Ironhouse Sanitary District

2025 Wastewater Collection System

Master Plan

Final
March 15, 2026

Prepared By

V.W. HOUSEN
& ASSOCIATES

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ACRONYMS & TERMS

ADWF	Average Dry Weather Flow
BWWF	Base Wastewater Flow
CCTV	Closed Circuit Television
CIP	Capital Improvement Program
CMMS	Computerized Maintenance Management System
d/D	Depth over Diameter
District	Ironhouse Sanitary District
Diurnal	Daily Variation in base wastewater flow
ENR CCI	Engineering News Record Construction Cost Index
FPS	Feet per Second
GIS	Geographic Information System
GWI	Groundwater Infiltration
HDPE	High Density Polyethylene
I&I	Inflow and Infiltration
ISD	Ironhouse Sanitary District
MGD	Million Gallons per Day
NASSCO	National Association of Sewer Service Companies
NOAA	National Oceanographic and Atmospheric Administration
O&M	Operations & Maintenance
PACP	Pipeline Assessment and Certification Program
PDWF	Peak Dry Weather Flow
PVC	Polyvinyl Chloride
PWWF	Peak Wet Weather Flow
RDII	Rainfall-Dependent Inflow and Infiltration
R&R	Rehabilitation and Replacement
SWRCB	State Water Resources Control Board
V&A	V&A Consulting Engineers
WDR	2023 Statewide WDR or Order 2022-0103-DWQ
WRF	Water Reclamation Facility
WWPF	Wet Weather Peaking Factor
WWTP	Wastewater Treatment Plant

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We would like to extend our sincere appreciation for the contributions of the following ISD project team members, who provided necessary data, documents, and feedback throughout the development of the hydraulic model and rehabilitation and replacement recommendations.

Jean-Marc Petit, General Manager
Tyson Zimmerman, Assistant General Manager
Danea Gemmell, District Engineer
Louis Solana, Maintenance Superintendent
Dominick Gardiner, Collections System Operator

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EXECUTIVE SUMMARY

The 2025 Wastewater Collection System Master Plan presents results and recommendations from two separate but related studies that were completed for the Ironhouse Sanitary District's ("District") wastewater collection system. These studies include: 1) Hydraulic Model and Capacity Assessment and 2) Linear Asset Management Plan, also known as the Rehabilitation and Replacement ("R&R") plan. Together, the studies provide recommended projects, priorities, and costs for input into the District's capital improvement program ("CIP").

The District's previous Wastewater Collection System Master Plan was completed in 2004 and reviewed in 2018. The 2018 evaluation confirmed that the 2004 recommendation for a second 14-inch diameter force main from the E. Cypress corridor was still required in light of reduced buildout projections. The 2004 Master Plan utilized results from a hydraulic model that had not been verified through flow monitoring. The 2025 update includes a new hydraulic model that is calibrated to measured flow data from February and March 2024. The 2025 update also incorporates information gained through a significant wet weather event that occurred on December 31, 2022.

In addition to evaluating system capacity needs for use in developing the District's CIP, the Master Plan addresses topics that are discussed in the 2023 State Water Resources Control Board Order No. WQ 2022-0103-DWQ ("Statewide Waste Discharge Requirements") as related to system capacity.

ES-1 EXISTING SERVICE AREA

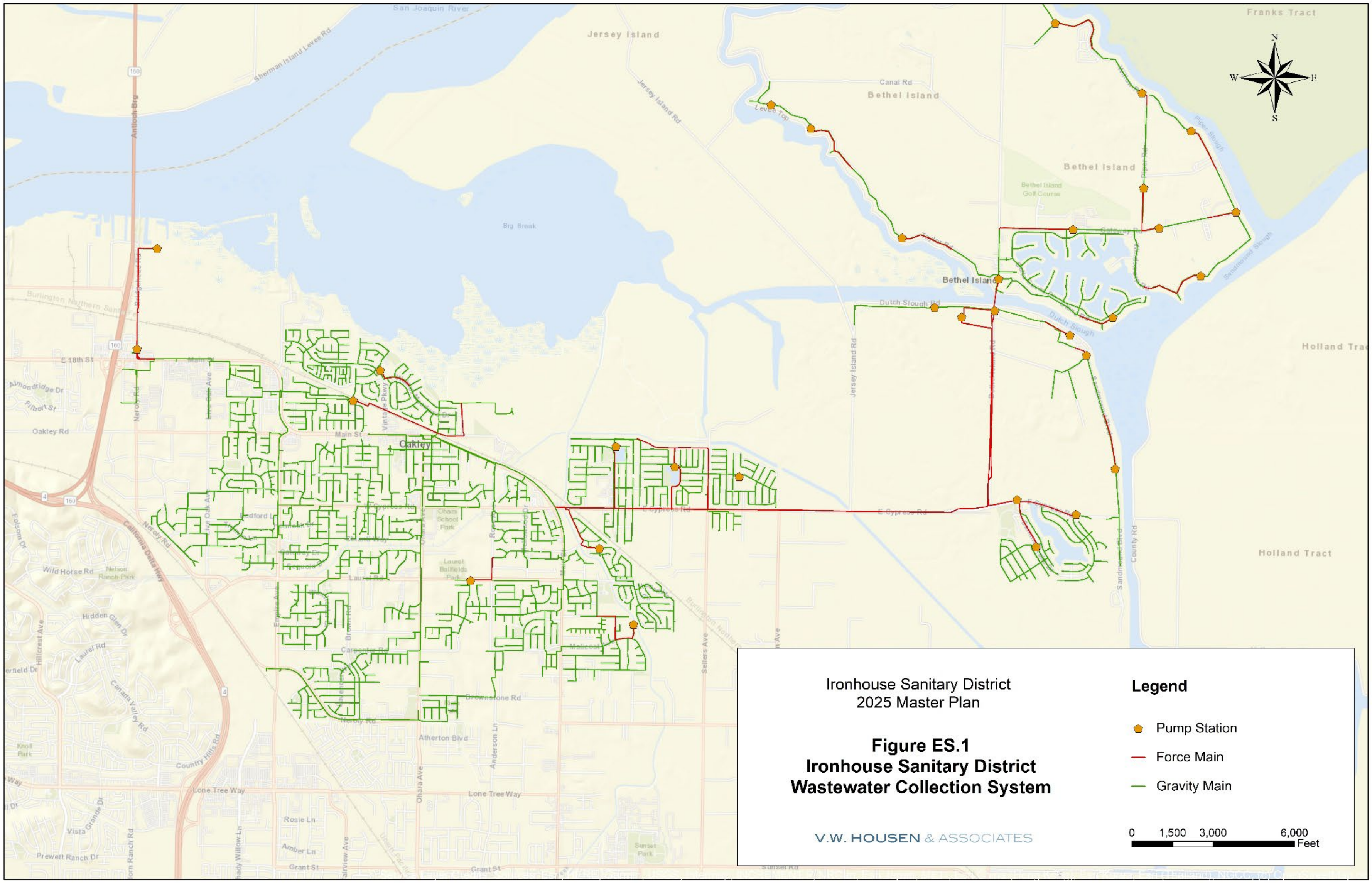
Ironhouse Sanitary District is located in eastern Contra Costa County near the intersection of Highway 160 and Highway 4. Ironhouse Sanitary District is situated with the San Joaquin River to the north, City of Brentwood to the south, and City of Antioch to the west. The District's sewer system is shown on Figures ES-1 on the following page and Figure 1.2 in Section 1.

Ironhouse Sanitary District at a Glance

- The District was formed in 1945
- The District serves approximately 41,361 residents in Oakley and Bethel Island
- The current wastewater treatment plant (WWTP) has been operating since 2011
- Historically, a majority of the District's recycled water was used on Jersey Island to grow hay, which was in turn used to feed ISD-owned cattle. Currently, the recycled water is discharged into the Sacramento-San Joaquin River Delta.
- The District comprises a mix of residential, commercial, and agricultural land uses
- Temperatures vary from average lows in the high 30s (in degrees Fahrenheit) in January to average highs in the low 90s in July
- The District's service area has a mean annual rainfall of 13.3 inches

ISD provides stewardship of City of Oakley and Bethel Island's sanitary sewer assets. The District owns and operates a wastewater collection system that discharges flow to a 4.3 million gallon per day ("mgd") Water Reclamation Facility ("WRF"). The District's geographical information system ("GIS") maps include 140 miles of gravity pipelines and 18.6 miles of force main pipelines. The system includes

Figure ES.1 ISD Wastewater Collection System



approximately 3,095 access structures and 32 sewer pump stations, with over half of the system having been constructed after 1980.

The District's WRF is located north of the intersection of Rose Avenue and Main Street in Oakley, California. Wastewater flow receives tertiary treatment and is either recycled for seasonal use on Bethel Island or discharged to the San Joaquin River.

Land use in the District's service area is primarily residential (76 percent) with a dense business corridor located along Main Street. Significant areas of new development are under construction and/or in planning stages along E. Cypress Road, between Main Street and Bethel Island Road.

Approximately 70 percent of the gravity system is comprised of PVC pipe. Sixteen percent of the system is comprised of vitrified clay pipe, 13 percent does not have its material labeled in GIS, and less than one percent of pipe is comprised of ductile iron pipe.

The District owns and operates an emergency storage facility on the mainland, south and west of the bridge crossing to Bethel Island. This facility, called the "Bethel Island Ponds," is used from time to time to temporarily store or manage wastewater flows from Bethel Island.

ES-2 HYDRAULIC MODEL AND CAPACITY ASSESSMENT

The District's hydraulic model is a tool for assessing the flows and capacities of the District's trunk sewers and for confirming the effectiveness of proposed capacity projects. The hydraulic model is also a tool for performing "what if" scenarios to assess the impacts of future developments, land use changes, and system configuration changes.

The hydraulic model includes the District's trunk sewers and associated facilities. The model also includes some smaller-diameter sewers as needed to provide system connectivity. The hydraulic model assigns flow from each of the District's parcels to the modeled network to simulate dry weather base wastewater flow ("BWWF").

The District conducted a flow monitoring program in February and March 2024. The flow monitoring period included two temporary gravity flow meters and five pump loggers and captured both dry and wet weather flow days. For this Master Plan, the selected dry weather flow day was February 27, 2024. On this day, the estimated average dry weather flow was 2.7 mgd.

Flow monitoring confirmed that in addition to BWWF, on February 27, 2024, the Bethel Island pipeline network received approximately 0.53 mgd of dry weather groundwater infiltration ("GWI"). This dry weather GWI was the result of high groundwater levels that were independent of any rainfall received. The dry weather GWI was entered into the hydraulic model separately from rainfall-dependent inflow and infiltration ("RDII").[1]

RDII is the collective description for stormwater and groundwater that enters the sewer system through pipe defects and unpermitted direct connections during wet weather events. Inflow describes water that enters through structures such as roof leaders and private drains, or from holes in manhole covers.

[1] After completion of the flow study, the District completed an extensive pipe grouting project on Bethel Island, named the Bethel Island I&I Reduction project. Following the completion of this project, lower plant flows were observed.

Infiltration describes water that enters through defects in pipes, joints, and manhole walls such as cracks, open joints, or breaks.

The hydraulic model calculates RDII through the application of industry standard wet weather parameters known as "RTK" that mimic RDII that enters the system during rainfall events. The wet weather factors add RDII as a percentage of rainfall volume and also distribute this additional flow over time.

Figure ES.2, also included as Figure 3.6 in this report and Figure 2-4 in the V&A Flow Monitoring Report (Appendix A), shows common sources of infiltration and inflow.

Figure ES.2 Typical Sources of Infiltration and Inflow taken from Figure 2-4 of the V&A Flow Monitoring Report (Appendix A)

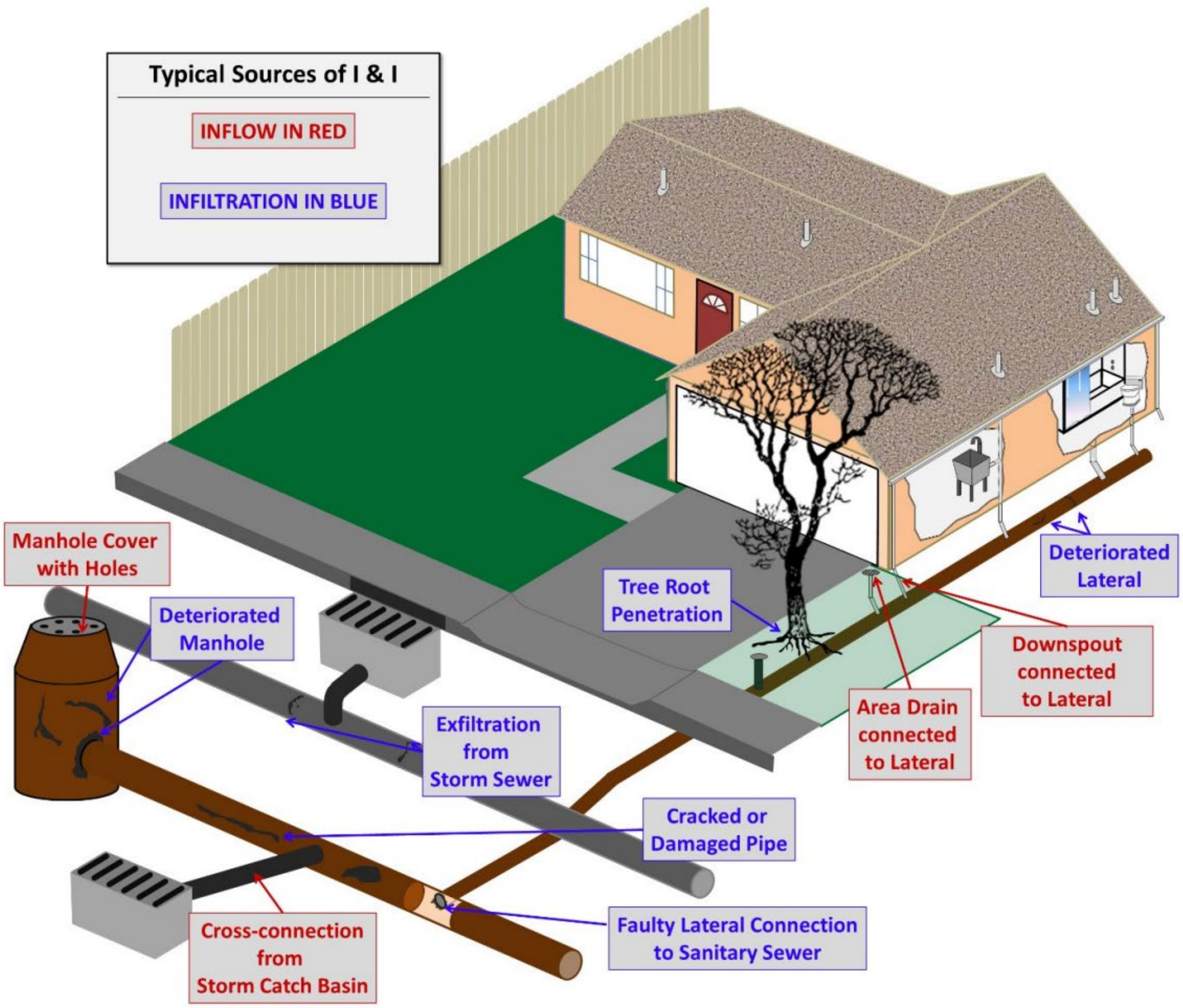


Figure 2-4. Typical Sources of Infiltration and Inflow

From February 17, 2024 to March 19, 2024, the District, through V&A Consulting Engineers, conducted a flow monitoring program. This program collected flow data using two temporary flowmeters within the gravity pipeline system and five pump station data loggers. The flow monitoring program included rainfall data from three rain gauges that are available through the website, Weather Underground (https://www.wunderground.com/wundermap). Figure ES.3, also included as Figure 3.1, shows the rainfall that was recorded at the three selected rainfall gauges. The most rainfall was received between February 17 and February 20, 2024, with a second event occurring between February 29 and March 3, 2024.

Figure ES.3 Rainfall Received during 2024 Flow Monitoring Period

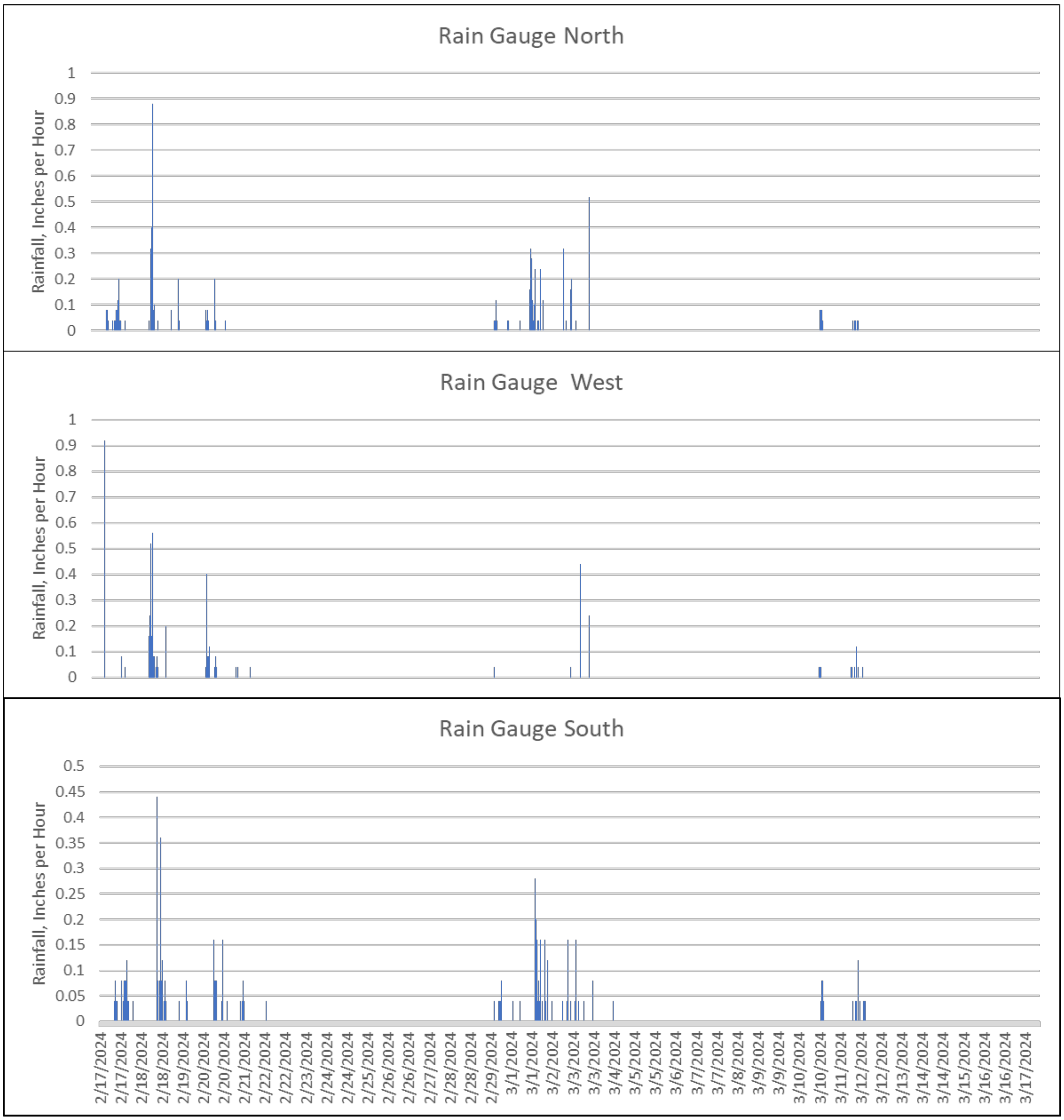
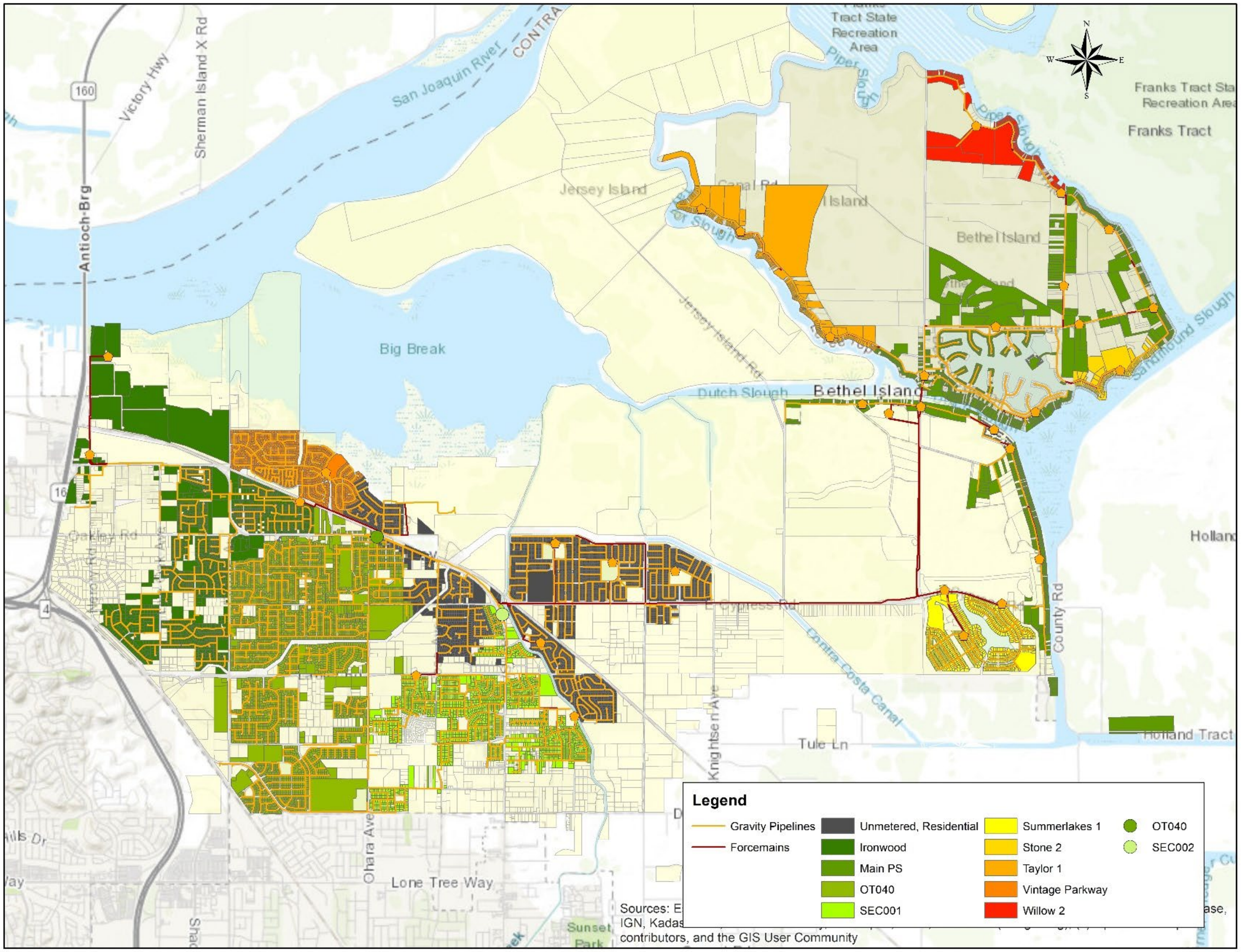


Figure ES.4 on the following page shows the locations of the meters and pump loggers and their associated sewer basins. Figure ES.4, also included as Figure 3.2, shows the areas of the District's service area, shaded in pale yellow, that are currently not connected to the public sewer system. The residential parcels on the western and southern boundaries of the services area are currently served by septic systems. As these system reach the end of their useful lives, the property owners will be required to consider whether to replace their existing systems or connect to the public sewer system. The final version of this Master Plan will include an evaluation of potential additional flow that could be generated by these properties, and their impact, if any, on the wastewater collection system.

The properties near the eastern boundary of the service area, west and east of Bethel Island Road, are part of the Summer Lakes North and Grand Cypress Preserve developments that are currently in planning or in the first phases of construction.

Figure ES.4 Metered Sewer Basins



ES-2.1 Capacity Assessment – Gravity Pipelines

The calibrated hydraulic model was used to estimate the capacity of the District's wastewater collection system under the design flow condition. The Design Storm is a hypothetical rainfall event that is configured based on criteria that is published by the National Oceanographic Atmospheric Administration ("NOAA").[2] The District has selected a 10-year, 24-hour design storm as its design flow condition. This storm has a recurrence interval of 10 years (*i.e.*, 10 percent probability of occurring in any given year) and duration of 24 hours. NOAA predicts that a 10-year, 24-hour design storm will deposit average rainfall depth of 2.43 inches in the City of Oakley. Larger storms are not as common, but do occur from time to time. For example, on December 31, 2022, local rainfall gauges measured between 4.6 and 6 inches of rain in a 24-hour period. This storm is characterized by NOAA as having a 100- to 500-year, 24-hour recurrence.

The hydraulic model identified no locations within the gravity collection system with predicted spills during the existing system design storm. During the existing scenario, the model predicted one location having a capacity constraint beginning in the 21-inch pipeline in Jordan Lane, directly upstream of the WRF, and continuing east on Main Street. This capacity constraint causes the hydraulic gradeline or water level to rise to within three feet of the manhole rim during existing conditions.

During the future scenario, which assumes full planned buildout of the E. Cypress Road communities, the hydraulic model predicts that the gravity interceptor on Main Street from East Cypress Road to Jordan Lane will experience spills.

It should be noted that no spills were observed from this pipeline during the December 31, 2022 rainfall event. The absence of a spill on December 31, 2022 confirmed that even though this section of the system is predicted to become surcharged during the design storm existing scenario, a spill is unlikely to occur under existing conditions.

The 2004 Master Plan recommended the installation of a new 14-inch force main from Bethel Island Road to the WRF. Portions of this force main have been constructed in conjunction with new development that has occurred along E. Cypress Road. This Master Plan confirms the need for this force main, which will reroute flows from the Summer Lakes, Summer Lakes North, Grand Cypress Preserve, Gilbert Ranch, Emerson Ranch, and Cypress Grove communities. After this flow is removed from the existing 14-inch force main, the model predicts that spills will no longer occur from the Main Street interceptor in the future wet weather flow scenario.

The existing and planned 14-inch force main sections are shown on Figure ES.5 on the following page and also Figure 5.9 of this report. The total estimated project cost for the remaining sections of 14-inch force main is $15.7 million, including design, construction, project and construction management, and other soft costs.

A buildout scenario was created that assumes that one-third of unsewered parcels will be connected to the ISD sewer system in the future. This scenario shows that even after the new 14-inch force main is completed as described above, the existing interceptor downstream of SEC001 will surcharge but not overflow in this future condition. The extent and timing of additional connections from the basins upstream of SEC001 are not known. Therefore this finding is presented for information only, and no new infrastructure was developed to address this future surcharge condition.

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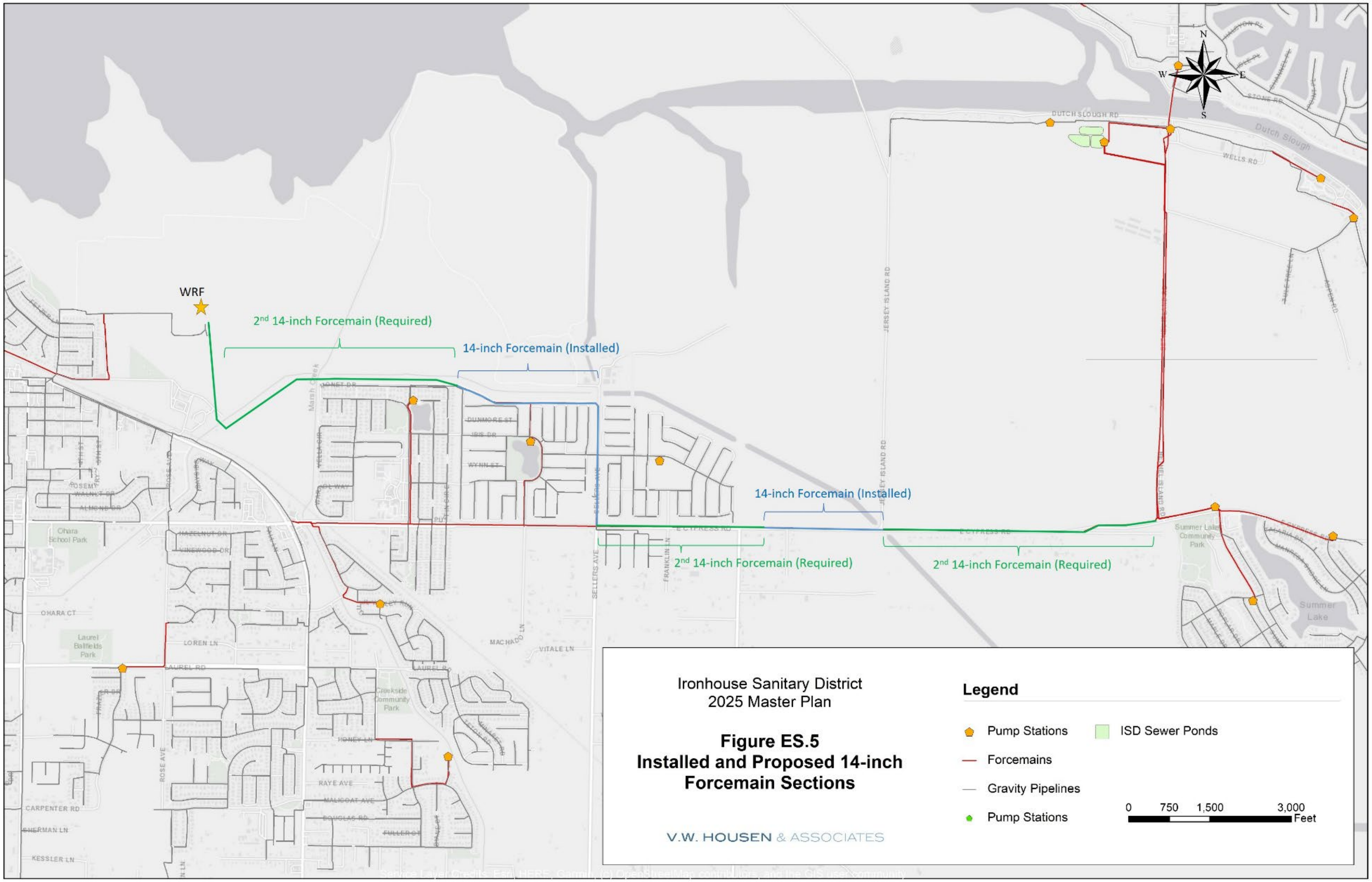
[2] Rainfall depths for an array of storms and associated temporal distribution curves are available at the NOAA Precipitation Frequency Data Server which is found at the following: https://hdsc.nws.noaa.gov/pfds/

Figure ES.5 Installed and Proposed 14-inch Force Main Pipeline Sections



ES-2.2 Capacity Assessment – Pump Stations

Model-generated flows from the design storm event were compared to firm pump station capacity (i.e., capacity with the largest pump out of service) for each pump station. Four of the District's 32 pump stations were monitored during the flow monitoring program and 29 of these stations were included in the model.

The hydraulic model predicts that influent flows exceed firm pumping capacity at four pump stations on or near Bethel Island. This Master Plan does not recommend any capacity improvements to these stations for several reasons. First, the hydraulic model did not predict any spills from the gravity pipelines that lead to each of these stations. In addition, no known spills occurred from these stations during the December 31, 2022 wet weather event. Therefore, it does not appear that additional pumping capacity is required to address the predicted influent flows.

Second, in 2024, after completion of the flow monitoring program, the District completed the Bethel Island I&I Reduction project. This project involved grout sealing of known locations of I&I within the Bethel Island wastewater collection system. Although the resulting reduction in GWI has not been documented, flow data from the WRF has confirmed that a measurable reduction in GWI has been achieved. This reduction in GWI should also help reduce the volume of wet weather flow that enters each pump station.

In addition, the District's Bethel Island pump stations operate in sequence. Upsizing one station will increase flows to all downstream stations and any such upgrades should be implemented judiciously.

In addition to pumping capacity, force main velocities were reviewed during the existing and buildout scenarios. The maximum force main velocity is predicted to be 7.9 feet per second ("fps") downstream of the Taylor 3 pump station. This velocity is below the maximum preferred design velocity of 8 fps, as discussed in Section 5. Therefore, no upgrades to the existing force main system is required as related to velocity.

Summer Lakes 1 pump station was constructed with sufficient capacity to convey flows from the Summer Lakes, Summer Lakes North, and Grand Cypress Preserve developments. The current pump station serves only the Summer Lakes development and has been downsized to more closely match existing flows. The rated capacity for this station is 1,925 gpm or 2.77 mgd. The existing system scenario assumes that the station has 1 mgd of pumping capacity that will be upgraded as the remainder of these developments are built out.

In the buildout scenario, after the full 2.77 mgd of Summer Lakes 1 pumping capacity is placed into service, the existing gravity system at the Main Street force main discharge manhole becomes overwhelmed. Consequently, the hydraulic model predicts spills from the gravity interceptor on Main Street and Jordan Lane. This capacity issue is mitigated after completion of the second 14-inch force main from the E. Cypress Road communities.

The second 14-inch force main will also serve as a redundant force main during dry weather conditions. In order to increase system reliability, the District may want to consider adding similar parallel (redundant) force mains downstream of the largest pumping stations, including the Main, Web, and Ironwood pump stations.

ES-2.3 Review of Statewide Waste Discharge Requirements for Capacity Analysis

The new State Water Resources Control Board Order WQ 2022-103-DWQ ("Statewide WDR") has been effective as of June 5, 2023. The hydraulic analysis and capacity assessment that were completed for this Master Plan address most of the requirements of the Statewide WDR without the need for supplemental analysis. However, two items from the Statewide WDR require additional discussion:

1) Capacity of systems subject to increased inflow and infiltration ("I&I") due to larger and/or higher-intensity storm events as a result of climate change; and

2) increase of erosive forces in canyons and streams near underground and aboveground system components due to larger and/or higher intensity storm events.

Chapter 5, Capacity Analysis, provides information to address these two topics from the Statewide WDR.

ES-3 REHABILITATION AND REPLACEMENT PLAN

The District's Rehabilitation and Replacement ("R&R") plan identifies gravity sewer pipelines with the highest risk of failure, develops rehabilitation recommendations for these pipelines, estimates costs, and prioritizes repairs to assist in capital project planning. The R&R plan focuses on pipes with the highest likelihood of failure as determined through CCTV inspection data. The R&R program provides a recommended project to address every known high-risk structural defect. If these pipes need to be further prioritized, this Master Plan provides a strategy for incorporating data that adds a consequence of failure lens to the rehabilitation plan, thereby providing a risk-based prioritization process based on Likelihood and Consequence of Failure.

ES-3.1 Likelihood of Failure

Likelihood of Failure for the District's gravity collection system is defined by several parameters:

- Sanitary sewer spill history (5 years)
- Structural and Operation & Maintenance defects

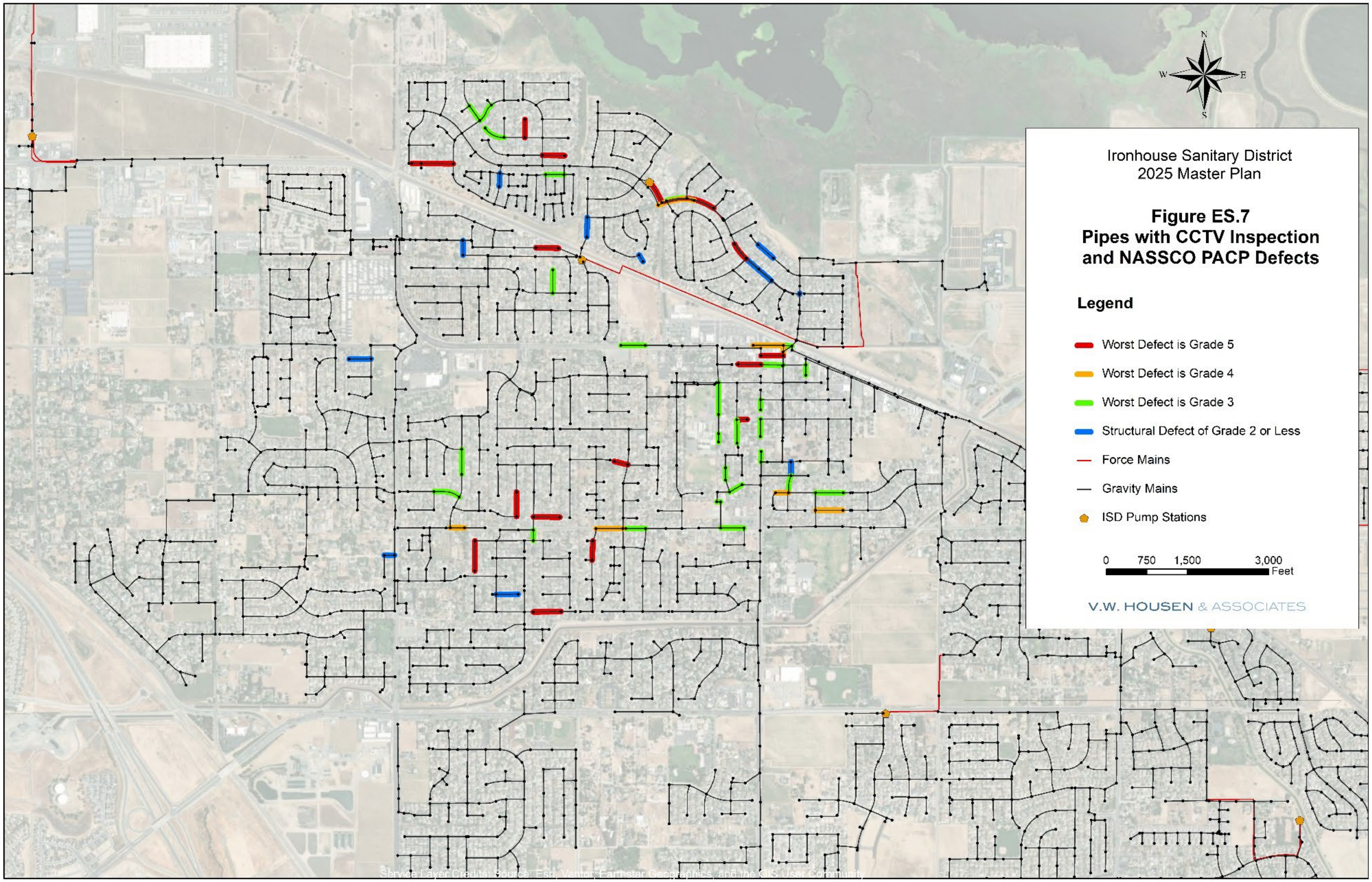
The R&R plan utilized CCTV inspection records for pipes bounded by the Vintage Parkway neighborhood to the north, Empire Avenue to the west, Chianti Way and the Contra Costa Canal to the south, and Fifth Street to the east. This area includes the oldest portion of the service area with the highest concentration of trouble spots. The remainder of the system is in very good condition, as most of the pipes were installed after 1980.

Approximately 3.6 miles or 10 percent of the inspected pipelines had recorded structural defects. 1.4 miles of pipes with defects had at least one National Association of Sewer Service Companies ("NASSCO") Pipeline Assessment Certification Program ("PACP") structural Grade 4 or 5 defect[3]. Additional lines have root balls or infiltration defects that impact system performance. These pipes are recommended for repair or replacement.

Figure ES-6 shows the areas that received CCTV inspection, and of the pipes that were inspected, the pipes with recorded defects. This figure is also included later in this Master Plan as Figure 6.1.

[3] The NASSCO PACP scoring scale ranges from 1 to 5, with 5 designating the most severe defect.

Figure ES-6 Pipes with CCTV Inspection and NASSCO PACP Defects



Consequence of Failure Factors

The District plans to address all pipes with known structural Grade 4 and 5 defects within the next five years. If the District decides to lengthen the proposed rehabilitation and replacement program timeline to a period that extends beyond ten years, further project prioritization should consider one or more of the following consequence-of-failure parameters. A pipe that has more of these consequence factors would be a candidate for earlier replacement in order to reduce the impacts of sanitary sewer spills on the public and environment.

- Proximity to a waterway
- Proximity to a primary or secondary transportation corridor
- Proximity to public facilities, including schools, parks, and hospitals
- Area served, as indicated by pipe size (i.e., greater than 12 inches in diameter)

ES-3.2 R&R Capital Replacements

The Rehabilitation and Replacement program is summarized in Table ES.1 below.

Table ES.1 Summary of Rehabilitation and Replacement Projects

Sewer Basin	Point Repair (Pipes)	Replace (Pipes)	Length (ft)	Cost
California Dawn	1	0	350	$ 19,500
Ironwood	3	0	867	$ 58,500
Old Town	28	13	11,364	$ 2,689,200
Vintage Parkway	12	2	4,058	$ 477,300
Walnut Meadows	4	0	998	$ 78,000
Allowance to Convert Point Repairs to Pipe Replacements (15 percent of designated point repairs in this table)				$ 1,587,600
Total	**48**	**15**	**17,637**	**$ 4,910,100**

ES-4.0 CAPITAL IMPROVEMENT PLAN

The Capacity Assessment (Chapter 5) and Pipeline Rehabilitation and Replacement program (Chapter 6) each evaluated infrastructure needs for the next 10 years and developed proposed recommendations, priorities, and costs. These projects, priorities, and costs are summarized in the Capital Improvement Program shown in Table ES.2. The basis behind each of the projects is discussed in further detail within each respective chapter in this Master Plan.

All costs are presented in current dollars and indexed to Engineering News Record (“ENR”) Construction Cost Index (“CCI”), San Francisco, August 2025, 15473.38.

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Table ES.2 Capital Improvement Plan

Project Description	FY 2025/26	FY 2026/27	FY 2027/28	FY 2028/29	FY 2029/30	FY 2030/31	FY 2031/32	FY 2032/33	FY 2033/34	FY 2034/35	10-Year Total
14-inch Parallel FM	$250,000	$350,000	$7,500,000	$7,500,000							$15,600,000
Collection System Renovation Program - Phase I - California Dawn (1 point repair) - Ironwood (3 point repairs) - Old Town (28 point repairs and 14 line replacements) - Vintage Parkway (13 point repairs and one line replacement) - Walnut Meadows: 4 point repairs) - Allowance to complete full replacement of 20 percent of pipes currently shown as receiving point repairs	$150,000	$900,000	$900,000	$900,000	$900,000	$900,000	$260,100				$4,910,100
Collection System Renovation Program - Phase 2							$589,900	$850,000	$850,000	$850,000	$3,139,900
Total - Collection System Program	**$400,000**	**$1,250,000**	**$8,400,000**	**$8,400,000**	**$900,000**	**$900,000**	**$850,000**	**$850,000**	**$850,000**	**$850,000**	**$23,650,000**

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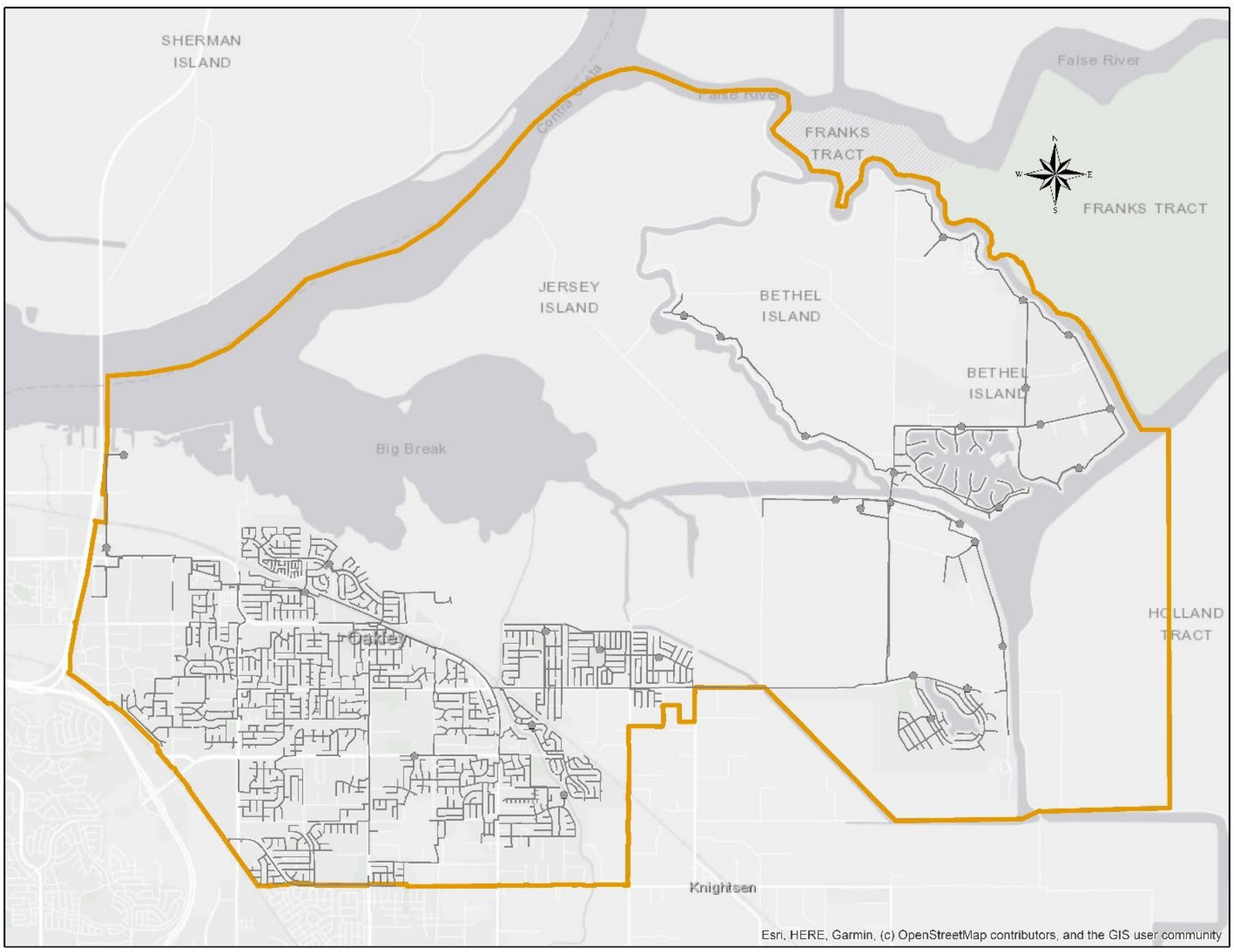
CHAPTER 1 INTRODUCTION AND EXISTING INFORMATION

1.0 DISTRICT OVERVIEW

The Ironhouse Sanitary District ("ISD or District") is located in eastern Contra Costa County at the intersection of Highway 4 and Highway 160 and is bordered by the City of Antioch on the west, the City of Brentwood on the south, the San Joaquin River on the north, and unincorporated areas of Contra Costa County on the east and south. The District's service area is shown on Figure 1.1.

This Chapter describes the District's existing system assets and summarizes the information that was used to develop this 2025 Master Plan.

Figure 1.1 Ironhouse Sanitary District Service Area



ISD owns and operates the City of Oakley and Bethel Island's wastewater collection system assets. The District conveys flow to a 4.3 mgd recycled water facility, located north of Main Street, near the intersection of Main Street and Rose Avenue in Oakley. The District's GIS maps include 140 miles of gravity pipelines and 18.6 miles of force main pipelines. Over half of the system was constructed after 1980. The system includes approximately 3,200 access structures and 32 sewer pump stations.

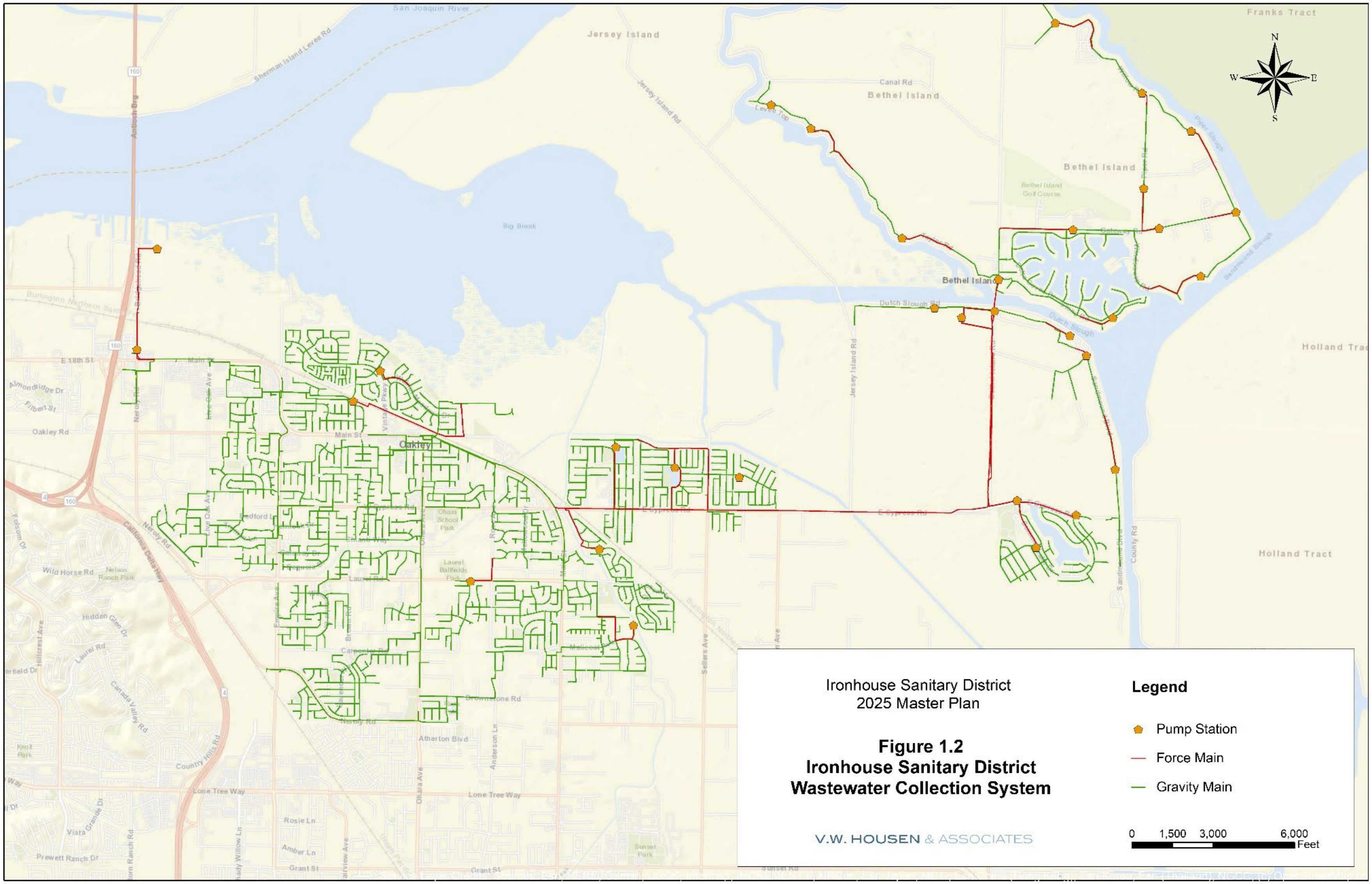
The District has compiled a comprehensive paper and digital library of documents that describe these assets, including a computerized maintenance management system ("CMMS") using CalCAD software, a computerized hydraulic model of the wastewater collection system, geographical information system ("GIS") maps, and paper and/or electronic records documenting system evaluations, repairs, replacements, new construction, operations, and maintenance. Figure 1.3 on the following page shows the District's wastewater collection system assets.

The District operates an emergency storage facility known as the Bethel Island Ponds. The Bethel Island Ponds are comprised of three shallow earthen ponds located on the mainland, west of the crossing to Bethel Island. These ponds are used to temporarily store flows during construction activities, or in heavy wet weather events when flows on and near Bethel Island exceed the capacity of the wastewater collection infrastructure.

Figure 1.2 below shows the location of the Bethel Island Ponds.

Figure 1.2 Location of Bethel Island Ponds

Figure 1.3 ISD Wastewater Collection System Assets.



The District's average dry weather flow as measured on February 27, 2024 was 2.7 million gallons per day (mgd). Approximately 0.53 mgd of this flow is attributed to dry weather groundwater infiltration ("GWI") from Bethel Island[4]. The measured flow, less calculated dry weather GWI, translates to approximately 53 gallons per capita per day.

1.1 SUMMARY OF EXISTING INFORMATION

This section describes existing information that was reviewed and utilized in the development of the 2025 Master Plan.

1.1.1 Prior District Studies, Documents, and Data

The following information was provided by the District for the 2025 Master Plan update.

Prior District Studies and Documents

- Ironhouse Sanitary District Sewer Master Plan (2004, updated in 2007)
- Sewer System Management Plan (2021 with updated Element 6 (2023))
- ISD Strategic Plan (2022 to 2027+)
- ISD GIS layers from CalCAD
- ISD CCTV inspection data from CalCAD
- ISD pump station characteristics and setpoints
- ISD connected parcels from tax roll
- Flow Monitoring Report, V&A Engineering (2024)
- Raw flow data to support the above Flow Monitoring Report
- City of Oakley General Plan
- City of Oakley E. Cypress Corridor Specific Plan
- City of Oakley River Oaks Crossing Specific Plan
- City of Oakley Downtown Specific Plan

In addition, to further confirm hydraulic model results, the District has provided water data from Diablo Water District and additional data from the Main pump station flowmeter. This information will be integrated into the final version of this Master Plan.

1.1.2 Other Resources

The following publicly-available information was also downloaded for use during this project.

[4] After completion of the flow study, the District completed the Bethel Island I&I Reduction Project on Bethel Island. Following the completion of this project, a measurable decrease in plant flows was observed.

- Contra Costa GIS Layers including: Parcels; City Boundaries; Natural Features; Landmark Features; and County Streets
- Land Use Documentation

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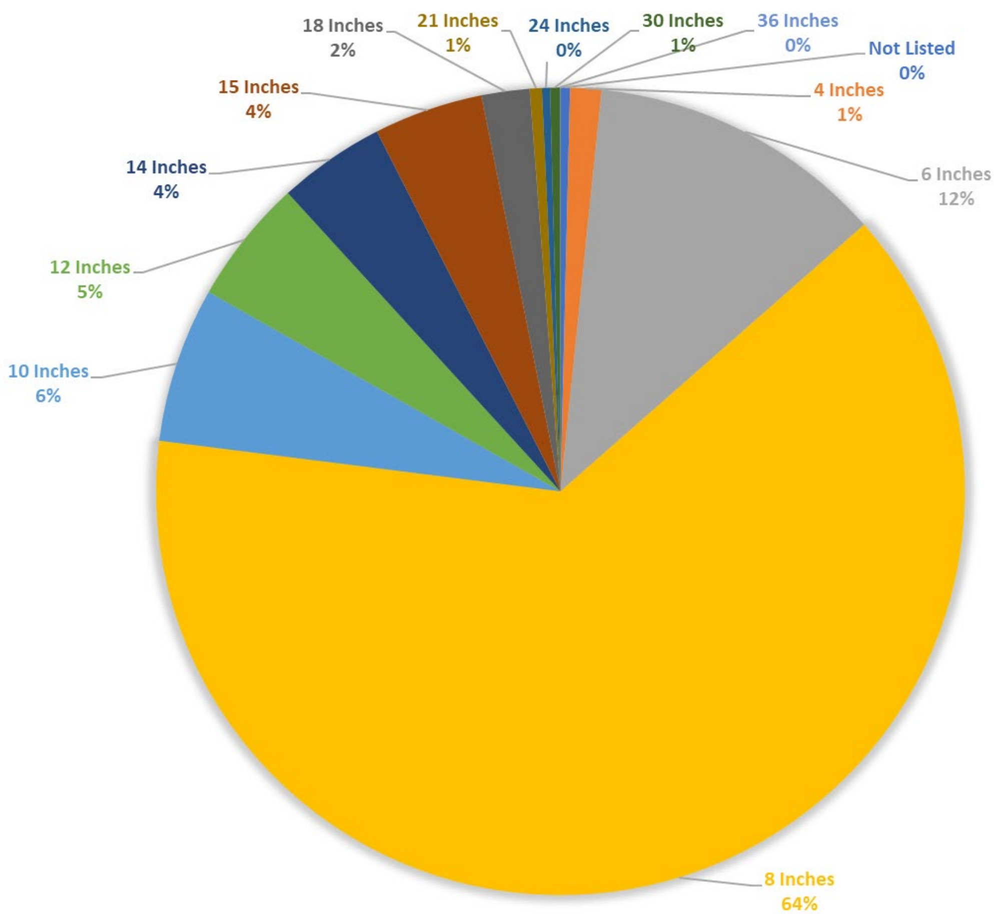
CHAPTER 2 SYSTEM ASSETS AND LAND USES

The purpose of this Chapter is to summarize the District's existing system, land use characteristics, and projected average dry weather ("ADWF") and base wastewater flow ("BWWF"). For the District's system, ADWF is the sum of BWWF plus dry weather groundwater infiltration.

2.1 EXISTING WASTEWATER SYSTEM

The District's wastewater collection system is represented in the District's GIS map layers as having approximately 140 miles of gravity sewer pipe, 18.6 miles of force mains, and 3,095 access structures. The District's gravity pipes range in diameter from 4 to 36 inches. Figure 2.1 shows the distribution of pipes by size.

Figure 2.1 Gravity Sewer Pipeline Inventory by Size



The predominant pipeline material is vitrified clay pipe. New gravity pipelines utilize polyvinyl chloride ("PVC") pipe. Force main pipes are comprised of ductile iron, PVC, and/or high density polyethylene ("HDPE").

The District's service area includes 36 sewer basins. Thirty of these basins flow to pumping stations; six basins flow by gravity to the District's WRF. The two largest basins of these six gravity-flow basins were monitored during the 2024 wet weather season. Table 2.1 lists the District's basins and their connectivity.

This connectivity is also shown on Figure 2.2. Figure 2.3 on the following page shows the District's wastewater access structures, color coded by sewer basin.

Table 2.1 ISD Wastewater Basins

Basin Name	Flows to PS with Same Name	Flows by Gravity to WRF	Notes
Bridgehead	x		Flows to Ironwood PS
California Dawn		x	182 parcels flow to Hwy 4
Cypress Grove	x		Flows to E. Cypress Road[3]
D1 (Dutch Slough)	x		Flows to WEB PS
Emerson	x		Flows to E. Cypress Road[5]
G1 (Gateway)	x		Flows to MPS
G2 (Gateway)	x		Flows to G1 PS
G3 (Gateway)	x		Flows to G2 PS
Gilbert	x		Flows to Emerson PS[3]
Hwy 4		x	5 parcels flow to Walnut Meadows
Ironwood	x		Flows to Walnut Meadows
Laurel Heights	x		Flows to Rose Ave
Lauritzen	x		Flows to Bridgehead PS
Marsh Creek	x		Flows to Southeast Corridor
MPS (Main)	x		Flows to E. Cypress Road
Old Town		x	Flows to Walnut Meadows
P1 (Piper)	x		Flows to G1 PS
Quail Valley	x		Flows to Hwy 4
Rose Ave		x	482 parcels flow to Hwy 4
S1 (Stone)	x		Flows to MPS
S2 (Stone)	x		Flows to G1 PS
SL1 (Summer Lakes)	x		Flows to E. Cypress Road[3]

[5] When the redundant 14-inch force main is completed, the Emerson Ranch, Gilbert, Cypress Grove, and Summer Lakes 1 pump stations will be connected to the new force main and flow directly to the WRF.

Basin Name	Flows to PS with Same Name	Flows by Gravity to WRF	Notes
SL2 (Summer Lakes)	x		Flows to SL1 PS
SL3 (Summer Lakes)	x		Flows to SL1 PS
SM1 (Stonemound)	x		Flows to WEB
SM2 (Stonemound)	x		Flows to SM1 PS
South East Corridor		x	Flows to Hwy 4
T1 (Taylor)	x		Flows to MPS
T2 (Taylor)	x		Flows to T1 PS
T3 (Taylor)	x		Flows to T2 PS
Vintage Parkway	x		Flows to Walnut Meadows
W1 (Willow)	x		Flows to G3 PS
W2 (Willow)	x		Flows to MPS
W3 (Willow)	x		Flows to W2 PS
Walnut Meadows		x	269 parcels flows to WRF
WEB	x		Flows to E. Cypress Road

Figure 2.2 Wastewater Collection System Subcatchment Connectivity

Notes:

1. Basins shaded in dark blue were metered during the 2024 flow monitoring program
2. The Junction Structure is added for clarity and does not comprise a separate sewer basin

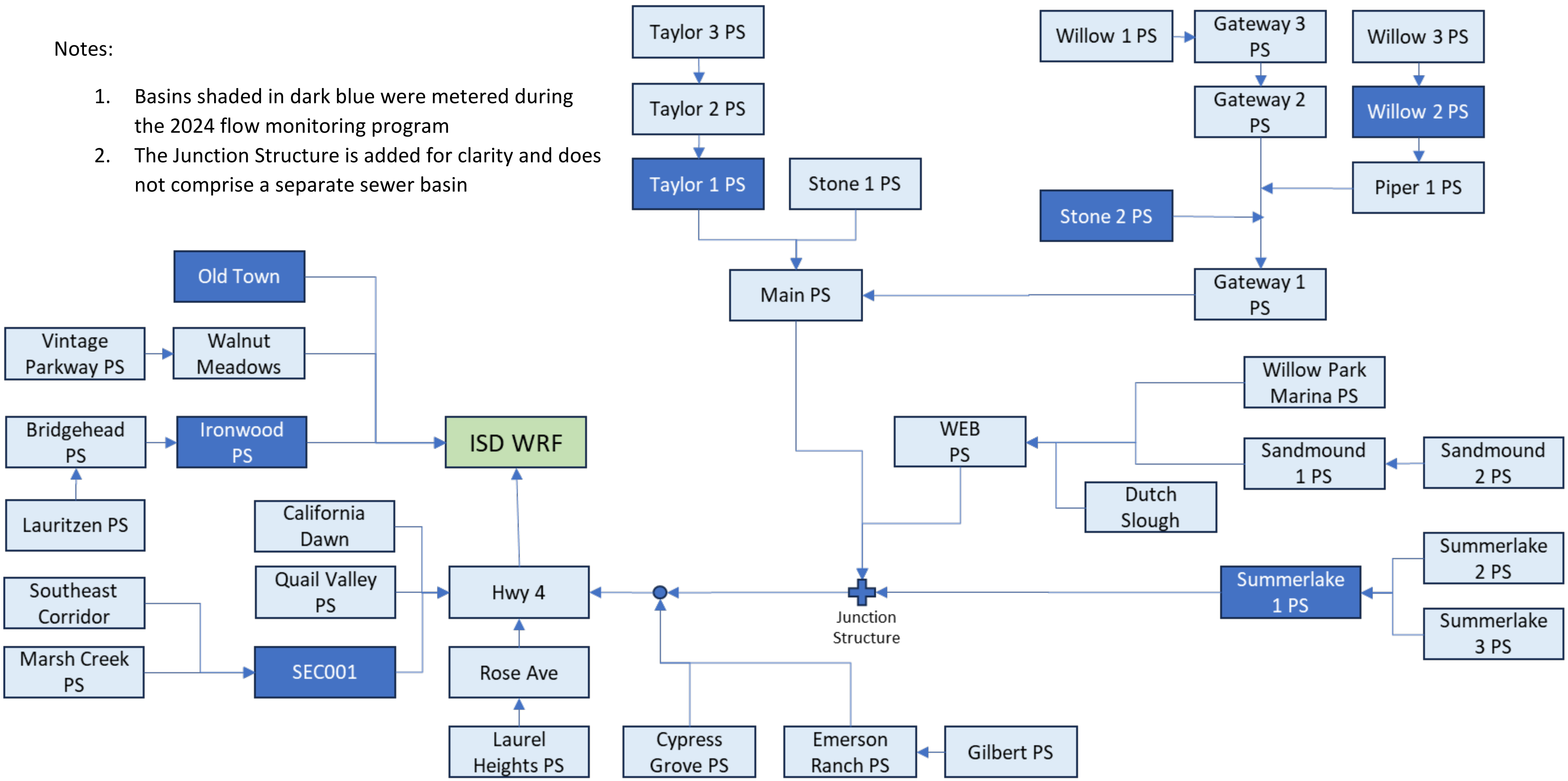
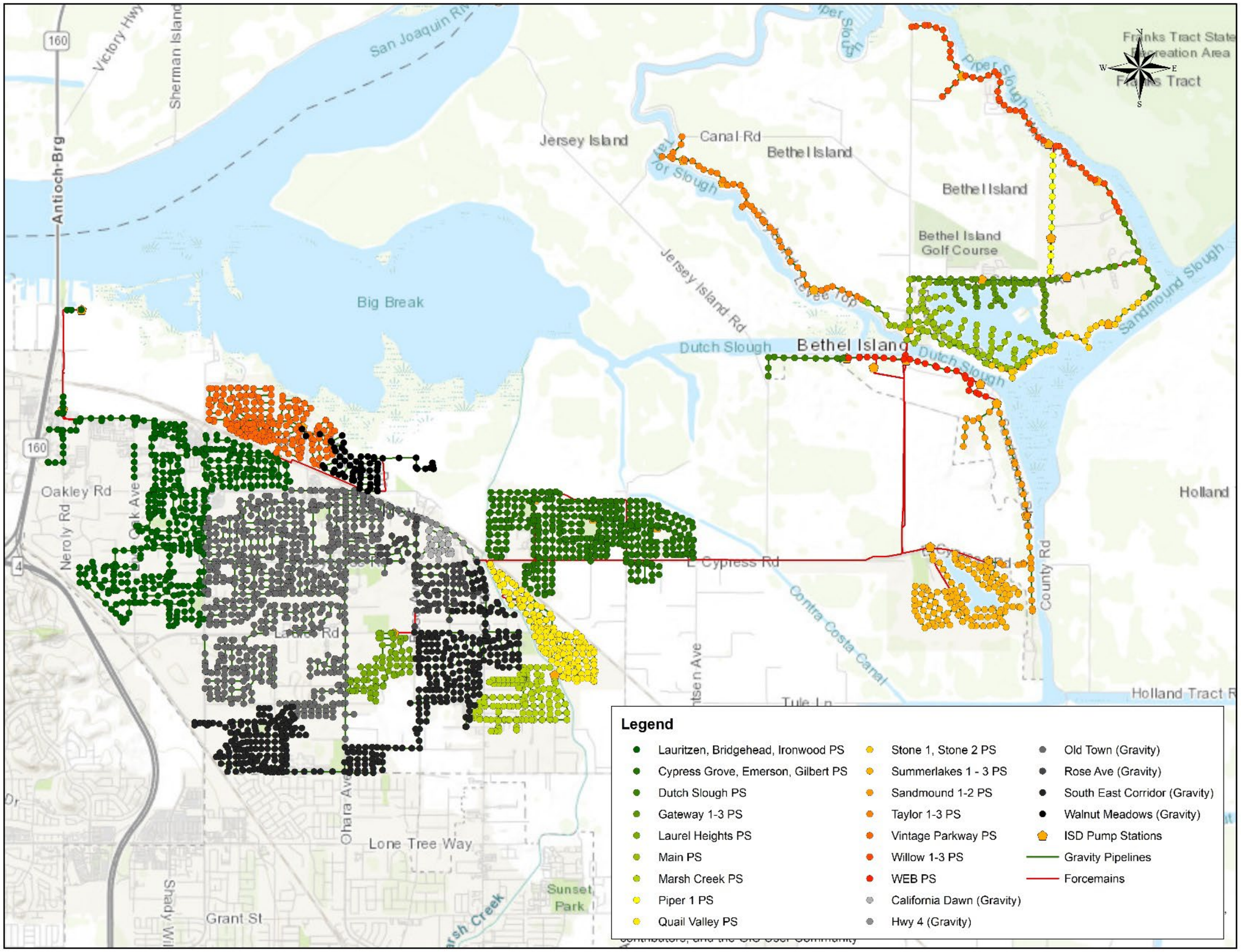


Figure 2.3 Wastewater Collection System Access Structures Color Coded by Sewer Basin



The District operates 32 pump stations. Four these pump stations were monitored during the flow monitoring period using data loggers as shown in bold font; 29 of the stations were included in the model, as shown on Table 2.2.

Table 2.2 Pump Station Characteristics

Pump Station Name		Where Does Flow Discharge	Associated Meter	Firm Capacity (gpm)
1	Bridgehead	Main Street	IW PS data logger	180
2	Cypress Grove	Main Street	Unmetered[6]	355
3	Dutch Slough	Dutch Slough Road	WEB. See Note 6.	150
4	Emerson Ranch	Main Street	Unmetered	449
5, 6, 7	Gateway 1, 2, 3	Gateway Road	Main PS. See Note 6.	158 to 836
8	Gilbert Ranch 1	Main Street	Unmetered	Unmodeled
9	**Ironwood**	**Main Street**	**IW PS data logger**	**1000**
10	Lauritzen	Bridgehead PS	IW PS data logger	Unmodeled
11	Laurel Heights	Clearwood Drive	Unmetered	110
12	Marsh Creek	Creekside Way	SEC002 flowmeter	329
13	Main (MPS)	Bethel Island Road	Unmetered[7]	1400
14	Piper 1	Gateway Road	Main PS. See Note 6.	277
15	Quail Valley	Main Street	OT040 flowmeter	600
16	Stone 1	Stone Road	Main PS. See Note 6.	240
17	Stone 2	Windsweep Road	Main PS. See Note 6.	200
18	**Summer Lakes 1**	**Main Street**	**SL-1 PS data logger**	**1925**
19, 20	Summer Lakes 2, 3	Summer Lakes 1 PS	SL-1 PS data logger	250 to 350
21	Sandmound 1	Sandmound Blvd	Unmetered. See WEB.	150
22	Sandmound 2	Mariner Road	Unmetered. See WEB.	190
23	**Taylor 1**		**Taylor 1 PS data logger**	**270**
24	Taylor 2	Taylor 1	Taylor 1 PS data logger	190
25	Taylor 3	Taylor 2	Taylor 1 PS data logger	310

[6] Pump stations for new developments along E. Cypress Corridor were not metered.
[7] The Main pump station is metered and connected to SCADA. Flow data for the flow monitoring period was unavailable during the calibration phase, however, due to issues with the SCADA clock. Therefore, I&I values from the metered Bethel Island stations were assigned to Main and WEB pump stations. Since this time, new data has been provided for Main pump station that is under review.

Pump Station Name		Where Does Flow Discharge	Associated Meter	Firm Capacity (gpm)
26	Vintage Parkway	WRF	Unmetered	600
27	Willow 1	Gateway 1	Main PS. See Note 6.	150
28	Willow 2	Piper 1	Main PS. See Note 6.	106
29	Willow 3	Willow 2	Main PS. See Note 6.	200
30	WEB	WRF	Unmetered	796
31	Willow Park Marina	WEB	Unmetered	Unmodeled
32	Bethel Island Ponds	WEB	Unmetered	Unmodeled

2.2 LAND USE CHARACTERISTICS

Land use in the District's service area is primarily residential, with a dense business corridor located along Main Street and rapidly-developing subdivisions along East Cypress Road.

Land use information is available through the following sources:

- City Land Use Database – General Plan land use shapefiles were provided by the City of Oakley. Adopted Specific Plans for the East Cypress Corridor, Downtown, and River Oaks Crossing were available in pdf format on the City's website.
- County Parcels – Information on County Parcels were used to assign land uses to the Bethel Island parcels
- Ironhouse Sanitary District – Billing records denoting connected parcels and their associated commercial billing classification were provided by the District
- Aerial Imagery – In addition to land use information, aerial imagery was reviewed for the District's service area to identify parcels that are currently vacant, or where actual uses may vary significantly from the designated land use

Figure 2.4 on the following page shows the distributions of land use designations that were assigned to parcels within the District's service area, using the references described above. The land use descriptions used in the District's billing records were used where the land use designations differed between the General Plan and billing information.

Seventy nine percent of the service area is comprised of residential land uses, with the predominant land use being Single Family Residence. Of the remaining land uses, eight percent are commercial and industrial, eight percent are for recreational uses, and five percent are public buildings.

Figure 2.5 shows how land uses were assigned to the connected parcels within the service area.

Figure 2.4 Distribution of Land Uses

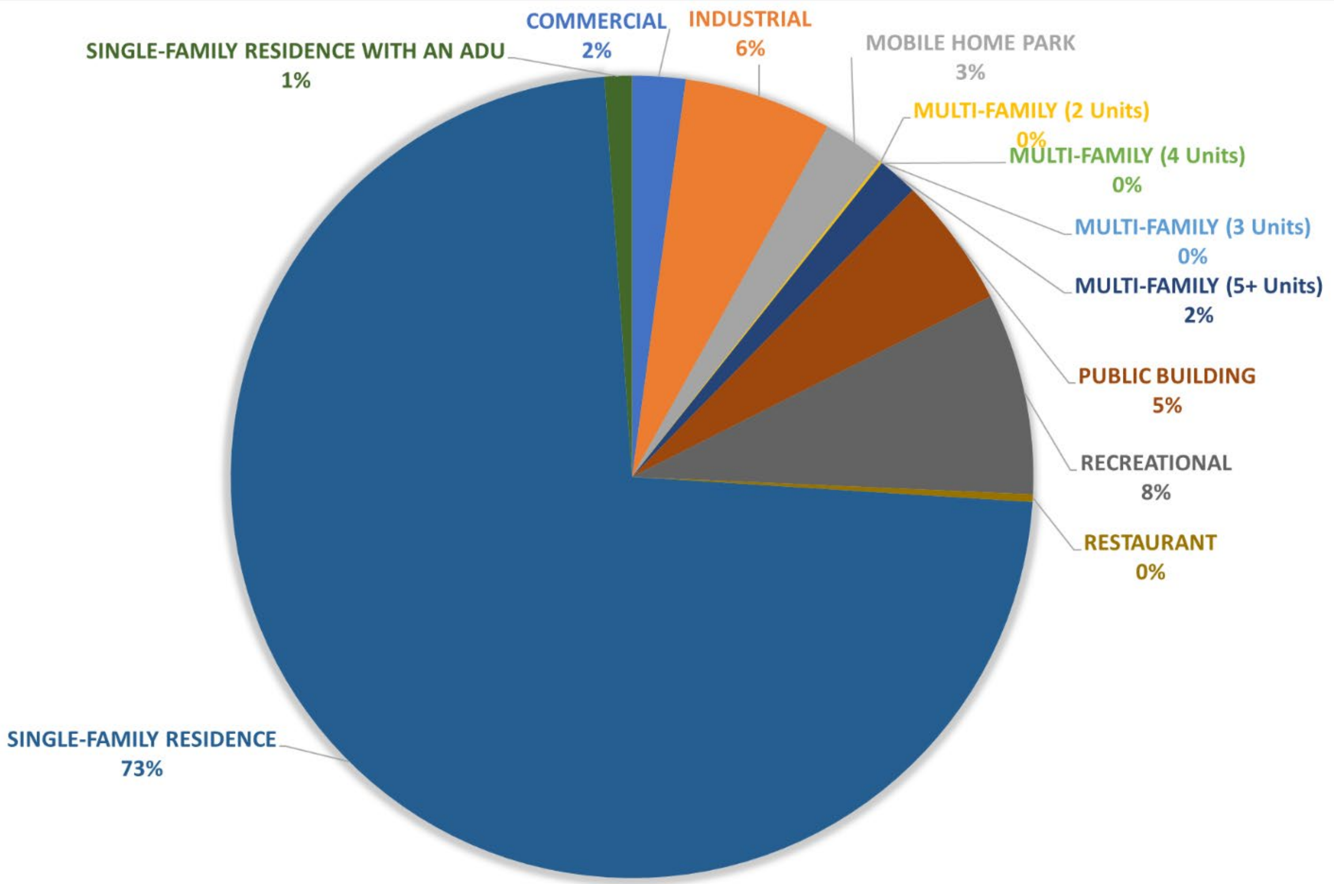
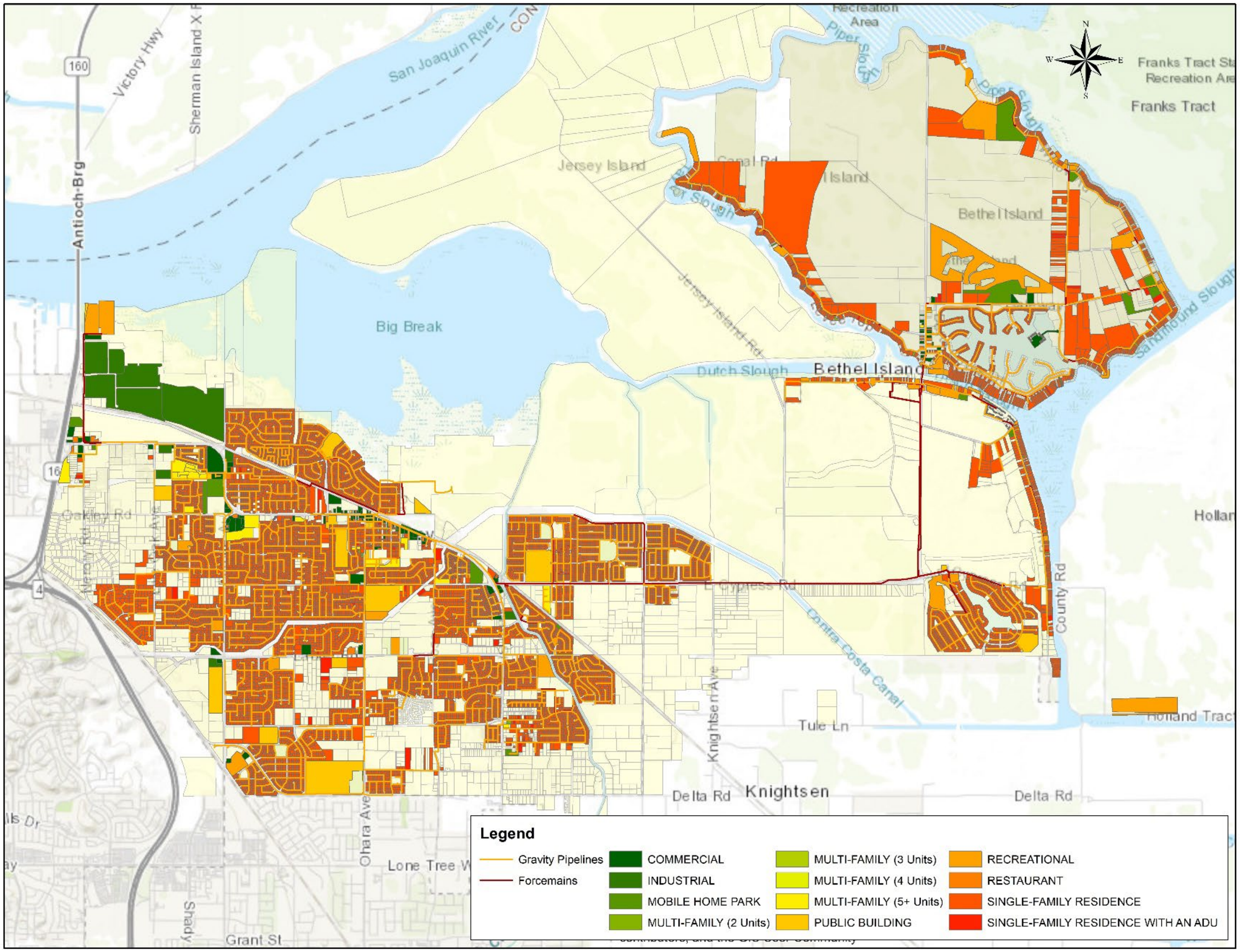


Figure 2.5 Parcels and Land Uses



2.3 INITIAL ESTIMATE OF SYSTEM FLOWS

Unit loads were applied to the District's land use factors to develop an initial estimated dry weather flow for each of the connected parcels. The flows were then adjusted as part of the model calibration process until system-wide dry weather flows were consistent with measured flows on February 27, 2024. Specific basins were further adjusted on a case by case basis. The final unit load factors by land use are shown in Table 2.3. Additional information on unit flows is presented in Chapter 4, Hydraulic Model Development.

Table 2.3 Unit Load Factors by Land Use

Customer Class	Modeled Unit Flow
Single Family Residential	120 gallons per unit
Single Family Residential + ADU	180.5 gallons per unit
Multifamily Residential (2 units)	375 gallons per acre
Multifamily Residential (3 units)	500 gallons per acre
Multifamily Residential (4 units)	750 gallons per acre
Multifamily Residential (5 units)	750 gallons per acre
Mobile Home	500 gallons per acre
Commercial	750 gallons per acre
Industrial	750 gallons per acre
Restaurant	750 gallons per acre
Public Building	118.75 gallons per acre
Recreational	0 gallons

CHAPTER 3 FLOW MONITORING PROGRAM

The purpose of this Chapter is to summarize key components and findings from the District's 2024 flow monitoring program that was conducted by V&A Consulting Engineers. The full report from V&A Consulting Engineers is included in Appendix A.

This Chapter is organized as follows.

- Introduction
- Flowmeter Locations
- Flow Integrity
- System Flows

3.1 INTRODUCTION

From February 16 to March 17, 2024, V&A Consulting Engineers conducted a flow monitoring program for the District. This program collected flow data using two temporary flowmeters and 5 pump station data loggers. The flowmeters were placed in locations that would capture the majority of system flows that do not pass through an existing pump station. The five pump station data loggers were located in stations that would help to provide information on the dry and wet weather flow conditions on Bethel Island and the Summer Lakes development.

Three rainfall events occurred during the flow monitoring period. These events are shown on Figure 3.1 on the following page. The flow monitoring study included three publicly-owned rain gauges that captured rainfall from the following locations:

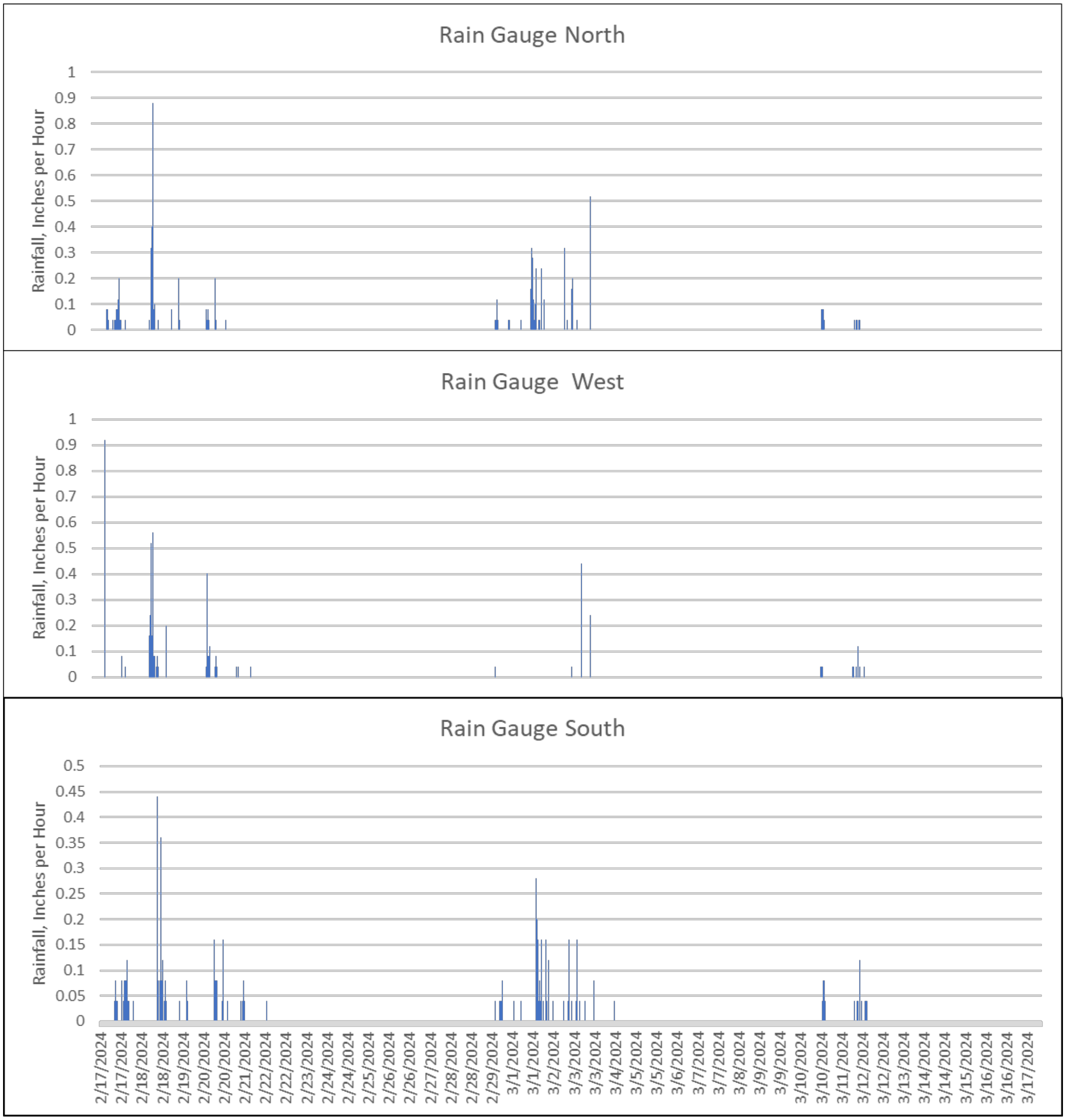
1. Rain Gauge West was located near Bridgehead Road, north of Main Street. This gauge captured 1.09 inches of rain over the flow monitoring period..
2. Rain Gauge North was located on Bethel Island, near the intersection of Piper Road and Willow Road. This gauge captured 2.57 inches of rain over the flow monitoring period.
3. Rain Gauge South was located near Main Street and Malicoat Avenue. This gauge captured 2.25 inches of rain over the flow monitoring period.

V&A used a mathematical equation to triangulate and assign rainfall to each of the metered locations.

Between February 17 and February 20, 2024, the District received approximately 1.49 inches of rain near Rain Gauge South. Since this amount of rain fell over multiple days, the event was too small to be categorized in terms of recurrence interval. However, during this event, the meters recorded measurable wet weather response. Most importantly, the meters confirmed that the system on Bethel Island received a substantial amount of dry weather groundwater infiltration.

February 27, 2024 was selected as the representative dry weather flow day, as the two temporary flowmeters indicated that wet weather response from the prior rainfall events had receded by this date. The District's BWWF as measured on February 27, 2024 was approximately 2.7 million gallons per day ("mgd"). The flow was calculated by calibrating the hydraulic model to measured flows and then recording the predicted total system flow at the District's WRF. This value is consistent with measured flow recorded at the WRF on this date.

Figure 3.1 Rainfall Received during 2024 Flow Monitoring Period



Approximately 0.53 mgd of this flow is attributed to dry weather GWI from Bethel Island[8]. The measured flow, less calculated I&I, translates to approximately 53 gallons per capita per day.

[8] After completion of the flow study, the District completed the Bethel Island I&I Reduction Project. Following the completion of this project, lower flows were observed at the WRF.

3.2 FLOWMETER LOCATIONS

The 2024 flow monitoring program included two temporary meters and five pump station data loggers. The District meters were calibrated by V&A before the beginning of the flow monitoring period. Figure 3.2 on the following pages shows the metered sewer basins.

Table 3.1 lists the meters, their associated manhole ID, pipe diameter, and meter location. The two gravity pipeline flow monitors were installed in the pipe directly upstream of the named manhole.

Table 3.1 Meter Names, Locations, and Whether Temporary or Permanent

Meter Name	Manhole ID	Pipe Diameter	Location
FM01	SEC002	24”	Main Street south of E. Cypress Road
FM02	OT040	16”	2nd Street south of Main Street
Ironwood PS		PS	5286 Ironwood Lane
Stone 2 PS		PS	4420 Windsweep Road, Bethel Island
Summer Lakes 1 PS		PS	Summer Lakes Drive north of E. Cypress Road
Taylor 1 PS		PS	2358 Taylor Road, Bethel Island
Willow 1 PS		PS	4315 Willow Road, Bethel Island

The connectivity of the District’s basins is shown on Figure 3.3. Dark blue squares indicate pump station and/or gravity system flows that were measured during the 2024 flow monitoring program.

