CITY OF MORGAN HILL

April 2024

City of Morgan Hill

Wastewater Collection System Master Plan Update

AKEL
ENGINEERING GROUP, INC.

FINAL

CITY OF MORGAN HILL

2024

WASTEWATER COLLECTION SYSTEM MASTER PLAN UPDATE

FINAL

April 2024

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Smart Planning Our Water Resources

April 23, 2024

FINAL

City of Morgan Hill
17575 Peak Avenue
Morgan Hill, CA 95037

Attention: James F. Sylvain, P.E.
Deputy Director of Utilities Services

Subject: 2024 Wastewater Collection System Master Plan Update – Final Report

Dear James,

We are pleased to submit one (1) digital copy of the final report for the City of Morgan Hill's Wastewater Collection System Master Plan Update. This report summarizes the City's existing collection system facilities, planning area characteristics, design criteria, and hydraulic modeling analysis to recommend a Capital Improvement Program for the 2035 General Plan horizon.

The model development phase included integrating all-pipes from the City's GIS database and calibrating flows based on the 2023 Flow Monitoring Program. A capacity evaluation was subsequently performed to identify potential deficiencies in the collection system. The recommended improvements consist of both hydraulic capacity and rehabilitation projects with an opinion of probable construction costs.

We are extending our thanks to you; Chris Ghione, Director of Public Services; Maria Angeles, Senior Civil Engineer, and other City staff whose courtesy and cooperation were valuable components in completing this Master Plan Update.

Sincerely,

AKEL ENGINEERING GROUP, INC.

Tony Akel, P.E., D. WRE
President

Enclosure: 2024 Wastewater Collection System Master Plan Update

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CITY OF MORGAN HILL

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APPENDICES

LIST OF ABBREVIATIONS

AACE International	Association for the Advancement of Cost Engineering International
ADWF	Average Dry Weather Flow
CCI	Construction Cost Index
CIP	Capital Improvement Program
City	City of Morgan Hill
CoF	Consequence of Failure
d/D	Depth of Flow to Pipe Diameter Ratio
DU	Dwelling Units
EDU	Equivalent Dwelling Units
ENR	Engineering News Record
fps	Feet per Second
ft	Feet
GIS	Geographic Information Systems
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
HGL	Hydraulic Grade Line
I&I	Infiltration and Inflows
IDF	Intensity Duration Frequency
in/hr	Inch per Hour
JPA	Joint Powers Agreement
LoF	Likelihood of Failure
mgd	Million Gallons per Day
NASSCO	National Association of Sewer Service Companies
NOAA	National Oceanic and Atmospheric Administration
PDWF	Peak Dry Weather Flow
PWWF	Peak Wet Weather Flow
R&R	Rehabilitation and Replacement
SCRWA	South County Regional Wastewater Authority
WCSMP	Wastewater Collection System Master Plan
WWTP	Wastewater Treatment Plant

UNIT CONVERSIONS

Volume Unit Calculations

To Convert From:	To:	Multiply by:
acre feet	gallons	325,851
acre feet	cubic feet	43,560
acre feet	million gallons	0.3259
cubic feet	gallons	7.481
cubic feet	acre feet	2.296×10^{-5}
cubic feet	million gallons	7.481×10^{-6}
gallons	cubic feet	0.1337
gallons	acre feet	3.069×10^{-6}
gallons	million gallons	1×10^{-6}
million gallons	gallons	1,000,000
million gallons	cubic feet	133,672
million gallons	acre feet	3.069

Flow Rate Calculations

To Convert From:	To:	Multiply By:
acre feet per year	million gallons per day	8.93×10^{-4}
acre feet per year	cubic feet per second	1.381×10^{-3}
acre feet per year	gallons per minute	0.621
acre feet per year	gallons per day	892.7
cubic feet per second	million gallons per day	0.646
cubic feet per second	gallons per minute	448.8
cubic feet per second	acre feet per year	724
cubic feet per second	gallons per day	646,300
gallons per day	million gallons per day	1×10^{-6}
gallons per day	cubic feet per second	1.547×10^{-6}
gallons per day	gallons per minute	6.944×10^{-4}
gallons per day	acre feet per year	1.12×10^{-3}
gallons per minute	million gallons per day	1.44×10^{-3}
gallons per minute	cubic feet per second	2.228×10^{-3}
gallons per minute	acre feet per year	1.61
gallons per minute	gallons per day	1,440
million gallons per day	cubic feet per second	1.547
million gallons per day	gallons per minute	694.4
million gallons per day	acre feet per year	1,120
million gallons per day	gallons per day	1,000,000

AT-A-GLANCE

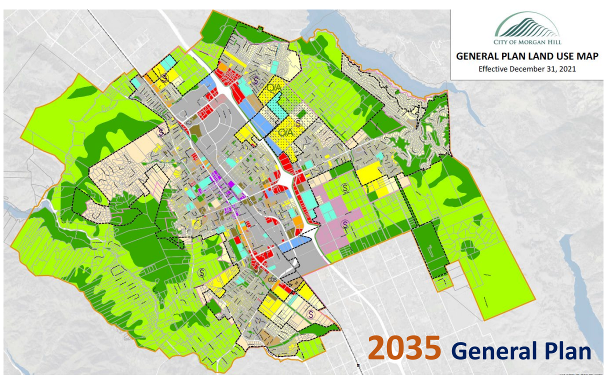
1. MASTER PLAN OBJECTIVES

The 2024 Wastewater Collection System Master Plan (WCSMP) Update was initiated to assure safe and efficient operation of the City's collection system. This Update provides a capacity assessment and recommends infrastructure needed to maintain the target level of service.

What does the 2024 WCSMP address?
What wastewater infrastructure improvements need to be constructed?
How are we addressing aging wastewater infrastructure?
When do we need them?
How much do they cost?

2. PLANNING AREA CHARACTERISTICS

Buildout land uses are consistent with the City's 2035 General Plan, which focuses on balancing future housing and employment needs to achieve sustainable growth.



3. SYSTEM PERFORMANCE & DESIGN CRITERIA

The system performance criteria define the adequate levels of service for gravity pipes, lift stations and force mains. These criteria were used to evaluate system capacity and identify future improvements.

What Defines an Adequate Level of Service?

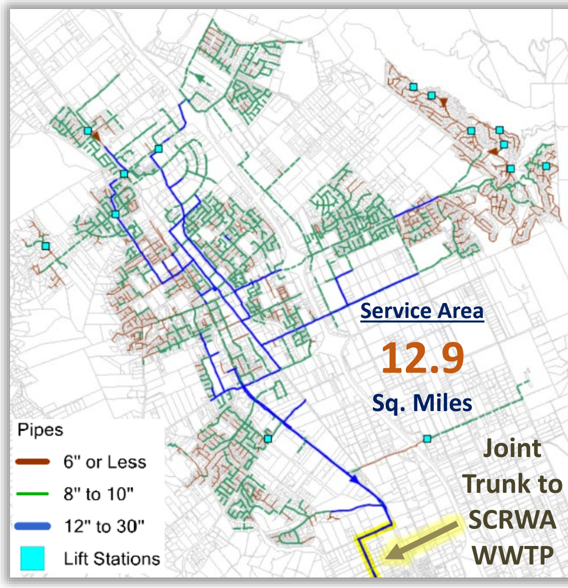
Asset	Condition	Criteria
Gravity Pipes	Dry	Flow Depth <= 92% Full
	Wet	Freeboard >= 3 feet
Lift Stations	Wet	Convey peak flow with 1 standby pump
Force Mains	Wet	Max. Velocity = 10 ft/s

4. EXISTING WASTEWATER SYSTEM FACILITIES

The City owns and maintains a collection system comprised of 164 miles of pipes and 14 lift stations:

What are the Main Elements of the Wastewater Collection System?	
159	Miles of Gravity Pipes
3	Miles of Force Mains
2	Miles of Siphons
3,800+	Manholes
14	Lift Stations

The overall system discharges into a Joint Trunk that conveys the City's wastewater to the SCRWA WWTP.



5. WASTEWATER FLOWS

Existing wastewater flows were quantified from the 2023 Flow Monitoring Program whereas future wastewater flows were projected based on the 2035 General Plan land uses.

How much wastewater does the City convey to the SCRWA WWTP? What are future flow projections?	
Existing Average Dry Weather Flow	**2.8 mgd**
Future Average Dry Weather Flow	**4.3 mgd**

6. HYDRAULIC MODEL DEVELOPMENT

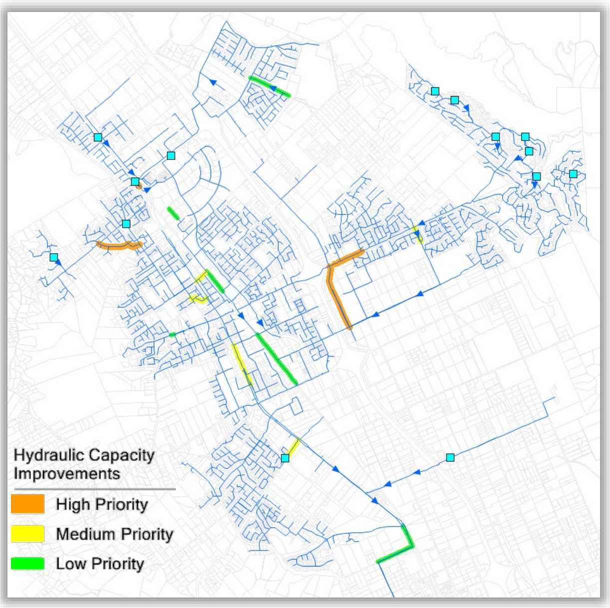
The City's GIS database was used to develop a **Digital Twin** of the collection system. A thorough calibration was performed to mimic current conditions and instill a strong level of confidence in the modeled flows.

How was the Digital Twin Developed?	
1	Physical attributes were based on GIS. (pipe diameter, inverts etc.)
2	Operational attributes were based on the 2023 Flow Monitoring Program.
3	Peak dry and wet weather flows were calibrated to mimic current conditions at 11 Sites, including the Joint Trunk.

7. CAPACITY EVALUATION & IMPROVEMENTS

A capacity evaluation was completed to identify system deficiencies, potential bottlenecks, and locations susceptible to overflows.

Hydraulic capacity improvements were recommended and prioritized into 3 categories, as shown on the following graphic.



8. REHABILITATION AND REPLACEMENT PROGRAM

The City has developed a comprehensive R&R program to address aging infrastructure based on condition and risk analyses. This Program is supported by routine CCTV inspections, lift station condition assessments and annual projects that target high risk infrastructure.

9. CAPITAL IMPROVEMENT PROGRAM

The Capital Program (CIP) provides a list of both hydraulic capacity and rehabilitation improvements for the **2035 planning horizon**. The CIP also provides AACE Class 5 opinion of probable construction costs, itemized by projects.

What is the Cost Estimating Methodology?	
1	Class 5 Opinion of Probable Construction Cost
2	Unit Costs applied to Project Quantities ($/ft)
3	Construction Contingency Allowance of 30%
4	Project Related Cost Allowance of 30%
5	20-City Average ENR CCI of 13,532 (April 2024).

The total CIP implementation costs are estimated at approximately **$138.1 Million dollars**, distributed as follows:

How Much does the CIP Cost?

Suggested Implementation Schedule	Cost (Millions)
A. Hydraulic Capacity Improvements	
Imminent / Under Design	$68.5
2024 – 2026	$2.4
2027 - 2030	$4.5
2031 - 2035	$2.2
Beyond 2035 (Long-Term)	$9.4
Subtotal	**$87.0**
B. Rehabilitation Improvements	
Imminent / Under Design	$2.3
2024 – 2026	$7.0
2027 - 2030	$13.9
2031 - 2035	$19.6
Subtotal	**$42.8**
C. Existing Joint Trunk Improvements	
2027 - 2030	$1.2
2031 - 2035	$7.1
Subtotal	**$8.3**
Capital Improvement Program Cost	
Total	**$138.1**

EXECUTIVE SUMMARY

This executive summary presents a background of the City of Morgan Hill's wastewater collection system, the planning area characteristics, the design criteria, and the development of a GIS-based hydraulic model.

The hydraulic model was calibrated based on observations from the 2023 Flow Monitoring Program and used to evaluate the capacity adequacy of the existing system. A prioritized Capital Improvement Program was subsequently developed to mitigate capacity deficiencies and rehabilitate aging infrastructure.

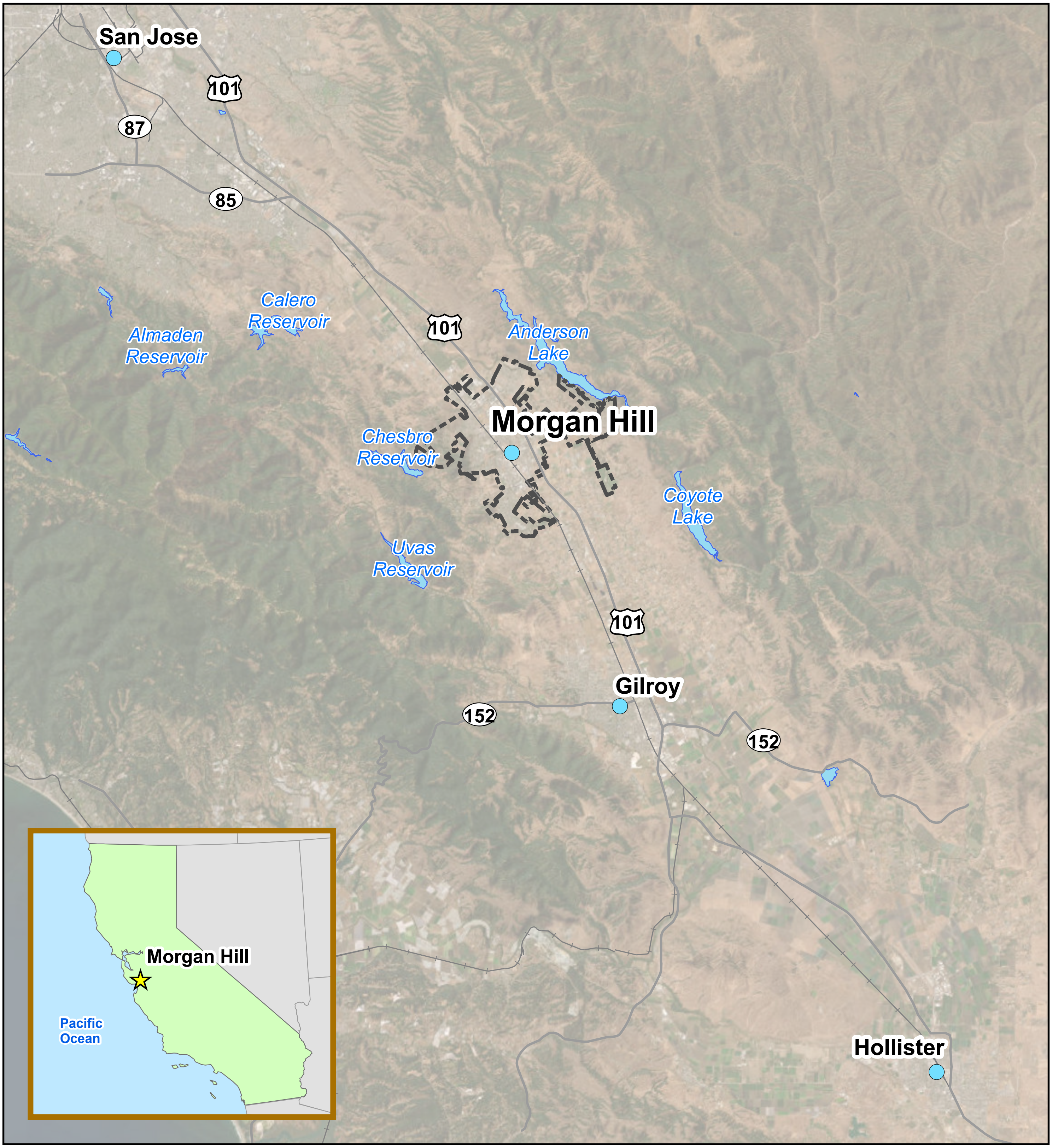
ES.1 STUDY OBJECTIVES

The City of Morgan Hill (City) initiated the 2024 Wastewater Collection System Master Plan Update to develop, finance, and continue to provide reliable wastewater collection services to both current and future customers. Key objectives of this Update are as follows:

- Summarize the City's existing wastewater collection system facilities.
- Document growth assumptions based on the City's 2035 General Plan.
- Summarize the wastewater collection system performance and design criteria.
- Project future wastewater flows for the General Plan horizon.
- Develop a Geographic Information Systems (GIS)-based hydraulic model of the City's wastewater collection system.
- Complete a Flow Monitoring Program to collect data from 11 strategic sites and calibrate the hydraulic model.
- Evaluate system capacity under existing and future flow conditions.
- Identify capacity improvements needed to maintain the target level of service.
- Document the City's Rehabilitation and Replacement (R&R) Program improvements.
- Recommend a Capital Improvement Program with an opinion of probable construction costs.
- Prepare a 2024 Wastewater Collection System Master Plan Update Report.

ES.2 STUDY AREA DESCRIPTION

The City is located in the Santa Clara County, approximately 22 miles southeast from the City of San Jose's downtown and 10 miles north from the City of Gilroy. A regional location map illustrating the neighboring cities is displayed on **Figure ES.1**.



Legend

- Cities
- Highways
- Railroads
- City Limits
- Lakes

Updated: April 2, 2024

0 0.5 1 2
Miles

N
W GIS E
S

Figure ES.1
Regional Location Map

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Final\Fig_ES.1_RLM_040224.aprx

The City owns and maintains a wastewater collection system with a service area of approximately 12.9 square miles. Based on the natural topography, the City is mostly flat at the center with steeper slopes along the foothills at the east and west boundaries. The City limits, Urban Growth Area Boundary and the Sphere of Influence are displayed on **Figure ES.2** for reference purposes.

ES.3 SYSTEM PERFORMANCE AND DESIGN CRITERIA

System performance and design criteria were developed to assure safe and efficient operation of the wastewater collection system. The hydraulic criteria for gravity pipes, lift stations and force mains are listed on **Table ES.1** and summarized in the following sections.

Gravity Pipe Criteria

The depth of flow to pipe diameter ratio (d/D) is used as a key hydraulic indicator to evaluate the conveyance capacity in gravity pipes. For circular pipes, the highest capacity is generally reached at 92 percent of the full height of the pipe (d/D = 0.92). Therefore, the d/D ratio in existing pipes should not exceed 0.92 during dry weather conditions.

During wet weather conditions, pipes are allowed to surcharge (d/D = 1) provided the Hydraulic Grade Line remains 3 feet below the manhole ground elevation. This criterion is implemented to avoid premature replacements while minimizing the risk of overflows.

Lift Station and Force Main Criteria

Wastewater lift stations are evaluated and designed to accommodate the peak wet weather flow with the largest pump out-of-service. This is typically known as the firm capacity of a lift station. The standby pump provides a safety factor in case the duty pump malfunctions or requires maintenance. Force mains are designed to flow at a minimum self-cleansing velocity of 2 feet per second and a maximum velocity of 10 feet per second.

ES.4 EXISTING WASTEWATER COLLECTION SYSTEM

The City owns and maintains a wastewater collection system comprised of 164 miles of pipes and 14 lift stations, as shown on **Figure ES.3**.

Infrastructure Inventory	
159	Miles of Gravity Pipes
3	Miles of Force Mains
2	Miles of Siphons
3,800+	Manholes
14	Lift Stations

The wastewater pipes vary in size from 4-inches to 30-inches in diameter, and service approximately 46,000 residents through more than 12,500 lateral connections.

