results per basin.

Table 6-1. Phase 2 Inflow Analysis Summary

Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Peak I/I per IDM (gpd/IDM)	Peak I/I per ACRE (gpd/AC)	Peak I/I per ADWF Ratio	Inflow Ranking
Basin 3.1	0.032	0.55	265,700	34,400	17.3	2
Basin 3.2	0.011	0.06	52,600	10,000	5.7	5
Basin 5.1	0.023	0.12	35,900	5,200	5.2	6
Basin 5.2	0.011	0.23	171,600	25,600	20.9	1
Basin 5.3	0.022	0.08	24,600	2,800	3.6	8
Basin 6.1	0.011	0.10	41,500	7,700	8.9	4
Basin 6.2	0.016	0.03	9,000	2,300	1.9	9
Basin 6.3	0.079	0.04	14,900	1,000	0.5	10
Basin 6.4	0.008	0.03	28,000	2,300	3.7	7
Basin 6.5	0.009	0.11	31,400	6,500	12.6	3

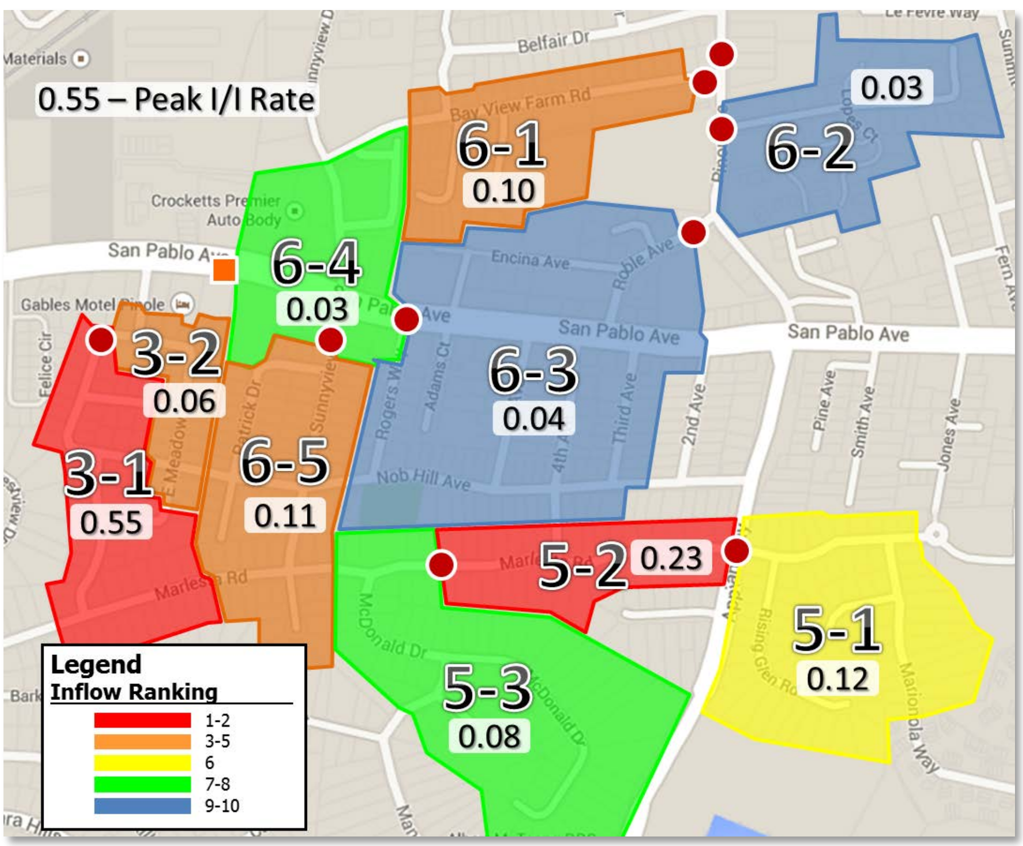


Figure 6-2. Phase 2 Inflow Analysis Temperature Map

6.3 Phase 2 Summary

On March 18, 2014, V&A met with the City to discuss the Phase 2 findings. At the time of the discussion, the measured peak I/I rates were the main topics of discussion, presented in the format of I/I response curves per each monitoring site. The following bullet items highlight important topics of discussion between V&A and the City:

- **Basin 3.1/Basin 3.2:** Basin 3.1 had significantly more peak I/I than Basin 3.2. Basin 3.1 had 0.55 mgd of peak I/I flow occurring within only approximately 2,700 lineal feet of pipe.
    - Rehabilitating the 2,700 lineal feet of pipe is a realistic CIP project that can have significant impact at the treatment plant. For Storm Event 2, at the treatment plant, there was approximately 2.47 mgd of peak I/I. Though not a perfect comparison due to attenuation and holding times within the collection system, 0.55 mgd peak inflow from Basin 3.1 is approximately 22% of the 2.47 mgd peak inflow measured from the City of Pinole for Storm Event 2.
- **Basin 5.2** and **Basin 6.5** also had high peak I/I rates.
- The City was interested in obtaining the percentage volume of flow contribution at the intersection of Roble Avenue and Appian Way where two main sewers combine.
- **Basin 6.3:** The City was interested in directly monitoring flows in the numbered streets if possible (2nd Avenue to 5th Avenue).

6.3.1 Recommendations

Final group recommendations were as follows:

- Focus the Phase 3 monitoring within Basins 3.1, 5.2 and 6.5 for focused I/I analysis.
- Monitor the Roble/Appian junction and capture the sewerage basins for Third Avenue and 5th Avenue.

6.3.2 Actions

- Remove seven flow meters from the following sites: M3.2, M5.1, M6.1, M6.2, M6.4, M7 and M13.
- Utilize the seven available flow meters plus one extra meter to further analyze and gather data on the high I/I basins as follows:
    - **Basin 3.1:** Sub-divide into three basins (3.1, 3.1A, 3.1B) for Phase 3 flow monitoring.
    - **Basin 5.2:** Sub-divide into three basins (5.2, 5.2A and 5-3) for Phase 3 flow monitoring.
    - **Basin 6.3:** Sub-divide into three basins (6.3, 6.3A and 6.3B) for Phase 3 flow monitoring.
    - **Basin 6.5:** Sub-divide into three basins (6.5, 6.5A and 6.5B) for Phase 3 flow monitoring.

7.0 PHASE 3 I/I RESULTS

7.1 Preface

The 13 flow monitoring sub-basins designated after the Phase 2 analysis are illustrated in Figure 1-6. The following items are noted regarding the early-season analysis:

- Results are presented on a basin-by-basin basis (not a site-by-site basis).
- Results are for the 13 Phase 3 basins only.
- Results presented to the City during Phase 3 were considered preliminary and based upon the information known at the time of presentation.
- Results presented in the following pages of analyses have been updated from the initial presentation to reflect final data and results. The results may differ from the initial presentation; however, the conclusions and recommendations are the same.

7.2 I/I Summary of Results

V&A performed flow monitoring and I/I analysis similar to Phase 1 and Phase 2. Figure 7-1 shows the I/I response curve for Basin 3.1A

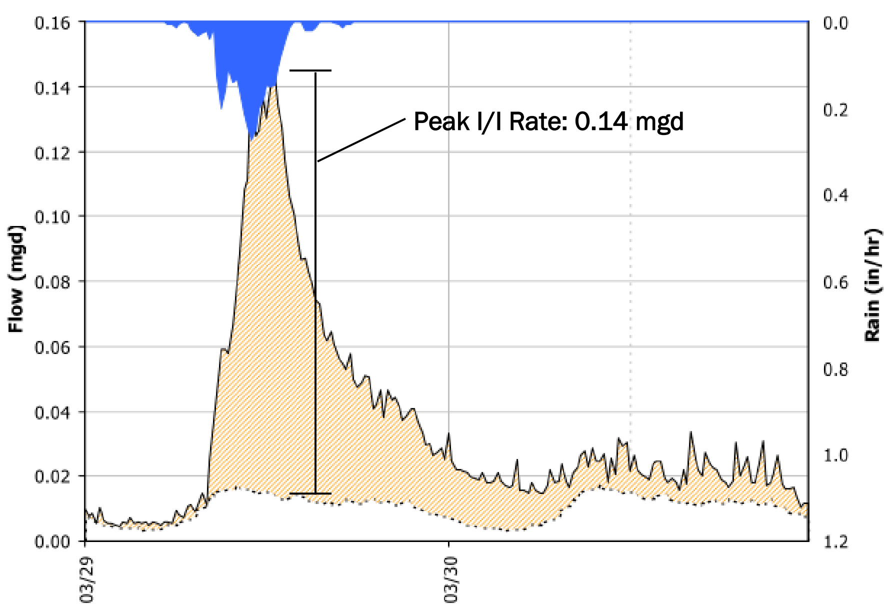


Figure 7-1. Peak I/I Rate, Basin 3.1A (Event 3)

Table 7-1 summarizes the peak measured I/I flows and inflow analysis results for the Phase 3 basins for Storm Event 3. Basins that ranked in the top 5 have been color coded red. Figure 7-2 illustrates a temperature map summary of the inflow analysis results per basin.

Table 7-1. Phase 3 Inflow Analysis Summary

Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Peak I/I per IDM (gpd/IDM)	Peak I/I per ACRE (gpd/AC)	Peak I/I per ADWF Ratio	Inflow Ranking
Basin M3.1	0.0036	0.168	243,600	54,200	46.6	3
Basin M3.1A	0.0038	0.044	66,000	18,400	11.7	7
Basin M3.1B	0.0057	0.087	47,700	8,600	15.2	8
Basin M5.2	0.0025	0.220	292,700	61,000	89.3	1
Basin M5.2A	0.0016	0.000	0	0	0	13
Basin M5.3	0.0074	0.115	33,500	4,000	15.6	10
Basin M6.0A	0.0459	0.703	69,300	12,700	15.3	6
Basin M6.3	0.0146	0.283	52,500	7,800	19.4	5
Basin M6.3A	0.0049	0.295	304,100	38,800	60.0	2
Basin M6.3B	0.0036	0.052	37,600	8,800	14.5	9
Basin M6.5	0.0016	0.068	80,600	20,100	41.8	4
Basin M6.5A	0.0029	0.028	20,400	4,300	9.4	12
Basin M6.5B	0.0033	0.034	25,800	4,700	10.4	11

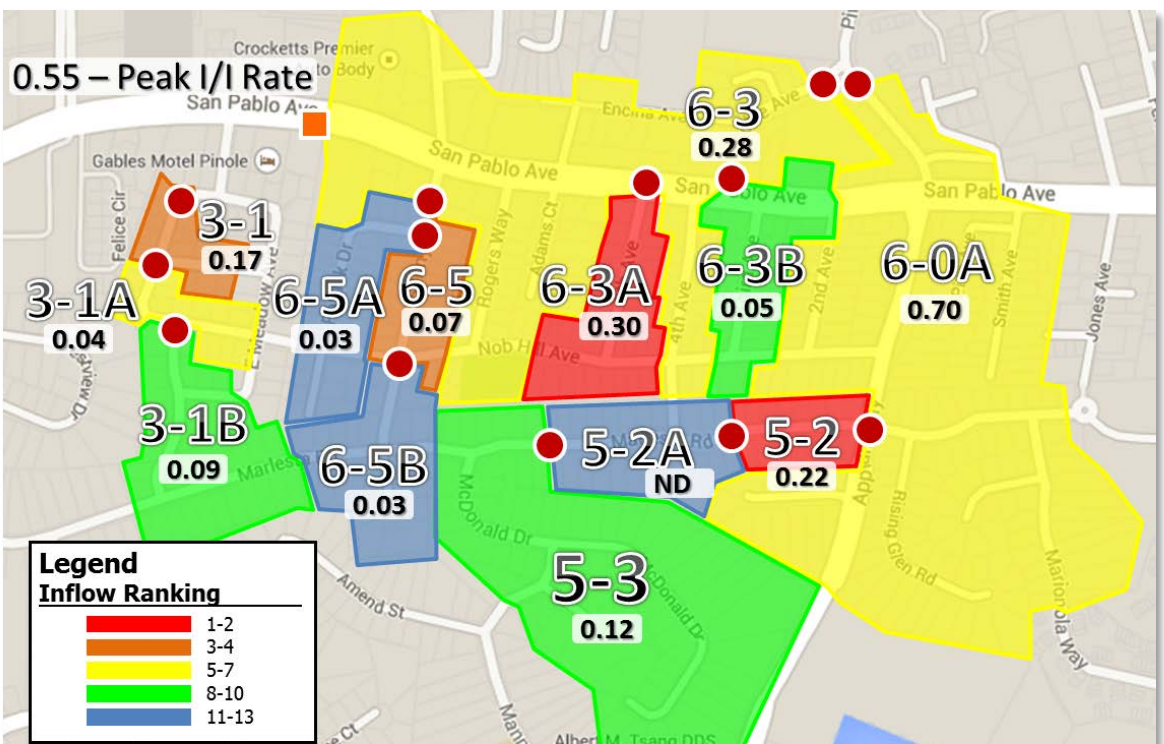


Figure 7-2. Phase 3 Inflow Analysis Temperature Map

7.3 Phase 3 Summary

On April 15, 2014, V&A met with the City to discuss the Phase 3 findings. The following bullet items highlight important topics of discussion between V&A and the City:

- **Basin 3.1:** Basin 3.1 had previously been broken into three parts in order to refine the potential location for the high rates of inflow.
  - The high rates of inflow were once more confirmed in Basin 3.1. Basin 3.1 had the highest rate of inflow amongst the three Basin 3.1 meters (3.1, 3.1A and 3.1B)
- **Basin 5.2:** Previously, Basin 5.2 had very high rates of inflow for a relatively small service area. This basin was divided approximately in half to further refine the problem area.
  - Of the two sub-basins (5.2 and 5.2A), Basin 5.2 had the vast majority of I/I.
- **Basin 6.5:** During Phase 2, Basin 6.5 was one of the two highest ranked basins. This was divided into three sub-basins (6.5, 6.5A and 6.5B).
  - Basin 6.5 had the highest rate of inflow
- **Basin 6.3A:** The rate of inflow observed in Basin 6.3A was higher than expected.

8.0 RECOMMENDATIONS

V&A advises that future I/I reduction plans consider the following recommendations:

1. **Potential CIP Projects for I/I Mitigation and Reduction**
   a. The City should conduct I/I mitigation and reduction measures in the following mini-basins:
      i. Basin 3-1
      ii. Basin 3-1A
      iii. Basin 5-2
      iv. Basin 6-3A
      v. Basin 6-5

   b. For I/I reduction, V&A recommends rehabilitation of the sewer mains, laterals and side sewers.
      i. The most comprehensive study on the percent of I/I reduction has been conducted by King County, *Initial Infiltration and Inflow Reduction Project Alternatives Analysis Report*. This study confirmed the popular theory that over 50% of infiltration and inflow enters from private lateral connections. The report also makes the following recommendations for I/I mitigation:
         (a) CCTV work is best performed during a rainfall event after groundwater levels have begun to rise, allowing visual confirmation of specific I/I entry points, including determining the source of potential lateral I/I source. A generally consistent deficiency was observed with regards to the joint conditions in the laterals and side sewers.
         (b) Rehabilitation of sewer mains, manholes, laterals and side sewers results in approximately 80% reduction of I/I.
2. **Future I/I Identification – Continued Sub-Basin Flow Monitoring and I/I Analysis:** V&A recommends that the City continue to locate and mitigate potential sources of I/I. Already identified as known contributing sub-basins with high volumes of I/I are Basins 9, 11, 12, 14 and 15. It is possible that a study similar to this study may identify CIP projects that can significantly reduce the overall I/I within the City collection system.
3. **Other I/I Investigation Methods:** Potential other I/I investigation methods include the following:
   a. Smoke testing
   b. Night-time reconnaissance work to (1) investigate and determine direct point sources of inflow, and (2) determine the areas and pipe reaches responsible for high levels of infiltration contribution.
4. **I/I Reduction Cost Effectiveness Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow/infiltration and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional rainfall dependent I/I flow.

Appendix A. Additional Analysis Request: Flow Split between M6.0A and M6.3

A.1 Flow Split between M6.0A and M6

As a part of Phase 3, the City asked that the flow split between Sites M6.0A and M6.3 be monitored. Site M6.0A monitored a 10-inch line running southwest along Pinon Ave that gathers flow from Basin 5 (Phase 1) as well as the area near the intersection of Appian Way and San Pablo Ave. Site M6.3 monitors an 8-inch line that gathers flow from Basin 3, Basin 6.5, Basin 6.4 and Basin 6.3. The flows from these two sites eventually travel through Site M6.

The average and peak flow splits for the two sites are illustrated in Figure A-1 and Figure A-2.

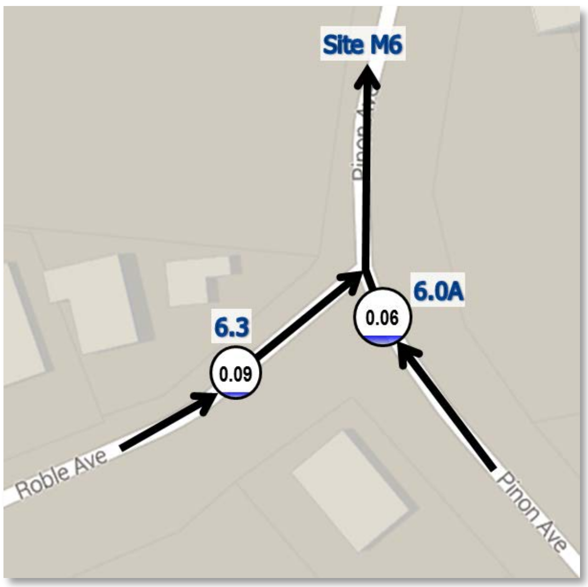


Figure A-1. ADWF Split for Site M6.3 and M6.0A

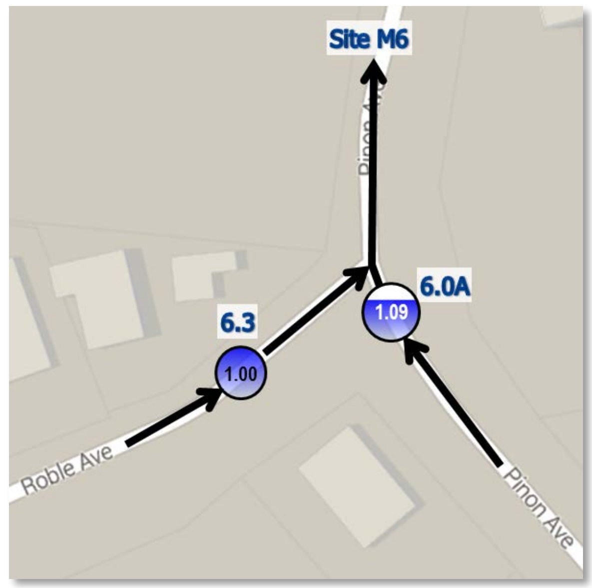


Figure A-2. Peak Flow Split for Site M6.3 and M6.0A

The split in flows between the two lines was approximately 40% for Site M6.0A and 60% for Site M6.3 for Average Dry Weather Flow.

The split in flows between the two lines was approximately 52% for Site M6.0A and 48% for Site M6.3 for Peak Flow during Storm Event 3.

Appendix B. Additional Analysis Request: Treatment Plant Inflow Contribution

B.1 Treatment Plant Inflow Contribution

The City of Pinole shares ownership of the Wastewater Treatment Plant with the City of Hercules. One aspect that is currently not addressed between the cities is the cost of treating the rainfall that falls on the footprint of the treatment plant itself.

The footprint of the treatment plant (Figure B-3) is approximately 228,300 ft^2 (5.28 acres). This equates to 142,300 gallons per inch of rain that falls. The City of Pinole averages approximately 20.25 inches of rain per year. With the drainage on the treatment plant property, the majority of this rainfall would flow into the treatment process. In total, this results in approximately 2,880,000 gallons of rainfall that is treated by the treatment facility each year.

Figure B-3. Footprint of City of Pinole Wastewater Treatment Plant

Appendix C. Additional Analysis Request: Comparison of Sanitary Sewer Flows from City of Pinole, City of Hercules, and the Pinole/ Hercules Wastewater Treatment Plant

This request was processed and submitted as a separate Technical Memorandum, but included within Appendix C as follows.

TECHNICAL MEMORANDUM

COMPARISON OF SANITARY SEWER FLOWS FROM CITY OF PINOLE, CITY OF HERCULES, AND THE PINOLE/HERCULES WASTEWATER TREATMENT PLANT

Prepared for: Dean Allison, P.E., City of Pinole

Prepared by: Kevin Krajewski, P.E., V&A Consulting Engineers

Reviewed by: Oliver Pohl, P.E., V&A Consulting Engineers
Glenn Willson, P.E., V&A Consulting Engineers

V&A

Date: February 23, 2015

V&A Project No.: 13-0276

1.0 INTRODUCTION

The Pinole/Hercules Wastewater Treatment Plant treats sanitary sewage flow from both the City of Pinole and the City of Hercules. Presently, the Hercules flows are measured directly through a Parshall flume located at the treatment plant. Pinole flows are measured using two methods:

- **Method 1:** Indirect measurement by subtracting the Hercules flows from the totalized treatment plant effluent flows, monitored at an effluent weir structure. This method has traditionally been the primary method for determining flows from the City of Pinole.
- **Method 2:** Direct measurement using a Hach FloDar flow meter located on the 30-inch line on Tennant Avenue as it enters into the treatment facility. This 30-inch line captures the entirety of the sanitary sewer waste from the City of Pinole collection system.

V&A Consulting Engineers (V&A) was retained by the City of Pinole (City) to compare both methods of flow calculation to temporary flow monitoring conducted on the 30-inch line on Tennent Avenue, just east of the railroad tracks, approximately 240 feet upstream from the City Influent Meter. The 30-inch line captures the entirety of the sanitary sewer waste from the City of Pinole collection system.

Flows from the Influent, Hercules and Effluent meters were provided by the City. V&A was provided two different types of data sets from the influent (FloDar) meter:

- **Data Set 1:** The City provided 15-minute interval data for the Influent flow meter from November 1 through December 10, 2014.
- **Data Set 2:** V&A accessed the flow meter directly and was able to retrieve five days of 1-minute interval data from January 18 to 23, 2015.

These data sets were analyzed separately and for different purposes that will be outlined later in this report.

V&A was initially retained by the City to perform sanitary sewer flow monitoring during the 2013/2014 wet weather season as part of the City's efforts to reduce inflow and infiltration (I/I) within the City collection system. Through the 2014 summer months, V&A maintained the flow meter that was already installed on Tennent Avenue. This work was performed under the same contract as the 2013/2014 Flow Monitoring and I/I Analysis work.

Figure 1-1 illustrates a map of the treatment plant and the Pinole and Hercules trunk sewer lines.

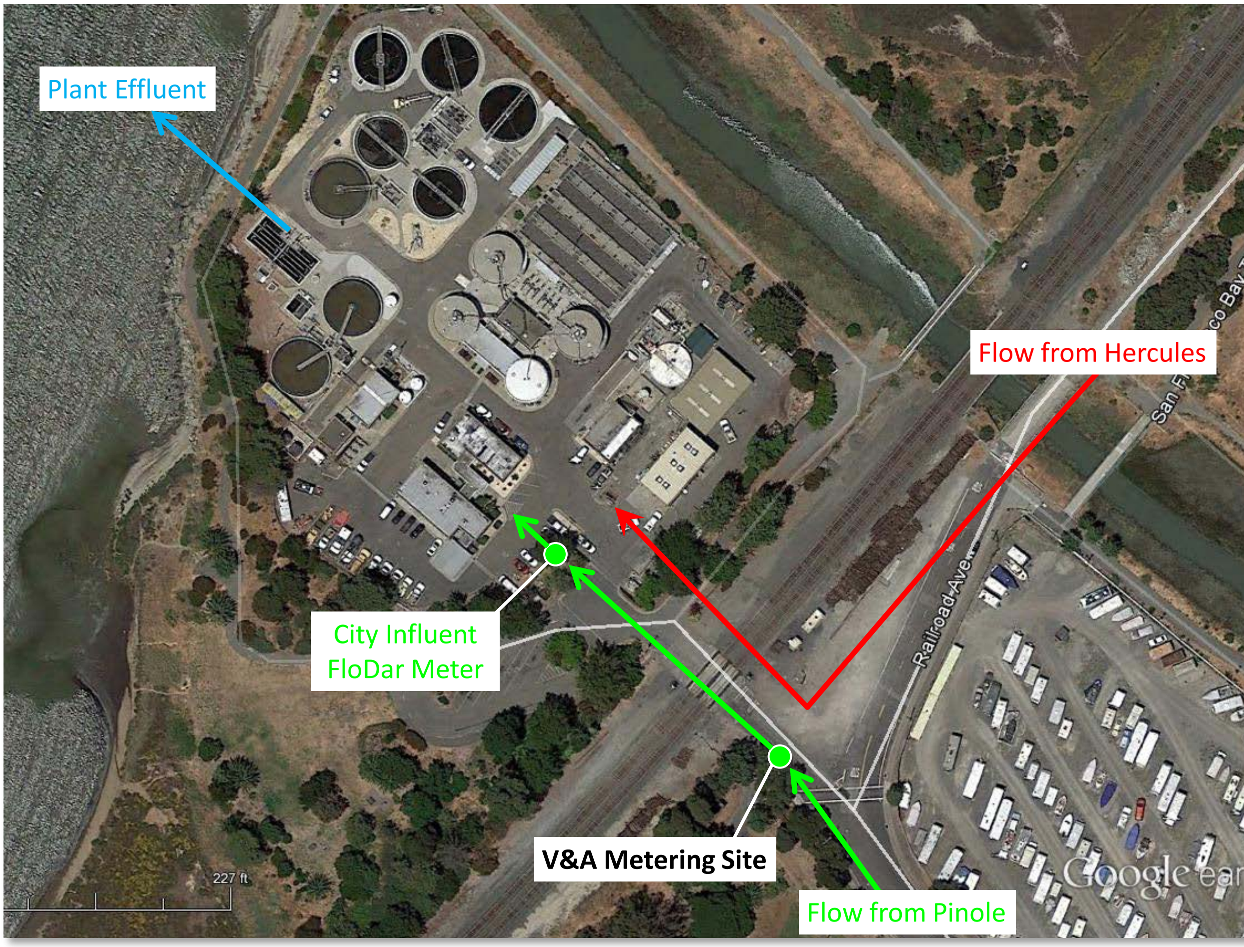


Figure 1-1. Map of Pinole/Hercules Treatment Plant, Contributing Flows

2.0 METHOD 1 ANALYSIS (EFFLUENT LESS HERCULES)

2.1 Review of Validity of City Data for Analysis

The City provided 15-minute interval data for the Hercules Parshall flume and for the treatment plant effluent from June 14, 2014 to September 25, 2014. V&A performed a cursory QA/QC on the data provided. The data from June 14 through July 20, 2014 appeared to have been reported correctly and is considered as valid data sets for comparison. From July 21, 2014 forward, there were only a few valid data points for each data set per day, resulting in several “flat-lines” or repeated values in the data sets, illustrated in Figure 2-1.

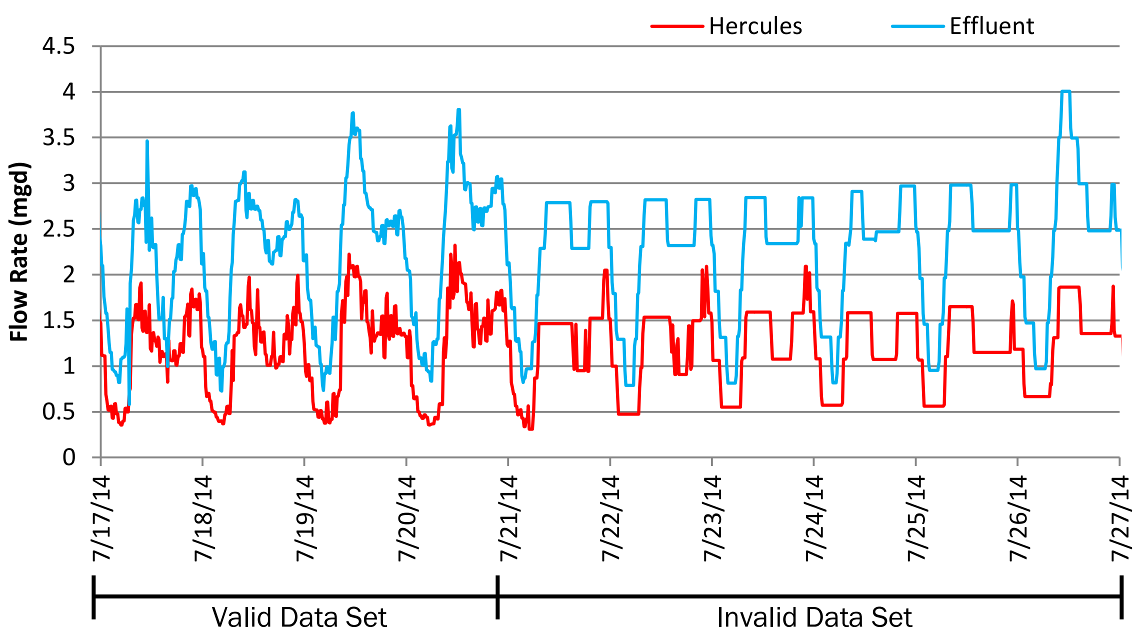


Figure 2-1. Hercules and Effluent Flow Data, 7/17/2014 – 7/27/2014

For Method 1 Flows, the data from July 21 forward was not considered valid for comparison for the purposes of this study.

2.2 Micro Analysis

V&A first looked at the data at the 15-minute data interval level. Though the City does not report totalized flows at this interval level; however, analyzing the data on a micro level may lend confidence or provide valuable information as to the operational viability of the metering methods currently in use. Figure 2-2 shows a graph of the flow monitoring data sets evaluated for this study. The purple line labeled “Pinole+Hercules” is simply a sum of the directly monitored values for Pinole and Hercules and is intended to be shown as a direct comparison to the Effluent data.

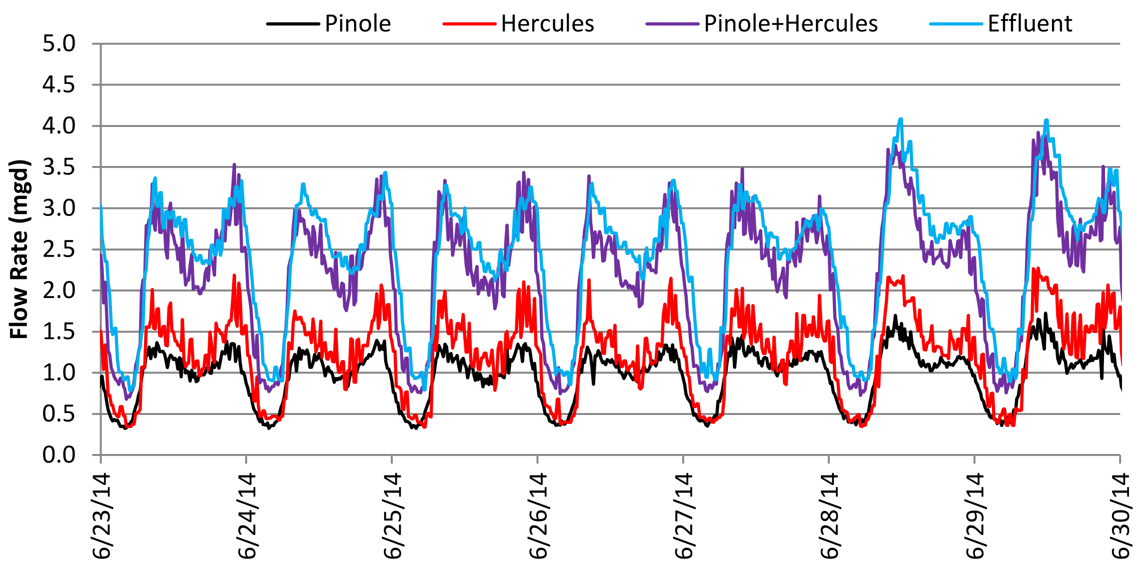


Figure 2-2. Hercules and Effluent Flow Data, 6/23/2014 – 6/30/2014

It is noted that the Effluent flows undergo some degree of attenuation; there is detention time in between the influent and effluent stages of the treatment process. The following items are noted: analysis:

- The Effluent flow data and the sum of the directly monitored data sets (“Pinole + Hercules”) match each other well in terms of magnitude and range.
- When operating correctly, the diurnal curves and trends of the flow data from the Hercules Parshall flume and the Effluent Weir appear to provide solid and repeatable data.

2.3 Macro Analysis

Flow data was analyzed on a day-by-day basis to determine the estimated difference in the calculated flow for Pinole (present method -- Effluent less Hercules) versus the measured flows per the V&A flow monitoring conducted on the 30-inch trunk sewer on Tennent Avenue. Table 2-1 shows the daily flow comparison of the daily calculated flows versus the measured City flows.

Table 2-1. Comparison of Average Daily Flow Rates (Method 1)

Date	Effluent (mgd)	Hercules (mgd)	Method 1 Pinole Flows (Effl – Herc) (mgd)	Pinole Measured (mgd)	% Difference
6/18/2014	2.32	1.17	1.15	0.97	19%
6/19/2014	2.34	1.18	1.16	0.97	19%
6/20/2014	2.39	1.18	1.21	0.98	22%
6/21/2014	2.49	1.30	1.19	0.99	20%
6/22/2014	2.50	1.34	1.16	1.01	14%
6/23/2014	2.37	1.21	1.16	0.96	20%
6/24/2014	2.36	1.19	1.18	0.94	25%
6/25/2014	2.32	1.20	1.12	0.94	19%
6/26/2014	2.33	1.15	1.19	0.94	26%
6/27/2014	2.40	1.20	1.21	1.00	21%
6/28/2014	2.51	1.26	1.25	1.01	24%
6/29/2014	2.59	1.34	1.25	1.02	22%
6/30/2014	2.42	1.21	1.21	0.97	24%
7/1/2014	2.32	1.15	1.17	0.95	23%
7/2/2014	2.30	1.15	1.15	0.97	19%
7/3/2014	2.28	1.16	1.12	0.96	17%
7/4/2014	2.37	1.23	1.15	0.97	18%
7/5/2014	2.32	1.20	1.12	0.99	12%
7/6/2014	2.40	1.26	1.14	1.04	10%
7/7/2014	2.29	1.18	1.11	1.03	8%
7/8/2014	2.27	1.15	1.12	1.02	9%
7/9/2014	2.28	1.18	1.10	1.00	10%
7/10/2014	2.20	1.15	1.05	0.98	7%
7/11/2014	2.24	1.12	1.12	1.00	13%
7/12/2014	2.39	1.27	1.12	1.02	9%
7/13/2014	2.44	1.29	1.15	1.05	10%
7/14/2014	2.33	1.21	1.12	1.03	8%
7/15/2014	2.35	1.18	1.17	0.99	18%
7/16/2014	2.41	1.18	1.22	1.01	21%
7/17/2014	2.02	1.15	0.87	0.98	-11%
7/18/2014	2.22	1.13	1.09	1.00	9%
7/19/2014	2.31	1.24	1.07	1.00	7%
7/20/2014	2.37	1.26	1.11	1.04	7%
Average:	**2.35**	**1.20**	**1.14**	**0.99**	**15.2%**

Note: V&A has no knowledge of the accuracy of the flow data from Hercules and the plant. It is also noted that the industry standard for open-channel flow monitoring is expected to have an accuracy of approximately ±5%.

Method 1 Flow Measurement Summary

Using the Method 1 determination of the City of Pinole flows, the flows for Pinole are over-reported by approximately 15.2%.

3.0 METHOD 2, DATA SET 1

3.1 Review of Validity of City Data for Analysis

Similar to the data sets provided for the Hercules and Effluent flows of the previous section, the data set had many repeated values and flat-lines, illustrated in Figure 3-1.

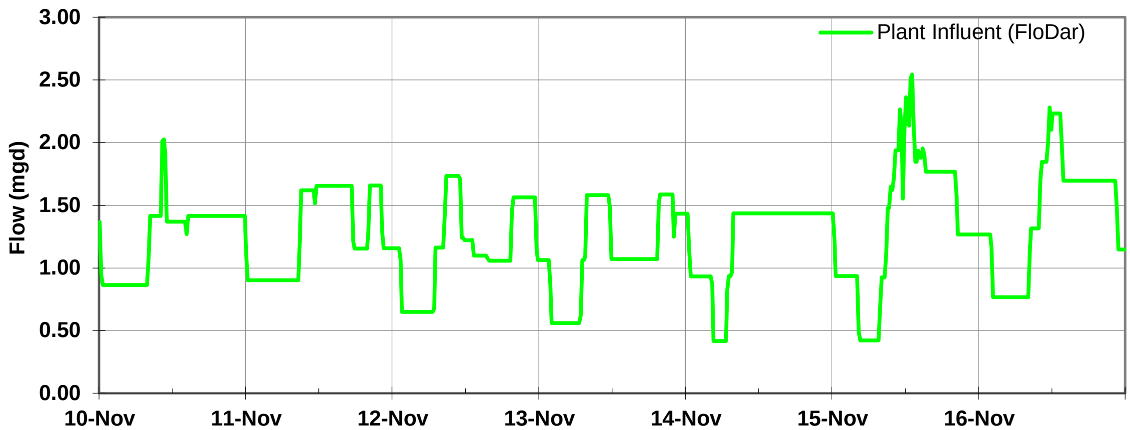


Figure 3-1. Pinole Influent Data, 11/10/2014 – 11/17/2014

Given that the repeated values occurred with all data sets, and occurred for the same date/time stamps for concurrent data sets of different meters, it is believed the repeated values are not an indication of meter failure, but an indication of data storage error within the City SCADA system.

For the purposes of the Method 2, Data Set 1 analysis, V&A assumed that non-repeated data points were valid at the time of the date/time stamp, but that repeated values were not valid data points. Figure 3-2 illustrates an example of valid data points for analysis and direct comparison to the V&A temporary flow meter.

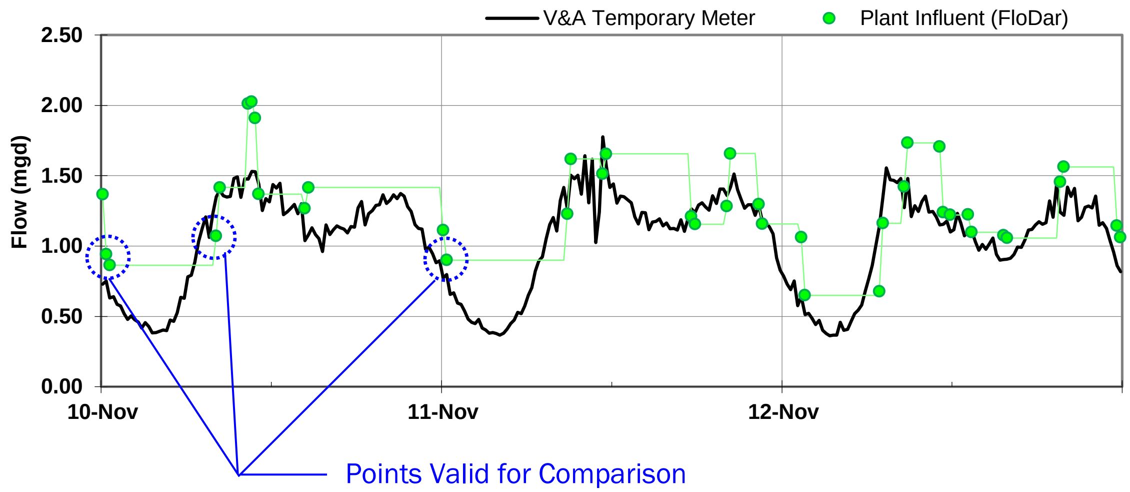
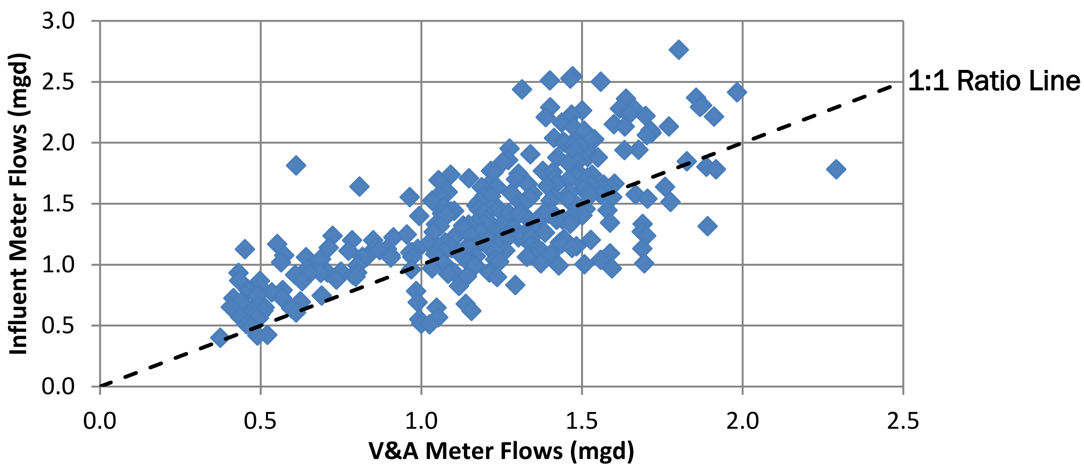


Figure 3-2. Pinole Influent Data, 11/10/2014 – 11/13/2014, Example of Valid Points

3.2 Method 2, Data Set 1 Analysis

From Data Set 1 (November 1 through December 10, 2014), there were 316 valid data points from the plant influent meter that could be directly compared to the V&A temporary flow meter. Figure 3-3 shows a scatter plot comparison of the Influent Meter to the V&A Meter.



Average of Valid Influent Meter Flow Values: 1.361 mgd
Average of Like V&A Meter Flow Values: 1.195 mgd
Percent Difference: 13.9%

Figure 3-3. Pinole Influent Data Comparison of All Valid Points

<u>Method 2 Flow Measurement Summary (Data Set 1)</u>
Using the Method 2 determination of the City of Pinole flows, the flows for Pinole are over-reported by approximately 13.9%.

4.0 METHOD 2, DATA SET 2

Data Set 2 was 1-minute interval data accessed directly from the flow meter and thus did not have the “repeated-value” issue noted in previous sections. This data set was utilized to better analyze the sensor measurements of the FloDar flow meter and perhaps lend information to understand why the values in Data Set 1 were over-reported.

4.1 Method 2, Data Set 2 Analysis

Figure 4-1 illustrates hydrographs of the 1-minute level, velocity and flow data measured by the FloDar meter from January 18 through 23, 2015.

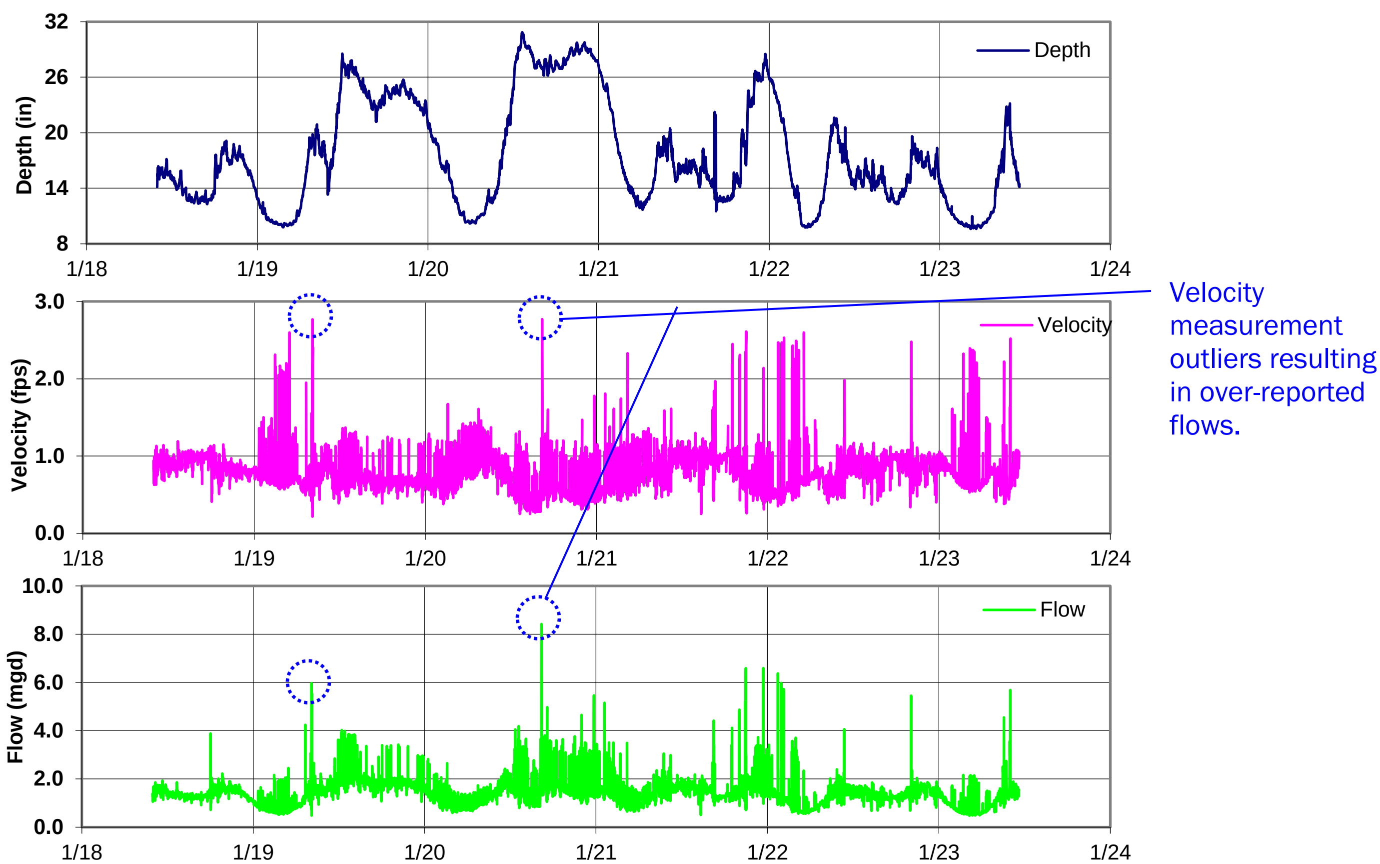
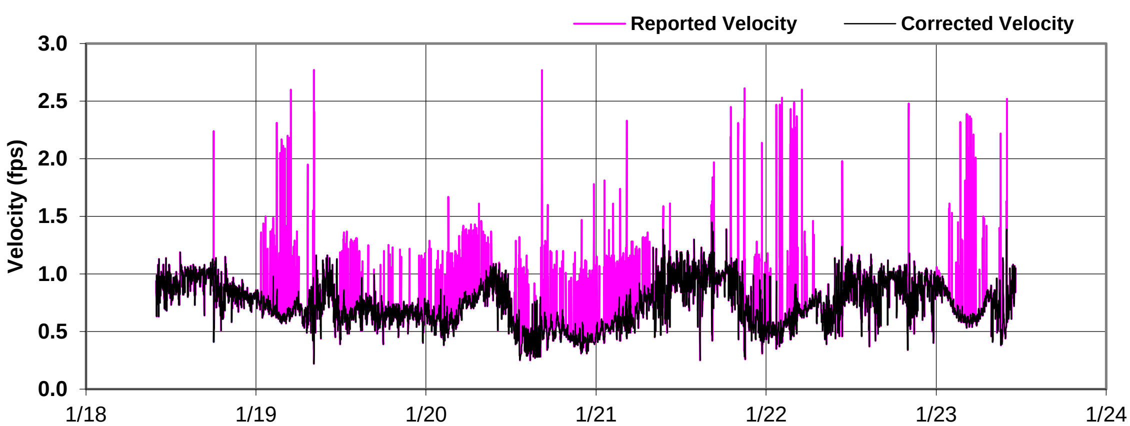


Figure 4-1. Pinole Influent Level, Velocity and Flow Data

Velocity Outliers

There are several velocity point outliers that, based on the hydraulic conditions at the treatment plant, are known to be incorrect and over-reported. This is not an uncommon occurrence with flow monitoring equipment. The FloDar meter measures surface velocity of the flow stream; excessive floating debris or bubbles or foam on the flow surface could cause this issue.

For this data set, V&A went through the exercise of correcting the velocity values to determine the overall effect of these outliers on the flow data. Figure 4-2 illustrates the corrected velocity measurements.



Average of Reported Velocity Measurements: 0.794 fps
Average of Like V&A Meter Flow Values: 0.733 fps
Percent Difference: 9.1%

Figure 4-2. Pinole Influent Velocity Measurements, Reported and Corrected

Flow Calculation and Sediment

The flow calculation programmed into the FloDar meter did not consider the volume of sediment within the pipe channel. The cross-sectional area filled with sediment is not available for conveyance of waste stream flow. It is known that this particular trunk sewer has a sediment issue. Assuming no sediment would result in the over-reporting of flow values.

The exact level of sediment at the Influent Meter location was not known. The sediment at the V&A Meter approximately 240 feet upstream from the Influent Meter manhole measured approximately 2 inches. For Data Set 2, assuming 2 inches of sediment in the pipe channel would result in a 5.7% reduction of flow.

Data Set 2 Analysis Summary

If the velocity outliers were corrected and 2 inches of sediment was accounted for in the pipeline, the overall impact on the flows reported by the FloDar would be approximately 14.8%. This number corresponds well with the percentage difference in flows analyzed in Data Set 1 (13.9%).

5.0 DISCUSSION

The following discussion items are presented for review by the City:

- **Primary Devices vs. Flow Meters:** Primary devices, flumes and weirs, traditionally have been a popular and reliable method to measure flows in permanent installations. The main limitations of these methods is that there are constraints on where such devices can be installed and once installed, moving them to a different location is difficult. Primary devices are less reliant on technology because one of the parameters used to calculate flow is removed from the equation as a variable.
- **Method 1 (Effluent less Hercules):** Using the Method 1 determination of the City of Pinole flows, the flows for Pinole are over-reported by approximately 15.2%.
    - The flow monitoring methods used presently to determine the Hercules flows and the Effluent flows appear to be repeatable.
- **Method 2 (Influent meter direct measurement):** The influent meter was over-reporting flows by approximately 13.9% per comparison of valid data points within Data Set 1 and the V&A temporary meter.
    - Detailed analysis of the 1-minute level and velocity data reported from the FloDar meter indicates that there are likely two causes for this over-reporting:
        - **Cause #1:** There are velocity outliers reported by the FloDar that cause the velocity to be over-reported. For the five days of data analyzed, these outliers caused the flows to be over-reported by 9.1%.
        - **Cause #2:** There are known sediment issues in the pipeline where the influent meter is located. The FloDar meter was not programmed to account for sediment when calculating flow. The exact level of sediment in that manhole was not known at the time of this report. If one assumes 2 inches of sediment as measured 240 feet upstream from the influent meter, then the net effect of accounting for the sediment would be a reduction of flow of approximately 5.7% (Data Set 2).
- **Preferred Method:** Given the known sediment issues at the influent meter location, and also given the repeatability and sustainability of the Primary Device used at the Effluent Meter location, Method 1 would be considered the preferred method for determining the City of Pinole flows generated from the collection system on a mass flow basis.
    - Note: Instantaneous flows from the City of Pinole cannot be determined due to attenuation and hold times through the treatment process.

Appendix D. Flow Monitoring Sites Data, Graphs, Information: Phase 1

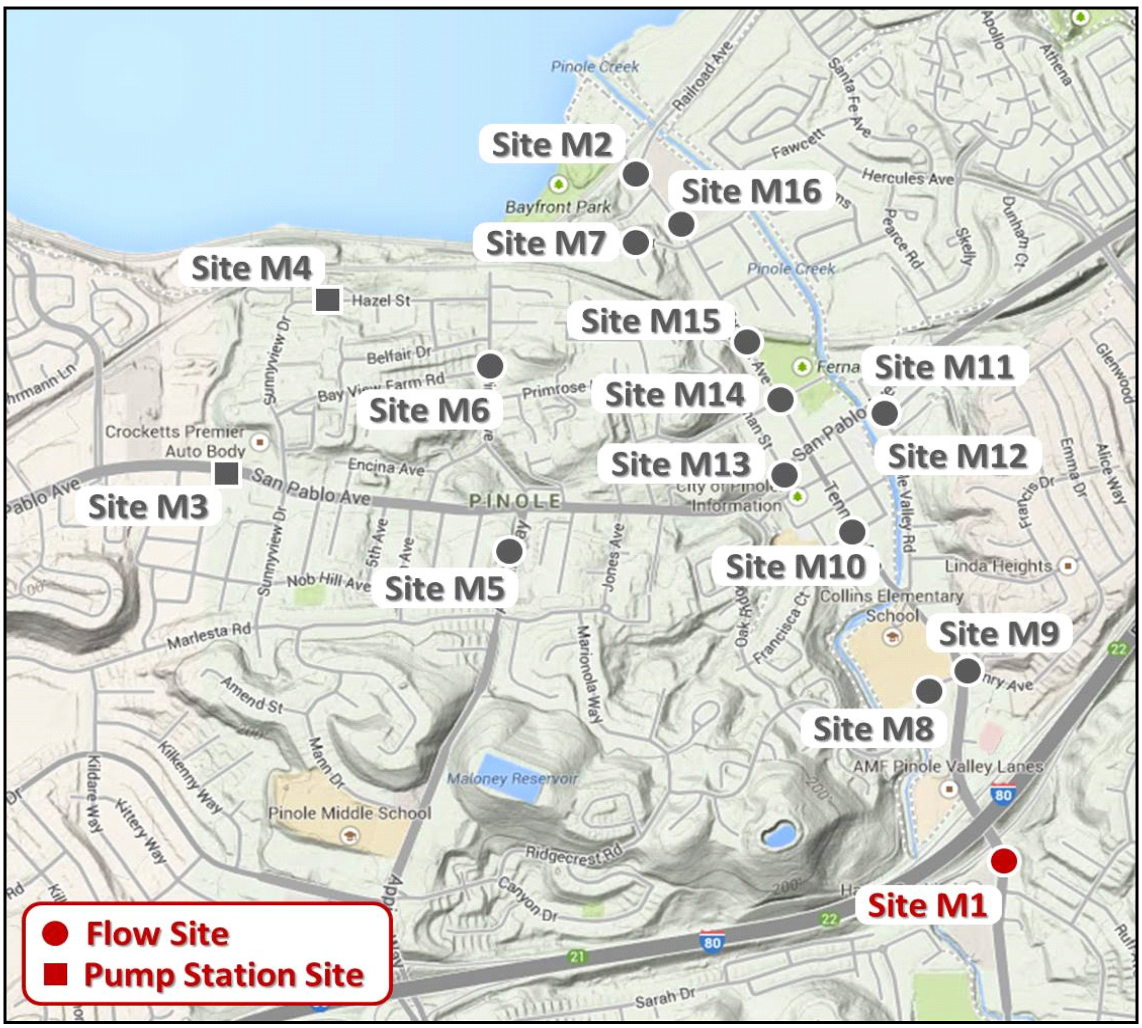
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M1

Location: Pinole Valley Rd., just south of Highway 80

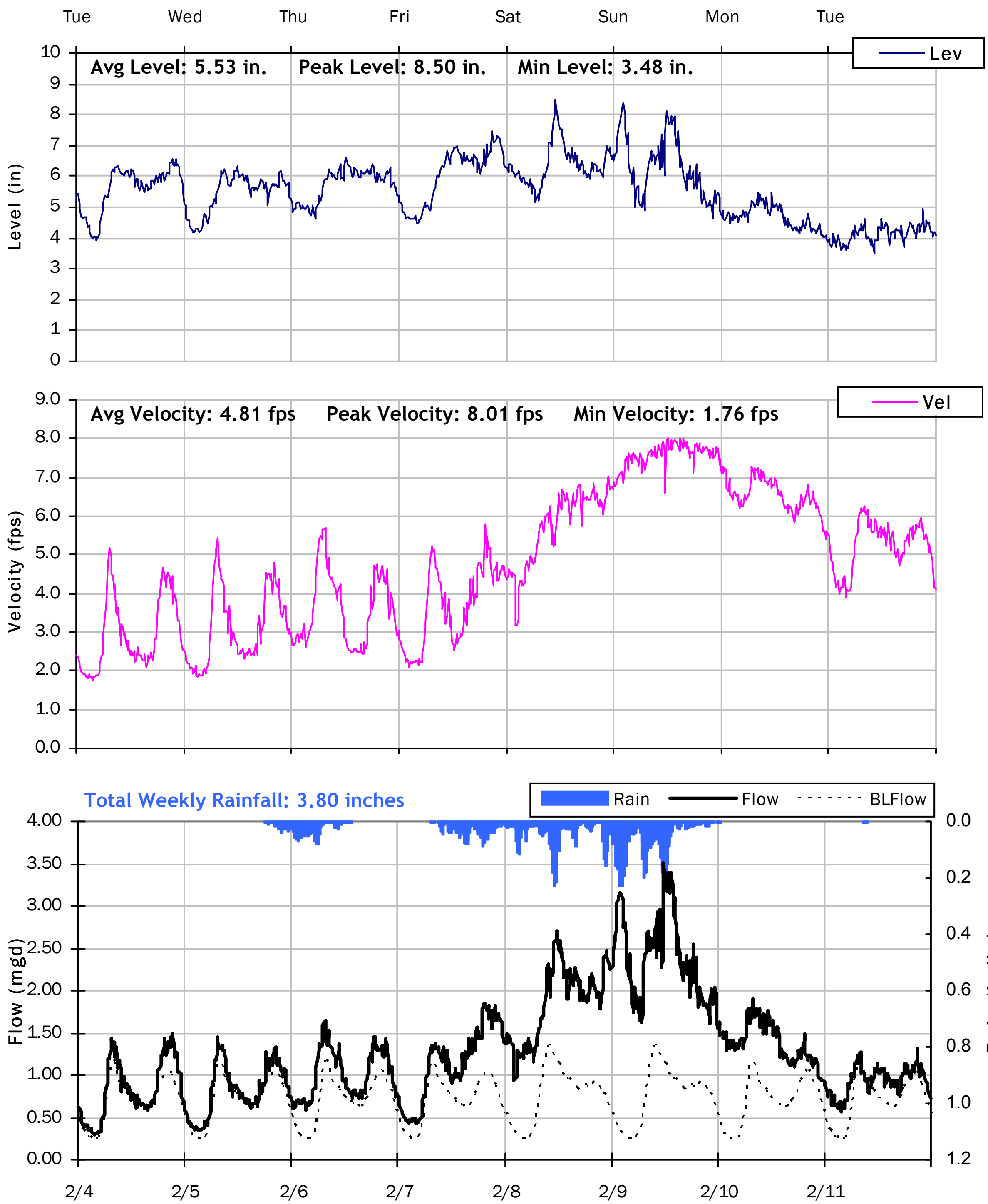
Data Summary Report



Vicinity Map: Site M1

SITE M1

Weekly Level, Velocity and Flow Hydrographs

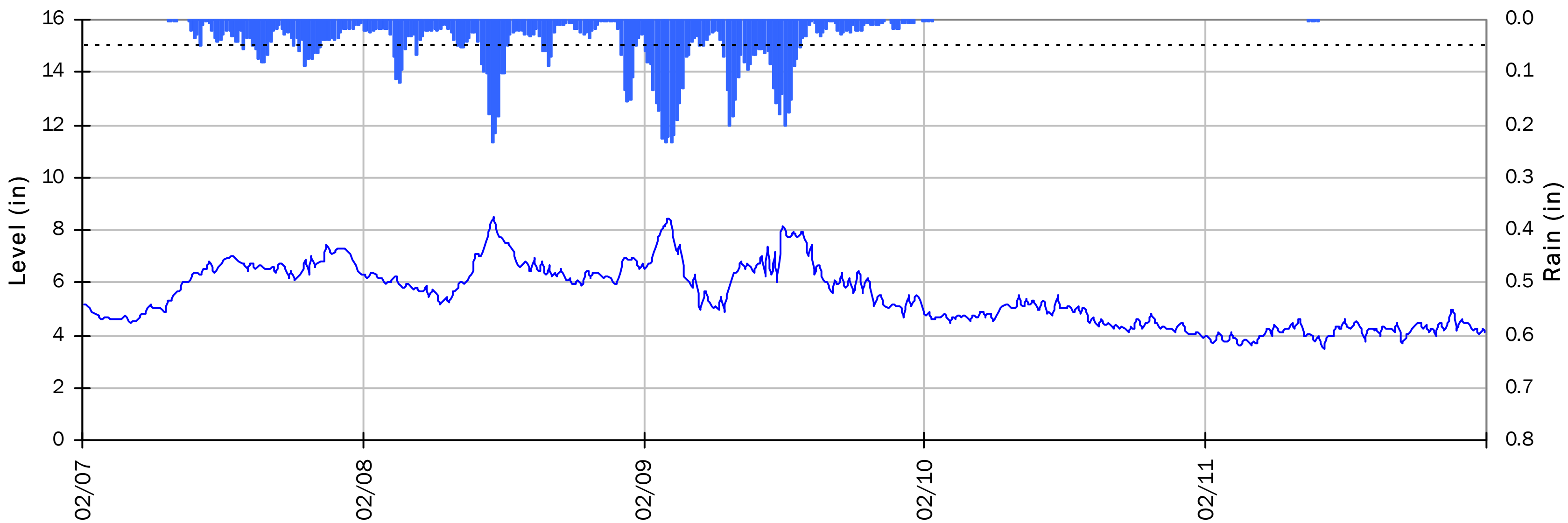


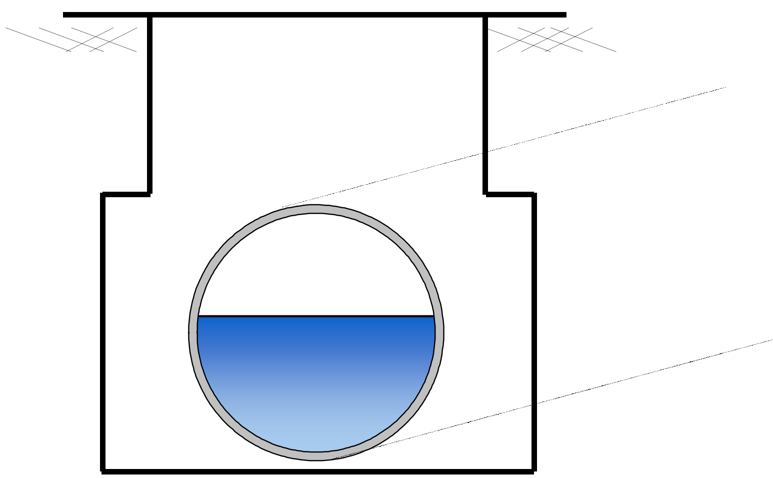
Avg Flow: 1.295 mgd Peak Flow: 3.500 mgd Min Flow: 0.299 mgd

SITE M1

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



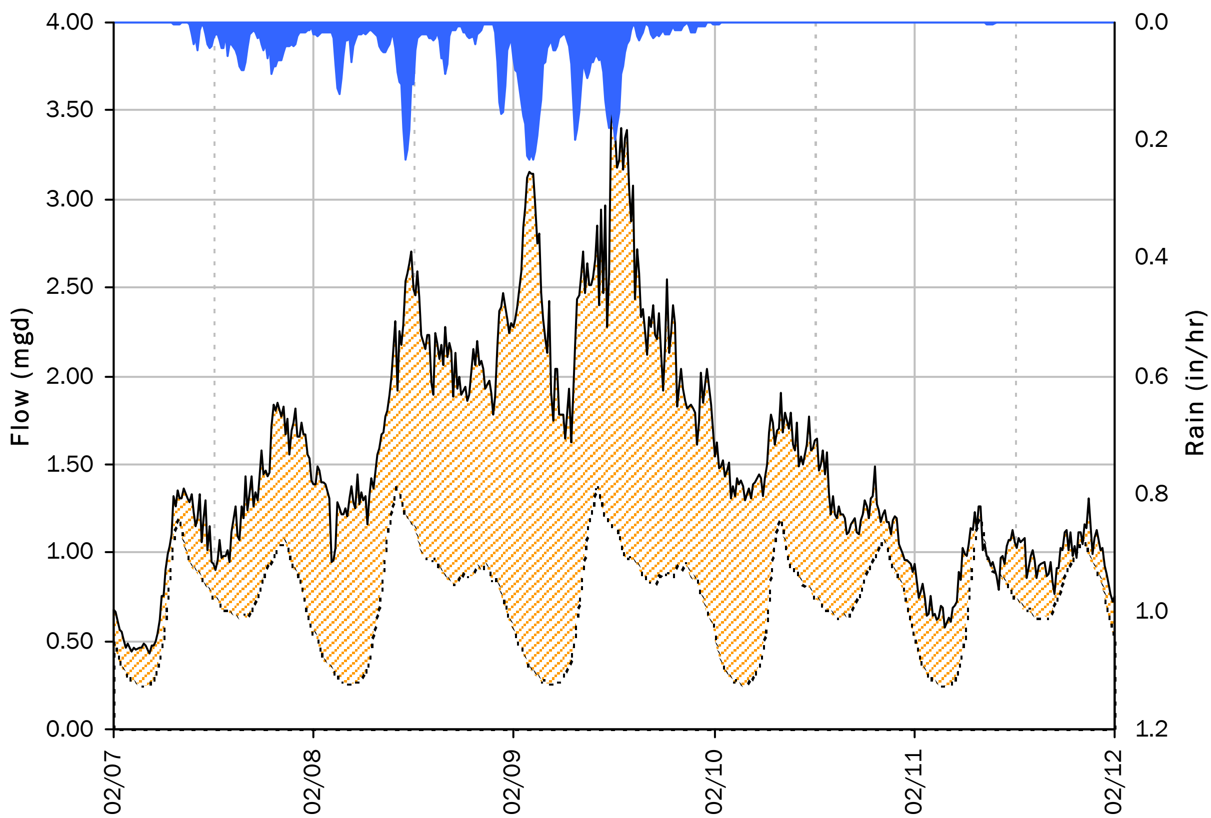


Pipe Diameter:	15	*inches*
Peak Measured Level:	8.5	*inches*
Peak d/D Ratio:	0.57	

SITE M1
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow:	3.50 *mgd*
PF:	4.90
Peak Level:	8.50 *in*
d/D Ratio:	0.57

Inflow / Infiltration

Peak I/I Rate:	2.83 *mgd*
Total I/I:	4,093,000 *gallons*

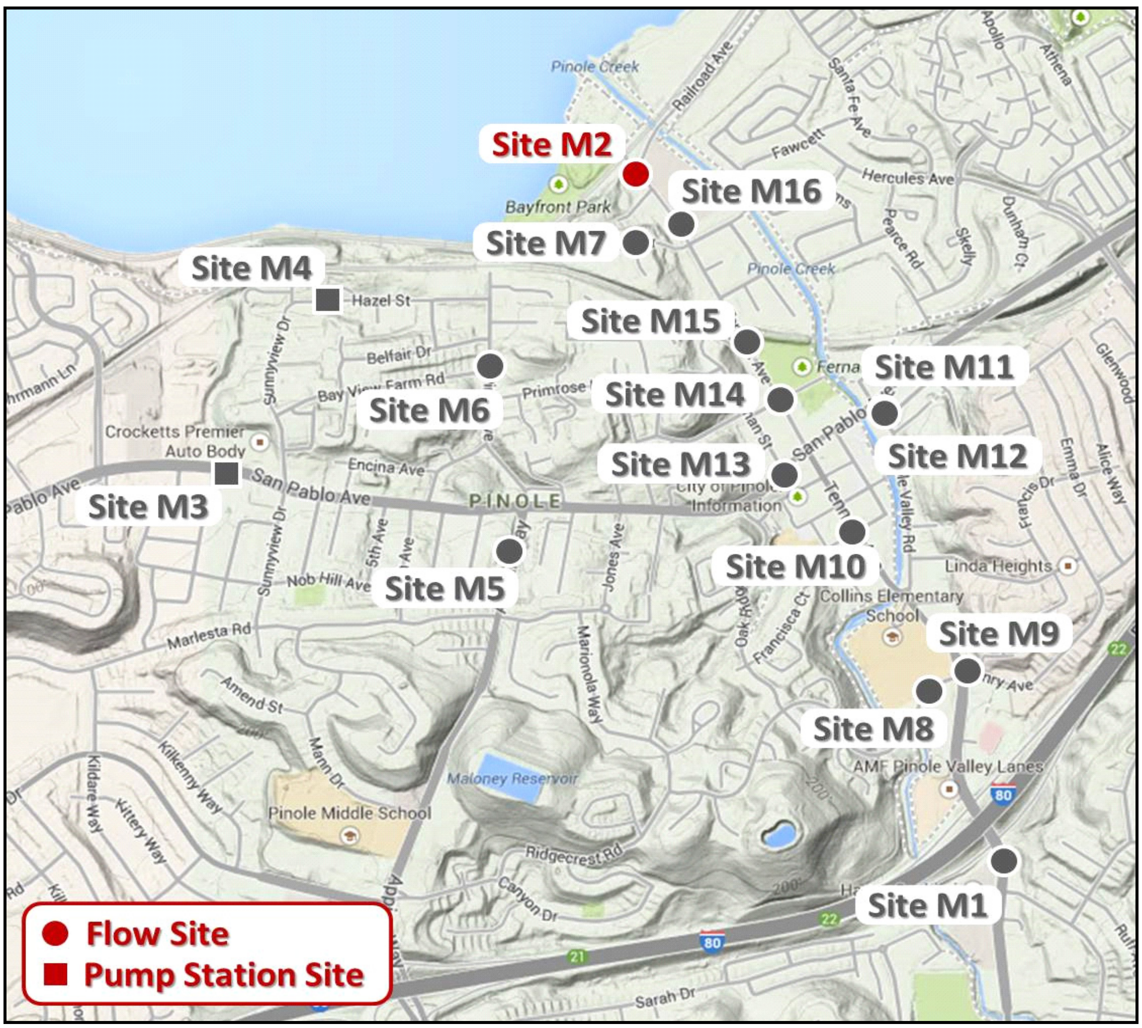
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M2

Location: Tennant Ave., just outside WWTP

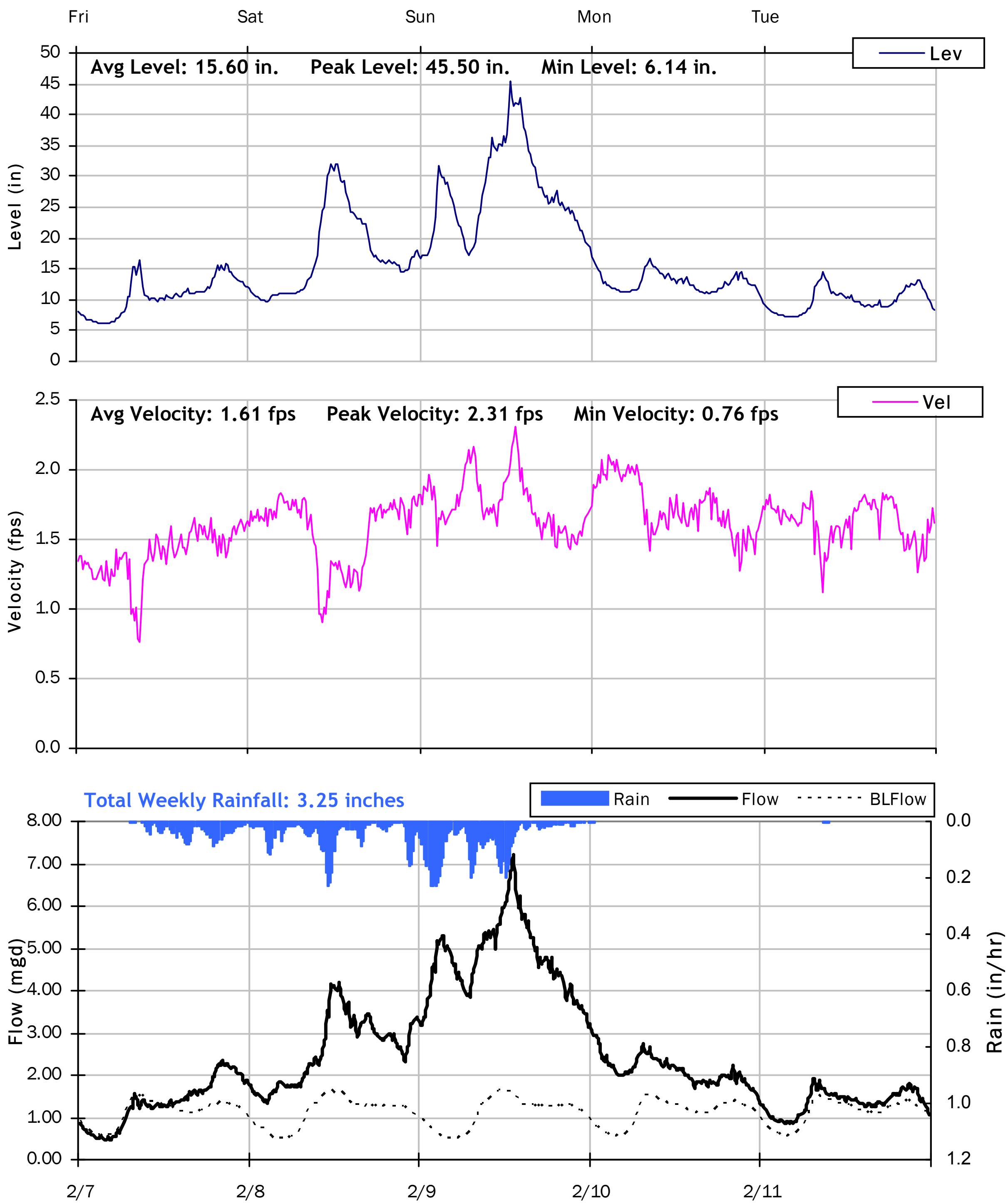
Data Summary Report



Vicinity Map: Site M2

SITE M2

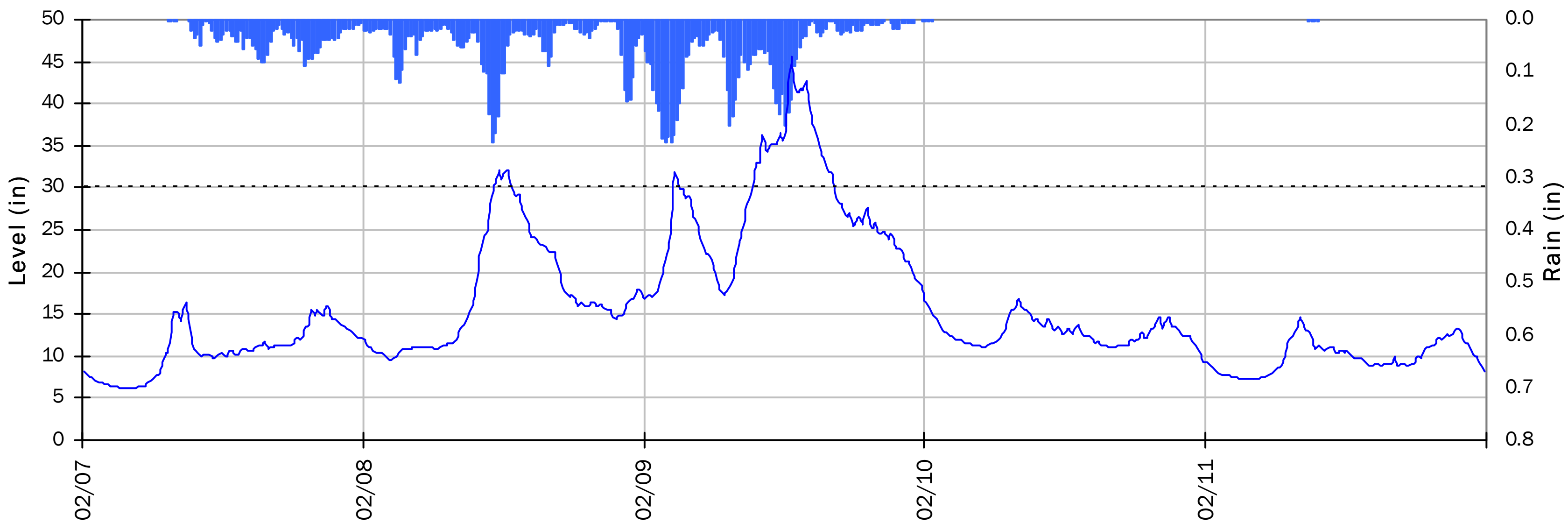
Weekly Level, Velocity and Flow Hydrographs

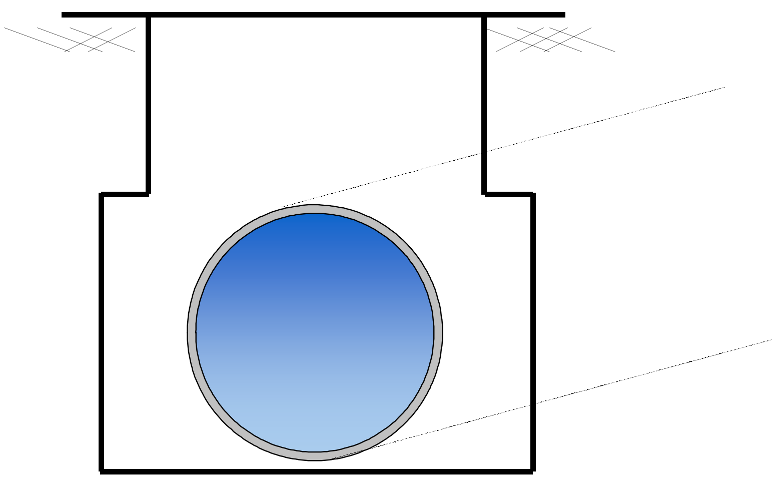


Avg Flow: 2.432 mgd **Peak Flow: 7.201 mgd** **Min Flow: 0.478 mgd**

SITE M2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





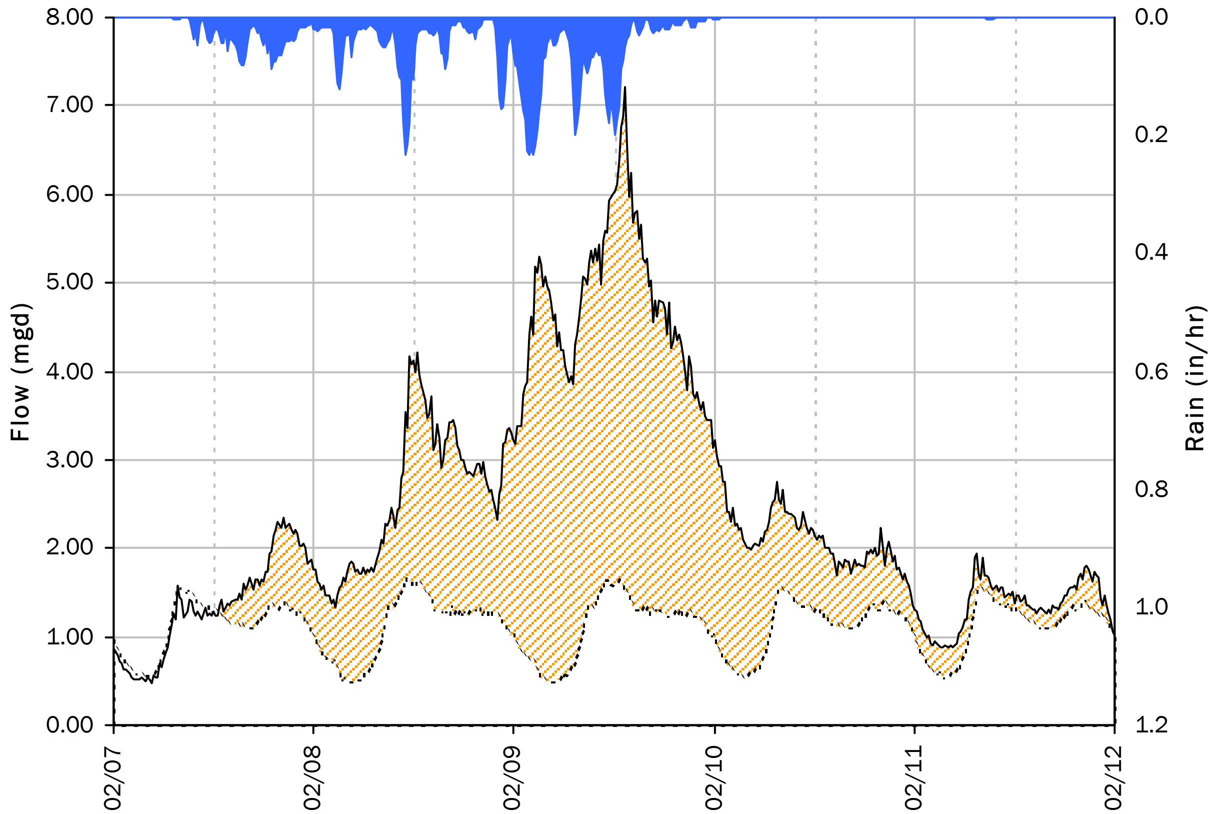
Pipe Diameter:	30	*inches*
Peak Measured Level:	45.5	*inches*
Peak d/D Ratio:	1.52	

Surcharged 15.5 inches over crown

SITE M2
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 7.20 *mgd*
PF: 6.50

Peak Level: 45.50 *in*
d/D Ratio: 1.52

Inflow / Infiltration

Peak I/I Rate: 5.70 *mgd*
Total I/I: 6,623,000 *gallons*

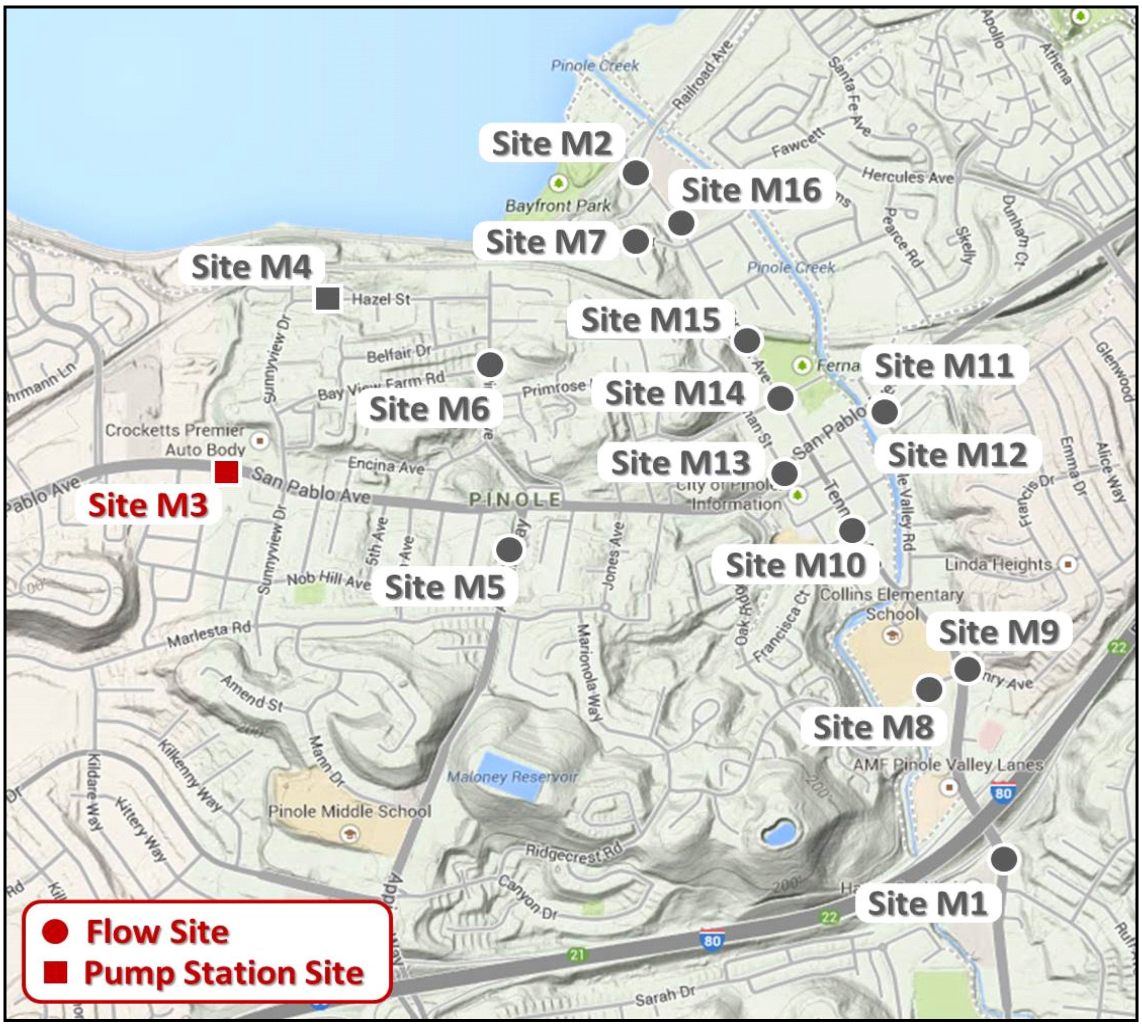
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M3

Location: San Pablo Ave., west of Sunnyview Dr.

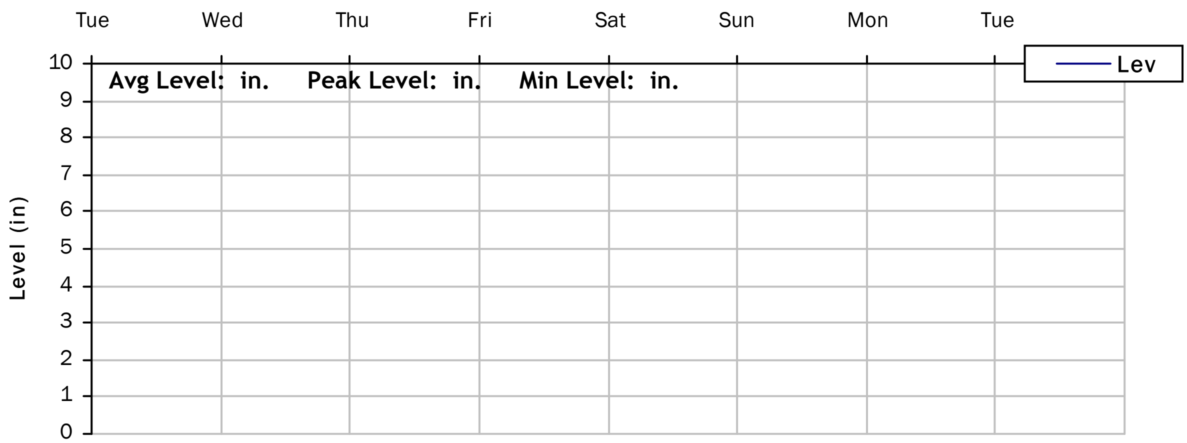
Data Summary Report

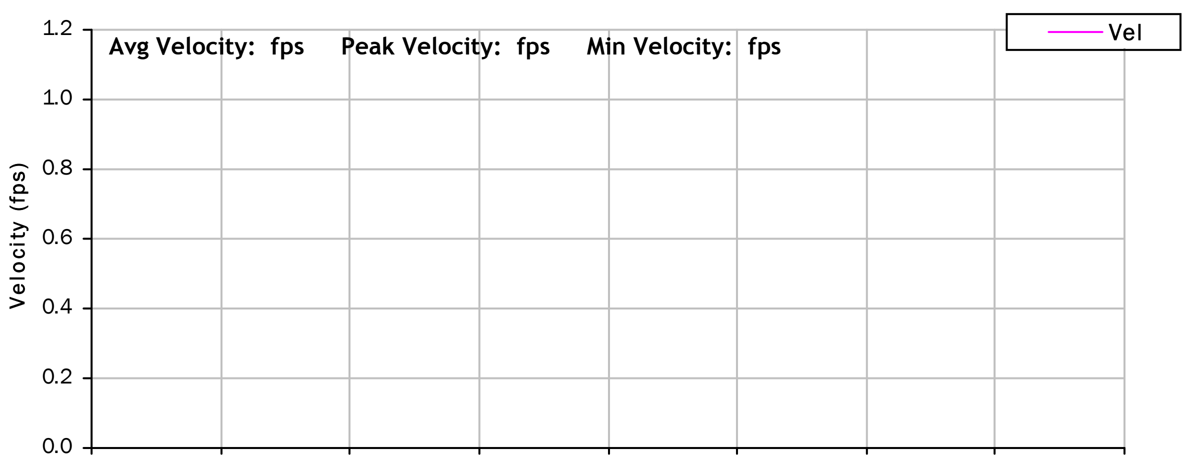


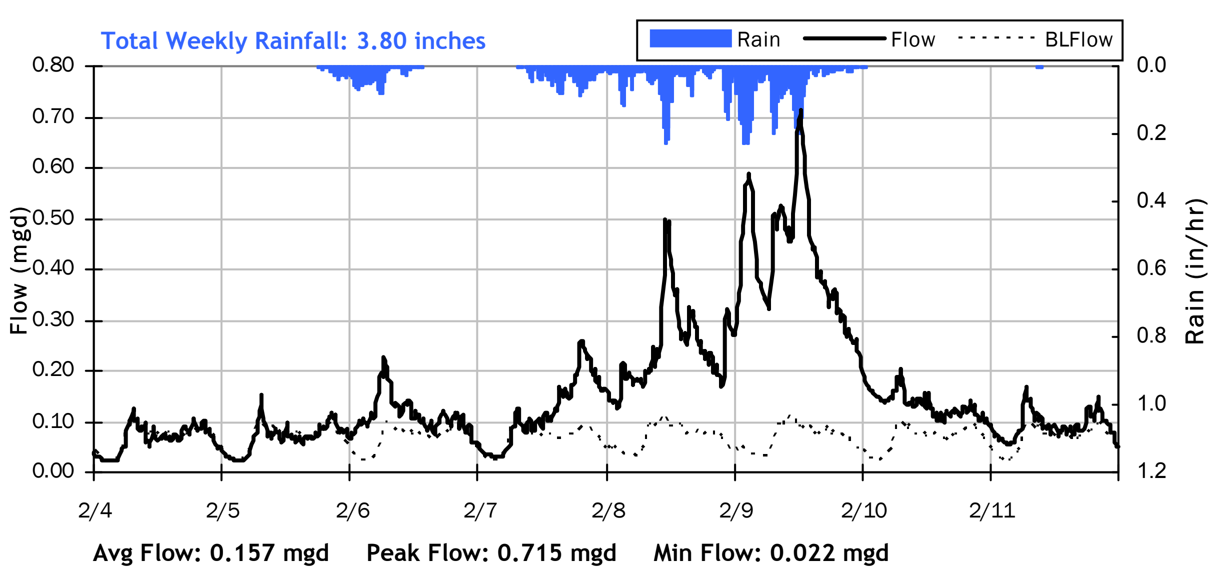
Vicinity Map: Site M3

SITE M3

Weekly Level, Velocity and Flow Hydrographs



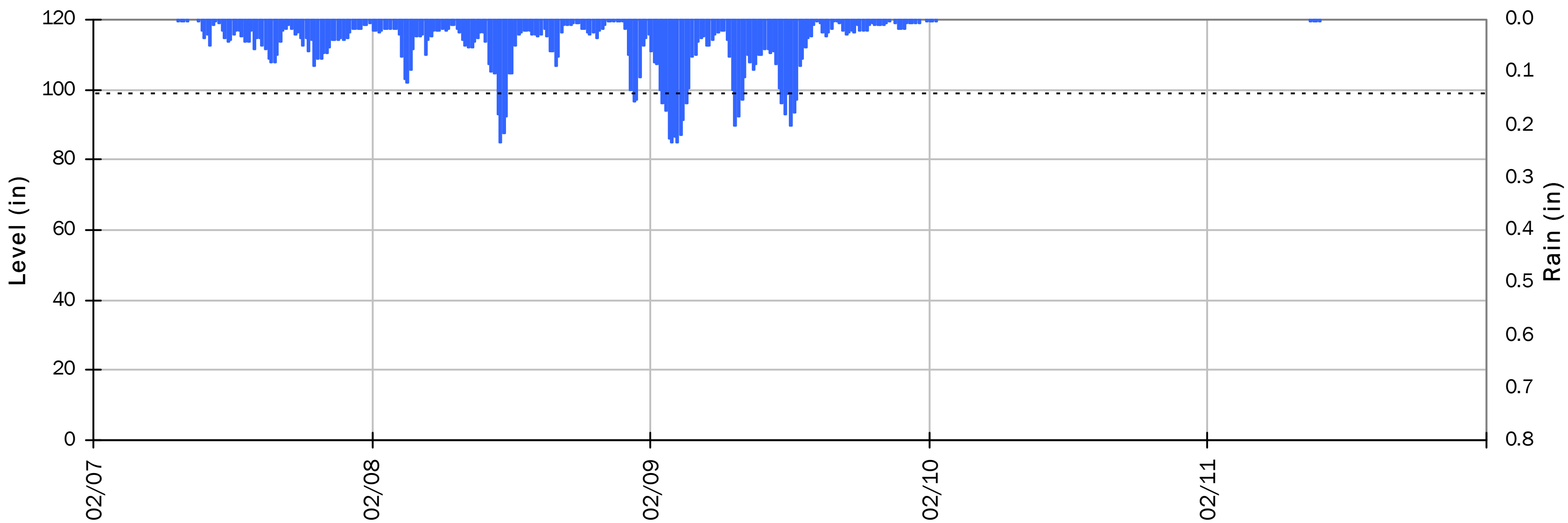


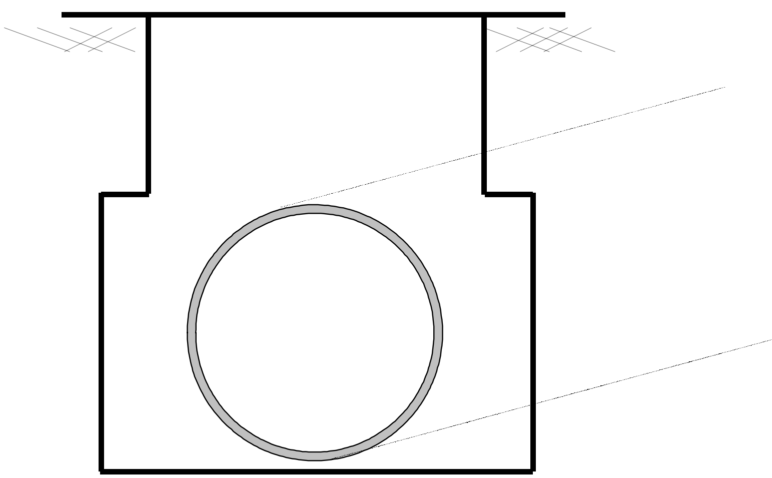


SITE M3

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



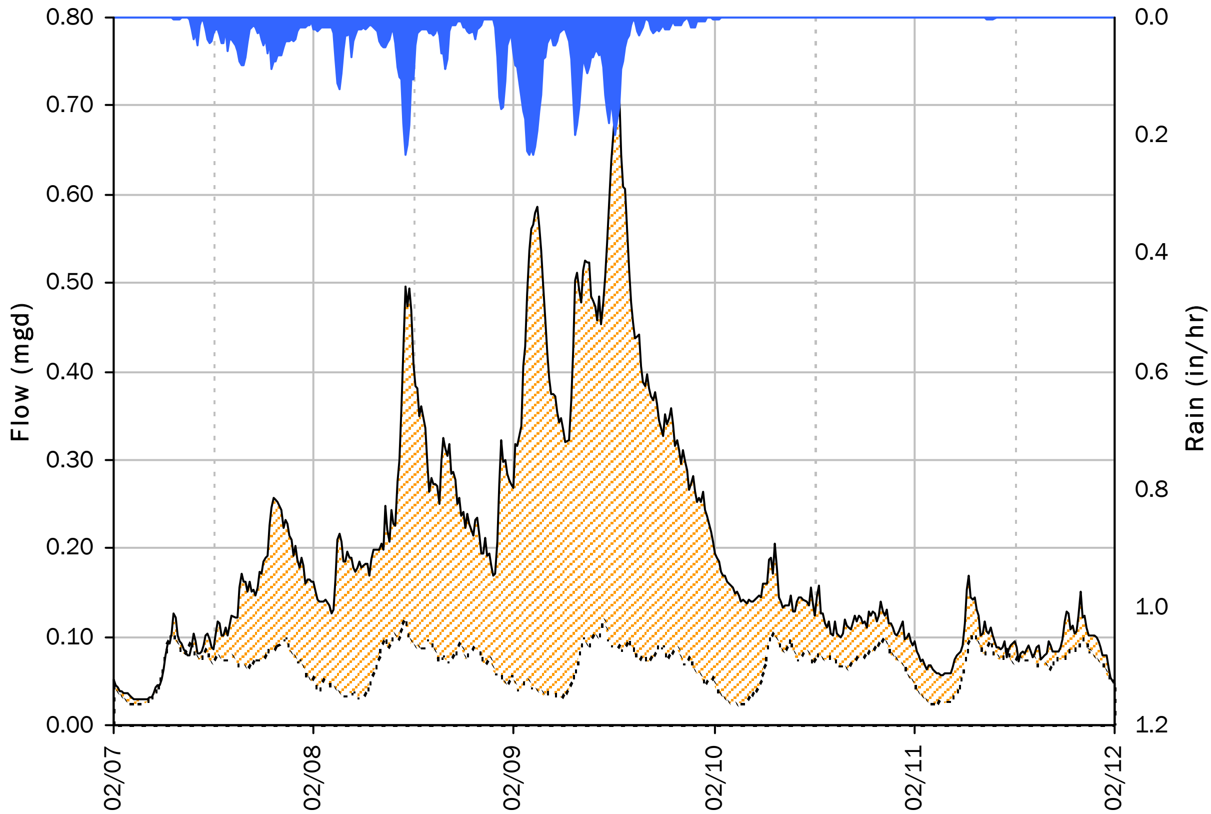


Pipe Diameter:	99	*inches*
Peak Measured Level:		*inches*
Peak d/D Ratio:		

SITE M3
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.71 *mgd*
PF: 10.66

Peak Level: *in*
d/D Ratio:

Inflow / Infiltration

Peak I/I Rate: 0.63 *mgd*
Total I/I: 670,000 *gallons*

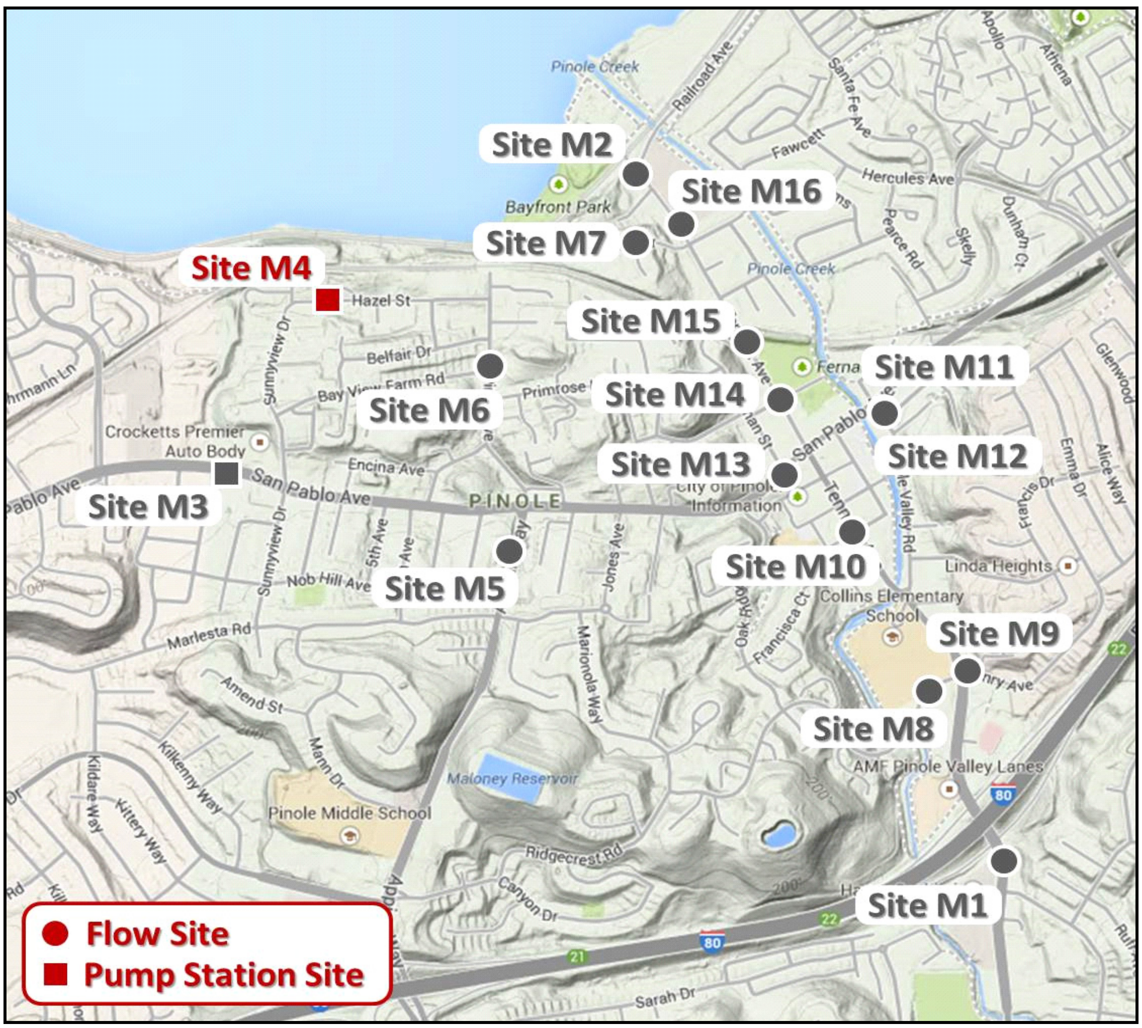
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M4

Location: In easement at west end of Hazel St.