Figure 3.2 Metered Sewer Basins

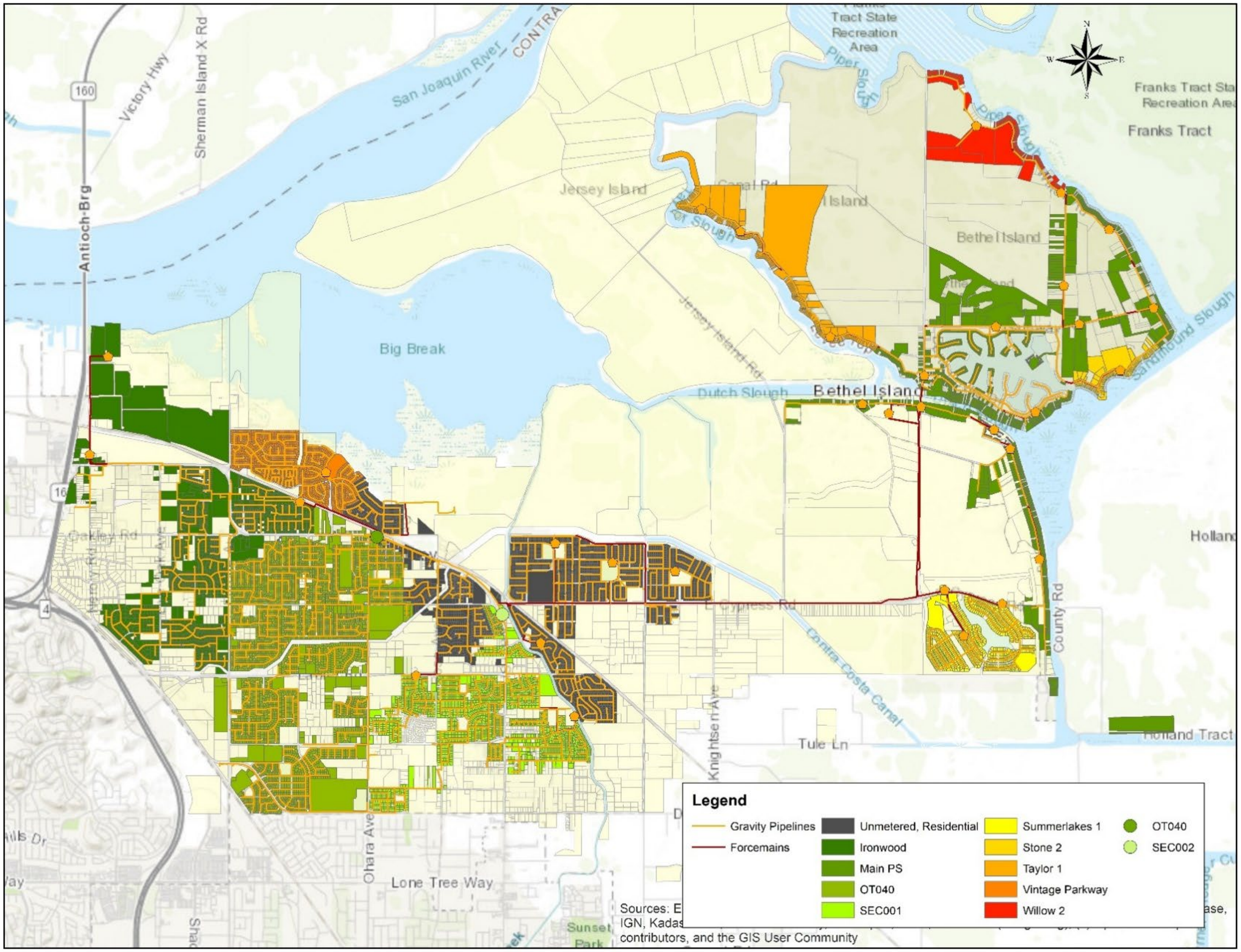
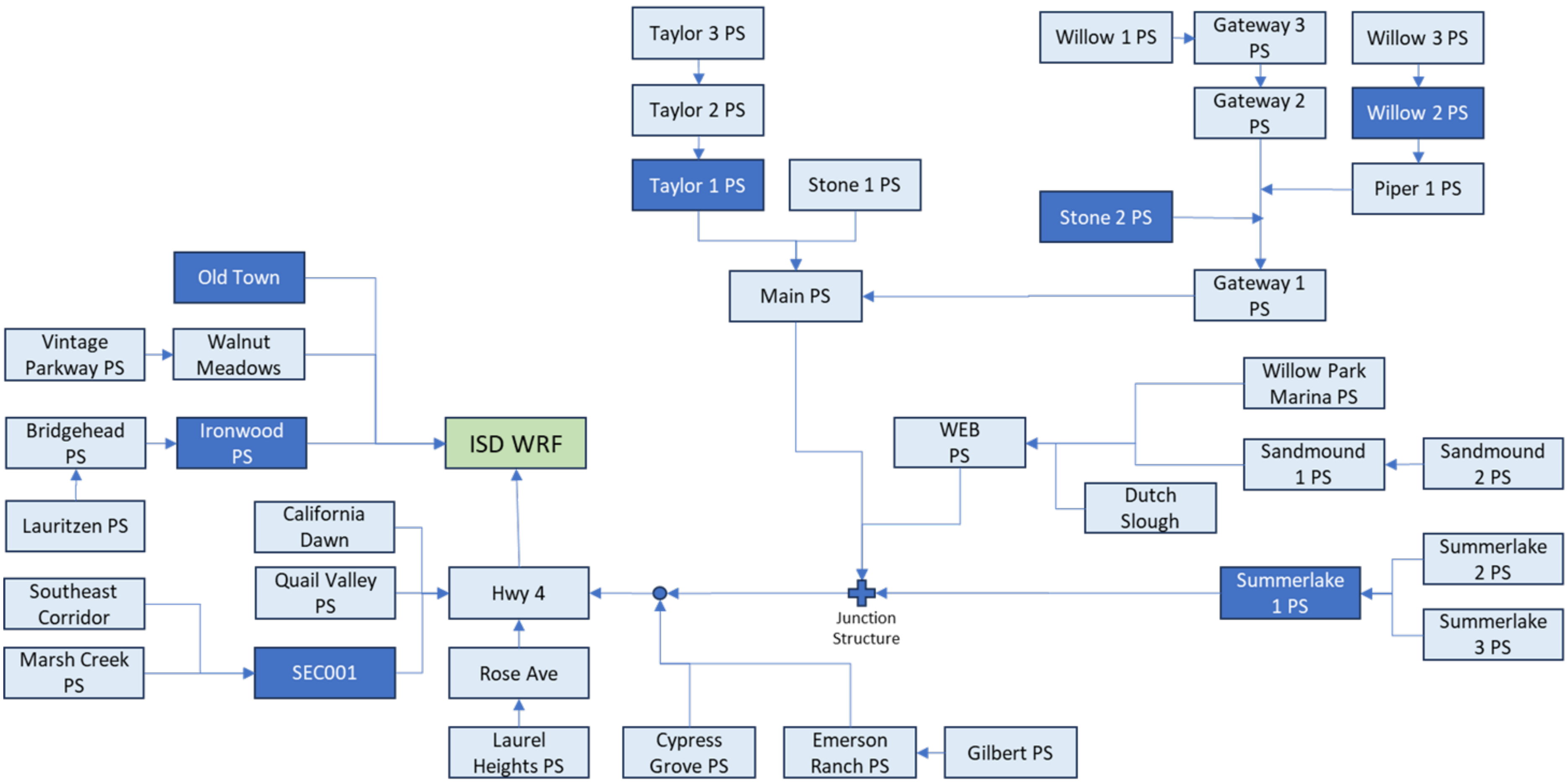


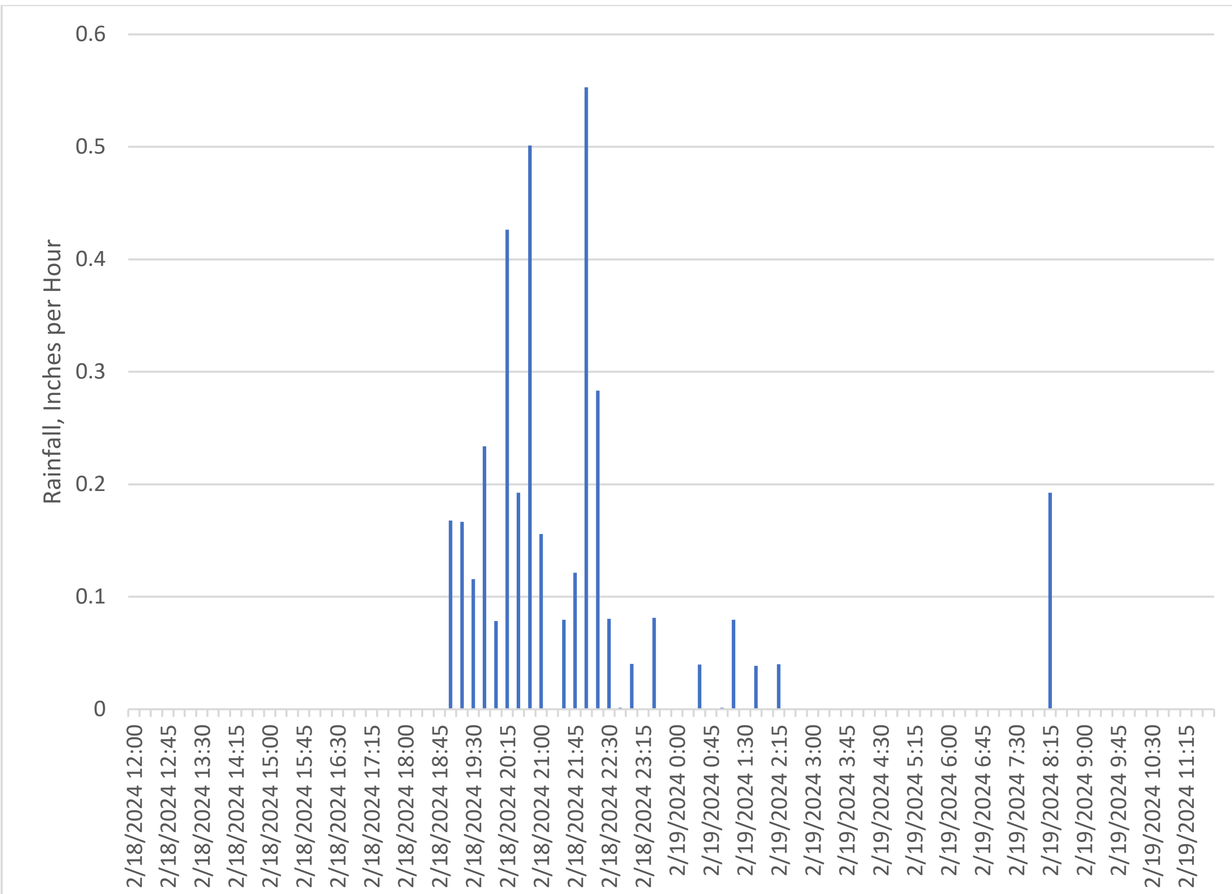
Figure 3.3 Basin Connectivity



The February 18 to 19, 2024 rainfall event was selected as the calibration storm for four reasons: 1) preceding rainfall events created favorable antecedent conditions (i.e., groundwater saturation); 2) this was the largest event that occurred during the flow monitoring period; 3) the system exhibited distinct response characteristics; and 4) the system had no known spills during this period.

Figure 3.4 shows the rainfall that is reported in the V&A Flow Monitoring Report at manhole OT040 during the selected calibration event from February 18 to 19, 2024.

Figure 3.4 Calibration Rainfall Event 1 (February 18 to 19, 2024)



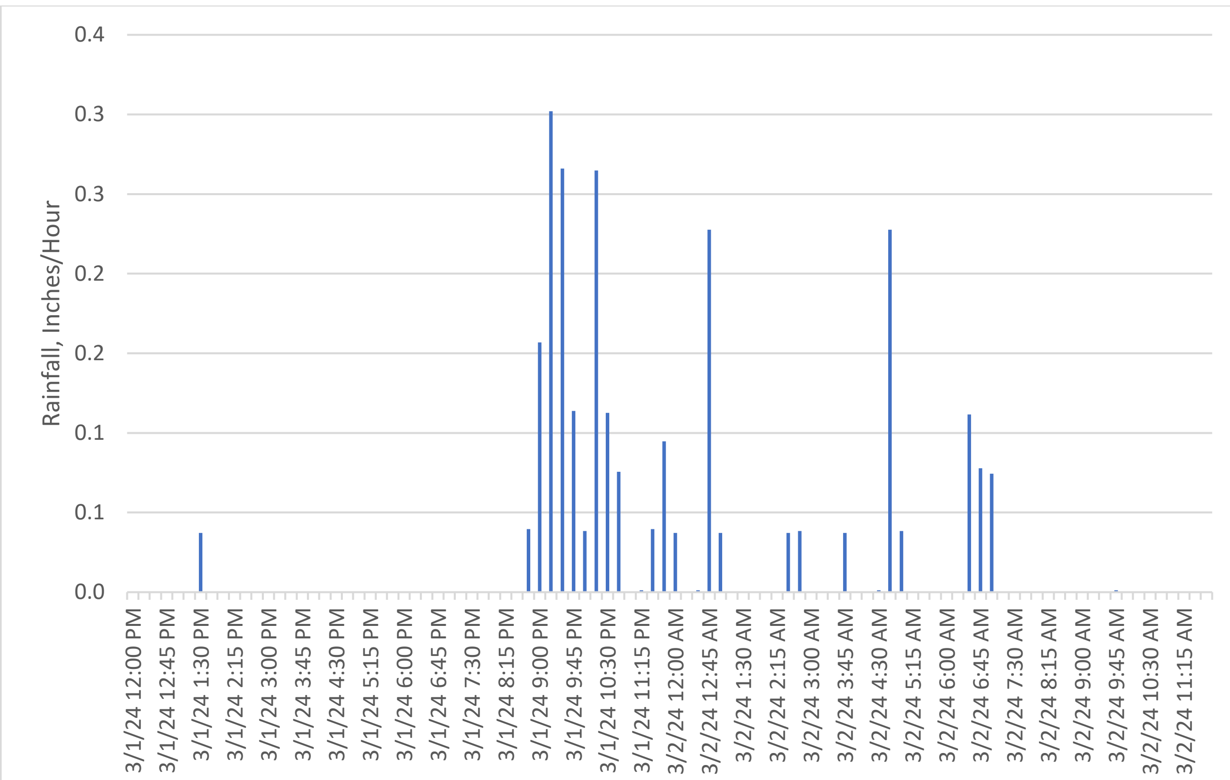
The pump station data loggers became active later in the flow monitoring period. Therefore, a March date was selected as the wet weather calibration event for the pump station data loggers as follows:

1. Ironwood pump station. Data logger was in service on February 16. Therefore, the February 18 to 19 rainfall event was selected for wet weather calibration.
2. Taylor 1 pump station. Data logger began collecting data on February 22, 2024. The March 2 rainfall event was selected for wet weather calibration.
3. Willow 2 pump station. Data logger began collecting data on February 22, 2024. The March 2 rainfall event was selected for wet weather calibration.

4. Stone 2 pump station. Data logger began collecting data on February 22, 2024. The March 2 rainfall event was selected for wet weather calibration.
5. Summer Lakes 1 pump station. Data logger began collecting data on February 22, 2024. The March 2 rainfall event was selected for wet weather calibration.

Figure 3.5 shows the rainfall that is reported in the V&A Flow Monitoring Report at the Taylor 1 pump station during the second calibration date of March 1-2, 2024.

Figure 3.5 Calibration Rainfall Event 2 (March 1-2, 2024)



3.3 SYSTEM FLOWS

This section summarizes wastewater system dry and wet weather flow characteristics as measured during the 2024 flow monitoring program.

3.3.1 Base Wastewater Flows and Dry Weather Groundwater Infiltration

Table 3.2 on the following page lists average dry weather flows that were measured at the two temporary flowmeters and five pump station data loggers on February 27, 2024. Table 3.2 also shows estimated dry weather flows for the parcels that were not captured by these seven meters. Dry weather groundwater infiltration was estimated by applying trends measured by the data loggers at Willow 2, Taylor 1, and Stone 2 pump stations to the other stations on Bethel Island and directly to the south. The estimated GWI flow was calculated as the difference between actual (measured) dry weather flow and expected dry weather flow based on land uses. After reviewing calculated GWI values, it appears that the Taylor, Willow, and Stone 2 pump station influent sewer pipelines can accommodate as much as, but possibly not more than 100,000 gpd of dry weather GWI.

Dry weather GWI was added to the Sandmound 1 and 2, Willow Park Marina, Dutch Slough, WEB, Gateway, Willow 1 Stone 1, Piper 1, and Main pump station wastewater basins in the same proportion as found in the Taylor, Willow, and Stone 2 basins. For these other expected dry weather flow based on land use was assumed to comprise 20 percent of dry weather flow. The remaining 80 percent of flow, to a maximum limit of 100,000 gallons, was assumed to enter as dry weather GWI..

Table 3.2 Average Dry Weather Flows (February 27, 2024)

Meter or Node Location	Measured ADWF (gpd)	Calibrated DWF Based on Land Use (gpd)	Calculated Dry Weather GWI (gpd)	DWF to Use (gpd)	% GWI of Measured Flow
FM1 (SEC001)	274,300	263,700	--	274,300	--
FM2 (OT040)	342,000	413,570	--	342,000	--
Ironwood Data Logger	108,600	154,700	--	108,600	--
Taylor 1 Data Logger	115,000	33,400	81,600	115,000	70.9%
Willow 2 Data Logger	97,800	16,750	81,050	97,800	82.9%
Stone 2 Data Logger	102,900	5,000	97,900	102,900	95.1%
Summerlake 1 Data Logger	71,300	78,500	--	71,300	--
Cypress Grove; Emerson; Gilbert; TAP2	--	246,064	--	246,064	--
Vintage Park; Walnut Meadows	--	159,750	--	159,750	--
Rose Ave; Laurel Heights	--	95,600	--	95,600	--
Sandmound 1 and 2; Willow Park Marina; Dutch Slough; WEB	--	52,140	248,000	300,140	83%
Gateway 1,2,3; Willow 1; Stone 1; Piper 1; MPS	--	110,300	386,000	496,300	77.8%
Hwy 4; Quail Valley; California Dawn	--	282,750		282,750	--
Total				**2,692,504**	

As shown on Table 3.2, after BWWF and dry weather GWI were combined, the expected ADWF for the system on February 27, 2024 was 2.7 mgd. This value is consistent with the flow that was measured at the District's water reclamation facility.

3.3.2 Wet Weather Flow

Wet weather flow is created when rainfall-dependent inflow and infiltration ("RDII") enters the gravity sewer system during wet weather events.

RDII is the collective description for stormwater and groundwater that enters the sewer system through pipe defects and unpermitted direct connections. Inflow describes water that enters through structures such as roof leaders and private drains, or from holes in manhole covers. Infiltration describes water that enters through defects in pipes, joints, and manhole walls such as cracks, open joints, or breaks.

Figure 3.6 shows sources of RDII as presented in Figure 2-4 of the V&A report. Wet weather peaking factors are calculated as peak flow divided by average flow. Every wet weather event has a unique wet weather peaking factor and smaller basins usually have higher peaking factors. Therefore, peaking factor alone cannot be used to assess whether a basin has high I&I.

Figure 3.6 Typical Sources of Infiltration and Inflow from Figure 2-4 of V&A Report

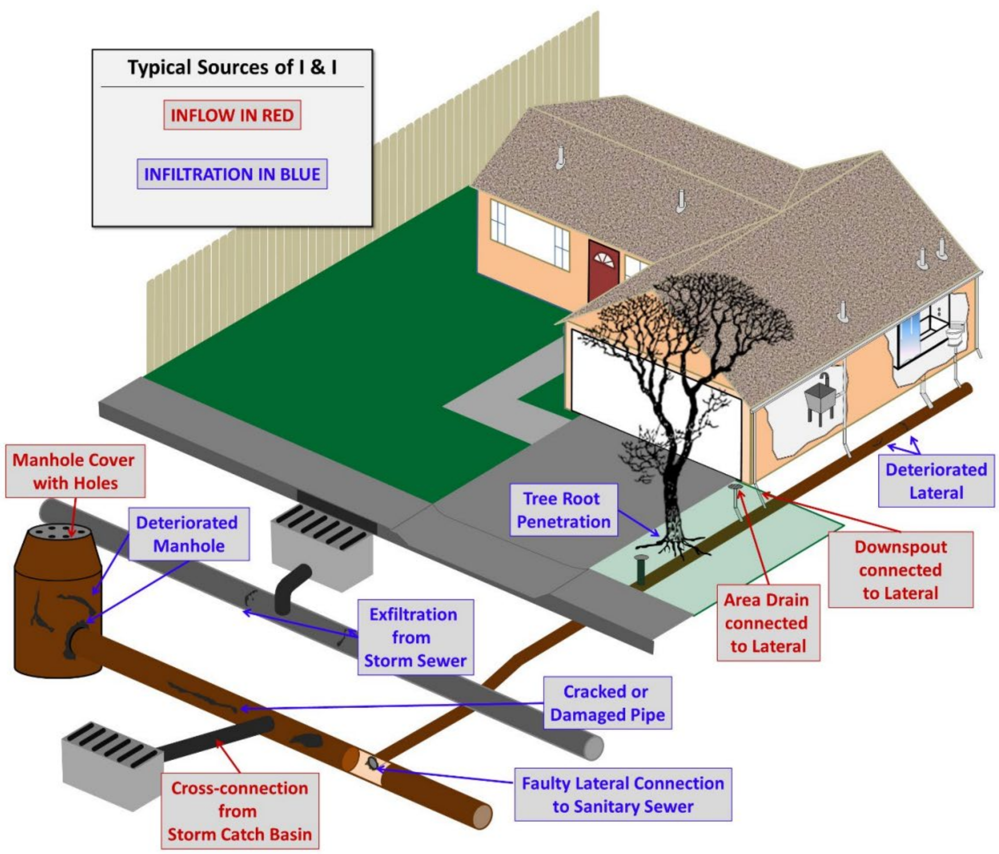


Figure 2-4. Typical Sources of Infiltration and Inflow

Table 3.3 shows average wet weather flow measured on February 19, 2024 at SEC001, OT040, and Ironwood pump station, and on March 3 at the remaining metered pump stations. The purpose of this comparison was to determine whether the high volumes of dry weather groundwater found in the Taylor 1, Willow 2, and Stone 2 data could be considered sufficient to prevent additional RDII from entering pipes within these sewer basins.

Ironwood pump station flows reached an average value of approximately 123,000 gpd, indicating that this is the maximum flow that the basin will convey, independent of storm size. Taylor 1 pump station flows do not appear to be impacted by wet weather, potentially because the basin sewer pipes are full due to dry weather groundwater. The remaining pump station loggers show that during wet weather events, additional RDII does enter the stations above and beyond dry weather GWI.

Table 3.3 Average Wet Weather Flow vs Average Dry Weather Flow

Meter or Node Location	Measured ADWF on 2/27/2024 (gpd)	Measured Average WWF on 2/19/2024 (gpd)	Measured Average WWF on 3/3/2024 (gpd)	Max Percent Change (%)
FM1 (SEC001)	274,300	325,500	282,000	20%
FM2 (OT040)	342,000	596,800	583,300	75%
Ironwood Data Logger	108,600	120,300	123,400	14%
Taylor 1 Data Logger	115,000	--	111,400	--
Willow 2 Data Logger	97,800	--	136,500	40%
Stone 2 Data Logger	102,900	--	124,900	21%
Summerlake 1 Data Logger	71,300	--	91,500	28%

A review of minimum wet weather flows provided additional information on whether additional RDII could enter a pipe that is already full with GWI. A study of minimum flows during dry and wet weather is summarized in Table 3.4 on the following page. Table 3.4 shows that minimum flows measured at the Ironwood PS, Willow 2 PS, and Stone 2 PS were elevated during the wet weather events. Minimum flows at the Taylor 1 and Summerlake 1 pump stations during wet weather did not change from dry weather. The results from the data reviews summarized in Tables 3.3 and 3.4 are consistent and confirm that Ironwood PS should have a mild response to wet weather flow, whereas Willow 2 and Stone 2 pump stations have a greater capacity for increased flow due to RDII.

Table 3.4 Review of Minimum Flows during Dry and Wet Weather Periods

Meter or Node Location	Minimum DWF on 2/27/2024 (gpd)	Minimum WWF on 2/19/2024 (gpd)	Minimum WWF on 3/3/2024 (gpd)	Max Percent Change (%)
FM1 (SEC001)	77,900	67,600	75,500	--
FM2 (OT040)	73,500	71,900	74,200	--
Ironwood Data Logger	29,750	34,500	39,600	33%
Taylor 1 Data Logger	61,100	--	57,900	--
Willow 2 Data Logger	48,000	--	89,000	85%
Stone 2 Data Logger	95,000	--	111,000	17%
Summerlake 1 Data Logger	25,000	--	21,000	--

A study of peak flows and peaking factor is presented in Table 3.5 on the following page. PWWF divided by ADWF represents the wet weather peaking factor for a given wet weather event. Wet weather peaking factor was calculated in two ways for basins with dry weather GWI. For these basins, which drain to the Taylor 1, Willow 2, and Stone 2 pump stations, a wet weather peaking factor was calculated for the basin with and without consideration of dry weather GWI.

As shown on Table 3.5, the sites without dry weather GWI (not highlighted) have a wet weather peaking factor of approximately 3. The Ironwood basin was less sensitive to the size of the rainfall event and reached a maximum flow of approximately 250,000 gpd.

The three pump stations with dry weather GWI (highlighted in green) had lower wet weather peaking factors of 1.6 to 2.2. However, when dry weather GWI was removed, wet weather peaking factors increased, ranging from 5.28 to 13.07[9]. This calculation confirms that the pipes within basins that contribute dry weather GWI are likely to admit less RDII because they are already partially full with groundwater when the wet weather event occurs.

The hydraulic model estimates wet weather flow as a percentage of the volume of rainfall that falls during a given storm. Therefore, as the storm gets larger, the amount of model-generated RDII and, as a result, model-generated peak wet weather flow increases. The model does not consider whether groundwater will eventually fill up and stop entering the pipe, as reflected in the data that is presented above. Therefore, the model results are expected to be conservative for the basins with significant dry weather groundwater infiltration (i.e., in basins with dry weather GWI, the model is expected to overpredict wet weather flow). This information was considered during the analysis of system flows as discussed in Section 5.

[9] Stone 2 shows a WWPF of 32.6. However, since this basin is very small, this WWPF cannot be compared directly to Taylor 1 and Willow 2 wet weather peaking factors.

Table 3.5 Wet Weather Peaking Factor on 3/3/2024

Meter or Node Location	(A) Measured DWF on 2/27/2024 (gpd)	(B) DWF on 2/27/2024 without Dry Weather GWI (gpd)	(C) Peak WWF on 2/19/2024 (gpd)	(D) Peak WWF on 3/3/2024 (gpd)	WWPF with GWI Max of (C) or (D) / (A)	WWPF without GWI Max of (C) or (D) / (B)
FM1 (SEC001)	274,300	274,300	788,300	481,800	2.87	2.87
FM2 (OT040)	342,000	342,000	1,065,700	1,061,300	3.12	3.12
Ironwood Data Logger	108,600	108,600	248,200	250,300	2.30	2.30
Taylor 1 Data Logger	115,000	33,400	--	176,300	1.53	5.28
Willow 2 Data Logger	97,800	16,750	--	219,000	2.24	13.07
Stone 2 Data Logger	102,900	5,000	--	163,000	1.58	32.60
Summerlake 1 Data Logger	71,300	71,300	--	209,000	2.93	2.93

CHAPTER 4 HYDRAULIC MODEL DEVELOPMENT

The purpose of this Chapter is to summarize assumptions, items considered during model development and calibration, and key components of the completed hydraulic model.

This Chapter is organized as follows.

1.1 Model Components

1.2 Wastewater Loads

1.3 Model Calibration

The 2025 hydraulic model was developed using Innovyze InfoWorks ICM software. The model can be used as a tool for assessing the flows and capacities of the District's trunk sewers and for identifying solutions to pipeline capacity issues. The hydraulic model is also a tool for performing "what if" scenarios to assess the impacts of future developments, land use changes, and system configuration changes.

The hydraulic model includes the District's trunk sewers (10-inch diameter and larger) and associated pipelines and is a skeletonized representation of the wastewater collection system in its configuration and operation. The model also includes smaller diameter sewers as needed to provide system connectivity and to include pump station facilities.

4.1 MODEL COMPONENTS

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

The hydraulic model comprises a skeletonized network of nodes (*e.g.*, manholes) and conduits (*e.g.,* pipelines). The following descriptions provide additional information on elements used in the hydraulic model.

- Nodes represent manholes, split manholes, and lift station wet wells. The existing system model network discharges to a single an open outfall on the west side of the District's WRF. The future network includes a second force main that discharges to a second open outfall on the south side of the WRF.
- All flows are applied at node structures. The data required for node structures include elevation data (pipe invert and manhole rim), manhole diameter, and whether the system is open or sealed.
- Conduits represent facilities that convey wastewater from one point in the system to another. Conduits include gravity pipes, force mains, and pumps. The physical data for gravity pipes and force mains include invert elevation, size, length, and friction factor. The physical data for pumps include type of pump, elevation, pump capacity, and operational parameters such as

on/off setpoint elevations and sequencing.

- Subcatchments represent a tributary area that flows to an individual node in the model. Each parcel in the system is assigned a subcatchment and this subcatchment is then connected to the nearest trunk sewer manhole. The subcatchment layer serves several purposes, including defining land use, assigning diurnal curves, and assigning dry and wet weather flow inputs. The data required for subcatchments are node connection, land use, flow factors, total and contributing area, diurnal curve profile, rainfall profile, I&I parameters, and groundwater parameters.

Pipelines and Manholes

The initial model network was developed using the District's *ISD_Sewer_Mains, ISD_Sewer_Nodes, ISD_Parcels, and ISD_Pump Stations,* shapefiles. Model development and validation involved the following steps:

- All pipes 10-inches and diameter were isolated from the GIS layers. These pipes were inspected to find locations where pipes were discontinuous or otherwise ambiguous. The GIS file was then reviewed further to identify smaller diameter pipes, force mains, or other infrastructure needed to assure connectivity and these lines were added to the trunkline network. The resulting network has upstream and downstream manhole IDs, pipe sizes, and lengths for each pipe segment.
- At the same time, node structures associated with the pipes discussed above were added to the network. Manhole rim and invert elevations were added to the network using available information from the District's GIS layers. In cases where a single invert elevation was provided for a manhole that had multiple pipes attached, the invert was assumed to be the elevation of the center of the manhole base.
- Rim elevations and invert elevations for the remaining pipes were interpolated and adjusted to create a continuous slope from top to bottom of each pipe reach. The following approach was taken to assign rim and invert elevations:
    - Inverts were first set at six feet below the rim elevation
    - Where gravity flow could not be maintained using these elevations, inverts were adjusted using engineering judgment to first provide minimum allowable slopes, and then provide continuous slopes relative to adjacent manhole and pipe structures
    - Where necessary, ground surface elevations were reviewed using Google Earth to reconcile locations where the pipe appeared to get too close to the ground surface using the approach above
    - Where flows split between two pipes, the larger pipe was assumed to take the bulk of the flow. The invert for smaller downstream pipe was raised to match the crown of the primary downstream pipe to avoid dry weather flow splitting at this intersection.
    - All gravity pipelines were assigned a Manning's friction factor ("n") of 0.013
    - Force main pipelines were assigned a Hazen-Williams Coefficient of 130

The hydraulic model consists of 862 pipe segments comprising approximately 36.7 miles of gravity pipelines and 15.4 miles of force main pipe. Modeled gravity pipelines range from 6 to 36 inches in diameter. The model includes all 10-inch diameter and larger trunk lines, and associated manholes plus additional smaller diameter pipelines. The 36.7 miles of pipeline represent approximately 26 percent of the District's gravity collection system.

The hydraulic model includes 29 of the District's pump stations and associated force main pipes. Force mains vary in diameter from 4 to 18 inches. The modeled collection system pipelines are shown on Figure 4.1 on the following page.

Pump Stations

Table 4.1, which follows Figure 4.1, lists information used to model the District's pump stations. Each station includes fixed pumps with specific pumping capacities and on and off elevations to define pump setpoints. Pump station capacities and wet well configurations were provided by the District or obtained from as-built drawings. Pump setpoints were approximated based on the wetwell depth, unless shown on drawings.

Subcatchments

The District's service area includes 15,822 parcels. Each parcel is represented in the model as a subcatchment with an assigned land use. Loads were developed and assigned to each subcatchment based on the assigned land use, as follows:

- Single Family Residential land uses (SFR and SFR + ADU) were assigned a flow per parcel based on the assigned land use
- Multifamily Residential land uses (2 units, 3 units, 4 units, and 5 units), as well as Mobile Homes were a flow per acre
- All other land uses (commercial, industrial, restaurant, public building, recreational) were assigned a flow per acre based on the assigned land use. The applicable acreage for buildings on large parcels with substantial undeveloped land was reduced on a parcel by parcel basis.

Unit flows and land uses are discussed in more detail later in this Chapter, as part of the discussion on dry weather calibration.

Figure 4.1 Modeled Gravity Sewer and Force Main Pipelines

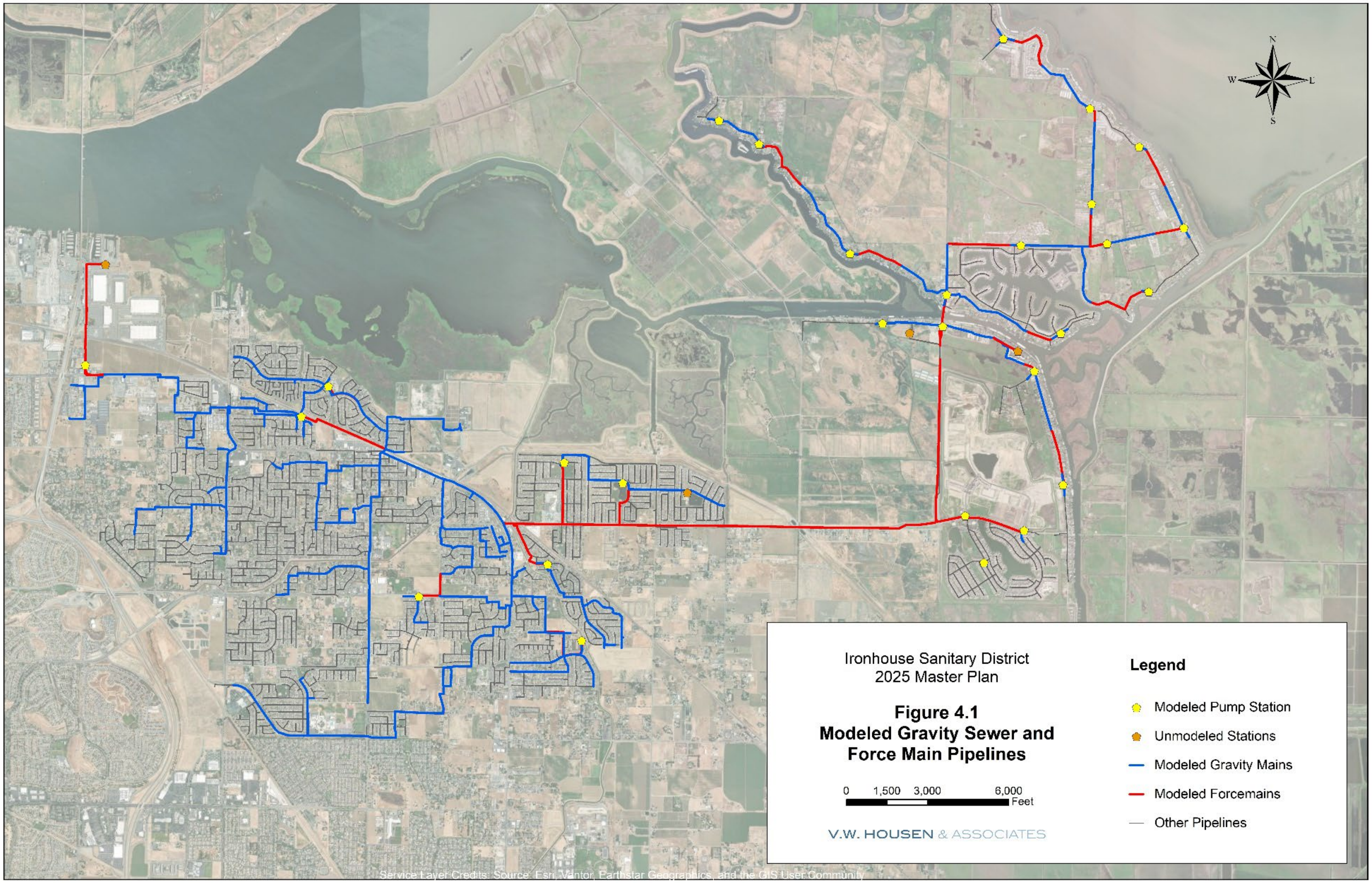


Table 4.1 Pump Station Parameters

Pump Station Name	Modeled Wet Well Size	Modeled Ground Elevation (ft)	Modeled Wet Well Floor Elevation (ft)	No. of Pumps	Pumping Capacity (gpm) (Note 1)	Modeled Pump On / Off Elevations (ft)
Bridgehead	8' diameter x 13.7' deep	16.3	2.6	2	180	5.9'/5.1'
Cypress Grove	8' diameter x 20.2' deep	6.8	-13.4	2	355	-9.2' / -10.9'
Dutch Slough	8' diameter x 17.9' deep	-2.0	-19.9	2	150	-13.6' / -16.1'
Emerson Ranch	Not Modeled	--	--	--	--	--
Gateway 1	10' diameter x 21.6' deep	-7.0	-28.6	2	836	-21.1' / -25.3'
Gateway 2	8' diameter x 14.5' deep	-4.5	-19.0	2	184	-14.4' / -16.5'
Gateway 3	8' diameter x 18.4' deep	0.0'	-18.4	2	158	-14.2' / -15.9'
Gilbert Ranch 1	Not modeled	--	--	--	--	--
Ironwood	17.1' X 9' X 23.6' deep	11.8'	-11.8'	2	1000	-7.6' / -9.3'
Lauritzen	Not modeled	--	--	--	--	--
Laurel Heights	8' diameter x 15.2' deep	27.1'	11.9'	2	110	14.4' / 13.1'
Marsh Creek	10' diameter x 18.2' deep	28.6'	10.4'	2	329	13.7' / 12.5'
Main (MPS)	8' diameter x 20.8' deep	-3.0'	-23.8'	2	1400	-16.3' / -19.6'
Piper 1	8' diameter x 14.8' deep	-7.5'	-22.3'	2	277	-17.3' / -19.8'
Quail Valley	10' diameter x 20.5' deep	12.73'	7.8'	2	600	-14.0' / 11.6'
Stone 1	8' diameter x 13.1' deep	-0.5'	-13.6'	2	240	-9.4' / -11.1'
Stone 2	8' diameter x 14.7' deep	0.0'	-14.7'	2	242	-11.4' / -12.2'

Pump Station Name	Modeled Wet Well Size	Modeled Ground Elevation (ft)	Modeled Wet Well Floor Elevation (ft)	No. of Pumps	Pumping Capacity (gpm) (Note 1)	Modeled Pump On / Off Elevations (ft)
Summer Lakes 1	10’ diameter x 12.5’ deep[8]	-3.7’	-16.2’[10]	4[11]	1925	-10.4’ / -12.4’
Summer Lakes 2	6.1’ diameter x 14.2’ deep	-4.3’	-18.5’	2	350	-12.7’ / -15.2’
Summer Lakes 3	6’ diameter x 17.5’ deep	-3.7’	-21.2’	2	250	-15.8’ / -18.3’
Sandmound 1	8’ diameter x 16.1’ deep	1.0’	-17.1’	2	150	-13.3’ / -15.0’
Sandmound 2	8’ diameter x 19.5’ deep	-4.0’	-23.5’	2	190	-18.1’ / -21.0’
Taylor 1	8’ diameter x 14.6’ deep	0.0’	-14.6’	2	270	-9.2’ / -12.1’
Taylor 2	8’ diameter x 13.1’ deep	-7.5’	-20.6’	2	190	-16.0’ / -18.1’
Taylor 3	8’ diameter x 13.7’ deep	-6.5’	-20.2’	2	310	-16.4’ / -17.7’
Vintage Parkway	8’ diameter x 18.0’ deep	-5.7’	-23.7’	2	600	-18.7’ / -20.4’
Willow 1	8’ diameter x 13.4’ deep	-7.0’	-20.4’	2	150	-15.0’ / -18.0’
Willow 2	8’ diameter x 15.4’ deep	-5.5’	-20.9’	2	106	-15.5’ / -18.4’
Willow 3	8’ diameter x 14.3’ deep	-6.5’	-20.8’	2	200	-16.6’ / -18.3’
WEB	8’ diameter x 16.4’ deep	2.0’	-14.4’	2	796	-8.6’ / -11.1’
Willow Park Marina	Not modeled	--	--	--	--	--
Bethel Island Ponds	Not modeled	--	--	--	--	--

Note 1. Pumping capacity assumes the largest pump is out of service.

[10] To be confirmed for final report
[11] Current station configuration utilizes two of the four pump slots.

4.2 WASTEWATER LOADS

Wastewater loads or flows are divided into three categories:

- Base Wastewater Flow ("BWWF") is the average daily dry weather sanitary flow contribution from permitted connections to the collection system
- Groundwater infiltration ("GWI") includes a constant flow that is found in the flow monitoring in addition to BWWF. Different dry weather and wet weather GWI values were evaluated
- Average Dry Weather Flow ("ADWF") is the combination of BWWF and dry weather GWI
- Rainfall-dependent inflow and infiltration ("RDII") results when flows from wet weather events infiltrate the system through defects in existing wastewater collection system assets

4.2.1 Dry Weather Flow Generation

This section describes the tasks completed to calculate dry weather flows.

Dry Weather Sewer Flows

Dry weather flows were calculated by applying a unit flow factor to each parcel based on its land use. The key elements of dry weather flow generation in the hydraulic model include BWWF, Peak Dry Weather Flow ("PDWF"), and dry weather GWI.

The initial step in assigning BWWF in the hydraulic model was to assign a unit flow to each assigned land use designation. Following is the process that was followed to assigned BWWF.

1. Land use categories from the City of Oakley General Plan were grouped into a shortlist of land use descriptions that were consistent with the ISD tax roll records.
2. A unique land use designation was assigned to each modeled parcel within the service area.
3. Initial unit flows were assigned to each land use designation, following general protocols for sewer modeling.
4. The hydraulic model was used to generate flows for the service area. Model-generated flows were compared to the measured dry weather wastewater flow, taken from data for February 27, 2024.
6. The initial land use factors were adjusted globally for each land use category until general consistency was found between the model-generated and metered flows. Table 4.2 lists the final initial unit flow factors. These flow factors received further refinement for specific basins, as discussed later in this section.

Table 4.2 Unit Factors for Base Wastewater Flow

Customer Class	Modeled Unit Flow
Single Family Residential	120 gallons per unit
Single Family Residential + ADU	180.5 gallons per unit
Multifamily Residential (2 units)	375 gallons per acre
Multifamily Residential (3 units)	500 gallons per acre
Multifamily Residential (4 units)	750 gallons per acre
Multifamily Residential (5 units)	750 gallons per acre
Mobile Home	500 gallons per acre
Commercial	750 gallons per acre
Industrial	750 gallons per acre
Restaurant	750 gallons per acre
Public Building	118.75 gallons per acre
Recreational	0 gallons

Diurnal (24-Hour) Flows

24-hour diurnal patterns were developed for each monitored basin from the V&A 2024 flow monitoring data. Diurnal curves for the locations without measurable dry weather GWI are shown on Figures 4.2 and 4.3. In these figures, the peak water use/sewer discharge that occurs mid-morning and again in the evening, as well as lower use at night and mid-day, are visible.

Figures 4.4 through 4.6 show dry weather flows at Taylor 1, Willow 2, and Stone 2 pump stations on Bethel Island. Each of these figures also shows the calculated BWWF based on land use, and measured ADWF. The difference between these two values was allocated to the model as dry weather GWI. Dry weather GWI was added to the hydraulic model by adding a constant flow component to each parcel in each impacted basin[12]

[12] See footnote for Figure 4.4

Figure 4.2 Diurnal Patterns Upstream of Manholes SEC001 and OT040

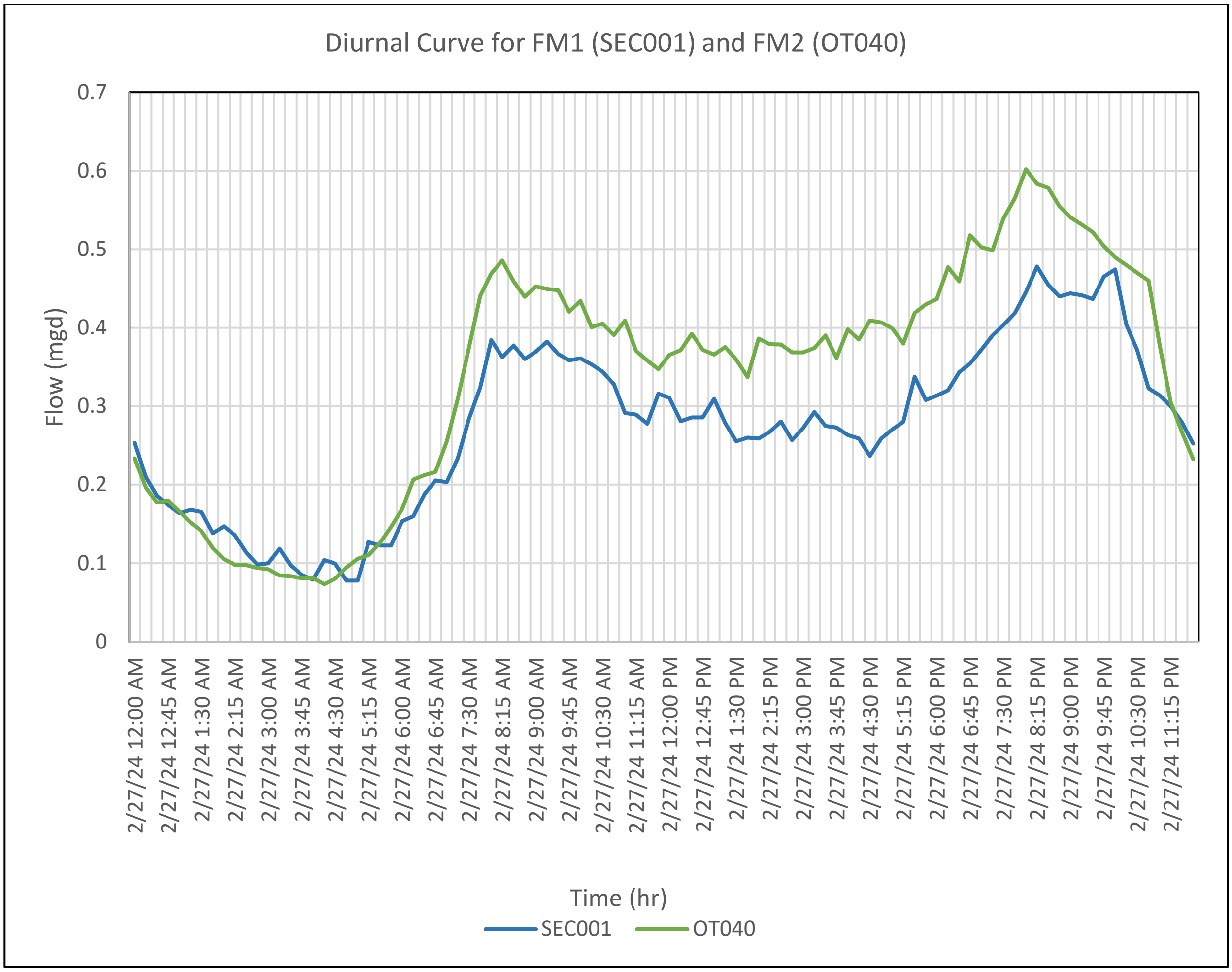


Figure 4.3 Diurnal Patterns Upstream of the Ironwood PS and Summer Lakes 1 PS

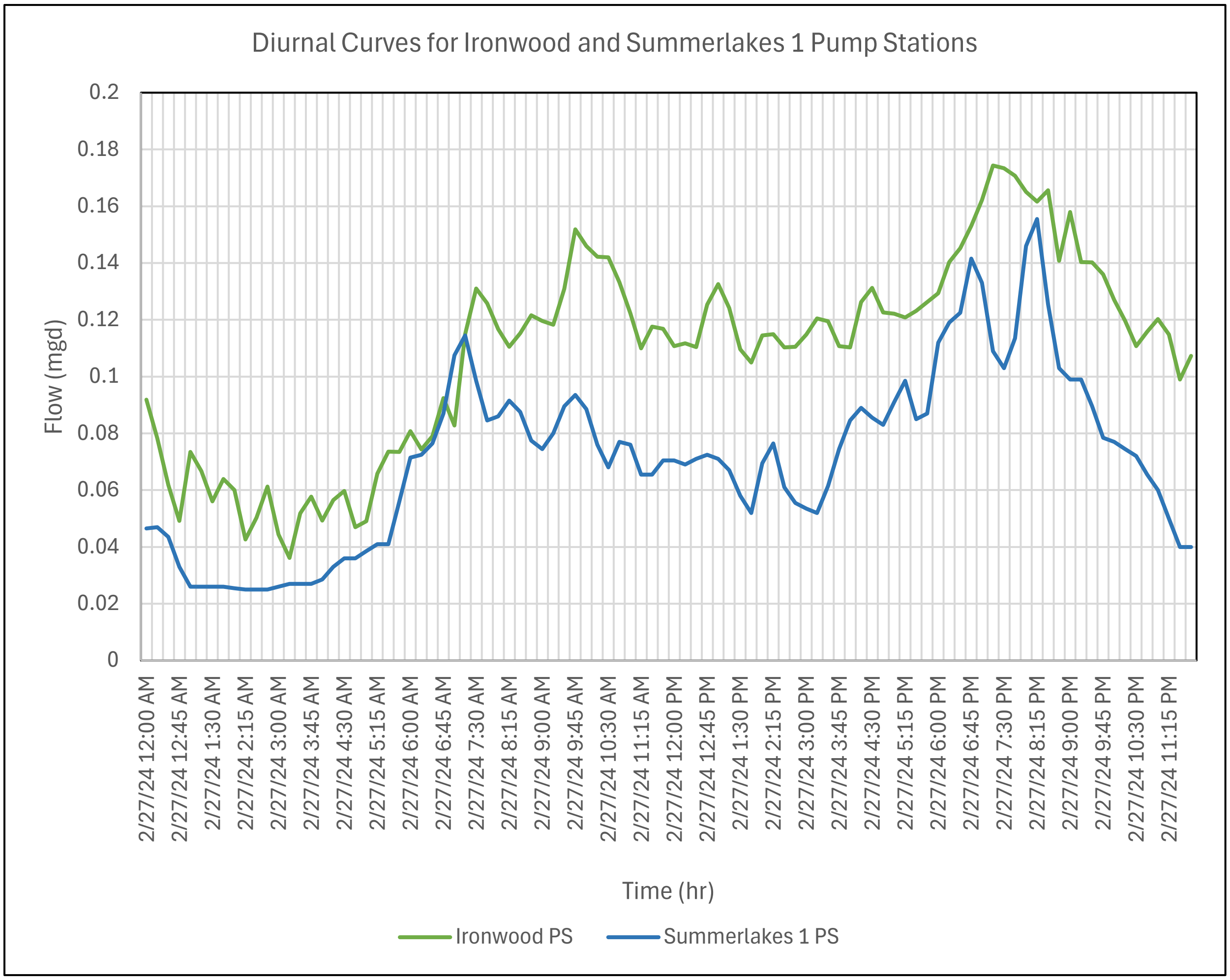
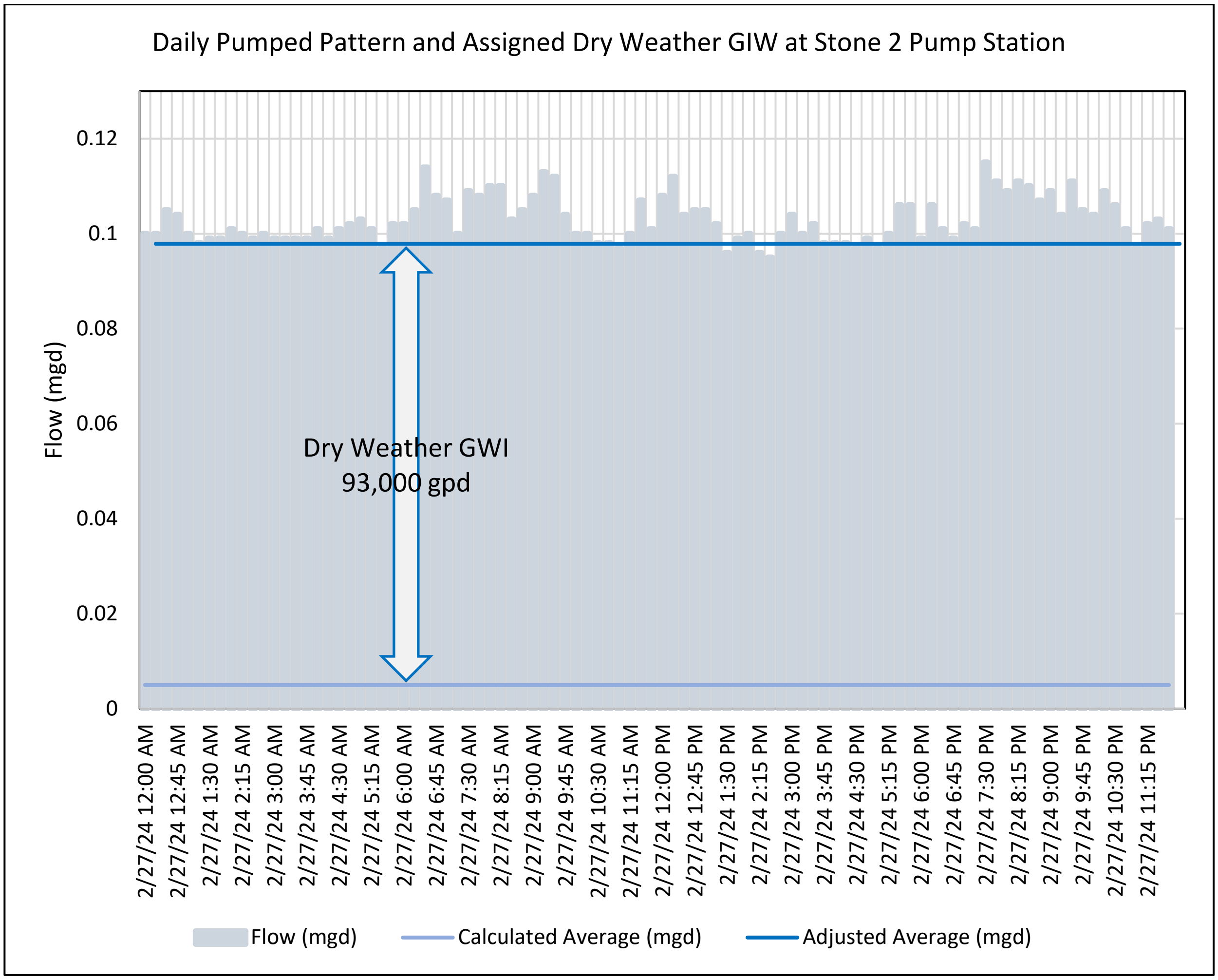
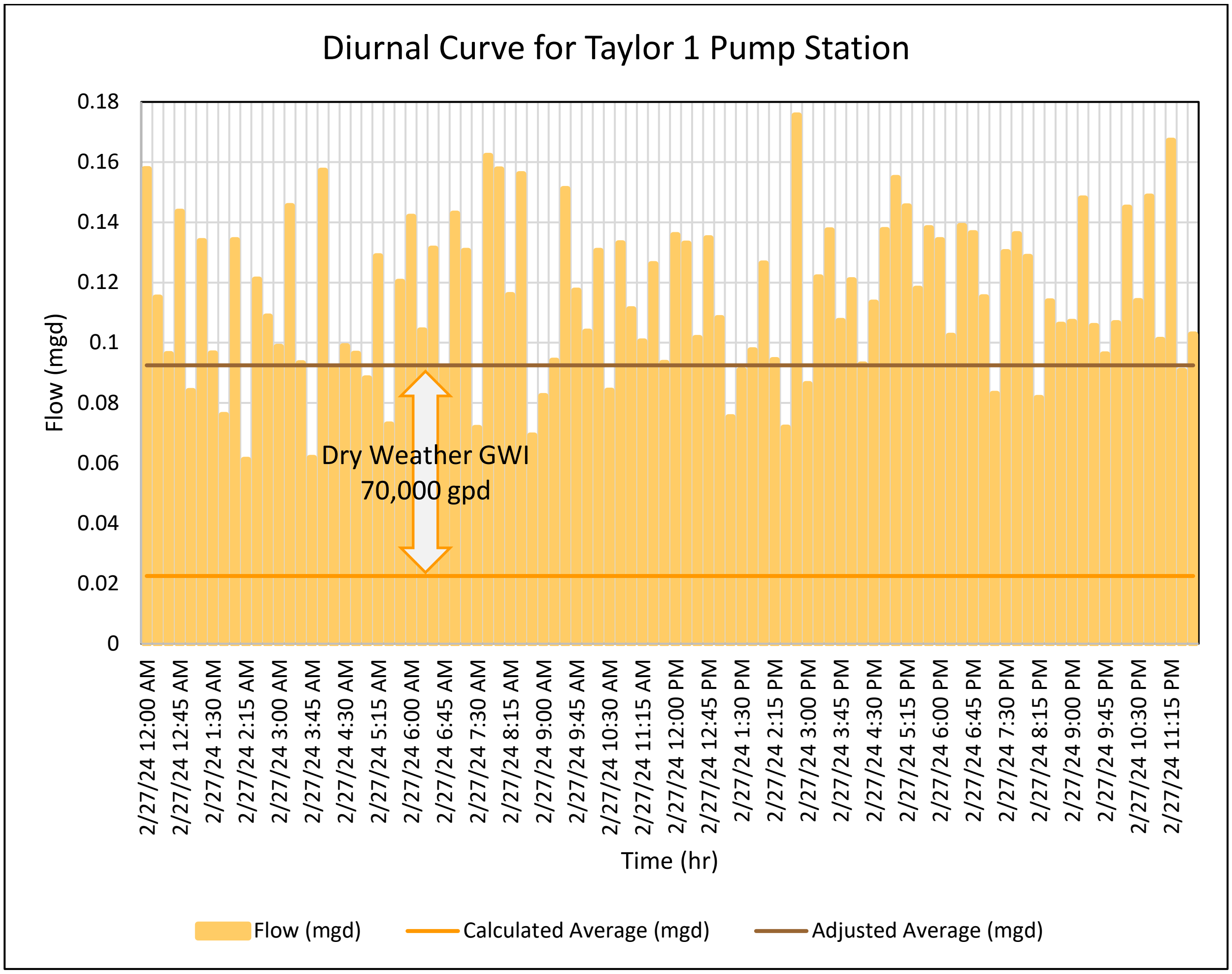


Figure 4.4 Diurnal Pattern and Dry Weather GWI Upstream of Stone 2 PS[13]



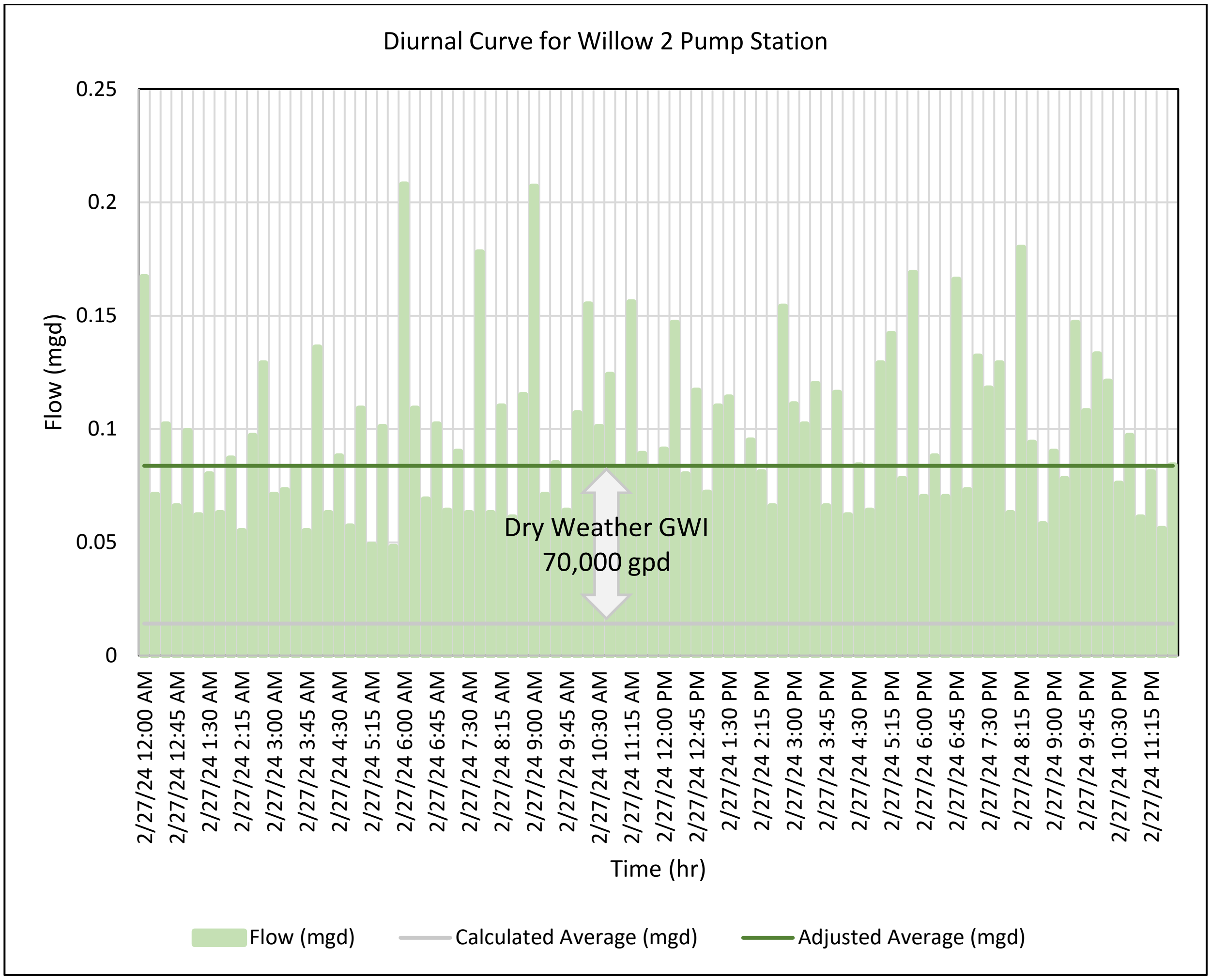
[13] In 2024, the District completed the Bethel Island I&I Reduction project. This project used extensive pipe grouting to seal visible defects within the Bethel Island gravity system. The purpose of the project was to reduce dry weather GWI that results from the high water table. Flow data measured at the WRF has confirmed that a measurable flow reduction was achieved after completion of the project. As a result, the dry weather GWI that is shown on Figure 4.4, 4.5, and 4.6 should be significantly lower now than measured in February and March 2024.

Figure 4.5 Diurnal Pattern and Dry Weather GWI Upstream of Taylor 1 PS[14]



[14] See footnote to Figure 4.4.

Figure 4.6 Diurnal Pattern and Dry Weather GWI Upstream of Willow 2 PS[15]



4.2.2 Wet Weather Flow Generation

The wet weather calibration process assigns parameters that represent the amount of RDII that enters the gravity sewer pipeline during a wet weather event. The RTK method was used to model rainfall-dependent inflow and infiltration "RDII". The RTK method generates three hydrographs for each metered basin that represent the three different patterns of I&I that can enter the system during a wet weather event.

The three triangular hydrographs represent short-term, medium-term, and long-term RDII. RTK parameters include:

R = the area of the graph representing the portion of rainfall falling on a subcatchment that enters the sewer collection system.

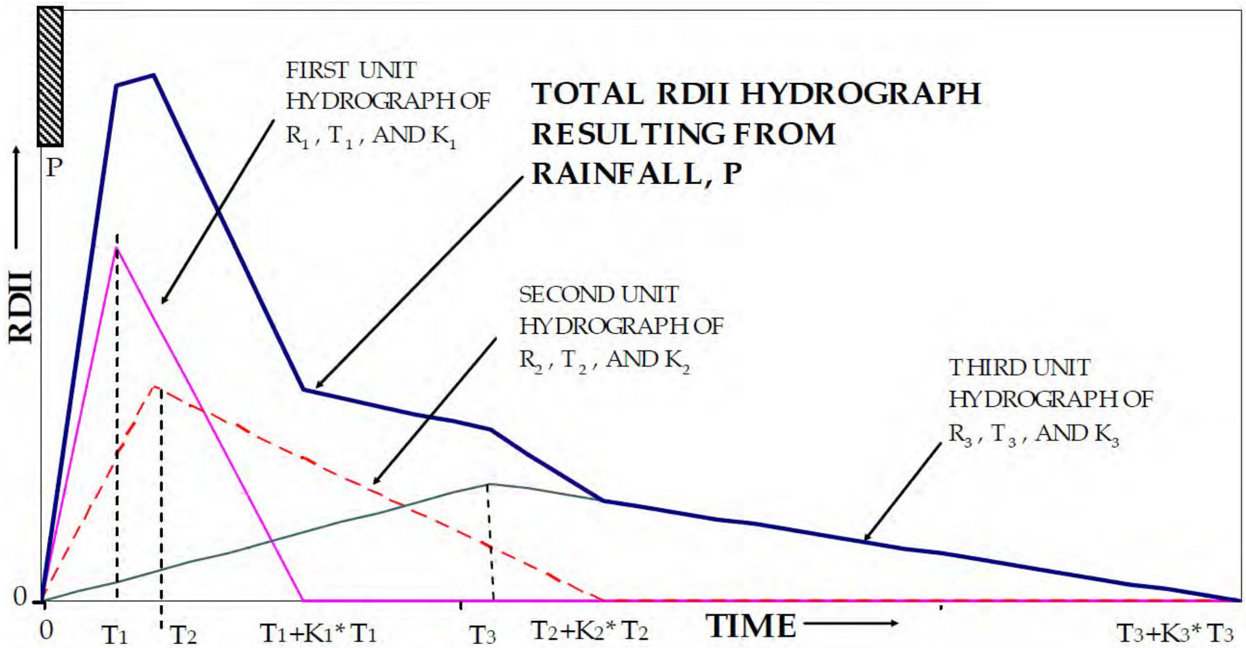
T = the time from the onset of rainfall to the peak of the triangle.

[15] See footnote to Figure 4.4.

K = the ratio of the “time to recession” to the “time to peak” of the hydrograph.

Components of the RTK hydrograph are provided courtesy of the United States Environmental Protection Agency (EPA) Office of Research and Development, and are presented in Figure 4.7.

Figure 4.7 Components of RTK Hydrograph



The hydraulic model includes fifteen separate profiles, each with an independent set of RTK hydrographs. The model predicts wet weather flows by adding RDII to the dry weather values that were obtained through the dry weather calibration process.

4.3 MODEL CALIBRATION

The hydraulic model was calibrated for the dry and wet weather conditions. This section provides more information on the calibration effort and presents results.

4.3.1 Dry Weather Flow Calibration

The initial dry weather flow values discussed in Table 4.2 were assigned to each subcatchment in the hydraulic model. Subcatchments were grouped into the seven basins that monitored in 2024 by two temporary flowmeters and five pump station data loggers.

Modeled dry weather flows were then compared to average metered dry weather flows from February 27, 2024. Although some rainfall had occurred prior to this time, flow data indicates that the system flows on this day were not elevated as a result. The total system ADWF measured by V&A Engineering on February 27, 2024, expanded to include calculated dry weather flows for the unmetered basins, was 2.7 mgd.

Dry weather flow components were then adjusted as follows, until average modeled flows were, for most basins, within five to ten percent of measured flows:

- If modeled flows were different than measured flows, the land use distribution within the basin was reviewed and unit flow factors increased or decreased as needed to adjust generated flows
- A constant dry weather groundwater infiltration component was added to the basins that flow to each of the pump stations on Bethel Island. The same dry weather groundwater infiltration component was also added to the pump stations that border the waterway directly south of Bethel Island.
- The groundwater was individually adjusted until calculated flows were within 10 percent of measured flows. In addition, low density residential unit flows were reduced for the Ironwood pump station basin, as measured flows were approximately half of the expected BWWF based on land uses.

A completed dry weather calibration was achieved when minimum, maximum, and average modeled flows, as well as the temporal distribution of flow over a 24-hour period, were within ten percent of measured flows. Dry weather calibration results are shown in Table 4.3.

Table 4.3 Dry Weather Calibration Results (February 27, 2024)

2024 Meter Name	Manhole ID	Measured ADWF from Feb 27, 2024 (gpd)	Model-Generated ADWF Incl. Dry Weather GWI (gpd)	Model Accuracy	Comments
FM01	SEC001	274,300	268,800	98%	
FM02	OT040	342,000	387,600	113%	
Ironwood PS	IW	108,600	164,200	151%	After minimizing assigned unit flow from residential land uses, model still over-predicts flow.
Stone 2 PS	S2	102,900	97,600	95%	
Taylor 1 PS	T1	115,000	113,300	99%	
Willow 2 PS	W2	97,800	98,400	101%	
Summer Lakes 1 PS	SL1	71,300	72,600	102%	

4.3.2 Wet Weather Calibration

After the hydraulic model was calibrated to dry weather flows, the system was evaluated under the selected calibration storm, which occurred from February 18 to 19, 2024. The Ironwood Pump Station pump logger was operational on this date. However, the remaining pump station loggers were not active.

Therefore, March 1 to 2, 2024 was used as the wet weather calibration date for Stone 2, Taylor 1, Willow 2, and Summer Lakes one pump loggers.

Wet Weather Flow Calibration

Wet weather flow calibration consisted of the following steps:

- Identify a wet weather calibration event with heavy rainfall, visible collection system response (increased flows), and without any spills. The selected rainfall event occurred on February 18 to 19, 2024 for FM01, FM02, and Ironwood PS, and on March 1-2 for the remaining pump stations.
- Assign and adjust R, T, and K parameters for the three wet weather hydrographs and assigned to the appropriate metering basins
- Complete a hydraulic model run using the initial wet weather scenario and compare metered data with model simulation results
- Adjust R, T, and K parameters and conduct subsequent runs to maximize agreement for the calibration event, beginning with upstream and proceeding through all downstream metered basins

A completed calibration was achieved when average modeled flows, as well as the temporal distribution of flow over the calibration period were within ten percent of measured flows. Wet weather calibration results are shown in Table 4.4.

Validation of Wet Weather Model Results

After the hydraulic model was calibrated for wet and dry weather conditions, the model was validated by reviewing the long-term rainfall response for the monitored period in February and March, 2024. The two-month timeframe includes the calibration storms and additional rainfall events. The purpose of the validation was to provide a level of confidence in the model's ability to predict flow under a range of wet weather events. These confirmation results for both flow and water level fluctuations are presented in Figure 4.8 and Figure 4.9, respectively.

Figure 4.8 Longterm Confirmation Results - Flow

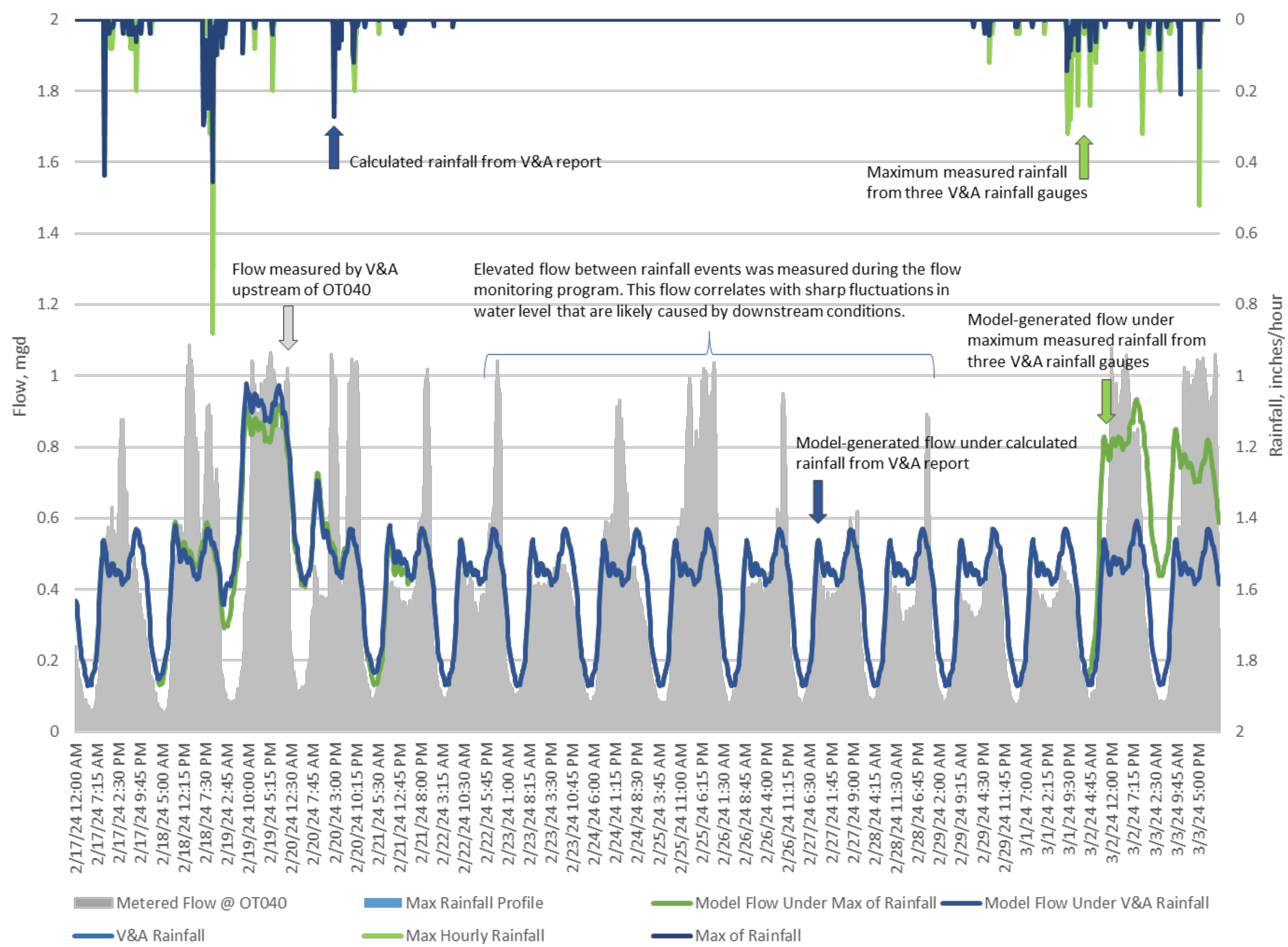


Figure 4.9 Longterm Confirmation Results – Water Level Fluctuations as Compared to Flow

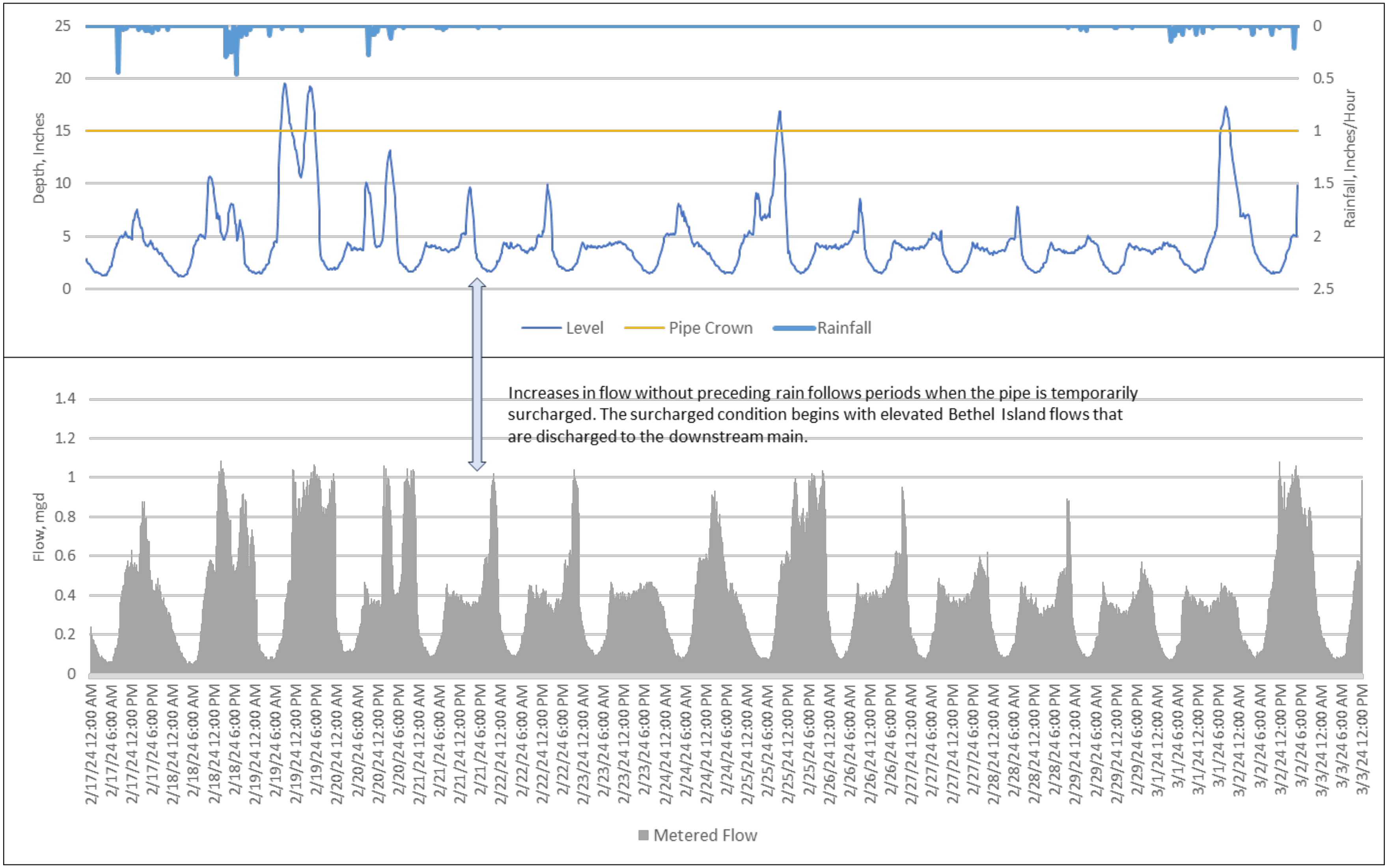


Table 4.4 Wet Weather Calibration Results on February 19, 2024 and March 1 to 2, 2024

2024 Meter Name	Manhole ID	Average WW Flow			Peak WW Flow			Comments
		Meter	Model	Accuracy	Meter	Model	Accuracy	
FM01[16]	SEC001	308,000	278,000	90%	788,000	814,000	103%	
FM02	OT040	545,000	583,300	107%	1,087,000	1,039,000	96%	Metered flow shows a flattened profile that follows 12 hours after rainfall, indicating that rainfall near OT040 was delayed and more distributed than in V&A report. Basin calibrated to average flow.
Ironwood PS	I12086	121,000	167,800	139%	248,000	278,300	112%	Model-generated AWWF includes high ADWF. Basin calibrated to peak flow.
Stone 2 PS[17]	H14165	129,000	123,000	95%	165,000	232,000	141%	Model-generated peak influent flows are higher than measured. Actual pumped flows appear to be dampened for pumps in series. Pumped basins were calibrated to average wet weather flow.
Taylor 1 PS	G14189/G14071	113,000	113,300	100%	176,000	359,000	204%	
Willow 2 PS	H12067	140,000	124,300	89%	251,000	235,900	94%	
Summer Lakes 1 PS	G13222	87,000	93,100	107%	348,000	504,000	145%	

[16] Wet Weather flow for FM01, FM02, and Ironwood PS measured on February 18 and 19, 2024

[17] Wet Weather flow for Stone 2, Taylor 1, Willow 2, and Summer Lakes 1 PS measured on March 1 and 2, 2024

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CHAPTER 5 CAPACITY ANALYSIS

Chapter 4, Hydraulic Model Development, provides information on the model network, sewer loads, and wet weather calibration factors. The calibrated hydraulic model was used to evaluate the District's wastewater collection system for capacity constraints resulting from flow conditions that are predicted to occur during the District's design storm. The purpose of this Chapter is to summarize planning and capacity criteria, to discuss predicted capacity constraints, and present recommended solutions and strategies to address the identified issues. This Chapter also discussed specific capacity-related considerations that are required by Statewide Order 2022-0103-DWQ ("Statewide WDR").

This Chapter is organized as follows.

- 5.1 Planning and System Deficiency Criteria
- 5.2 Capacity Analysis
- 5.3 Project Costs
- 5.4 Review of Statewide Waste Discharge Requirements for Capacity Analysis

5.1 PLANNING AND SYSTEM DEFICIENCY CRITERIA

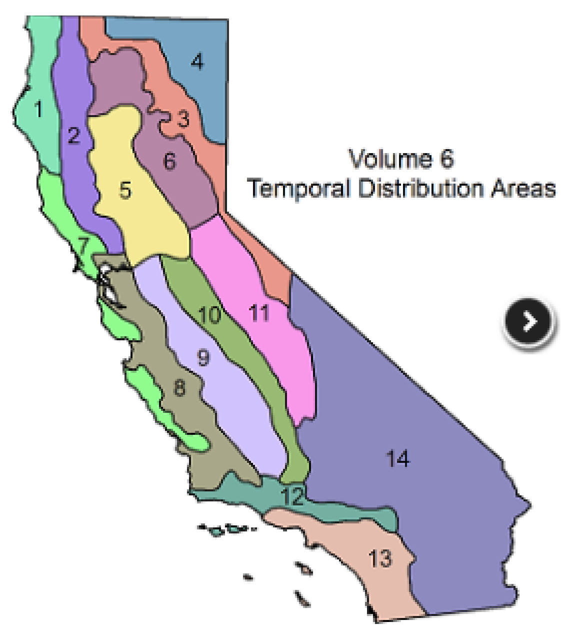
This section includes information on the selected design storm and also presents criteria that was used to evaluate system deficiencies that are predicted to occur during the design storm wet weather event.

5.1.1 Design Storm

The hydraulic model evaluates the predicted capacity of the District's wastewater collection system under flow loading from a hypothetical design storm. The selected design storm has a recurrence interval of 10 years (*i.e.*, 10 percent probability of occurring in any given year) and duration of 24 hours. Flow characteristics for the 10-year, 24-hour design storm were derived from data that is published by the National Oceanographic Atmospheric Administration ("NOAA"), which provides rainfall depths and also distribution over time based on location, as shown on Figure 5.1.

NOAA publishes statistically-derived rainfall depths for use in assigning a rainfall recurrence event[18]. The NOAA rainfall depth table for the City of Oakley is included as Figure 5.2 on the following pages. As shown on the table, the most likely rainfall depth for a 10-year, 24 hour rainfall event is 2.45 inches.

Figure 5.1 NOAA Rainfall Temporal Distribution Systems for Volume 6: California



[18] https://hdsc.nws.noaa.gov/pfds/pfds_map_cont.html?bkmrk=ca

Figure 5.2 NOAA Rainfall Depths for Various Storm Frequencies and Durations

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 6, Version 2

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.083** (0.074-0.095)	**0.101** (0.090-0.115)	**0.127** (0.112-0.145)	**0.151** (0.132-0.175)	**0.188** (0.157-0.228)	**0.220** (0.178-0.275)	**0.257** (0.201-0.331)	**0.298** (0.224-0.401)	**0.362** (0.257-0.515)	**0.418** (0.284-0.623)
10-min	**0.119** (0.106-0.136)	**0.145** (0.128-0.165)	**0.182** (0.161-0.208)	**0.217** (0.189-0.250)	**0.270** (0.225-0.327)	**0.316** (0.255-0.394)	**0.368** (0.288-0.475)	**0.428** (0.322-0.574)	**0.519** (0.369-0.738)	**0.600** (0.407-0.893)
15-min	**0.144** (0.128-0.164)	**0.175** (0.155-0.199)	**0.220** (0.195-0.252)	**0.262** (0.229-0.303)	**0.326** (0.272-0.395)	**0.382** (0.309-0.477)	**0.445** (0.348-0.575)	**0.517** (0.389-0.694)	**0.628** (0.446-0.893)	**0.725** (0.492-1.08)
30-min	**0.195** (0.174-0.222)	**0.237** (0.210-0.269)	**0.298** (0.264-0.340)	**0.354** (0.310-0.409)	**0.441** (0.367-0.534)	**0.516** (0.418-0.645)	**0.602** (0.470-0.777)	**0.699** (0.526-0.939)	**0.849** (0.604-1.21)	**0.981** (0.665-1.46)
60-min	**0.278** (0.248-0.316)	**0.337** (0.299-0.383)	**0.425** (0.376-0.485)	**0.505** (0.441-0.583)	**0.628** (0.523-0.761)	**0.736** (0.595-0.918)	**0.857** (0.670-1.11)	**0.996** (0.750-1.34)	**1.21** (0.860-1.72)	**1.40** (0.948-2.08)
2-hr	**0.410** (0.364-0.465)	**0.485** (0.431-0.552)	**0.601** (0.532-0.687)	**0.710** (0.621-0.821)	**0.885** (0.737-1.07)	**1.04** (0.843-1.30)	**1.22** (0.957-1.58)	**1.44** (1.08-1.93)	**1.78** (1.26-2.53)	**2.09** (1.42-3.11)
3-hr	**0.516** (0.459-0.586)	**0.608** (0.540-0.691)	**0.750** (0.663-0.856)	**0.884** (0.773-1.02)	**1.10** (0.918-1.33)	**1.30** (1.05-1.62)	**1.53** (1.20-1.98)	**1.80** (1.36-2.42)	**2.24** (1.60-3.19)	**2.65** (1.80-3.94)
6-hr	**0.732** (0.651-0.831)	**0.861** (0.765-0.980)	**1.06** (0.939-1.21)	**1.25** (1.09-1.45)	**1.56** (1.30-1.89)	**1.83** (1.48-2.29)	**2.16** (1.69-2.79)	**2.54** (1.91-3.42)	**3.16** (2.25-4.49)	**3.73** (2.53-5.55)
12-hr	**0.974** (0.866-1.11)	**1.16** (1.03-1.32)	**1.45** (1.28-1.66)	**1.72** (1.50-1.99)	**2.14** (1.79-2.60)	**2.52** (2.04-3.15)	**2.96** (2.32-3.83)	**3.48** (2.62-4.67)	**4.29** (3.05-6.10)	**5.02** (3.41-7.48)
24-hr	**1.32** (1.22-1.46)	**1.62** (1.49-1.79)	**2.06** (1.89-2.28)	**2.45** (2.23-2.74)	**3.03** (2.66-3.52)	**3.52** (3.02-4.19)	**4.07** (3.39-4.97)	**4.68** (3.78-5.90)	**5.58** (4.32-7.37)	**6.36** (4.74-8.72)
2-day	**1.60** (1.48-1.77)	**2.09** (1.92-2.31)	**2.72** (2.50-3.02)	[illegible] (2.94-3.62)	**3.93** (3.45-4.57)	**4.47** (3.83-5.32)	**5.01** (4.18-6.13)	**5.57** (4.51-7.02)	**6.33** (4.89-8.35)	**6.92** (5.15-9.48)
3-day	**1.81** (1.67-2.00)	**2.42** (2.23-2.68)	**3.19** (2.93-3.54)	**3.79** (3.45-4.25)	**4.57** (4.01-5.32)	**5.15** (4.42-6.13)	**5.72** (4.77-6.99)	**6.28** (5.08-7.92)	**7.01** (5.42-9.26)	**7.56** (5.63-10.4)
4-day	**1.98** (1.83-2.20)	**2.66** (2.45-2.94)	**3.50** (3.21-3.88)	**4.14** (3.77-4.64)	**4.97** (4.36-5.78)	**5.58** (4.78-6.64)	**6.16** (5.14-7.53)	**6.73** (5.45-8.49)	**7.47** (5.77-9.85)	**8.00** (5.96-11.0)
7-day	**2.49** (2.30-2.76)	**3.25** (3.00-3.60)	**4.18** (3.85-4.65)	**4.90** (4.46-5.49)	**5.82** (5.10-6.76)	**6.47** (5.55-7.70)	**7.10** (5.93-8.69)	**7.72** (6.25-9.73)	**8.50** (6.57-11.2)	**9.06** (6.75-12.4)
10-day	**2.80** (2.59-3.10)	**3.60** (3.31-3.98)	**4.57** (4.20-5.07)	**5.31** (4.84-5.95)	**6.25** (5.49-7.27)	**6.93** (5.94-8.25)	**7.58** (6.33-9.27)	**8.21** (6.64-10.4)	**9.00** (6.96-11.9)	**9.58** (7.13-13.1)
20-day	**3.60** (3.32-3.98)	**4.51** (4.16-5.00)	**5.63** (5.17-6.25)	**6.48** (5.90-7.26)	**7.55** (6.63-8.78)	**8.32** (7.13-9.90)	**9.05** (7.55-11.1)	**9.76** (7.90-12.3)	**10.6** (8.23-14.0)	**11.3** (8.40-15.5)
30-day	**4.29** (3.96-4.74)	**5.31** (4.89-5.88)	**6.56** (6.02-7.28)	**7.50** (6.84-8.41)	**8.71** (7.64-10.1)	**9.56** (8.20-11.4)	**10.4** (8.66-12.7)	**11.2** (9.04-14.1)	**12.2** (9.40-16.0)	**12.9** (9.58-17.6)
45-day	**5.22** (4.82-5.77)	**6.38** (5.88-7.06)	**7.78** (7.15-8.64)	**8.86** (8.07-9.93)	**10.2** (8.97-11.9)	**11.2** (9.60-13.3)	**12.1** (10.1-14.8)	**13.0** (10.5-16.4)	**14.2** (10.9-18.7)	**15.0** (11.1-20.5)
60-day	**6.37** (5.88-7.04)	**7.68** (7.08-8.51)	**9.30** (8.54-10.3)	**10.5** (9.59-11.8)	**12.1** (10.6-14.1)	**13.2** (11.3-15.7)	**14.3** (11.9-17.5)	**15.3** (12.4-19.3)	**16.6** (12.9-21.9)	**17.6** (13.1-24.1)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Estimates from the table in CSV format: Precipitation frequency estimates | Submit

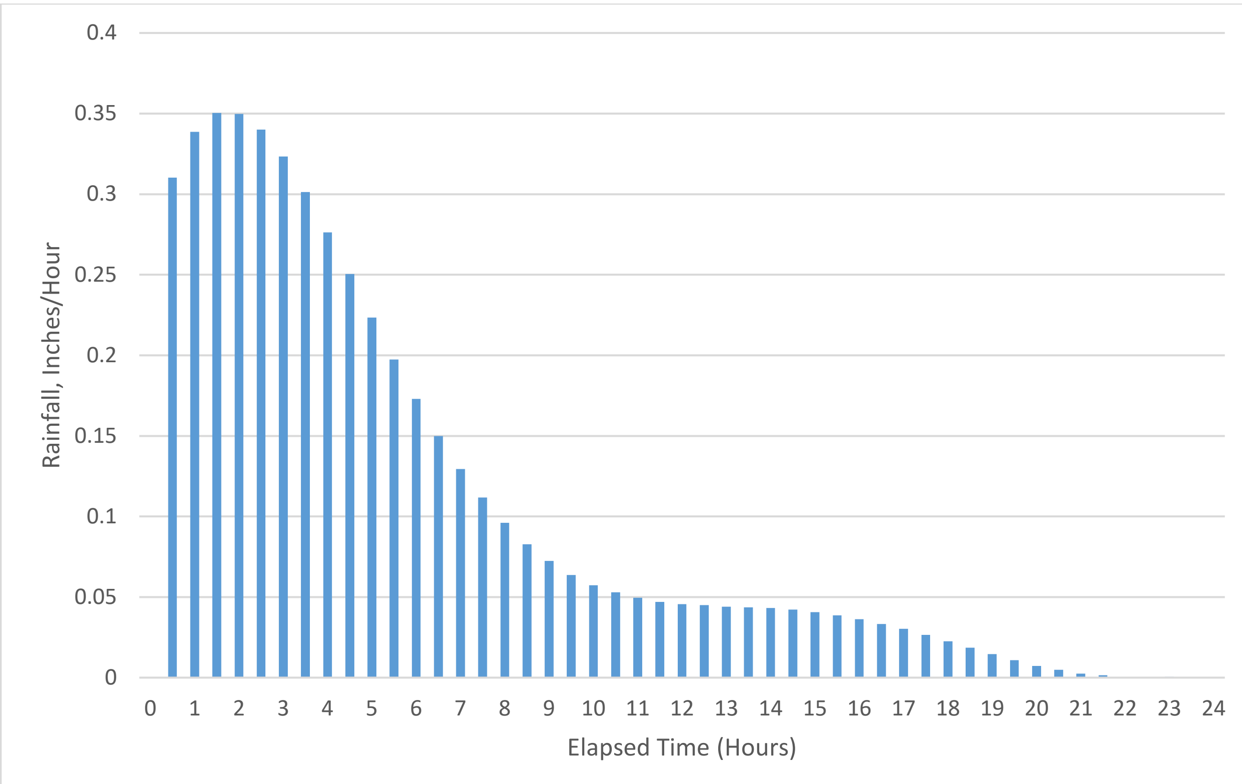
In addition to providing rainfall depths, NOAA provides statistically probable distribution profiles for the rainfall over the defined period[19]. Rainfall temporal distributions were taken from Volume 6 (California); Temporal Distribution System 8, as shown in Figure 5.3 on the previous page.

NOAA provides 36 temporal distributions for rainfall that could occur during each of the four quartiles

[19] https://hdsc.nws.noaa.gov/pfds/pfds_temporal.html

of the storm duration, with zero to 90 percent chance of occurring within each quartile. For the purposes of evaluating capacity needs, the temporal distribution that resulted in the highest hourly peak flow was selected for the hydraulic model analysis. This distribution has a 10 percent chance of occurring during the first quartile, and is shown in Figure 5.3 for both design storms. The rainfall distribution was shifted to 9:30 a.m. in each case so the peak rainfall would occur at approximately the same time as the peak diurnal flow.

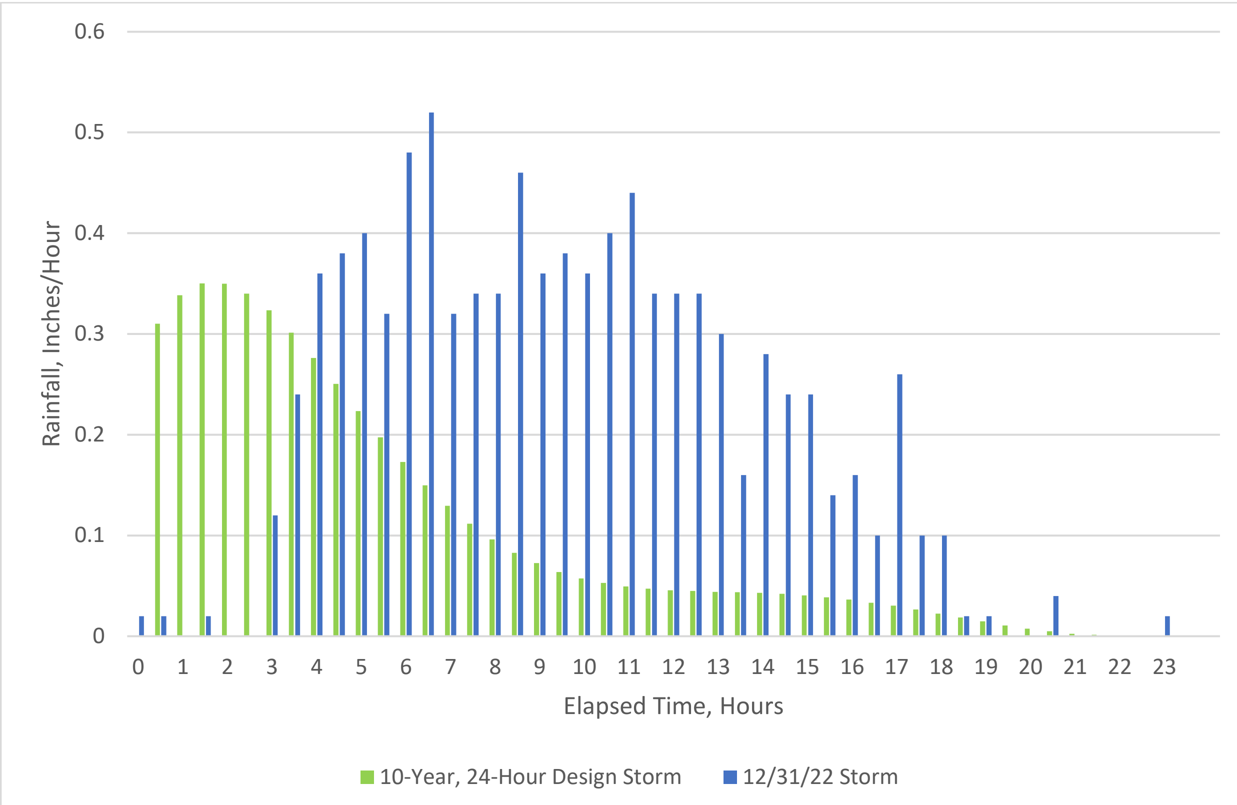
Figure 5.3 NOAA Temporal 24-Hour Rainfall Distribution for the District's Design Storm (10% Probability of Occurring in the First Quartile)



By comparison, on December 31, 2022, the District received a rainfall event with reported rainfall depths ranging from 4.6 to over 6 inches over a 24-hour period. Using the NOAA precipitation depth table from Figure 5.2, the December 31, 2022 storm had a recurrence interval of between 100 and 500 years (for a 24-hour event) and exceeded the District's design storm. During this rainfall event, the District utilized the emergency storage basins at Bethel Island ponds to store diverted flows from the Main and Web pump stations. By following this flow management approach, no spills occurred from the wastewater collection system.

Figure 5.4 compares the December 31, 2022 wet weather event to the Design Storm.