Ultimately, wastewater collected from the City is conveyed south via a 12-mile Joint Trunk system that discharges into a Wastewater Treatment Plant in the City of Gilroy. This plant is owned and operated by the South County Regional Wastewater Authority (SCRWA) under a Joint Powers Agreement (JPA) between the City of Morgan Hill and the City of City of Gilroy. Similarly, the Joint

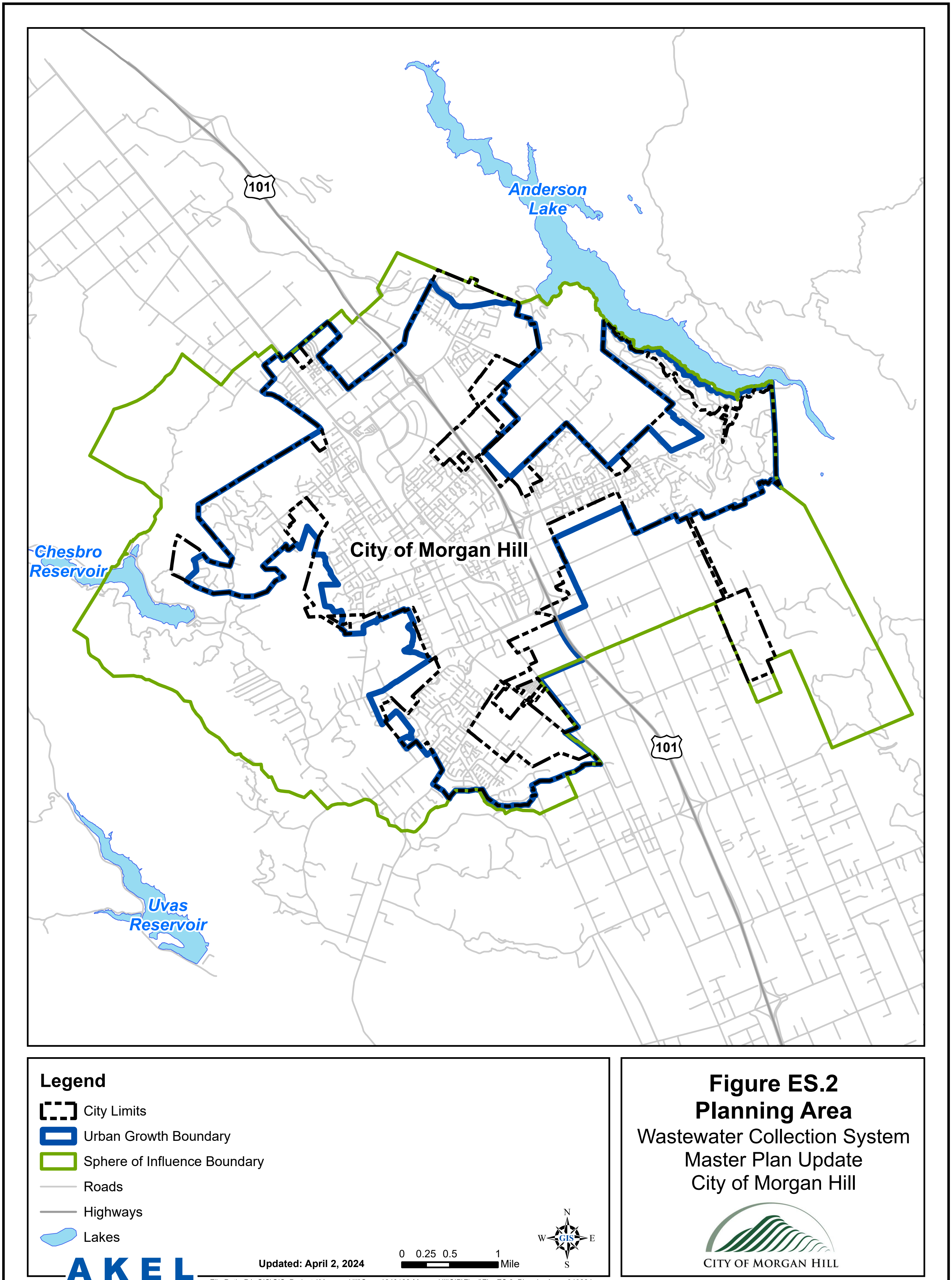

Figure ES.2
Planning Area
Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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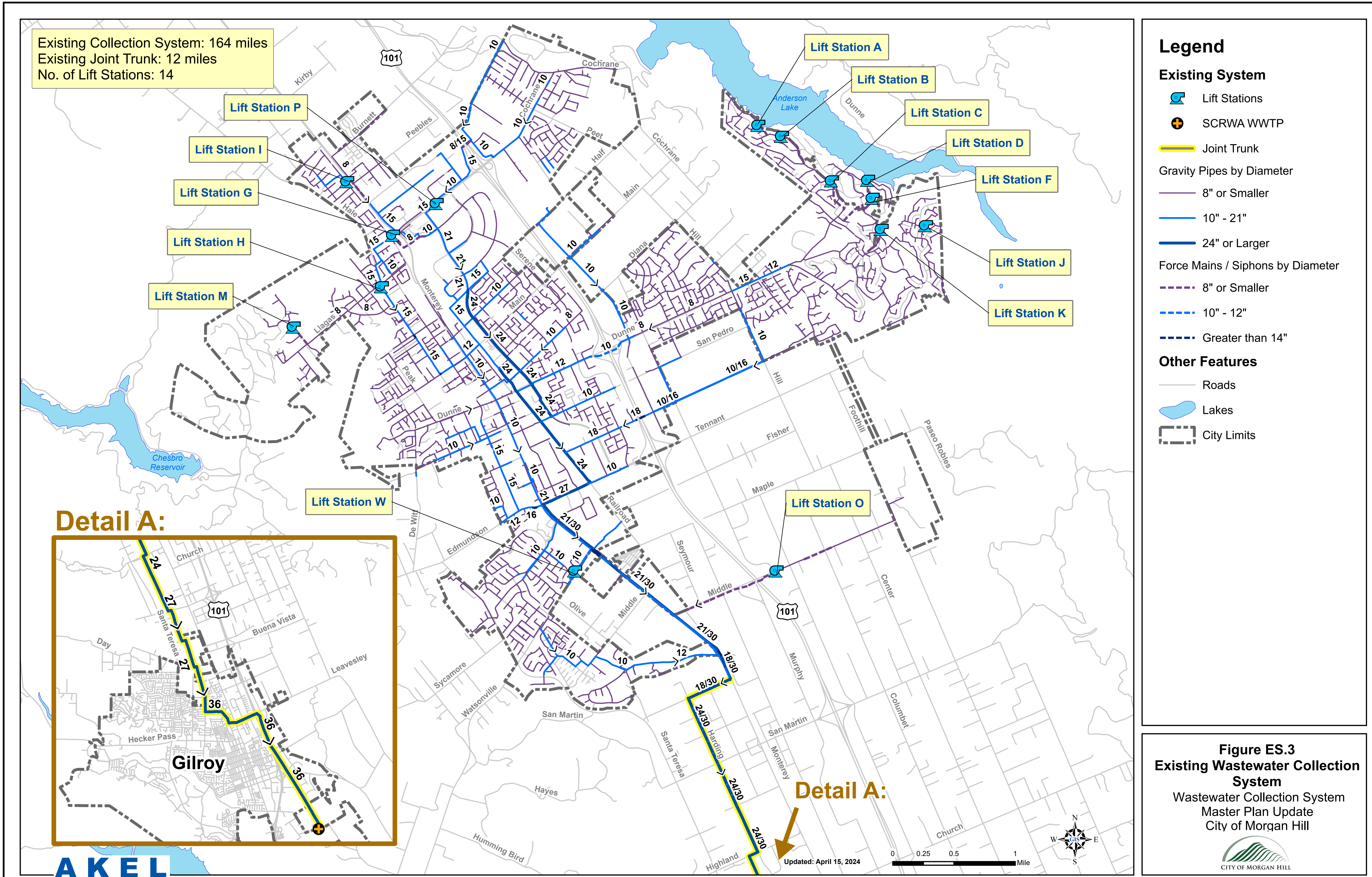

Figure ES.3
Existing Wastewater Collection System
Wastewater Collection System
Master Plan Update
City of Morgan Hill

Table ES.1 System Performance and Design Criteria

Wastewater Collection System Master Plan Update
City of Morgan Hill

Gravity Pipe Criteria [1]

Peak Dry Weather Flow Conditions

Diameter (in)	Maximum Allowable d/D Existing Pipes	Maximum Allowable d/D Proposed Pipes
6 to 10"	**0.92**	**0.50**
12" or Higher	**0.92**	**0.75**

Peak Wet Weather Flow Conditions

Hydraulic Grade Line (HGL) should be at least **3 feet** below the manhole ground elevation.

Minimum Design Slope and Capacity [2]

Pipe Size (in)	Minimum Slope (%)	Full Flow Capacity (n = 0.013) (mgd)	Half Full Velocity (ft/s)
8''	0.400%	0.49	2.19
10''	0.280%	0.75	2.12
12''	0.220%	1.08	2.13
15''	0.150%	1.62	2.04
18''	0.120%	2.35	2.06
21''	0.100%	3.24	2.08
24''	0.080%	4.13	2.04
27''	0.067%	5.18	2.02
30''	0.058%	6.38	2.01
33''	0.052%	7.79	2.03
36''	0.046%	9.24	2.02
42''	0.037%	12.51	2.01

Lift Station and Force Main Criteria [1]

Lift Station shall be sized to accommodate:	**Peak Wet Weather Flow with Largest Pump Out of Service** (Firm Capacity)
Force Main Velocity:	**2 to 10 ft/s**

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Notes:

1. Recommended based on typical industry standards and a survey of local municipalities.
2. Recommended based on the Ten State Wastewater Collection System Design Standards.

Trunk, south of Highland Avenue, is maintained by both cities based on the capacity allocations documented in the JPA.

An overall wastewater infrastructure inventory listing the pipe length by diameter is also shown on **Table ES.2**. The inventory indicates that 97 percent of the collection system consists of gravity pipes whereas the remaining 3 percent consists of force mains and siphons. Additionally, 6-inch and 8-inch diameter pipes account for approximately 80 percent of the total pipe length.

ES.5 WASTEWATER FLOWS

Existing wastewater flows in the City's collection system were quantified from the 2023 Flow Monitoring Program, which measured flows at 11 strategic locations, including the Joint Trunk. The existing wastewater flows were quantified as follows:

- **Average Dry Weather Flow (ADWF).** Represents the daily average flow during dry weather conditions. This flow was quantified at approximately 2.8 mgd.
- **Peak Dry Weather Flow (PDWF).** Represents the hourly peak flow during dry weather conditions. This flow was quantified at approximately 4.7 mgd.
- **Peak Wet Weather Flow (PWWF).** Represents the hourly peak flow during wet weather conditions simulated through a synthetic 10-Year 24-Hour design storm event. This flow was estimated at approximately 13.9 mgd. It should be noted that the hydraulic analysis indicated overflows at select locations during the wet weather simulation. The gravity pipes in these locations were upsized to capture design peak flows in the system.

Dry weather flows are largely influenced by customer uses and vary depending on weekdays and weekends. In contrast, wet weather flows are influenced by the severity of storm events and corresponding Infiltration and Inflows (I&I) that enter the collection system.

Future wastewater flows were estimated from land uses in the City's 2035 General Plan and unit flow factors developed through calibration. This land-use based approach accounts for urbanization of undeveloped lands as well as re-development or intensification in specific growth areas.

The future ADWF, PDWF and PWWF were projected to be 4.3 mgd, 7.3 mgd and 15.2 mgd, respectively. It should be noted that the future wet weather flows include a 20 percent reduction in I&I to account for the City's planned R&R improvements. A wastewater flow summary of existing and future flows is shown on **Table ES.3**.

ES.6 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION

A hydraulic model is an analytical tool that combines physical and operational characteristics of a wastewater collection system. The hydraulic model then solves a series of equations to simulate

Table ES.2 Wastewater Infrastructure Inventory

Wastewater Collection System Master Plan Update
City of Morgan Hill

Pipe Diameter (in)	Total Length [1] (feet)	(miles)
Gravity Pipes		
4'' or Smaller	1,965	0.4
6''	284,936	54.0
8''	375,629	71.1
10''	78,442	14.9
12''	17,813	3.4
14''	456	0.1
15''	23,421	4.4
16''	7,065	1.3
18''	6,096	1.2
20''	491	0.1
21''	14,444	2.7
24''	12,982	2.5
27''	2,396	0.5
30''	10,360	2.0
Subtotal	**836,495**	**158.4**
Force Mains		
4'' or Smaller	6,322	1.2
6''	8,703	1.6
8''	1,490	0.3
Subtotal	**16,515**	**3.1**
Siphons		
6''	1,168	0.2
8''	2,846	0.5
10''	3,194	0.6
12'' or Higher	3,609	0.7
Subtotal	**10,817**	**2.0**
Joint Trunk (Shared Ownership between City of Morgan Hill and City of Gilroy)		
21'' or Smaller	2,681	0.5
24''	14,997	2.8
27''	9,684	1.8
30''	12,507	2.4
36'' or Higher	22,368	4.2
Subtotal	**62,237**	**11.8**
Summary (Excluding Joint Trunk)		
	Total Pipe Length	**163.6**
	Number of Manholes / Cleanouts	**3,888**
	Number of Siphons	**25**
	Number of Lift Stations / Force Mains	**14**

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Note:

1. Inventory was tabulated from the City's GIS database received on April 3, 2023, and updated through discussions with the City staff to include recently constructed infrastructure.

Table ES.3 Wastewater Collection System Flows

Wastewater Collection System Master Plan Update
City of Morgan Hill

Hydraulic Model Scenarios	Average Dry Weather Flow ADWF (mgd)	Peak Dry Weather Flow PDWF (mgd)	Peak Wet Weather Flow PWWF (10-Year 24-Hour Design Storm) (mgd)
Existing Conditions [1,2]	2.8	4.7	13.9
Future Conditions [3]	4.3	7.3	15.2

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Notes:

1. Existing flows were obtained from the permanent meter located along the Joint Trunk at the intersection of Harding Avenue and Highland Avenue.
2. Select pipes within the upstream tributary areas were upsized to alleviate hydraulic model overflows and capture the design peak wet weather flow.
3. Future conditions include buildout flow projections and a 20% Infiltration and Inflow reduction to account for the City's planned R&R improvements.

flows in pipes, including backwater calculations for surcharged conditions. The selection of a particular software ultimately depends on user preferences and costs for purchasing the software. InfoWorks ICM Ultimate by Autodesk Inc. was selected as the preferred hydraulic modeling software due to its ability to manage multiple scenarios and integrate GIS datasets. This software uses the fully dynamic St. Venant equations with a SWMM5 calculation engine for simulating backwater effects.

Model Development

The City's GIS database was used as the primary source to develop an all-pipes hydraulic model. This database contained key physical attributes such as pipe sizes, upstream and downstream manhole connections, and invert/ground elevations.

A thorough quality control review was completed to identify notable data gaps such as missing invert elevations and network connectivity issues. These data gaps were resolved through field verification surveys conducted by the City staff and a review of record drawings. In some cases, missing elevation data was populated from topographic information (Digital Elevation Model) or via linear interpolation, minimum design slopes and typical cover assumptions.

Model Calibration

Model calibration is an iterative process of comparing the model flows with observations and revising the input parameters until the predicted results are acceptable. This process is intended to instill a strong level of confidence in the hydraulic model results.

As part of this Master Plan Update, the City commissioned a 7-week Flow Monitoring Program from January 9th to February 22nd, 2023. The City retained V&A Consulting Engineers to deploy 10 temporary meters and collect data from 1 permanent meter located in the Joint Trunk. The hydraulic model was calibrated to reflect the following measured conditions:

- Peak dry weather flows from January to February 2023.
- Peak wet weather flows from January 2023 wet weather events.

After calibration, the hydraulic model was used to evaluate the capacity adequacy of the collection system under dry and wet weather conditions. The hydraulic model is a valuable investment that will continue to prove its worth to the City as future planning issues or other operational conditions surface. It is recommended that the model be maintained and updated with new construction projects to preserve its integrity.

ES.7 CAPACITY EVALUATION AND IMPROVEMENTS

The existing peak dry weather flow simulation indicated that the wastewater pipes are generally less than 75 percent full, with a few exceptions north of East Dunne Avenue. In contrast, the existing wet weather simulation indicated several capacity deficiencies dispersed across the system. These deficiencies can be attributed to the volume of I&I observed in select basins.

Similar to existing conditions, the future peak dry weather flow simulation indicated potential deficiencies north of East Dunne Avenue whereas the future peak wet weather flow simulation identified several locations with surcharged flow conditions.

The future simulations included two imminent hydraulic capacity improvements that are currently at the design phase (Relief Trunk and Condit Road Diversion). Additionally, the future simulations also included a 20 percent reduction in I&I to account for the City's planned R&R improvements.

The hydraulic capacity improvements needed to mitigate existing and future system deficiencies are illustrated in **Figure ES.4**. These improvements were discussed with City staff and prioritized to accommodate growth envisioned in the City's 2035 General Plan.

ES.8 REHABILITATION AND REPLACEMENT (R&R) PROGRAM

Since the completion of the 2017 Wastewater Collection System Master Plan, the City of Morgan Hill has developed a comprehensive R&R program to address aging infrastructure. This program was developed from condition and risk analyses documented in the following studies:

- **2021 Sanitary Sewer System Asset Management Plan.** This plan defines appropriate Likelihood of Failure (LoF) and Consequence of Failure (CoF) criteria for wastewater facilities and includes a risk assessment of the collection system. A decision tree is used to recommend prioritized R&R improvements with capital cost estimates.
- **2021 Joint Trunk Pipeline Condition Assessment Report.** This report was completed in coordination with the City of Gilroy and documents condition defects in the existing Joint Trunk system. This report is also used to recommend prioritized rehabilitation improvements with capital cost estimates.

The City currently maintains a list of known structural deficiencies and continues to implement R&R improvements on an annual basis. The R&R program is supported by routine CCTV inspections, lift station condition assessments and annual capital projects that target high risk infrastructure.

It should be noted that the CCTV inspections follow the NASSCO rating system established for pipelines, manholes and laterals. Additionally, the City's Private Sewer Lateral Inspection Ordinance also requires residents to inspect old service laterals and repair deficiencies before sale or re-modeling of properties.

ES.9 CAPITAL IMPROVEMENT PROGRAM

The Capital Improvement Program consists of both hydraulic capacity and rehabilitation improvements for the 2035 horizon. The wastewater infrastructure unit costs and contingencies established for the Capital Improvement Program are shown on **Table ES.4** and were developed for master planning purposes. The unit costs were benchmarked using a 20-City average Engineering News Record Construction Cost Index of 13,532, reflecting a date of April 2024.

The program is summarized on **Table ES.5** with a uniquely coded project ID, description, implementation schedule and cost information. The capital costs estimates include a 30 percent contingency allowance to account for unforeseen events and unknown field conditions, and an additional 30 percent to account for project related costs such as engineering design and construction management.

In total, the Capital Improvement Program consists of 11.6 miles of hydraulic capacity improvements and 15.7 miles of rehabilitation improvements. The implementation costs are estimated at approximately $138.1 million dollars, with $80.2 million attributed to the existing customers and $57.9 million dollars attributed to future customers. A suggested 10-year expenditure budget is also provided on **Table ES.6**.

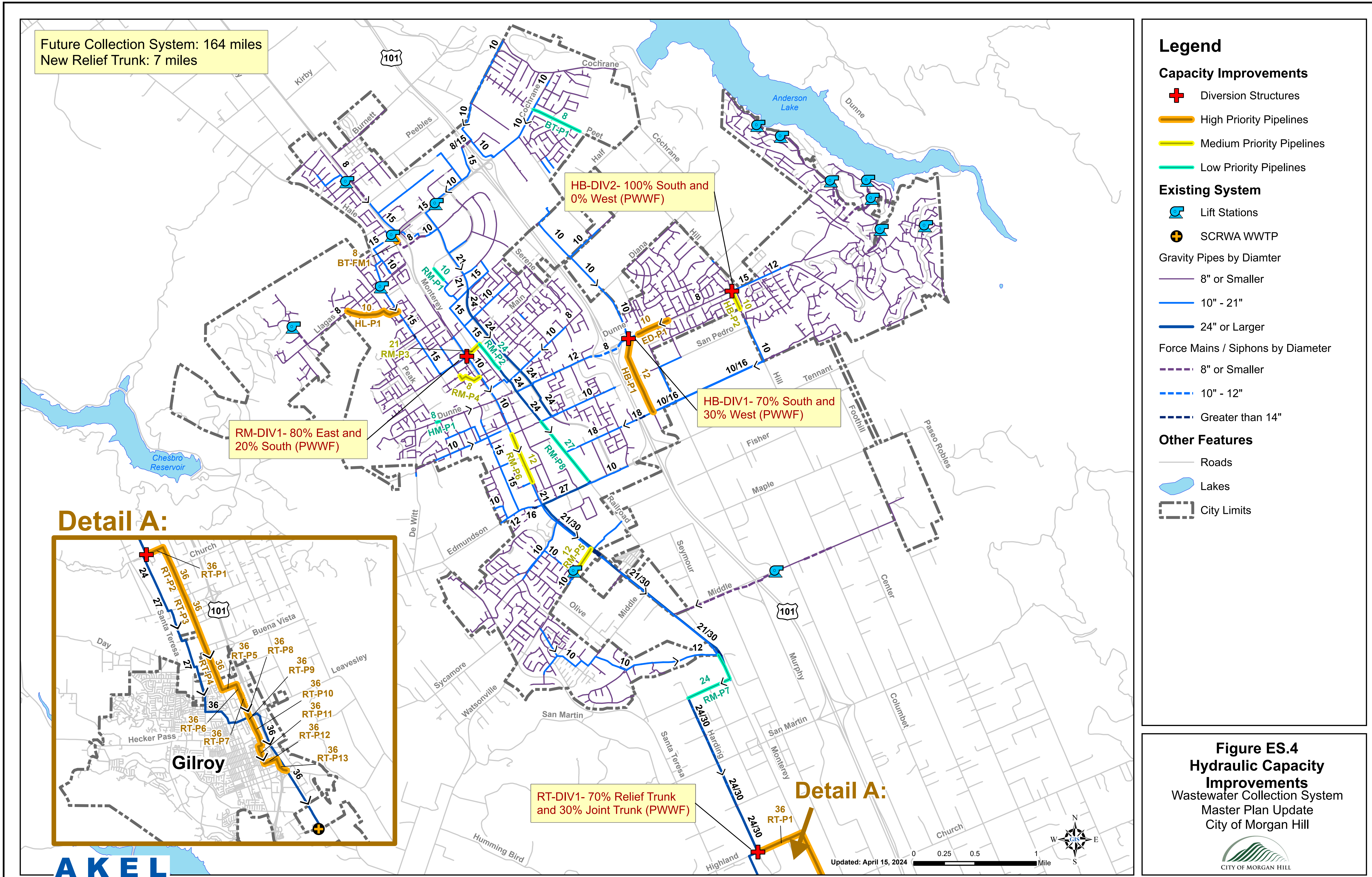

Figure ES.4
Hydraulic Capacity Improvements
Wastewater Collection System
Master Plan Update
City of Morgan Hill

Table ES.4 Infrastructure Unit Costs

Wastewater Collection System Master Plan Update
City of Morgan Hill

Pipelines [1]	
Pipe Size (in)	**Cost** ($ / Linear Foot)
8	288
10	360
12	432
15	463
18	543
21	633
24	723
27	814
30	904
36	1,085
Force Mains [1]	
Pipe Size (in)	**Cost** ($ / Linear Foot)
8	174
10	217
Miscellaneous Improvements	
Diversion Manhole with Slide Gates / Weir [1]	$ 35,000 / Each
Infrastructure Security [2]	$ 500,000 / Every 5 Years
Master Plan Contingencies [3]	
Unknown Field Conditions	30%
Project Related Costs	30%

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Notes :

1. Unit Costs were based on typical industry trends and adjusted using a 20-city average ENR CCI of 13,532 from April 2024
2. Costs estimated based on discussions with the City staff.
3. Master plan contingencies established from typical industry trends.