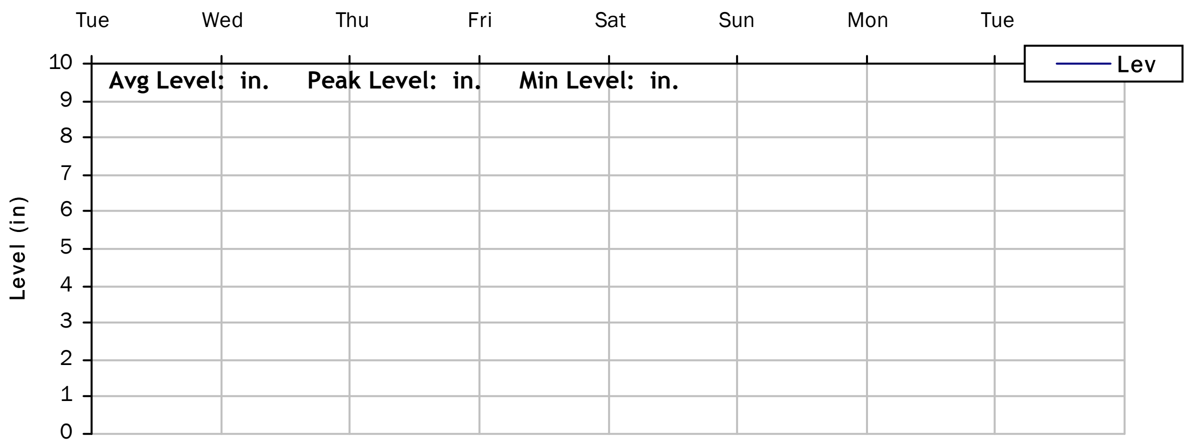
Data Summary Report

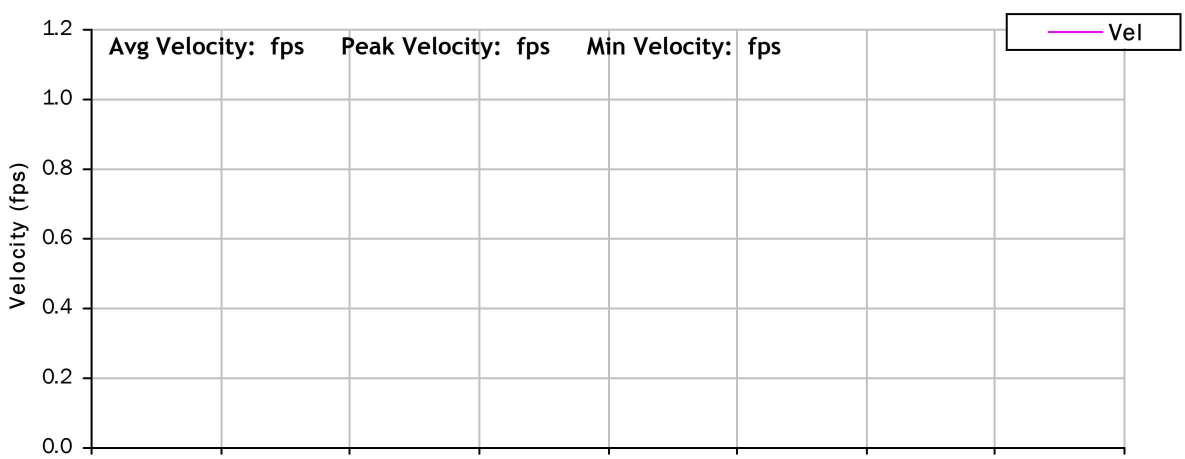


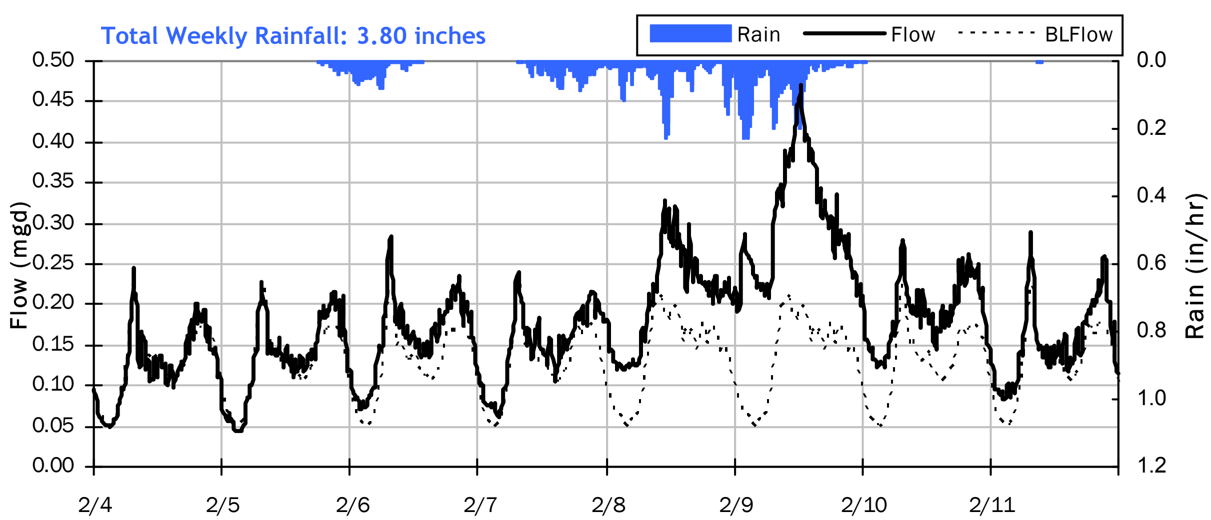
Vicinity Map: Site M4

SITE M4

Weekly Level, Velocity and Flow Hydrographs





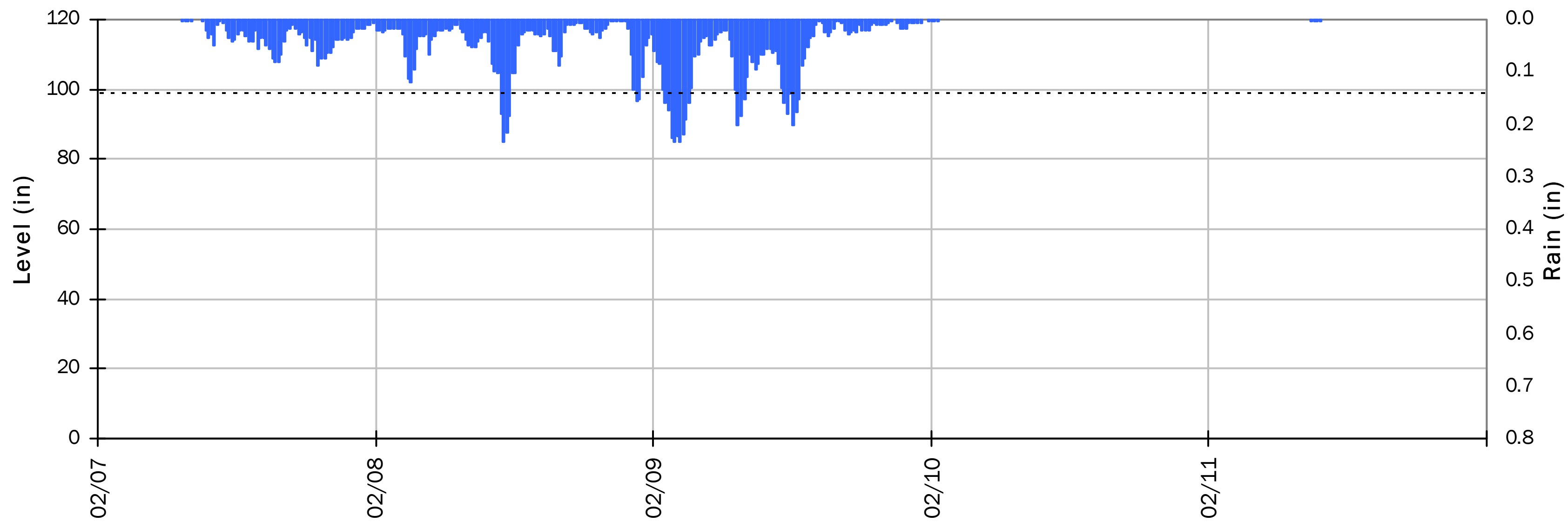


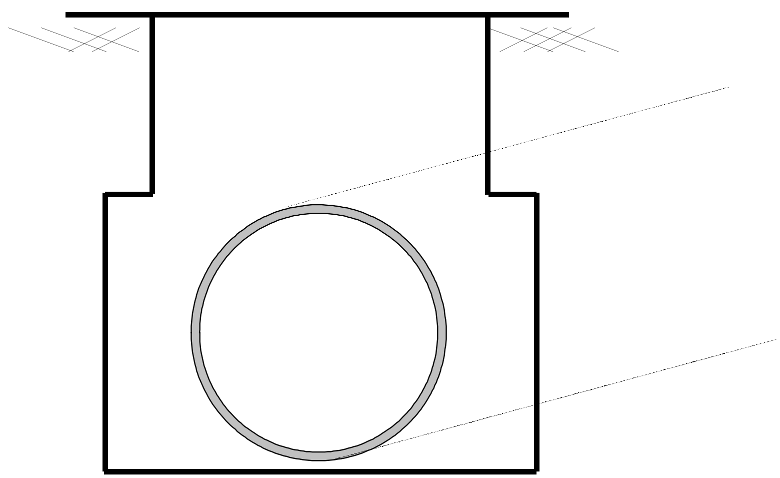
Avg Flow: 0.178 mgd Peak Flow: 0.470 mgd Min Flow: 0.044 mgd

SITE M4

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



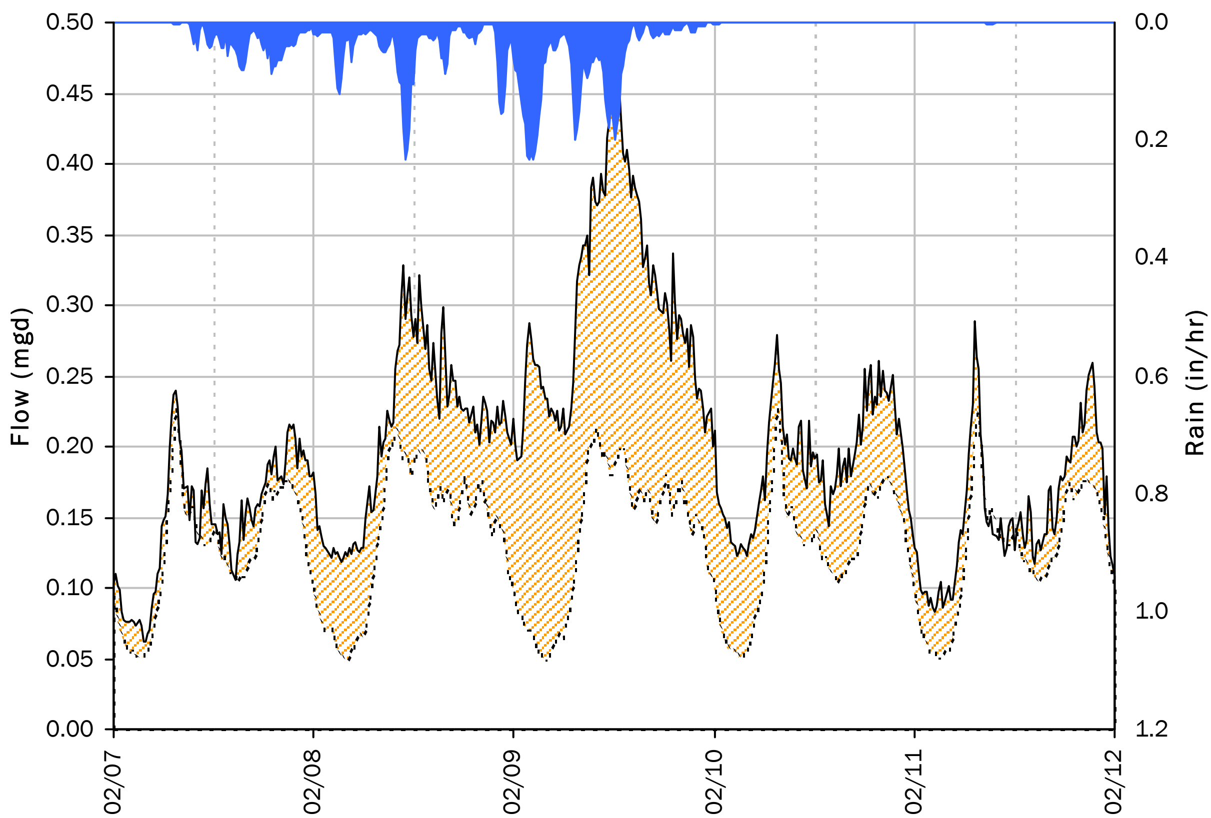


Pipe Diameter:	99	*inches*
Peak Measured Level:		*inches*
Peak d/D Ratio:		

SITE M4
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.47 *mgd*
PF: 3.66

Peak Level: *in*
d/D Ratio:

Inflow / Infiltration

Peak I/I Rate: 0.28 *mgd*
Total I/I: 355,000 *gallons*

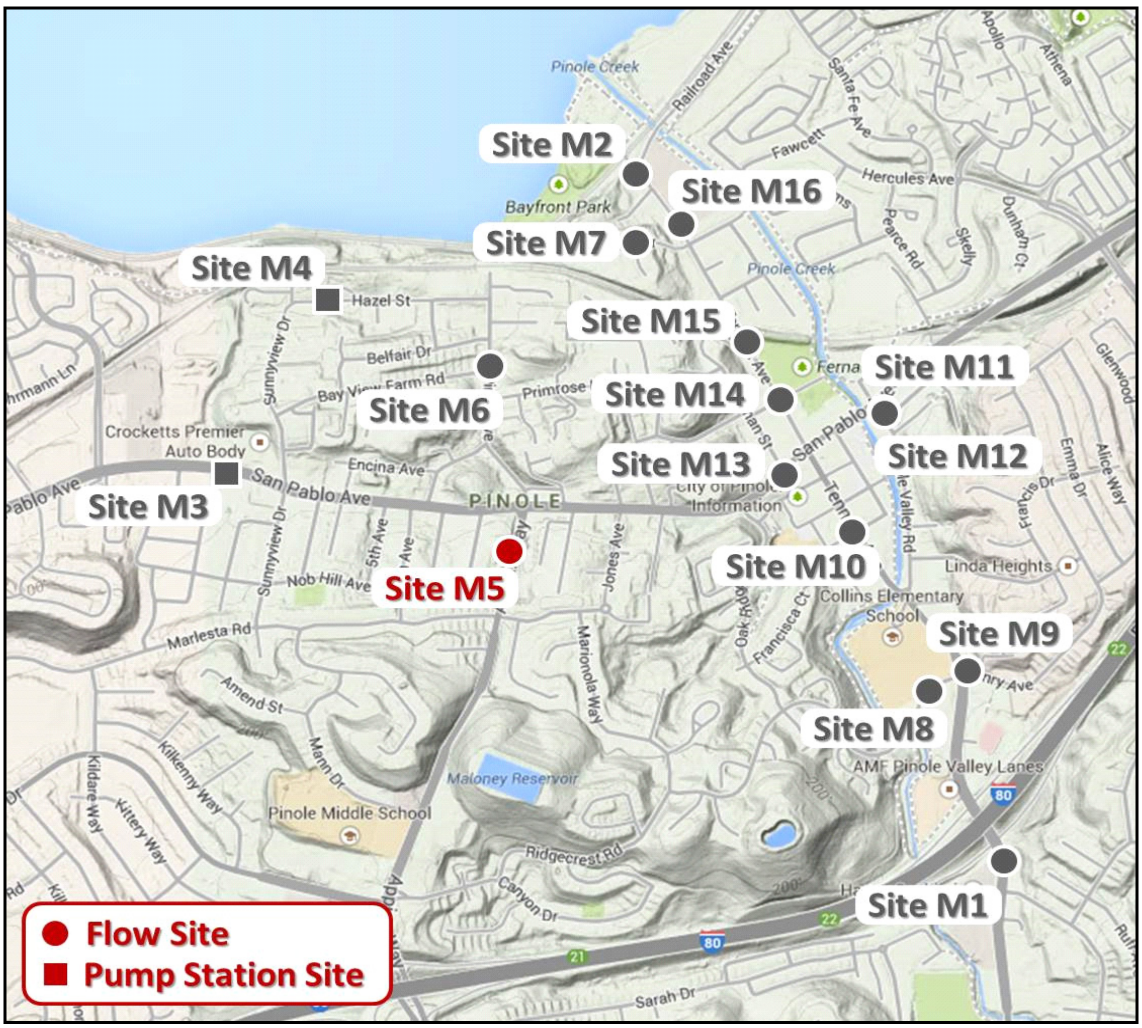
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M5

Location: Appian Way, south of San Pablo Ave.

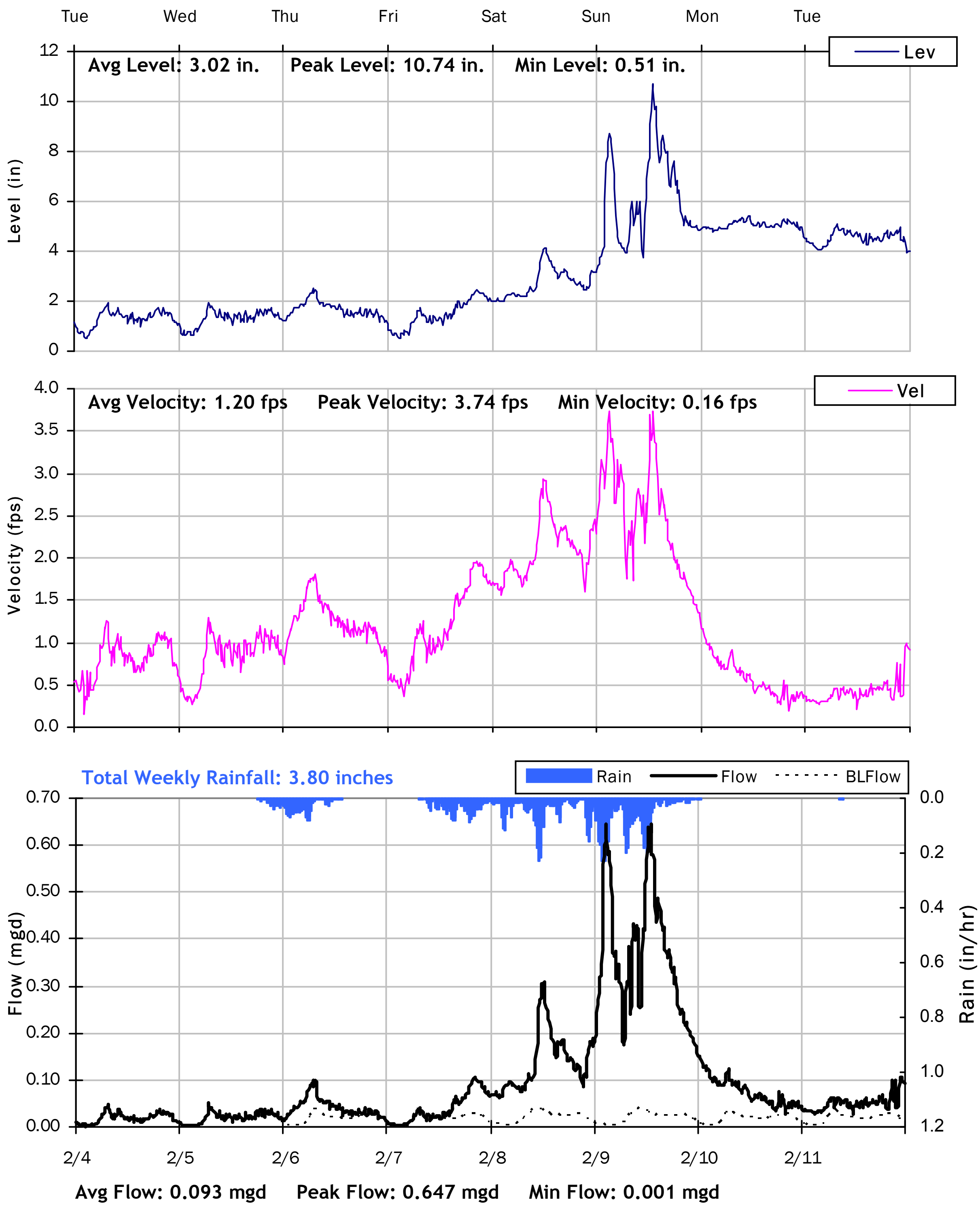
Data Summary Report



Vicinity Map: Site M5

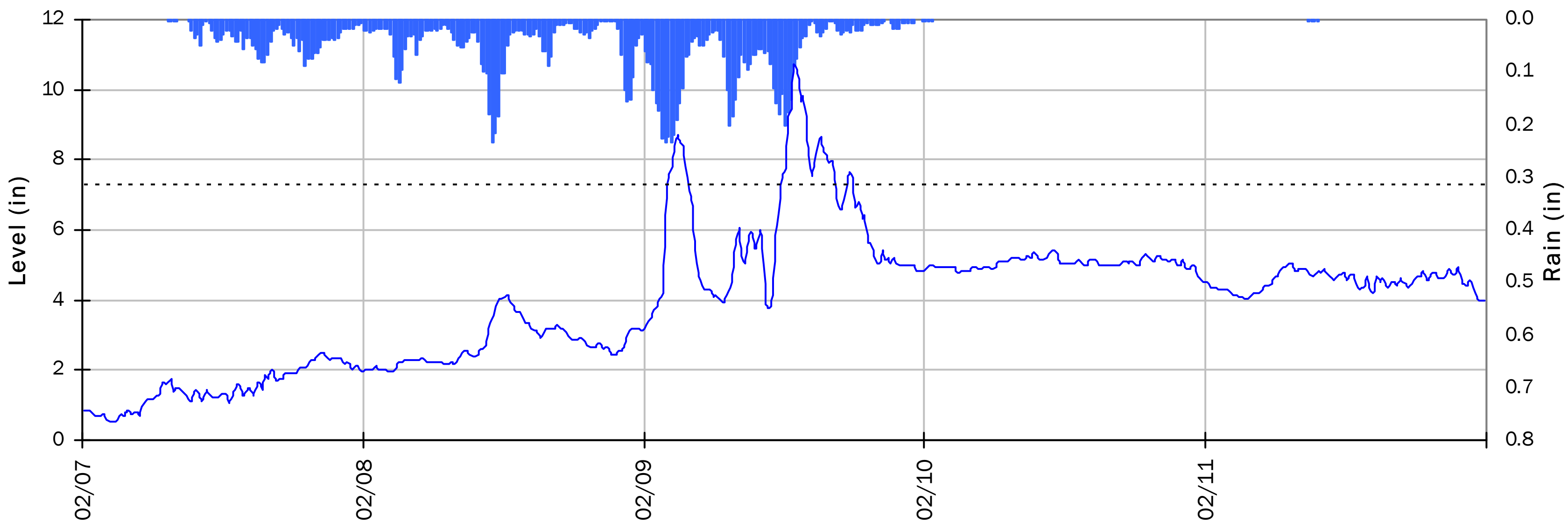
SITE M5

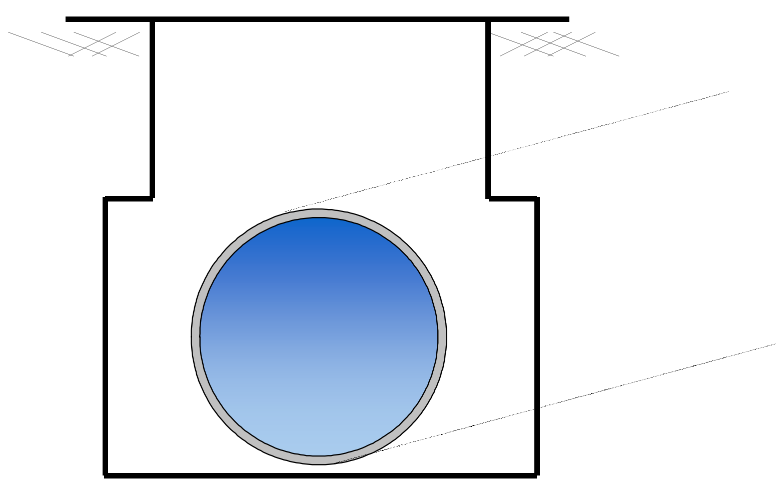
Weekly Level, Velocity and Flow Hydrographs



SITE M5
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





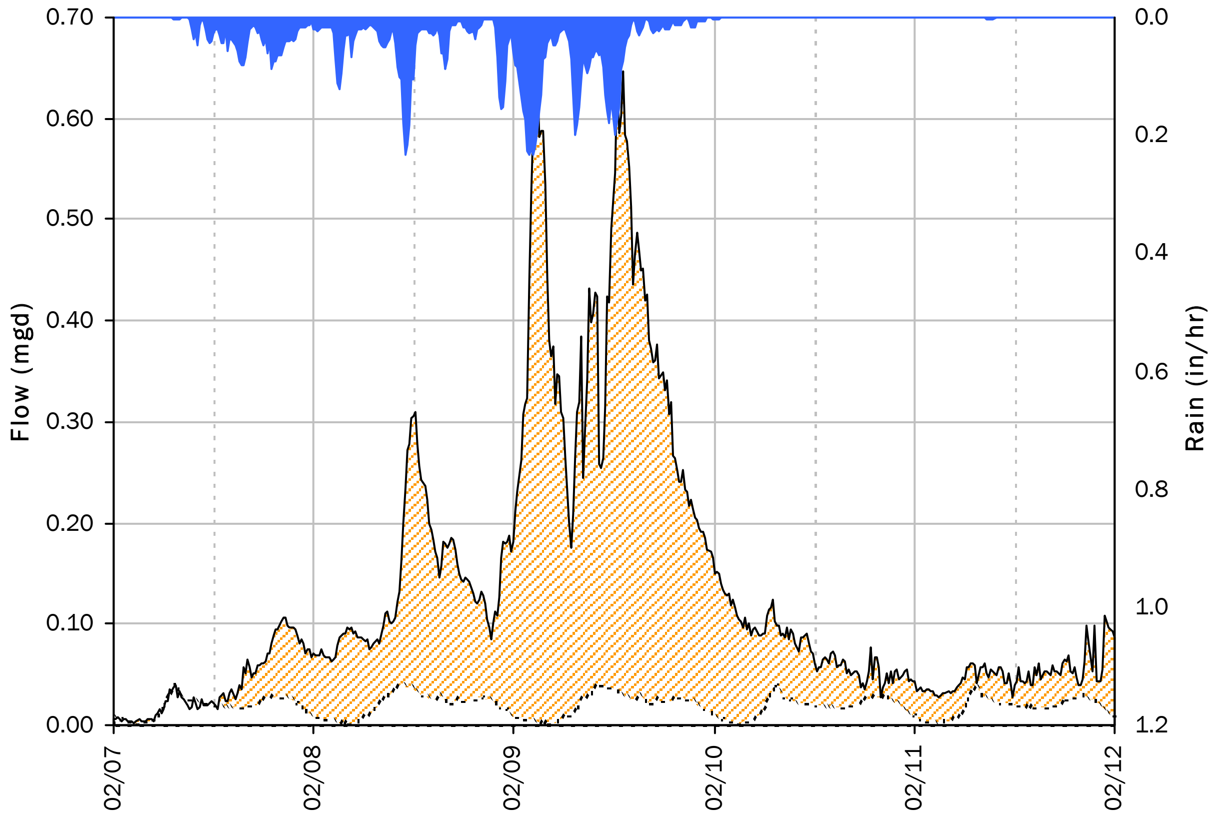
Pipe Diameter:	7.25	*inches*
Peak Measured Level:	10.7	*inches*
Peak d/D Ratio:	1.48	

Surcharged 3.5 inches over crown

SITE M5
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.65 *mgd*
PF: 33.85

Peak Level: 10.74 *in*
d/D Ratio: 1.48

Inflow / Infiltration

Peak I/I Rate: 0.64 *mgd*
Total I/I: 570,000 *gallons*

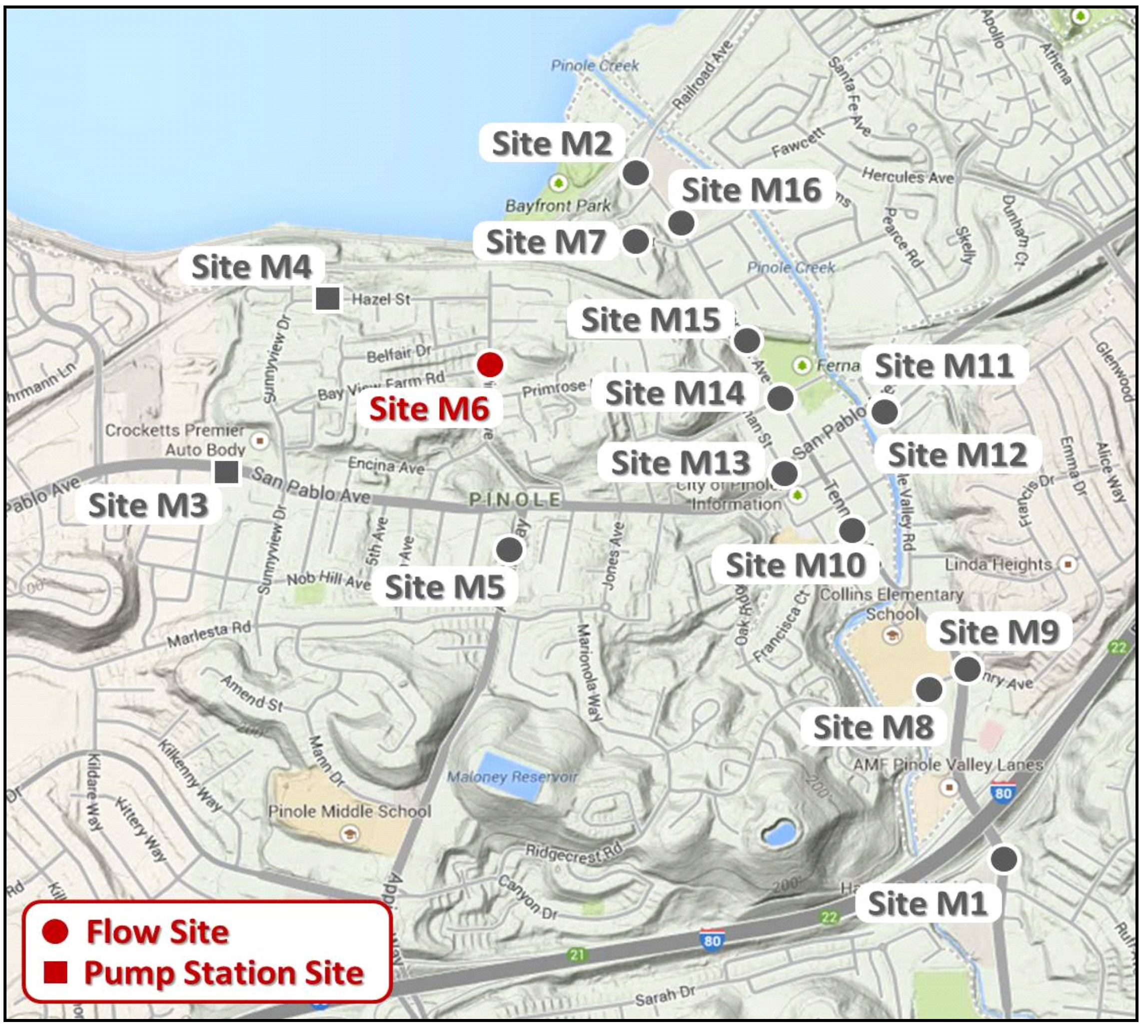
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M6

Location: Pinon Ave., north of Bay View Farm Rd.

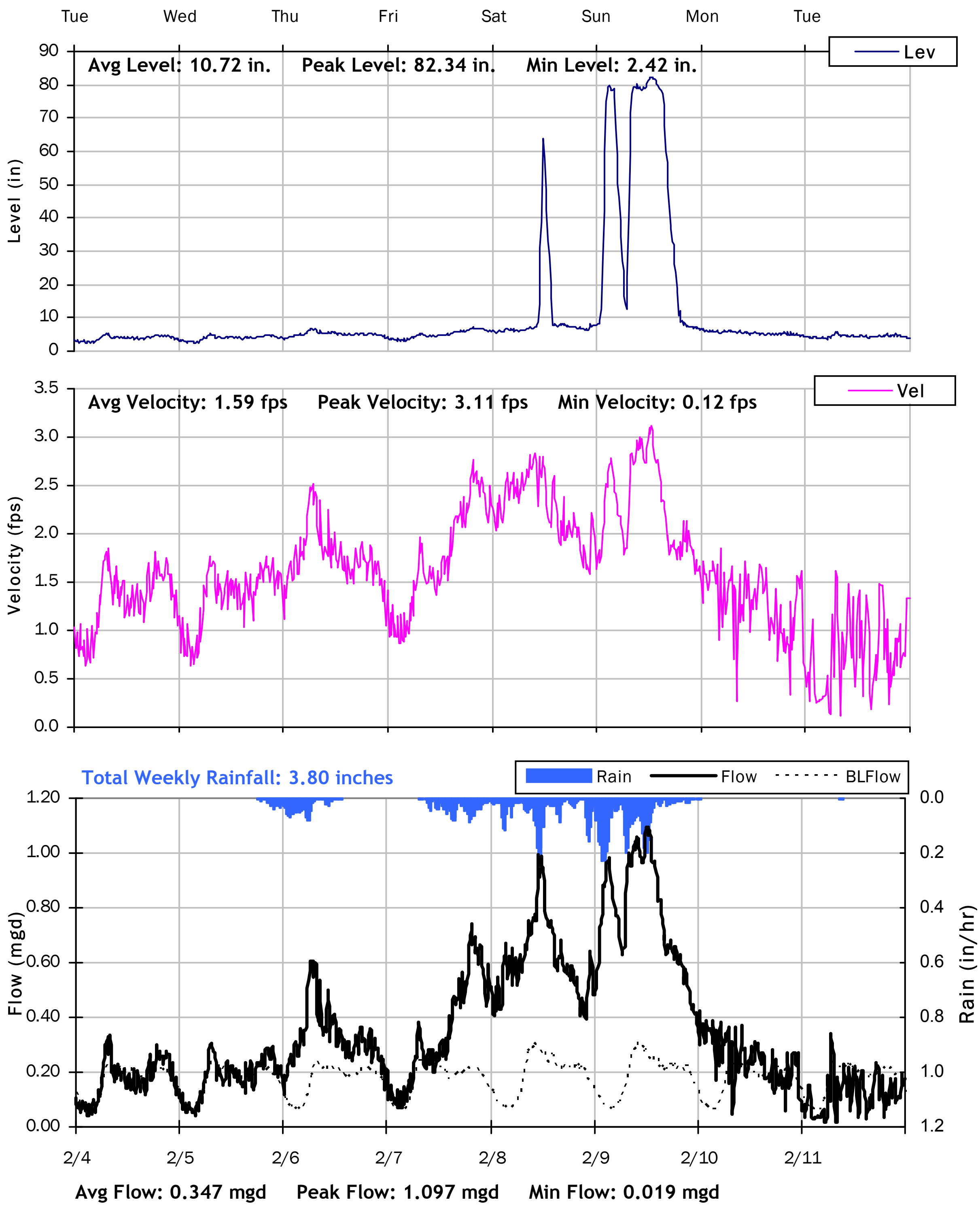
Data Summary Report



Vicinity Map: Site M6

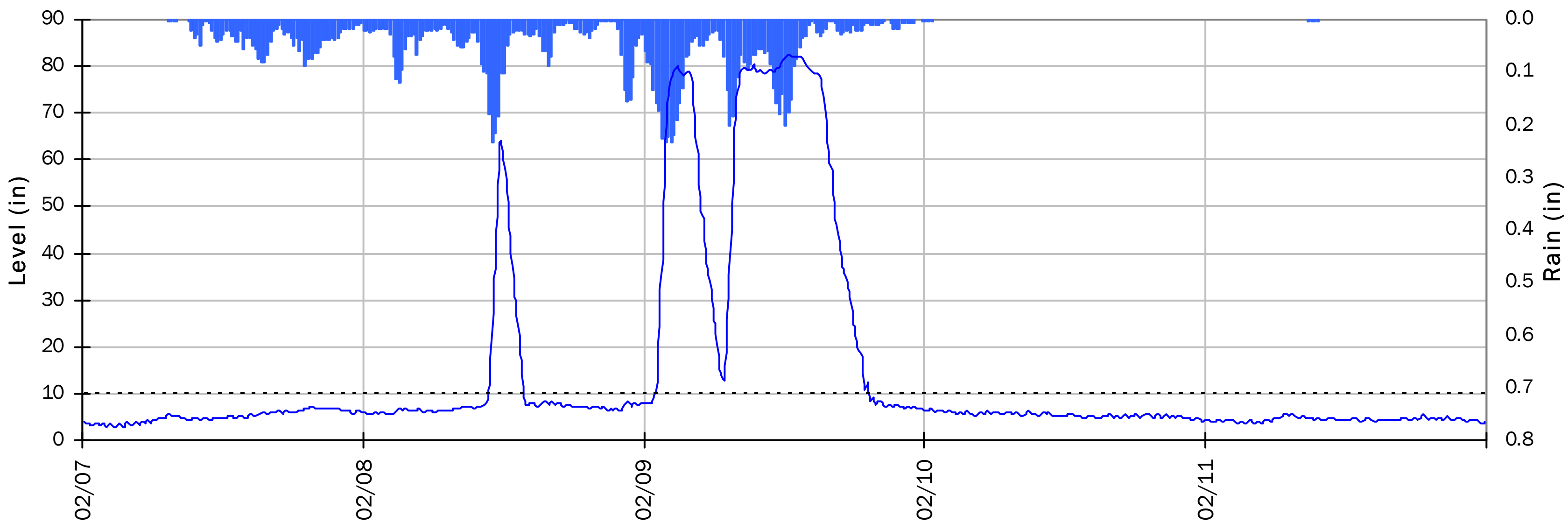
SITE M6

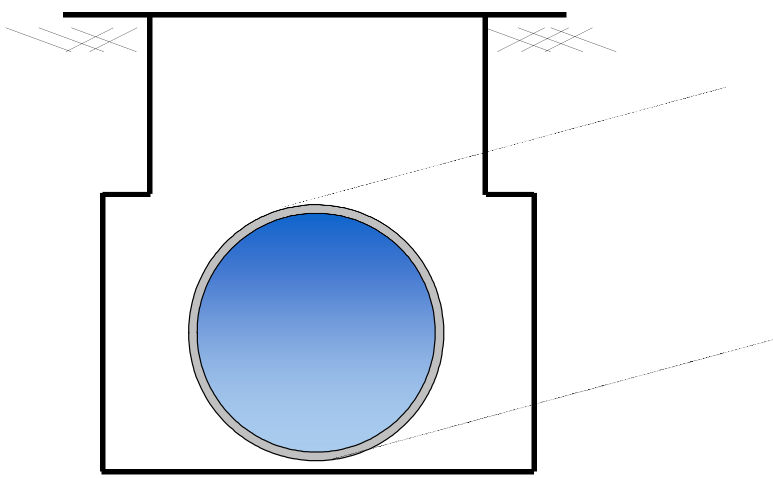
Weekly Level, Velocity and Flow Hydrographs



SITE M6
Site Capacity and Surcharge Summary

<u>Realtime Flow Levels with Rainfall Data over Monitoring Period</u>





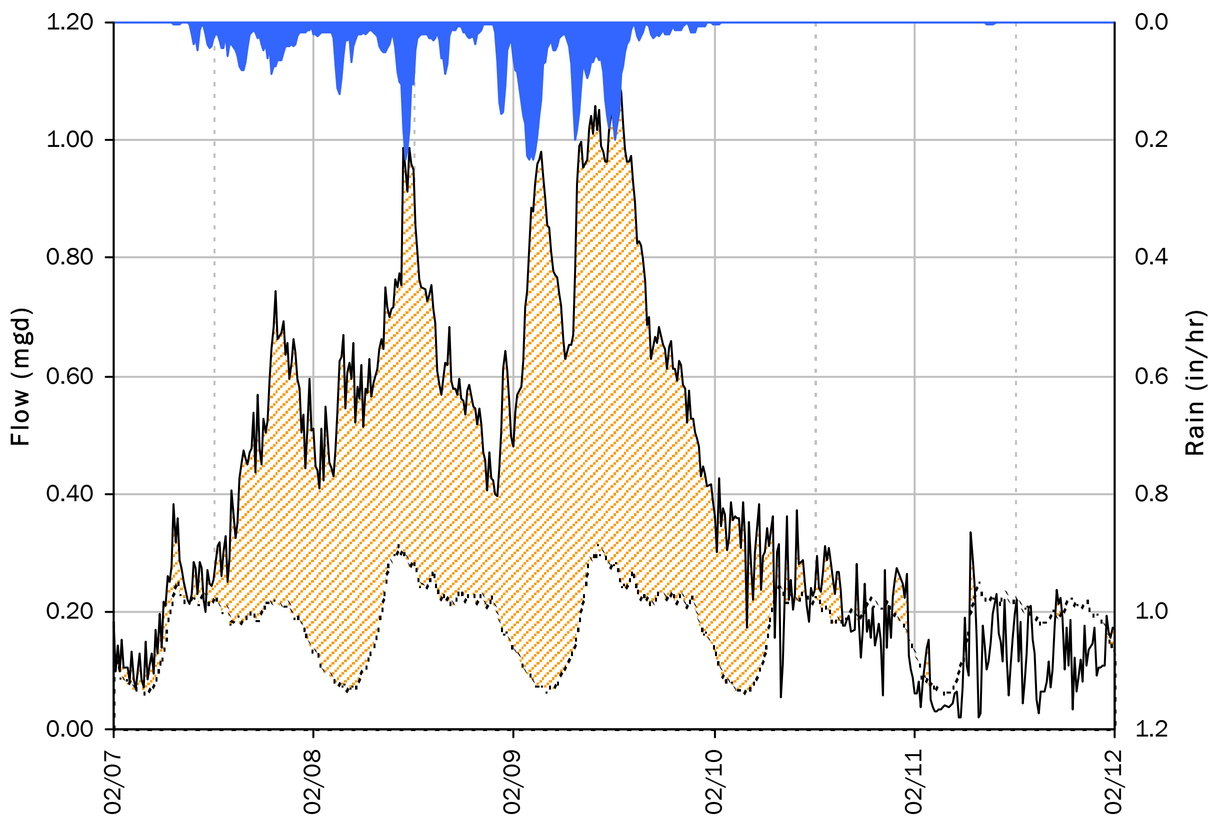
Pipe Diameter:	10	*inches*
Peak Measured Level:	82.3	*inches*
Peak d/D Ratio:	8.23	

Surcharged 72.3 inches over crown

SITE M6
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 1.10 *mgd*
PF: 6.21

Peak Level: 82.34 *in*
d/D Ratio: 8.23

Inflow / Infiltration

Peak I/I Rate: 0.91 *mgd*
Total I/I: 1,202,000 *gallons*

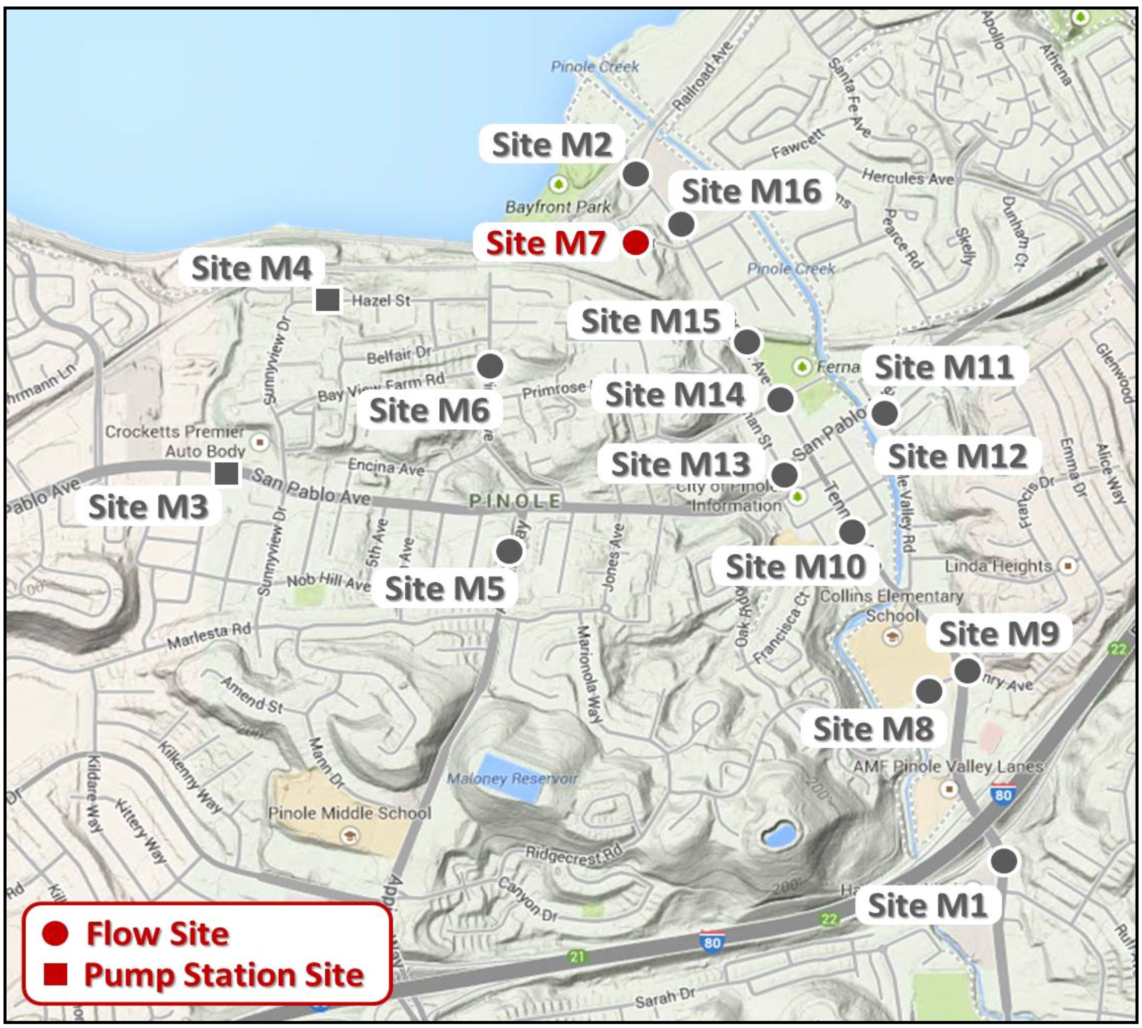
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M7

Location: Intersection of Orleans Dr. and Zoe Ct.

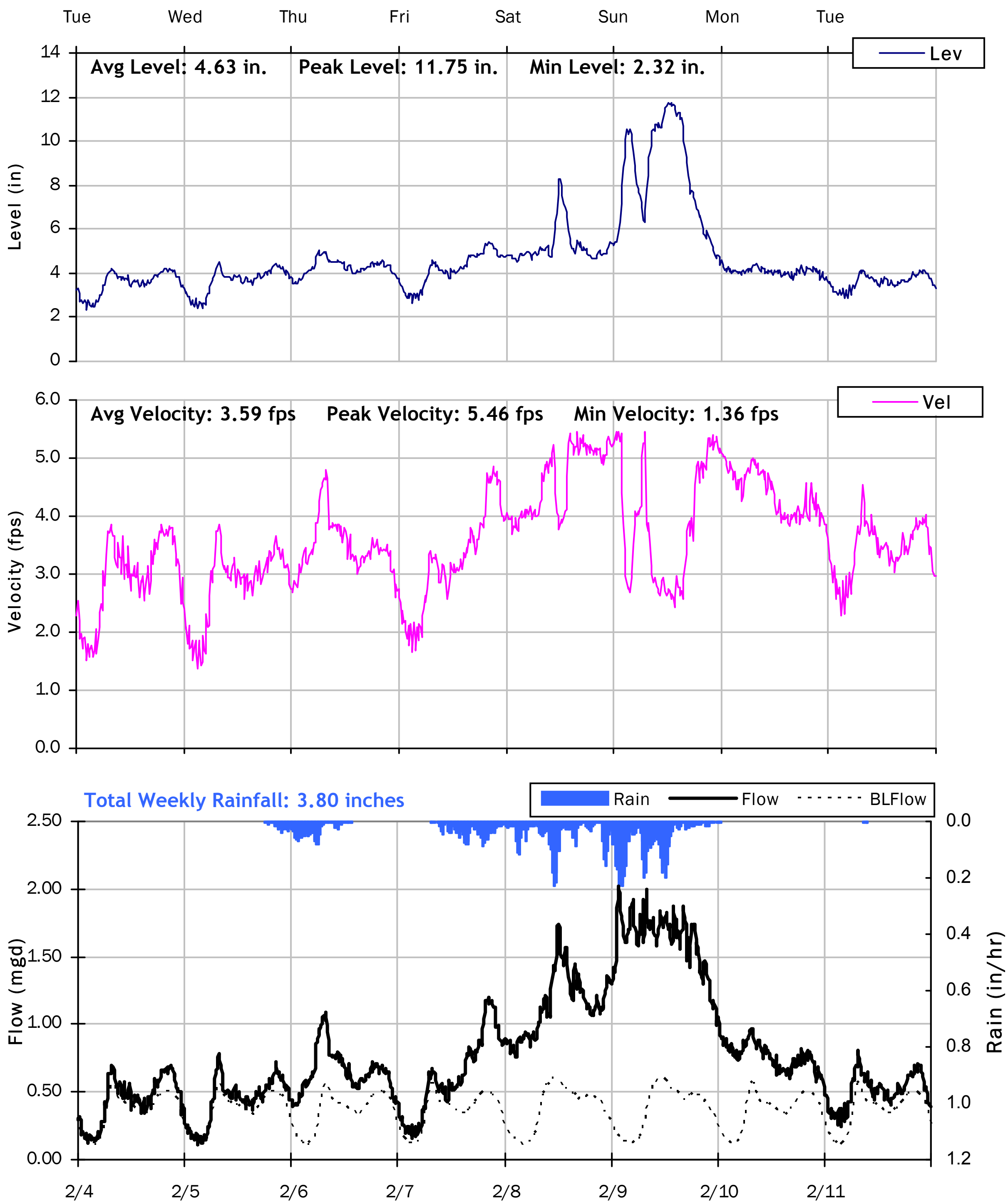
Data Summary Report



Vicinity Map: Site M7

SITE M7

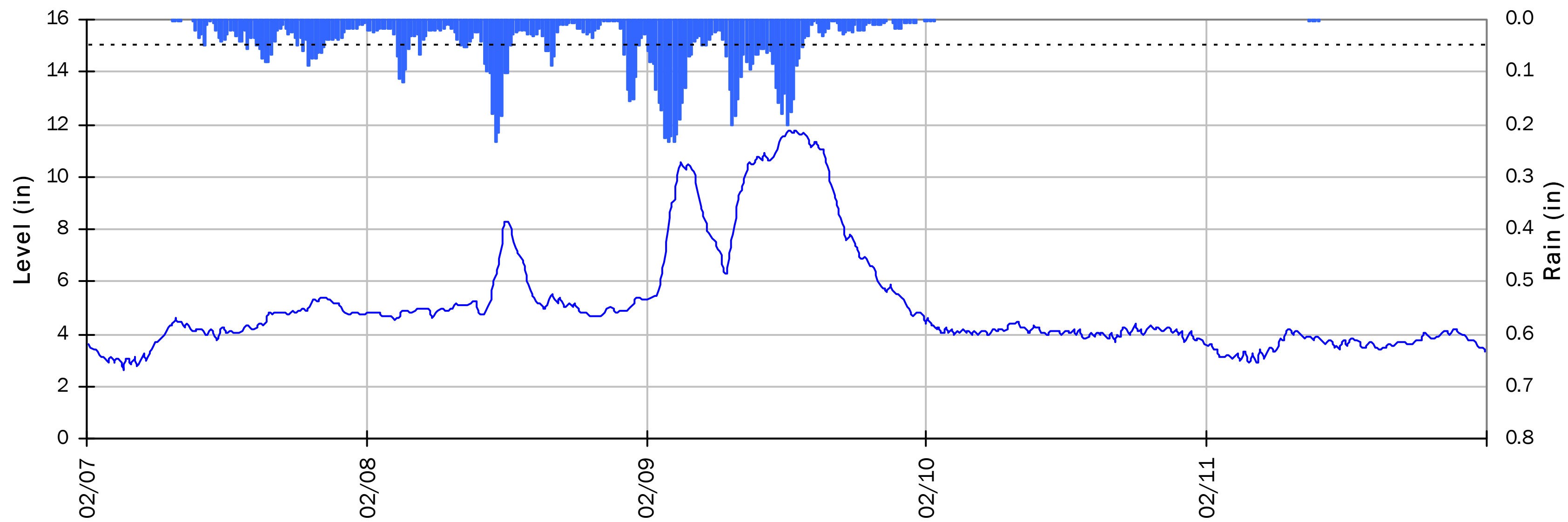
Weekly Level, Velocity and Flow Hydrographs

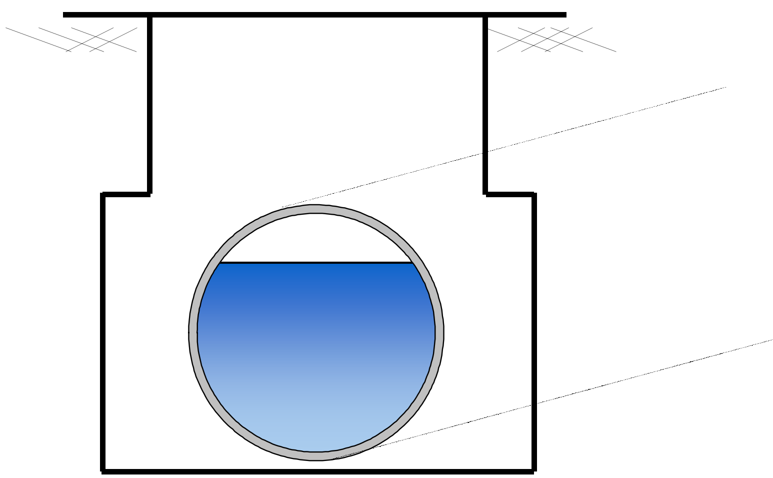


Avg Flow: 0.776 mgd Peak Flow: 2.020 mgd Min Flow: 0.110 mgd

SITE M7
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



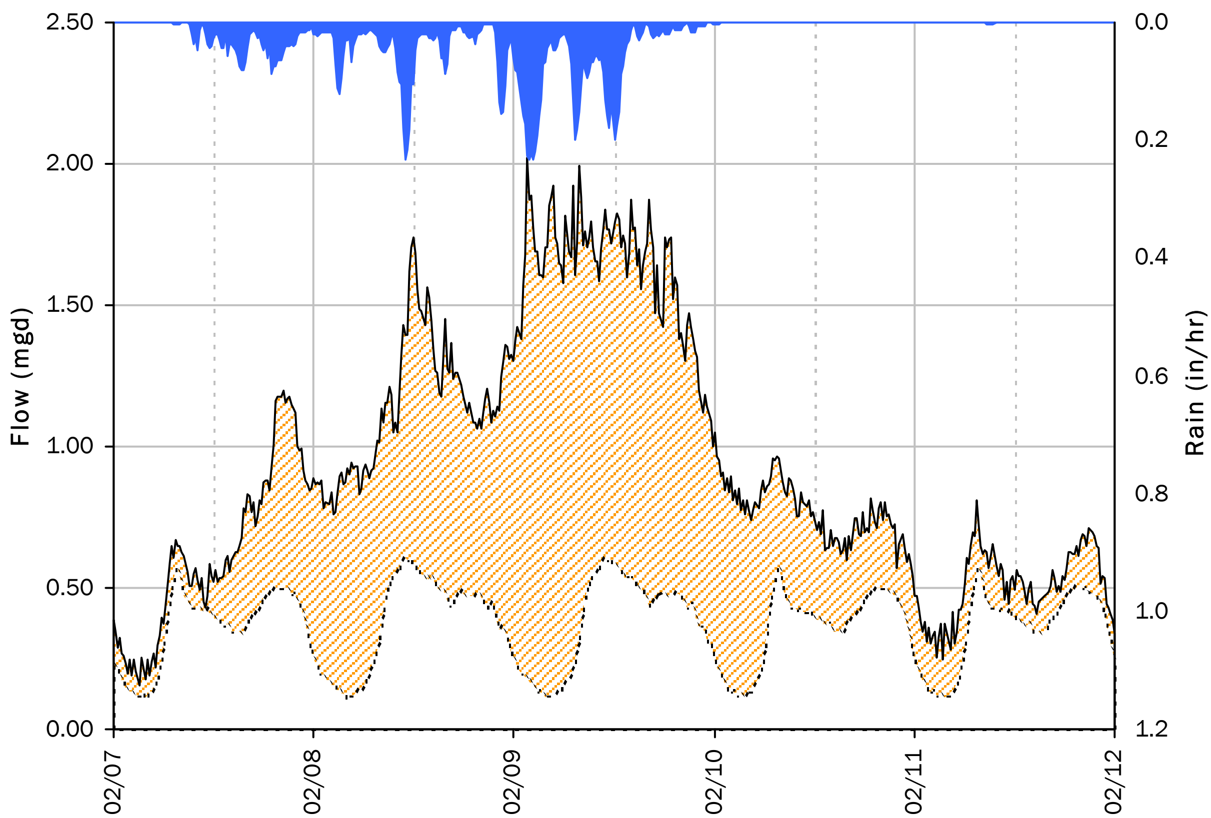


Pipe Diameter:	15	*inches*
Peak Measured Level:	11.8	*inches*
Peak d/D Ratio:	0.78	

SITE M7
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 2.02 *mgd*
PF: 5.56

Peak Level: 11.75 *in*
d/D Ratio: 0.78

Inflow / Infiltration

Peak I/I Rate: 1.85 *mgd*
Total I/I: 2,841,000 *gallons*

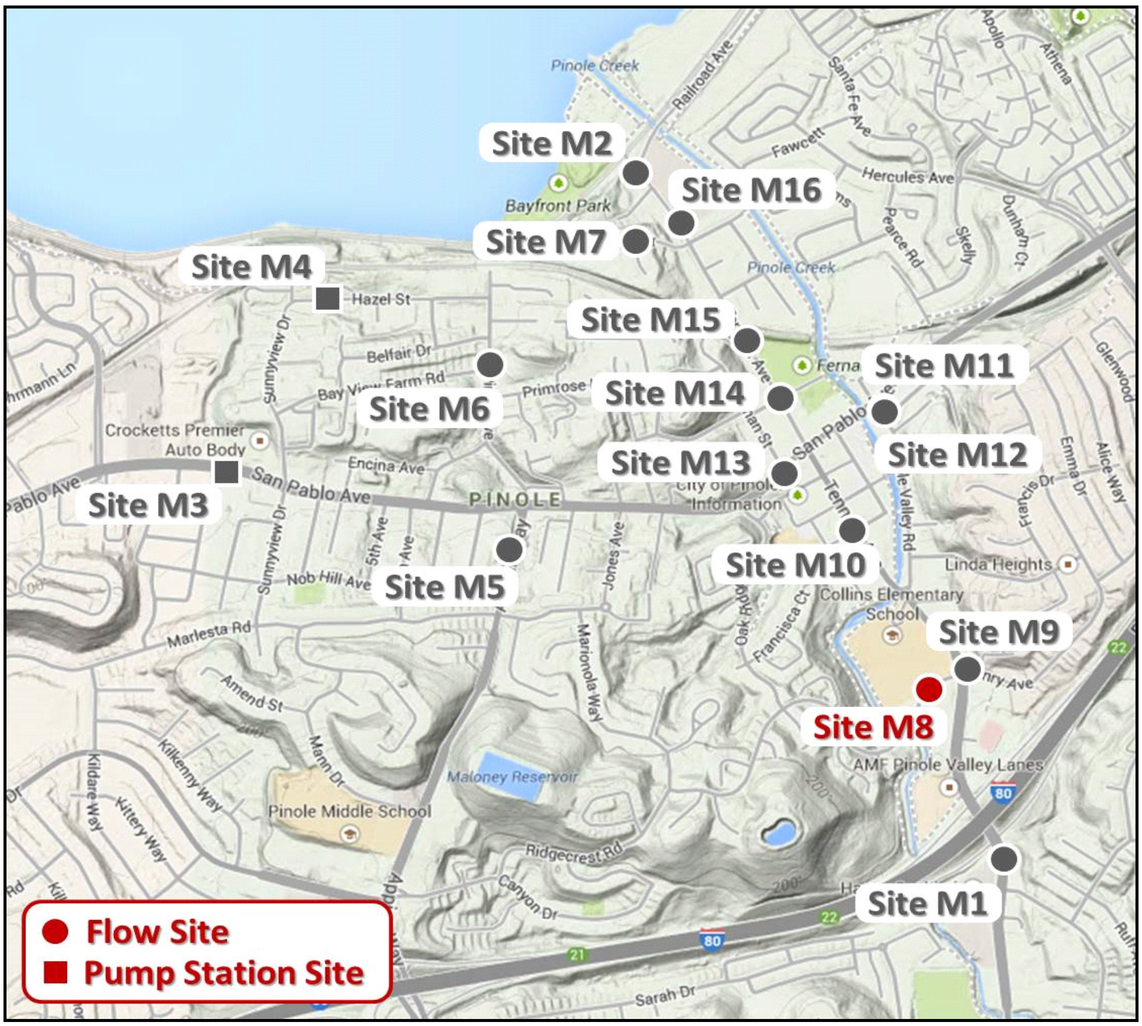
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M8

Location: Henry Ave., west of Pinole Valley Rd.

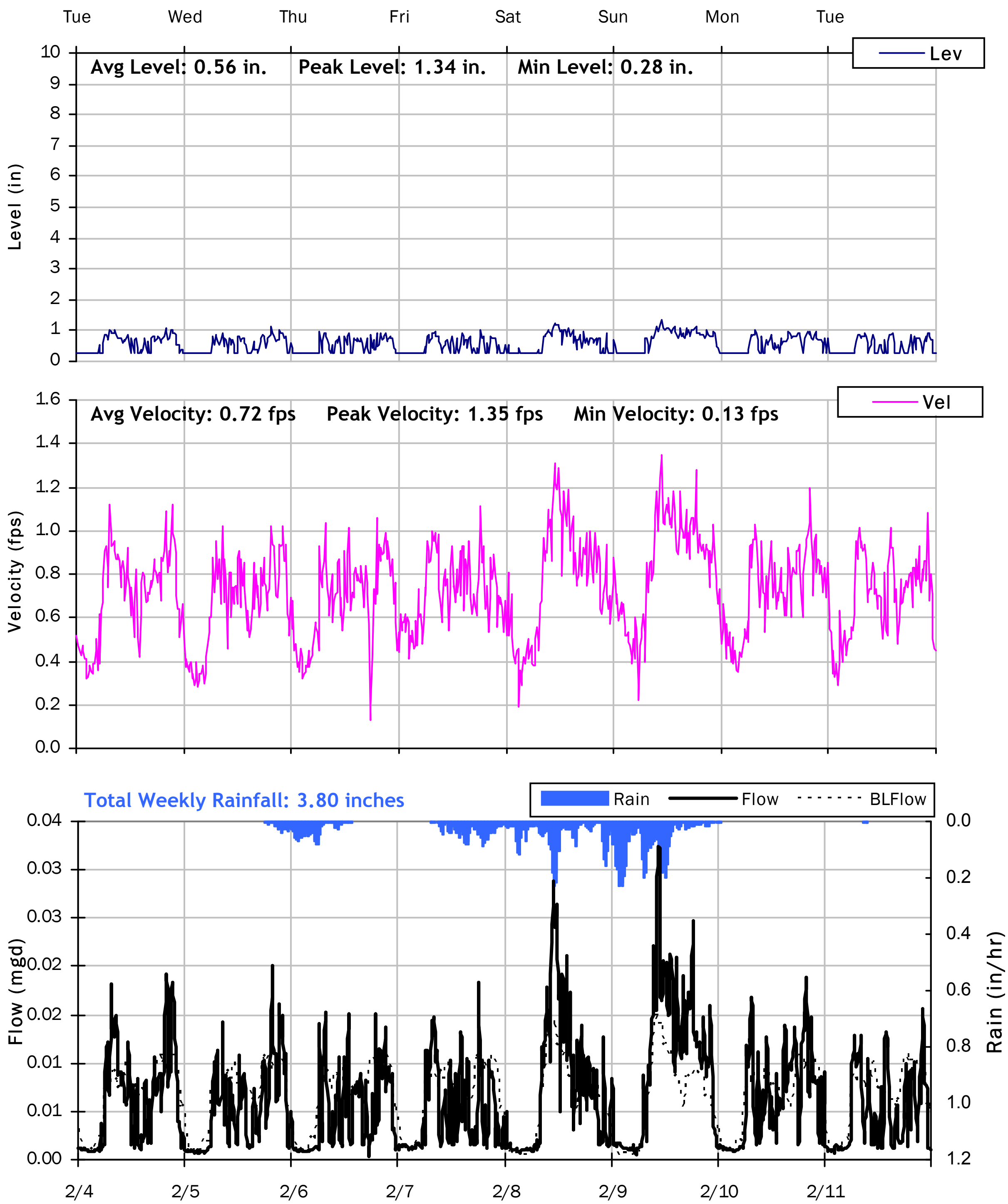
Data Summary Report



Vicinity Map: Site M8

SITE M8

Weekly Level, Velocity and Flow Hydrographs

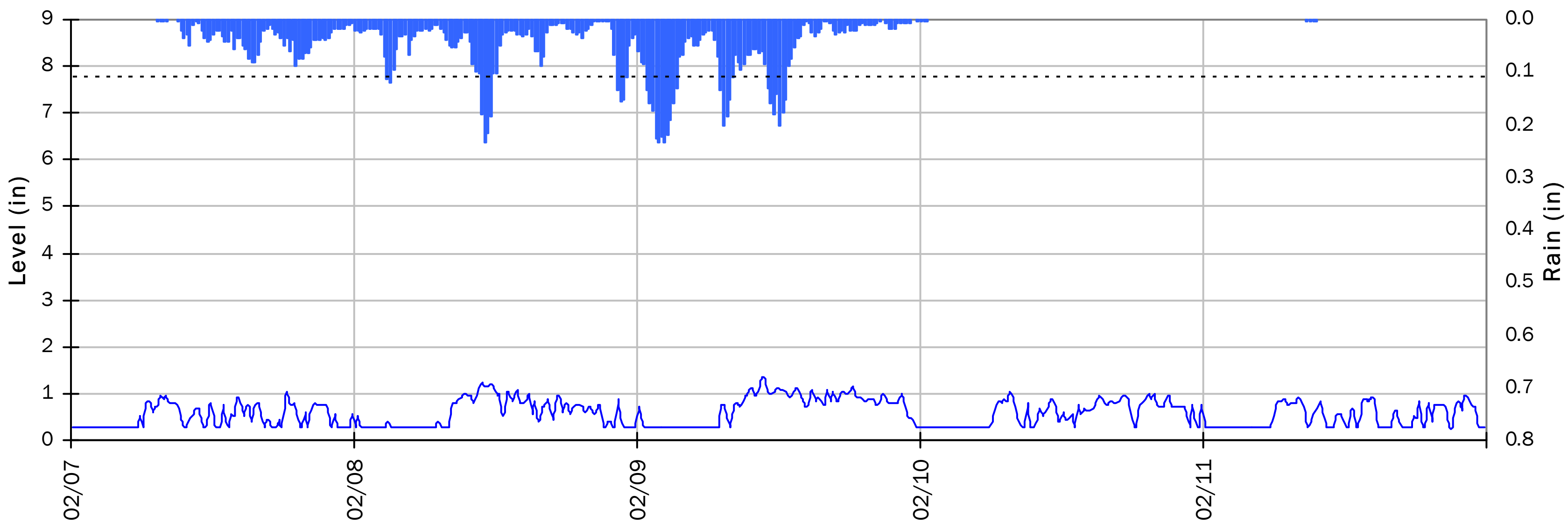


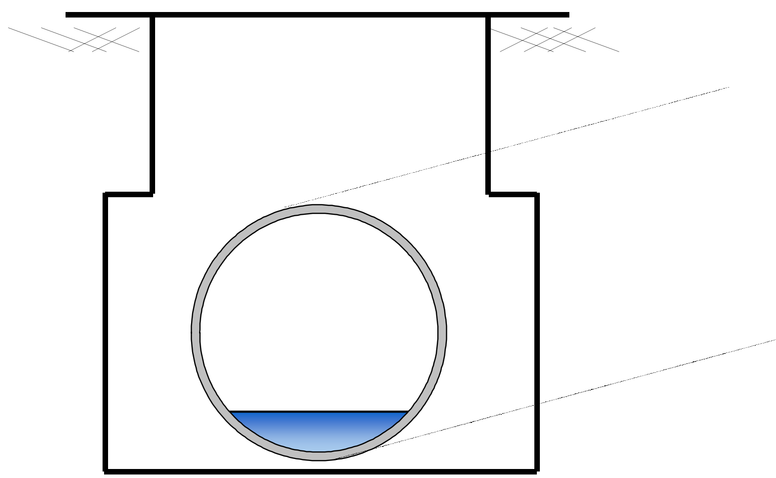
Avg Flow: 0.006 mgd **Peak Flow: 0.032 mgd** **Min Flow: 0.000 mgd**

SITE M8

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



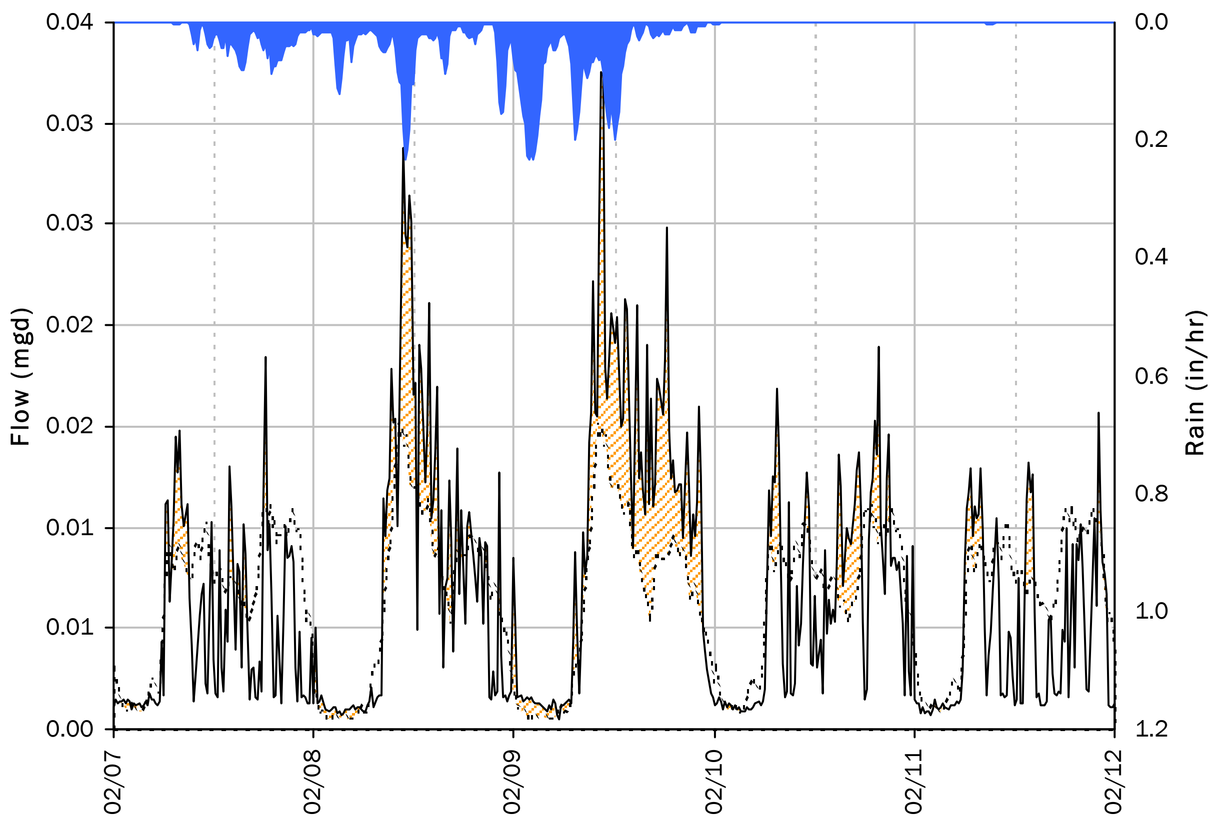


Pipe Diameter:	7.75	*inches*
Peak Measured Level:	1.34	*inches*
Peak d/D Ratio:	0.17	

SITE M8
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow:	0.03 *mgd*
PF:	4.98
Peak Level:	1.34 *in*
d/D Ratio:	0.17

Inflow / Infiltration

Peak I/I Rate:	0.02 *mgd*
Total I/I:	1,000 *gallons*

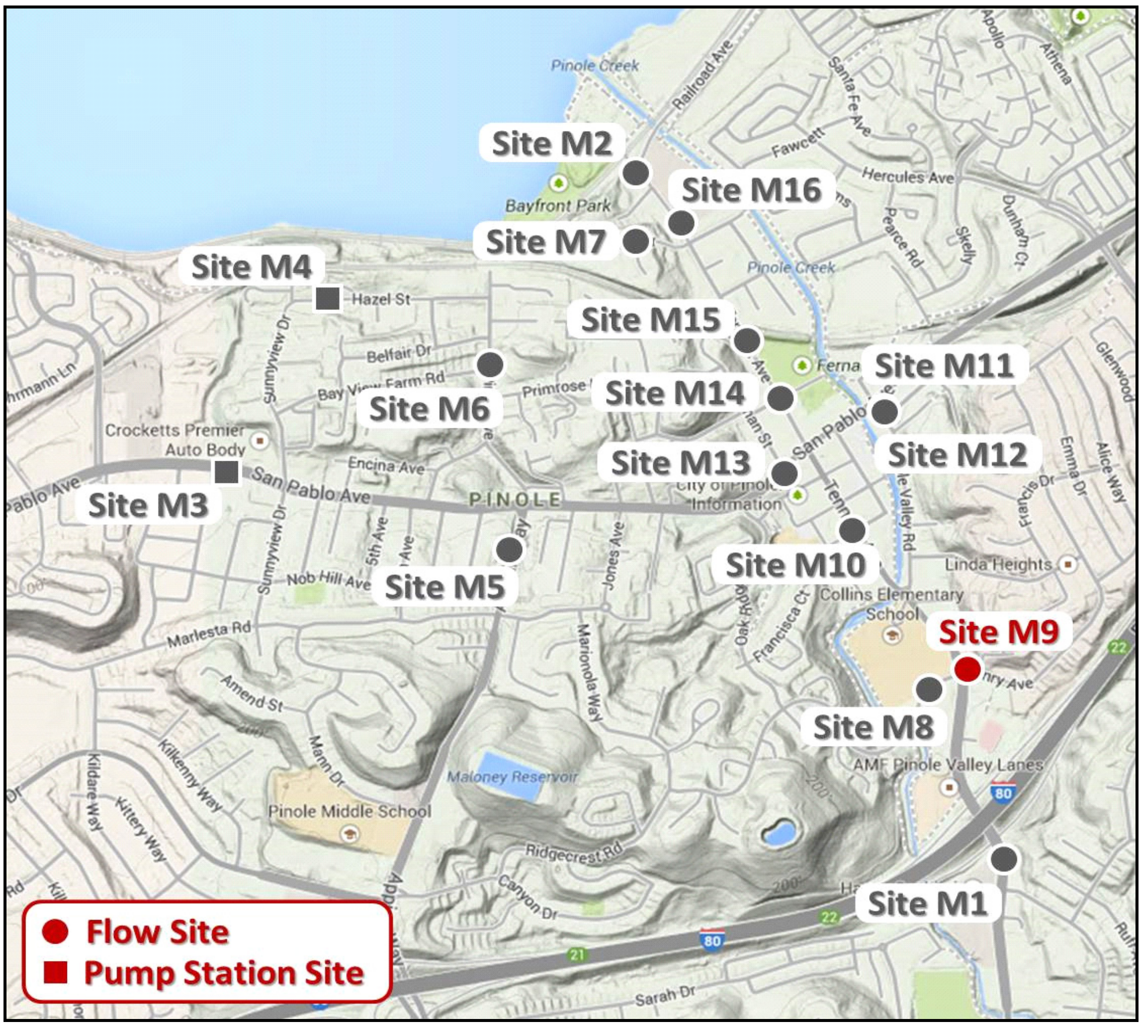
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M9

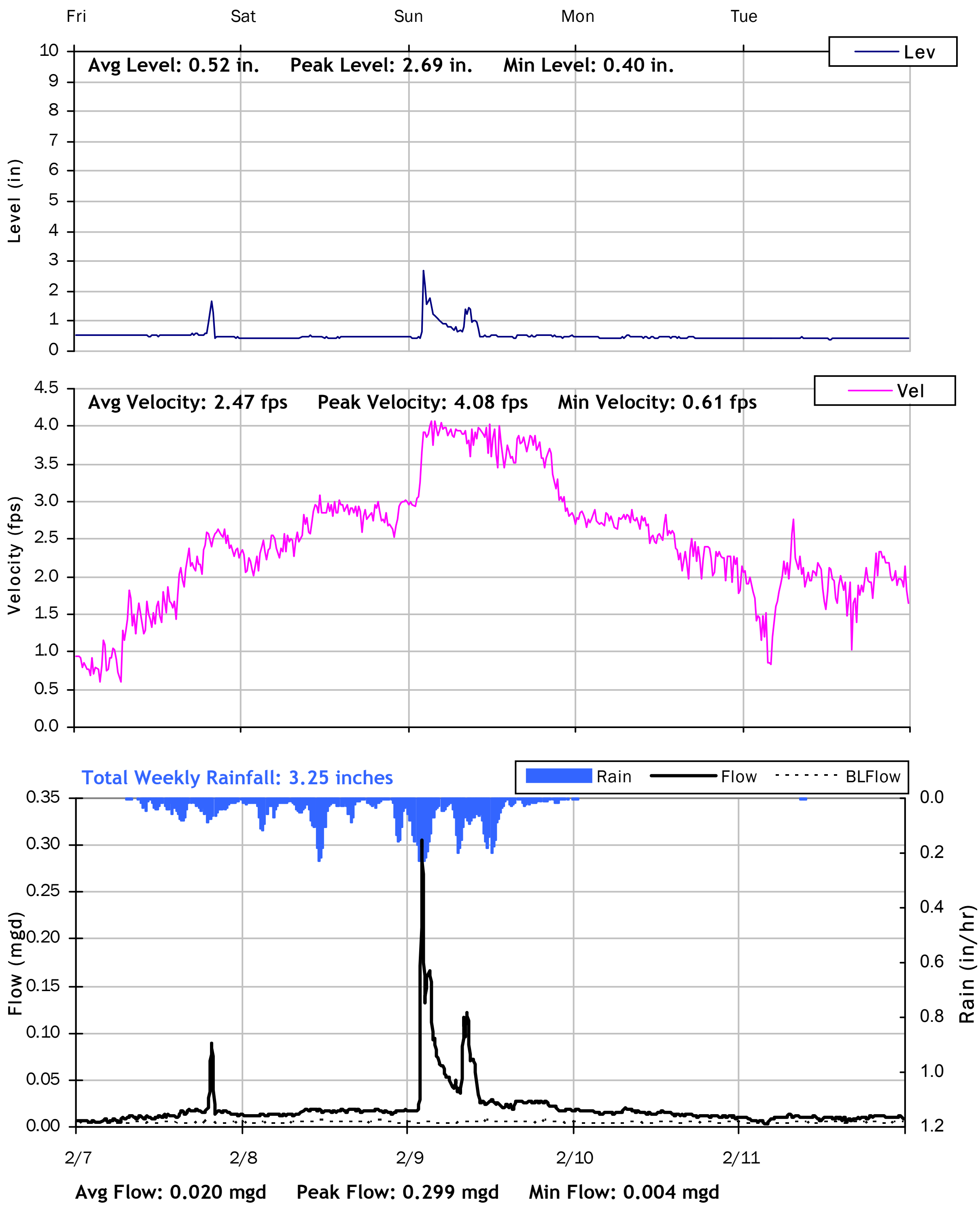
Location: Intersection of Henry Ave. and Pinole Valley Rd.

Data Summary Report



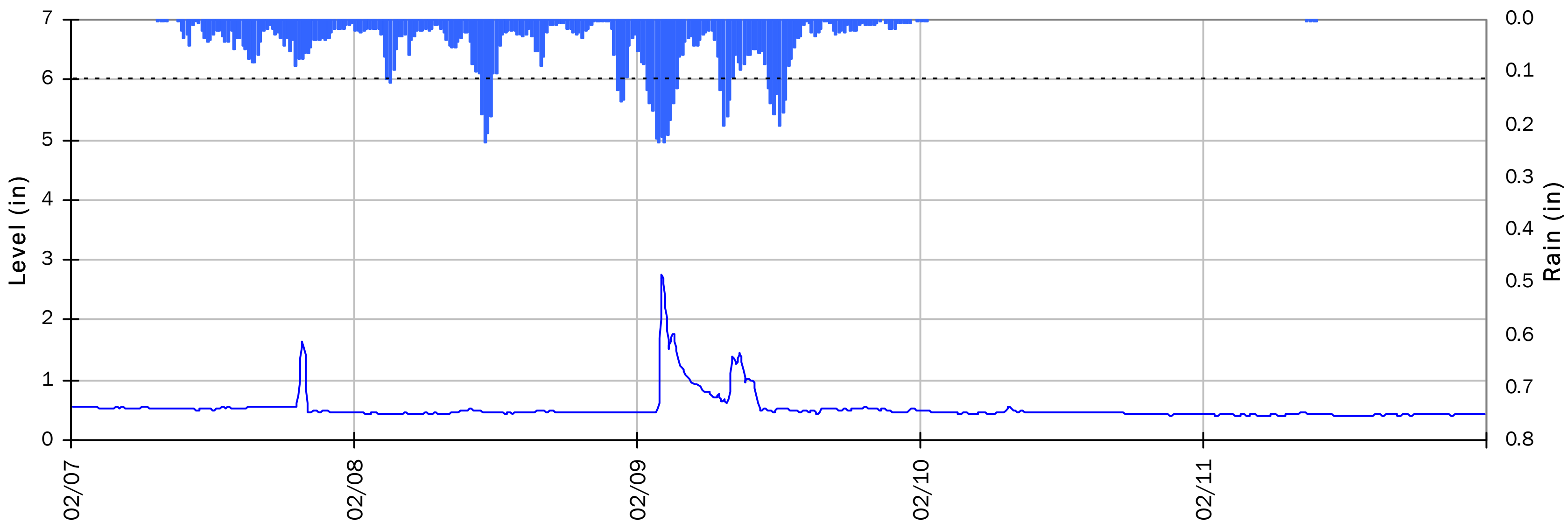
Vicinity Map: Site M9

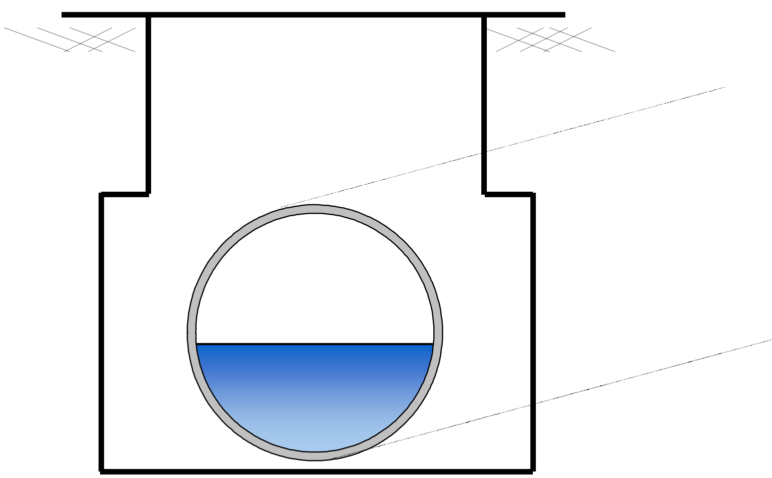
SITE M9
Weekly Level, Velocity and Flow Hydrographs



SITE M9
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



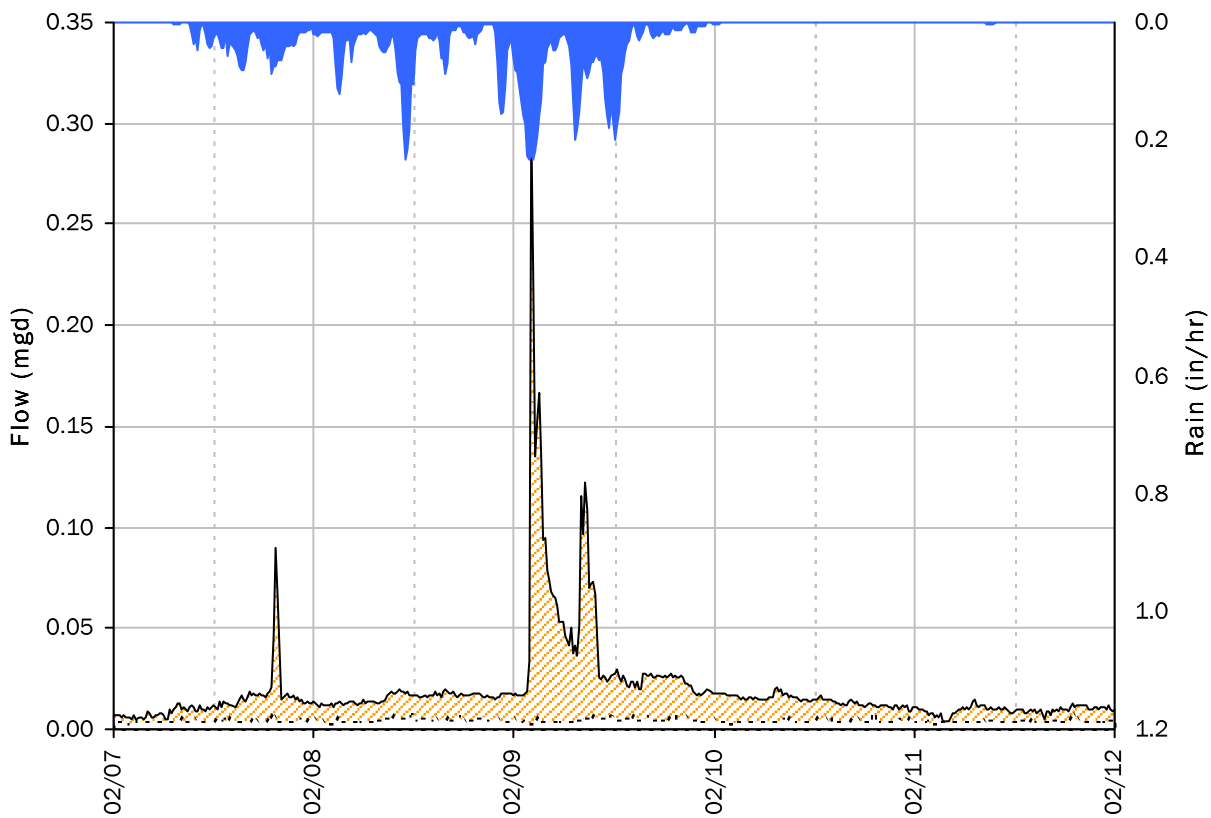


Pipe Diameter:	6	*inches*
Peak Measured Level:	2.69	*inches*
Peak d/D Ratio:	0.45	

SITE M9
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.30 *mgd*
PF: 67.45

Peak Level: 2.69 *in*
d/D Ratio: 0.45

Inflow / Infiltration

Peak I/I Rate: 0.30 *mgd*
Total I/I: 76,000 *gallons*

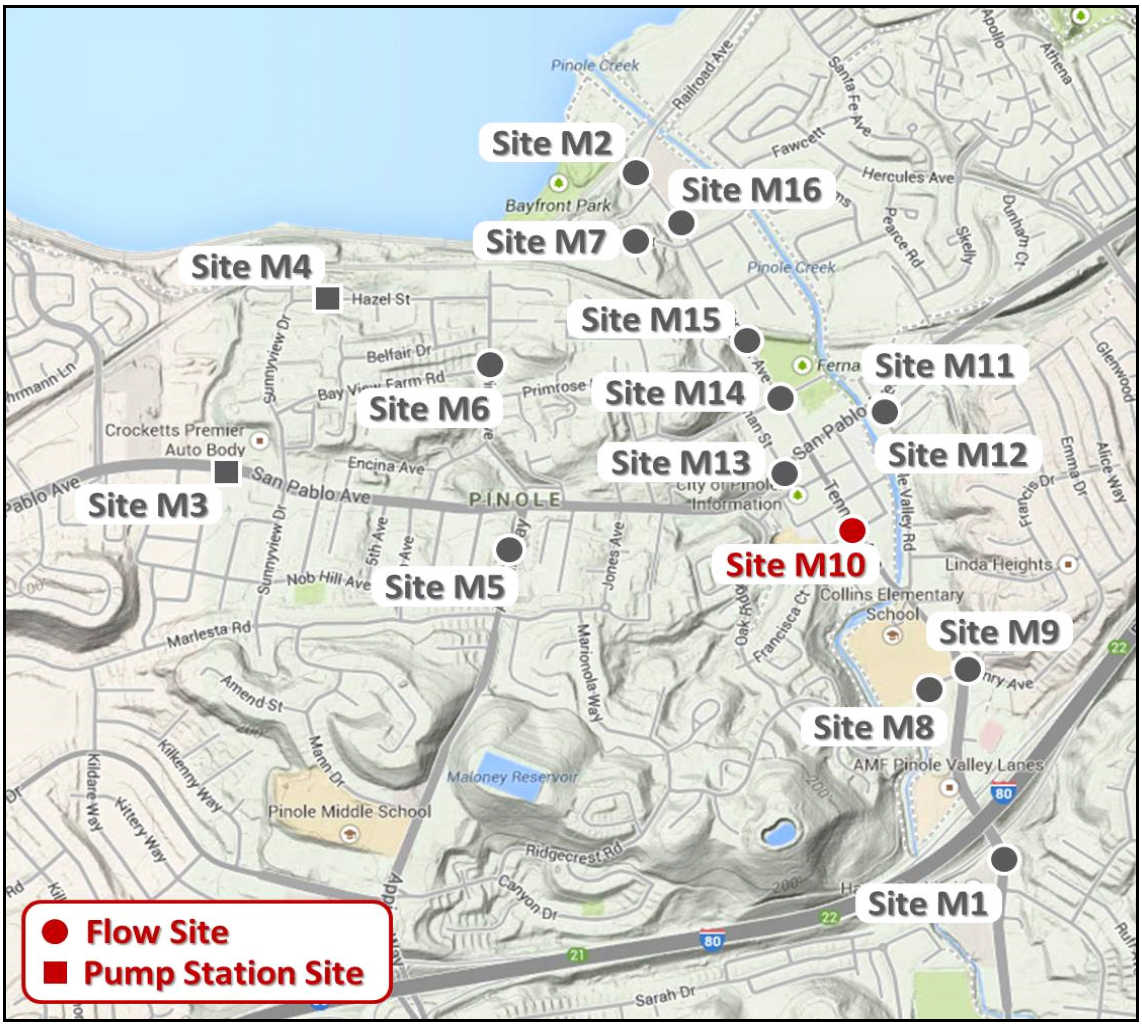
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M10

Location: Intersection of Tennant Ave. and Prune St.