Figure 5.4 Comparison of December 31, 2022 Wet Weather Event with 10-Year, 24-Hour NOAA Design Storm



5.1.2 Hydraulic Deficiency and Pipeline Design Criteria

The goal of this Master Plan is to identify and address capacity deficiencies by upsizing, replacing, or otherwise addressing existing pipelines and pump station capacity as needed. The primary purpose of developing each capacity recommendation is to establish a budget for planning future capital improvement projects. During the design phase for each project, alternative solutions may be identified. In addition, if the District is able to sufficiently reduce I&I through collector sewer rehabilitation and replacement, and can thereby avoid or reduce the scope of a proposed capacity upgrade for a comparable cost, then the District should consider completing the I&I reduction project in lieu of upsizing undersized infrastructure.

<u>Existing Pipelines</u>

- For existing pipelines, the pipe is considered to have a capacity deficiency when, under peak wet weather flow conditions for the design storm, the water level or hydraulic gradeline is higher than the pipe crown and within three feet of rim elevation (i.e., freeboard is less than or equal to three feet). This constraint should be reviewed on a case-by-case for shallow pipes, which may breach this criteria without actually having a spill risk.
- A force main is be considered capacity-deficient if the maximum velocity exceeds 8 feet per second during peak hourly flows.

- A lift station is determined to require capacity upgrades if the tributary (i.e., upstream) system experiences a spill because the existing pump or pumps are undersized. This evaluation is completed with the largest pump at each pump station out of service.

New Pipelines

- Under peak dry weather flow conditions and where feasible, velocity should remain above 2.5 feet per second to facilitate self-cleaning
- Under peak wet weather flow conditions, maximum depth of flow divided by diameter ("d/D") should be equal to or less than the following where practical:
  - 10-inch diameter and smaller: Max d/D = 0.67
  - 12-inch diameter and above: Max d/D = 0.80
- Under all conditions, maximum allowable velocity should be 10 feet per second. Under sustained operations, maximum velocity of 6 feet per second is recommended.

5.1.3 Emergency Storage at the Bethel Island Ponds

The District's Bethel Island Ponds are available to store peak flows during construction bypassing activities or very heavy wet weather events. During the December 31, 2022 wet weather event, the District used the combination vacuum and excavation truck to remove flow from the Main and Web pump stations. This flow was stored in the Bethel Island ponds during the wet weather event and pumped back into the system after the rainfall ended and system flows stabilized.

Diversion and return flow from these ponds are currently managed manually and the hydraulic model does not include any flow to or from the ponds. However, the master plan recognizes that if flows from the Main, Web, or Summer Lakes 1 pump station exceed capacity, Bethel Island Pond storage will be available for use by the District.

5.2 CAPACITY ANALYSIS

The District's modeled collection system network was evaluated for its capacity to convey flows that are predicted to occur during a peak dry weather scenario and also the District's 10-year, 24-hour design storm event. The hydraulic model identified no locations with predicted spills from the existing system during either scenario. However, the model identified one location with freeboard of less than three feet. The gravity interceptor with reduced freeboard begins on Main Street, heads west and then north on Jordan Lane in the Vintage Parkway community, and then east to the WRF.

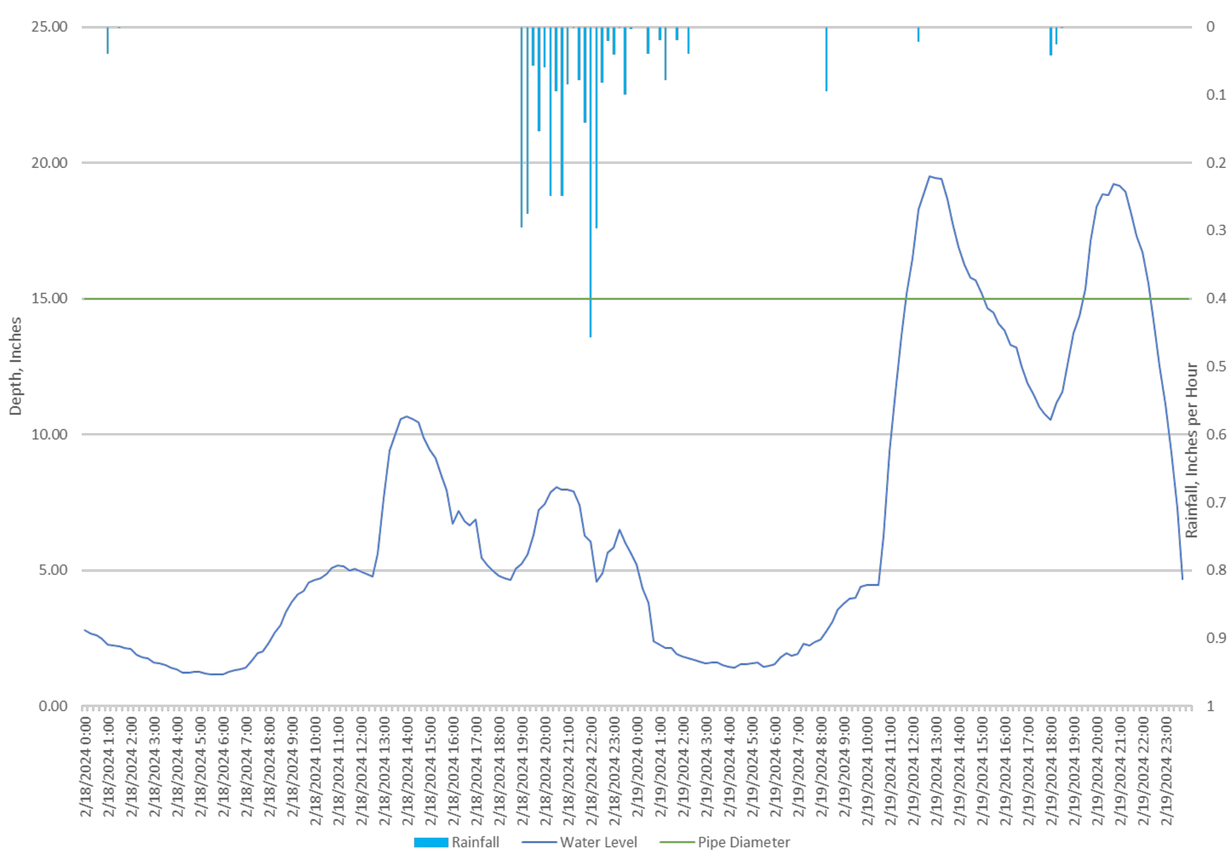
Of note is that no spills occurred from this part of the system during the December 31, 2022 wet weather event. The December 31, 2022 event was between a 100- and 500-year, 24-hour storm based on measured flow and NOAA precipitation forecast data. The absence of actual spills during this larger event indicates that although the model predicts freeboard of less than three feet during the design storm, this result does not trigger the immediate need for a capacity improvement in this location. This conclusion, however, cannot be applied to the future scenario.

During the future system design storm scenario, this same pipeline is predicted to have spills. The District is in the process of constructing a dual 14-inch force main. This force main, which was also discussed in the 2004 Master Plan, will route future flow from the Summer Lakes, Summer Lakes North, Emerson, Gilbert, Cypress Grove, and Grand Cypress Preserve communities to a discharge location south of the WRF. When this force main is placed in service, the predicted spills will be addressed. Removing this flow from the existing 14-inch force main in the future scenario lowers the freeboard in the existing force main to a level that meets the District's capacity criteria.

5.2.1 Surcharged Condition on Main Street and Jordan Lane

As introduced above, during some wet weather events, the existing 15-inch pipe at manhole OT040 becomes surcharged, as shown on Figure 5.5. The green line depicts the pipe crown and the blue line shows the water level within the pipe over the 48-hour period from February 18 to 19, 2024. As shown, although the pipe becomes surcharged, freeboard is greater than three feet and this line does not require action based on the capacity criteria discussed above. The surcharge is created when the Main Street interceptor, which is directly downstream of this location, surcharges during wet weather events.

Figure 5.5 Water Level at Meter Site 2 (OT040) during February 18 to 19 Calibration Storm



During the existing system 10-year, 24-hour design storm, the predicted freeboard within the Main Street interceptor decreases to less than two feet on Jordan Lane and less than one foot on Main Street, as shown on Figure 5.6. As discussed further above, spills were not observed from any portion of this line beginning on Main Street through Jordan Lane to the WRF during the December 31, 2022 rainfall event. Therefore, also the hydraulic model predicts that surcharge conditions will occur during the existing system design storm, the limited freeboard does not in itself trigger a capacity project.

Figure 5.7 shows the same profile during the future (build-out) design storm scenario. As shown on Figure 5.7, added flows from the East Cypress Corridor reduce freeboard further, leading to predicted spills.

Figure 5.8 shows the profile after flows from the East Cypress Corridor are diverted to the new, 14-inch force main. As shown on Figure 5.8, the proposed 14-inch force main is successful in reducing the hydraulic profile and associated freeboard to a level that is lower than the existing condition. The proposed 14-inch force main is discussed further under Capital Improvements.

Figure 5.6 Surcharge Condition on Main Street and Jordan Lane leading to WRF during 10-year, 24-hour Design Storm (Existing Scenario)

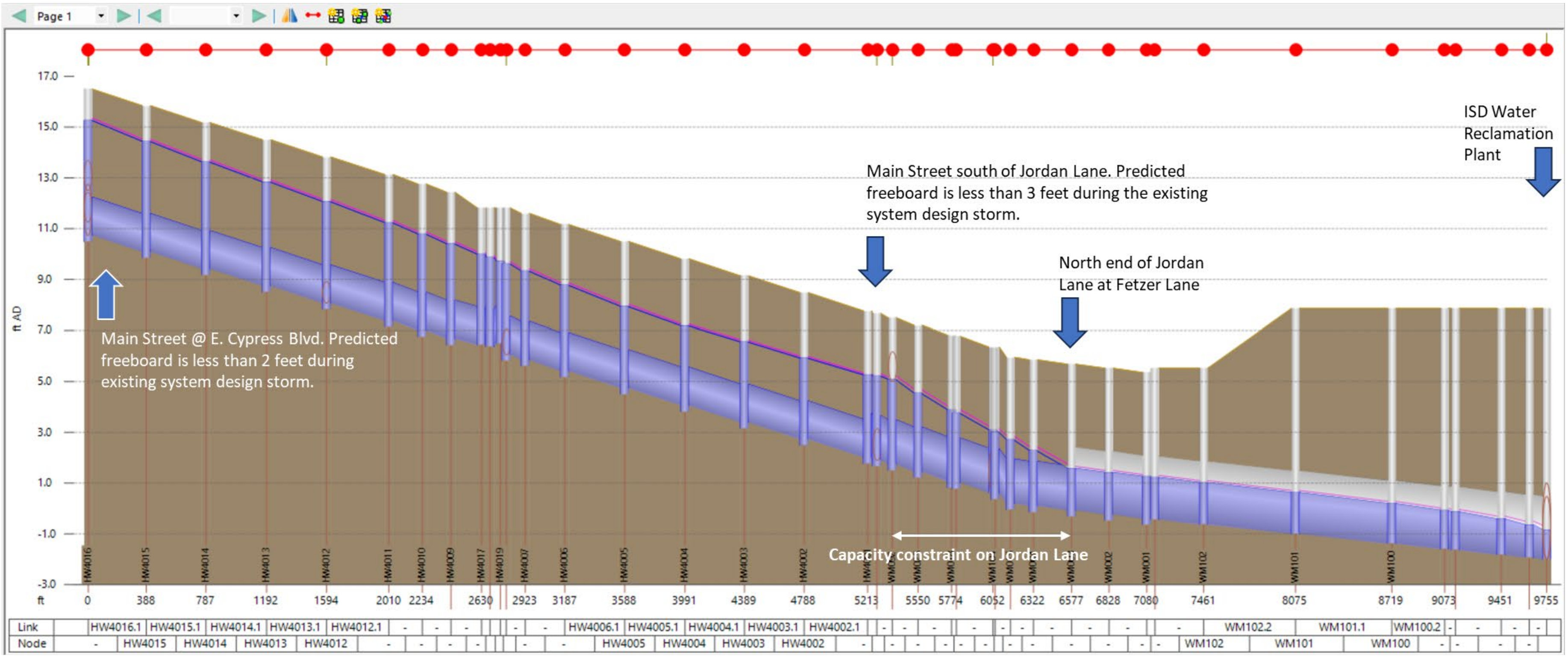


Figure 5.7 Surcharge Condition on Main Street and Jordan Lane leading to WRF during 10-year, 24-hour Design Storm (Future Scenario)

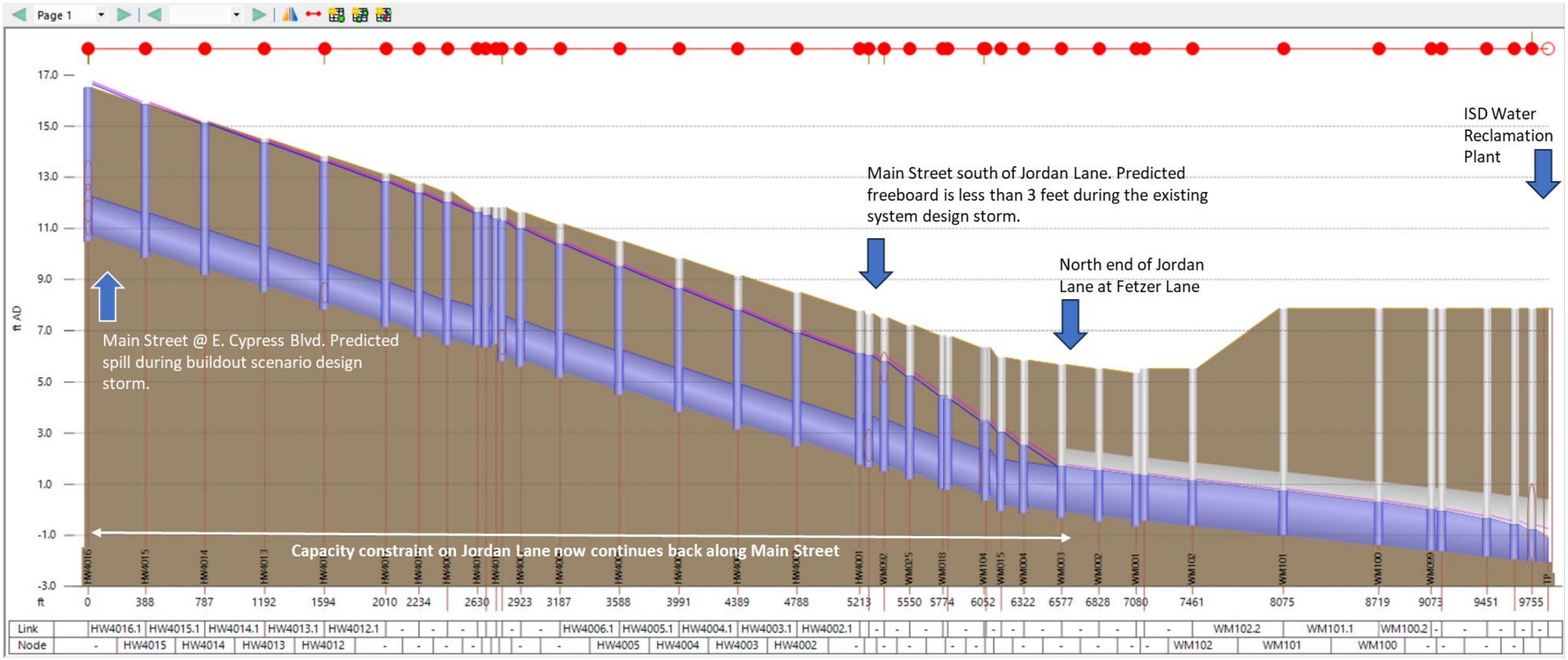
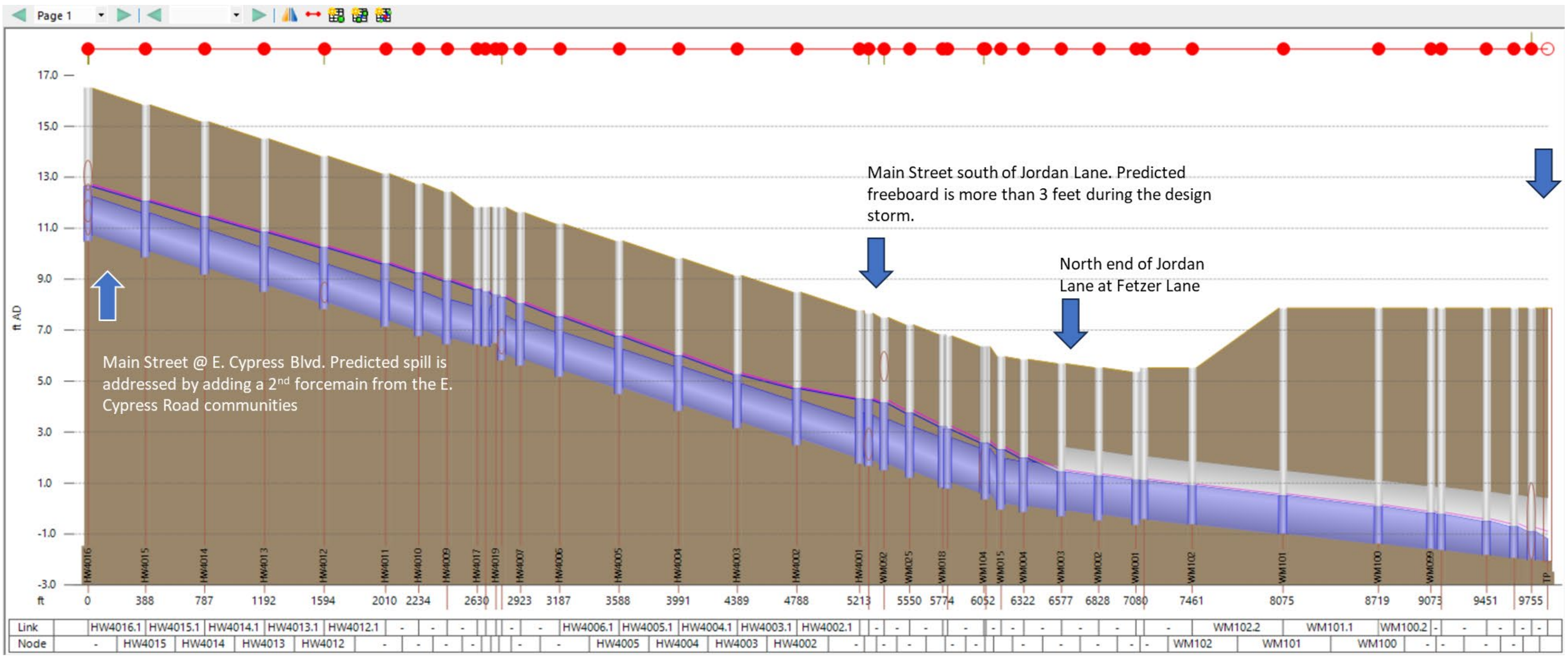


Figure 5.8 Future Hydraulic Profile on Main Street and Jordan Lane leading to WRF during 10-year, 24-hour Design Storm After E. Cypress Corridor Flows are Diverted to New 14-Inch Force main



5.2.2 Pump Station Capacity Analysis

Model-generated influent flow during the District's design storm event was compared to firm pump station capacity, as shown on Table 5.1.The pumping capacities shown are taken from data provided by District staff. The land-based stations showed relatively low I&I and minimal or no buildout. Therefore, planned flows for the land-based stations were not increased for the future condition.[20]

After completion of the flow monitoring program, the District completed the Bethel Island I&I Reduction project which involved grout-sealing of known defects and locations of visible I&I within the Bethel Island gravity sewer system. After this project was completed, District staff observed lower flows to the WRF. Therefore, the Bethel Island pump station flows listed in Table 5.1 are likely higher than actual current flows. For this reason, no further increase in these pump station influent flows were assumed for the future condition.

One exception was made to this approach and additional future flow was added to the Summer Lakes 1 pump station. This station will pump all new flows from the planned expansions at Summer Lakes North and Grand Cypress Preserve.

Table 5.1 Pump Station Parameters

		Firm Capacity (gpm)	Model-Predicted Peak Wet Weather Influent Flow (gpm)
1	Bridgehead	180	19.0
2	Cypress Grove	355	151.2
3	Dutch Slough	150	100.8
4	Emerson Ranch	449	Not modeled
5	Gateway 1	836	63.5
6	Gateway 2	184	177.7
7	**Gateway 3**	**158**	**205.1**
8	Gilbert Ranch 1	Not modeled	
9	Ironwood	1000	187.2
10	Lauritzen	Not modeled	
11	Laurel Heights	110	109.6
12	Marsh Creek	329	321.5
13	Main (MPS)	1400	1,126.6
14	**Piper 1**	**277**	**310.8**
15	Quail Valley	600	239.3
16	Stone 1	240	160
17	**Stone 2**	**200**	**238.4**

[20] The Final Master Plan will include an analysis of currently unsewered parcels in the western and southern portions of the service area, including an estimate of future flows and a desktop evaluation of whether existing infrastructure can accommodate these flows.

		Firm Capacity (gpm)	Model-Predicted Peak Wet Weather Influent Flow (gpm)
18	Summer Lakes 1	1925	501.9[21]
19	Summer Lakes 2	350	282.5
20	Summer Lakes 3	250	63.9
21	**Sandmound 1**	**150**	**190.8**
22	Sandmound 2	190	176.9
23	**Taylor 1**	**270**	**307.3**
24	Taylor 2	190	185.2
25	Taylor 3	310	105
26	Vintage Parkway	600	594.6
27	Willow 1	150	151.0
28	**Willow 2**	**106**	**106.2**
29	Willow 3	200	200.0
30	WEB	796	393.2
31	Willow Park Marina	Not modeled	
32	Bethel Island Ponds	Not modeled	

The model predicts that several pump stations on Bethel Island will have influent flow that exceeds pumping capacity during the design storm. However, no capacity improvements to these stations are recommended at this time for the reasons below. Future flow monitoring during heavy wet weather events could be used to help refine the model further in the Bethel Island sewer basin.

First, the hydraulic model does not predict spills within the gravity sewer system upstream of these stations, indicating that the collection system has sufficient storage capacity to provide a buffer for these peak flows. In addition, the gravity pipelines leading into the pump stations did not have recorded spills during the December 31, 2022 wet weather event.

Second, the Bethel Island pump stations receive significant dry weather groundwater infiltration as discussed above. After pipes have filled with infiltration during normal dry weather high groundwater conditions, it is expected that additional rainfall will contribute some I&I, but potentially less I&I as a percentage of the rainfall volume than a system with empty pipes. During the flow monitoring period, the District did not receive any storms that approached the design storm in volume or intensity. Therefore, this hypothesis could not be confirmed. However, this hypothesis would explain why the hydraulic model predicts higher than observed wet weather influent flows to several Bethel Island pump stations.

Third, as discussed throughout this Master Plan, the District completed the Bethel Island I&I Reduction project in 2024 that has reduced both dry weather GWI and RDII in the Bethel Island gravity system, including RDII within the pipelines that are influent to the Bethel Island pump stations. Further flow monitoring and recalibration of the Bethel Island sewer basins are expected to yield influent flows that are lower than shown in Table 5.1.

[21] Summer Lakes 1 pump station receives flow from future Summer Lakes North and Grand Cypress Preserve communities. Predicted buildout wet weather flow is 1,414 gpd.

5.2.3 Review of Force Main Velocities

Table 5.2 lists force main velocities reported by the hydraulic model. The highest force main velocity of 7.9 fps is recorded for the Taylor 3 pump station. This velocity is below the 8 fps criteria that would trigger potential capacity improvements. Therefore, no improvements are recommended to the existing force main pipelines as related to velocity.

Table 5.2 Force Main Velocities

Pump Station Name		Force Main Velocity (fps)
1	Bridgehead	4.6
2	Cypress Grove	4.0
3	Dutch Slough	3.8
4	Emerson Ranch	1.1
5	Gateway 1	5.3
6	Gateway 2	4.7
7	Gateway 3	4.0
8	Gilbert Ranch 1	Not modeled
9	Ironwood	2.1
10	Lauritzen	Not modeled
11	Laurel Heights	2.8
12	Marsh Creek	2.1
13	Main (MPS)	5.7
14	Piper 1	3.1
15	Quail Valley	1.7
16	Stone 1	2.7
17	Stone 2	2.3
18	Summer Lakes 1	5.5
19	Summer Lakes 2	4.0
20	Summer Lakes 3	1.7
21	Sandmound 1	2.2
22	Sandmound 2	4.9
23	Taylor 1	3.1
24	Taylor 2	2.2
25	Taylor 3	7.9
26	Vintage Parkway	3.8
27	Willow 1	3.8
28	Willow 2	2.7
29	Willow 3	2.3
30	WEB	1.7

	Pump Station Name	Force Main Velocity (fps)
31	Willow Park Marina	Not modeled
32	Bethel Island Ponds	Not modeled

5.2.4 14-inch Parallel Force Main

As shown on Figure 5.9 on the following page and discussed above, a second 14-inch force main from the East Cypress corridor properties is required prior to buildout. The City of Oakley General Plan buildout projections result in the addition of approximately 600,000 gpd of flow from these communities. During the design storm, flows are projected to increase to 1.1 mgd.

The District has installed a portion of this second force main in conjunction with new developments along the E. Cypress Road corridor. When completed, the force main will begin at Bethel Island Road and continue west to Sellers Avenue, head north on Sellers Avenue, turn west behind the Emerson Ranch and Cypress Grove neighborhoods, south for a short period along the Contra Costa Canal, and then north to the WRF.

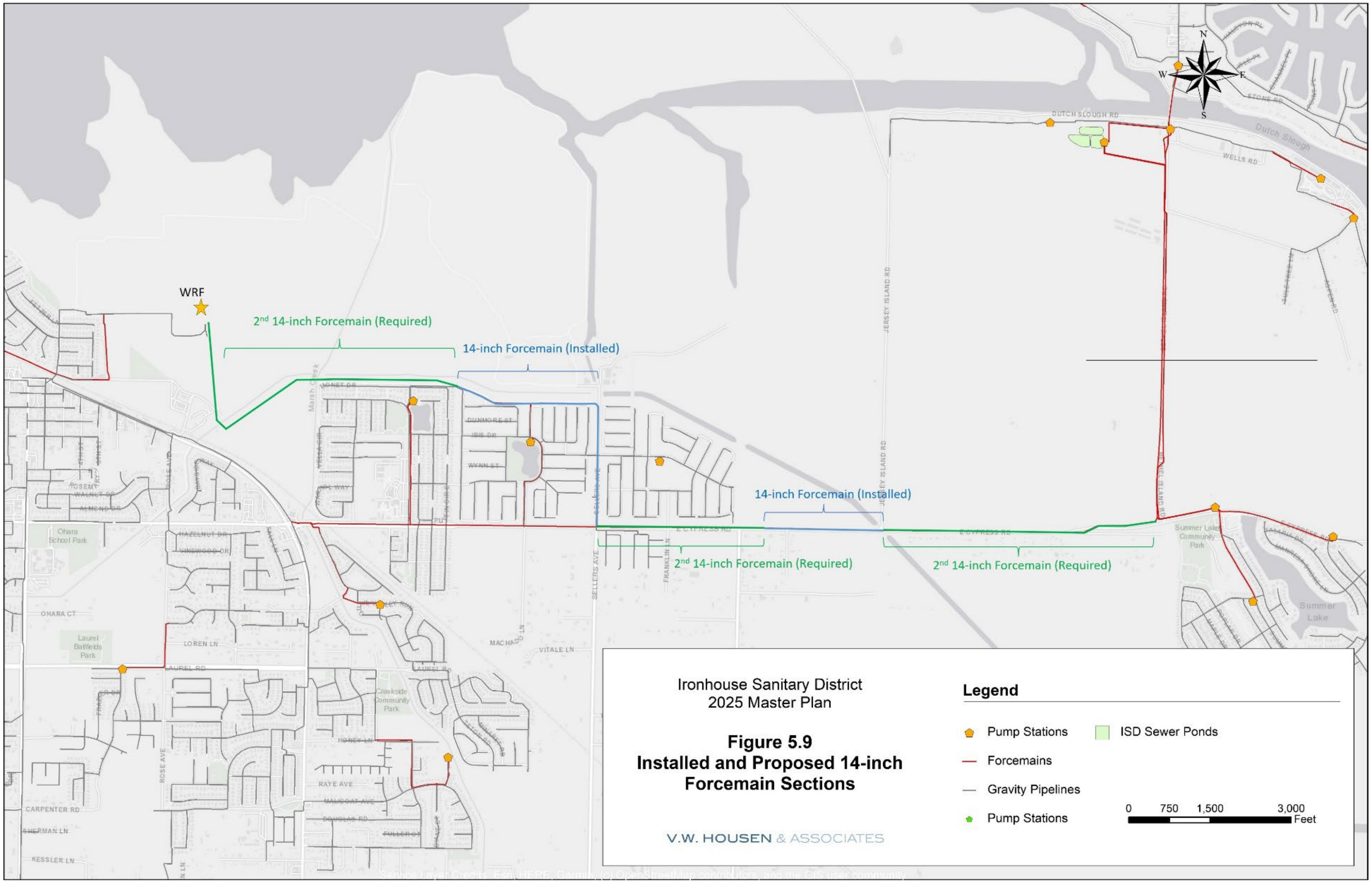
The installed sections are located on E. Cypress Blvd. between Jersey Island Road and Knightsen Avenue, on Sellers Avenue, and north of Emerson Ranch. New sections are required from Bethel Island Road to Jersey Island Road, from Knightsen Avenue to Sellers Avenue, and from the eastern border of Cypress Grove to the WRF.

The total estimated force main length is 4.2 miles. Of this length, approximately 1.4 miles of pipe has been installed. The remaining portions of pipe to be installed include the following:

- Bethel Island Road to Jersey Island Road, including valve structure @ Bethel Island Road (5,100 lineal feet ("lf")
- East of Knightsen Avenue to Sellers Avenue (2,800 lf)
- Emerson Ranch through Cypress Grove to WRF (6,400 lf)

In summer 2024, the District repaired the existing approach lines upstream and downstream of the Contra Costa Canal bridge crossing at E. Cypress Road and Main Street due to an unforeseen pipe failure. This portion of the system will be reviewed and potentially reinstalled/reconfigured as part of the 14-inch force main project. As an additional component of this project, the potential to route Cypress Grove, Emerson Ranch, and Gilbert Ranch pump station flows back to the Summer Lakes 1 pump station so that this flow might be routed back to the Bethel Island ponds during future emergency and/or construction operations will be evaluated. Because these projects are not yet defined, the cost for these two potential project add-ons are not included in the force main cost estimate below.

Figure 5.9 Installed and Proposed 14-inch Force Main Pipeline Section



5.2.5 Evaluation of System Capacity to Accommodate Future Conversions from Septic to Sewered

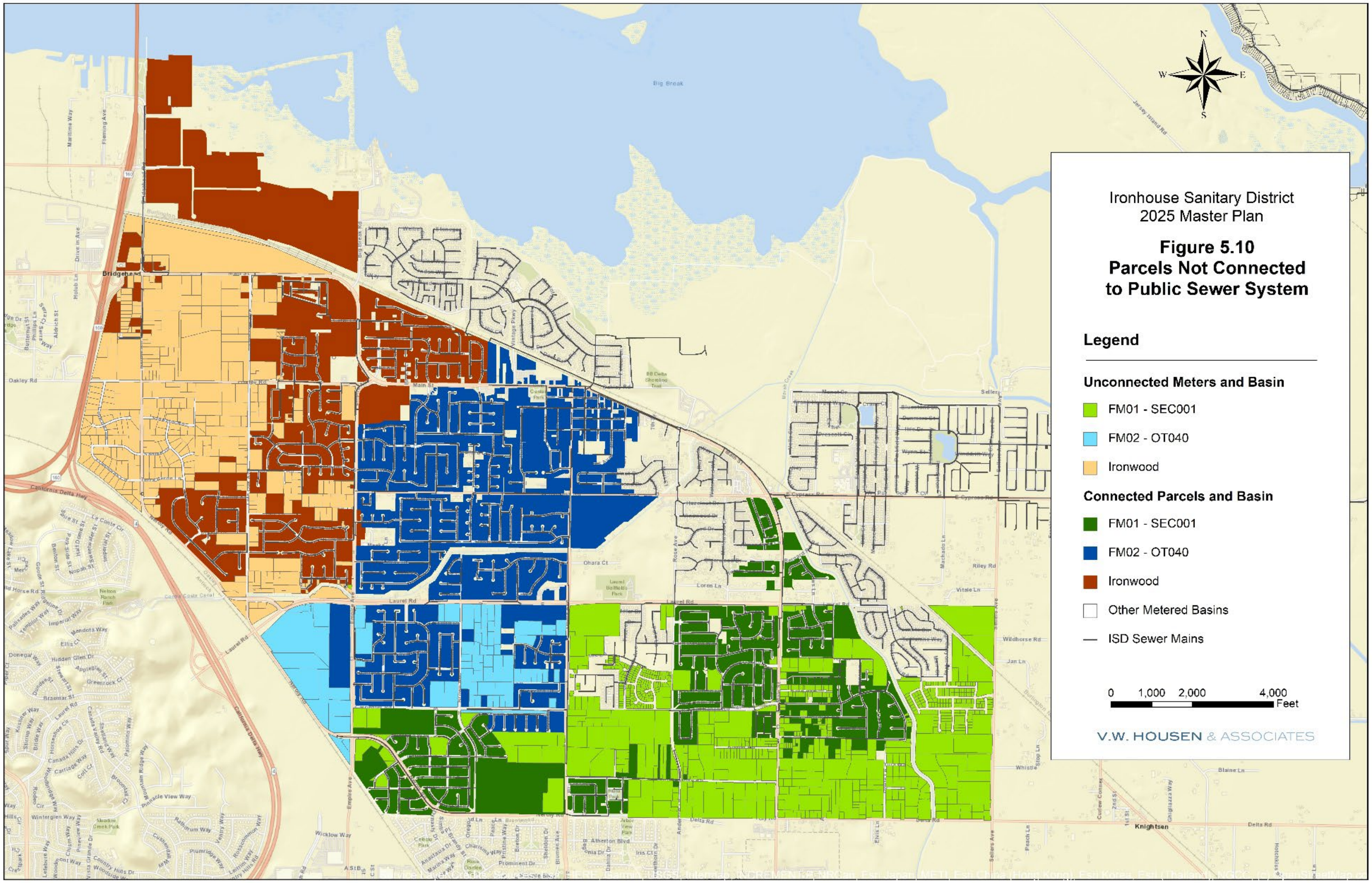
Approximately 1,150 parcels comprising approximately 1,500 acres of land have assigned land uses in the City of Oakley General Plan with the potential to generate sewer flows. These parcels are currently not connected to the District's sewer system. Figure 5.10 on the following page shows the locations of the unconnected parcels, and the meter that these parcels would be associated with if connected.

VWHA completed an analysis of flows for a hypothetical buildout condition that assumes that one third of these parcels will become connected to the District's system. The analysis also assigned a wet weather peaking factor of 3.0 to the calculated dry weather flows. Table 5.3 shows the parcels and flows that are assumed to require sewer service in the buildout scenario.

Table 5.3 Summary of Unconnected Parcels and Estimated Buildout Flows

Meter	Land Use	# Parcels	# Acres	Estimated Flow (all parcels)	Estimated DW Flow (one-third of parcels connected)	Estimated WW Flow (PF = 3.0)
Ironwood	Single-Family Residential	255	322	33,800	10,900	33,800
	Multi-Family Residential	3	0.6	1,500	500	1,500
	Commercial and Industrial	107	312	156,200	52,100	156,200
	Other	4	11	0	0	0
				Subtotal, Ironwood	*63,500*	*190,600*
FM-01 (SEC001)	Single-Family Residential	725	646	118,300	39,500	118,300
	Multi-Family Residential	3	3	1,100	375	1,100
	Commercial and Industrial	28	73	44,000	14,700	44,000
	Other	7	29	0	0	0
				Subtotal, FM-01	*54,500*	*163,500*
FM-02 (OT040)	Single-Family Residential	9	20	1,600	500	1,600
	Multi-Family Residential	1	2.8	750	250	750
	Commercial and Industrial	4	3	2,200	700	2,200
	Other	59	173	0	0	0
				Subtotal, FM-02	*1,500*	*4,500*
TOTAL		**1,205**	**1,596**	**358,500**	**119,500**	**358,500**

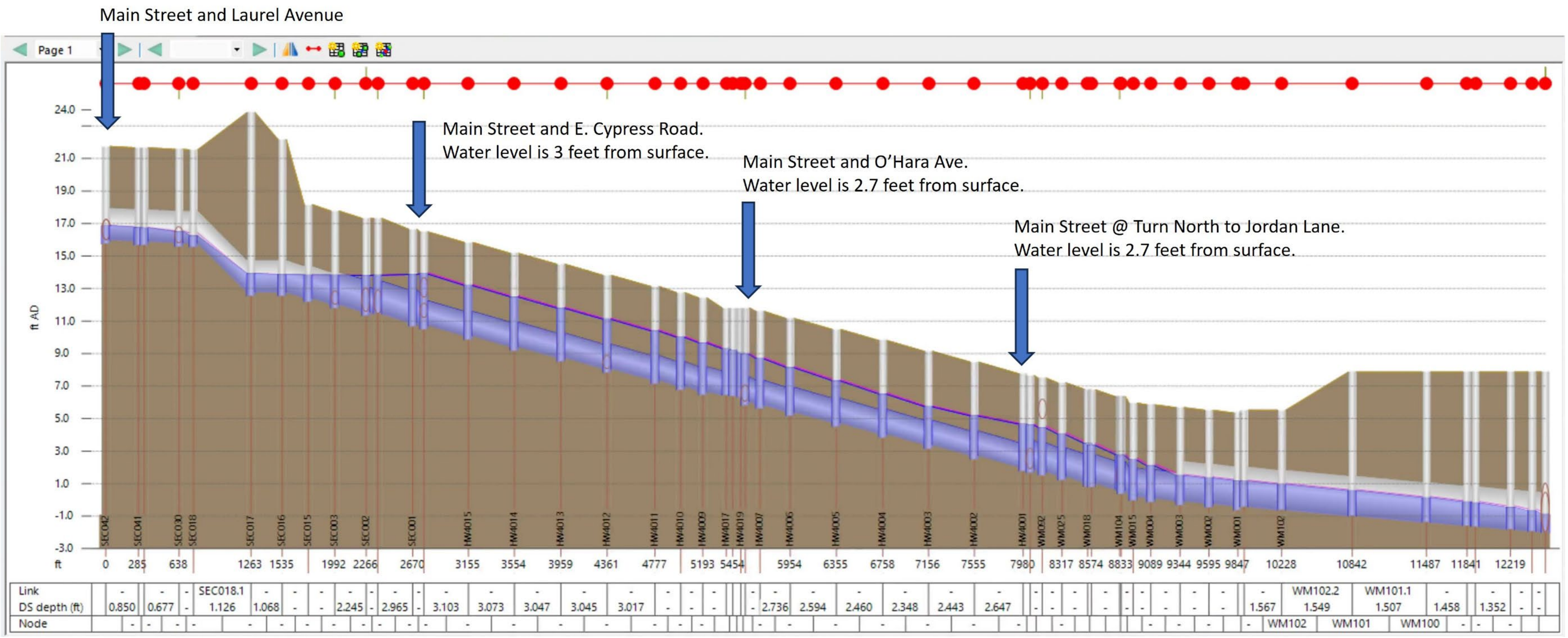
Figure 5.10 Currently Unconnected Parcels and Associated District Meter



This buildout analysis indicates that during the wet weather scenario, assuming a wet weather peaking factor of 3.0, if one third of all unsewered parcels in the FM-01 basin (shown in light green on Figure 5.10) are connected to the District's existing sewer system, the interceptor on Main Street heading north to Jordan Avenue and the WRF will become surcharged during the design storm scenario. The surcharged state shows water level as within 2.7 to 3 feet below the manhole rim for a portion of this alignment.

The hydraulic profile for this alignment is shown on Figure 5.11 on the following page. The actual projected number and timing for new connections from the FM-01 parcels have not been identified. Therefore, it is recommended that the District monitor the number of new connections that occur in the FM-01 basin over the next 5 to 10 years, and to develop a projection for future buildout that will help establish an appropriate timeframe for future capacity upgrades.

Figure 5.11 Predicted Surcharge from SEC-001 and Contributing Basin if One Third of Unsewered Parcels are Connected to the ISD Sewer System



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5.3 CAPACITY PROJECT COSTS

The following planning level costs are recommended for proposed pipeline improvements. These costs are also applicable to the rehabilitation and replacement projects that are discussed later in this report.

Cost Assumptions:

- Pipe replacement unit cost: $30 per inch-diameter-foot of pipe. This unit cost was reduced to $27 per inch-diameter-foot of pipe for the 14-inch force main pipe that will be installed within open fields.
- Appurtenances, laterals, mobilization, and shoring: 50% of pipe installation unit cost
- Construction contingency: 30%
- Engineering and Administration: 40% of construction cost, including contingencies

All costs are indexed to Engineering News Record Construction Cost Index, San Francisco, August 2025, 15285.78.

Following this cost estimating approach, the estimated construction cost to complete the new 14-inch force main is $15.7 million, as shown on Table 5.4.

Table 5.4 Conceptual Cost for Remaining Sections of 14-inch Force Main Pipe

Pipe Segment	Diameter (in)	Length (ft)	Pipe Cost	Appurtenances & Other	Contingency	Engineering & Admin	Total
Bethel Island Road to Jersey Island Road	14	5,100	$ 2,142,000	$ 1,071,000	$ 963,900	$ 1,670,760	$ 5,847,660
Knightsen Avenue to Sellers Avenue	14	2,800	$ 1,176,000	$ 588,000	$ 529,200	$ 917,280	$ 3,210,480
Emerson Ranch/Cypress Grove to WRF	14	6,400	$ 2,419,200	$ 1,209,600	$ 1,088,640	$ 1,886,976	$ 6,604,416
Total Project		**14,300**	**$ 5,737,200**	**$ 2,868,600**	**$ 2,581,740**	**$ 4,475,016**	**$ 15,662,556**

5.4 REVIEW OF STATEWIDE WASTE DISCHARGE REQUIREMENTS FOR CAPACITY ANALYSIS

The new State Water Resources Control Board Order WQ 2022-103-DWQ (Statewide WDR) became effective as of June 5, 2023. The Statewide WDR includes specific requirements for capacity analyses that are discussed in this section. The requirements of the Statewide WDR are summarized in Section 5.10 as follows:

The Enrollee shall maintain the system capacity necessary to convey: (1) base flows during dry weather conditions, and (2) wet weather peak flows consistent with designated local historic storms. Design storms must take into account system-specific stormwater contributions via inflow and infiltration, and location-specific depth of groundwater and storm frequencies. The Enrollee shall implement capital improvements to provide adequate hydraulic capacity to: • Meet or exceed the design criteria as defined in the Enrollee's System Evaluation and Capacity Assurance element of its Sewer System Management Plan; and • Prevent system capacity-related spills, and adverse impacts to the treatment efficiency of downstream wastewater treatment facilities.

The capacity analysis described in this Chapter addresses all of these requirements. However, Section 8.2 of the WDR further describes the following:

The capacity assessment must consider: • Data from existing system condition assessments, system inspections, system audits, spill history, and other available information; • Capacity of flood-prone systems subject to increased infiltration and inflow, under normal local and regional storm conditions; • Capacity of systems subject to increased infiltration and inflow due to larger and/or higher-intensity storm events as a result of climate change; • Increases of erosive forces in canyons and streams near underground and aboveground system components due to larger and/or higher-intensity storm events; • Capacity of major system elements to accommodate dry weather peak flow conditions, and updated design storm and wet weather events; and • Necessary redundancy in pumping and storage capacities.

The District's Master Plan addresses data from system condition assessments via the Rehabilitation and Replacement program, considers both wet and dry weather peak flow conditions, and reviews the system under a number of wet weather events including the design storm event. Further, the hydraulic analysis reviews and confirms that the District has sufficient pump station capacity. The District has a redundant pump at each station.

This section discusses two areas of the Statewide WDR that are not otherwise addressed and evaluated through the capacity assessment described above: 1) Capacity of systems subject to increased I&I due to larger and/or higher-intensity storm events as a result of climate change; and 2) increase of erosive forces in canyons and streams near underground and aboveground system components due to larger and/or higher intensity storm events.

5.4.1 Capacity of Systems Subject to Increased I&I from Larger or Higher-Intensity Storms

As discussed above, on December 31, 2022, the District received a rainfall event with reported rainfall between 4.6 to over 6 inches over a 24-hour period. Using the NOAA precipitation depth table from Figure 5.2, above the December 31, 2022 storm had a recurrence interval of between 100 and 500 years (for a 24-hour event) and exceeded the District's design storm. During this rainfall event, the District utilized the emergency storage basins at Bethel Island ponds to manage flows from Bethel Island, and otherwise had no spills from the wastewater collection system. This storm can be used as an example of how the existing system has sufficient capacity to manage increased flows that could result from larger and/or higher-intensity storm events.

The future system will include a new 14-inch force main that will relieve the only predicted surcharge condition beginning on Main Street and leading into the WRF. These improvements, when constructed, will improve the capacity of the system further to not only manage the design storm flows, but also increased future flows similar to the December 31, 2022 wet weather event.

As related to climate change, sea level rise along the coastal areas of the District's service area will most likely impact the Bethel Island properties and associated sewers. These sewers are currently subjected to dry weather GWI due to high groundwater, some of which has been addressed through the Bethel Island I&I Reduction project. The remaining gravity sewers are set back from the shoreline. Therefore, the District's gravity system is not expected to be inundated by flooding as a result of sea level rise resulting from climate change.

5.4.2 Increase Of Erosive Forces in Canyons and Streams due to Higher Intensity Storm Events

The District's service area, including waterways, are mostly built-out and/or channelized. The District did not observe any new erosion over existing sewer pipelines in natural waterway crossings and/or marshlands as a result of the December 31, 2022 rainfall event. However, a closer inspection of the system should be conducted after future wet weather seasons to identify any areas of concern related to potential future erosion in the vicinity of existing gravity sewers.

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CHAPTER 6 PIPELINE REHABILITATION AND REPLACEMENT

The District's wastewater collection system is considered young, with over 80 percent of the system having been installed after 1980. The oldest portion of the system is located near Main Street, east of the City of Oakley downtown core. In 2020, the District completed the "Old Town Pipeline Replacement project," which replaced approximately 9,700 lineal feet of 6-inch to 15-inch gravity sewer pipe within this area. In 2024, the District completed the Bethel Island I&I Reduction project, which used grout sealing to address open structural defects within 15 miles of gravity sewers that are located on Bethel Island. Although these two projects addressed the majority of known structural defects and associated inflow and infiltration within the gravity collection system, the Old Town project did not replace all of the known severe structural defects within the oldest part of the system. Therefore, as a component of this Master Plan, available CCTV data for this area was reviewed for the purposes of developing a funding plan for the District's future pipeline rehabilitation and replacement ("R&R") capital improvement plan.

This Chapter presents recommended pipeline rehabilitation projects and priorities, describes the cost estimating methodology, and provides an estimated budget for the proposed projects.

This Chapter is organized as follows.

6.1 Introduction and Approach for Rehabilitation and Replacement

6.2 Results and Project Recommendations

6.3 Estimated Costs

6.1 INTRODUCTION AND APPROACH TO PIPELINE REHABILITATION

The District records pipeline defects using the National Association of Sewer Service Companies ("NASSCO") Pipeline Assessment and Certification Program ("PACP"). NASSCO PACP software is used to assign both Structural and Operations & Maintenance ("O&M") defect scores and codes to a pipe segment during closed circuit television ("CCTV") inspection. The NASSCO PACP scoring system assigns a score of "1" for minor defects up to a score of "5" for the most severe defects. Pipes without defects receive a score of "0." Separate scores are assigned to structural defects that may lead to pipe failure, as compared to O&M defects that can be addressed through routine cleaning and maintenance operations.

CCTV inspection records were provided for pipes in the area bounded by and including the Vintage Parkway neighborhood to the north, Empire Avenue to the west, Chianti Way and the Contra Costa Canal to the south, and Fifth Street to the east. Within this area, the oldest pipes in the system are found. Approximately 36 miles of pipes were included in this portion of the inspection program, as shown on Figure 6.1.

Approximately 3.6 miles or 10 percent of the inspected pipelines have recorded structural defects. 1.4 miles of the pipes with recorded structural defects had at least one NASSCO PACP structural Grade 4 or 5 defect. A structural Grade 4 defect is a fracture, crack, hinge, or similar defect that is still considered a closed fracture and is therefore not indicative of imminent pipe failure. A structural Grade 5 defect is an open hole or break with soil or a void visible beyond the defect, or a hinge fracture or similar defect that could fail within the foreseeable future, which could be within up to five years. The District's R&R CIP prioritizes the repair of structural Grade 5 and 4 defects. When reviewing the pipes with structural Grade 5 and 4 defects, the CIP also considers adjacent pipes with structural Grade 3 defects,

Figure 6.1 on the following page shows pipes with NASSCO PACP structural defects and their associated maximum structural defect scores.

After all known structural Grade 4 and 5 defects are addressed through the CIP, the District's risk will be substantially reduced. If the District decides to defer the repair of known structural Grade 4 and 5 defects beyond a 10-year timeframe, then additional prioritization criteria should be considered. These criteria, which assign additional Likelihood of Failure and Consequence of Failure metrics to each pipe, will help to identify pipes with known defects that present the greatest risk to the District. This risk profile can then be used to determine which projects should be accelerated and which can be deferred.

Additional Likelihood of Failure factors may include criteria such as unresolved spills, liquefaction potential, and/or pipes that cannot be accessed. Recommended Consequence of Failure parameters are discussed below.

Consequence of Failure

Consequence of Failure metrics are used to assess how the failure of an asset will impact the ability of the District to meet its Level of Service goals.

- **Proximity to Waterway**. Waterways are available through the County of Contra Costa open GIS data portal. A pipe would receive an elevated risk of failure if it is located in proximity (e.g., within 50 or 100 feet) to an active waterway.

- **Proximity to Parks, Schools, and Other Critical Facilities.** Parks and schools with potentially higher impacts to human health could be elevated in the event of a sewer spill. Other similar facilities, such as police, fire, and commercial districts, may be identified by the District. These facilities can be identified from the County's parcel database/shapefile. A pipe that crosses a critical facility polygon would have the highest risk.
- **Area Impacted**. Failure of a large pipe, which typically serves a larger area, would have a higher consequence than failure of a small collector sewer. The District may consider pipes that are 10 inches or larger as more critical pipelines when prioritizing repairs.
- **Transportation Impact**. Roadways are designated as a line feature in the County streets shapefile. Any pipes that cross ore are adjacent to a primary or secondary arterial roadway, or a railroad, would receive a higher consequence of failure score in the risk assessment. Example primary and secondary arterials are listed in Table 6.1.

Figure 6.1 Pipes with NASSCO PACP Structural Defects and Scores

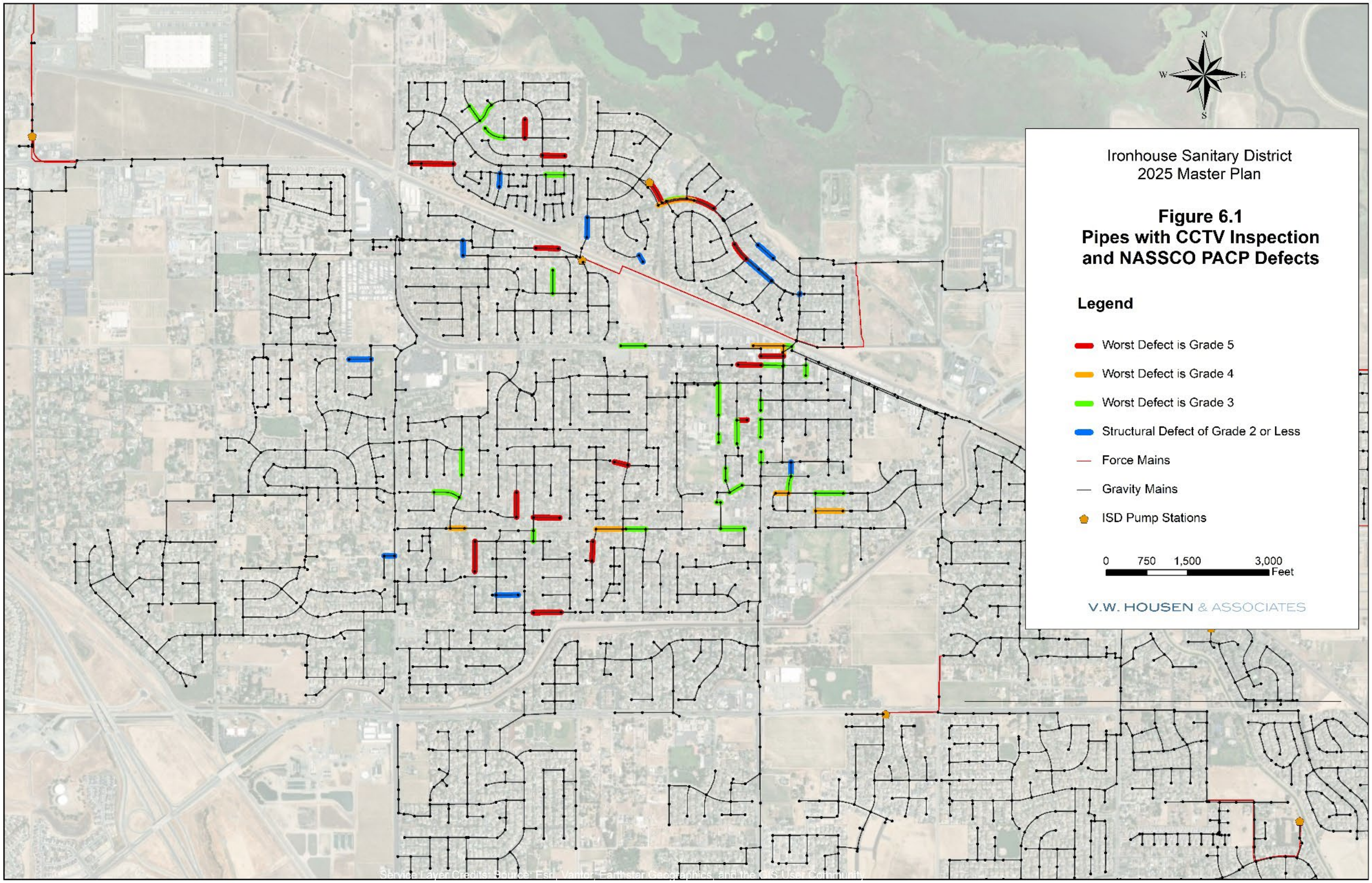


Table 6-1. Example Primary and Secondary Arterials

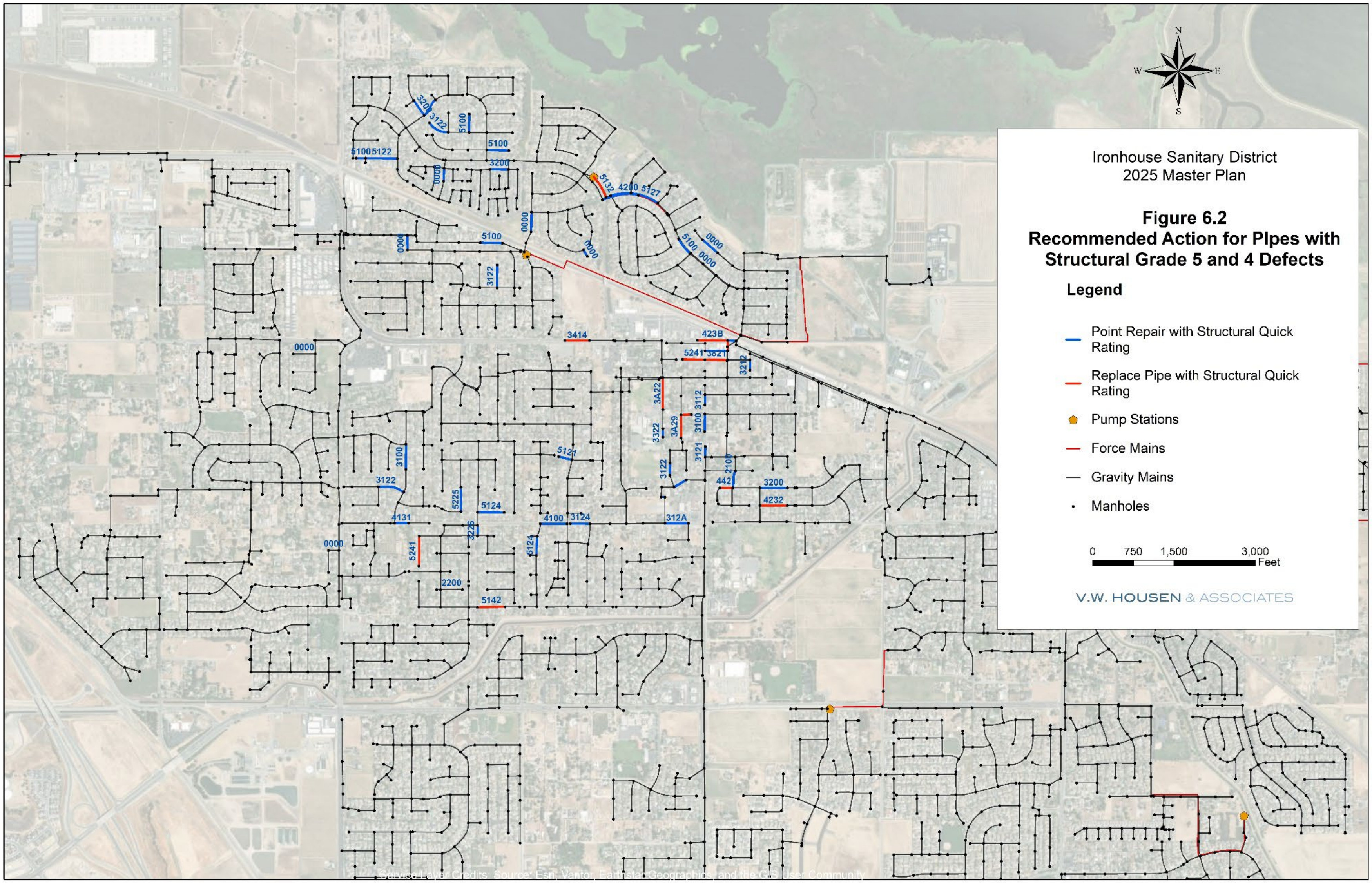
Example Primary Arterial	Example Secondary Arterial
• Main Street • E. Cypress Road • Bethel Island Road • Laurel Road	• Neroly Road • Empire Avenue • O'Hara Road • W. Cypress Road

6.2 RESULTS AND PROJECT RECOMMENDATIONS

63 lines comprising 17,631 combined lineal feet (3.3 miles) had structural Grade 4 or 5 defects, or structural Grade 3 defects that are recommended for repair. Of these lines, 52 pipes had one to three isolated defects that are identified as receiving point repairs. 11 lines with a combined length of 2,730 lineal feet (0.5 miles) are recommended for replacement. In total, this program would address 3.3 miles of pipe within the oldest part of the system with known structural Grade 4 and 5 defects, and some structural Grade 3 defects.

Figure 6.2 on the following page shows the minimum recommended action for pipes with structural Grade 4 and 5 defects.

Figure 6.2 Minimum Recommended Action for Pipes with Structural Grade 4 and 5 Defects



The total estimated cost to address the pipes shown on Figure 6.2 is $4,574,500. This cost was developed using the same cost estimating basis that was discussed in Section 5, as follows:

- Pipes to be replaced were assigned costs as follows. All existing 6-inch diameter pipes were assumed to be replaced with 8-inch diameter pipe. All other pipes were assumed to be replaced with the same size of pipe.
  - $30.00 per inch-diameter of pipe * replacement pipe diameter
  - Factor of 1.5 applied to account for mobilization, demobilization, trenching, shoring, and lateral reinstatement
  - Factor of 1.3 applied to account for unforeseen conditions such as high water table, concrete pavement, deep pipe, conflicting utilities, etc.
  - Factor of 1.4 to include design (10%), construction management (10%), District administration including soils testing (5%), and contingency (15%)
- Pipes to receive point repairs were assigned costs as follows:
  - $15,000 per point repair
  - Factor of 1.3 applied to account for unforeseen conditions such as high water table, concrete pavement, deep pipe, conflicting utilities, etc.
  - Point repairs were assigned based on the number of Grade 3 through 5 defects, and without reviewing actual CCTV footage. Review of CCTV footage may determine that some pipes shown as requiring only point repairs should actually be replaced. $1 million was added to the project budget to cover these costs, assuming 20 percent of pipes noted as having point repairs are actually replaced from manhole to manhole.

This program is shown in the CIP as being initiated in fiscal year 2025-26 and completed by fiscal year 2030-31. Maps showing the proposed projects are included in Appendix B. Additional information regarding identified defects, NASSCO PACP scores, and proposed projects is included in Appendix C. All costs are indexed to Engineering News Record Construction Cost Index, San Francisco, August 2025, 15285.78. The rehabilitation and replacement CIP is summarized in Table 6.2.

Table 6.2 Rehabilitation and Replacement Project Summary

Sewer Basin	Point Repair (Pipes)	Replace (Pipes)	Length (ft)	Cost
California Dawn	1	0	350	$ 19,500
Ironwood	3	0	867	$ 58,500
Old Town	28	13	11,364	$ 2,689,200
Vintage Parkway	12	2	4,058	$ 477,300
Walnut Meadows	4	0	998	$ 78,000
Allowance to Convert Point Repairs to Pipe Replacements (15 percent of designated point repairs in this table)				$ 1,587,600
Total	**48**	**15**	**17,637**	**$ 4,910,100**

After all NASSCO PACP Structural Grade 4 and 5 defects are addressed through the program above, it is recommended that the District maintain an annual budget of $850,000 per year to continue to maintain the system on an ongoing basis. This budget would be sufficient to address approximately ½ of one percent of the system annually through currently unplanned pipe replacements and point repairs.

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CHAPTER 7 CAPITAL IMPROVEMENT PROGRAM

This Chapter consolidates the Capital Improvement Program (“CIP”) components of Chapter 5 - Capacity Analysis and Chapter 6 – Pipeline Rehabilitation and Replacement Plan. For more information about each of these planning efforts and about the individual projects listed in this consolidated CIP, please see the individual plans that are included in the respective Chapters.

The Capital Improvement Program is designed with the expectation that Summer Lakes North and Grand Cypress Preserve developments will not be fully built out for at least four years. Therefore, construction of the new 14-inch force main is planned in phases, beginning in fiscal year 2025-26 and ending in fiscal year 2029-30.

The Rehabilitation and Replacement Program is structured so that all known NASSCP PACP structural Grade 4 and 5 defects are addressed through either point repairs or pipe replacement within the next five years, ending in fiscal year 2030-31.

Table 7.1 presents the project descriptions, costs, and proposed year of implementation, designating project priority.

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Table 7.1 Capital Improvement Program

Project Description	FY 2025/26	FY 2026/27	FY 2027/28	FY 2028/29	FY 2029/30	FY 2030/31	FY 2031/32	FY 2032/33	FY 2033/34	FY 2034/35	10-Year Total
14-inch Parallel FM	$250,000	$350,000	$7,500,000	$7,500,000							$15,600,000
Collection System Renovation Program - Phase I - California Dawn (1 point repair) - Ironwood (3 point repairs) - Old Town (28 point repairs and 14 line replacements) - Vintage Parkway (13 point repairs and one line replacement) - Walnut Meadows: 4 point repairs) - Allowance to complete full replacement of 20 percent of pipes currently shown as receiving point repairs	$150,000	$900,000	$900,000	$900,000	$900,000	$900,000	$260,100				$4,910,100
Collection System Renovation Program - Phase 2							$589,900	$850,000	$850,000	$850,000	$3,139,900
Total - Collection System Program	**$400,000**	**$1,250,000**	**$8,400,000**	**$8,400,000**	**$900,000**	**$900,000**	**$850,000**	**$850,000**	**$850,000**	**$850,000**	**$23,650,000**

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Appendix A

2024 Flow Monitoring Report (V&A)

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Housen/Ironhouse Sanitary District

2024 Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study

Prepared for: V.W. Housen & Associates
1470 Maria Lane, Suite 320
Walnut Creek, CA 94596

Report Date: November 19th, 2024

Prepared by: V&A consulting engineers

V&A Project No. 24-0061

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Abbreviations and Acronyms

Abbreviations/Acronyms	Definition
ADWF	Average Dry Weather Flow
AVG.	Average
CCTV	Closed-Circuit Television
CIP	Capital Improvement Plan
CO	Carbon Monoxide
DIA.	Diameter
d/D	Depth/Diameter Ratio
FPS	Feet/Second
ft.	Feet
FM	Flow Monitor
GPD	Gallons per Day
GPM	Gallons per Minute
GWI	Groundwater Infiltration
H2S	Hydrogen Sulfide
in.	Inch
I/I	Inflow and Infiltration
IDM	Inch-Diameter Mile
IDW	Inverse Distance Weighting
LEL	Lower Explosive Limit
MAX.	Maximum
MGD	Million Gallons per Day
MIN.	Minimum
NOAA	National Oceanic and Atmospheric Administration
N/A	Not applicable
PF	Peaking Factor
PS	Pump Station
PWS	Personal Weather Station
Q	Flow Rate
QAQC	Quality Assurance Quality Control
RDI	Rainfall-Dependent Infiltration
RG	Rain Gauge
SSO	Sanitary Sewer Overflow
V&A	V&A Consulting Engineers, Inc.
WEF	Water Environment Federation
WRCC	Western Regional Climate Center
WU	Weather Underground

Terms and Definitions

Term	Definition
Average dry weather flow (ADWF)	The 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 is expressed as a numeric average and may include the influence of normal groundwater infiltration (not related to a rain event).
Basin	Sanitary sewer collection systems upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances, also refer 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	The depth of water in a pipe is a fraction of the pipe's diameter. A measure of the fullness of the pipe used in the 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. **Combined 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 are 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 stormwater. The stormwater 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.
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.
Peak Wet Weather Flow	The highest daily flow during and immediately after a significant storm event includes sanitary flow, infiltration, and inflow.
Peaking factor (PF)	PF is the 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 the 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.

Executive Summary

Scope and Purpose

This executive summary is a general overview of V&A Consulting Engineers' (V&A) findings, conclusions, and recommendations of the temporary flow performed on portions of the Ironhouse Sanitary District's (ISD) sanitary sewer collection system. V&A Consulting Engineers was retained by V.W. Housen & Associates (Housen) to perform sanitary sewer flow monitoring and rainfall monitoring with I/I analysis over a period of four weeks from February 16th to March 20th, 2024. Open-channel flow monitoring was conducted at two scoped flow monitoring locations along with level loggers at five pump stations in Oakley and Bethel Island, California. There were three general purposes of this study:

1. Establish the baseline sanitary sewer flows at the flow monitoring sites
2. Establish the peak flow condition during the rainfall events and indicate the relative available sewer capacity at the flow monitoring nodes.
3. Quantify I/I at the applicable flow monitoring sites, isolate flow monitoring basins (where applicable), and conduct I/I analysis to determine basins with the highest relative I/I contributions.

Monitoring Sites

The flow monitoring site locations were selected and approved by Housen and the ISD and are listed in Table ES-1.

Table ES-1. List of Monitoring Sites

Monitoring Site	Manhole No.	Monitored Pipe	Measured Pipe Diameter (in)	Location
FM01	SEC002	Southwest Inlet	23.5	In the grass at 4532 Main St., Oakley
FM02	OT040	South Inlet	16	102 2nd St., Oakley
Ironwood PS[1]	N/A	N/A	N/A	West of 5286 Ironwood Ln, Oakley
Stone 2 PS[1]	N/A	N/A	N/A	4420 Windsweep Rd, Bethel Island
Summer Lakes 1 PS[1]	N/A	N/A	N/A	3200 E Cypress Rd, Oakley
Taylor 1 PS[1]	N/A	N/A	N/A	Southwest of 2383 Taylor Rd, Bethel Island
Willow 2 PS[1]	N/A	N/A	N/A	3995 N Willow Road #9, Bethel Island
[1]Pump Station wet well level logger only.				

The rainfall monitoring site locations were selected by V&A and are listed in Table ES-2.

Table ES-2. List of Rain Gauge Sites

Rain Gauge	Source	X (Longitude)	Y (Latitude)	Total Rain (in)
RG West	WU (KCAOAKLE32)	-121.698	37.976	1.89
RG North	WU (KCABETHE16)	-121.622	38.030	2.57
RG South	WU (KCAOAKLE62)	-121.729	38.007	2.25

Findings

The following briefly discusses the findings of V&A's evaluation of the wastewater collection system flow data within the Study Area.

- A total of 460,420 linear feet of sanitary sewer was flow monitored as part of the project.
- The maximum pipe diameter within the metered collection system was 24 inches.
- The minimum pipe diameter with the metered collection system was 4 inches.
- The metered collection system contained VCP, PVC, C900, and unidentified material. Most of the piping was unidentified material.
- A triangulated total of 1.97 inches of rainfall was recorded during the flow metering period. One event from this period was analyzed: Rain Event 1 (February 17th – 23rd), which had a triangulated average of 1.47 inches. It is classified as <5-year, 1-hour storm.
- All sites appear to have adequate dry weather flow capacity with d/D ratios ranging from 0.14 to 0.25.
- All sites had a peaking factor below 7.
- Site FM02 was the only site to have a max d/D > 0.75. Site FM02 had a d/D ratio of 1.22 and surcharged 3.5 inches above the pipe crown.
- Flow monitoring Sites FM01 and FM02 ranked highest for normalized peak I/I rates per the three inflow categories utilized for analysis.
- Flow monitoring Sites FM01 and FM02 ranked highest for overall RDI per the three RDI categories utilized for analysis.
- Flow monitoring Sites FM01 and FM02 ranked highest for overall combined I/I per the three categories utilized for analysis.
- All pump stations indicated GWI rates higher than typical standards, indicating possible high groundwater infiltration rates.

Figure ES-1 shows the location of all monitoring devices utilized as part of this study. Figure ES-1 also indicates which sites/basins are recommended for potential follow-up investigation work. These recommendations are detailed in the following section.

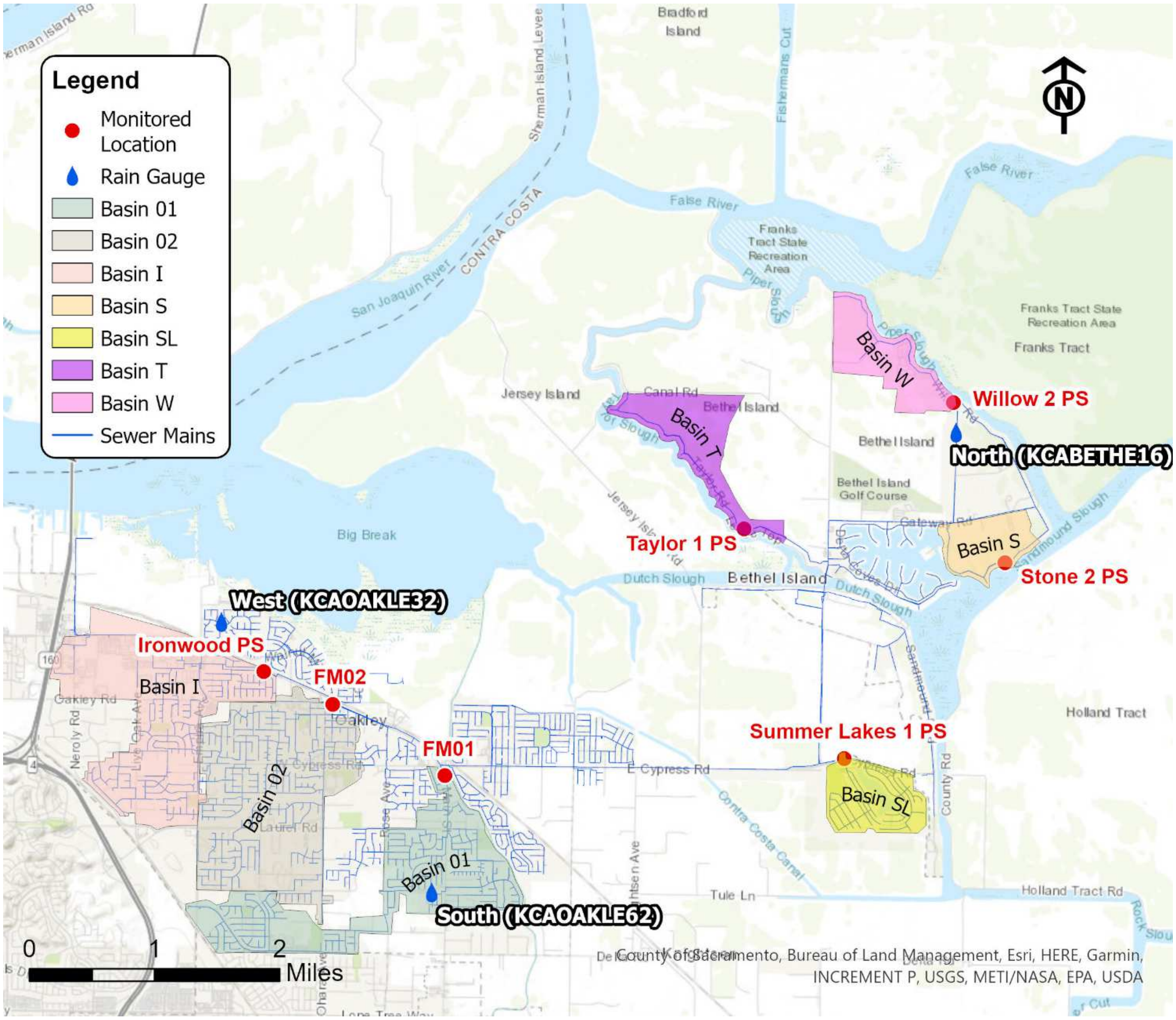


Figure ES-1. Map of Monitoring Sites and Basin

Recommendations

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

1. Master Plan and Model Implementation: This study focuses on inflow and infiltration generation; the study results can be used to update the master plan and compare with previous model assumptions and flow monitoring results.
2. Capacity Analysis: One site (FM02) was surcharged during the monitoring period, approximately 3.5 inches above the pipe crown. The following possible capacity concerns are noted:
    a. Dry weather: No site showed an average flow level greater than 50% full pipe conditions (greater than 0.5 d/D).
        i. Site FM02 appears to have experienced a surcharge immediately after the rain event. Levels returned to normal afterward.
        ii. No site had a peaking factor greater than 7.
    b. Wet Weather: With the exception of Site FM02, wet weather peaking factors ranged from 1.5 to 4.9. This is typically considered to be within acceptable limits, and no further action is recommended unless the City wishes to pursue I/I identification. It should be noted that while these limits may be within normally acceptable limits, higher-intensity storms than those that occurred during the monitoring period could cause higher wet weather responses.
3. Determine I/I Reduction Program: If the City should decide to pursue some I/I reduction in the future, consider the following I/I reduction programs:
    a. If peak flows, sanitary sewer overflows, and pipeline capacity issues are of greater concern; then priority can be given to investigating and reducing sources of inflow within the basins with the greatest inflow problems. The highest inflow occurred within the basins for Sites FM02 and Willow 2 Pump Station.
    b. Suppose total infiltration and general pipeline deterioration are of greater concern. In that case, the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems. The highest RDI occurred in the basins for Sites FM01 and FM02. Additionally, all basins for the pump stations showed possible evidence of elevated GWI.
4. I/I Investigation Methods: Potential I/I investigation methods include the following:
    a. Smoke testing.
    b. Mini-basin flow monitoring.
    c. Night-time reconnaissance work is done to (1) investigate and determine direct point sources of inflow and (2) determine the areas and/or pipe reaches responsible for high levels of infiltration contribution.
    d. CCTV inspection.
5. I/I Reduction Cost Effective Analysis: If the City is considering pursuing I/I reduction, a study can be conducted 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 Introduction

1.1 Scope and Purpose

V&A Consulting Engineers (V&A) was retained by Housen & Associates (Housen) to perform sanitary sewer flow monitoring and rainfall monitoring with I/I analysis within the Ironhouse Sanitary District (ISD) collection system. Flow and rainfall monitoring were performed over the period of four weeks from February 16th to March 20th, 2024. Open-channel flow monitoring was conducted at two scoped flow monitoring locations along with level loggers at five pump stations in Oakley and Bethel Island, California. There were three general purposes of this study:

1. Establish the baseline sanitary sewer flows at the flow monitoring sites
2. Establish the peak flow condition during the rainfall events and indicate the relative available sewer capacity at the flow monitoring nodes.
3. Quantify I/I at the applicable flow monitoring sites, isolate flow monitoring basins (where applicable), and conduct I/I analysis to determine basins with the highest relative I/I contributions.

1.2 Flow Monitoring Sites and Isolated Sewerage Basins

Flow monitoring sites are defined as the manholes where flow monitors are secured and the pipelines in which flow sensors are placed. Capacity analysis and flow rate information are presented on a site-by-site basis. The flow monitoring sites were selected and approved by Housen and the City. Information regarding the flow monitoring locations is listed in Table 1-1 and illustrated in

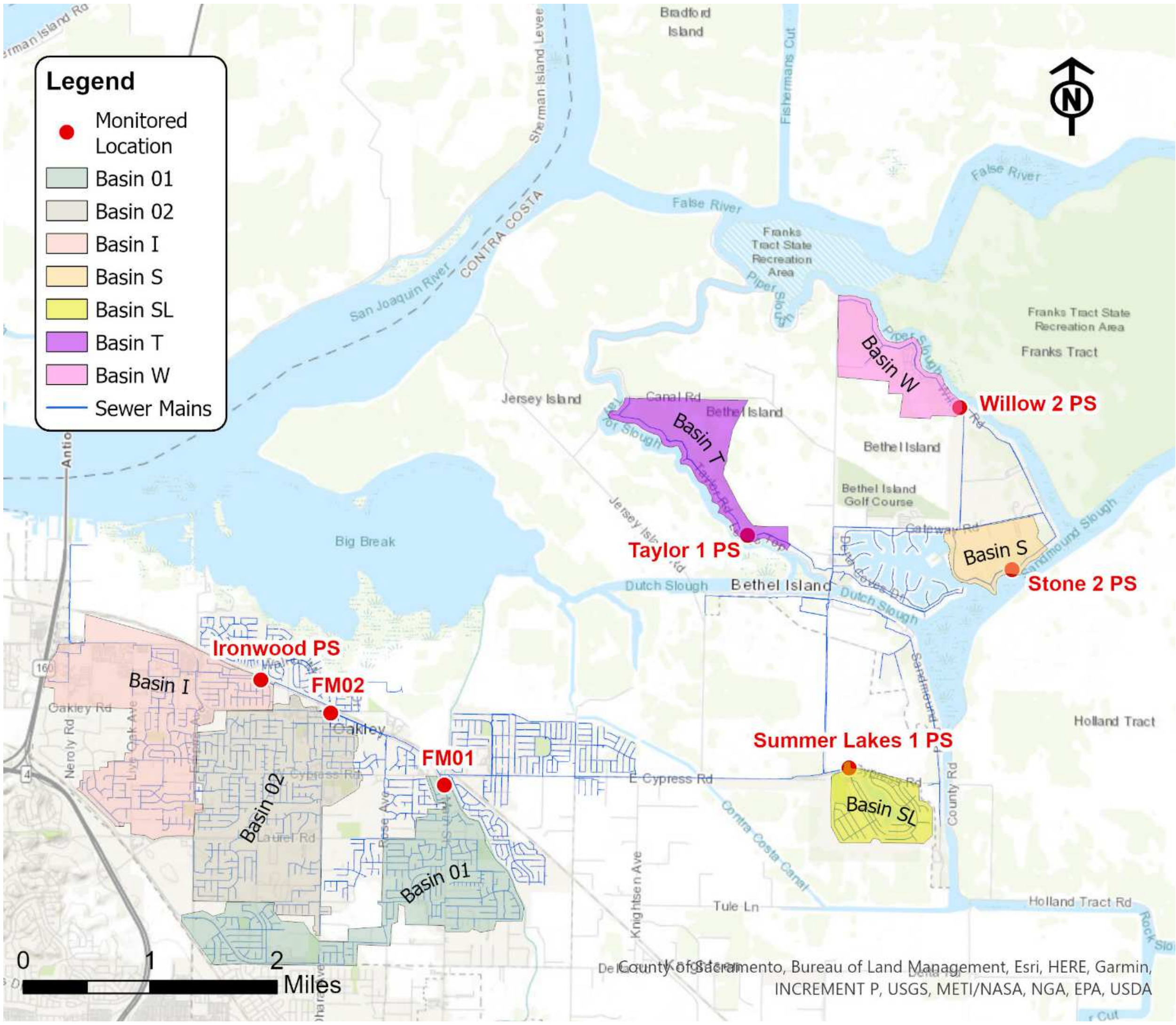


Figure 1-1. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in Appendix A.

Flow monitoring site data may include the flows of one or many drainage 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, requiring basin isolation (subtraction of upstream flows). The I/I analysis results will be presented on an isolated basin basis. The basins, basin attributes, and basin isolation equations are listed in Table 1-2.

Table 1-1. List of Monitoring Locations

Monitoring Site	Manhole No.	Monitored Pipe	Expected Pipe Diameter (in)	Measured Pipe Diameter (in)	Location
FM01	SEC002	Southwest Inlet	24	23.5	In the grass at 4532 Main St., Oakley
FM02	OT040	South Inlet	15	16	102 2nd St., Oakley
Ironwood PS[1]	N/A	N/A	N/A	N/A	West of 5286 Ironwood Ln, Oakley
Stone 2 PS[1]	N/A	N/A	N/A	N/A	4420 Windsweep Rd, Bethel Island
Summer Lakes 1 PS[1]	N/A	N/A	N/A	N/A	3200 E Cypress Rd, Oakley
Taylor 1 PS[1]	N/A	N/A	N/A	N/A	Southwest of 2383 Taylor Rd, Bethel Island
Willow 2 PS[1]	N/A	N/A	N/A	N/A	3995 N Willow Road #9, Bethel Island

[1]Pump Station wet well level logger only.

Table 1-2. Isolated Flow Monitoring Basin Characteristics

Isolated Basin	Flow Meter	Flow Isolation Calculation	Iso Area (Acres)	IDM[2]	Pipe Length (miles)
Basin 01	FM01	$Q_{01} = Q_{01}$	883	204	22.4
Basin 02	FM02	$Q_{02} = Q_{02}$	1,086	221	29.5
Basin I	Ironwood PS	$Q_{I} = Q_{I}$	1,003	205	21.4
Basin S	Stone 2 PS	$Q_{S} = Q_{S}$	196	13	1.8
Basin SL	Summer Lakes 1 PS	$Q_{SL}= Q_{SL}$	257	48	6.8
Basin T	Taylor 1 PS	$Q_{T}= Q_{T}$	322	22	3.0
Basin W	Willow 2 PS	$Q_{W}= Q_{W}$	288	17	2.3

[2]Inch-diameter miles (IDM) is the summation of the miles of sewer multiplied by the diameter (in inches) of sewer pipe.

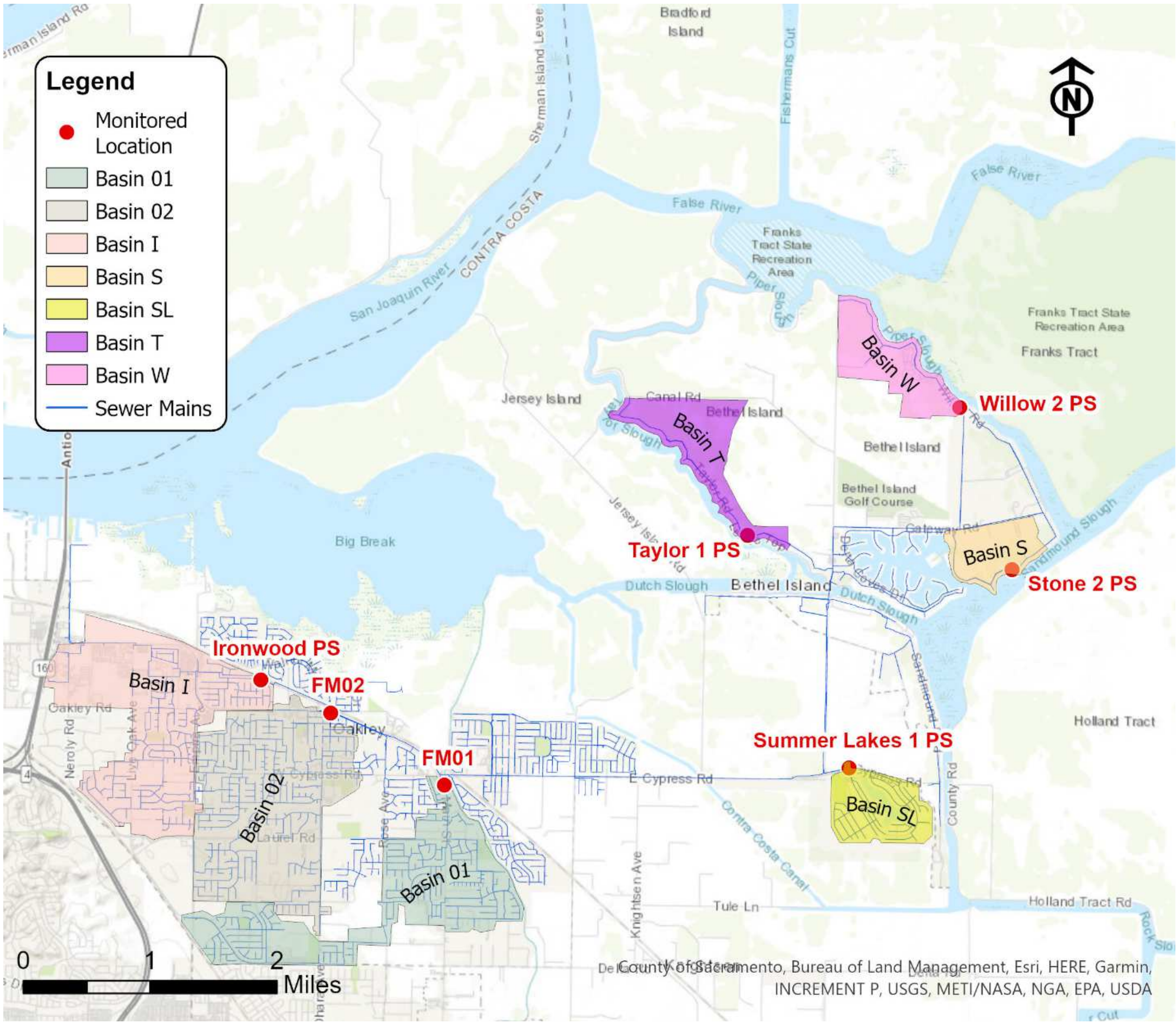


Figure 1-1. Map of Flow Monitoring Sites and Basins

2 Methods and Procedures

2.1 Confined Space Entry

A confined space (Photo 2-1) is defined as any space that is 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%), the presence 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 OSHA-defined responsibilities of Entrant, Attendant, and Supervisor. The Entrant is the individual performing the work. He or she is equipped with the necessary personal protective equipment 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 if there is more than one. The Supervisor is responsible for developing the safe work plan for the job at hand prior to entering.

Photo 2-1. Confined Space Entry

Photo 2-2. Typical Personal Four-Gas Monitor

2.2 Flow Meter Installation

V&A installed two area-velocity flow meters for temporary monitoring within the collection system using ISCO 2150 manufactured equipment. 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 of the pipe 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 flow meters, and again, when they were removed, they were compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. Figure 2-1 shows a typical installation for a flow meter with a submerged sensor.

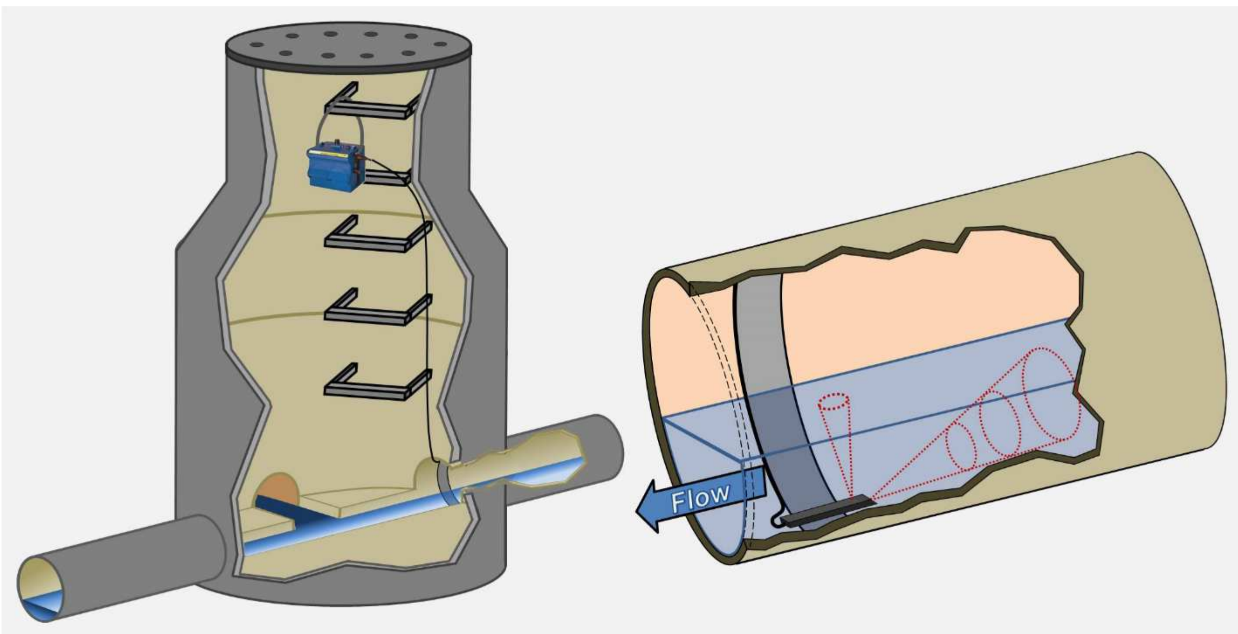


Figure 2-1. Typical Installation for ISCO 2150 Flow Meter with Submerged Sensor

2.3 Flow Calculation

Data retrieved from the flow meters is placed into a spreadsheet program for analysis. Data analysis includes 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 the 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 level and pipe invert

d_S: depth of sediment

D: pipe diameter

2.4 Measurement Error and Uncertainty

For traditional engineering applications, measurement "error" is a difference between a computed, estimated, or measured value and the generally accepted true or theoretically correct value. It can also be considered a difference between the desired and the actual performance of equipment. For equipment, an error is usually expressed as a percentage relative to accuracy (i.e., "…the velocity sensor has an accuracy of ±2% of the reading…").

However, for this study and flow monitoring applications, the cause of the measurement difference is important, and a distinction will be made between the equipment not performing to industry standards ("error") and expected inaccuracies ("uncertainty") associated with monitoring technology limitations.

Gauging "error" occurs when the equipment does not meet industry standards. This can occur as a result of the following common categories of conditions that can be encountered at a wastewater monitoring site.

- Malfunctioning equipment (i.e., a sensor is damaged, battery life ends, or a desiccant canister becomes saturated)
- Improper equipment choice or maintenance (i.e., the selected gauging equipment technologies are incompatible with hydraulic conditions within the sewer, or excessive gravel deposits are allowed to accumulate around the sensors without being removed)
- Improper equipment calibration (i.e., depth and/or velocity measurements are incorrectly taken within the sewer, or equipment is allowed to drift out of calibration)
- Field conditions within the sewer (i.e., foaming at the water surface that "blinds" an ultrasonic depth sensor or toilet paper catching and accumulating on a combination sensor, blinding the acoustic Doppler velocity meter)

For flow monitoring applications, gauging "uncertainty" is used to describe and quantify the expected inaccuracies that result from the limitations of the technologies that utilize indirect measurements to quantify wastewater flow.

It is important to try to install flow meters in "ideal" flow conditions. Ideal flow conditions are generally defined as laminar flow in a straight-through, constant-slope pipeline with no disturbances (elbows, tees, hydraulic shifts, etc.) 10 diameters upstream and 5 diameters downstream from the flow monitoring location. If ideal flow conditions are met, then an expected uncertainty of final flow calculation from an open-channel flow meter may be approximately ±5%. In many situations, ideal flow conditions cannot be met, and uncertainties increase.

2.4.1 Flow Addition versus Flow Subtraction

Due to the uncertainties involved in subtracting flows of similar magnitudes, adding flows at multiple monitoring sites is usually preferred over subtracting flows. Subtraction becomes an issue, especially when the flow difference from the subtraction falls within the measurement uncertainty range of the two larger flow data sets (i.e., subtracting a large flow from another large flow to obtain a small difference).

The following example best demonstrates this concept:

1. Meter A measures 2.00 MGD of flow and has an expected uncertainty of ±5%; thus, the uncertainty range of the flow measurement is ±0.10 MGD.

2. Meter B measures 2.50 MGD of flow and has an expected uncertainty of ±6%; thus, the uncertainty range of the flow measurement is ±0.15 MGD.
3. Meter C measures 0.50 MGD of flow and has an expected uncertainty of ±8%; thus, the uncertainty range of the flow measurement is ±0.04 MGD.

Scenario 1 – Flow Addition

- Meter A + Meter B = 2.00 MGD (±0.10) + 2.50 MGD (±0.15) = 4.50 MGD (±0.25)
- Overall uncertainty = ±0.25 / 4.50 = ±5.6%
- The final uncertainty is essentially a weighted average of the component uncertainties for flow addition.