Table ES.5 Capital Improvement Program

Wastewater Collection System Master Plan Update

City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Users ($)	Future Users ($)
A. Hydraulic Capacity Improvements										Includes +30% Contingency	Includes +30% Contingency			1 EDU = 180 gpd [6]				
Butterfield Trunk															Cost Based on Proportional Average Wastewater Flows			
BT-P1	Gravity Pipe	Peet Rd	From approx. 420 ft e/o Avenida De Los Padres to Cochrane Rd	New	-	8	2,250	288	$648,000	$843,000	$1,096,000	Low	Beyond 2035	326 EDUs	0%	100%	$0	$1,096,000
BT-FM1	Force Main	Cochrane Road / Lift Station G	From Lift Station G to approx. 340 ft w/o Monterey Rd	Replace	6	8	350	174	$61,000	$80,000	$104,000	High	Imminent	Under Design	60%	40%	$62,400	$41,600
BT-FM1	Temporary Diversion	Cochrane Road / Lift Station G	Temporary pumped diversion to bypass Lift Station G during project BT-FM1	-	-	-	-	110,000	$110,000	$143,000	$186,000	High	Imminent	Under Design	60%	40%	$111,600	$74,400
						Butterfield Trunk Subtotal	**2,600**		**$819,000**	**$1,066,000**	**$1,386,000**						**$174,000**	**$1,212,000**
Hale-Llagas Trunk																		
HL-P1	Gravity Pipe	Llagas Creek Dr	From Llagas Rd to Hale Ave	Replace	8	10	2,250	360	$810,000	$1,053,000	$1,369,000	High	2024 - 2026	Existing Deficiency	80%	20%	$1,095,200	$273,800
						Hale-Llagas Trunk Subtotal	**2,250**		**$810,000**	**$1,053,000**	**$1,369,000**						**$1,095,200**	**$273,800**
East Dunne Trunk																		
ED-P1	Gravity Pipe	East Dunne Ave	From Peppertree Dr to 300 ft e/o of Condit Rd	Replace	8	10	1,600	360	$576,000	$749,000	$974,000	High	2024 - 2026	Existing Deficiency	100%	0%	$974,000	$0
ED-P1	Gravity Pipe	East Dunne Ave	Traffic Control Costs	-	-	-	-	50,000	$50,000	$65,000	$85,000	High	2024 - 2026	Existing Deficiency	100%	0%	$85,000	$0
						East Dunne Trunk Subtotal	**1,600**		**$626,000**	**$814,000**	**$1,059,000**						**$1,059,000**	**$0**
Hill-Barrett Trunk																		
HB-DIV1	Diversion Manhole	East Dunne Ave / Condit Rd	Route Flows 70% South along Condit Rd and 30% West alond Dunne Ave	Replace	-	-	-	Condit Rd Diversion Project Cost Provided by the City			$60,000	High	Imminent	Under Design	25%	75%	$15,000	$45,000
HB-P1	Gravity Pipe	Condit Rd	From E Dunne Ave to Barrett Ave	New	-	12	3,450	Condit Rd Diversion Project Cost Provided by the City			$2,140,000	High	Imminent	Under Design	25%	75%	$535,000	$1,605,000
HB-DIV2	Diversion Manhole	East Dunne Ave / Hill Rd	Route Flows 100% South along Hill Rd	Replace	-	-	-	35,000	$35,000	$46,000	$60,000	Medium	2027 - 2030	103 EDUs	90%	10%	$54,000	$6,000
HB-P2	Gravity Pipe	Hill Rd	From East Dunne Ave to Sundance Dr	Replace	8	10	850	360	$306,000	$398,000	$518,000	Medium	2027 - 2030	103 EDUs	90%	10%	$466,200	$51,800
						Hill-Barrett Trunk Subtotal	**4,300**		**$341,000**	**$444,000**	**$2,778,000**						**$1,070,200**	**$1,707,800**
Hale-Monterey Trunk																		
HM-P1	Gravity Pipe	Peak Ave / W Dunne Ave	From Peak Ave to 150 ft e/o Evergreen Dr	Replace	6	8	150	288	$44,000	$58,000	$76,000	Low	2031 - 2035	112 EDUs	80%	20%	$60,800	$15,200
						Hale-Monterey Trunk Subtotal	**150**		**$44,000**	**$58,000**	**$76,000**						**$60,800**	**$15,200**
Railroad-Monterey Trunk																		
RM-P1	Gravity Pipe	North of Digital Dr	From 635 ft n/o Digital Drive to Digital Dr	New	-	10	650	360	$234,000	$305,000	$397,000	Low	2031 - 2035	114 EDUs	0%	100%	$0	$397,000
RM-P2	Gravity Pipe	Mason Ln	From East Main Ave to 150 ft n/o East 4th St	Replace	15	24	1,400	723	$1,013,000	$1,317,000	$1,713,000	Low	2031 - 2035	2264 EDUs	65%	35%	$1,113,450	$599,550
RM-DIV1	Diversion Manhole	East Main Ave / Monterey Rd	Route Flows 80% East along Main Ave and 20% South along Monterey Rd	Replace	-	-	-	35,000	$35,000	$46,000	$60,000	Medium	2027 - 2030	518 EDUs	80%	20%	$48,000	$12,000
RM-P3	Gravity Pipe	East Main Ave	From Monterey Rd to Mason Ln	Replace	15 and 12	21	750	633	$475,000	$618,000	$804,000	Medium	2027 - 2030	518 EDUs	80%	20%	$643,200	$160,800
RM-P4	Gravity Pipe and Siphon	West 2nd St / West 3rd St	From Del Monte Ave to Monterey Rd	Replace	6	8	1,050	288	$303,000	$394,000	$513,000	Medium	2027 - 2030	78 EDUs	40%	60%	$205,200	$307,800
RM-P5	Gravity Pipe and Siphon	Watsonville Rd	From 400 ft w/o Calle Sueno to Monterey Rd	Replace	10	12	1,200	432	$519,000	$675,000	$878,000	Medium	2027 - 2030	130 EDUs	90%	10%	$790,200	$87,800
RM-P6	Gravity Pipe	Monterey Rd	From San Pedro Ave to Edes St	Replace	10	12	2,250	432	$972,000	$1,264,000	$1,644,000	Medium	2027 - 2030	269 EDUs	50%	50%	$822,000	$822,000
RM-P7	Gravity Pipe	Monterey Rd / California Ave	From South of Llagas Creek to Harding Ave	Replace	18	24	3,150	723	$2,278,000	$2,962,000	$3,851,000	Low	Beyond 2035	4172 EDUs	70%	30%	$2,695,700	$1,155,300
RM-P8	Gravity Pipe	Railroad Ave	From San Pedro Ave to 100 ft n/o Tennant Ave	Replace	24	27	3,250	814	$2,646,000	$3,440,000	$4,472,000	Low	Beyond 2035	4922 EDUs	70%	30%	$3,130,400	$1,341,600
						Railroad-Monterey Trunk Subtotal	**13,700**		**$8,475,000**	**$11,021,000**	**$14,332,000**						**$9,448,150**	**$4,883,850**

Table ES.5 Capital Improvement Program

Wastewater Collection System Master Plan Update
City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Users ($)	Future Users ($)
Relief Trunk - Currently under Design															**25% Attributed to Existing Users and 75% to Future Users**			
RT-DIV1	Diversion Manhole	Highland Ave / Harding Ave	Route Flows 70% East into the Relief Trunk and 30% West into the existing Joint Trunk	New	-	-	-	35,000	$35,000	$46,000	$60,000	High	Imminent	Under Design	-	-	$15,000	$45,000
RT-P1	Gravity Pipe	Highland Ave	From Harding Ave to Monterey Rd	New	-	36	2,100	1,085	$2,279,000	$2,963,000	$3,852,000	High	Imminent	Under Design	-	-	$963,000	$2,889,000
RT-P2	Gravity Pipe	Monterey Rd	From Highland Ave to Masten Ave	New	-	36	7,550	1,085	$8,192,000	$10,650,000	$13,845,000	High	Imminent	Under Design	-	-	$3,461,250	$10,383,750
RT-P3	Gravity Pipe	Monterey Rd	From Masten Ave to Buena Vista Ave	New	-	36	5,650	1,085	$6,131,000	$7,971,000	$10,363,000	High	Imminent	Under Design	-	-	$2,590,750	$7,772,250
RT-P4	Gravity Pipe	Monterey Rd	From Buena Vista Ave to Las Animas Ave	New	-	36	4,650	1,085	$5,046,000	$6,560,000	$8,528,000	High	Imminent	Under Design	-	-	$2,132,000	$6,396,000
RT-P5	Gravity Pipe	Las Animas Ave	From Monterey Rd to Murray Ave	New	-	36	1,750	1,085	$1,899,000	$2,469,000	$3,210,000	High	Imminent	Under Design	-	-	$802,500	$2,407,500
RT-P6	Gravity Pipe	Murray Ave	From Las Animas Ave to 550 ft n/o of Kishimura Dr	New	-	36	1,100	1,085	$1,194,000	$1,553,000	$2,019,000	High	Imminent	Under Design	-	-	$504,750	$1,514,250
RT-P7	Siphon	Murray Ave	From 550 ft n/o of Kishimura Dr to Kishimura Dr	New	-	Twin 24 and 12 Siphon	1,700	723 and 432	$1,065,000	$1,385,000	$1,801,000	High	Imminent	Under Design	-	-	$450,250	$1,350,750
RT-P8	Gravity Pipe	Murray Ave	From Kishimura Dr to Leavesley Rd	New	-	36	2,200	1,085	$2,387,000	$3,104,000	$4,036,000	High	Imminent	Under Design	-	-	$1,009,000	$3,027,000
RT-P9	Gravity Pipe	Murray Ave	From 150 ft n/o Leavesley Rd to 150 ft s/o Leavesley Rd	New	-	Twin 24	600	723	$434,000	$565,000	$735,000	High	Imminent	Under Design	-	-	$183,750	$551,250
RT-P10	Gravity Pipe	Murray Ave	From 150 ft s/o Leavesley Rd to Chestnut St	New	-	36	3,550	1,085	$3,852,000	$5,008,000	$6,511,000	High	Imminent	Under Design	-	-	$1,627,750	$4,883,250
RT-P11	Gravity Pipe	Chestnut St	From Murray Ave to E 7th St	New	-	36	2,600	1,085	$2,821,000	$3,668,000	$4,769,000	High	Imminent	Under Design	-	-	$1,192,250	$3,576,750
RT-P12	Gravity Pipe	E 7th St	From Chestnut St to Renz Ln	New	-	36	1,500	1,085	$1,628,000	$2,117,000	$2,753,000	High	Imminent	Under Design	-	-	$688,250	$2,064,750
RT-P13	Gravity Pipe	Renz Ln	From E 7th St to Tie-in 250 ft n/o of Hwy 152	New	-	36	1,900	1,085	$2,062,000	$2,681,000	$3,486,000	High	Imminent	Under Design	-	-	$871,500	$2,614,500
						Relief Trunk Subtotal	**36,850**		**$39,025,000**	**$50,740,000**	**$65,968,000**						**$16,492,000**	**$49,476,000**
						A. Hydraulic Capacity Subtotal	**61,450**		**$50,140,000**	**$65,196,000**	**$86,968,000**						**$29,399,350**	**$57,568,650**
B. Rehabilitation and Miscellaneous Improvements																		
Annual Pipeline and Manhole Rehabilitation Plan [7]						Total Length	Length w/o Capacity Projects [8]				Total Capital Cost Excluding Capacity Projects [9]				**Rehabilitation Attributed to Existing Users**			
RR-2025	Complete Approx. 40% of Priority 1A Rehab Projects				Varies	6,154	5,975				$2,500,000	1A	2025	Defects	100%	0%	$2,500,000	$0
RR-2026	Complete Approx. 40% of Priority 1A Rehab Projects				Varies	6,154	5,975				$2,500,000	1A	2026	Defects	100%	0%	$2,500,000	$0
RR-2027	Complete Approx. 20% of Priority 1A, 100% of Priority 1B and 3% of Priority 2A Rehab Projects				Varies	10,689	9,881				$2,500,000	1A, 1B & 2A	2027	Defects	100%	0%	$2,500,000	$0
RR-2028	Complete Approx. 19% of Priority 2A Rehab Projects				Varies	8,935	8,745				$2,500,000	2A	2028	Defects	100%	0%	$2,500,000	$0
RR-2029	Complete Approx. 19% of Priority 2A Rehab Projects			Repair / Replace	Varies	8,935	8,745				$2,500,000	2A	2029	Defects	100%	0%	$2,500,000	$0
RR-2030	Complete Approx. 19% of Priority 2A Rehab Projects				Varies	8,935	8,745				$2,500,000	2A	2030	Defects	100%	0%	$2,500,000	$0
RR-2031	Complete Approx. 19% of Priority 2A Rehab Projects				Varies	8,935	8,745				$2,500,000	2A	2031	Defects	100%	0%	$2,500,000	$0
RR-2032	Complete Approx. 21% of Priority 2A and 14% of Priority 2B Rehab Projects				Varies	10,187	9,987				$2,500,000	2A & 2B	2032	Defects	100%	0%	$2,500,000	$0
RR-2033	Complete Approx. 86% of Priority 2B and 44% of Priority 3 Rehab Projects				Varies	13,081	12,871				$2,500,000	2B & 3	2033	Defects	100%	0%	$2,500,000	$0
RR-2034	Complete Approx. 56% of Priority 3 Rehab Projects				Varies	3,473	3,344				$2,334,000	3	2034	Defects	100%	0%	$2,334,000	$0
						Pipeline and Manhole 10-Year Rehabilitation Plan Subtotal	**83,012**				**$24,834,000**						**$24,834,000**	**$0**
Lift Station Rehabilitation Plan [10]				Force Main Diameter	No. of Pumps	Total Capacity	Firm Capacity				Total Capital Cost							
RR-LS1	Lift Station F (Wet well, pumps, electrical, control panels and slab) [11]			4	2	1012 gpm	506 gpm				$1,328,700	1A	Imminent	Varies	100%	0%	$1,328,700	$0
RR-LS2	Lift Station K (Wet well, pumps, electrical and slab) [11]			4	2	1030 gpm	515 gpm				$959,300	1A	Imminent	Varies	100%	0%	$959,300	$0

Table ES.5 Capital Improvement Program

Wastewater Collection System Master Plan Update
City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Users ($)	Future Users ($)
Lift Station Rehabilitation Plan [10]				Force Main Diameter	No. of Pumps	Total Capacity	Firm Capacity				Total Capital Cost							
RR-LS3	Lift Station D (Wet well, electrical, control panels, slab, roof, walls and lighting)			4	2	1012 gpm	506 gpm				$1,506,000	1B	2024 - 2027	Varies	100%	0%	$1,506,000	$0
RR-LS4	Lift Station D (Land Acquisition Cost for Site Access) [11]			Future Land Acquisition Costs for Site Access							$1,600,000	3	2032	Varies	100%	0%	$1,600,000	$0
RR-LS5	Lift Station A (Wet well, electrical, control panels, roof, lighting and generator)			4	2	1012 gpm	506 gpm				$1,506,000	1B	2027 - 2030	Varies	100%	0%	$1,506,000	$0
RR-LS6	Lift Station P (Wet well, roof, fencing and lighting)			4	2	916 gpm	458 gpm				$1,506,000	1B	2027 - 2030	Varies	100%	0%	$1,506,000	$0
RR-LS7	Lift Station B (Wet well and lighting)			6	2	1310 gpm	655 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS8	Lift Station C (Wet well and roof)			6	2	1012 gpm	506 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS9	Lift Station G (Wet well Only)			6 and 8	2	1568 gpm	784 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS10	Lift Station H (Roof only)			4	2	1030 gpm	515 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS11	Lift Station I (Wet well Only)			6	2	988 gpm	494 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS12	Lift Station M (Wet well, roof and lighting)			6	2	968 gpm	484 gpm				$1,506,000	2B	2031 - 2035	Varies	100%	0%	$1,506,000	$0
RR-LS13	Lift Station O (Roof Only)			6	2	1074 gpm	537 gpm				$728,000	2B	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS14	Lift Station J (Rehabilitation completed in 2018)			6	2	1108 gpm	554 gpm				$728,000	3	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS15	Lift Station W (Rehabilitation completed in 2018)			6	2	1030 gpm	515 gpm				$728,000	3	2031 - 2035	Varies	100%	0%	$728,000	$0
						Lift Station Rehabilitation Subtotal	**13 Projects**				**$15,736,000**						**$15,736,000**	**$0**
Miscellaneous [10]				Purpose			Start Year				Total Capital Cost							
RR-M1	Wastewater Infrastructure Security Improvements ($500,000 Every 5 Years)			Infrastructure Security Upgrades			2024, 2029 and 2034				$1,500,000	3	2024 - 2035	Security Risk	100%	0%	$1,500,000	$0
RR-M2	Sanitary Sewer System Management Plan Updates ($50,000 Every 5 Years)			To Comply with Discharge Requirements			2027 and 2032				$100,000	3	2027 - 2035	Discharge Requirements	100%	0%	$100,000	$0
RR-M3	Wastewater Collection System Master Plan Updates ($250,000 Every 5 Years)			Identify Infrastructure Needs based on Growth			2029 and 2034				$500,000	3	2029 - 2035	General Plan Update	50%	50%	$250,000	$250,000
RR-M4	Wastewater Rate Study Updates ($50,000 Every 5 Years)			Identiy Funding Needs for Capital Improvements			2029 and 2034				$100,000	3	2029 - 2035	General Plan Update	50%	50%	$50,000	$50,000
						Miscellaneous Subtotal	**4 Projects**				**$2,200,000**						**$1,900,000**	**$300,000**
						B. Rehabilitation / Miscellaneous Subtotal	**#######**				**$42,770,000**						**$42,531,000**	**$300,000**
C. Existing Joint Trunk Improvements								**Cost Based on 1992 JPA Capacity Allocation [13]**										
Pipeline and Manhole Rehabilitation [12]						No. of Manholes	Total Length		1992 JPA City of Morgan Hill Capacity / Cost Allocation	1992 JPA City of Gilroy Capacity / Cost Allocation	Total Capital Cost for City of Morgan Hill [14]						Rehabilitation Attributed to Existing Users	
RR-J1	Priority 1 Lining Manholes North of Highland Ave			Repair	Varies	6	-		100%	0%	$39,000	1	2027 - 2030	Defects	100%	0%	$39,000	$0
RR-J2	Priority 1 Lining Manholes / Raising Buried Manhole South of Fitzgerald Ave			Repair	Varies	3	-		57%	43%	$12,000	1	2027 - 2030	Defects	100%	0%	$12,000	$0
RR-J3	Priority 1 Lining Manhole North of Day Rd			Repair	Varies	1	-		47%	53%	$3,000	1	2027 - 2030	Defects	100%	0%	$3,000	$0
RR-J4	Priority 1 Pipe and Manhole Lining Between Highland Ave and Fitzgerald Ave			Repair	27	3	533		46%	54%	$136,000	1	2027 - 2030	Defects	100%	0%	$136,000	$0
RR-J5	Priority 1 Pipe and Manhole Lining / Open Cut Repair/ Raising Manholes South of Day Rd			Repair	36	27	2,612		50%	50%	$1,007,000	1	2027 - 2030	Defects	100%	0%	$1,007,000	$0
RR-J6	Priority 2 Pipe Lining Between Highland Ave and Fitzgerald Ave			Repair	36	-	1,072		46%	54%	$326,000	2	2031 - 2035	Defects	100%	0%	$326,000	$0
RR-J7	Priority 2 Pipe Lining / Open Cut Repair South of Day Rd			Repair	36	-	20,627		50%	50%	$6,812,000	2	2031 - 2035	Defects	100%	0%	$6,812,000	$0
						C. Joint Trunk Subtotal	**24,844**				**$8,335,000**						**$8,335,000**	**$0**

Table ES.5 Capital Improvement Program

Wastewater Collection System Master Plan Update
City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Users ($)	Future Users ($)
Total Capital Cost Estimates															Cost Distribution			
A. Hydraulic Capacity Improvements									$50,140,000	$65,196,000	$86,968,000	High, Medium and Low	2024 - 2035 and Beyond	Varies	34%	66%	$29,399,350	$57,568,650
B. Rehabilitation and Miscellaneous Improvements									-	-	$42,770,000	1A, 1B, 2A, 2B and 3	2024 - 2035	Varies	99%	1%	$42,531,000	$300,000
C. Existing Joint Trunk Improvements									-	-	$8,335,000	1 and 2	2027 - 2035	Defects	100%	0%	$8,335,000	$0
Total Capital Improvement Program Cost											$138,073,000						$80,265,350	$57,868,650

AKEL ENGINEERING GROUP, INC.