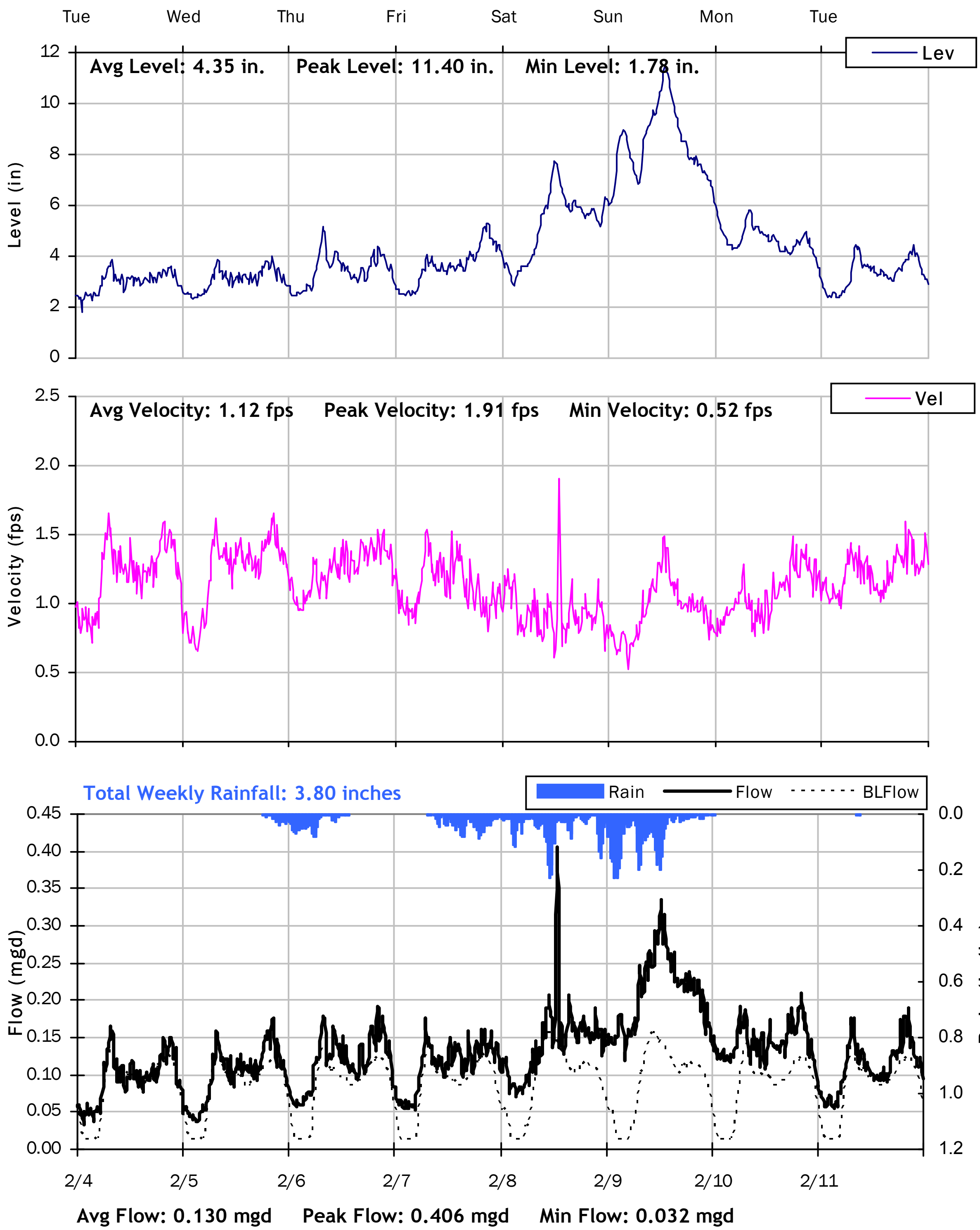
Data Summary Report



Vicinity Map: Site M10

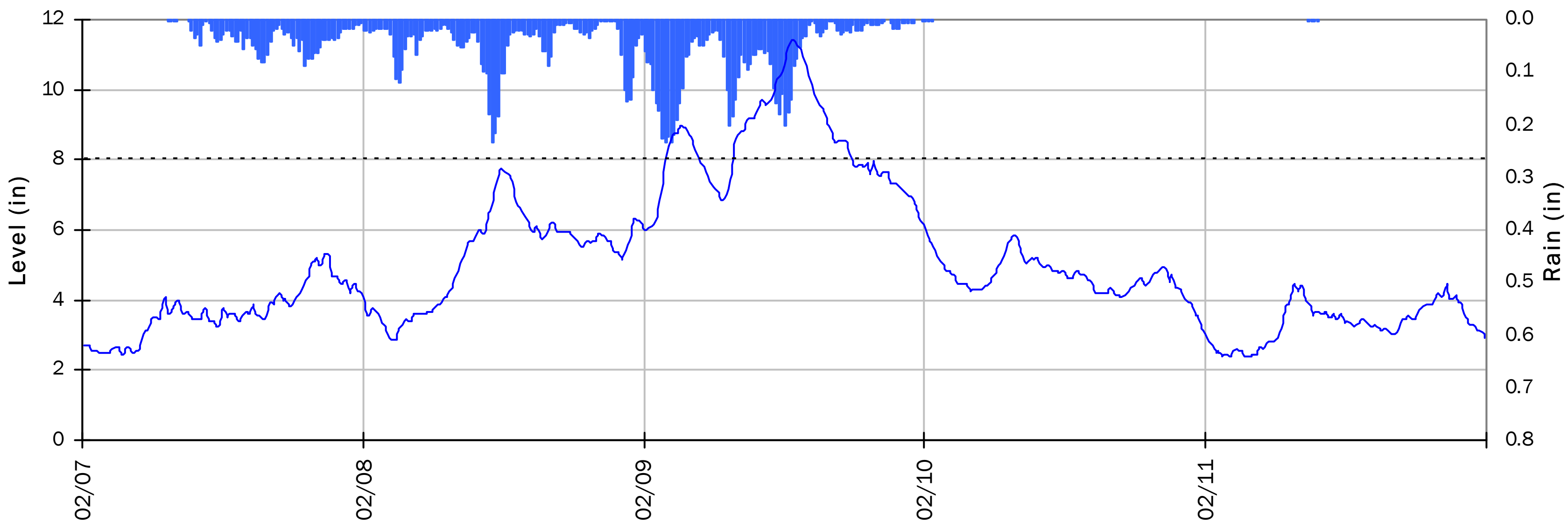
SITE M10

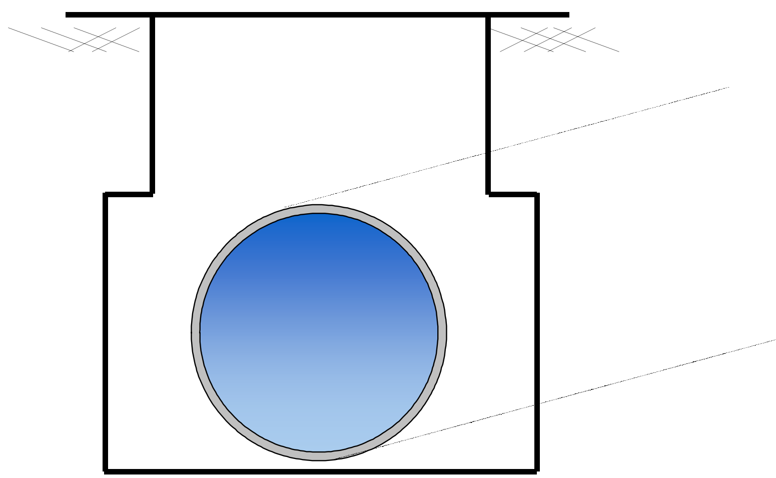
Weekly Level, Velocity and Flow Hydrographs



SITE M10
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





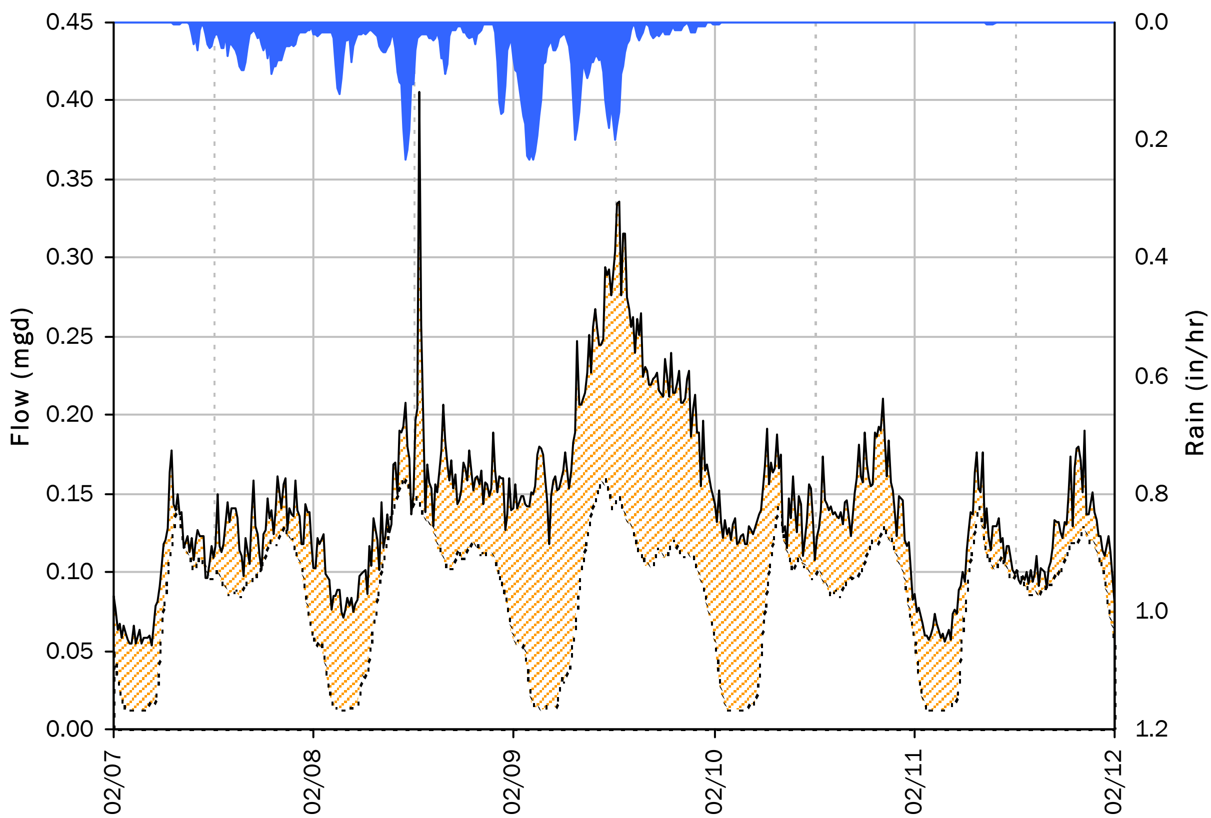
Pipe Diameter:	8	*inches*
Peak Measured Level:	11.4	*inches*
Peak d/D Ratio:	1.43	

Surcharged 3.4 inches over crown

SITE M10
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow:	0.41 *mgd*
PF:	4.76
Peak Level:	11.40 *in*
d/D Ratio:	1.43

Inflow / Infiltration

Peak I/I Rate:	0.26 *mgd*
Total I/I:	291,000 *gallons*

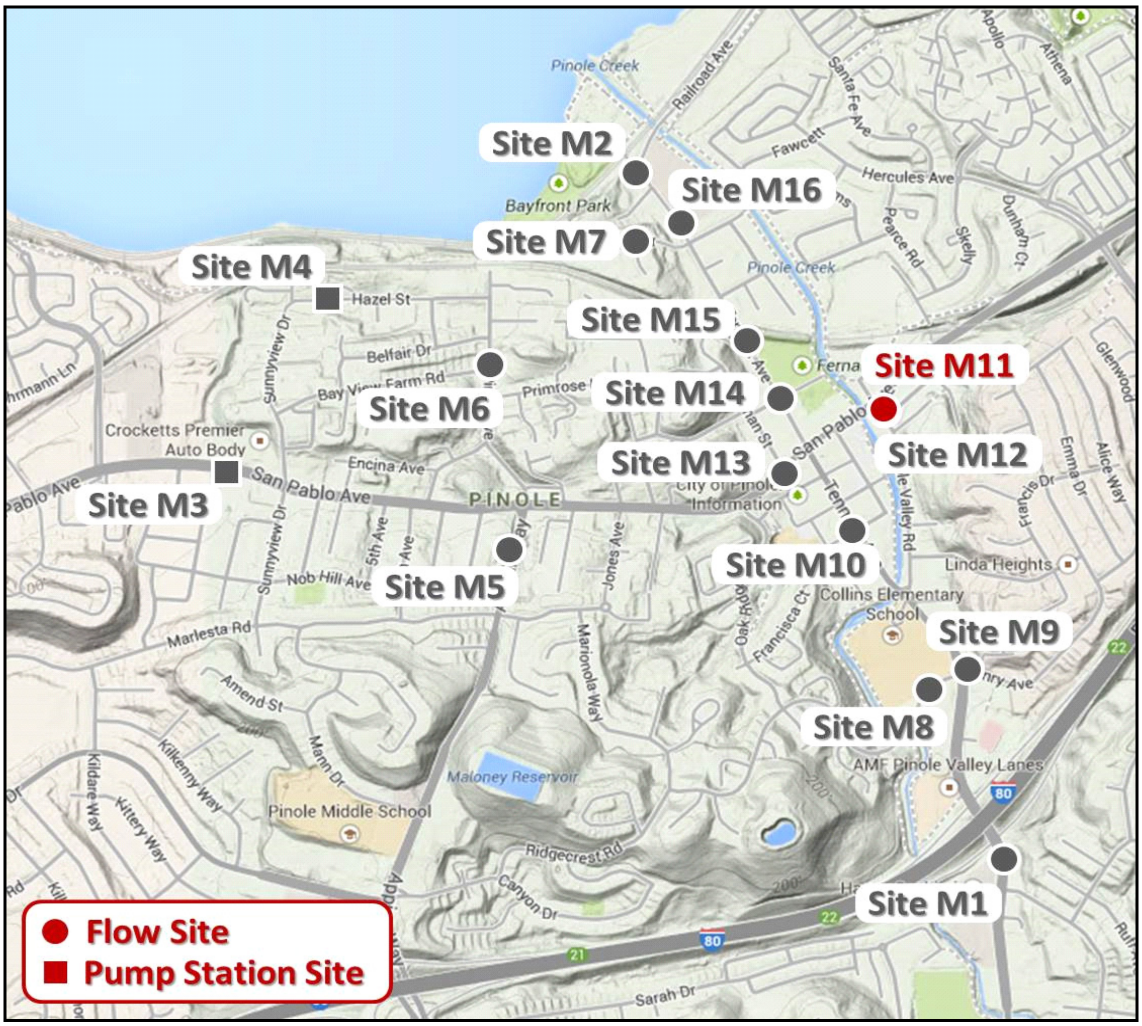
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M11

Location: Intersection of Pinole Valley Rd. and Rafaela St.

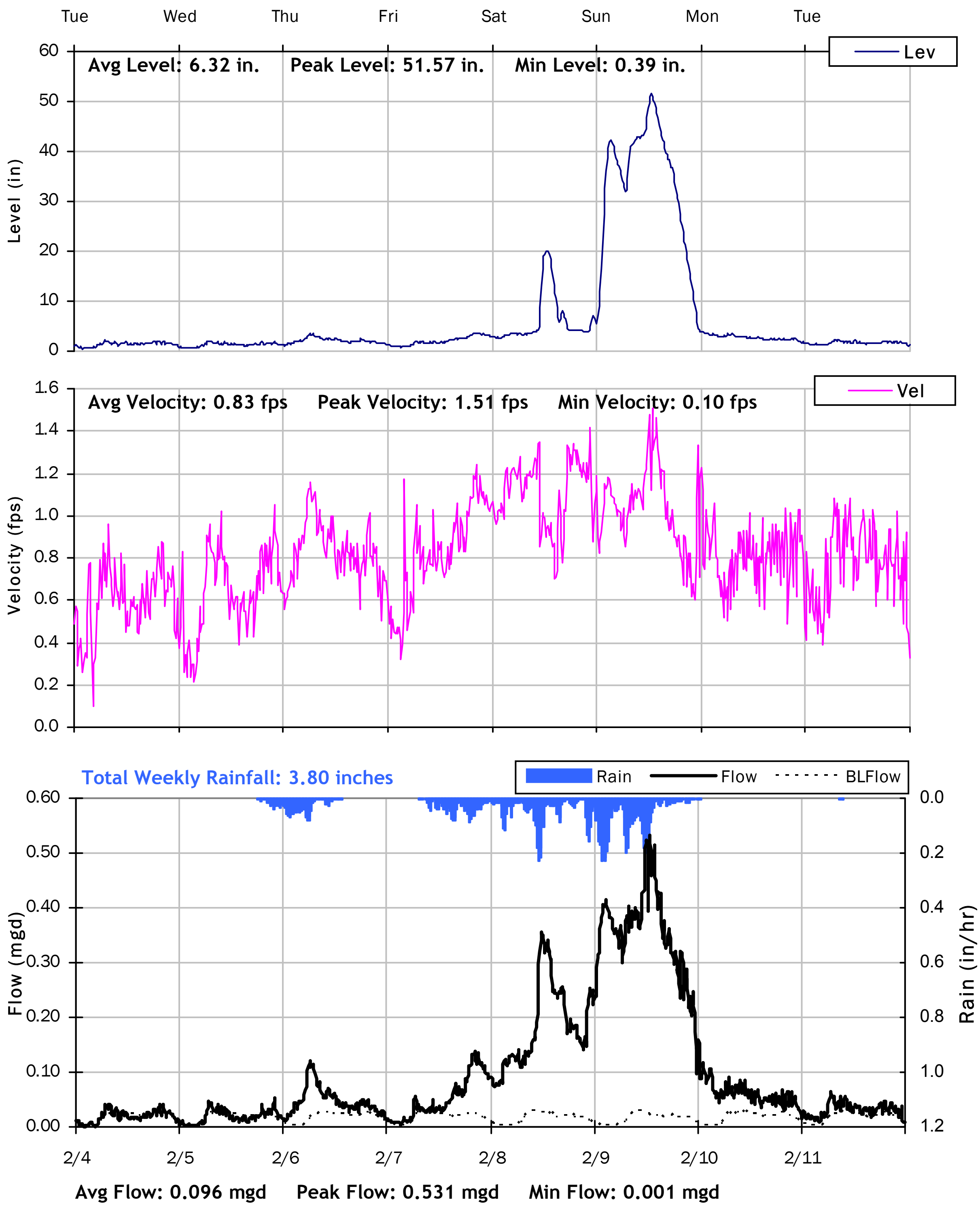
Data Summary Report



Vicinity Map: Site M11

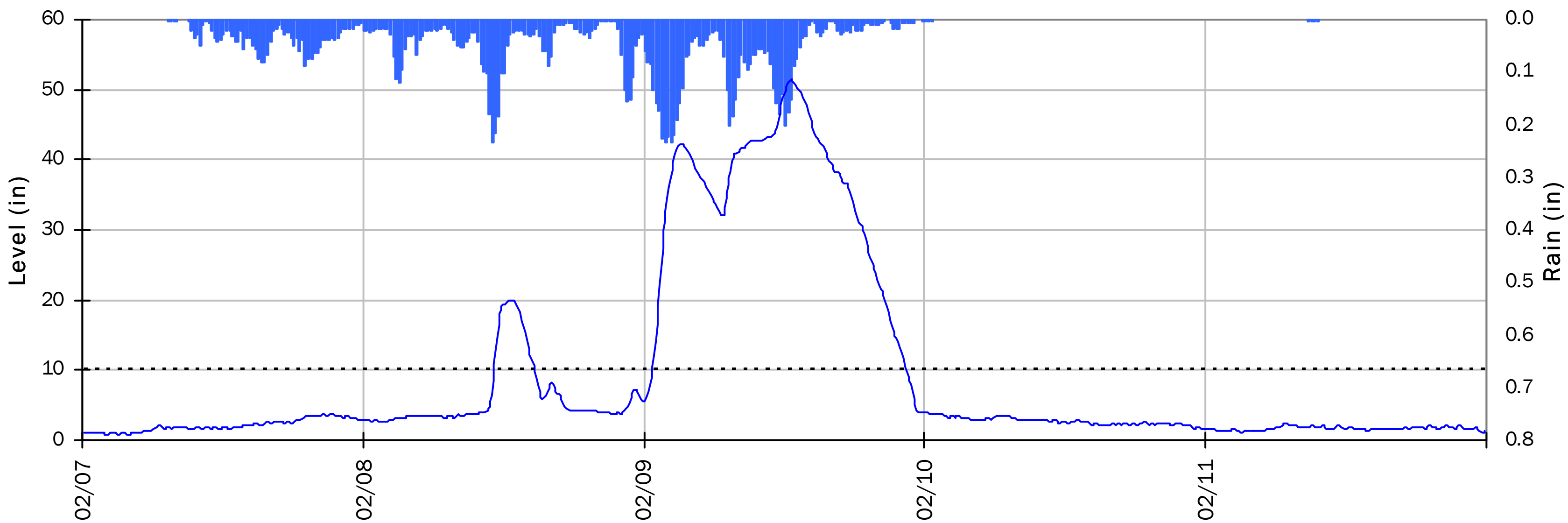
SITE M11

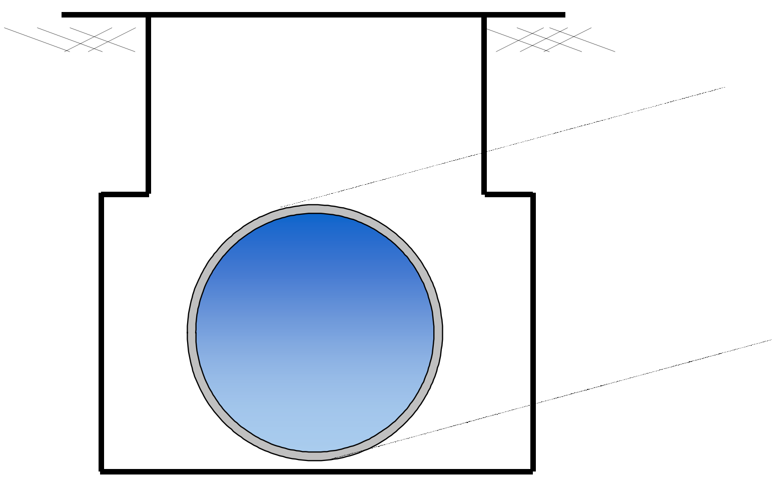
Weekly Level, Velocity and Flow Hydrographs



SITE M11
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





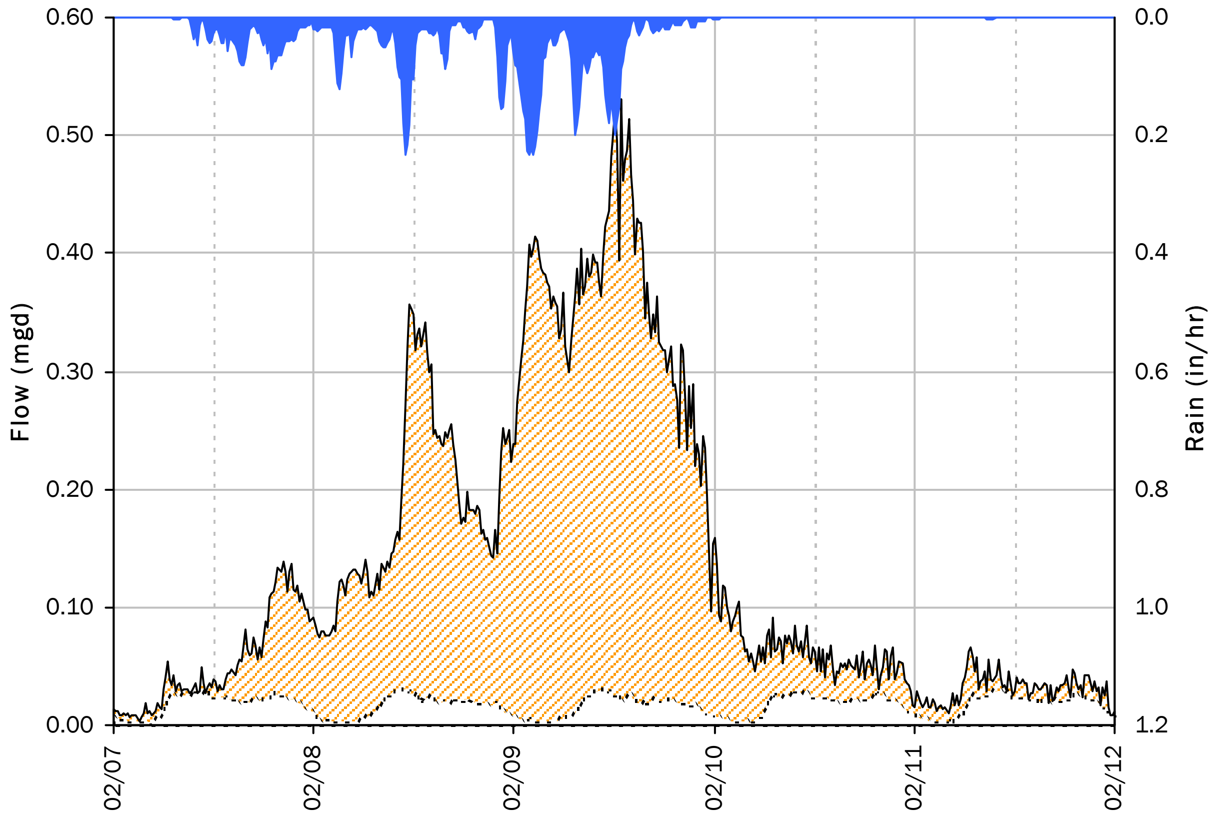
Pipe Diameter:	10	*inches*
Peak Measured Level:	51.6	*inches*
Peak d/D Ratio:	5.16	

Surcharged 41.6 inches over crown

SITE M11
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.53 *mgd*
PF: 30.59

Peak Level: 51.57 *in*
d/D Ratio: 5.16

Inflow / Infiltration

Peak I/I Rate: 0.51 *mgd*
Total I/I: 594,000 *gallons*

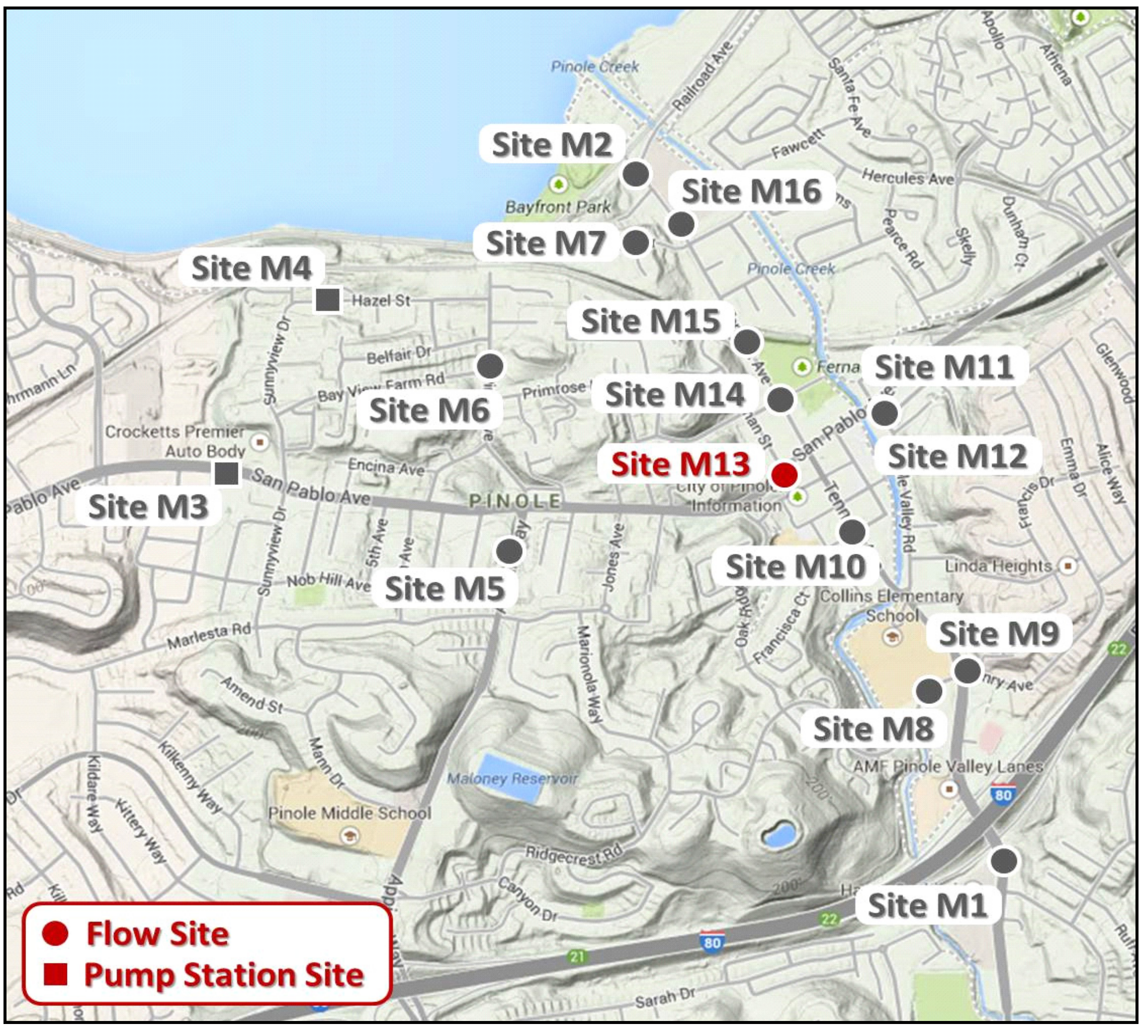
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M13

Location: San Pablo Ave. just west of Quinan St.

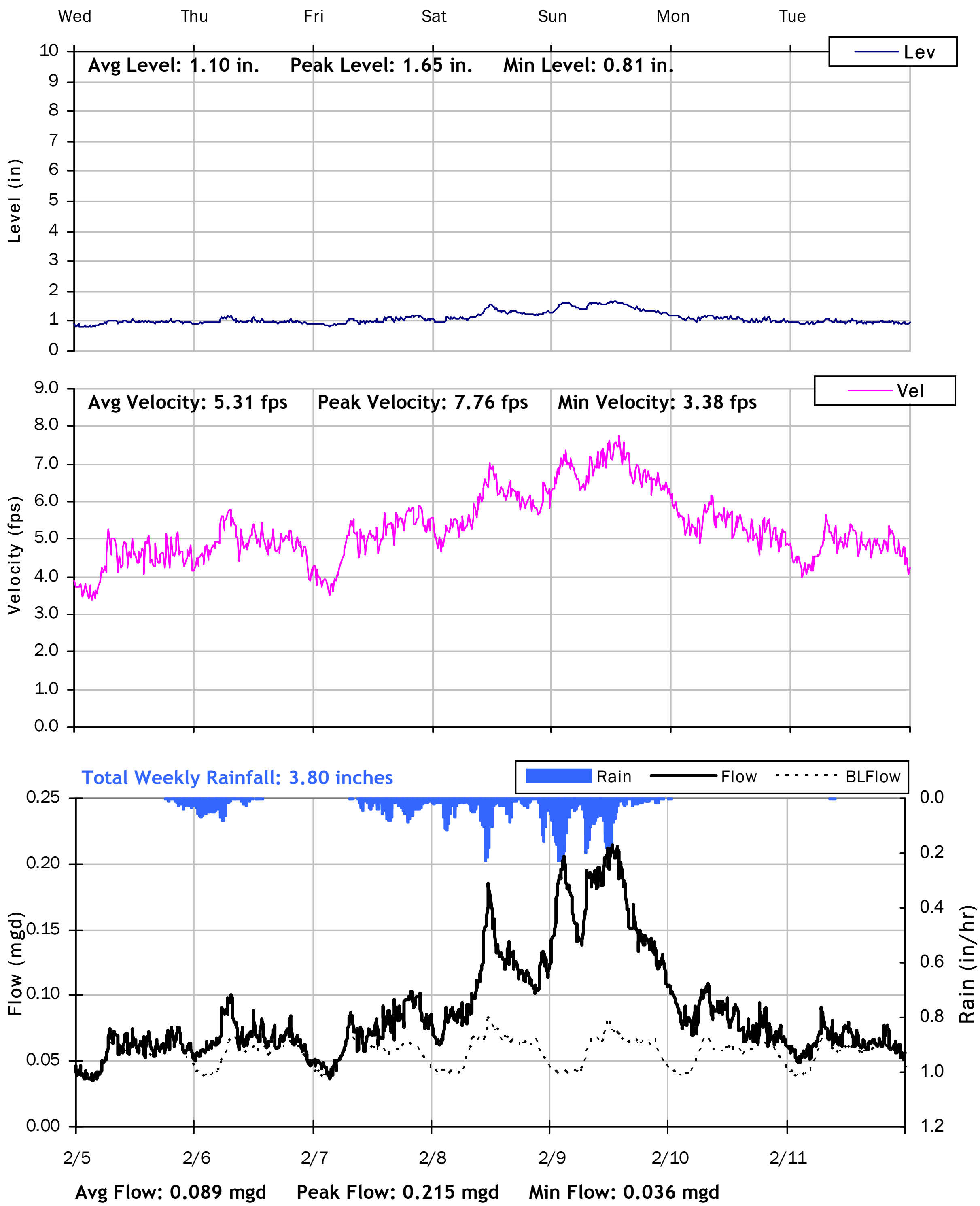
Data Summary Report



Vicinity Map: Site M13

SITE M13

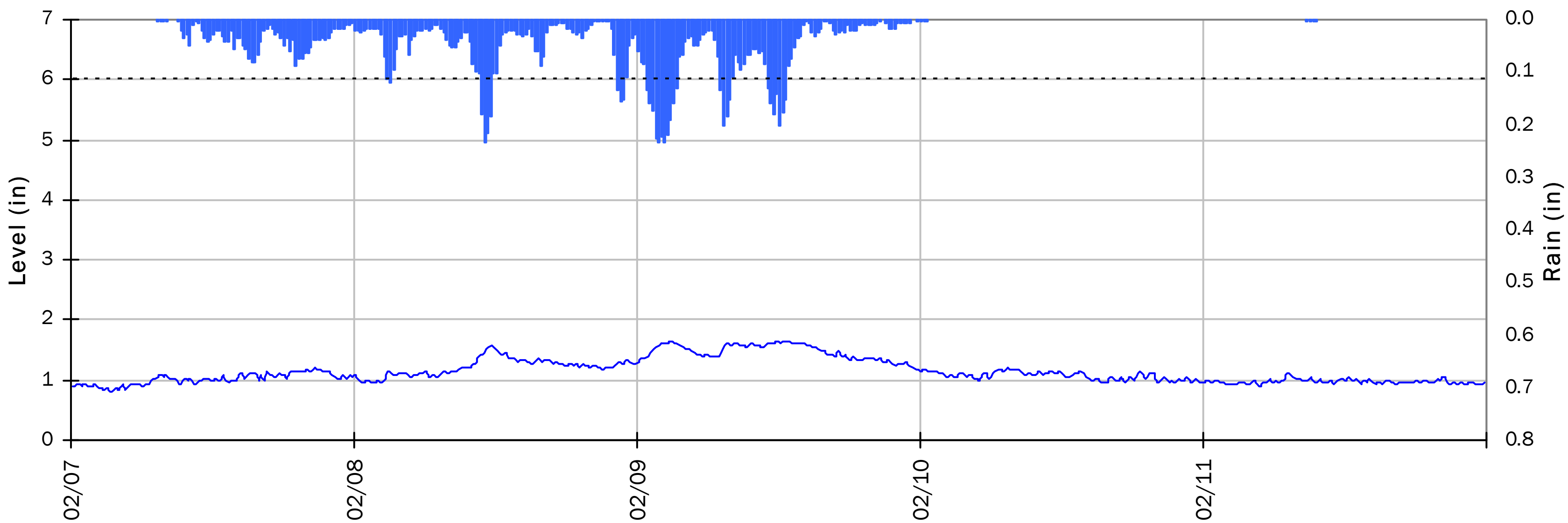
Weekly Level, Velocity and Flow Hydrographs

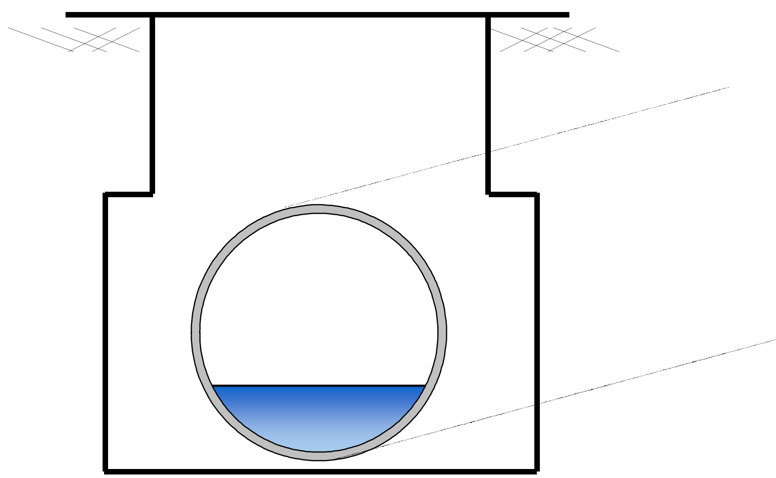


SITE M13

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



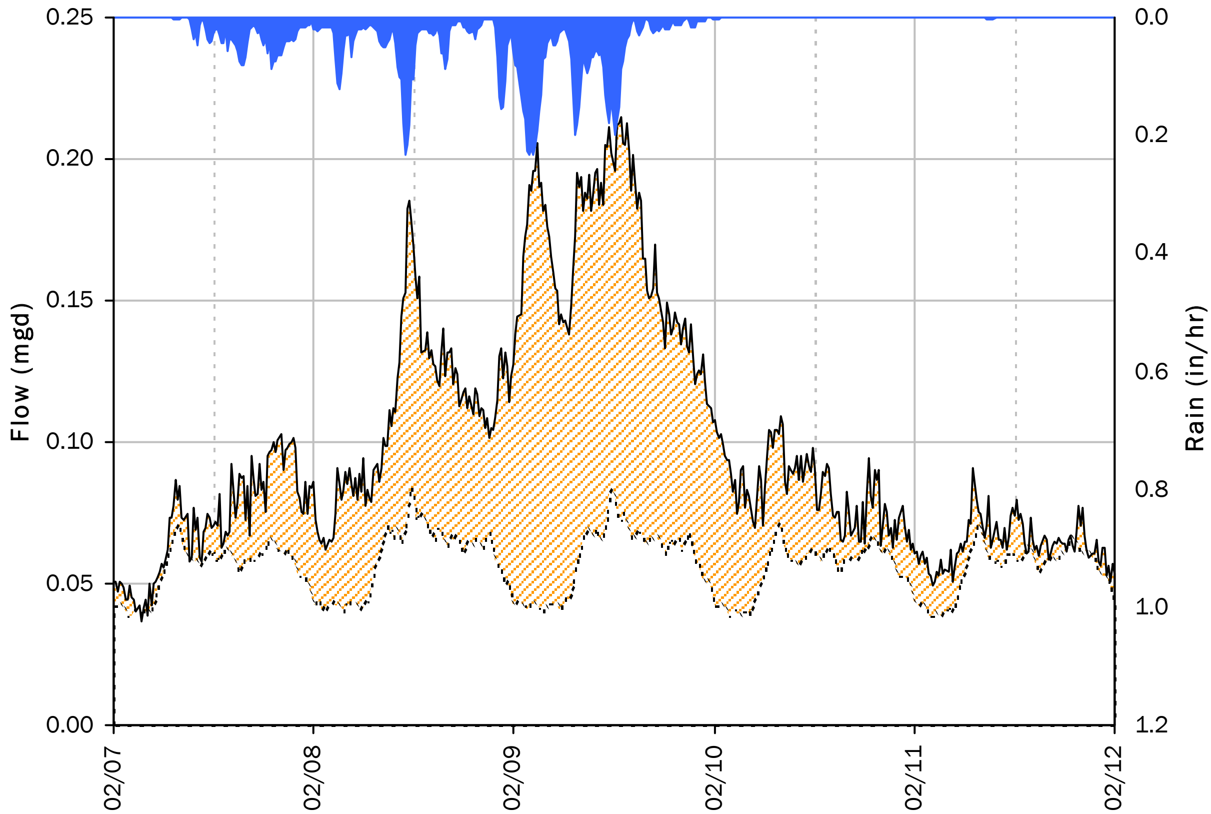


Pipe Diameter:	6	*inches*
Peak Measured Level:	1.65	*inches*
Peak d/D Ratio:	0.28	

SITE M13
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.21 *mgd*
PF: 3.84

Peak Level: 1.65 *in*
d/D Ratio: 0.28

Inflow / Infiltration

Peak I/I Rate: 0.16 *mgd*
Total I/I: 214,000 *gallons*

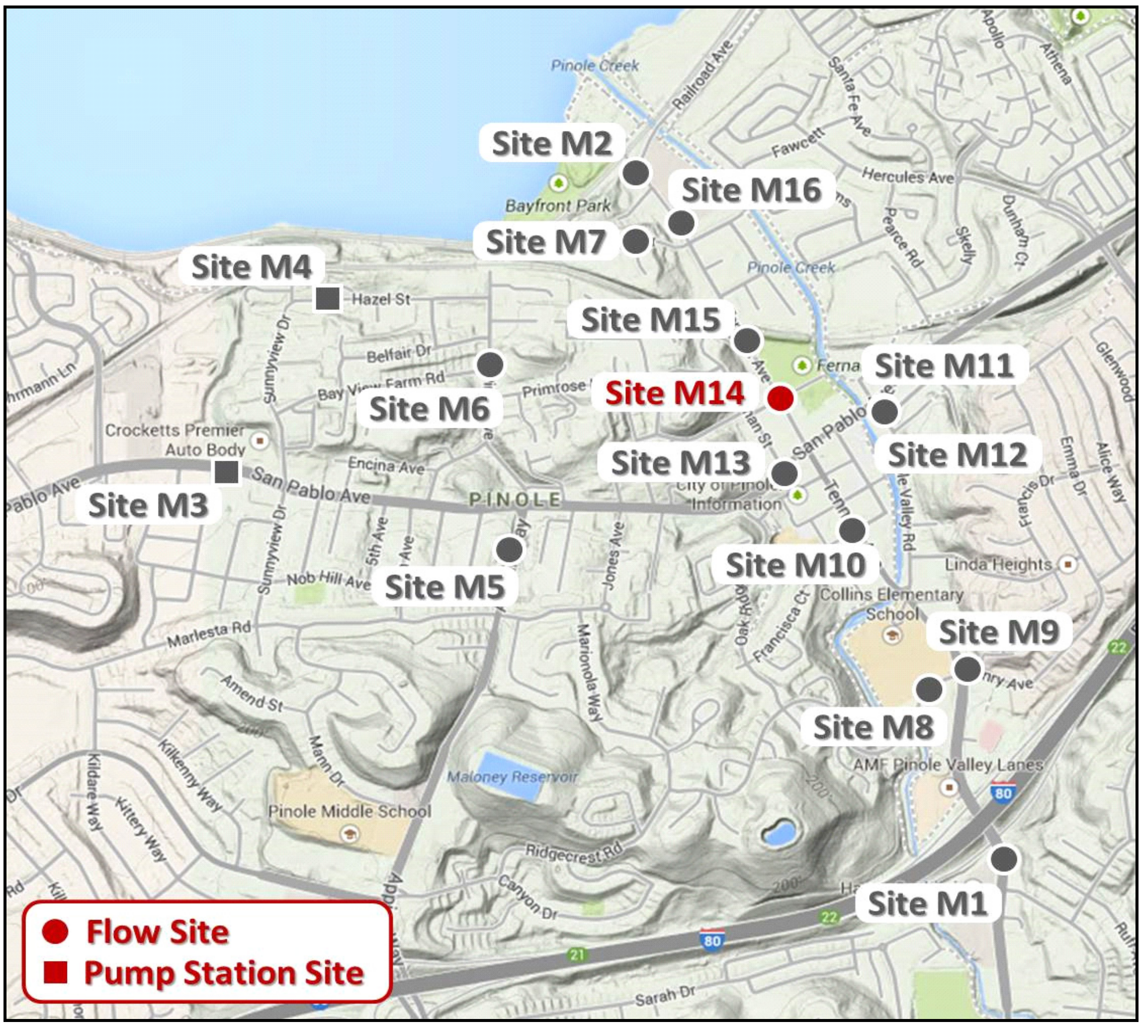
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M14

Location: Intersection of Tennant Ave. and Park St.

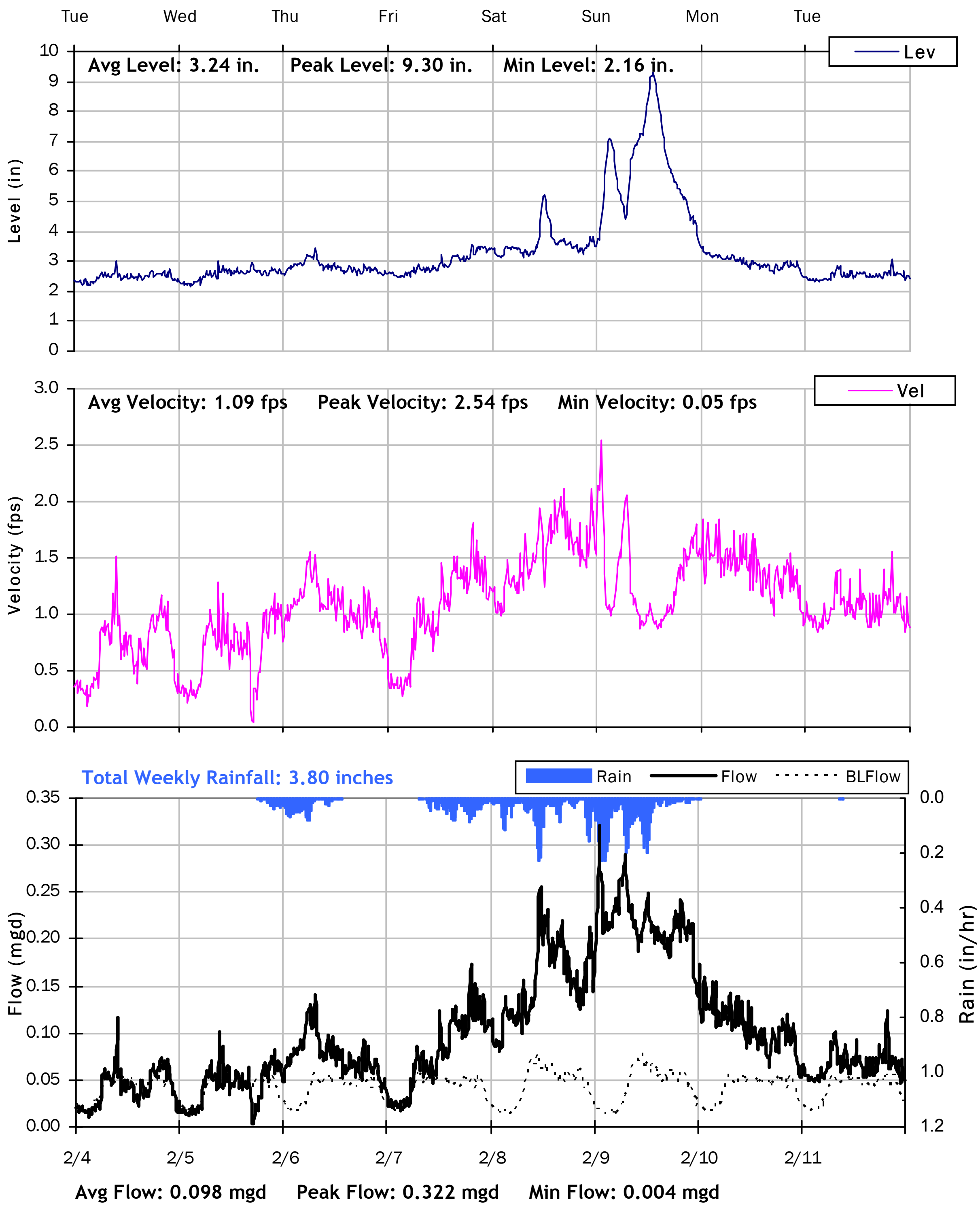
Data Summary Report



Vicinity Map: Site M14

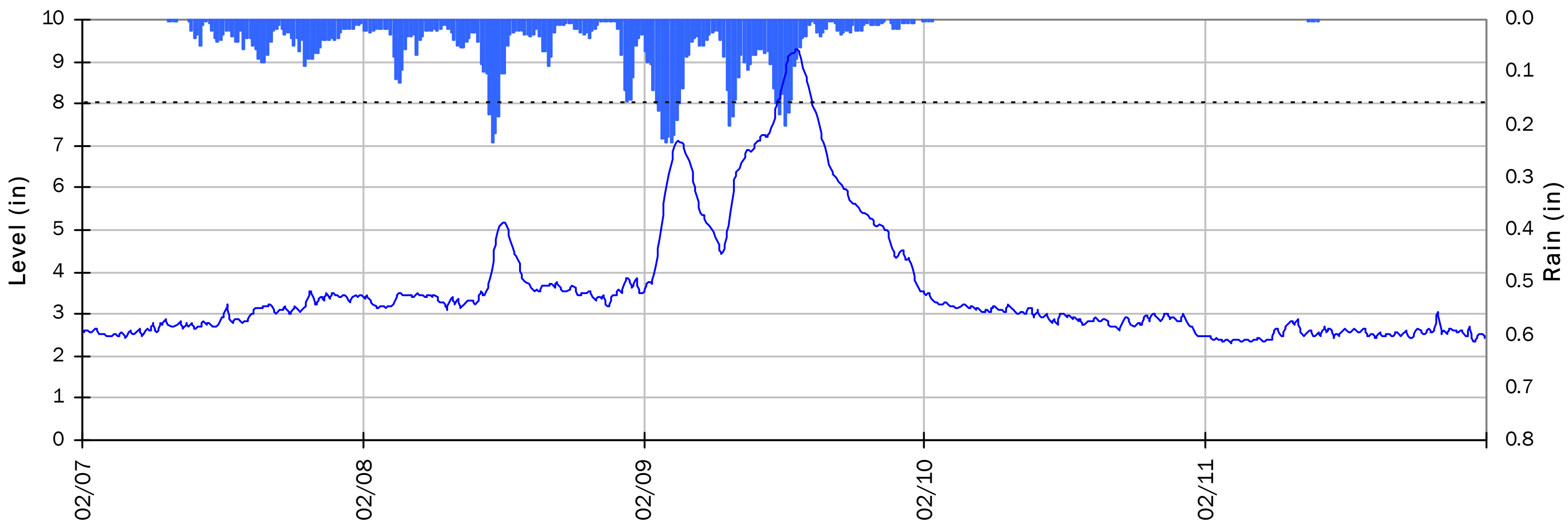
SITE M14

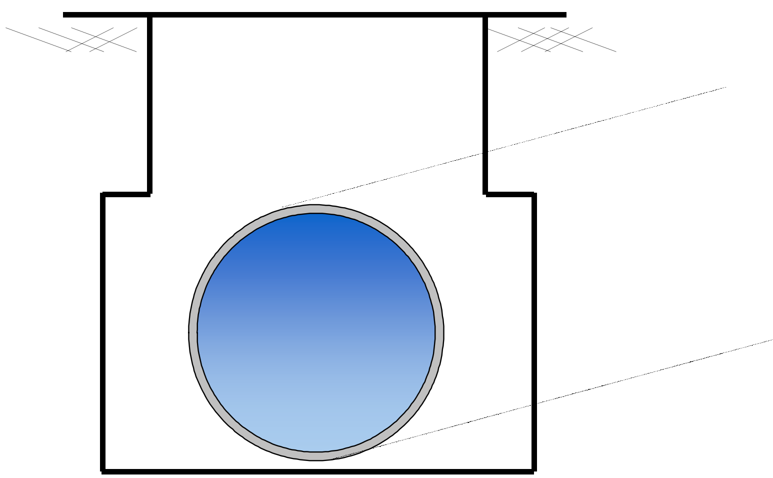
Weekly Level, Velocity and Flow Hydrographs



SITE M14
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





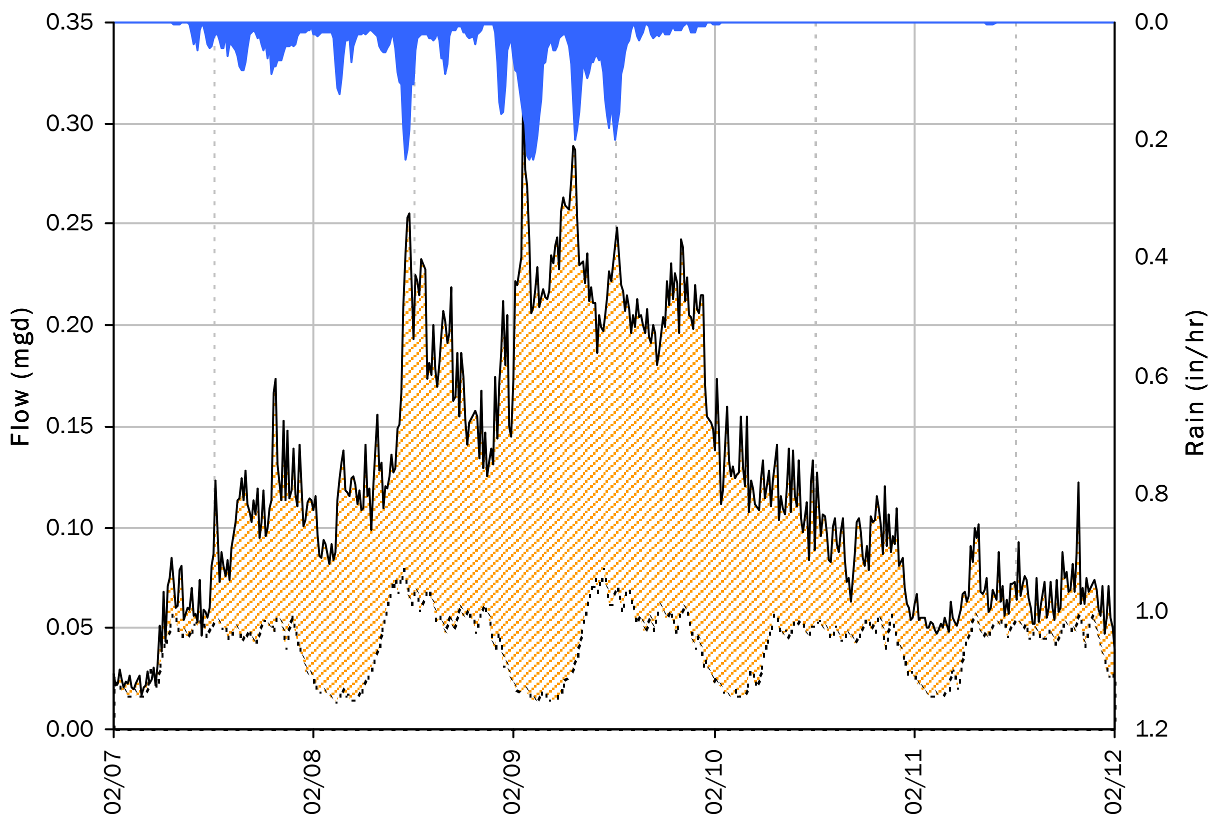
Pipe Diameter:	8	*inches*
Peak Measured Level:	9.3	*inches*
Peak d/D Ratio:	1.16	

Surcharged 1.3 inches over crown

SITE M14
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.32 *mgd*
PF: 7.68

Peak Level: 9.30 *in*
d/D Ratio: 1.16

Inflow / Infiltration

Peak I/I Rate: 0.30 *mgd*
Total I/I: 412,000 *gallons*

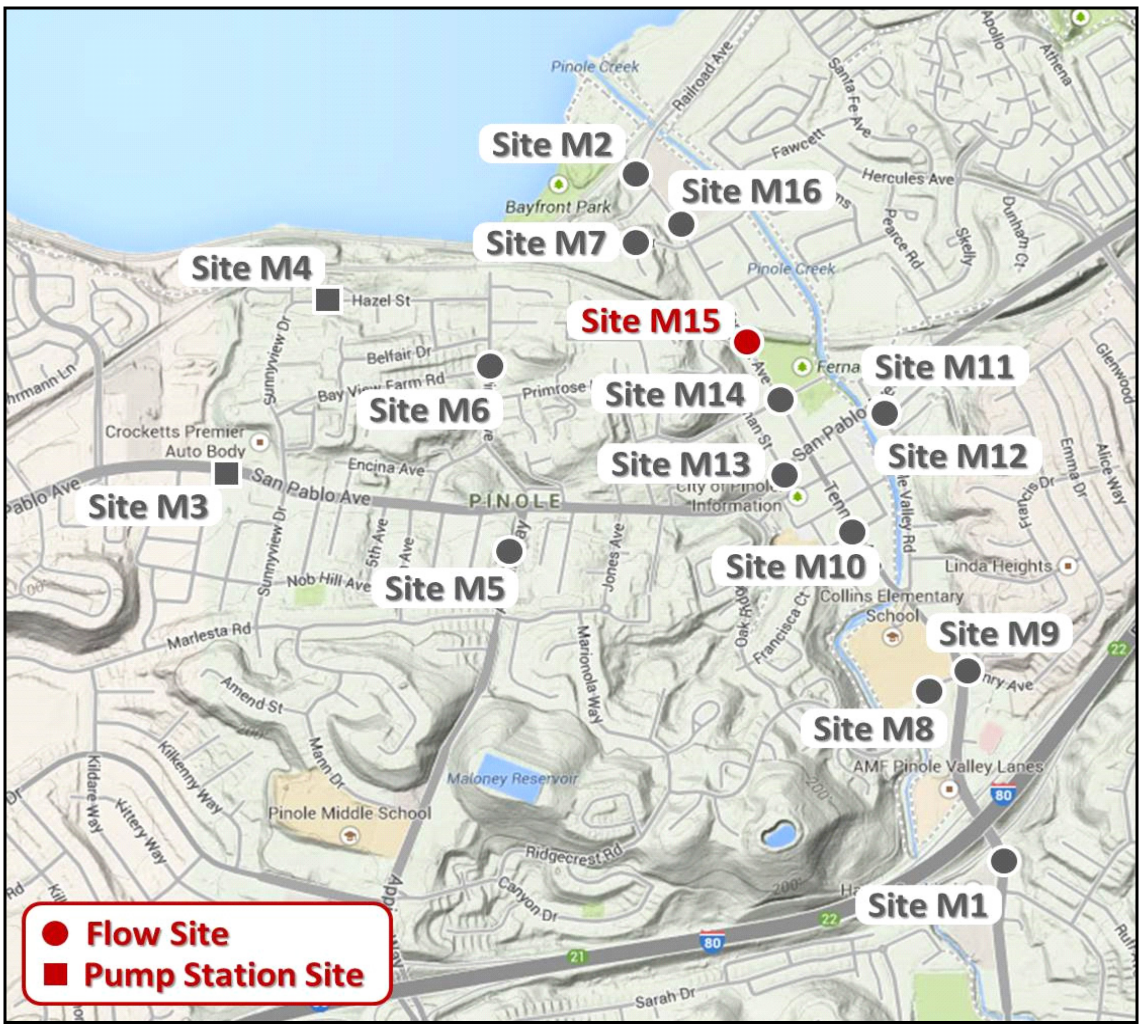
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M15

Location: Tennant Ave., south of train tracks, west of Fernandez Park

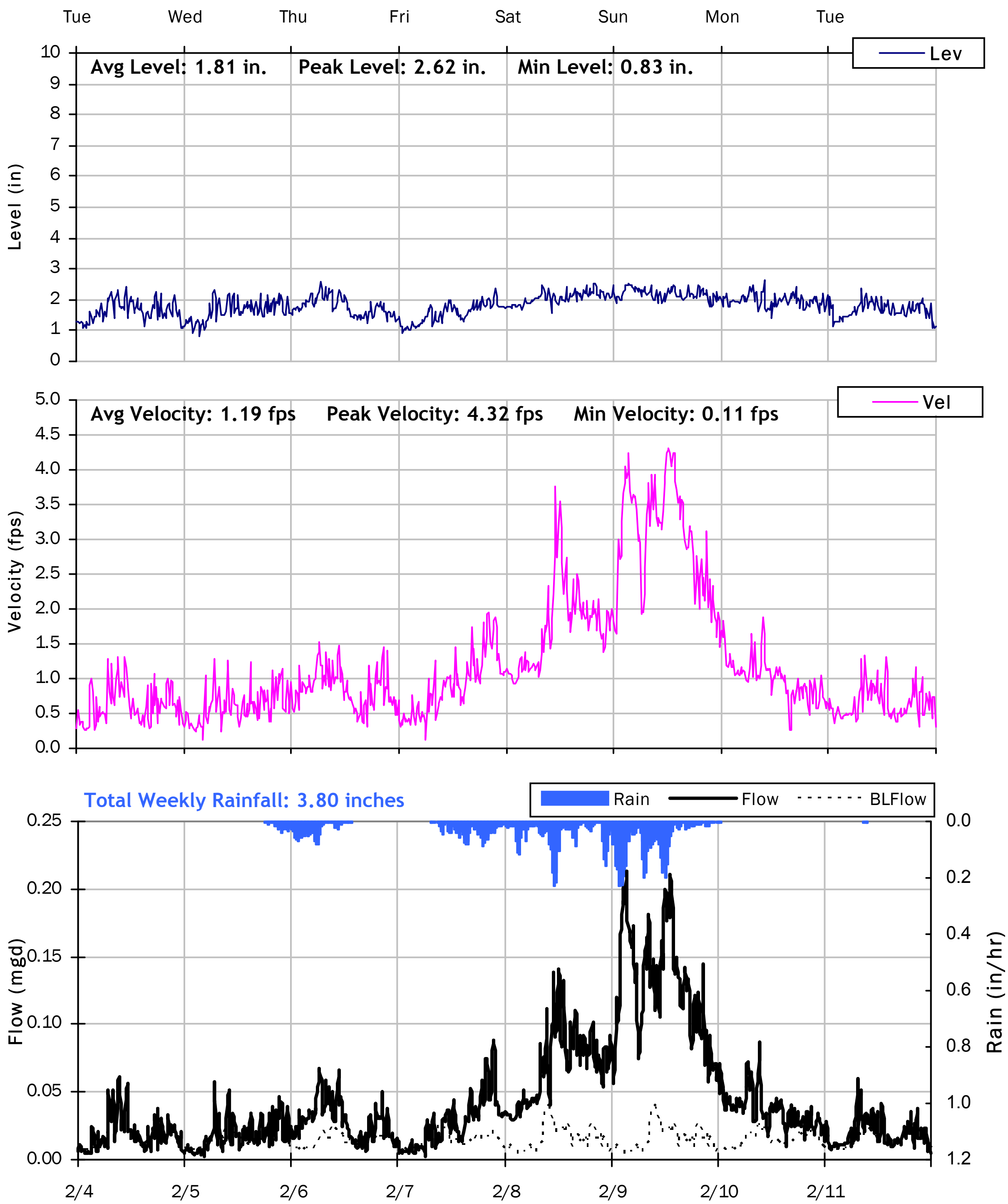
Data Summary Report



Vicinity Map: Site M15

SITE M15

Weekly Level, Velocity and Flow Hydrographs

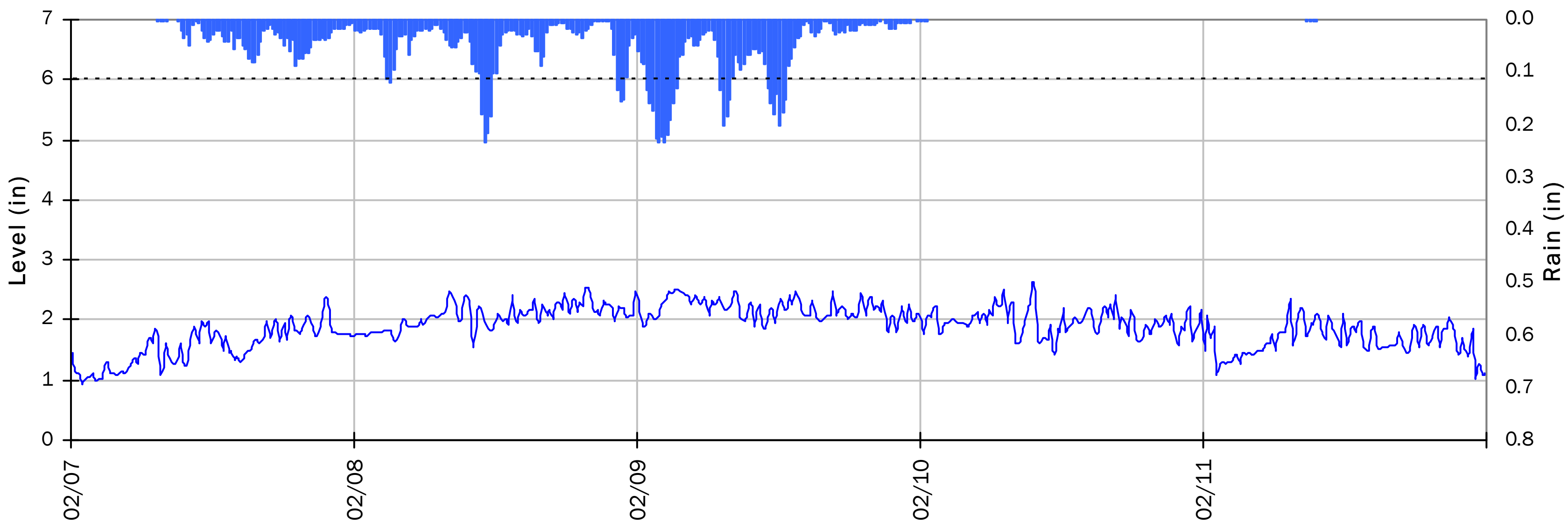


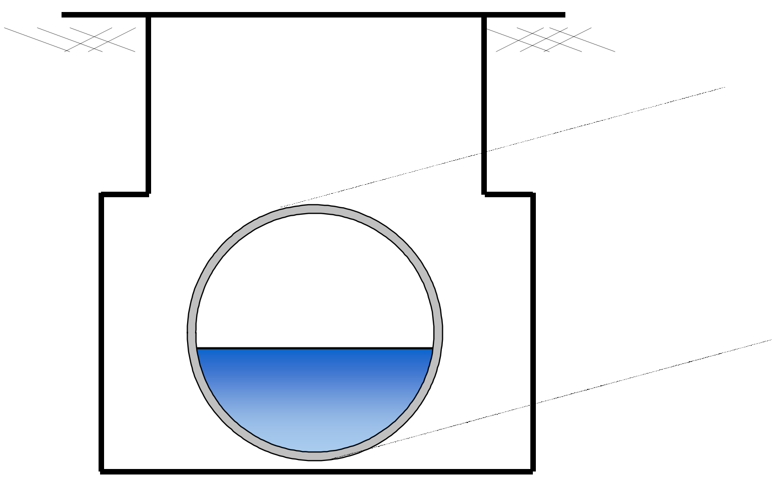
Avg Flow: 0.043 mgd **Peak Flow: 0.214 mgd** **Min Flow: 0.002 mgd**

SITE M15

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



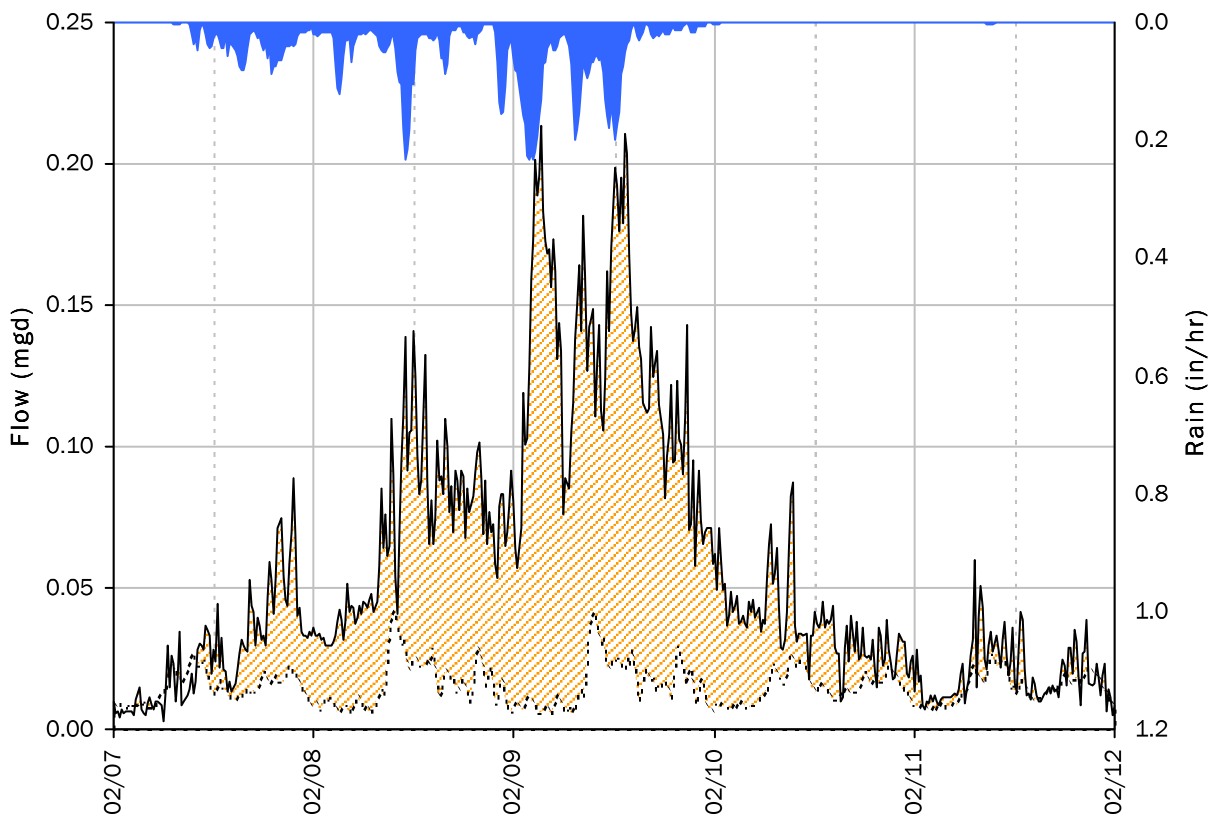


Pipe Diameter:	6	*inches*
Peak Measured Level:	2.62	*inches*
Peak d/D Ratio:	0.44	

SITE M15
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.21 *mgd*
PF: 14.14

Peak Level: 2.62 *in*
d/D Ratio: 0.44

Inflow / Infiltration

Peak I/I Rate: 0.21 *mgd*
Total I/I: 205,000 *gallons*

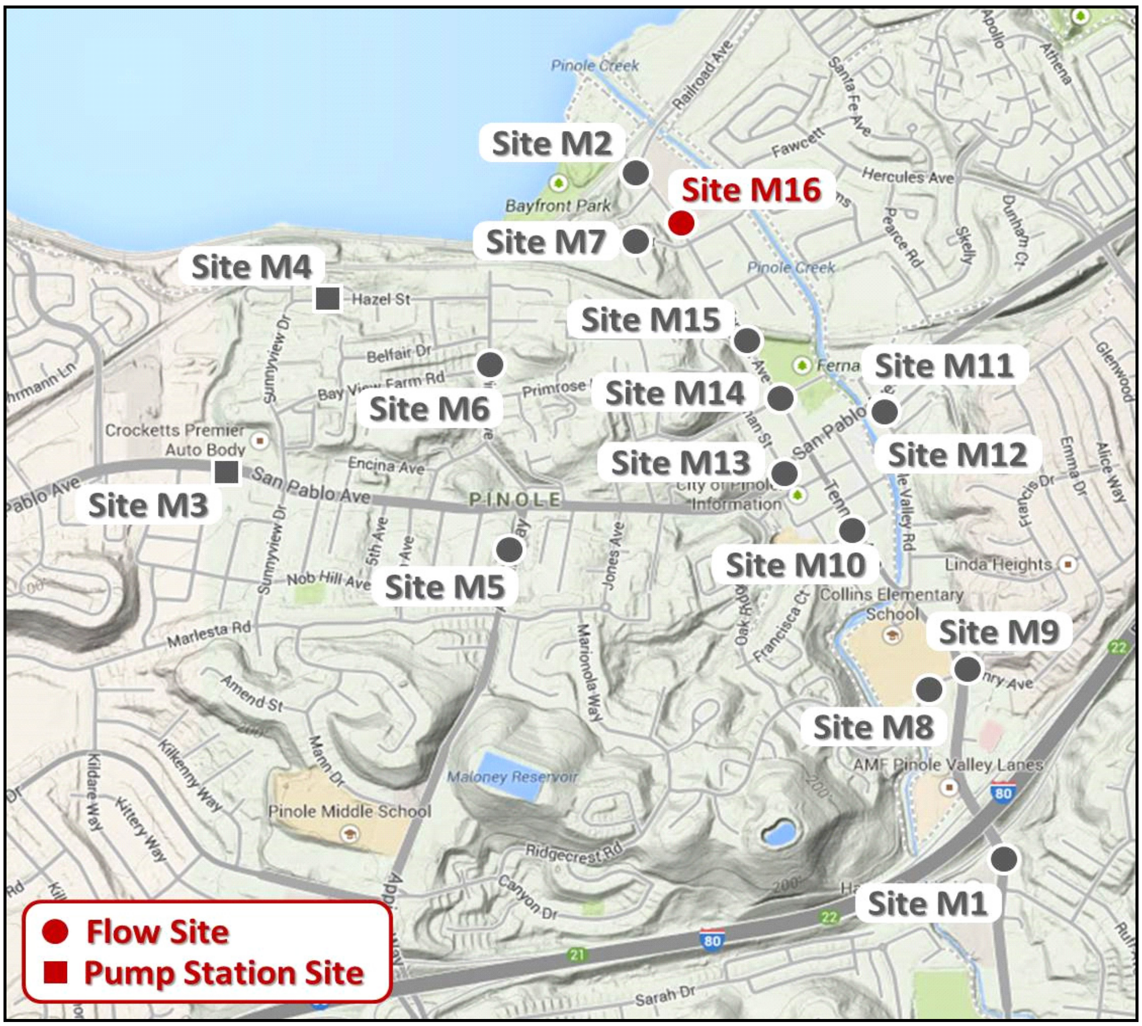
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M16

Location: Tennant Ave. north of Orleans Dr.

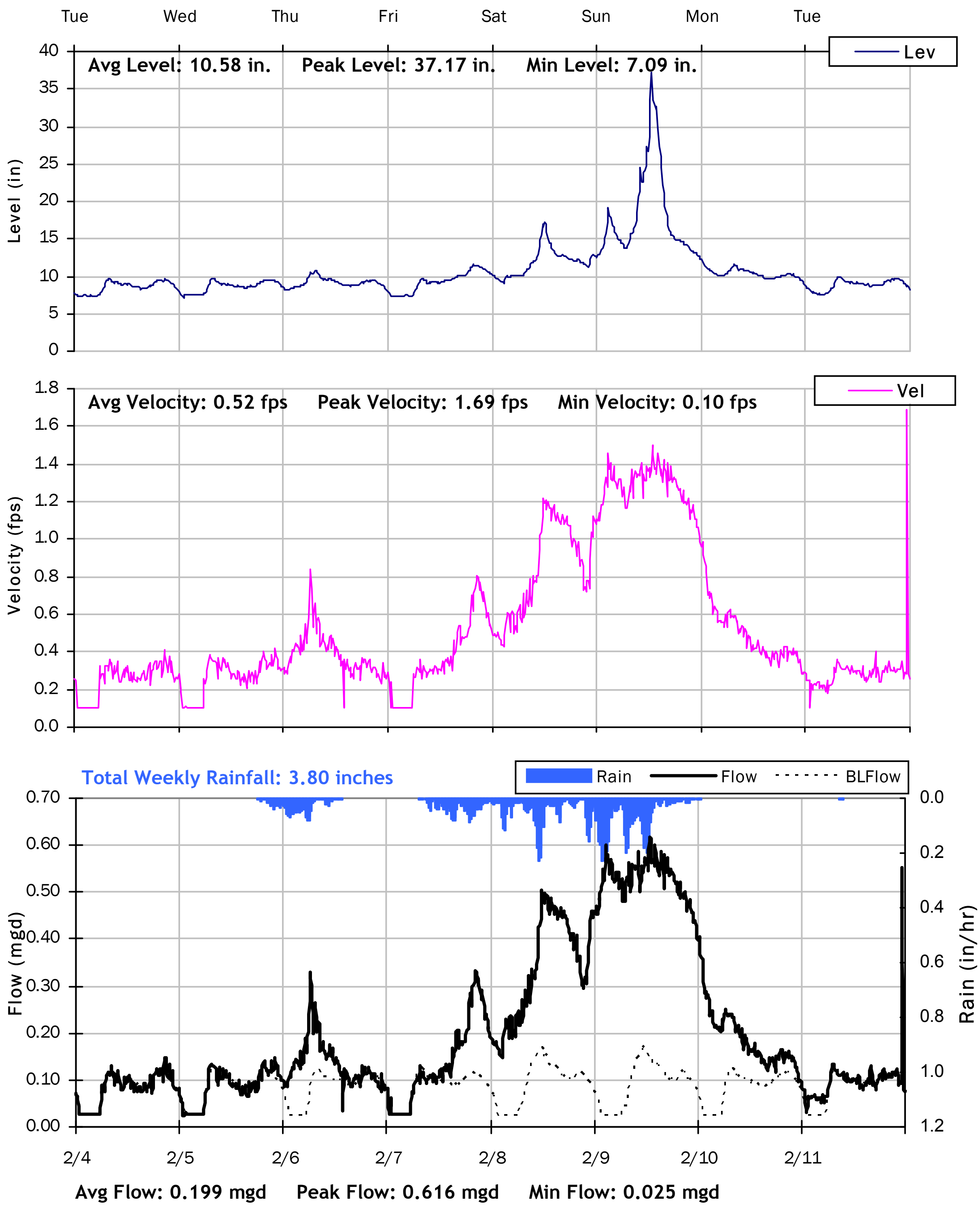
Data Summary Report



Vicinity Map: Site M16

SITE M16

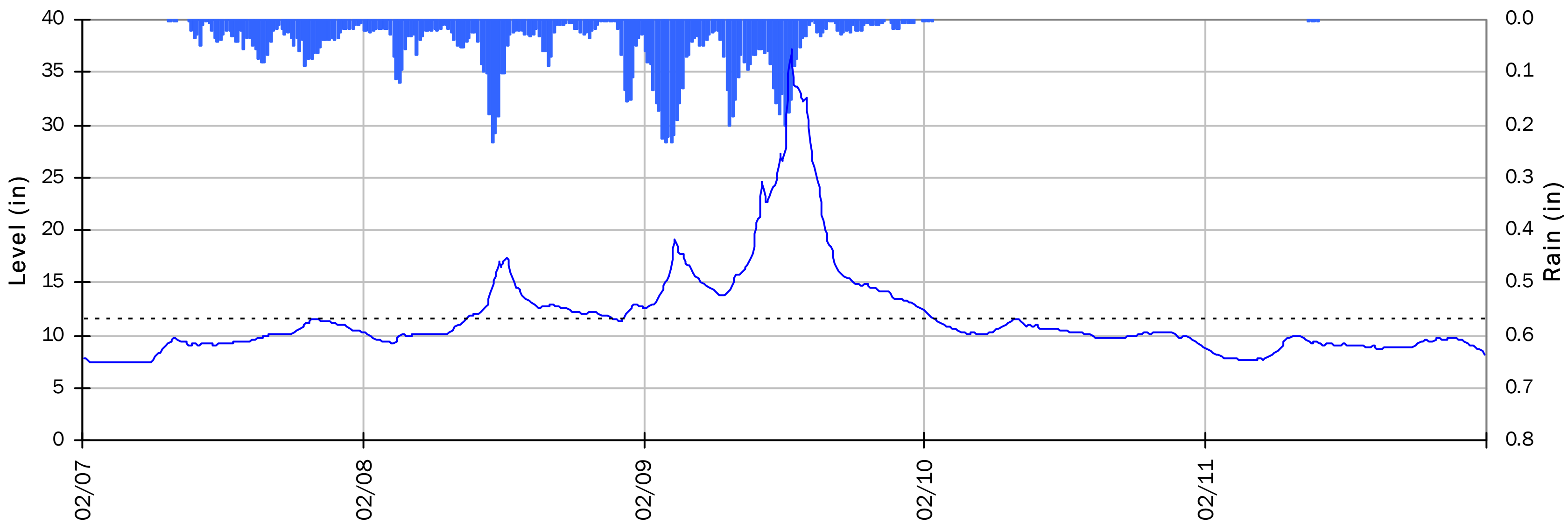
Weekly Level, Velocity and Flow Hydrographs

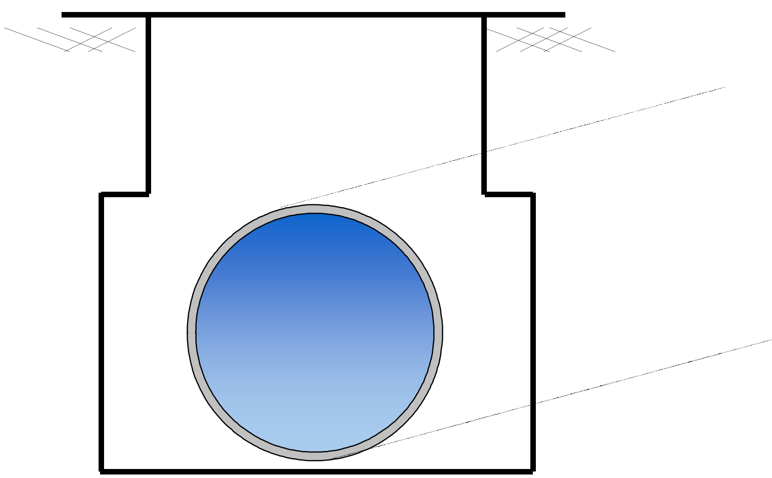


SITE M16

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 11.5 *inches*

Peak Measured Level: 37.2 *inches*

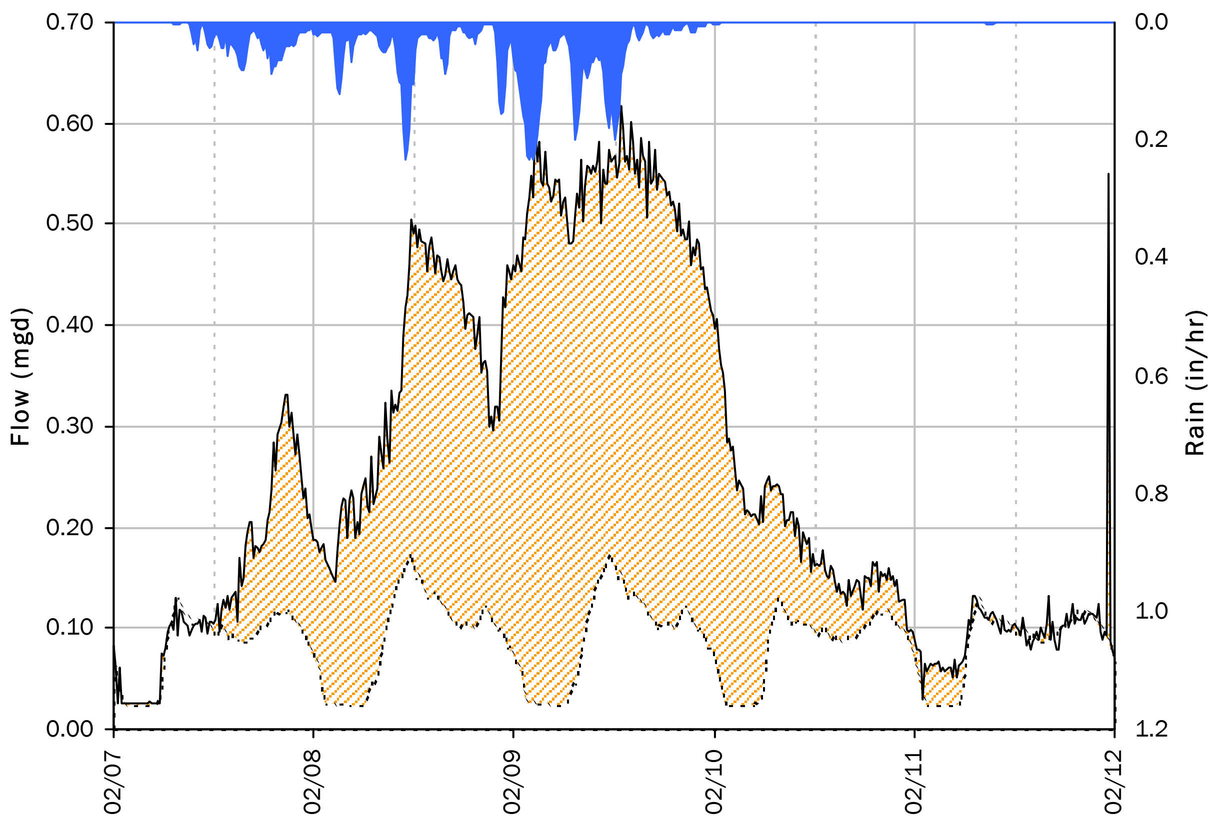
Peak d/D Ratio: 3.23

Surcharged 25.7 inches over crown

SITE M16
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 3.25 inches)

Capacity

Peak Flow: 0.62 *mgd*
PF: 7.20

Peak Level: 37.17 *in*
d/D Ratio: 3.23

Inflow / Infiltration

Peak I/I Rate: 0.58 *mgd*
Total I/I: 858,000 *gallons*

Appendix E. Flow Monitoring Sites Data, Graphs, Information: Phase 2

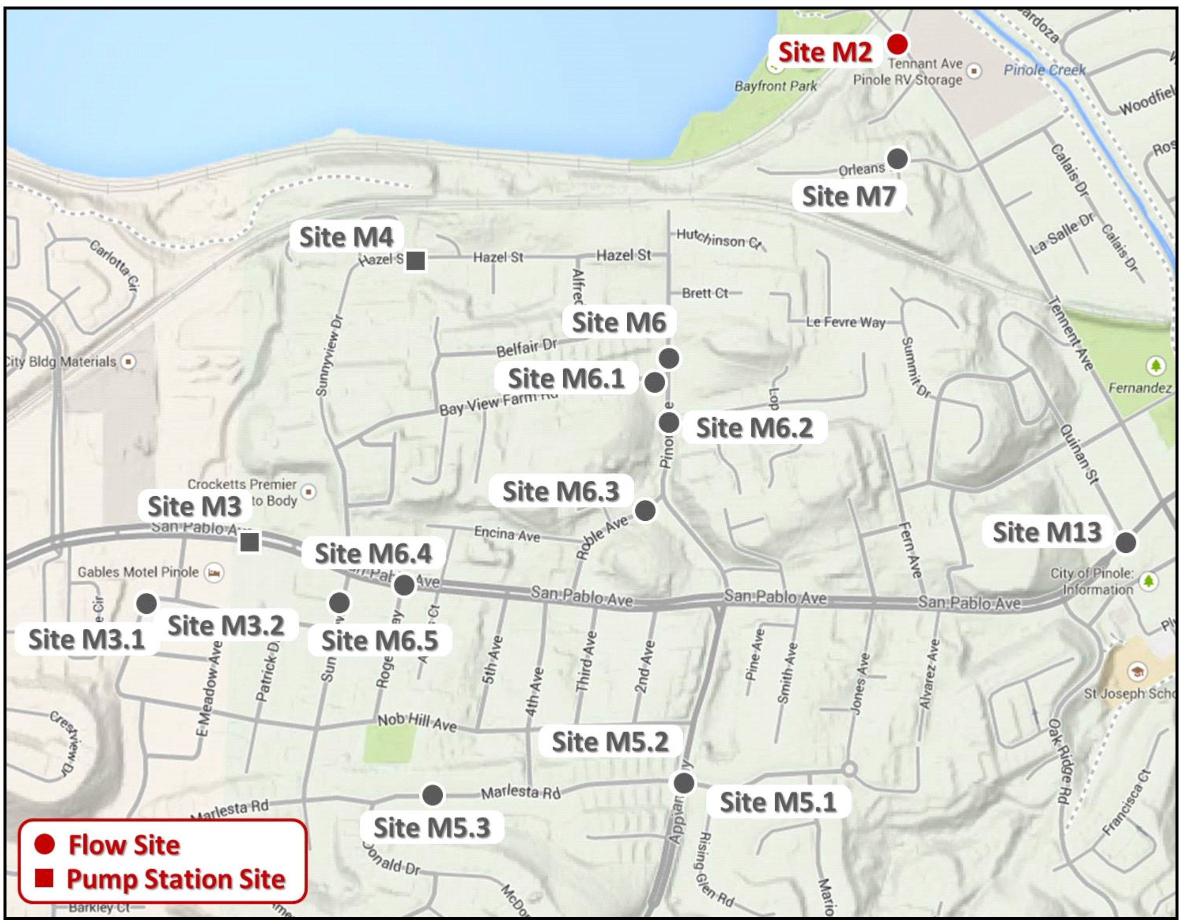
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M2

Location: Tennant Ave., just outside WWTP

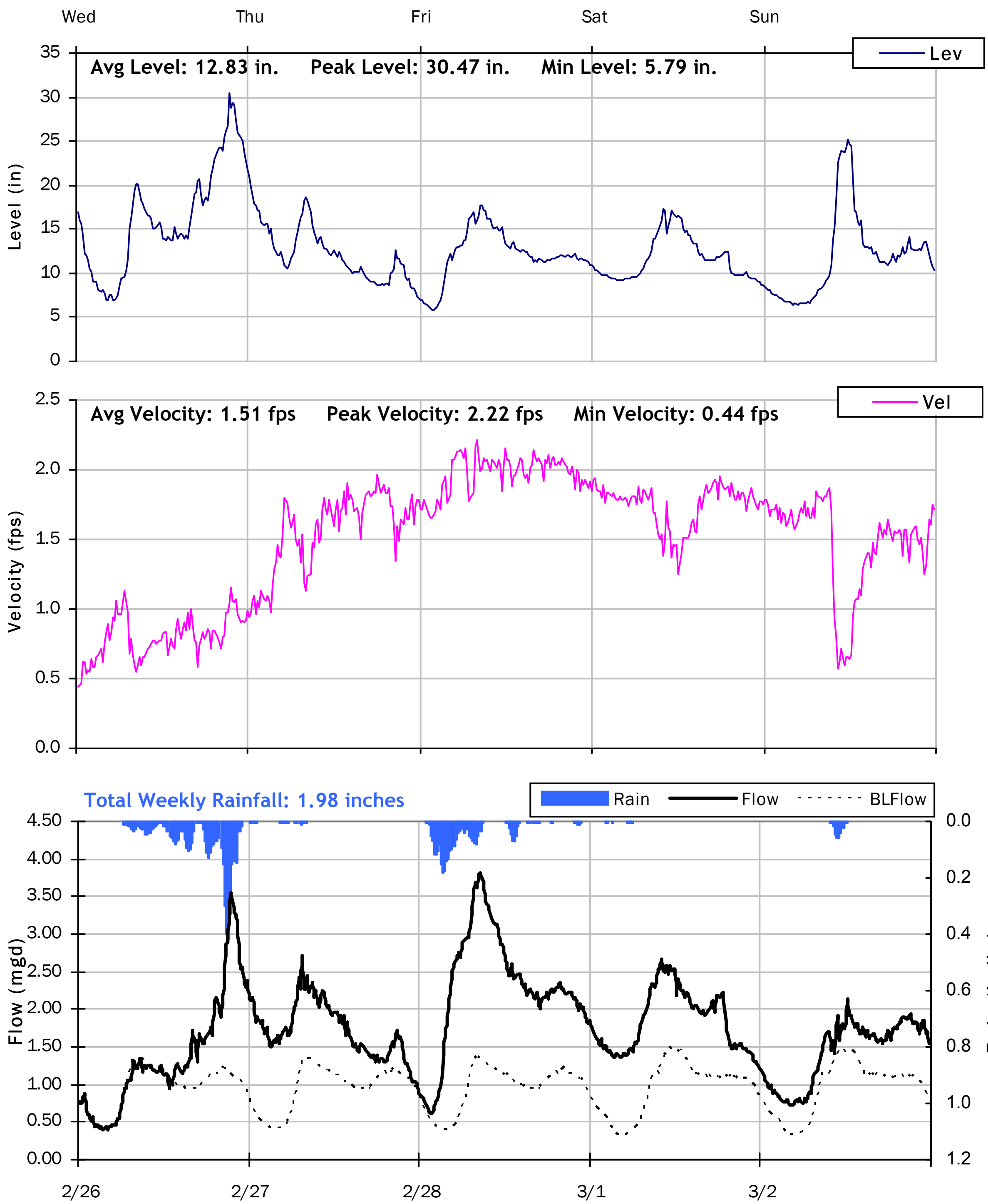
Data Summary Report



Vicinity Map: Site M2

SITE M2

Weekly Level, Velocity and Flow Hydrographs

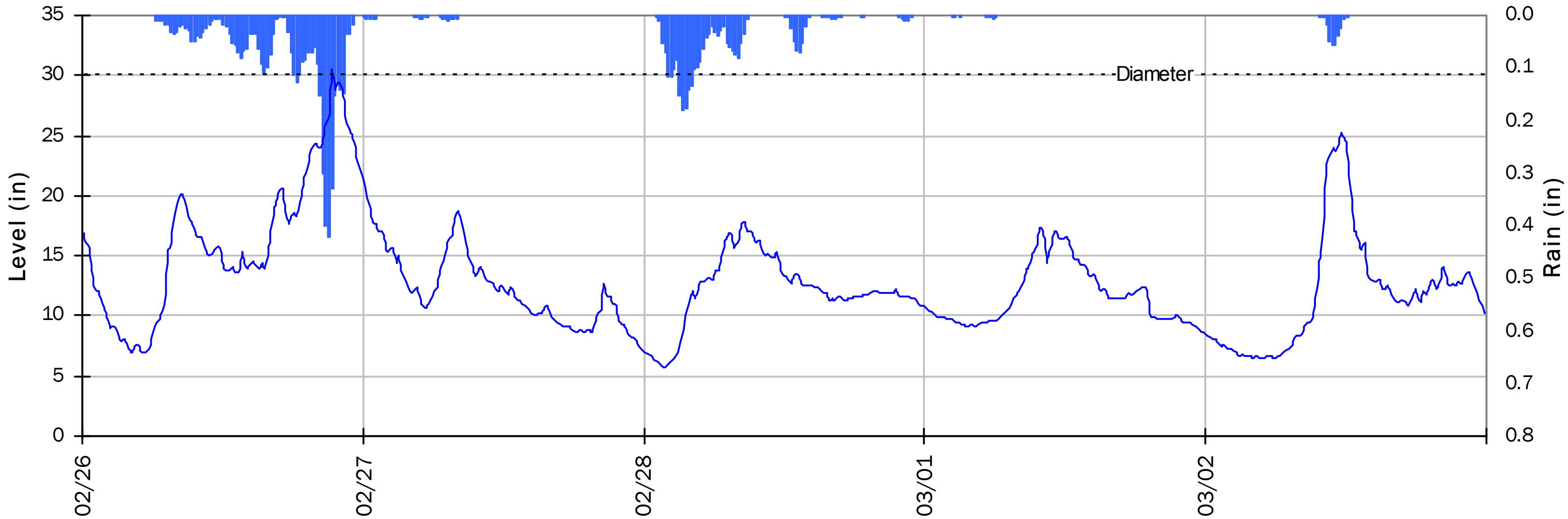


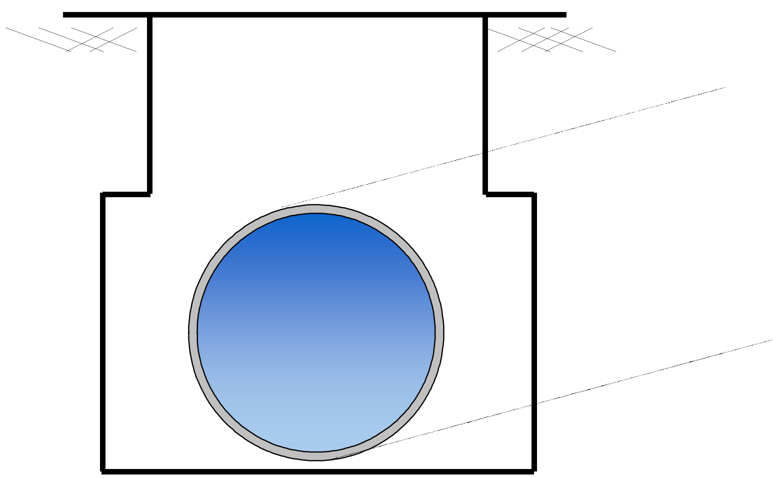
Avg Flow: 1.732 mgd **Peak Flow: 3.813 mgd** **Min Flow: 0.395 mgd**

SITE M2

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





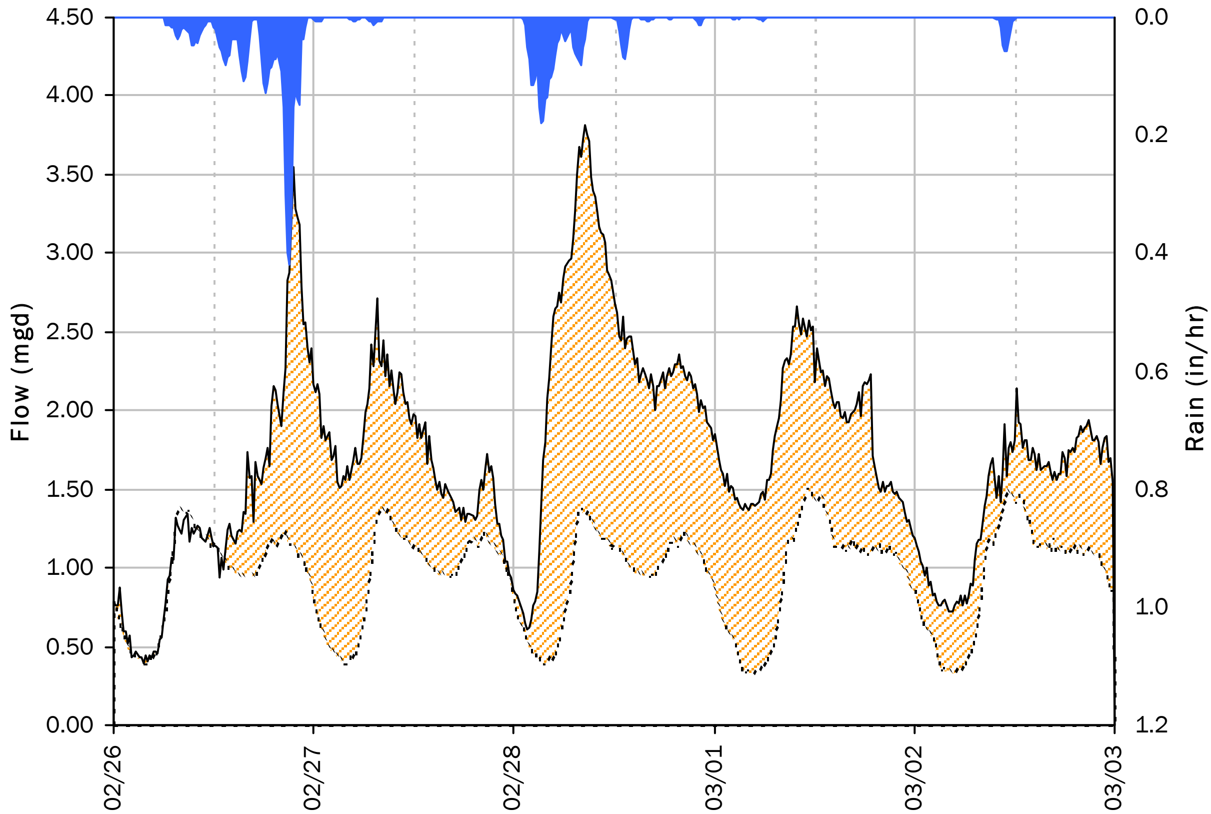
Pipe Diameter:	30	*inches*
Peak Measured Level:	30.5	*inches*
Peak d/D Ratio:	1.02	

Surcharged 0.5 inches over crown

SITE M2
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 3.81 *mgd*
PF: 4.02

Peak Level: 30.47 *in*
d/D Ratio: 1.02

Inflow / Infiltration

Peak I/I Rate: 2.47 *mgd*
Total I/I: 3,432,000 *gallons*

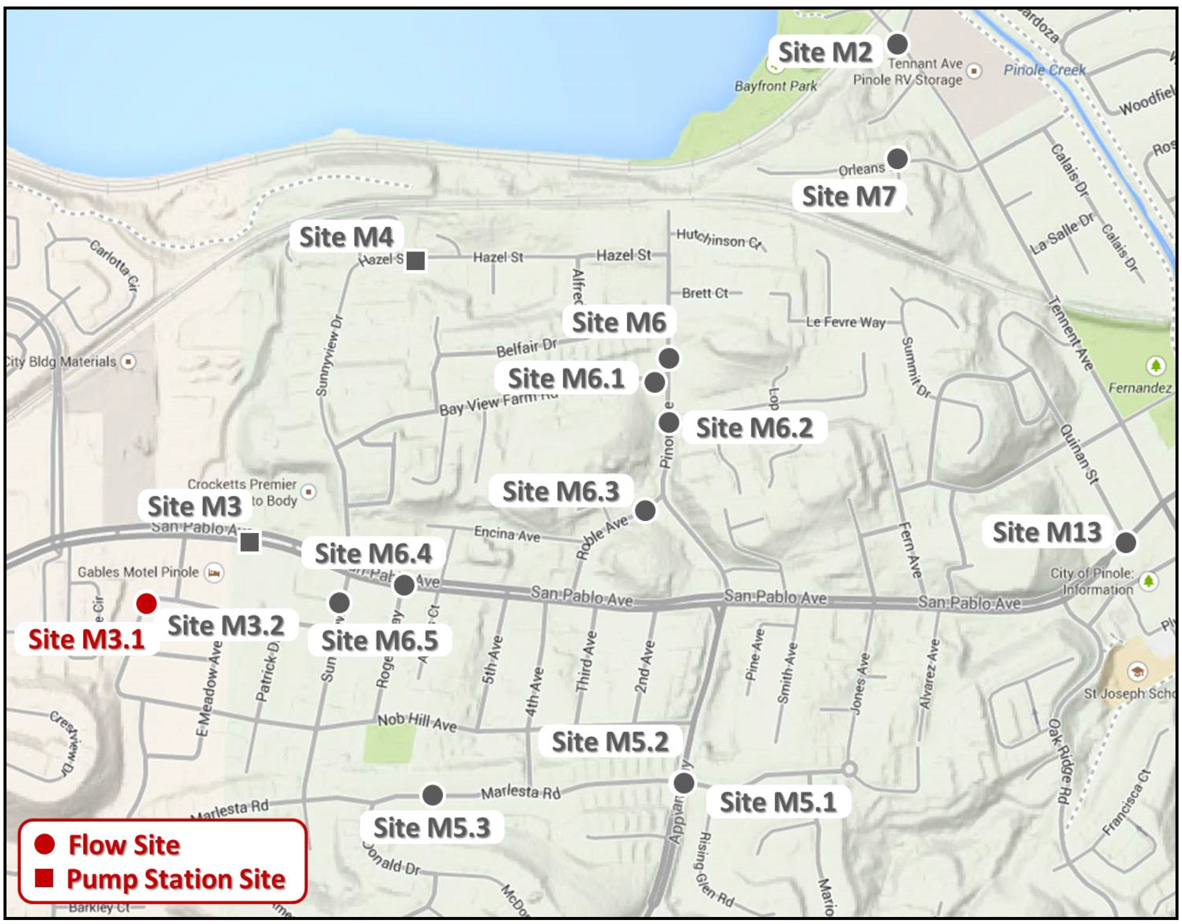
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M3.1

Location: 830 Meadows Ave.