Scenario 2 – Flow Subtraction, Large Flow less Small Flow

- Meter B - Meter C = 2.50 MGD (±0.15) - 0.50 MGD (±0.04) = 2.00 MGD (±0.19)
- Overall uncertainty = ±0.19 / 2.00 = ±9.5%
- The final uncertainty will always be greater than the component uncertainties for flow subtraction.
- When subtracting a small flow from a large flow, the resulting uncertainties can still be manageable.

Scenario 3 – Flow Subtraction, Large Flow less a similarly Large Flow

- Meter B - Meter A = 2.50 MGD (±0.15) – 2.00 MGD (±0.10) = 0.50 MGD (±0.25)
- Overall uncertainty = ±0.25 / 0.50 = ±50%
- The resulting uncertainties may not be manageable when subtracting similarly sized flow rates. In this example, an uncertainty of ±50% may be considered unacceptable for confident analyses.

Scenario 3 is a very "real-world" situation. The uncertainties for Meter A and Meter B are extremely reasonable (indeed, most flow monitoring service providers would be extremely pleased with true meter uncertainties of ±5% to ±6%). However, the reality of the math is clear, and the above example demonstrates the concept of flow subtraction and compounding or inflating uncertainty ranges.

The following points are emphasized in relation to the items of this section:

- For subtraction of flows, the overall uncertainty can be an inflated value that far exceeds the component uncertainties.
- The smaller the resultant flow from the subtraction equation, the larger the percentage uncertainty.
- Whenever possible, basin flows should be directly measured rather than calculated as a subtraction of two or more flow meters.
- If flow subtraction cannot be avoided, it is better to have the magnitudes of the component flows be as dissimilar as possible.

2.5 Average Dry Weather Flow Determination

For this study, four distinct average dry weather flow curves were established for each site location:

- Mondays – Thursdays
- Fridays
- Saturdays
- Sundays

Flows for many sites differ on Friday evenings compared to Mondays through Thursdays. Starting around 7 p.m., the flows often decrease (compared to Monday through Thursday). Similarly, flow patterns for Saturday and Sunday were also separated due to their unique evening flow pattern. This type of differentiation can be important when determining I/I response, especially if a rain event occurs on a Friday, Saturday, or Sunday evening.

Figure 2-2 illustrates a sample of varying flow patterns within a typical dry week[1].

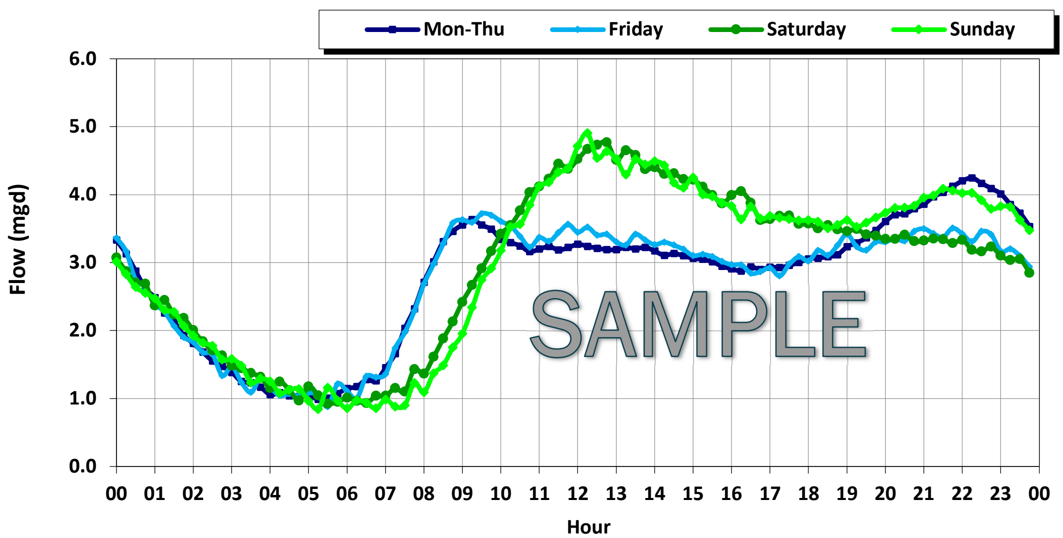


Figure 2-2. Sample ADWF Diurnal Flow Patterns

ADWF curves are taken from "Dry Days" when RDI had the least impact on the baseline flow. The overall average dry weather flow (ADWF) is calculated using the following equation:

$$ADWF = \left(ADWF_{Mon-Thu} \times \frac{4}{7}\right) + \left(ADWF_{Fri} \times \frac{1}{7}\right) + \left(ADWF_{Sat} \times \frac{1}{7}\right) + \left(ADWF_{Sun} \times \frac{1}{7}\right)$$

[1] Holiday flows can be extremely variable. Christmas flows are different from Thanksgiving flows and different from MLK Day flows. See Section 3.3 for details on whether holiday ADWF curves were established for this project's I/I analysis.

2.6 Flow Attenuation

Flow attenuation in a sewer collection system is the natural process of reducing the peak flow rate through redistribution of the same flow volume 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 2-3.

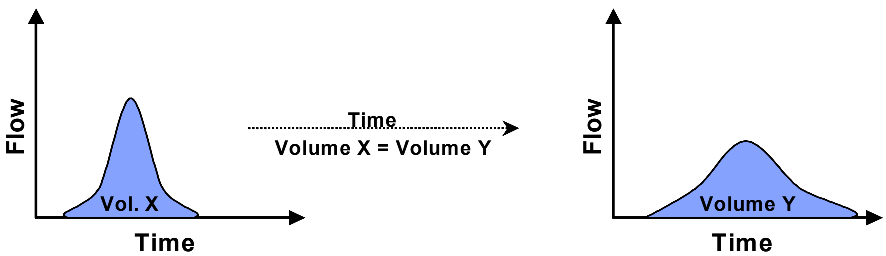


Figure 2-3. Attenuation Illustration

Each basin will have a specific flow profile within a sanitary sewer collection system. As the flows from the basins combine within the trunk sewer lines, the peaks from each basin will not necessarily coincide at the same time, and peak flows may attenuate prior to reaching the treatment facility due to the length and time of travel through the trunk sewers. 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.

2.7 Inflow / Infiltration Analysis: Definitions and Identification

Inflow and infiltration (I/I) consist of stormwater and groundwater that enters the sewer system through pipe defects and improper storm drainage connections and are defined as follows:

- Inflow: Stormwater inflow is defined as water discharged into the sewer system, including private sewer laterals, from direct connections such as downspouts, yard and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins.
- Infiltration: 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 2-4 illustrates the possible sources and components of I/I.

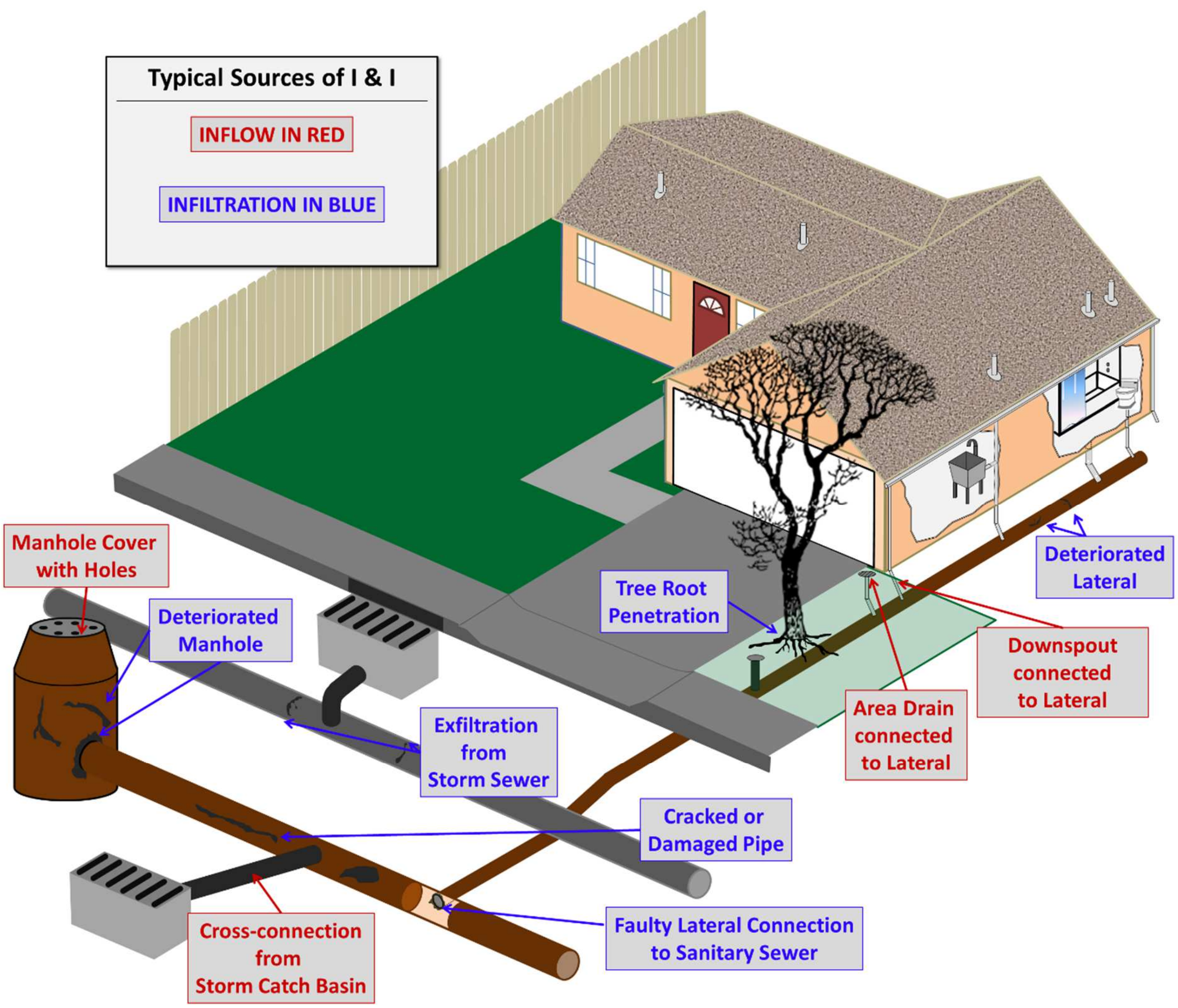


Figure 2-4. Typical Sources of Infiltration and Inflow

2.7.1 Infiltration Components

Infiltration can be further subdivided into components as follows:

- Groundwater Infiltration: Groundwater infiltration 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 groundwater infiltration rates are seasonal by nature. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.
- Rainfall-Dependent Infiltration: This component occurs as a result of stormwater and enters the sewer system through pipe defects, as with groundwater infiltration. The stormwater first percolates directly into the soil and then migrates to an infiltration point. Typically, the time of concentration for rainfall-related infiltration maybe 24 hours or longer, but this depends on the soil permeability and saturation levels.
- Rainfall-Responsive Infiltration is stormwater 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 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 is bedded and backfilled with 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.

2.7.2 Impact and Cost of Source Detection and Removal

- Inflow:
  - Impact: 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. Because the response and magnitude of inflow are closely tied to the storm event's intensity, the short-term peak instantaneous flows may result in surcharging and overflows within a collection system. Severe inflow may result in sewage dilution, resulting in upsetting the biological treatment (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. 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 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.

2.7.3 Graphical Identification of I/I

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 is typically sustained after rainfall stops and then gradually drops off as soils become less saturated and groundwater levels recede to normal levels. Real-time flows are plotted against ADWF to analyze the I/I response to rainfall events. Figure 2-5 illustrates a sample of how this analysis is conducted and some of the measurements that are used to distinguish infiltration and inflow. Similar graphs have been generated for the individual flow monitoring sites and can be found in Appendix A.

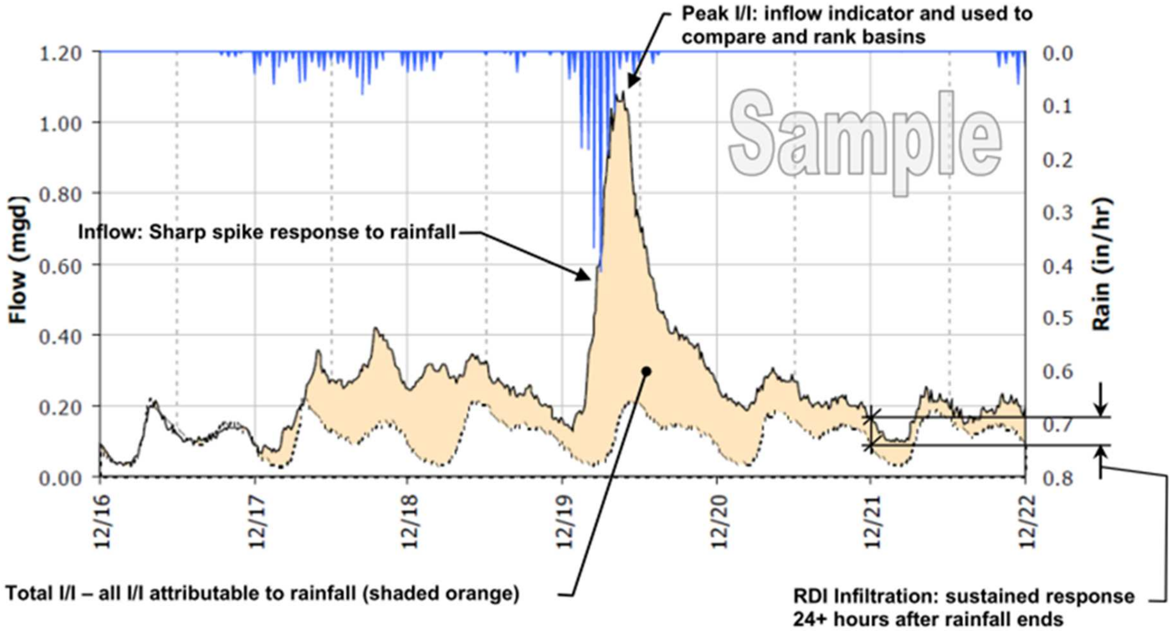


Figure 2-5. Sample Infiltration and Inflow Isolation Graph

2.7.4 Analysis Metrics

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 – Peak I/I Flow Rate: Inflow is characterized by sharp, direct spikes occurring during rainfall. Peak I/I rates are used for inflow analysis.
- Groundwater Infiltration (GWI): GWI analysis is conducted by looking at minimum dry weather flow to average dry weather flow ratios and comparing them to established standards to quantify the rate of excess groundwater infiltration.
- Rainfall-Dependent Infiltration (RDI): RDI Analysis is conducted by looking at the infiltration rates at set periods after the conclusion of a storm event. Depending on the collection system and the time required for flows to return to ADWF levels, different periods may be examined to determine the basins with the greatest or most sustained rainfall-dependent infiltration rates.
- Combined I/I: The combined inflow and infiltration are measured in gallons per site and storm event. Because it is based on combined I/I volume, it is used to identify the overall volumetric influence of I/I within the monitoring basin.

2.7.5 Normalization Methods

There are three ways to *normalize* the I/I analysis metrics for an "apples-to-apples" comparison among the different drainage basins:

- Per-IDM: The metric is divided by the length of pipe (IDM (inch-diameter mile)) contained within the upstream basin. Final units typically are gallons per day (gpd) per IDM.
- Per-Acre: The metric is divided by the acreage of the upstream basin. Final units typically are gallons per day (gpd) per Acre.
- Per-ADWF: The metric is divided by the established average dry weather flow rate and is typically expressed as a ratio. Peaking Factors are examples of using ADWF to normalize data from different sites.

The infiltration and inflow indicators were normalized by the methods listed above in this report, and these results will be shown in the sections relating to the I/I analysis results. For the purposes of basin rankings, the following weighting decisions are given:

- Per-IDM: Per-IDM values were assigned 30% weighting towards final rankings.
- Per-Acre: Per-ACRE values were assigned 30% weighting towards final rankings. The topography is known and should result in valid per-Acre analyses.
- Per-ADWF: Per-ADWF metrics were assigned 40% weighting towards final rankings. It is noted that abnormal waste usage could result in low ADWF values, which could skew results and lead to possible misinterpretation of data.

3 Results and Analysis

3.1 Rainfall Monitoring

3.1.1 Rain Gauge Locations

V&A analyzed rainfall data from three (3) publicly available private weather stations (PWS) on Weather Underground[2], allowing for solid coverage over the collection system, which has a diverse range of topographical features. Table 3-1 lists the recorded rainfall over the monitoring period at the site. Figure 3-1 illustrates the locations and labeling convention used for the rain gauge as well as the location of the metered basins.

Table 3-1. Recorded Rainfall

Rain Gauge	Source	X (Longitude)	Y (Latitude)	Total Rain (in)
RG West	WU (KCAOAKLE32)	-121.698	37.976	1.89
RG North	WU (KCABETHE16)	-121.622	38.030	2.57
RG South	WU (KCAOAKLE62)	-121.729	38.007	2.25

3.1.2 Flow Study Rainfall Data

One rainfall event elicited minor I/I responses during the flow monitoring period. The selected event is summarized in **Error! Reference source not found.** and illustrated in Figure 3-2 to provide context.

- Event 1 was a multi-day event from February 17th through the 23rd, which resulted in an I/I response. A triangulated average of 1.47 inches of rainfall was recorded over the 6-day period. This event was classified as approximately a <5-year, 1-hour storm.

Table 3-2. Summary of Rainfall Data

Rain Gauge	Rain Event 1 Feb 17th – 23rd	
	Inches of Rain	Classification
RG West (KCAOAKLE32)	1.53	5-year, 3-hour
RG North (KCABETHE16)	1.23	5-year, 1-hour
RG South (KCAOAKLE62)	1.26	<1-year, 1-hour
Triangulated to Basin Area Centroid	**1.47**	**<5-year, 1-hour**

[2] Weather Underground (wunderground.com) collects data from 180,000+ weather stations across the country, including Automated Surface Observation System (ASOS) at airports, personal weather stations (PWS), and Meteorological Assimilation Data Ingest System (MADIS) managed by the National Oceanic and Atmospheric Administration (NOAA). While V&A has no direct control over the rain gauges, V&A performs additional QA/QC on the data to assure its suitability for use.

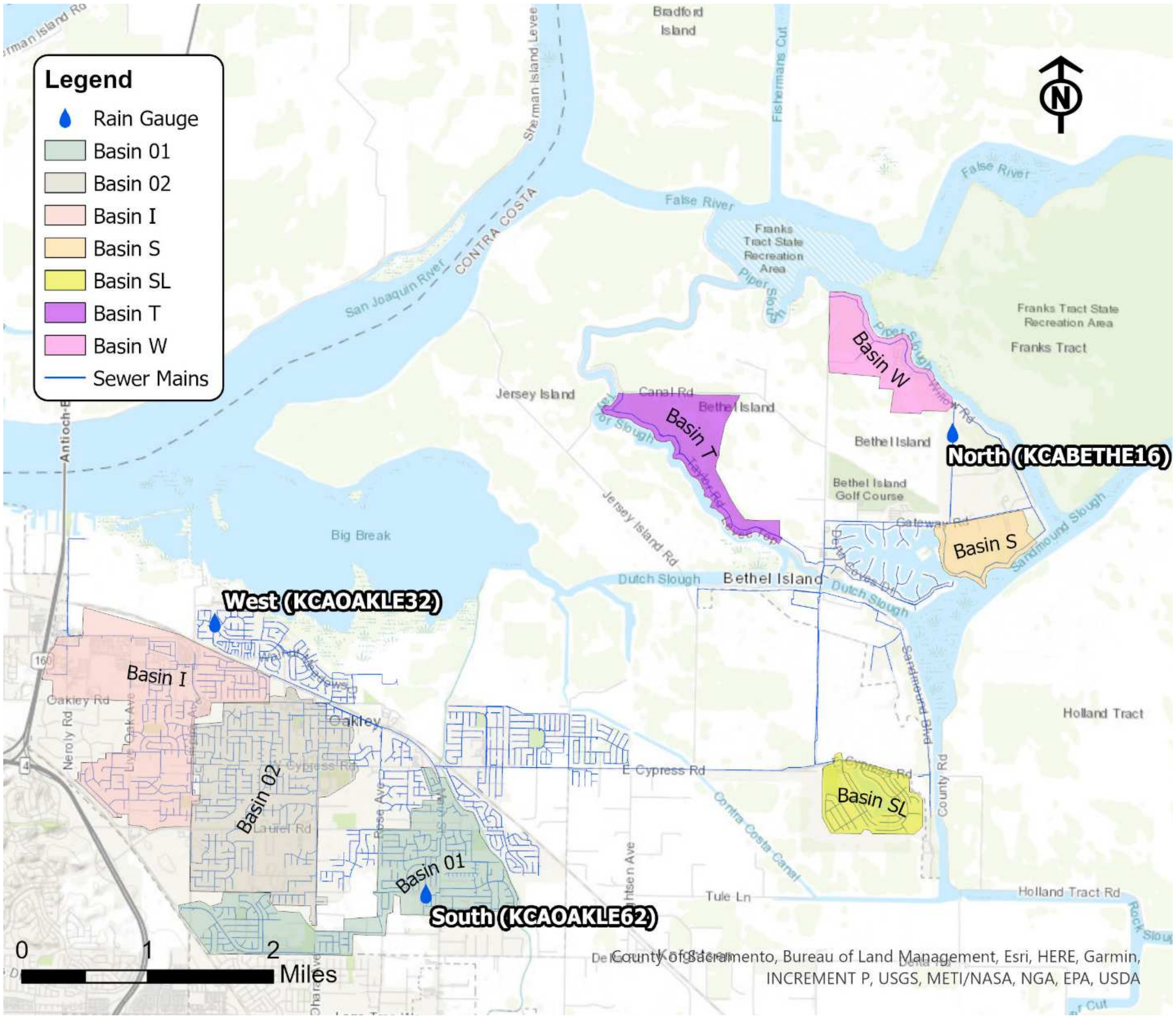


Figure 3-1. Location of Rain Gauges

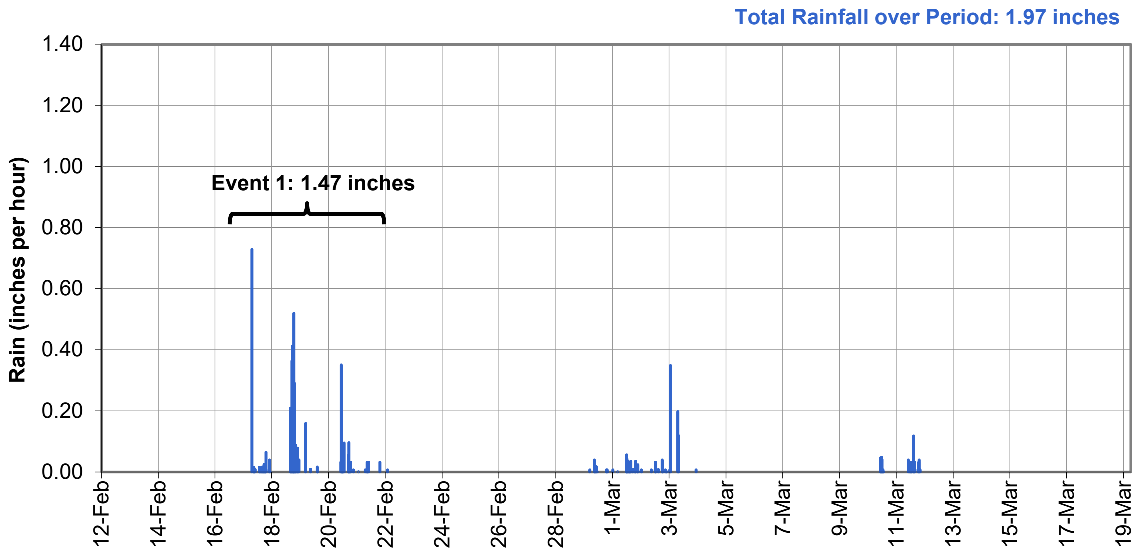


Figure 3-2. Rainfall Monitoring (triangulated to Basin Area Centroid)

Figure 3-3 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[3] (triangulated to the Basin Area Centroid), which was approximately 1.97/2.59 or 76% of the historical precipitation averages over the same period.

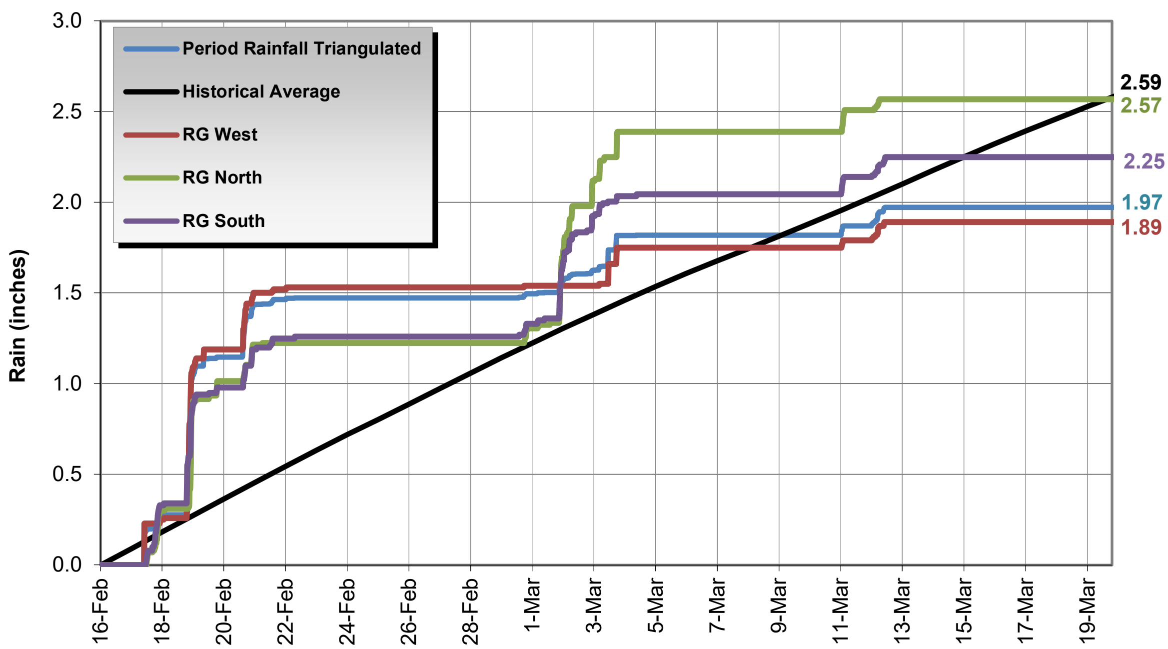


Figure 3-3. Rainfall Accumulation Plot

[3] Historical data taken from the WRCC (Station 047769 at San Fransico WSO AP): http://www.wrcc.dri.edu/summary/climsmnca.html

3.1.3 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[4]. Rainfall events are classified by intensity and duration. Based on historical data, the NOAA has developed frequency contour maps for storm events of given intensity and duration for all areas within the continental United States (Figure 3-4).

For example, the NOAA Rainfall Frequency Atlas[5] classifies a 10-year, 24-hour storm event at the West rain gauge as 2.46 inches. This means that in any given year, at this specific location, there is a 10% chance that 2.46 inches of rain will fall in any 24-hour period.

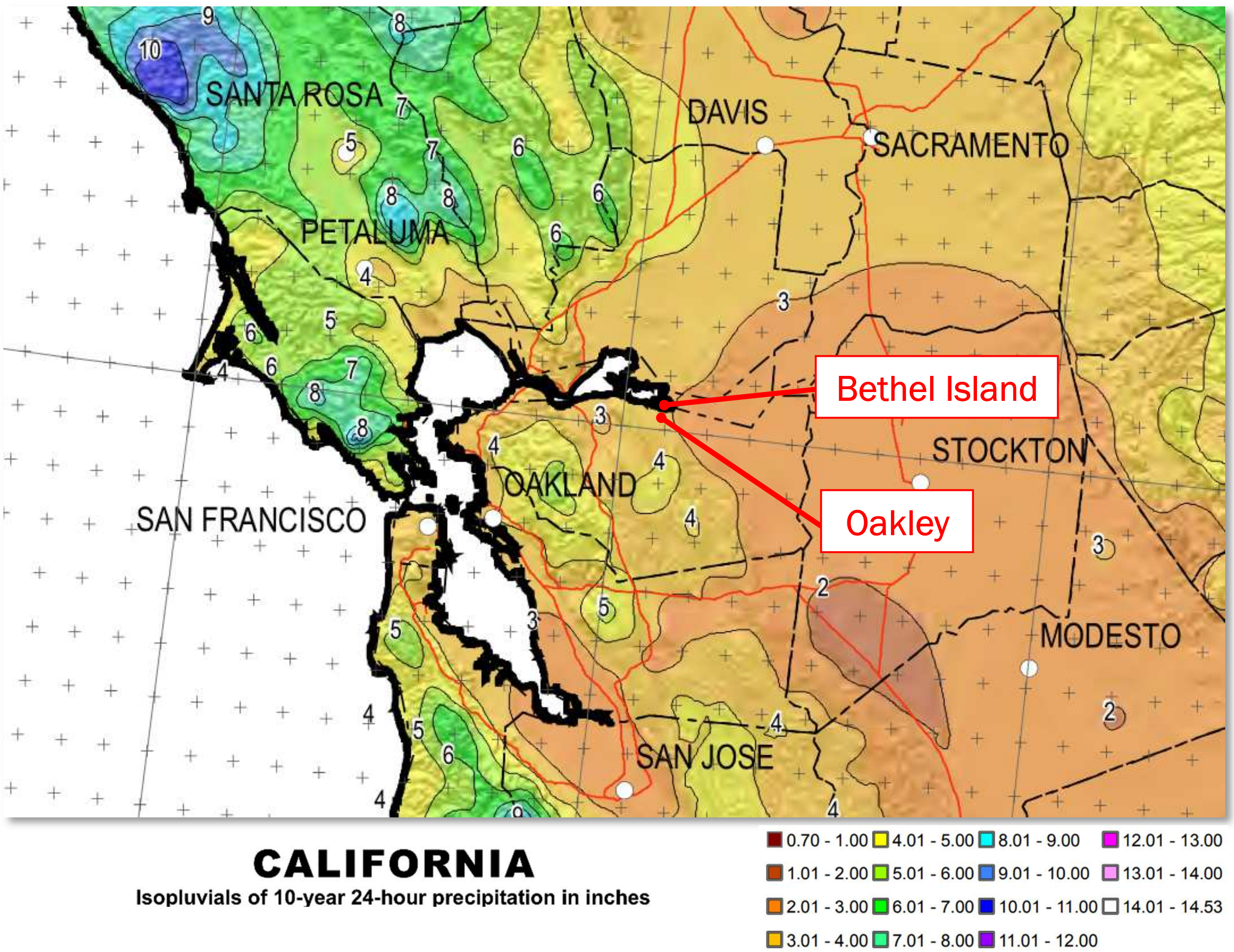


Figure 3-4. NOAA Northern California Rainfall Frequency Map

From the NOAA frequency maps, for a specific latitude and longitude, the rainfall densities for period durations ranging from 1 hour to 20 days are known for rain events ranging from 1-year to 10-year intensities. These are 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 frequency plot determines the classification of the rainfall event.

[4] Sanitary sewers are often designed to withstand I/I contribution to sanitary flows for specific-sized "design" storm events.

[5] NOAA Western U.S. Precipitation Frequency Maps Atlas 14, Volume 6, 2011: ftp://hdsc.nws.noaa.gov/pub/hdsc/data/sw/ca10y24h.pdf

Figure 3-5, Figure 3-6, and Figure 3-7 show the classification plots for the event from the rain gauges over the course of the flow monitoring period. Note that the event is classified as a 5-year, 3-hour storm at RG West but classified as a <1-year, 1-hour storm at RG South.

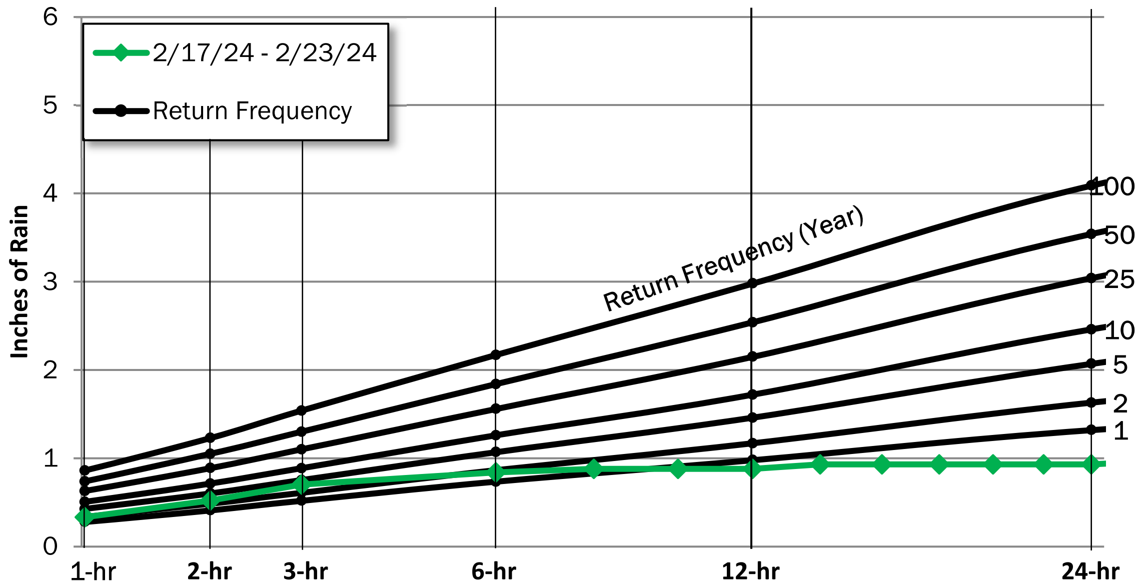


Figure 3-5. Rainfall Event Classification – 24-Hour Period (RG West)

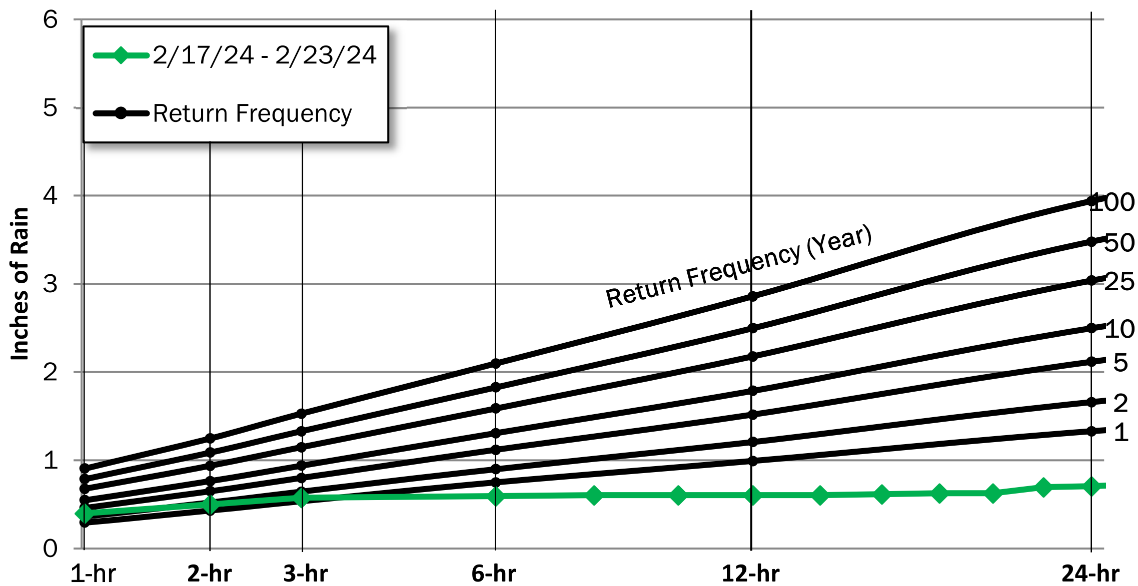


Figure 3-6. Rainfall Event Classification – 24-Hour Period (RG North)

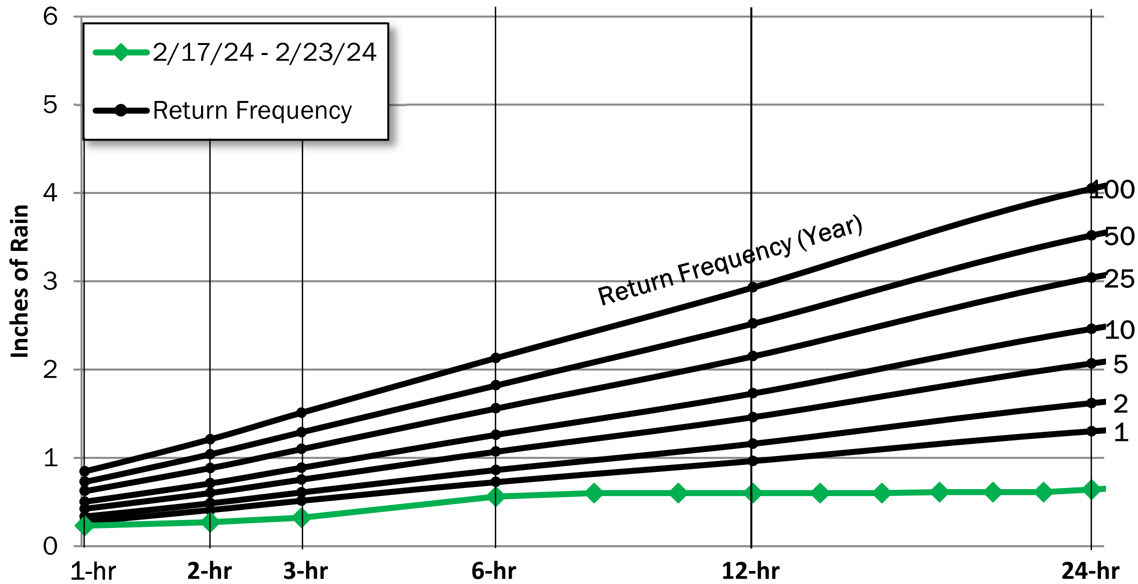


Figure 3-7. Rainfall Event Classification – 24-Hour Period (RG South)

3.1.4 Rain Gauge Triangulation Distribution

The rainfall affecting the sanitary sewer collection system basins must be calculated based on their proximity to the rain gauge locations. The mean precipitation for each site's upstream basin was calculated by taking data from the rain gauges and using the inverse distance weighting (IDW) method. IDW is an interpolation method that assumes the influence of each rain gauge location diminishes with distance. The center of an upstream basin[6] is identified, and a weighted triangulated average of the precipitation data from nearby rain gauge locations is taken.

The IDW function is as follows:

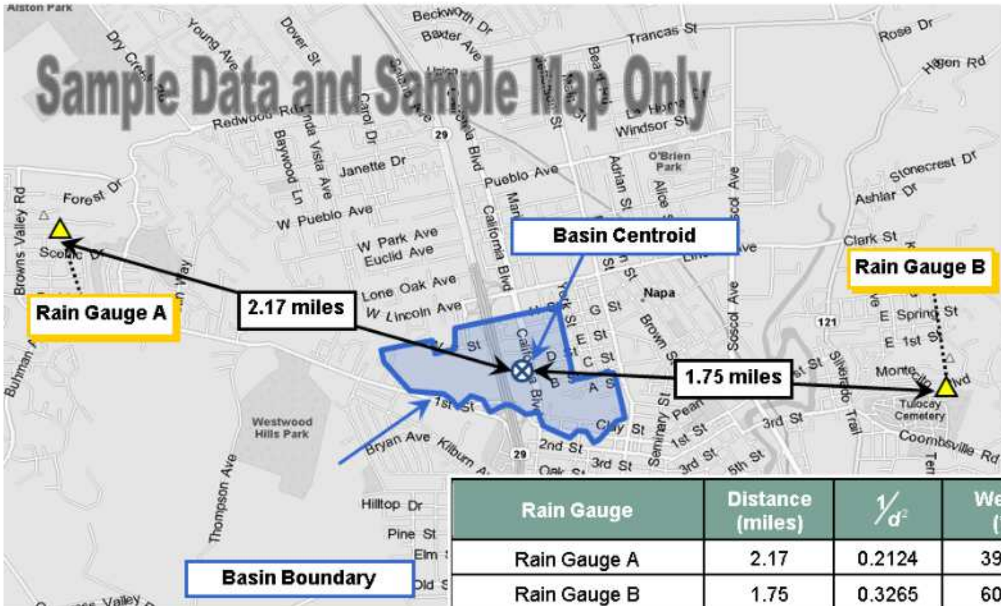
$$weight(d) = \frac{1/d^p}{\sum 1/d^p},$$

where: d = distance
p = power ($p > 0$)

The value of p is user-defined. The most common choice for hydrological studies of watershed areas is p = 2.

Figure 3-8 illustrates the IDW method with sample data. The rain gauge distribution, as calculated for each flow monitoring site, is shown in Table 3-3.

[6] Note that the full basin upstream of the site was used instead of the isolated basins as the rain data will be compared to the flow at each site



Rain Gauge	Distance (miles)	$\frac{1}{d^2}$	Weight (%)
Rain Gauge A	2.17	0.2124	39.4%
Rain Gauge B	1.75	0.3265	60.6%
Totals		**0.5389**	**100%**

Figure 3-8. Rainfall Inverse Distance Weighting Method

Table 3-3. Rain Gauge Distribution per Monitoring Site

Site	RG West	RG North	RG South
FM01	96%	1%	3%
FM02	47%	3%	50%
Ironwood PS	5%	1%	94%
Stone 2 PS	4%	93%	3%
Summer Lakes 1 PS	33%	52%	15%
Taylor 1 PS	17%	70%	13%
Willow 2 PS	0%	100%	0%

3.2 Flow Monitoring

3.2.1 Average Flow Analysis

Average dry weather flow (ADWF) curves were established during dry days when I/I had the least impact on the baseline flow. Table 3-4 summarizes the dry weather flow data measured for this study. ADWF curves for each site can be found in Appendix A. Figure 3-9 shows a flow schematic of the average daily flows and levels. The following ADWF analysis results are noted:

- No site showed an average dry weather flow level greater than 50% pipe full conditions (greater than 0.5 d/D).

Table 3-4. Dry Weather Flow

Monitored Site	Sediment (in.)	Average d/D Ratio	Mon-Thu ADWF (MGD)	Friday AWDF (MGD)	Saturday ADWF (MGD)	Sunday ADWF (MGD)	Overall ADWF (MGD)
FM01	0	0.14	0.274	0.267	0.284	0.308	0.279
FM02	0	0.25	0.355	0.328	0.407	0.539	0.385
Ironwood PS	N/A	N/A	0.111	0.110	0.116	0.122	0.113
Stone 2 PS	N/A	N/A	0.107	0.105	0.106	0.107	0.106
Summer Lakes 1 PS	N/A	N/A	0.071	0.065	0.071	0.080	0.072
Taylor 1 PS	N/A	N/A	0.112	0.109	0.110	0.111	0.111
Willow 2 PS	N/A	N/A	0.103	0.101	0.099	0.097	0.101

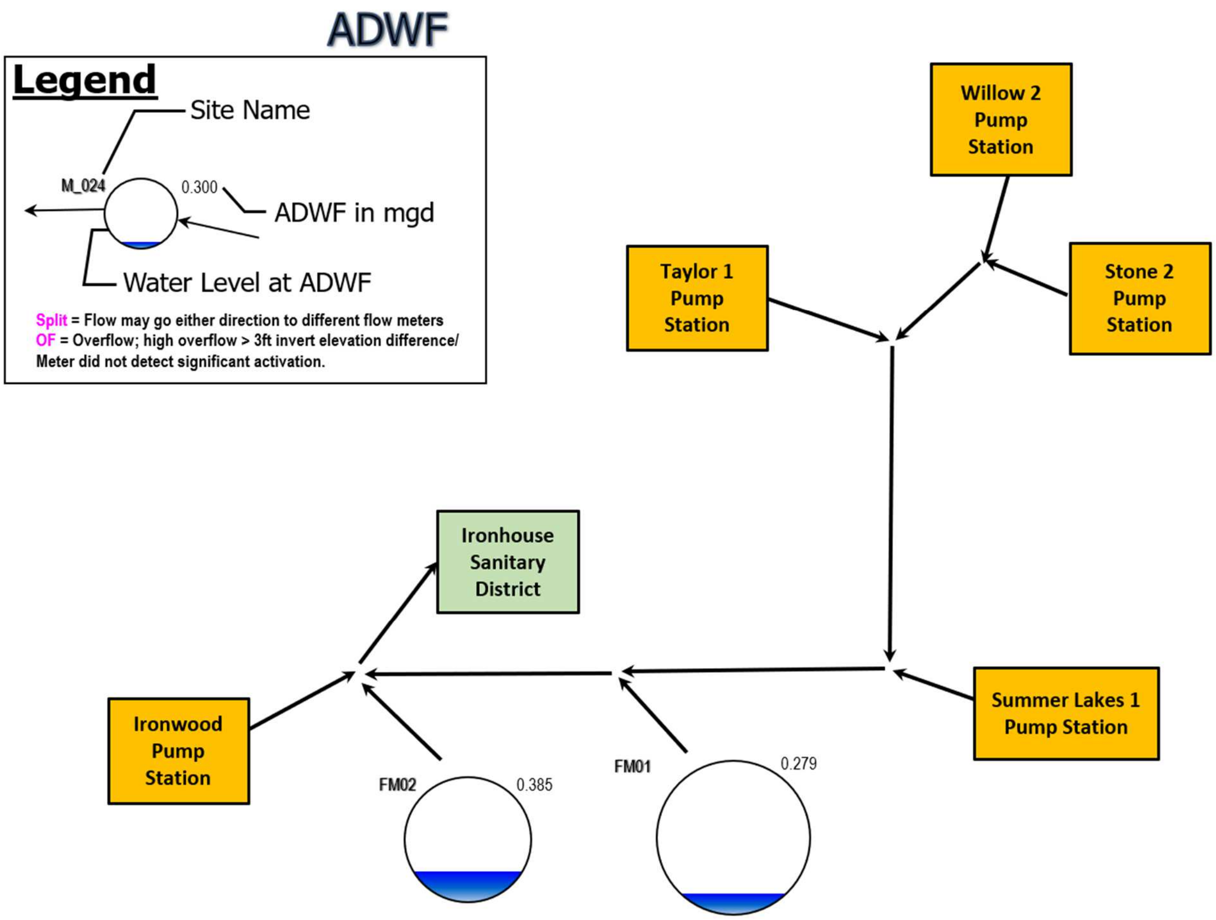


Figure 3-9. Average Dry Weather Flow (Flow Schematic)

3.2.2 Peak Measured Flows and Pipeline Capacity Analysis

Peak measured flows and the hydraulic grade line data (flow depths) are important to understanding the capacity limitations of a collection system. The peak flows and flow levels are the peak measurements taken across the entirety of the flow monitoring period. For this study, peak flows and peak levels correspond to rainfall events. The following capacity analysis definitions will be used:

- The Peaking Factor (PF) is defined as the peak measured flow divided by the average dry weather flow (ADWF). Peaking factors are influenced by many factors, including size and topography of the tributary area, flow attenuation, flow restrictions, characteristics of I/I entering the collection system, and hydraulic features such as pump stations.
  - For this report, if PF > 7 , it will be highlighted in RED[7]; however, the City should refer to City standards when evaluating peaking factors. Peaking factor data should be used at the discretion of the City Engineer.
- d/D Ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). The d/D ratio for each site is computed based on the maximum depth of flow for the study. Standards for d/D ratio vary from agency to agency but typically range between d/D ≤ 0.5 and d/D ≤ 0.75
  - For this report, if d/D ratio > 0.75 , it will be highlighted in RED; however, the City should refer to City standards when evaluating d/D ratios, to be used at the discretion of the City Engineer.

Table 3-5 summarizes the peak recorded flows, depths, *d/D* ratios, and peaking factors per site during the flow monitoring period. Capacity analysis data are presented on a site-by-site basis and represent the hydraulic conditions only at the site locations; hydraulic conditions in other areas of the collection system will differ. Figure 3-10 and Figure 3-11 show bar graph summaries of the peaking factors and d/D ratios, respectively. Figure 3-2 shows the schematic diagram of the peak measured flows in each section with peak flow levels.

The following capacity analysis results are noted:

- Peaking Factors
  - All sites had a peaking factor below 7.
- d/D Ratio:
  - Site FM02 surcharged 3.5 inches above the crown.

[7] WEF Manual of Practice FD-6 and ASCE Manual No. 62 suggests typical peaking factor ratios range between 3 and 4, with higher values possibly indicative of pronounced I/I flows.

Table 3-5. Capacity Analysis Summary

Site	ADWF (MGD)	Peak Measured Flow (MGD)	Peaking Factor	Pipe Diameter, *D* (in)	Max Depth, *d* (in)	Max d/D Ratio	Surcharge above pipe crown (in)
FM01	0.279	0.788	2.8	23.5	8.2	0.35	N/A
FM02	0.385	1.087	2.8	16	19.5	1.22	3.5
Ironwood PS	0.113	0.284	2.5	N/A	N/A	N/A	N/A
Stone 2 PS	0.106	0.165	1.5	N/A	N/A	N/A	N/A
Summer Lakes 1 PS	0.072	0.348	4.9	N/A	N/A	N/A	N/A
Taylor 1 PS	0.111	0.192	1.7	N/A	N/A	N/A	N/A
Willow 2 PS	0.101	0.255	2.5	N/A	N/A	N/A	N/A

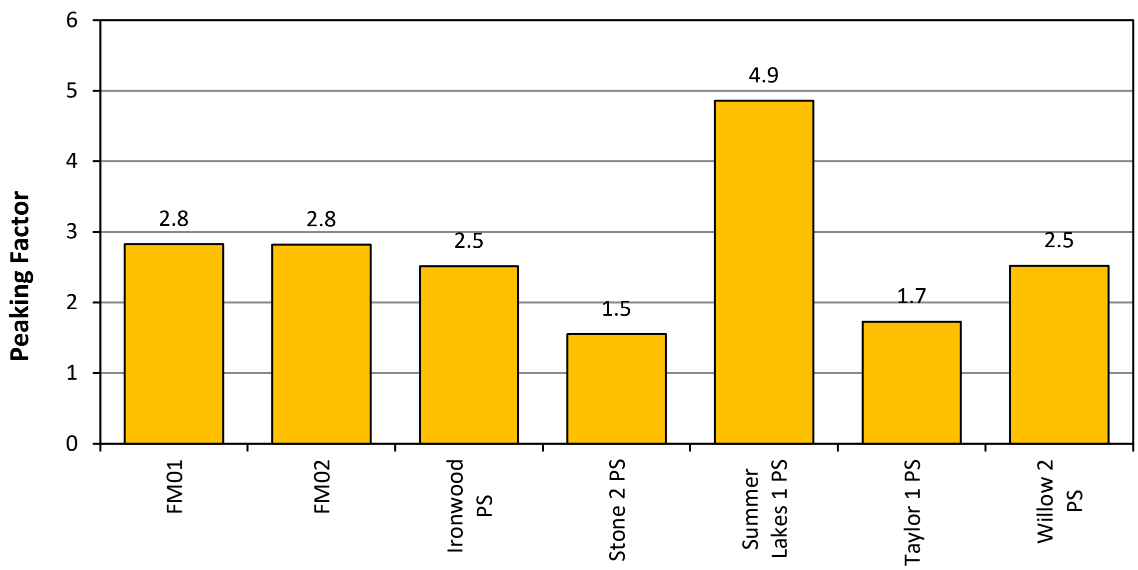


Figure 3-8. Peaking Factors

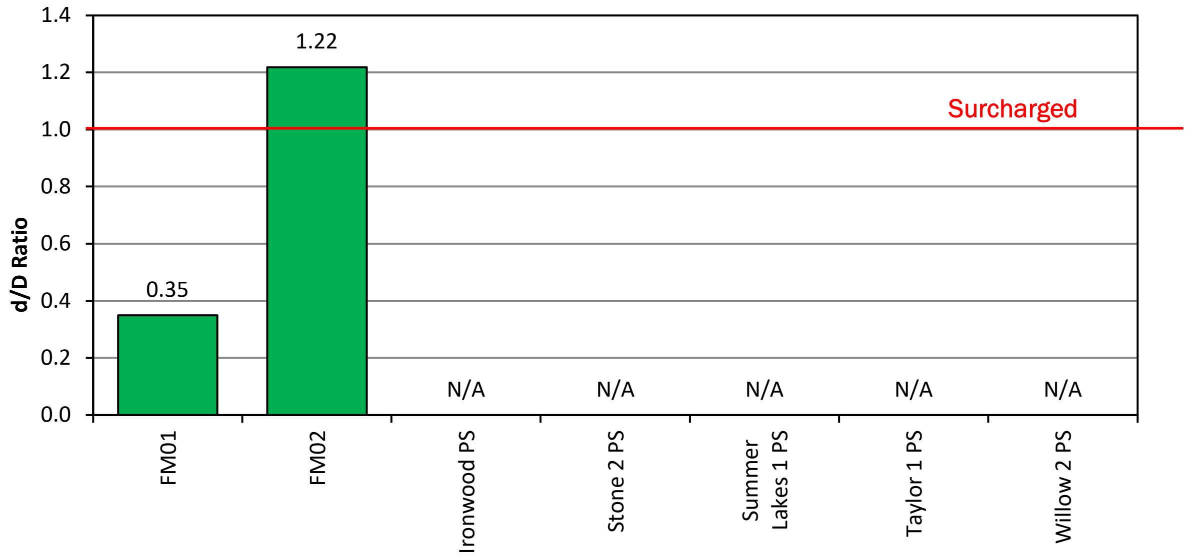


Figure 3-9. Capacity Summary: Max d/D Ratios

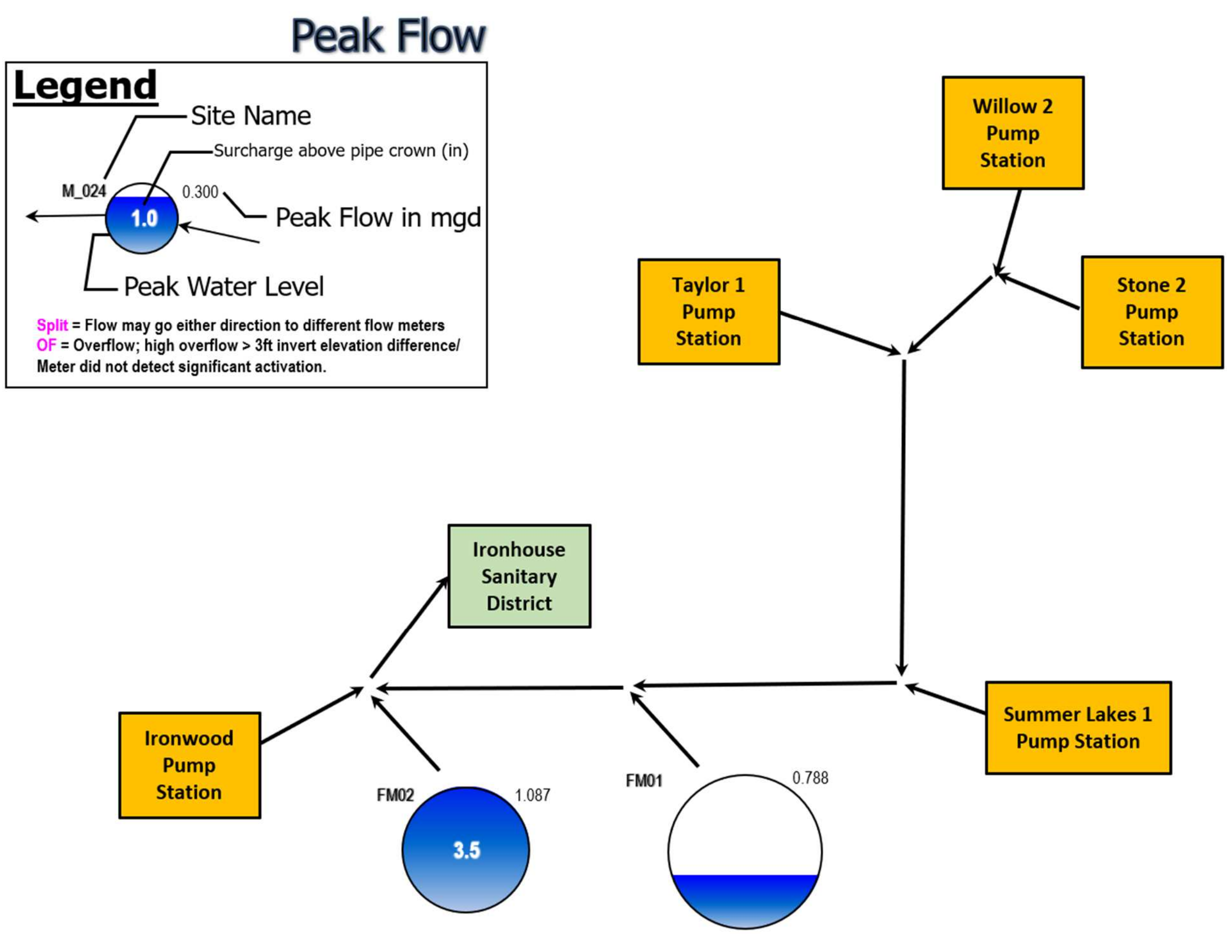


Figure 3-10. Peak Measured Flow (Flow Schematic)

3.3 Inflow and Infiltration: Results

3.3.1 Preface

I/I analyses are presented on a basin-by-basin basis. Items relevant to the analysis in this study are noted below and referenced in Figure 3-61:

- I/I Isolation: The I/I flow rate is the real-time flow less the estimated average dry weather flow rate (shown below as the RED line).
- Inflow: Inflow is usually recognized graphically by large-magnitude, short-duration spikes immediately following a rain event. The peak inflow rate is the highest spike in the isolated I/I hydrograph immediately following the evaluated rainfall event.
- RDI: RDI is typically taken as the average I/I flow rate measured approximately 24 hours after the rainfall event has concluded.
- Combined I/I: the totalized volume (in gallons) of inflow and RDI over a rainfall event (shown below as the shaded orange area).

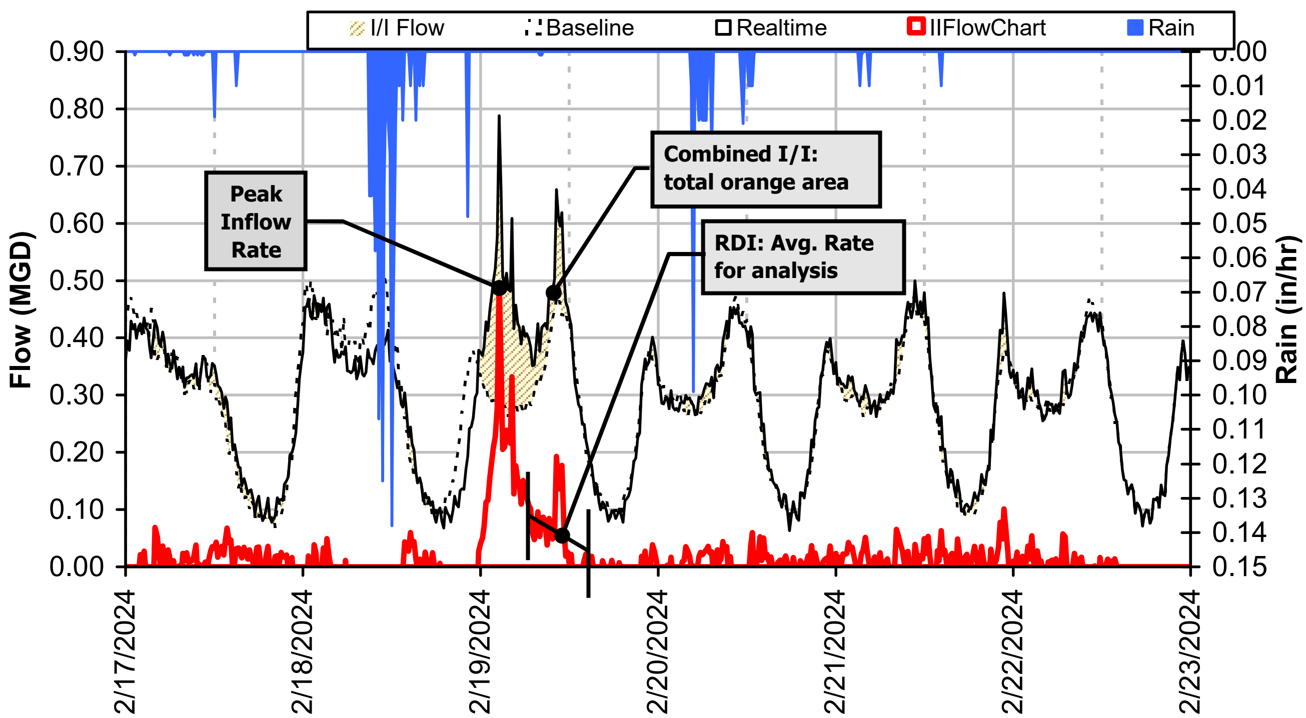


Figure 3-61. I/I Isolation, Site FM01 Event #1

3.3.2 Inflow Results Summary

Inflow is stormwater discharged into the sewer system through direct connections such as downspouts, area drains, cross-connections to catch basins, etc. These sources transport rainwater directly into the sewer system, and the corresponding flow rates are tied closely to the intensity of the storm. This component of I/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.

Table 3-6 and Figure 3-72 through Figure 3-4 summarize the peak measured inflow and inflow analysis results for the relevant flow monitoring basins. The "Top 2" basins for each category have been shaded in RED. The following inflow results are noted:

- Flow monitoring Sites FM01 and FM02 had the highest normalized peak I/I rates per the 3 inflow categories utilizing the method listed in Section 2.7.5.
- Willow 2 Pump Station ranked the highest for inflow per IDM and overall, with a rate of 5,488 gpd/IDM.
- Flow monitoring Site FM02 ranked the highest for inflow per Acre, with a rate of 660 gpd/Acre.
- Flow monitoring Site FM02 ranked the highest for inflow per ADWF, with a ratio of 1.9.

Table 3-6. Results and Rankings of Inflow Analysis

Site ID	ISO ADWF (MGD)	ISO IDM	ISO Basin Acreage	Weighted Inflow Rate (MGD)	Inflow per-IDM Ranking	Inflow per-Acre Ranking	Inflow per-ADWF Ranking	Final Inflow Ranking
FM01	0.279	204	883	0.50	4	2	2	2
FM02	0.385	221	1,086	0.72	3	1	1	1
Ironwood PS	0.113	205	1,003	0.12	7	6	3	6
Stone 2 PS	0.106	13	196	0.05	2	4	6	4
Summer Lakes 1 PS	0.072	48	257	0.05	5	5	5	5
Taylor 1 PS	0.111	22	322	0.02	6	7	7	7
Willow 2 PS	0.101	17	288	0.09	1	3	4	3

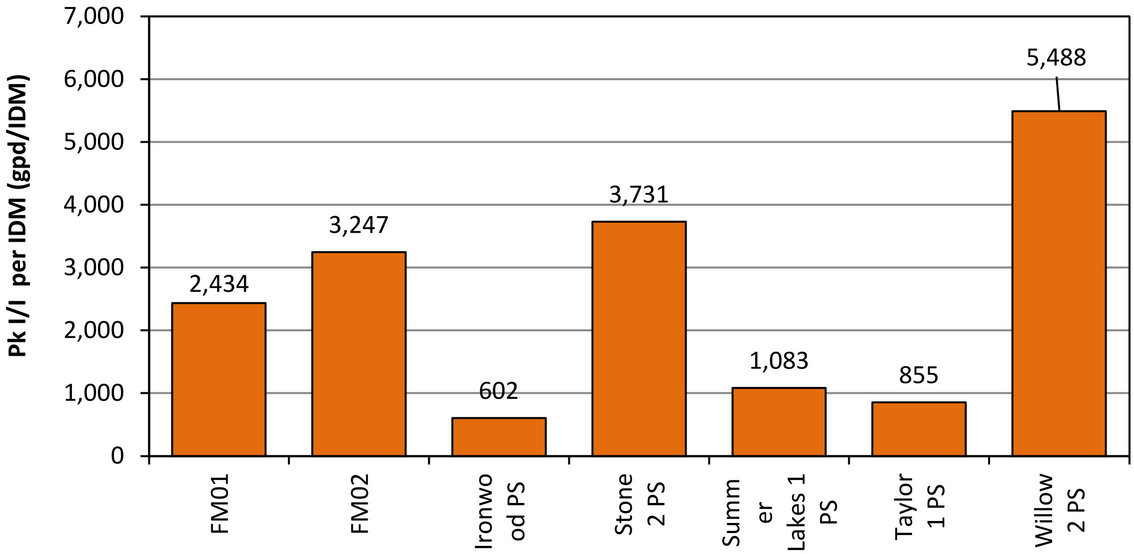


Figure 3-72. Averaged Peak I/I per IDM Analysis

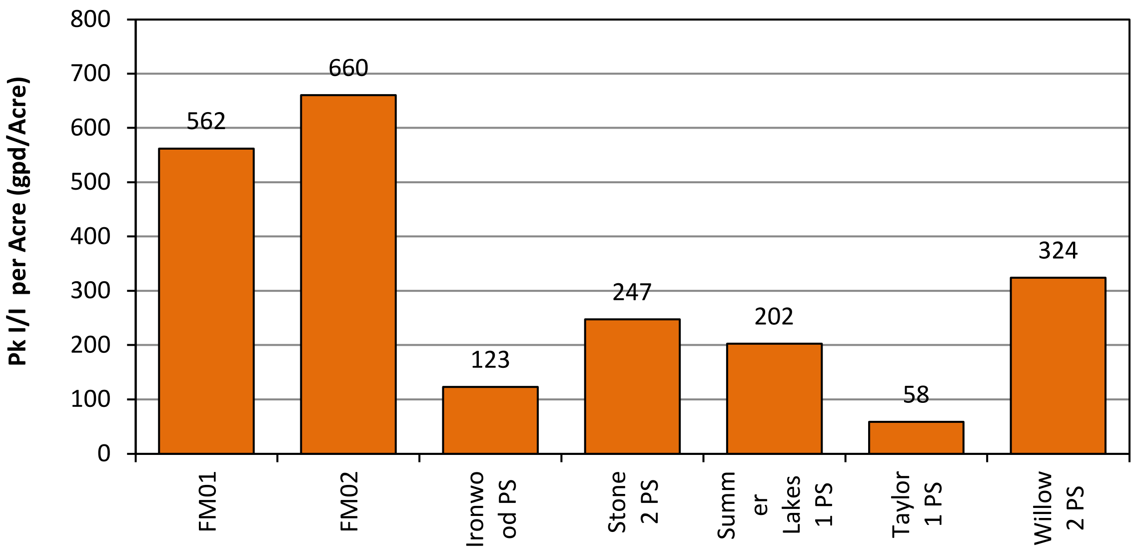


Figure 3-83. Averaged Peak I/I per Acre Analysis

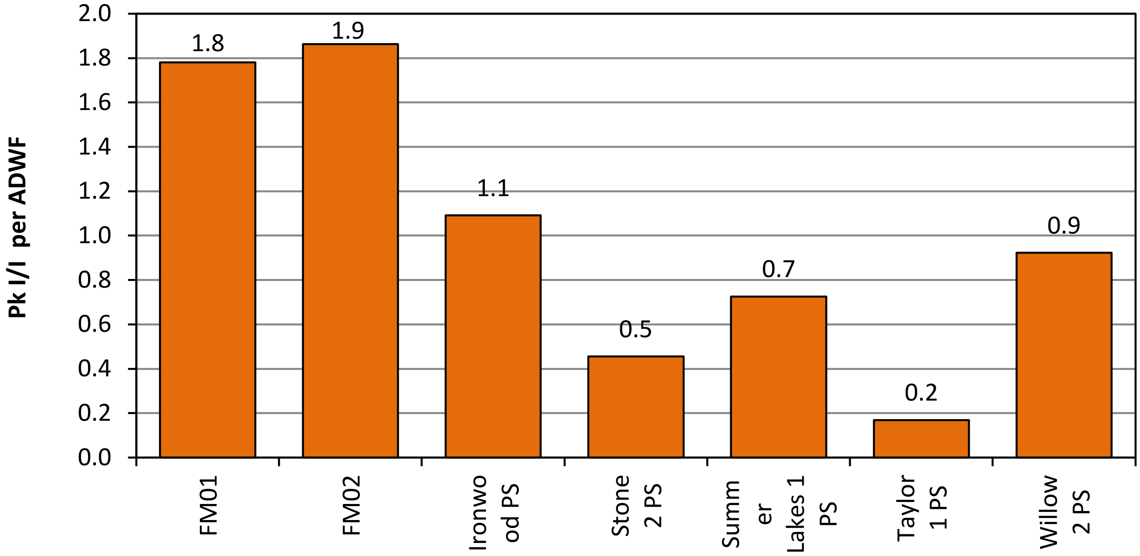


Figure 3-14. Averaged Peak I/I per ADWF Analysis

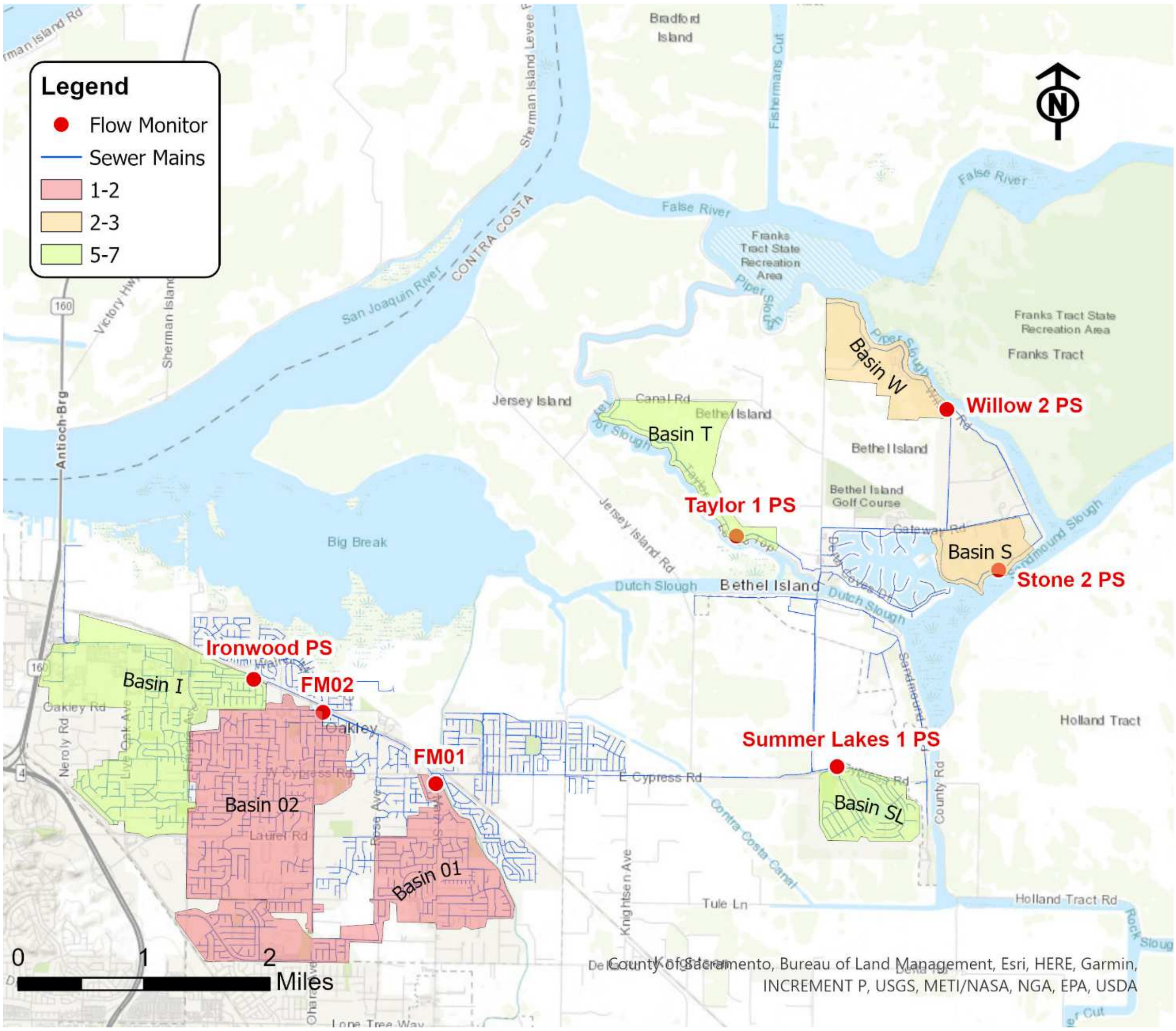


Figure 3-95. Temperature Map: Inflow Final Basin Rankings

3.3.3 Rainfall-Dependent Infiltration Results Summary

Infiltration is water entering the sanitary sewer system through defects in pipes, pipe joints, and manhole walls, including cracks, offset joints, root intrusion points, and broken pipes. Increased flows into the sanitary sewer system are usually tied to groundwater and soil saturation levels. Infiltration sources transport rainwater into the system indirectly; flow levels in the sanitary system increase gradually, are typically sustained for a period after rainfall has stopped, and then gradually decrease as soils become less saturated and groundwater levels recede to normal.

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).

RDI results were taken from February 19th for Event 1. Table 3-7 and Figure 3-10 through Figure 3-12 summarize the RDI analysis results for the relevant flow monitoring basins. The "Top 2" basins for each category have been shaded in RED. The following inflow results are noted:

- Flow monitoring Sites FM01 and FM02 ranked highest overall RDI per IDM, per Acre, and per ADWF
- Flow monitoring Site FM02 ranked the highest for RDI per IDM and overall, with a rate of 2,499 gpd/IDM.
- Flow monitoring Site FM02 ranked the highest for RDI per Acre, with a rate of 508 gpd/Acre.
- Flow monitoring Site FM02 ranked the highest for RDI per ADWF, with a ratio of 1.4.

Table 3-7. Results and Rankings of RDI Analysis

Site ID	ISO ADWF (MGD)	ISO IDM	ISO Basin Acreage	RDI Rate (MGD)	RDI per-IDM Ranking	RDI per-Acre Ranking	RDI per-ADWF Ranking	Final RDI Ranking
FM01	0.279	204	883	0.09	2	2	2	2
FM02	0.385	221	1,086	0.55	1	1	1	1
Ironwood PS	0.113	205	1,003	0.03	3	3	3	3
Stone 2 PS	0.106	13	196	No data[1]	No data[1]	No data[1]	No data[1]	No data[1]
Summer Lakes 1 PS	0.072	48	257	No data[1]	No data[1]	No data[1]	No data[1]	No data[1]
Taylor 1 PS	0.111	22	322	No data[1]	No data[1]	No data[1]	No data[1]	No data[1]
Willow 2 PS	0.101	17	288	No data[1]	No data[1]	No data[1]	No data[1]	No data[1]

[1]No infiltration data is available for February 19th during the rain event.

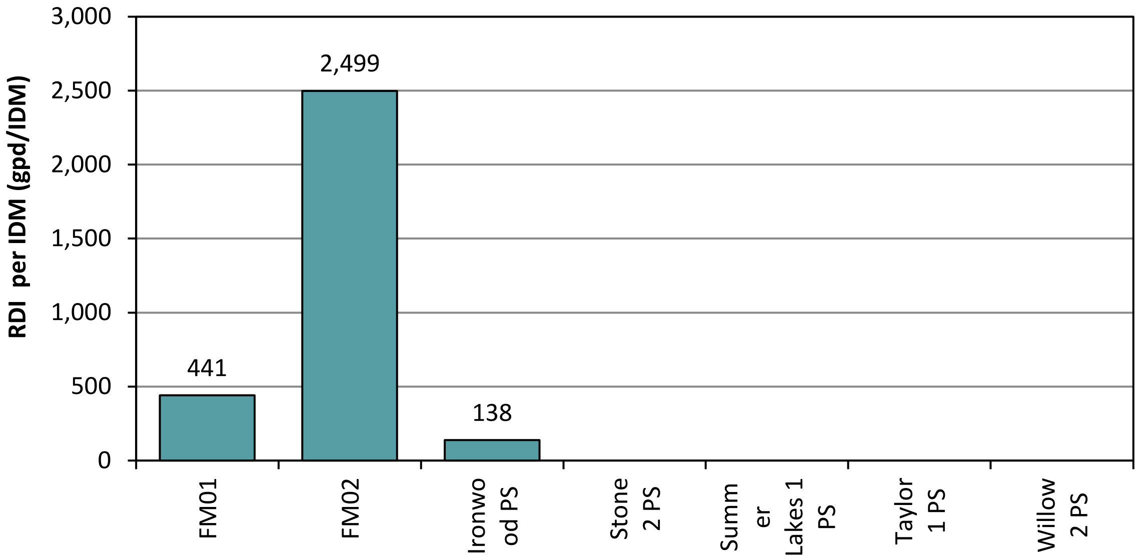


Figure 3-10. RDI per IDM

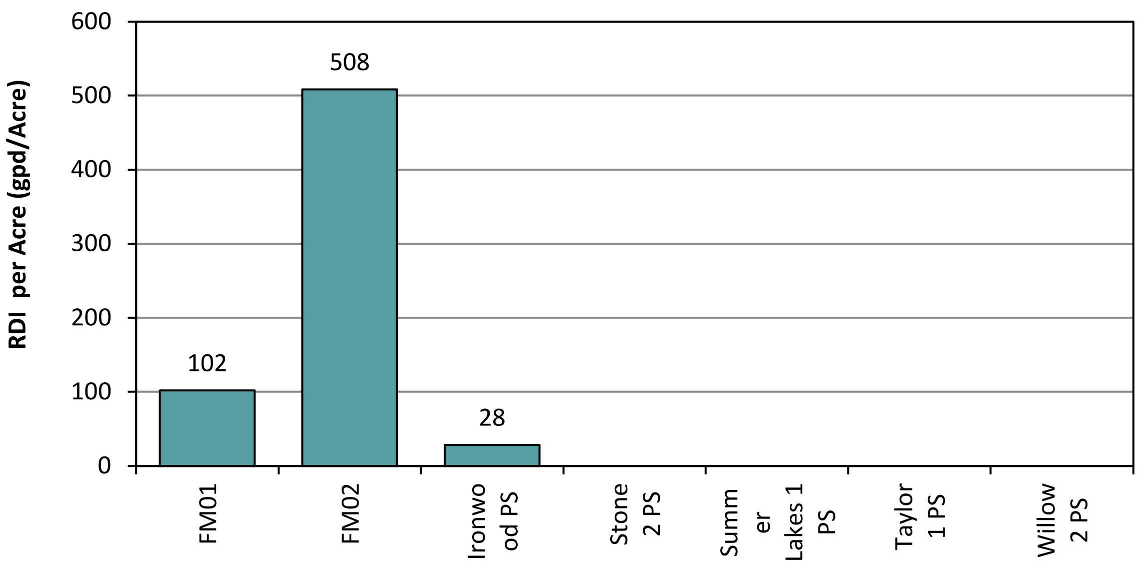


Figure 3-11. RDI per Acre

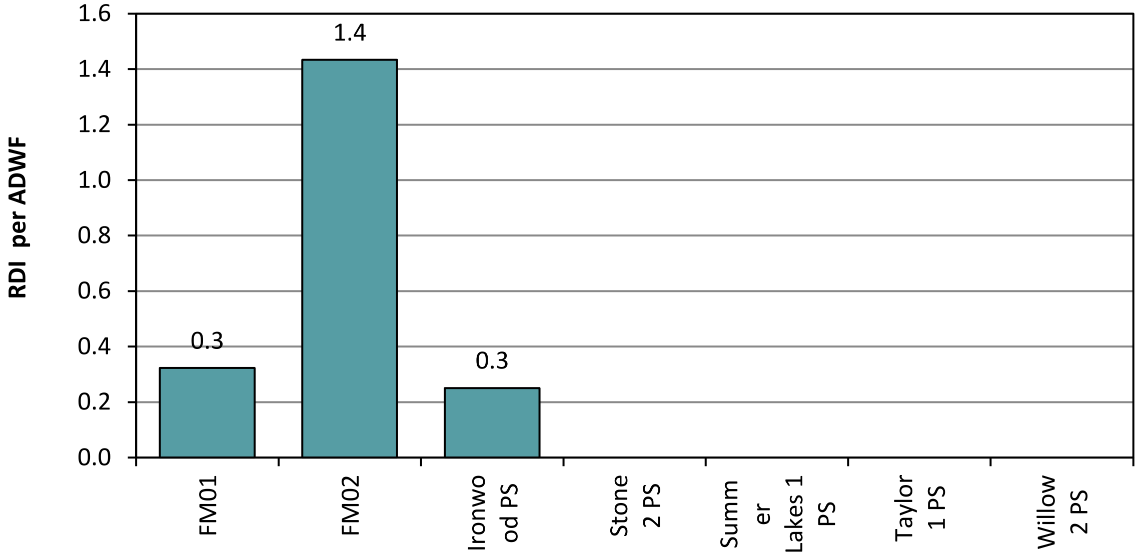


Figure 3-12. RDI per ADWF

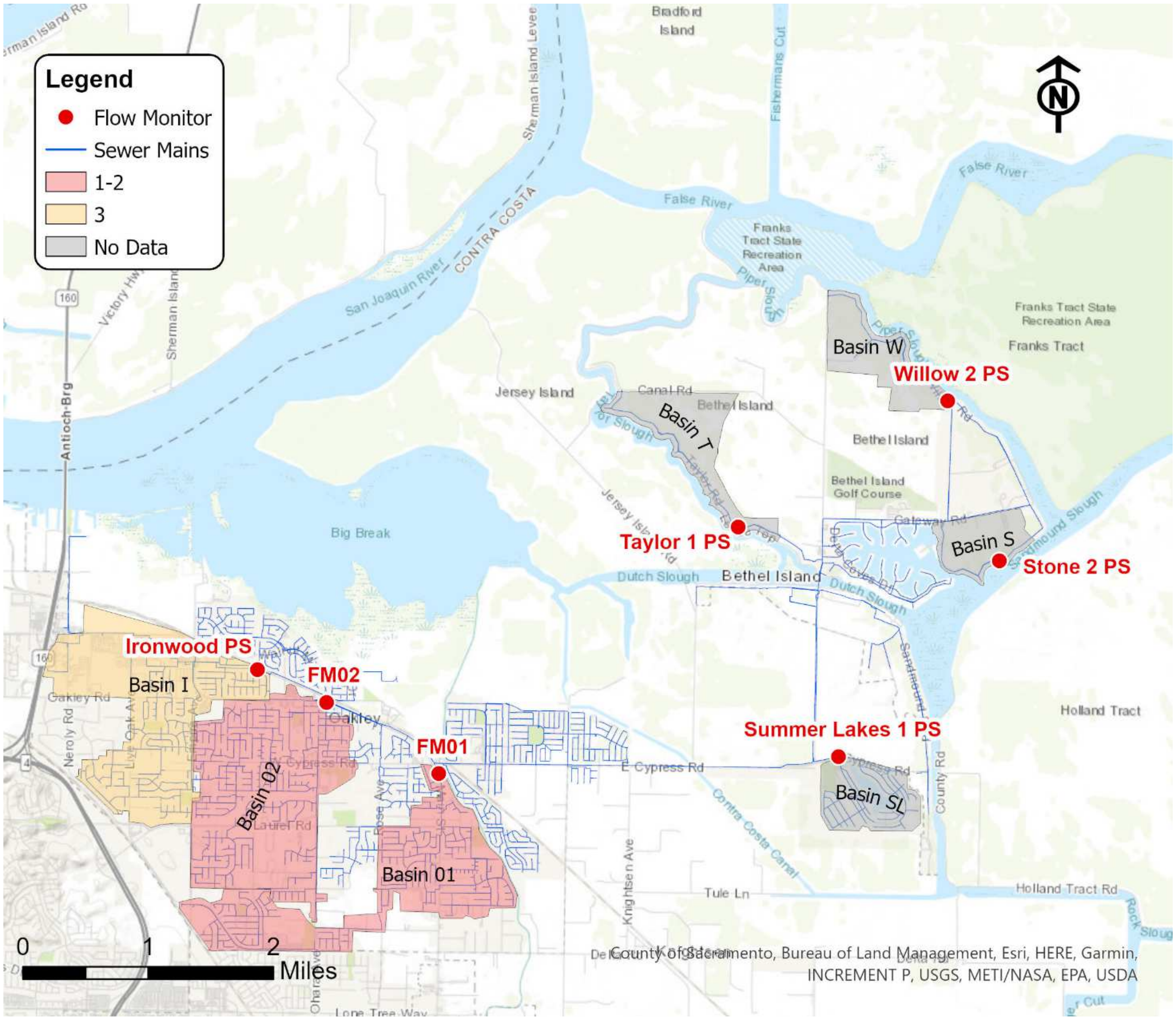


Figure 3-13. Temperature Map: RDI Final Basin Rankings

3.3.4 Combined I/I Results

Combined I/I analysis considers the totalized volume (in gallons) of inflow and rainfall-dependent infiltration over a storm event.

Table 3-8 and Figure 3-14 through Figure 3-16 summarize the combined I/I analysis results for the relevant flow monitoring basins. The "Top 2" basins for each category have been shaded in RED. The following inflow results are noted:

- Flow monitoring Sites FM01 and FM02 ranked highest overall combined I/I per IDM, per Acre, and per ADWF.
- Flow monitoring Site FM02 ranked the highest for combined I/I per IDM and overall, with a rate of 1,251 gallons per IDM.
- Flow monitoring Site FM02 ranked the highest in R-value, with a value of 0.9%.
- Flow monitoring Site FM02 ranked the highest for combined I/I per ADWF, with a ratio of 0.72.

Table 3-8. Combined I/I Analysis Summary

Site ID	ISO ADWF (MGD)	ISO IDM	ISO Basin Acre	Avg Event I/I (gallons)	Combined I/I per IDM per inch-rain Ranking	Combined I/I per Acre per inch-rain (R-Value) Ranking	Combined I/I per ADWF per inch-rain Ranking	Final Combined I/I Ranking
FM01	0.279	204	883	60,507	3	2	2	2
FM02	0.385	221	1,086	353,498	1	1	1	1
Ironwood PS	0.113	205	1,003	11,107	5	5	4	4
Stone 2 PS	0.106	13	196	3,199	4	4	6	5
Summer Lakes 1 PS	0.072	48	257	2,564	6	6	5	6
Taylor 1 PS	0.111	22	322	Unusable data	Unusable data	Unusable data	Unusable data	Unusable data
Willow 2 PS	0.101	17	288	14,598	2	3	3	3

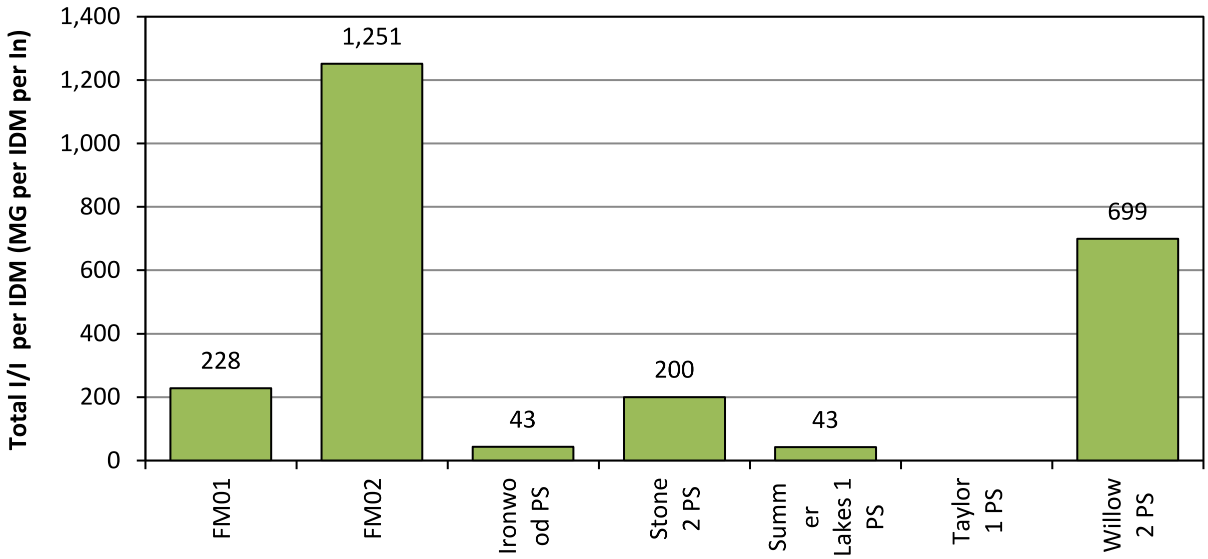


Figure 3-14: Combined I/I per IDM

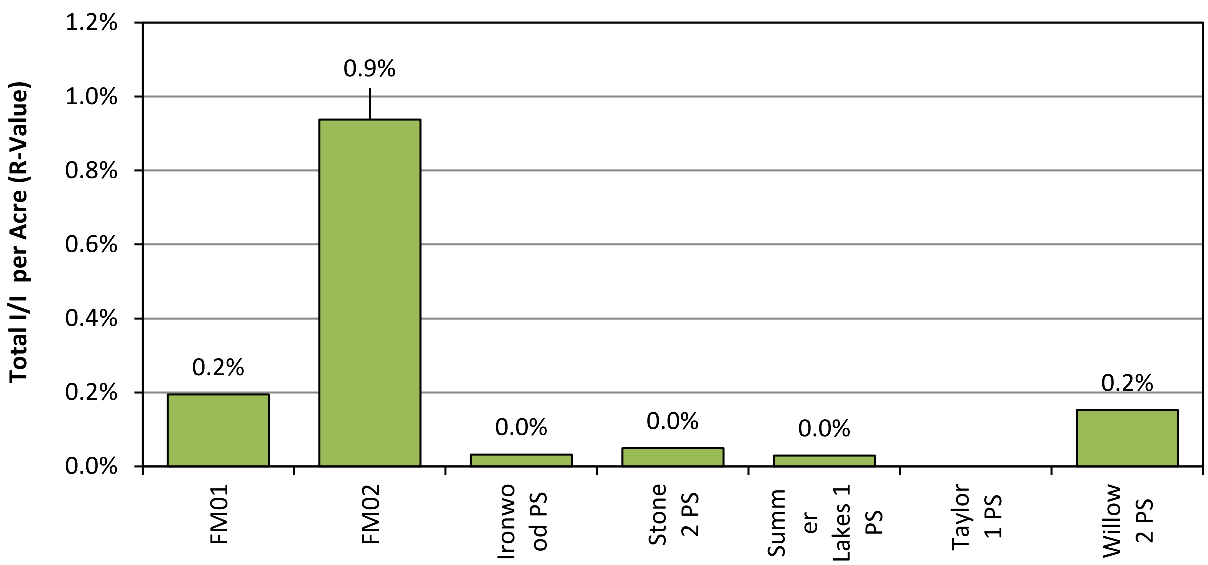


Figure 3-15: Combined I/I per Acre

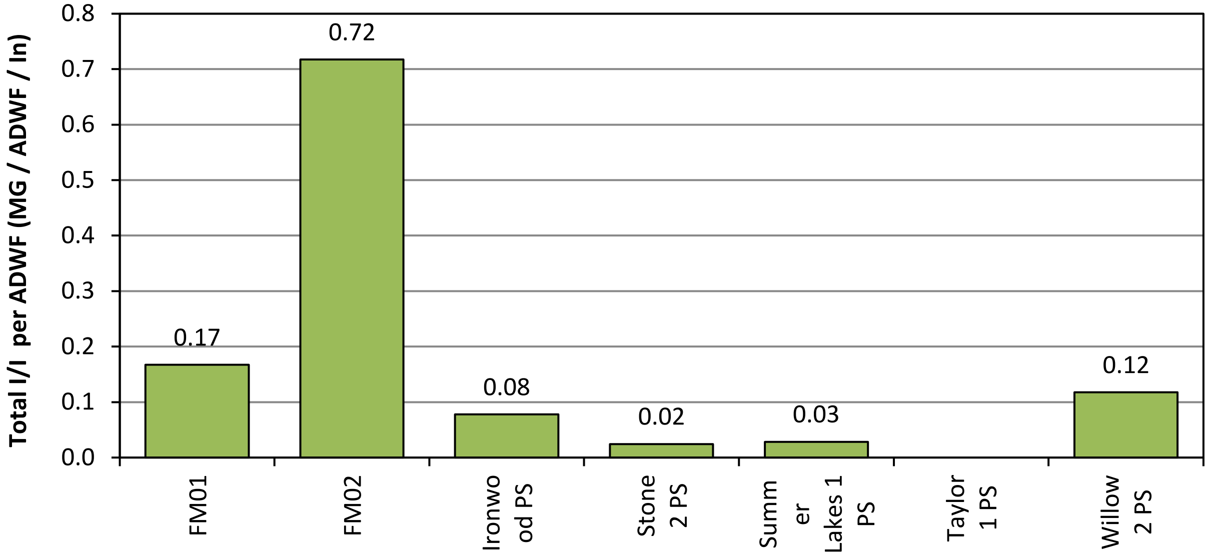


Figure 3-16: Combined I/I per ADWF

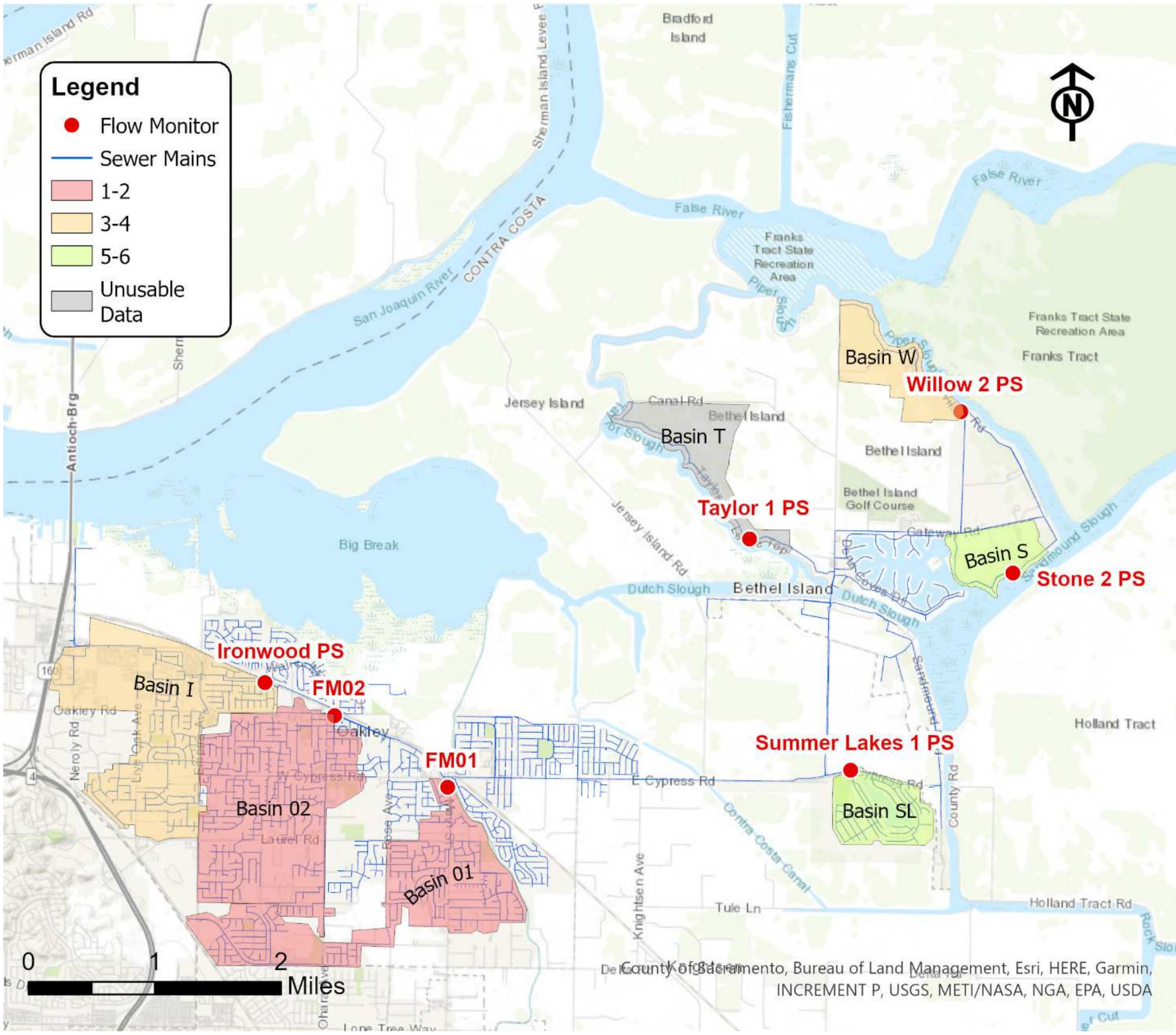


Figure 3-17. Temperature Map: Combined I/I Final Basin Rankings

3.3.5 Groundwater Infiltration Results Summary

Dry weather flow can be expected to have a predictable diurnal flow pattern. While each site is unique, experience has shown that, given a reasonable volume of flow and typical loading conditions, the daily flows fall into a predictable range when compared to the daily average flow. If a site has a large percentage of groundwater infiltration occurring during the periods of dry weather flow measurement, the amplitudes of the peak and low flows will be dampened[8]. Figure 3-18 shows a sample of two flow monitoring sites, both with nearly the same average daily flow but with considerably different peak and low flows. In this sample case, Site B1 may have a considerable volume of groundwater infiltration.