4/23/2024

Notes:

1. Total length was obtained from the hydraulic model and rounded up to the nearest 50 feet.
2. Unit Costs were based on typical industry trends and adjusted using a 20-city average ENR CCI of 13,532 from April 2024
3. Baseline costs were calculated by multiplying the pipeline length by the unit cost and rounding up to the nearest $1,000.
4. Construction costs were calculated by applying a 30% contingency to the baseline costs and rounding up to the nearest $1,000. These costs account for unknown field conditions and site-specific constraints.
5. Total capital costs were calculated by applying a 30% contingency to the construction costs and rounding up to the nearest $1,000. These costs account for project related expenses such as engineering, legal fees, contract administration and construction management.
6. According to the 2023 wastewater flow monitoring program and annual water billing records, a single-family dwelling unit generates an average wastewater flow of approximately 180 gallons per day (gpd).
7. Pipeline, manhole and O&M rehabilitation improvements were sourced from the City's 2021 Sanitary Sewer System Asset Management Plan (SSAMP).
8. Adjusted length and number of manholes were calculated by excluding rehabilitation projects that overlap with hydraulic capacity projects. A list of overlapping capacity and rehabilitation projects are as follows:
   a. Project RR-P1A (Priority 1A) length excludes 451 feet that overlaps with hydraulic capacity projects HL-P1 (310 feet) and RM-P4 (141 feet).
   b. Project RR-P1B (Priority 1B) length excludes 681 feet that overlaps with hydraulic capacity projects HL-P1 (483 feet) and RM-P4 (198 feet).
   c. Project RR-P2A (Priority 2A) length excludes 979 feet that overlaps with hydraulic capacity projects ED-P1 (439 feet) and RM-P6 (540 feet).
   d. Project RR-P2B (Priority 2B) length excludes 124 feet that overlaps with hydraulic capacity project ED-P1.
   e. Project RR-P3 (Priority 3) length excludes 231 feet that overlaps with hydraulic capacity projects HL-P1 (70 feet) and RM-P4 (161 feet).
   f. Project RR-M2 excludes 2 manholes that overlap with hydraulic capacity projects HL-P1 (1 manhole) and RM-P4 (1 manhole).
9. Pipeline and manhole rehabilitation capital cost estimates were obtained from the City's 2021 SSAMP with their compounded contingencies (30% costs and 30% engineering). These costs were also adjusted using a 20-city average ENR CCI of 13,532 from April 2024 and rounded up to the nearest $1,000. It should be noted that the costs are based on the adjusted length/manhole count, which excludes overlapping hydraulic capacity projects (See Note 8).
10. Lift station rehabilitation improvements, capital costs, and implementation schedule was provided by City staff on February 8, 2024.
11. Costs for Lift Stations F and K were provided by City staff on February 8, 2024. Additionally, the City also provided land acquisition costs for Lift Station D.
12. Joint Trunk rehabilitation improvements were obtained from the City's 2021 Joint Trunk Pipeline Condition Assessment Report.
13. Joint Trunk capacity allocations for the City of Gilroy and the City of Morgan Hill were obtained from the 1992 Joint Powers Agreement (JPA). These allocations were used to calculate the proportional capital cost for the City of Morgan Hill.
14. Joint Trunk cost estimates were obtained from the City's 2021 Joint Trunk Pipeline Condition Assessment Report with their total contingencies (45% total for priority 1 projects and 40% total for priority 2 projects). Subsequently, these costs were escalated using a 20-city average ENR CCI of 13,532 from April 2024 and rounded up to the nearest $1,000.

Table ES.6 Suggested 10-Year Expenditure Budget

Wastewater Collection System Master Plan Update
City of Morgan Hill

Phasing Schedule	Hydraulic Capacity Improvements ($)	Rehabilitation / Miscellaneous Improvements ($)	Existing Joint Trunk Improvements ($)	Total Capital Costs ($)
	A	B	C	A+B+C
Imminent / Under Design	$68,458,000	$2,288,000	-	$70,746,000
2024 - 2026	$2,428,000	$7,006,000	-	$9,434,000
2027 - 2030	$4,477,000	$13,862,000	$1,197,000	$19,536,000
2031 - 2035	$2,186,000	$19,614,000	$7,138,000	$28,938,000
Beyond 2035	$9,419,000	-	-	$9,419,000
Total	**$86,968,000**	**$42,770,000**	**$8,335,000**	**$138,073,000**

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CHAPTER 1 – INTRODUCTION

This chapter provides a background of the City of Morgan Hill's wastewater collection system, the need for this Master Plan Update, and the objectives of the study.

1.1 BACKGROUND

The City of Morgan Hill (City) is located in the Santa Clara County, approximately 22 miles southeast from the City of San Jose's downtown and 10 miles north from the City of Gilroy. A regional location map illustrating the neighboring cities is displayed on **Figure 1.1**.

The City owns and maintains a wastewater collection system comprised of 164 miles of pipes, and 14 lift stations. Wastewater from the City is conveyed south into the City of Gilroy via a Joint Trunk system shared by the City of Morgan Hill and the City of Gilroy. Ultimately, wastewater from both cities is discharged into a Wastewater Treatment Plant (WWTP) located in southeast Gilroy. This plant is operated by the South County Regional Wastewater Authority (SCRWA), which was formed by both cities under a Joint Powers Agreement (JPA) dated May 19th, 1992.

The City had originally adopted a Wastewater Collection System Master Plan (WCSMP) in 2002, which was most recently updated in 2017 with recommendations to service future growth.

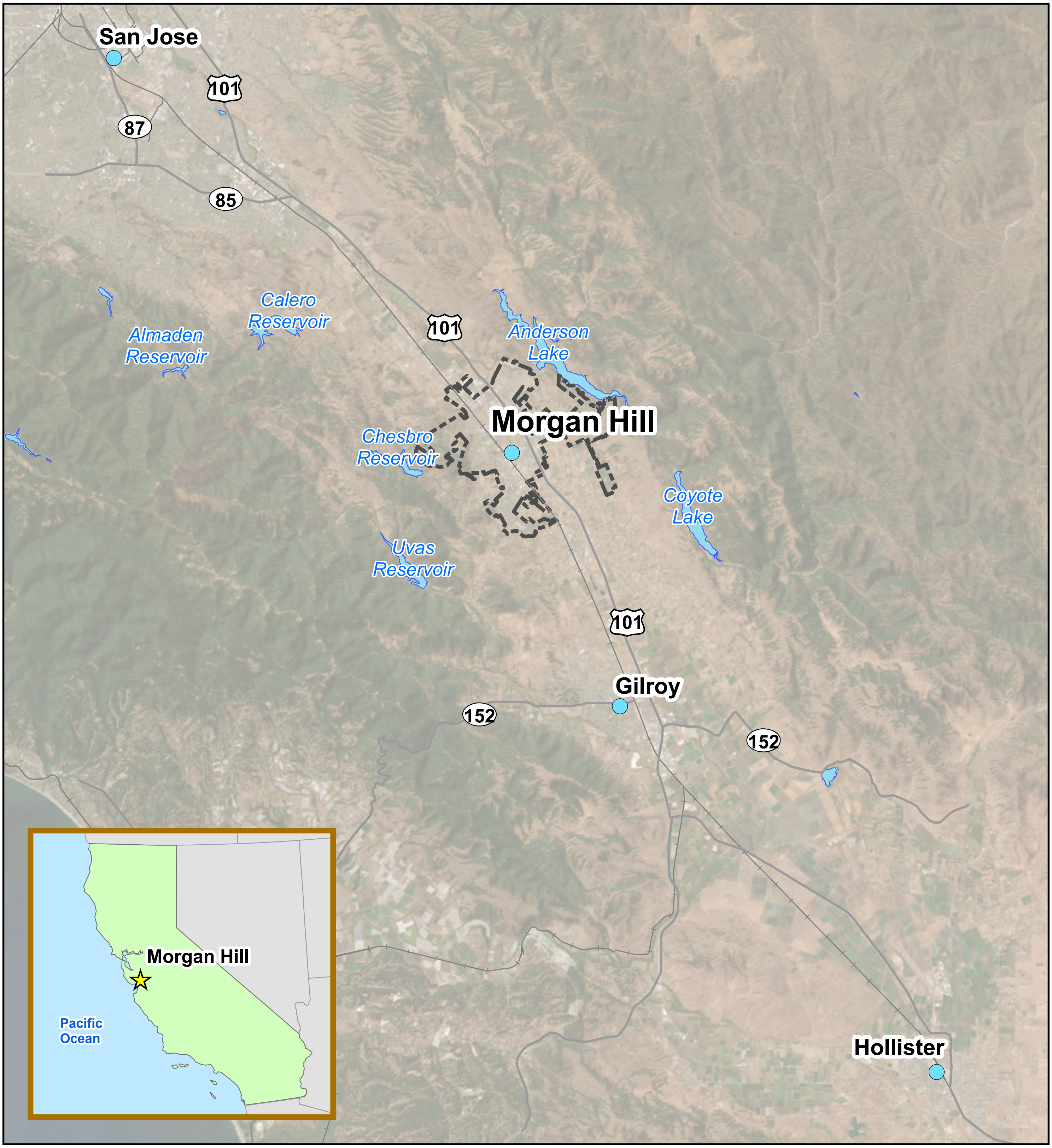
Since then, the City has constructed most of the recommended improvements and developed a comprehensive plan to rehabilitate aging infrastructure. The objectives of this 2024 WCSMP Update are to incorporate recently constructed projects, document General Plan land use amendments, and evaluate the capacity adequacy of the existing collection system.

1.2 SCOPE OF WORK

The City Council approved Akel Engineering Group, Inc. to prepare the 2024 WCSMP Update in January 2023. This update is intended to serve as a planning tool and support the construction of critical wastewater infrastructure required to facilitate growth.

The 2024 WCSMP Update included the following tasks:

- Summarize the City's existing wastewater collection system facilities.
- Document growth assumptions based on the 2035 General Plan.
- Summarize the wastewater collection system performance and design criteria.
- Project future wastewater flows for the General Plan horizon.
- Develop a Geographic Information Systems (GIS)-based hydraulic model of the City's wastewater collection system.



Legend

- Cities
- Highways
- Railroads
- City Limits
- Lakes

Updated: April 2, 2024

0 0.5 1 2 Miles

N W E S

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Figure 1.1
Regional Location Map

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

- Complete a Flow Monitoring Program to collect data from 11 strategic sites and calibrate the hydraulic model.
- Evaluate system capacity under existing and projected flows.
- Identify capacity improvements needed to maintain the target level of service.
- Document the City's Rehabilitation and Replacement (R&R) Program improvements.
- Recommend a Capital Improvement Program (CIP) with an opinion of probable construction costs.
- Prepare a 2024 WCSMP Update Report.

1.3 PREVIOUS MASTER PLANS AND RELEVANT STUDIES

The City has completed several planning studies to assess the wastewater collection system and prioritize capital improvements. The following list provides a summary of previous planning efforts:

- **2002 Sewer System Master Plan.** This master plan documents the capacity evaluation of the City's 6 major trunk collectors and recommends a CIP to mitigate deficiencies. This plan also recommends the construction of a Relief Trunk that runs parallel to the existing Joint Trunk.
- **2017 Sewer System Master Plan Update.** This plan details hydraulic model updates, buildout flow projections and provides a revised CIP to mitigate capacity deficiencies.
- **2022 Sanitary Sewer Management Plan.** This regulatory plan documents the effective management, operation and maintenance of the City's wastewater collection system and adheres to the requirements established by the State Water Resources Control Board.
- **2035 General Plan/General Land Use Map (Effective December 31, 2021).** The original General Plan was adopted by the City Council in July 2016 and land uses were most recently amended on December 31, 2021. The updated General Plan land use map identifies future rezoning areas and potential development sites within the City's Sphere of Influence.

1.4 REPORT ORGANIZATION

The 2024 WCSMP Update contains the following chapters:

Chapter 1 – Introduction. This chapter provides a background of the City's wastewater collection system, the need for this master plan, and the objectives of the study.

Chapter 2 – Planning Area Characteristics. This chapter presents a discussion of the planning area characteristics, which include current and General Plan land uses, population growth and regional climate patterns.

Chapter 3 – System Performance and Design Criteria. This chapter details capacity study requirements for private developers and presents the City's design criteria that were used to evaluate the capacity of the existing system.

Chapter 4 – Existing Wastewater Collection System Facilities. This chapter provides a description of the City's existing wastewater facilities, including 6 major trunk collectors, Joint Trunk system and lift stations. This chapter also describes the SCRWA WWTP, which treats and recycles the City's wastewater.

Chapter 5 – Wastewater Flows. This chapter discusses the existing and future design flows in the wastewater collection system.

Chapter 6 – Hydraulic Model Development. This chapter describes the development and calibration of the City's GIS-based wastewater collection system hydraulic model.

Chapter 7 – Capacity Evaluation and Improvements. This chapter summarizes capacity evaluation results under existing and future flow conditions. The hydraulic capacity improvements needed to mitigate system deficiencies are also discussed in this chapter.

Chapter 8 – Rehabilitation and Replacement (R&R) Program. This chapter documents R&R improvements needed to address aging wastewater infrastructure.

Chapter 9 – Capital Improvement Program. This chapter presents a Capital Improvement Program with hydraulic capacity, rehabilitation, and Joint Trunk improvements. This chapter also presents the cost criteria and methodologies for developing capital improvement costs.

1.5 GEOGRAPHIC INFORMATION SYSTEMS

This master planning effort made extensive use of GIS technology to complete the following tasks:

- Develop the physical characteristics of the hydraulic model (gravity pipes, force mains, and lift stations).
- Perform a table join of annual water billing records and parcel shapefiles using the Assessor Parcel Number.
- Allocate/distribute flows from each parcel to the nearest manhole.
- Generate maps and exhibits.

1.6 ACKNOWLEDGEMENTS

Akel Engineering Group, Inc. acknowledges the support and cooperation of the City of Morgan Hill and extends its appreciation to the Public Services Department for their assistance in preparing this report. Obtaining the necessary information to successfully complete this Update was accomplished with the strong commitment and very active input from dedicated team members including:

- Chris Ghione, Director of Public Services
- James F. Sylvain, P.E., Deputy Director of Utilities Services
- Scott Creer, P.E., City Engineer
- Maria Angeles, P.E., C.F.M., Senior Civil Engineer
- Mario Parraz, Utilities Operations Manager
- Pamela Van Der Leeden, GIS Manager
- Yat Cho, Senior Project Manager
- Other City Engineering and Operations Staff

CHAPTER 2 – PLANNING AREA CHARACTERISTICS

This chapter presents a discussion of the planning area characteristics, which include current and General Plan land uses, population growth and regional climate patterns.

2.1 STUDY AREA DESCRIPTION

The City of Morgan Hill is generally bound by Tilton Avenue to the north, Anderson Lake/Foothill Avenue to the east, Village of San Martin to the south and Sunnyside Drive to the west. Highway 101 bisects the eastern boundary of the City in the north-south direction.

There are several creeks flowing through and along the boundaries of the City, including: Fisher Creek, West Little Llagas Creek, and Llagas Creek. Based on the natural topography, the City is mostly flat at the center with steeper slopes along the foothills at the east and west boundaries. The City limits, Urban Growth Area Boundary and the Sphere of Influence are displayed on **Figure 2.1** for reference purposes.

The City owns and maintains a wastewater collection system with a service area of approximately 12.9 square miles. Wastewater from the City is ultimately conveyed through the City of Gilroy into the SCRWA WWTP.

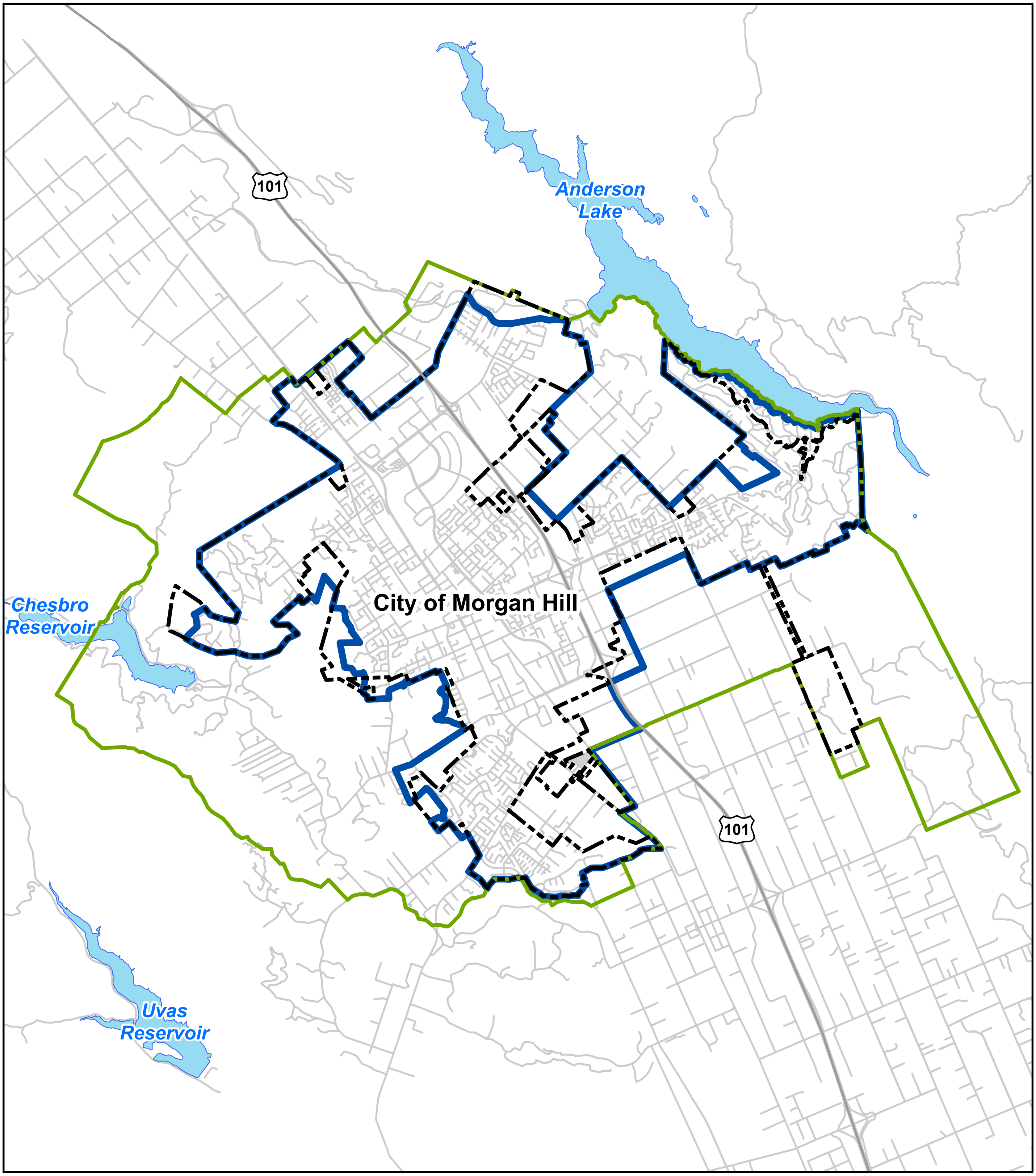
2.2 EXISTING LAND USE

The City of Morgan Hill mostly consists of single-family residential neighborhoods with multi-family apartment complexes in more urbanized areas, as illustrated on **Figure 2.2.** The non-residential land uses include commercial/retail facilities along major transportation corridors and two industrial centers: business parks north and south of Cochrane Road, and light industrial facilities west of Highway 101 between Dunne and Tennant Avenues. The majority of the rural areas outside the City limits and within its Sphere of Influence are designated as agricultural lands or open space.

2.3 2035 GENERAL PLAN LAND USE

The 2035 General Plan was adopted by the City Council on July 27, 2016, and provides a strategic vision for long-term growth. The General Plan focuses on balancing future housing and employment needs to facilitate sustainable growth while complying with regulatory requirements.

The General Plan land use map is displayed on **Figure 2.3**, and was last amended on December 31, 2021. Potential rezoning areas and development sites are subsequently identified on **Figure 2.4**, and were obtained by comparing existing, known planned developments at the time of this report, and General Plan land uses.