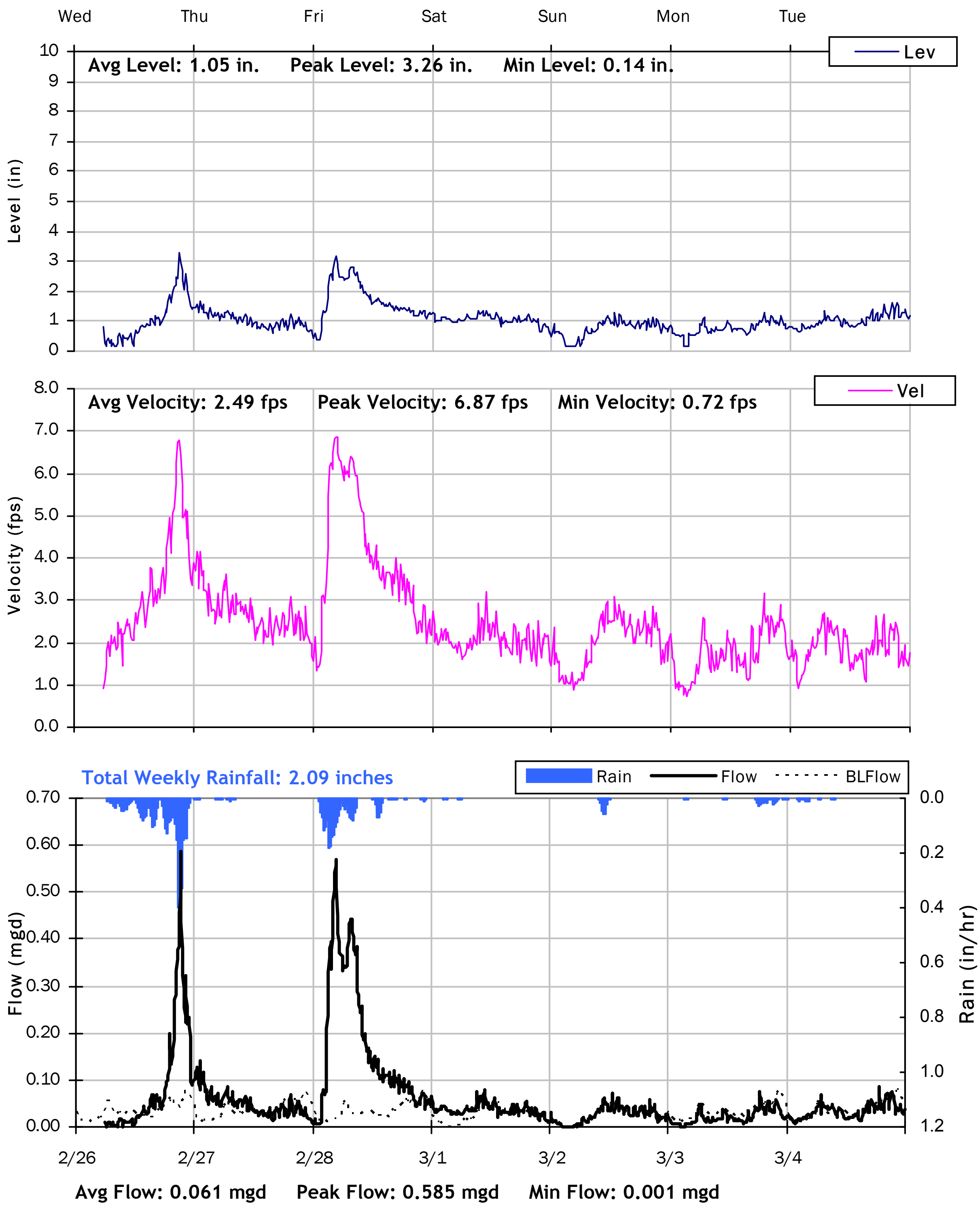
Data Summary Report



Vicinity Map: Site M3.1

SITE M3.1

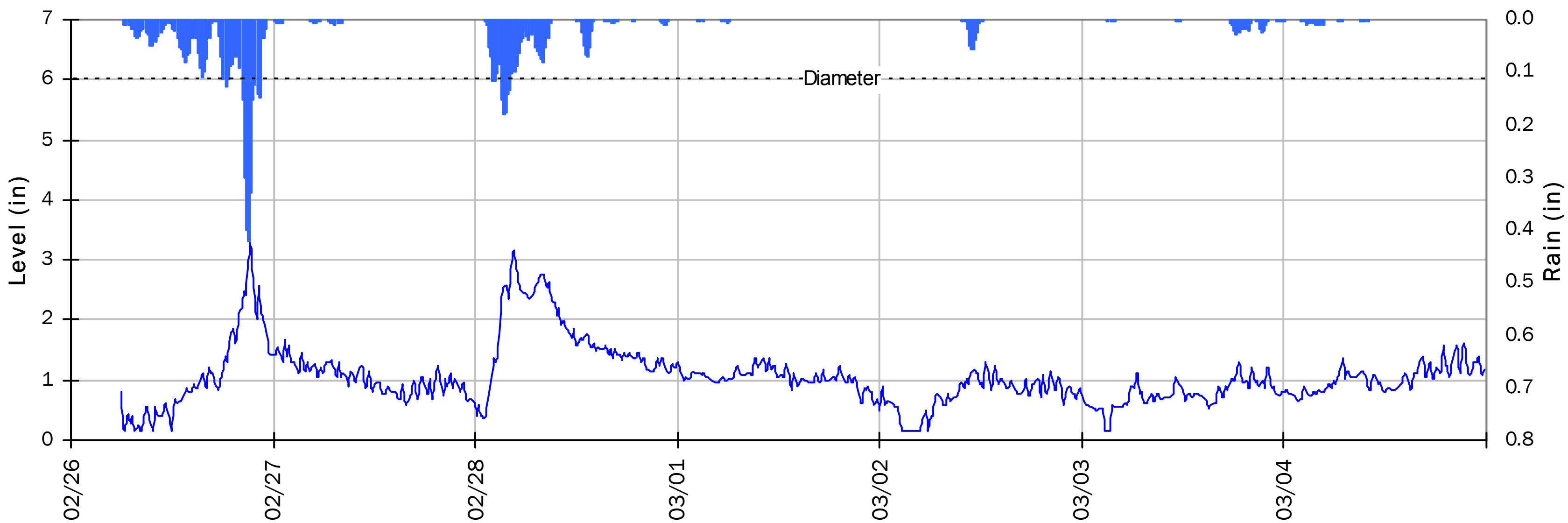
Weekly Level, Velocity and Flow Hydrographs

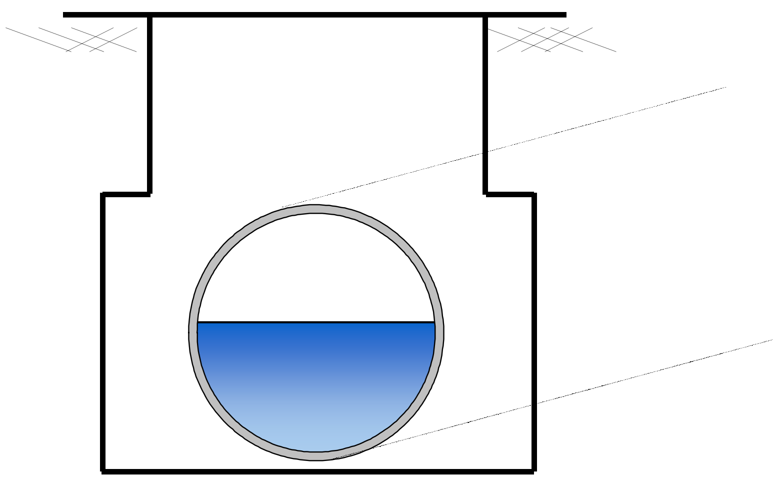


SITE M3.1

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



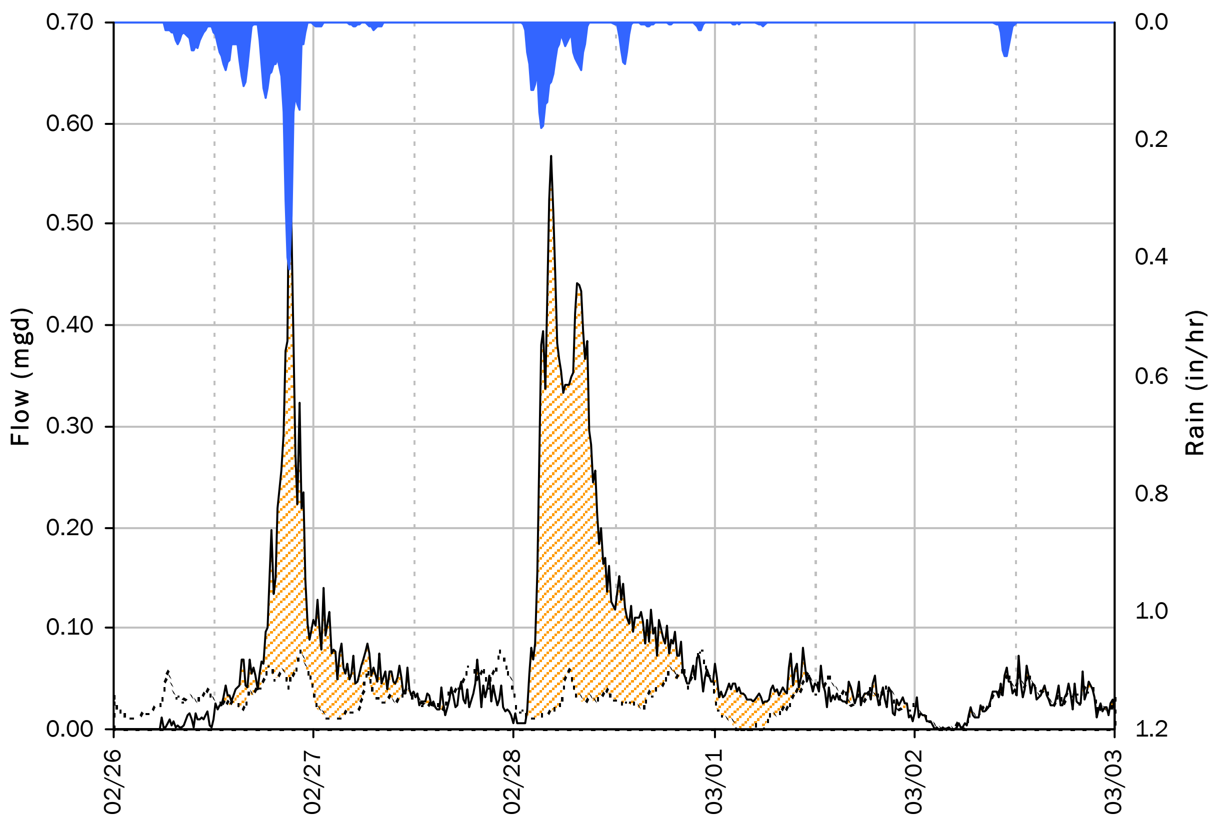


Pipe Diameter:	6	*inches*
Peak Measured Level:	3.26	*inches*
Peak d/D Ratio:	0.54	

SITE M3.1
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.58 *mgd*
PF: 18.34

Peak Level: 3.26 *in*
d/D Ratio: 0.54

Inflow / Infiltration

Peak I/I Rate: 0.55 *mgd*
Total I/I: 213,000 *gallons*

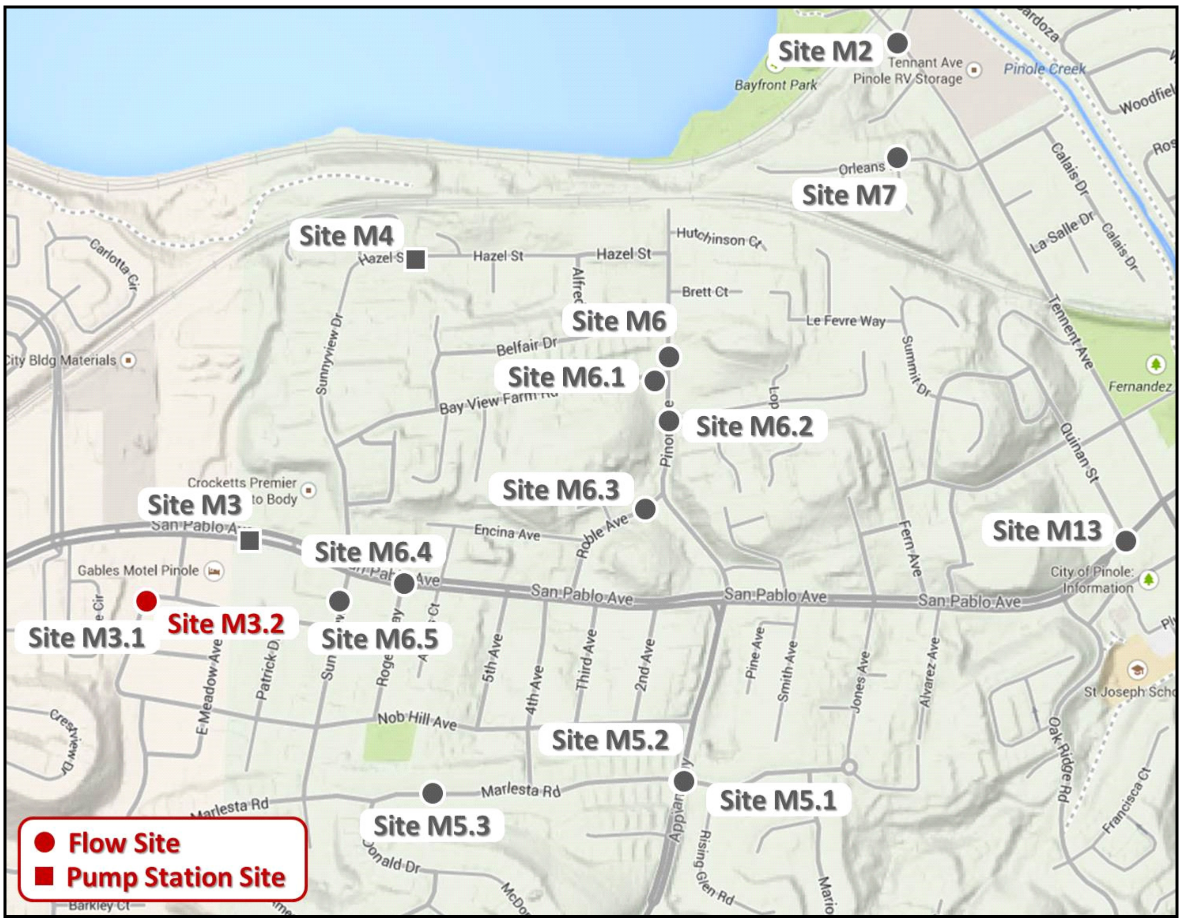
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M3.2

Location: 830 Meadows Ave.

Data Summary Report



Vicinity Map: Site M3.2

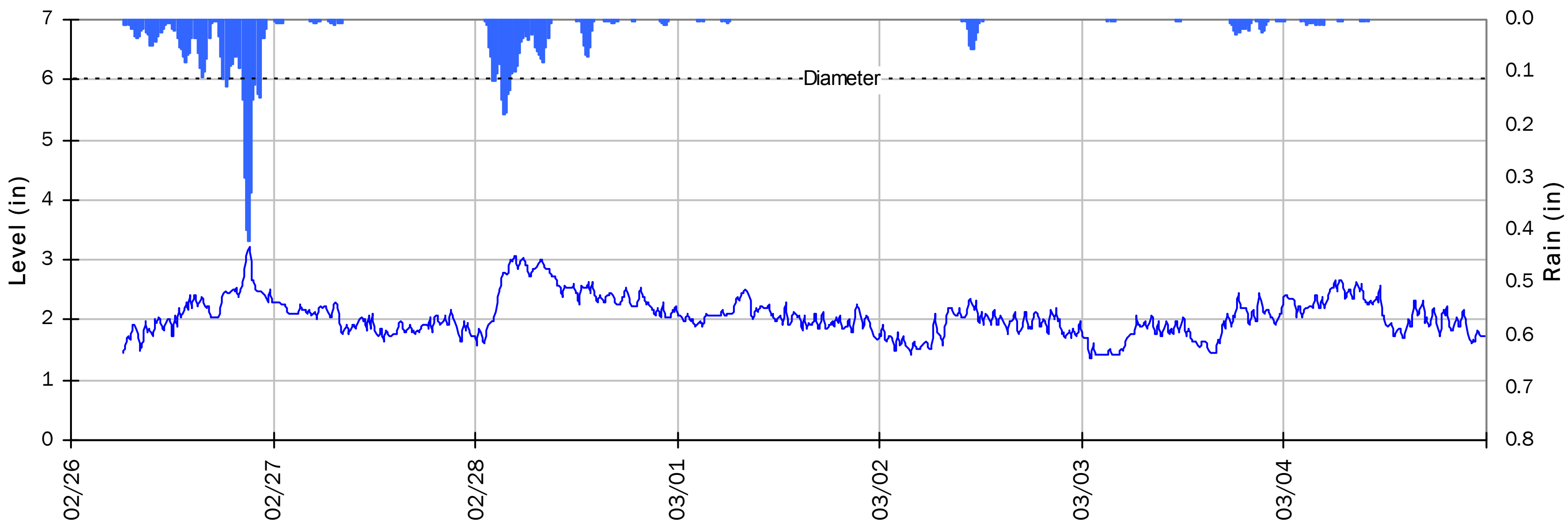
SITE M3.2

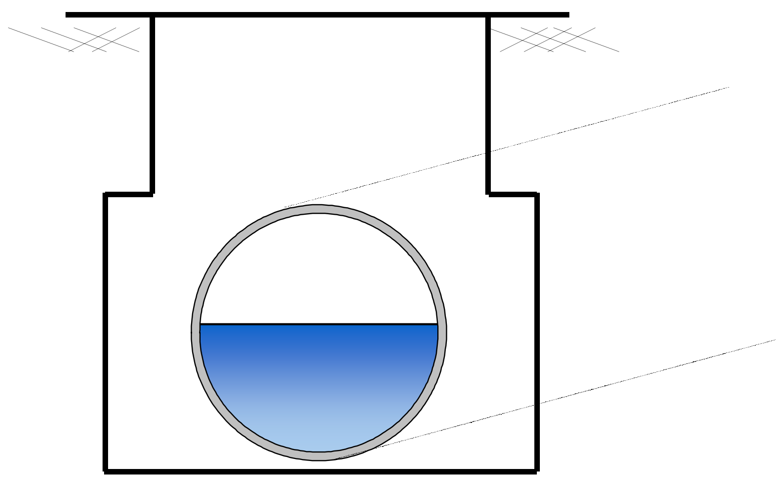
Weekly Level, Velocity and Flow Hydrographs

Avg Flow: 0.017 mgd **Peak Flow: 0.078 mgd** **Min Flow: 0.003 mgd**

SITE M3.2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



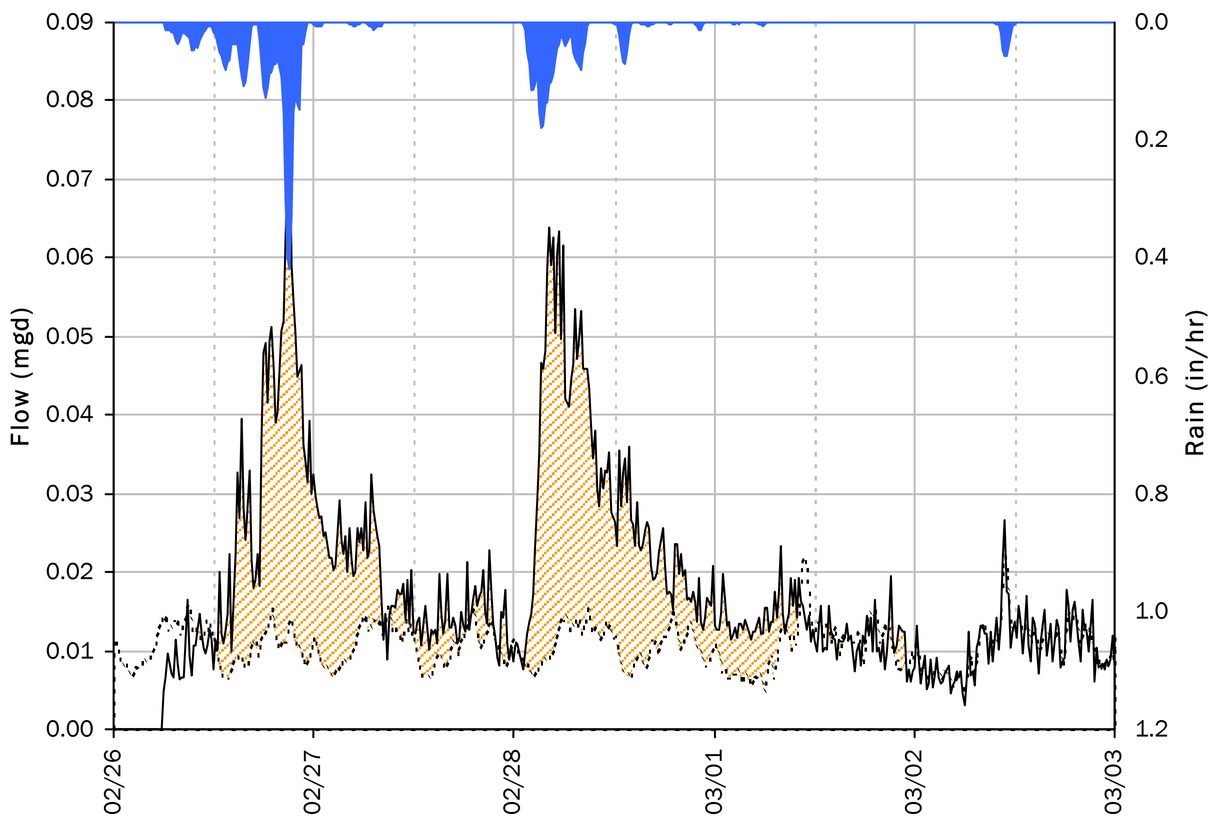


Pipe Diameter:	6	*inches*
Peak Measured Level:	3.22	*inches*
Peak d/D Ratio:	0.54	

SITE M3.2
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.08 *mgd*
PF: 7.34

Peak Level: 3.22 *in*
d/D Ratio: 0.54

Inflow / Infiltration

Peak I/I Rate: 0.06 *mgd*
Total I/I: 41,000 *gallons*

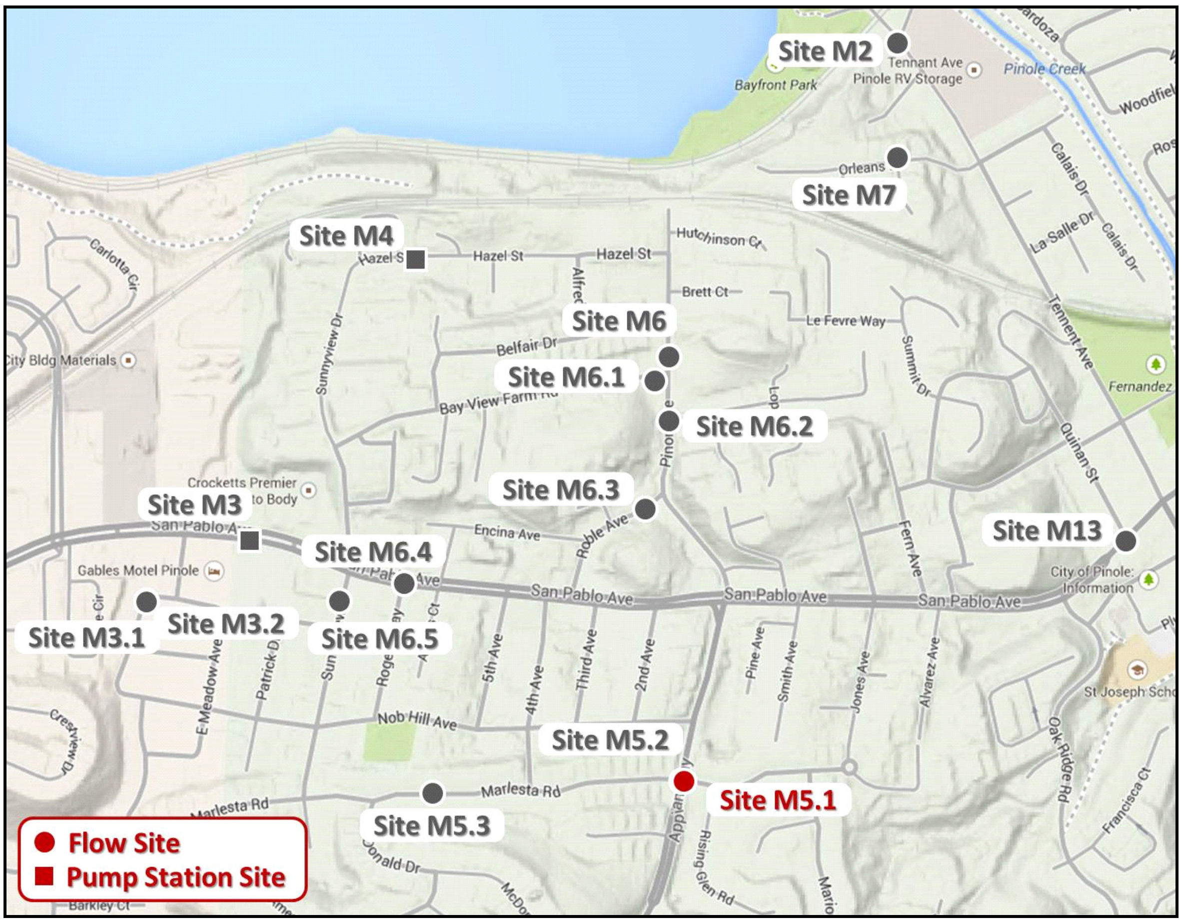
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M5.1

Location: Intersection of Appian Way and Marlesta Rd.

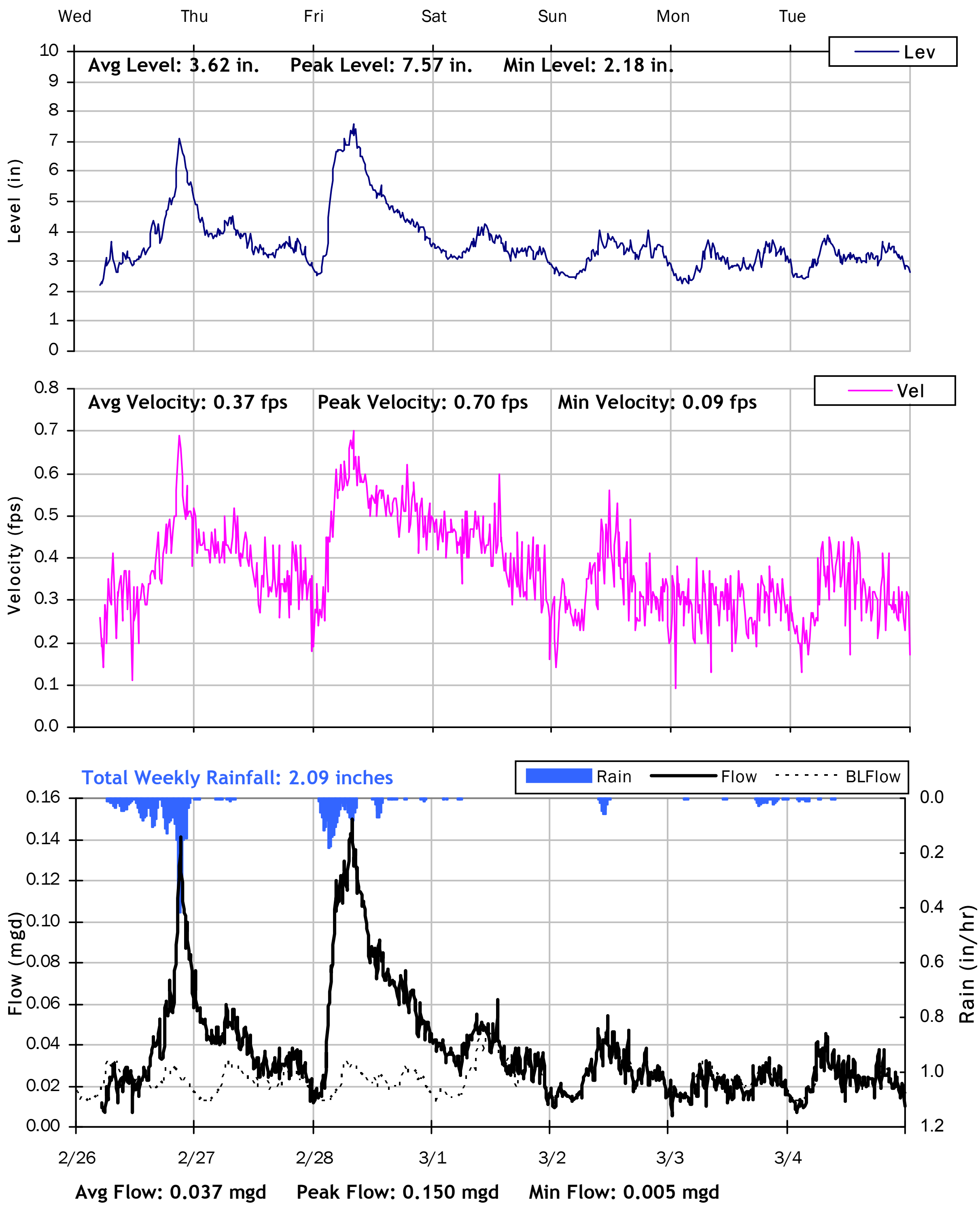
Data Summary Report



Vicinity Map: Site M5.1

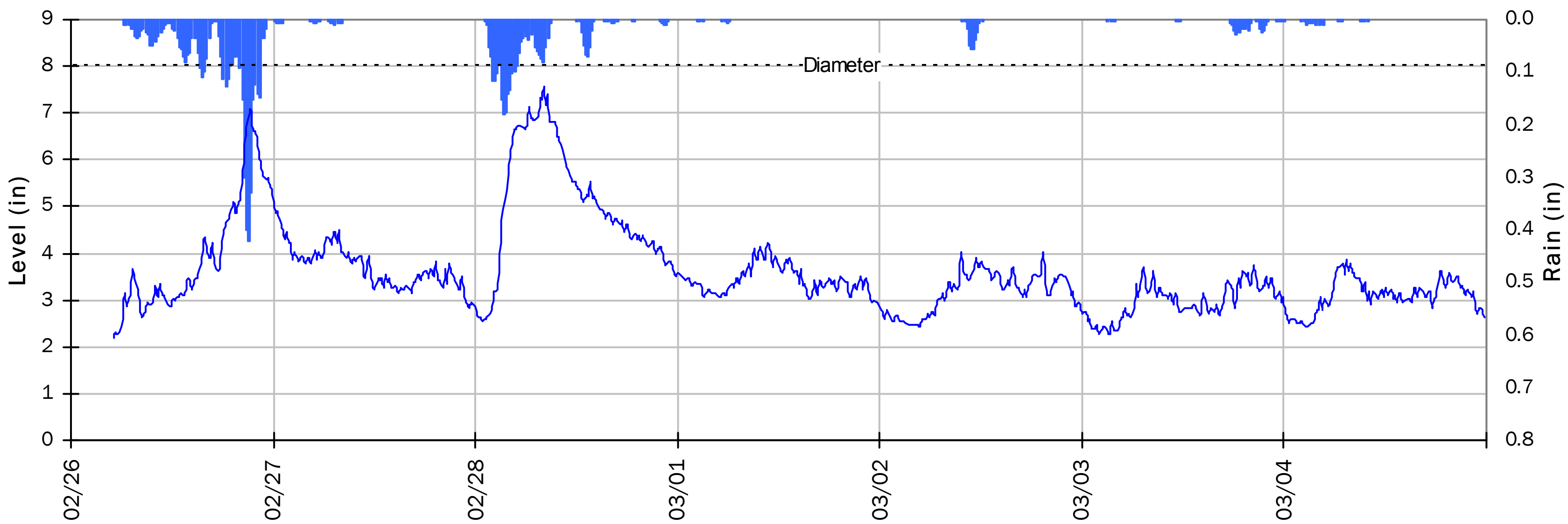
SITE M5.1

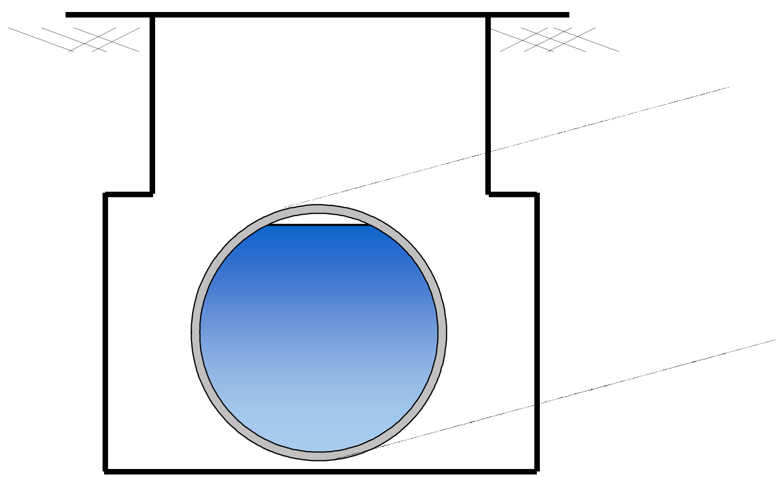
Weekly Level, Velocity and Flow Hydrographs



SITE M5.1
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



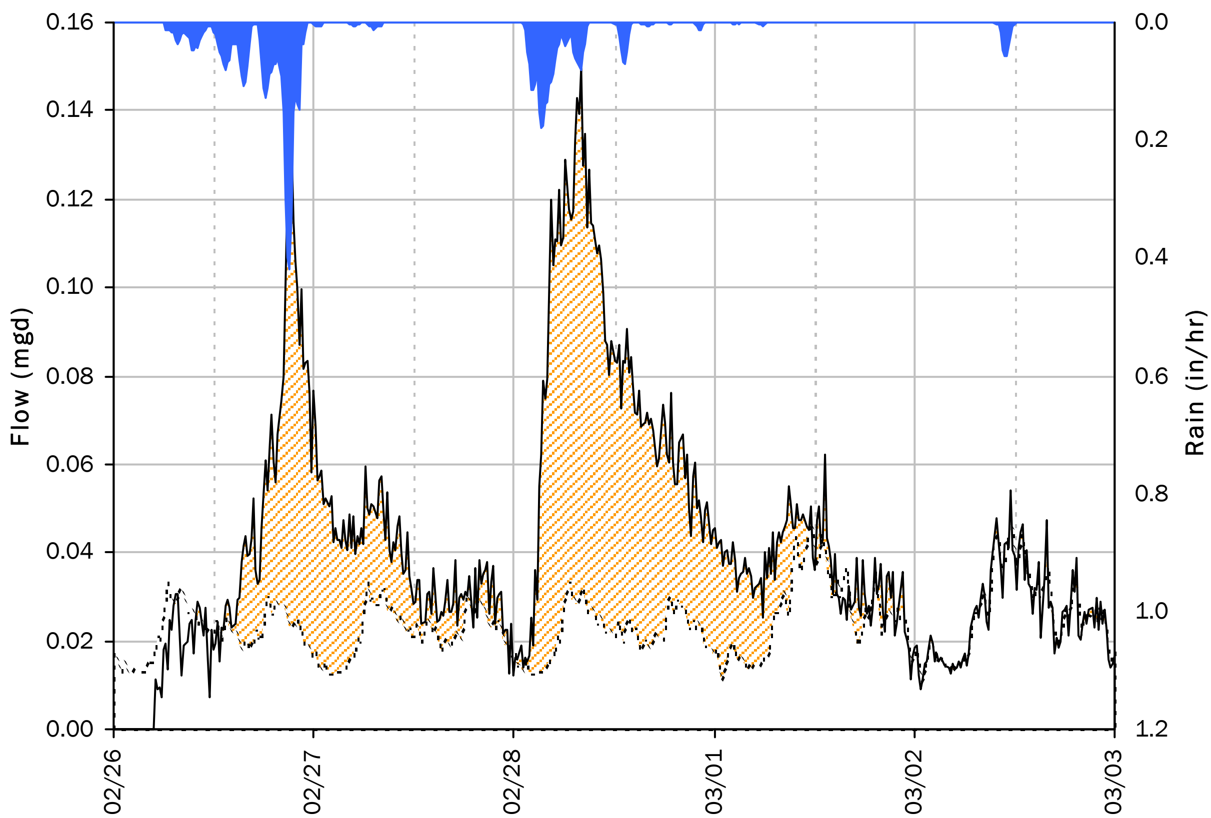


Pipe Diameter:	8	*inches*
Peak Measured Level:	7.57	*inches*
Peak d/D Ratio:	0.95	

SITE M5.1
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow:	0.15 *mgd*
PF:	6.49
Peak Level:	7.57 *in*
d/D Ratio:	0.95

Inflow / Infiltration

Peak I/I Rate:	0.12 *mgd*
Total I/I:	98,000 *gallons*

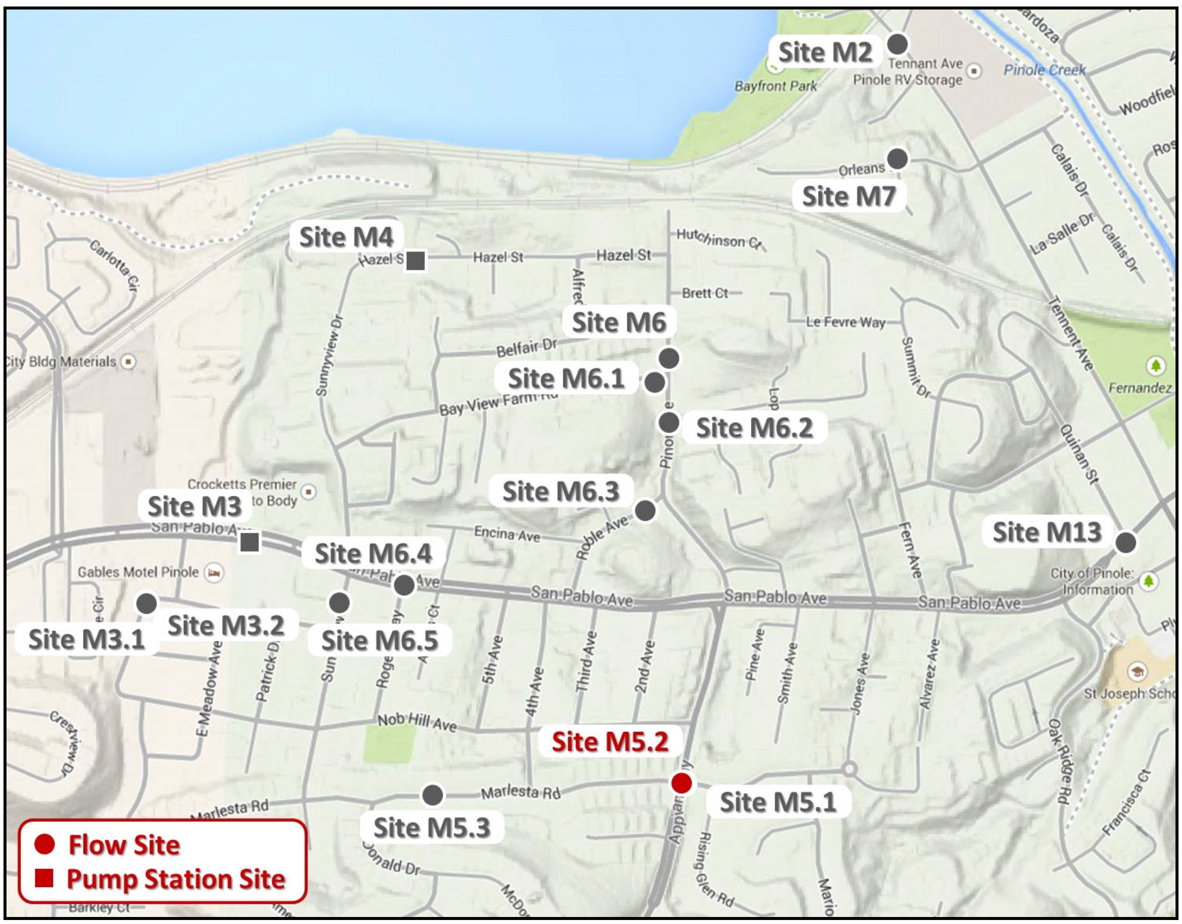
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M5.2

Location: Intersection of Appian Way and Marlesta Rd.

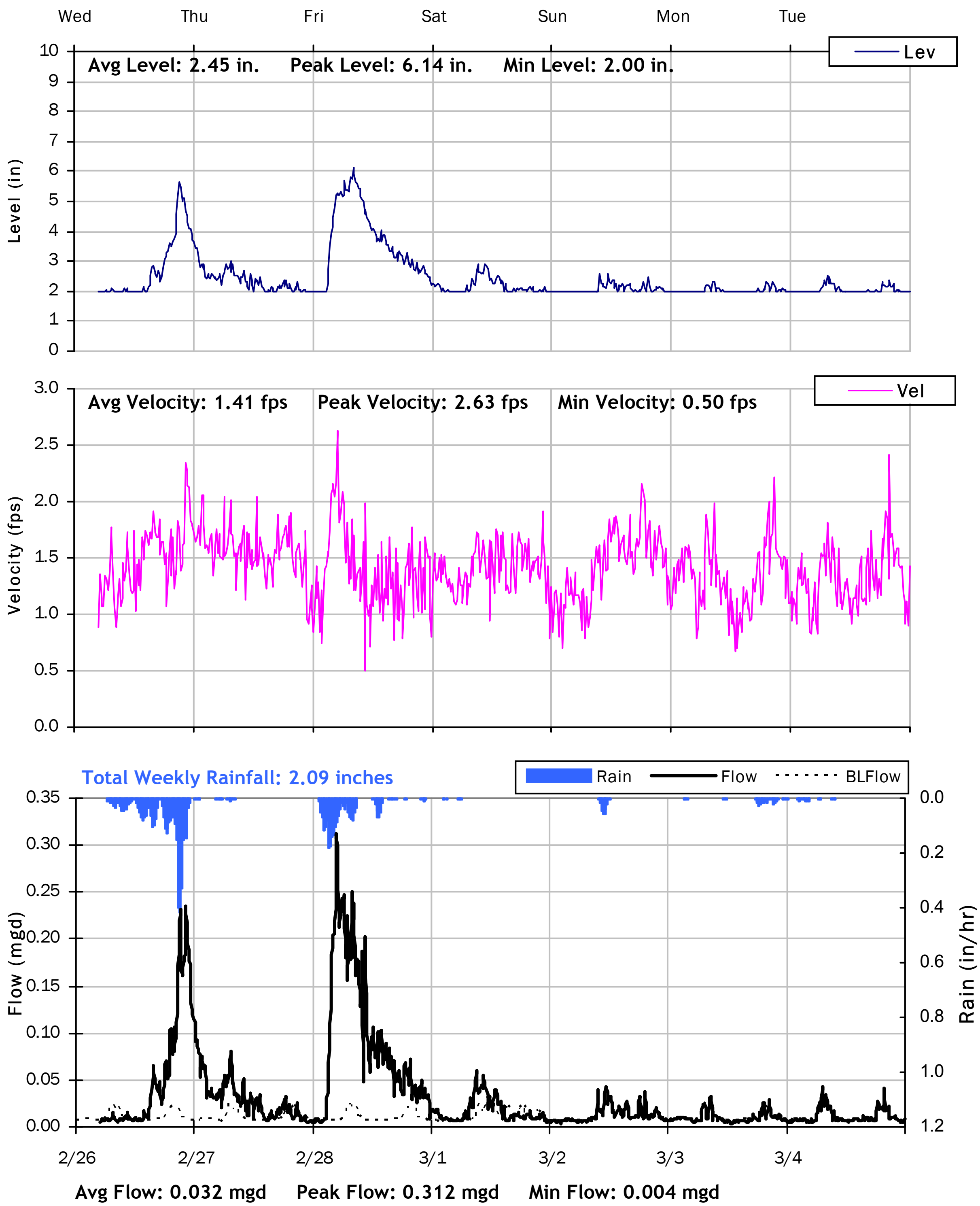
Data Summary Report



Vicinity Map: Site M5.2

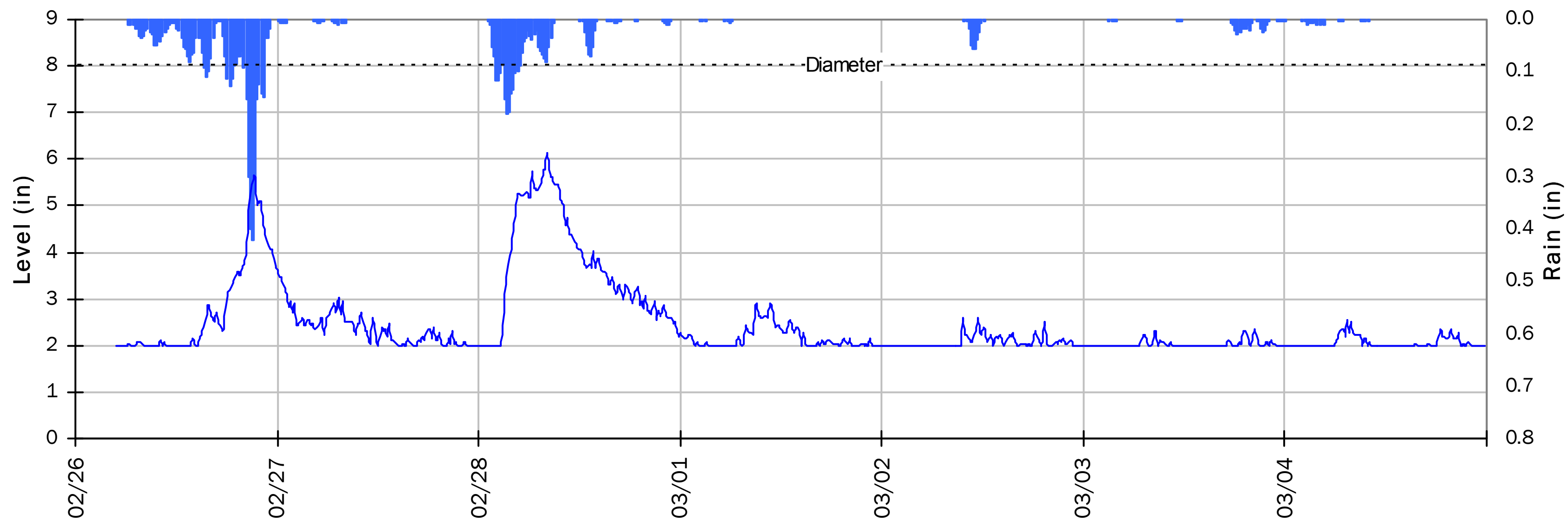
SITE M5.2

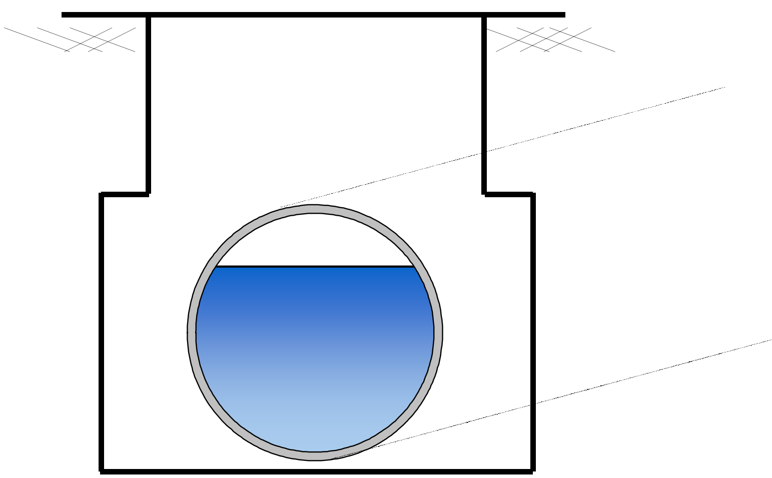
Weekly Level, Velocity and Flow Hydrographs



SITE M5.2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



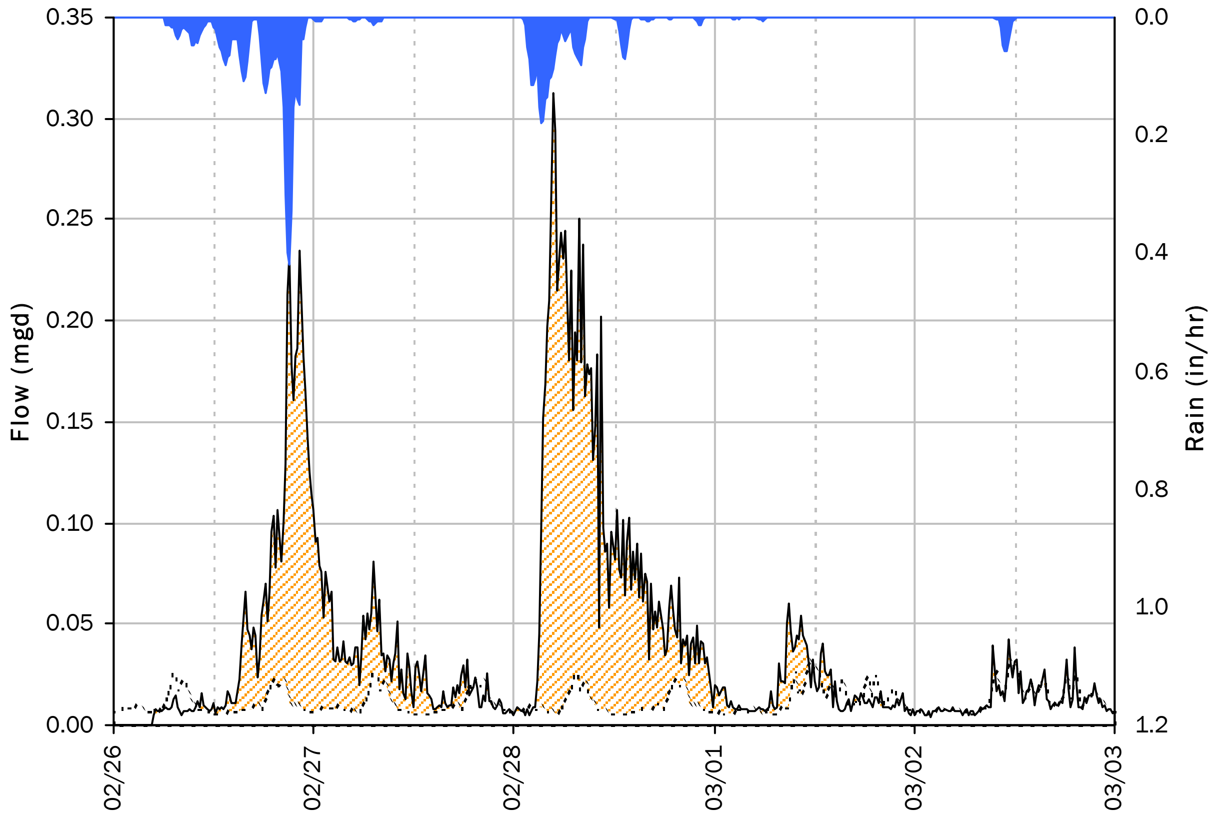


Pipe Diameter:	8	*inches*
Peak Measured Level:	6.14	*inches*
Peak d/D Ratio:	0.77	

SITE M5.2
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.31 *mgd*
PF: 27.55

Peak Level: 6.14 *in*
d/D Ratio: 0.77

Inflow / Infiltration

Peak I/I Rate: 0.31 *mgd*
Total I/I: 142,000 *gallons*

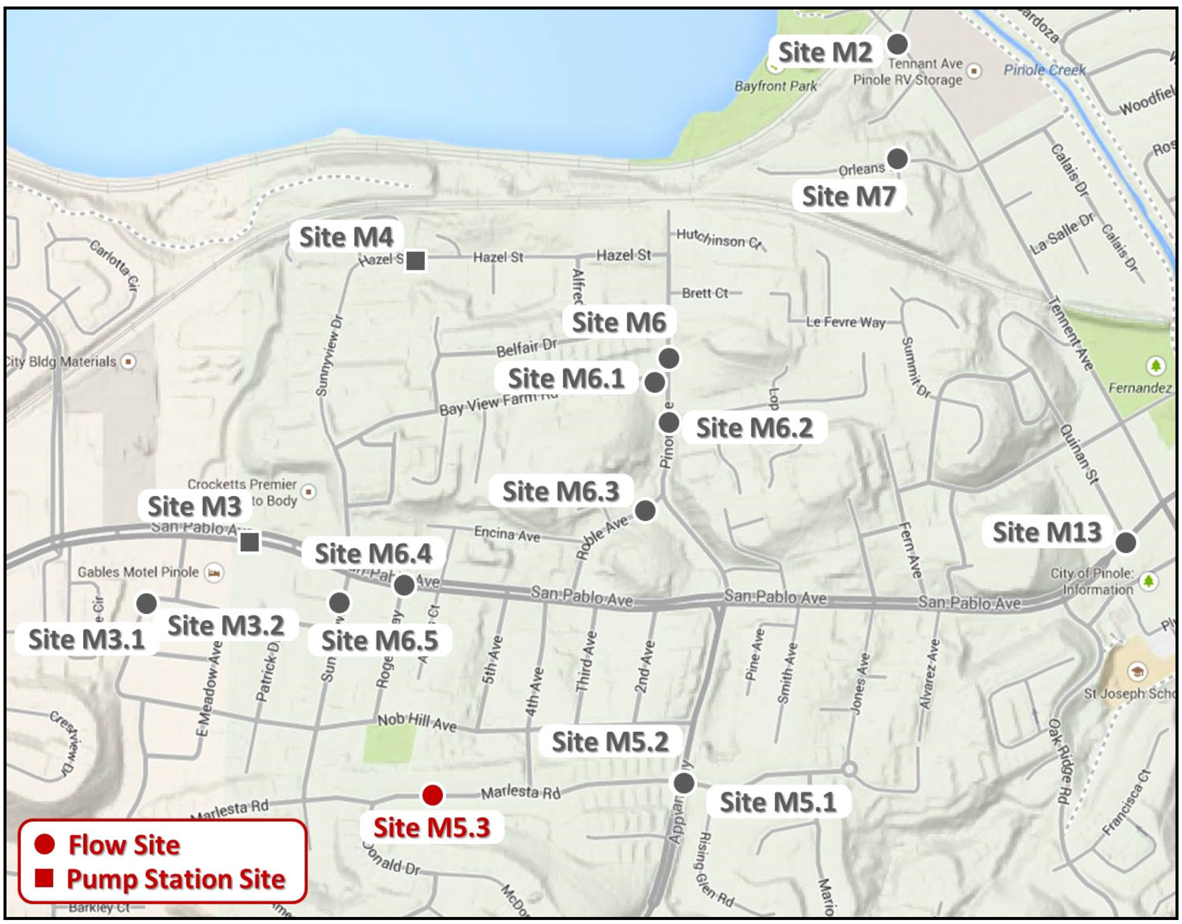
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M5.3

Location: 1171 Marlesta Rd.

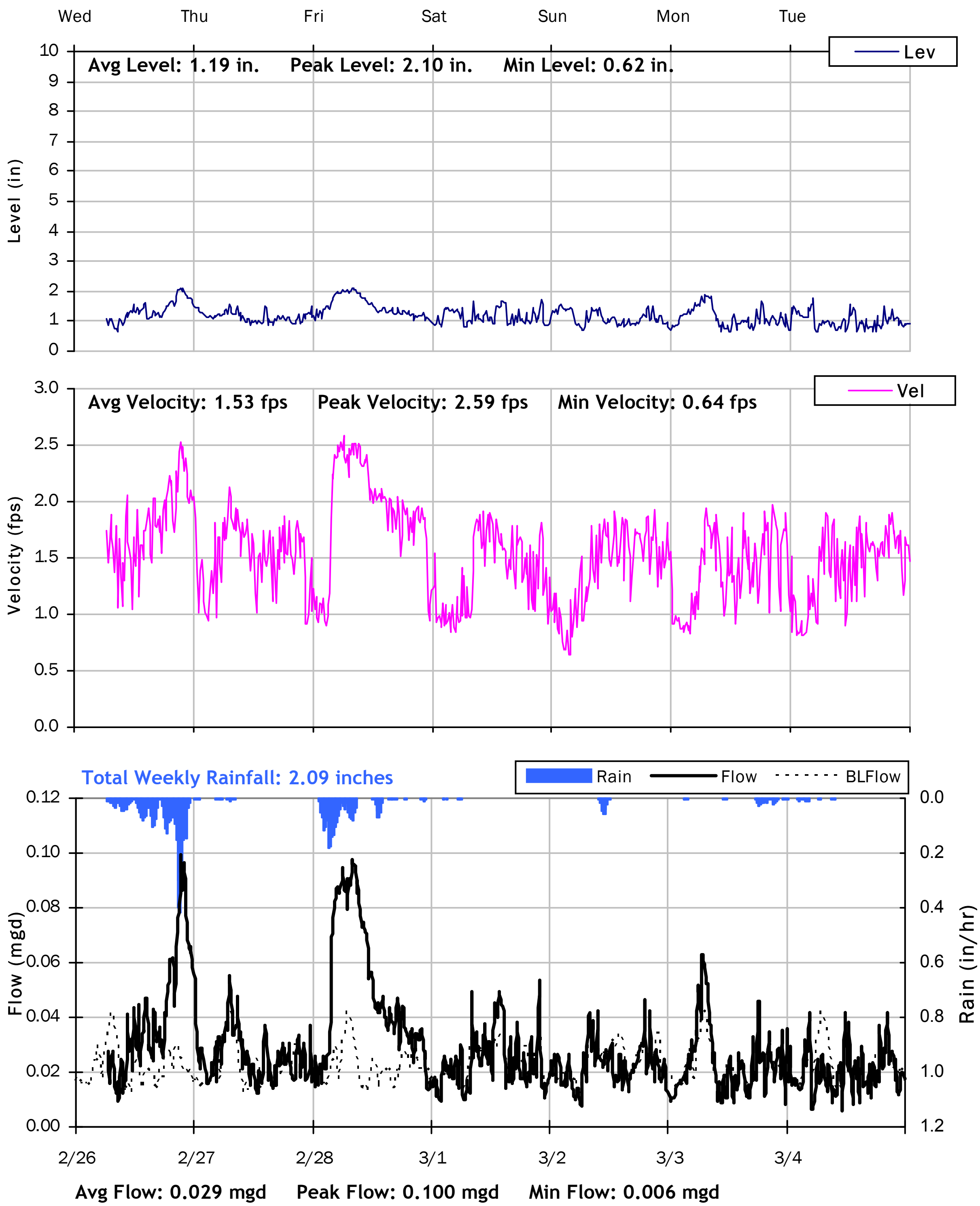
Data Summary Report



Vicinity Map: Site M5.3

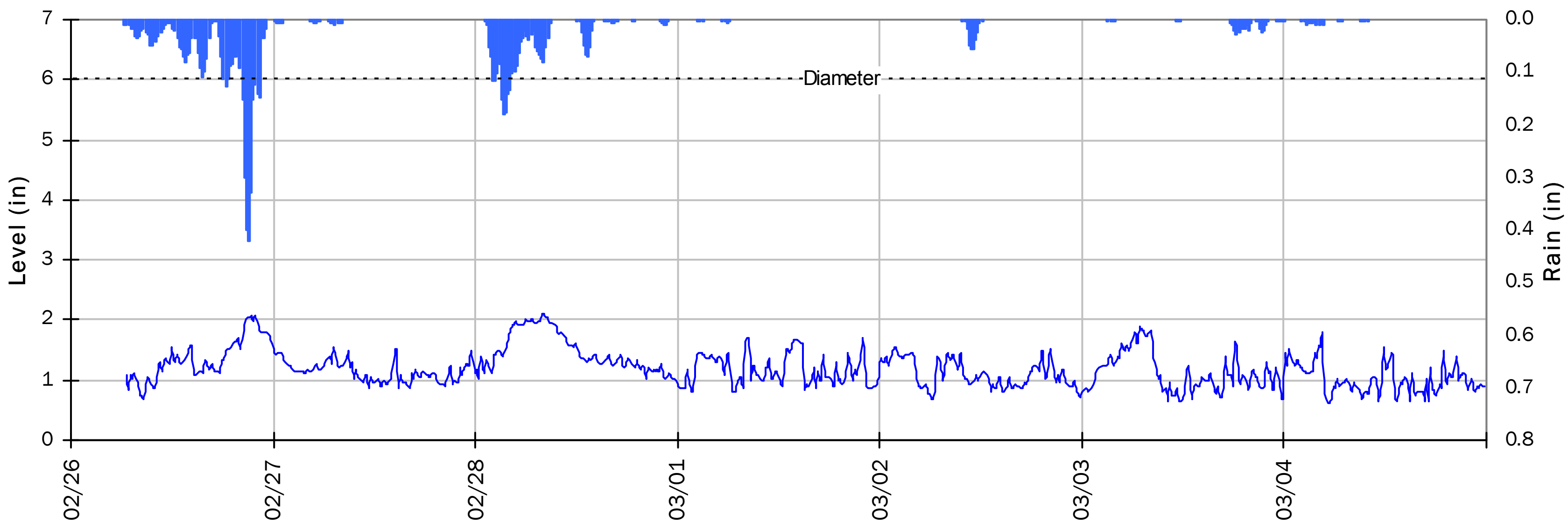
SITE M5.3

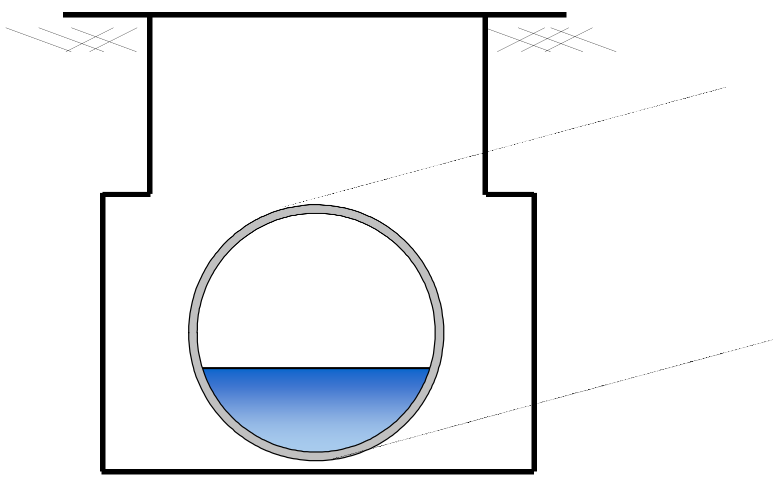
Weekly Level, Velocity and Flow Hydrographs



SITE M5.3
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



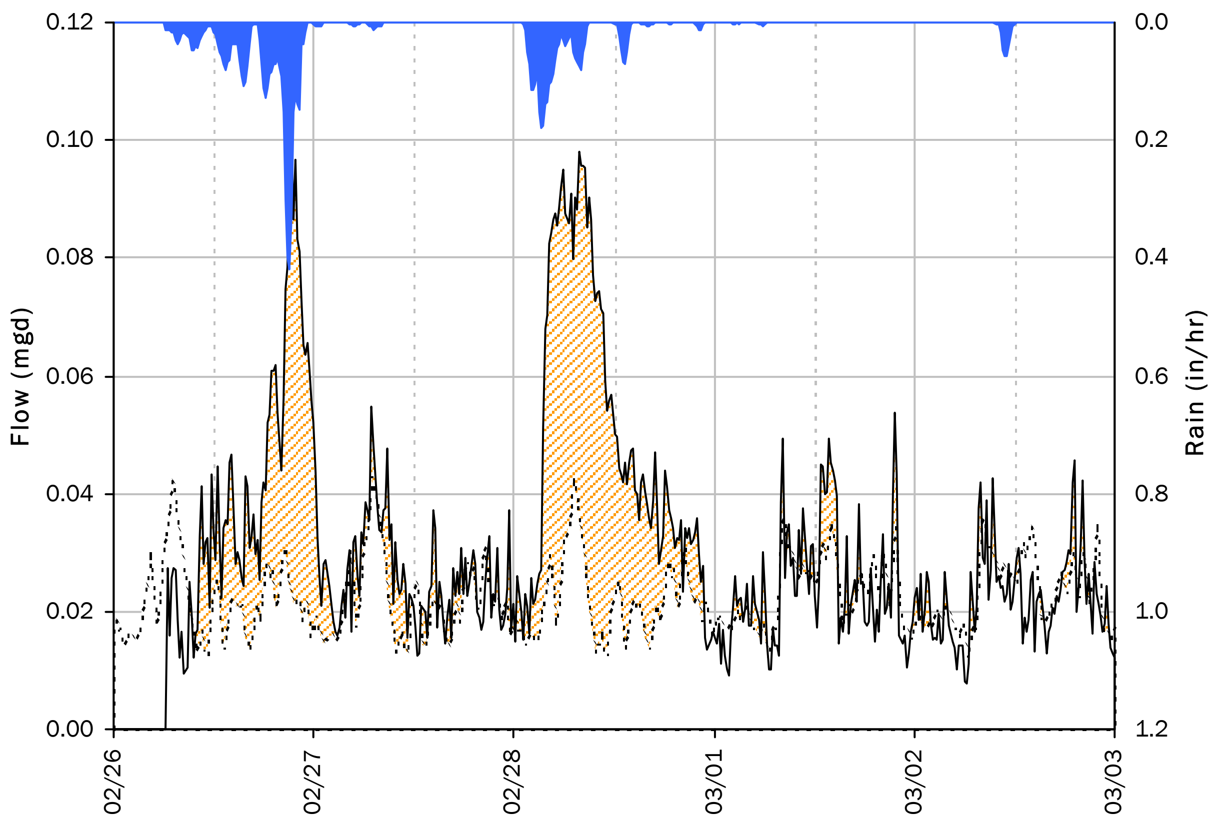


Pipe Diameter:	6	*inches*
Peak Measured Level:	2.1	*inches*
Peak d/D Ratio:	0.35	

SITE M5.3
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.10 *mgd*
PF: 4.49

Peak Level: 2.10 *in*
d/D Ratio: 0.35

Inflow / Infiltration

Peak I/I Rate: 0.08 *mgd*
Total I/I: 47,000 *gallons*

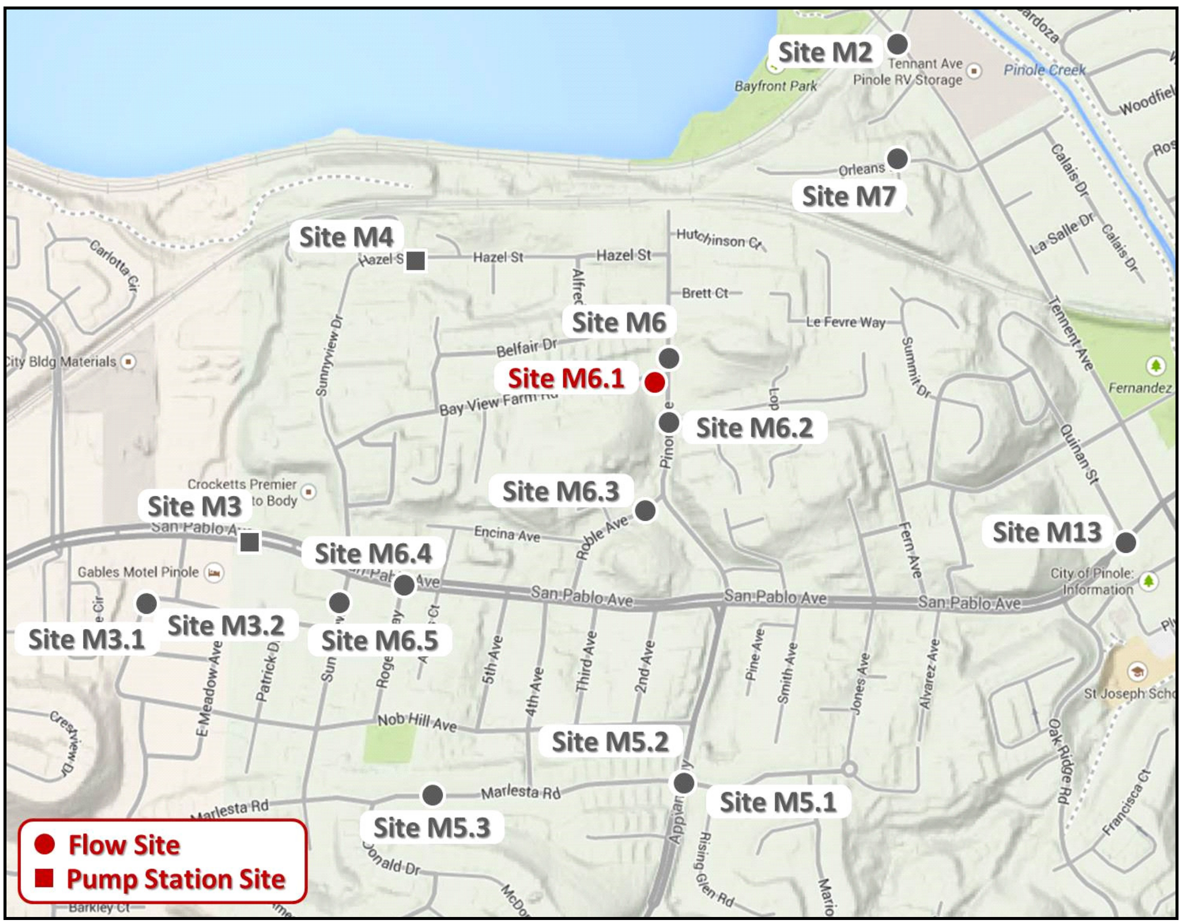
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M6.1

Location: Just west of intersection of Bay View Farm Rd. and Pinon Ave.

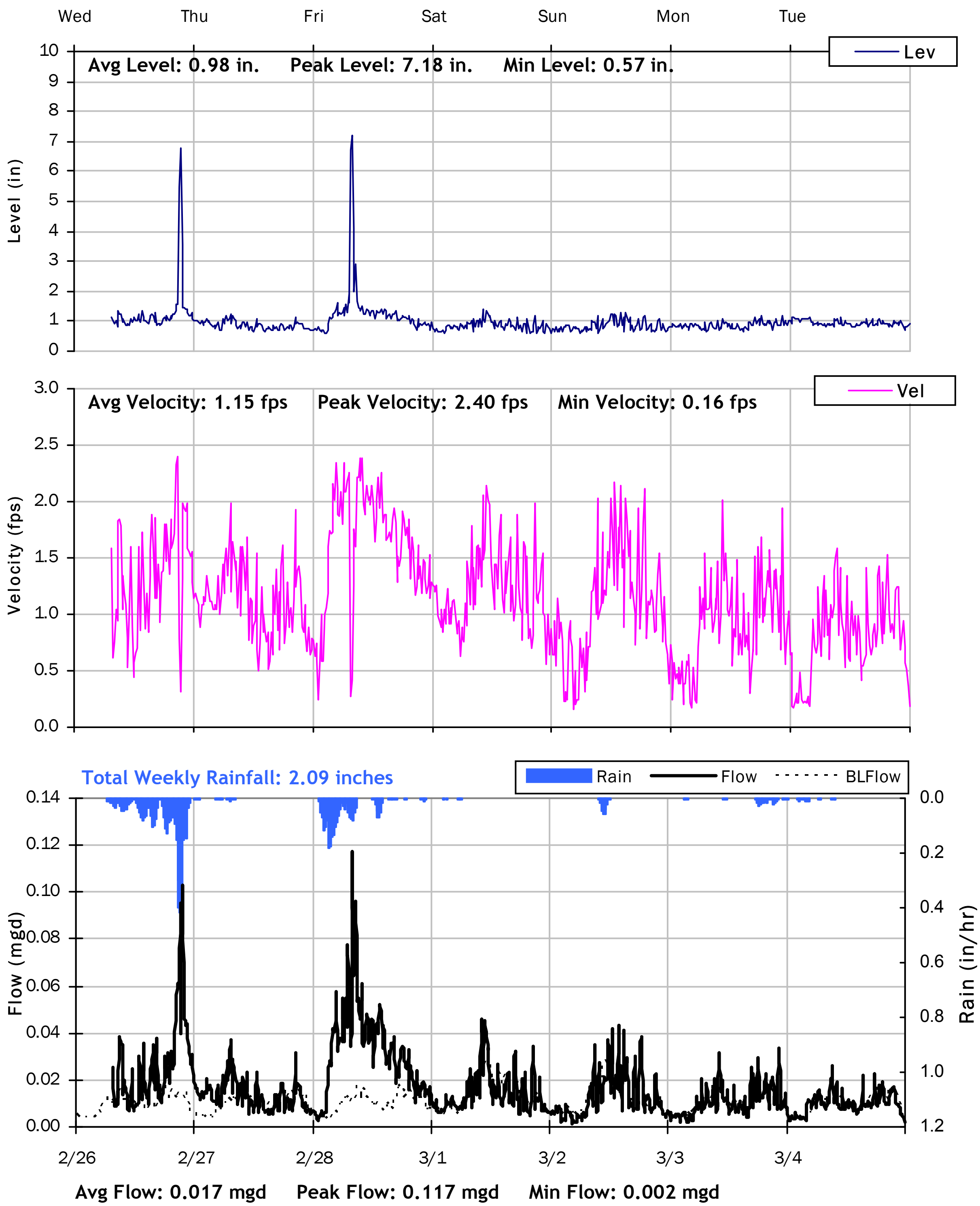
Data Summary Report



Vicinity Map: Site M6.1

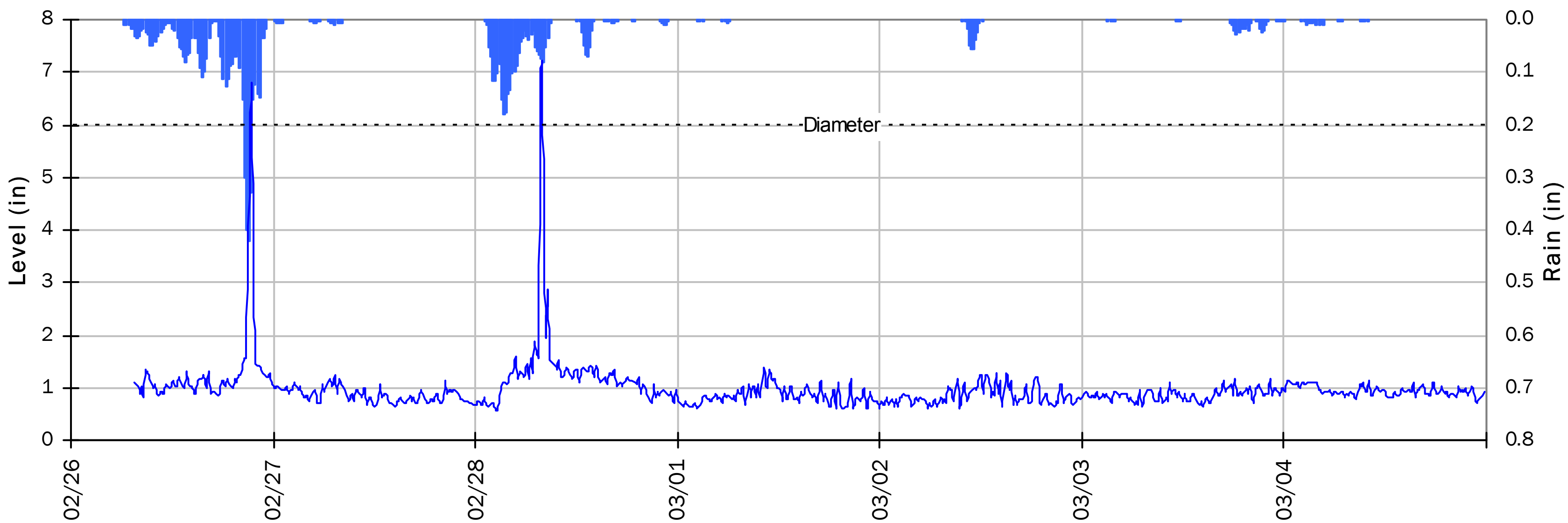
SITE M6.1

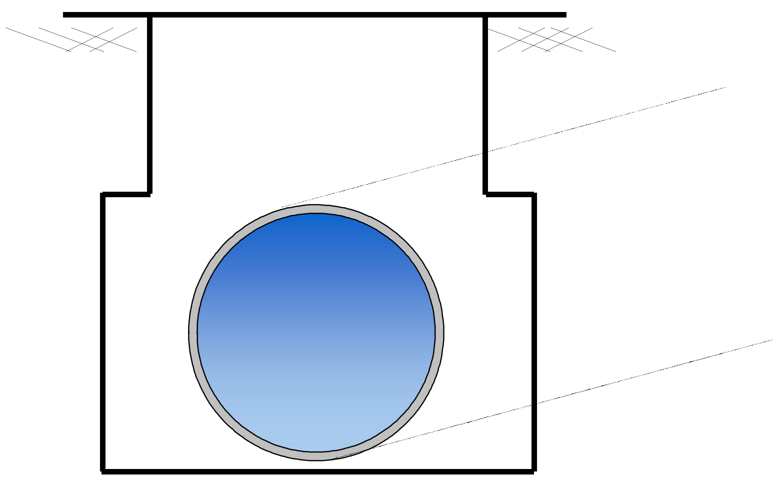
Weekly Level, Velocity and Flow Hydrographs



SITE M6.1
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





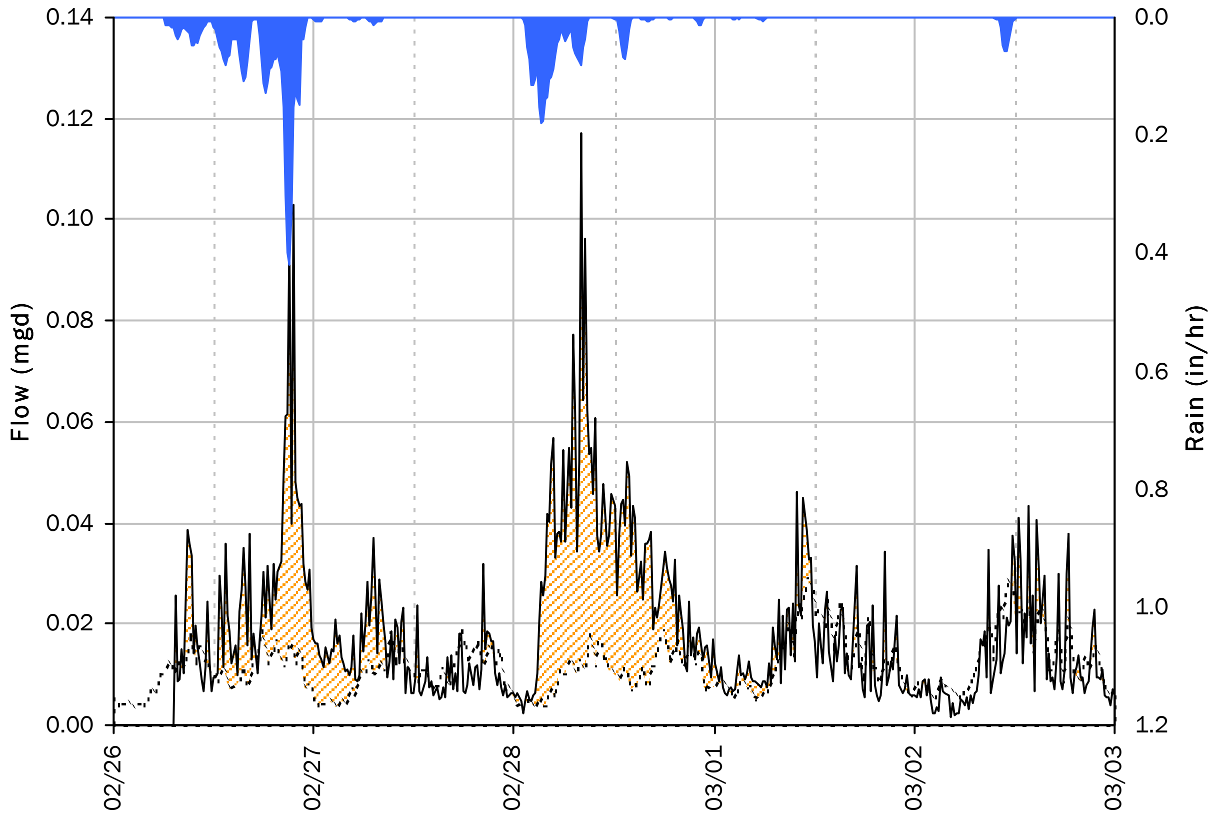
Pipe Diameter:	6	*inches*
Peak Measured Level:	7.18	*inches*
Peak d/D Ratio:	1.20	

Surcharged 1.2 inches over crown

SITE M6.1
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.12 *mgd*
PF: 10.37

Peak Level: 7.18 *in*
d/D Ratio: 1.20

Inflow / Infiltration

Peak I/I Rate: 0.10 *mgd*
Total I/I: 35,000 *gallons*

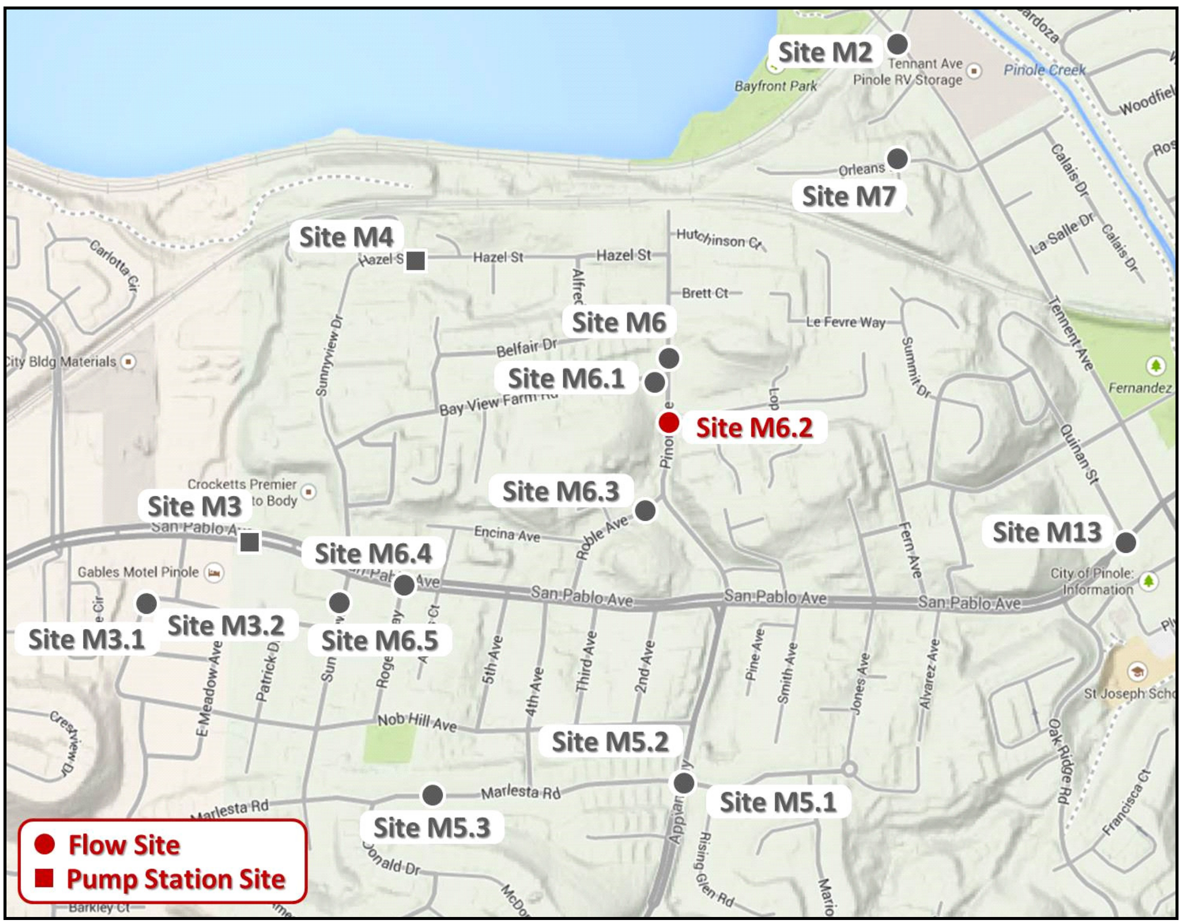
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M6.2

Location: Intersection of Pinon Ave. and Primrose Ln.