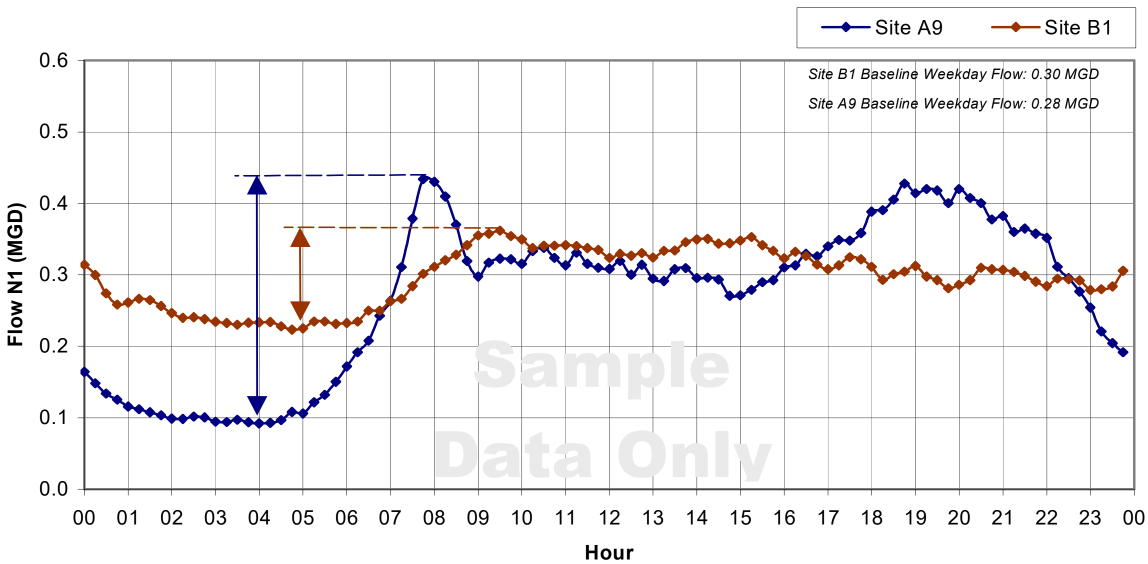


Figure 3-18. Groundwater Infiltration Sample Figure

It can be useful to compare the low-to-ADWF flow ratios for the flow monitoring sites. A site with abnormal ratios and no other reasons to suspect abnormal flow patterns (such as proximity to a pump station, treatment facilities, etc.) has a possibility of higher levels of groundwater infiltration in comparison to the rest of the collection system.

Figure 3-19 plots the low-to-ADWF flow ratios[9] against the ADWF flows for the relevant flow monitoring sites. The brown dashed line shows "typical" low-to-ADWF ratios per the Water Environment Federation (WEF).

WEF derived these ratios from residential sanitary sewer data. It is noted that the type of service in this project (airport and shipping) is not residential, and there exists the possibility of excessive early-morning flows due to abnormal working hours. This analysis is presented for reference only. The following GWI results are noted:

- All pump station sites indicated GWI rates higher-than typical standards, indicating possible high groundwater infiltration rates. However, because all are pump stations, it may be an effect of the water flowing to the pump stations.

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[8] In an extreme case, perhaps 0.2 mgd of ADWF flow and 2.0 mgd of groundwater infiltration, the peaks and lows would be barely recognizable; the ADWF flow would be nearly a straight line.

[9] The Minimum to Average flow ratio is calculated by taking the minimum flow and dividing by the ADWF value (using the Mon-Thu ADWF curve).

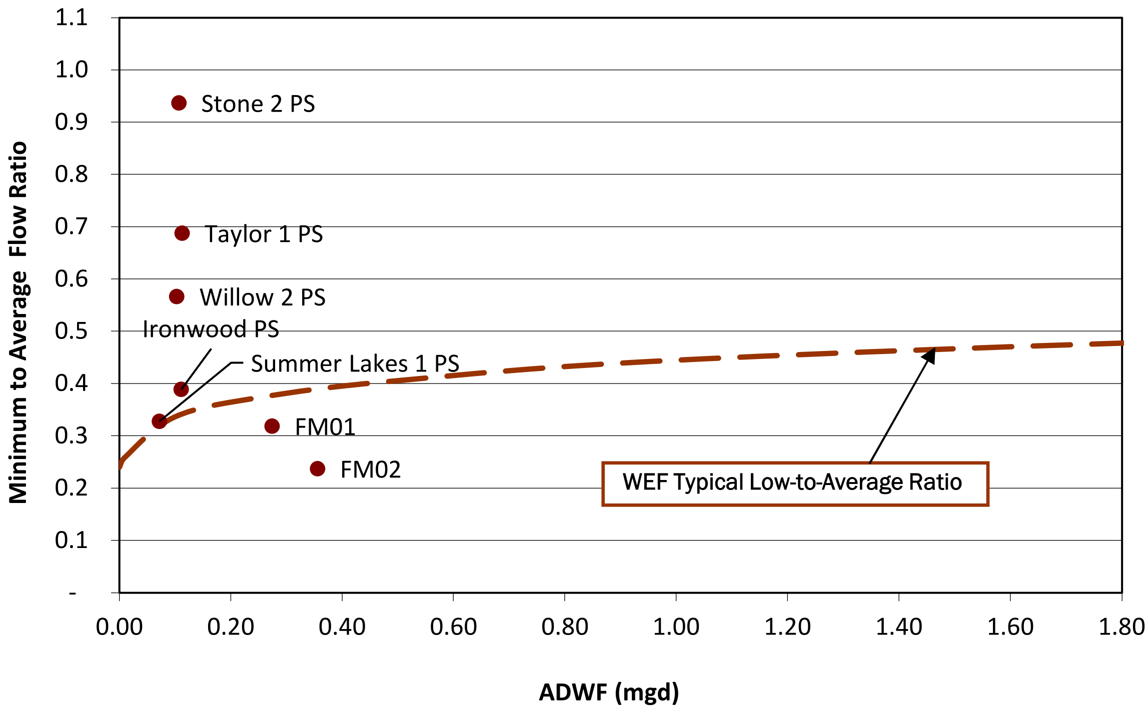


Figure 3-19. Minimum Flow Ratios vs ADWF[10]

[10] Due to attenuation, it should be expected that sites with larger flow volumes should not have quite the peak-to-average and low-to-average flow ratios as sites with lesser flow volumes. This is why the WEF typical trend line's slope is closer to 1.0 as the ADWF increases, as shown in the figure.

4 Recommendations

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

1. **Master Plan and Model Implementation:** This study focuses on inflow and infiltration generation; the study results can be used to update the master plan and compare with previous model assumptions and flow monitoring results.
2. **Capacity Analysis:** One site (FM02) was surcharged during the monitoring period, approximately 3.5 inches above the pipe crown. The following possible capacity concerns are noted:
    a. **Dry weather:** No site showed an average flow level greater than 50% full pipe conditions (greater than 0.5 d/D).
        i. Site FM02 appears to have experienced a surcharge immediately after the rain event. Levels returned to normal afterward.
        ii. No site had a peaking factor greater than 7.
    b. **Wet Weather:** With the exception of Site FM02, wet weather peaking factors ranged from 1.5 to 4.9. This is typically considered to be within acceptable limits, and no further action is recommended unless the City wishes to pursue I/I identification. It should be noted that while these limits may be within normally acceptable limits, higher-intensity storms than those that occurred during the monitoring period could cause higher wet weather responses.
3. **Determine I/I Reduction Program:** If the City should decide to pursue some I/I reduction in the future, consider the following I/I reduction programs:
    a. If peak flows, sanitary sewer overflows, and pipeline capacity issues are of greater concern; then priority can be given to investigating and reducing sources of inflow within the basins with the greatest inflow problems. The highest inflow occurred within the basins for Sites FM02 and Willow 2 Pump Station.
    b. Suppose total infiltration and general pipeline deterioration are of greater concern. In that case, the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems. The highest RDI occurred in the basins for Sites FM01 and FM02. Additionally, all basins for the pump stations showed possible evidence of elevated GWI.
4. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
    a. Smoke testing.
    b. Mini-basin flow monitoring.
    c. Night-time reconnaissance work is done to (1) investigate and determine direct point sources of inflow and (2) determine the areas and/or pipe reaches responsible for high levels of infiltration contribution.
    d. CCTV inspection.
5. **I/I Reduction Cost Effective Analysis:** If the City is considering pursuing I/I reduction, a study can be conducted 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
Flow Monitoring Sites: Data, Graphs, Information

Monitoring Site: FM01

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: In the grass at 4532 Main St, Oakley

Data Summary Report

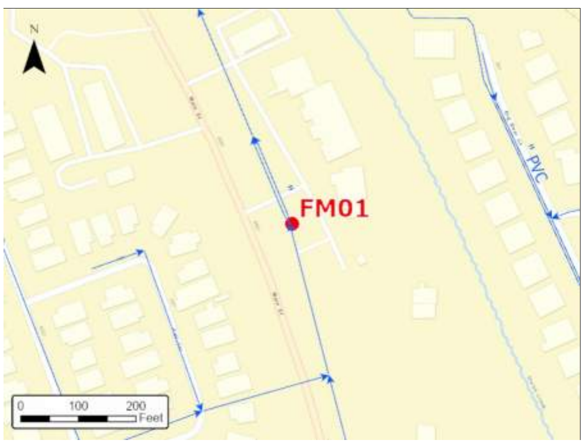
Vicinity Map: FM01

FM01

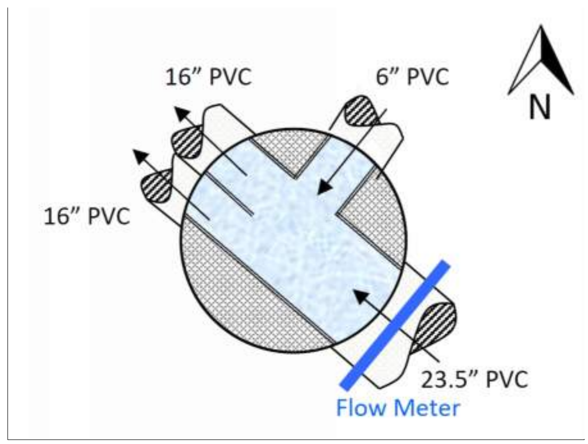
Site Information — MH ID: SEC002

Location:	In the grass at 4532 Main St, Oakley
Coordinates:	121.6963° W, 37.9895° N
Rim Elevation:	24 feet
Expected Pipe Diameter:	24 inches
Measured Pipe Diameter:	23.5 inches
ADWF:	0.279 mgd
Peak Measured Flow:	0.79 mgd
Sediment:	None

Satellite Map



Sanitary Map



Flow Sketch


Street View


Plan View

FM01

Additional Site Photos

Northwest Effluent Pipe 1

Northwest Effluent Pipe 2

FM01

Additional Site Photos

Northeast Influent Pipe

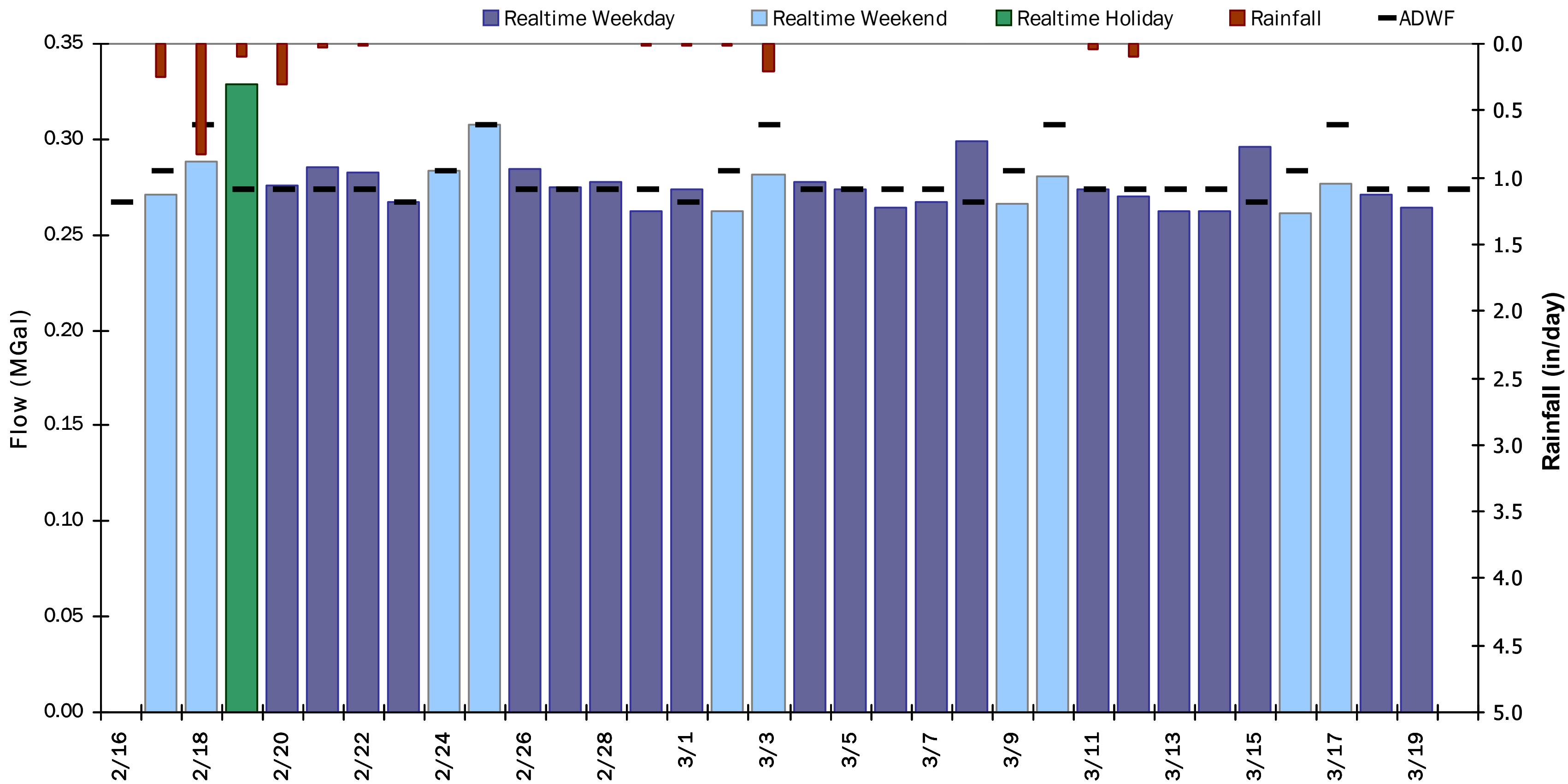
Monitored Southeast Influent Pipe

FM01

Period Flow Summary: Daily Flow Totals

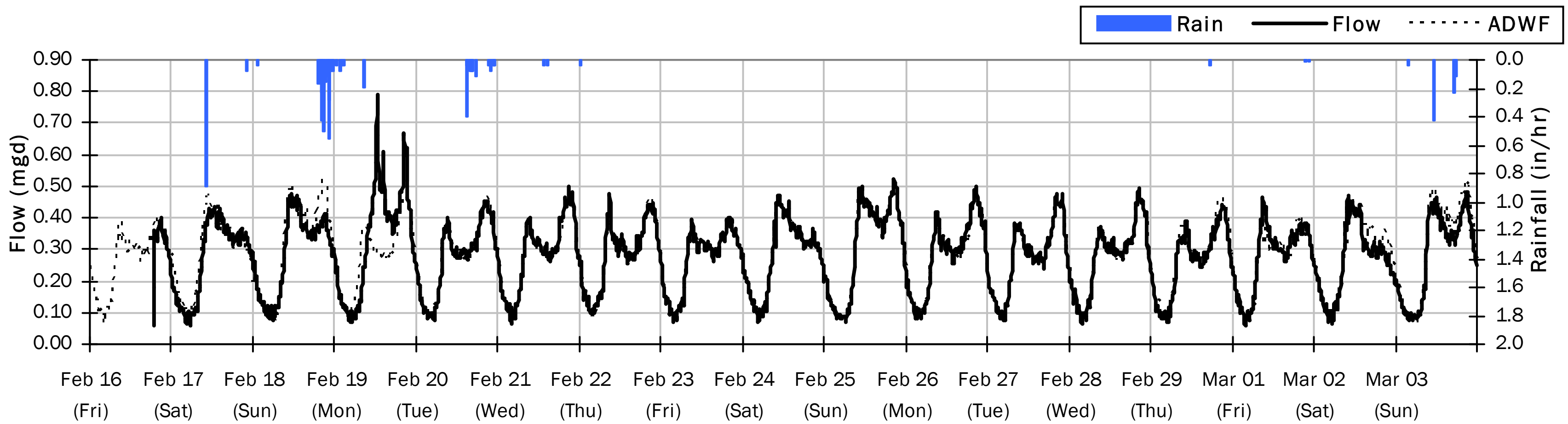
Avg Daily Flow: 0.275 MGal   Peak Daily Flow: 0.329 MGal   Min Daily Flow: 0.152 MGal

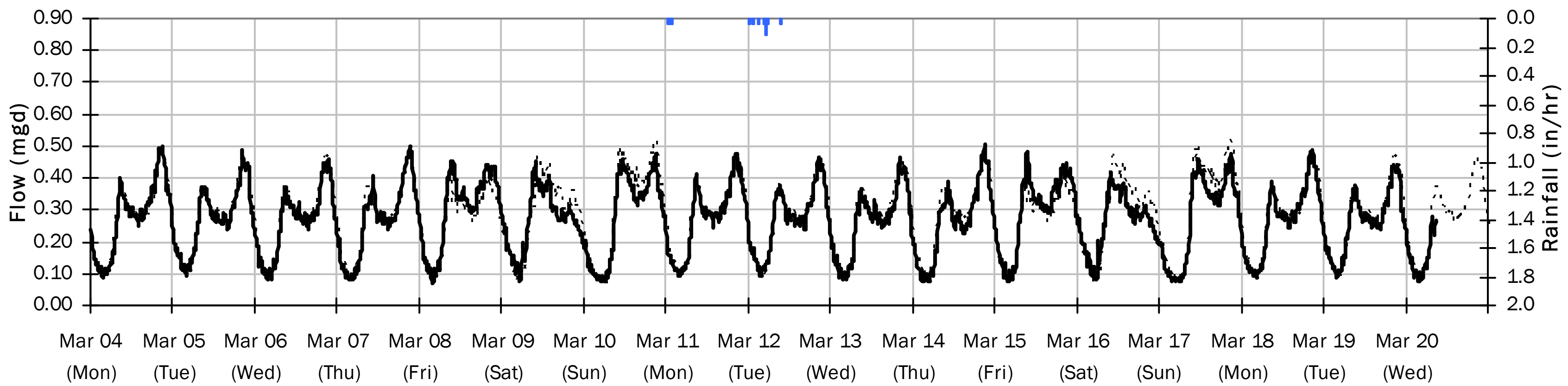
Total Rainfall: 1.91 inches



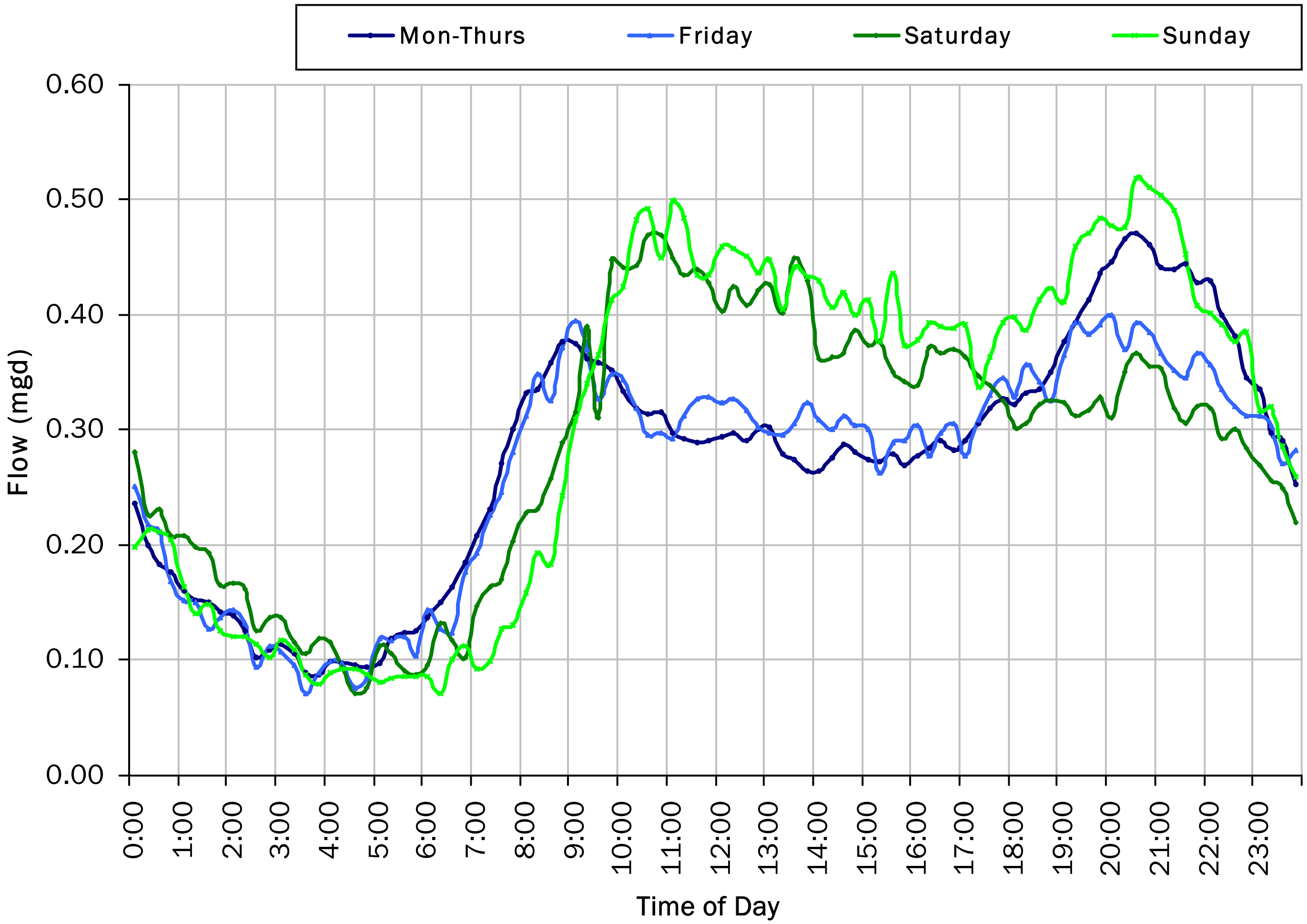
FM01
Flow Summary: 2/16/2024 to 3/20/2024

Period Rainfall: 1.91 inches Period Avg Flow: 0.276 mgd Period Peak Flow: 0.788 mgd Period Min Flow: 0.055 mgd

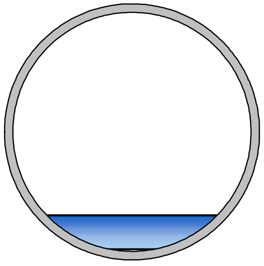




FM01
Average Dry Weather Flow Hydrographs

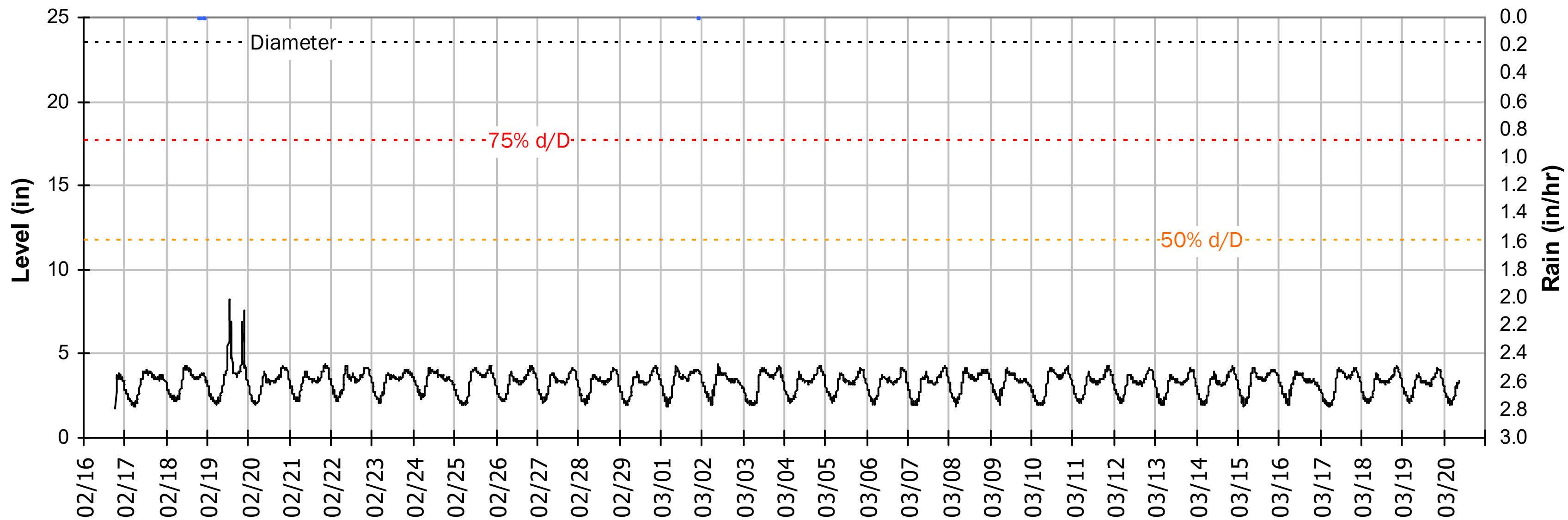


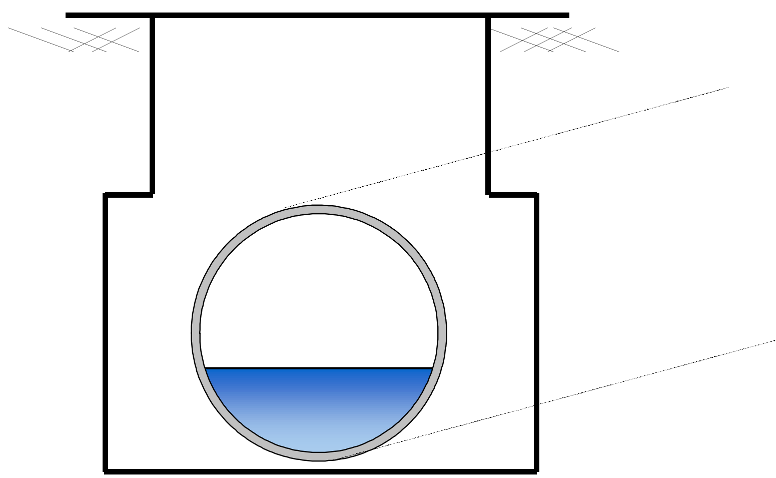
ADWF:
0.278 *mgd*



FM01
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



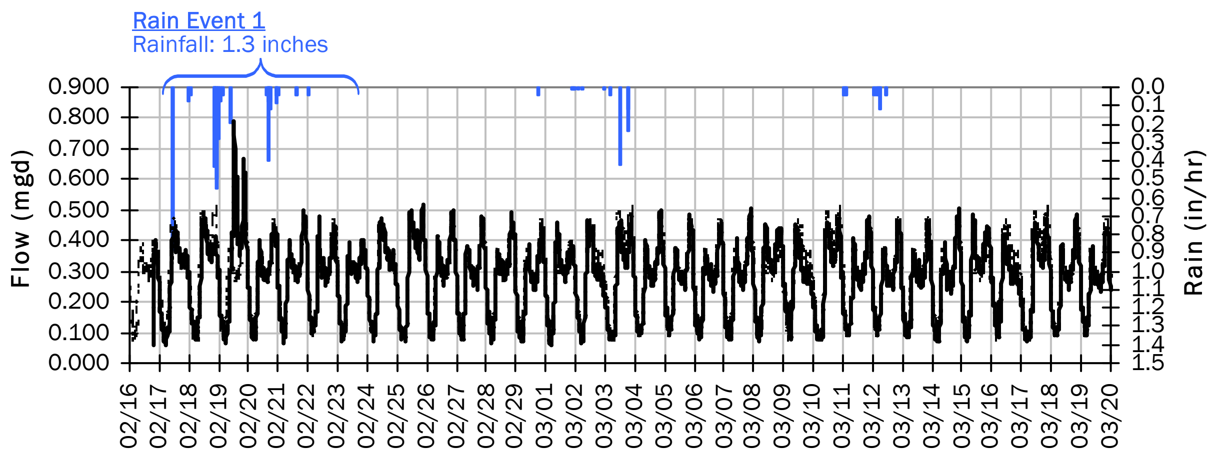


Pipe Diameter:	23.5	*inches*
Peak Measured Level:	8.20	*inches*
Peak d/D Ratio:	0.35	

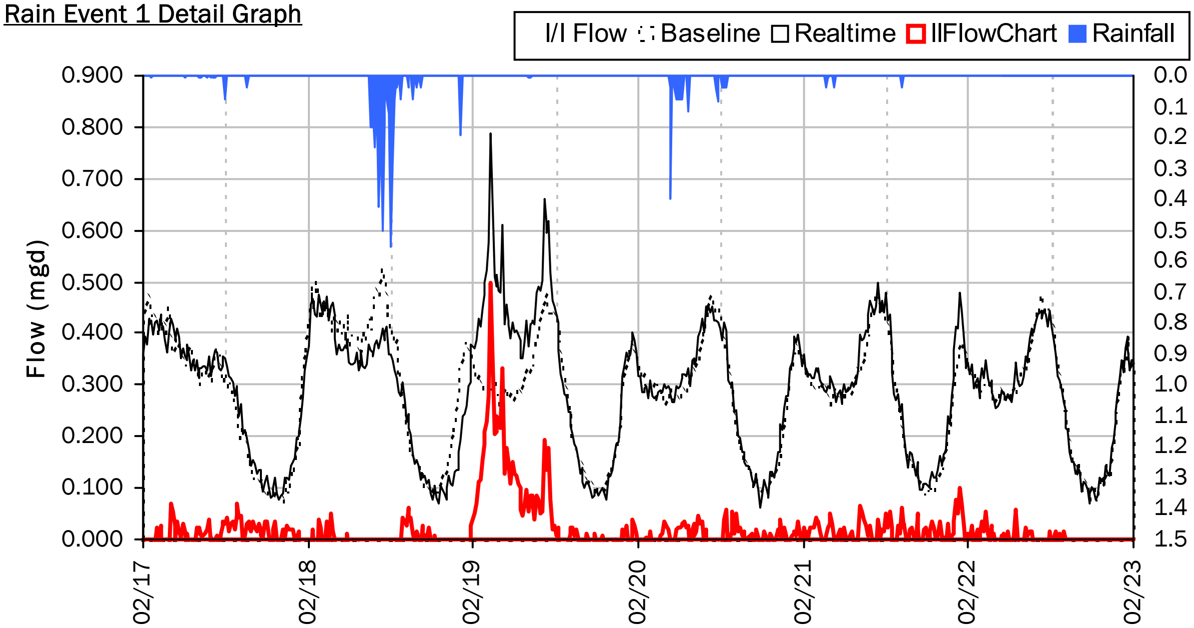
FM01

I/I Summary: Rain Event 1

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



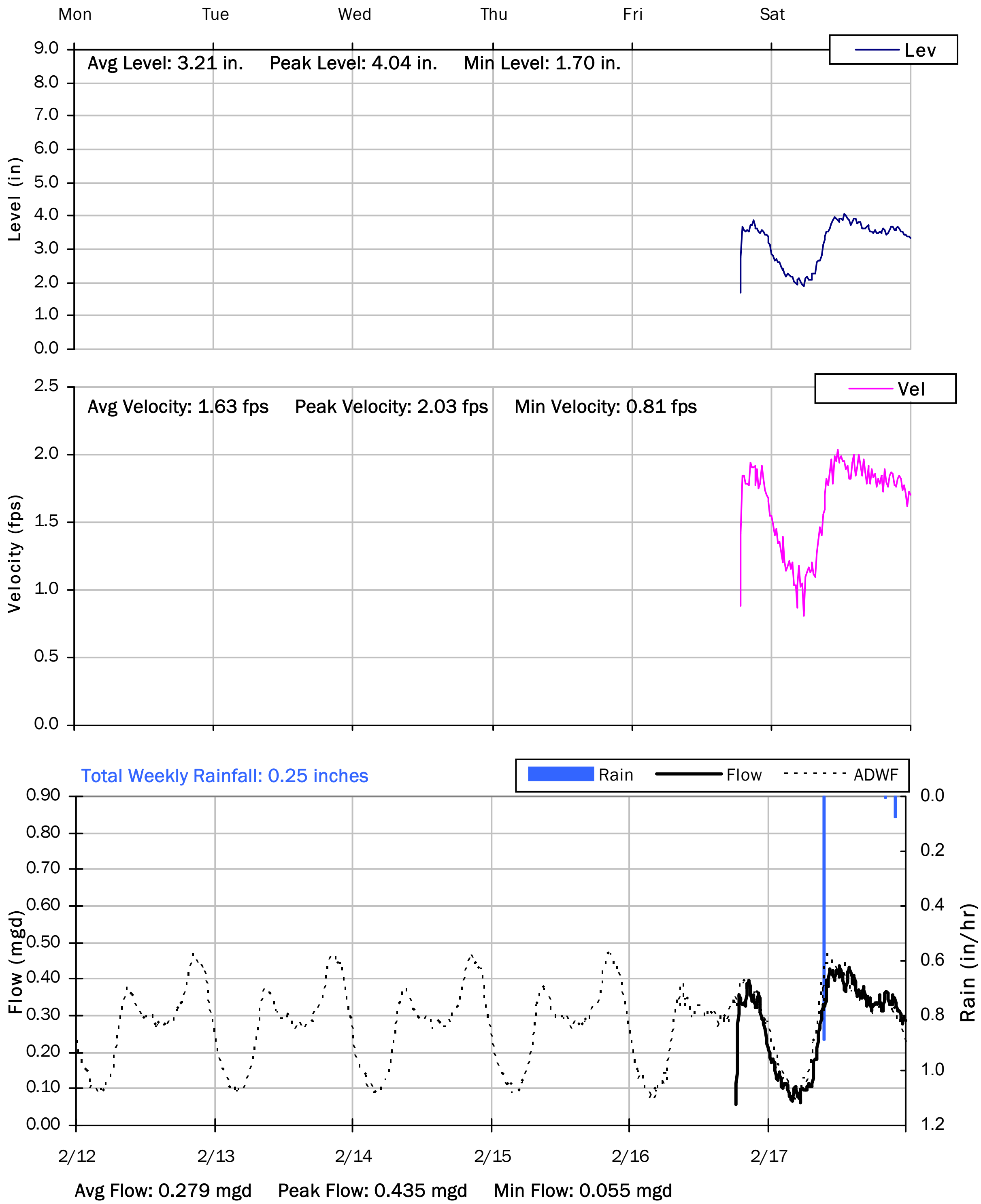
<u>Rain Event 1 Detail Graph</u>



Storm Event I/I Analysis (Rain = 1.30 inches)

<u>Capacity</u>		<u>Inflow / Infiltration</u>	
Peak Flow:	0.788 *mgd*	Peak I/I Rate:	0.497 *mgd*
PF:	2.83	Total I/I:	61,000 *gallons*
Peak Level:	8.20 *in*		
d/D Ratio:	0.35		

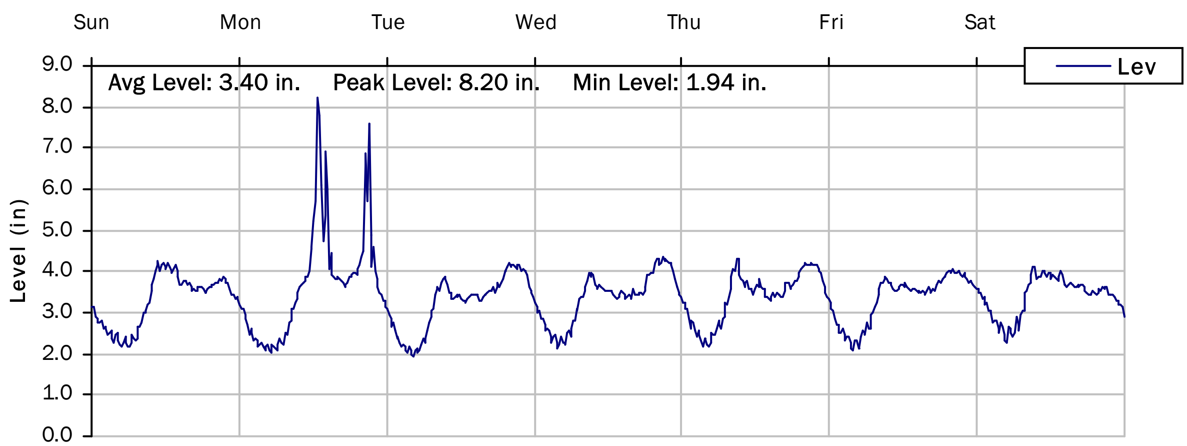
FM01
Weekly Level, Velocity and Flow Hydrographs
2/12/2024 to 2/18/2024

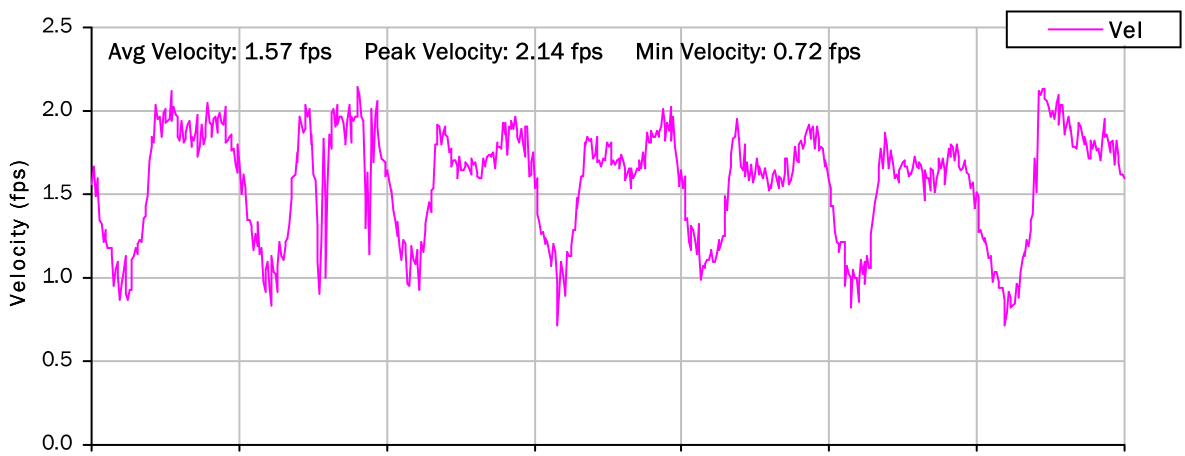


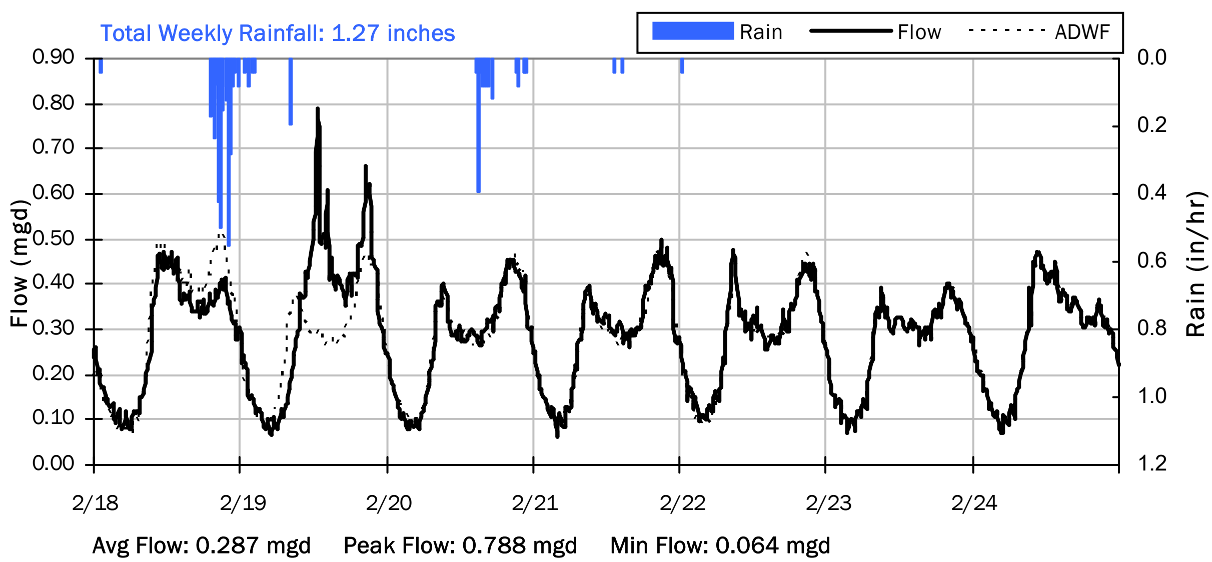
FM01

Weekly Level, Velocity and Flow Hydrographs

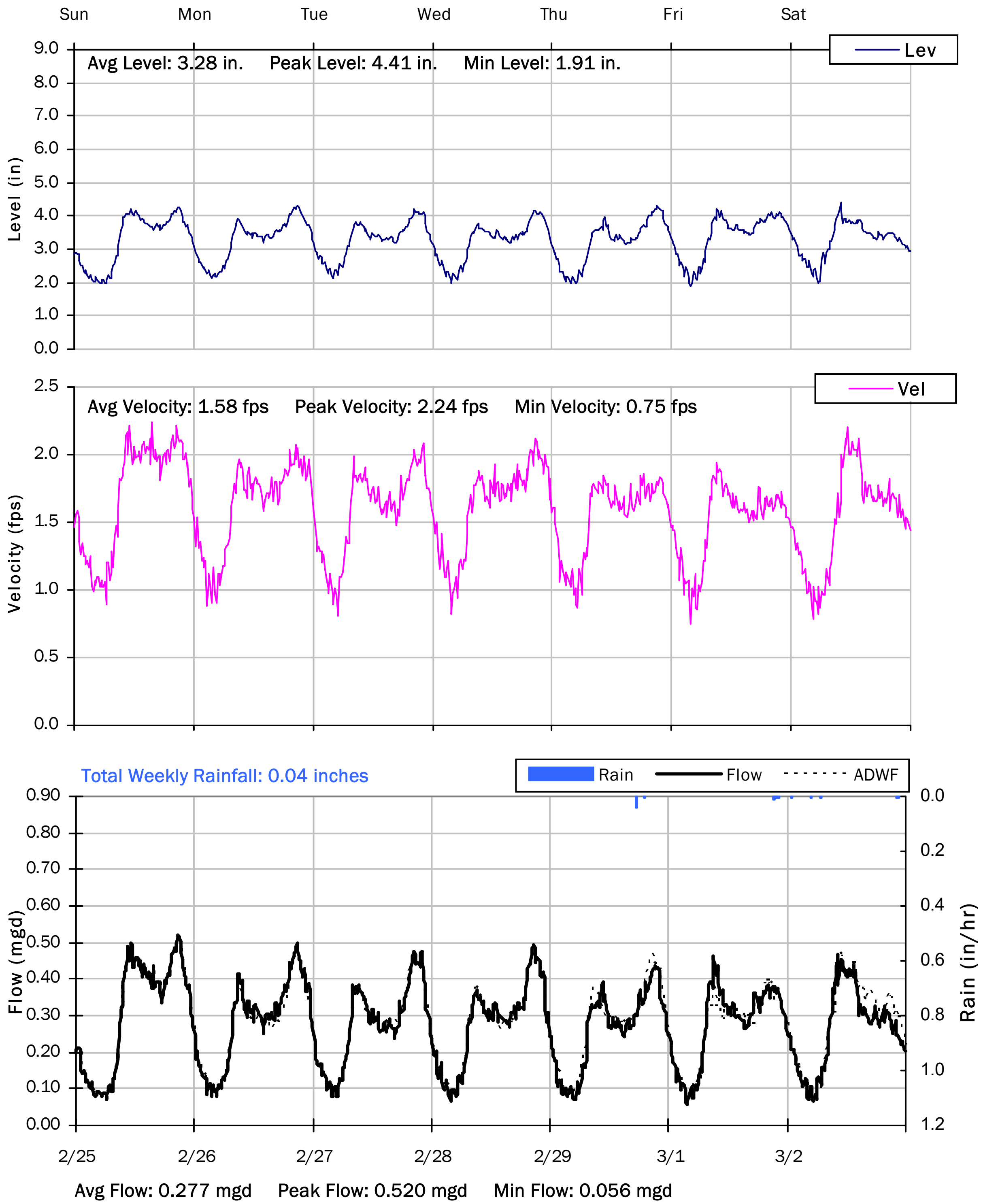
2/18/2024 to 2/25/2024







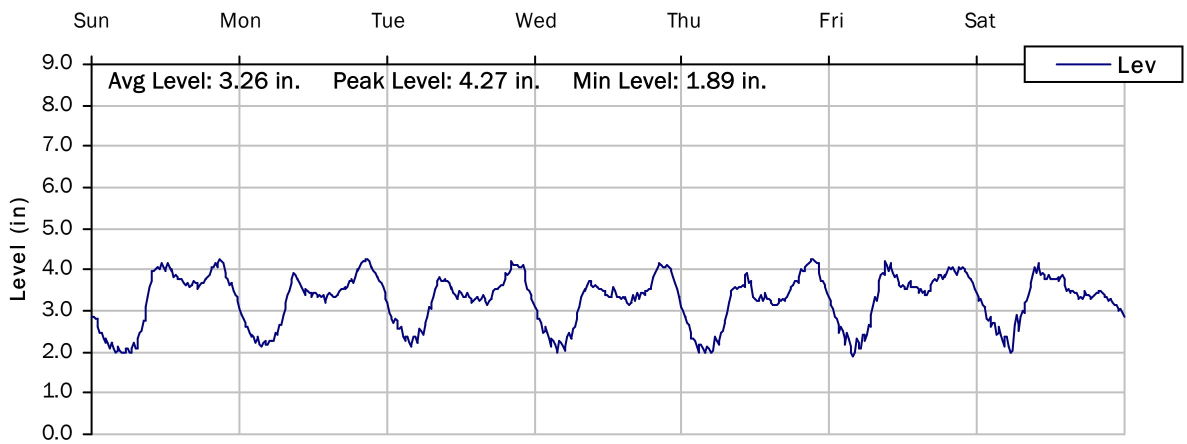
FM01
Weekly Level, Velocity and Flow Hydrographs
2/25/2024 to 3/3/2024

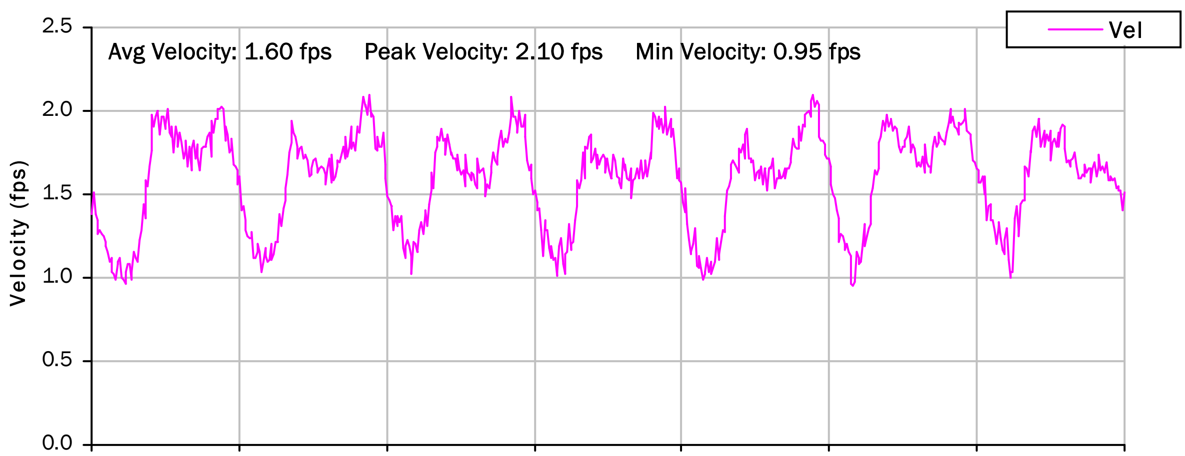


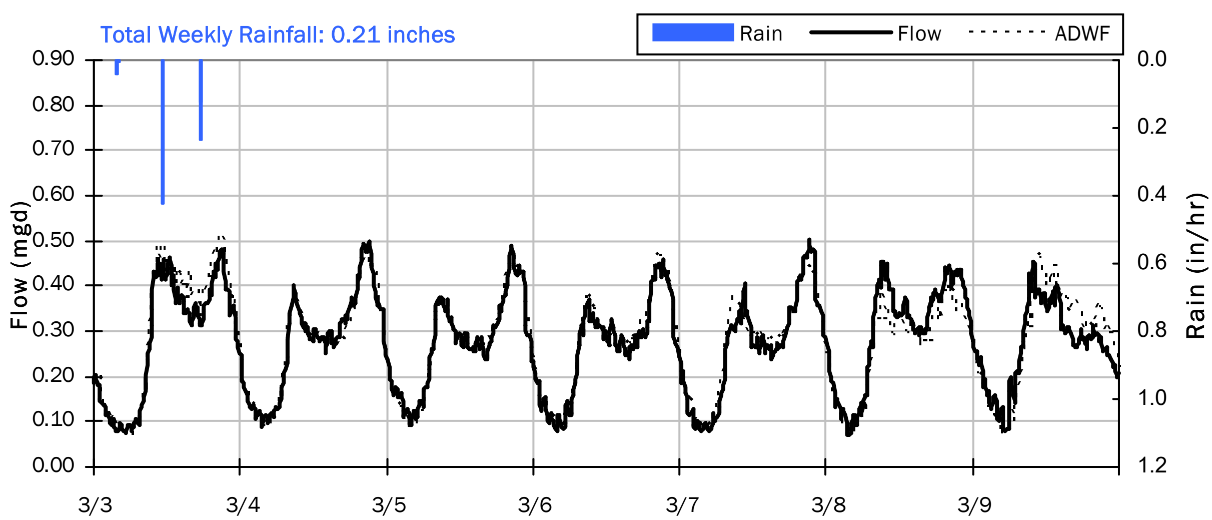
FM01

Weekly Level, Velocity and Flow Hydrographs

3/3/2024 to 3/10/2024

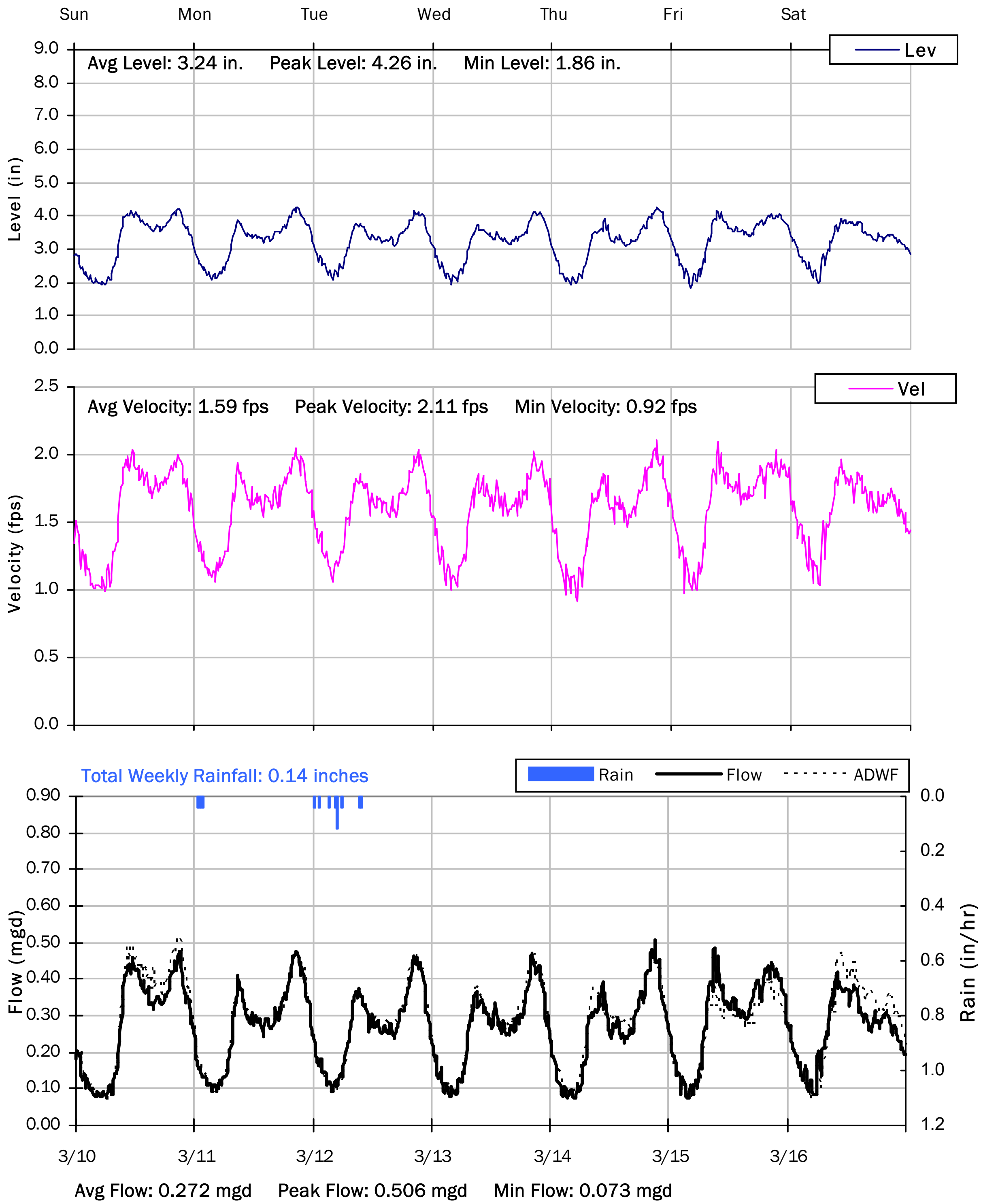




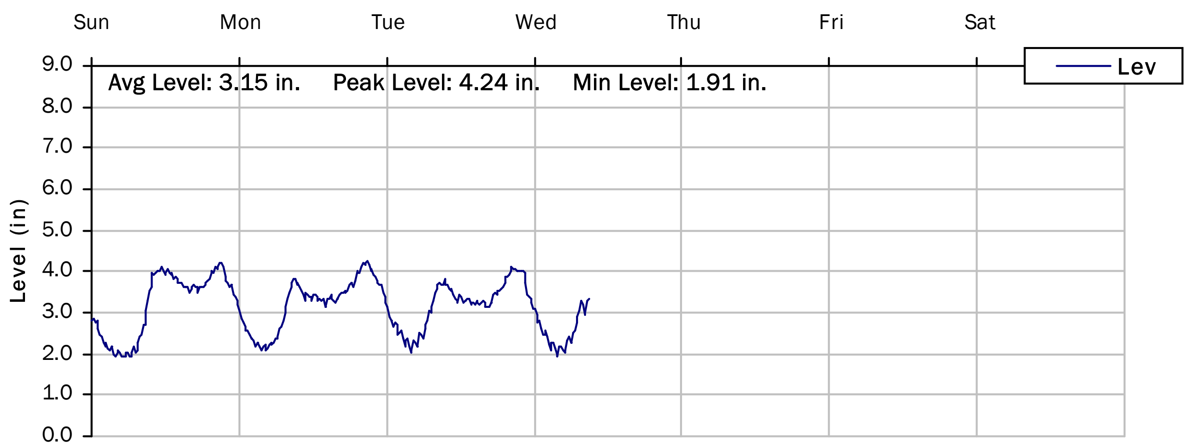


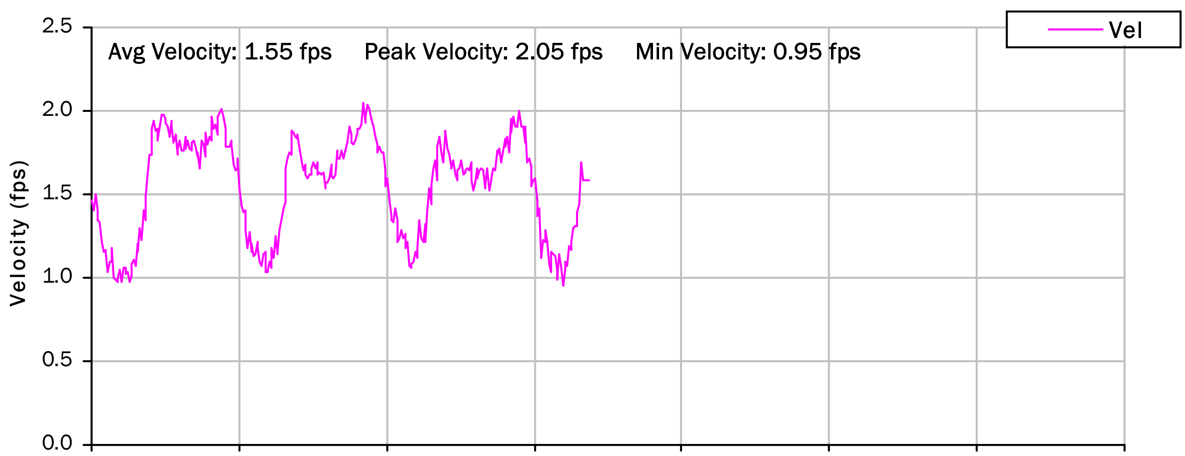
Avg Flow: 0.276 mgd Peak Flow: 0.502 mgd Min Flow: 0.070 mgd

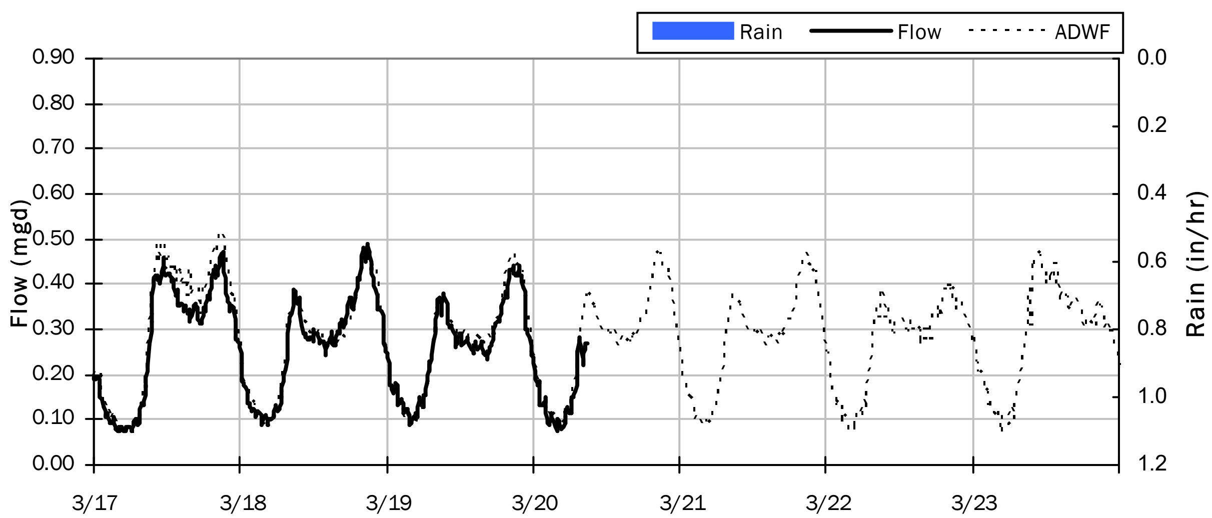
FM01
Weekly Level, Velocity and Flow Hydrographs
3/10/2024 to 3/17/2024



FM01
Weekly Level, Velocity and Flow Hydrographs
3/17/2024 to 3/24/2024







Avg Flow: 0.257 mgd   Peak Flow: 0.488 mgd   Min Flow: 0.074 mgd

Monitoring Site: FM02

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: 102 2nd St, Oakley

Data Summary Report

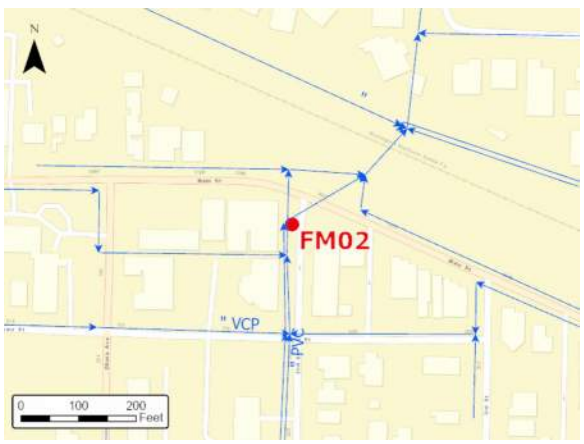
Vicinity Map: FM02

FM02

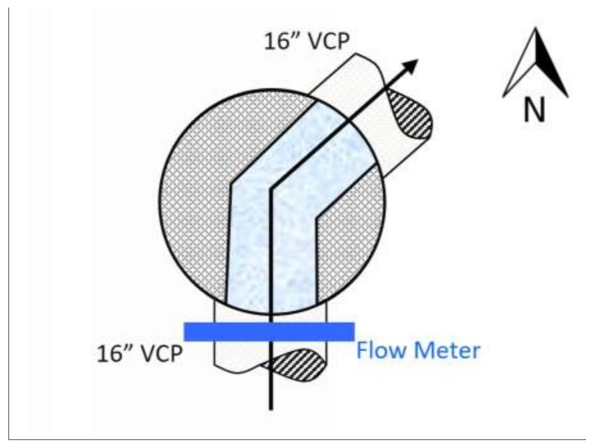
Site Information MH ID: OT040

Location:	102 2nd St, Oakley
Coordinates:	121.7129° W, 37.9976° N
Rim Elevation:	21 feet
Expected Pipe Diameter:	15 inches
Measured Pipe Diameter:	16 inches
ADWF:	0.385 mgd
Peak Measured Flow:	1.09 mgd
Sediment:	None

Satellite Map



Sanitary Map



Flow Sketch


Street View


Plan View

FM02

Additional Site Photos

Northeast Effluent Pipe

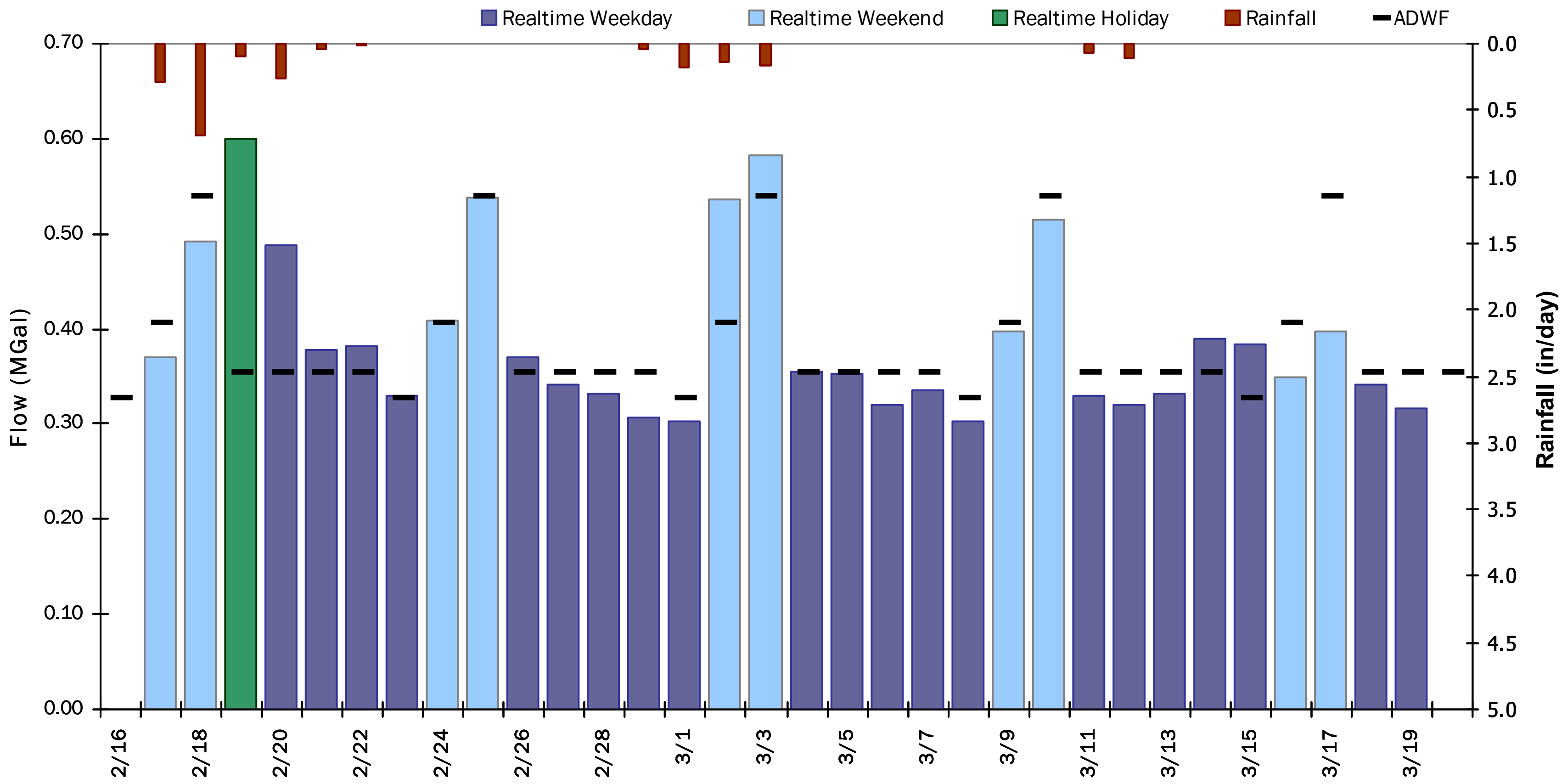
Monitored South Influent Pipe

FM02

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.383 MGal   Peak Daily Flow: 0.599 MGal   Min Daily Flow: 0.120 MGal

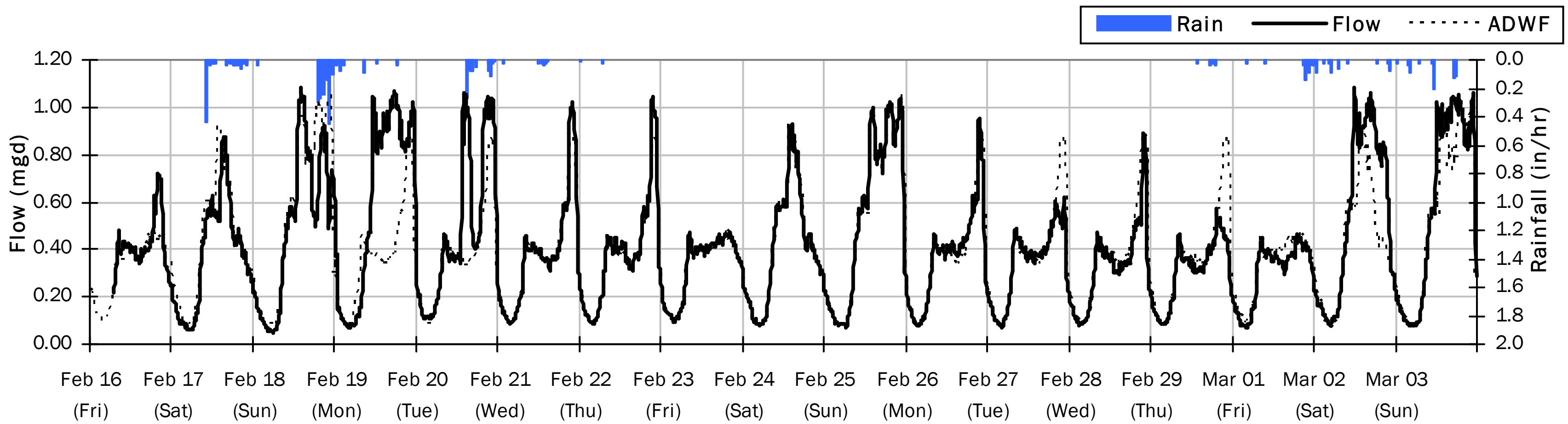
Total Rainfall: 2.09 inches

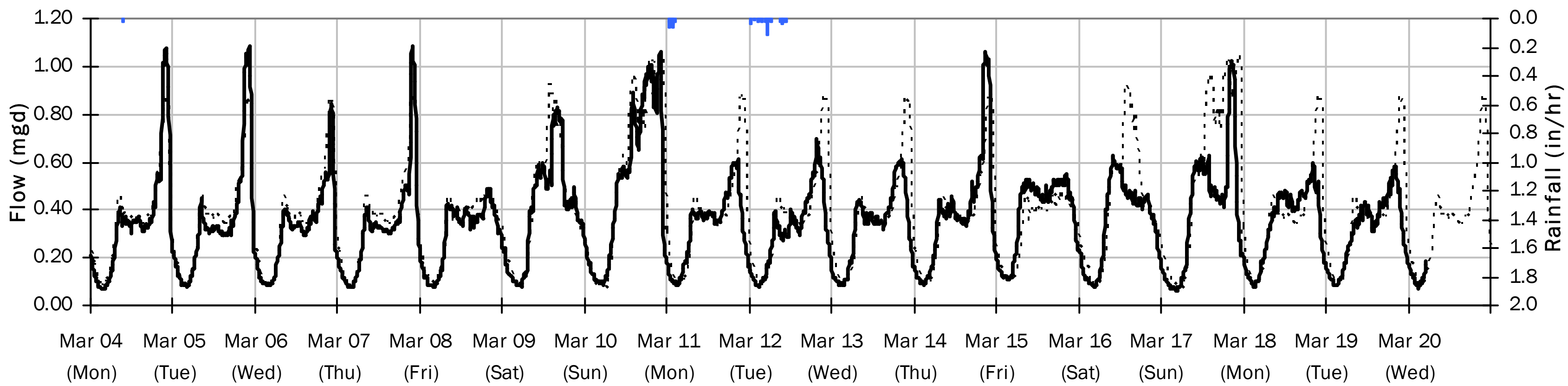


FM02
Flow Summary: 2/16/2024 to 3/20/2024

Period Rainfall: 2.09 inches

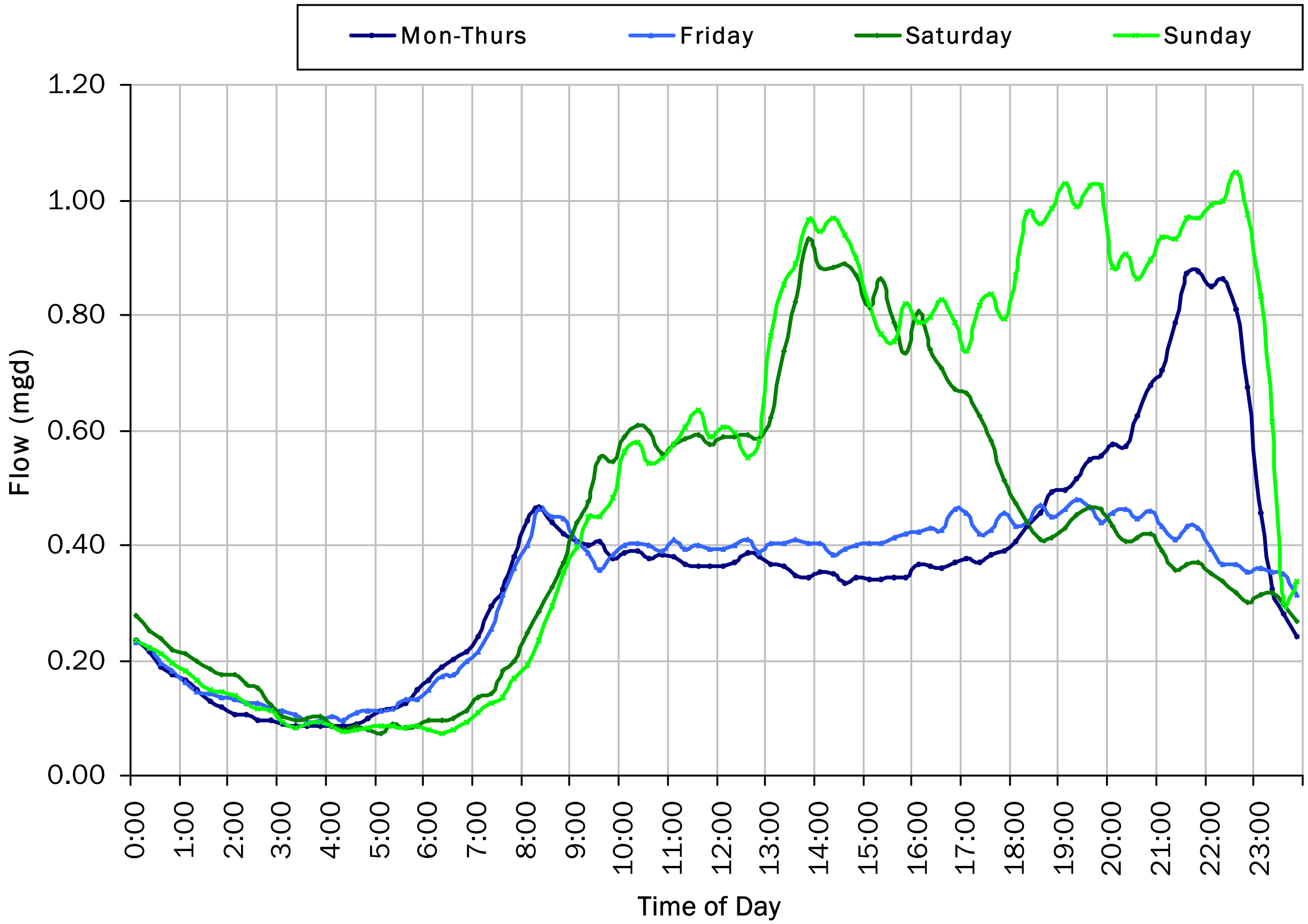
Period Avg Flow: 0.389 mgd Period Peak Flow: 1.087 mgd Period Min Flow: 0.049 mgd





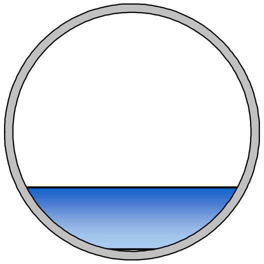
FM02

Average Dry Weather Flow Hydrographs



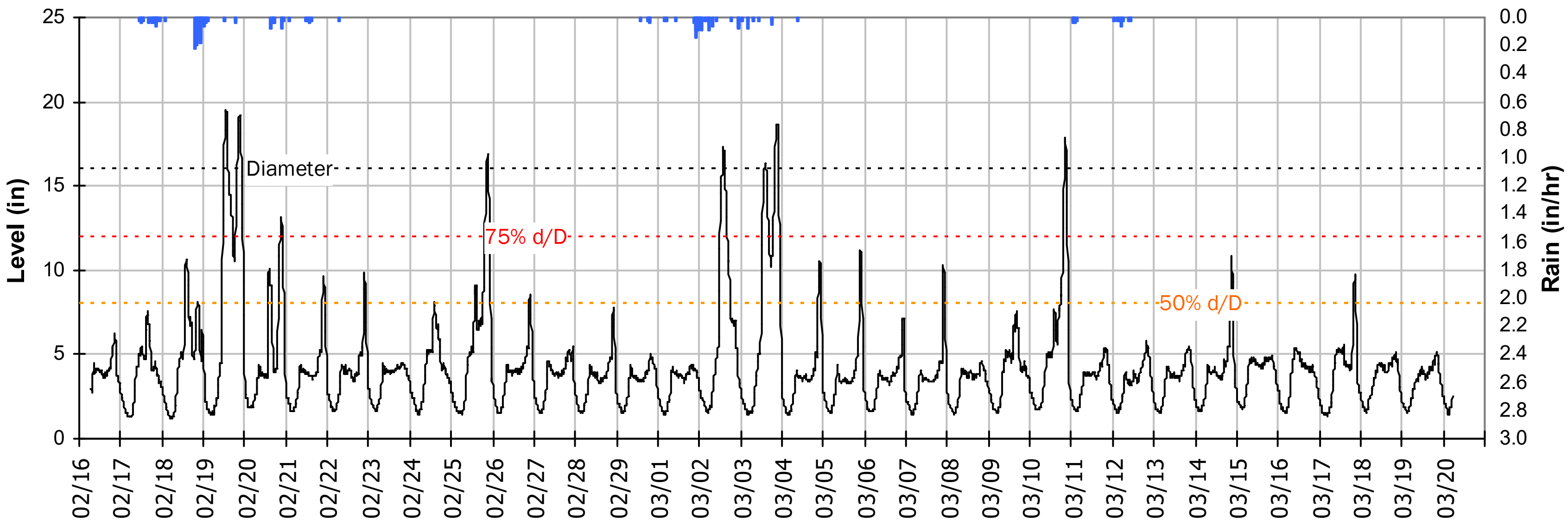
ADWF:

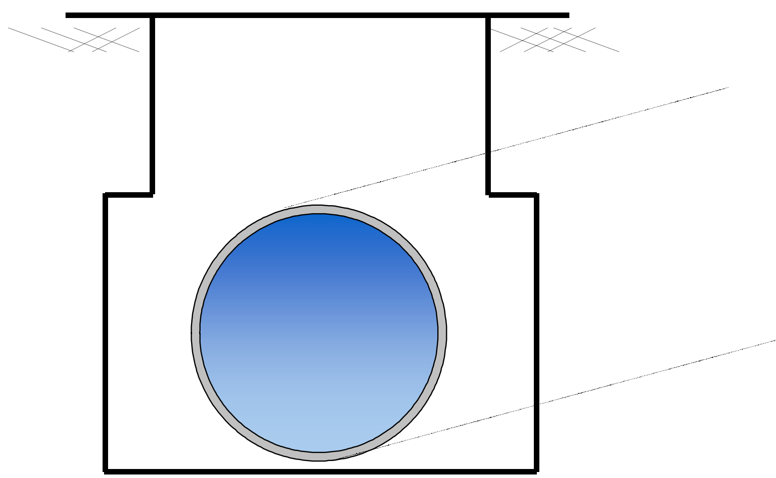
0.382 *mgd*



FM02
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





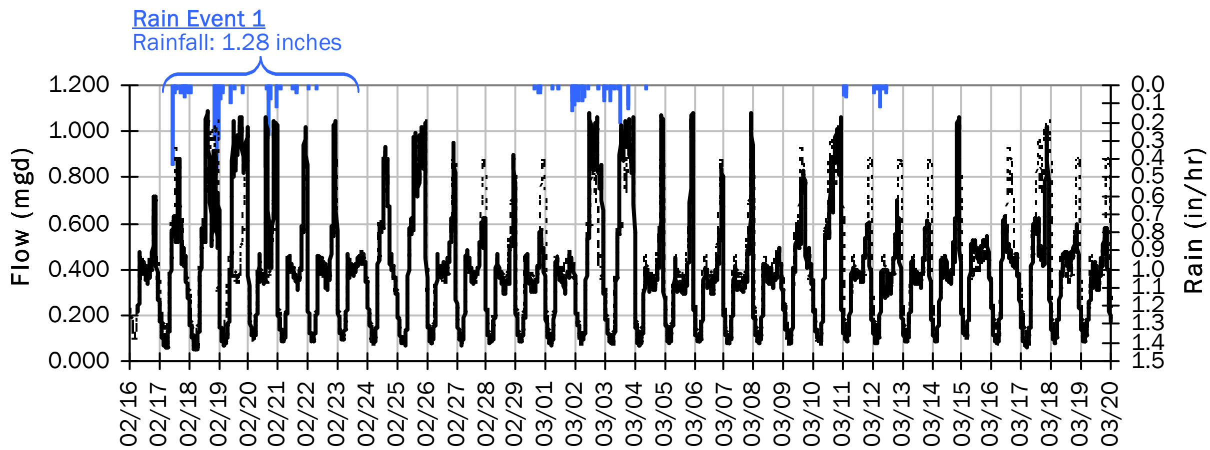
Pipe Diameter:	16	*inches*
Peak Measured Level:	19.5	*inches*
Peak d/D Ratio:	1.22	

Surcharged 3.5 inches over crown

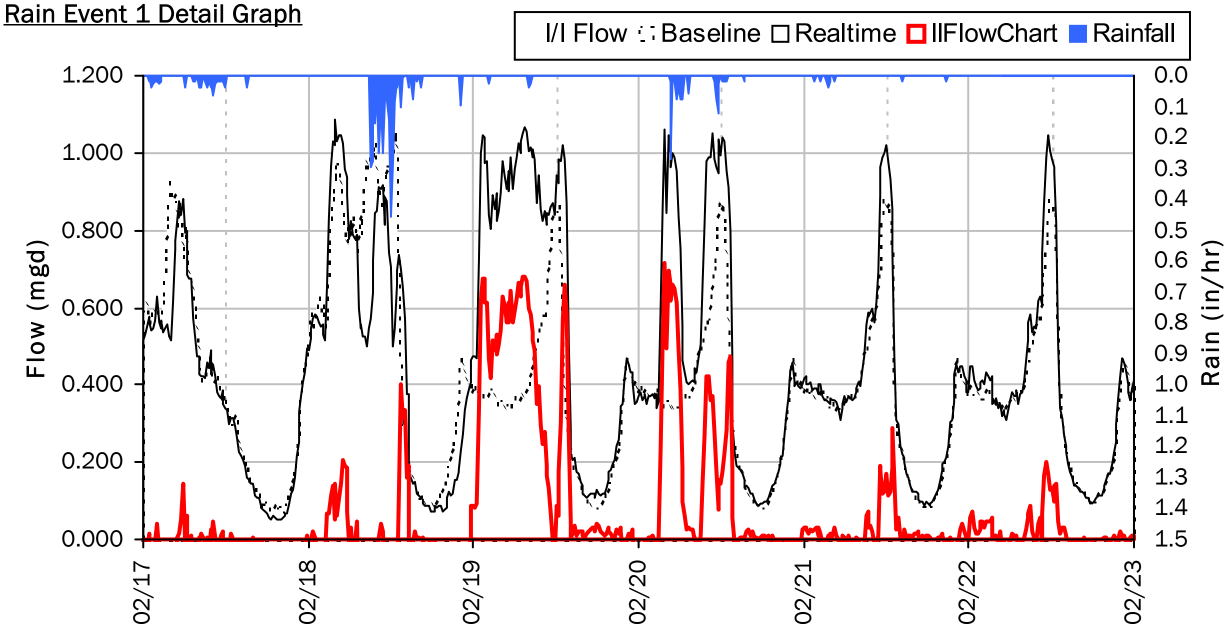
FM02

I/I Summary: Rain Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Rain Event 1 Detail Graph



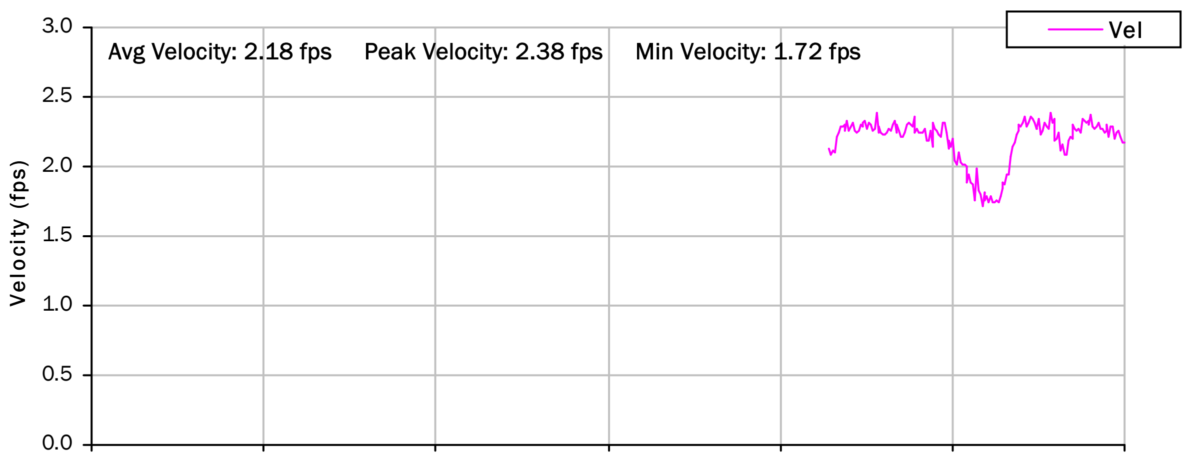
Storm Event I/I Analysis (Rain = 1.28 inches)

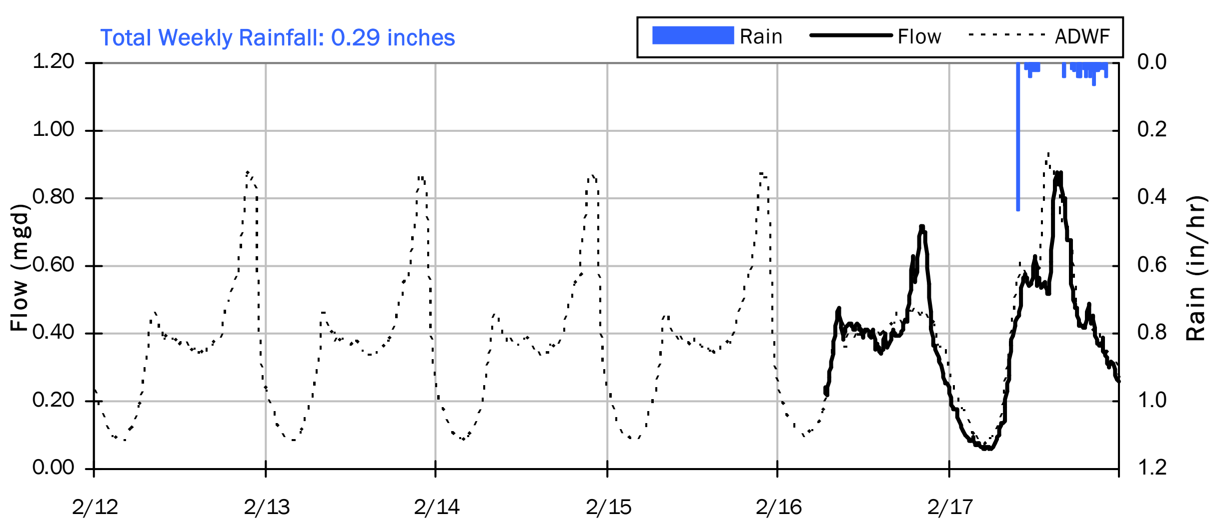
Capacity		Inflow / Infiltration	
Peak Flow:	1.087 *mgd*	Peak I/I Rate:	0.718 *mgd*
PF:	2.82	Total I/I:	353,000 *gallons*
Peak Level:	19.52 *in*		
d/D Ratio:	1.22		