Legend

- City Limits
- Urban Growth Boundary
- Sphere of Influence Boundary
- Roads
- Highways
- Lakes

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Updated: April 2, 2024

0 0.25 0.5 1 Mile

N S W E

Figure 2.1
Planning Area

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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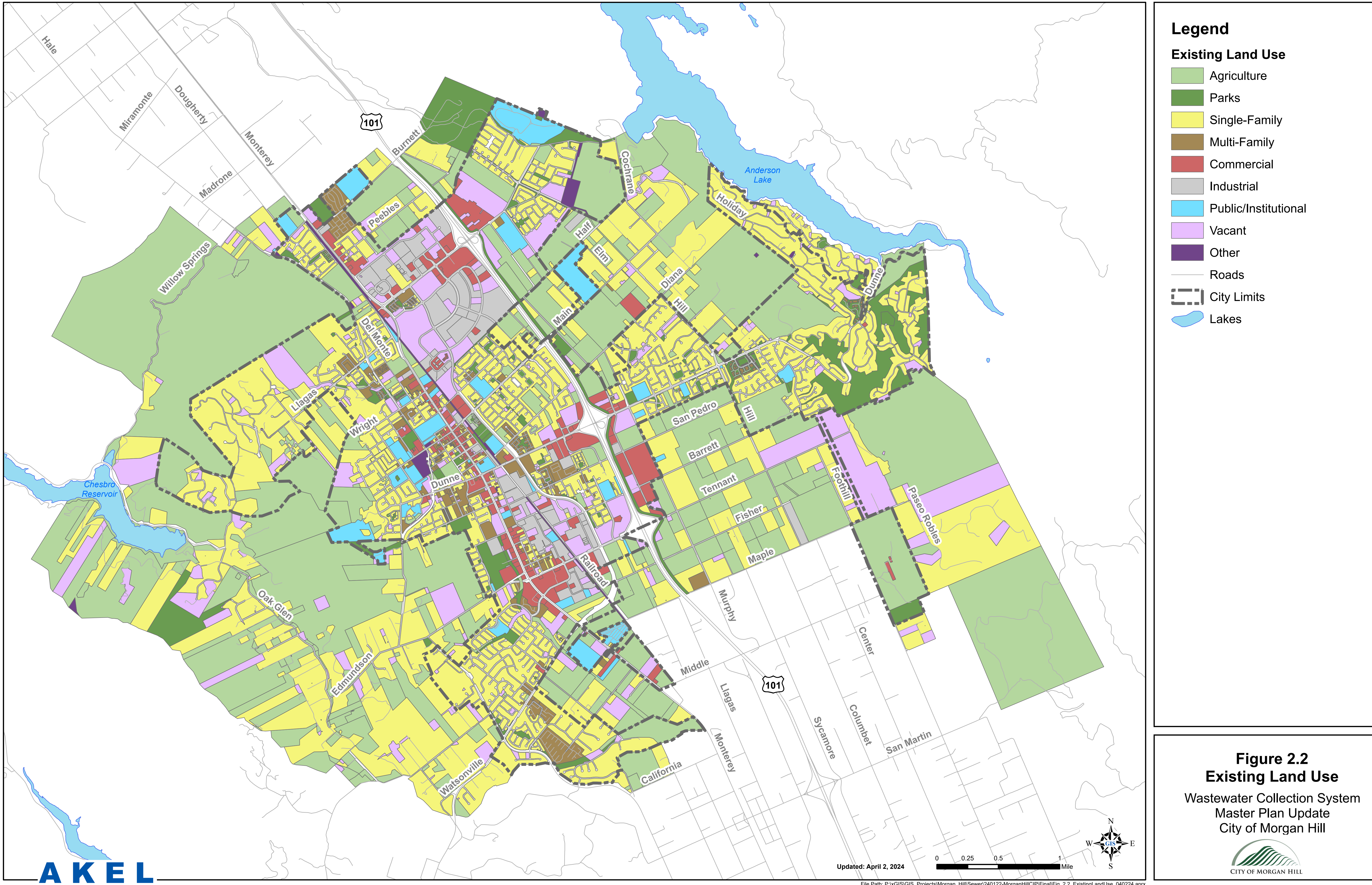

Figure 2.2
Existing Land Use

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

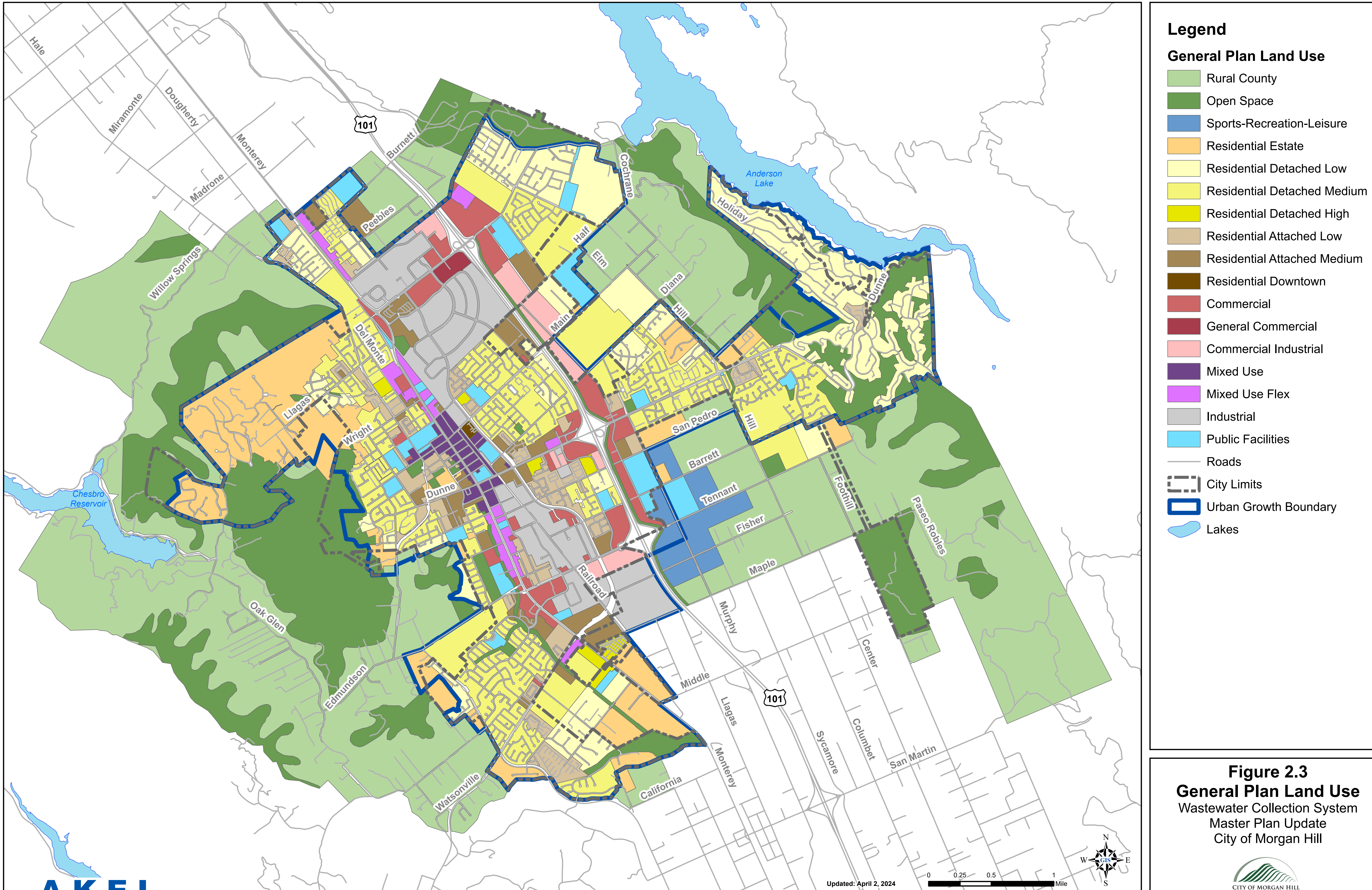

Figure 2.3
General Plan Land Use
Wastewater Collection System
Master Plan Update
City of Morgan Hill

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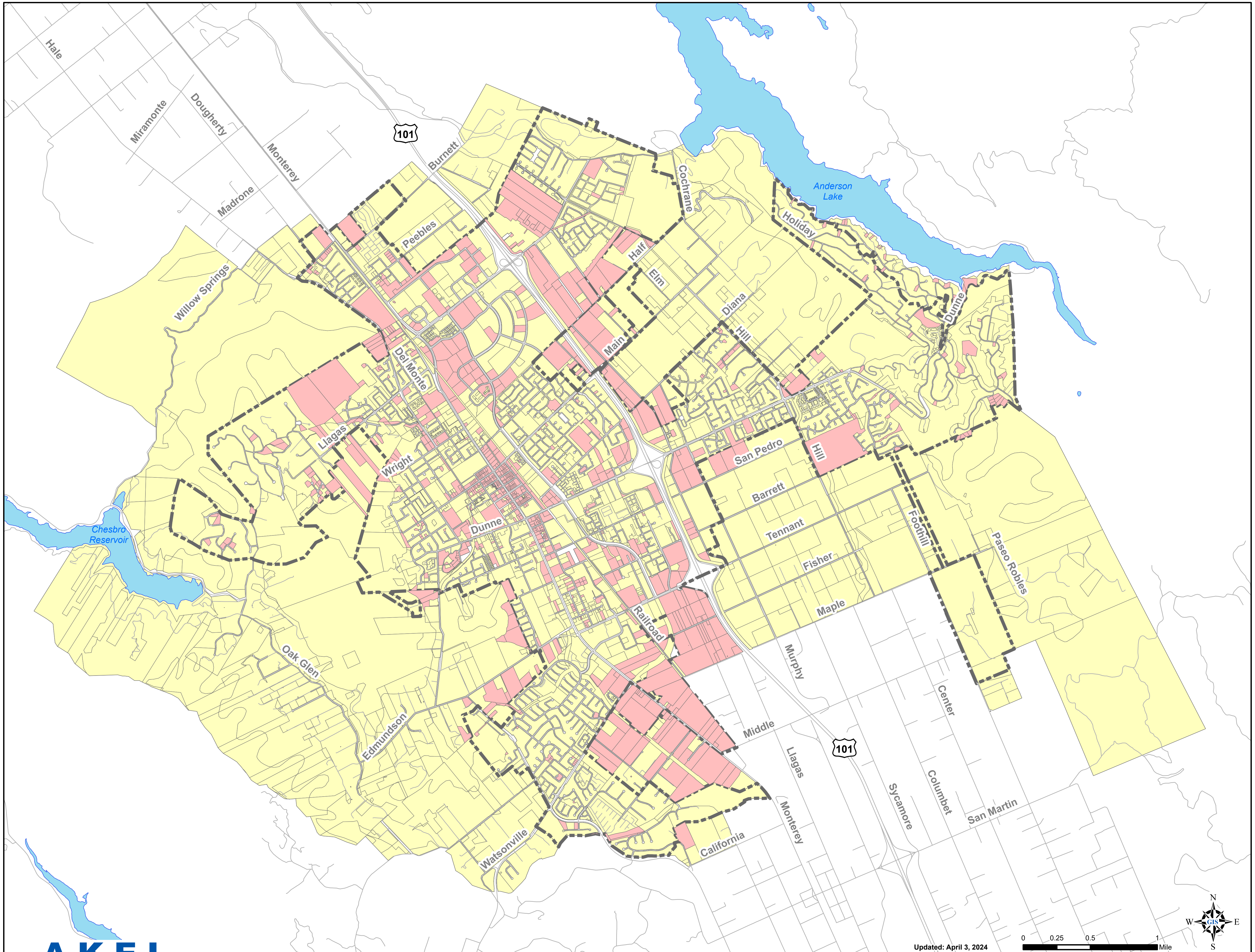

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Legend

Parcels to Change Land Use in Future Plan

- Land Use Stays the Same
- Existing Land Use Changes
- Roads
- City Limits
- Lakes

Figure 2.4
Areas of Change

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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The General Plan includes three special planning areas that may experience considerable growth:

- **Downtown.** This intensification will include new commercial and mixed-use growth around Monterey Road from Main Avenue to Dunne Avenue, and from Del Monte Avenue to Depot Street.
- **Monterey Corridor.** This corridor located north and south of Downtown will include rezoning of existing commercial lands to mixed-use lands with increased pedestrian mobility.
- **East of 101.** The area bound by Half Road to the north, Hill Road to the east, Diana Avenue to the south and Condit Road to the west will include new residential neighborhoods with a variety of commercial and open space amenities (parks and trails).

Overall, the General Plan will facilitate the development of vibrant employment districts (business park or industrial centers) to maintain a steady balance between jobs and housing needs.

2.4 HISTORICAL AND PROJECTED POPULATION

The City population has increased from 37,882 in 2010 to 46,454 in 2020, which equates to an annual growth rate of approximately 2 percent. This can be attributed to an increase in the number of jobs and the construction of accessory dwelling units, commonly known as second units. However, it should be noted that the current population has remained relatively stable and slightly decreased since 2020.

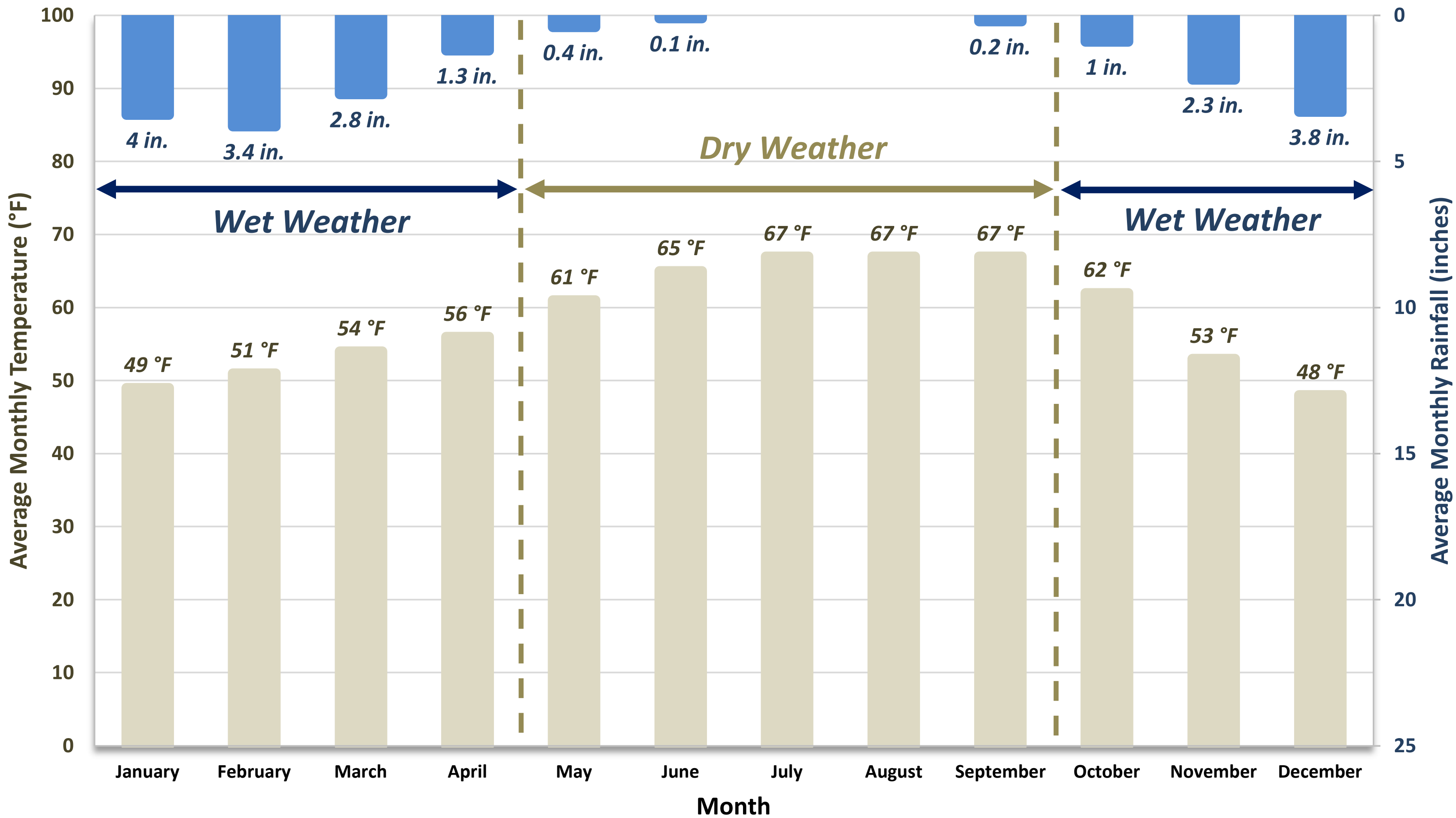
The historical population data from 1990 to 2023 is shown on **Table 2.1** and was obtained from the City's 2023 – 2031 Housing Element. Future population projections are also shown on this table and were estimated based on policy CNF-3.4 of the 2035 General Plan:

- **Policy CNF-3.4 Population Limit.** Plan for a January 1, 2035, population of 58,200 residents.

Given the current population of 45,892 (2023), the City anticipates an annual growth rate of approximately 1.8 percent.

2.5 REGIONAL CLIMATE

The City experiences a mediterranean climate with mild summers and relatively cool winters, as displayed on **Figure 2.5**. The average values are based on historical observations and indicate that the dry weather season typically begins in May and ends in September with July or August being the hottest month. The wet weather season then begins in October and ends in April with January or December being the coldest month.



LEGEND

Average Monthly Rainfall (inches)

Average Monthly Temperature (° F)

Note:

Climate data for the City of Morgan Hill was obtained from Weather Spark, and estimated from statistical models that cover a period from January 1, 1980 to December 31, 2016.

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Figure 2.5

Regional Climate Patterns

Wastewater Collection System Master Plan Update

City of Morgan Hill

CITY OF MORGAN HILL

Table 2.1 Historical and Projected Population

Wastewater Collection System Master Plan Update
City of Morgan Hill

Year	Population	Percent Growth (%)
Historical [1]		
1990	23,928	-
1995	26,924	12.5%
2000	33,586	24.7%
2005	36,292	8.1%
2010	37,882	4.4%
2015	42,380	11.9%
2020	46,454	9.6%
Existing 2023 [2]	45,892	-1.2%
Projected Based on 1.8% Annual Growth [3]		
2024	46,738	1.8%
2025	47,601	1.8%
2025	48,478	1.8%
2026	49,373	1.8%
2027	50,283	1.8%
2028	51,211	1.8%
2029	52,155	1.8%
2030	53,117	1.8%
2031	54,097	1.8%
2032	55,095	1.8%
2033	56,111	1.8%
2034	57,146	1.8%
2035	58,200	1.8%

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Notes:

1. Historical population was obtained from Table H 1-2 of the City's 2023 - 2031 Housing Element.
2. 2023 population was obtained from the California Department of Finance Estimates.
3. The General Plan policy CNF-3.4 anticipates the current population to increase to 58,200 by January 1, 2035. As such, future population from 2024 to 2035 was estimated based on an annual growth rate of 1.84%.

CHAPTER 3 – SYSTEM PERFORMANCE AND DESIGN CRITERIA

This chapter details capacity study requirements for private developers and presents the City's performance and design criteria that were used to evaluate the capacity of the existing system.

3.1 CAPACITY STUDY REQUIREMENTS FOR PRIVATE DEVELOPERS

Proposed private development projects that have been determined by the City to have an impact on the capacity of the existing wastewater collection system will be required to complete a capacity study. The purpose of the study is to ensure the existing system can accommodate the proposed development and, if needed, identify new improvements required to maintain the current level of service.

The capacity study will include hydraulic modeling and shall be performed by the City of Morgan Hill's WCSMP consultant. The developer will be responsible for the full cost of the study, which will document a minimum of three hydraulic model scenarios:

- **Existing Conditions.** This scenario will represent pre-development conditions.
- **Existing Conditions with the Proposed Development.** This scenario will represent post-development conditions and is needed to identify additional deficiencies that may be triggered by the proposed development.
- **Buildout Conditions with the Proposed Developments.** This scenario will demonstrate compliance to the existing WCSMP improvement recommendations.

Refer to the City's Policy on Wastewater Collection System Capacity Studies for additional information. The City's Policy details specific requirements for private development projects, including wastewater capacity modeling, design capacity criteria, mitigation methods and cost responsibilities.

3.2 HYDRAULIC CAPACITY CRITERIA

The hydraulic capacity criteria for gravity pipes, lift stations and force mains is presented in the following sections.

3.2.1 Gravity Pipes

Gravity pipe capacities depend on several factors including material, roughness, slope, and the maximum allowable depth of flow. InfoWorks ICM by Autodesk Inc. was selected as the preferred hydraulic modeling software to evaluate the capacity adequacy of the City's collection system.

This software uses the fully dynamic St. Venant's equation and has a more accurate engine for simulating backwater effects for surcharged conditions.

Continuity and Manning's Equation for Pipe Capacity

Continuity Equation

$Q = V \times A$

Where:
- Q = peak flow, in cubic feet per second (cfs)
- V = velocity, in feet per second (fps)
- A = cross-sectional area of pipe, in square feet (sq. ft)

Manning's Equation

$V = (1.486\, R^{2/3}\, S^{1/2})/n$

Where:
- V = velocity, in feet per second (fps)
- n = Manning's roughness coefficient
- R = hydraulic radius (area divided by wetted perimeter), ft
- S = slope of pipe, in feet per foot

The Continuity equation and the Manning equation for steady-state flow are used to calculate pipe capacities in open channel flow. Open channel flow can consist of either open conduits or, in the case of gravity pipes, partially full conduits. Gravity full flow occurs when the conduit is flowing full but has not reached a pressure condition.

St. Venant Equations for Pipe Capacity

Dynamic modeling facilitates the analysis of unsteady and non-uniform flows (dynamic flows) within a collection system. Some hydraulic modeling programs can analyze these flows using the St. Venant equations, which consider unsteady and non-uniform conditions that occur over changes in time and cross-section of pipes.

The St. Venant equation is a set of two equations, a continuity equation, and a dynamic equation. The first equation, the continuity equation, relates the continuity of flow mass within the system pipes in terms of: (A) the change in the cross-sectional area of flow at a point over time and (B) The change of flow over the distance of piping in the system.

The second equation, the dynamic equation, relates changes in flow to fluid momentum in the system using: (A) changes in acceleration at a point over time, (B) changes in convective flow acceleration, (C) changes in momentum due to fluid pressure at a given point, (D) changes in momentum from the friction slope of the pipe and fluid momentum provided by gravitational forces.

Continuity Equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

(A) (B)

Where:
- t = time
- x = distance along the longitudinal direction of the channel
- Q = discharge flow
- A = flow cross-sectional area perpendicular to the x directional axis

Dynamic Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial t}\left(\beta \frac{Q^2}{A}\right) + gA\frac{\partial y}{\partial x} + gAS_f - gAS_o = 0$$

(A) (B) (C) (D) (E)

Where:
- t = time
- x = distance along the longitudinal direction of the channel
- Q = discharge flow
- A = flow cross-sectional area perpendicular to the x directional axis
- y = flow depth measured from the channel bottom and normal to the x directional axis
- S_f = friction slope
- S_o = channel slope
- β = momentum
- g = gravitational acceleration

Use of this method of analysis provides a more accurate and precise analysis of flow conditions within the system compared to steady state flow analysis methods. The two critical assumptions for the St. Venant equations are as follows:

- **Flow is one dimensional.** This means it is only necessary to consider velocities in the downstream direction and not in the transverse or vertical directions.
- **Flow is gradually varied.** This means the vertical pressure distribution increases linearly with depth in the pipe.

Manning's Roughness Coefficient (n)

The Manning roughness coefficient 'n' is a friction coefficient that is used in the Manning formula for flow calculation in open channel flow. In wastewater collection systems, the coefficient can vary between 0.009 and 0.017 depending on pipe material, size, depth of flow, root intrusion, smoothness of joints, and other structural factors. A "n" value of 0.013 is an acceptable practice in planning studies and was therefore used in the hydraulic model.

Partial Flow Criteria (d/D)

The depth of flow to pipe diameter ratio (d/D) is used as a key hydraulic indicator to evaluate the conveyance capacity in gravity pipes. For circular pipes, the highest capacity is generally reached at 92 percent of the full height of the pipe (d/D ratio of 0.92). This is due to the additional wetted perimeter and increased friction of a gravity pipe.

When designing wastewater pipes, it is common practice to use variable flow depth criteria that allow higher safety factors in larger sizes. Thus, design d/D ratios may range between 0.5 and 0.75, with the lower values used for smaller pipes. The smaller pipes may experience flow peaks greater than planned or may experience blockages from debris.