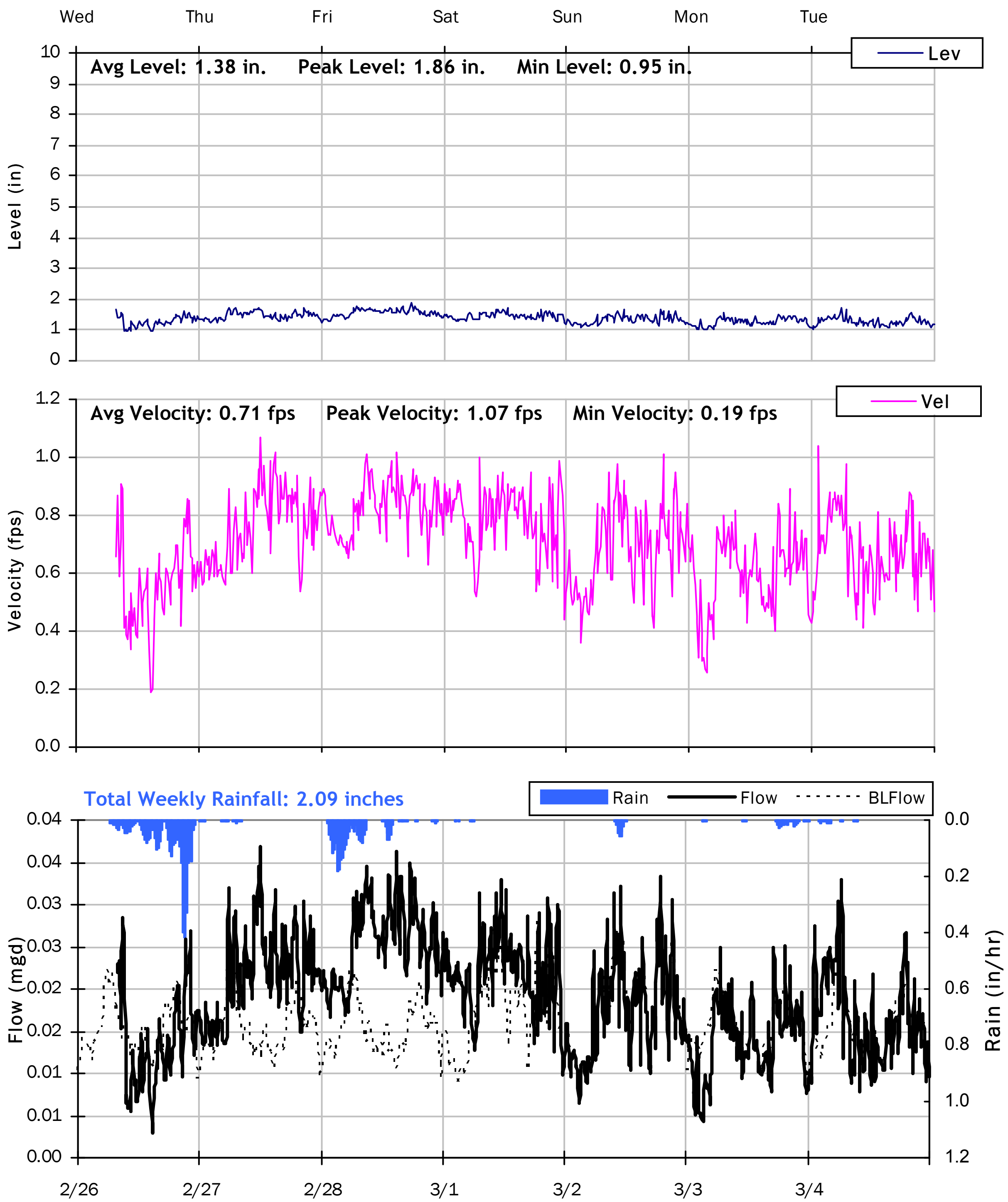
Data Summary Report



Vicinity Map: Site M6.2

SITE M6.2

Weekly Level, Velocity and Flow Hydrographs

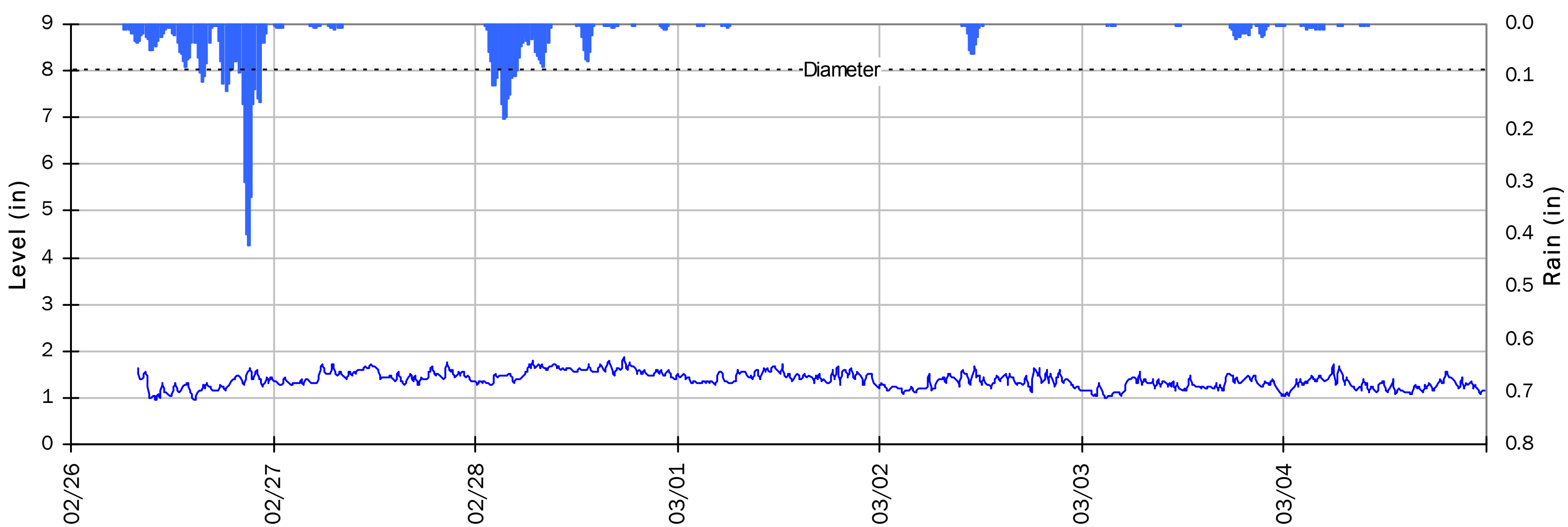


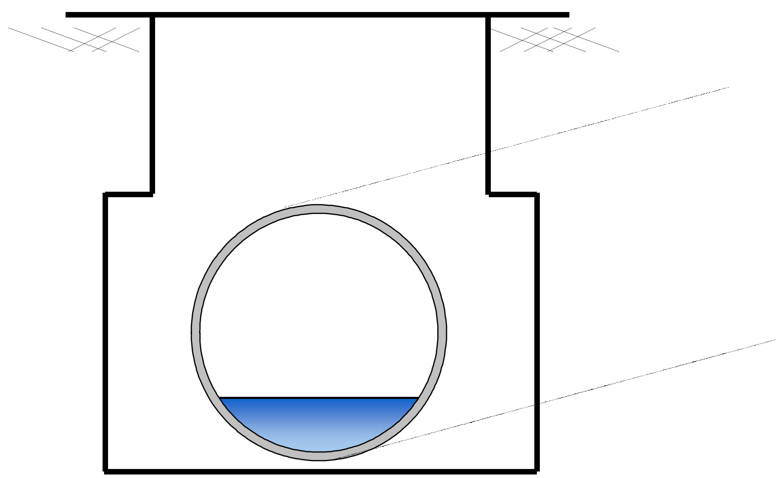
Avg Flow: 0.019 mgd **Peak Flow: 0.037 mgd** **Min Flow: 0.003 mgd**

SITE M6.2

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



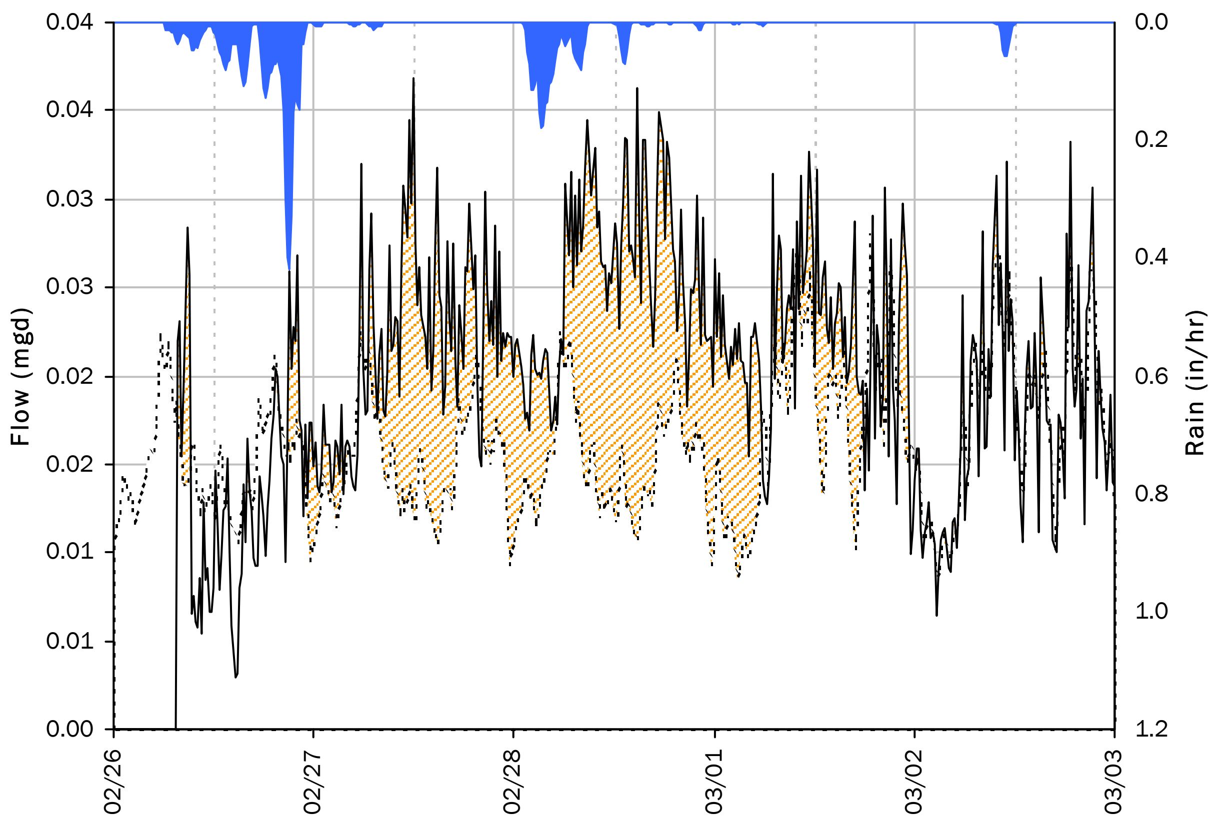


Pipe Diameter:	8	*inches*
Peak Measured Level:	1.86	*inches*
Peak d/D Ratio:	0.23	

SITE M6.2
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.04 *mgd*
PF: 2.34

Peak Level: 1.86 *in*
d/D Ratio: 0.23

Inflow / Infiltration

Peak I/I Rate: 0.03 *mgd*
Total I/I: 21,000 *gallons*

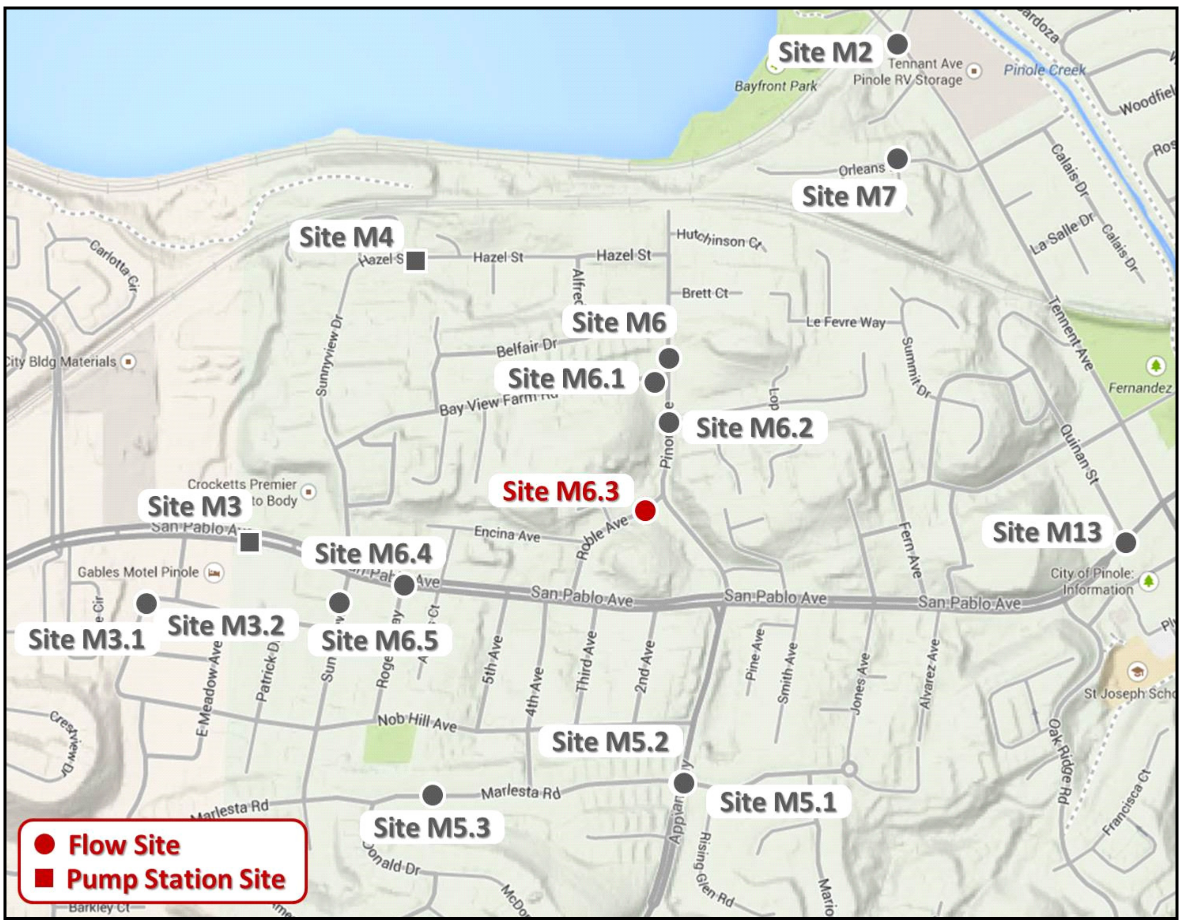
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M6.3

Location: Roble Ave., west of Pinon Ave.

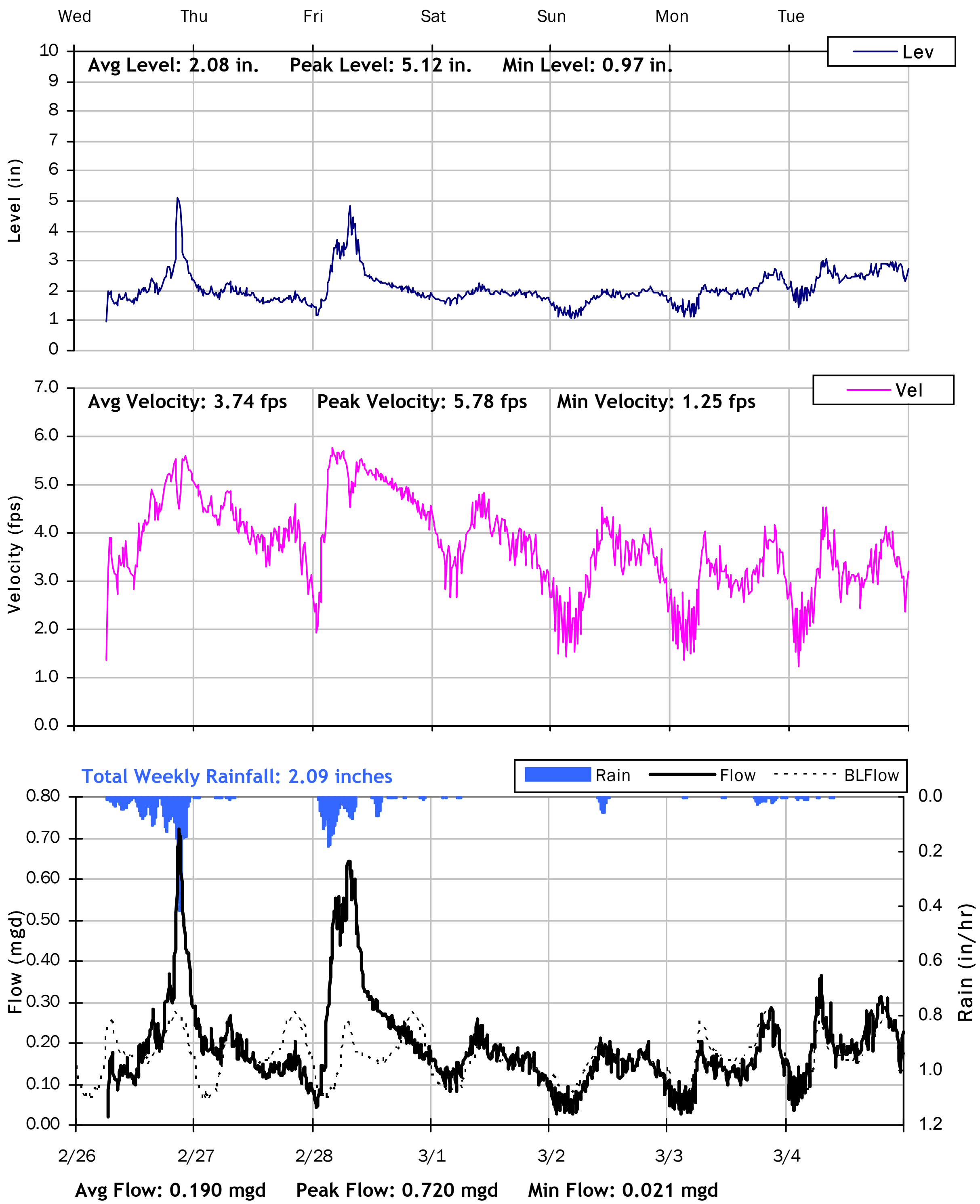
Data Summary Report



Vicinity Map: Site M6.3

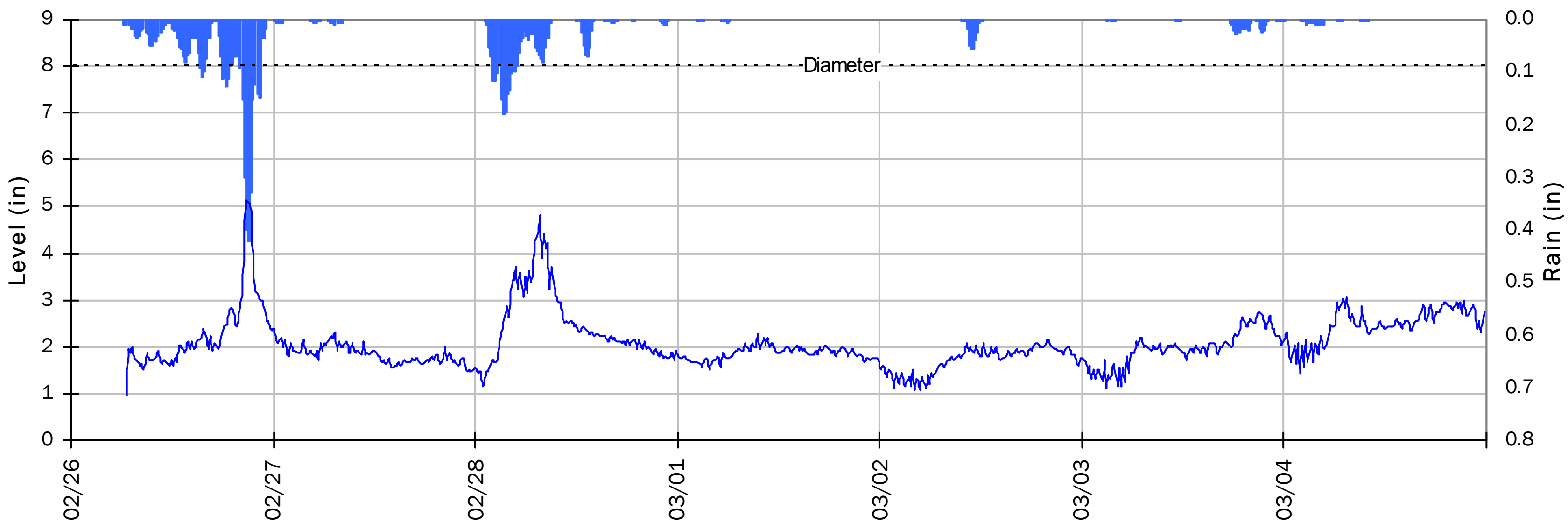
SITE M6.3

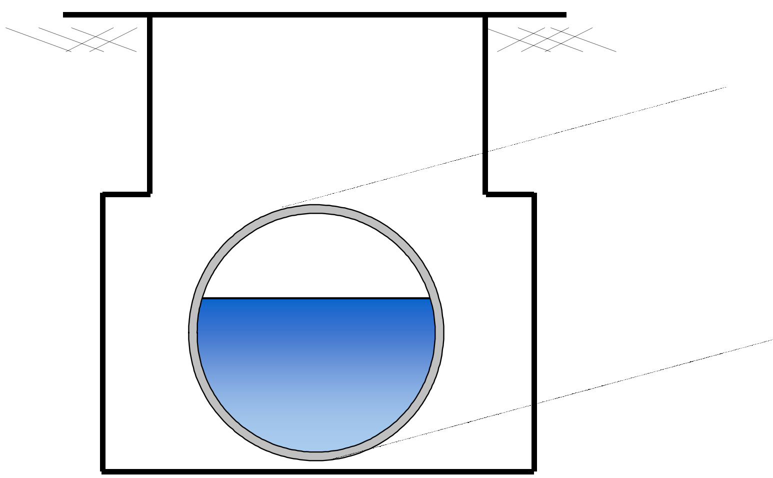
Weekly Level, Velocity and Flow Hydrographs



SITE M6.3
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



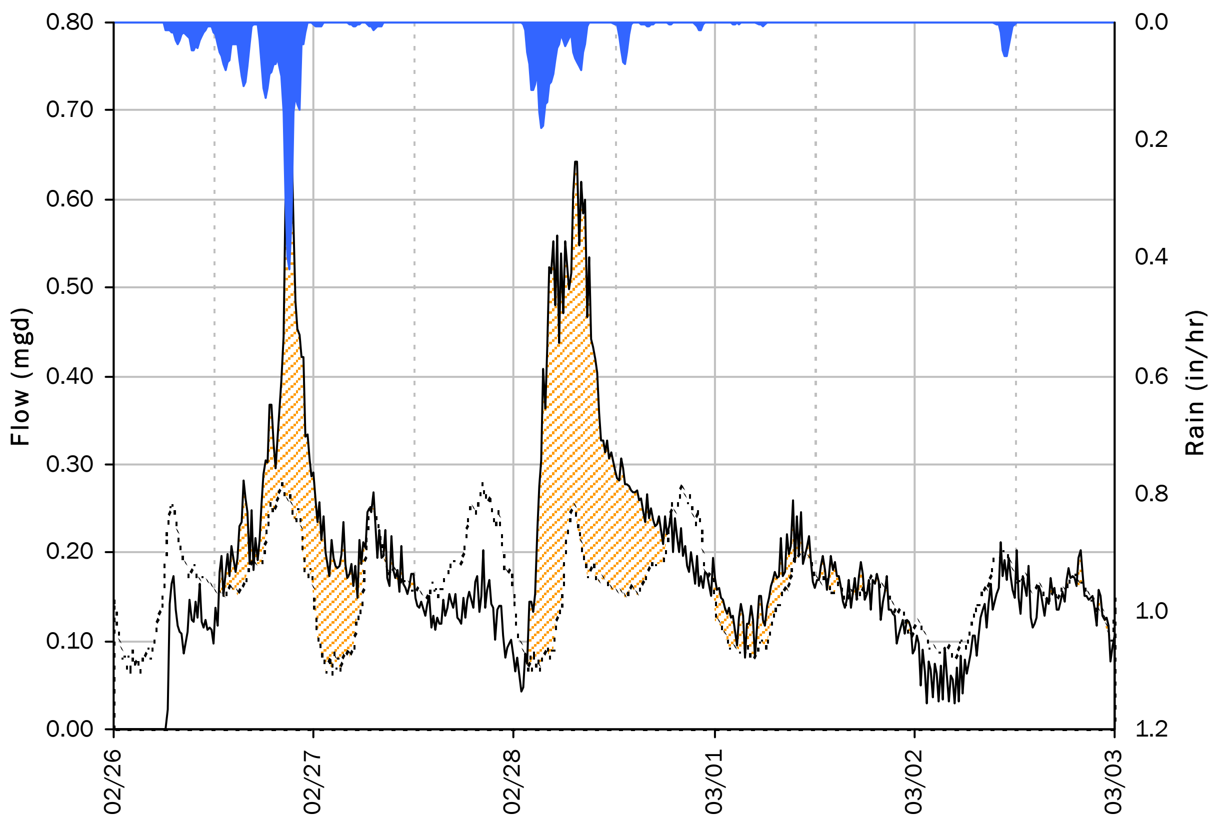


Pipe Diameter:	8	*inches*
Peak Measured Level:	5.12	*inches*
Peak d/D Ratio:	0.64	

SITE M6.3
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.72 *mgd*
PF: 4.43

Peak Level: 5.12 *in*
d/D Ratio: 0.64

Inflow / Infiltration

Peak I/I Rate: 0.46 *mgd*
Total I/I: 198,000 *gallons*

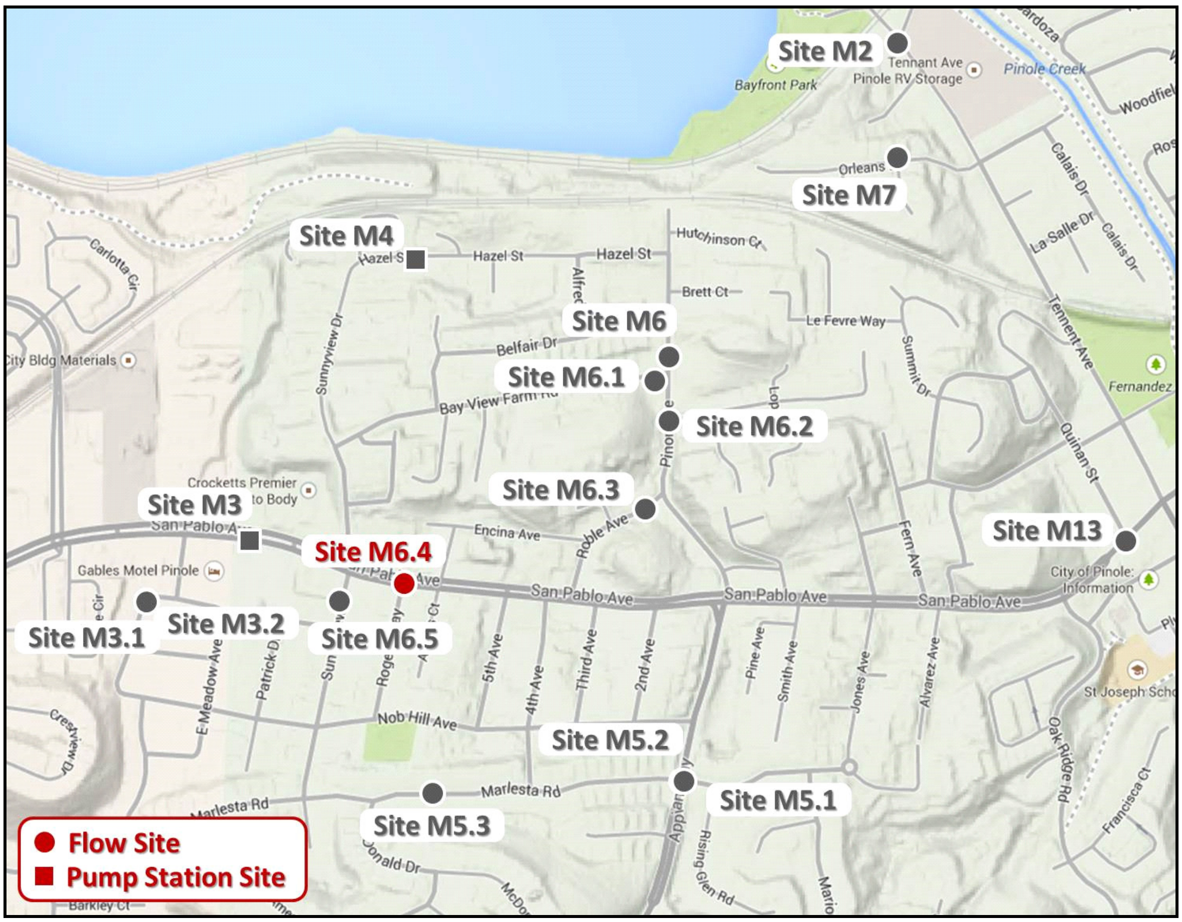
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M6.4

Location: Intersection of San Pablo Ave. and Rogers Way

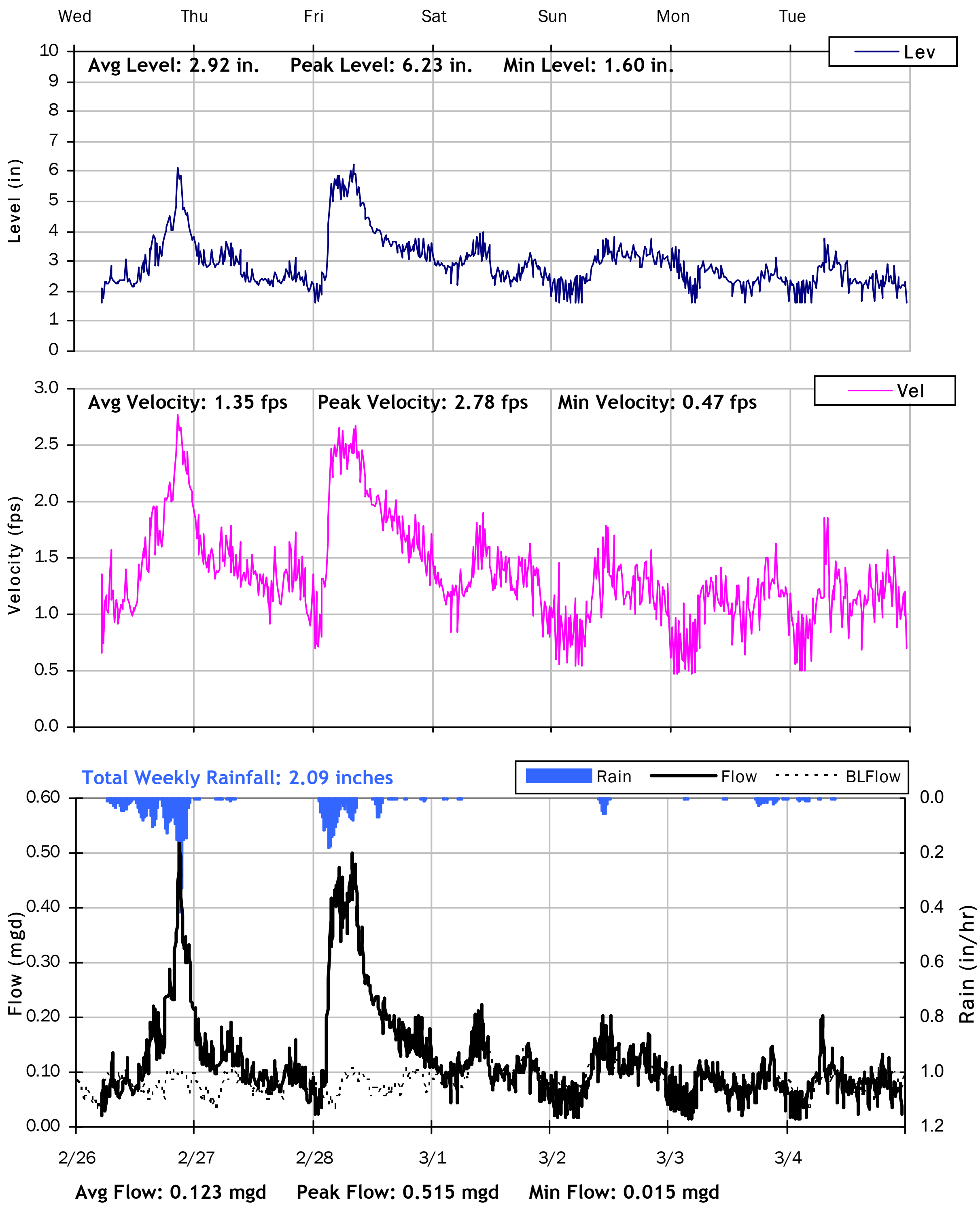
Data Summary Report



Vicinity Map: Site M6.4

SITE M6.4

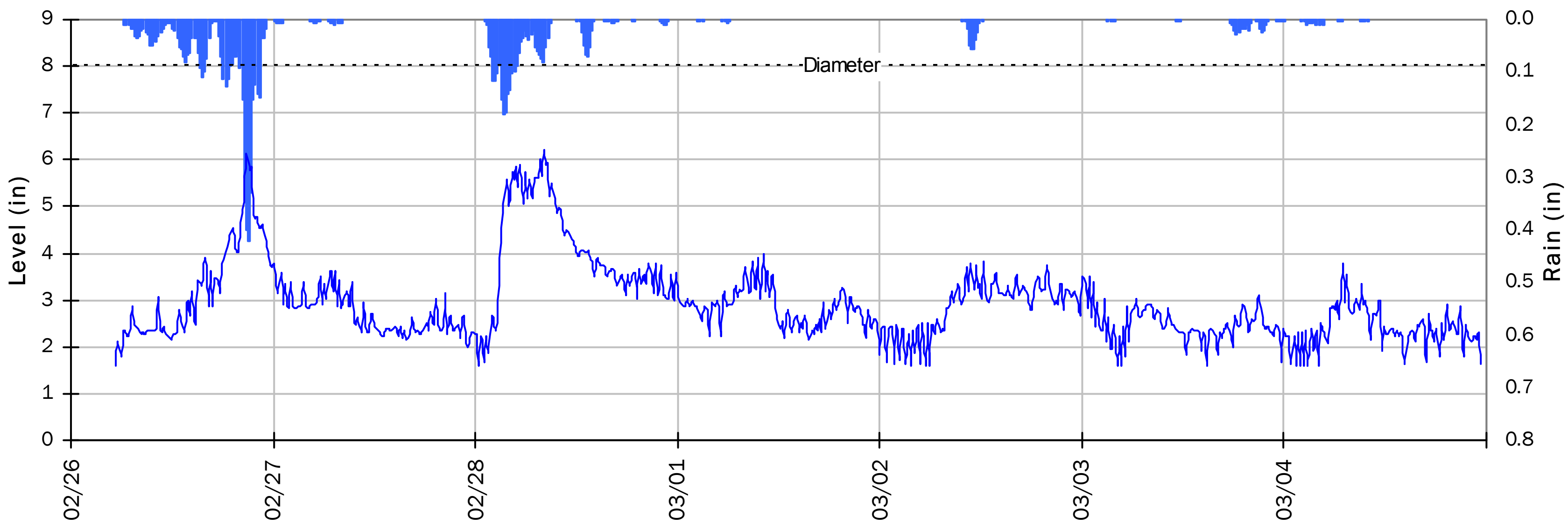
Weekly Level, Velocity and Flow Hydrographs

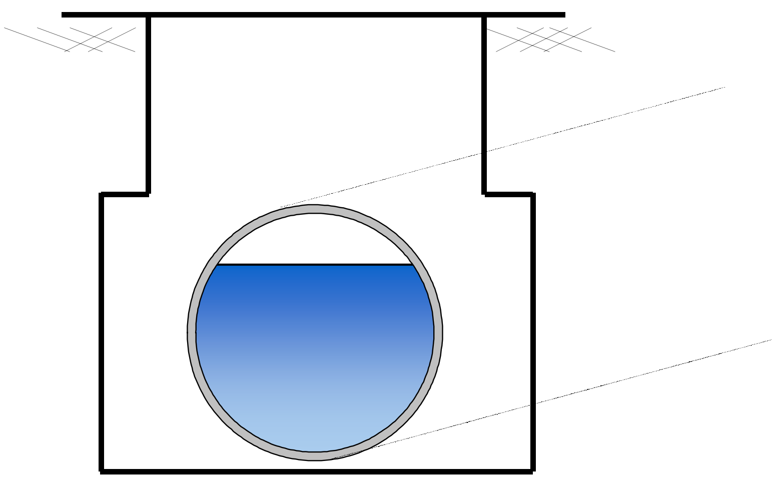


SITE M6.4

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



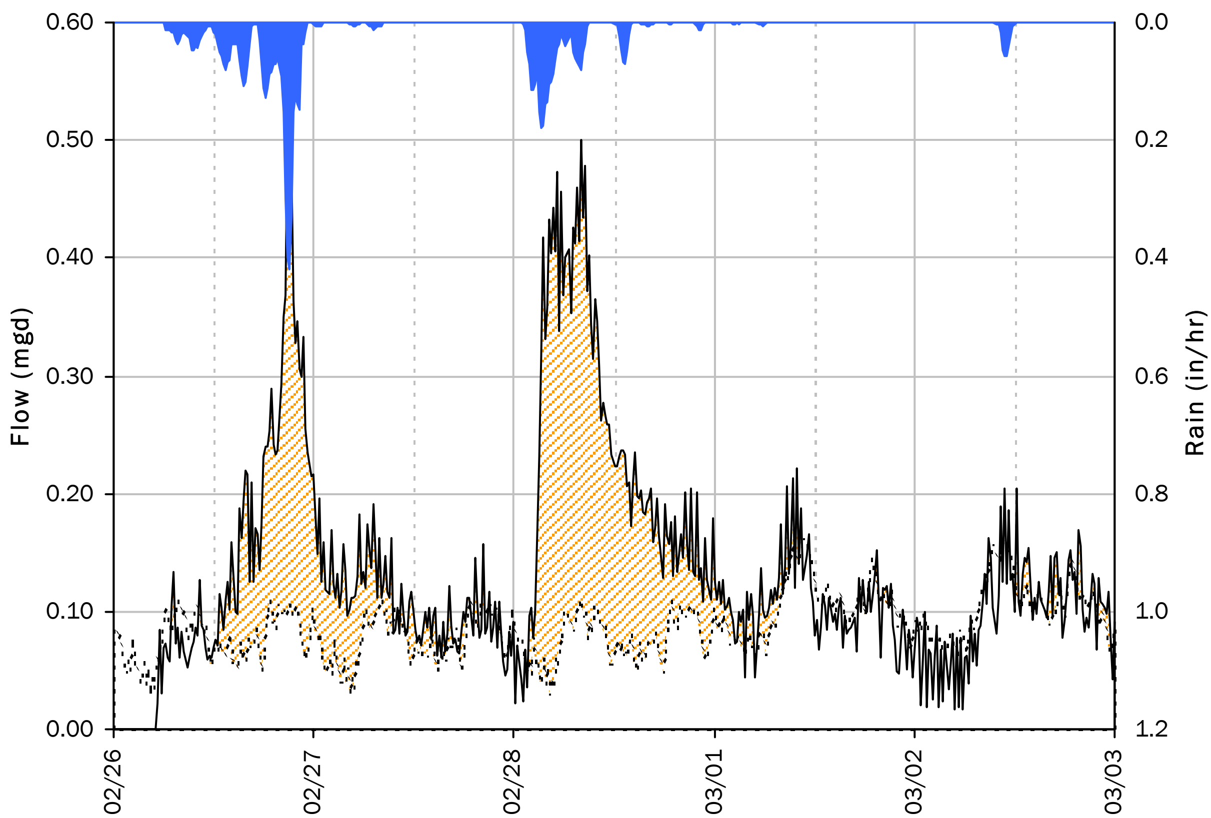


Pipe Diameter:	8	*inches*
Peak Measured Level:	6.23	*inches*
Peak d/D Ratio:	0.78	

SITE M6.4
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.51 *mgd*
PF: 6.14

Peak Level: 6.23 *in*
d/D Ratio: 0.78

Inflow / Infiltration

Peak I/I Rate: 0.42 *mgd*
Total I/I: 267,000 *gallons*

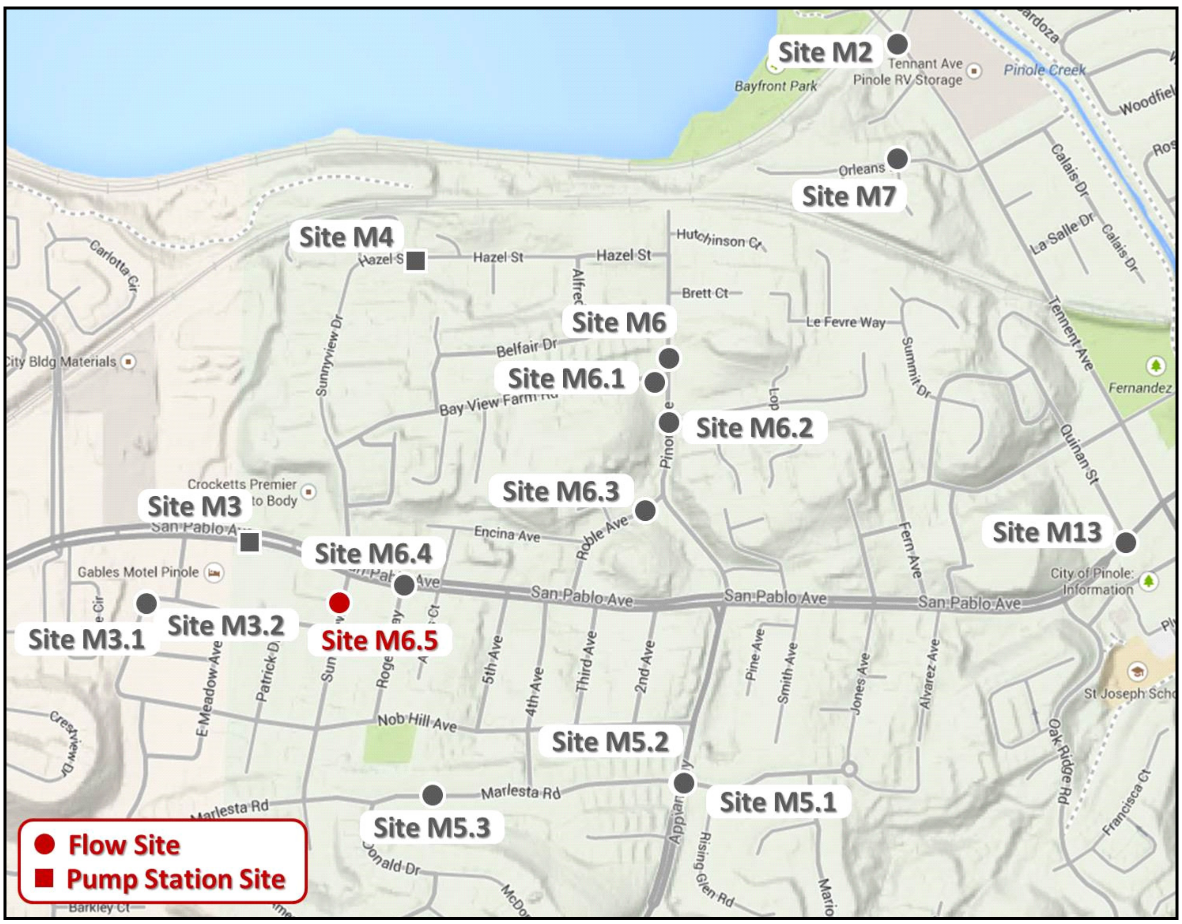
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 2014

Monitoring Site: Site M6.5

Location: 747 Sunnyview Dr.

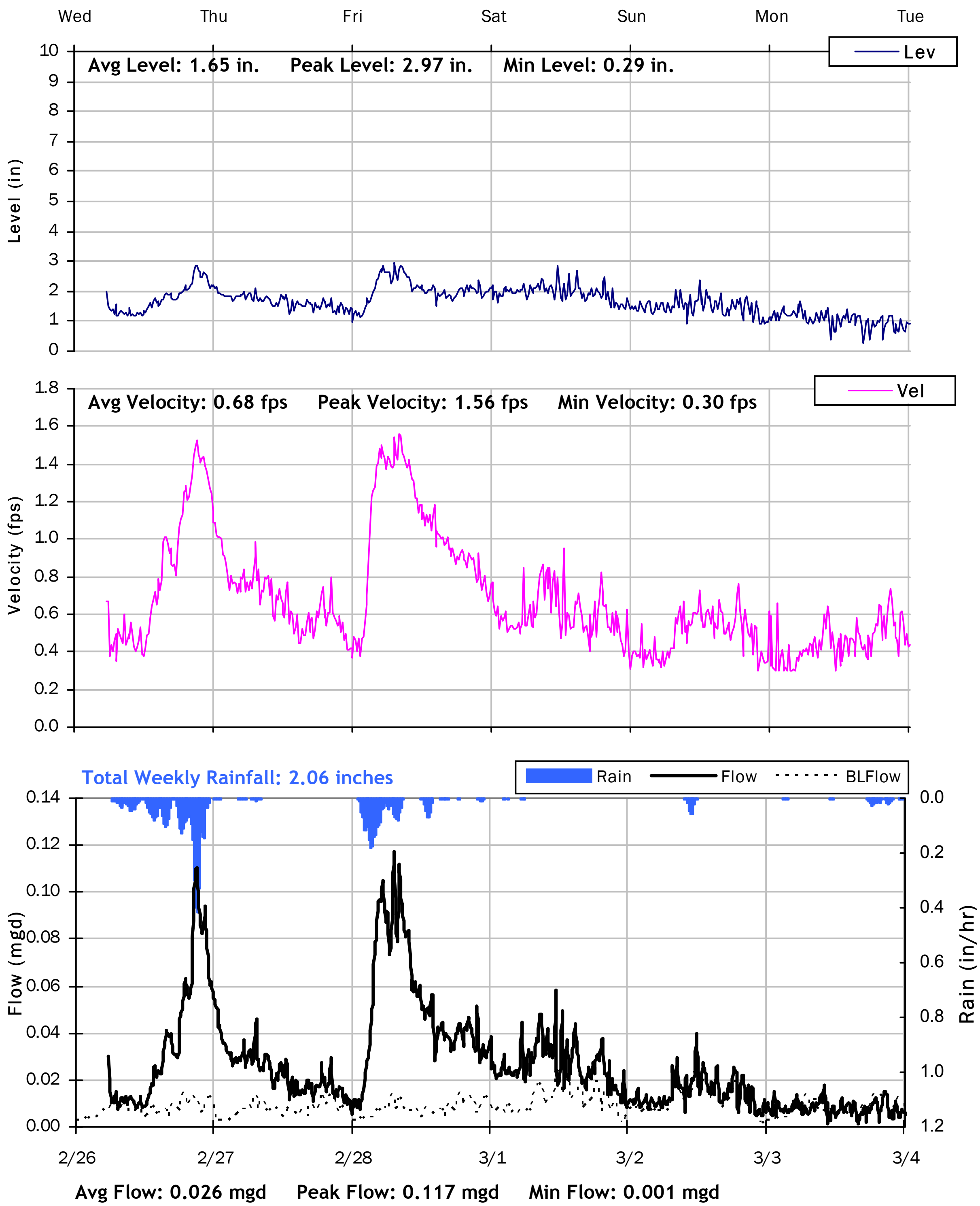
Data Summary Report



Vicinity Map: Site M6.5

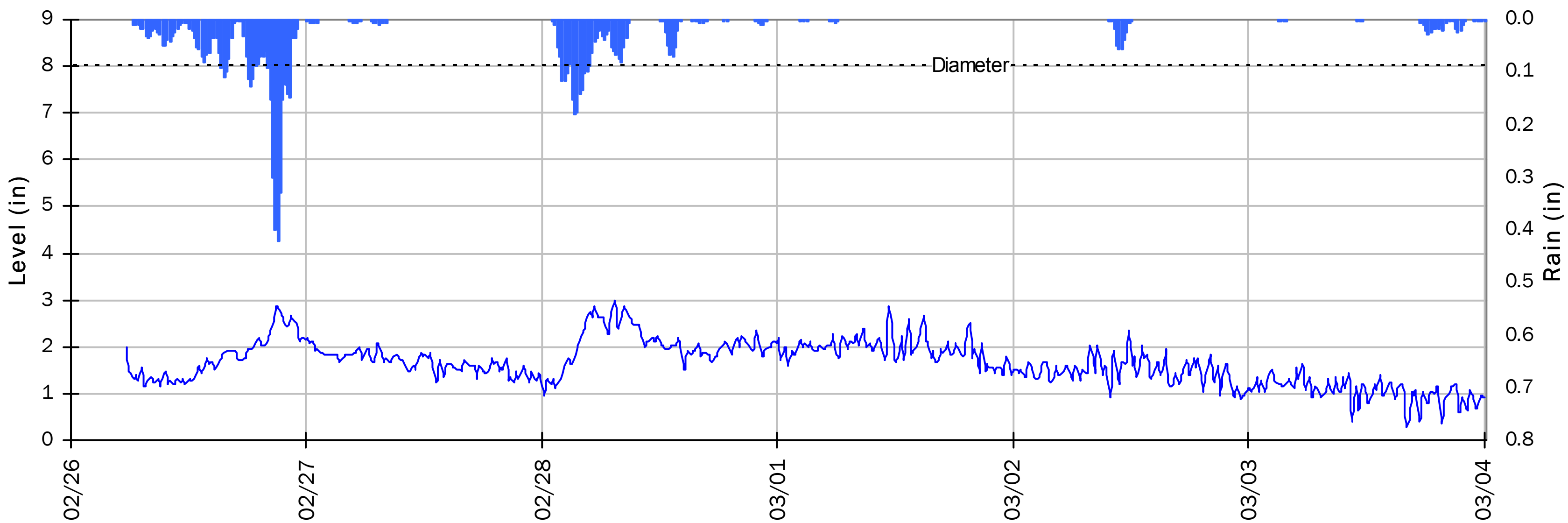
SITE M6.5

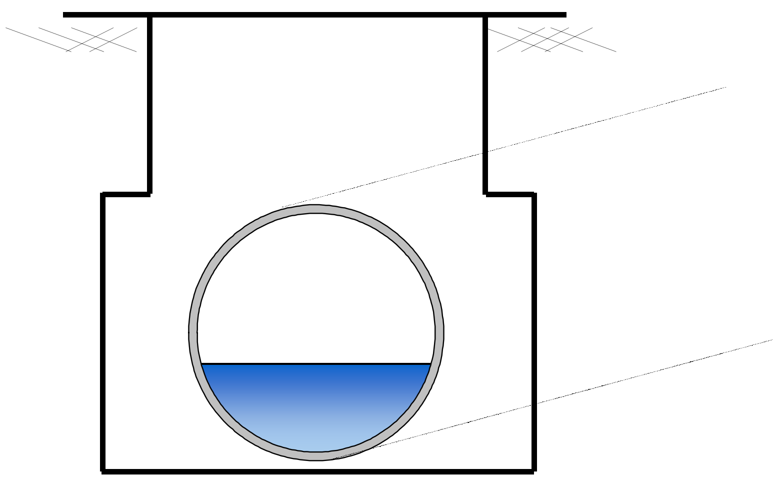
Weekly Level, Velocity and Flow Hydrographs



SITE M6.5
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



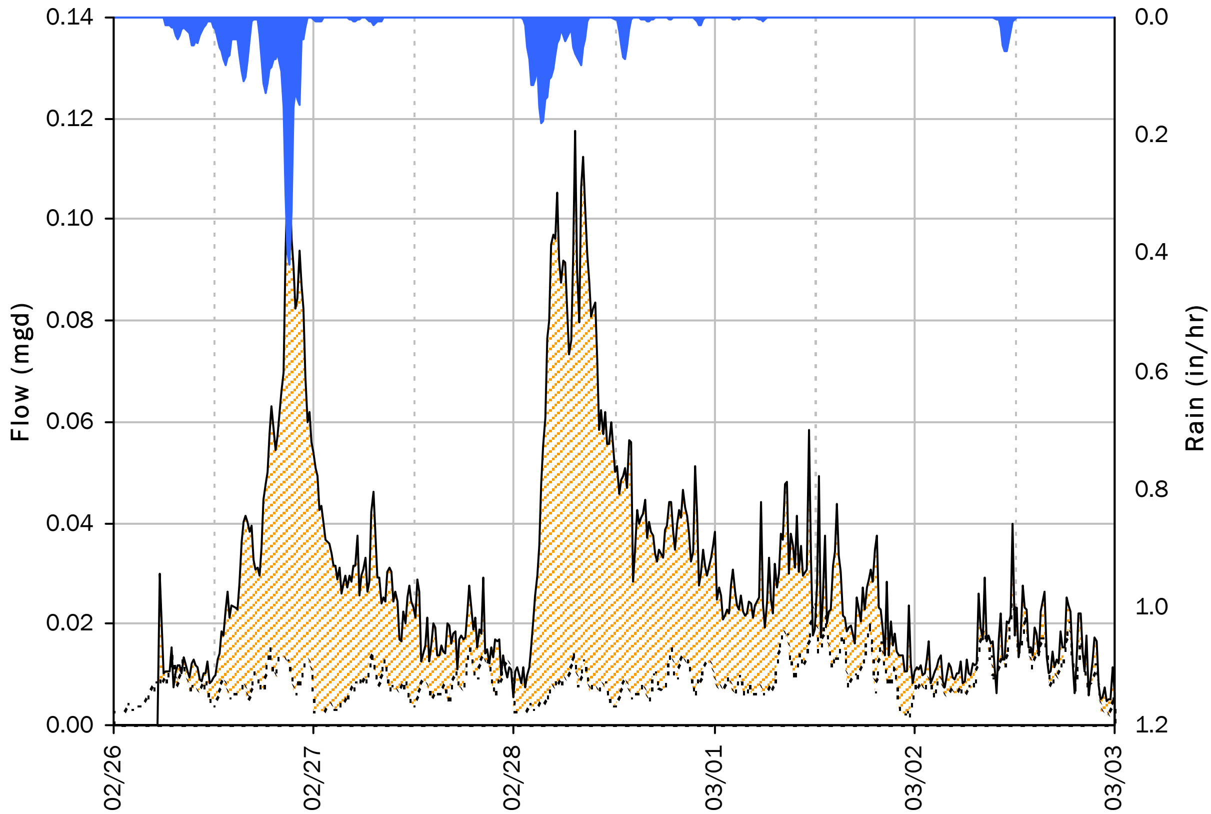


Pipe Diameter:	8	*inches*
Peak Measured Level:	2.97	*inches*
Peak d/D Ratio:	0.37	

SITE M6.5
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 1.98 inches)

Capacity

Peak Flow: 0.12 *mgd*
PF: 13.49

Peak Level: 2.97 *in*
d/D Ratio: 0.37

Inflow / Infiltration

Peak I/I Rate: 0.11 *mgd*
Total I/I: 96,000 *gallons*

Appendix F. Flow Monitoring Sites Data, Graphs, Information: Phase 3

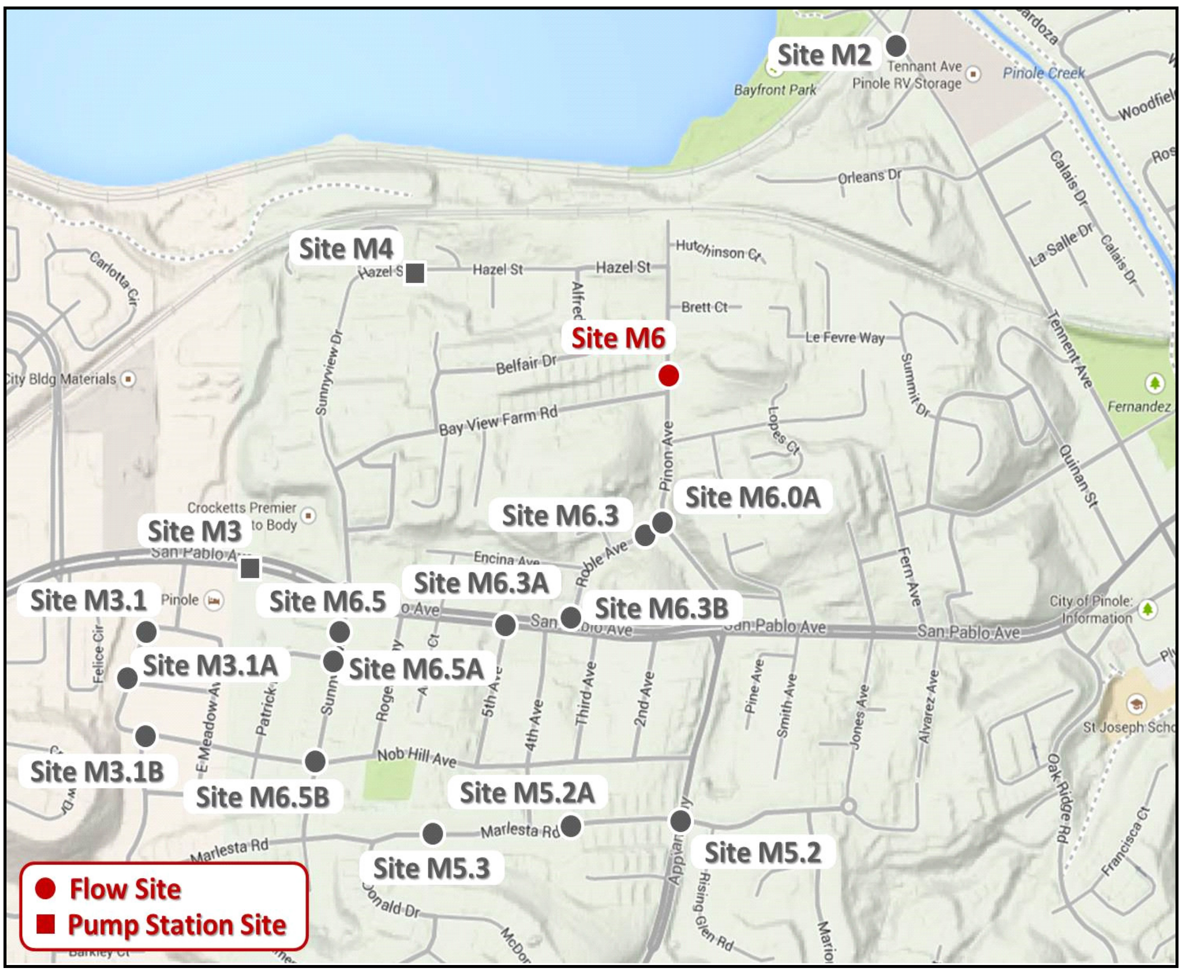
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6

Location: Pinon Ave., north of Bay View Farm Rd.

Data Summary Report

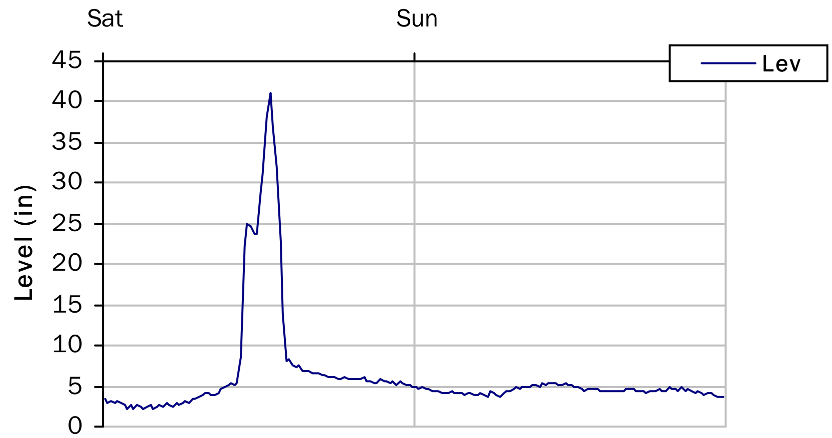


Vicinity Map: Site M6

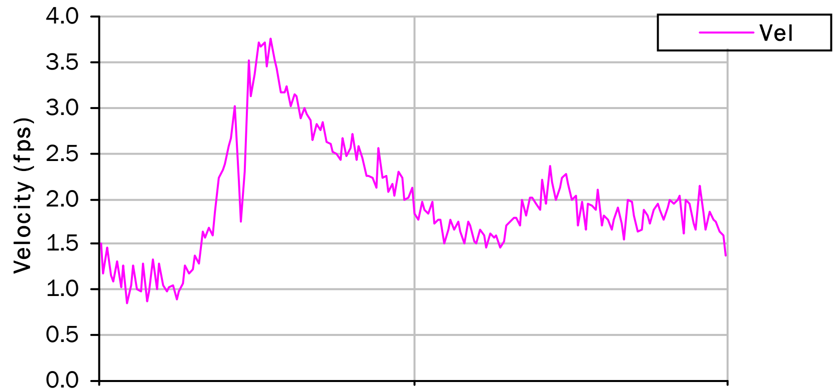
SITE M6

Weekly Level, Velocity and Flow Hydrographs

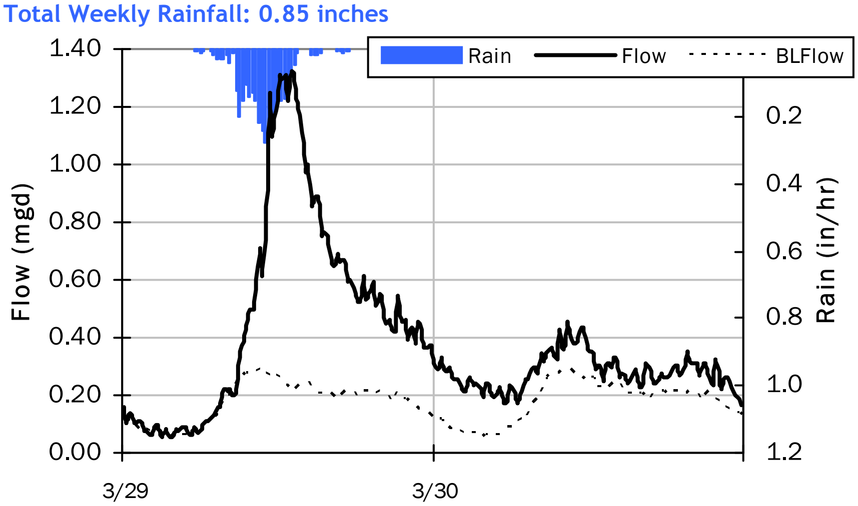
Avg Level: 6.18 in.   Peak Level: 41.01 in.   Min Level: 2.12 in.



Avg Velocity: 1.98 fps   Peak Velocity: 3.76 fps   Min Velocity: 0.86 fps



Total Weekly Rainfall: 0.85 inches

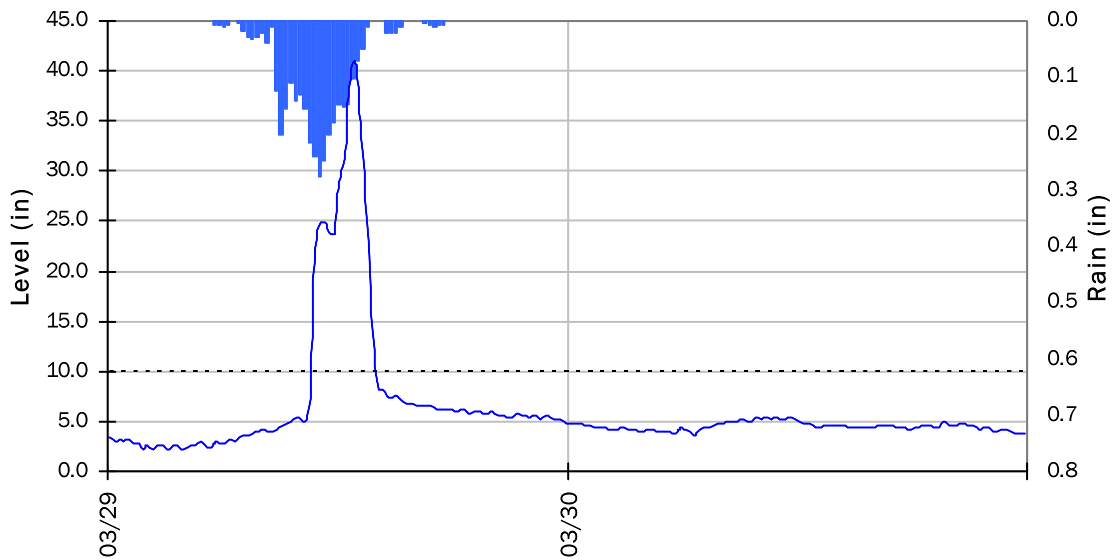


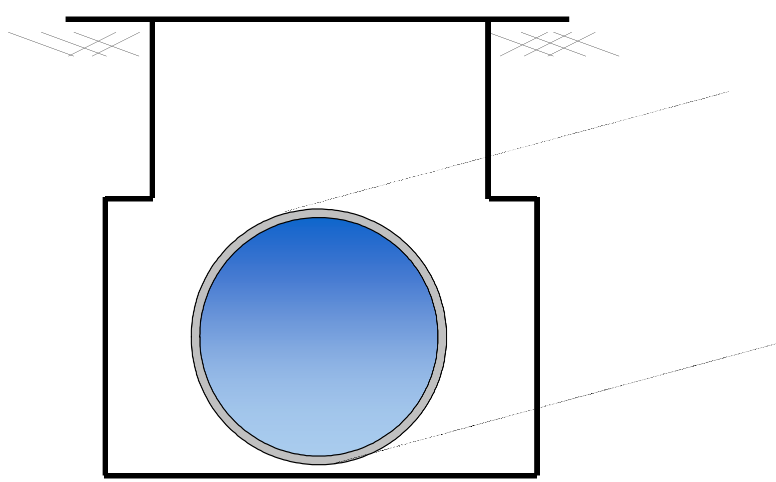
Avg Flow: 0.380 mgd   Peak Flow: 1.324 mgd   Min Flow: 0.053 mgd

SITE M6

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





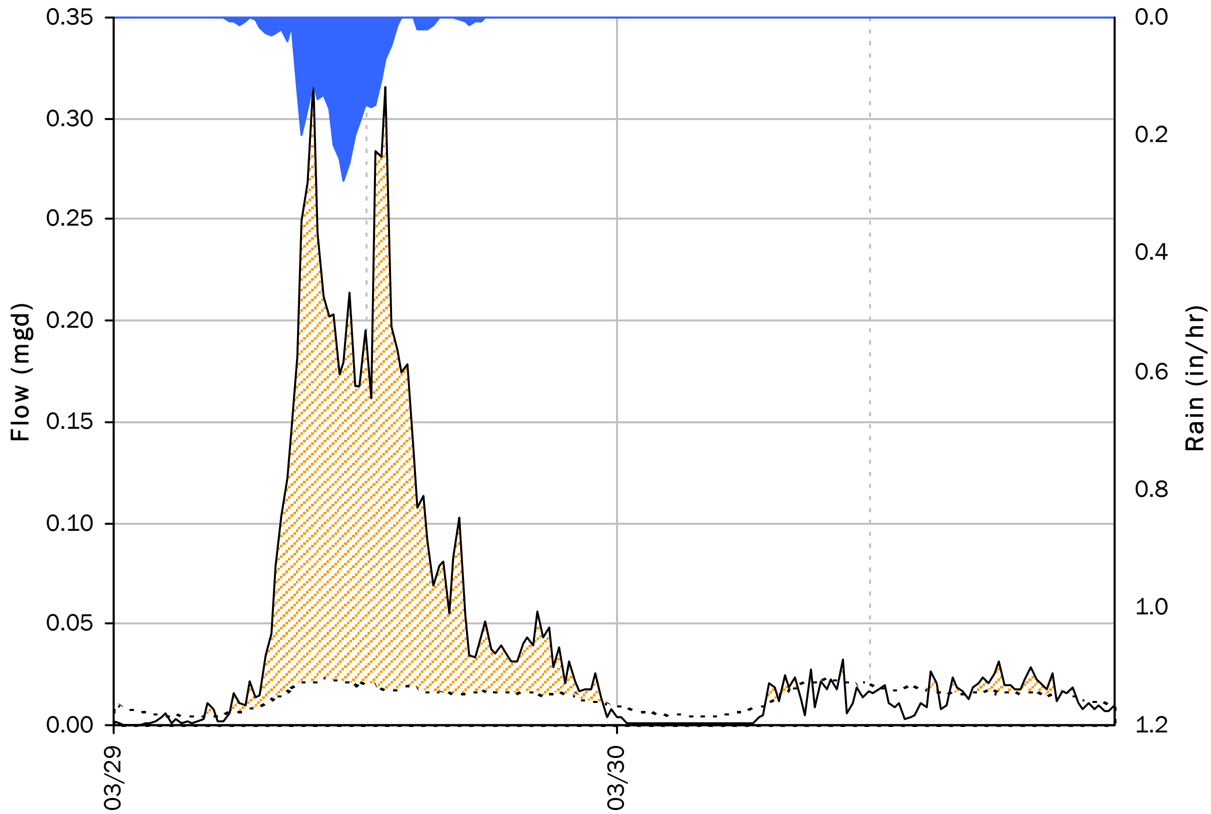
Pipe Diameter:	10	*inches*
Peak Measured Level:	41.0	*inches*
Peak d/D Ratio:	4.10	

Surcharged 31.0 inches over crown

SITE 3.1
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.32 *mgd*
PF: 24.33

Peak Level: 2.61 *in*
d/D Ratio: 0.44

Inflow / Infiltration

Peak I/I Rate: 0.30 *mgd*
Total I/I: 60,000 *gallons*

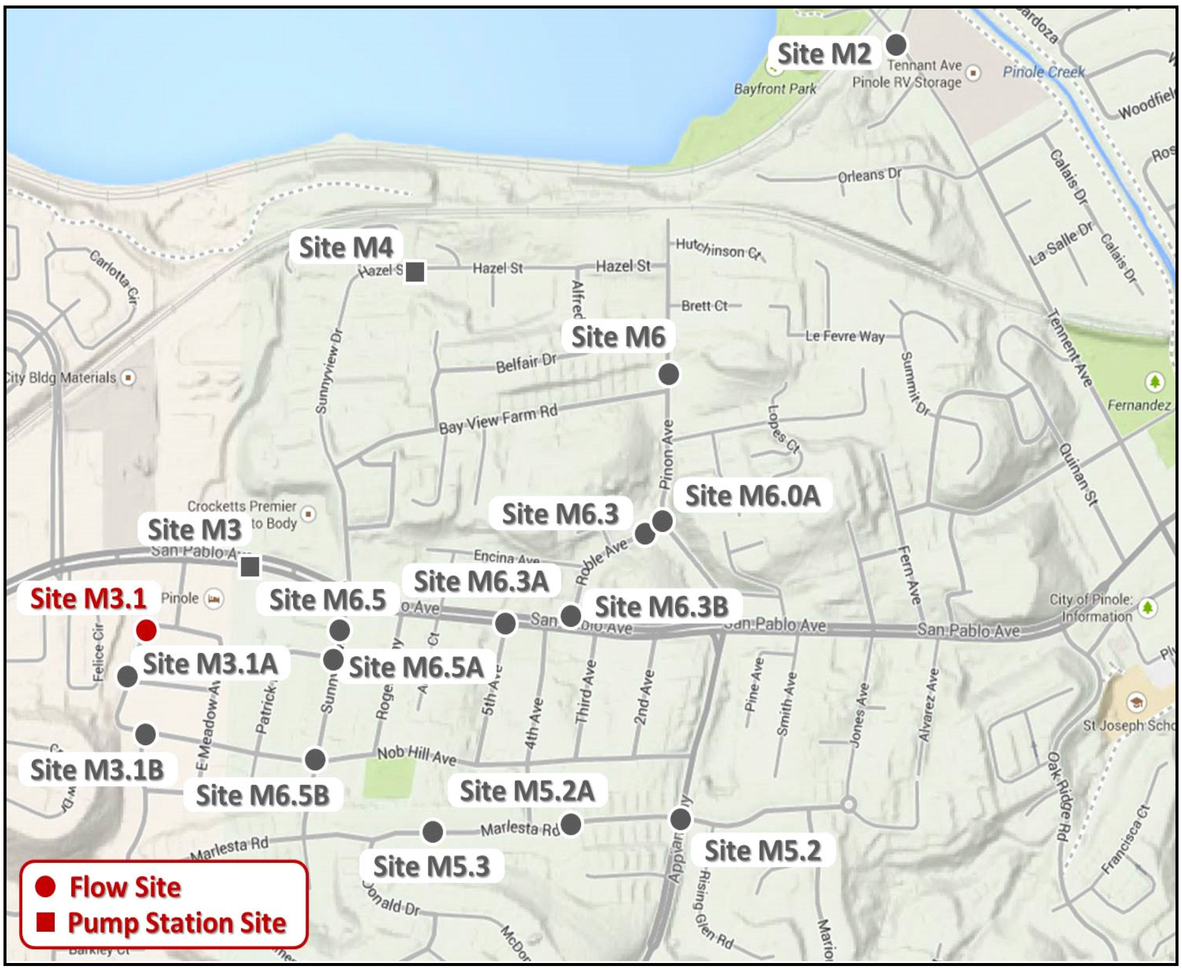
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M3.1

Location: 830 Meadows Ave.

Data Summary Report

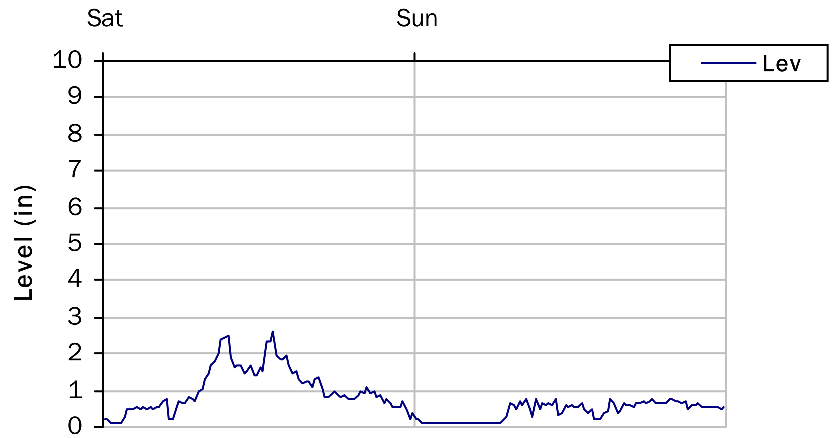


Vicinity Map: Site M3.1

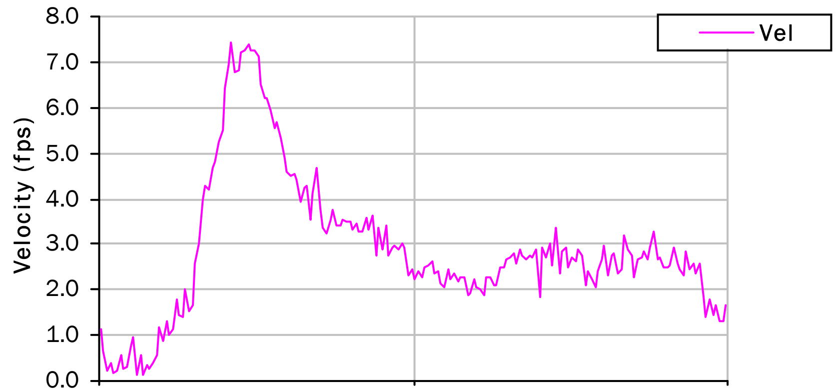
SITE M3.1

Weekly Level, Velocity and Flow Hydrographs

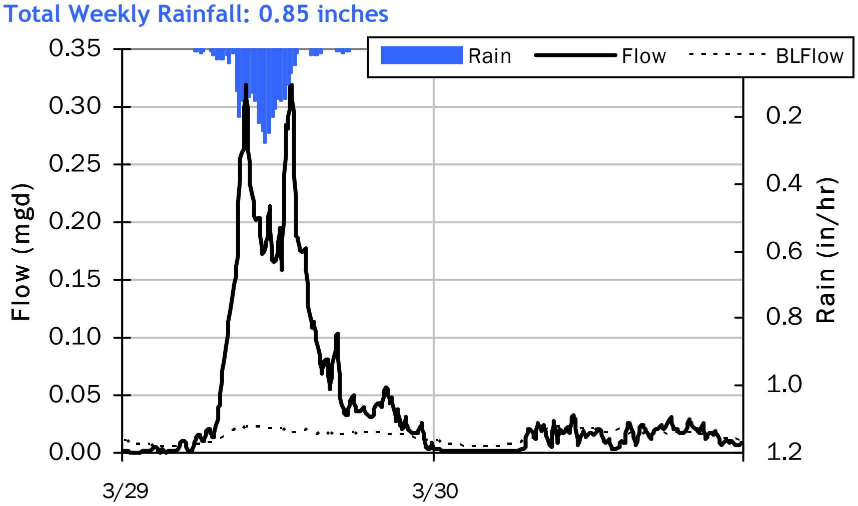
Avg Level: 0.74 in. Peak Level: 2.61 in. Min Level: 0.11 in.



Avg Velocity: 2.90 fps Peak Velocity: 7.44 fps Min Velocity: 0.13 fps



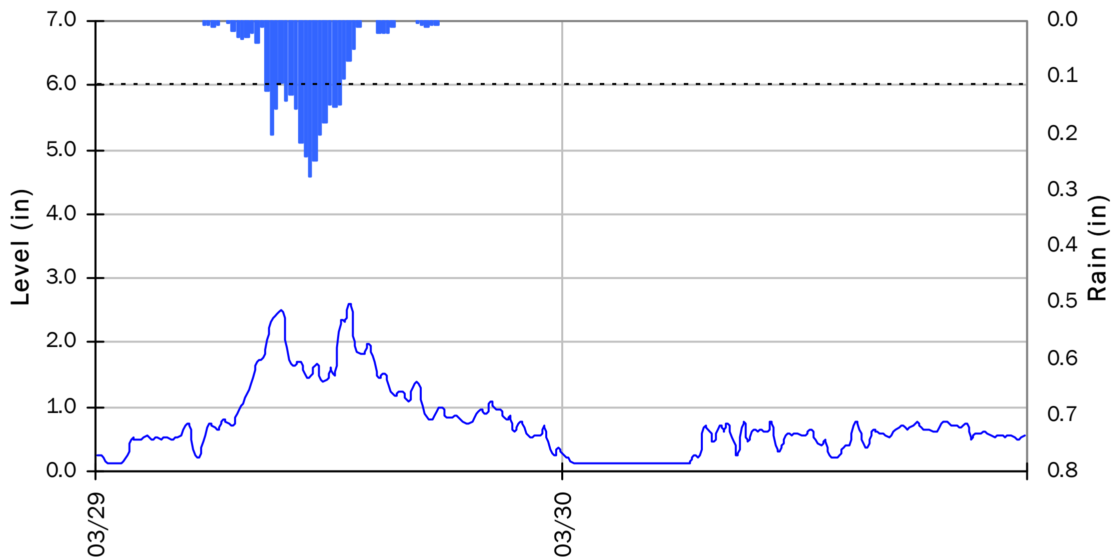
Total Weekly Rainfall: 0.85 inches

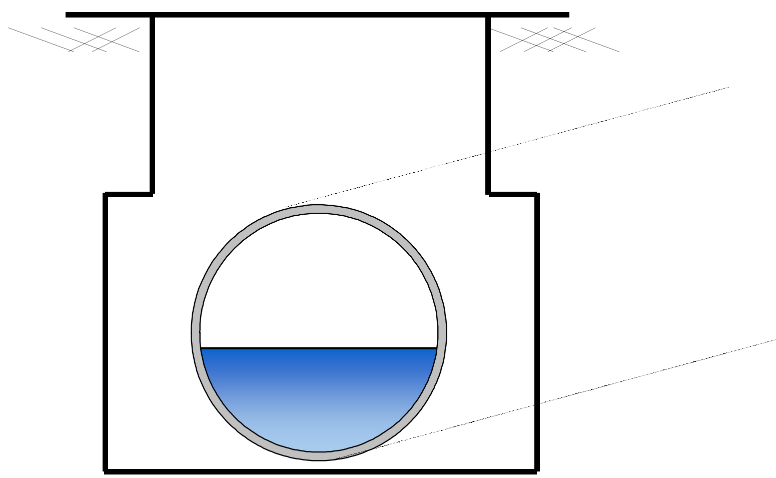


Avg Flow: 0.044 mgd Peak Flow: 0.319 mgd Min Flow: 0.000 mgd

SITE M3.1
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



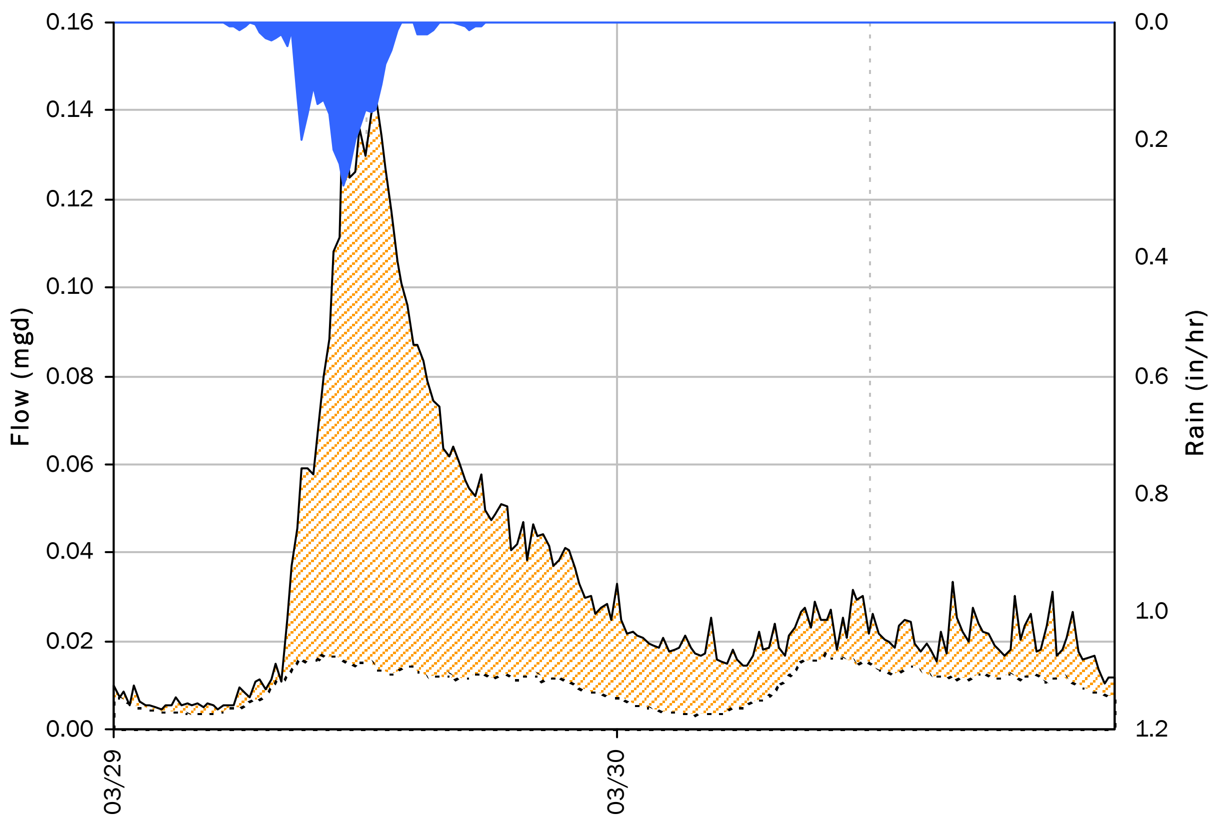


Pipe Diameter:	6	*inches*
Peak Measured Level:	2.61	*inches*
Peak d/D Ratio:	0.44	

SITE 3.1A
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.15 *mgd*
PF: 15.37

Peak Level: 5.85 *in*
d/D Ratio: 0.98

Inflow / Infiltration

Peak I/I Rate: 0.13 *mgd*
Total I/I: 48,000 *gallons*

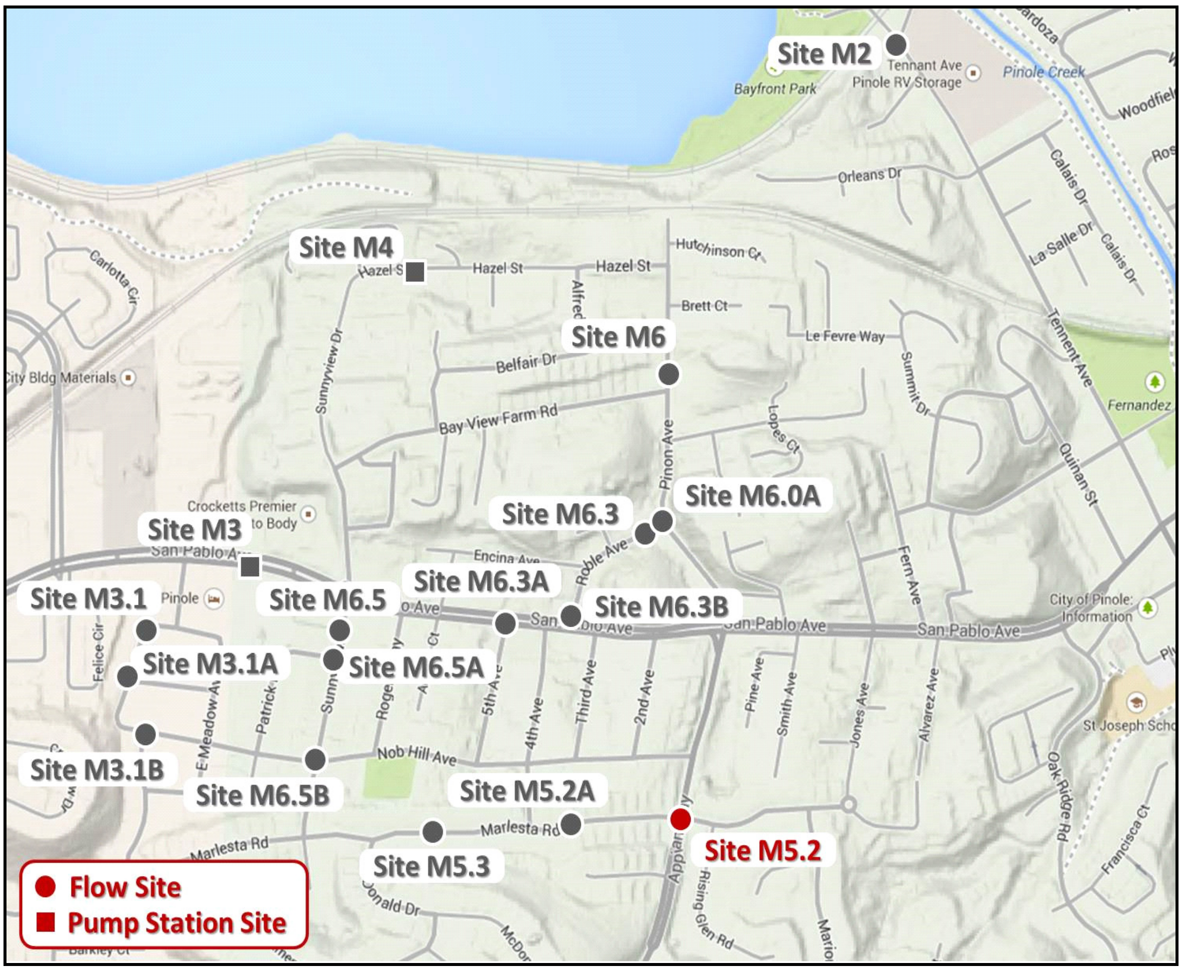
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M5.2

Location: Intersection of Appian Way and Marlesta Rd.

Data Summary Report

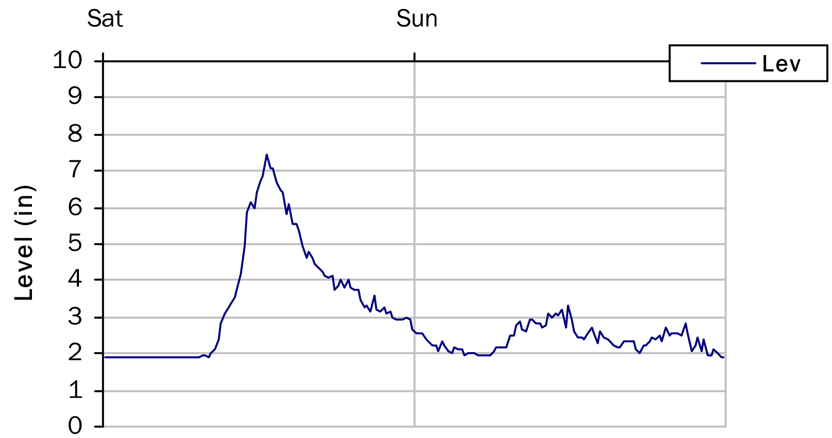


Vicinity Map: Site M5.2

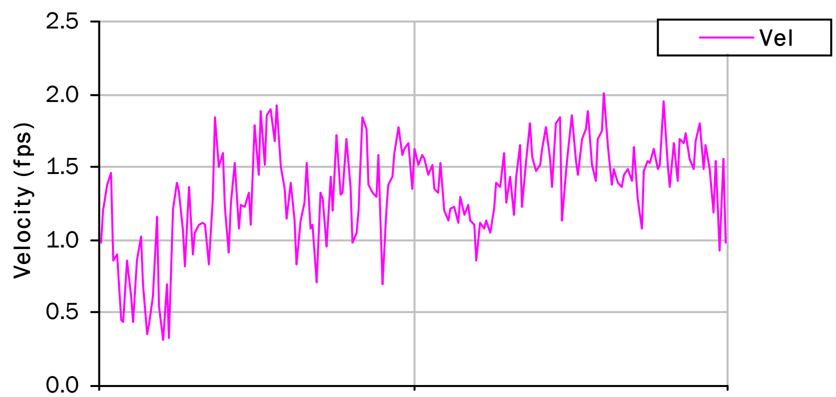
SITE M5.2

Weekly Level, Velocity and Flow Hydrographs

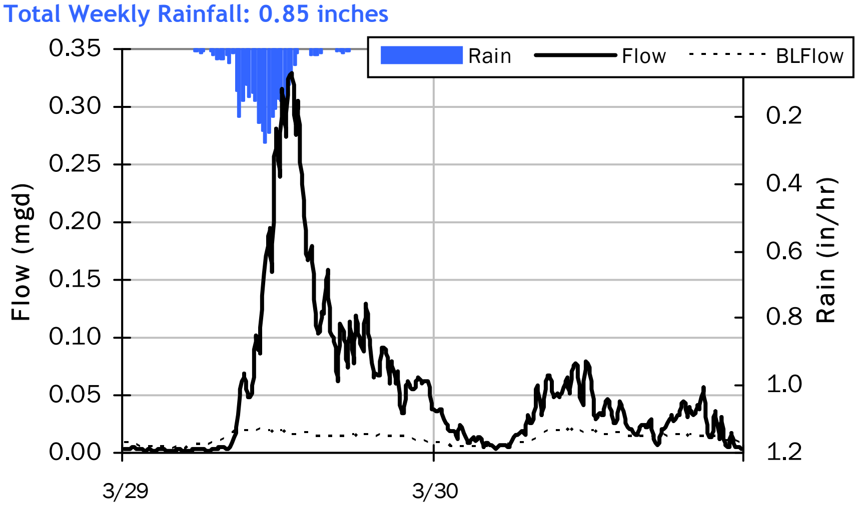
Avg Level: 2.94 in. **Peak Level: 7.47 in.** **Min Level: 1.90 in.**



Avg Velocity: 1.33 fps **Peak Velocity: 2.01 fps** **Min Velocity: 0.32 fps**



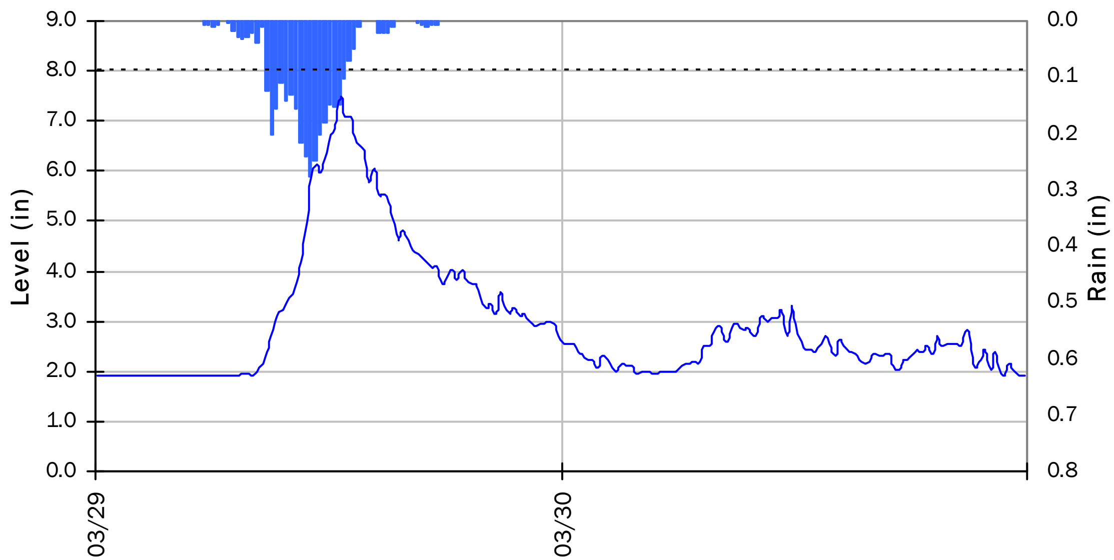
Total Weekly Rainfall: 0.85 inches

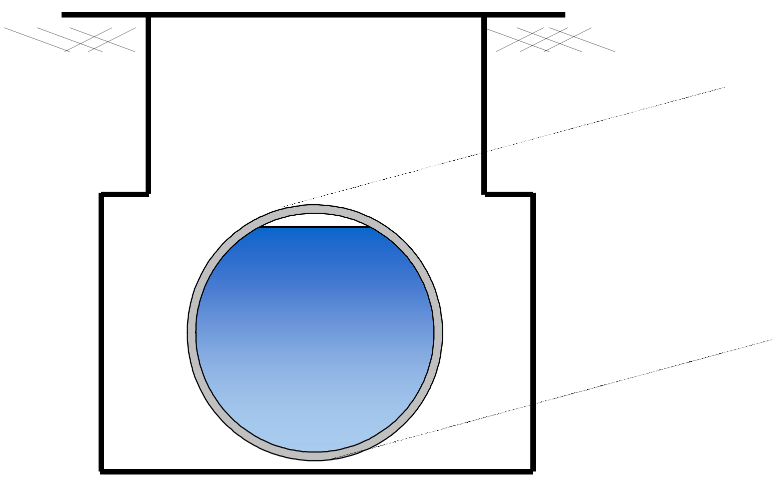


Avg Flow: 0.054 mgd **Peak Flow: 0.329 mgd** **Min Flow: 0.001 mgd**

SITE M5.2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



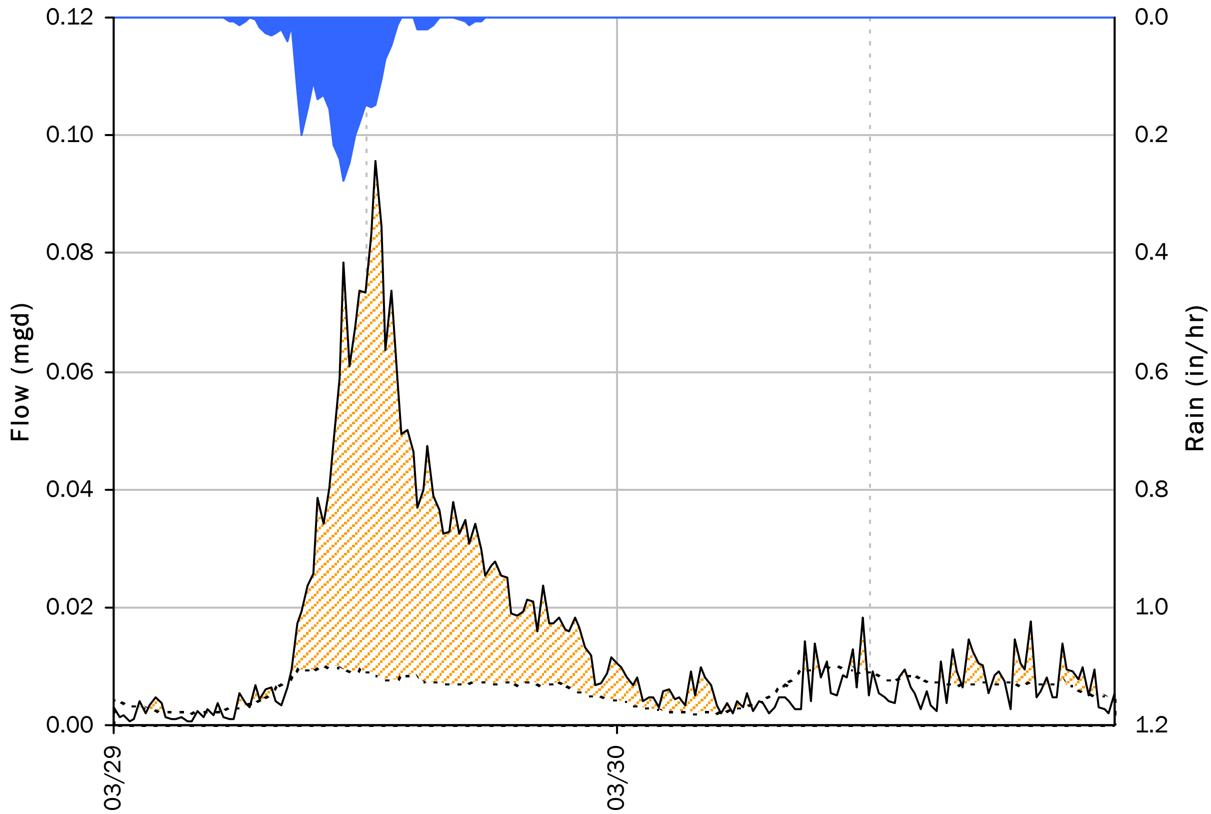


Pipe Diameter:	8	*inches*
Peak Measured Level:	7.47	*inches*
Peak d/D Ratio:	0.93	

SITE 3.1B
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.10 *mgd*
PF: 16.66

Peak Level: 2.03 *in*
d/D Ratio: 0.34

Inflow / Infiltration

Peak I/I Rate: 0.09 *mgd*
Total I/I: 18,000 *gallons*

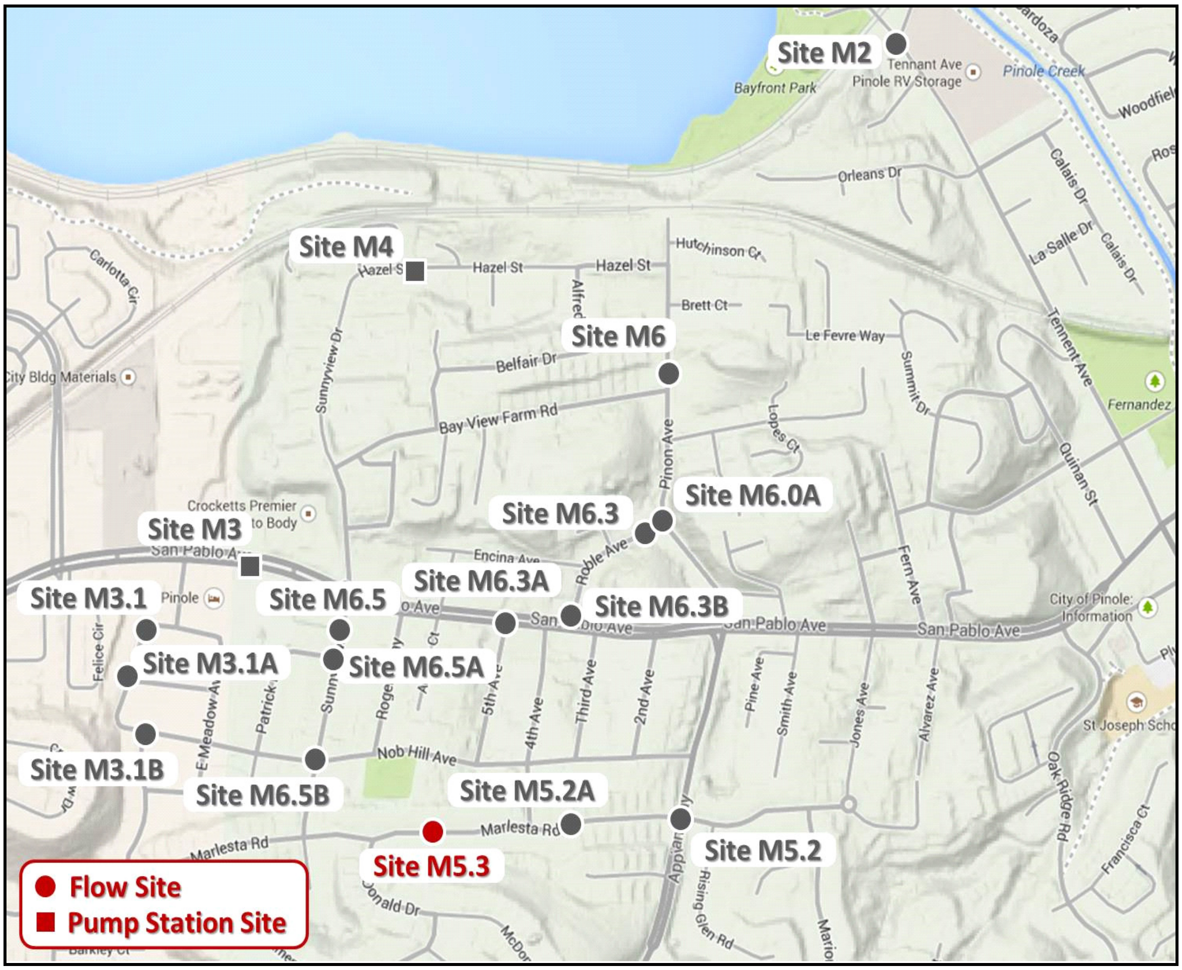
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M5.3

Location: 1171 Marlesta Rd.