FM02

Weekly Level, Velocity and Flow Hydrographs

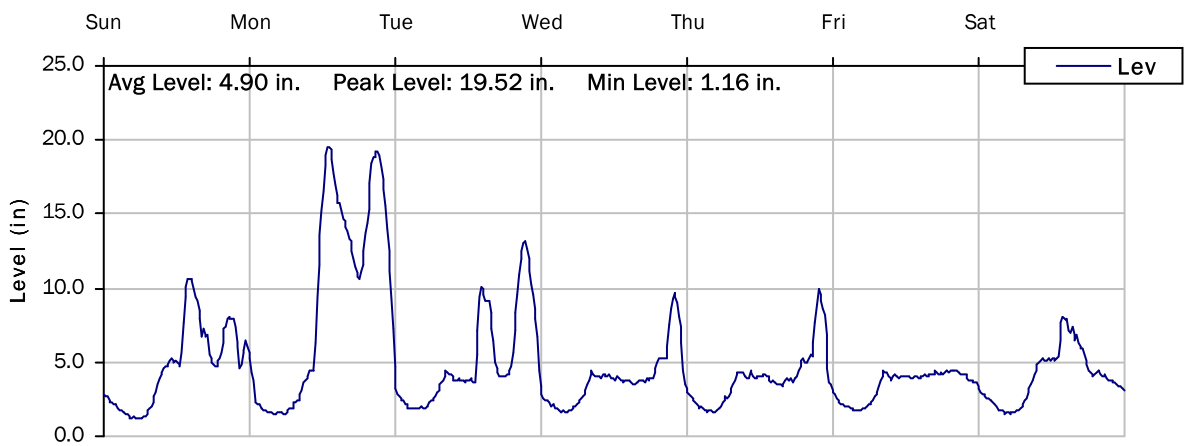
2/12/2024 to 2/18/2024

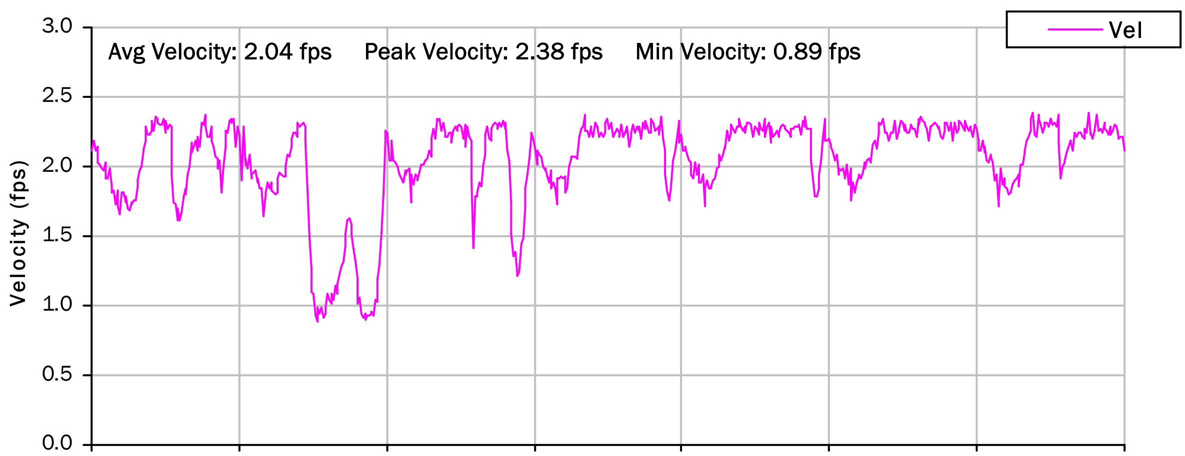


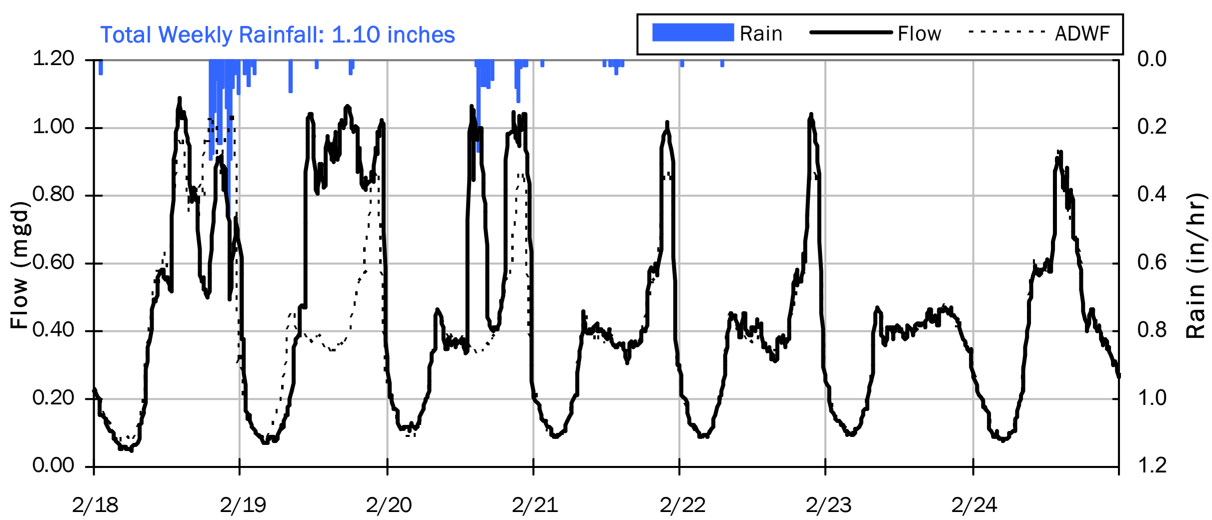


Avg Flow: 0.391 mgd   Peak Flow: 0.878 mgd   Min Flow: 0.058 mgd

FM02
Weekly Level, Velocity and Flow Hydrographs
2/18/2024 to 2/25/2024





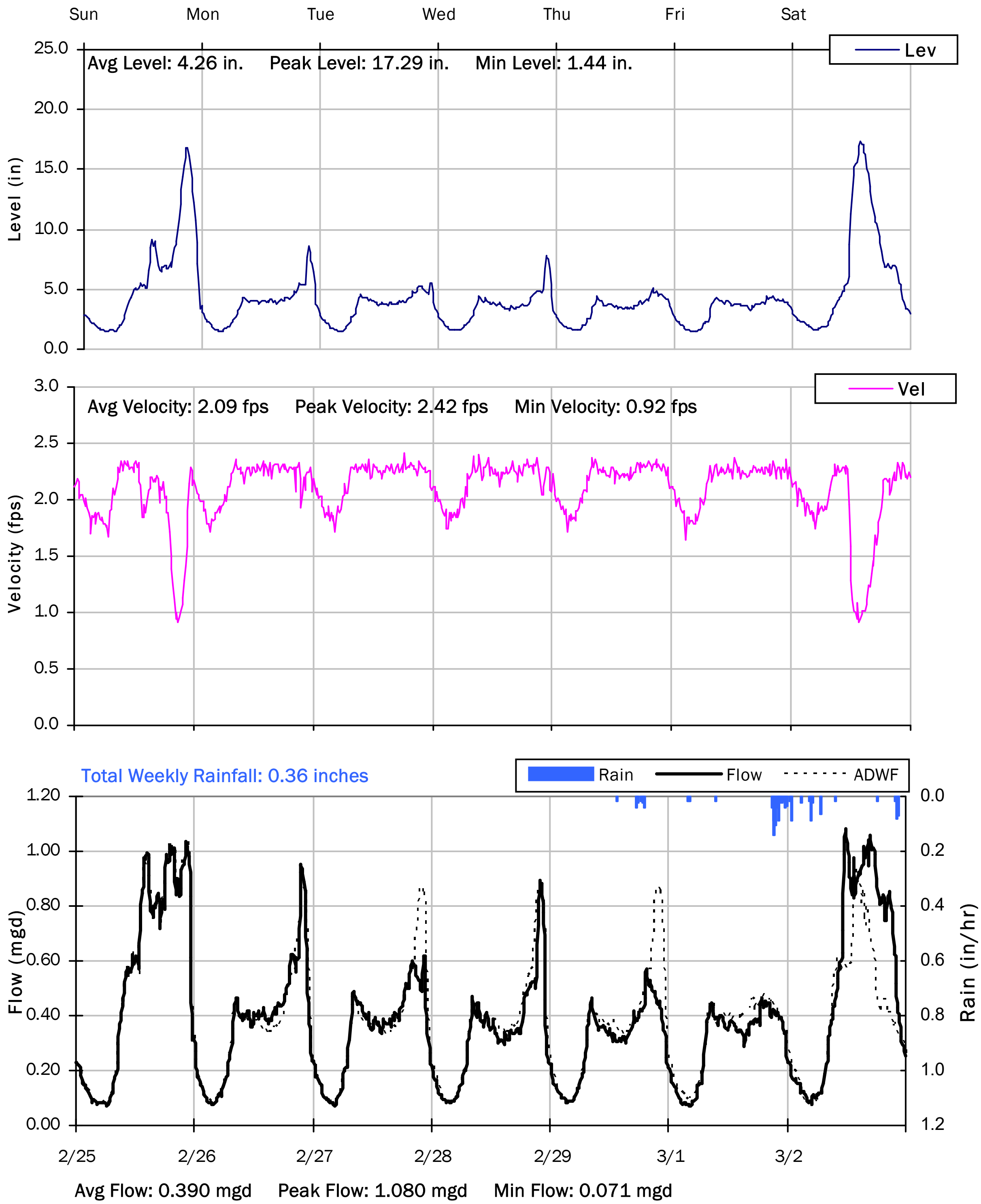


Avg Flow: 0.439 mgd Peak Flow: 1.087 mgd Min Flow: 0.049 mgd

FM02

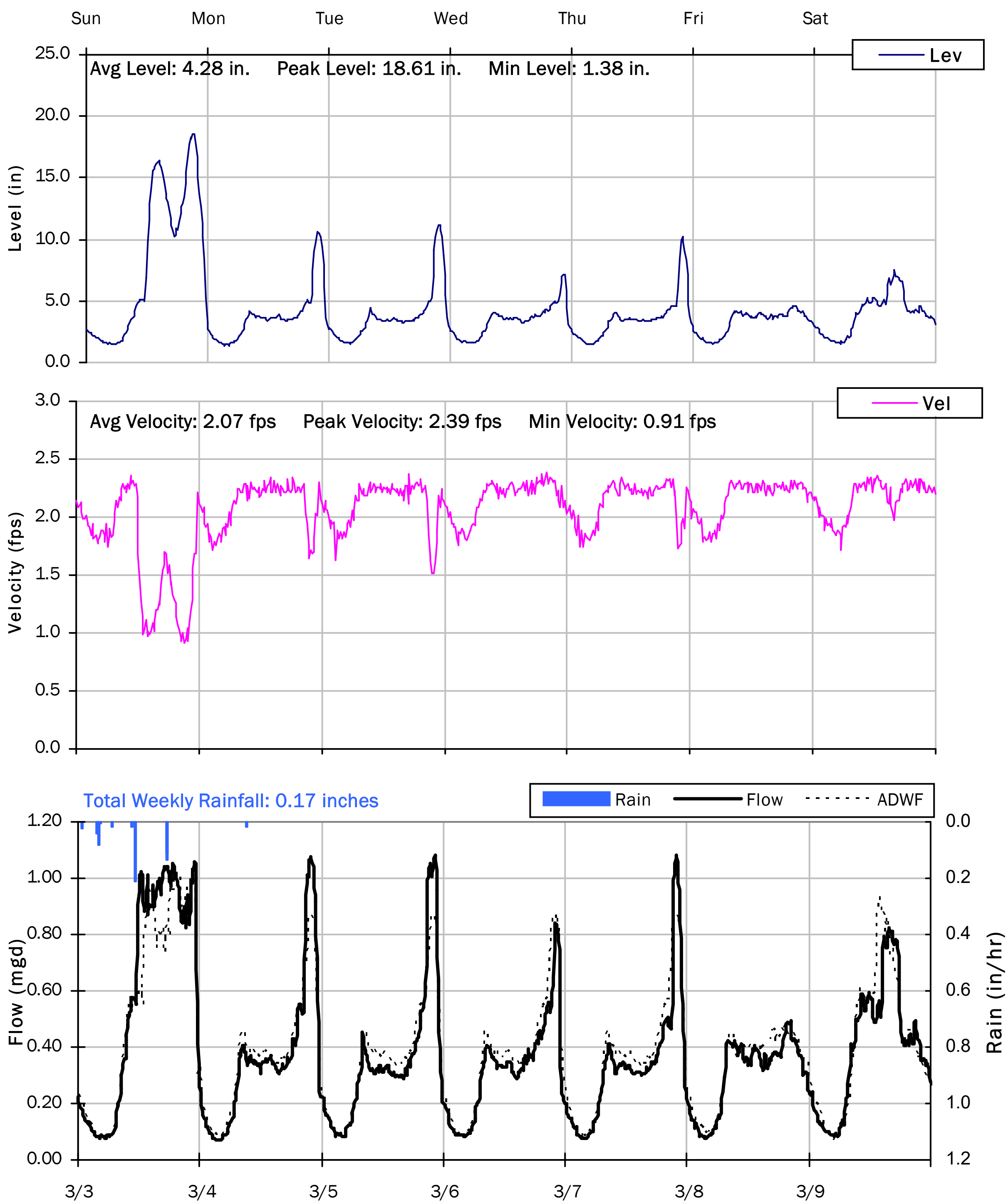
Weekly Level, Velocity and Flow Hydrographs

2/25/2024 to 3/3/2024



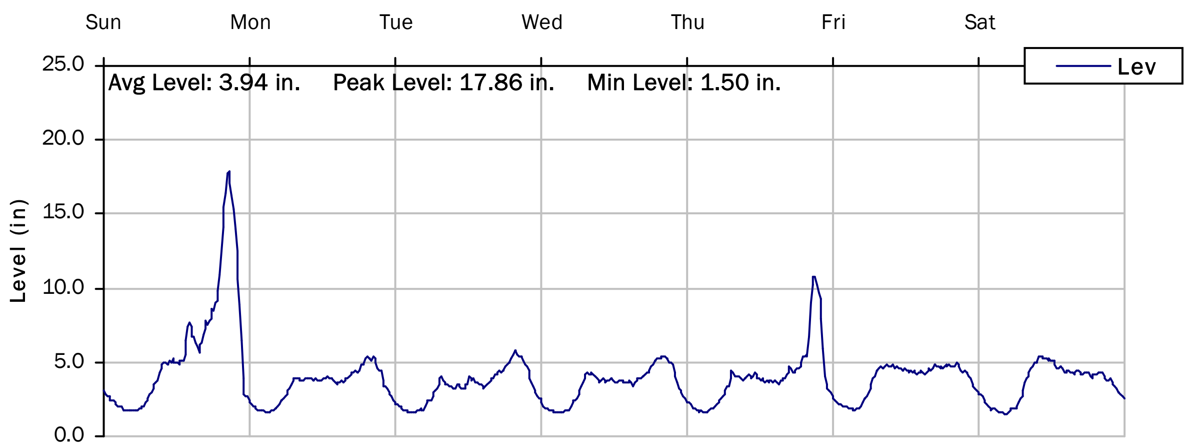
FM02

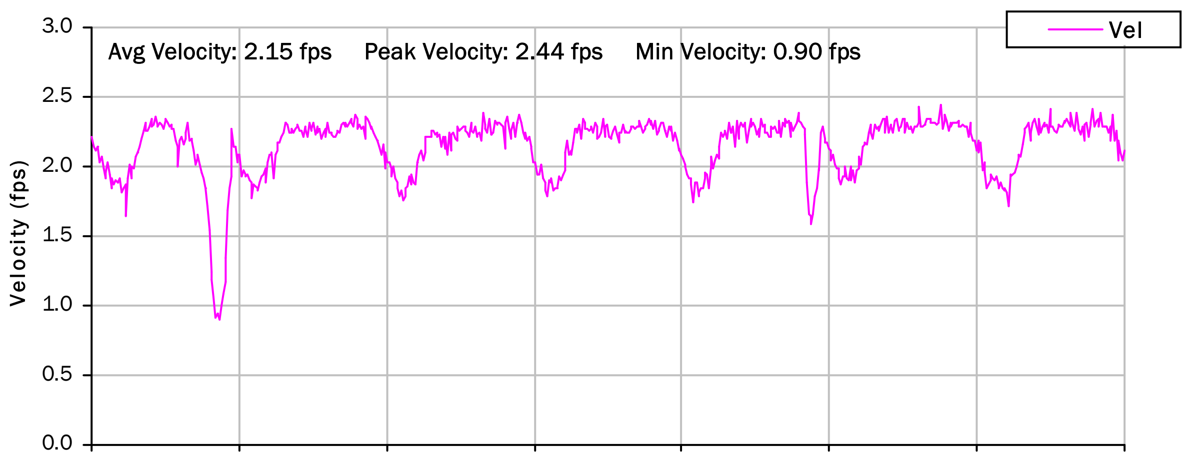
Weekly Level, Velocity and Flow Hydrographs
3/3/2024 to 3/10/2024

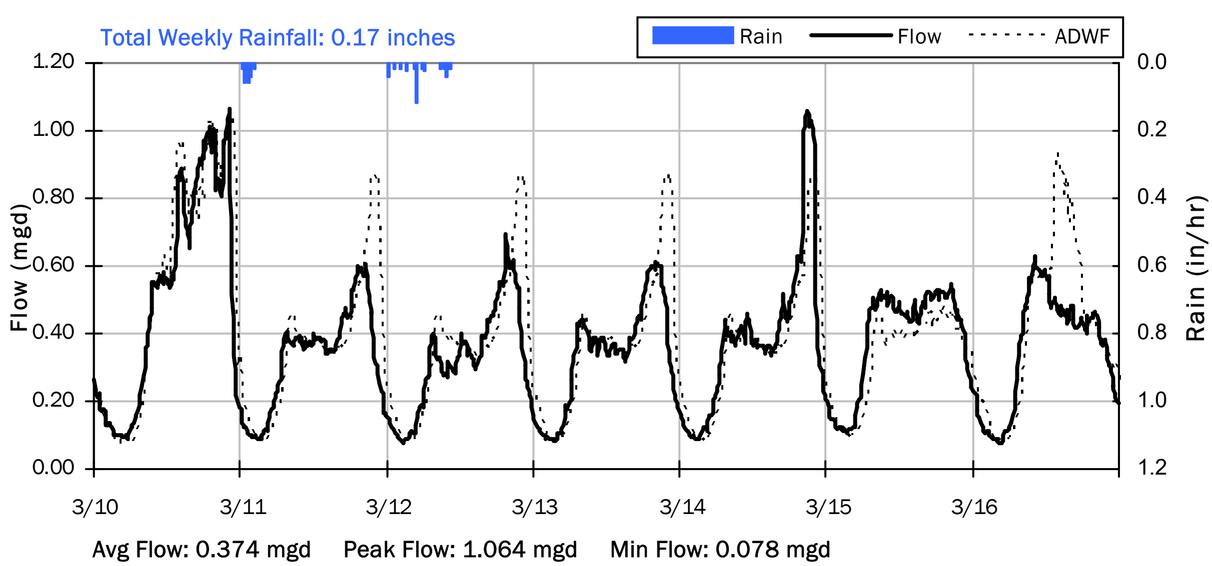


Avg Flow: 0.378 mgd Peak Flow: 1.083 mgd Min Flow: 0.068 mgd

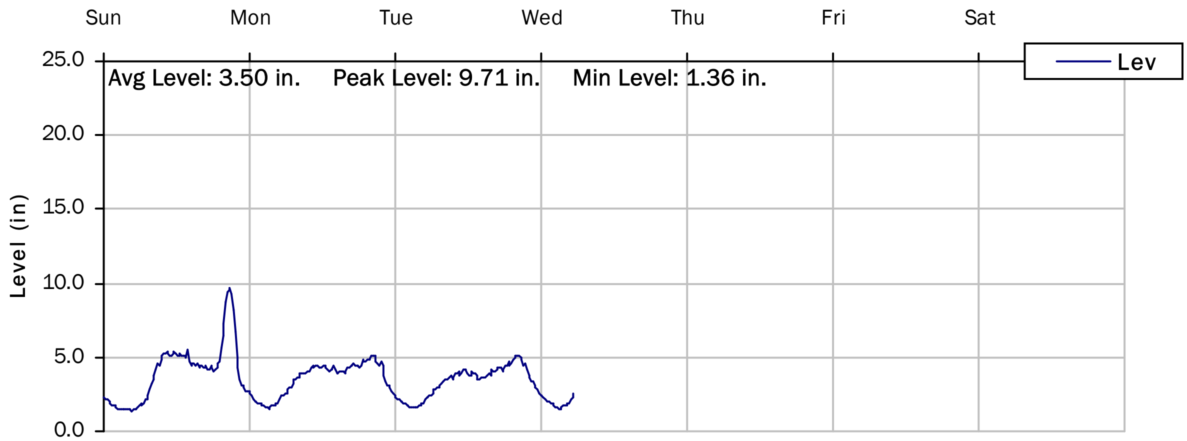
FM02
Weekly Level, Velocity and Flow Hydrographs
3/10/2024 to 3/17/2024

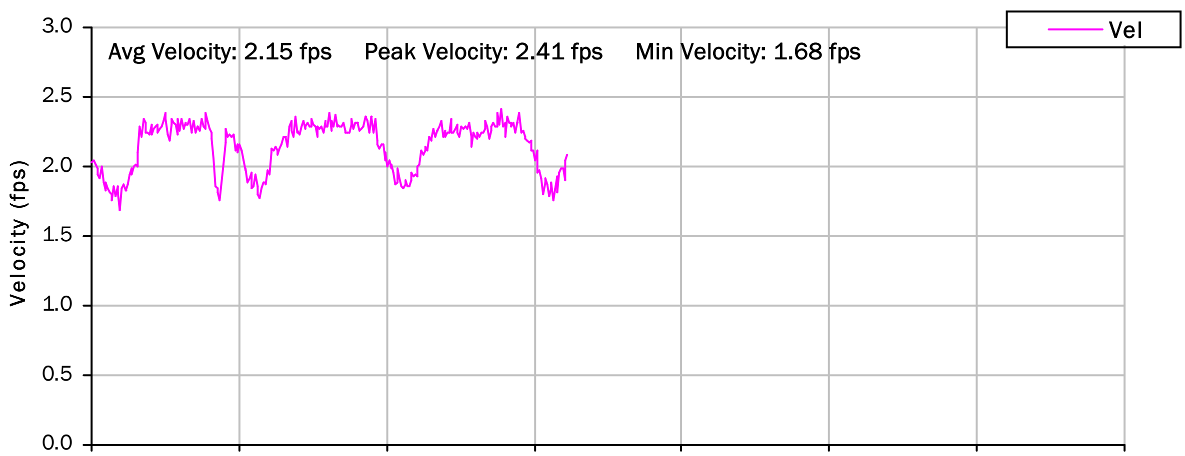


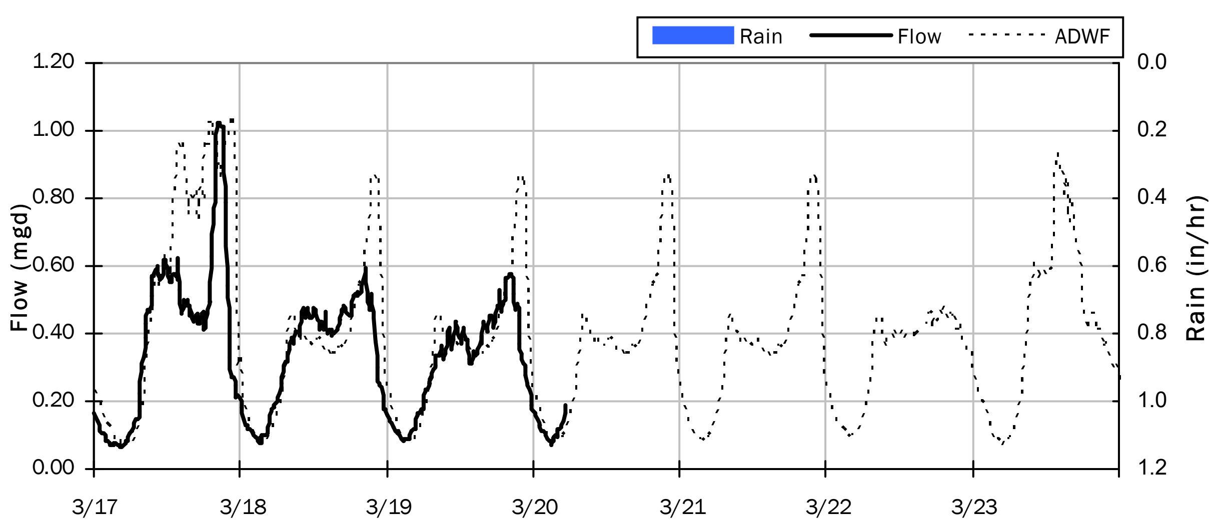




FM02
Weekly Level, Velocity and Flow Hydrographs
3/17/2024 to 3/24/2024







Avg Flow: 0.335 mgd Peak Flow: 1.021 mgd Min Flow: 0.062 mgd

Monitoring Site: Ironwood PS

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: West of 5286 Ironwood Ln, Oakley

Data Summary Report

Vicinity Map: Ironwood PS

IRONWOOD PS

Site Information

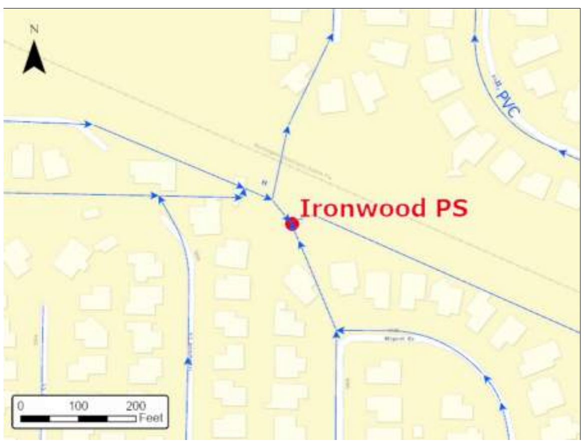
Location: West of 5286 Ironwood Ln, Oakley

Coordinates: 121.7230° W, 38.0013° N

ADWF: 0.113 mgd

Peak Measured Flow: 0.28 mgd

Satellite Map



Sanitary Map

Street View

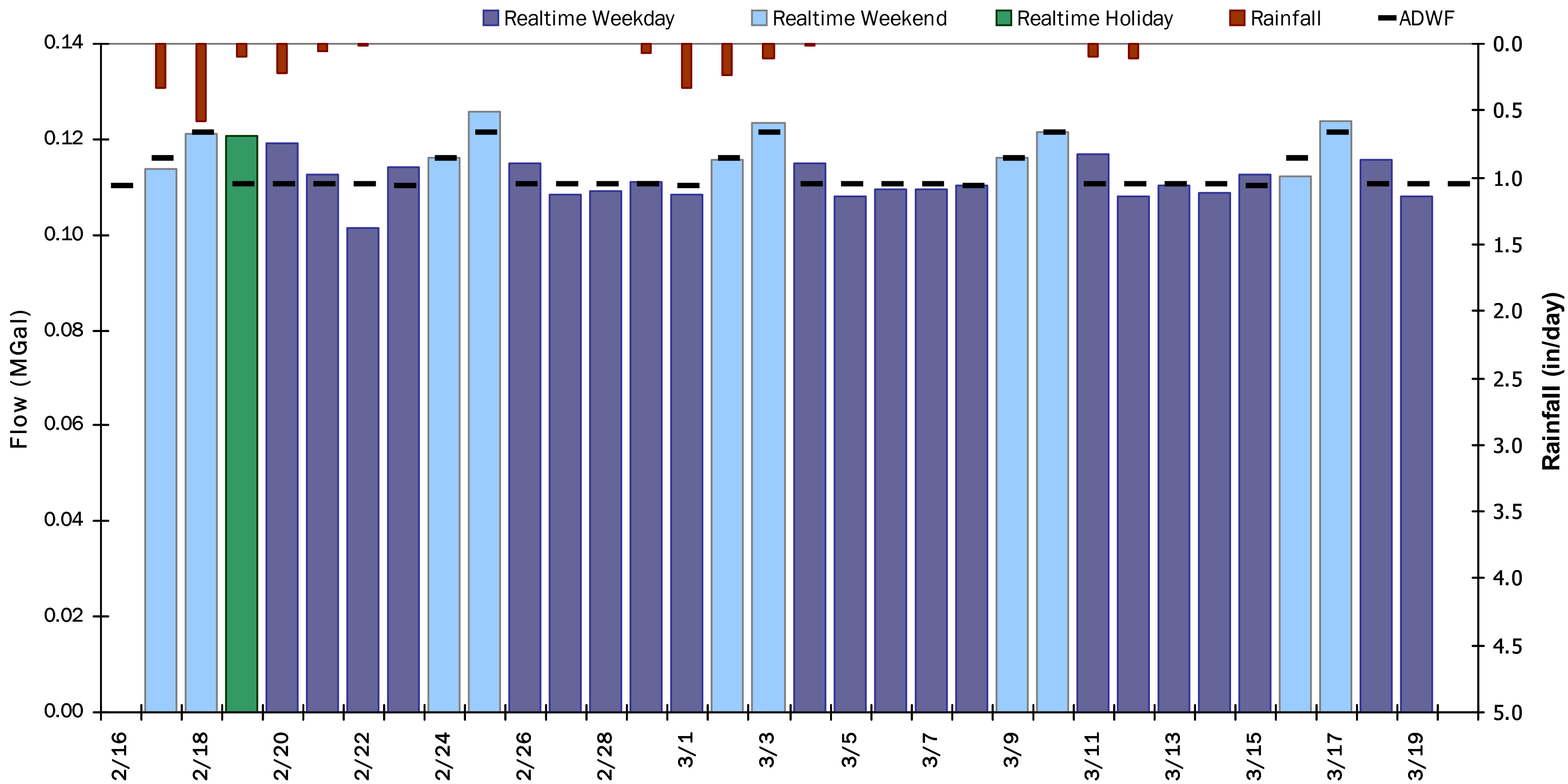
Control Panel

IRONWOOD PS

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.124 MGal   Peak Daily Flow: 0.124 MGal   Min Daily Flow: 0.124 MGal

Total Rainfall: 2.23 inches

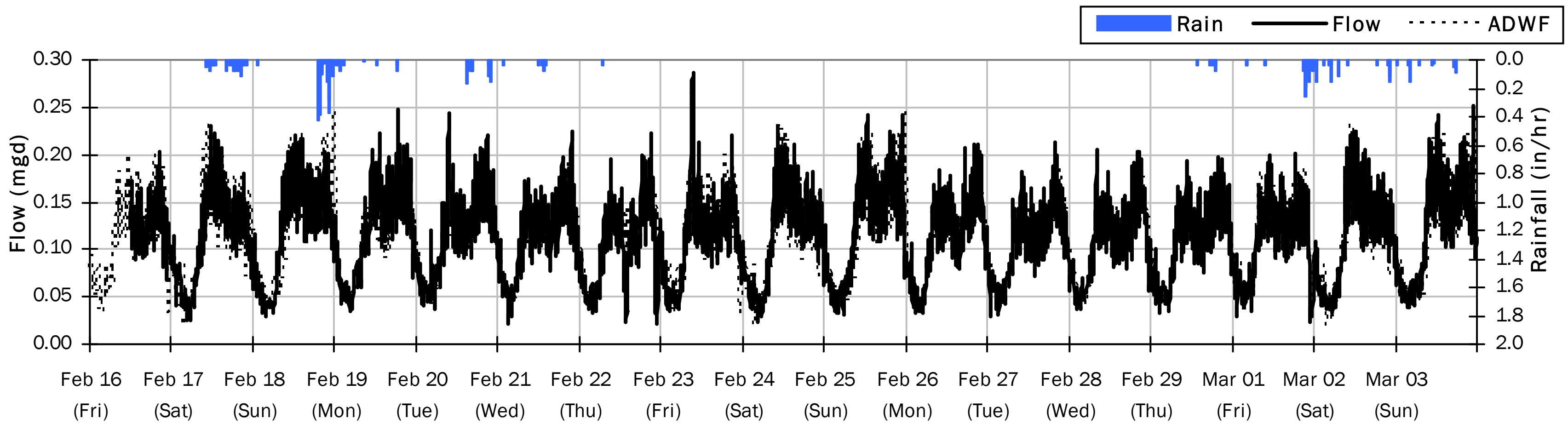


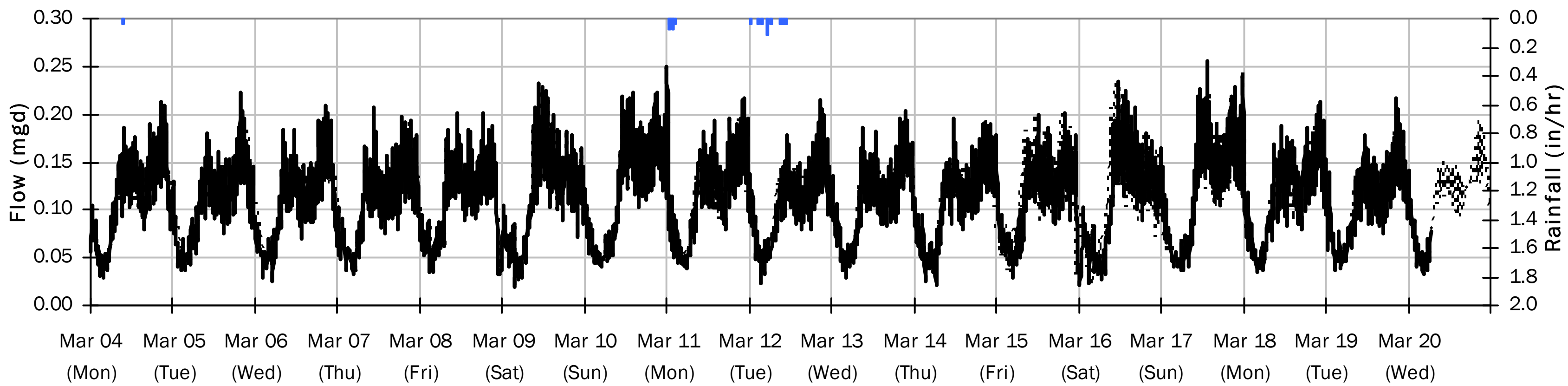
IRONWOOD PS

Flow Summary: 2/16/2024 to 3/20/2024

Period Rainfall: 2.23 inches

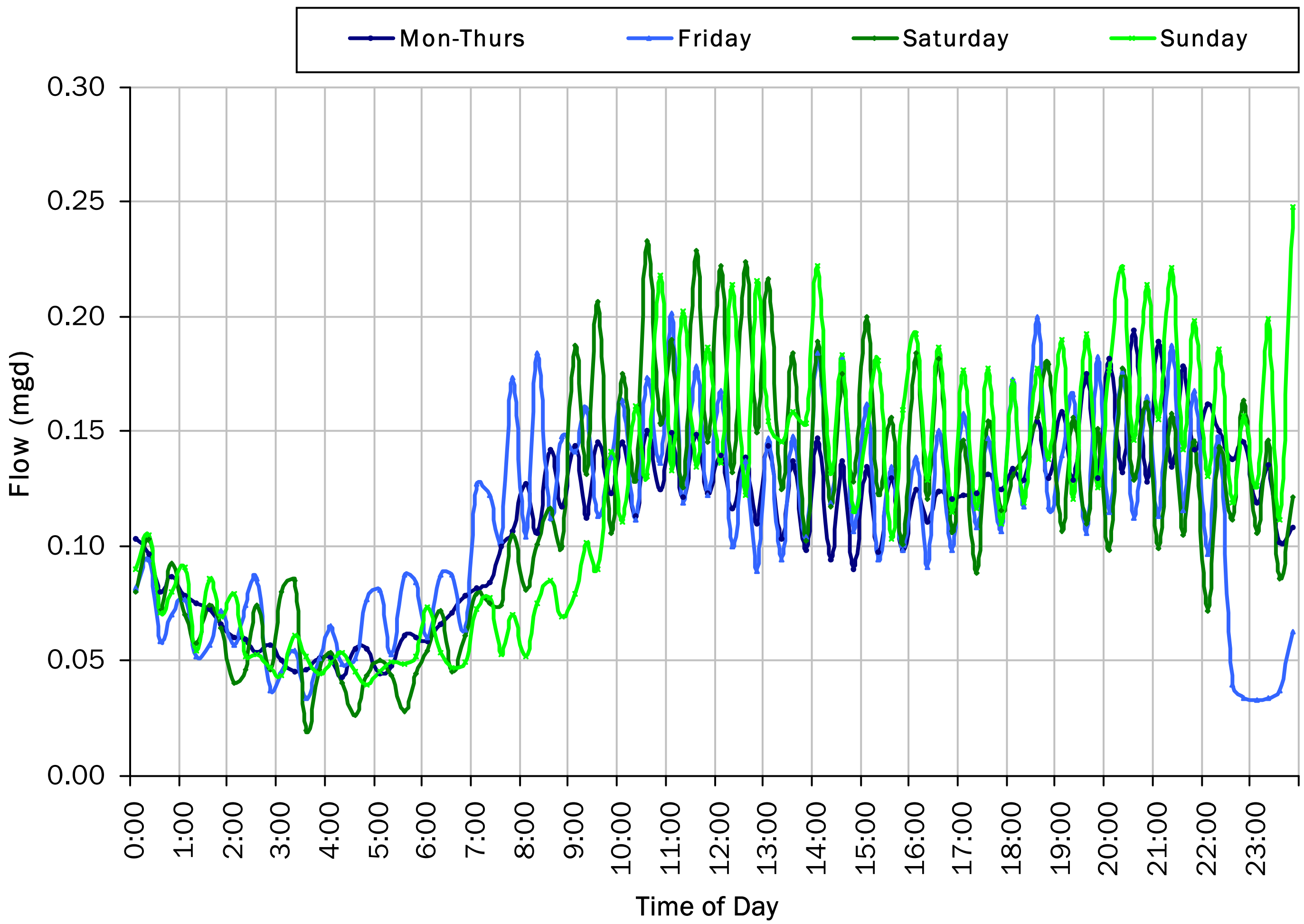
Period Avg Flow: 0.114 mgd Period Peak Flow: 0.284 mgd Period Min Flow: 0.020 mgd





IRONWOOD PS

Average Dry Weather Flow Hydrographs



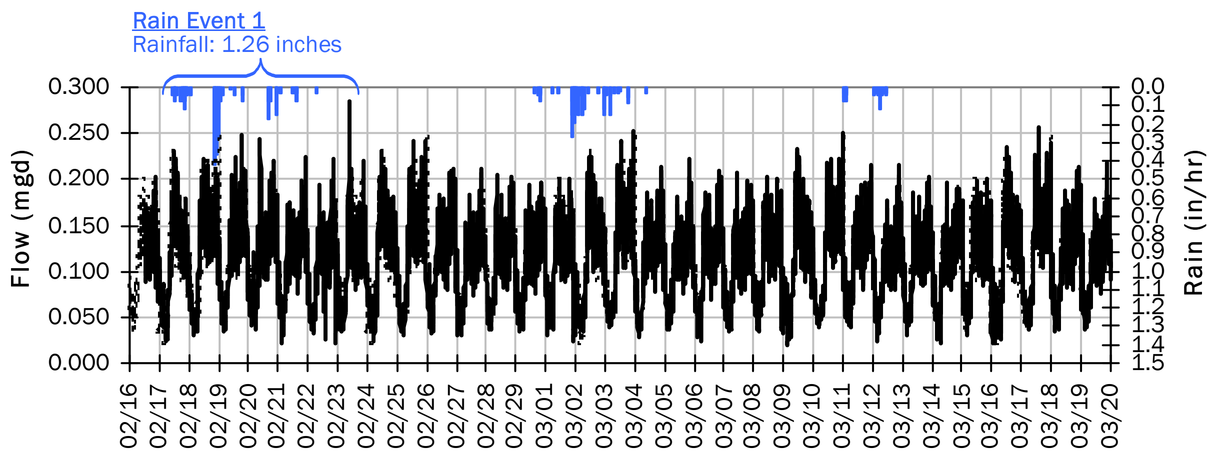
ADWF:

0.113 *mgd*

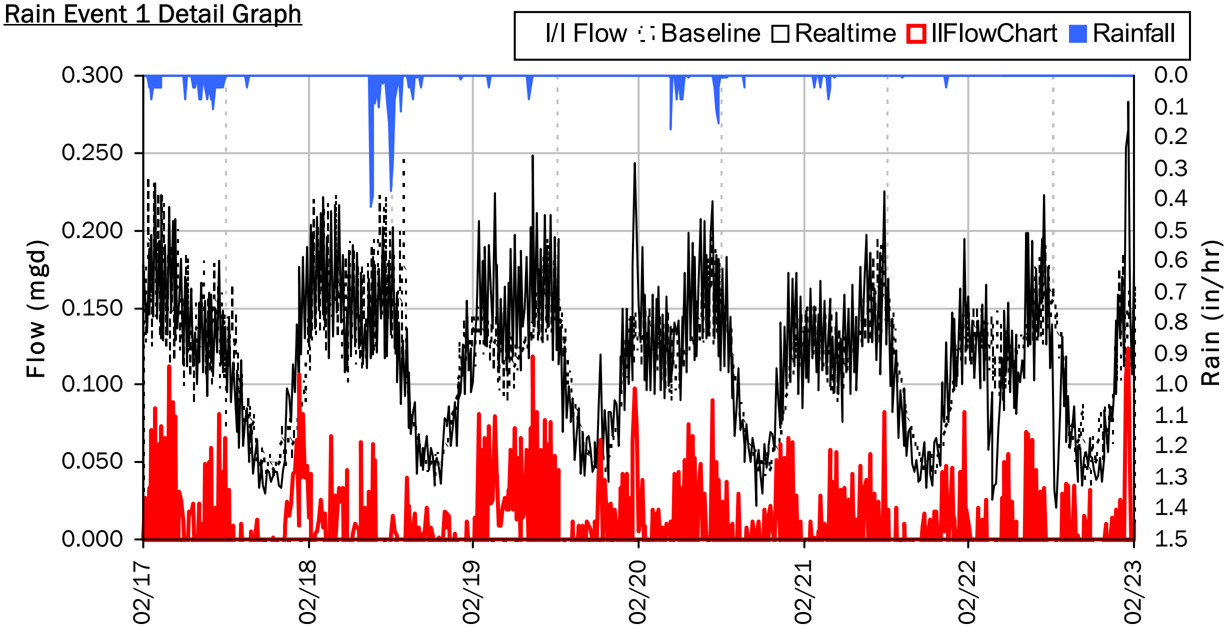
IRONWOOD PS

I/I Summary: Rain Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Rain Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 1.26 inches)

Capacity

Peak Flow: 0.284 *mgd*

PF: 2.51

Peak Level: *in*

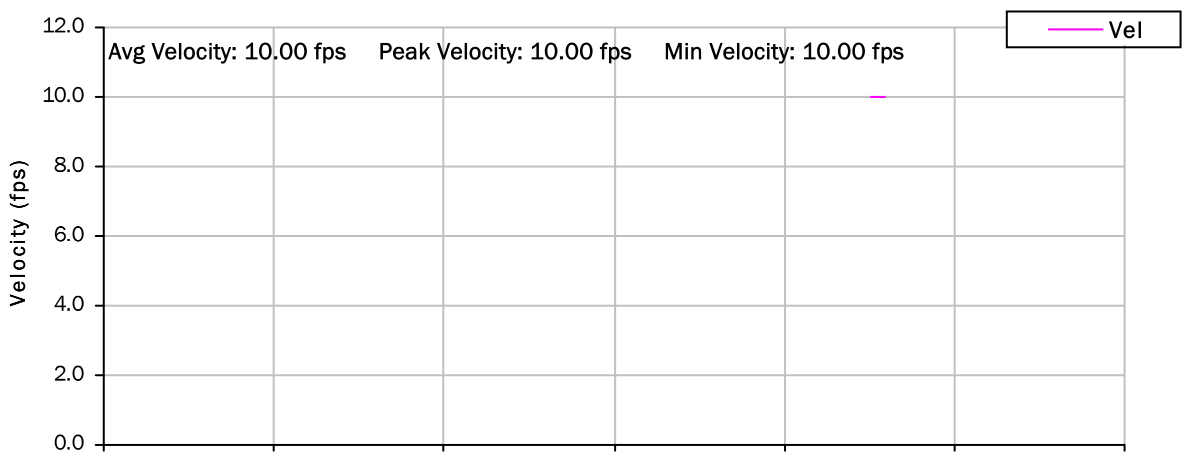
d/D Ratio:

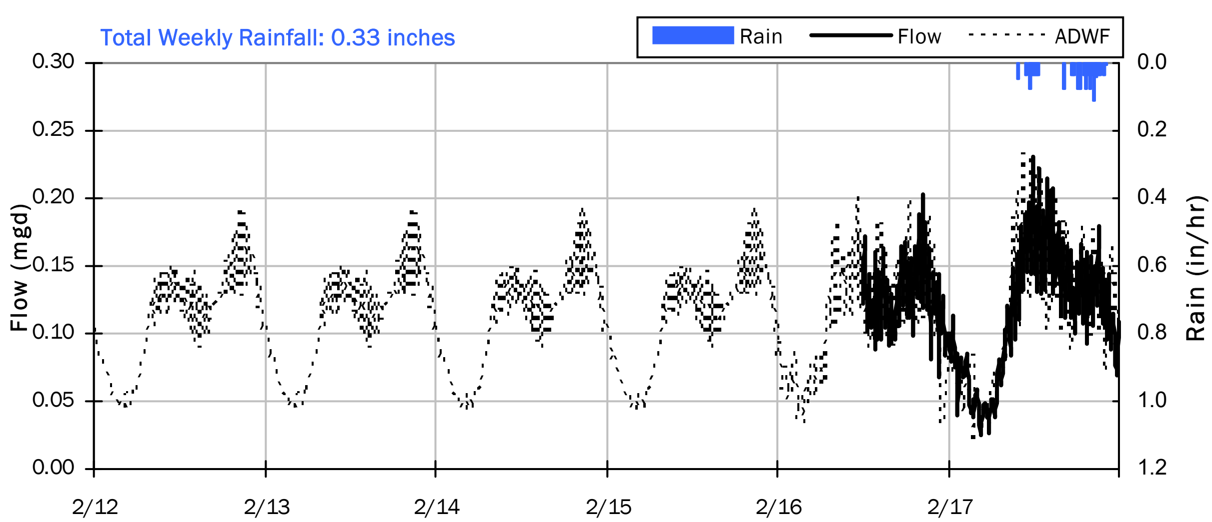
Inflow / Infiltration

Peak I/I Rate: 0.123 *mgd*

Total I/I: 11,000 *gallons*

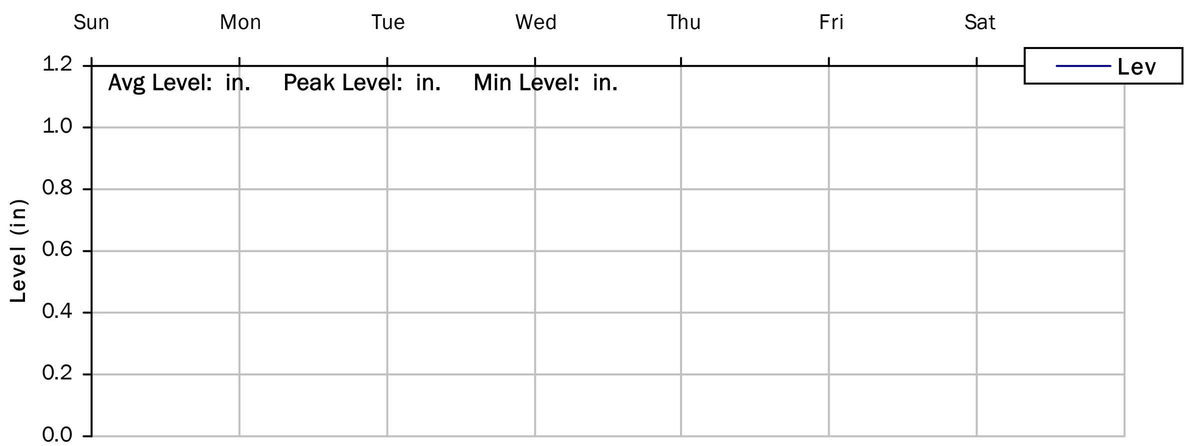
IRONWOOD PS
Weekly Level, Velocity and Flow Hydrographs
2/12/2024 to 2/18/2024

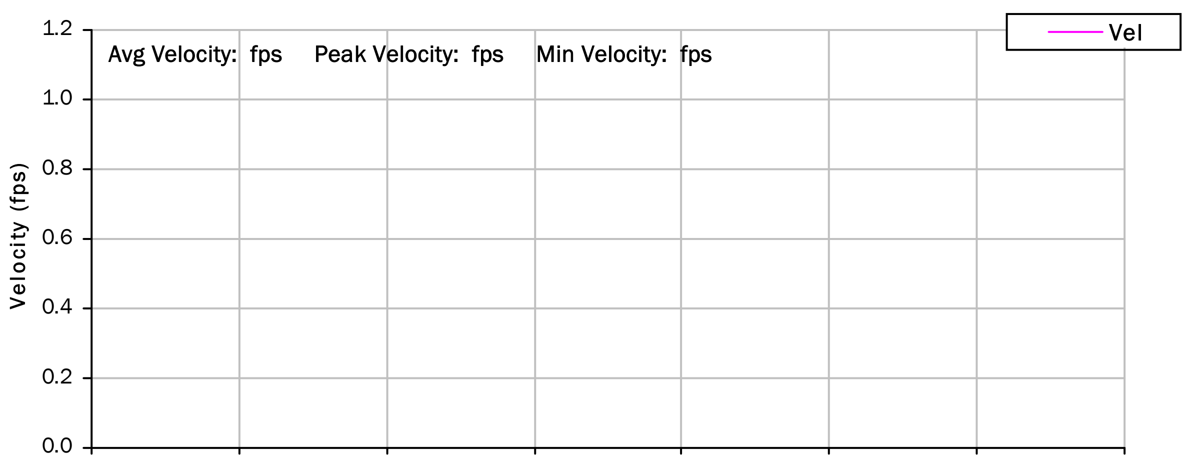


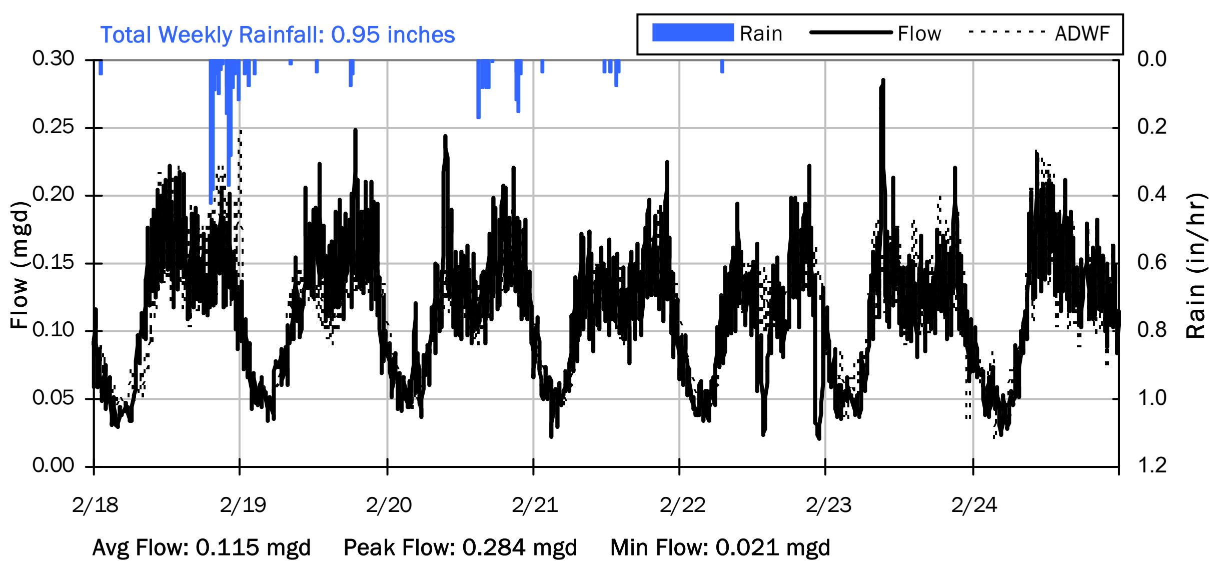


Avg Flow: 0.118 mgd Peak Flow: 0.231 mgd Min Flow: 0.024 mgd

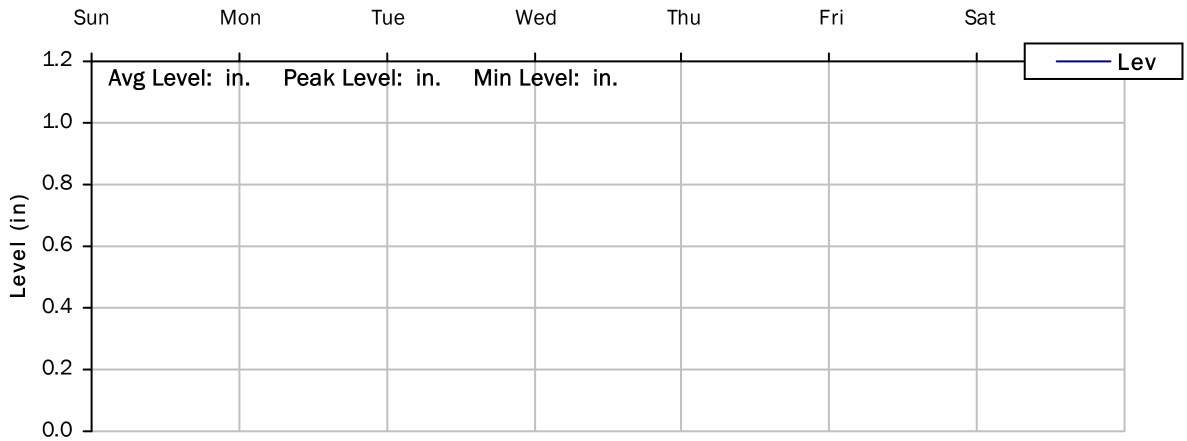
IRONWOOD PS
Weekly Level, Velocity and Flow Hydrographs
2/18/2024 to 2/25/2024

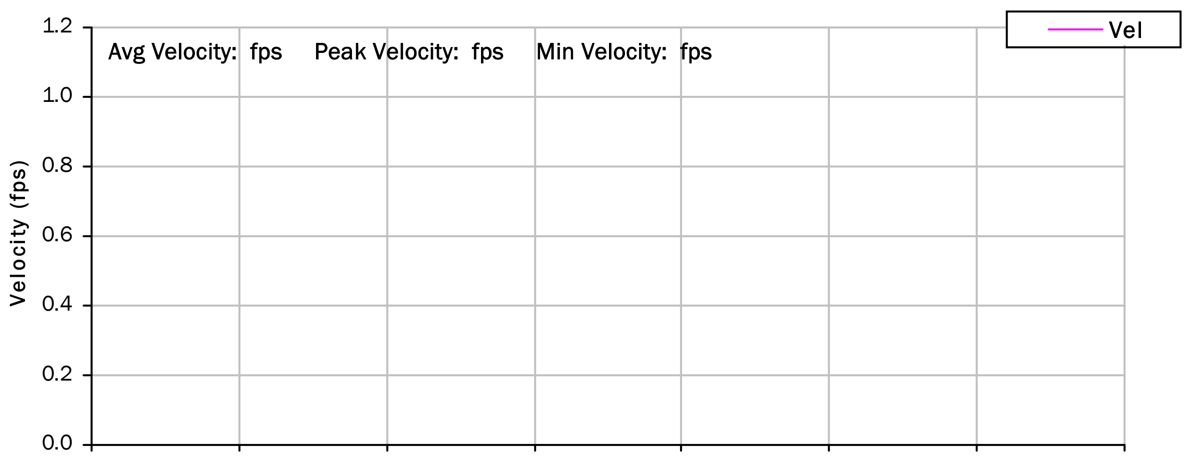


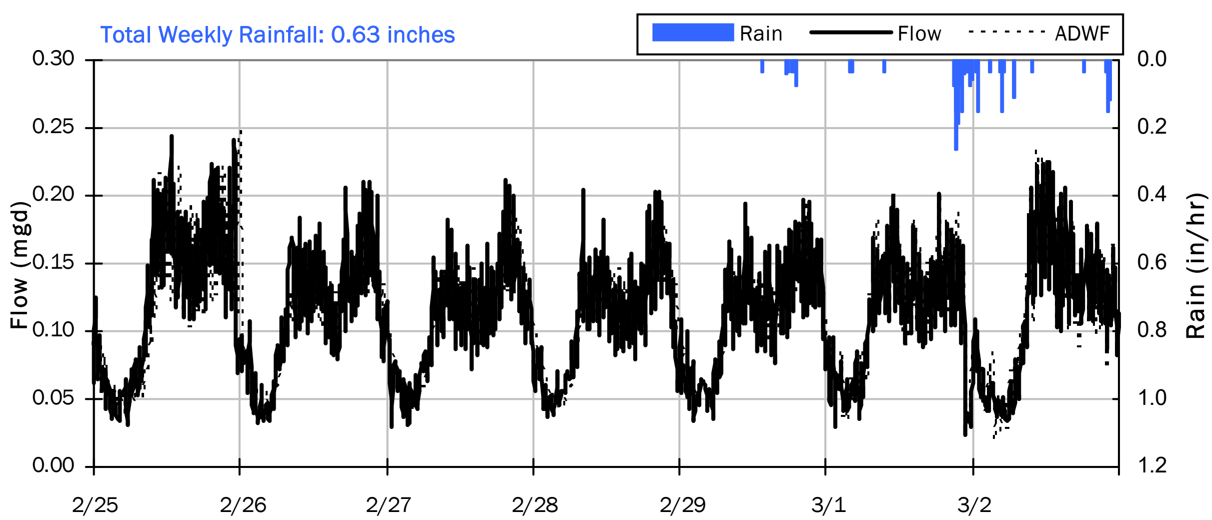




IRONWOOD PS
Weekly Level, Velocity and Flow Hydrographs
2/25/2024 to 3/3/2024





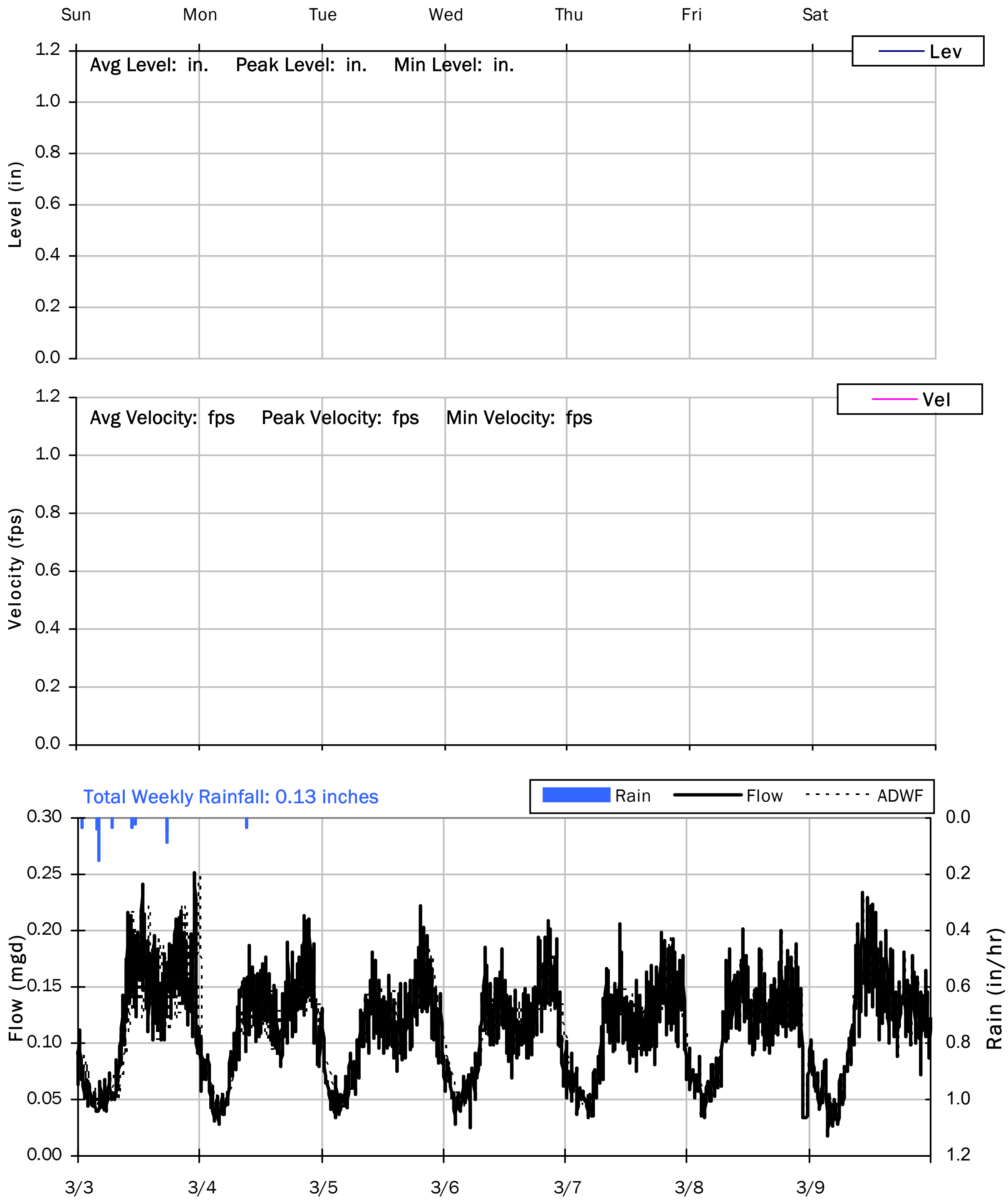


Avg Flow: 0.113 mgd Peak Flow: 0.242 mgd Min Flow: 0.024 mgd

IRONWOOD PS

Weekly Level, Velocity and Flow Hydrographs

3/3/2024 to 3/10/2024

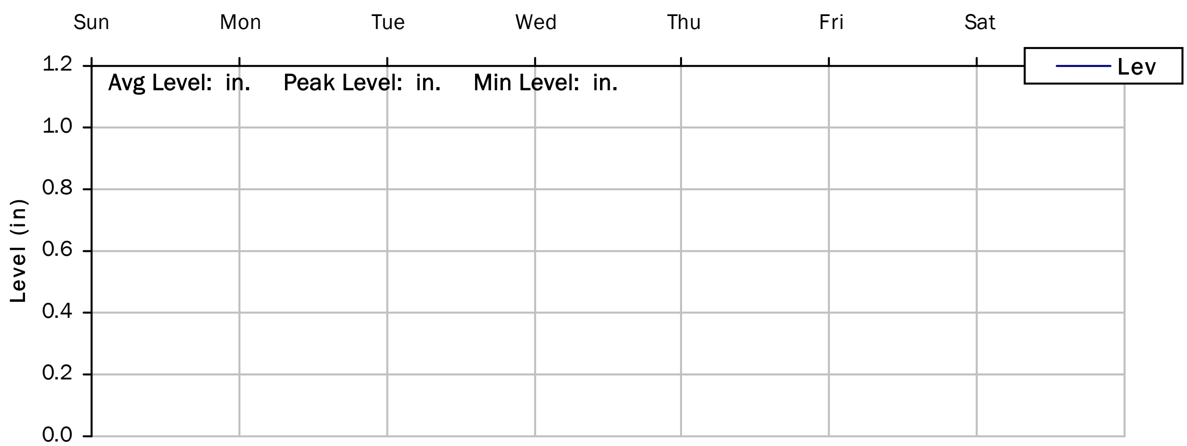


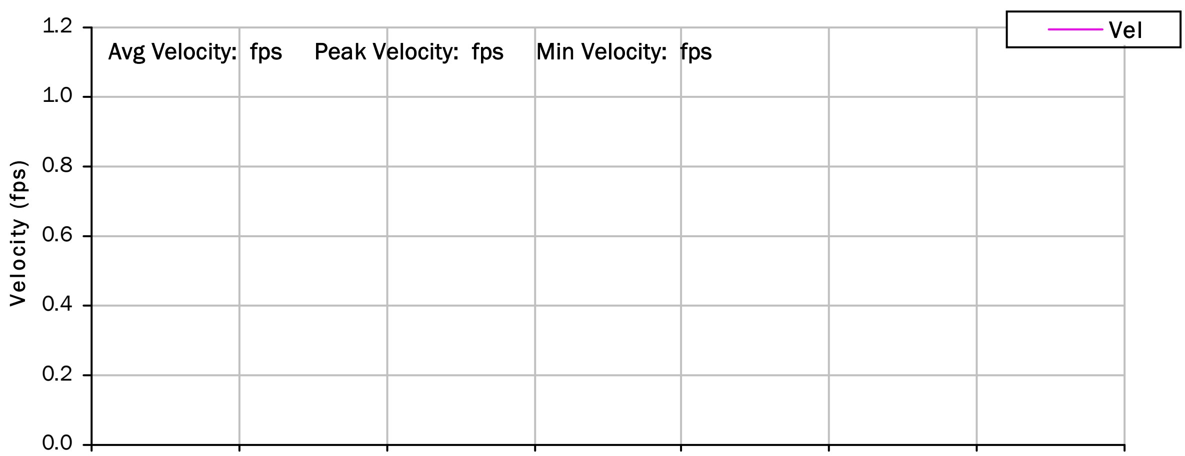
Avg Flow: 0.113 mgd   Peak Flow: 0.250 mgd   Min Flow: 0.020 mgd

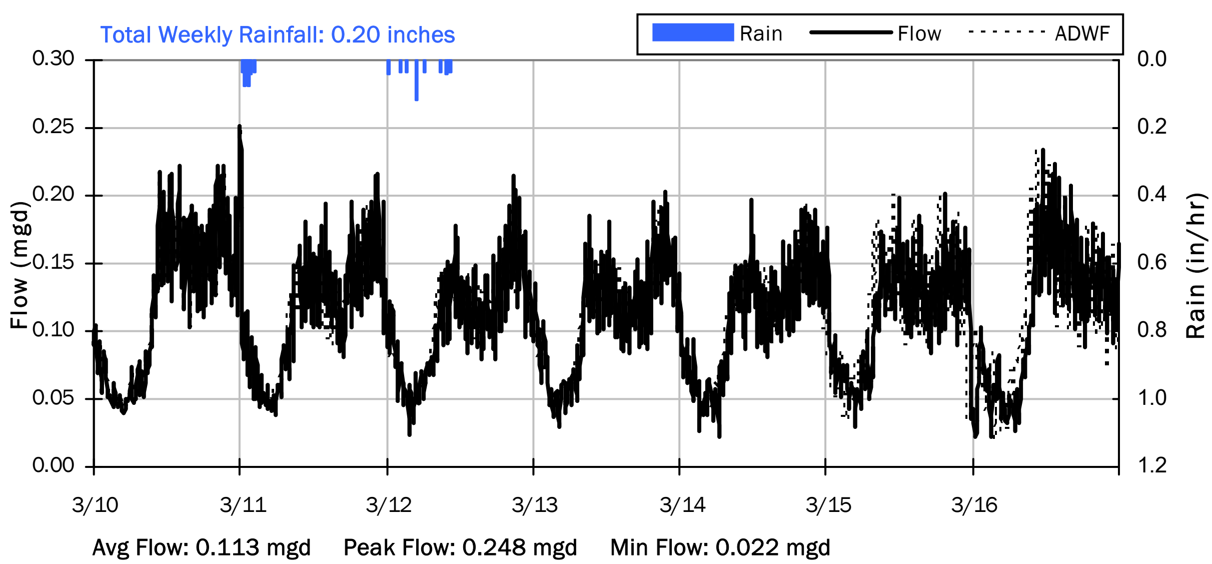
IRONWOOD PS

Weekly Level, Velocity and Flow Hydrographs

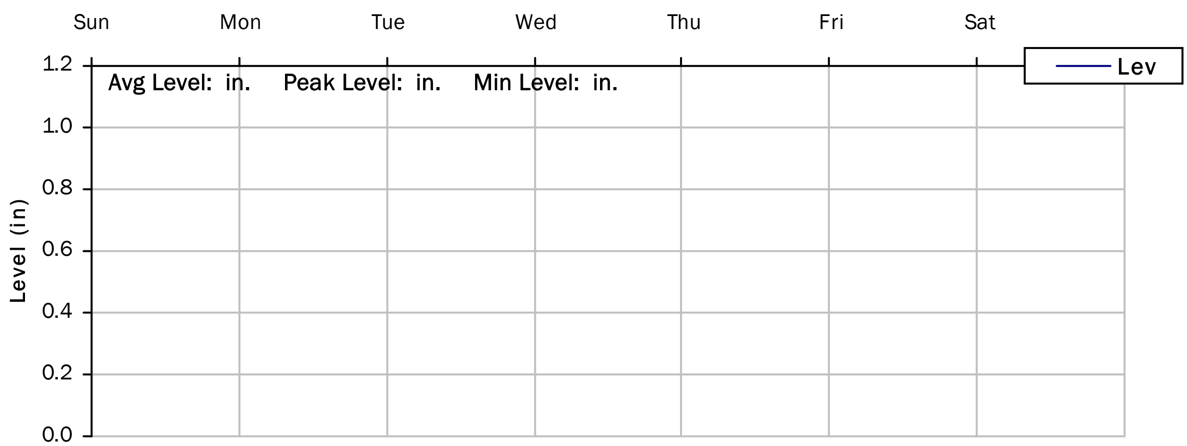
3/10/2024 to 3/17/2024

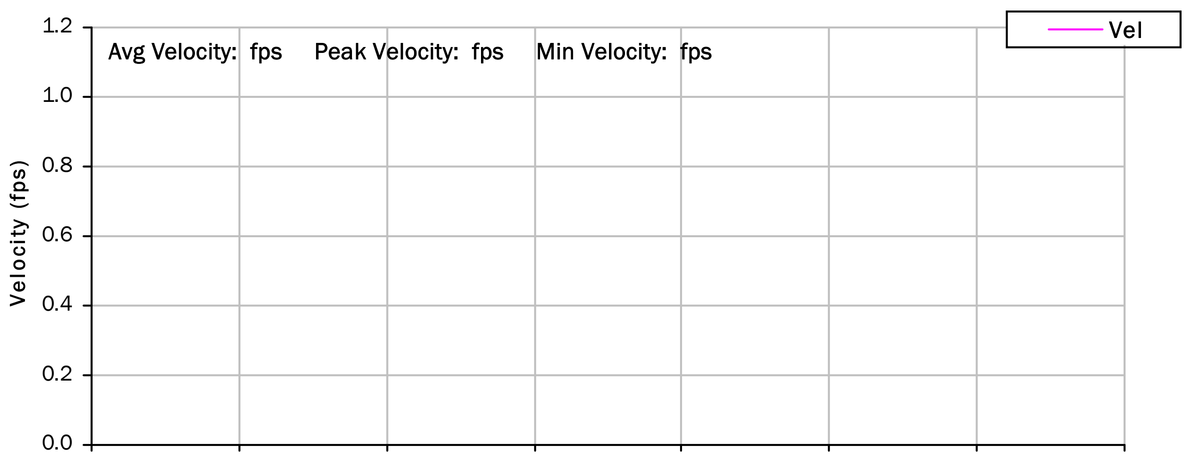


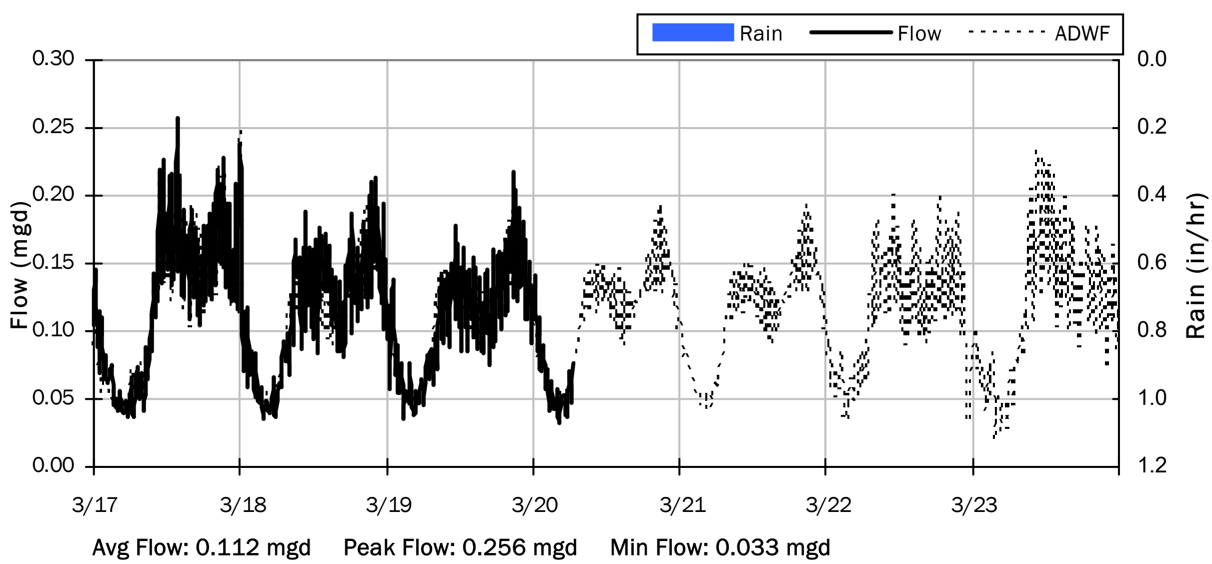




IRONWOOD PS
Weekly Level, Velocity and Flow Hydrographs
3/17/2024 to 3/24/2024







Monitoring Site: Stone 2 PS

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: 4420 Windsweep Rd, Bethel Island

Data Summary Report

Vicinity Map: Stone 2 PS

STONE 2 PS

Site Information

Location: 4420 Windsweep Rd, Bethel Island

Coordinates: 121.6146° W, 38.0150° N

ADWF: 0.106 mgd

Peak Measured Flow: 0.17 mgd

Satellite Map

Sanitary Map

Street View

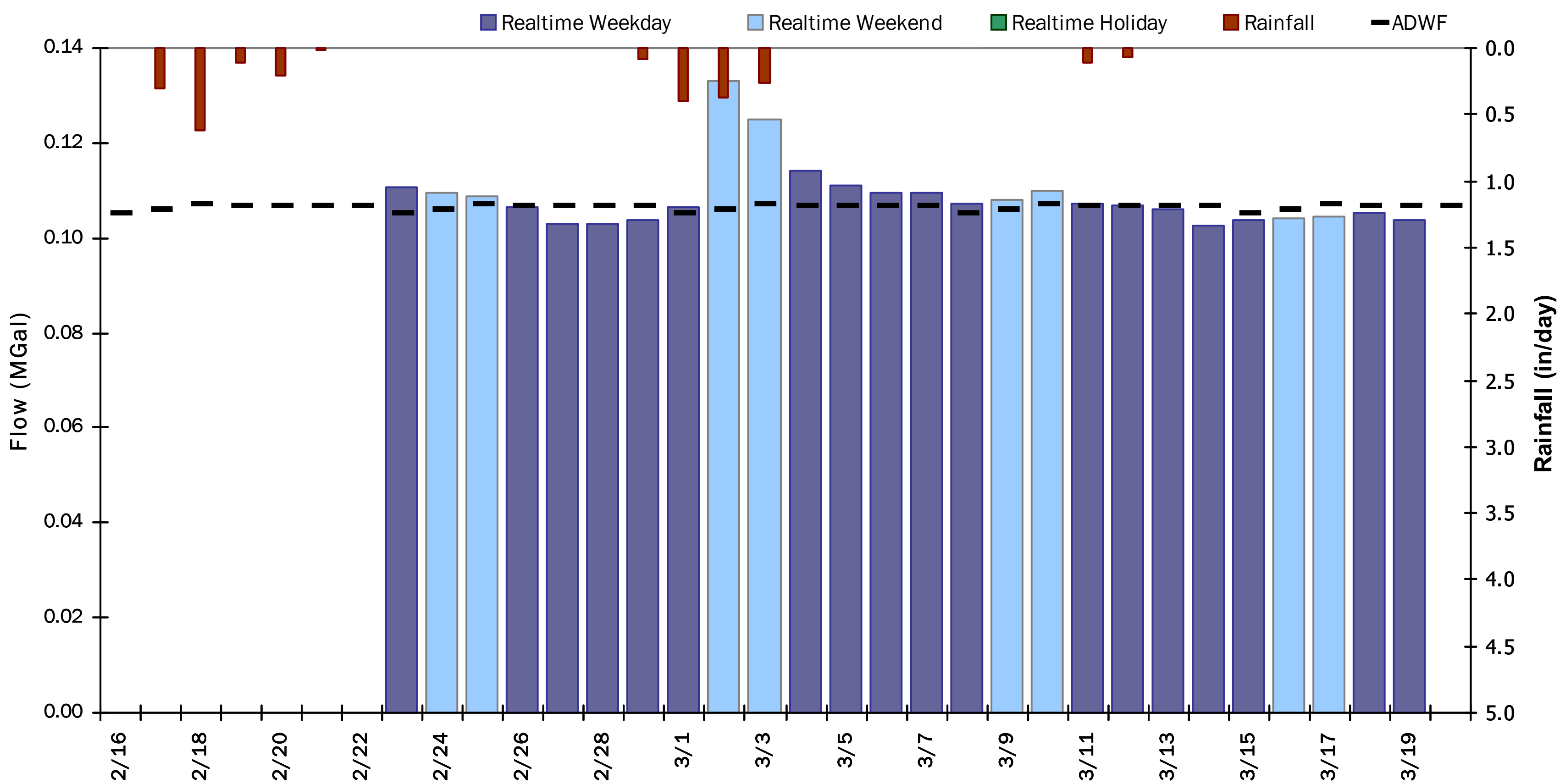

Control Panel

STONE 2 PS

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.115 MGal   Peak Daily Flow: 0.115 MGal   Min Daily Flow: 0.115 MGal

Total Rainfall: 2.53 inches

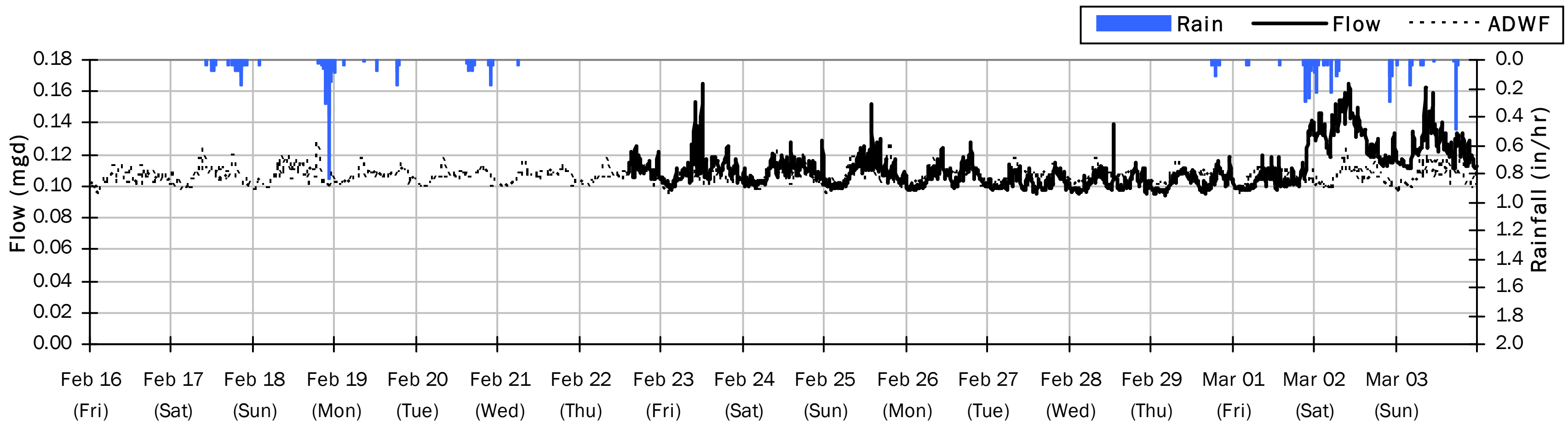


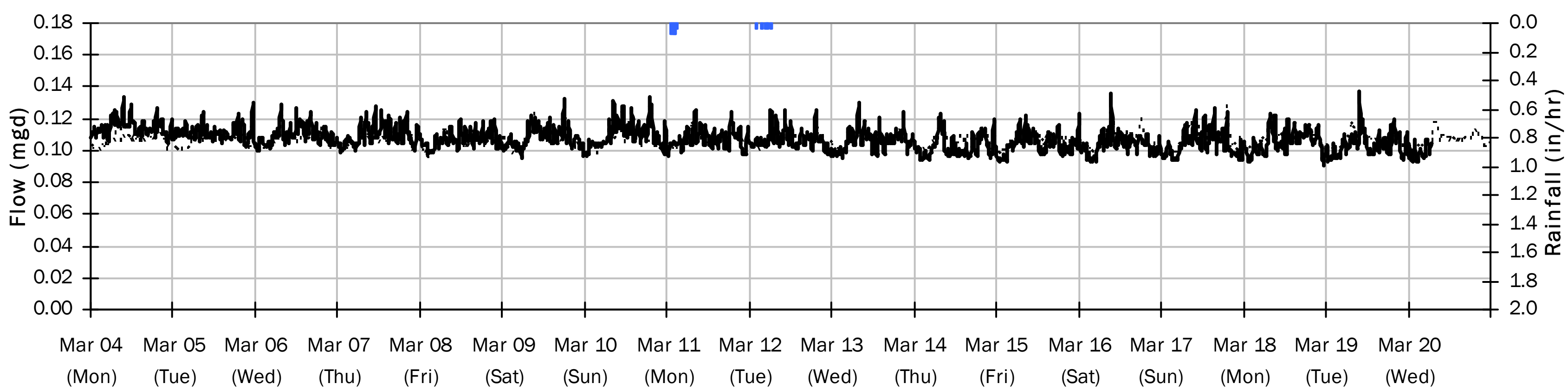
STONE 2 PS

Flow Summary: 2/16/2024 to 3/20/2024

Period Rainfall: 2.53 inches

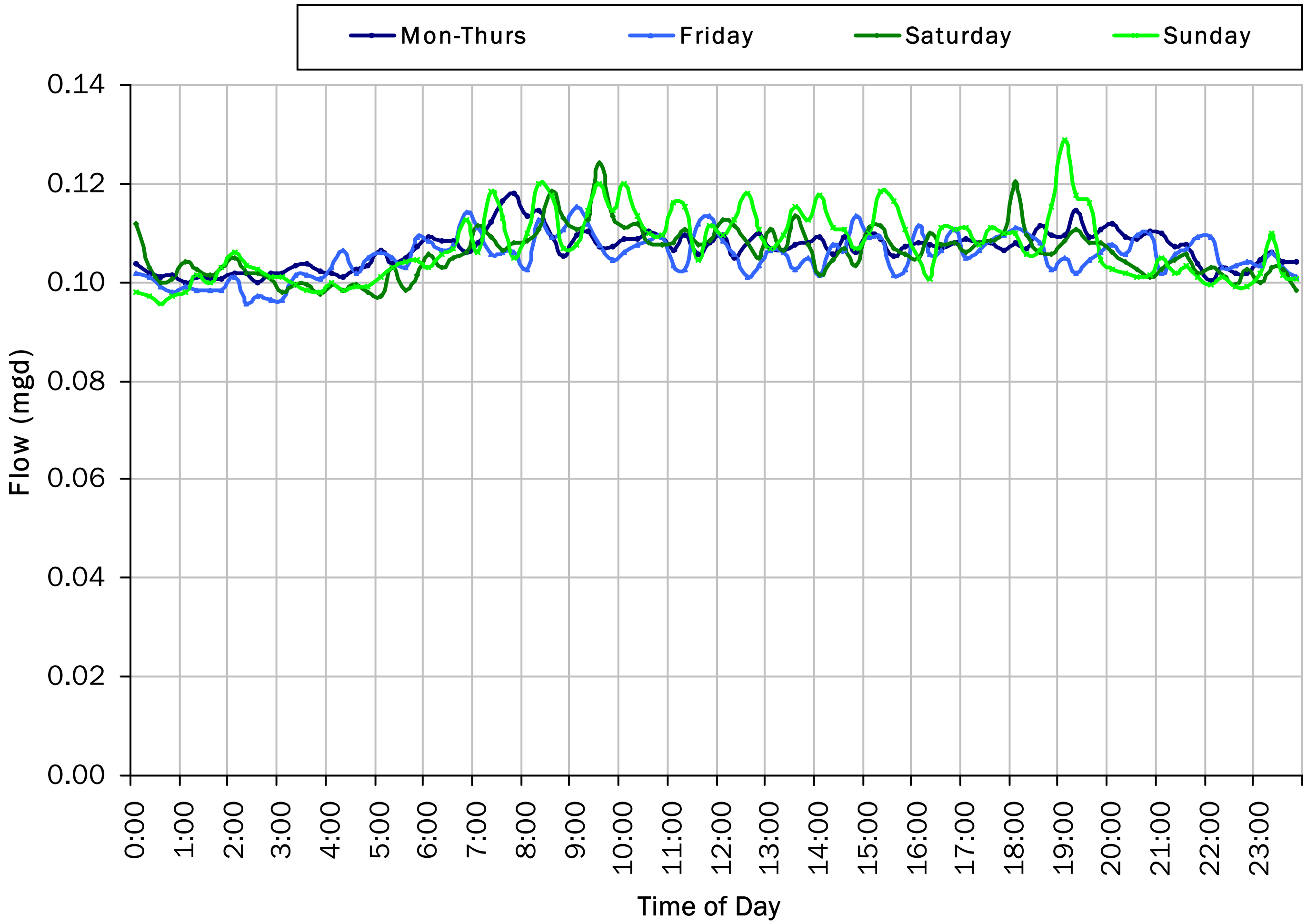
Period Avg Flow: 0.109 mgd Period Peak Flow: 0.165 mgd Period Min Flow: 0.091 mgd





STONE 2 PS

Average Dry Weather Flow Hydrographs



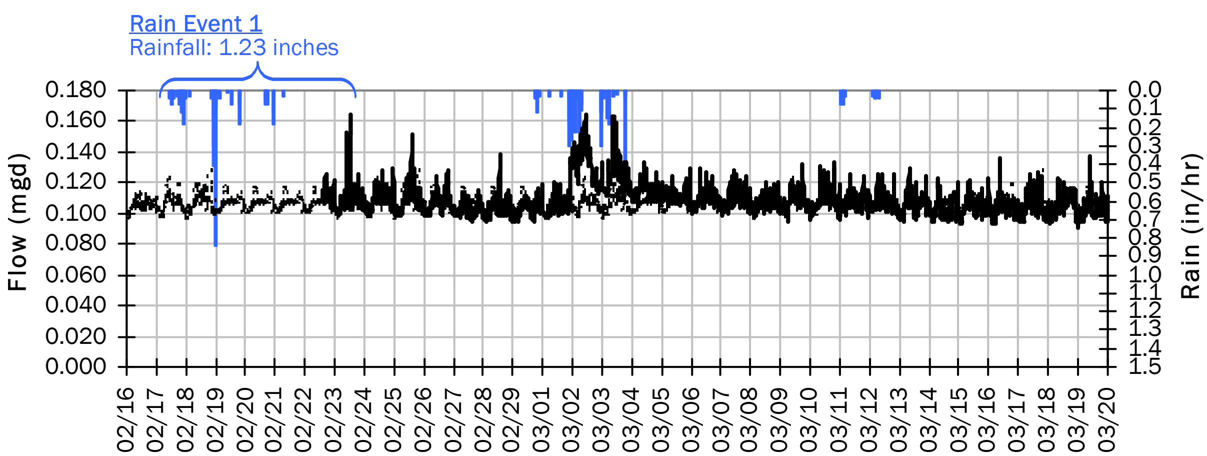
<u>ADWF:</u>

0.106 *mgd*

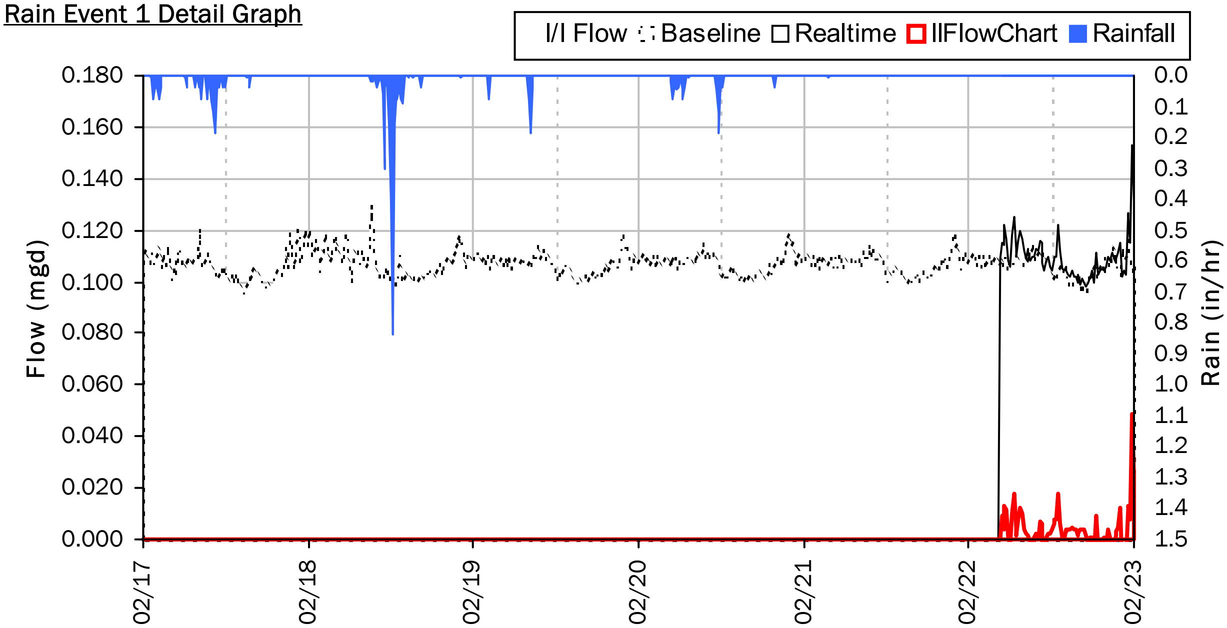
STONE 2 PS

I/I Summary: Rain Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Rain Event 1 Detail Graph



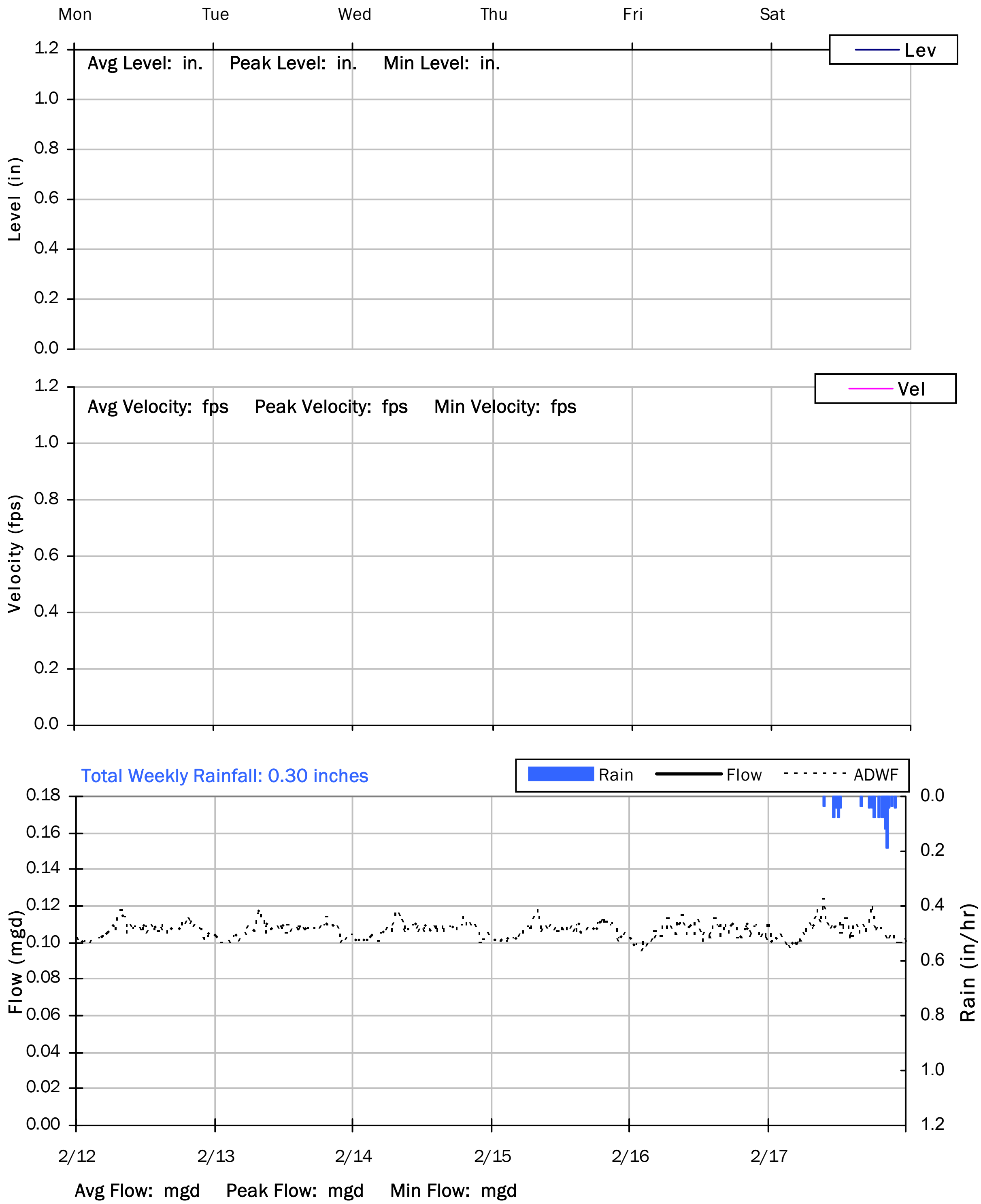
Storm Event I/I Analysis (Rain = 1.23 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.153 *mgd*	Peak I/I Rate:	0.049 *mgd*
PF:	1.44	Total I/I:	3,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