During peak dry weather flow (PDWF), the maximum allowable d/D ratio for proposed and existing pipes are 0.75 and 0.92, respectively, as listed on **Table 3.1**. The criterion for existing pipes is relaxed to maximize asset life before triggering costly improvements.

During peak wet weather flow (PWWF), pipes are allowed to surcharge provided the connected manholes have a minimum freeboard depth of three feet, as listed on **Table 3.1**. This criterion is typically implemented to avoid premature replacements while minimizing the risk of overflows.

Minimum Slopes and Design Velocities

The minimum recommended slopes for gravity pipes are listed on **Table 3.1** for various pipe sizes. In order to minimize the settlement of sewage solids, it is standard practice in the design of gravity pipes to specify that a minimum velocity of 2 feet per second (fps) be maintained when the pipe is half-full. At this velocity, the wastewater flow will typically result in self-cleaning of the pipe.

Table 3.1 System Performance and Design Criteria

Wastewater Collection System Master Plan Update
City of Morgan Hill

Gravity Pipe Criteria [1]

Peak Dry Weather Flow Conditions

Diameter (in)	Maximum Allowable d/D Existing Pipes	Maximum Allowable d/D Proposed Pipes
6 to 10"	**0.92**	**0.50**
12" or Higher	**0.92**	**0.75**

Peak Wet Weather Flow Conditions

Hydraulic Grade Line (HGL) should be at least **3 feet** below the manhole ground elevation.

Minimum Design Slope and Capacity [2]

Pipe Size (in)	Minimum Slope (%)	Full Flow Capacity (n = 0.013) (mgd)	Half Full Velocity (ft/s)
8''	0.400%	0.49	2.19
10''	0.280%	0.75	2.12
12''	0.220%	1.08	2.13
15''	0.150%	1.62	2.04
18''	0.120%	2.35	2.06
21''	0.100%	3.24	2.08
24''	0.080%	4.13	2.04
27''	0.067%	5.18	2.02
30''	0.058%	6.38	2.01
33''	0.052%	7.79	2.03
36''	0.046%	9.24	2.02
42''	0.037%	12.51	2.01

Lift Station and Force Main Criteria [1]

Lift Station shall be sized to accommodate:	**Peak Wet Weather Flow with Largest Pump Out of Service** (Firm Capacity)
Force Main Velocity:	**2 to 10 ft/s**

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Notes:
1. Recommended based on typical industry standards and a survey of local municipalities.
2. Recommended based on the Ten State Wastewater Collection System Design Standards.

Changes in Pipe Size

When a smaller gravity pipe joins a larger pipe, the invert of the larger pipe is generally lowered to maintain the same energy gradient. One of the methods used to approximate this condition includes placing the 80 percent depth point (d/D at 0.8) from both pipes at the same elevation. For master planning purposes, and in the absence of known field data, pipe crowns were matched at the manholes.

3.2.2 Lift Station and Force Main Criteria

Wastewater lift stations are evaluated and designed to meet the PWWF with one standby pump having a capacity equal to the largest operating unit. This is also commonly referred to as the firm capacity. The standby pump provides a safety factor in case the duty pump malfunctions or requires maintenance.

The Hazen-Williams Equation is commonly used for the design of force mains. The value of the Hazen-Williams 'C' depends on the pipe material and is also influenced by pipe age. An industry standard 'C' value of 120 was used for capacity evaluation.

Hazen Williams Velocity Equation

$$V = 1.32\ C\ R^{0.63}\ S^{0.54}$$

Where:
- V = mean velocity, fps
- C = roughness coefficient
- R = hydraulic radius, ft
- S = slope of the energy grade line, ft/ft

The minimum recommended velocity in force mains is at 2 feet per second (fps). The economical pumping velocity ranges between 3 and 5 fps, and a maximum not-to-exceed velocity is at 10 fps.

3.3 DRY WEATHER FLOW CRITERIA

Wastewater unit flow factors are applied in planning studies to estimate average dry weather flow (ADWF) for areas with predetermined land uses. The unit factors are multiplied by the number of dwelling units or gross acreages, to yield future flow projections.

3.3.1 Unit Flow Factors Methodology

Wastewater unit factors are developed by multiplying water consumption records with an appropriate return to sewer rate for each land use. This analysis relied on the City's 2022 water billing records for relative flow distribution within the system. The return to sewer rates were then adjusted to reflect measured wastewater flows from the 2023 Flow Monitoring Program.

3.3.2 Average Dry Weather Flow Unit Factors

Average wastewater flows were initially calculated from the City's 2022 water billing records, current land use database and a typical return to sewer rate of 60 percent. The water billing records were used for relative flow distribution purposes only and do not impact the quantity of wastewater flows.

After completion of the 2023 Flow Monitoring Program, the return to sewer rates were revised for each metered basin to match observations from February 2023. The existing unit factor analysis is shown on **Table 3.2** and indicates a system-wide return to sewer rate of approximately 55 percent. This analysis also provides the following wastewater coefficients:

- A system-wide average wastewater generation rate of 176 gallons per day per dwelling unit (gpd/DU) based on 16,178 residential units (2023).
- A system-wide average wastewater generation rate of 62 gallons per capita per day (gpcd) based on a population of 45,892 (2023).

The unit flow factors were also adjusted to account for vacant areas and specific land use densities. The final recommended unit flow factors are listed on on **Table 3.3** and were used to project buildout flows for the 2035 General Plan horizon.

3.3.3 Peaking Factors

The wastewater collection system is evaluated based on its ability to convey peak flows. A peaking factor represents the increase in flows experienced above the ADWF. This factor is calculated from historical data and, at times, tempered by engineering judgement.

The critical peaking conditions for the collection system include:

- Peak Dry Weather Flow (PDWF)
- Peak Wet Weather Flow (PWWF)

A typical factor of 1.1 to 2.0 is commonly used to estimate peak flows at wastewater treatment plants. The plant experiences smaller peaks because fluctuations are smoothed out during the time of travel in the collection system. A factor of 3.0 or 4.0 is used to estimate peak flows in the smaller upstream areas of the system where low flow conditions are prone to greater fluctuations.

This master plan used calibrated 24-hour diurnal patterns developed from the 2023 Flow Monitoring Program. These patterns simulate hourly changes in dry weather flows, averaging a factor of 1.0 over 24 hours.

3.4 WET WEATHER FLOW CRITERIA

The wet weather flow criteria accounts for Infiltration and Inflows (I&I) that seep into the City's wastewater collection system during storm events.

3.4.1 Infiltration and Inflows

I&I is associated with extraneous water entering the wastewater collection system through structural defects in pipes, manholes, cleanouts, or laterals. Infiltration occurs when groundwater rises, or the soil is saturated due to a storm event. Inflow occurs when surface water enters the collection system from illegal storm drain cross connections, defective manhole covers, or

Table 3.2 Wastewater Unit Factor Analysis
Wastewater Collection System Master Plan Update
City of Morgan Hill

Land Use Classification within City Limits	Existing Area (Gross Acres)	Existing Dwelling Units [1] (DUs)	Avg. Water Consumption (mgd)	Return to Sewer Rates [2] (%)	Avg. Wastewater Flows (mgd)	Unadjusted Wastewater Unit Factor [3] (gpd / acre)
Residential			2022 Average Day Demand	Calibrated per 2023 Flow Monitoring Program	2023 Average Dry Weather Flow	
Detached (Low to High) [4]	2,749	9,244	2.9	60%	1.7	625
Attached (Low to High) [5]	441	6,934	0.8	65%	0.5	1,135
Residential Subtotal	***3,190***	***16,178***	***3.6***		***2.2***	
Non-Residential						
Commercial / Industrial [6]	758		0.7	65%	0.5	624
Mixed Use [7]	157		0.2	40%	0.1	608
Public Facilities	302		0.2	40%	0.1	202
Non-Residential Subtotal	***1,217***		***1.1***		***0.6***	
Other (Non-Flow Generating)				System-Wide Return to Sewer Rate		
Agriculture / Open Space / Parks	1,622		0.4	-	-	-
Other Subtotal	***1,622***		***0.4***	-	-	-
Total	**6,028**	**16,178**	**5.2**	**55%**	**2.8**	

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Notes:
1. Dwelling units were obtained from the 2023 California Department of Finance Housing Survey database.
2. Calibrated return to sewer rates vary in each of the metered basins. The values on this table represent a system-wide average return to sewer rate for each land use.
3. The unadjusted wastewater unit flow factor represent a sytem-wide average and requires adjustments to account for vacant areas and specific land use densities (See Table 3.3).
4. "Detached" combines Residential Estate, Residential Detached Low, Medium and High into 1 category.
5. "Attached" combines Residential Attached Low, Medium and High into 1 category.
6. "Commercial / Industrial" combines commercial, general commercial, industrial, sports recreation leisure and commercial/industrial into 1 category.
7. "Mixed Use" combines Mixed Use and Mixed Use Flex into 1 category.

Table 3.3 Recommended Wastewater Unit Flow Factors

Wastewater Collection System Master Plan Update
City of Morgan Hill

General Plan Land Use	Adjusted Wastewater Unit Flow Factor (gpd / acre)
Single-Family Residential	
Residential Estate (up to 1 du / acre)	150
Residential Detached Low (up to 4 du / acre)	340
Residential Detached Medium (up to 7 du / acre)	630
Residential Detached High (6 - 12 du / acre)	840
Multi-Family Residential	
Residential Attached Low (6 - 16 du / acre)	1,100
Residential Attached Medium (16 - 24 du / acre)	1,700
Residential Downtown (24 - 46 du /acre)	2,930
Mixed Use	
Mixed Use (Variety of Land Uses)	960
Mixed Use Flex (7 - 24 du /acre)	900
Non-Residential	
Commercial	1,000
General Commercial	1,340
Commercial / Industrial	900
Industrial	900
Public Facilities (Includes Sports Recreation/Leisure)	220

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Note:

1. The recommended wastewater unit factors were adjusted for design purposes and account for vacant areas as well as specific land use densities (Example - Detached Medium and Detached High).

roof/footing drains. Potential I&I sources in a wastewater collection system are illustrated on **Figure 3.1**.

Several accepted methodologies for estimating I&I include:

- **Methodology 1.** Based on Tributary Sewershed Acreages. In this methodology, an I&I factor varying from 400 to 1,500 gallons per day per acre (gpd/acre) is applied to the tributary sewershed. The receiving pipes should be designed to accommodate this area based I&I allowance.
- **Methodology 2.** Based on Linear Feet (LF) of pipe or Linear Feet per Diameter (LF/inch diameter) of pipe. In this methodology, factors that may range between 12 to 30 gpd per inch diameter per 100 LF (gpd/inch diameter/100 LF) are applied to gravity pipes.
- **Methodology 3.** Based on a percentage of the ADWFs. In this methodology, the I&I component is estimated as a percentage of the dry weather flows.
- **Methodology 4.** Based on a flow monitoring program. In this methodology, the I&I component is determined by analyzing dry and wet weather flows in a collection system. The PDWF is initially determined from dry weather observations and subtracted from wet weather observations. The remaining volume can be attributed to rainfall and defined as I&I. The volume of I&I varies depending on the intensity of the wet weather event.

The I&I analysis for this Master Plan Update was completed using the 2023 Flow Monitoring Program (**Methodology 4**) detailed in **Appendix A**. This study was completed by V&A Consulting Engineers, Inc.

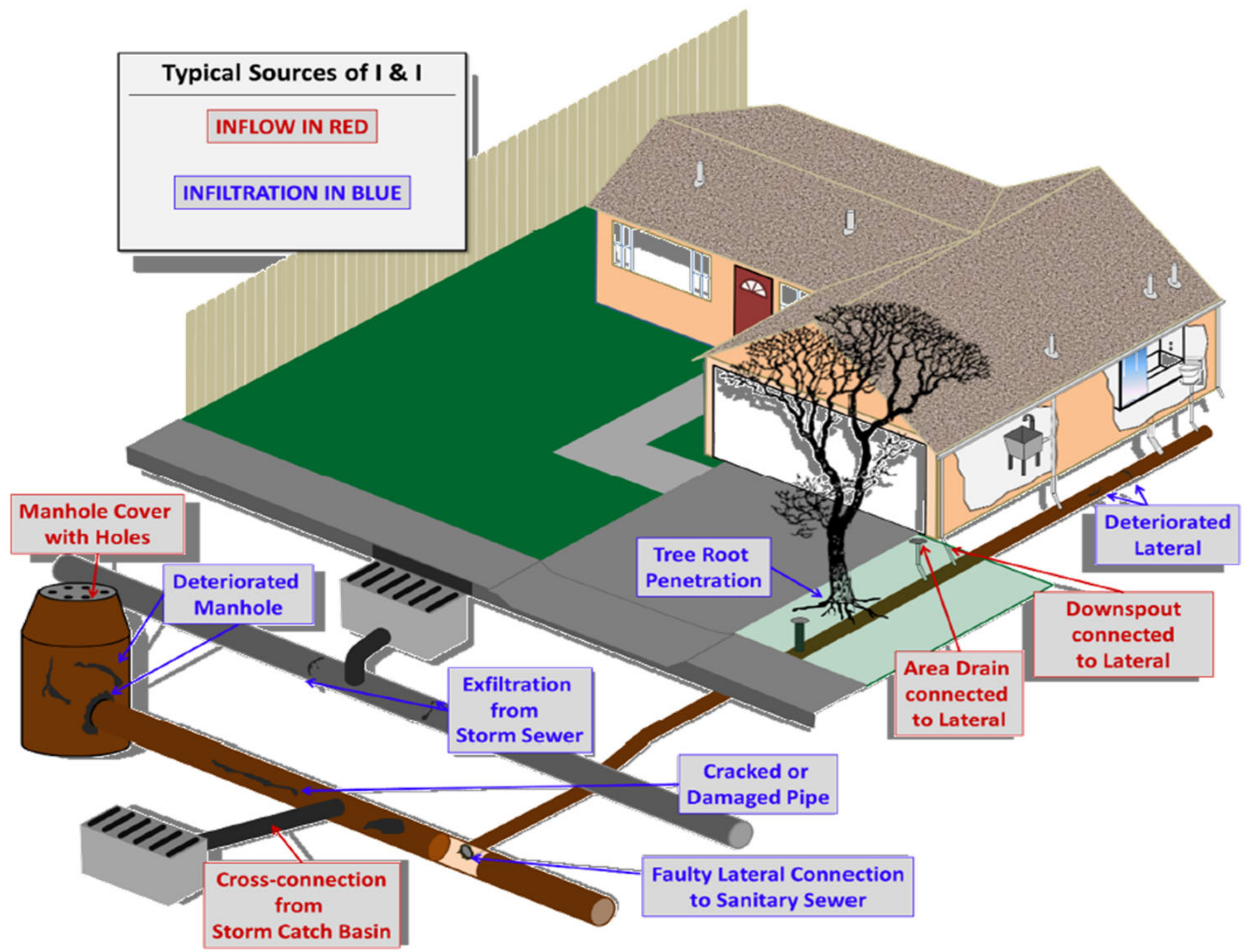
3.4.2 Design Storm

A design storm is a synthetic rainfall event used to simulate peak wet weather flows in a collection system. Currently, there is no regulatory policy that stipulates the use of a specific storm for wastewater collection systems. As such, Akel Engineering Group, Inc. reviewed the National Oceanic and Atmospheric Administration (NOAA) guidelines to develop a design storm for wet weather capacity evaluation, as summarized in **Appendix B**.

The selected design storm is illustrated on **Figure 3.2** and summarized as follows:

- **10-Year 24-Hour Synthetic.** This 24-hour storm has a total depth of 4.2 inches and a peak hourly intensity of approximately 0.8 inch/hour.

The storm return period (10-Year), duration (24-Hour) and distribution (synthetic) are consistent with the City's previous master plans.



LEGEND

- Inflow
- Infiltration

Source: 2023 City of Morgan Hill Inflow / Infiltration Study by V&A (Appendix A)

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Figure 3.1

Potential Sources of Infiltration and Inflows

Wastewater Collection System
Master Plan Update
City of Morgan Hill

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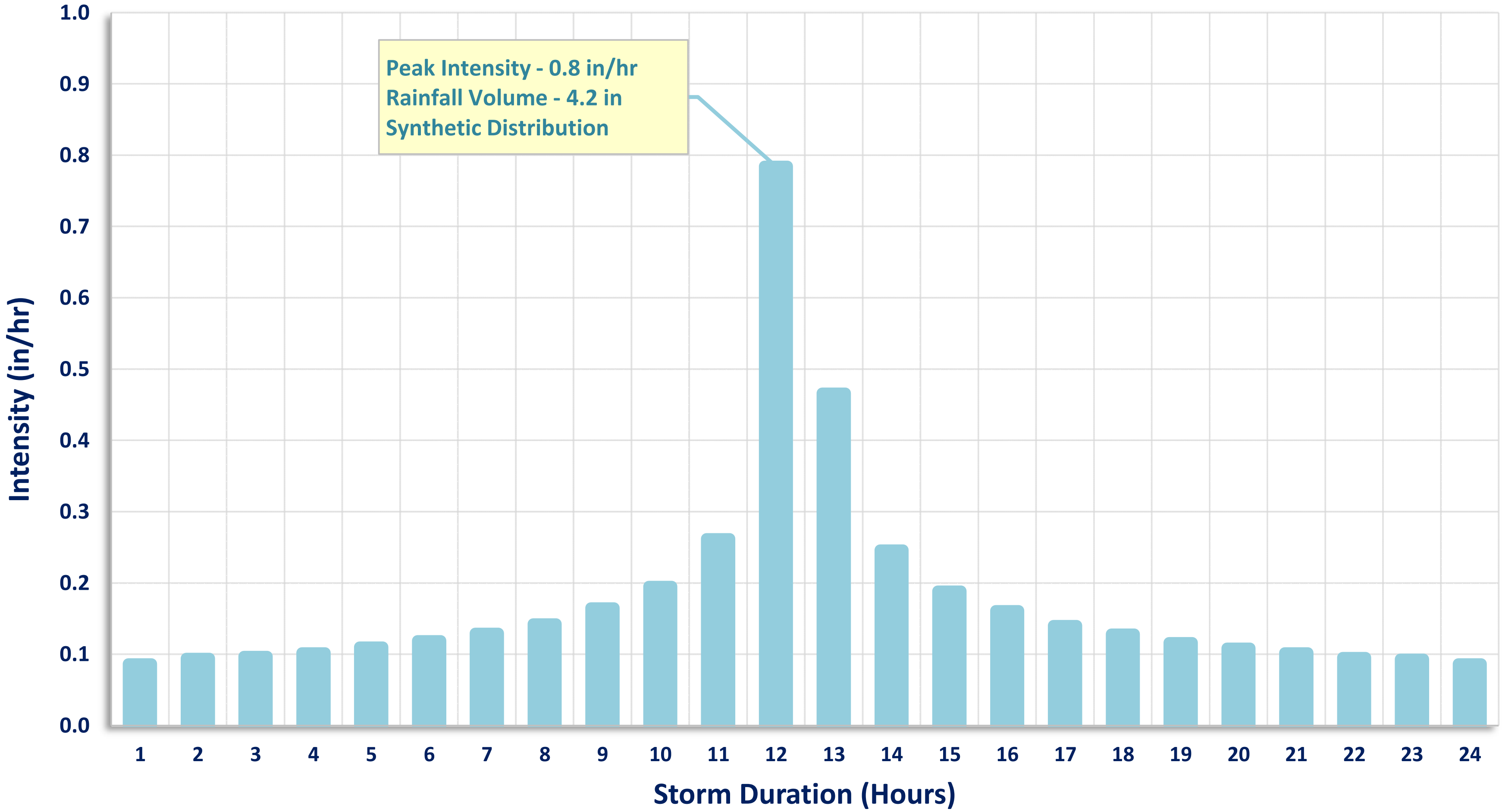

LEGEND

10-Year 24-Hour Design Storm (Appendix B)

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Figure 3.2

Design Storm for Wet Weather Capacity Evaluation

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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CHAPTER 4 – EXISTING WASTEWATER COLLECTION SYSTEM FACILITIES

This chapter provides a description of the City's existing wastewater facilities, including 6 major trunk collectors, Joint Trunk system and lift stations. This chapter also describes the SCRWA WWTP, which treats and recycles the City's wastewater.

4.1 WASTEWATER COLLECTION SYSTEM OVERVIEW

The City provides wastewater collection services to approximately 11,500 residential and non-residential customers. This service is currently funded by a fixed annual fee that was last amended by the City Council in July 2019. The service fees are allocated towards capital improvements and maintenance of the system.

According to the City's GIS database, the existing collection system consists of approximately 164 miles of pipes and 14 lift stations, as shown on Figure 4.1. The wastewater pipes vary in size from 4-inches to 30-inches in diameter, and service approximately 46,000 residents through more than 12,500 lateral connections. An infrastructure inventory listing the pipe length by diameter is also shown on Table 4.1.