Data Summary Report

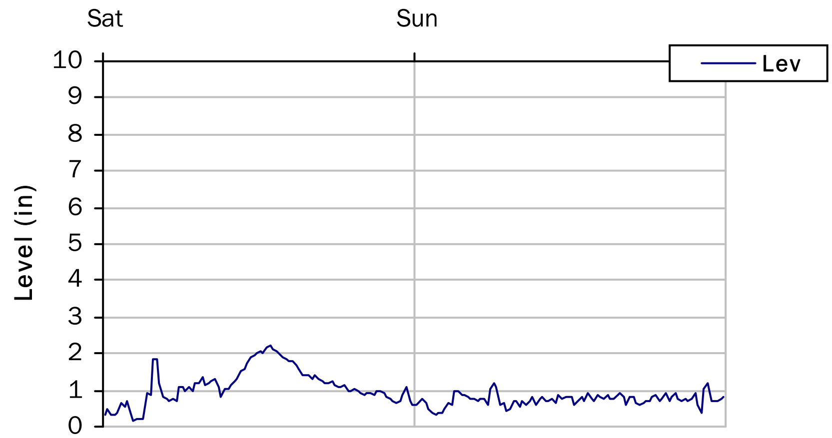


Vicinity Map: Site M5.3

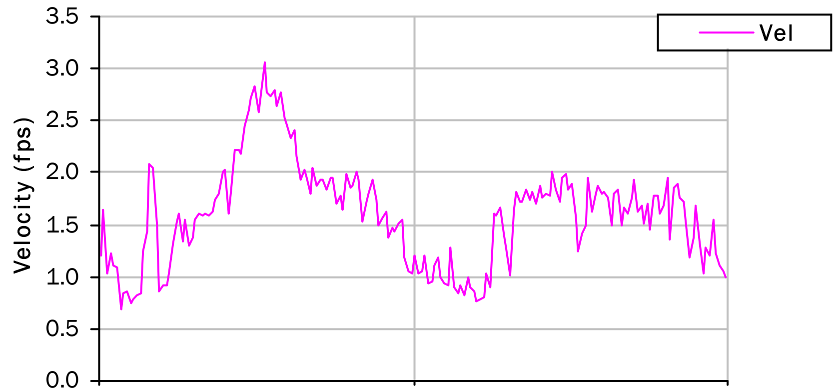
SITE M5.3

Weekly Level, Velocity and Flow Hydrographs

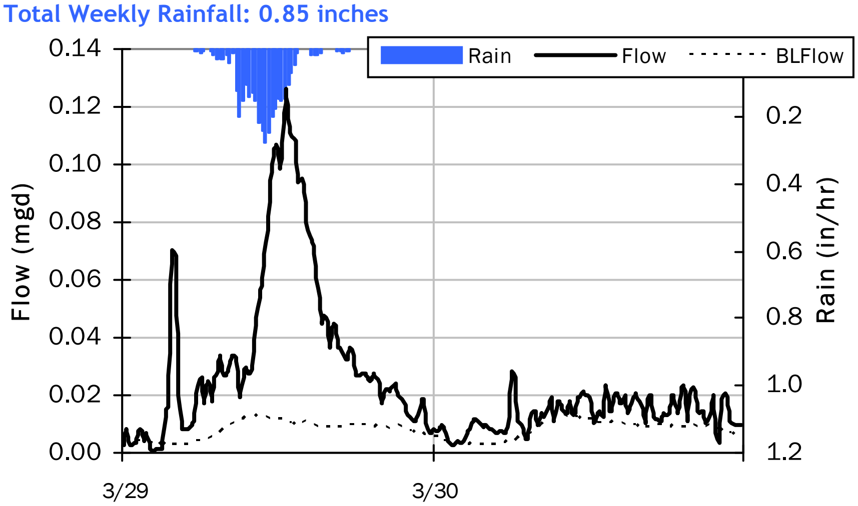
Avg Level: 0.93 in. **Peak Level: 2.21 in.** **Min Level: 0.18 in.**



Avg Velocity: 1.60 fps **Peak Velocity: 3.06 fps** **Min Velocity: 0.69 fps**



Total Weekly Rainfall: 0.85 inches

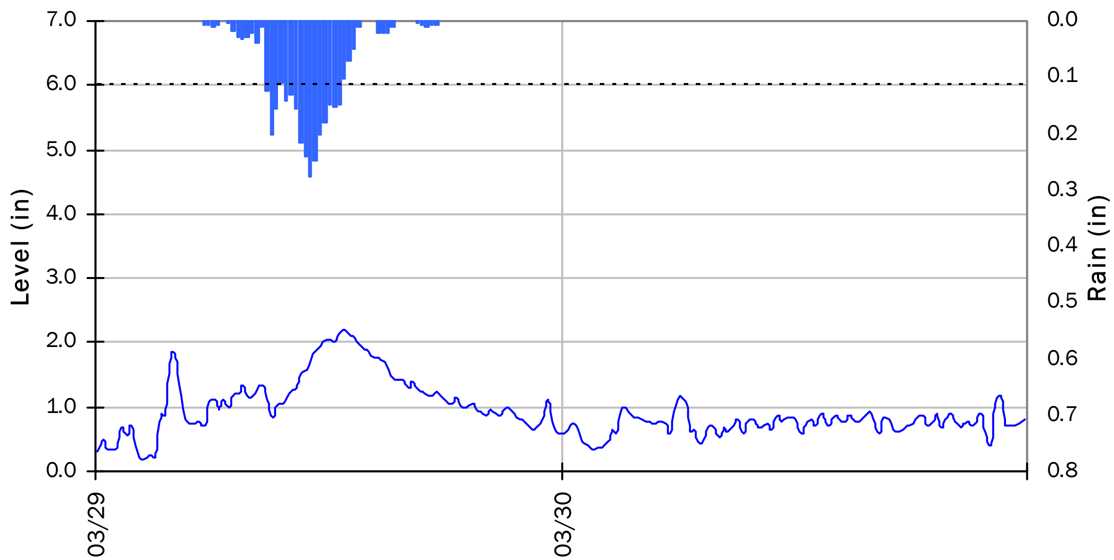


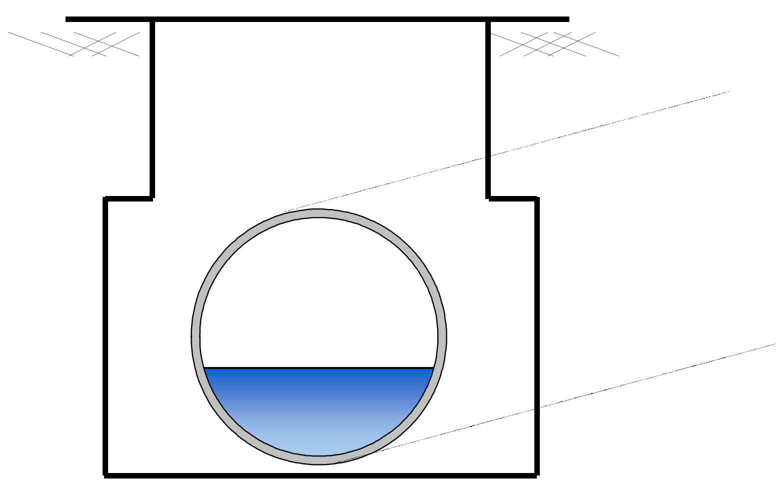
Avg Flow: 0.024 mgd **Peak Flow: 0.125 mgd** **Min Flow: 0.001 mgd**

SITE M5.3

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



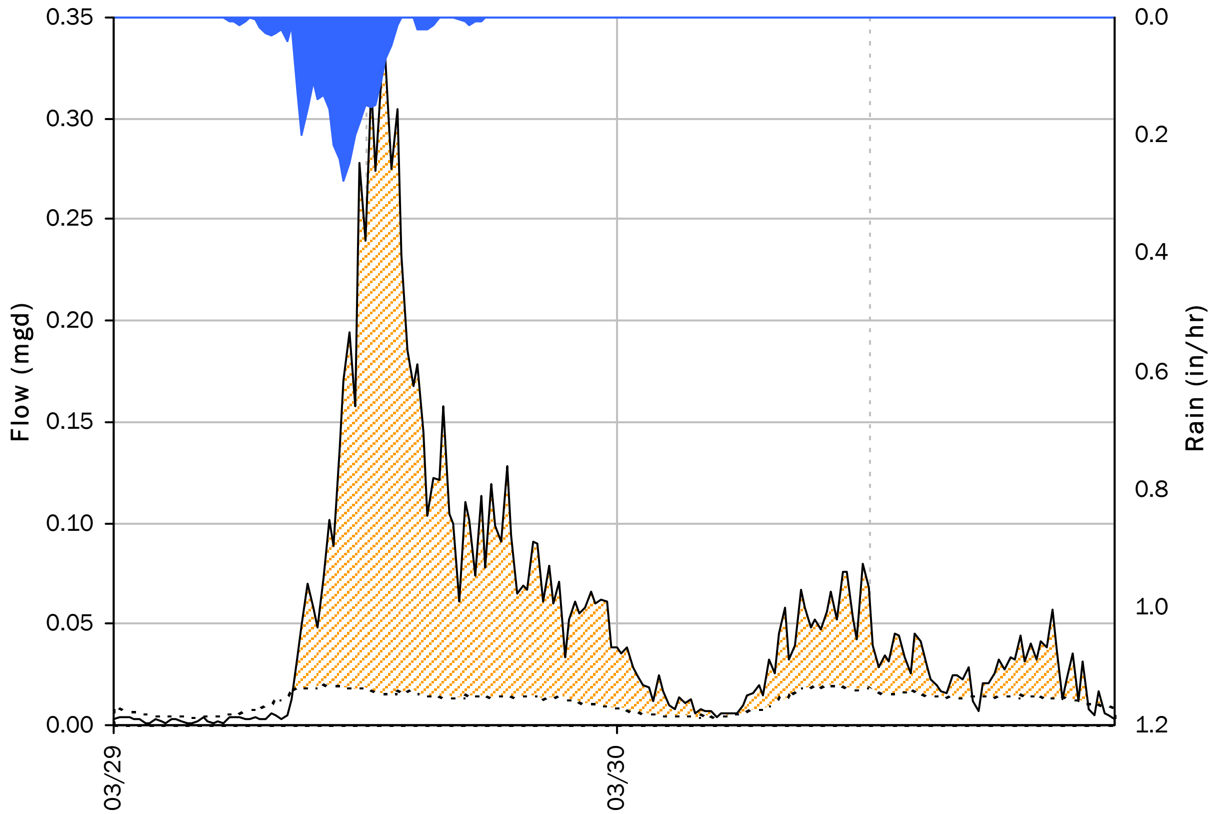


Pipe Diameter:	6	*inches*
Peak Measured Level:	2.21	*inches*
Peak d/D Ratio:	0.37	

SITE 5.2
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.33 *mgd*
PF: 28.70

Peak Level: 7.47 *in*
d/D Ratio: 0.93

Inflow / Infiltration

Peak I/I Rate: 0.31 *mgd*
Total I/I: 83,000 *gallons*

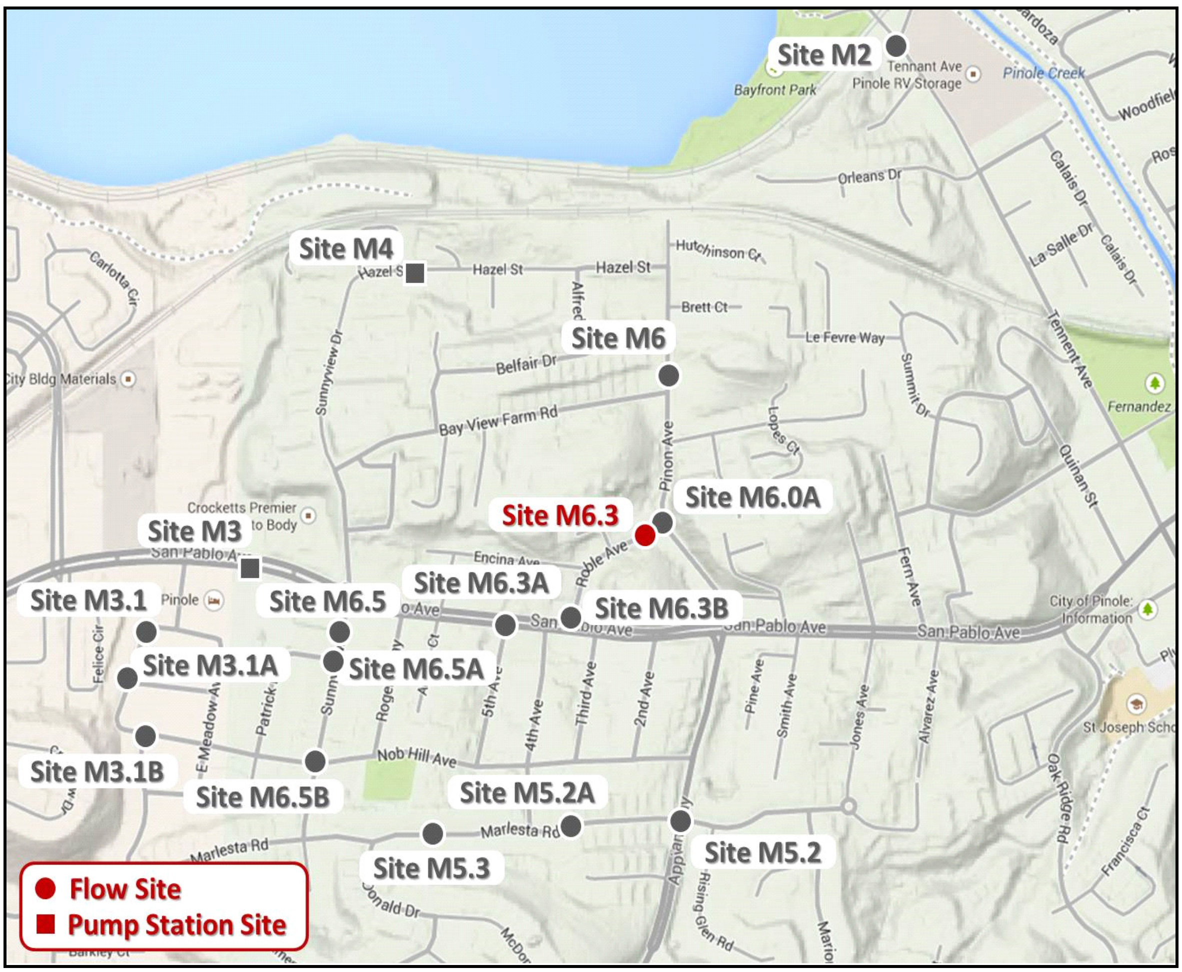
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.3

Location: Roble Ave., west of Pinon Ave.

Data Summary Report

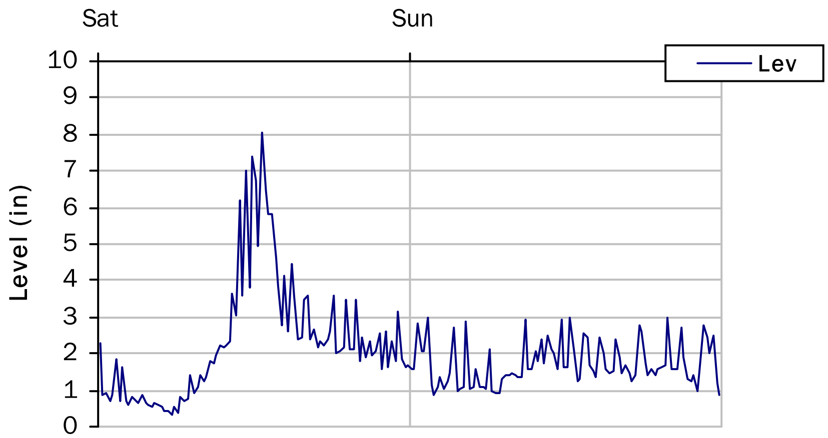


Vicinity Map: Site M6.3

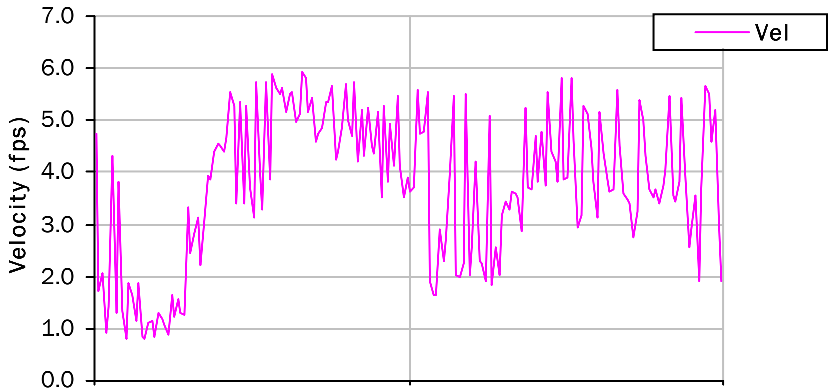
SITE M6.3

Weekly Level, Velocity and Flow Hydrographs

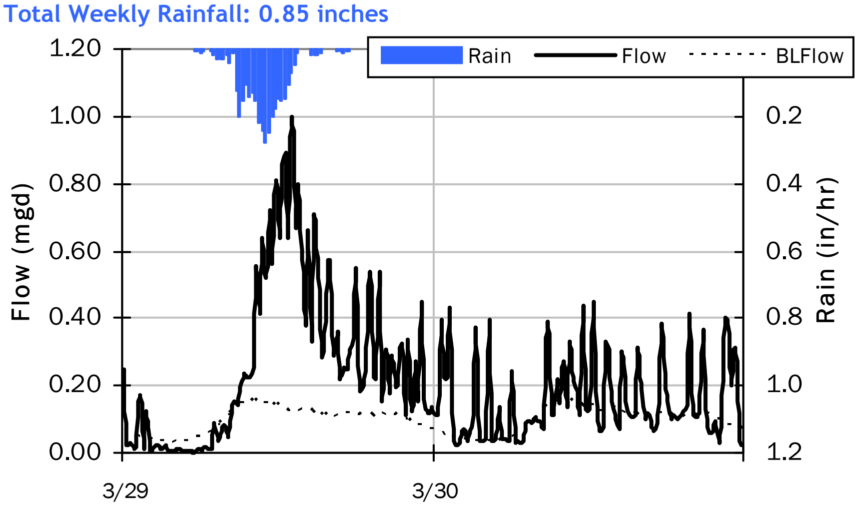
Avg Level: 2.03 in. **Peak Level: 8.07 in.** **Min Level: 0.32 in.**



Avg Velocity: 3.76 fps **Peak Velocity: 5.91 fps** **Min Velocity: 0.80 fps**



Total Weekly Rainfall: 0.85 inches

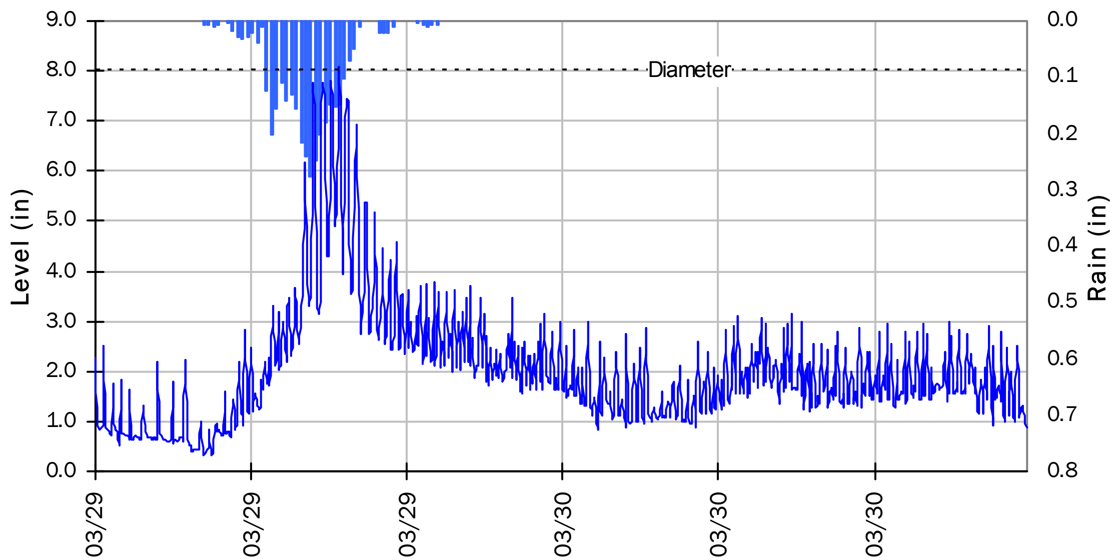


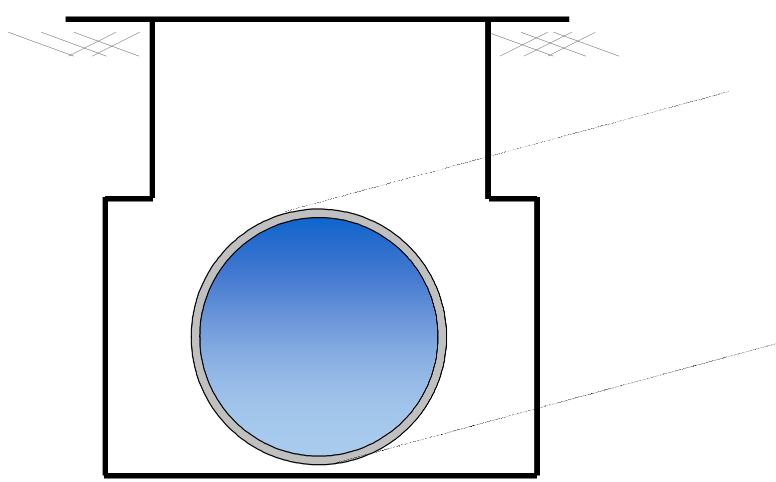
Avg Flow: 0.212 mgd **Peak Flow: 1.002 mgd** **Min Flow: 0.003 mgd**

SITE M6.3

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





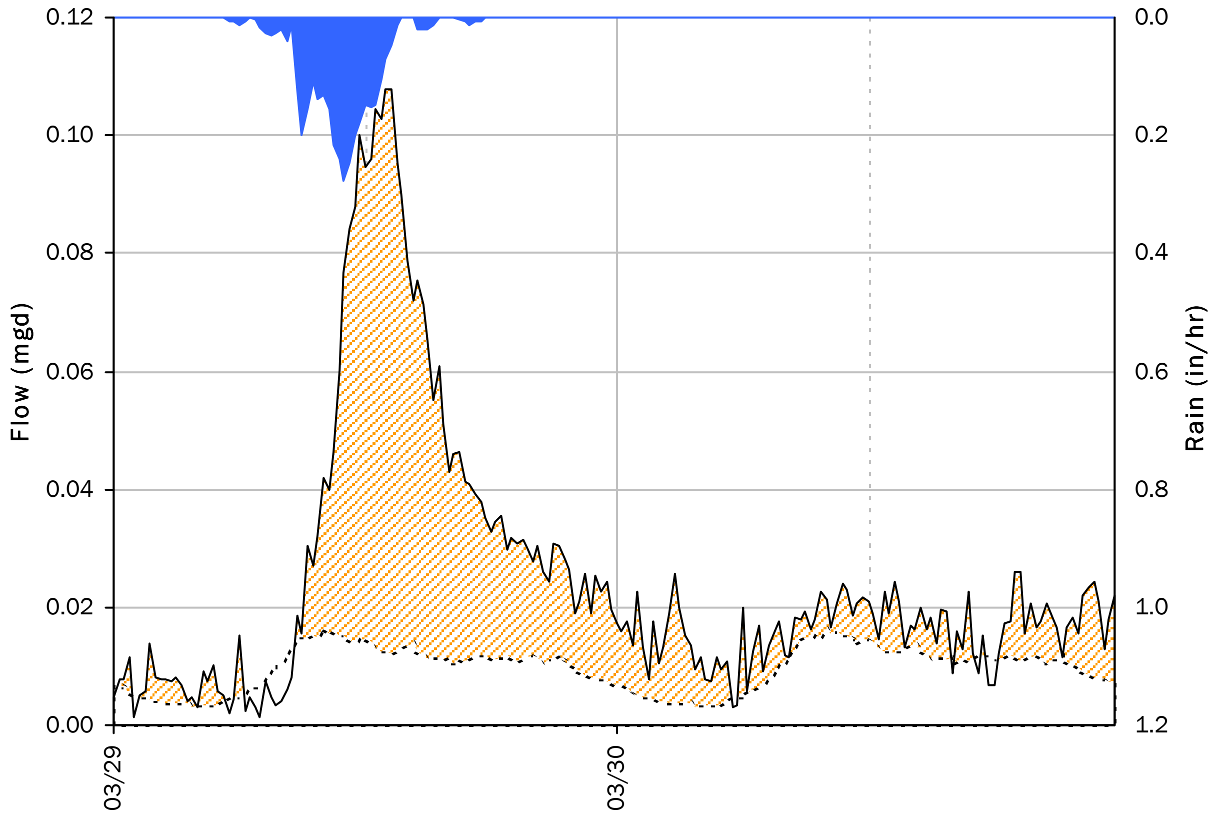
Pipe Diameter:	8	*inches*
Peak Measured Level:	8.07	*inches*
Peak d/D Ratio:	1.01	

Surcharged 0.1 inches over crown

SITE 5.2A
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.11 *mgd*
PF: 11.99

Peak Level: 3.20 *in*
d/D Ratio: 0.53

Inflow / Infiltration

Peak I/I Rate: 0.10 *mgd*
Total I/I: 30,000 *gallons*

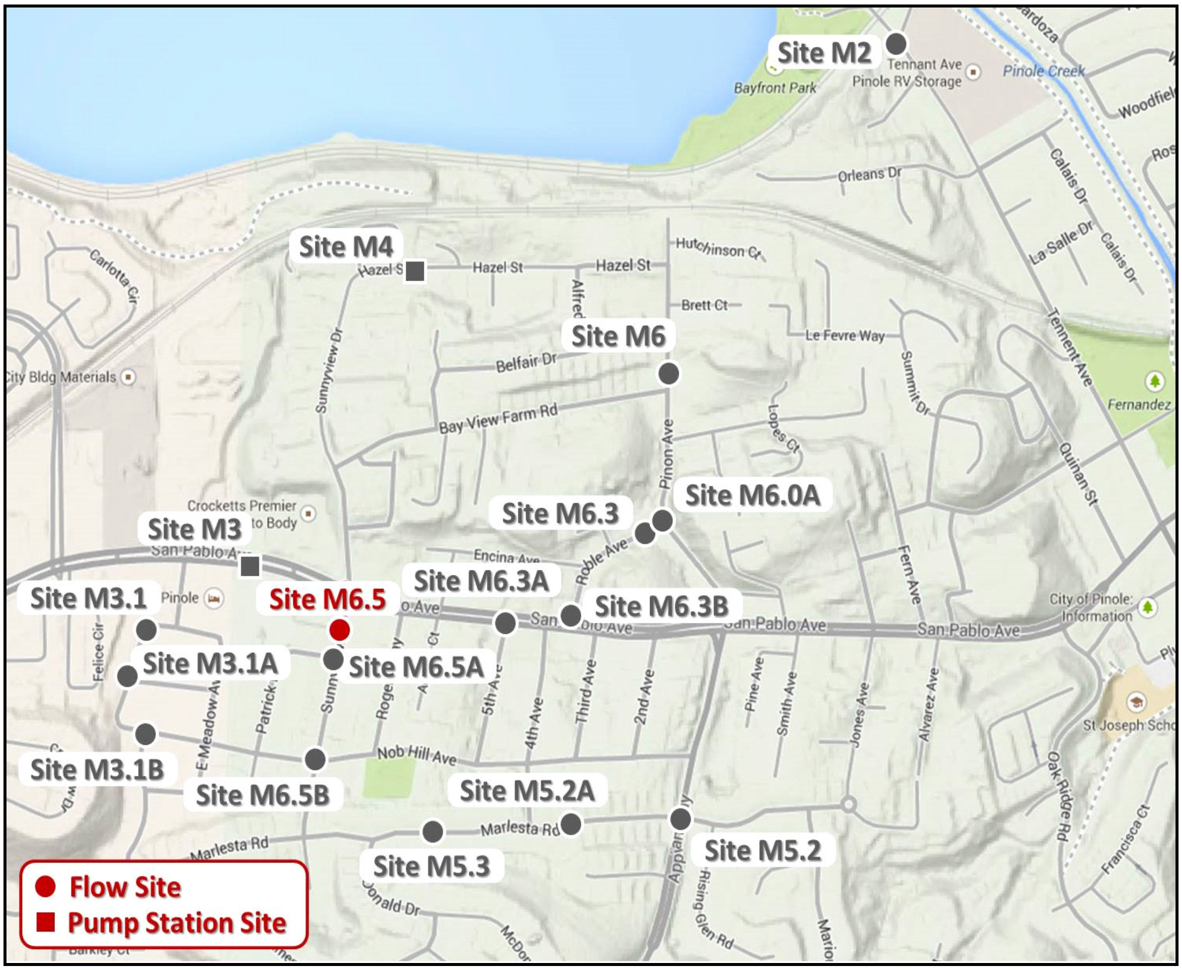
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.5

Location: 747 Sunnyview Dr.

Data Summary Report

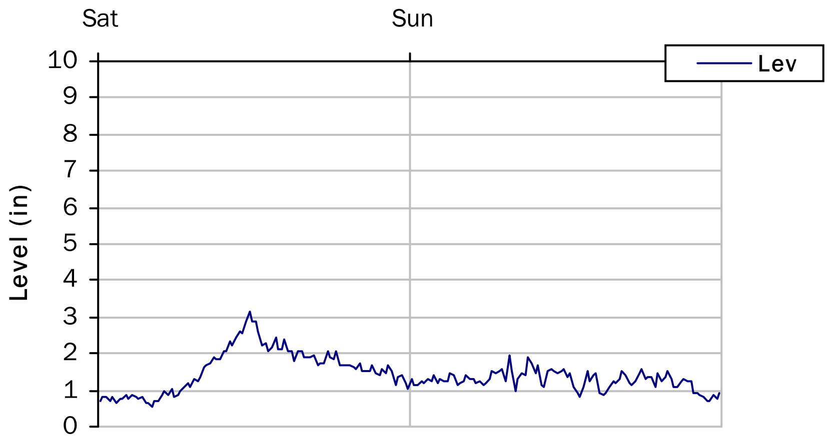


Vicinity Map: Site M6.5

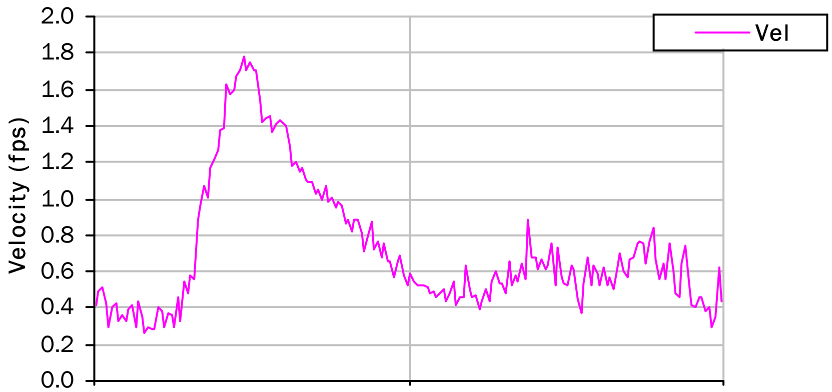
SITE M6.5

Weekly Level, Velocity and Flow Hydrographs

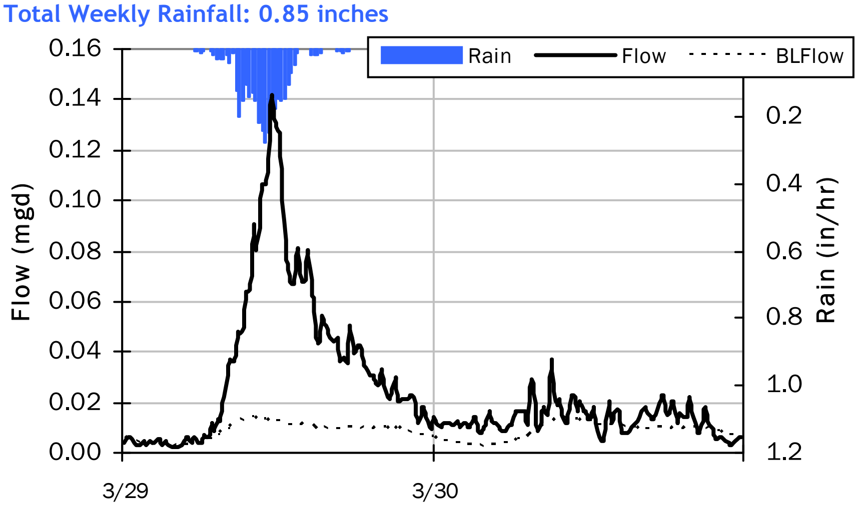
Avg Level: 1.42 in. Peak Level: 3.15 in. Min Level: 0.54 in.



Avg Velocity: 0.72 fps Peak Velocity: 1.78 fps Min Velocity: 0.26 fps



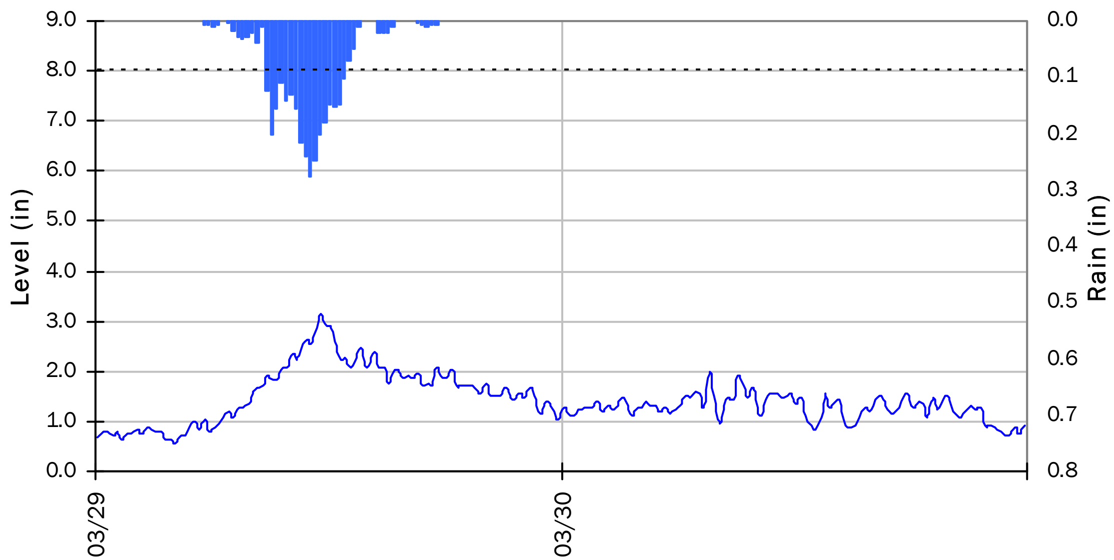
Total Weekly Rainfall: 0.85 inches

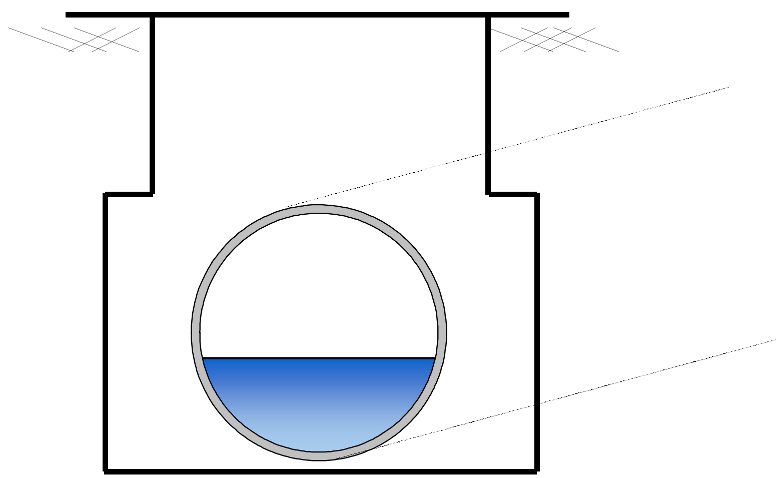


Avg Flow: 0.025 mgd Peak Flow: 0.141 mgd Min Flow: 0.002 mgd

SITE M6.5
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



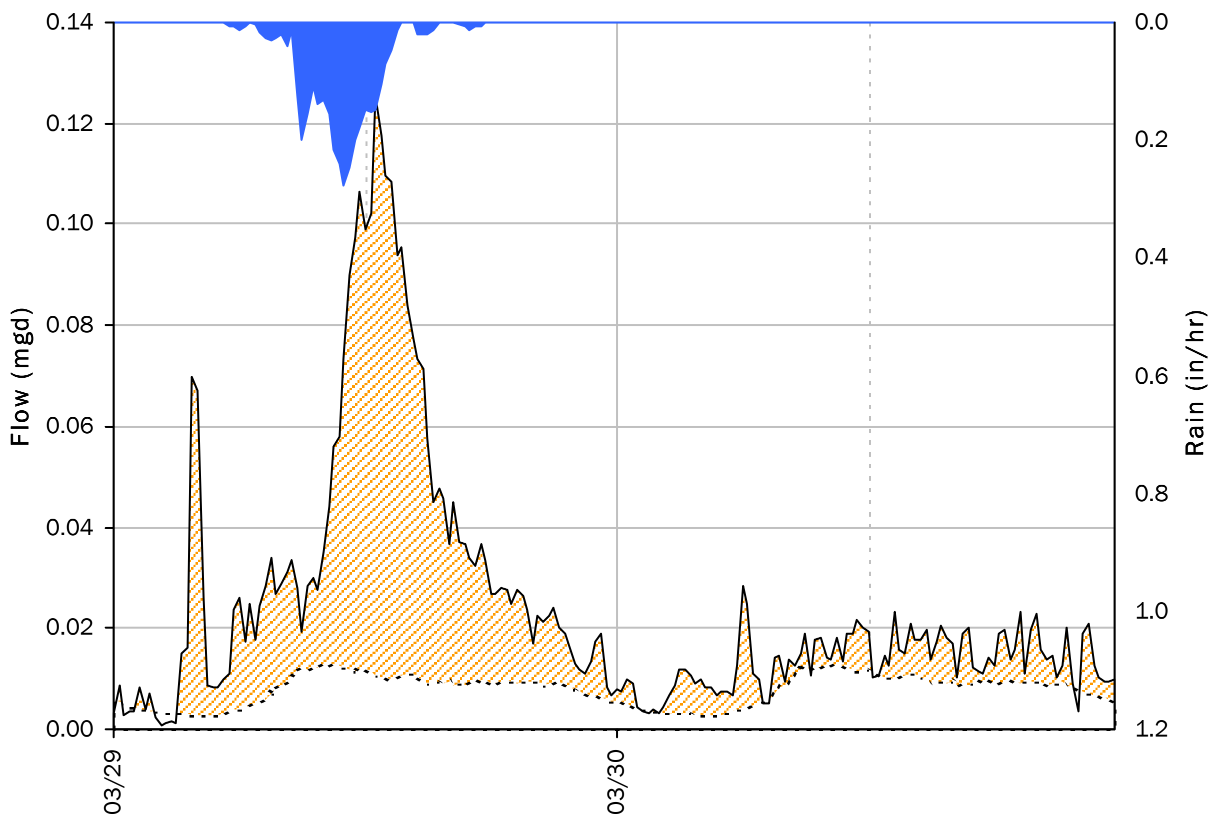


Pipe Diameter:	8	*inches*
Peak Measured Level:	3.15	*inches*
Peak d/D Ratio:	0.39	

SITE 5.3
I/I Summary: Event 3

<u>Baseline and Realtime Flows with Rainfall Data over Monitoring Period</u>

<u>Event 3 Detail Graph</u>



Storm Event I/I Analysis (Rain = 0.85 inches)

<u>Capacity</u>

Peak Flow: 0.13 *mgd*
PF: 17.01

Peak Level: 2.21 *in*
d/D Ratio: 0.37

<u>Inflow / Infiltration</u>

Peak I/I Rate: 0.11 *mgd*
Total I/I: 33,000 *gallons*

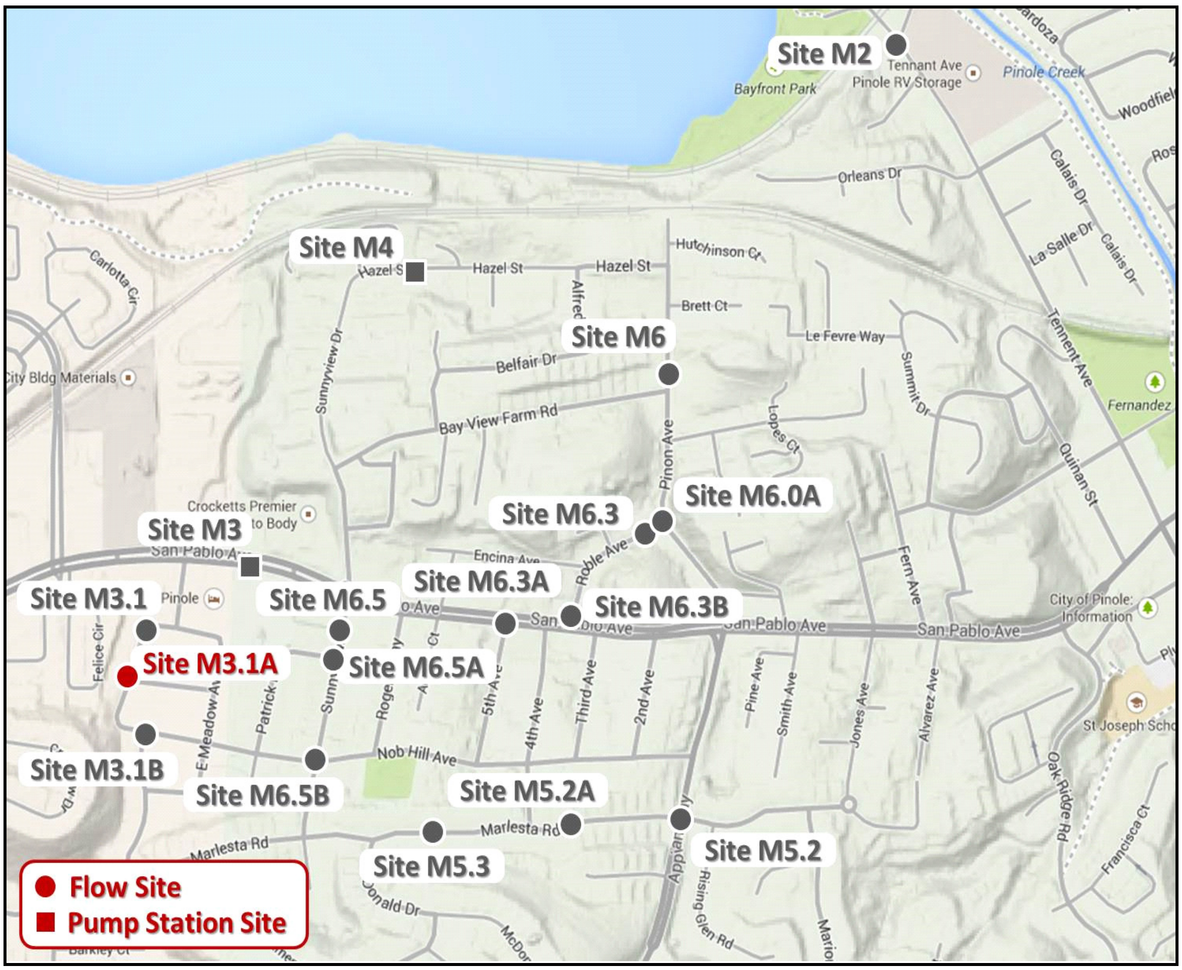
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M3.1A

Location: Intersection of Meadow Ave. and Betty Ave.

Data Summary Report

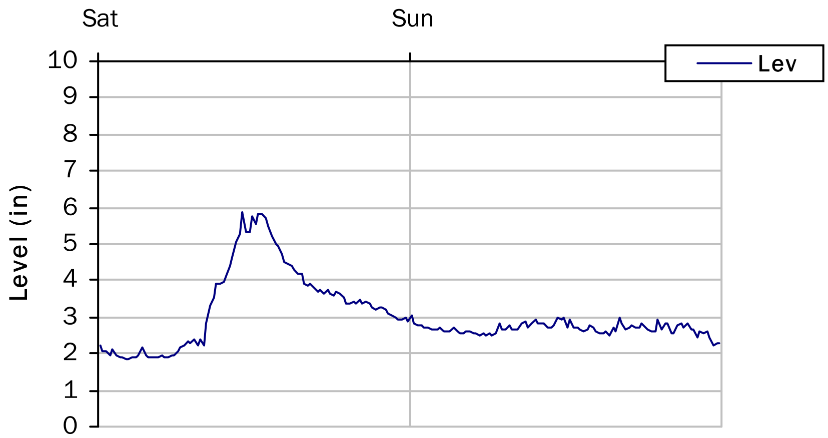


Vicinity Map: Site M3.1A

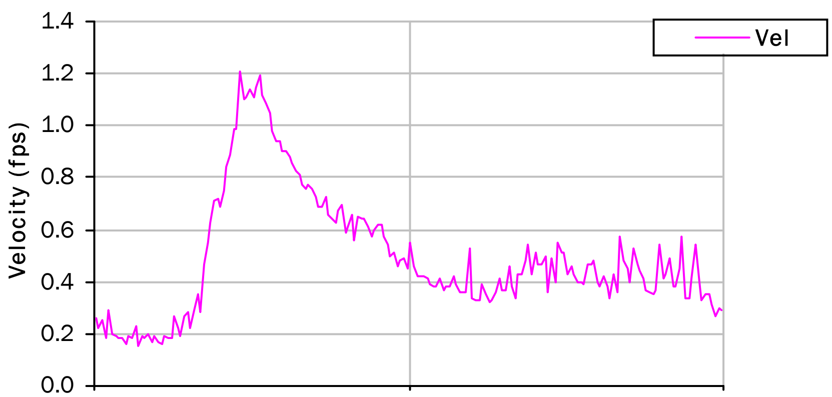
SITE M3.1A

Weekly Level, Velocity and Flow Hydrographs

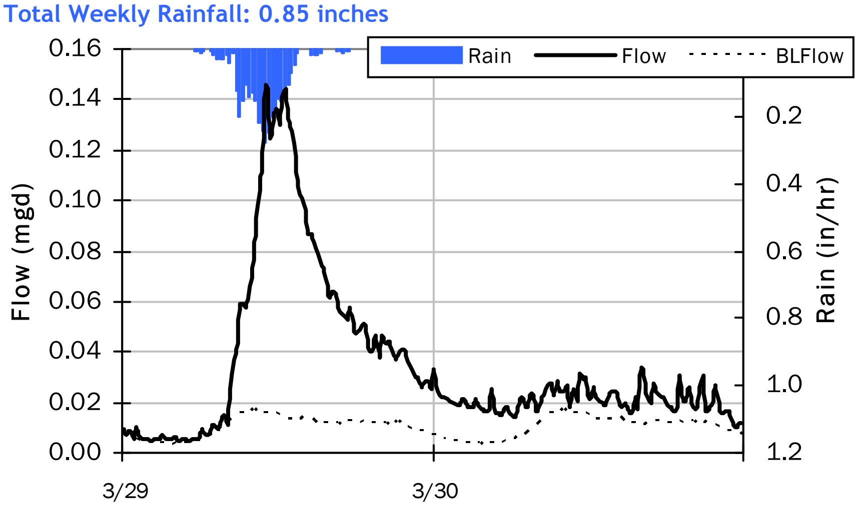
Avg Level: 3.02 in. Peak Level: 5.85 in. Min Level: 1.85 in.



Avg Velocity: 0.49 fps Peak Velocity: 1.21 fps Min Velocity: 0.15 fps



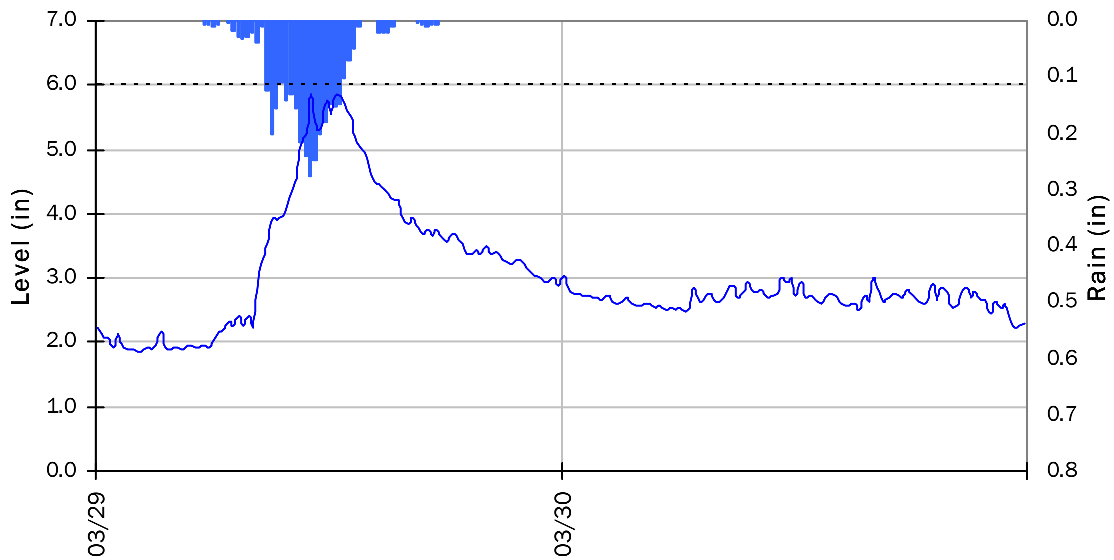
Total Weekly Rainfall: 0.85 inches

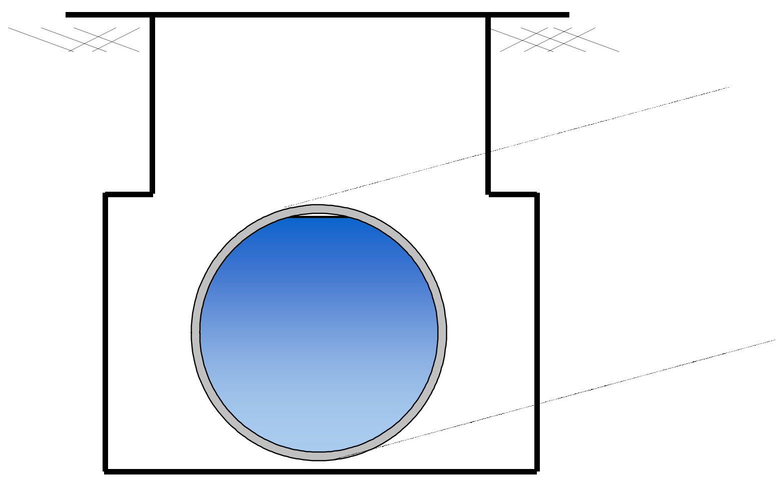


Avg Flow: 0.034 mgd Peak Flow: 0.146 mgd Min Flow: 0.005 mgd

SITE M3.1A
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



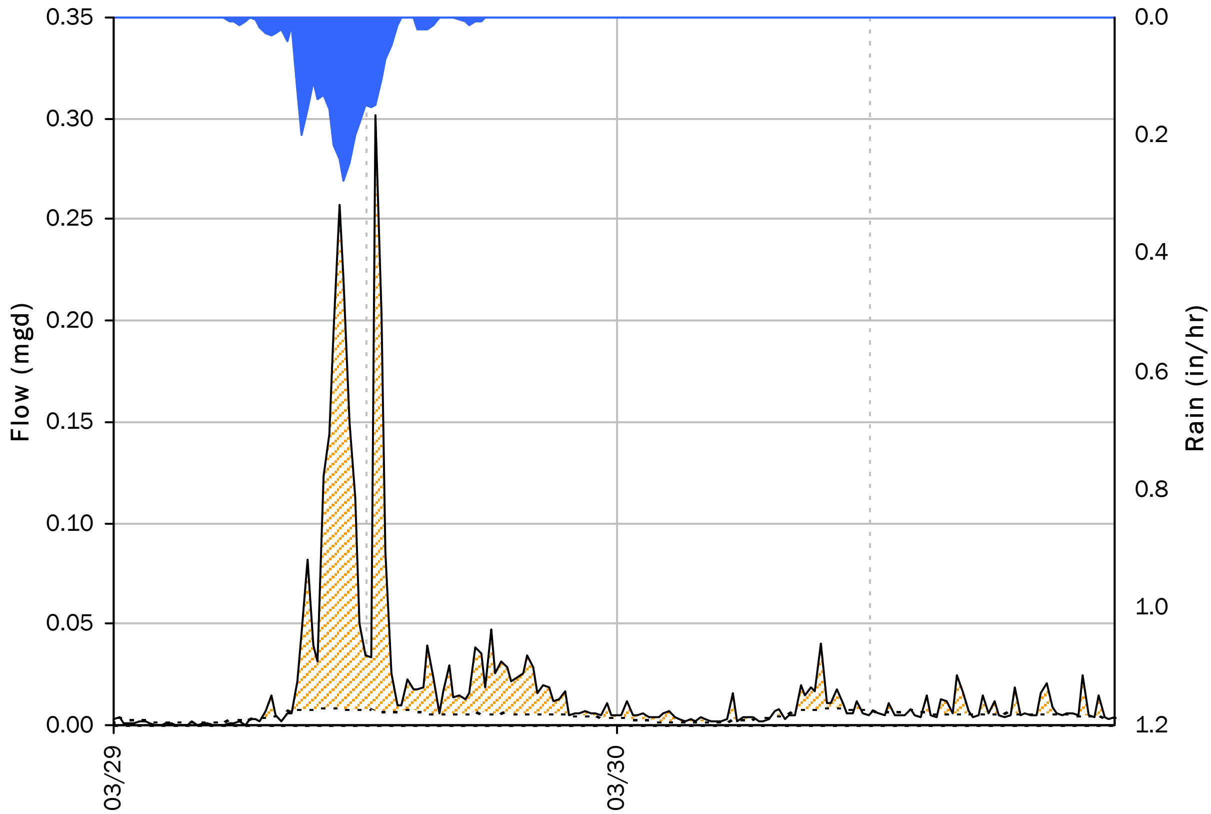


Pipe Diameter:	6	*inches*
Peak Measured Level:	5.85	*inches*
Peak d/D Ratio:	0.98	

SITE 6.3A
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.30 *mgd*
PF: 61.48

Peak Level: 3.15 *in*
d/D Ratio: 0.53

Inflow / Infiltration

Peak I/I Rate: 0.30 *mgd*
Total I/I: 29,000 *gallons*

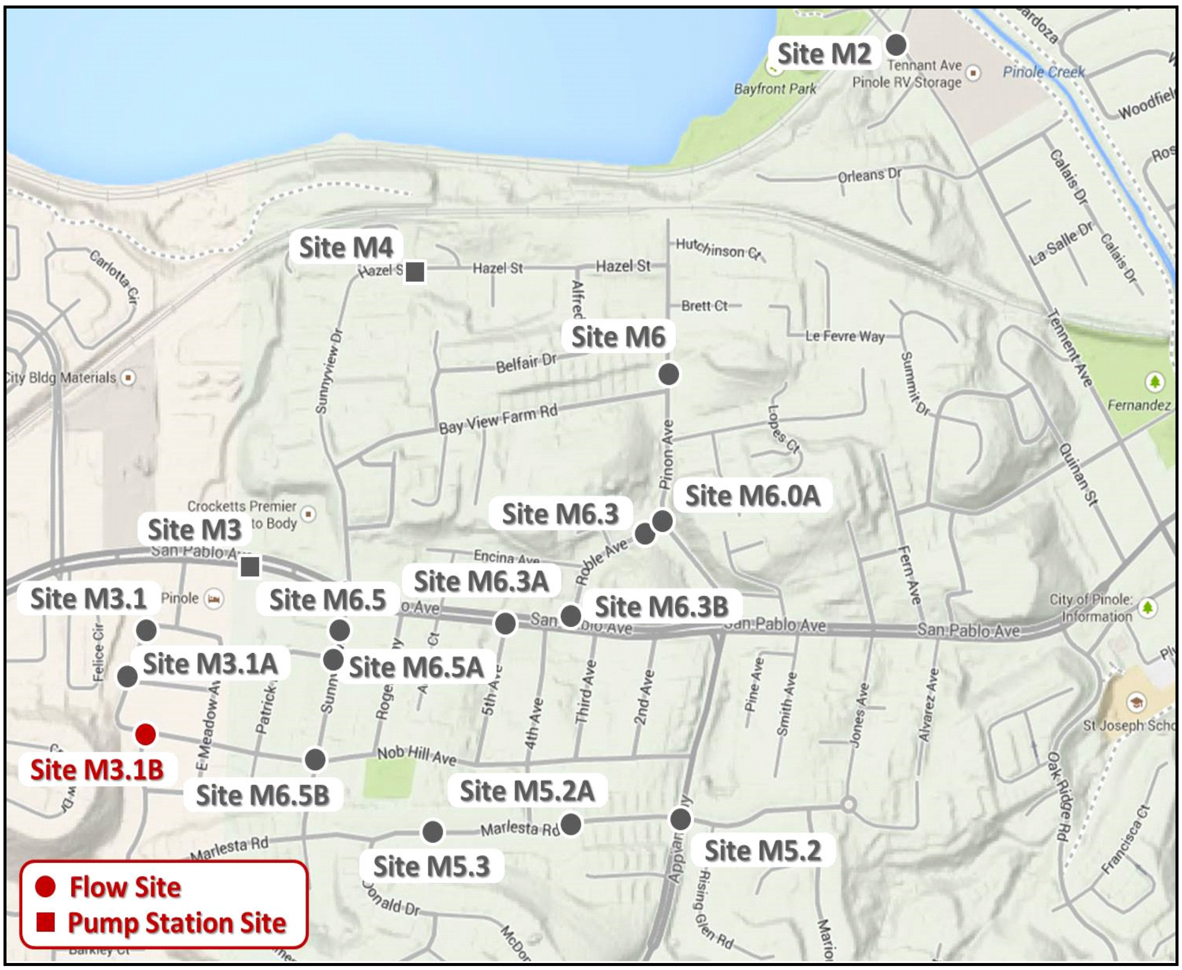
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M3.1B

Location: Intersection of Meadow Ave. and Nob Hill Ave.

Data Summary Report

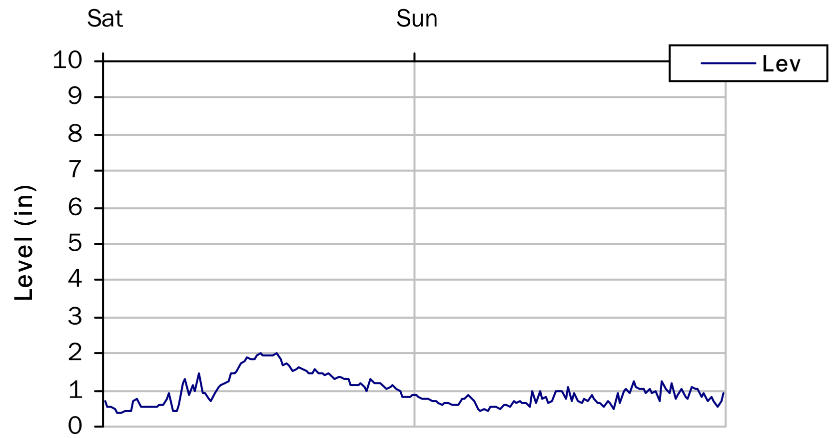


Vicinity Map: Site M3.1B

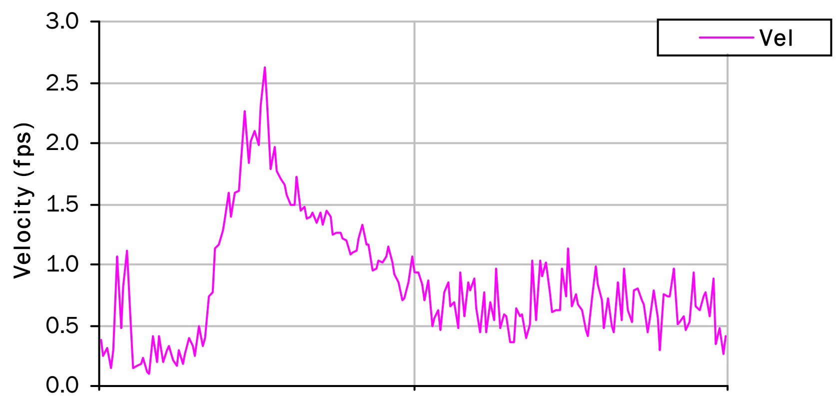
SITE M3.1B

Weekly Level, Velocity and Flow Hydrographs

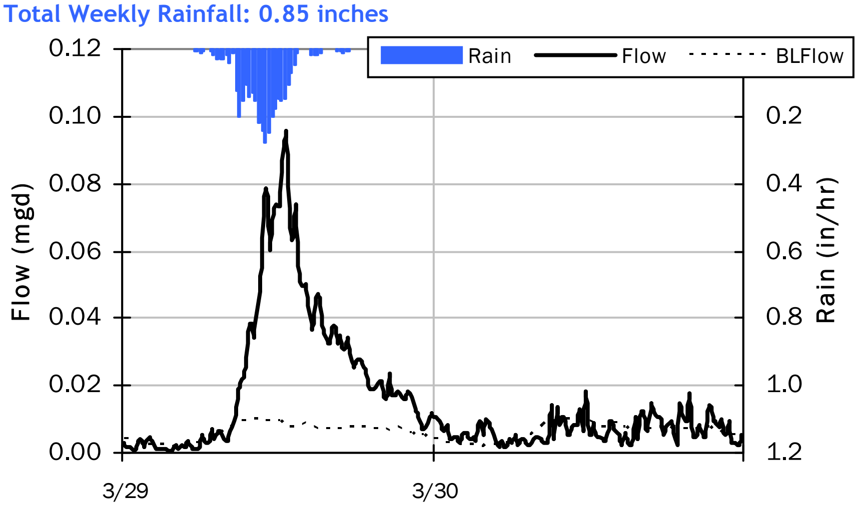
Avg Level: 0.97 in.   Peak Level: 2.03 in.   Min Level: 0.37 in.



Avg Velocity: 0.85 fps   Peak Velocity: 2.62 fps   Min Velocity: 0.10 fps



Total Weekly Rainfall: 0.85 inches

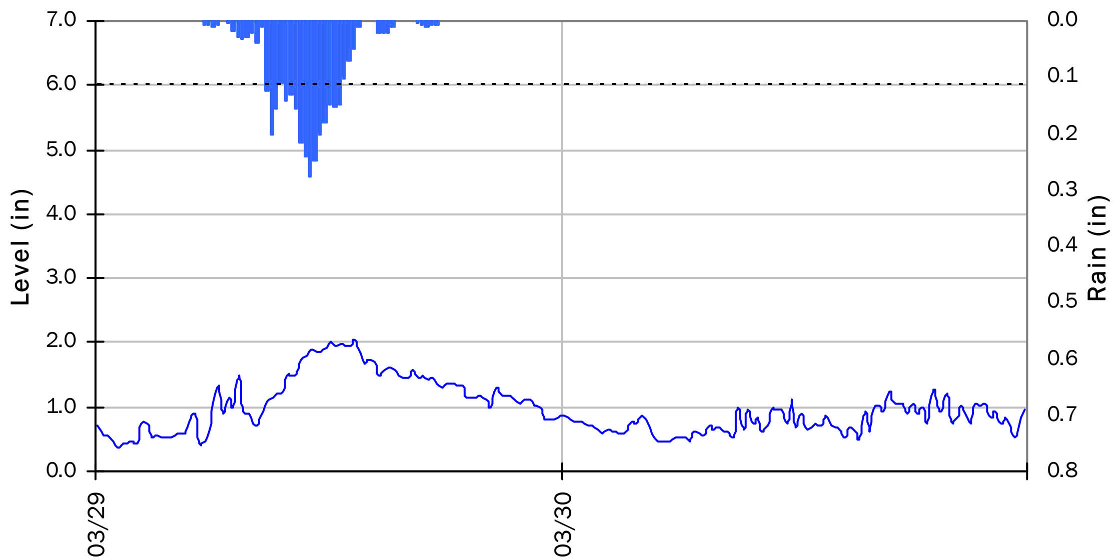


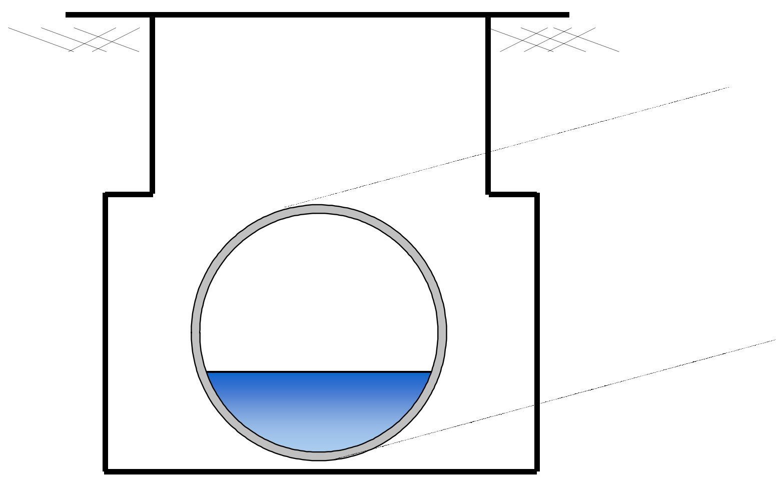
Avg Flow: 0.015 mgd   Peak Flow: 0.095 mgd   Min Flow: 0.001 mgd

SITE M3.1B

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



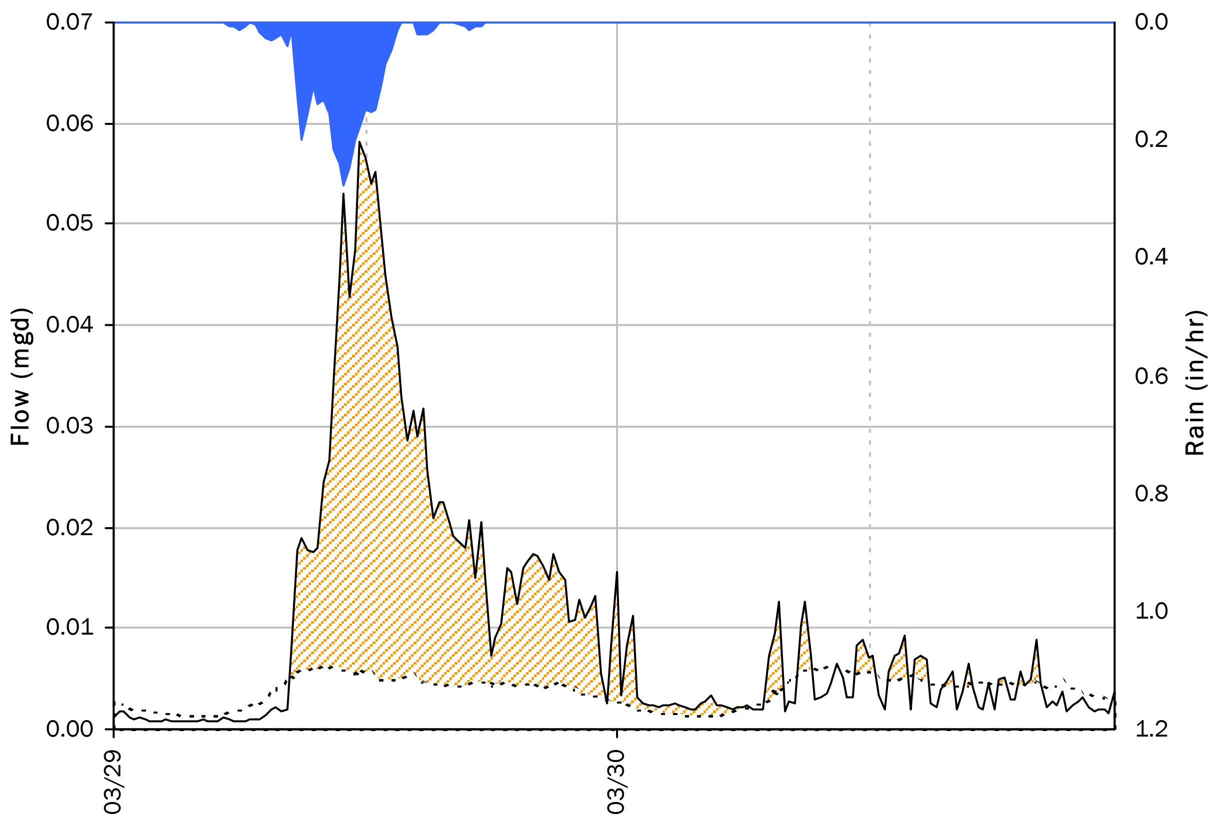


Pipe Diameter:	6	*inches*
Peak Measured Level:	2.03	*inches*
Peak d/D Ratio:	0.34	

SITE 6.3B
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.06 *mgd*
PF: 16.12

Peak Level: 1.26 *in*
d/D Ratio: 0.16

Inflow / Infiltration

Peak I/I Rate: 0.05 *mgd*
Total I/I: 12,000 *gallons*

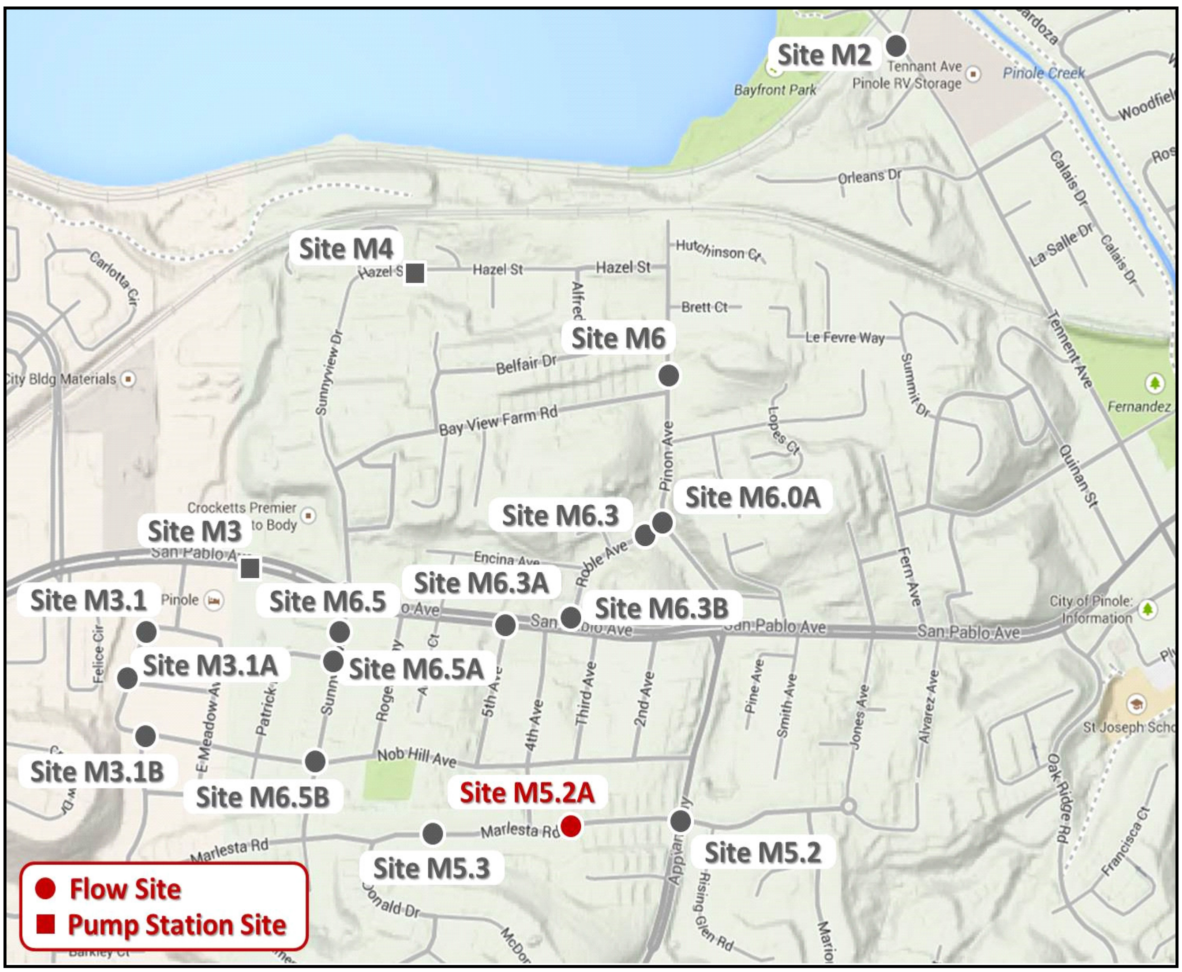
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M5.2A

Location: 1367 Marlesta Rd.

Data Summary Report

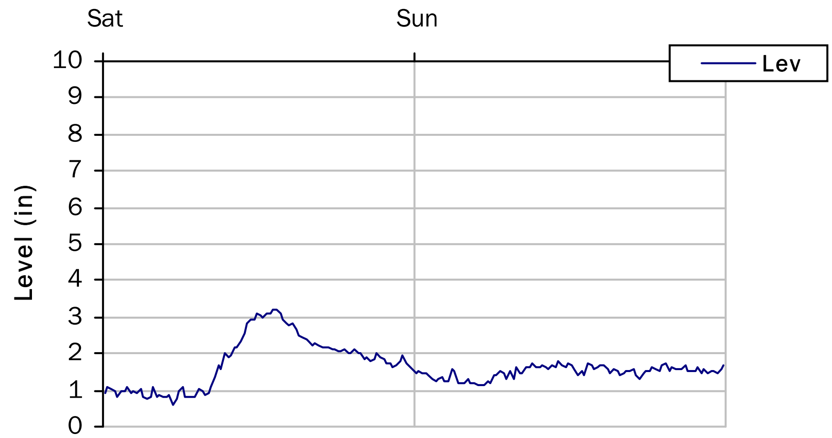


Vicinity Map: Site M5.2A

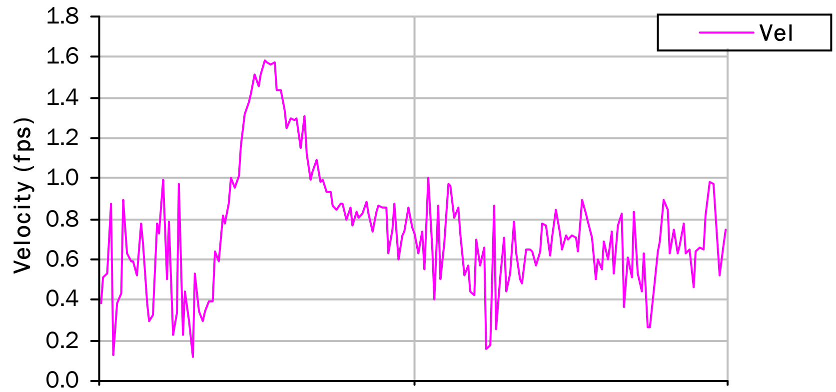
SITE M5.2A

Weekly Level, Velocity and Flow Hydrographs

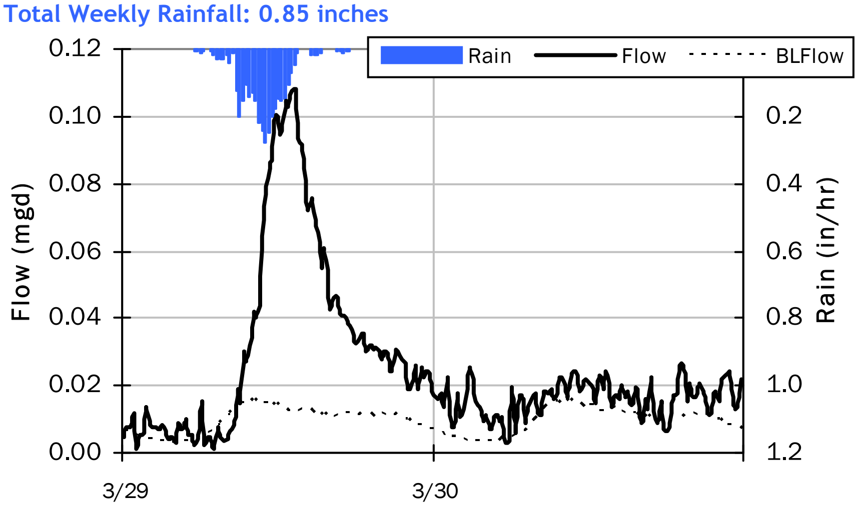
Avg Level: 1.64 in. Peak Level: 3.20 in. Min Level: 0.60 in.



Avg Velocity: 0.74 fps Peak Velocity: 1.58 fps Min Velocity: 0.12 fps



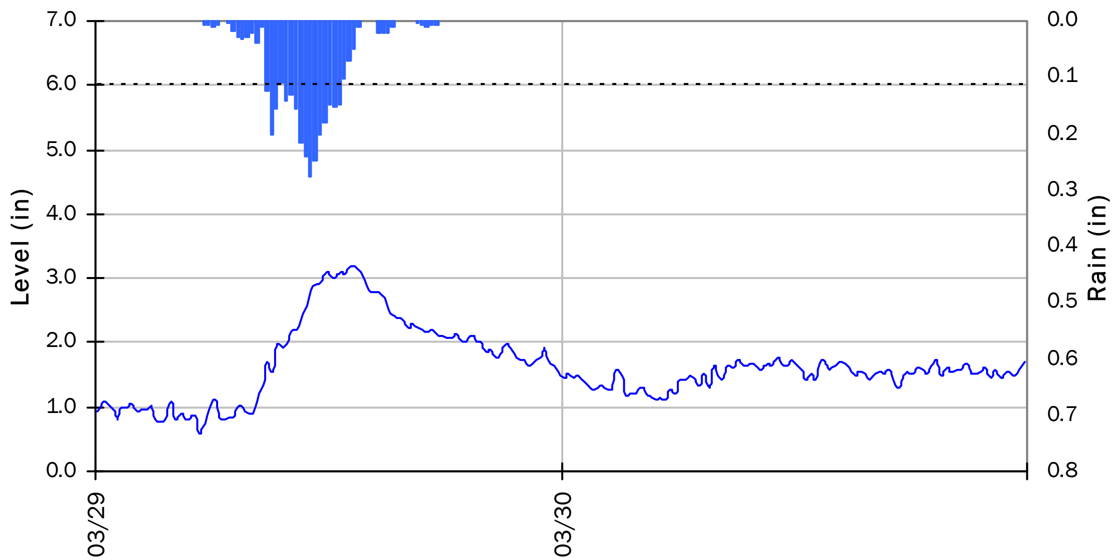
Total Weekly Rainfall: 0.85 inches

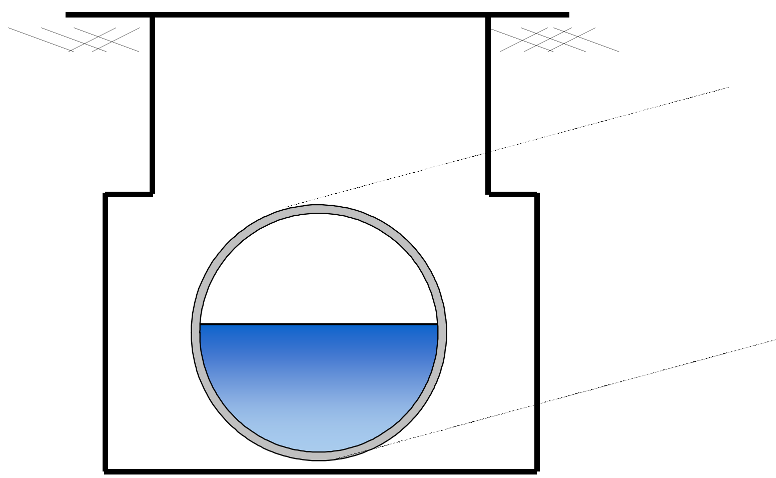


Avg Flow: 0.025 mgd Peak Flow: 0.108 mgd Min Flow: 0.001 mgd

SITE M5.2A
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



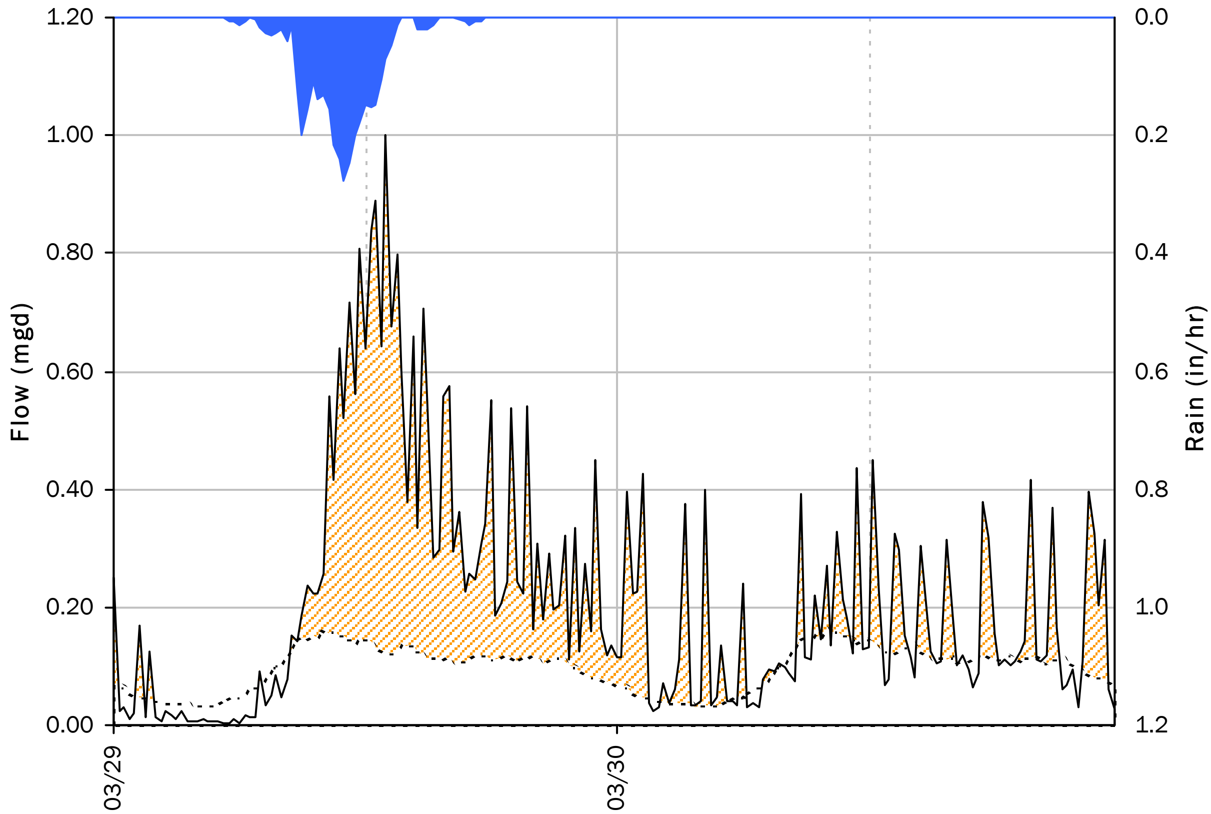


Pipe Diameter:	6	*inches*
Peak Measured Level:	3.2	*inches*
Peak d/D Ratio:	0.53	

SITE 6.3
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 1.00 *mgd*
PF: 11.12

Peak Level: 8.07 *in*
d/D Ratio: 1.01

Inflow / Infiltration

Peak I/I Rate: 0.88 *mgd*
Total I/I: 230,000 *gallons*

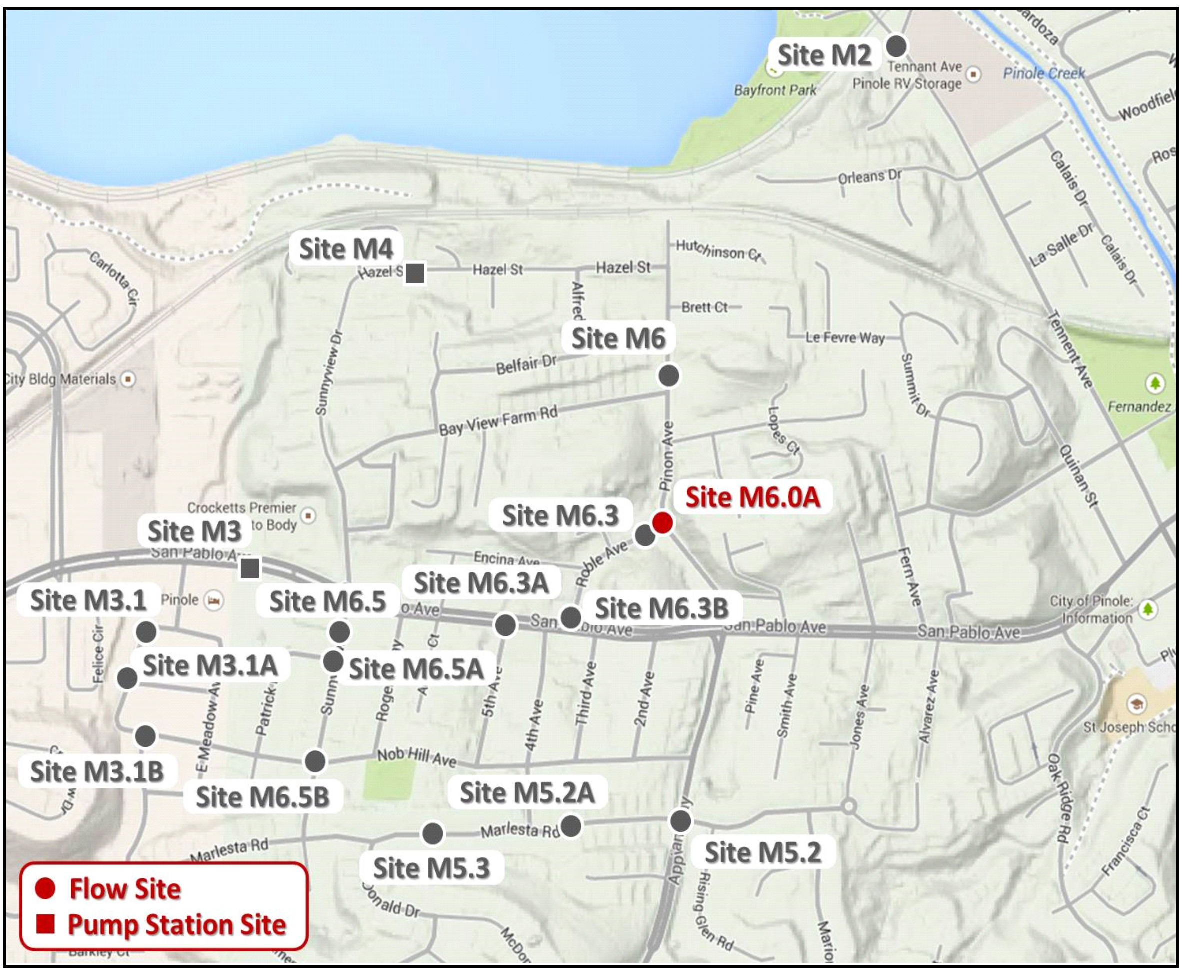
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.0A

Location: Intersection of Roble Ave. and Pinon Ave.