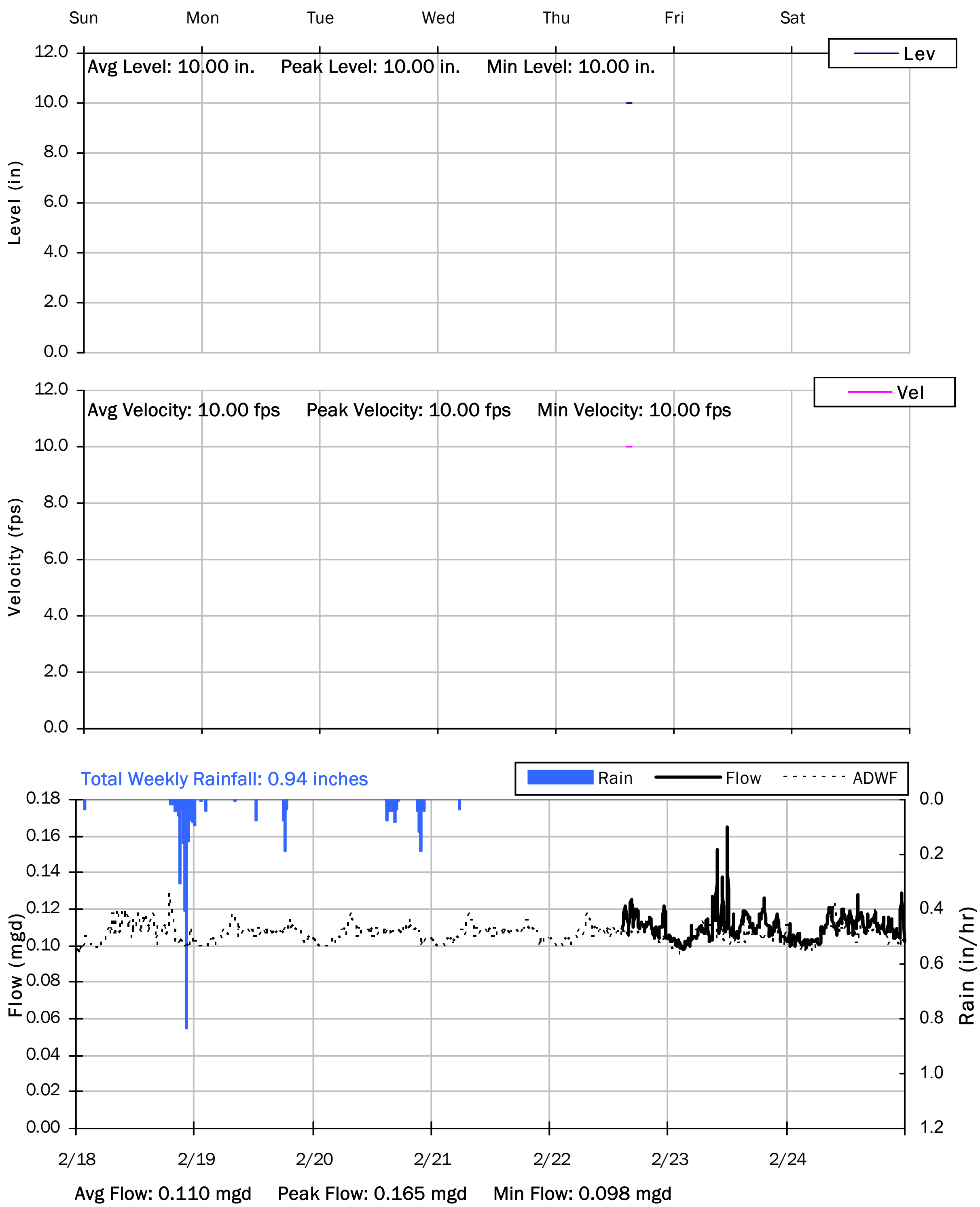
STONE 2 PS

Weekly Level, Velocity and Flow Hydrographs

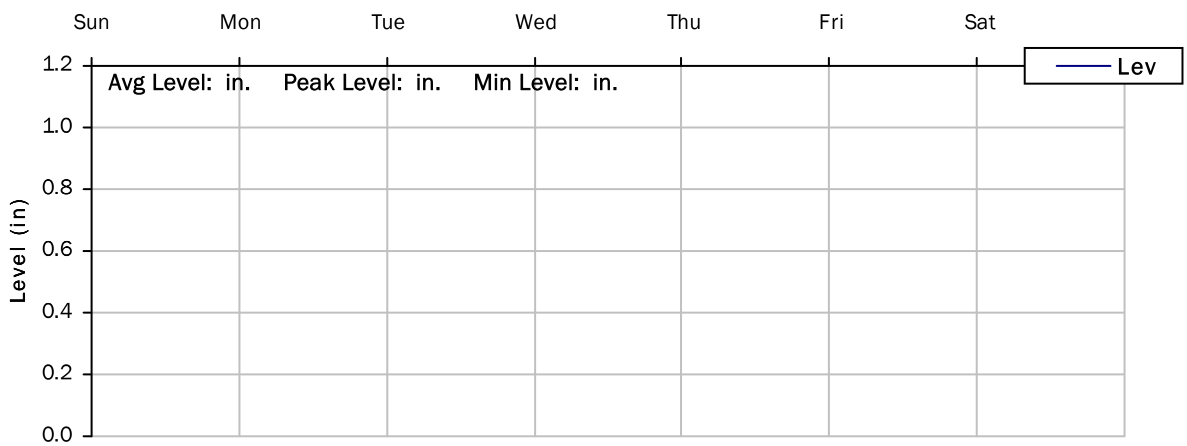
2/12/2024 to 2/18/2024

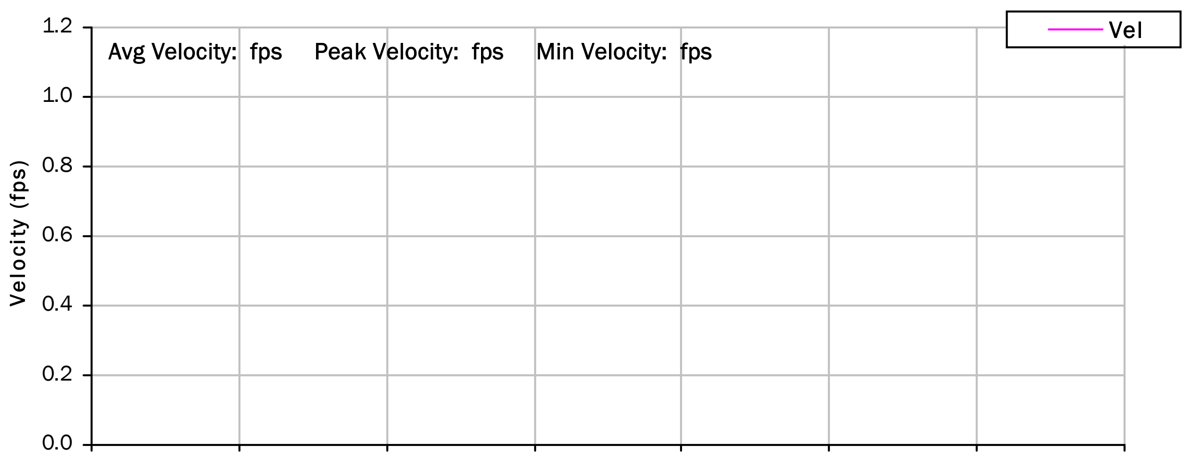


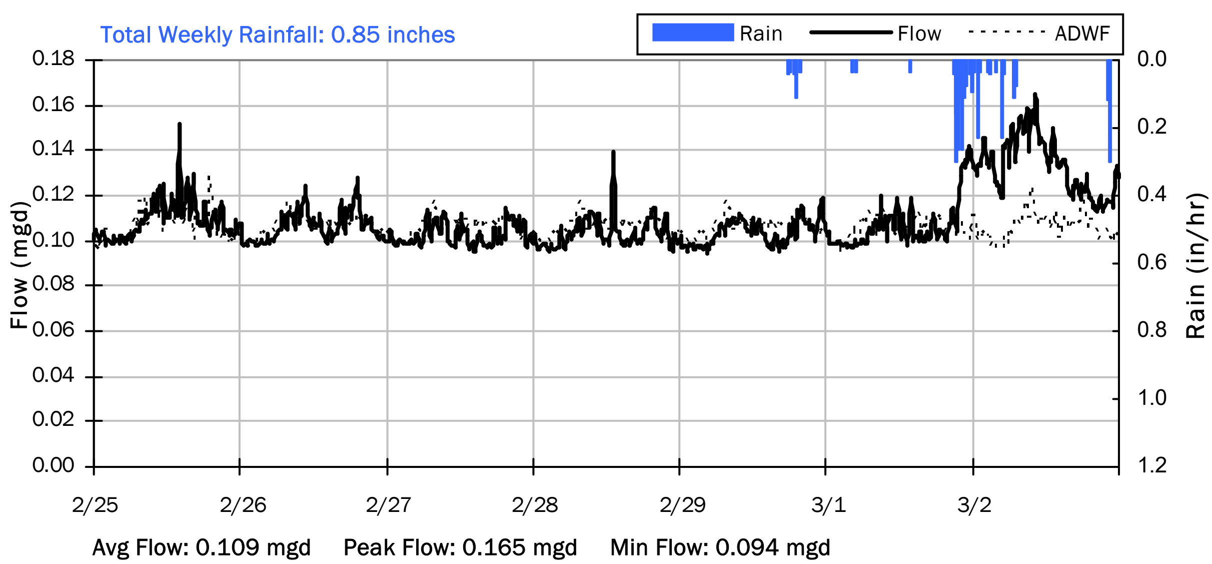
STONE 2 PS
Weekly Level, Velocity and Flow Hydrographs
2/18/2024 to 2/25/2024



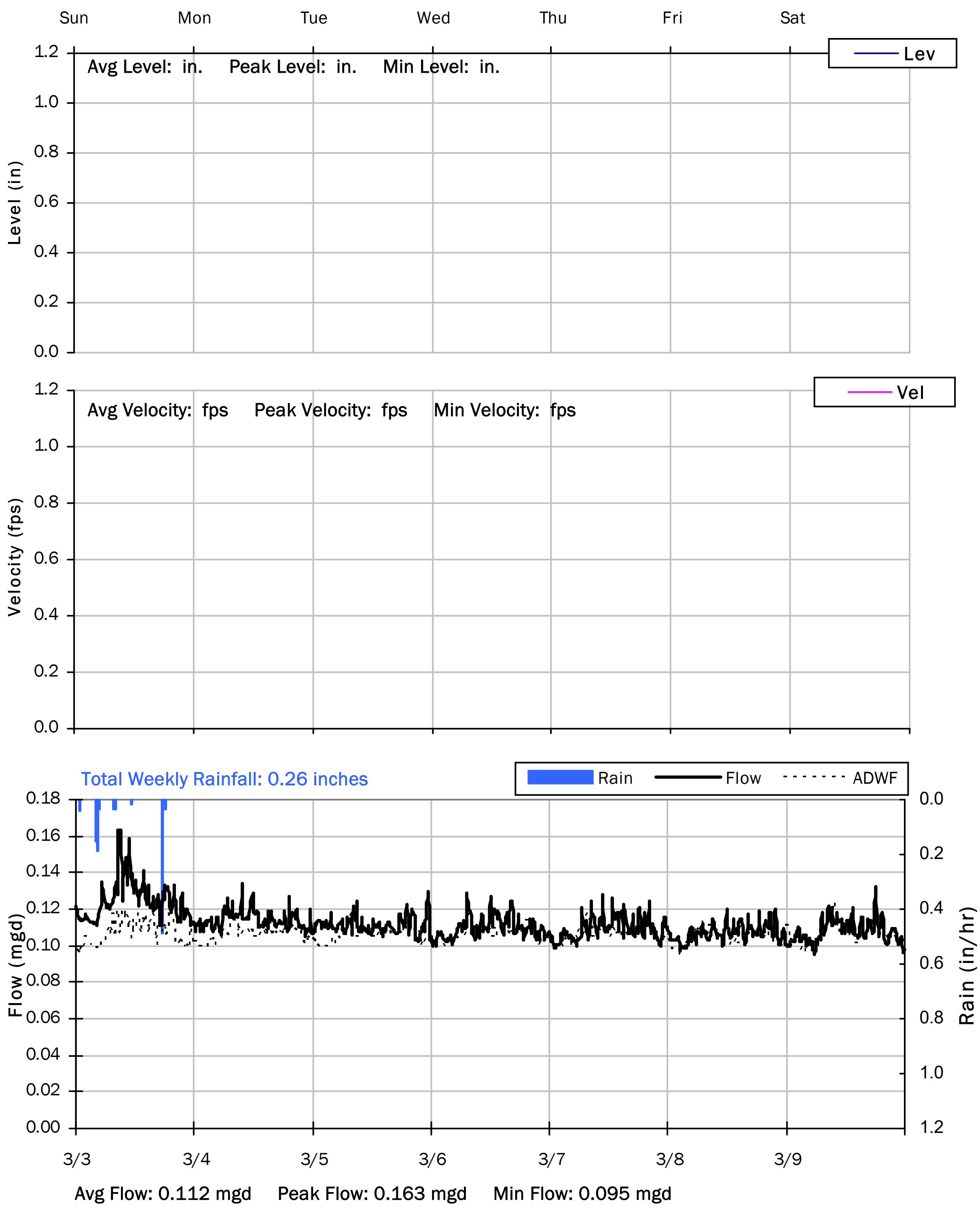
STONE 2 PS
Weekly Level, Velocity and Flow Hydrographs
2/25/2024 to 3/3/2024







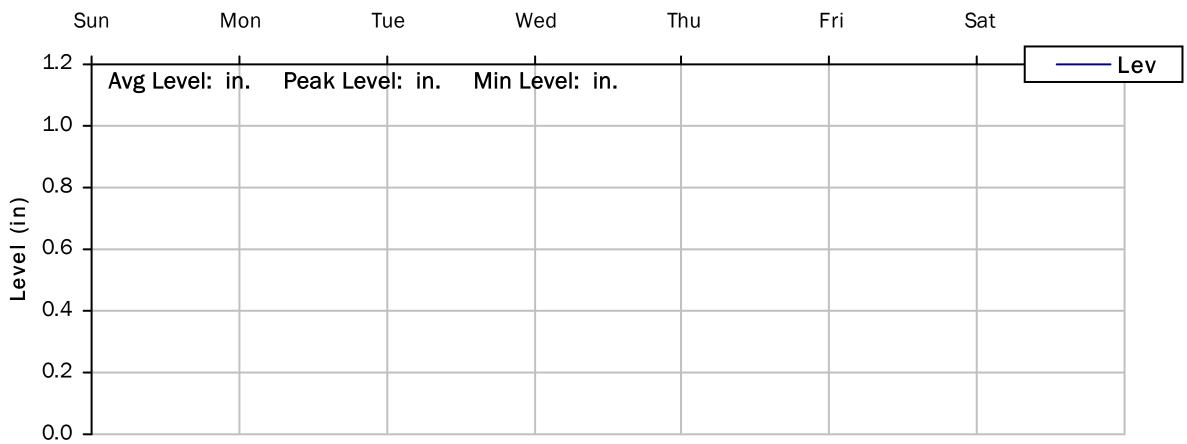
STONE 2 PS
Weekly Level, Velocity and Flow Hydrographs
3/3/2024 to 3/10/2024

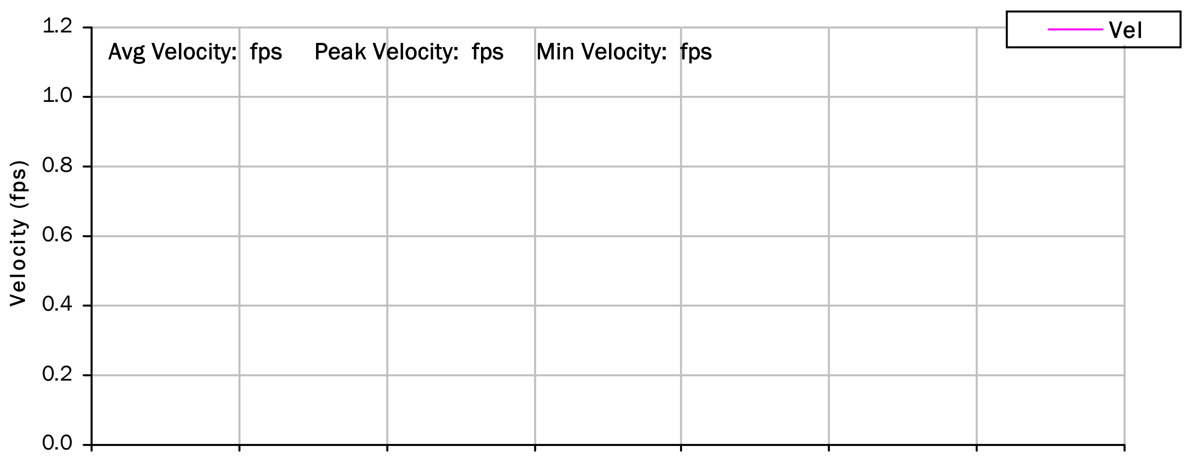


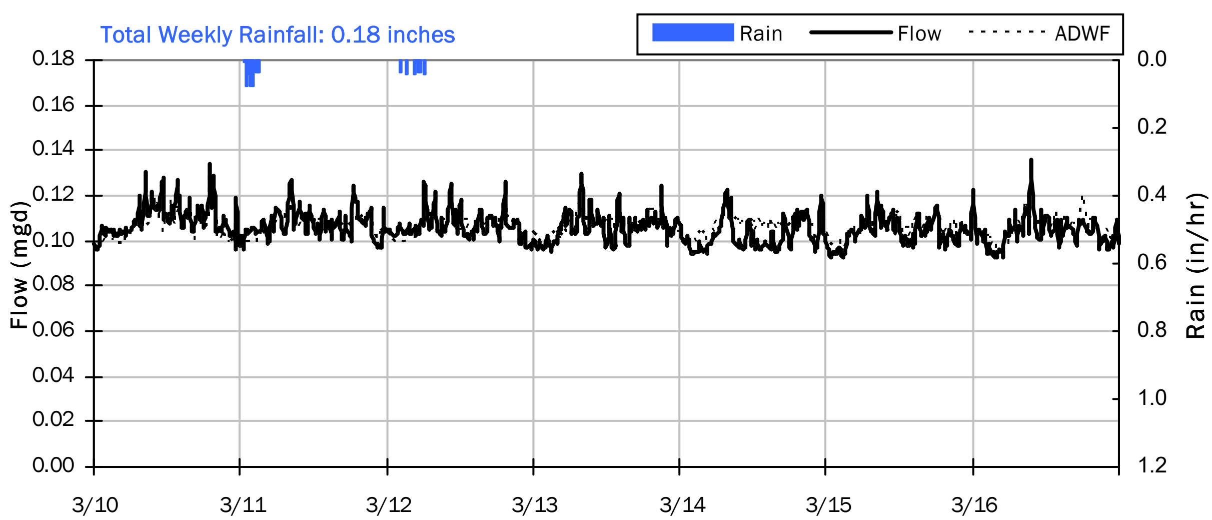
STONE 2 PS

Weekly Level, Velocity and Flow Hydrographs

3/10/2024 to 3/17/2024

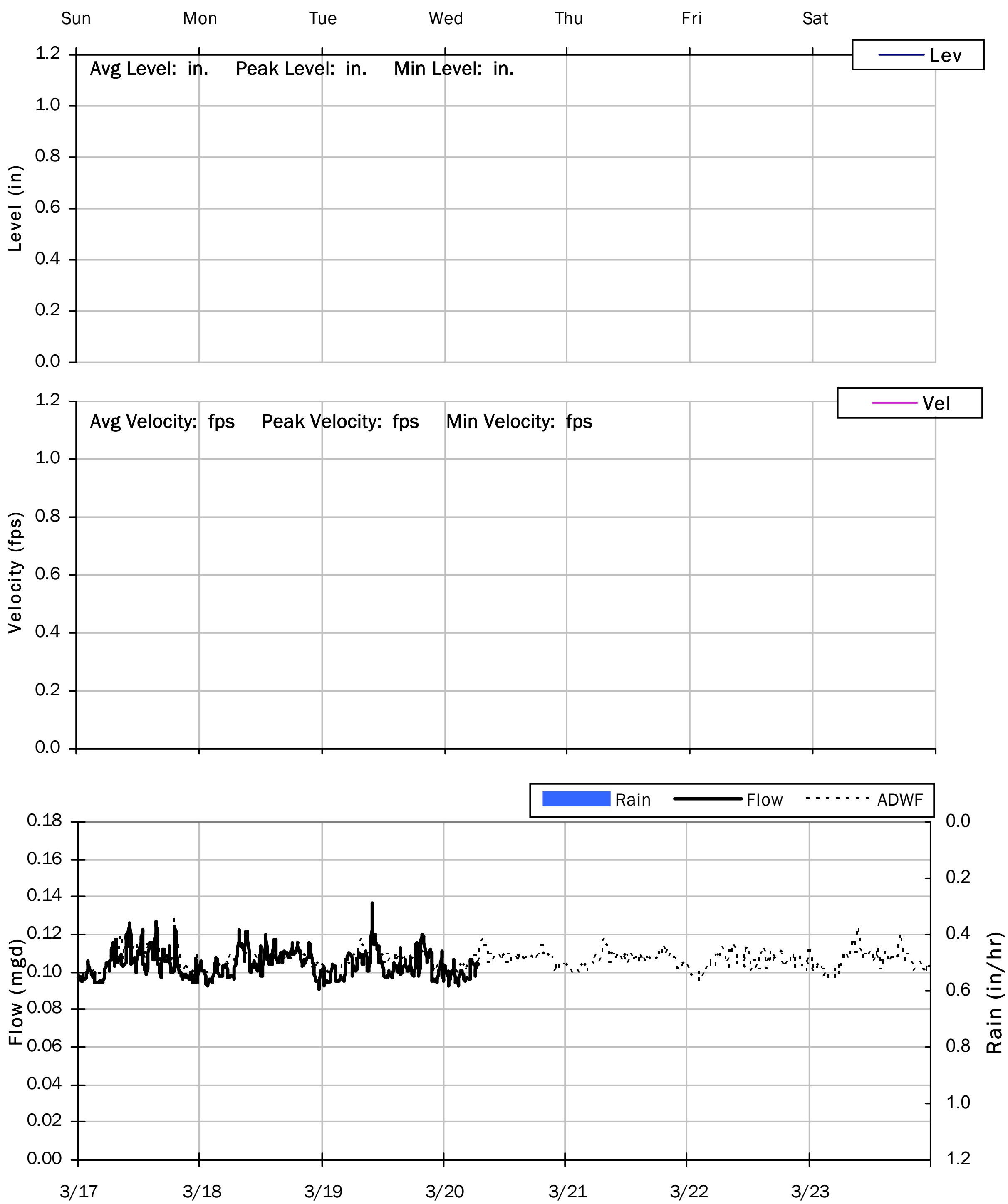






Avg Flow: 0.106 mgd Peak Flow: 0.136 mgd Min Flow: 0.093 mgd

STONE 2 PS
Weekly Level, Velocity and Flow Hydrographs
3/17/2024 to 3/24/2024



Avg Flow: 0.104 mgd Peak Flow: 0.137 mgd Min Flow: 0.091 mgd

Monitoring Site: Summer Lakes 1 PS

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: 3200 E Cypress Rd, Oakley

Data Summary Report

Vicinity Map: Summer Lakes 1 PS

SUMMER LAKES 1 PS

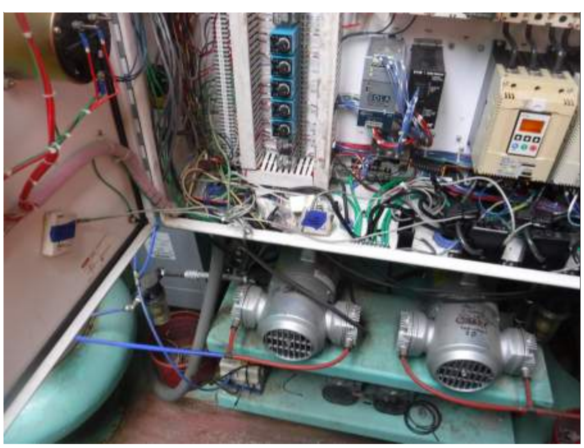
Site Information

Location:	3200 E Cypress Rd, Oakley
Coordinates:	121.6378° W, 37.9921° N
ADWF:	0.072 mgd
Peak Measured Flow:	0.35 mgd

Satellite Map

Sanitary Map

Street View

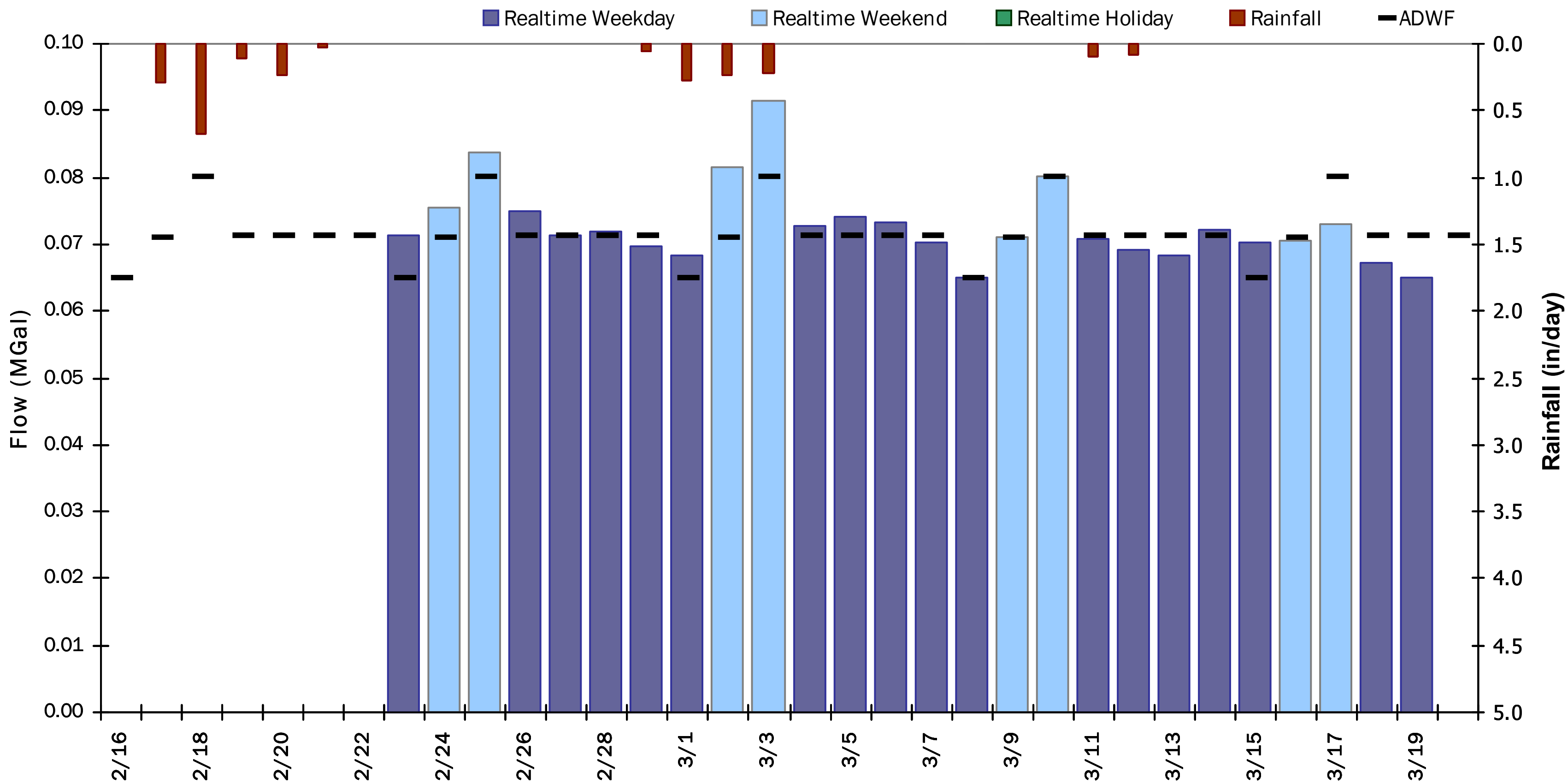

Control Panel

SUMMER LAKES 1 PS

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.079 MGal   Peak Daily Flow: 0.079 MGal   Min Daily Flow: 0.079 MGal

Total Rainfall: 2.30 inches

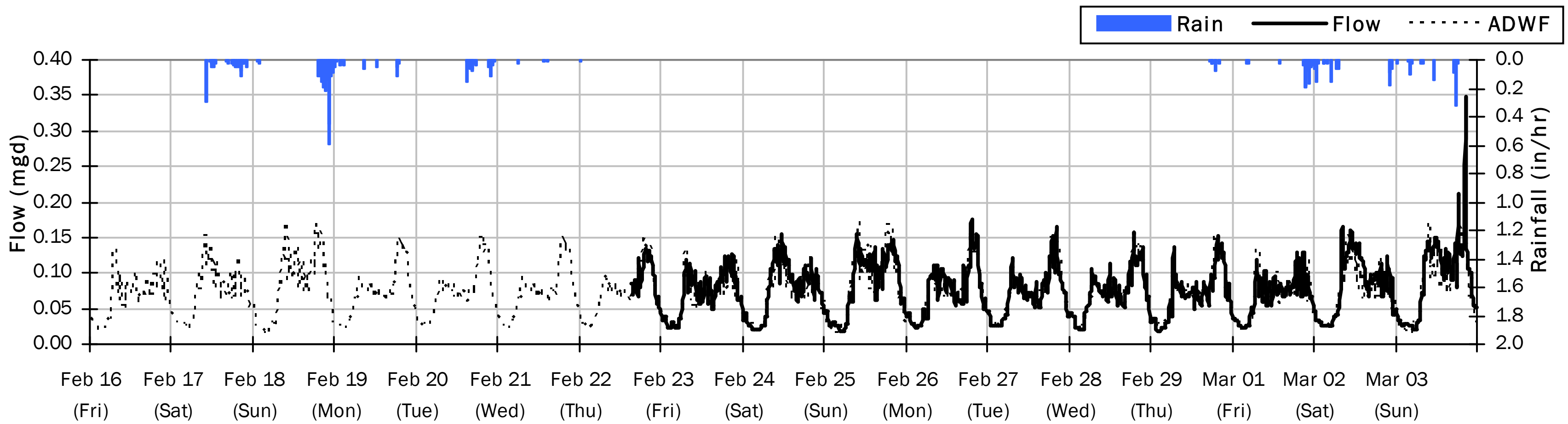


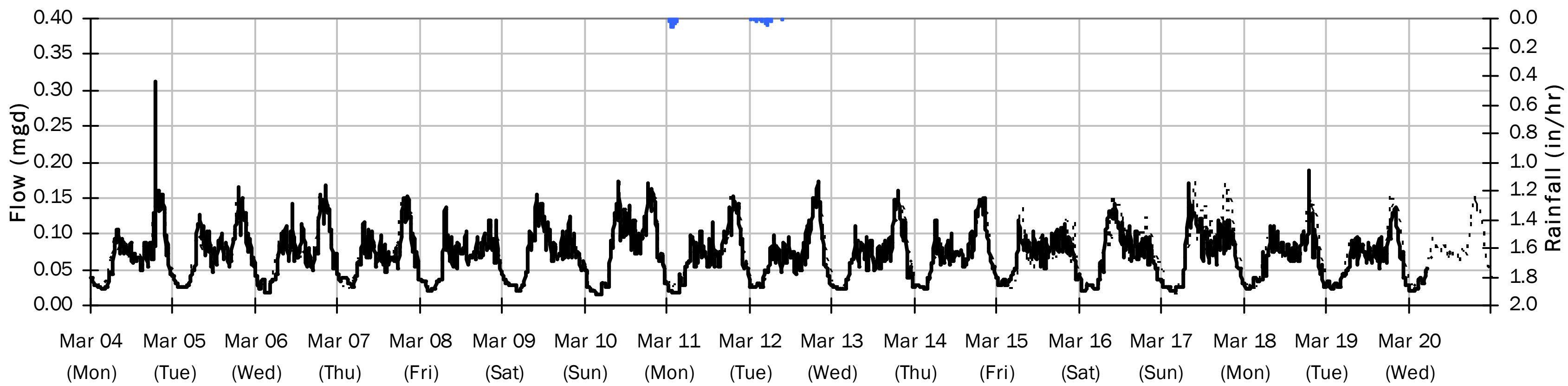
SUMMER LAKES 1 PS

Flow Summary: 2/16/2024 to 3/20/2024

Period Rainfall: 2.30 inches

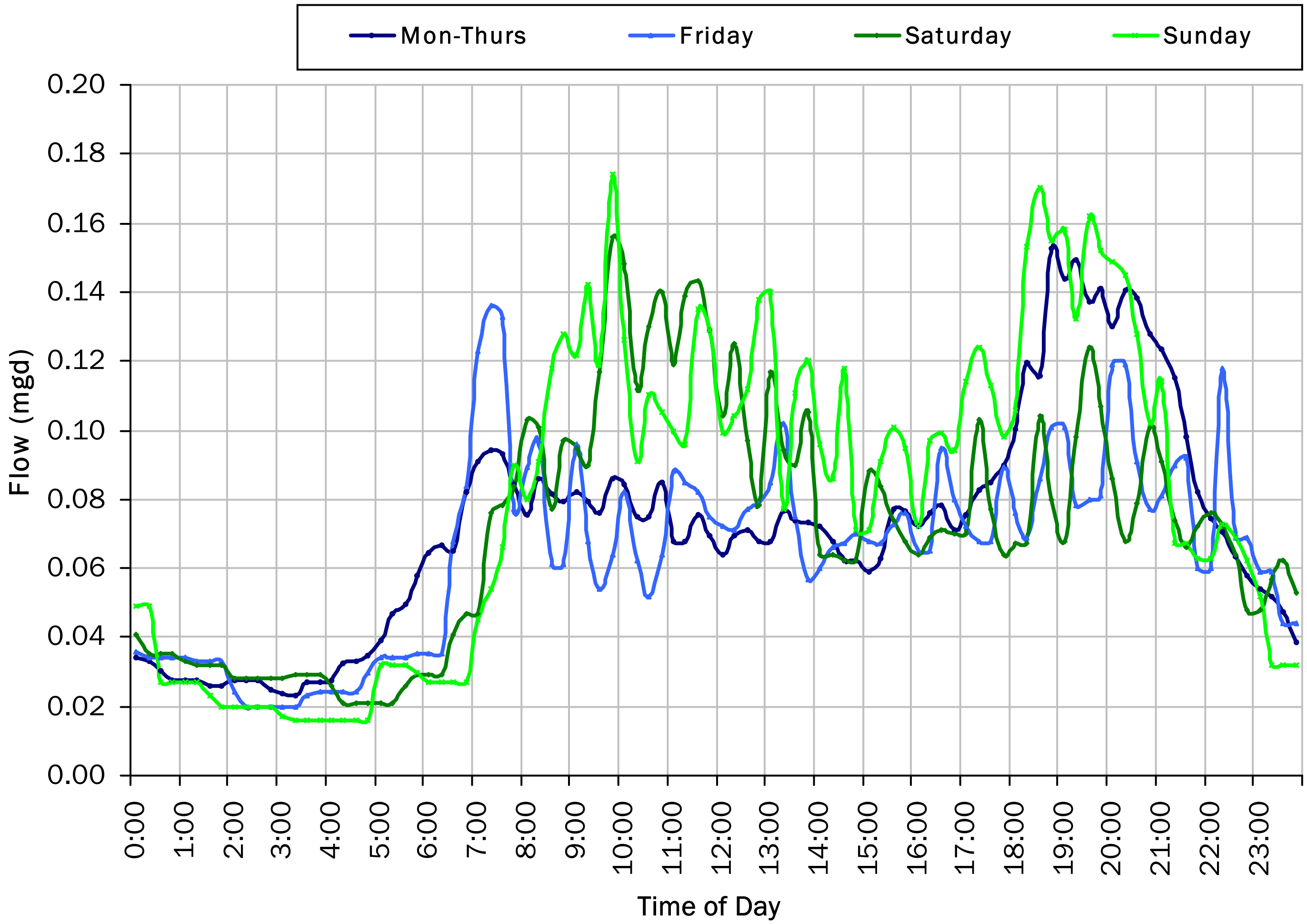
Period Avg Flow: 0.073 mgd Period Peak Flow: 0.348 mgd Period Min Flow: 0.016 mgd





SUMMER LAKES 1 PS

Average Dry Weather Flow Hydrographs

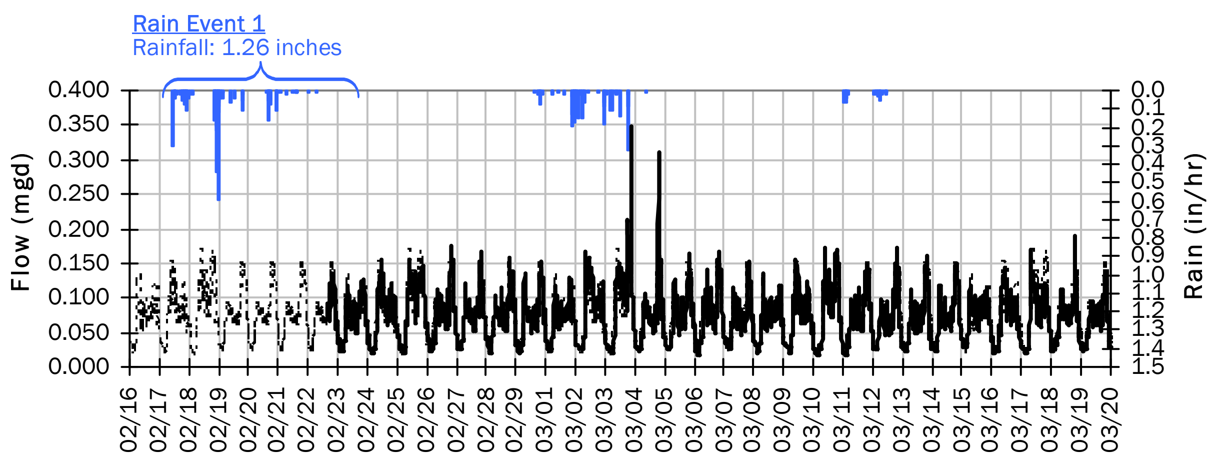


ADWF:

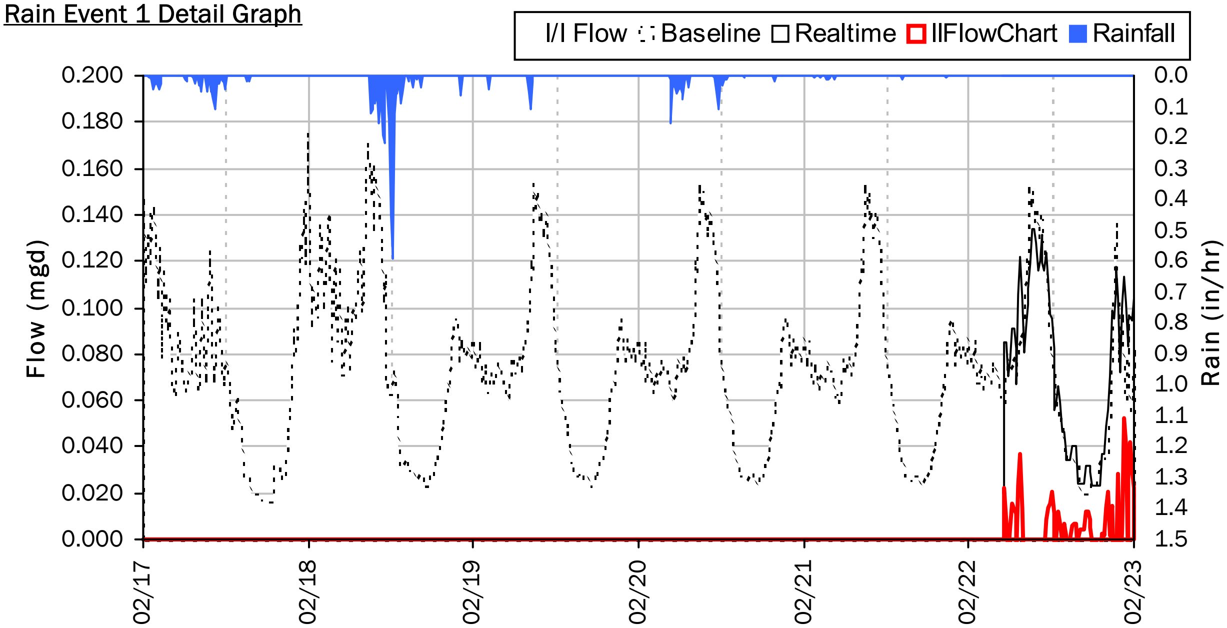
0.072 *mgd*

SUMMER LAKES 1 PS
I/I Summary: Rain Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Rain Event 1 Detail Graph



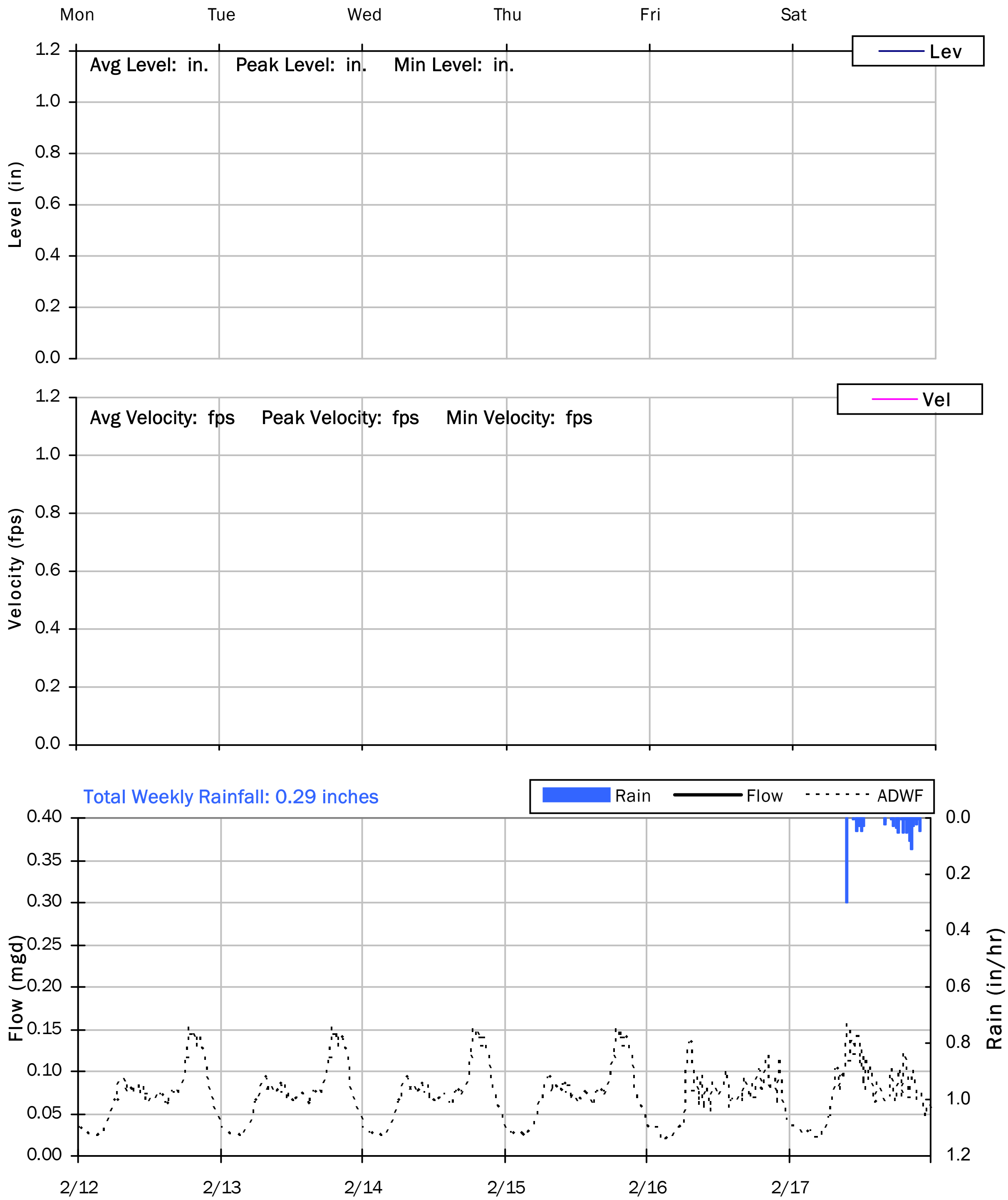
Storm Event I/I Analysis (Rain = 1.26 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.134 *mgd*	Peak I/I Rate:	0.052 *mgd*
PF:	1.87	Total I/I:	3,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

SUMMER LAKES 1 PS

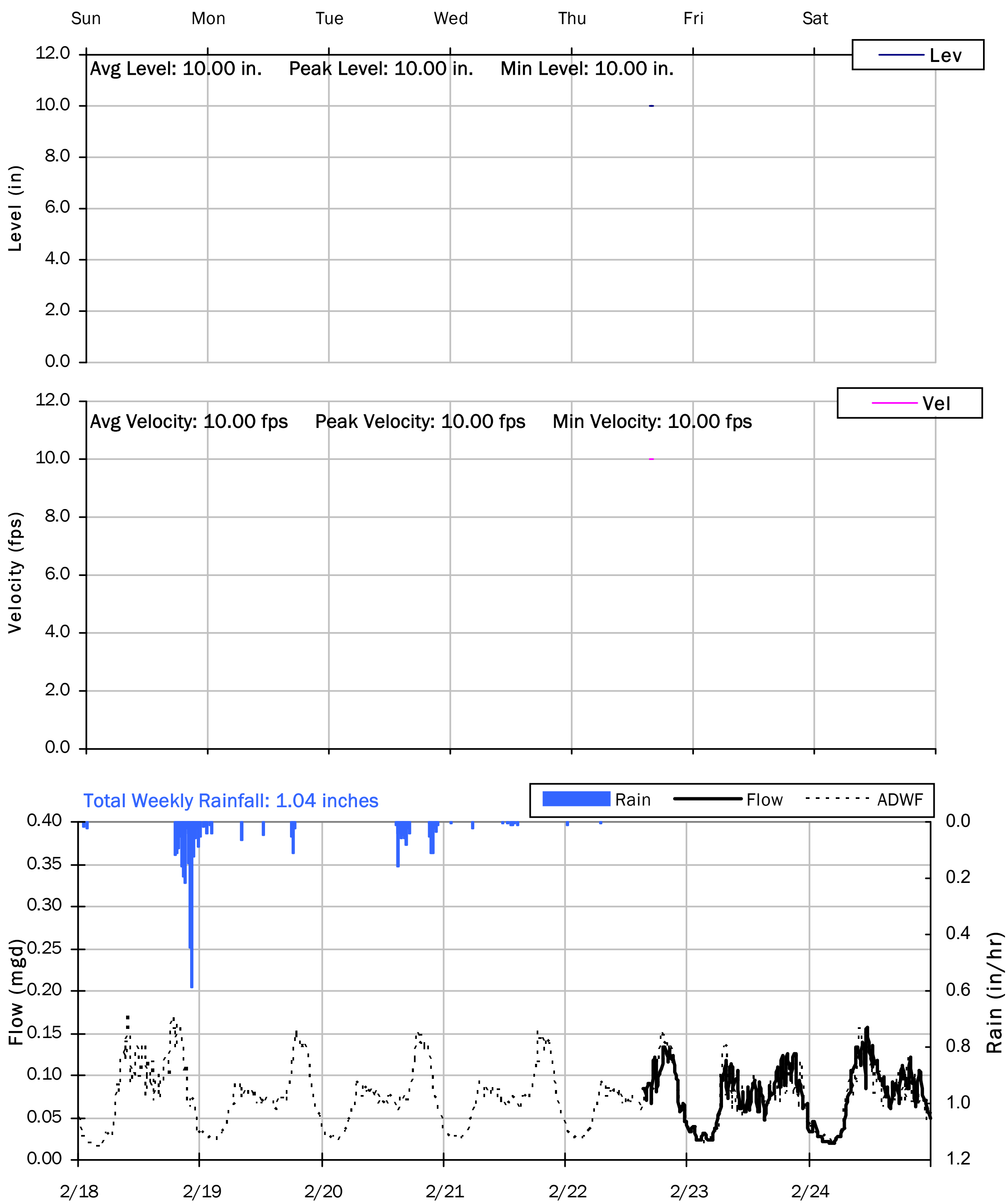
Weekly Level, Velocity and Flow Hydrographs

2/12/2024 to 2/18/2024



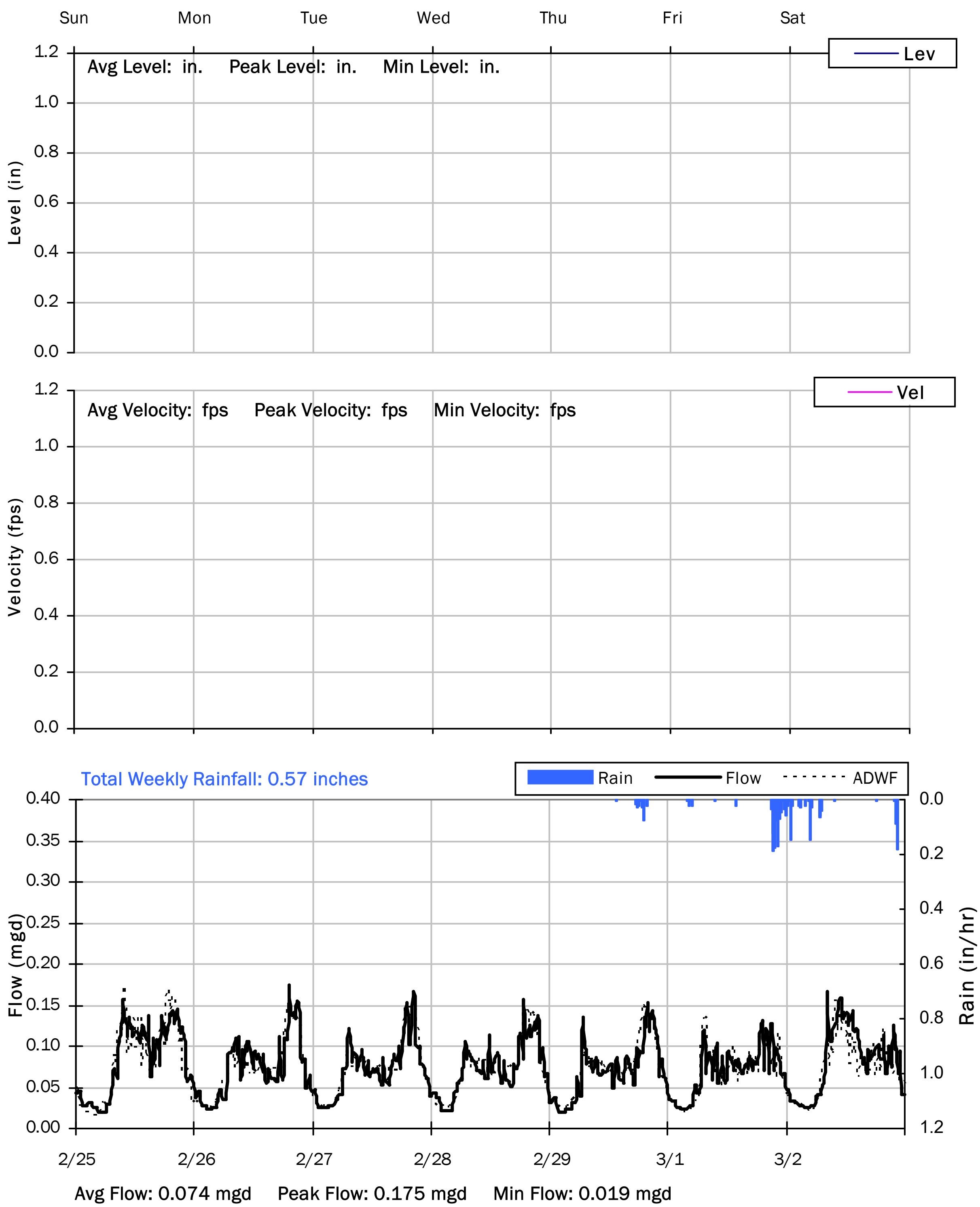
Avg Flow: mgd Peak Flow: mgd Min Flow: mgd

SUMMER LAKES 1 PS
Weekly Level, Velocity and Flow Hydrographs
2/18/2024 to 2/25/2024



Avg Flow: 0.077 mgd Peak Flow: 0.156 mgd Min Flow: 0.020 mgd

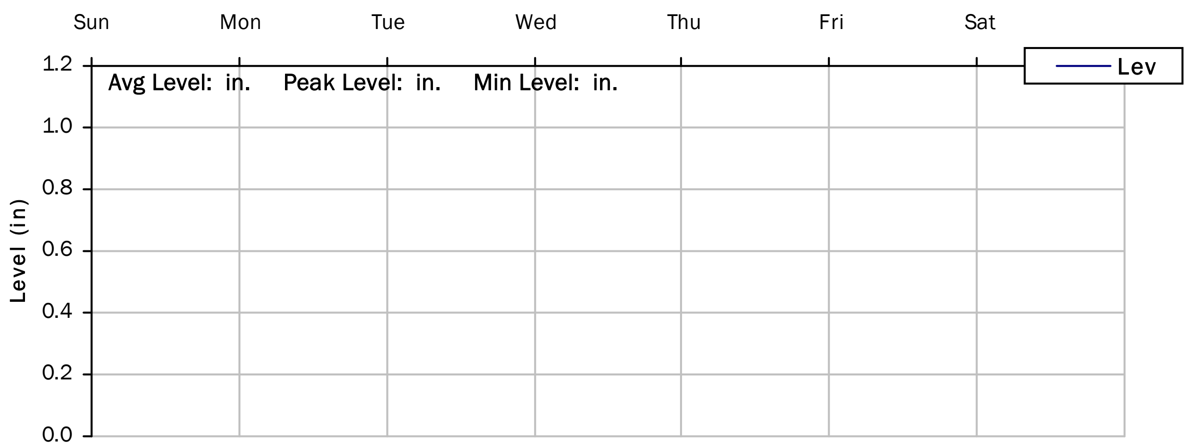
SUMMER LAKES 1 PS
Weekly Level, Velocity and Flow Hydrographs
2/25/2024 to 3/3/2024

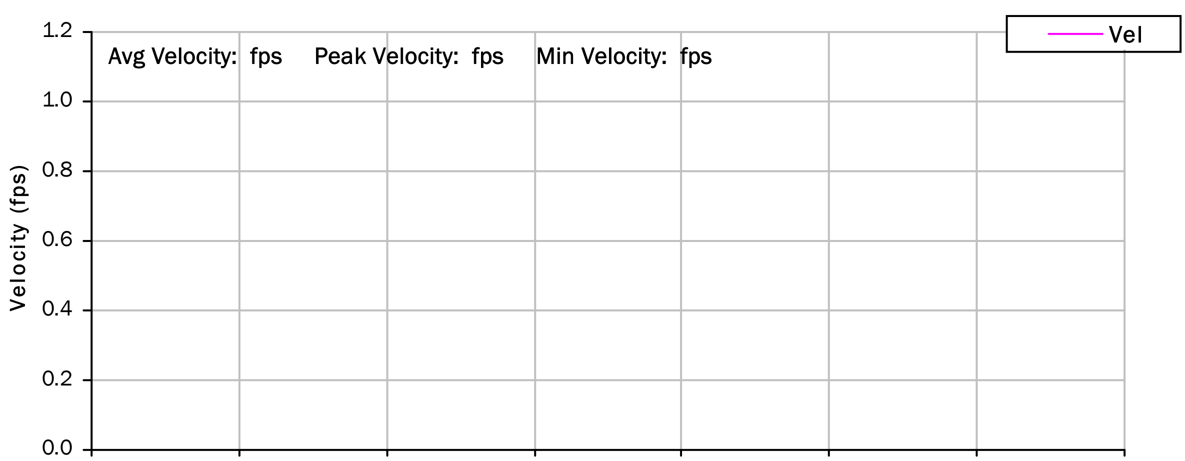


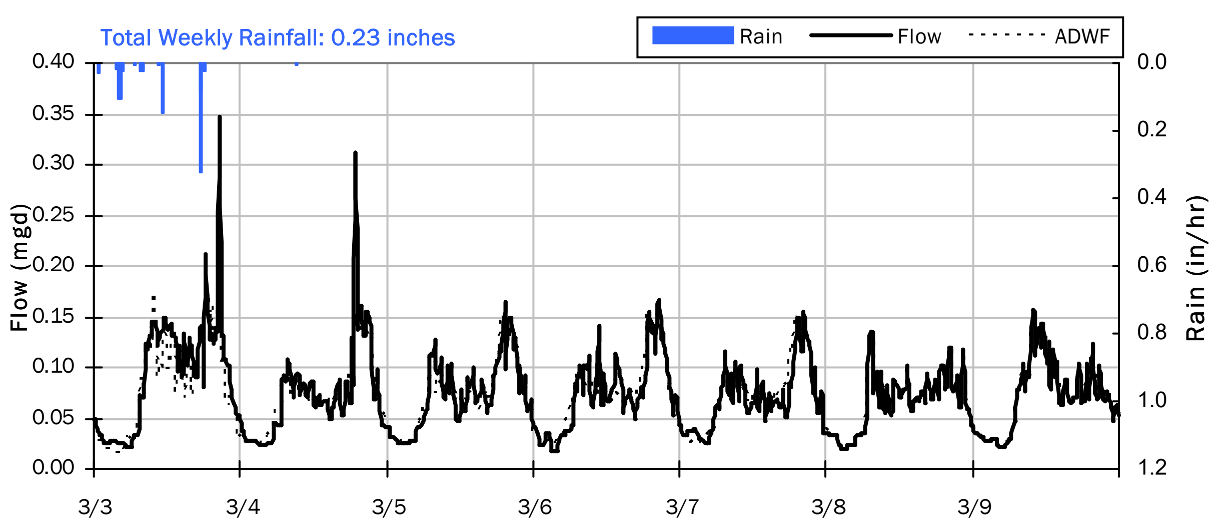
SUMMER LAKES 1 PS

Weekly Level, Velocity and Flow Hydrographs

3/3/2024 to 3/10/2024





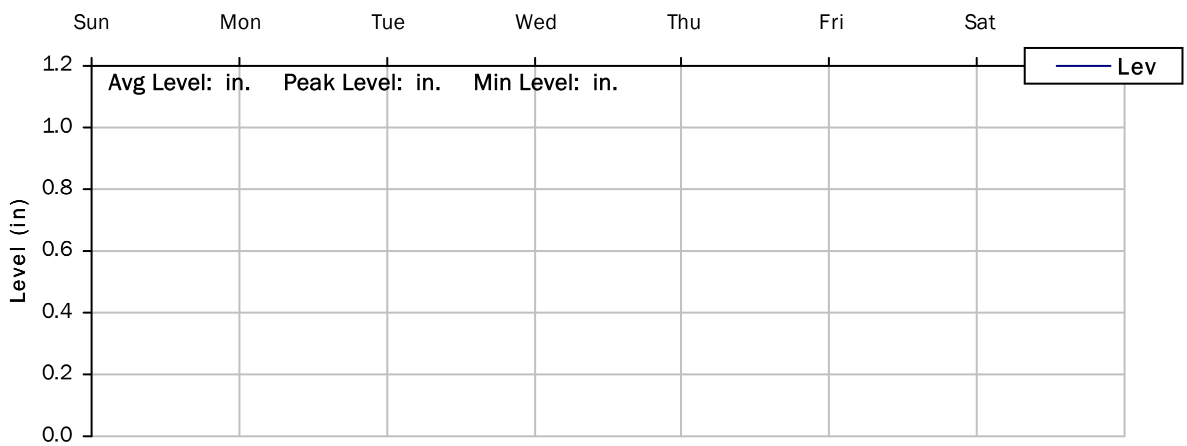


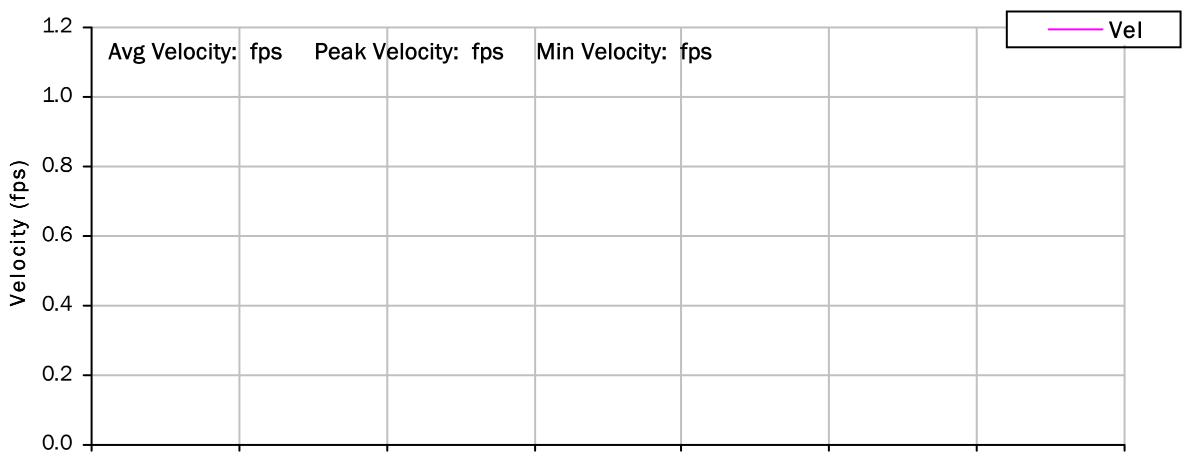
Avg Flow: 0.074 mgd Peak Flow: 0.348 mgd Min Flow: 0.018 mgd

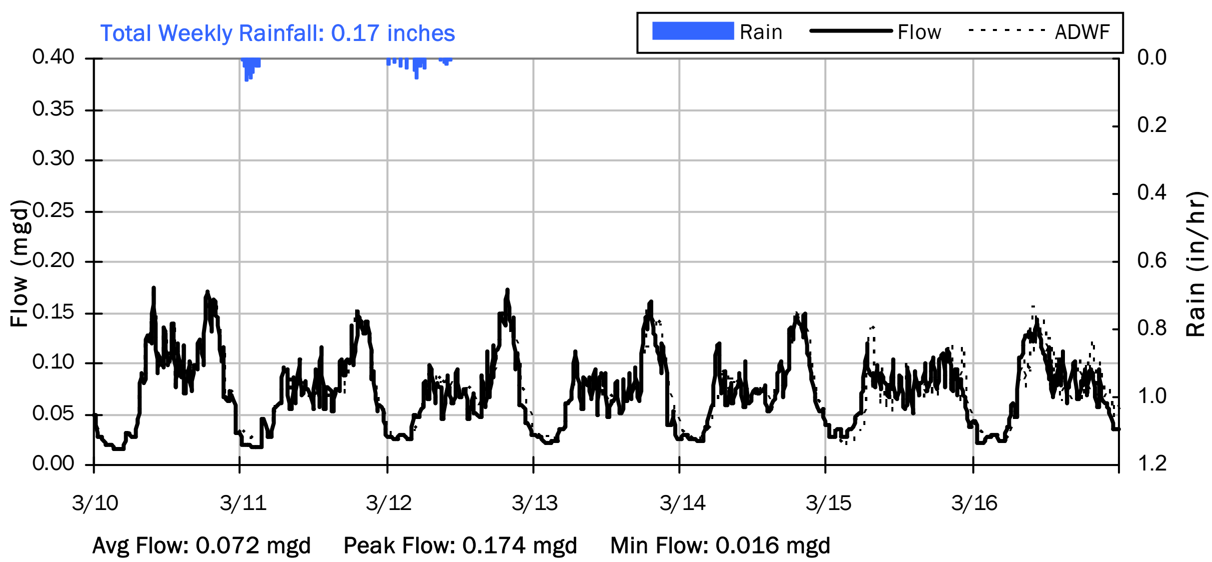
SUMMER LAKES 1 PS

Weekly Level, Velocity and Flow Hydrographs

3/10/2024 to 3/17/2024



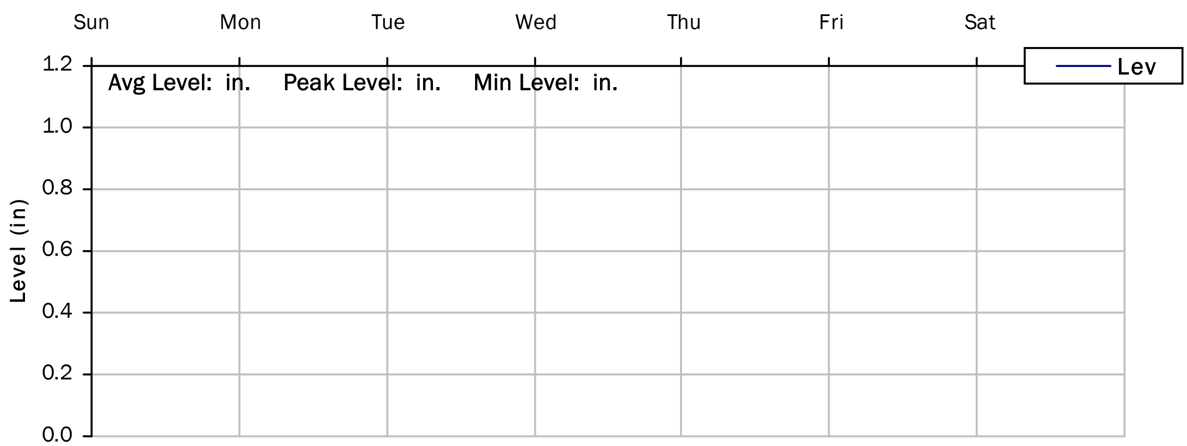


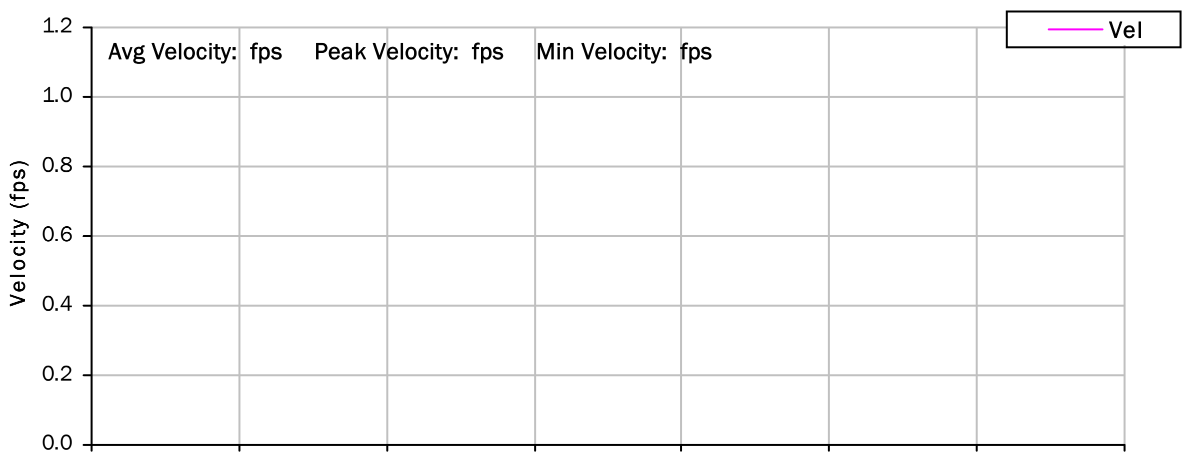


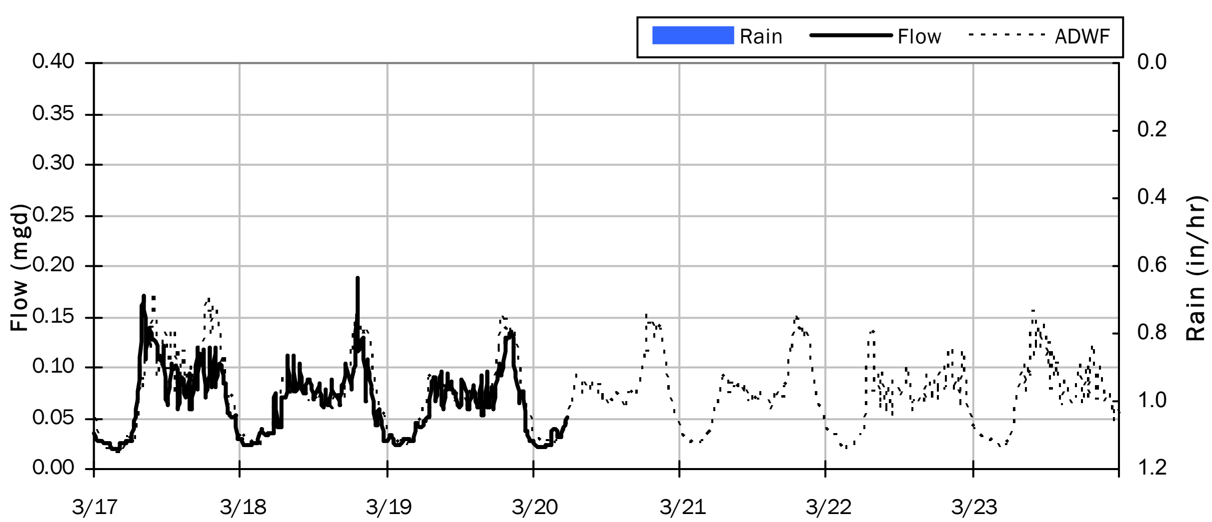
SUMMER LAKES 1 PS

Weekly Level, Velocity and Flow Hydrographs

3/17/2024 to 3/24/2024







Avg Flow: 0.066 mgd Peak Flow: 0.189 mgd Min Flow: 0.020 mgd

Monitoring Site: Taylor 1 PS

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: Southwest of 2383 Taylor Rd, Bethel Island

Data Summary Report

Vicinity Map: Taylor 1 PS

TAYLOR 1 PS

Site Information

Location: Southwest of 2383 Taylor Rd, Bethel Island

Coordinates: 121.6530° W, 38.0185° N

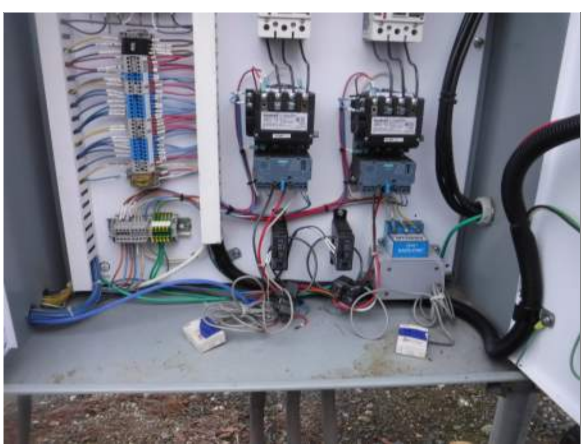
ADWF: 0.111 mgd

Peak Measured Flow: 0.19 mgd

Satellite Map

Sanitary Map

Street View

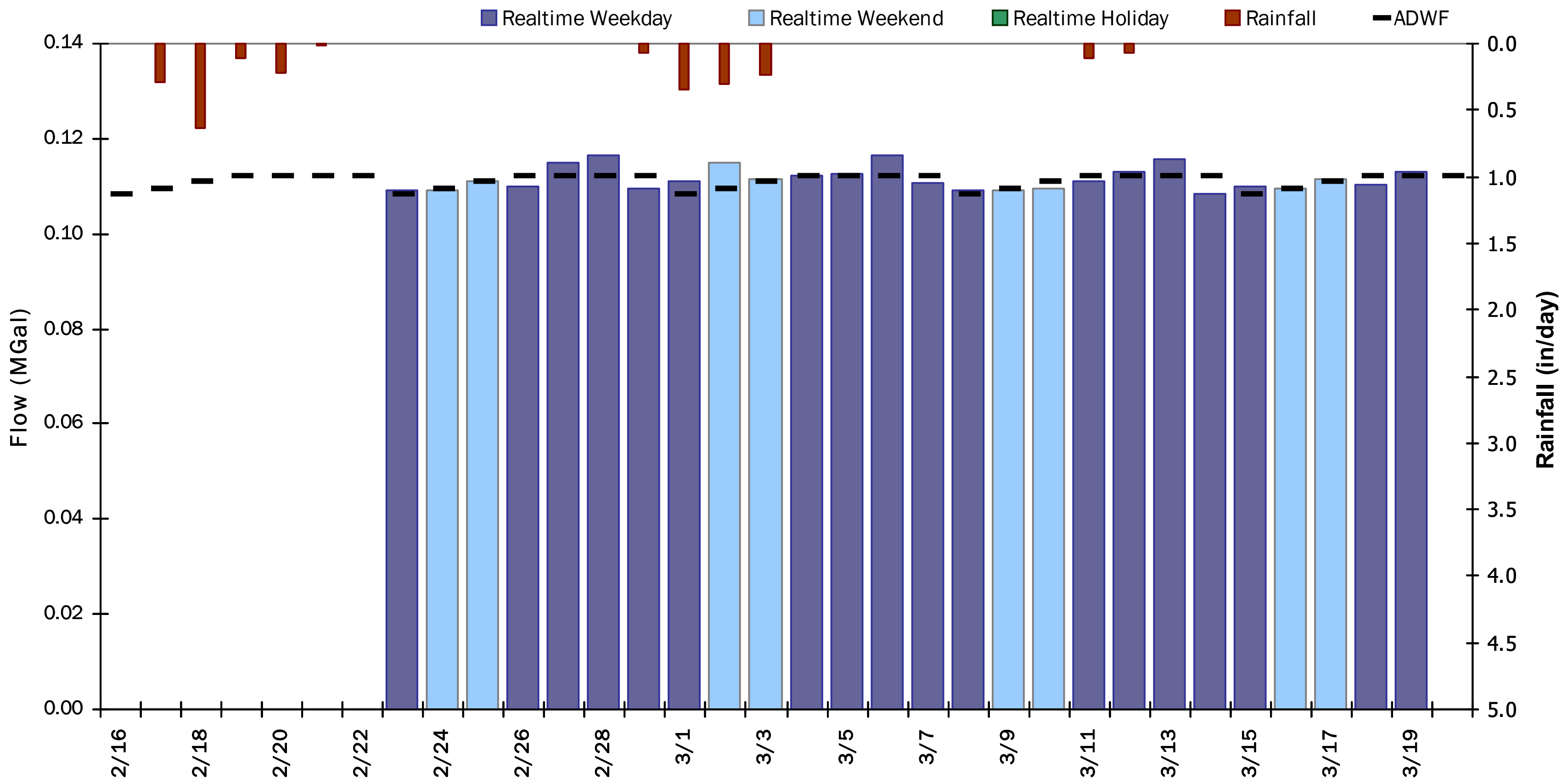

Control Panel

TAYLOR 1 PS

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.108 MGal   Peak Daily Flow: 0.108 MGal   Min Daily Flow: 0.108 MGal

Total Rainfall: 2.41 inches

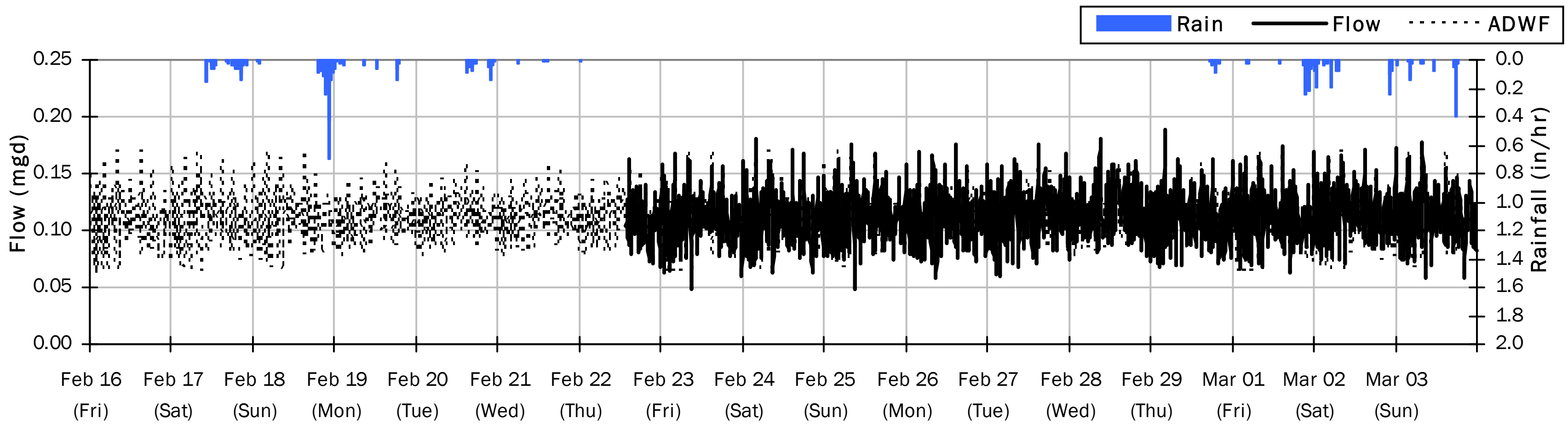


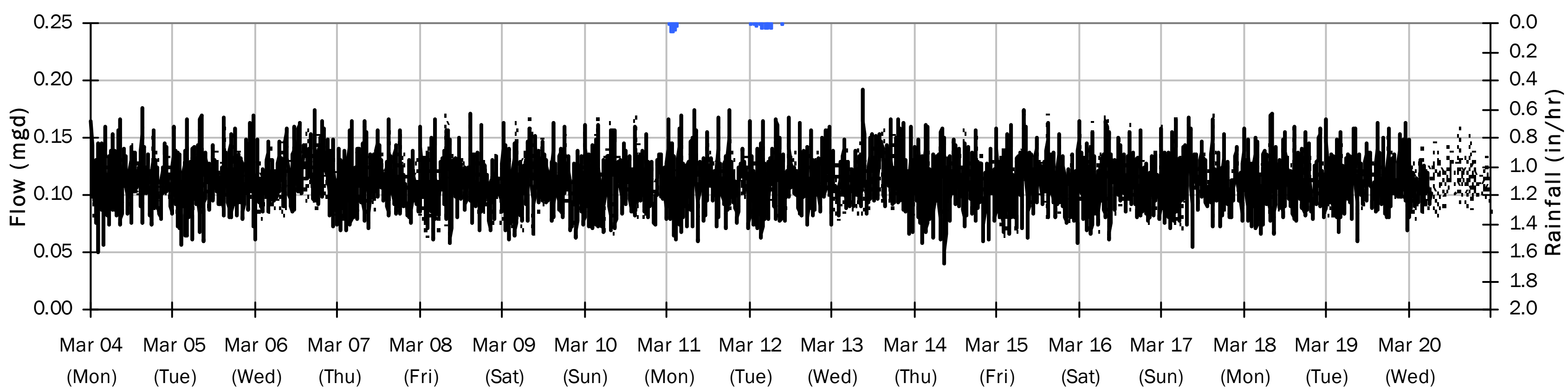
TAYLOR 1 PS

Flow Summary: 2/16/2024 to 3/20/2024

Period Rainfall: 2.41 inches

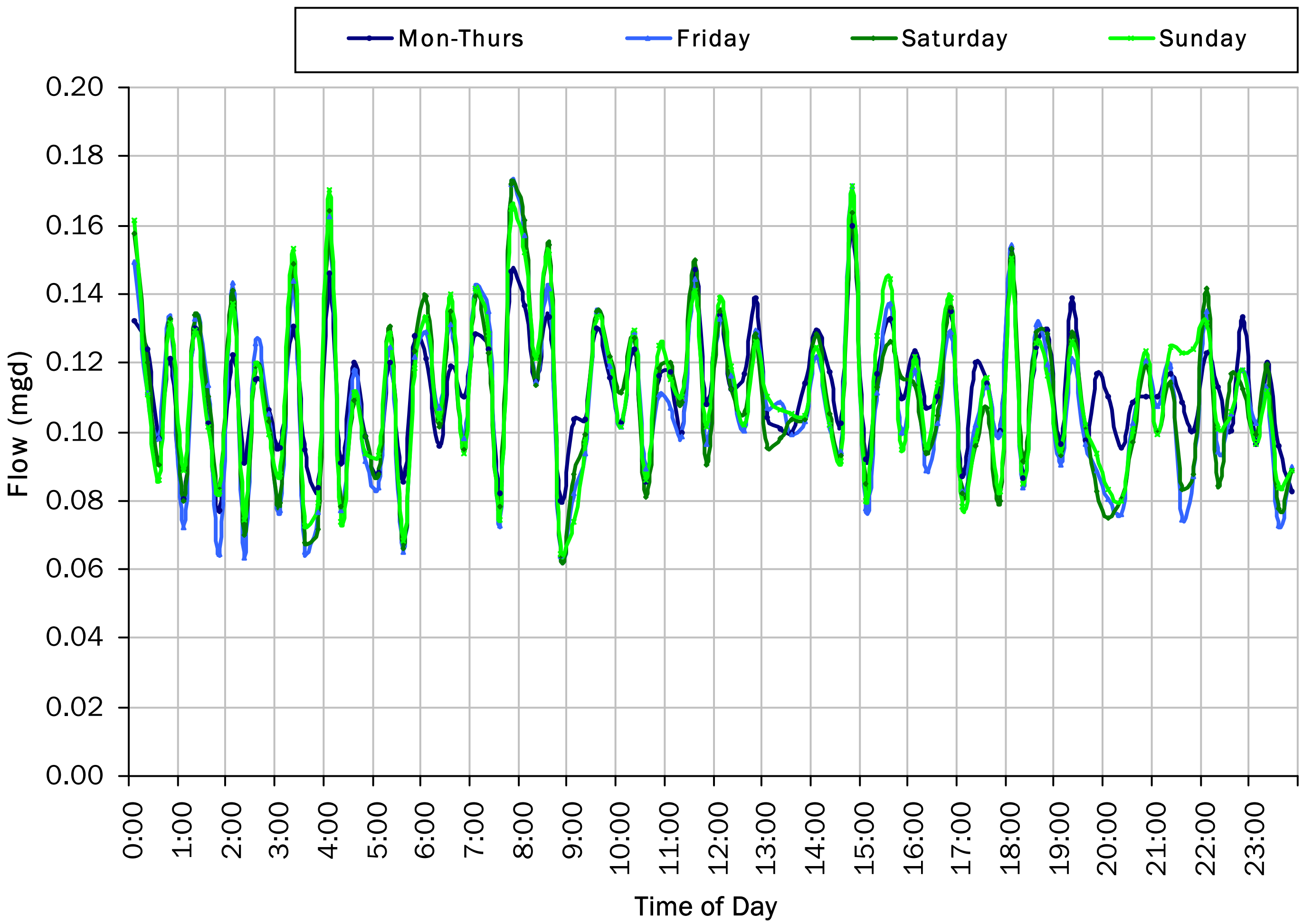
Period Avg Flow: 0.111 mgd   Period Peak Flow: 0.192 mgd   Period Min Flow: 0.043 mgd





TAYLOR 1 PS

Average Dry Weather Flow Hydrographs



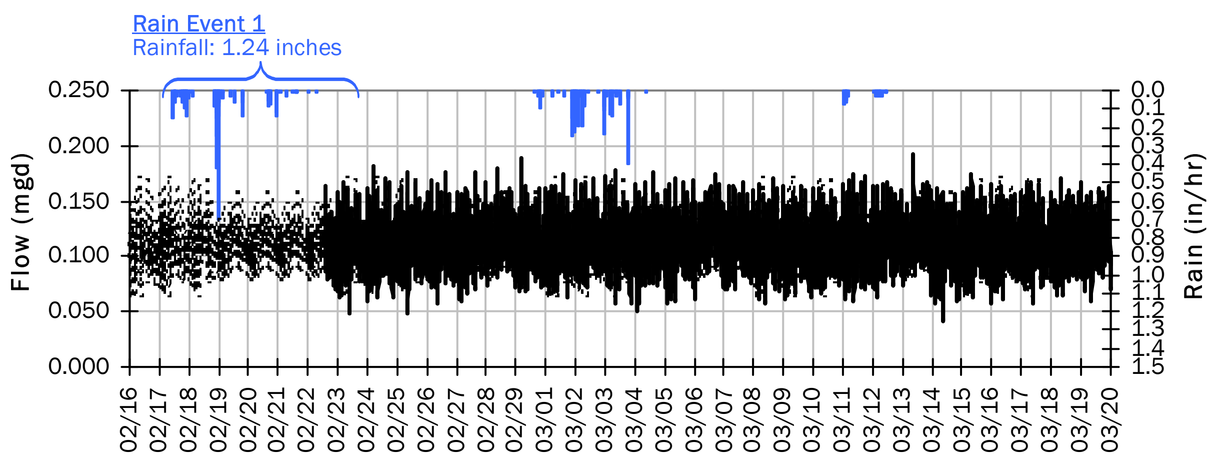
ADWF:

0.111 *mgd*

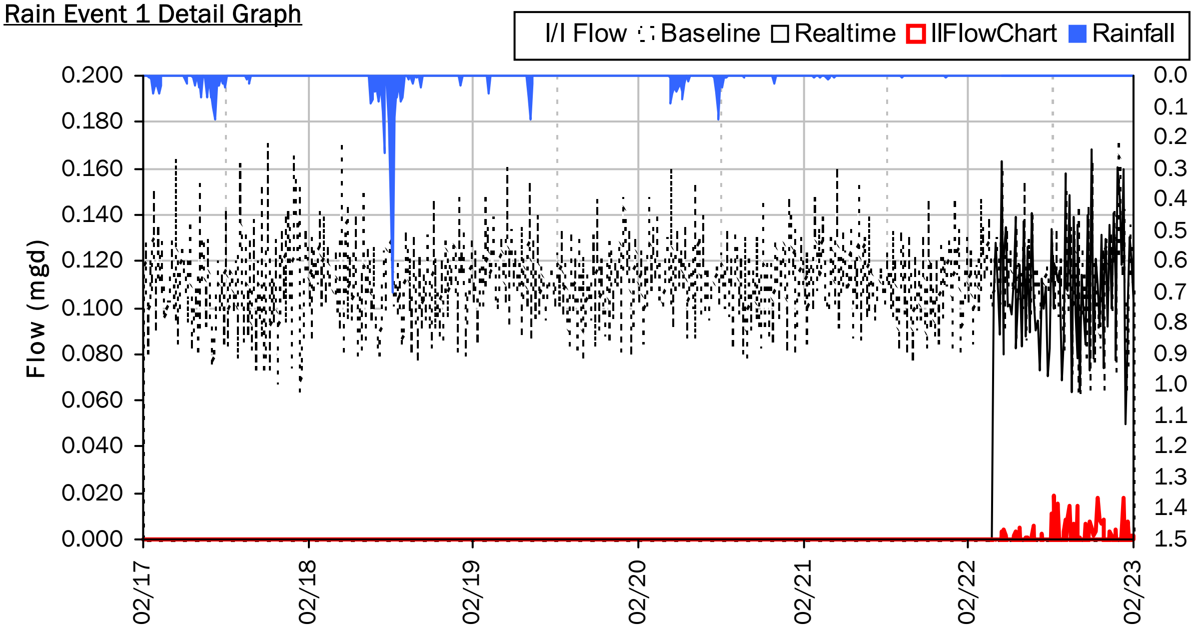
TAYLOR 1 PS

I/I Summary: Rain Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



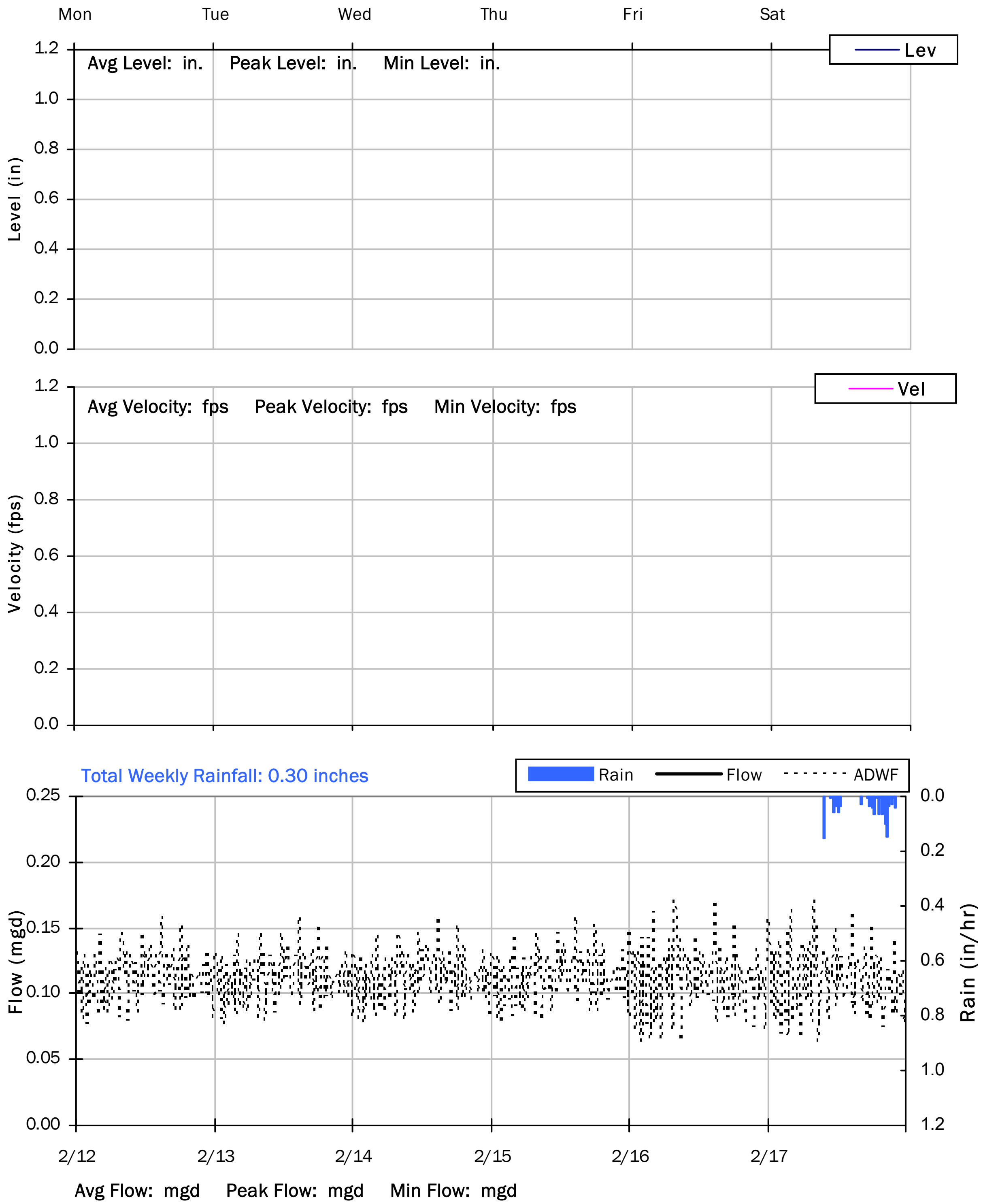
Rain Event 1 Detail Graph



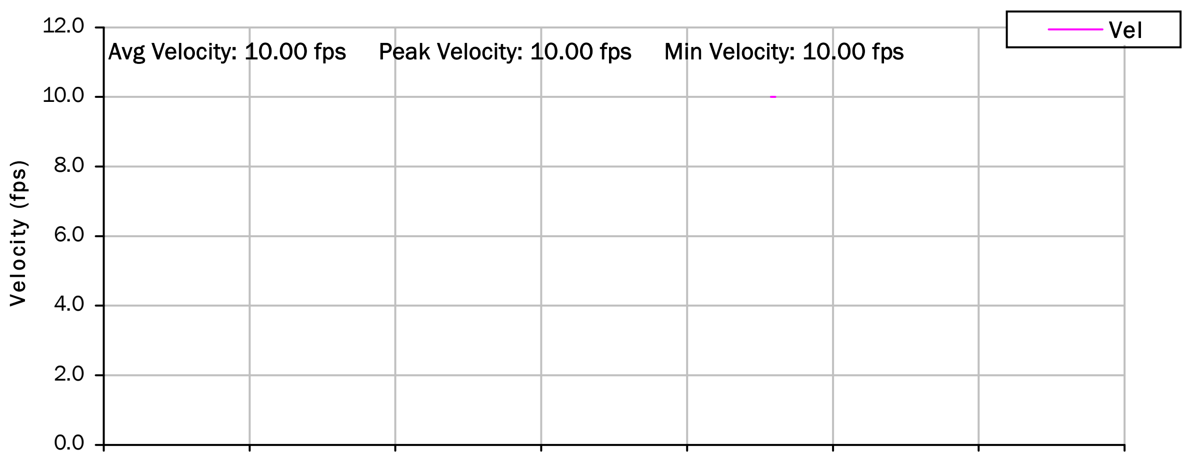
Storm Event I/I Analysis (Rain = 1.24 inches)

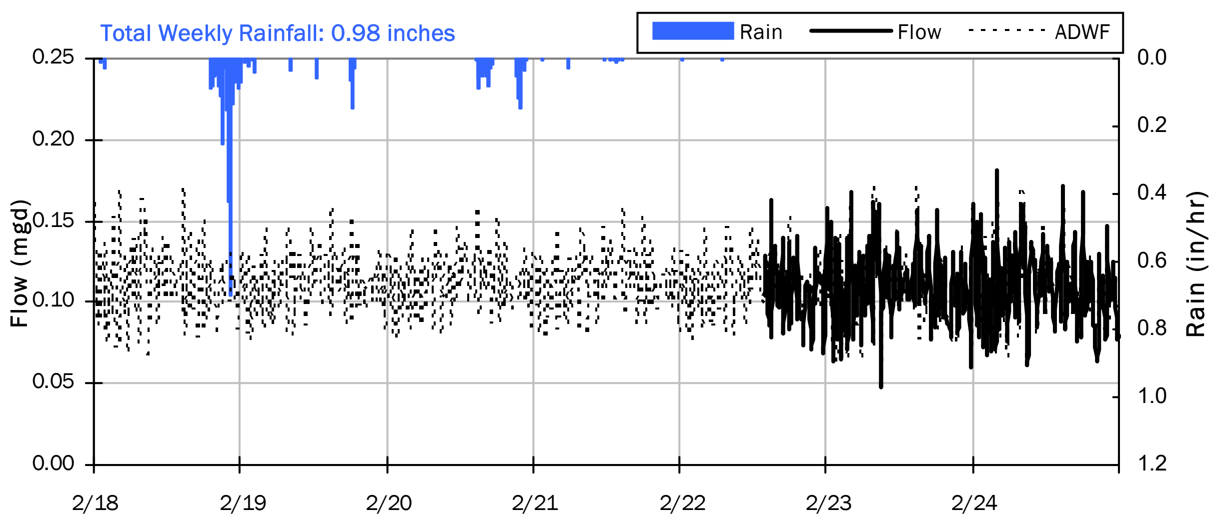
Capacity		Inflow / Infiltration	
Peak Flow:	0.168 *mgd*	Peak I/I Rate:	0.019 *mgd*
PF:	1.51	Total I/I:	-3,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