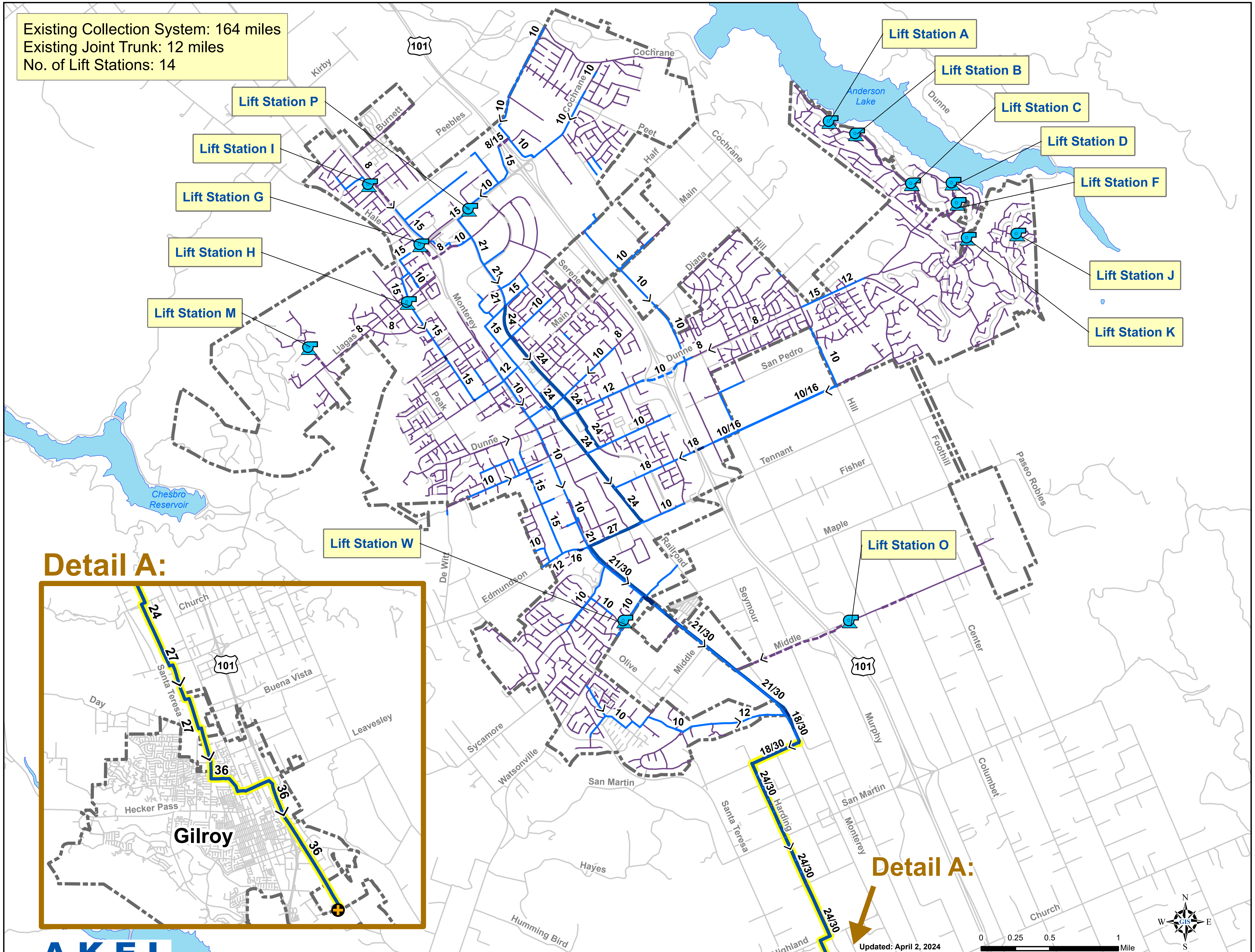
Infrastructure Inventory	
159	Miles of Gravity Pipes
3	Miles of Force Mains
2	Miles of Siphons
3,800+	Manholes
14	Lift Stations

Ultimately, wastewater collected from the City is conveyed south via a 12-mile Joint Trunk system that discharges into the SCRWA WWTP in the City of Gilroy. The WWTP is owned and operated under a JPA between the City of Morgan Hill and the City of Gilroy.

4.2 GRAVITY TRUNK COLLECTORS

Due to topography, the City's collection system is divided into 6 dendritic basins, which are serviced by 6 gravity trunk collectors:

- **Butterfield Trunk.** This trunk starts at the intersection of Cochrane Road and Butterfield Boulevard as a 21-inch diameter pipe in a southbound direction. The pipe follows Butterfield Boulevard before increasing to 24-inch near Jarvis Drive. The trunk ends near San Pedro Avenue, where it turns west and discharges into the Railroad-Monterey Trunk.
- **Llagas Trunk.** This trunk flows west from the intersection of Sanchez Drive and Monterey Road as a 15-inch diameter pipe. The pipe continues westerly and then turns south along Del Monte Avenue. The trunk then turns west and continues towards the intersection of Llagas Road and Hale Avenue. At this intersection, the trunk turns south and follows Hale Avenue, until it reaches Christine Lynn Drive and discharges into the Hale-Monterey Trunk.



Legend

Existing System

- Lift Stations
- SCRWA WWTP
- Joint Trunk

Gravity Pipes by Diameter

- 8" or Smaller
- 10" - 21"
- 24" or Larger

Force Mains / Siphons by Diameter

- 8" or Smaller
- 10" - 12"
- Greater than 14"

Other Features

- Roads
- Lakes
- City Limits

Figure 4.1
Existing Wastewater Collection System

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Final\Fig_4.1_ExistingWWCollectionSystem_040224.aprx

Table 4.1 Wastewater Infrastructure Inventory

Wastewater Collection System Master Plan Update
City of Morgan Hill

Pipe Diameter (in)	Total Length [1] (feet)	(miles)
Gravity Pipes		
4" or Smaller	1,965	0.4
6"	284,936	54.0
8"	375,629	71.1
10"	78,442	14.9
12"	17,813	3.4
14"	456	0.1
15"	23,421	4.4
16"	7,065	1.3
18"	6,096	1.2
20"	491	0.1
21"	14,444	2.7
24"	12,982	2.5
27"	2,396	0.5
30"	10,360	2.0
Subtotal	**836,495**	**158.4**
Force Mains		
4" or Smaller	6,322	1.2
6"	8,703	1.6
8"	1,490	0.3
Subtotal	**16,515**	**3.1**
Siphons		
6"	1,168	0.2
8"	2,846	0.5
10"	3,194	0.6
12" or Higher	3,609	0.7
Subtotal	**10,817**	**2.0**
Joint Trunk (Shared Ownership between City of Morgan Hill and City of Gilroy)		
21" or Smaller	2,681	0.5
24"	14,997	2.8
27"	9,684	1.8
30"	12,507	2.4
36" or Higher	22,368	4.2
Subtotal	**62,237**	**11.8**
Summary (Excluding Joint Trunk)		
	Total Pipe Length	**163.6**
	Number of Manholes / Cleanouts	**3,888**
	Number of Siphons	**25**
	Number of Lift Stations / Force Mains	**14**

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Notes:

1. Inventory was tabulated from the City's GIS database received on April 3, 2023, and updated through discussions with the City staff to include recently constructed infrastructure.

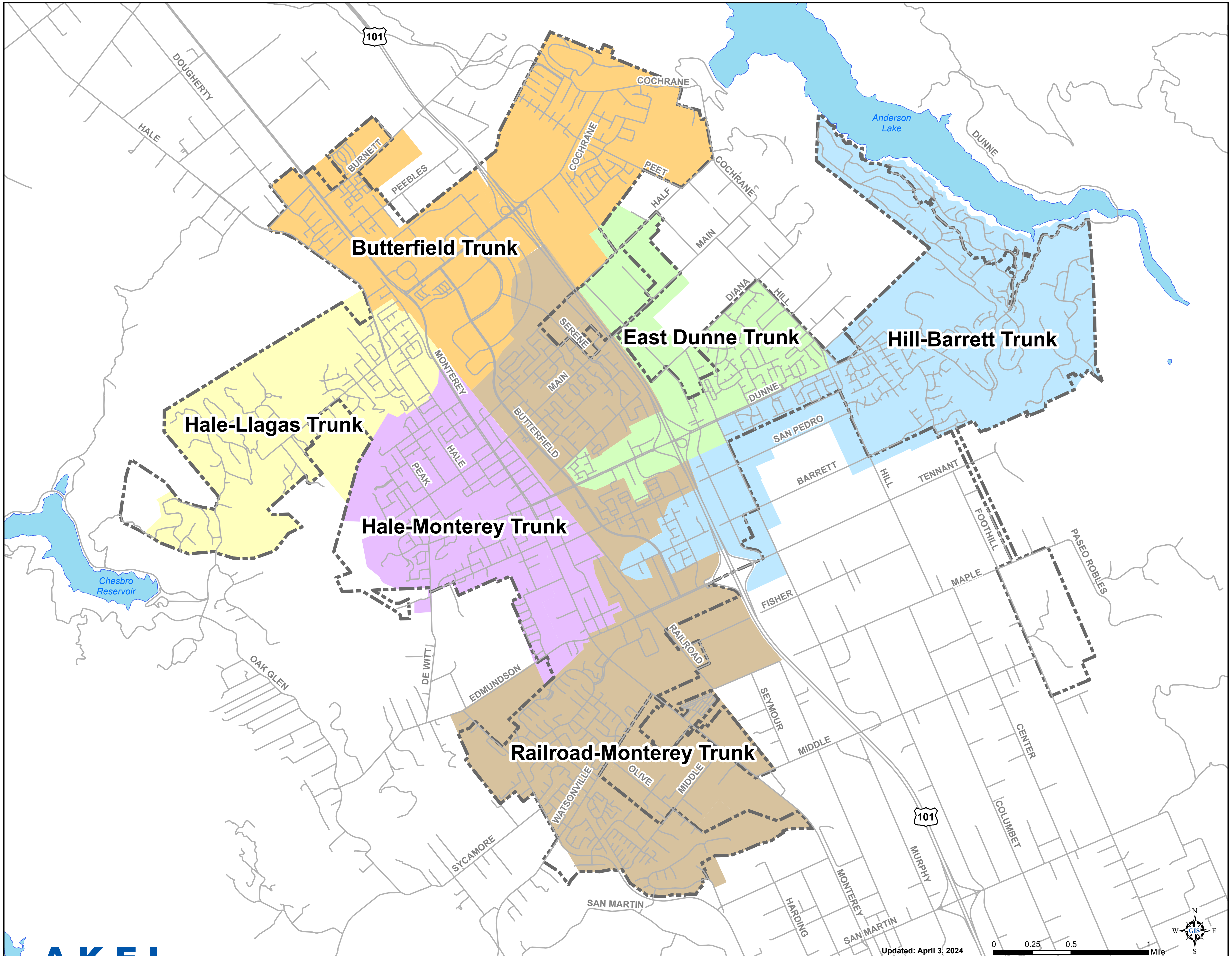
- **Hale-Monterey Trunk.** The trunk starts at the intersection of Hale Avenue and Christine Lynn Drive in a southerly direction following Hale Avenue as a 15-inch diameter pipe. The trunk increases in size for a short section near Main Avenue and then decreases back to 15-inch. The trunk then turns east on Main Avenue and south on Monterey Road. The trunk size decreases to 10-inch along Monterey Road and then increases to 21-inch at Edes Street, before consolidating into the Railroad-Monterey Trunk.
- **Hill-Barrett Trunk.** This trunk flows south from the intersection of East Dunne Avenue and Hill Road as a 8-inch diameter pipe. The pipe continues south along Hill Road and increases to 10-inch at Sundance Drive. The pipe then turns west at Barrett Avenue and runs in parallel with a 16-inch diameter pipe. Both pipes converge into a single 18-inch diameter pipe near Highway 101 and Condit Road. The trunk then continues in a westerly direction and ends at the intersection of Barrett Avenue and Railroad Avenue, where it consolidates into the Railroad-Monterey Trunk.
- **East Dunne Trunk.** This trunk flows west from the intersection of East Dunne Avenue and Hill Road as a 8-inch diameter pipe. The main continues westerly along East Dunne Avenue before increasing to 10-inch near Condit Road. The trunk increases to 12-inch after crossing Highway 101 and then discharges into the Butterfield Trunk at the intersection of East Dunne Avenue and Butterfield Boulevard.
- **Railroad-Monterey Trunk.** This trunk flows southeast from the intersection of Butterfield Boulevard and Jarvis Drive as a 15-inch diameter pipe. The trunk continues westerly along easements before turning south at Railroad Avenue and following the railroad alignment before increasing to 24-inch pipe near East Dunne Avenue. The trunk turns west at Tennant Avenue, where it increases in size to 27-inch and continues to Monterey Road. The trunk increases to a 30-inch pipe as it turns south on Monterey Road and runs parallel with 21-inch pipes to California Avenue, where it consolidates into the Joint Trunk system.

The tributary collection basins for each trunk are graphically shown on Figure 4.2. Additionally, the limits of each trunk are highlighted on Figure 4.3, followed by a connectivity schematic illustrated on Figure 4.4.

4.3 JOINT TRUNK SYSTEM

Wastewater flows from City are conveyed into a Joint Trunk System located at the intersection of Monterey Road and California Avenue in the Village of San Martin (Figure 4.1). This system includes approximately 12 miles of 18-inch to 30-inch diameter gravity pipes (Table 4.1) that discharge into the SCRWA WWTP in the City of Gilroy. The Joint Trunk is also equipped with a permanent flow meter near the intersection of Harding Avenue and Highland Avenue.

The JPA between the City of Morgan Hill and the City of Gilroy documents capacity allocations and maintenance responsibilities for each segment of the Joint Trunk. This agreement was signed on May 19th, 1992, and stipulates a 4.0 MGD capacity allocation for the City of Morgan Hill from



Legend

Collection Basins

- Butterfield Trunk
- East Dunne Trunk
- Hale-Llagas Trunk
- Hale-Monterey Trunk
- Hill-Barrett Trunk
- Railroad-Monterey Trunk
- Roads
- City Limits
- Lakes

Figure 4.2
Collection System Basins
Wastewater Collection System
Master Plan Update
City of Morgan Hill

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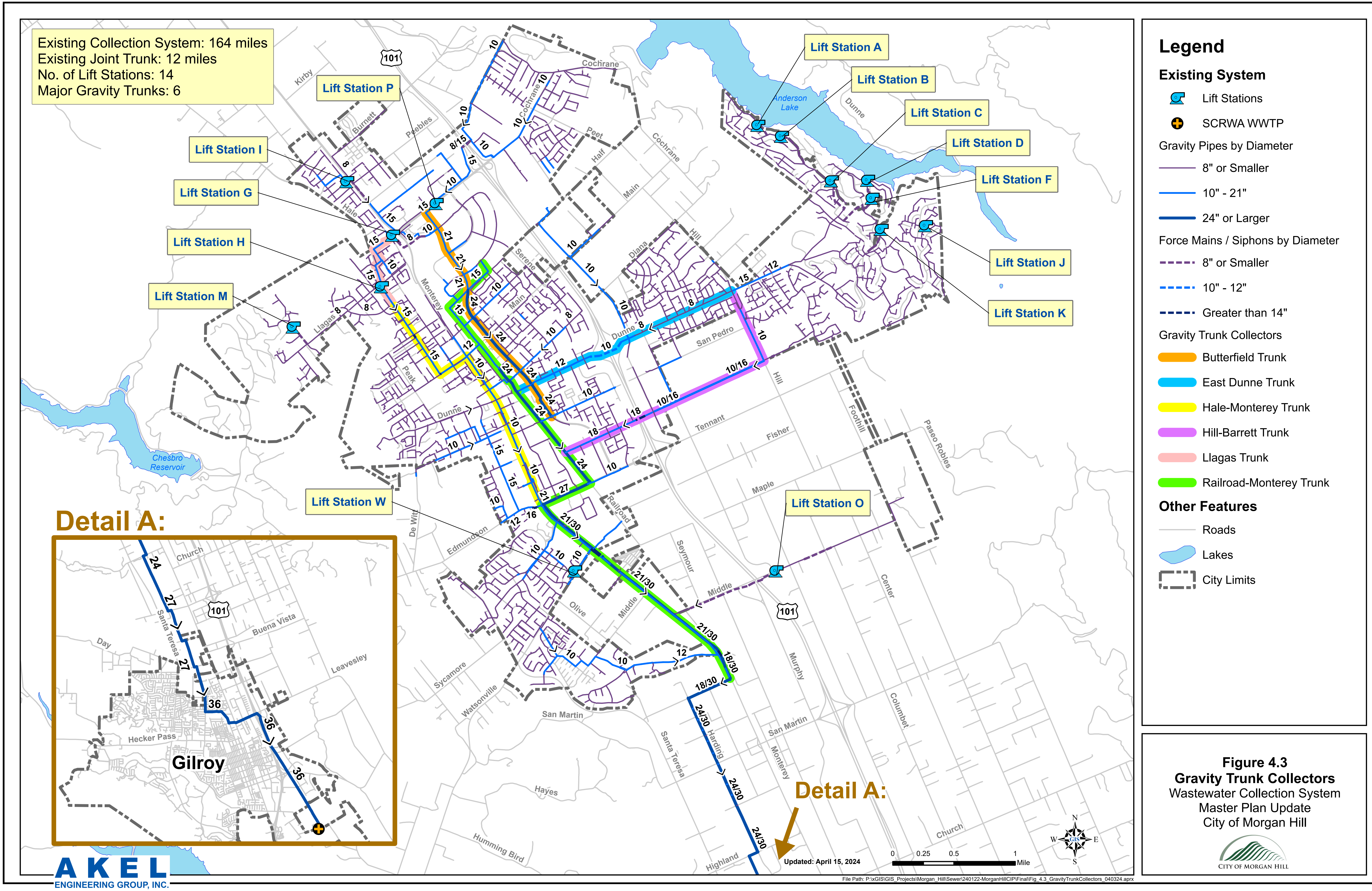
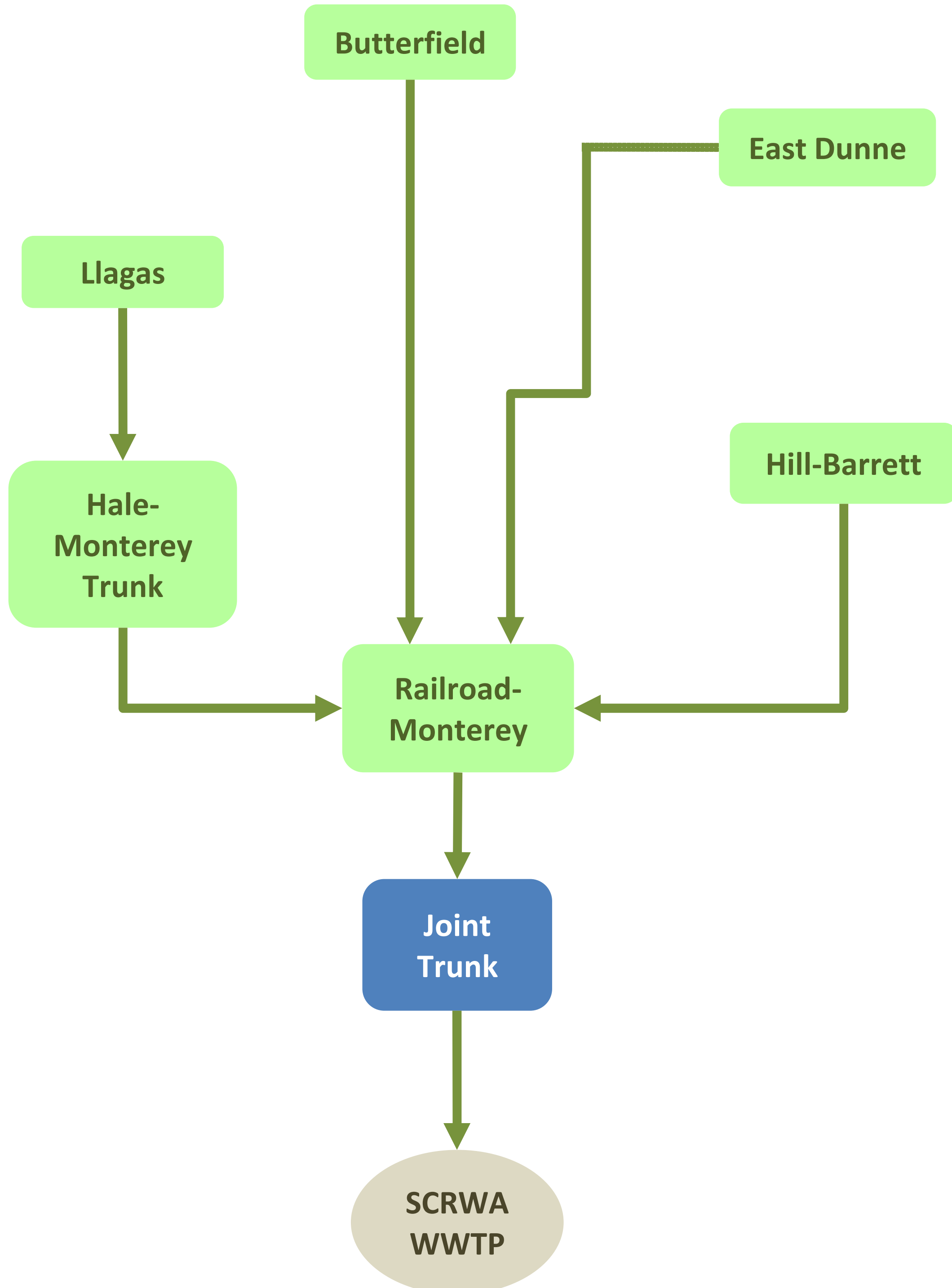


Figure 4.3
Gravity Trunk Collectors
Wastewater Collection System
Master Plan Update
City of Morgan Hill



LEGEND

TRUNK ID	Trunk Tributary Area	→	Pipelines
Joint Trunk	Permanemt Flow Meter		
WWTP	Wastewater Treatment Plant		

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Figure 4.4
Gravity Trunk Connectivity
Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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California Avenue in San Martin to Farrell Avenue in the City of Gilroy. Subsequently, the agreement stipulates a 7.7 MGD capacity allocation for the City of Morgan Hill in the final reaches between Farrell Avenue to the SCRWA WWTP. Previous master planning efforts included a thorough capacity evaluation of the Joint Trunk system and recommended constructing a parallel Relief Trunk. Since then and to facilitate the buildout of the 2035 General Plan, the City of Morgan Hill has initiated a capital project to construct a 36-inch diameter Relief Trunk.