Data Summary Report

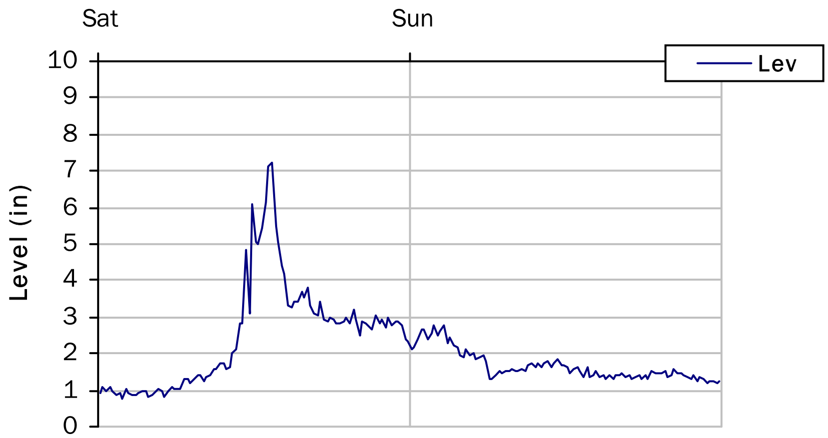


Vicinity Map: Site M6.0A

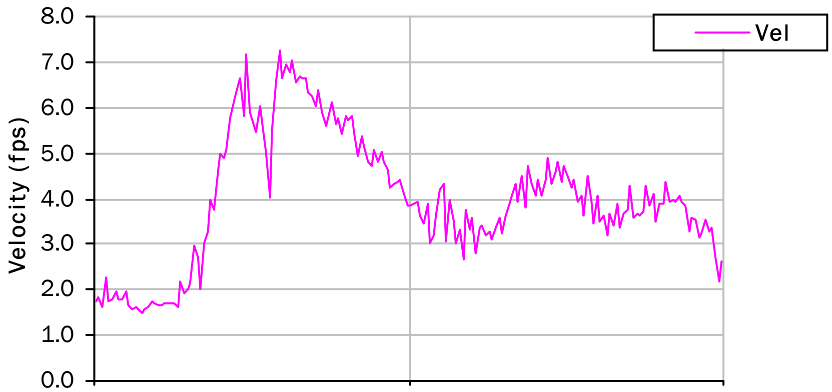
SITE M6.0A

Weekly Level, Velocity and Flow Hydrographs

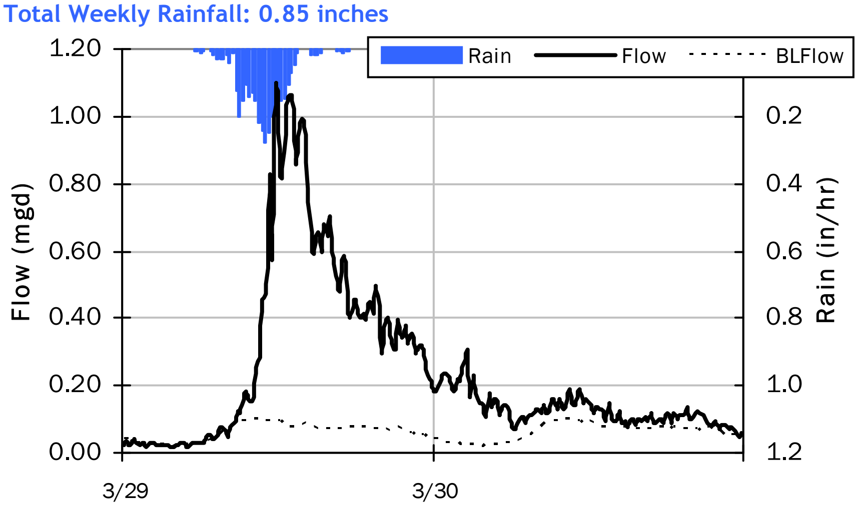
Avg Level: 2.08 in. **Peak Level: 7.24 in.** **Min Level: 0.77 in.**



Avg Velocity: 3.99 fps **Peak Velocity: 7.27 fps** **Min Velocity: 1.50 fps**



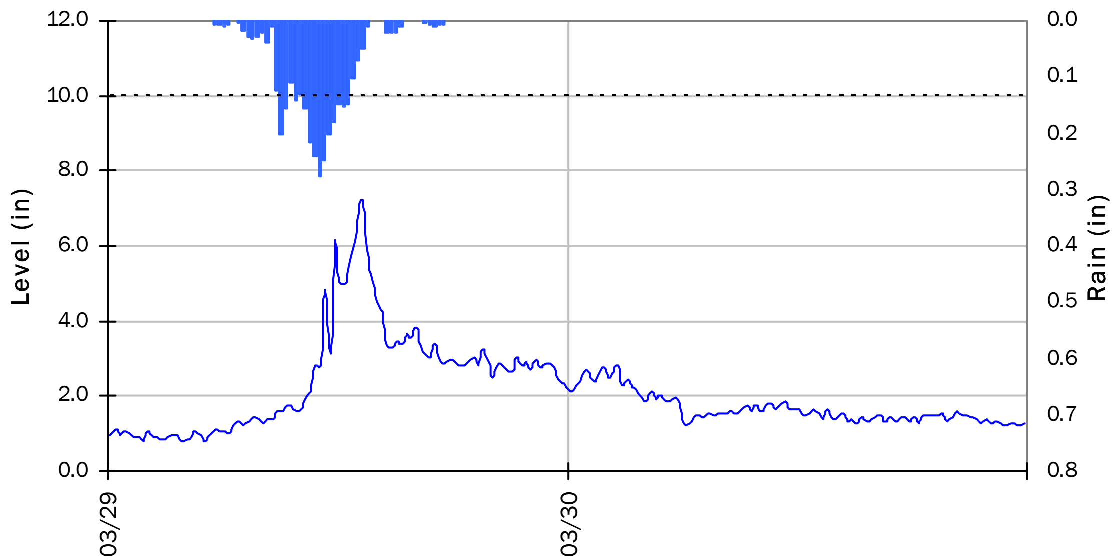
Total Weekly Rainfall: 0.85 inches

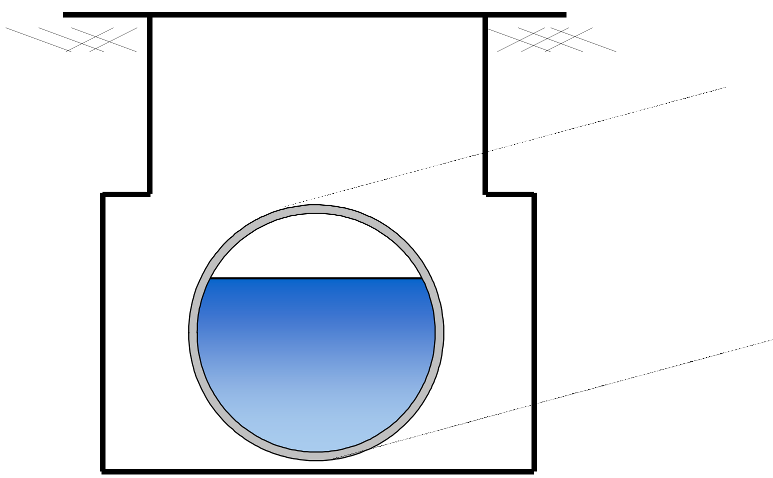


Avg Flow: 0.231 mgd **Peak Flow: 1.091 mgd** **Min Flow: 0.019 mgd**

SITE M6.0A
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



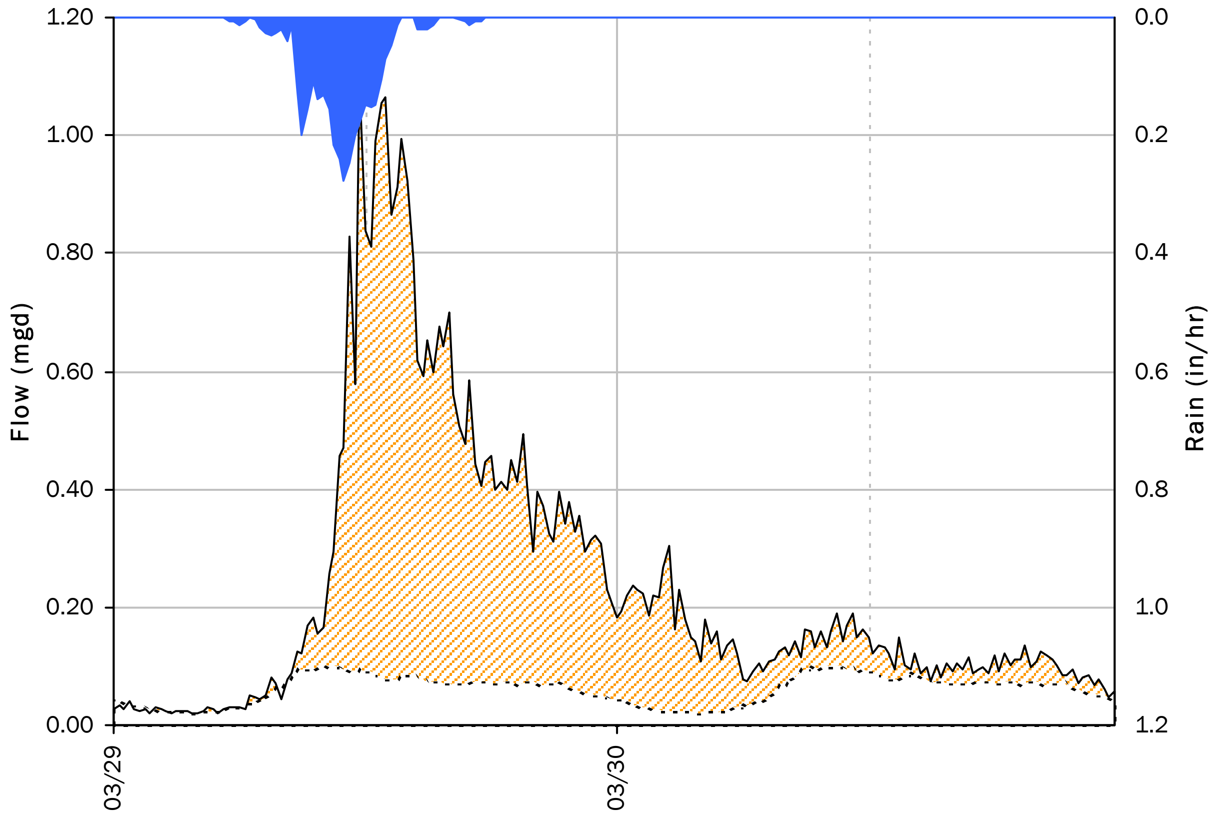


Pipe Diameter:	10	*inches*
Peak Measured Level:	7.24	*inches*
Peak d/D Ratio:	0.72	

SITE 6.0A
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 1.09 *mgd*
PF: 19.03

Peak Level: 7.24 *in*
d/D Ratio: 0.72

Inflow / Infiltration

Peak I/I Rate: 1.00 *mgd*
Total I/I: 339,000 *gallons*

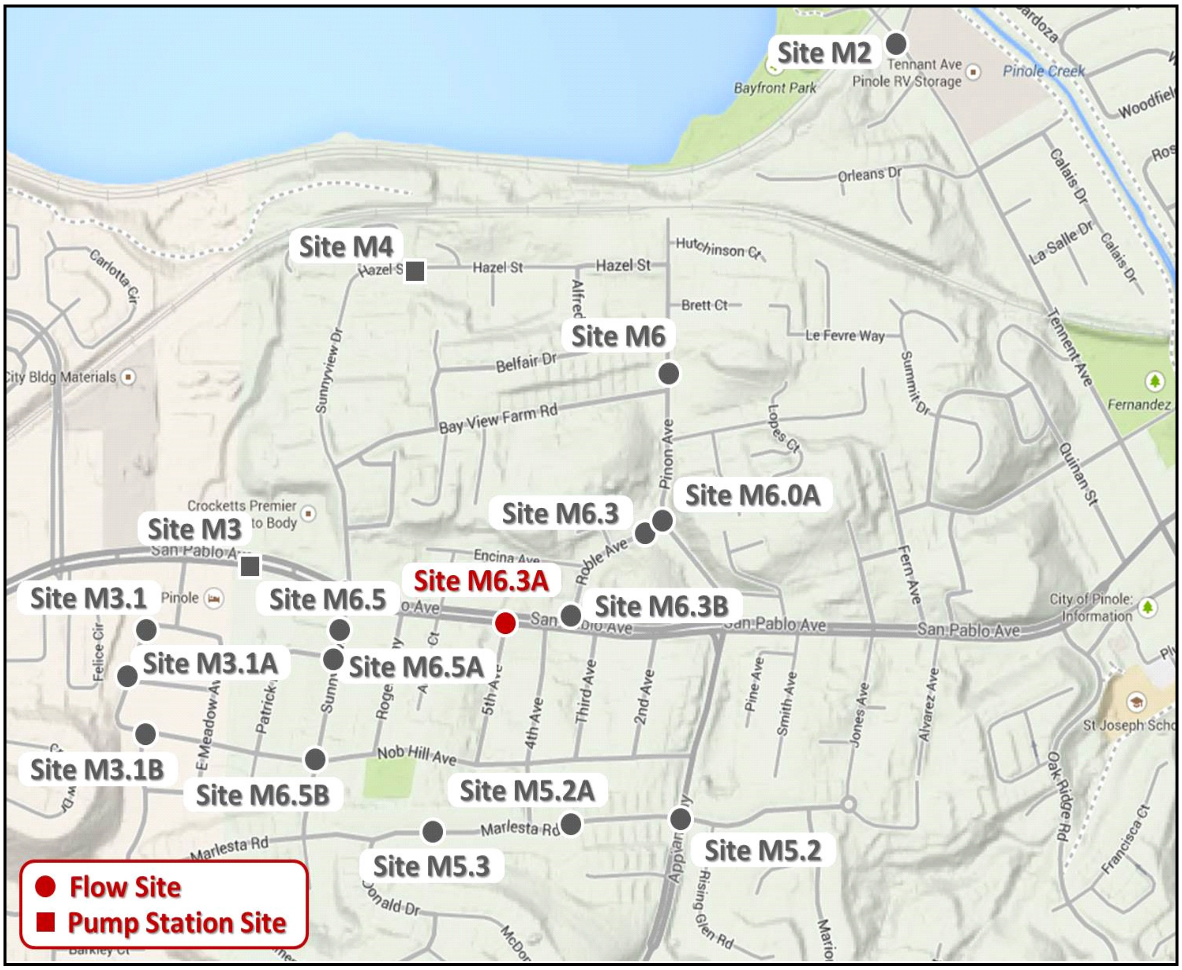
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.3A

Location: Intersection of San Pablo Ave. and 5th Ave.

Data Summary Report

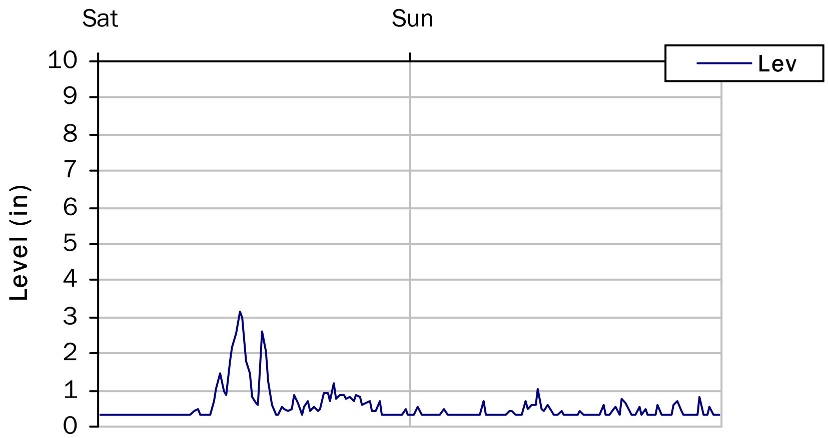


Vicinity Map: Site M6.3A

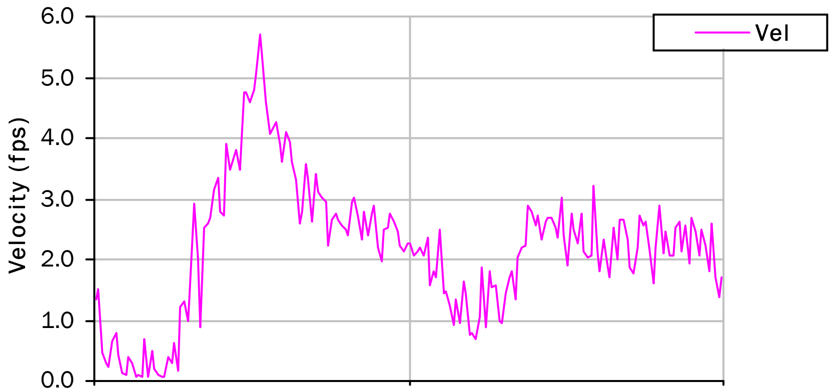
SITE M6.3A

Weekly Level, Velocity and Flow Hydrographs

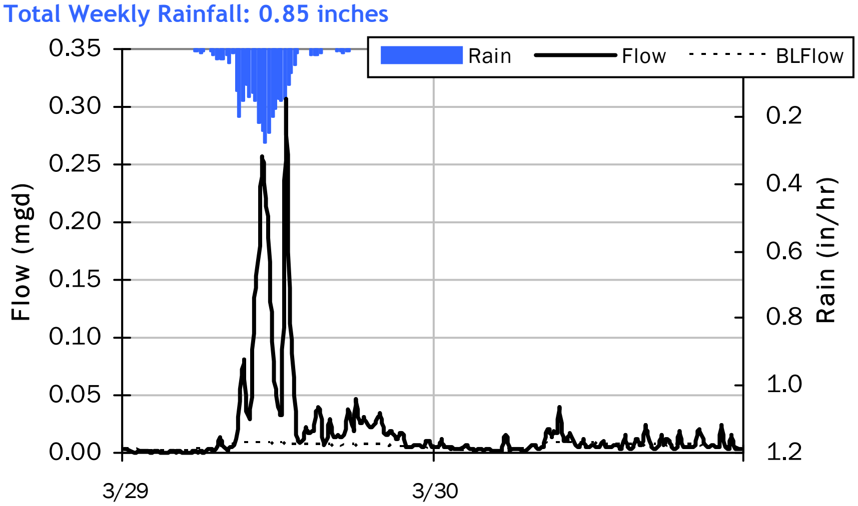
Avg Level: 0.53 in. **Peak Level: 3.15 in.** **Min Level: 0.30 in.**



Avg Velocity: 2.19 fps **Peak Velocity: 5.70 fps** **Min Velocity: 0.07 fps**



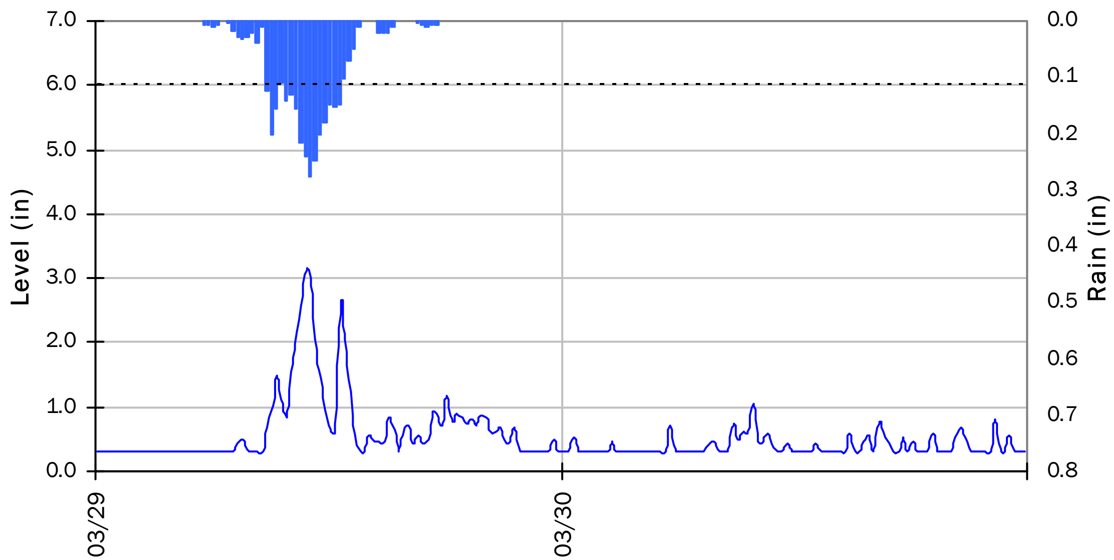
Total Weekly Rainfall: 0.85 inches

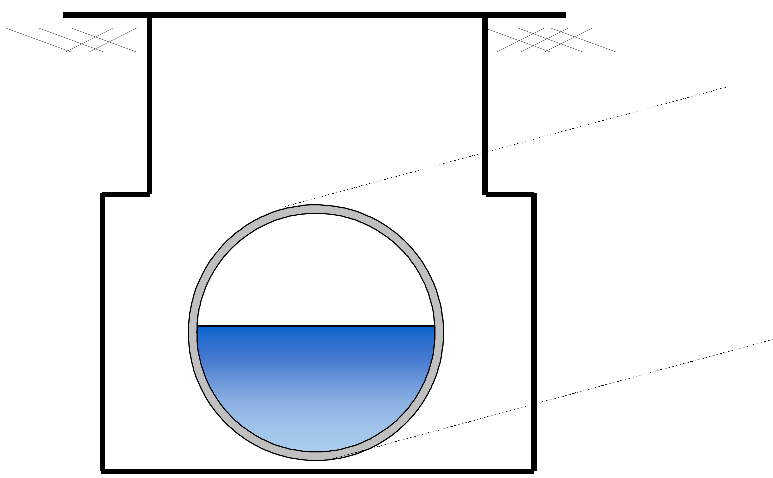


Avg Flow: 0.020 mgd **Peak Flow: 0.302 mgd** **Min Flow: 0.000 mgd**

SITE M6.3A
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



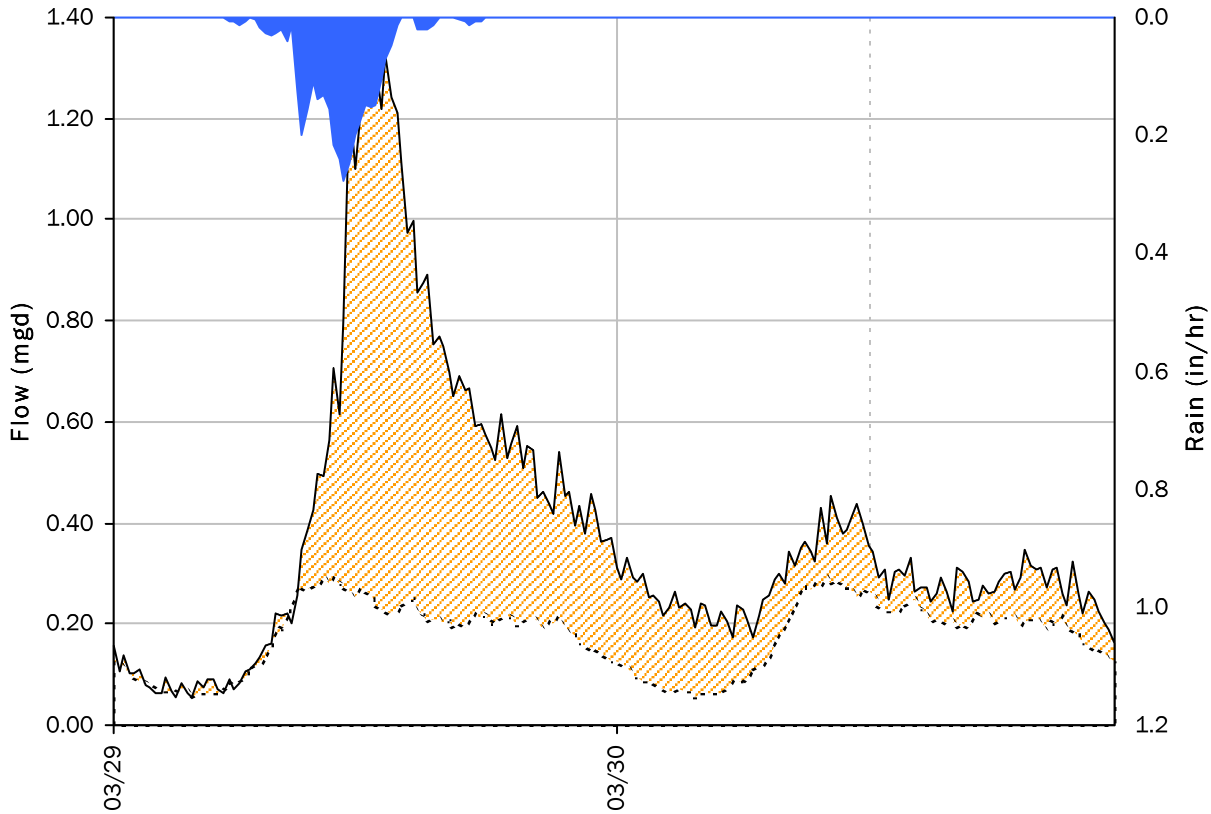


Pipe Diameter:	6	*inches*
Peak Measured Level:	3.15	*inches*
Peak d/D Ratio:	0.53	

SITE 6
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 1.32 *mgd*
PF: 8.09

Peak Level: 41.01 *in*
d/D Ratio: 4.10

Inflow / Infiltration

Peak I/I Rate: 1.10 *mgd*
Total I/I: 408,000 *gallons*

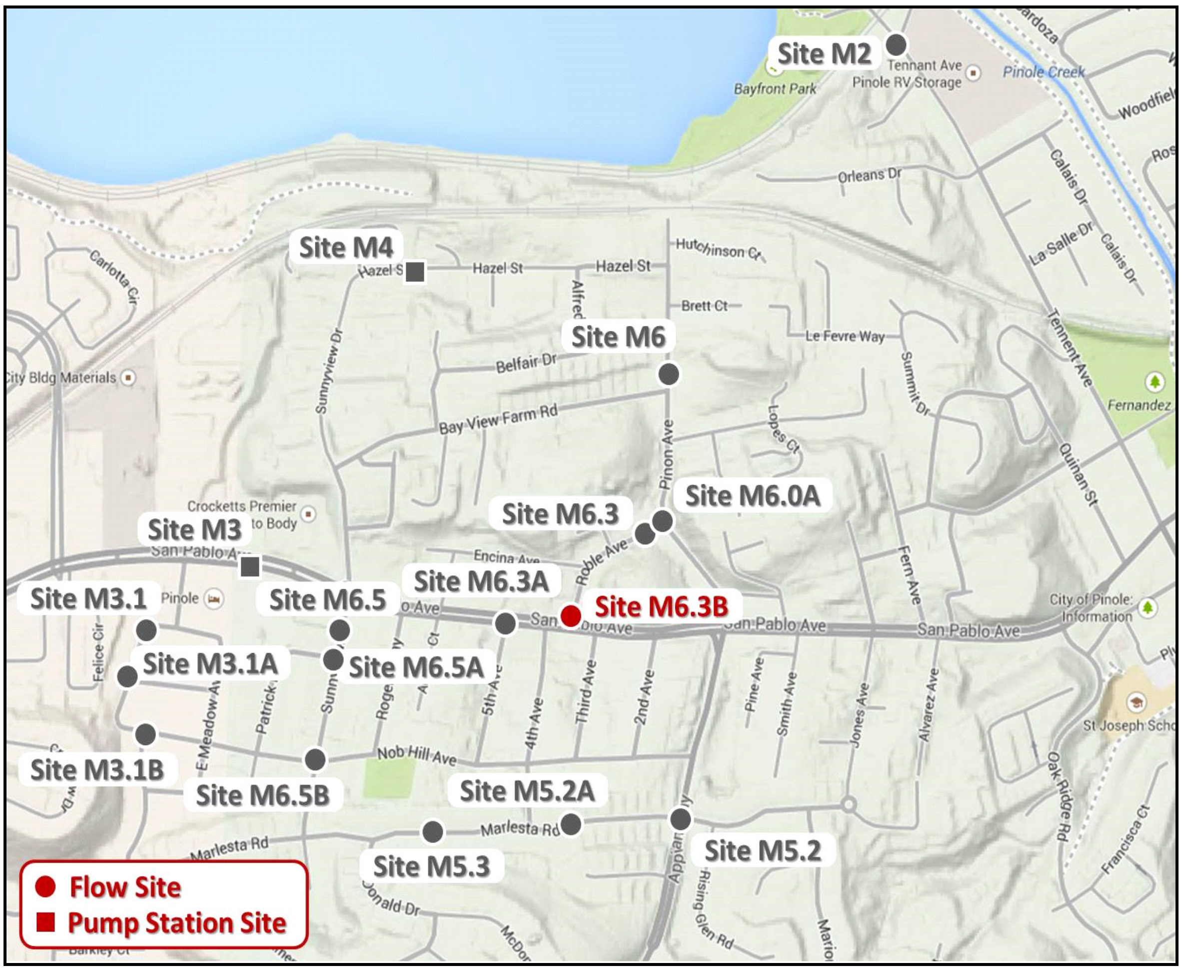
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.3B

Location: Intersection of San Pablo Ave. and Roble Ave.

Data Summary Report

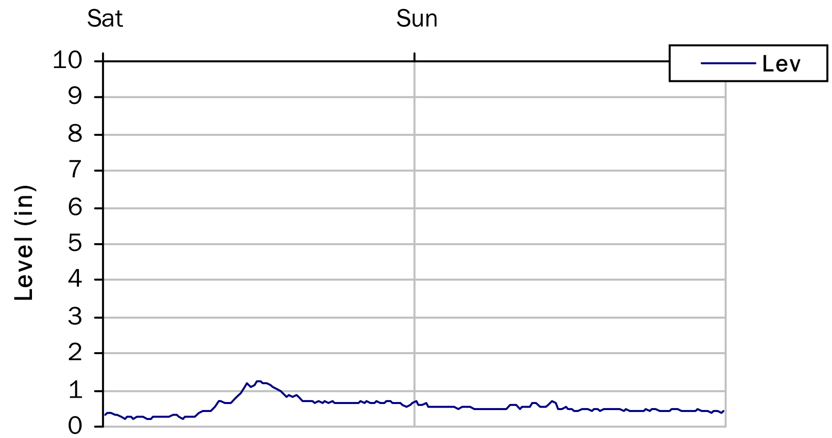


Vicinity Map: Site M6.3B

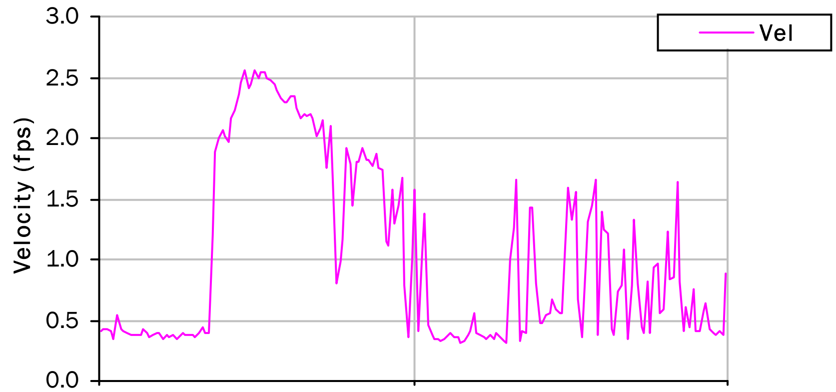
SITE M6.3B

Weekly Level, Velocity and Flow Hydrographs

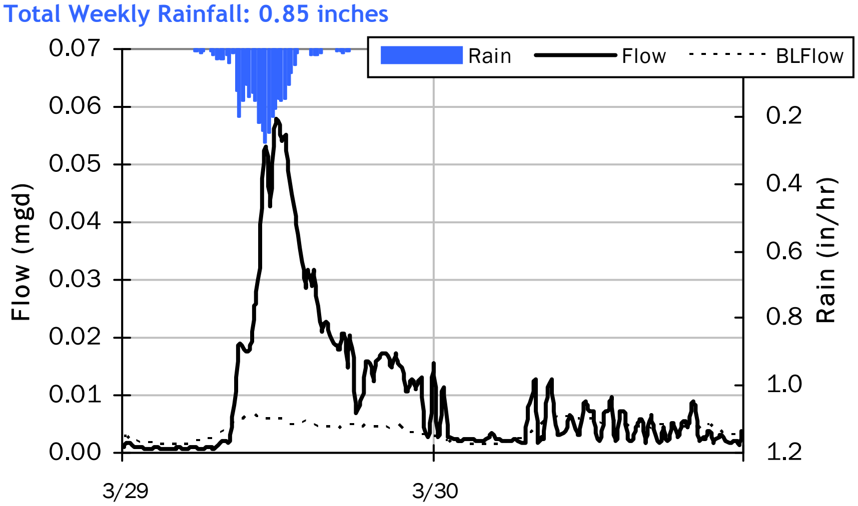
Avg Level: 0.56 in. Peak Level: 1.26 in. Min Level: 0.23 in.



Avg Velocity: 1.04 fps Peak Velocity: 2.56 fps Min Velocity: 0.31 fps



Total Weekly Rainfall: 0.85 inches

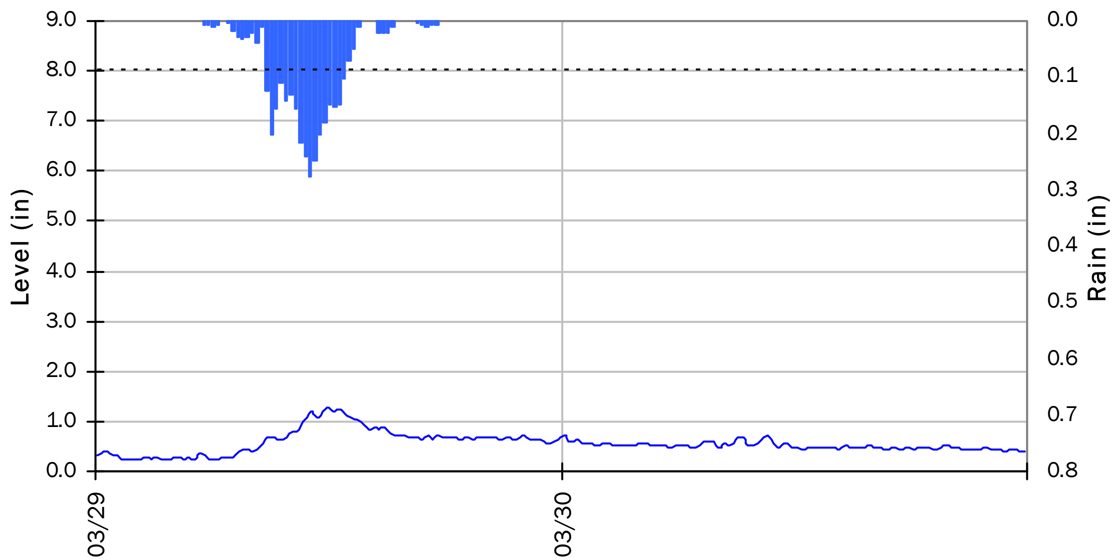


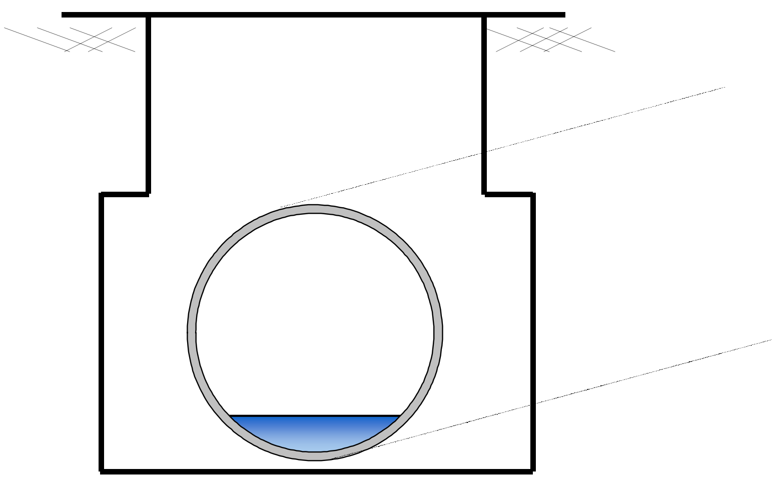
Avg Flow: 0.010 mgd Peak Flow: 0.058 mgd Min Flow: 0.001 mgd

SITE M6.3B

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 8 *inches*

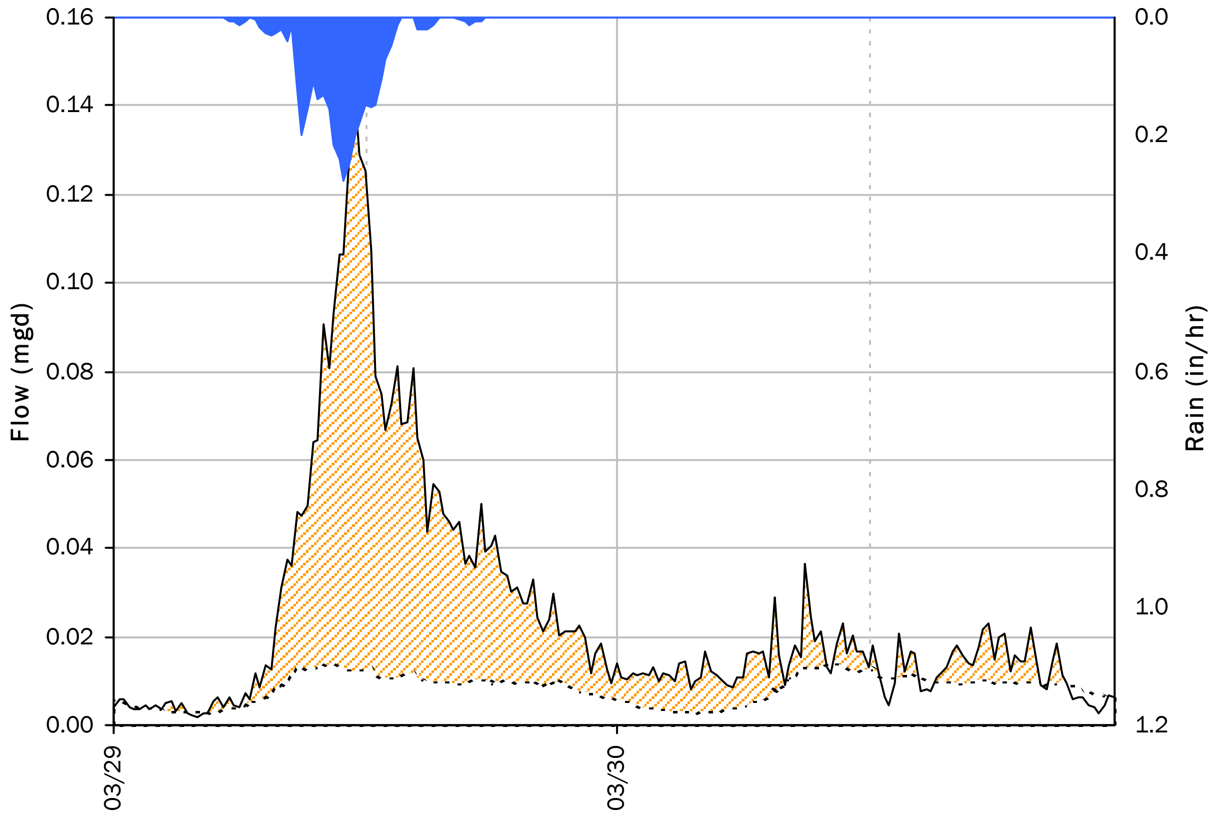
Peak Measured Level: 1.26 *inches*

Peak d/D Ratio: 0.16

SITE 6.5
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.14 *mgd*
PF: 18.00

Peak Level: 3.15 *in*
d/D Ratio: 0.39

Inflow / Infiltration

Peak I/I Rate: 0.13 *mgd*
Total I/I: 33,000 *gallons*

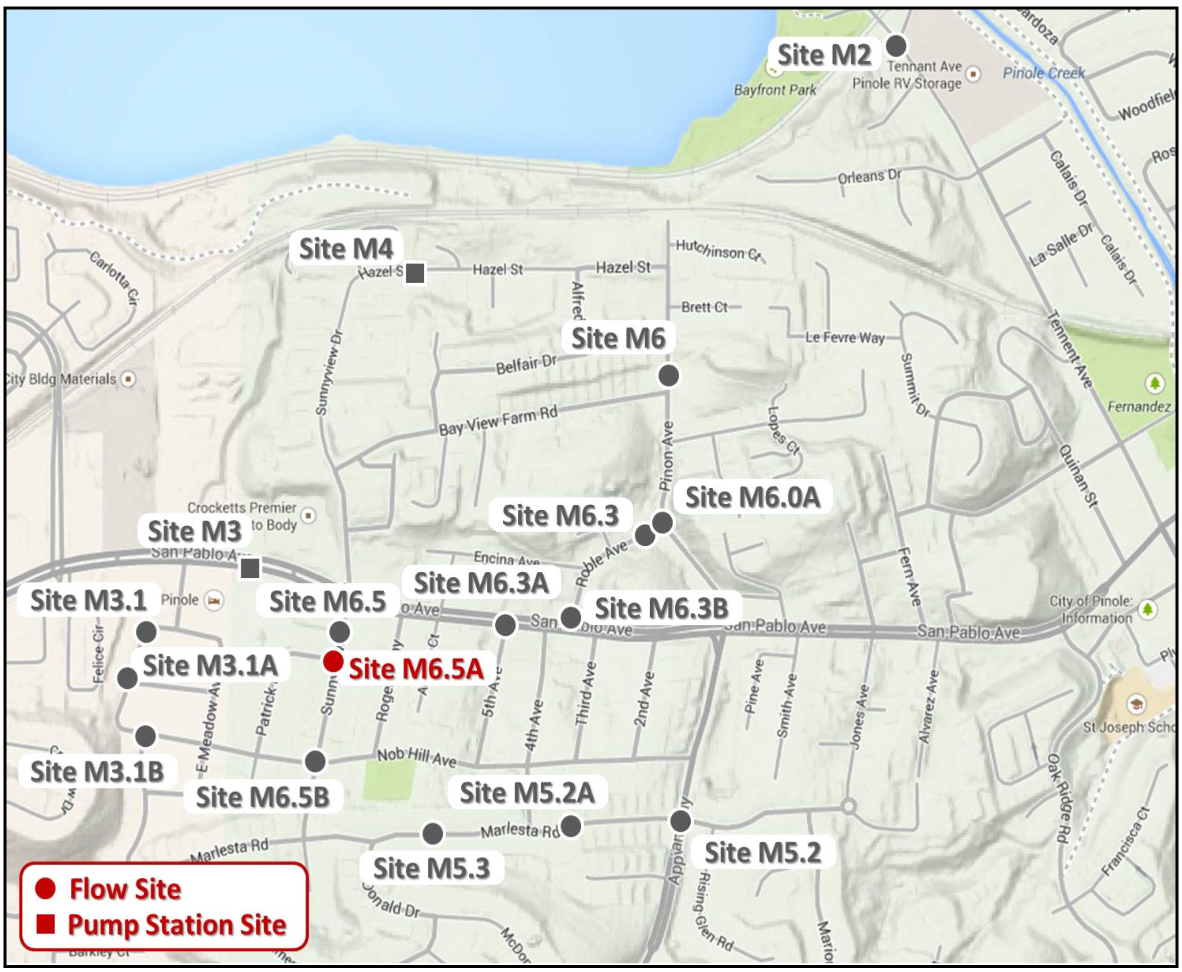
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.5A

Location: Intersection of Sunnyview Dr. and Patrick Dr.

Data Summary Report

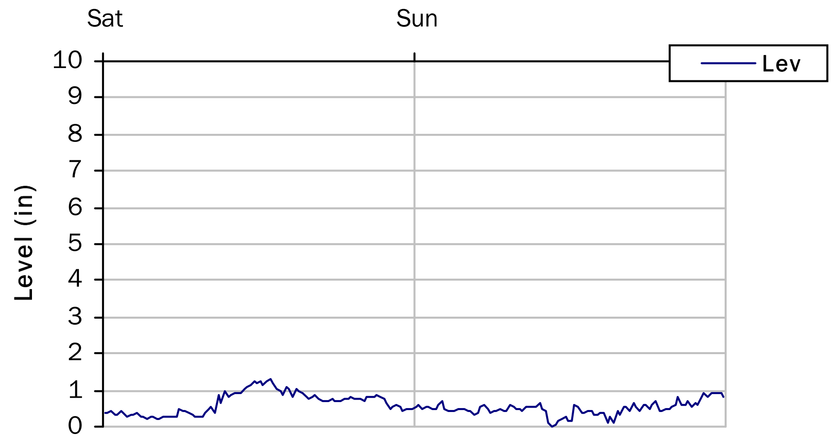


Vicinity Map: Site M6.5A

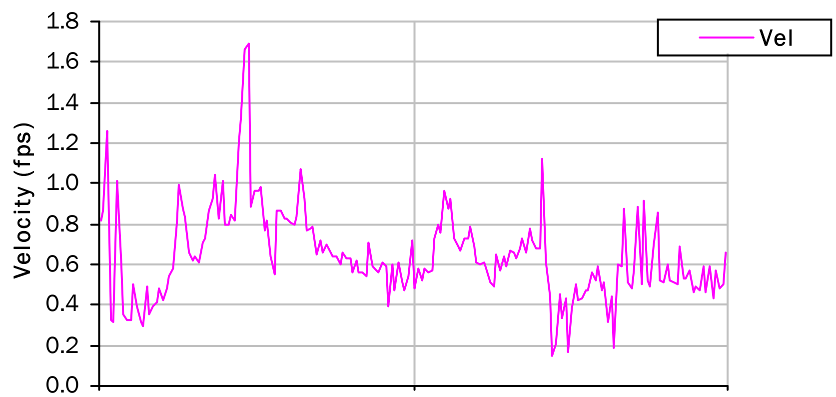
SITE M6.5A

Weekly Level, Velocity and Flow Hydrographs

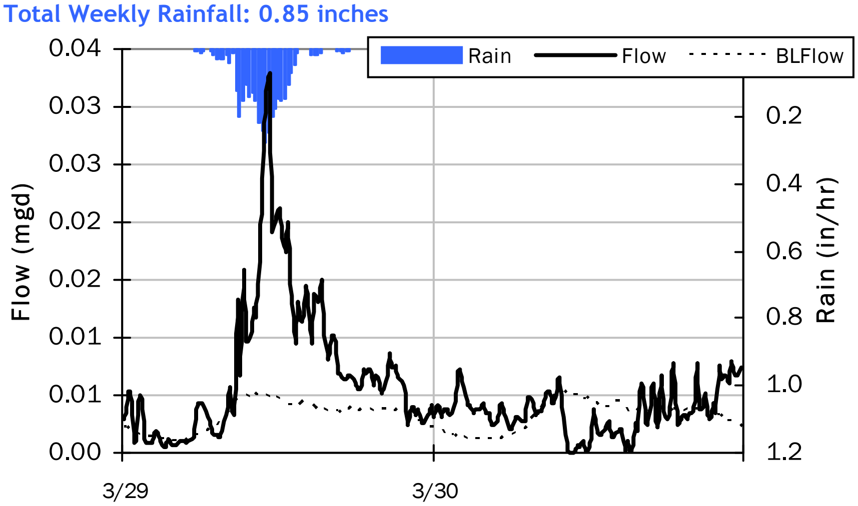
Avg Level: 0.58 in. Peak Level: 1.32 in. Min Level: 0.01 in.



Avg Velocity: 0.64 fps Peak Velocity: 1.69 fps Min Velocity: 0.15 fps



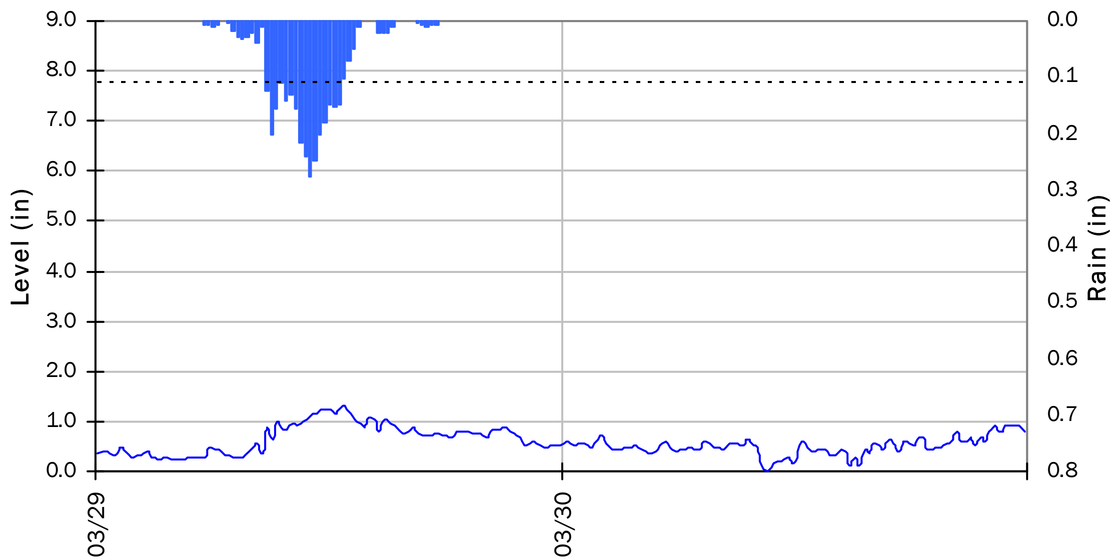
Total Weekly Rainfall: 0.85 inches

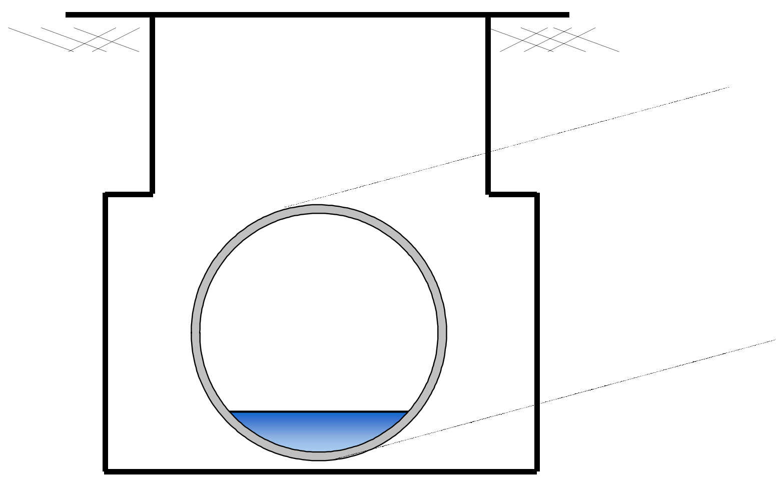


Avg Flow: 0.006 mgd Peak Flow: 0.033 mgd Min Flow: 0.000 mgd

SITE M6.5A
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



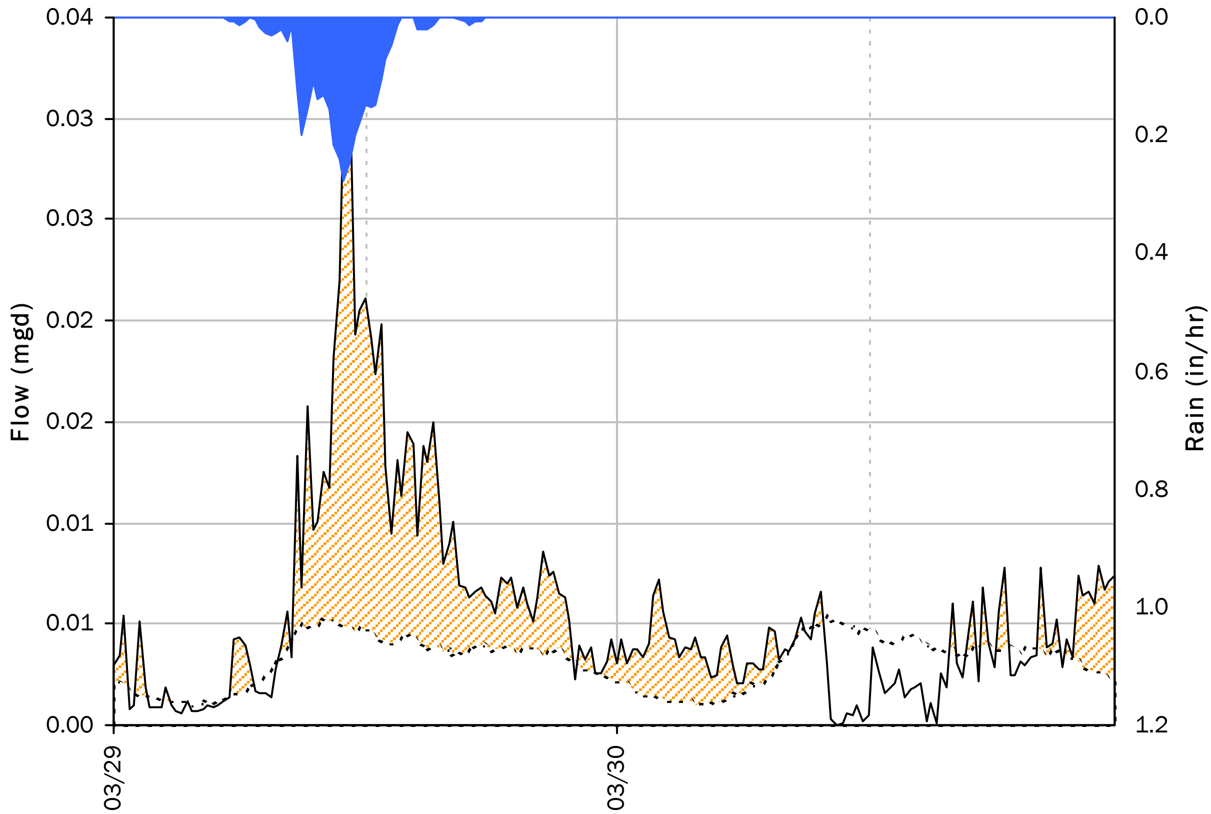


Pipe Diameter:	7.75	*inches*
Peak Measured Level:	1.32	*inches*
Peak d/D Ratio:	0.17	

SITE 6.5A
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.03 *mgd*
PF: 11.06

Peak Level: 1.32 *in*
d/D Ratio: 0.17

Inflow / Infiltration

Peak I/I Rate: 0.03 *mgd*
Total I/I: 5,000 *gallons*

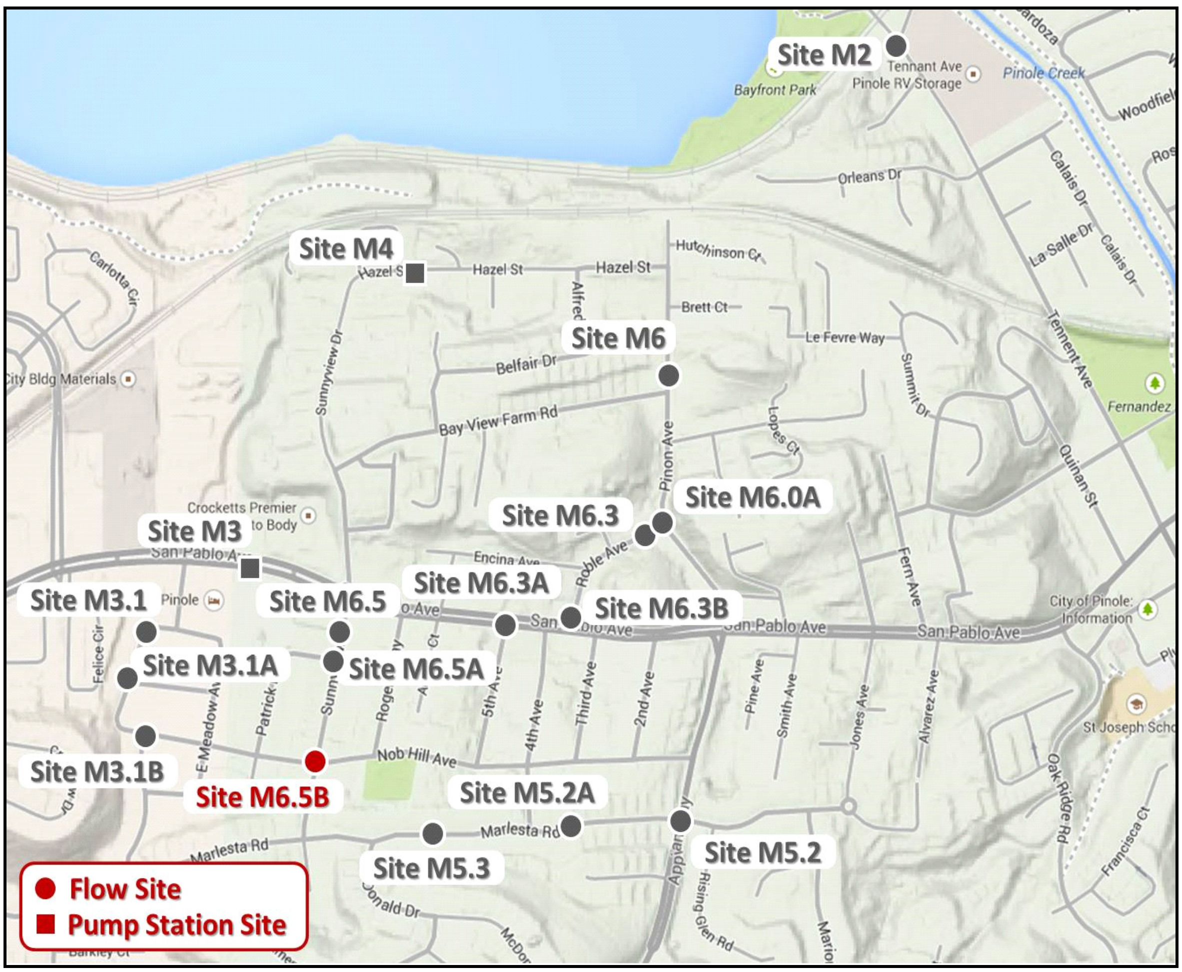
City of Pinole

Sanitary Sewer Flow Monitoring
Temporary Monitoring: March 2014

Monitoring Site: Site M6.5B

Location: Intersection of Sunnyview Dr. and Nob Hill Ave.

Data Summary Report

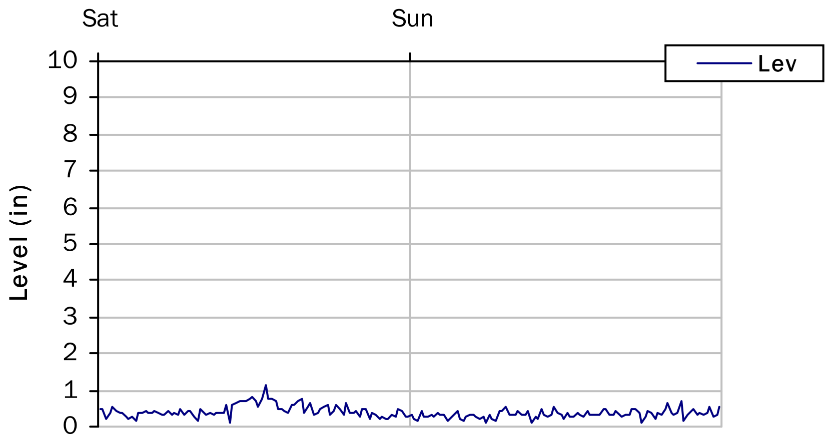


Vicinity Map: Site M6.5B

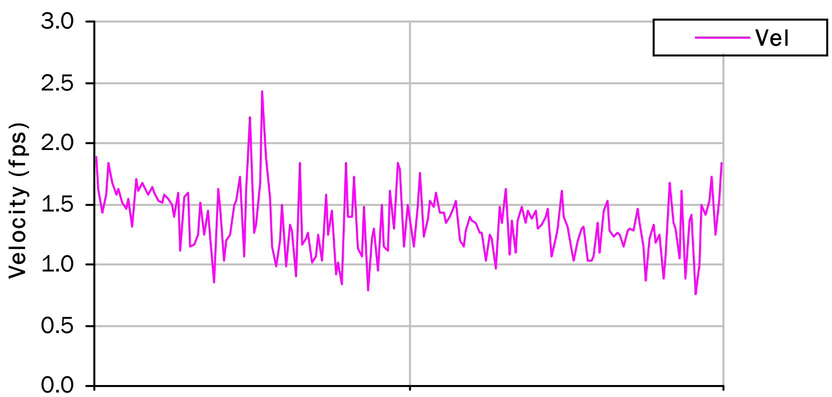
SITE M6.5B

Weekly Level, Velocity and Flow Hydrographs

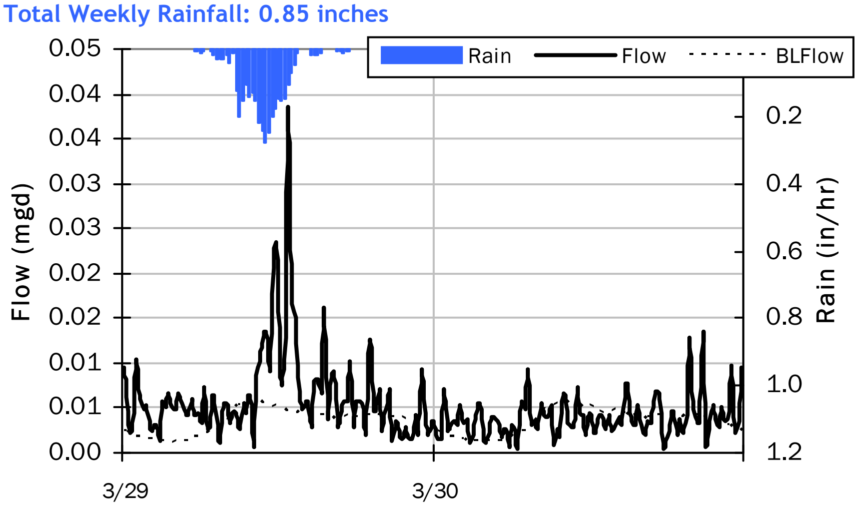
Avg Level: 0.39 in. Peak Level: 1.12 in. Min Level: 0.10 in.



Avg Velocity: 1.36 fps Peak Velocity: 2.42 fps Min Velocity: 0.76 fps



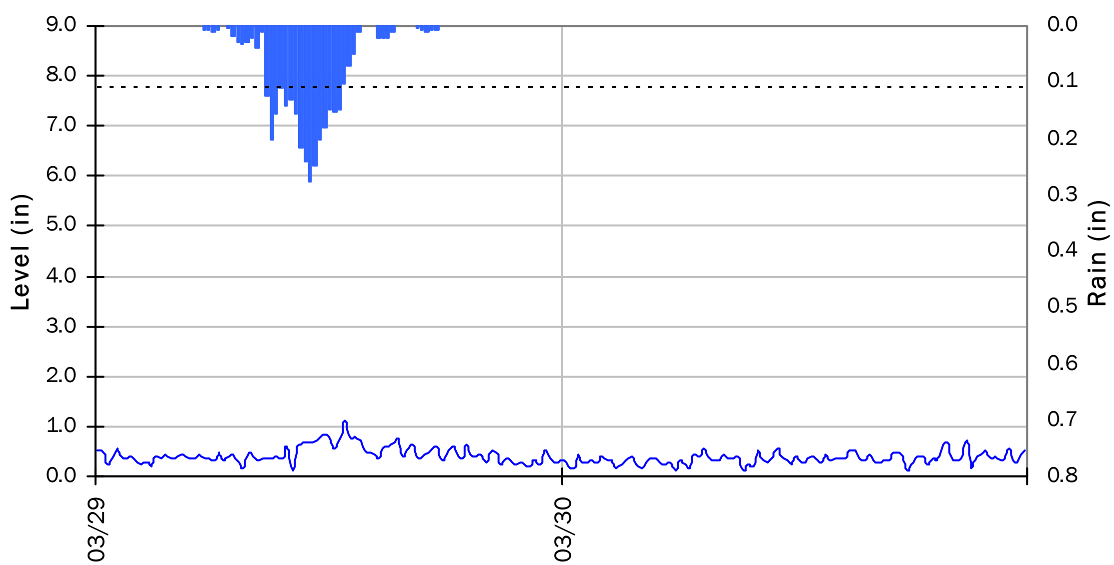
Total Weekly Rainfall: 0.85 inches

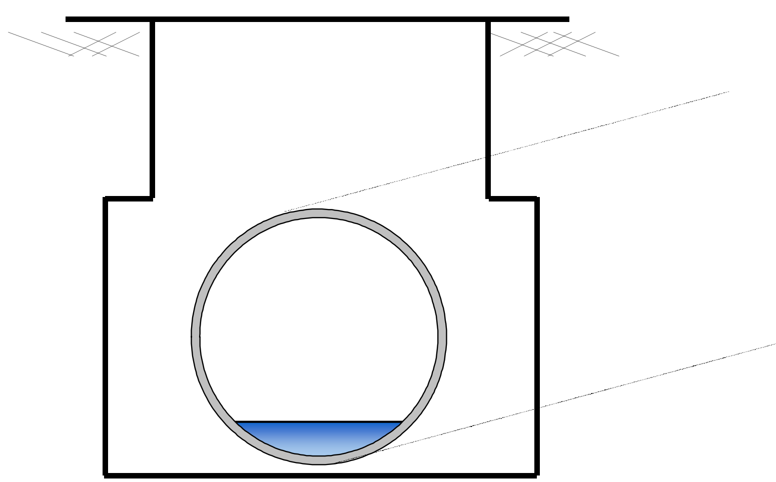


Avg Flow: 0.005 mgd Peak Flow: 0.039 mgd Min Flow: 0.001 mgd

SITE M6.5B
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



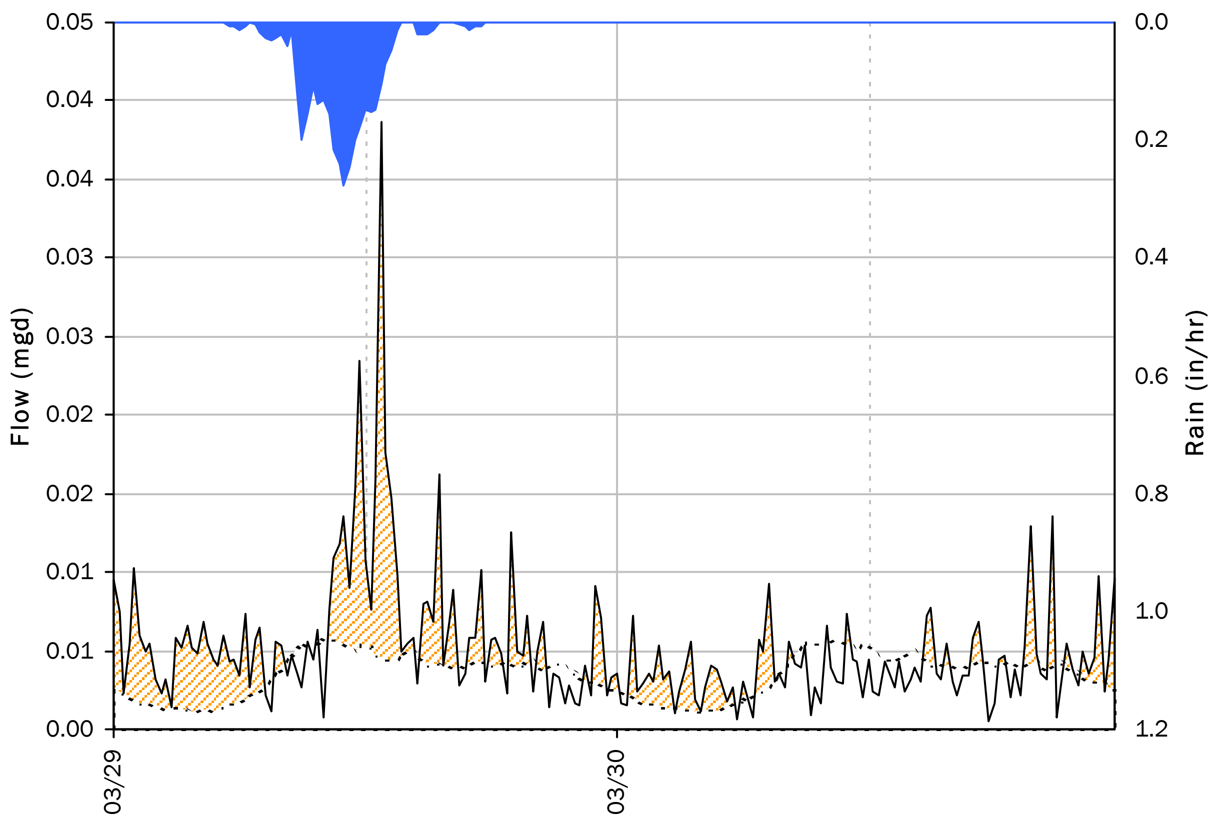


Pipe Diameter:	7.75	*inches*
Peak Measured Level:	1.12	*inches*
Peak d/D Ratio:	0.14	

SITE 6.5B
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period

Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity

Peak Flow: 0.04 *mgd*
PF: 11.78

Peak Level: 1.12 *in*
d/D Ratio: 0.14

Inflow / Infiltration

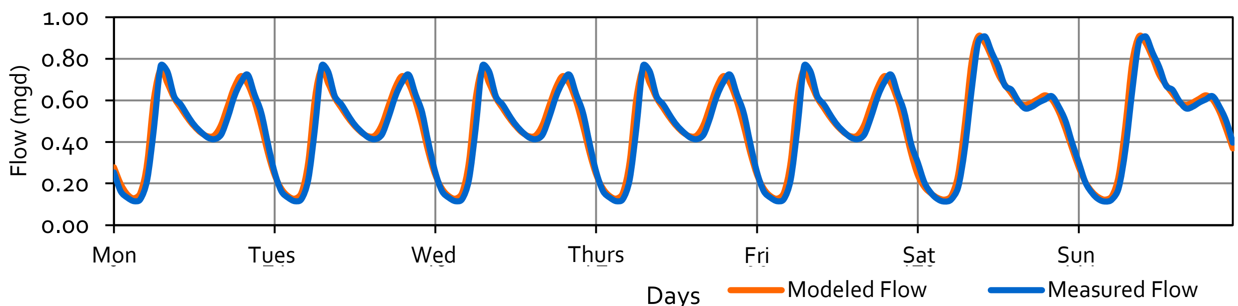
Peak I/I Rate: 0.03 *mgd*
Total I/I: 3,000 *gallons*

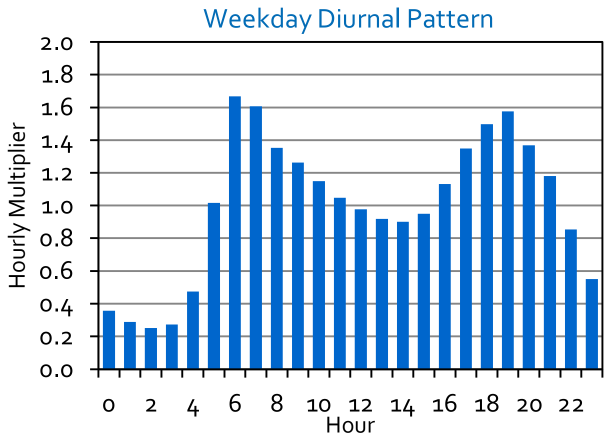
Appendix B

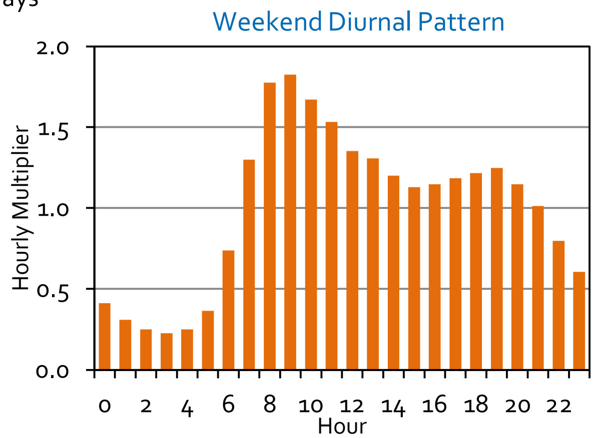
DRY WEATHER FLOW CALIBRATION SHEETS

Flow Monitoring Site M1, Dry Weather Flow Calibration

Location: Pinole valley Road just south of Highway 80
Measured Pipeline Diameter: 15"
Model Manhole ID: MH_1277
Model Pipe ID: P_691







Model Calibration Summary

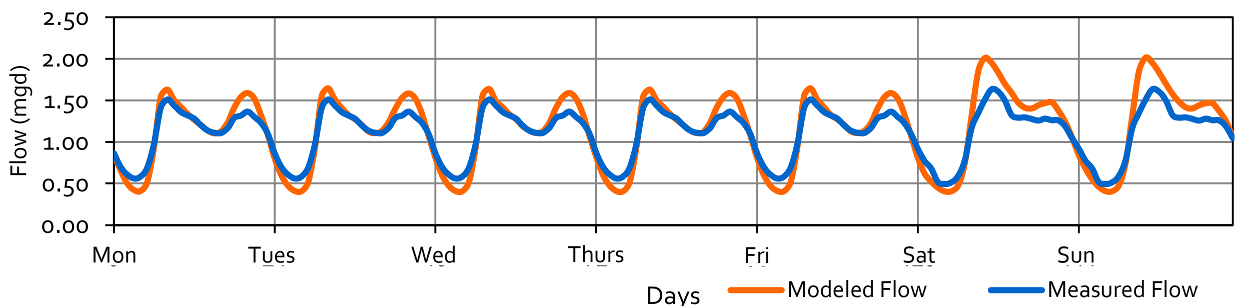
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.46	0.77	0.47	0.75	2%	-2%
Tues.	0.46	0.77	0.47	0.75	2%	-2%
Wed.	0.46	0.77	0.47	0.75	2%	-2%
Thur.	0.46	0.77	0.47	0.75	2%	-2%
Fri.	0.46	0.77	0.47	0.75	2%	-2%
Sat.	0.50	0.91	0.50	0.91	1%	0%
Sun.	0.50	0.91	0.51	0.91	2%	0%
Summary						
Weekday	0.459	--	0.467	--	2%	--
Weekend	0.497	--	0.505	--	2%	--
ADWF[4]	0.470	--	0.478	--	2%	--

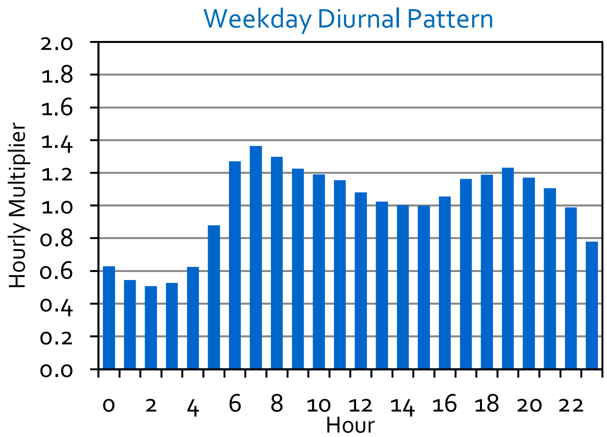
Notes:

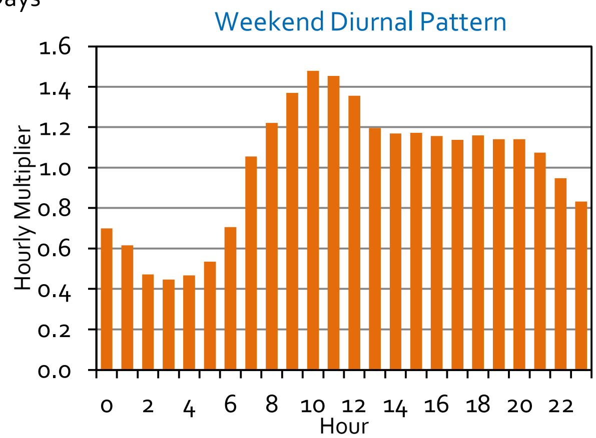
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M2, Dry Weather Flow Calibration

Location: Tennent Avenue just outside WPCP
Measured Pipeline Diameter: 30"
Model Manhole ID: MH_1106
Model Pipe ID: P_868







Model Calibration Summary

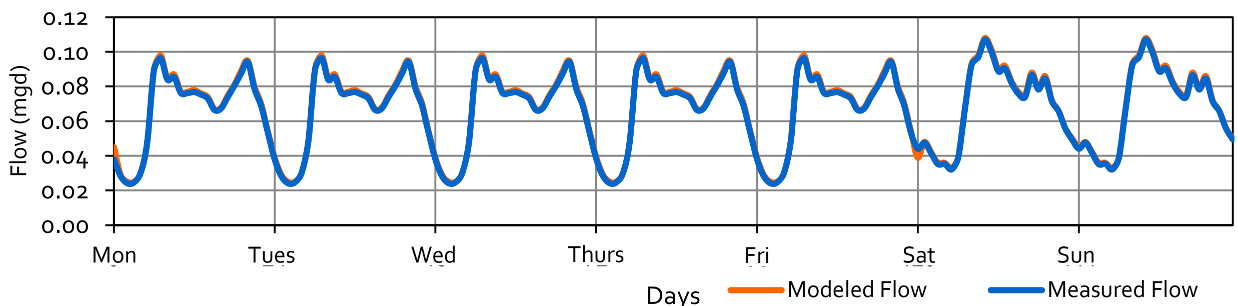
Day	Measured Data[1]		Modeled Data		Percent Error[3]	
	Avg. Flow (mgd)	Peak Flow[2] (mgd)	Avg. Flow (mgd)	Peak Flow[2] (mgd)	Avg. Flow (%)	Peak Flow (%)
Mon.	1.11	1.51	1.13	1.65	2%	9%
Tues.	1.11	1.51	1.13	1.65	2%	9%
Wed.	1.11	1.51	1.13	1.65	2%	9%
Thur.	1.11	1.51	1.13	1.65	2%	9%
Fri.	1.11	1.51	1.13	1.65	2%	9%
Sat.	1.10	1.63	1.21	2.01	10%	23%
Sun.	1.10	1.63	1.21	2.02	10%	23%
Summary						
Weekday	1.110	--	1.132	--	2%	--
Weekend	1.105	--	1.212	--	10%	--
ADWF[4]	1.108	--	1.155	--	4%	--

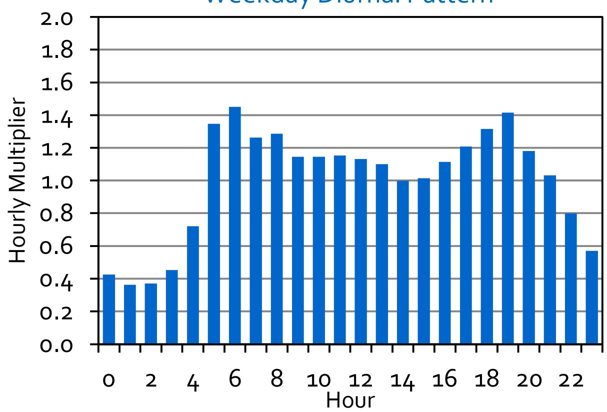
Notes:

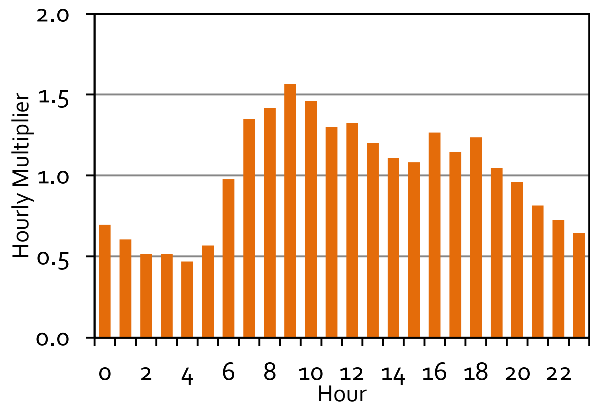
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M3, Dry Weather Flow Calibration

Location: San Pablo Lift Station (San Pablo Avenue west of Sunnyview Drive)
Measured Pipeline Diameter: N/A"
Model Manhole ID: SAN_PABLO
Model Pipe ID: San_Pablo







Model Calibration Summary

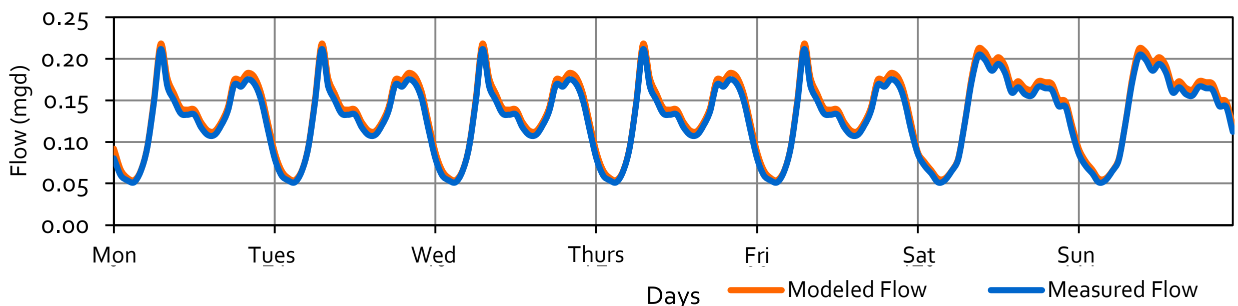
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.07	0.10	0.07	0.10	1%	2%
Tues.	0.07	0.10	0.07	0.10	1%	2%
Wed.	0.07	0.10	0.07	0.10	1%	2%
Thur.	0.07	0.10	0.07	0.10	1%	2%
Fri.	0.07	0.10	0.07	0.10	1%	2%
Sat.	0.07	0.11	0.07	0.11	1%	1%
Sun.	0.07	0.11	0.07	0.11	1%	1%
Summary						
Weekday	0.067	--	0.067	--	1%	--
Weekend	0.068	--	0.069	--	1%	--
ADWF[4]	0.067	--	0.068	--	1%	--

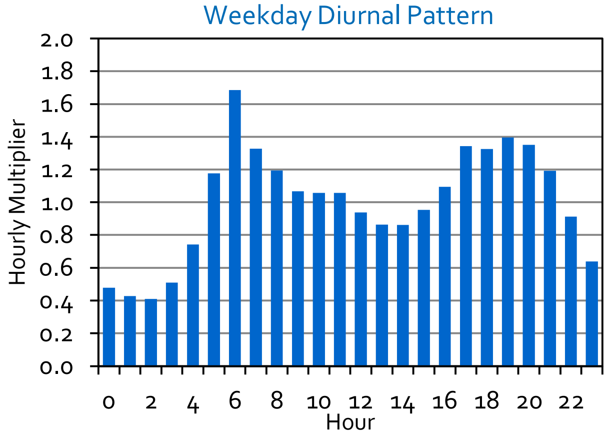
Notes:

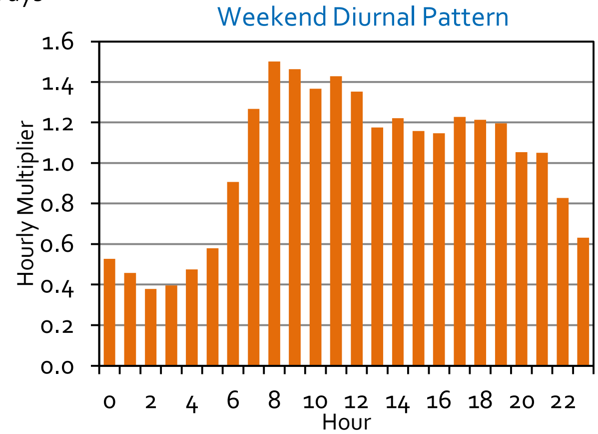
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M4, Dry Weather Flow Calibration

Location: Hazel Lift Station (In easement at west end of Hazel Street)
Measured Pipeline Diameter: N/A"
Model Manhole ID: MH_1175
Model Pipe ID: P_428







Model Calibration Summary

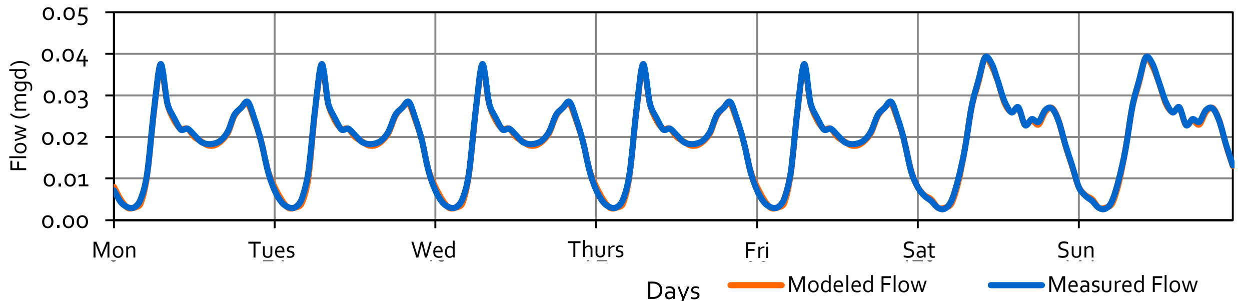
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.13	0.21	0.13	0.22	5%	3%
Tues.	0.13	0.21	0.13	0.22	5%	3%
Wed.	0.13	0.21	0.13	0.22	5%	3%
Thur.	0.13	0.21	0.13	0.22	5%	3%
Fri.	0.13	0.21	0.13	0.22	5%	3%
Sat.	0.14	0.20	0.14	0.21	4%	4%
Sun.	0.14	0.20	0.14	0.21	4%	4%
Summary						
Weekday	0.126	--	0.131	--	5%	--
Weekend	0.136	--	0.142	--	4%	--
ADWF[4]	0.129	--	0.134	--	5%	--

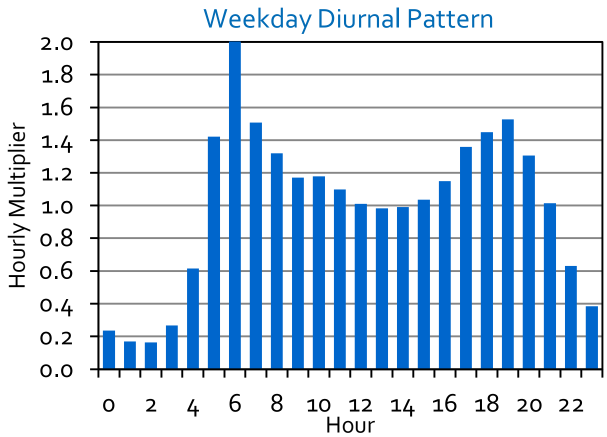
Notes:

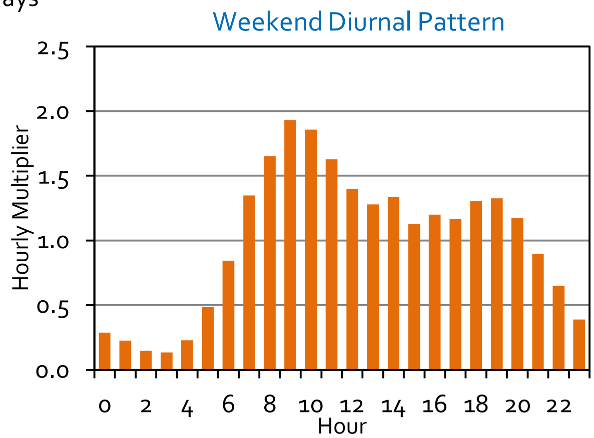
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M5, Dry Weather Flow Calibration

Location: Appian Way south of San Pablo Avenue
Measured Pipeline Diameter: 7.25"
Model Manhole ID: MH_497
Model Pipe ID: P_101







Model Calibration Summary

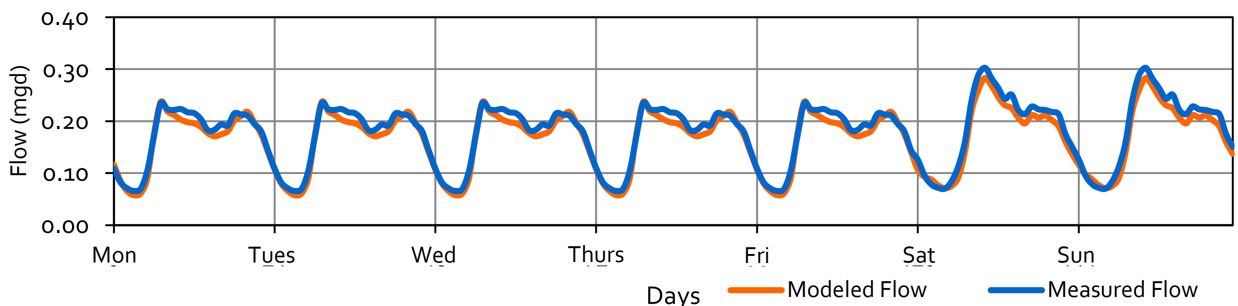
	Measured Data[1]		Modeled Data		Percent Error[3]	
Day	Avg. Flow (mgd)	Peak Flow[2] (mgd)	Avg. Flow (mgd)	Peak Flow[2] (mgd)	Avg. Flow (%)	Peak Flow (%)
Mon.	0.02	0.04	0.02	0.04	-1%	-1%
Tues.	0.02	0.04	0.02	0.04	-1%	-1%
Wed.	0.02	0.04	0.02	0.04	-1%	-1%
Thur.	0.02	0.04	0.02	0.04	-1%	-1%
Fri.	0.02	0.04	0.02	0.04	-1%	-1%
Sat.	0.02	0.04	0.02	0.04	-1%	0%
Sun.	0.02	0.04	0.02	0.04	-1%	0%
Summary						
Weekday	0.019	--	0.018	--	-1%	--
Weekend	0.020	--	0.020	--	-1%	--
ADWF[4]	0.019	--	0.019	--	-1%	--

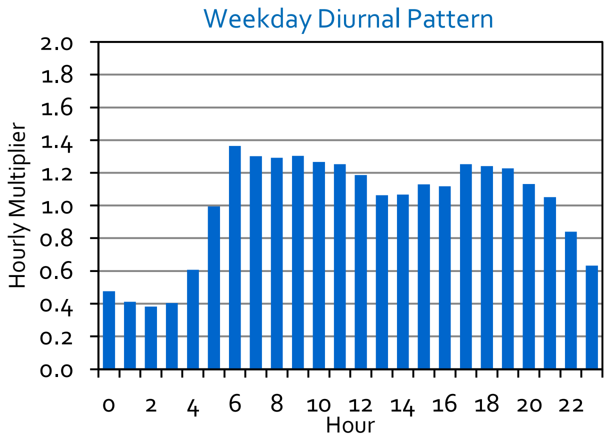
Notes:

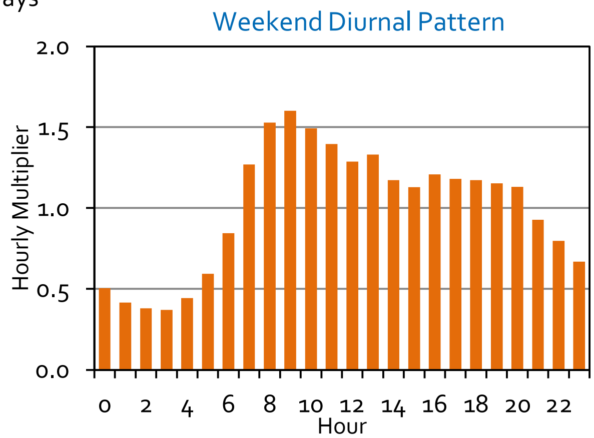
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M6, Dry Weather Flow Calibration

Location: Pinon Avenue north of Bay View Farm Road
Measured Pipeline Diameter: 10"
Model Manhole ID: MH_1143
Model Pipe ID: P_755







Model Calibration Summary

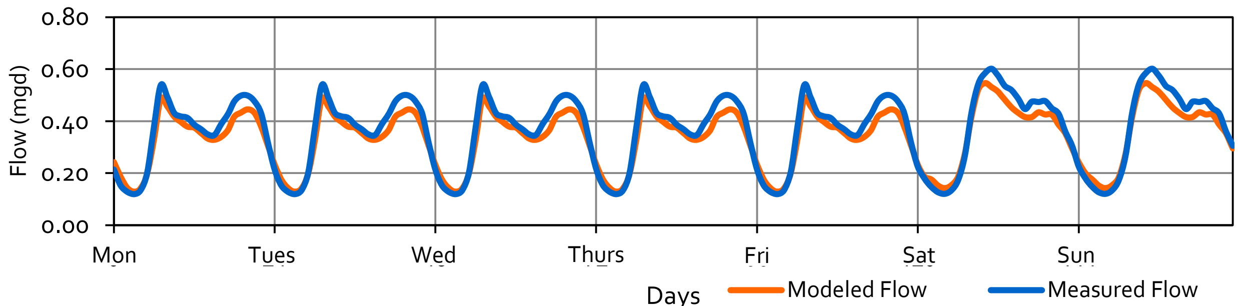
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.17	0.23	0.16	0.24	-4%	1%
Tues.	0.17	0.23	0.16	0.24	-4%	1%
Wed.	0.17	0.23	0.16	0.24	-4%	1%
Thur.	0.17	0.23	0.16	0.24	-4%	1%
Fri.	0.17	0.23	0.16	0.24	-4%	1%
Sat.	0.19	0.30	0.17	0.28	-7%	-6%
Sun.	0.19	0.30	0.18	0.28	-7%	-6%
Summary						
Weekday	0.172	--	0.164	--	-4%	--
Weekend	0.189	--	0.175	--	-7%	--
ADWF[4]	0.177	--	0.167	--	-5%	--

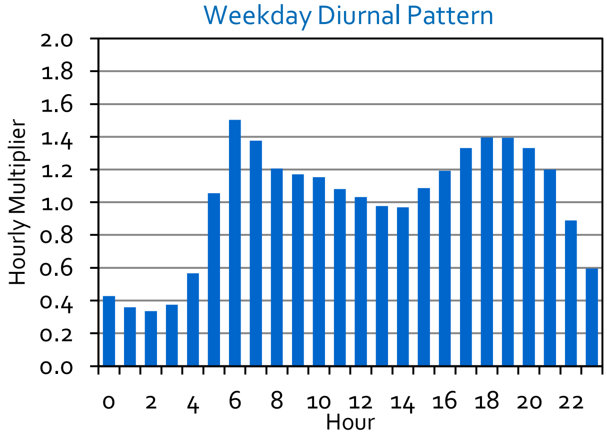
Notes:

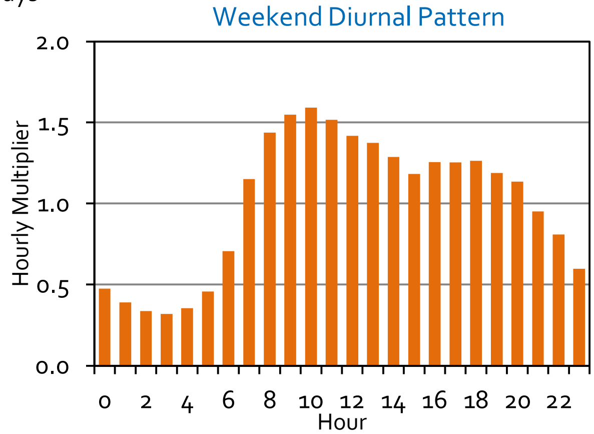
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M7, Dry Weather Flow Calibration

Location: Intersection of Orleans Drive and Zoe Court
Measured Pipeline Diameter: 15"
Model Manhole ID: MH_1093
Model Pipe ID: P_212







Model Calibration Summary

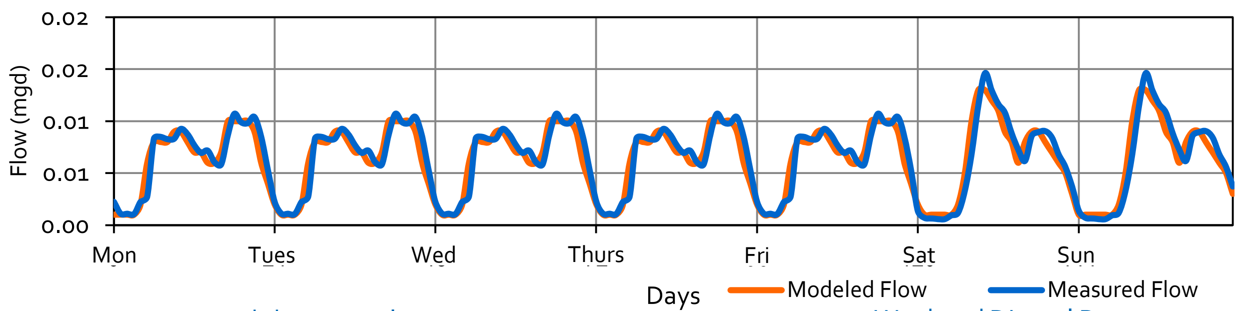
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.36	0.54	0.33	0.49	-7%	-10%
Tues.	0.36	0.54	0.33	0.49	-7%	-10%
Wed.	0.36	0.54	0.33	0.49	-7%	-10%
Thur.	0.36	0.54	0.33	0.49	-7%	-10%
Fri.	0.36	0.54	0.33	0.49	-7%	-10%
Sat.	0.38	0.60	0.36	0.55	-6%	-9%
Sun.	0.38	0.60	0.36	0.55	-5%	-9%
Summary						
Weekday	0.358	--	0.333	--	-7%	--
Weekend	0.378	--	0.357	--	-6%	--
ADWF[4]	0.364	--	0.340	--	-6%	--

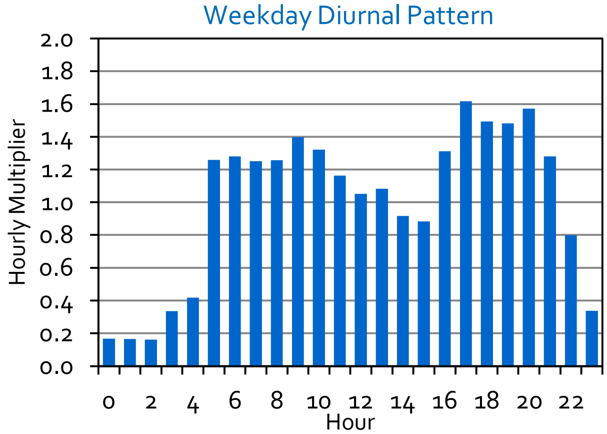
Notes:

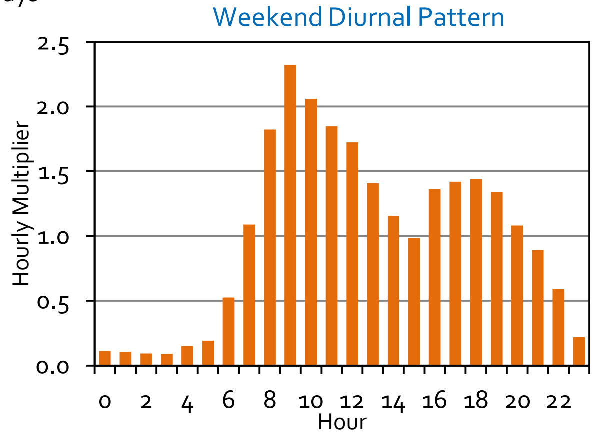
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M8, Dry Weather Flow Calibration

Location: Henry Avenue west of Pinole Valley Road
Measured Pipeline Diameter: 7.75"
Model Manhole ID: MH_230
Model Pipe ID: P_922







Model Calibration Summary

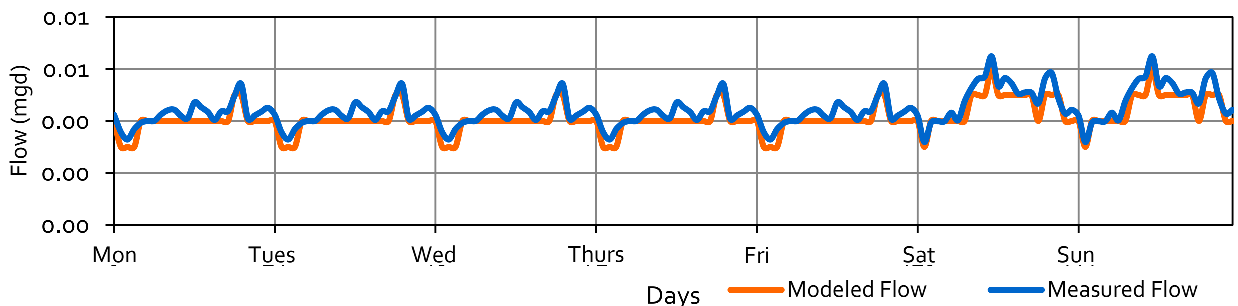
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.01	0.01	0.01	0.01	-3%	-7%
Tues.	0.01	0.01	0.01	0.01	-3%	-7%
Wed.	0.01	0.01	0.01	0.01	-3%	-7%
Thur.	0.01	0.01	0.01	0.01	-3%	-7%
Fri.	0.01	0.01	0.01	0.01	-2%	-7%
Sat.	0.01	0.01	0.01	0.01	0%	-11%
Sun.	0.01	0.01	0.01	0.01	-1%	-11%
Summary						
Weekday	0.007	--	0.006	--	-3%	--
Weekend	0.006	--	0.006	--	0%	--
ADWF[4]	0.007	--	0.006	--	-2%	--