TAYLOR 1 PS
Weekly Level, Velocity and Flow Hydrographs
2/12/2024 to 2/18/2024



TAYLOR 1 PS
Weekly Level, Velocity and Flow Hydrographs
2/18/2024 to 2/25/2024



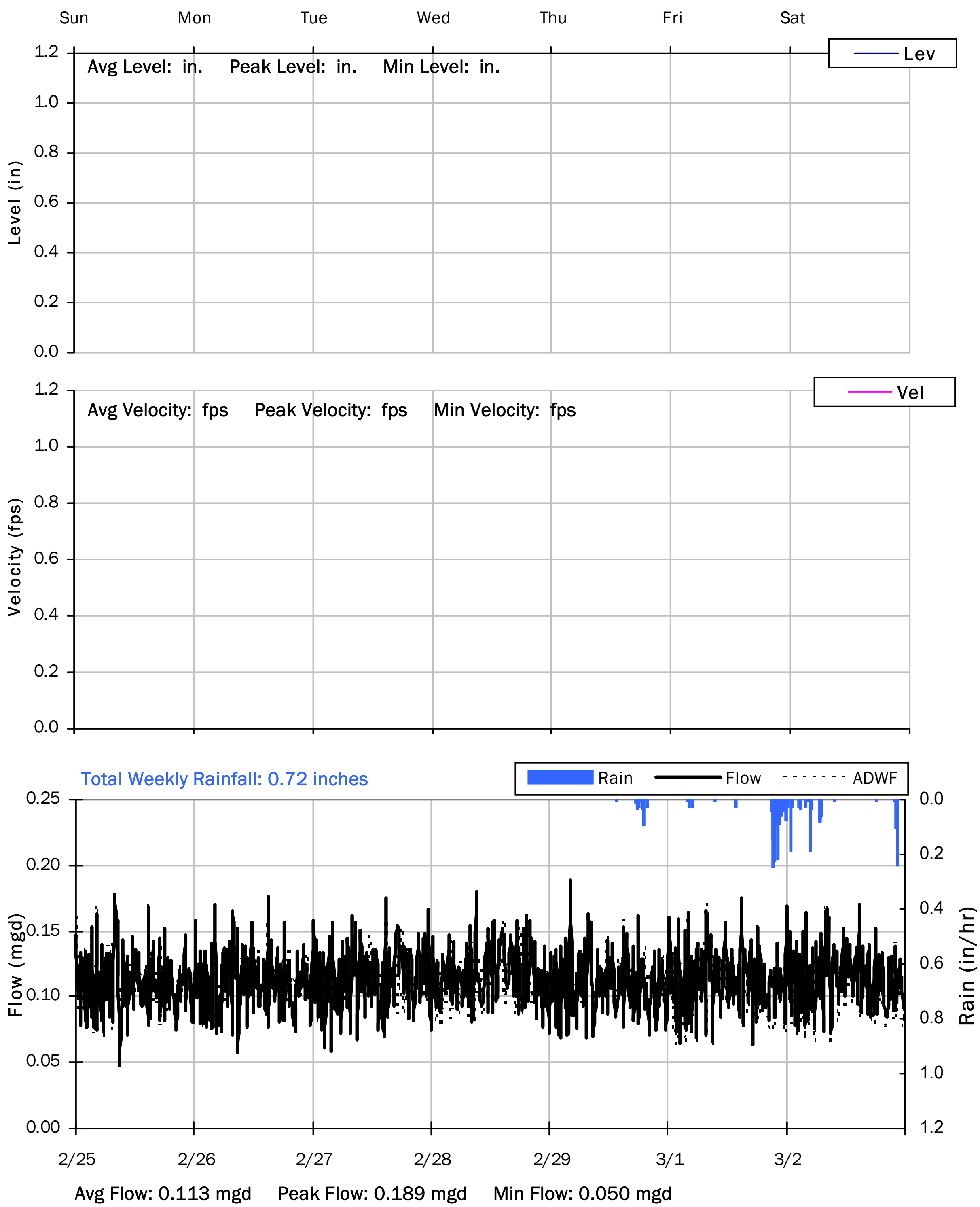


Avg Flow: 0.109 mgd Peak Flow: 0.181 mgd Min Flow: 0.050 mgd

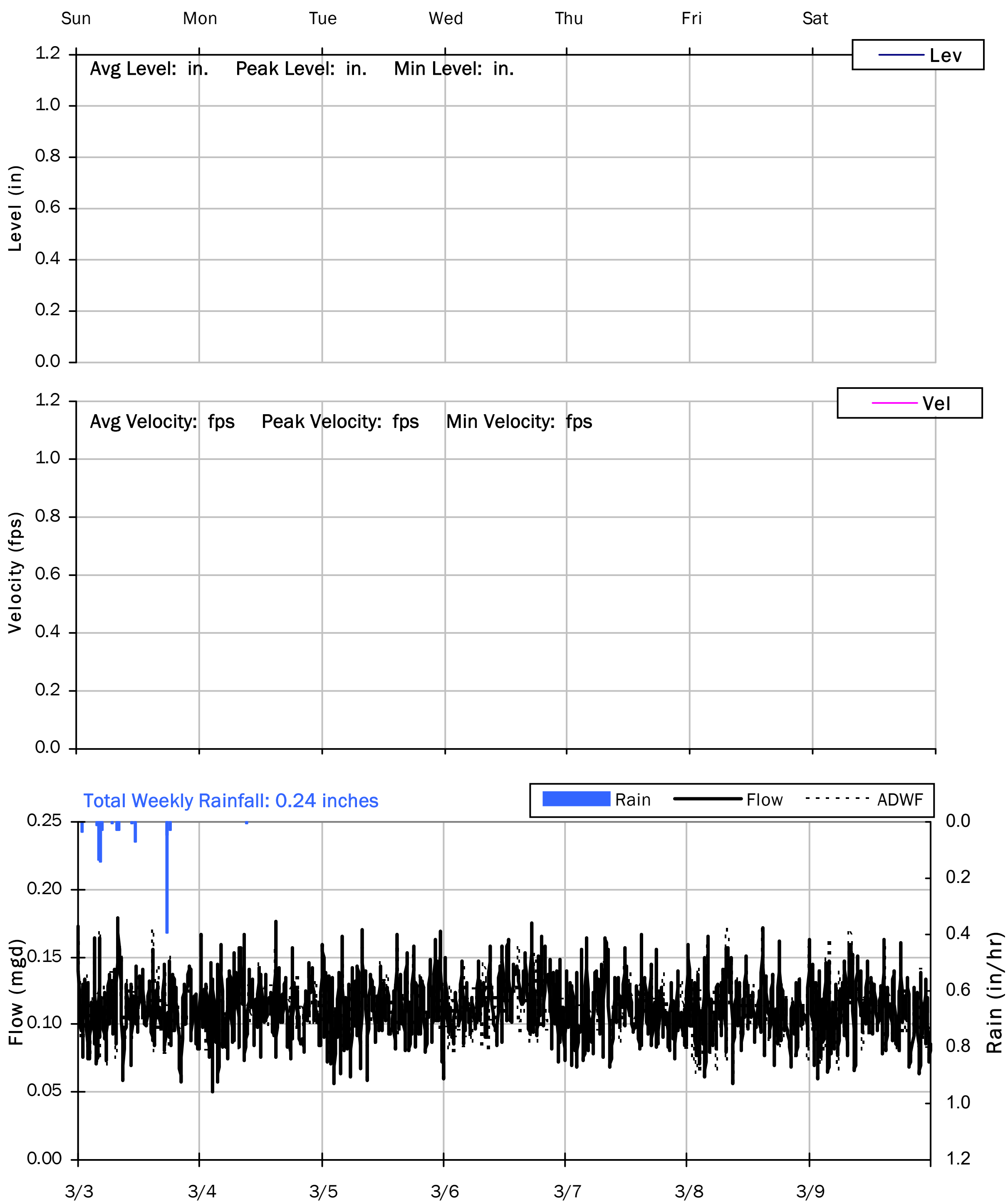
TAYLOR 1 PS

Weekly Level, Velocity and Flow Hydrographs

2/25/2024 to 3/3/2024

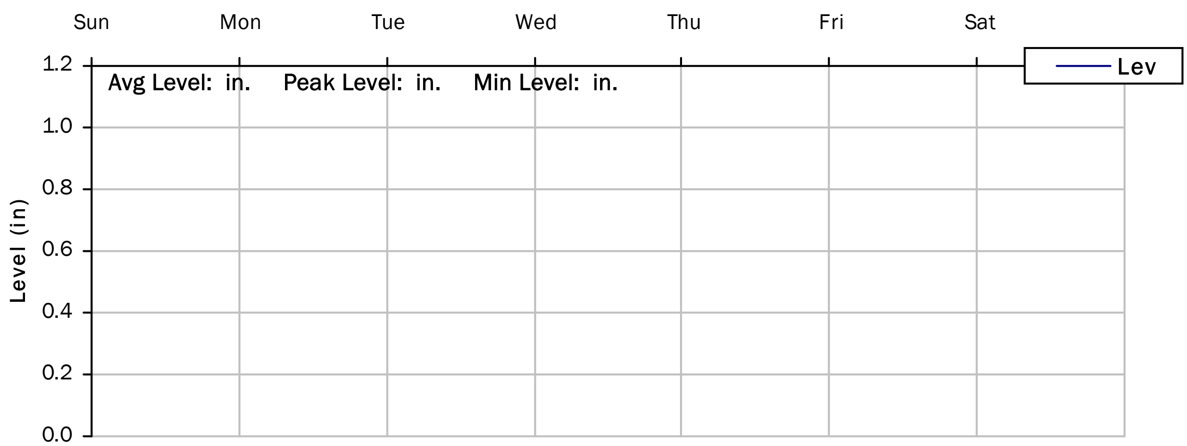


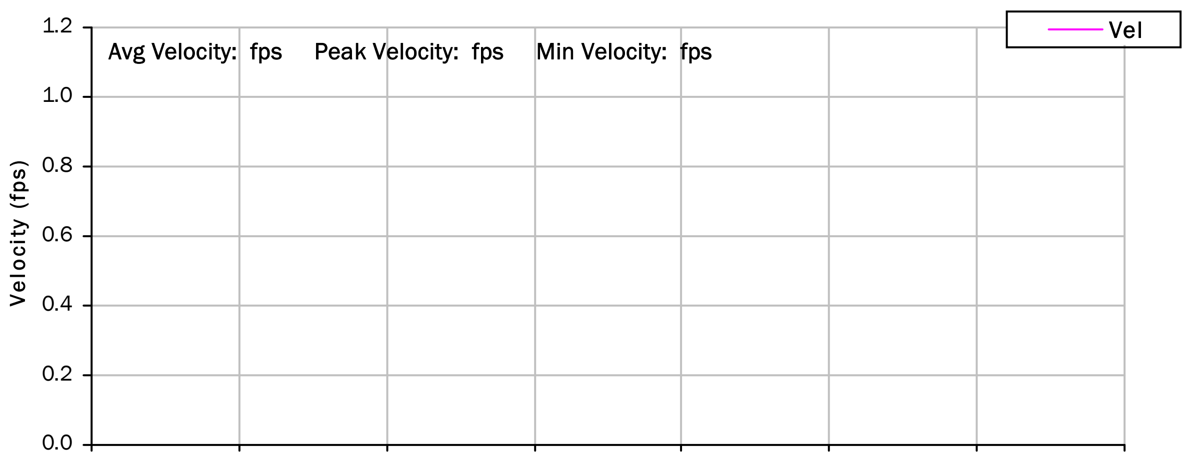
TAYLOR 1 PS
Weekly Level, Velocity and Flow Hydrographs
3/3/2024 to 3/10/2024

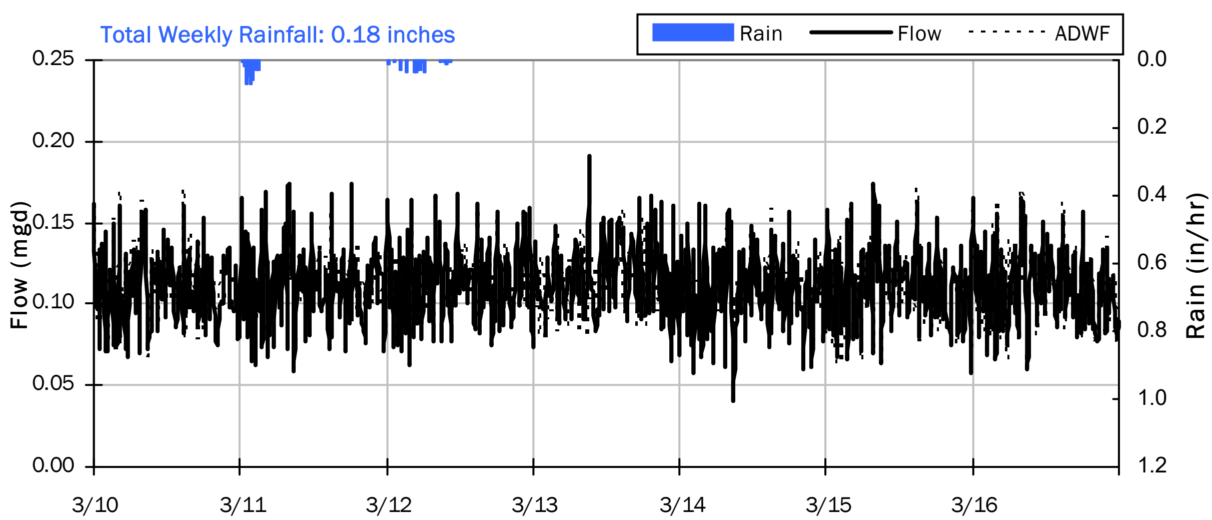


Avg Flow: 0.112 mgd   Peak Flow: 0.176 mgd   Min Flow: 0.050 mgd

TAYLOR 1 PS
Weekly Level, Velocity and Flow Hydrographs
3/10/2024 to 3/17/2024





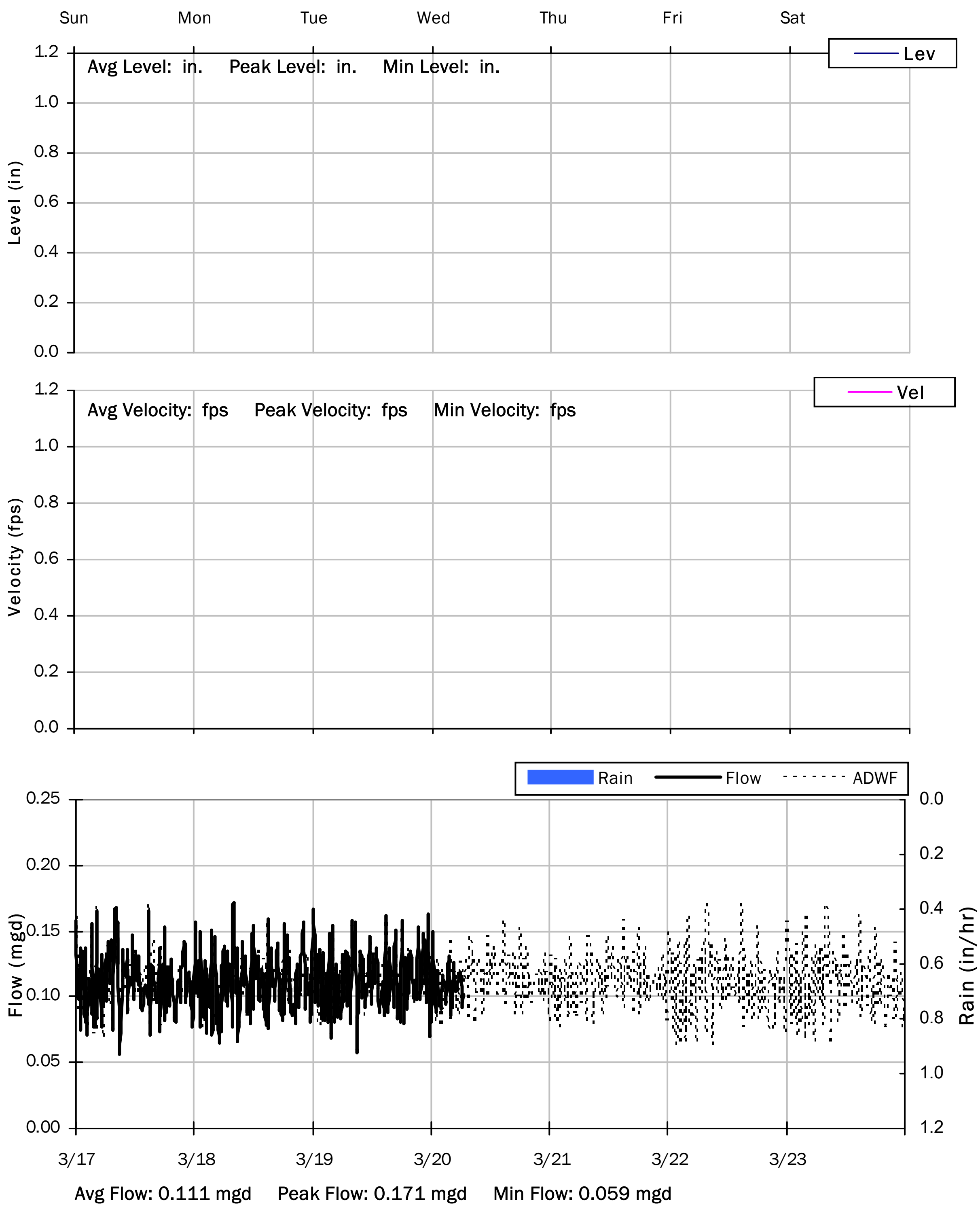


Avg Flow: 0.111 mgd Peak Flow: 0.192 mgd Min Flow: 0.043 mgd

TAYLOR 1 PS

Weekly Level, Velocity and Flow Hydrographs

3/17/2024 to 3/24/2024



Monitoring Site: Willow 2 PS

Ironhouse Sanitary District | Oakley, California

Sanitary Sewer Flow Monitoring

February 16, 2024 - March 20, 2024

Location: 3995 N Willow Road #9, Bethel Island

Data Summary Report

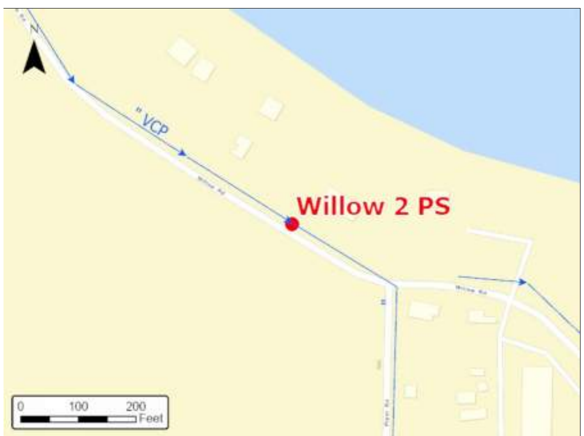
Vicinity Map: Willow 2 PS

WILLOW 2 PS

Site Information

Location:	3995 N Willow Road #9, Bethel Island
Coordinates:	121.6224° W, 38.0335° N
ADWF:	0.101 mgd
Peak Measured Flow:	0.26 mgd

Satellite Map



Sanitary Map

Street View

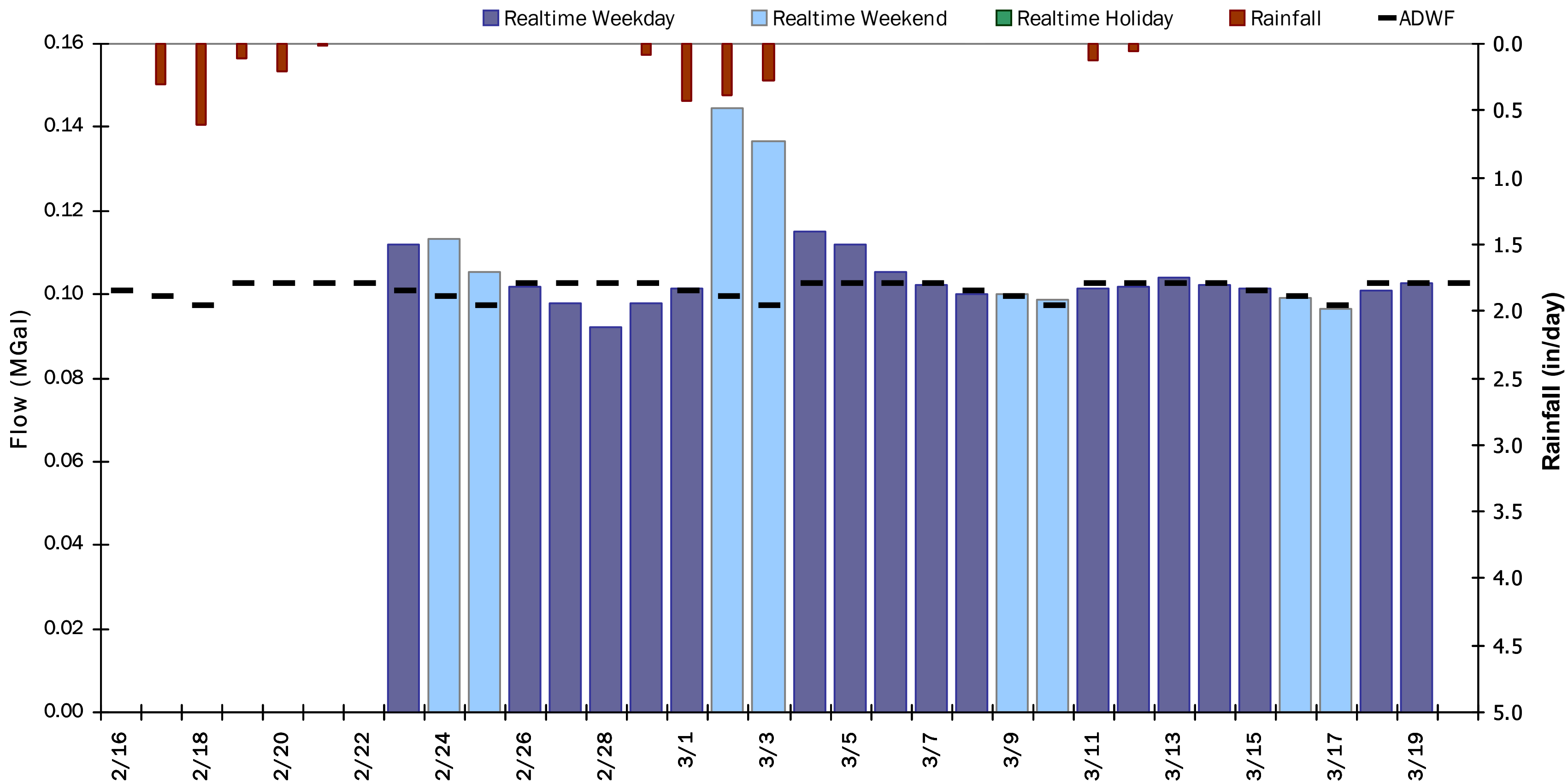

Control Panel

WILLOW 2 PS

Period Flow Summary: Daily Flow Totals

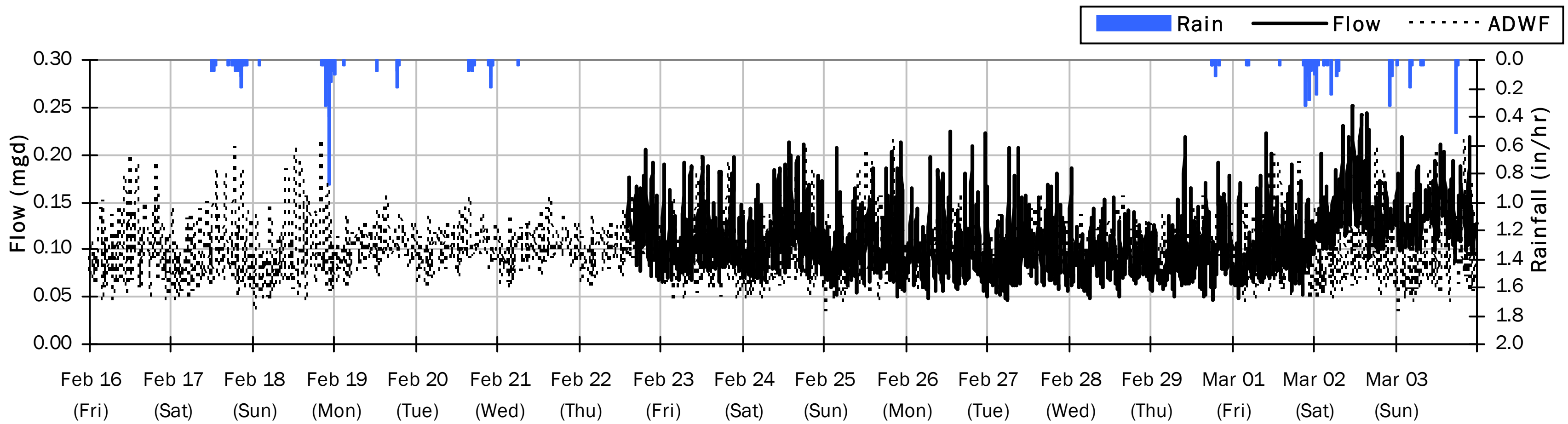
Avg Daily Flow: 0.142 MGal Peak Daily Flow: 0.142 MGal Min Daily Flow: 0.142 MGal

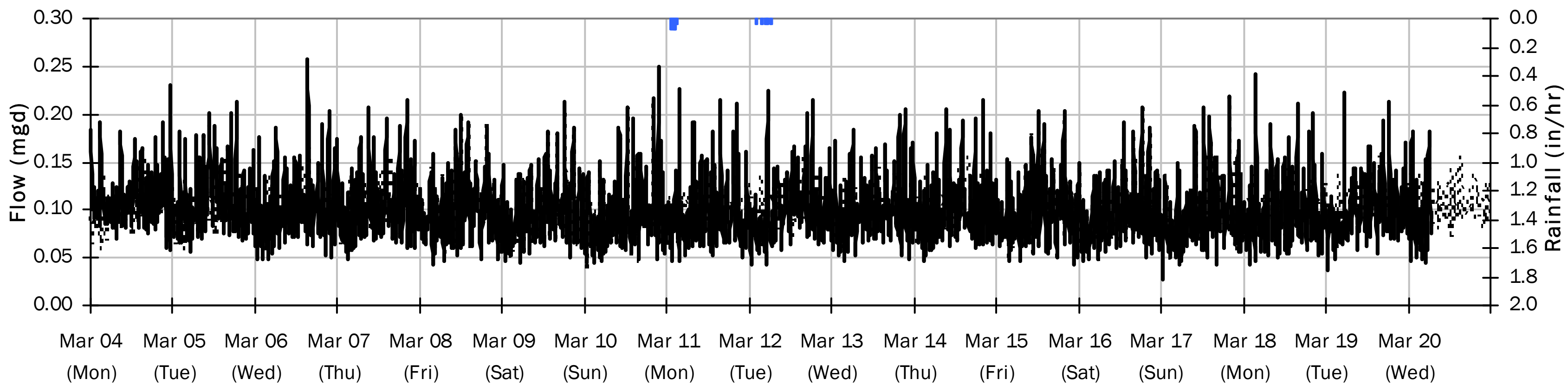
Total Rainfall: 2.57 inches



WILLOW 2 PS
Flow Summary: 2/16/2024 to 3/20/2024

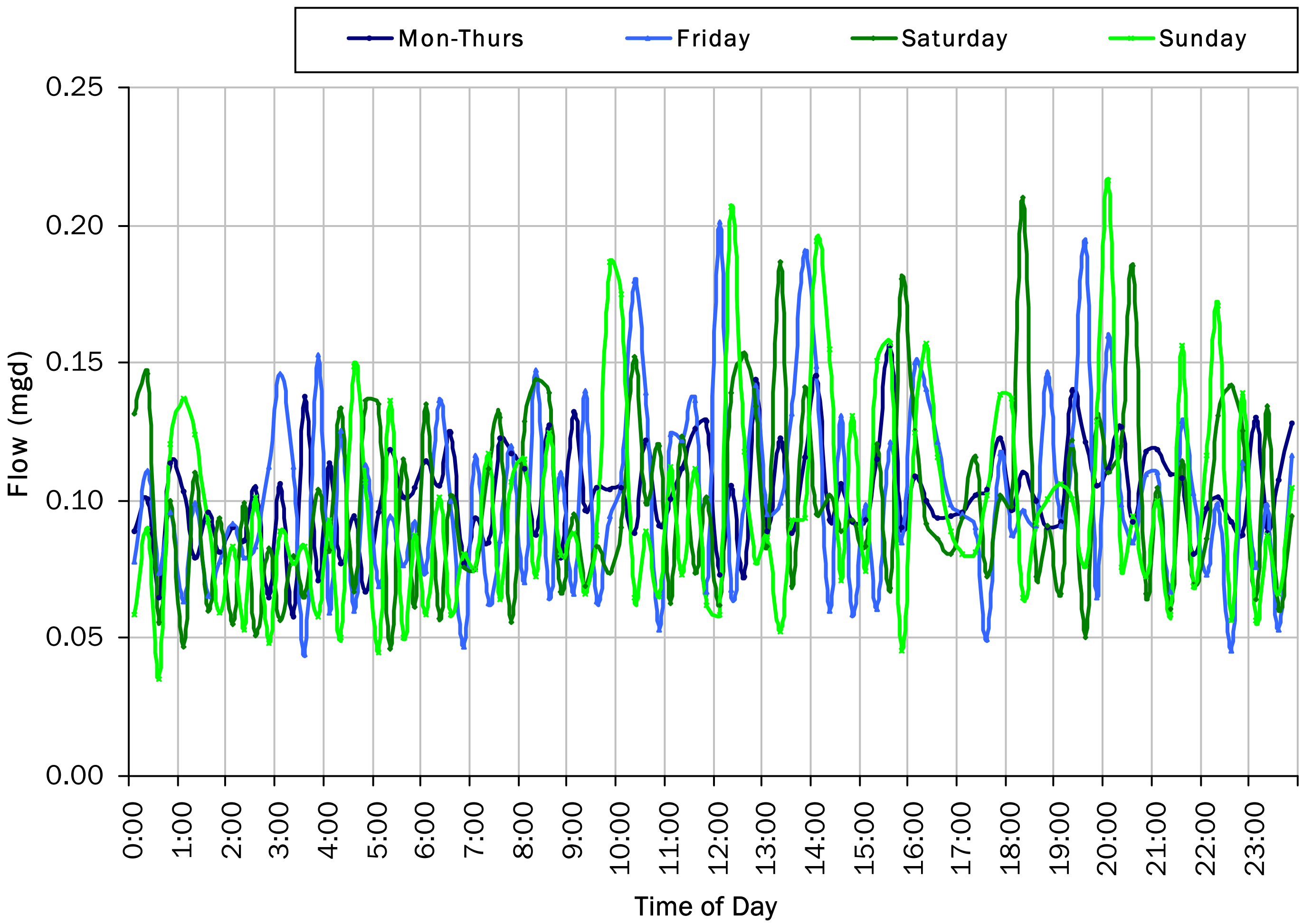
Period Rainfall: 2.57 inches   Period Avg Flow: 0.106 mgd   Period Peak Flow: 0.255 mgd   Period Min Flow: 0.026 mgd





WILLOW 2 PS

Average Dry Weather Flow Hydrographs



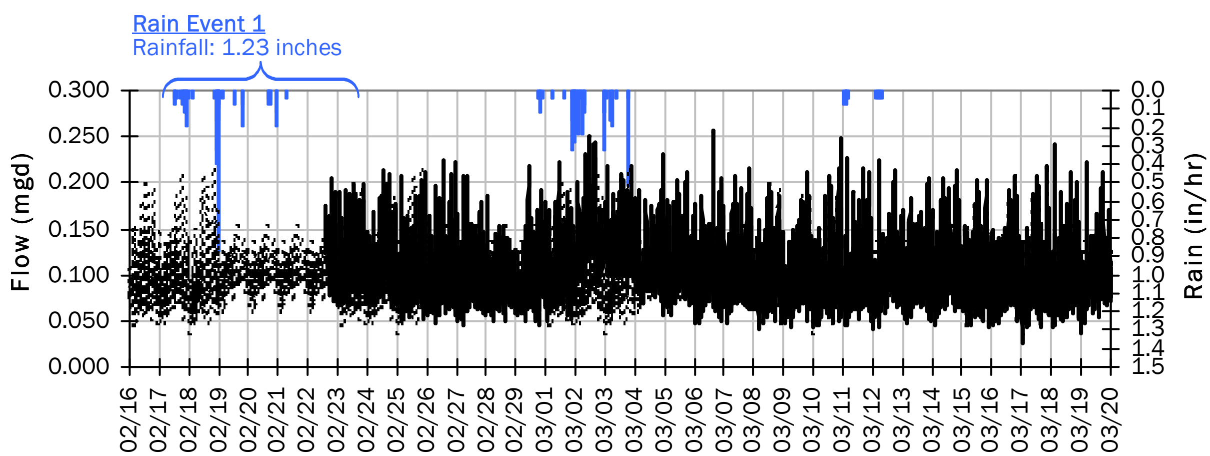
ADWF:

0.101 *mgd*

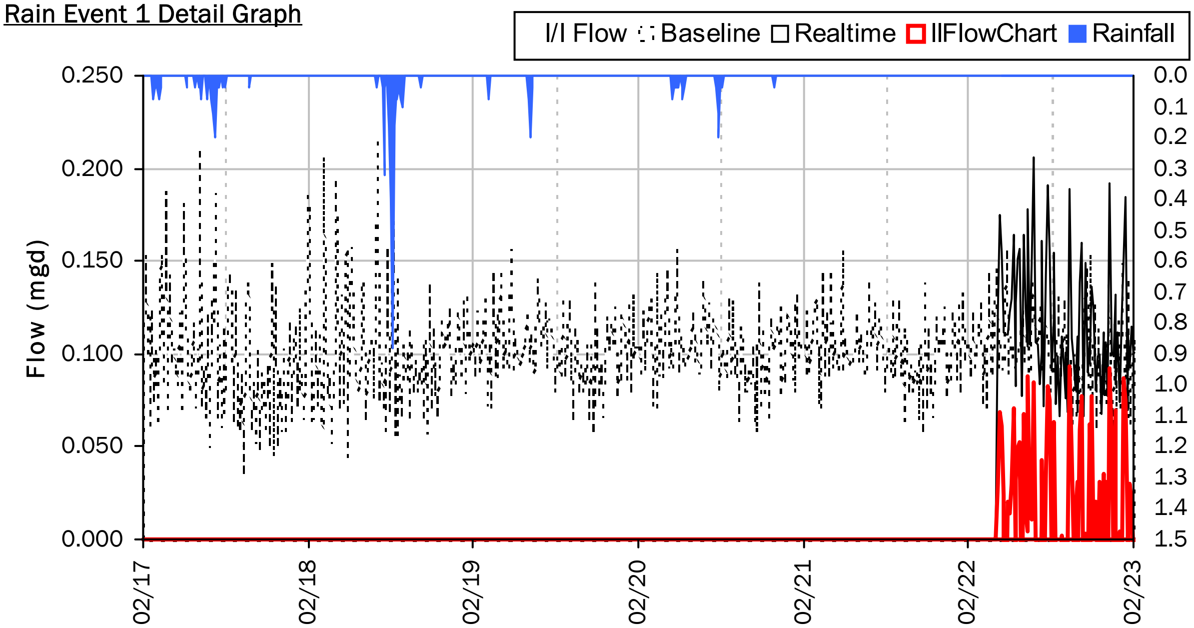
WILLOW 2 PS

I/I Summary: Rain Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



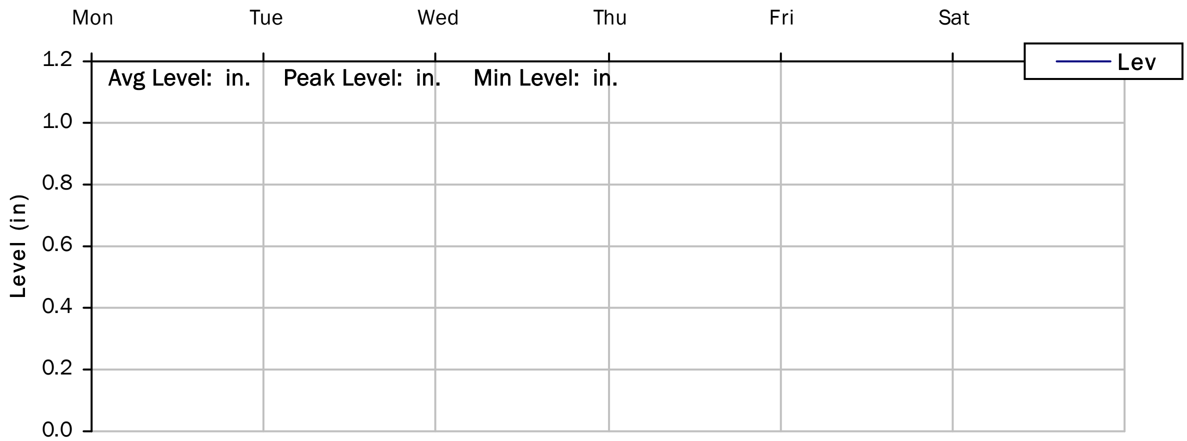
Rain Event 1 Detail Graph

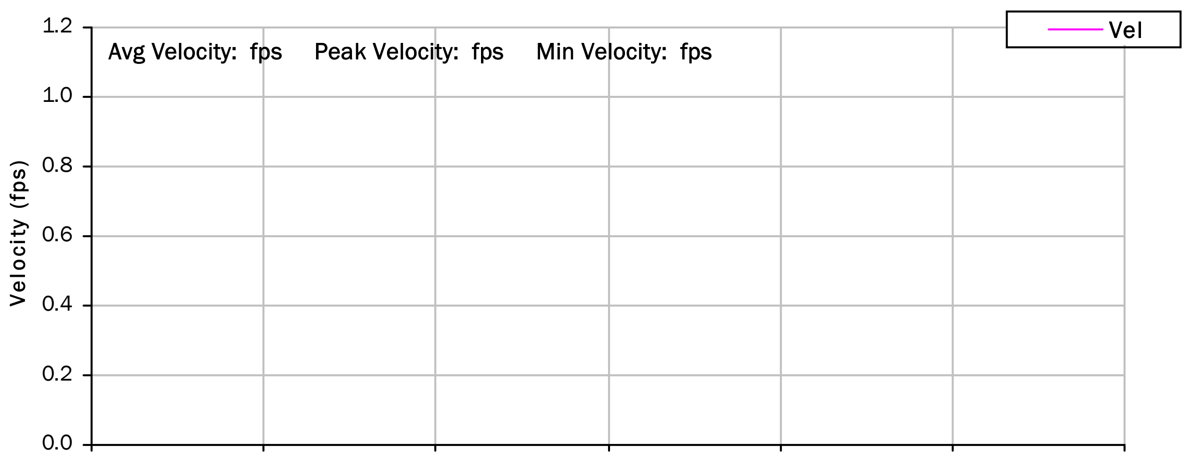


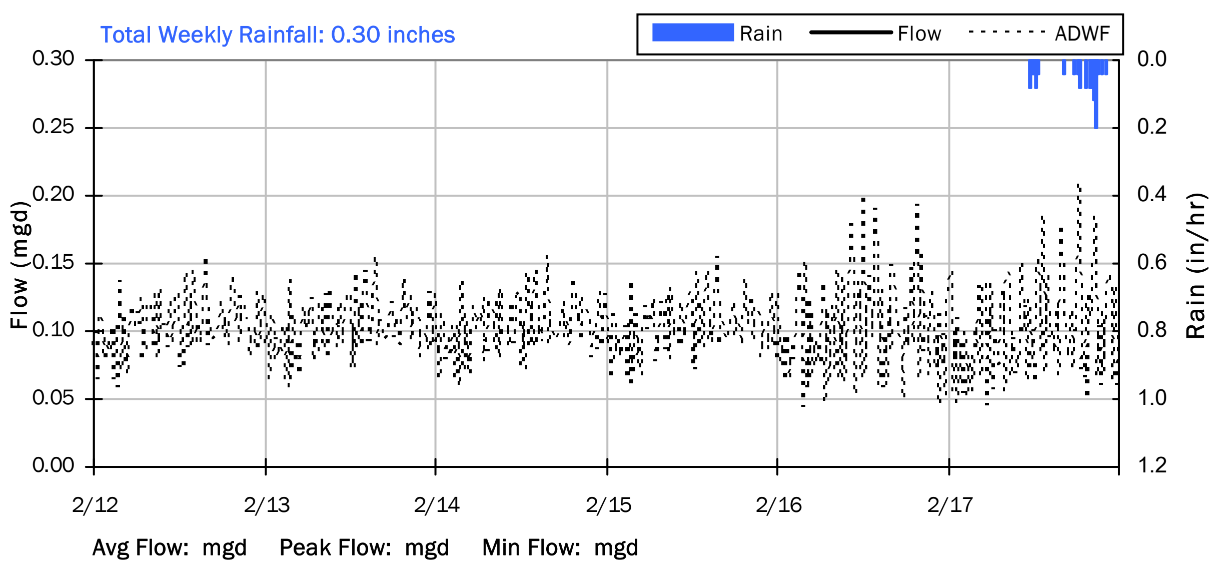
Storm Event I/I Analysis (Rain = 1.23 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.206 *mgd*	Peak I/I Rate:	0.093 *mgd*
PF:	2.04	Total I/I:	15,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

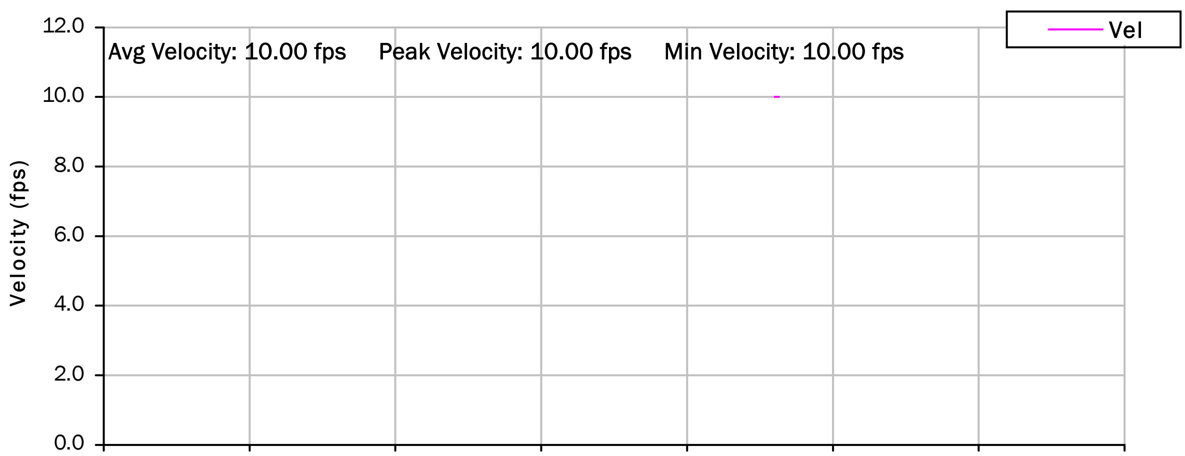
WILLOW 2 PS
Weekly Level, Velocity and Flow Hydrographs
2/12/2024 to 2/18/2024

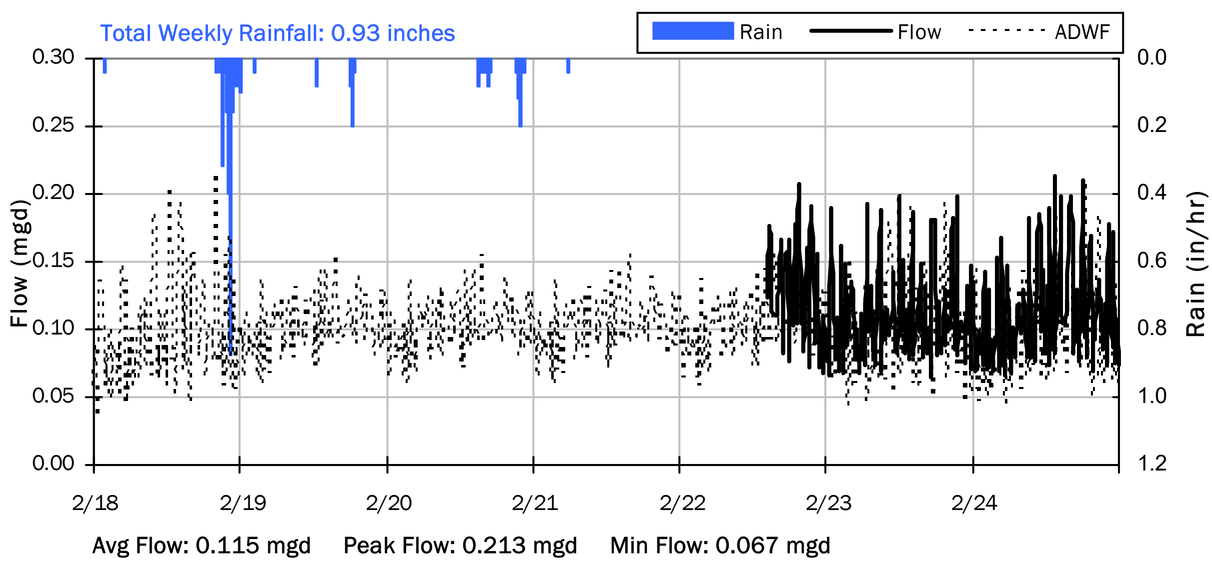




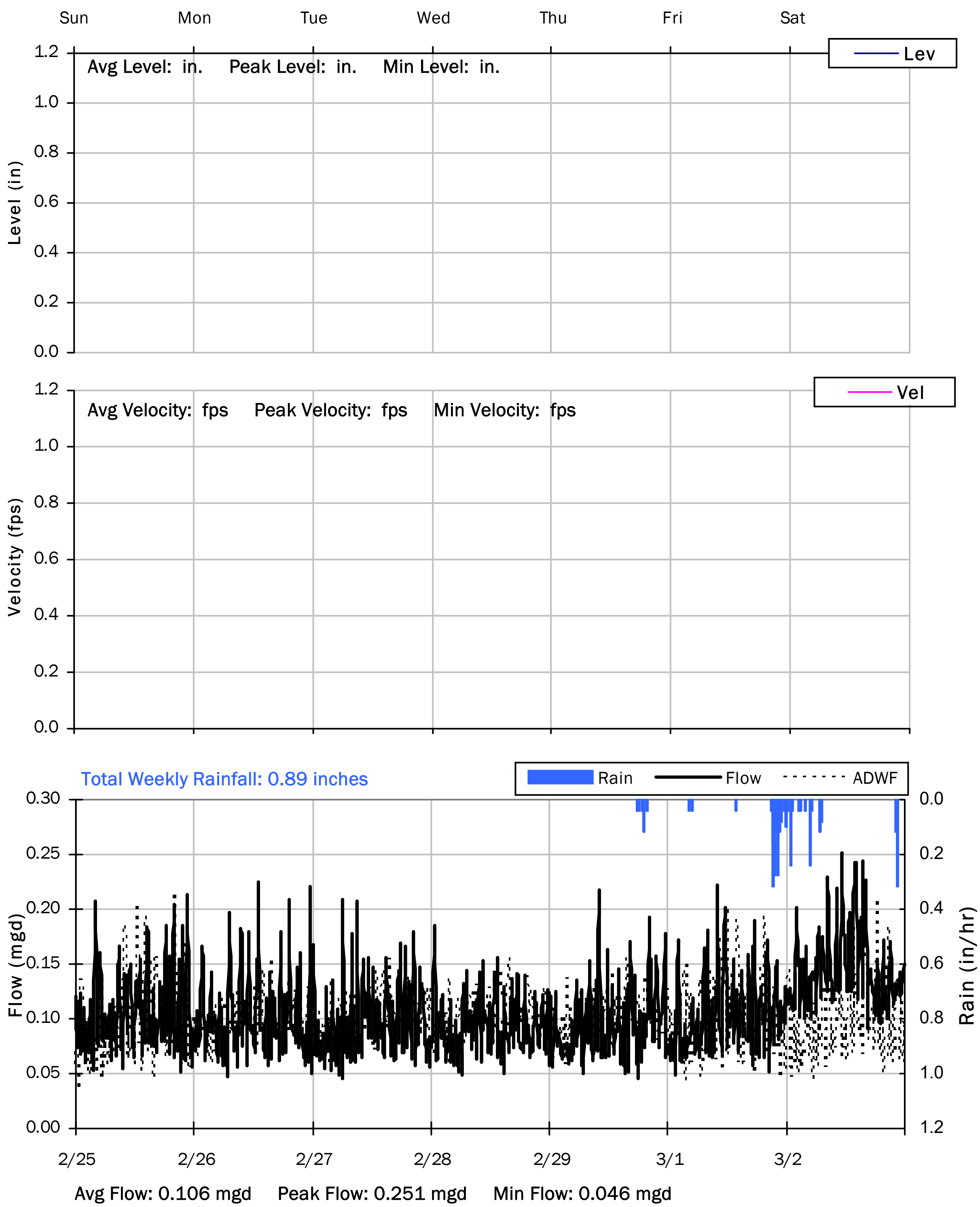


WILLOW 2 PS
Weekly Level, Velocity and Flow Hydrographs
2/18/2024 to 2/25/2024

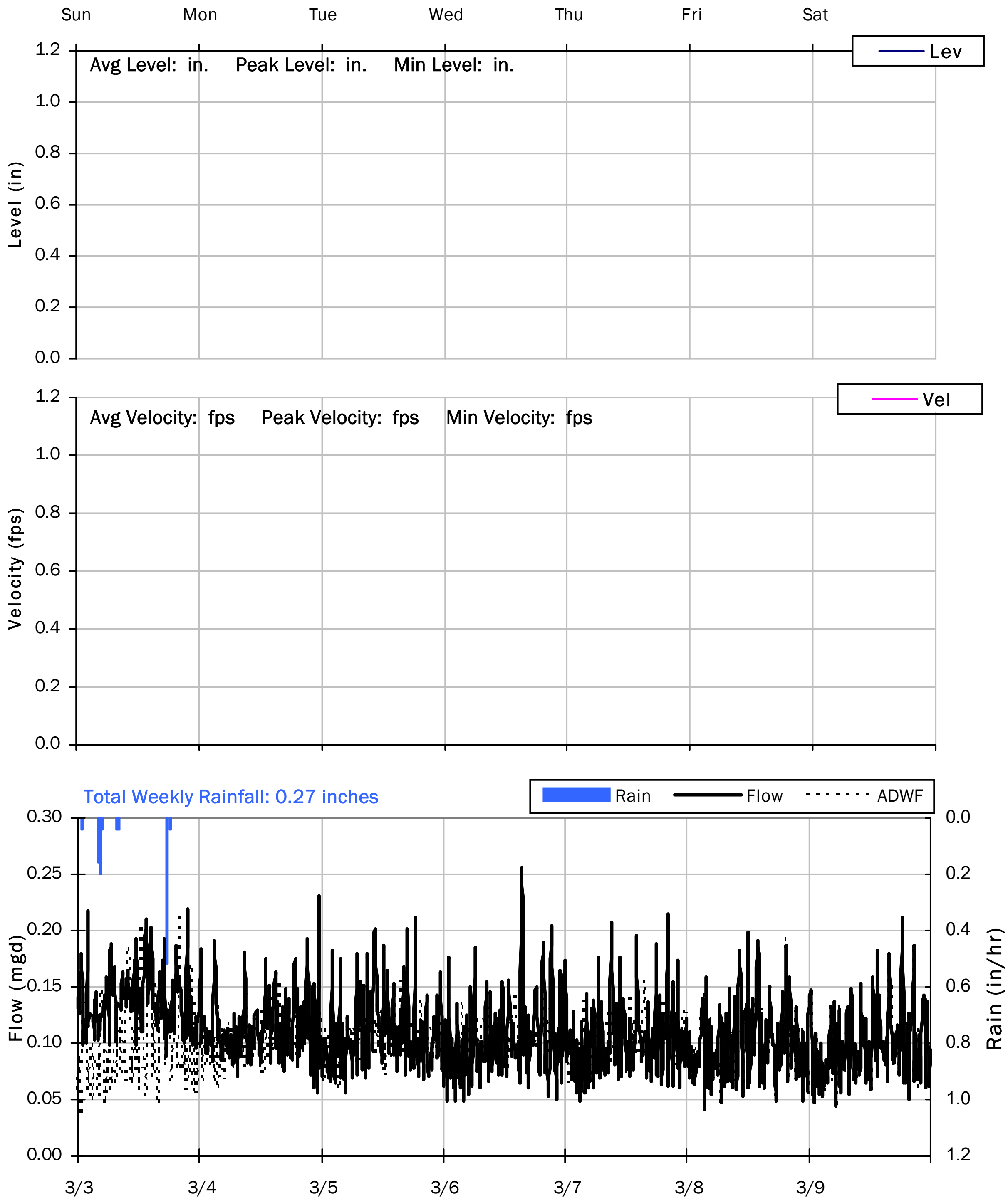




WILLOW 2 PS
Weekly Level, Velocity and Flow Hydrographs
2/25/2024 to 3/3/2024



WILLOW 2 PS
Weekly Level, Velocity and Flow Hydrographs
3/3/2024 to 3/10/2024

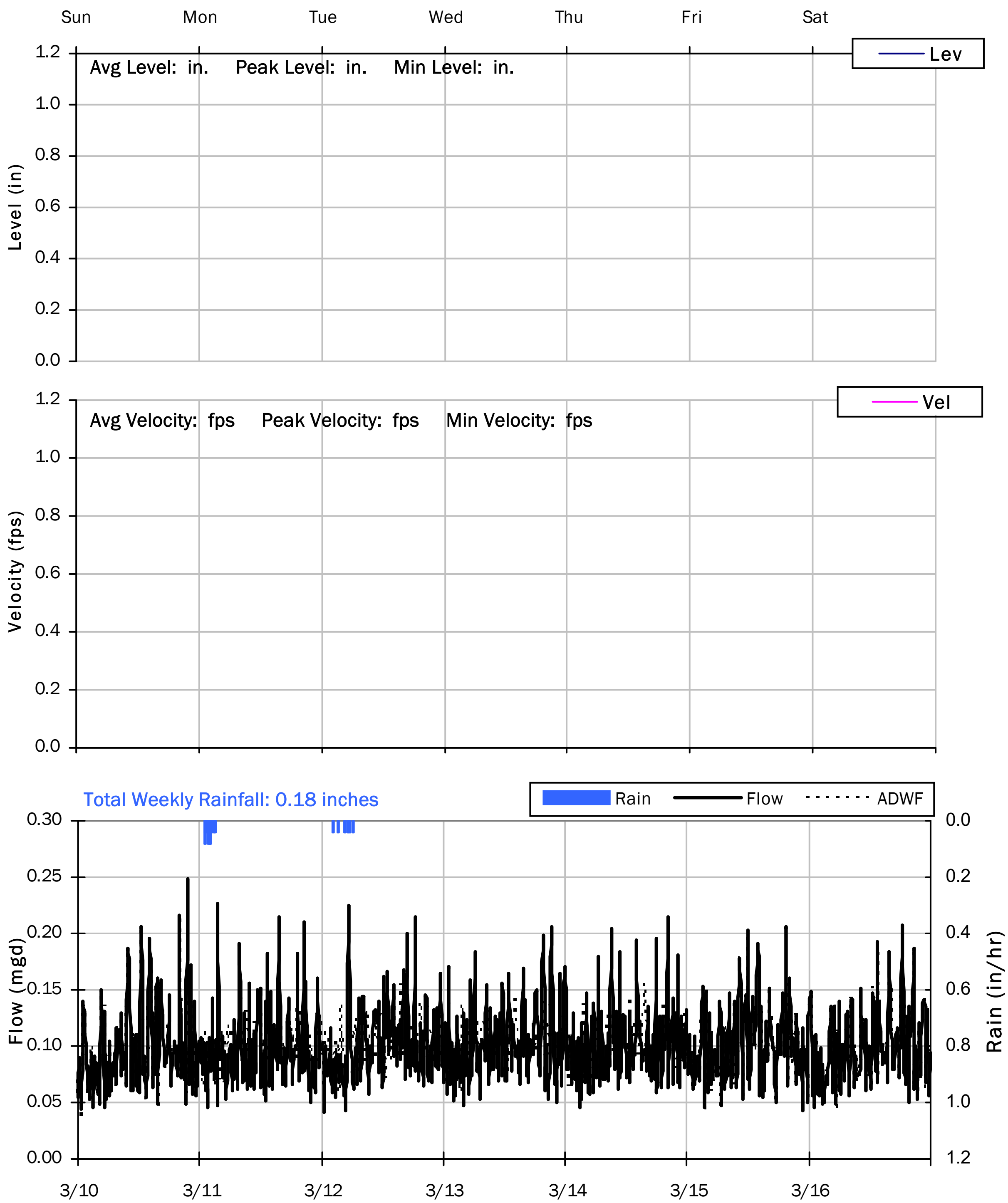


Avg Flow: 0.110 mgd Peak Flow: 0.255 mgd Min Flow: 0.042 mgd

WILLOW 2 PS

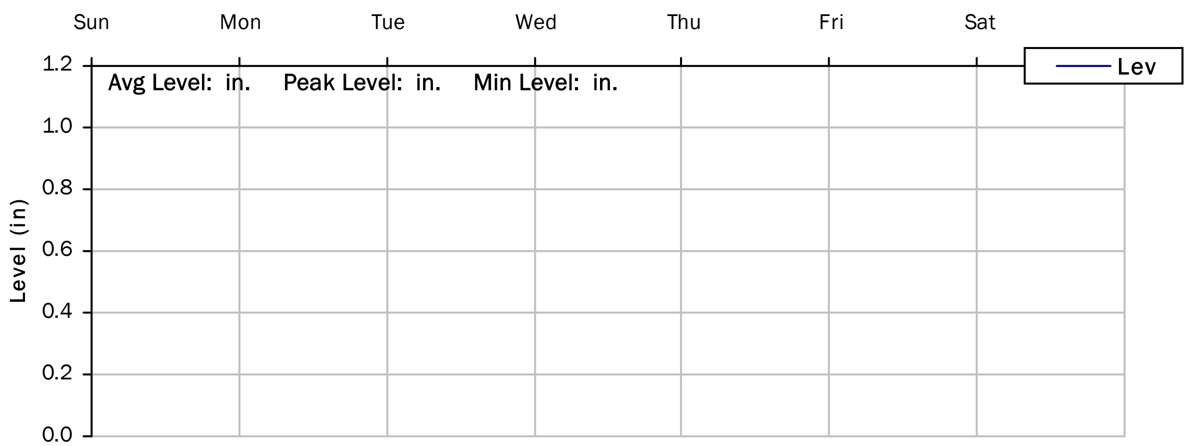
Weekly Level, Velocity and Flow Hydrographs

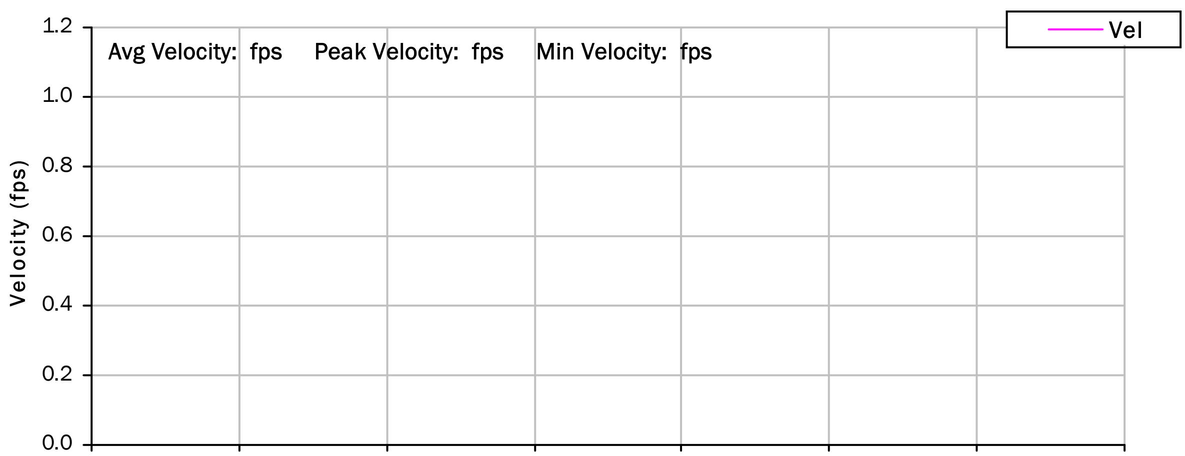
3/10/2024 to 3/17/2024

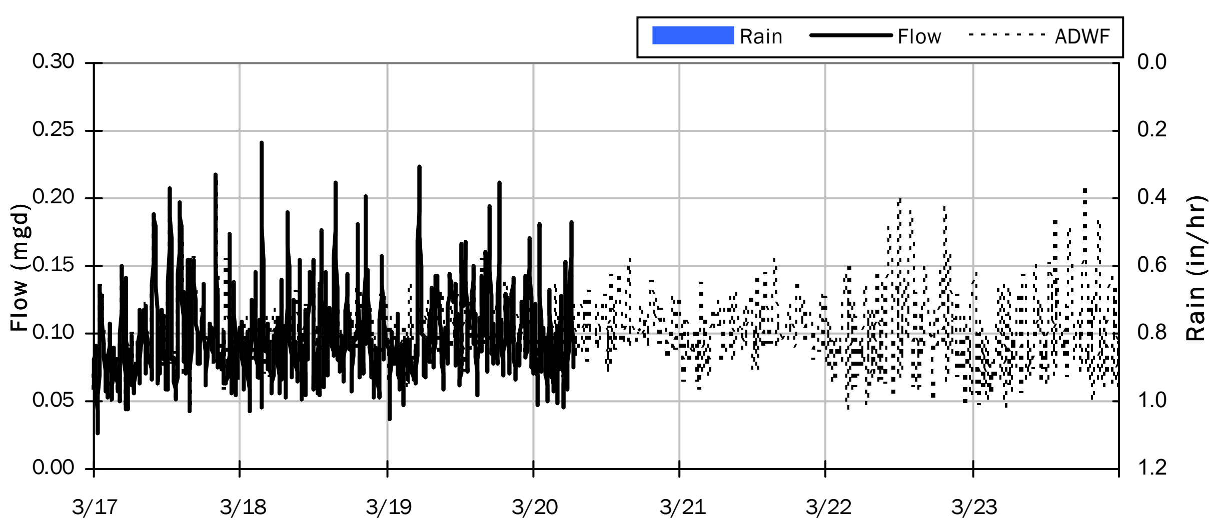


Avg Flow: 0.101 mgd Peak Flow: 0.249 mgd Min Flow: 0.042 mgd

WILLOW 2 PS
Weekly Level, Velocity and Flow Hydrographs
3/17/2024 to 3/24/2024







Avg Flow: 0.100 mgd Peak Flow: 0.241 mgd Min Flow: 0.026 mgd

V&A Project No. 24-0061

V&A
consulting engineers
1000 Broadway
Suite 320
Oakland, CA 94607
510.903.6600
510.903.6601, Fax

Appendix B

Rehabilitation and Replacement Project Maps

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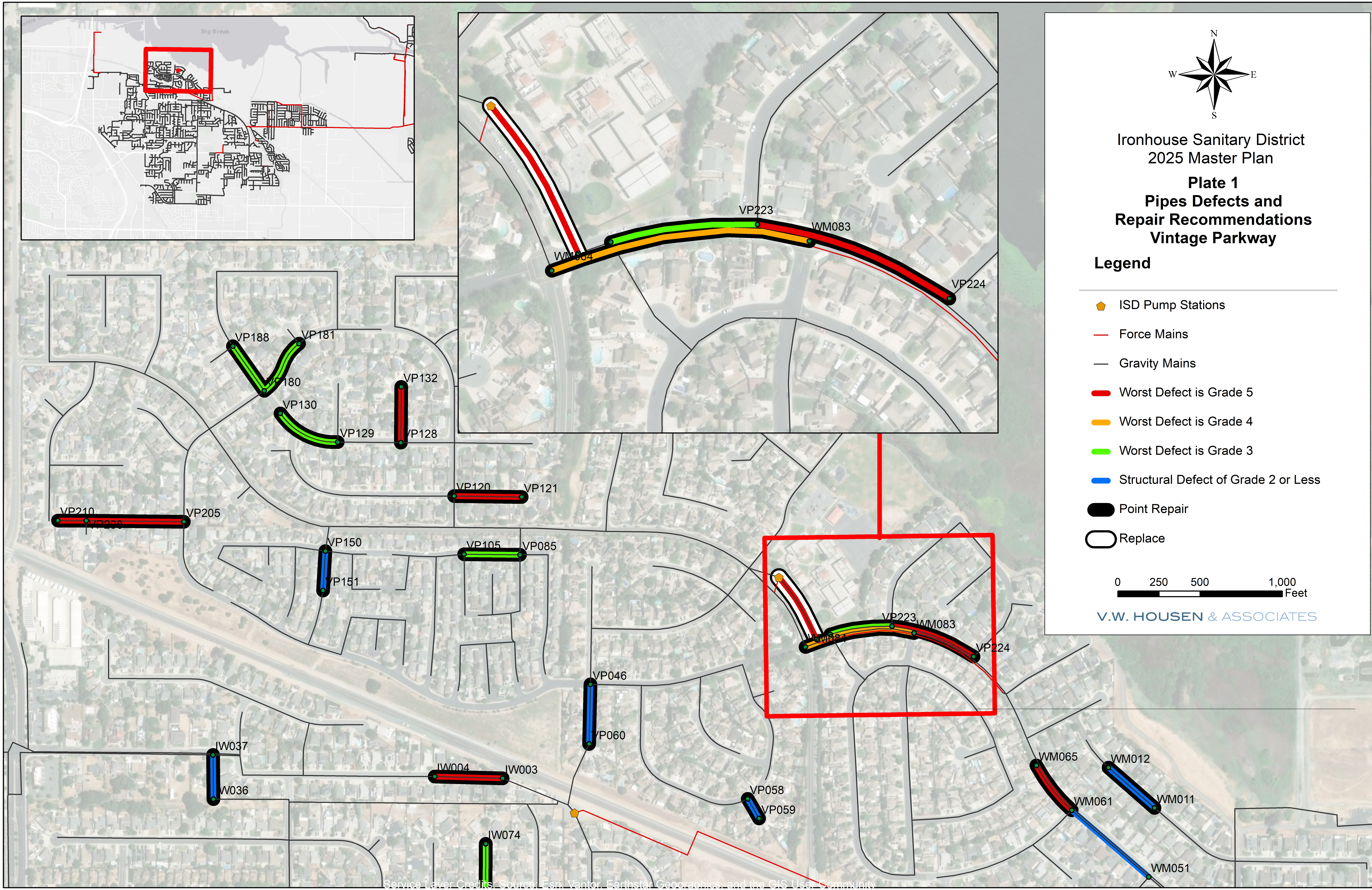
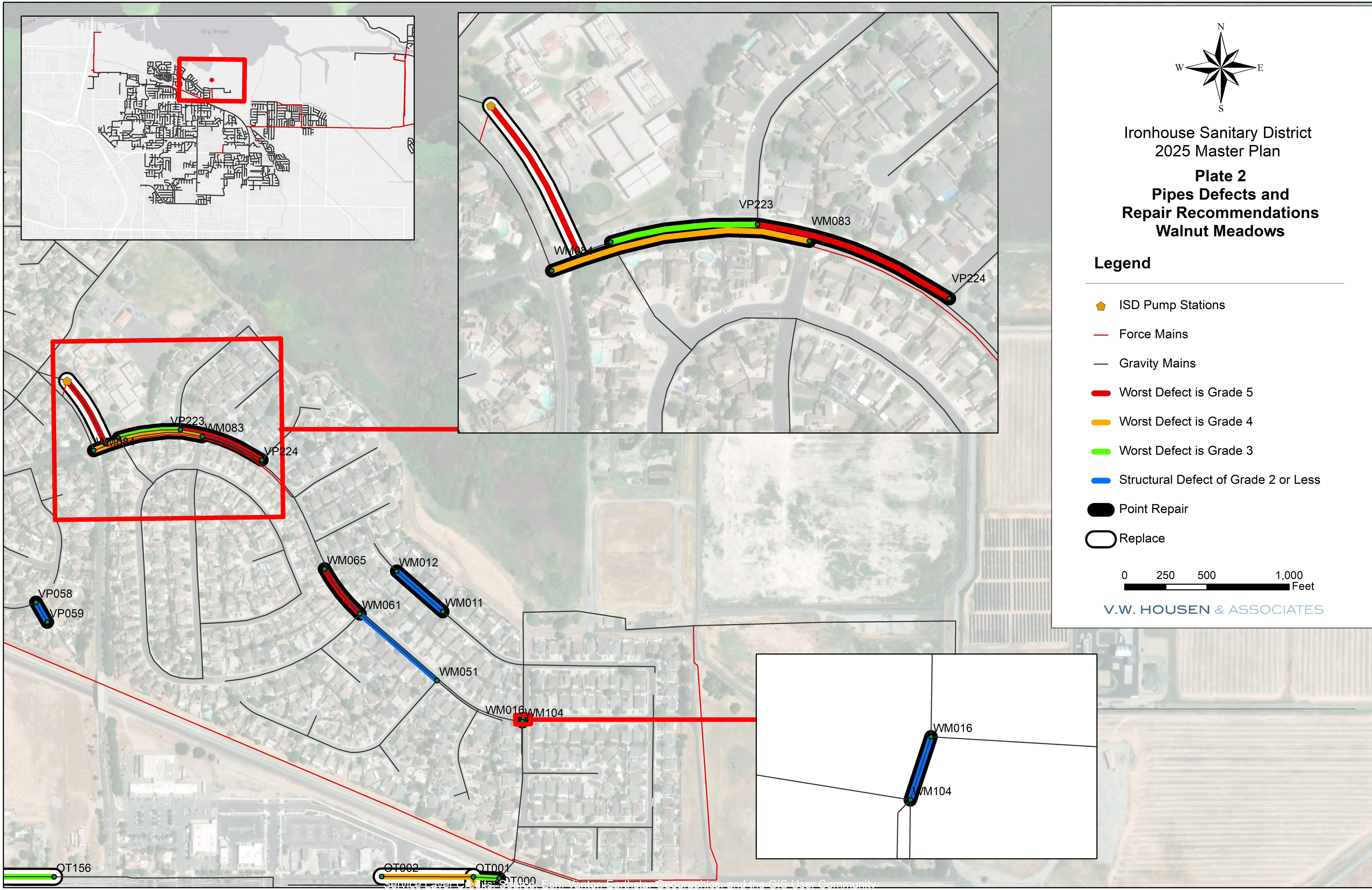
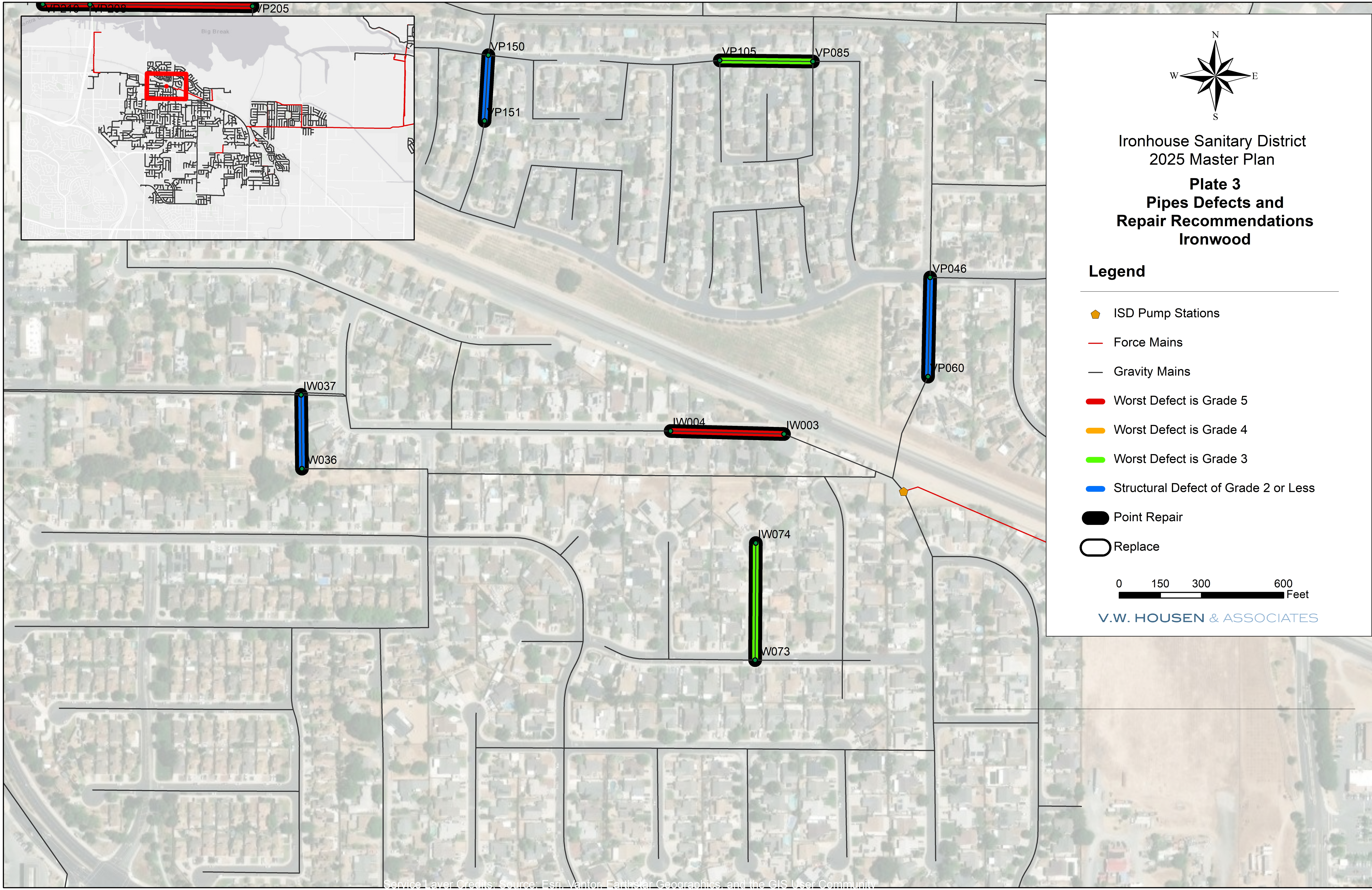


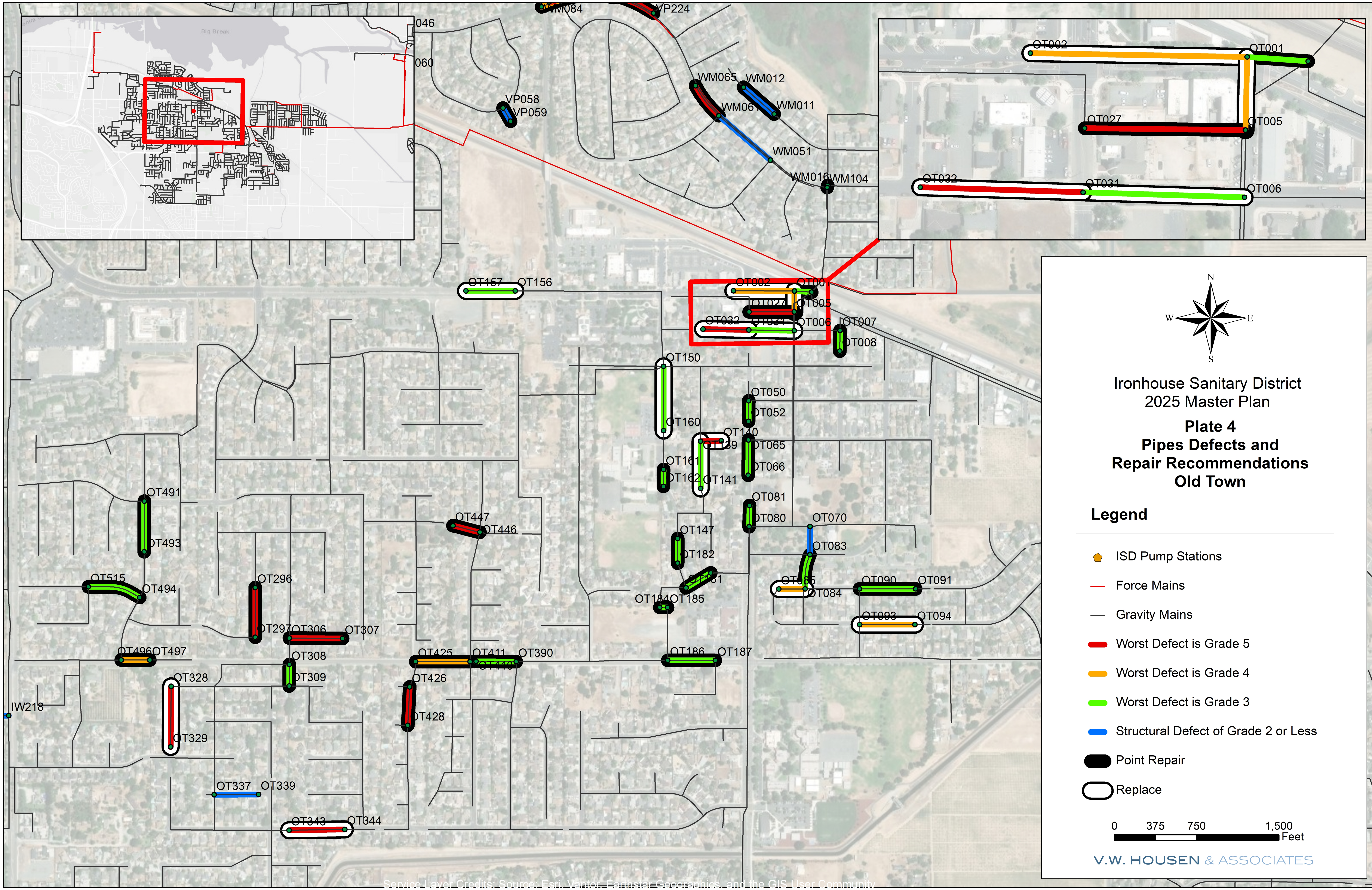
Ironhouse Sanitary District
2025 Master Plan
Plate 1
Pipes Defects and
Repair Recommendations
Vintage Parkway
Legend
ISD Pump Stations
Force Mains
Gravity Mains
Worst Defect is Grade 5
Worst Defect is Grade 4
Worst Defect is Grade 3
Structural Defect of Grade 2 or Less
Point Repair
Replace
0 250 500 1,000
Feet
N
S
E
W
V.W. HOUSEN & ASSOCIATES
Big Break
VP223
WM083
VP224
VP188
VP181
VP180
VP132
VP130
VP129
VP128
VP120
VP121
VP210
VP205
VP150
VP105
VP085
VP151
VP046
VP060
IW037
IW004
IW003
W036
VP058
VP059
WM065
WM012
WM061
WM011
IW074
WM051
Service Layer Credits: Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community



Ironhouse Sanitary District
2025 Master Plan
Plate 2
Pipes Defects and
Repair Recommendations
Walnut Meadows
Legend
ISD Pump Stations
Force Mains
Gravity Mains
Worst Defect is Grade 5
Worst Defect is Grade 4
Worst Defect is Grade 3
Structural Defect of Grade 2 or Less
Point Repair
Replace
0
250
500
1,000
Feet
V.W. HOUSEN & ASSOCIATES
VP223
WM083
WM084
VP224
WM065
WM012
WM061
WM011
WM051
WM016
WM104
VP058
VP059
OT156
OT002
OT001
OT000
Big Break
Service Layer Credits: Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community



VP205
VP150
VP105
VP085
VP046
IW037
IW004
IW003
IW074
N
W
E
S
Ironhouse Sanitary District
2025 Master Plan
Plate 3
Pipes Defects and
Repair Recommendations
Ironwood
Legend
ISD Pump Stations
Force Mains
Gravity Mains
Worst Defect is Grade 5
Worst Defect is Grade 4
Worst Defect is Grade 3
Structural Defect of Grade 2 or Less
Point Repair
Replace
0 150 300 600
Feet
V.W. HOUSEN & ASSOCIATES
Service Layer Credits: Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community


Ironhouse Sanitary District
2025 Master Plan
Plate 4
Pipes Defects and
Repair Recommendations
Old Town
Legend
ISD Pump Stations
Force Mains
Gravity Mains
Worst Defect is Grade 5
Worst Defect is Grade 4
Worst Defect is Grade 3
Structural Defect of Grade 2 or Less
Point Repair
Replace
0 375 750 1,500
Feet
V.W. HOUSEN & ASSOCIATES
N
S
E
W
Service Layer Credits: Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community

Appendix C

Project Information, Priorities, and Costs

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Rehabilitation and Replacement Recommended Actions and Project Costs

Pipe ID	Node US	Node DS	Structural Defect	PACP Quick Rating - Structural	O&M Defect	PACP Quick Rating - O&M	Diameter	Length (ft)	Length Surveyed (ft)	Location	Count of Grade 3,4,5	# Defects per 100 Ft	Action	Cost	Cost if Replaced
OT156_OT155	OT156	OT155	BVV	5341	RFJ	1100	6	12	347.8	Main St	4	33.33	Replace	$ 20,000	$ 20,000
OT002_OT001	OT002	OT001	MCU	423B	DAGS	2414	6	266	432.8	Main St	21	7.91	Replace	$ 173,956	$ 173,956
OT160_OT150	OT160	OT150	CM	3A22	RFJ	1100	6	201	456.6	Norcross Ln	14	6.98	Replace	$ 131,433	$ 131,433
OT141_OT139	OT141	OT139	CM, MWLS	3A29	-	0000	6	271	344.5	Landis St	14	5.17	Replace	$ 177,494	$ 177,494
OT085_OT084	OT085	OT084	MWLS, RPRD, RPLD	4421	DAGS	2200	6	134	185.6	Walnut St	4	2.99	Replace	$ 87,731	$ 87,731
OT157_OT156	OT157	OT156	CM	3414	RFL, RFJ	1500	6	148	367.5	Main St	4	2.70	Replace	$ 97,002	$ 97,002
OT031_OT006	OT031	OT006	CM	3821	RMJ	3119	6	318	333.2	W Acme St	8	2.52	Replace	$ 208,026	$ 208,026
OT329_OT328	OT329	OT328	DFBI, DFBR	5241	RBJ	4100	6	187	294.5	Redwood St	4	2.14	Replace	$ 122,719	$ 122,719
OT140_OT139	OT140	OT139	BSV, BVV	5343	DAGS	2200	6	292	145.8	W Home St	6	2.05	Replace	$ 191,318	$ 191,318
VP132_VP128	VP132	VP128	DFC	5100	-	0000	6	57	262.6	Korbel Ct	1	1.77	Replace	$ 37,019	$ 37,019
OT524_OT496	OT524	OT496	MWLS	3200	DSGV	5131	8	148	268.6	Cypress Rd	2	1.35	Replace	$ 97,035	$ 97,035
OT032_OT031	OT032	OT031	BSV	5241	RML	3214	6	378	323.1	Acme Street	5	1.32	Replace	$ 247,731	$ 247,731
OT162_OT161	OT162	OT161	JSM, CM	3322	RFJ	1800	6	245	418.8	Norcross Ln	3	1.23	Replace	$ 160,327	$ 160,327
VP105_VP085	VP105	VP085	MWLS	3200	-	0000	8	196	266.6	Country Ln	2	1.02	Replace	$ 128,288	$ 128,288
OT344_OT343	OT344	OT343	D	5142	-	0000	6	297	378.1	Chianti Way	3	1.01	Replace	$ 194,398	$ 194,398
OT094_OT093	OT094	OT093	JOLD, JOL	4232	-	0000	6	418	393.9	Almond St	4	0.96	PR	$ 78,000	$ 273,808
OT307_OT306	OT307	OT306	DFBI	5124		0000	6	106	346.2	Gamay St	1	0.94	PR	$ 19,500	$ 69,517
VP221_VP	VP221	VP	DFBI	5132	-	0000	8	319	345.6	Vintage Pkwy	3	0.94	PR	$ 58,500	$ 208,812
OT005_OT001	OT005	OT001	B, FM	4321	DAGS	2200	8	353	143.4	2nd St	3	0.85	PR	$ 58,500	$ 231,089
OT081_OT080	OT081	OT080	JOM	3121	DAGS, MWLS,	2500	6	118	148.1	Ohara St	1	0.85	PR	$ 19,500	$ 77,117
OT519_OT518	OT519	OT518	BSV	5100	-	0000	8	119	117.7	Sautrene St	1	0.84	PR	$ 19,500	$ 77,805
OT425_OT411	OT425	OT411	B	4100	-	0000	8	123	396	Cypress Rd	1	0.81	PR	$ 19,500	$ 80,459
VP223_VP222	VP223	VP222	MWLS	3224	-	0000	8	246	296.7	Merlot Ln	2	0.81	PR	$ 39,000	$ 161,310
OT497_OT496	OT497	OT496	D	4131	DAGS	2400	8	253	212.3	Cypress Rd	2	0.79	PR	$ 39,000	$ 165,720
OT309_OT308	OT309	OT308	MWLS	3226	-	0000	6	283	152.1	Ponderosa/Cypress Easement	2	0.71	PR	$ 39,000	$ 185,291
OT027_OT005	OT027	OT005	BVV	5141	RFJ	1100	6	295	323.4	2nd St	2	0.68	PR	$ 39,000	$ 192,956
OT493_OT491	OT493	OT491	JOM	3100	-	0000	8	150	362.8	Burgandy St	1	0.67	PR	$ 19,500	$ 98,018
OT447_OT446	OT447	OT446	D	5121	-	0000	6	151	174.3	Mallard Ln	1	0.66	PR	$ 19,500	$ 99,027
WM012_WM011	WM012	WM011	-	0000	IGL	5100	8	152	292	Fetzer St	1	0.66	PR	$ 19,500	$ 99,656
WM084_WM083	WM084	WM083	MWLS	4200	DAGS	3100	21	155	542.2	Walnut Meadows Dr	1	0.64	PR	$ 19,500	$ 267,272
OT091_OT090	OT091	OT090	JOM	3200	-	0000	6	346	402.6	Walnut Drive	2	0.58	PR	$ 39,000	$ 226,437
OT008_OT007	OT008	OT007	CM	3212	-	0000	6	346	147.9	3rd St	2	0.58	PR	$ 39,000	$ 226,830
OT187_OT186	OT187	OT186	MWLS	312A	DSGV	2300	8	174	333.1	Cypress St	1	0.57	PR	$ 19,500	$ 114,201
VP121_VP120	VP121	VP120	DFC	5100	VZ	1100	6	180	306.8	Chandon Ct	1	0.55	PR	$ 19,500	$ 118,133
OT181_OT180	OT181	OT180	JOM	3100	DAGS	2114	6	186	219.5	Francisco Villa Rd	1	0.54	PR	$ 19,500	$ 121,605
IW037_IW036	IW037	IW036	-	0000	RBB	5131	18	208	208.1	Carol St	1	0.48	PR	$ 19,500	$ 306,781
OT182_OT147	OT182	OT147	CM	3122	-	0000	6	220	180.3	Norcross Ln	1	0.46	PR	$ 19,500	$ 143,816
OT084_OT083	OT084	OT083	MWLS	3200	-	0000	8	464	248.1	Walnut Drive	2	0.43	PR	$ 39,000	$ 304,144
VP188_VP180	VP188	VP180	JOM	3200	-	0000	8	484	246.2	Weibel Circle	2	0.41	PR	$ 39,000	$ 317,051
VP060_VP046	VP060	VP046	-	0000	IRL	4100	15	248	282.8	Walnut Meadows Dr	1	0.40	PR	$ 19,500	$ 304,791
VP210_VP208	VP210	VP208	DFBI	5100	-	0000	6	267	133.9	Beringer Way	1	0.38	PR	$ 19,500	$ 174,676
WM065_WM061	WM065	WM061	DFBI	5100	-	0000	8	363	270.9	Walnut Meadows Dr	1	0.37	PR	$ 19,500	$ 237,707
OT297_OT296	OT297	OT296	DFBI	5225	-	0000	8	542	352.7	Port St	2	0.37	PR	$ 39,000	$ 355,249
VP224_VP223	VP224	VP223	DFBI	5127	-	0000	8	281	417.9	Walnut Meadows Dr	1	0.36	PR	$ 19,500	$ 184,308
OT052_OT050	OT052	OT050	JOM	3112	RMJ	3221	6	283	147	Ohara St	1	0.35	PR	$ 19,500	$ 185,487
VP181_VP180	VP181	VP180	JOM	3100	ISSR	2200	8	285	281.3	Concannon Dr	1	0.35	PR	$ 19,500	$ 186,470
OT082_OT083	OT082	OT083	JOM	3100	DAGS	413E	6	307	428.1	Las Dunas Ave	1	0.33	PR	$ 19,500	$ 201,015
IW004_IW003	IW004	IW003	IG	5100	-	0000	18	327	326.7	Fairhaven Court	1	0.31	PR	$ 19,500	$ 481,621
IW074_IW073	IW074	IW073	JOM	3122	-	0000	8	327	332.4	Teresa St	1	0.31	PR	$ 19,500	$ 214,218
WM104_WM016	WM104	WM016	CC	1100	IGJ	5141	21	328	12	Jordan St	1	0.31	PR	$ 19,500	$ 563,439
OT428_OT426	OT428	OT426	HVV	5124	-	0000	8	329	283.1	Lorenzetti Dr	1	0.30	PR	$ 19,500	$ 215,764
VP208_VP205	VP208	VP205	DFBI	5122	DAR	4121	8	332	464.2	Walnut Meadows Dr	1	0.30	PR	$ 19,500	$ 217,788
VP151_VP150	VP151	VP150	-	0000	IR	4000	8	333	187.3	Delta Ranch Rd	1	0.30	PR	$ 19,500	$ 218,247
OT508_OT507	OT508	OT507	DFBI	5129	RFJ	1100	6	339	527.6	Claret Ct	1	0.29	PR	$ 19,500	$ 222,309
Las Dunas_La Brea Way	Las Dunas	La Brea Way	FM	4122	RML, RMJ	3321	8	350	244.7	Las Dunas Ave	1	0.29	PR	$ 19,500	$ 229,372

Rehabilitation and Replacement Recommended Actions and Project Costs

Pipe ID	Node US	Node DS	Structural Defect	PACP Quick Rating - Structural	O&M Defect	PACP Quick Rating - O&M	Diameter	Length (ft)	Length Surveyed (ft)	Location	Count of Grade 3,4,5	# Defects per 100 Ft	Action	Cost	Cost if Replaced
OT001_OT000	OT001	OT000	CM	3100	-	0000	8	353	114.5	Main St	1	0.28	PR	$ 19,500	$ 231,390
OT185_OT184	OT185	OT184	CM	3100	-	0000	8	394	56.5	Francisco Villa Rd	1	0.25	PR	$ 19,500	$ 258,083
OT246_OT245	OT246	OT245	DFBI	5124	-	0000	8	396	265.5	Gardeniea St	1	0.25	PR	$ 19,500	$ 259,459
VP130_VP129	VP130	VP129	JOM	3122	-	0000	8	403	318.7	Domaine St	1	0.25	PR	$ 19,500	$ 263,784
VP059_VP058	VP059	VP058	-	0000	IR	4100	6	428	106.1	Stony Hill Ct	1	0.23	PR	$ 19,500	$ 280,491
OT410_OT390	OT410	OT390	JOM	3124	-	0000	8	436	284.6	Cypress St	1	0.23	PR	$ 19,500	$ 285,962
OT515_OT494	OT515	OT494	JOM	3122	-	0000	8	458	383.1	Sautrene St	1	0.22	PR	$ 19,500	$ 300,114
OT066_OT065	OT066	OT065	CM	3100	JOM	3113	6	528	118.8	Ohara Ave	1	0.19	PR	$ 19,500	$ 345,684
OT083_OT070	OT083	OT070	CL	2100	-	0000	8	263	195.8	La Vina Way	0	0.00	None	$ -	N/A
IW206_IW199	IW206	IW199	-	0000	RFB	2100	8	327	327.6	Garden Ct	0	0.00	None	$ -	N/A
IW275_IW218	IW275	IW218	-	0000	RFB	2100	8	147	155.4	Bedford Lane	0	0.00	None	$ -	N/A
OT339_OT337	OT339	OT337	RFB, MGO	2200	-	0000	6	212	317.5	Pine Ct	0	0.00	None	$ -	N/A
WM061_WM051	WM061	WM051	-	0000	RML	3100	8	383	483.9	Walnut Meadows	0	0.00	None	$ -	N/A
Totals							**in Feet**	**18963**					**Subtotal**	**$ 3,322,478**	
							in Miles	**3.59**					**If 15 percent of PR are converted to Replace**	**$ 1,587,643**	
													Total	**$ 4,910,121**	