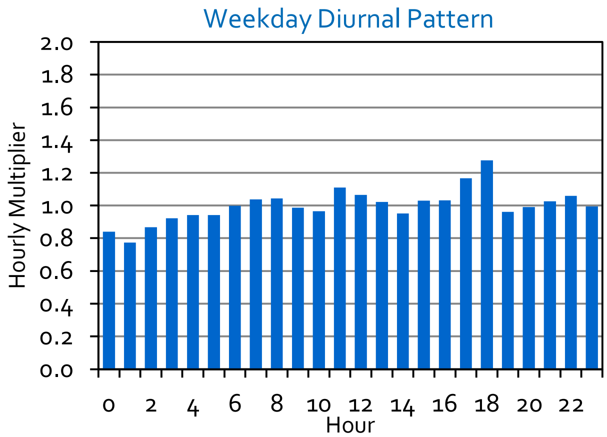
Notes:

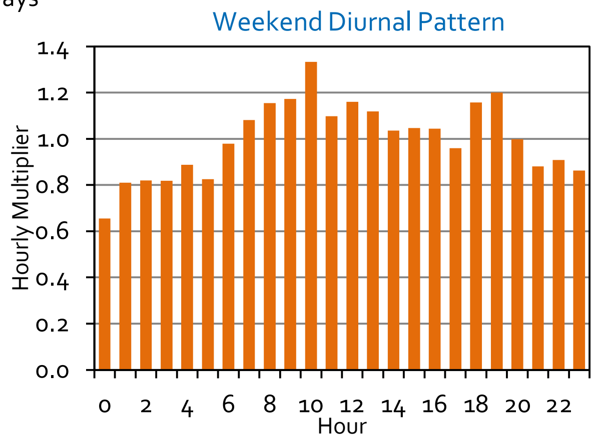
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M9, Dry Weather Flow Calibration

Location: Intersection of Henry Avenue and Pinole Valley Road
Measured Pipeline Diameter: 6"
Model Manhole ID: MH_234
Model Pipe ID: P_278







Model Calibration Summary

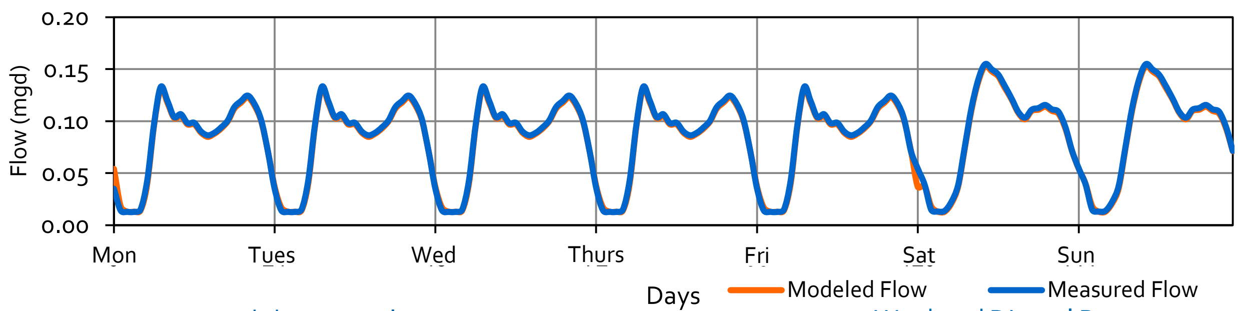
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.00	0.01	0.00	0.01	-7%	-8%
Tues.	0.00	0.01	0.00	0.01	-7%	-8%
Wed.	0.00	0.01	0.00	0.01	-7%	-8%
Thur.	0.00	0.01	0.00	0.01	-7%	-8%
Fri.	0.00	0.01	0.00	0.01	-7%	-8%
Sat.	0.00	0.01	0.00	0.01	-7%	-8%
Sun.	0.00	0.01	0.00	0.01	-7%	-8%
Summary						
Weekday	0.004	--	0.004	--	-7%	--
Weekend	0.005	--	0.005	--	-7%	--
ADWF[4]	0.004	--	0.004	--	-7%	--

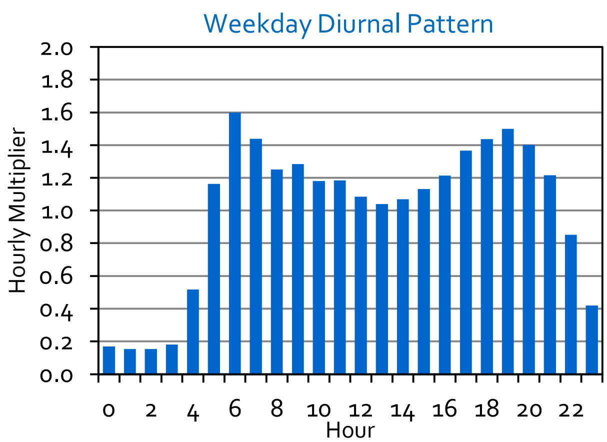
Notes:

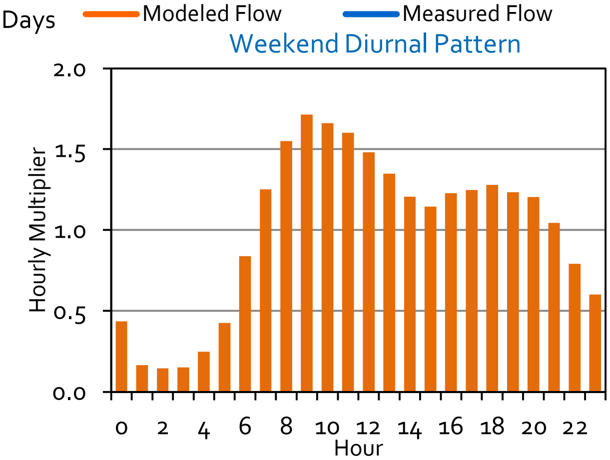
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M10, Dry Weather Flow Calibration

Location: Intersection of Tennent Avenue and Prune Street
Measured Pipeline Diameter: 8"
Model Manhole ID: MH_143
Model Pipe ID: P_355







Model Calibration Summary

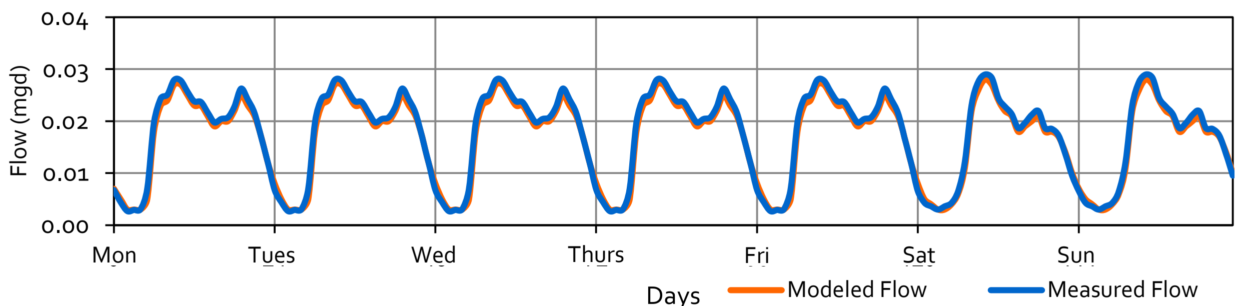
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.08	0.13	0.08	0.13	-1%	-1%
Tues.	0.08	0.13	0.08	0.13	-1%	-1%
Wed.	0.08	0.13	0.08	0.13	-1%	-1%
Thur.	0.08	0.13	0.08	0.13	-1%	-1%
Fri.	0.08	0.13	0.08	0.13	-1%	-1%
Sat.	0.09	0.15	0.09	0.15	-2%	-1%
Sun.	0.09	0.15	0.09	0.15	-1%	-1%
Summary						
Weekday	0.083	--	0.082	--	-1%	--
Weekend	0.090	--	0.089	--	-2%	--
ADWF[4]	0.085	--	0.084	--	-1%	--

Notes:

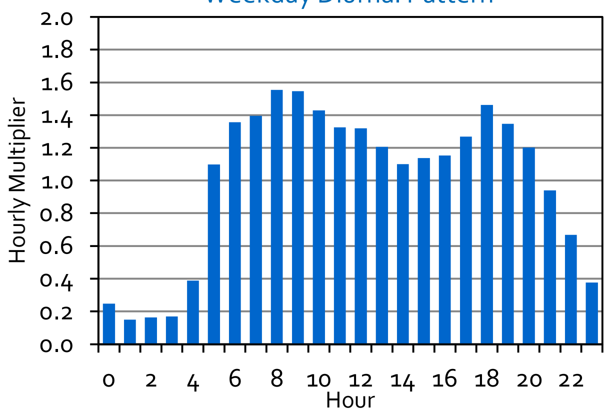
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M11, Dry Weather Flow Calibration

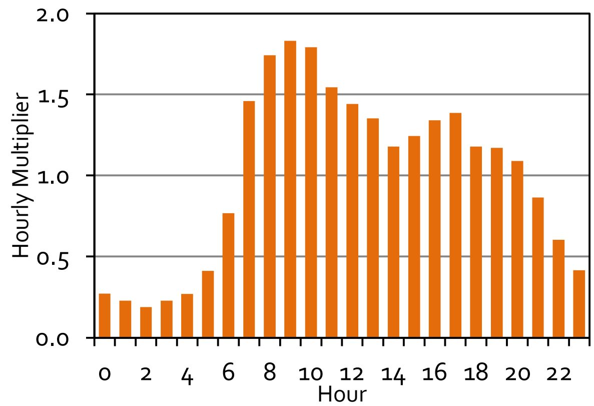
Location: Intersection of Pinole Valley Road and Rafaela Street
Measured Pipeline Diameter: 10"
Model Manhole ID: MH_1220
Model Pipe ID: P_233



Weekday Diurnal Pattern



Weekend Diurnal Pattern



Model Calibration Summary

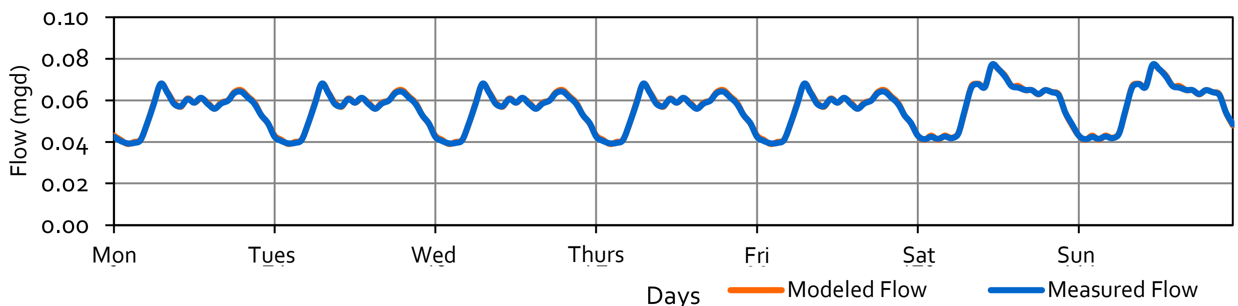
	Measured Data[1]		Modeled Data		Percent Error[3]	
Day	Avg. Flow (mgd)	Peak Flow[2] (mgd)	Avg. Flow (mgd)	Peak Flow[2] (mgd)	Avg. Flow (%)	Peak Flow (%)
Mon.	0.02	0.03	0.02	0.03	-3%	-3%
Tues.	0.02	0.03	0.02	0.03	-3%	-3%
Wed.	0.02	0.03	0.02	0.03	-3%	-3%
Thur.	0.02	0.03	0.02	0.03	-3%	-3%
Fri.	0.02	0.03	0.02	0.03	-3%	-3%
Sat.	0.02	0.03	0.02	0.03	-3%	-3%
Sun.	0.02	0.03	0.02	0.03	-3%	-3%
Summary						
Weekday	0.018	--	0.017	--	-3%	--
Weekend	0.016	--	0.015	--	-3%	--
ADWF[4]	0.017	--	0.017	--	-3%	--

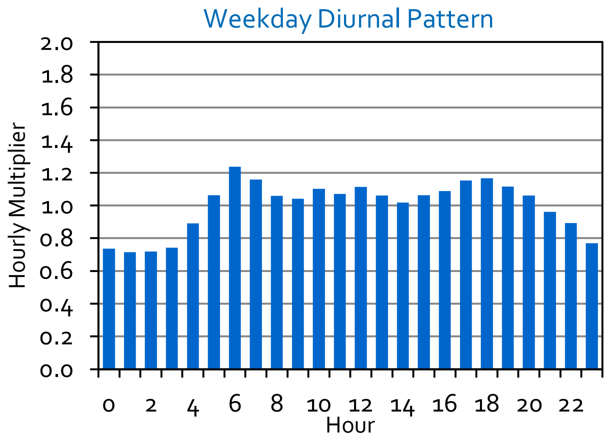
Notes:

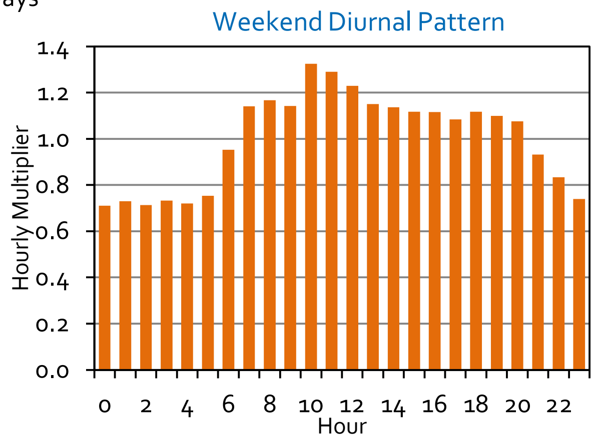
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M13, Dry Weather Flow Calibration

Location: San Pablo Avenue just west of Quinan Street
Measured Pipeline Diameter: 6"
Model Manhole ID: MH_1369
Model Pipe ID: P_351







Model Calibration Summary

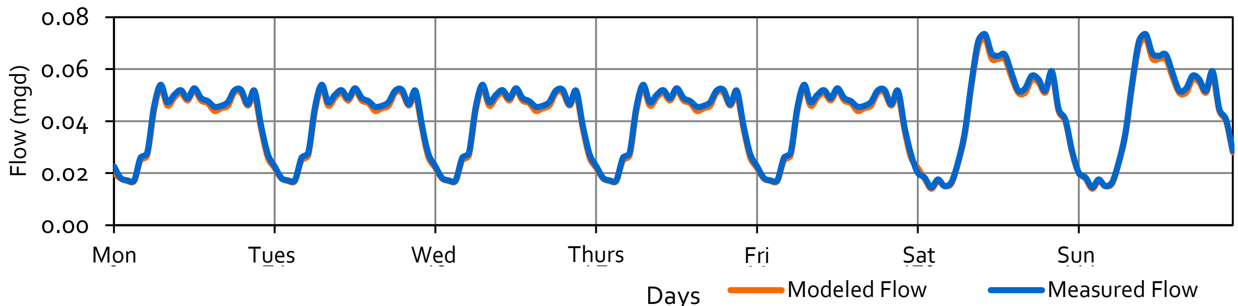
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.06	0.07	0.06	0.07	0%	0%
Tues.	0.06	0.07	0.06	0.07	0%	0%
Wed.	0.06	0.07	0.06	0.07	0%	0%
Thur.	0.06	0.07	0.06	0.07	0%	0%
Fri.	0.06	0.07	0.06	0.07	0%	0%
Sat.	0.06	0.08	0.06	0.08	0%	0%
Sun.	0.06	0.08	0.06	0.08	0%	0%
Summary						
Weekday	0.055	--	0.055	--	0%	--
Weekend	0.058	--	0.058	--	0%	--
ADWF[4]	0.056	--	0.056	--	0%	--

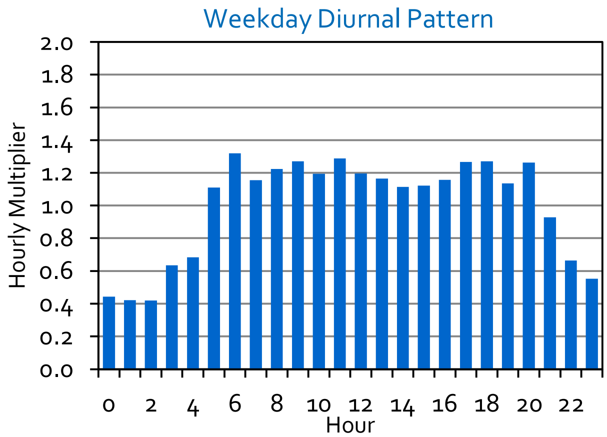
Notes:

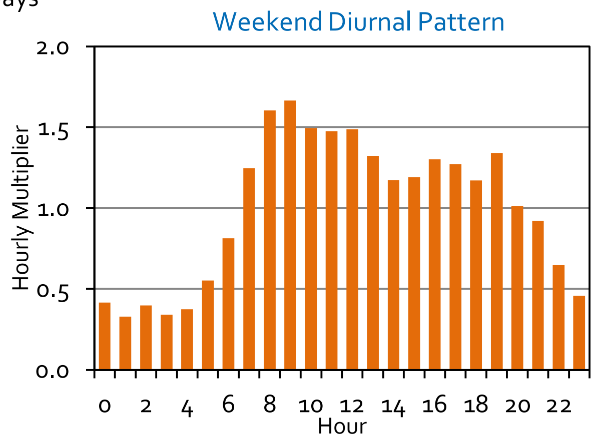
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M14, Dry Weather Flow Calibration

Location: Intersection of Tennent Avenue and Park Street
Measured Pipeline Diameter: 8"
Model Manhole ID: MH_1010
Model Pipe ID: P_998







Model Calibration Summary

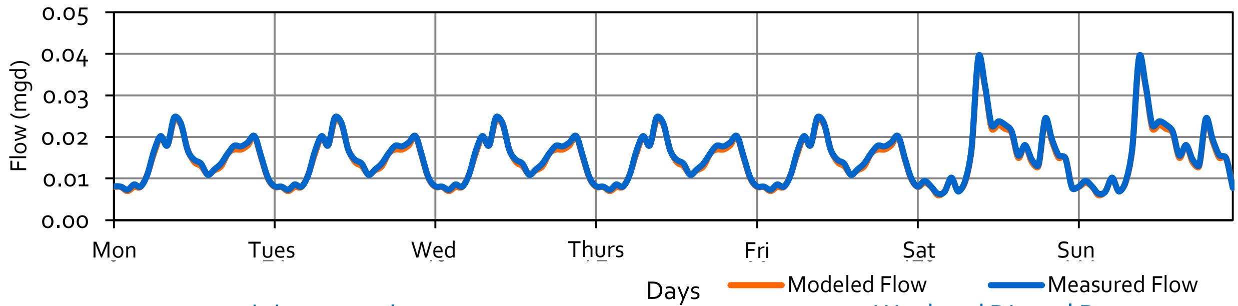
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.04	0.05	0.04	0.05	-2%	-2%
Tues.	0.04	0.05	0.04	0.05	-2%	-2%
Wed.	0.04	0.05	0.04	0.05	-2%	-2%
Thur.	0.04	0.05	0.04	0.05	-2%	-2%
Fri.	0.04	0.05	0.04	0.05	-2%	-2%
Sat.	0.04	0.07	0.04	0.07	-2%	-2%
Sun.	0.04	0.07	0.04	0.07	-2%	-2%
Summary						
Weekday	0.041	--	0.040	--	-2%	--
Weekend	0.044	--	0.043	--	-2%	--
ADWF[4]	0.042	--	0.041	--	-2%	--

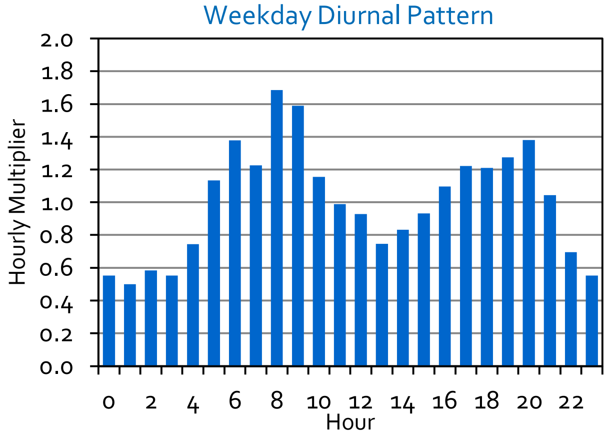
Notes:

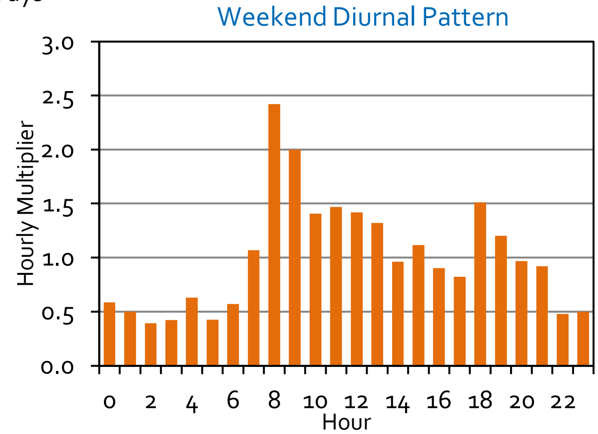
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M15, Dry Weather Flow Calibration

Location: Tennent Avenue south of the train tracks west of Fernandez Park
Measured Pipeline Diameter: 6"
Model Manhole ID: MH_1012
Model Pipe ID: P_885







Model Calibration Summary

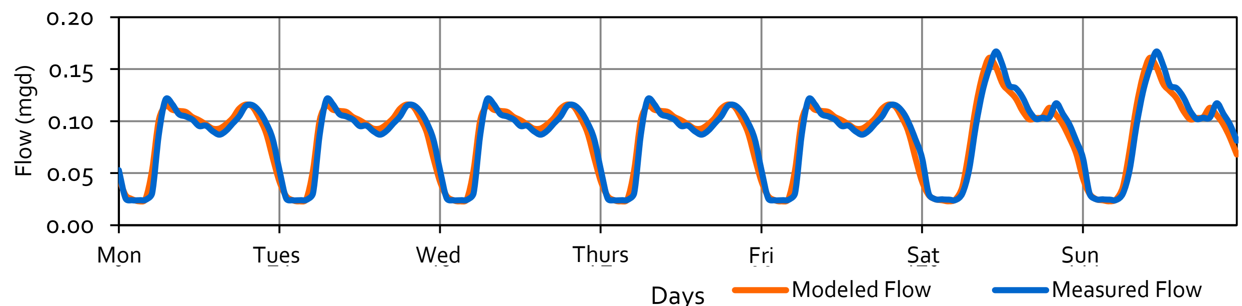
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.01	0.02	0.01	0.02	-2%	-3%
Tues.	0.01	0.02	0.01	0.02	-2%	-3%
Wed.	0.01	0.02	0.01	0.02	-2%	-3%
Thur.	0.01	0.02	0.01	0.02	-2%	-3%
Fri.	0.01	0.02	0.01	0.02	-2%	-3%
Sat.	0.02	0.04	0.02	0.04	-2%	-3%
Sun.	0.02	0.04	0.02	0.04	-2%	-3%
Summary						
Weekday	0.015	--	0.014	--	-2%	--
Weekend	0.016	--	0.016	--	-2%	--
ADWF[4]	0.015	--	0.015	--	-2%	--

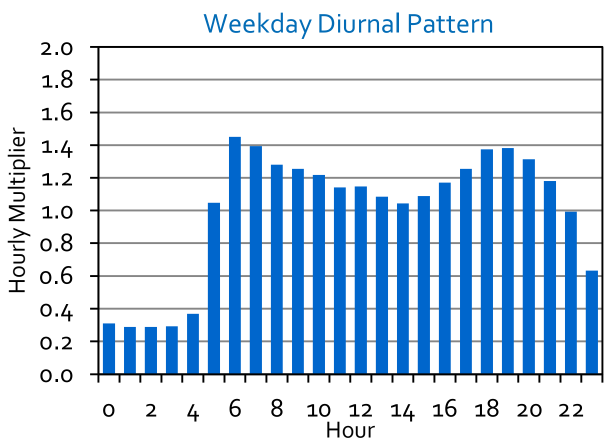
Notes:

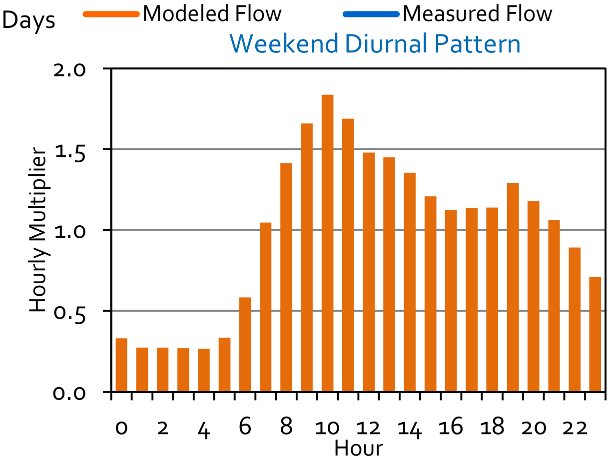
1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Flow Monitoring Site M16, Dry Weather Flow Calibration

Location: Tennent Avenue north of Orleans Drive
Measured Pipeline Diameter: 11.5"
Model Manhole ID: MH_1107
Model Pipe ID: P_986







Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)
Mon.	0.08	0.12	0.08	0.12	1%	-4%
Tues.	0.08	0.12	0.08	0.12	1%	-4%
Wed.	0.08	0.12	0.08	0.12	1%	-4%
Thur.	0.08	0.12	0.08	0.12	1%	-4%
Fri.	0.08	0.12	0.08	0.12	1%	-4%
Sat.	0.09	0.17	0.09	0.16	-3%	-4%
Sun.	0.09	0.17	0.09	0.16	-3%	-4%
Summary						
Weekday	0.083	--	0.085	--	1%	--
Weekend	0.091	--	0.089	--	-3%	--
ADWF[4]	0.086	--	0.086	--	0%	--

Notes:

1. Source: V&A Temporary Flow Monitoring Program
2. Peak flow is the hourly average hourly peak flow, which was derived based on the 15-minute flow data from V&A.
3. Percent Error = (Modeled - Measured) /Measured x 100
4. ADWF = (5xWeekday Average + 2xWeekend Average)/7

Appendix C

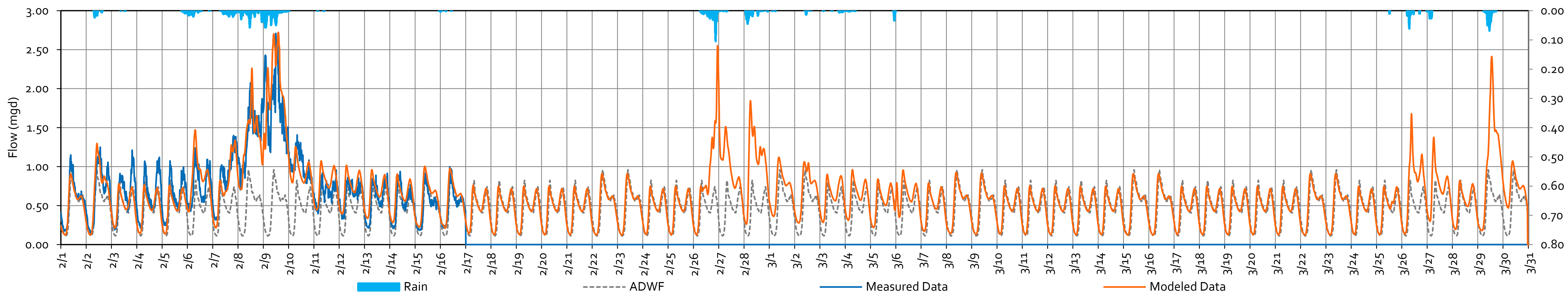
WET WEATHER FLOW CALIBRATION SHEETS

M1 Wet Weather Calibration

Location: Pinole valley Road just south of Highway 80

Measured Pipeline Diameter:15"

Model Pipe ID:P_691

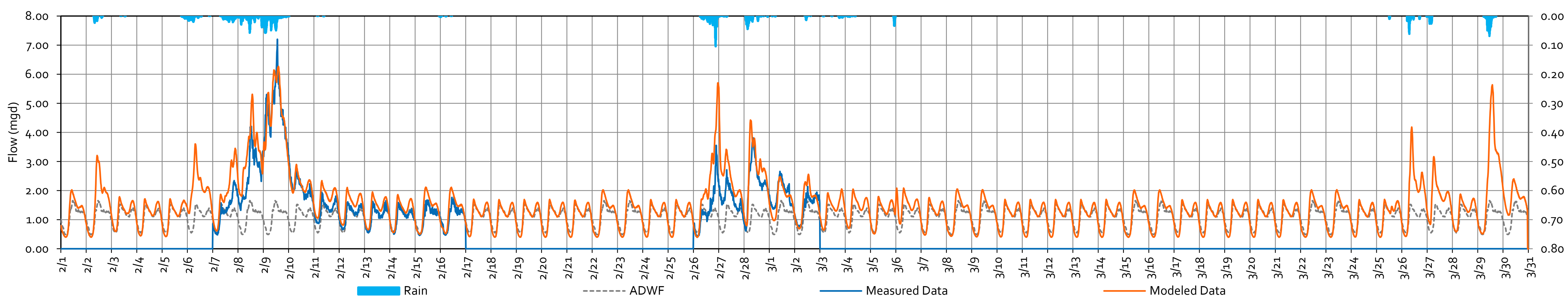


M2 Wet Weather Calibration

Location: Tennent Avenue just outside WPCP

Measured Pipeline Diameter:30"

Model Pipe ID: P_868

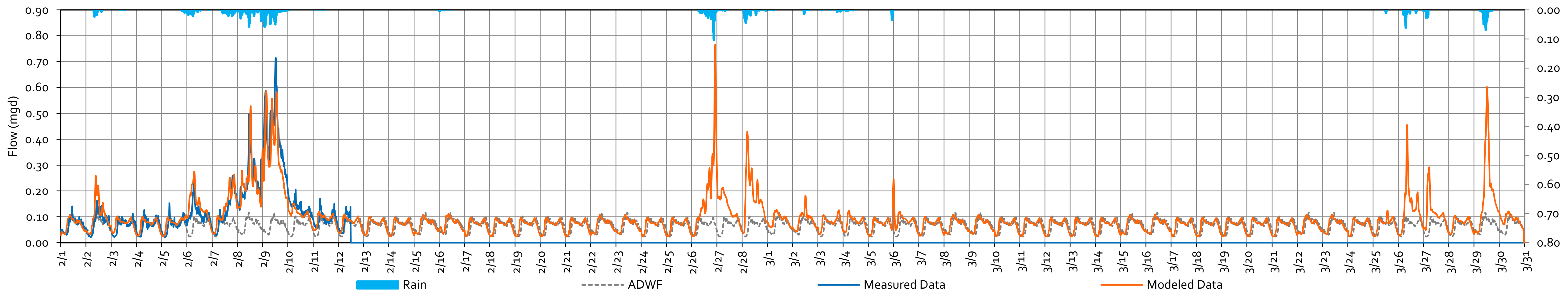


M3 Wet Weather Calibration

Location: San Pablo Lift Station (San Pablo Avenue west of Sunnyview Drive)

Measured Pipeline Diameter: N/A

Model Pipe ID: SAN_PABLO

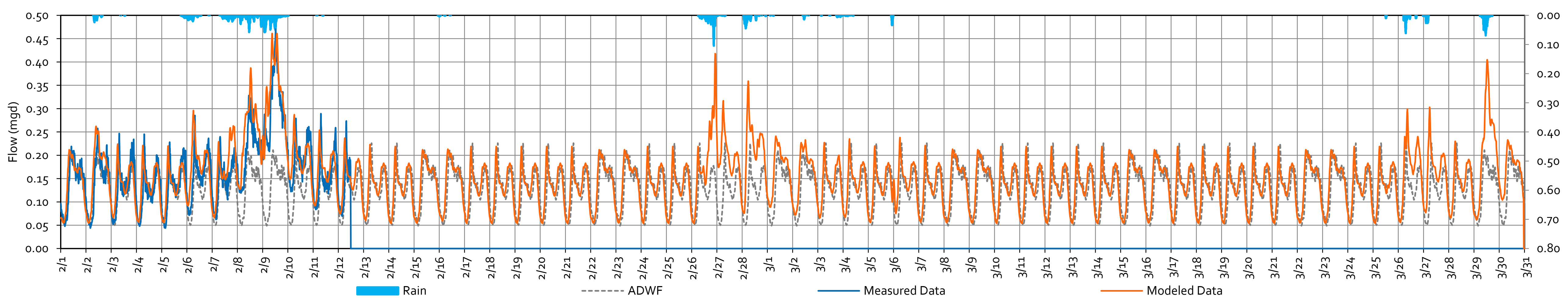


M4 Wet Weather Calibration

Location: Hazel Lift Station (In easement at west end of Hazel Street)

Measured Pipeline Diameter: N/A

Model Pipe ID: MH_1175

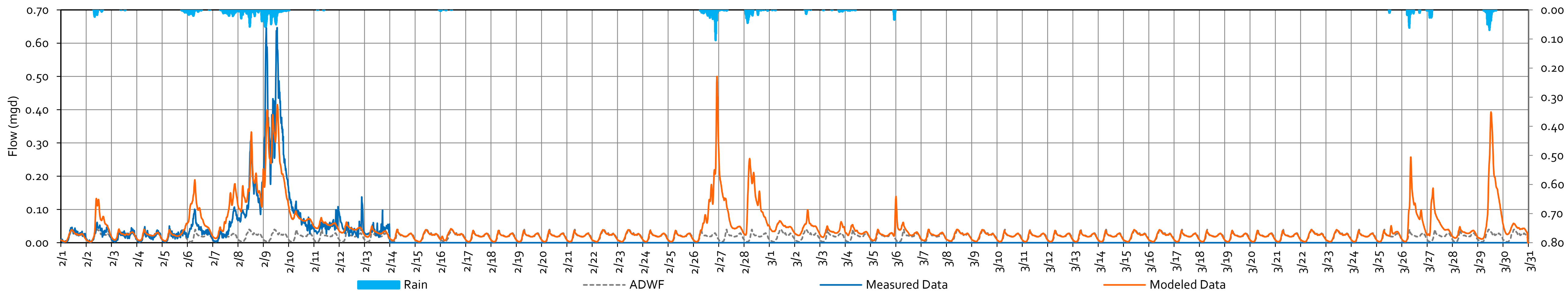


M5 Wet Weather Calibration

Location: Appian Way south of San Pablo Avenue

Measured Pipeline Diameter: 7.25"

Model Pipe ID: P_101

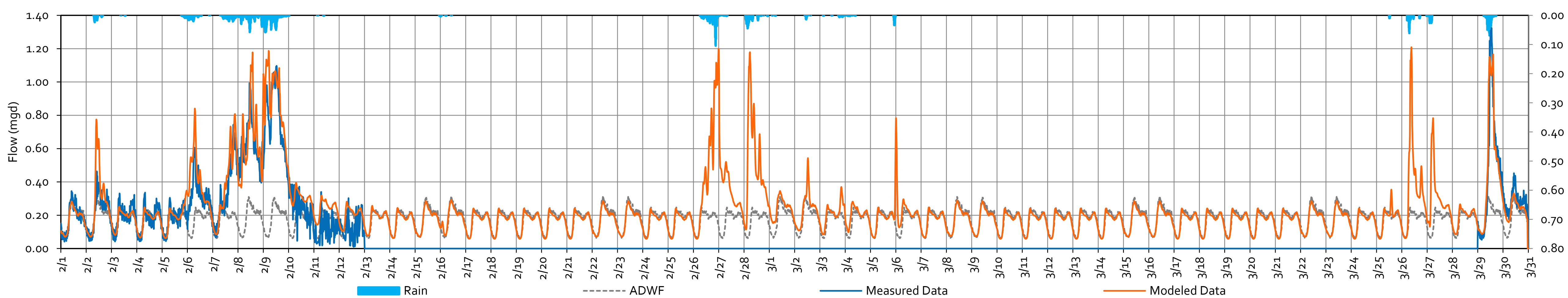


M6 Wet Weather Calibration

Location: Pinon Avenue north of Bay View Farm Road

Measured Pipeline Diameter: 10"

Model Pipe ID: P_755

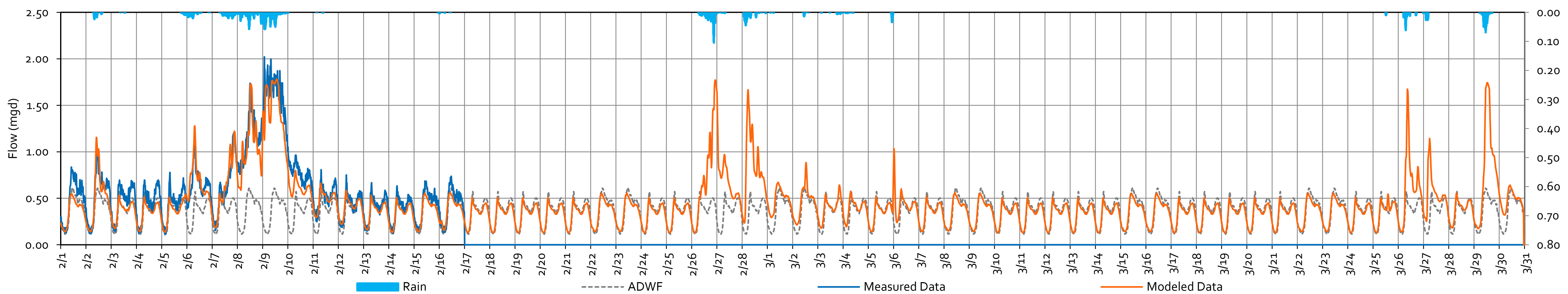


M7 Wet Weather Calibration

Location: Intersection of Orleans Drive and Zoe Court

Measured Pipeline Diameter: 15"

Model Pipe ID: P_212

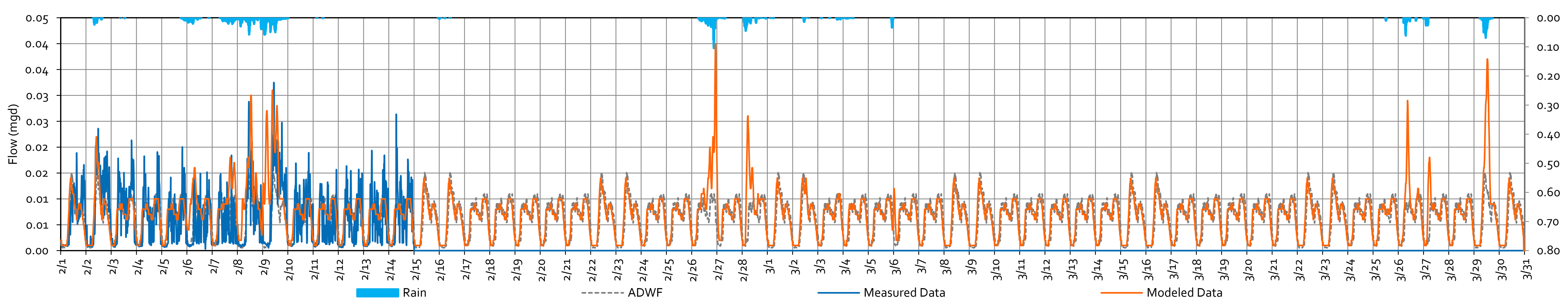


M8 Wet Weather Calibration

Location: Henry Avenue west of Pinole Valley Road

Measured Pipeline Diameter: 7.75

Model Pipe ID: P_922

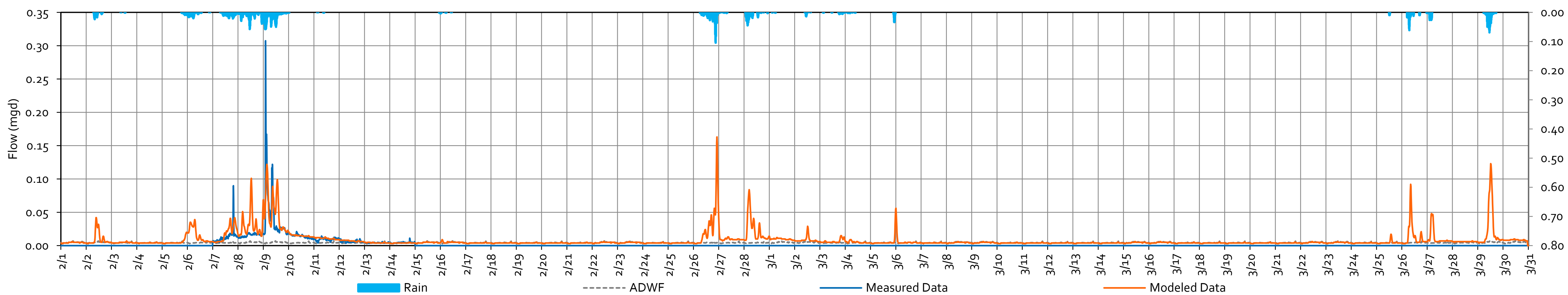


M9 Wet Weather Calibration

Location: Intersection of Henry Avenue and Pinole Valley Road

Measured Pipeline Diameter: 6"

Model Pipe ID: P_278

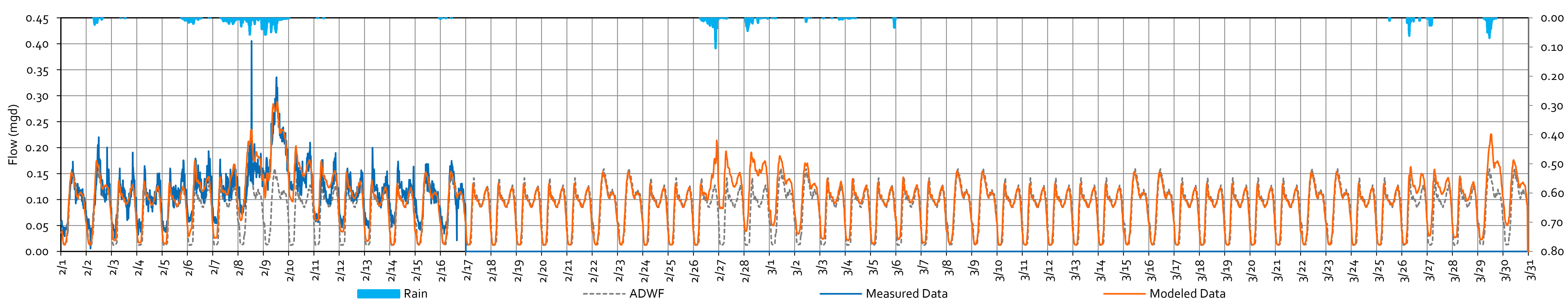


M10 Wet Weather Calibration

Location: Intersection of Tennent Avenue and Prune Street

Measured Pipeline Diameter: 8"

Model Pipe ID: P_355

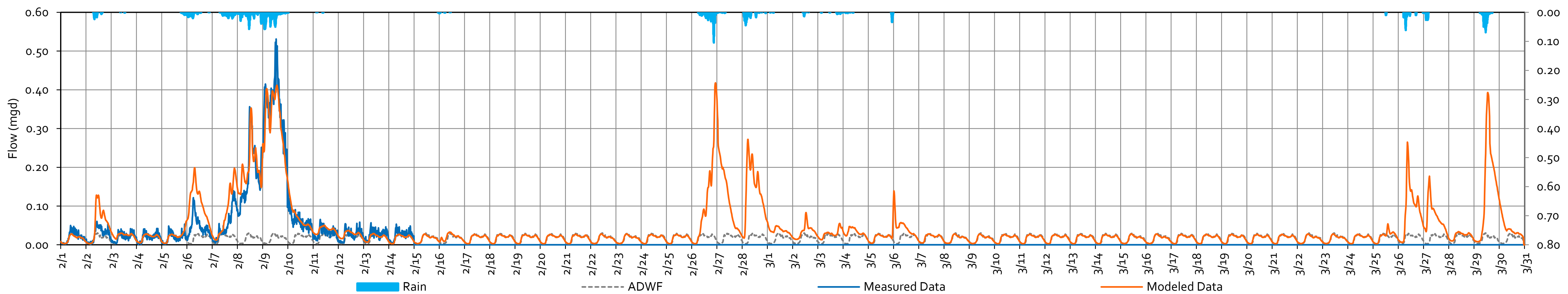


M11 Wet Weather Calibration

Location: Intersection of Pinole Valley Road and Rafaela Street

Measured Pipeline Diameter: 10"

Model Pipe ID: P_233

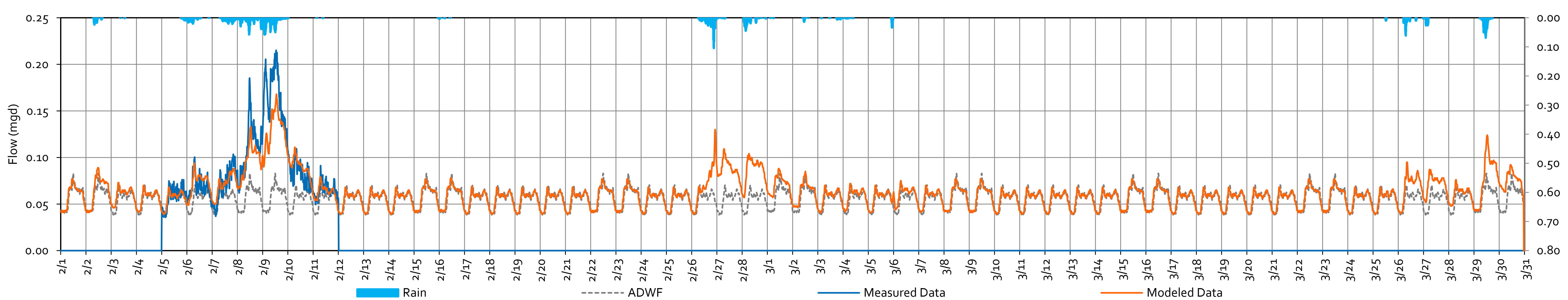


M13 Wet Weather Calibration

Location: San Pablo Avenue just west of Quinan Street

Measured Pipeline Diameter: 6"

Model Pipe ID: P_351

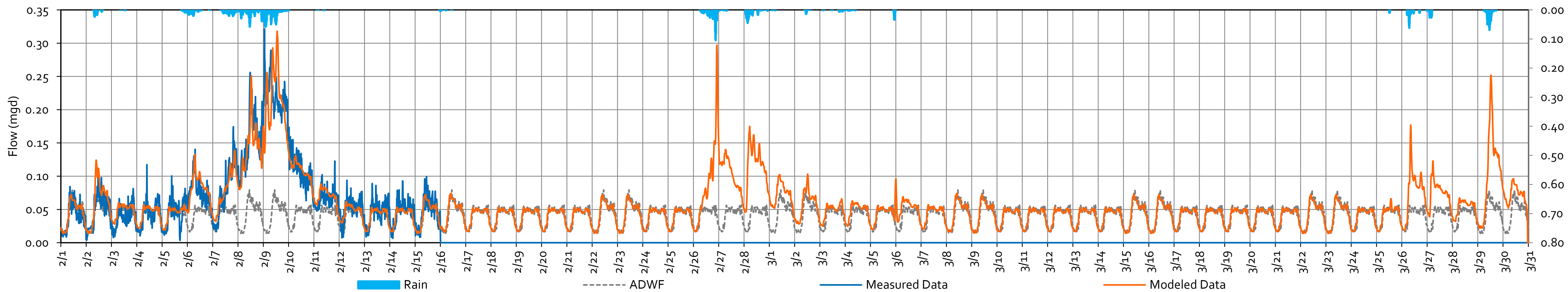


M14 Wet Weather Calibration

Location: Intersection of Tennent Avenue and Park Street

Measured Pipeline Diameter: 8"

Model Pipe ID: P_998

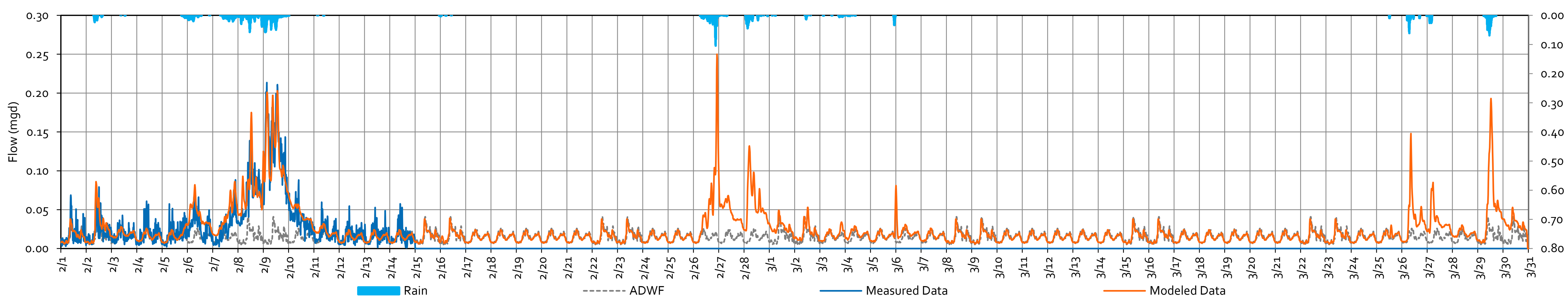


M15 Wet Weather Calibration

Location: Tennent Avenue south of the train tracks west of Fernandez Park

Measured Pipeline Diameter: 6"

Model Pipe ID: P_885

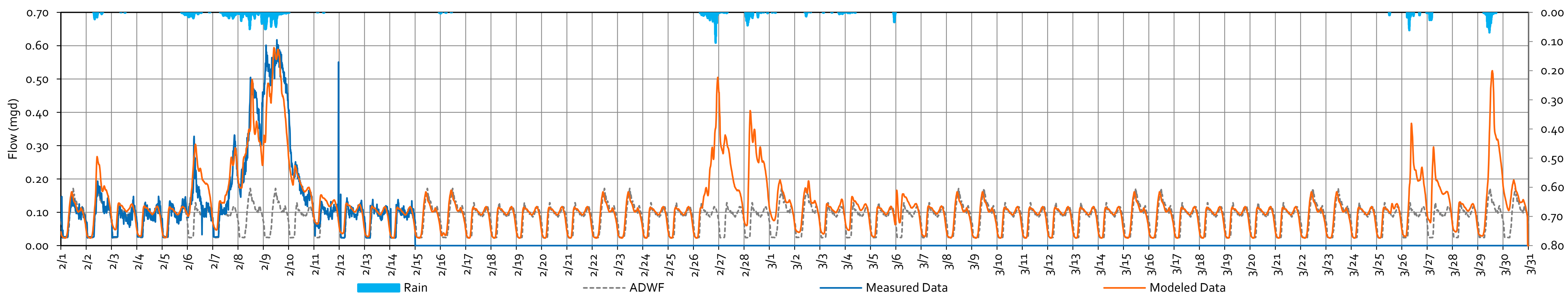


M16 Wet Weather Calibration

Location: Tennent Avenue north of Orleans Drive

Measured Pipeline Diameter: 11.5"

Model Pipe ID: P_986

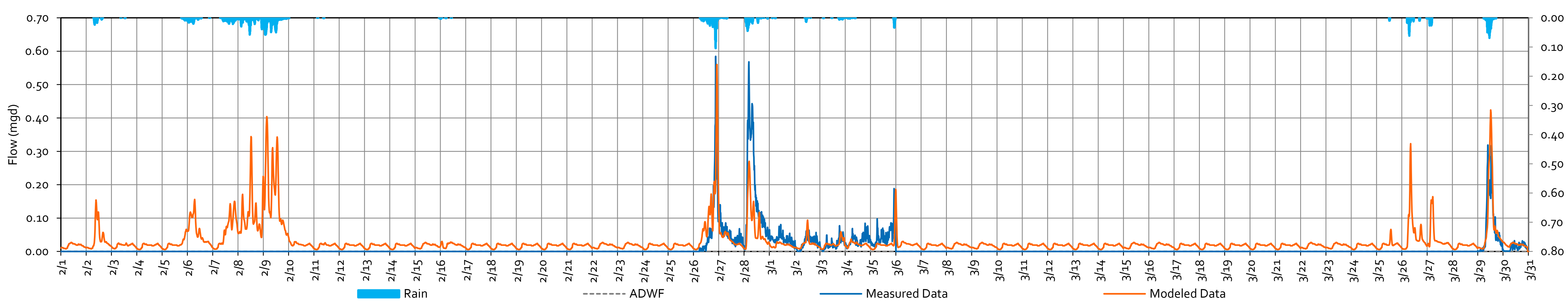


M3.1 Wet Weather Calibration

Location: 830 Meadows Avenue

Measured Pipeline Diameter: 6"

Model Pipe ID: P_60

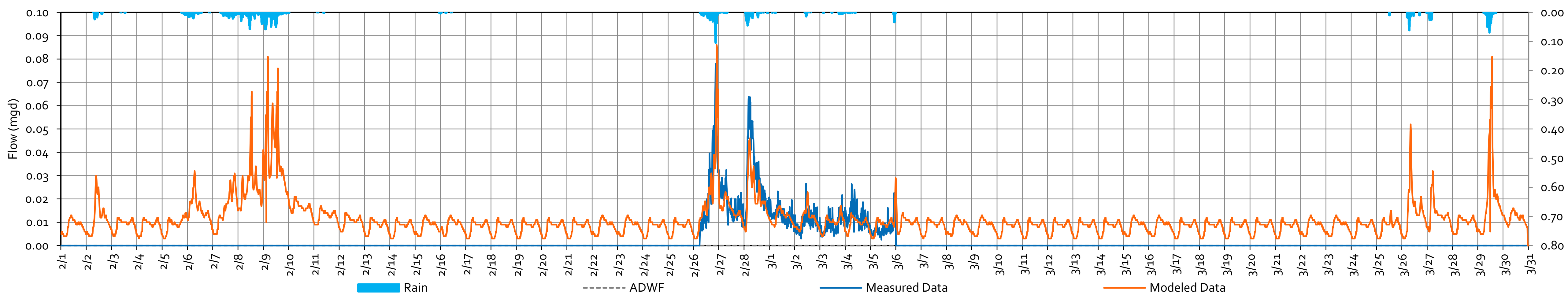


M3.2 Wet Weather Calibration

Location: 830 Meadows Avenue

Measured Pipeline Diameter: 6”

Model Pipe ID: P_61

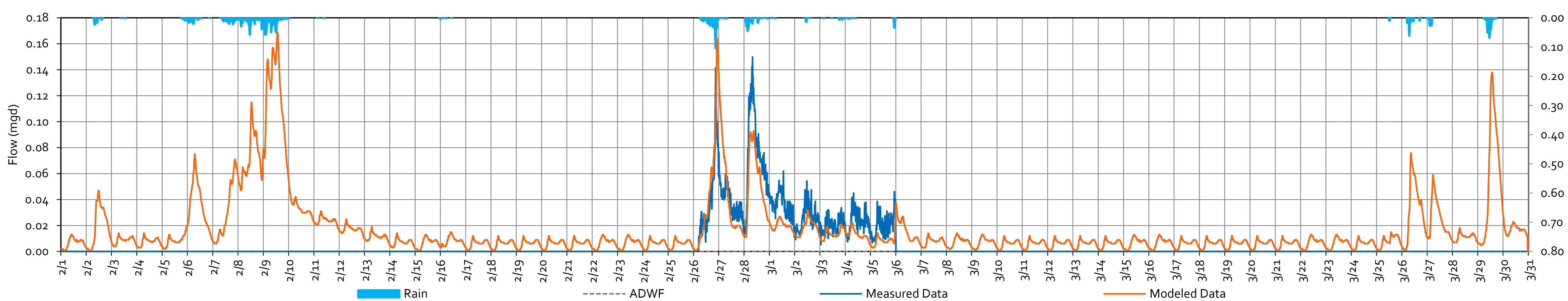


M5.1 Wet Weather Calibration

Location: Intersection of Appian Way and Marlesta Road

Measured Pipeline Diameter: 8”

Model Pipe ID: P_126

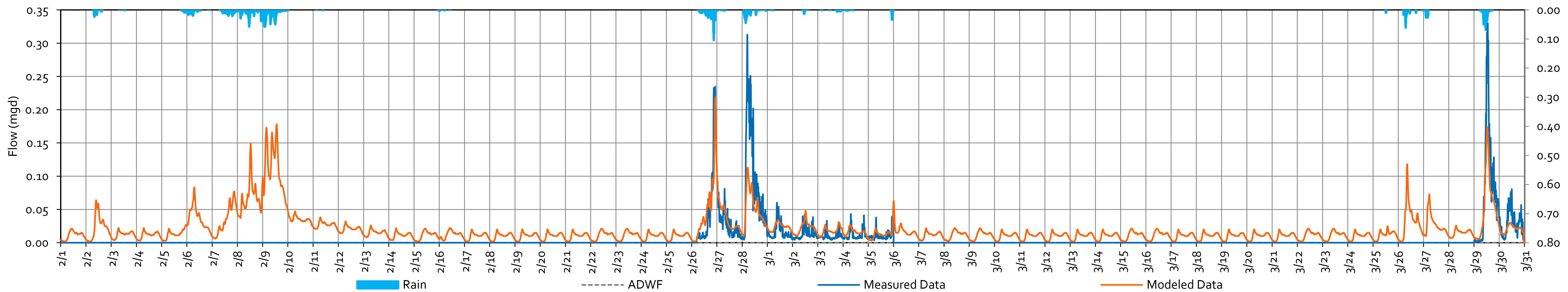


M5.2 Wet Weather Calibration

Location: Intersection of Appian Way and Marlesta Road

Measured Pipeline Diameter: 8"

Model Pipe ID: P_128

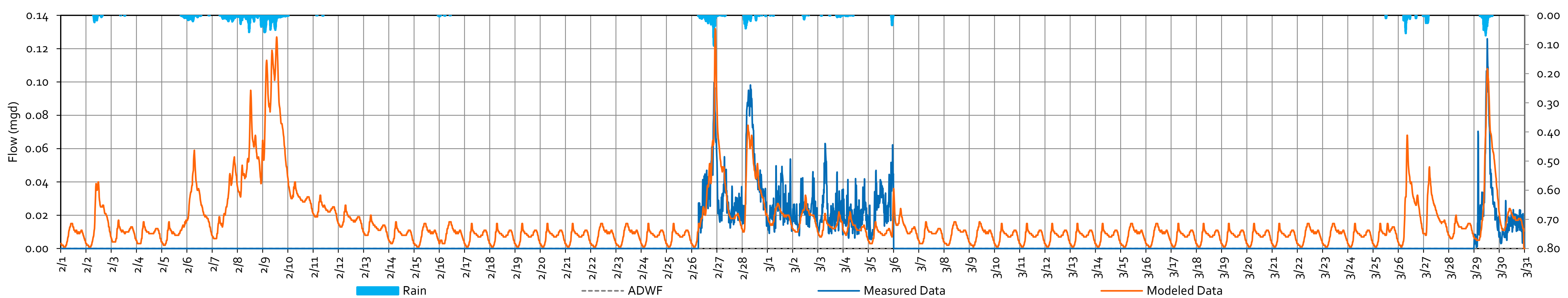


M5.3 Wet Weather Calibration

Location: 1171 Marlesta Road

Measured Pipeline Diameter: 6"

Model Pipe ID: P_5

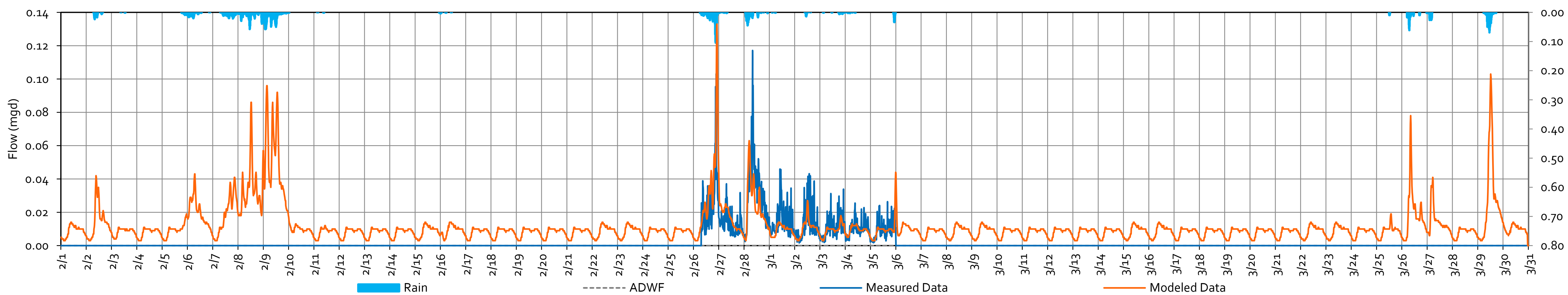


M6.1 Wet Weather Calibration

Location: Just west of the intersection of Bay View Farm Road and Pinon Avenel

Measured Pipeline Diameter: 6"

Model Pipe ID: P_219

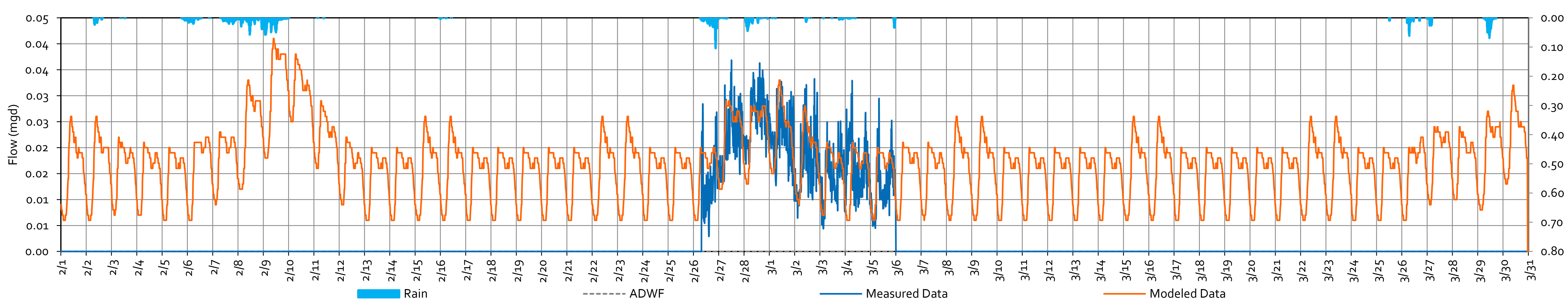


M6.2 Wet Weather Calibration

Location: Intersection of Pinon Avenue and Primrose Lane

Measured Pipeline Diameter: 8"

Model Pipe ID: P_243

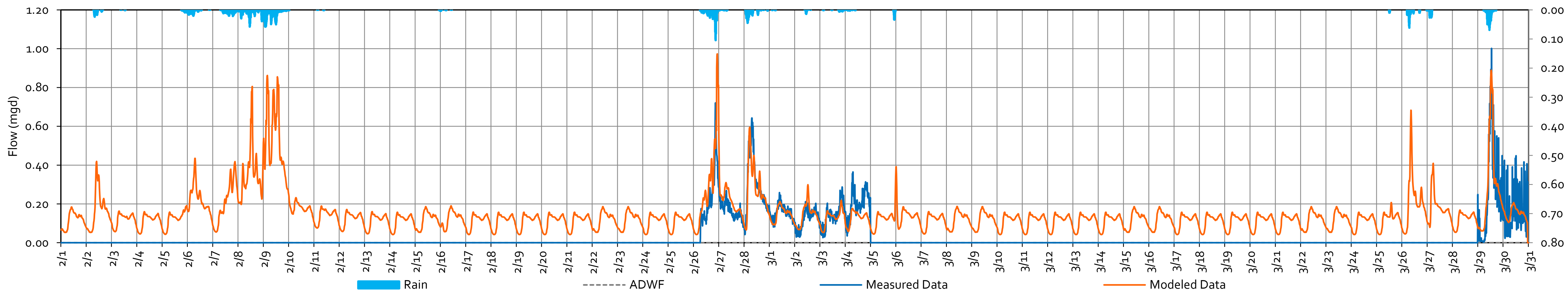


M6.3 Wet Weather Calibration

Location: Roble Avenue west of Pinon Avenue

Measured Pipeline Diameter: 8"

Model Pipe ID: P_88

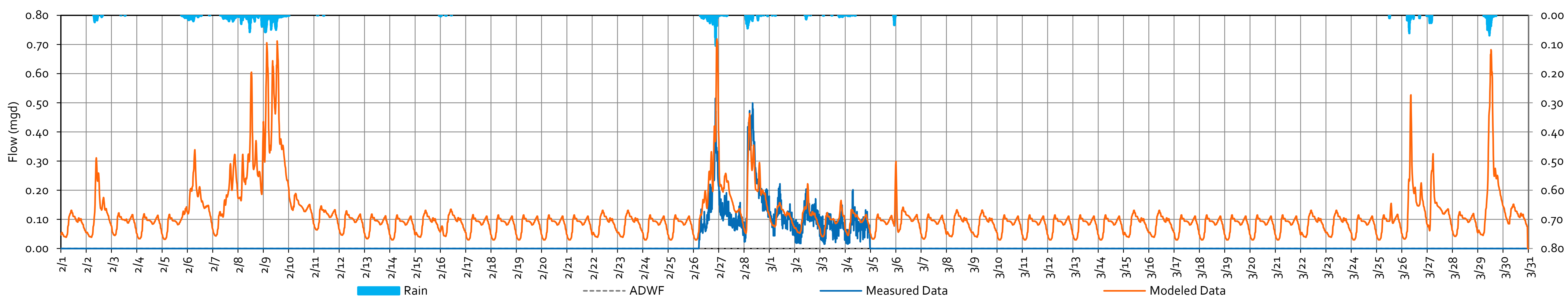


M6.4 Wet Weather Calibration

Location: Intersection of San Pablo Avenue and Rogers Way

Measured Pipeline Diameter: 8"

Model Pipe ID: P_32

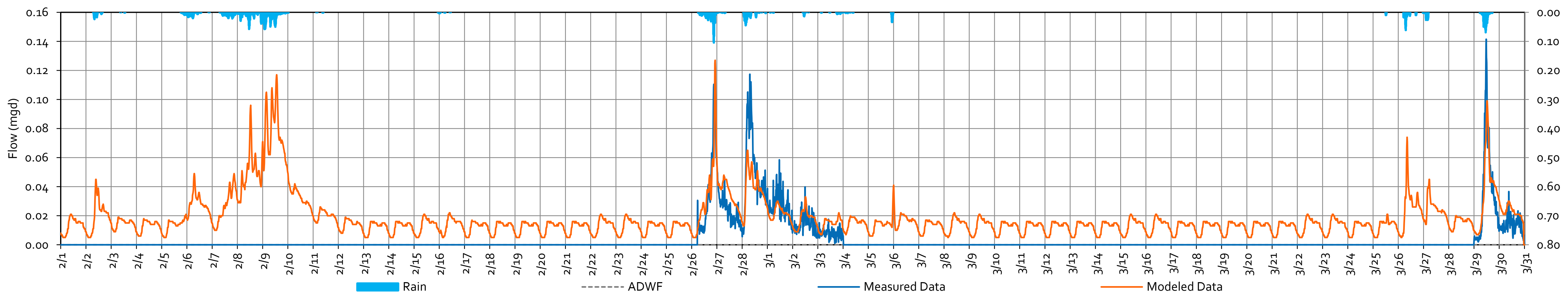


M6.5 Wet Weather Calibration

Location: 747 Sunnyview Drive

Measured Pipeline Diameter: 8"

Model Pipe ID: P_28

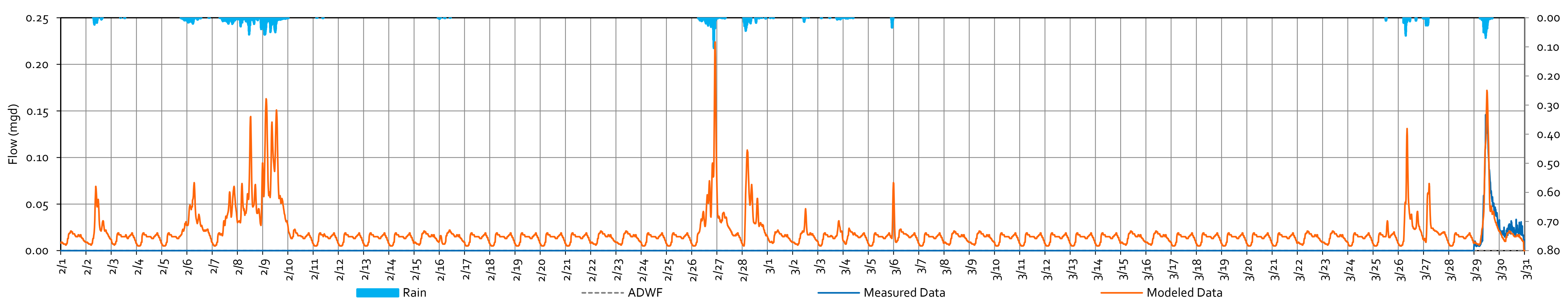


M3.1A Wet Weather Calibration

Location: Intersection of Meadow Avenue and Betty Avenue

Measured Pipeline Diameter: 6"

Model Pipe ID: P_57

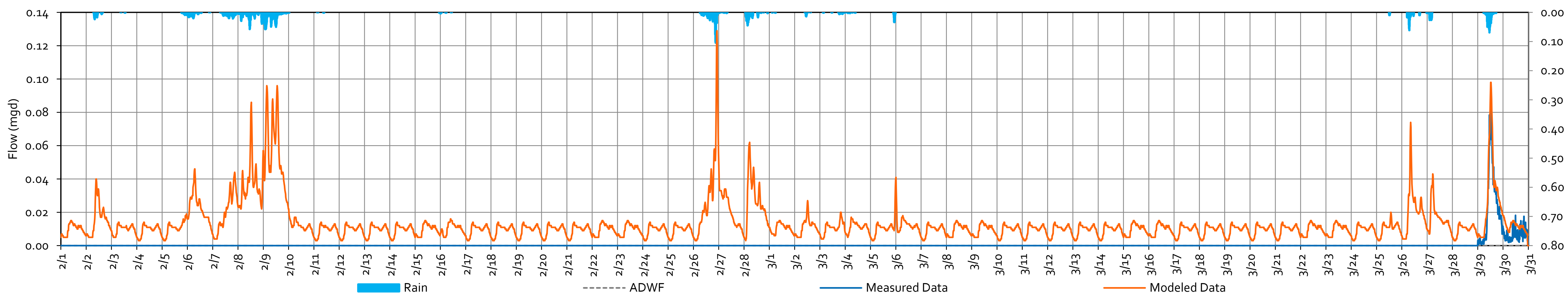


M3.1B Wet Weather Calibration

Location: Intersection of Meadow Avenue and Nob Hill Avenue

Measured Pipeline Diameter: 6”

Model Pipe ID: P_53

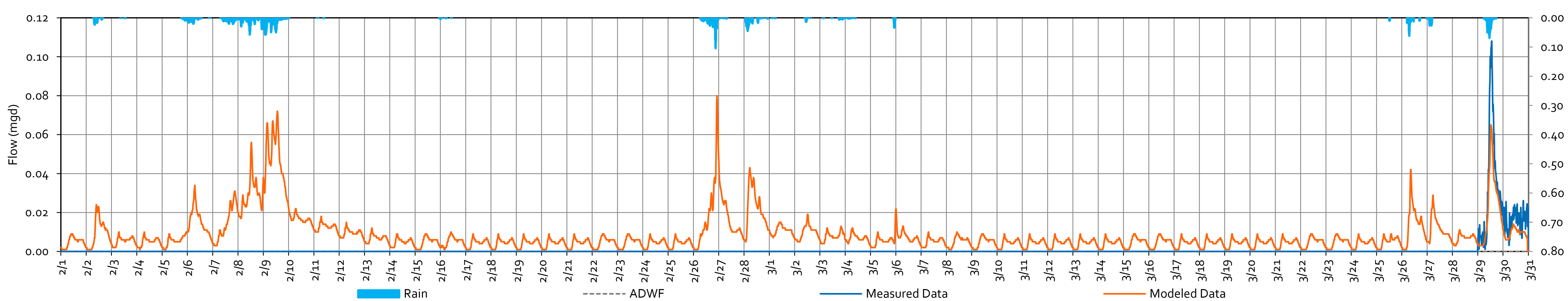


M5.2A Wet Weather Calibration

Location: 1367 Marlesta Road

Measured Pipeline Diameter: 6”

Model Pipe ID: P_40

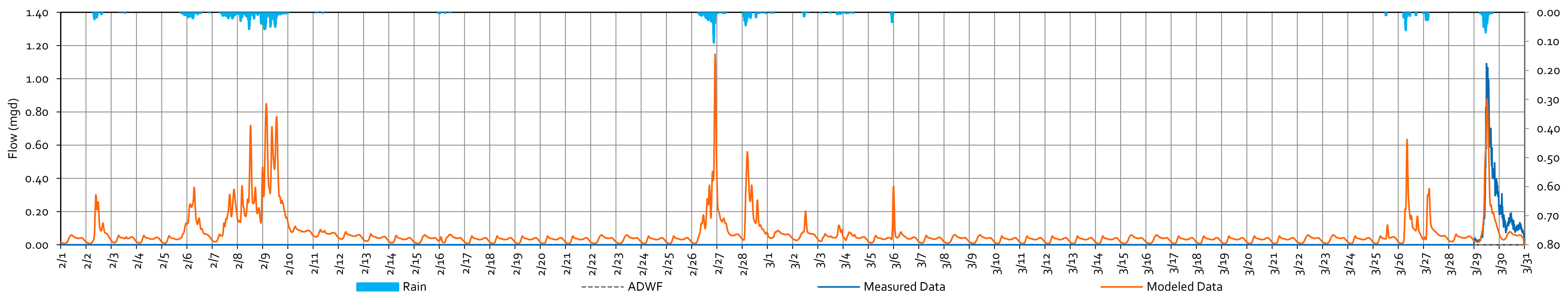


M6.0A Wet Weather Calibration

Location: Intersection of Roble Avenue and Pinon Avenue

Measured Pipeline Diameter: 10"

Model Pipe ID: P_520

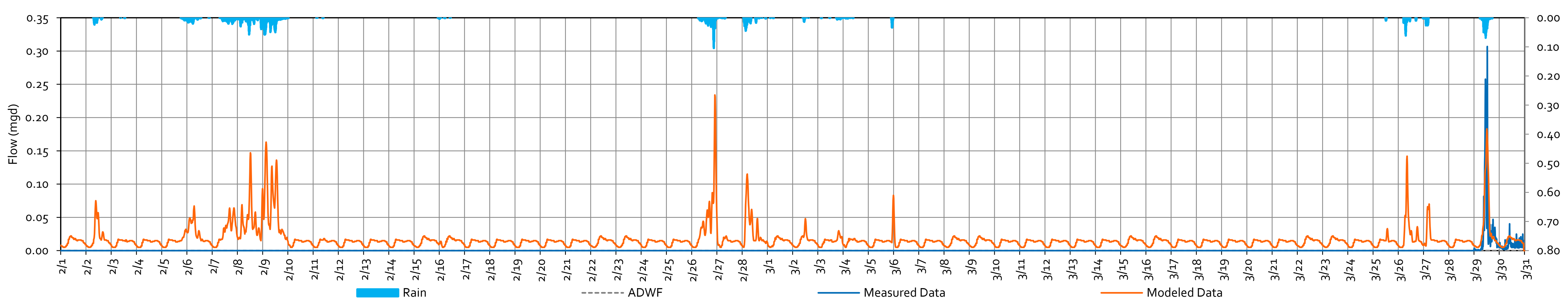


M6.3A Wet Weather Calibration

Location: Intersection of San Pablo Avenue and 5th Avenue

Measured Pipeline Diameter: 6"

Model Pipe ID: P_1

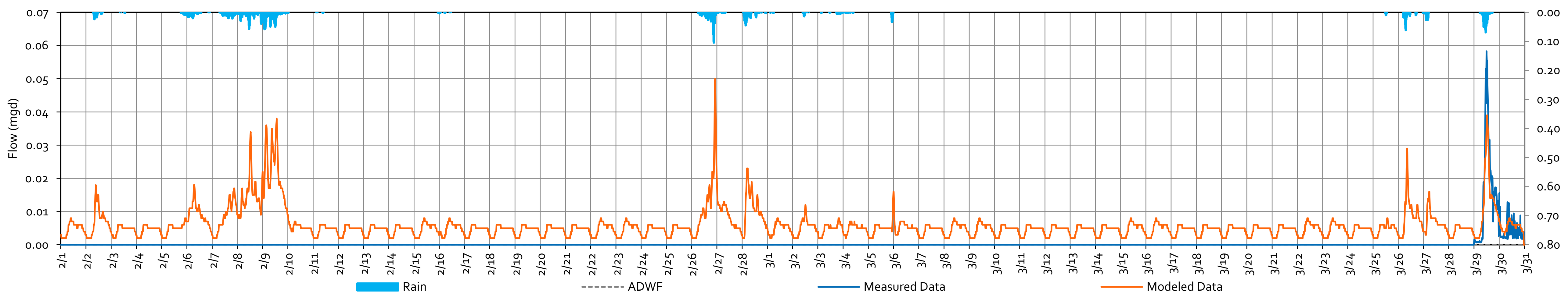


M6.3B Wet Weather Calibration

Location: Intersection of San Pablo Avenue and Roble Avenue

Measured Pipeline Diameter: 8"

Model Pipe ID: P_195

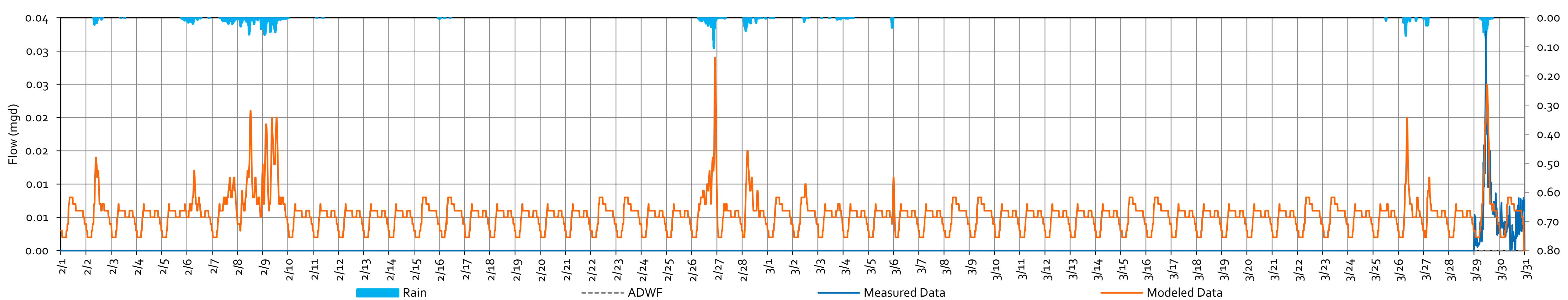


M6.5A Wet Weather Calibration

Location: Intersection of Sunnyview Drive and Patrick Drive

Measured Pipeline Diameter: 7.75"

Model Pipe ID: P_27

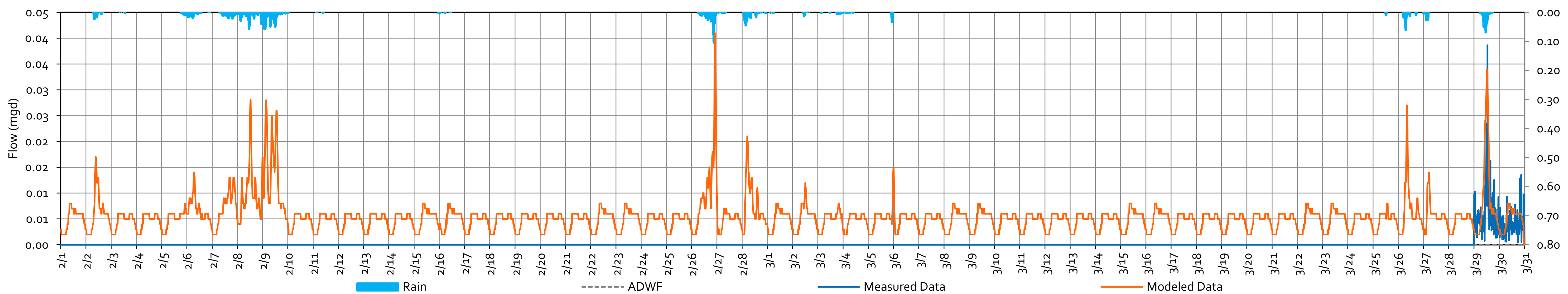


M6.5B Wet Weather Calibration

Location: Intersection of Sunnyview Drive and Nob Hill Avenue

Measured Pipeline Diameter: 7.75"

Model Pipe ID: P_20



Appendix D

CIP DETAIL SHEETS

Project Number:	Pinon-1
Project Name:	Gravity Main along Pinon Ave, Orleans Ave, Roble Ave, and San Pablo Ave
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,050 feet of 8-inch diameter pipeline along San Pablo Avenue, approximately 740 feet of 8-inch diameter pipeline along Roble Avenue, approximately 1,500 feet of 8-inch to 10-inch diameter pipeline along Pinon Avenue, approximately 520 feet of 12-inch diameter pipeline between Pinon Avenue and Orleans Avenue and approximately 1,160 feet of 8-inch to 15-inch diameter pipeline along Orleans Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 12-inch to

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	8-15	12-24	Replace	4,970	#N/A	$ 2,419,000	$ 3,145,000	$ 4,482,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	97%	$ 4,339,000
Future Users	3%	$ 143,000
Total	100%	$ 4,482,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	Pinon-2
Project Name:	Gravity Main along San Pablo Ave, Pinon Ave, Appian Way, and Meadow Ave
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 820 feet of 6-inch to 10-inch diameter pipeline along San Pablo Avenue, approximately 680 feet of 8-inch to 10-inch diameter pipeline along Pinon Avenue, approximately 890 feet of 6-inch to 8-inch diameter pipeline along Appian Way, approximately 290 feet of 6-inch diameter pipeline along Meadow Avenue, and approximately 290 feet of 6-inch diameter pipeline between Meadow Avenue and San Pablo Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	6-10	10-15	Replace	2,970	#N/A	$ 1,007,000	$ 1,310,000	$ 1,866,000	2023

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 1,858,000
Future Users	0%	$ 8,000
Total	100%	$ 1,866,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	Tennent-1
Project Name:	Gravity Main along Tennent Ave and at the WWTP
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 130 feet of 24-inch diameter pipeline, 1,250 feet of 30-inch diameter pipeline, and approximately 10 feet of 36-inch diameter pipeline along Tennent Avenue and inside of the WPCP. The surcharging of the gravity sewer cause SSO's upstream under PWWF conditions. To mitigate the risk of SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 36-inch to 42-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	24-36	36-42	Replace	1,390	#N/A	$ 1,438,000	$ 1,870,000	$ 2,664,000	2027-2031

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	97%	$ 2,582,000
Future Users	3%	$ 82,000
Total	100%	$ 2,664,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	Tennent-2
Project Name:	Gravity Main along Tennent Ave
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 3,360 feet of 18-inch diameter pipeline along Tennent Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate the risk of SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 24-inch to 36-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	18	24-36	Replace	3,360	#N/A	$ 2,288,000	$ 2,975,000	$ 4,239,000	2027-2031

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

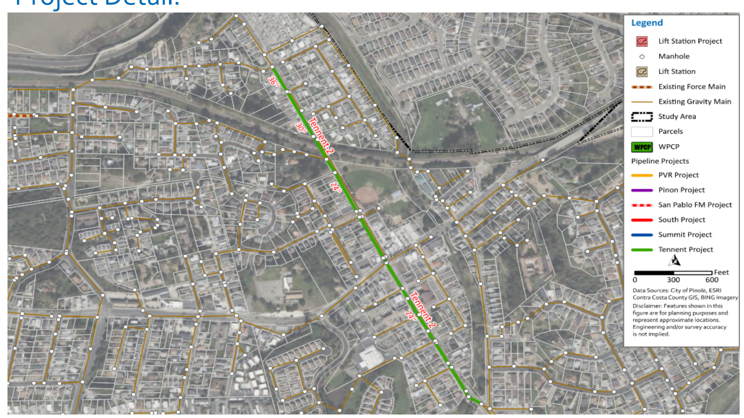
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	92%	$ 3,895,000
Future Users	8%	$ 344,000
Total	100%	$ 4,239,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:



Project Number:	PVR-1
Project Name:	Gravity Main along Pinole Valley Road, Orleans Drive, and Pinole Creek
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,130 feet of 6-inch to 10-inch diameter pipeline along Pinole Valley Road, approximately 1,830 feet of 8-inch diameter pipeline along Pinole Valley Creek, and approximately 530 feet of 12-inch diameter pipeline along Orleans Drive. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	6-12	12-15	Replace	4,020	#N/A	$ 1,629,000	$ 2,118,000	$ 3,018,000	2027-2031

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

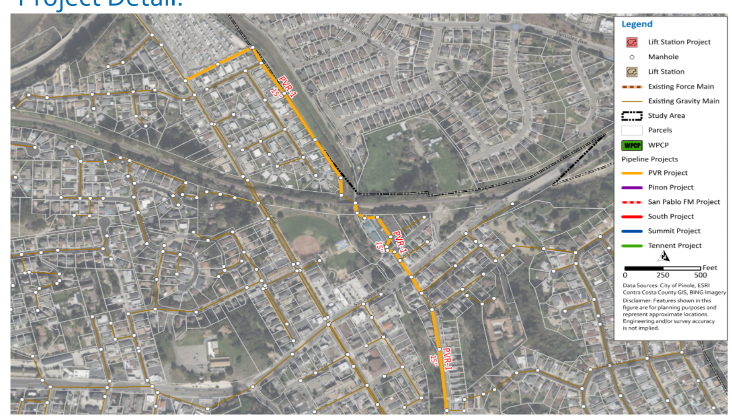
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	99%	$ 2,990,000
Future Users	1%	$ 28,000
Total	100%	$ 3,018,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:



Project Number:	PVR-2
Project Name:	Gravity Main along Pinole Valley Road
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,030 feet of 15-inch diameter pipeline and approximately 970 feet of 18-inch diameter pipeline along Pinole Valley Road. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[(1)] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[(2)] ($)	Capital Improvement Cost[(3)] ($)	Project Schedule
Gravity Main	15-18	24	Replace	2,000	#N/A	$ 1,240,000	$ 1,612,000	$ 2,298,000	2027-2031

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	92%	$ 2,122,000
Future Users	8%	$ 176,000
Total	100%	$ 2,298,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	South-1
Project Name:	Gravity Main along Pinole Valley Road, Sarah Drive, Shea Drive, and between Shea Driv
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,400 feet of 15-inch diameter pipeline along Pinole Valley Road, approximately 250 feet of 8-inch diameter pipeline along Sarah Drive, approximately 210 feet of 8-inch diameter pipeline along Shea Drive, and approximately 220 feet of 10-inch diameter pipeline between Shea Drive and Pinole Valley Road. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 15-inch to 21-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	8-15	15-24	Replace	2,080	#N/A	$ 1,066,000	$ 1,386,000	$ 1,975,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	93%	$ 1,836,000
Future Users	7%	$ 139,000
Total	100%	$ 1,975,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number: South-2

Project Name: Gravity Main along Pinole Valley Road

System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,090 feet of 15-inch diameter pipeline along Pinole Valley Road. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 18-inch to 21-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	15	18-21	Replace	1,090	#N/A	$ 524,000	$ 682,000	$ 971,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 949,000
Future Users	2%	$ 22,000
Total	100%	$ 971,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	South-3
Project Name:	Gravity Main along Pinole Valley Road and Simas Avenue
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 320 feet of 8-inch diameter pipeline along Simas Avenue and approximately 1,820 feet of 12-inch to 15-inch diameter pipeline along Pinole Valley Road. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 15-inch to 21-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	8-15	15-21	Replace	2,140	#N/A	$ 975,000	$ 1,268,000	$ 1,807,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 1,766,000
Future Users	2%	$ 41,000
Total	100%	$ 1,807,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	South-4
Project Name:	Gravity Main along Pinole Valley Road
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 2,500 feet of 10-inch to 12-inch diameter pipeline along Pinole Valley Road. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 15-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	10-12	15	Replace	2,500	#N/A	$ 1,013,000	$ 1,317,000	$ 1,877,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 1,835,000
Future Users	2%	$ 42,000
Total	100%	$ 1,877,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	South-5
Project Name:	Gravity Main along Pinole Valley Road, Doidge Avenue and Wright Avenue
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 980 feet of 8-inch to 10-inch diameter pipeline along Pinole Valley Road, approximately 290 feet of 8-inch diameter pipeline along Doidge Avenue and approximately 260 feet of 8-inch pipeline along Wright Avenue. The surcharging of the gravity sewer cause SSO's under PWWF conditions. To mitigate SSO's occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with pipelines ranging in size from 10-inch to 15-inch diameter pipeline. This project should be re-evaluated once the 2021 flow monitoring program has confirmed the flows in the pipes.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	8-10	10-15	Replace	1,530	#N/A	$ 620,000	$ 806,000	$ 1,149,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 1,128,000
Future Users	2%	$ 21,000
Total	100%	$ 1,149,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	Summit-1
Project Name:	Gravity Main along Summit Drive
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of approximately 410 feet of 6-inch diameter pipeline along Summit Drive. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	6	10	Replace	410	#N/A	$ 113,000	$ 147,000	$ 210,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	97%	$ 204,000
Future Users	3%	$ 6,000
Total	100%	$ 210,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number: Hazel-1
Project Name: Hazel Lift Station Replacement
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of the existing lift station. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. To mitigate the risk of a SSO occurring during PWWF conditions, it is recommended that the new lift station have a firm pumping capacity of 0.831 mgd.

Project Details:

Project Element	Existing Firm Capacity (mgd)	Proposed Firm Capacity (mgd)	Replace/ New	No. of Pumps (Units)	Unit Cost ($)	Baseline Construction Cost(1) ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Lift Station	0.43	0.831	Replace	2	--	$ 1,162,000	$ 1,511,000	$ 2,153,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 2,144,000
Future Users	0%	$ 9,000
Total	100%	$ 2,153,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost. Costs based on lift station cost curve.

Project Detail:

Project Number:	San Pablo-1
Project Name:	San Pablo Lift Station Replacement
System Type:	Wastewater Collection System

Project Description:

This project includes the replacement of the existing lift station. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. To mitigate the risk of a SSO occurring during PWWF conditions, it is recommended that the new lift station have a firm pumping capacity of 1.38 mgd.

Project Details:

Project Element	Existing Firm Capacity (mgd)	Proposed Firm Capacity (mgd)	Replace/ New	No. of Pumps (Units)	Unit Cost ($)	Baseline Construction Cost(1) ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Lift Station	0.58	1.38	Replace	2	--	$ 1,662,000	$ 2,161,000	$ 3,079,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	96%	$ 2,964,000
Future Users	4%	$ 115,000
Total	100%	$ 3,079,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost. Costs based on lift station cost curve.

Project Detail:

Project Number:	San Pablo-2
Project Name:	San Pablo Lift Station Forcemain Replacement
System Type:	Wastewater Collection System

Project Description:

The purpose of this project is to mitigate the high velocity (> 8 fps) that the existing forcemain experiences following San Pablo Lift Station under future conditions. It is recommended that an 8-inch diameter forcemain be constructed to replace the existing 6-inch diameter forcemain.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Force Main	6	8	Replace	640	#N/A	$ 141,000	$ 184,000	$ 262,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	96%	$ 252,000
Future Users	4%	$ 10,000
Total	100%	$ 262,000

Notes on Cost Estimation:

This project is an existing improvement. A cost percentage has been assigned to future users as a combination of existing and future users contribute to the deficiency.

Project Detail:

Project Number:	RR-2
Project Name:	Pipe Rehabilitation and Replacement Program
System Type:	Wastewater Collection System

Project Description:

This is an annual program to rehabilitate or replace aging pipes or pipes with poor condition. The results of the City's CCTV inspection program should be used to identify the pipes most in need of rehabilitation and replacement. Additionally, a long term risk assessment should be completed to identify long term rehabilitation and replacement projects and funding needs. It is also recommended that gravity pipes less than 8 inches in diameter be replaced with 8-inch pipe. The length/total cost recommended in the CIP is an estimate. Once the City completes the CCTV inspection and have a better idea of the condition of the

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Annual Length (ft/yr)	Unit Cost(1) ($/ft)	Baseline Construction Cost(1) ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2023
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2024
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2025
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2026
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2027
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2028
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2029
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2030
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 540,000	$ 702,000	$ 1,000,000	2031
Gravity Main	Varies	Varies	R&R	Varies	Varies	$ 5,400,000	$ 7,020,000	$ 10,000,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 19,000,000
Future Users	0%	$ -
Total	100%	$ 19,000,000

Notes on Cost Estimation:

As an R&R project, current users are assigned 100-percent of the project's cost.

Project Number:	RR-3
Project Name:	Inflow Identification Program
System Type:	Wastewater Collection System

Project Description:

The 2014 flow monitoring program revealed several subbasins within the collection system that exhibited higher rates of inflow. This project includes smoke testing and/or nighttime CCTV and/or field reconnaissance to identify potential sources of inflow.

Project Details:

Project Element	Existing Diameter	Proposed Diameter	Replace/ New	Length (ft)	Annual Cost ($/yr)	Capital Improvement Cost(3)	Project Schedule
I/I Reduction Projects	4-12	8-12	R&R	Varies	$ 500,000	$ 500,000	2022
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2023
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2024
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2025
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2026
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2027
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2028
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2029
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2030
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 500,000	2031
I/I Reduction Projects	4-12	> 6	R&R	Varies	$ 500,000	$ 5,000,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 10,000,000
Future Users	0%	$ -
Total	100%	$ 10,000,000

Notes on Cost Estimation:

As an existing R&R project, current users are assigned 100-percent of the project's cost. This CIP line item was developed so the City can budget for R&R Projects that could come from the I/I Reduction Program's inspection phases.

Project Number: O-1
Project Name: Sewer Master Plan Update
System Type: Wastewater Collection System

Project Description:

It is recommended that the City updates their Sewer Master Plan Update every 5 years to re-evaluate the wastewater collection system.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2027
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2032-2041
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2032-2041
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2032-2041

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	0%	$ -
Future Users	100%	$ 600,000
Total	100%	$ 600,000

Notes on Cost Estimation:

As a Sewer Master Plan Update, current users are assigned 100-percent of the project's cost.

Project Number:	O-2
Project Name:	Flow Monitoring Program
System Type:	Wastewater Collection System

Project Description:

It is recommended that the City conduct a flow monitoring program every 5 years to aid with the Sewer Master Plan Updates (O-1). It is assumed that each program will consist of 15 flow meters for a 1-month period. Flow monitoring should be timed to capture at least one major storm event, preferably following wet ground conditions.

Project Details:

Project Element	Existing Diameter (in)	Duration (weeks)	Replace/ New	No.	Unit Cost[(1)] ($/meter-week)	Baseline Construction Cost ($)	Estimated Construction Cost[(2)] ($)	Capital Improvement Cost[(3)] ($)	Project Schedule
Flow Monitoring Program	--	4 weeks	--	15	--	--	--		2027
Flow Monitoring Program	--	4 weeks	--	15	--	--	--		2028
Flow Monitoring Program	--	4 weeks	--	15	--	--	--		2032-2041
Flow Monitoring Program	--	4 weeks	--	15	--	--	--		2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for October 2021 is 14,452.

(2) Estimated Construction Cost include a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 15% for bid climate, 10% for engineering, 10% for construction management, and 7.5% for environmental and legal costs.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	0%	$ -
Future Users	100%	$ -
Total	100%	$ -

Notes on Cost Estimation:

As an existing inspection program, current users are assigned 100-percent of the programs's cost.