4.4 LIFT STATIONS

Lift stations are used to pump flows when wastewater cannot be routed via gravity. The City currently operates 14 lift stations, as listed on **Table 4.2** with location, last upgrade year, force main diameter and total pumping capacity. The firm capacity, which refers to the pumping capacity with the largest pump out-of-service, is also provided on this table. Each lift station is actively monitored by the City's SCADA system to maintain operational efficiency.

4.5 FLOW DIVERSIONS

The City's wastewater collection system consists of two main flow diversion structures that provide an opportunity to route flow away from trunks with capacity limitations. These diversions are summarized as follows:

- **Hill – East Dunne Diversion.** This structure is located at the intersection of Hill Road and East Dunne Avenue in the northeast quadrant of the City. This structure can convey flows south into a 8-inch diameter pipe along Hill Road (Hill-Barrett Trunk) or west into another 8-inch diameter pipe along Dunne Avenue (East Dunne Trunk). Field verifications completed by City staff indicate that all flows are routed south and the east pipe functions as an overflow for wet weather conditions.
- **Main – Monterey Diversion.** This structure is located near Downtown, at the intersection of Main Avenue and Monterey Road. This structure can convey flows east into a 15-inch diameter pipe along Main Avenue (Railroad-Monterey Trunk) or south into a 10-inch diameter pipe along Monterey Road (Hale-Monterey Trunk). The 2023 Flow Monitoring Program indicates that the majority of the flows are routed east while the south pipe functions as an overflow for wet weather conditions.

4.6 WASTEWATER TREATMENT

Wastewater flows from the City of Morgan Hill are treated and recycled at the SCRWA WWTP located at the end of Southside Drive in the City of Gilroy. SCRWA was formed as part of the JPA to collectivity treat wastewater from the City of Morgan Hill and the City of Gilroy.

The SCRWA WWTP was originally constructed in 1990 and most recently upgraded in 2007 to provide a primary, secondary, and tertiary treatment capacity of 8.5 mgd. The effluent from the WWTP is delivered to recycled water customers within the region or disposed via on-site percolation ponds.

Table 4.2 Lift Station Inventory
Wastewater Collection System Master Plan Update
City of Morgan Hill

Lift Station Name	Location	Last Upgraded	Force Main Diameter (inches)	No. of Pumps	Total Pumping Capacity (gpm)	Total Pumping Capacity (mgd)	Firm Pumping Capacity [2] (gpm)	Firm Pumping Capacity [2] (mgd)
Lift Station A	17670 Racoon Court	1995	4''	2	1,012	1.5	506	0.7
Lift Station B	17558 Holiday Drive	2003	6''	2	1,310	1.9	655	0.9
Lift Station C	3272 Quail Lane	2007	6''	2	1,012	1.5	506	0.7
Lift Station D	17110-B Shady Lane	1998	4''	2	1,012	1.5	506	0.7
Lift Station F	17109 Holiday Drive	1995	4''	2	1,012	1.5	506	0.7
Lift Station G	8615 Monterey Road	2005	6'' and 8''	2	1,568	2.3	784	1.1
Lift Station H	320 Llagas Road	1999	4''	2	1,030	1.5	515	0.7
Lift Station I	19160 Saffron Drive	2000	6''	2	988	1.4	494	0.7
Lift Station J	16035 Jackson Oaks Drive	1992	6''	2	1,108	1.6	554	0.8
Lift Station K	3300 East Dunne Avenue	-	4''	2	1,030	1.5	515	0.7
Lift Station M	1162 Llagas Road	1999	6''	2	968	1.4	484	0.7
Lift Station O	952 East Middle Avenue	-	6''	2	1,074	1.5	537	0.8
Lift Station P	320 Woodview Avenue	2009	4''	2	916	1.3	458	0.7
Lift Station W	15505 Watsonville Road	-	6''	2	1,030	1.5	515	0.7

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Notes:
1. Lift station and force main information was obtained from the City's 2022 Sanitary Sewer System Management Plan.
2. Firm capacity refers to the capacity available with the largest pump out-of-service.

CHAPTER 5 – WASTEWATER FLOWS

This chapter discusses the existing and future design flows in the wastewater collection system.

5.1 EXISTING WASTEWATER FLOWS

Existing wastewater flows in the City's collection system were quantified from the 2023 Flow Monitoring Program, which measured flows at 11 strategic locations, including the permanent meter near the intersection of Harding Avenue and Highland Avenue. This permanent meter is located in the Joint Trunk system and therefore captures existing wastewater flows generated by the City of Morgan Hill. The permanent meter data was also validated and confirmed using another temporary meter. Overall, the existing wastewater flows from the City are documented on **Table 5.1** and quantified as follows:

- **Average Dry Weather Flow (ADWF).** Represents the daily average flow during dry weather conditions. This flow was quantified at approximately 2.8 mgd.
- **Peak Dry Weather Flow (PDWF).** Represents the hourly peak flow during dry weather conditions. This flow was quantified at approximately 4.7 mgd.
- **Peak Wet Weather Flow (PWWF).** Represents the hourly peak flow during wet weather conditions simulated through a synthetic 10-Year 24-Hour design storm event. This flow was estimated at approximately 13.9 mgd. It should be noted that the hydraulic analysis indicated overflows at select locations during the wet weather simulation. The gravity pipes in these locations were upsized to alleviate overflows.

5.2 FUTURE WASTEWATER FLOWS

Future wastewater flows were projected based on land uses in the City's 2035 General Plan (**Figure 2.3**) and unit flow factors developed through calibration (**Table 3.3**). This land-use based approach accounts for urbanization of undeveloped lands as well as re-development or intensification in specific growth areas.

For future wet weather conditions, the volume of I&I entering the collection system was decreased by 20 percent to account for the City's planned R&R improvements. Future wastewater flow projections are documented on **Table 5.1** and estimated as follows:

- **ADWF** of 4.3 mgd
- **PDWF** of 7.3 mgd
- **PWWF** of 15.2 mgd

Table 5.1 Wastewater Collection System Flows

Wastewater Collection System Master Plan Update
City of Morgan Hill

Hydraulic Model Scenarios	Average Dry Weather Flow ADWF (mgd)	Peak Dry Weather Flow PDWF (mgd)	Peak Wet Weather Flow PWWF (10-Year 24-Hour Design Storm) (mgd)
Existing Conditions [1,2]	2.8	4.7	13.9
Future Conditions [3]	4.3	7.3	15.2

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Notes:

1. Existing flows were obtained from the permanent meter located along the Joint Trunk at the intersection of Harding Avenue and Highland Avenue.
2. Select pipes within the upstream tributary areas were upsized to alleviate hydraulic model overflows and capture the design peak wet weather flow.
3. Future conditions include buildout flow projections and a 20% Infiltration and Inflow reduction to account for the City's planned R&R improvements.

CHAPTER 6 – HYDRAULIC MODEL DEVELOPMENT

This chapter describes the development and calibration of the City's GIS-based wastewater collection system hydraulic model.

6.1 SOFTWARE SELECTION

A hydraulic model is an analytical tool that combines physical and operational characteristics of a wastewater collection system. The hydraulic model then solves a series of equations to simulate flows in pipes, including backwater calculations for surcharged conditions. The selection of a particular software ultimately depends on user preferences and costs for purchasing the software.

InfoWorks ICM Ultimate by Autodesk Inc. was selected as the preferred hydraulic modeling software due to its ability to manage multiple scenarios and integrate GIS datasets. This software uses the fully dynamic St. Venant equations with a SWMM5 calculation engine for simulating backwater effects for surcharged conditions.

6.2 HYDRAULIC MODEL DEVELOPMENT

The City's GIS database was used as the primary source for hydraulic model development. The GIS database contained key physical attributes such pipe sizes, manhole invert elevations and manhole ground elevations. Notable data gaps such as missing inverts or pipe sizes were resolved through discussions with the City staff and summarized as follows:

- GIS features identified as abandoned or private were not included in the hydraulic model.
- Laterals and cleanouts were not included in the hydraulic model.
- GIS features with missing or erroneous physical attributes were verified using record drawings or through discussions with the City staff.
- Dummy junctions and pipes were created in some areas for model connectivity.
- Missing ground elevations for manholes were inferred using the County of Santa Clara's 2020 Digital Elevation Model (1 feet resolution).
- Missing invert elevations were inferred via linear interpolation or assumed based on typical pipe cover and minimum design slopes.
- Operational characteristics such as pump curves and pump set points were obtained from lift station record drawings and relevant design reports.

The following section lists hydraulic model elements that represent the City's wastewater facilities.

6.2.1 Hydraulic Model Elements

The hydraulic component of the model routes wastewater flows from tributary catchments to the boundary condition, and consists of the following elements:

- **Tributary Catchments.** These elements represent parcels that generate wastewater flows. The City of Morgan Hill's parcel dataset contained a unique accessor parcel number that was cross-referenced with the water billing records for flow distribution. Based on the existing land use, a return to sewer rate was applied to estimate wastewater flows for each parcel.
- **Manholes.** These elements represent junctions that provide connectivity between gravity pipes and force mains. They receive wastewater flows from tributary catchments and contain information related to invert and ground elevations.
- **Gravity Pipes and Force Mains.** These elements convey wastewater flows either via gravity or pressurized mains. Gravity pipes and force mains are modeled with frictional losses simulated through the Manning's n roughness coefficient (gravity pipes) or a Hazen-Williams C-factor (force mains). The frictional losses are a measure of pipe smoothness and dependent on pipe material and age. Key inputs required to model these elements include invert elevation, diameter, and frictional loss coefficient.
- **Wet Wells.** These elements are defined as point features that represent the City's lift stations. Key inputs required to model these elements include invert elevation, ground elevation and wet well storage area.
- **Pumps.** These elements are defined as line features that pump wastewater flows from a lift station into a force main. Pumps are needed to convey wastewater flows to downstream areas located at a higher elevation. They are modeled with a pump curve, which correlates the relationship between the head delivered by the pump and the flow through the pump. Control rules with logical statements based on time or head are applied to turn pumps on and off.
- **Outfalls.** These elements function as downstream boundary conditions and represent discharge points such as the SCRWA WWTP.

6.3 MODEL CALIBRATION

Model calibration is an iterative process of comparing the model flows with observations and revising the input parameters until the predicted results are acceptable. This process is intended to instill a strong level of confidence in the hydraulic model results. In wastewater collection systems, it is common practice to calibrate flows to three dynamic conditions:

- Peak dry weather flows on one day during the week and one on the weekend.
- Peak wet weather flows from Wet Weather Event No. 1.
- Peak wet weather flows from Wet Weather Event No. 2.

An overview of the flow monitoring program and the calibration approach are detailed in the following sections.

6.3.1 2023 Flow Monitoring Program

As part of this WCSMP Update, the City commissioned a 7-week Flow Monitoring Program from January 9th to February 22nd, 2023. The City retained V&A Consulting Engineers to deploy 10 temporary meters and collect data from 1 permanent meter.

The selected flow monitoring sites and tributary basins are shown on **Figure 6.1**, and listed on **Table 6.1**. These sites were selected through discussions with City staff and capture major trunk collectors as well as strategic flow diversions.

A total of 8 rain gauges were also reviewed to determine spatial variation in rainfall. The flow monitoring report completed by V&A is included in **Appendix A** for reference purposes. The flow monitoring results were reviewed for outliers and adjusted where necessary.

During dry weather conditions, most flow meters showed typical diurnal patterns observed in a residential community. A peak flow was observed in the morning as residents start their day, followed by a decline in the afternoon, and a second peak in the evening as residents return home. During the weekends, most flow meters showed a peak flow during late morning or early afternoon hours, followed by a gradual decline until midnight. During wet weather conditions, the flow meters showed varying levels of I&I.

6.3.2 Dry Weather Calibration

The dry weather flows were calibrated by adjusting the return to sewer rates and applying synthetic weekday and weekend diurnal patterns measured in February 2023. A comparison of the modeled and observed dry weather flows are illustrated in **Appendix C** and indicate a strong correlation between both hydrographs.

6.3.3 Wet Weather Calibration

The wet weather flows were calibrated based on a wet weather event from January 13 to 16, 2023. This event had a total rainfall depth of 4.7 inches over a span of 3 days, resembling a 2-year return period storm. A peak intensity of 0.6 inch/hour was also measured during this event, as shown on **Table 6.2**.

The RTK Unit Hydrograph method was employed to calibrate hydraulic model flows. This method relies on three-unit hydrographs to control the hydraulic model peaks:

1. **Fast Inflow.** This hydrograph is defined as the short-term response in the collection system, represented by R_1, T_1 and K_1 parameters. This response occurs due to direct sources of inflow such as connected roof leaders, illegal storm drain connections or foundation drains.

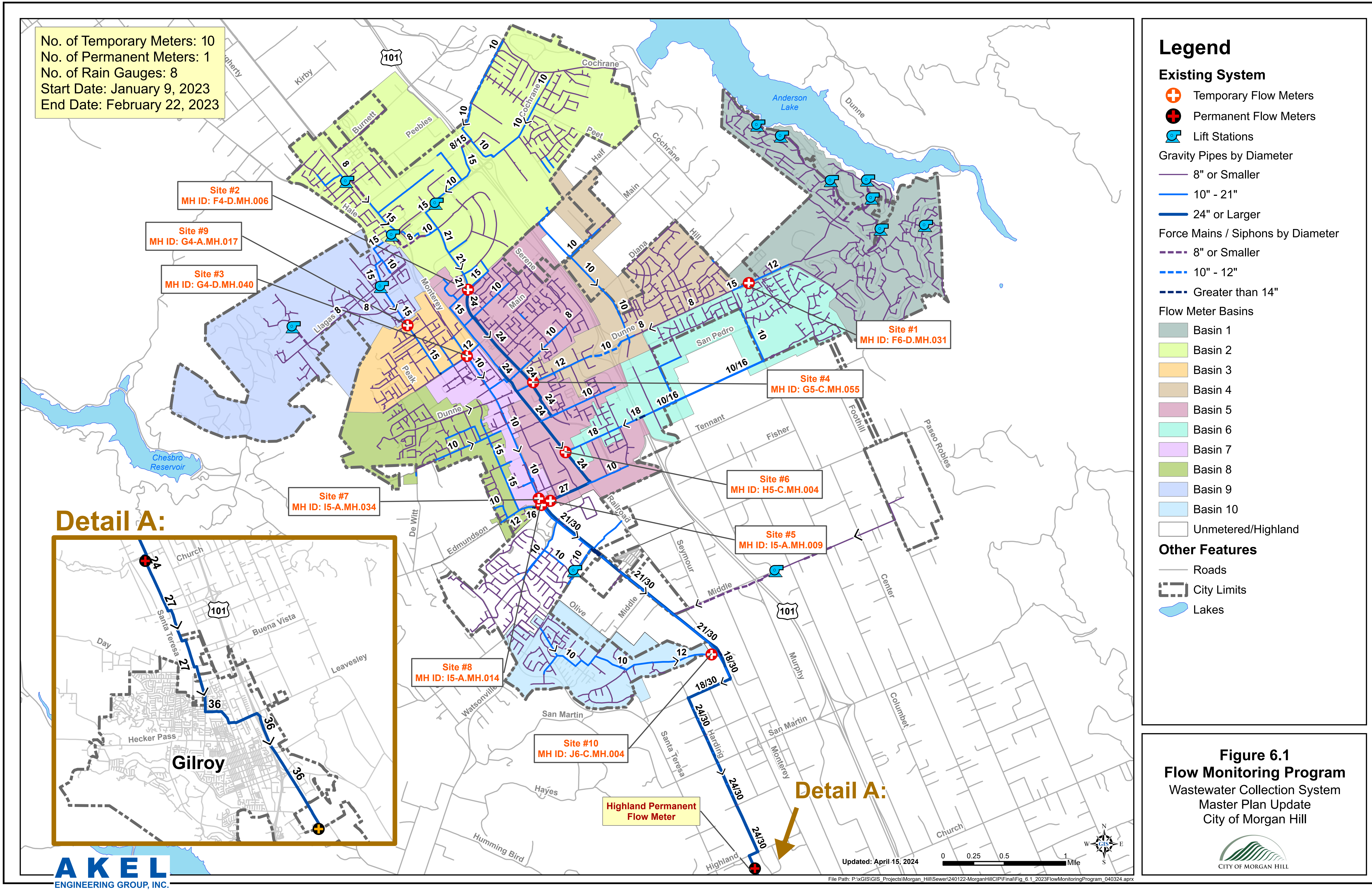

Figure 6.1
Flow Monitoring Program
Wastewater Collection System
Master Plan Update
City of Morgan Hill

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Table 6.1 Flow Monitoring Sites

Wastewater Collection System Master Plan Update
City of Morgan Hill

Site No.	GIS Manhole ID	Location	Pipe Information	
			GIS ID	Size (inches)
Temporary Flow Monitors				
1	F6-D.MH.031	Dunne Ave and Hill Rd	5450	15'' Inlet (NE)
2	F4-D.MH.006	600 ft South of Butterfield Blvd south of Digital Dr	6643	21'' Inlet (N)
3	G4-D.MH.040	Main Ave and Monterey Rd	4404	15'' Inlet (SW)
4	G5-C.MH.055	Dunne Ave east of Butterfield Blvd	3905	12'' Inlet (NE)
5	I5-A.MH.009	Monterey Rd and Tennant Ave	27673	27'' Inlet (NE)
6	H5-C.MH.004	Railroad Ave and Barrett Ave	4456	18'' Inlet (NE)
7	I5-A.MH.034	Monterey Rd and Edmundson Ave	3942	21'' Inlet (N)
8	I5-A.MH.014	Monterey Rd and Tennant Ave	6132	15'' Inlet (SW)
9	G4-A.MH.017	Hale Ave, SE of Hillwood Ln	6361	15'' Inlet (NW)
10	J6-C.MH.004	Monterey Rd, adjacent to Llagas Creek	4368	12'' Inlet (W)
Permanent Highland Avenue Flow Monitor (Joint Trunk)				
11	13105	Highland Ave and Harding Ave	19052	27'' Inlet (NW)

Table 6.2 January 2023 Wet Weather Event

Wastewater Collection System Master Plan Update
City of Morgan Hill

Wet Weather Event	Start Date/Time	End Date/Time	Total Duration (hours)	Total Depth (inches)	Peak Intensity (inches / hour)	Estimated Return Period [1]
Event No. 1	1/13/2023 10:00 AM	1/16/2023 7:00 AM	69	4.7	0.6	2-Year Return Period Event

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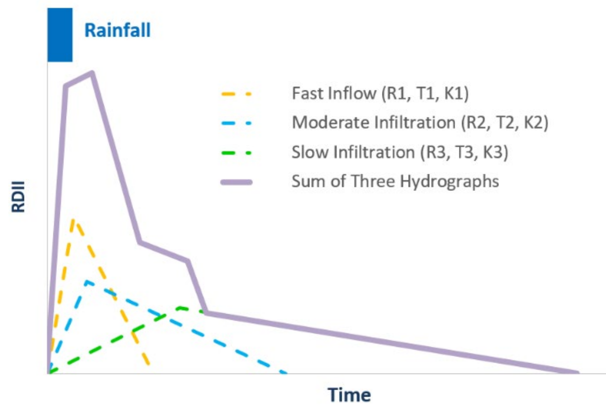
Note:

1. Return period was estimated from the City of Morgan Hill (Weather Station ID 04-5844) Intensity Duration Frequency Curves obtained from NOAA Atlas 14 Point Frequency Data Server.

2. **Moderate Infiltration.** This hydrograph is defined as the medium-term response in the collection system, represented by R_2, T_2 and K_2 parameters. This response typically indicates major structural defects in pipes, manholes or laterals.
3. **Slow Infiltration.** This hydrograph is defined as the long-term response in the collection system, represented by R_3, T_3 and K_3 parameters. This response typically indicates minor structural defects or presence of groundwater.

The sum of these hydrographs influences the wet weather response in the hydraulic model (**Exhibit B**). The shape of the modeled hydrograph can be controlled and optimized by varying 9 parameters from the 3-unit hydrographs:

- **R_1, R_2 and R_3.** Represents the fraction of rainfall that enters the collection system, where the 'R-value' is the sum of $R_1 + R_2 + R_3$.
- **T_1, T_2 and T_3.** Represents the time to peak in hours.
- **K_1, K_2 and K_3.** Represents the ratio of the time to recession, to the time to peak.



Several iterations were completed to adjust the RTK parameters and yield an acceptable comparison between the modeled and observed flows. The wet weather calibration results for each site are documented in **Appendix C**.

6.3.4 Use of the Calibrated Model

Calibration was completed to increase model confidence and enhance its ability to simulate current flows. After calibration, the hydraulic model was used to evaluate the capacity adequacy of the collection system under dry and wet weather conditions.

The hydraulic model is a valuable investment that will continue to prove its worth to the City as future planning issues or other operational conditions surface. It is recommended that the model be maintained with updated flows to represent real-world conditions.

CHAPTER 7 – CAPACITY EVALUATION AND IMPROVEMENTS

This chapter summarizes capacity evaluation results under existing and future flow conditions. The hydraulic capacity improvements needed to mitigate system deficiencies are also discussed in this chapter.

7.1 CAPACITY EVALUATION

A capacity evaluation was performed for the following hydraulic model scenarios:

- **Scenario 1 - Existing PDWF.** This scenario represents existing dry weather conditions.
- **Scenario 2 - Existing PWWF.** This scenario represents existing wet weather conditions analyzed using a 10-Year 24-Hour synthetic design storm event.
- **Scenario 3 - Future PDWF**. This scenario represents projected dry weather conditions developed from the City's General Plan (**Figure 2.3**) and calibrated unit flow factors (**Table 3.3**). This scenario also incorporates two imminent hydraulic capacity improvements that are currently at the design phase.
- **Scenario 4 - Future PWWF.** This scenario represents projected wet weather conditions, analyzed using a 10-Year 24-Hour synthetic design storm event. In addition to the General Plan land uses and two imminent hydraulic capacity improvements, this scenario assumes a 20 percent reduction in I&I to account for the City's planned R&R improvements.

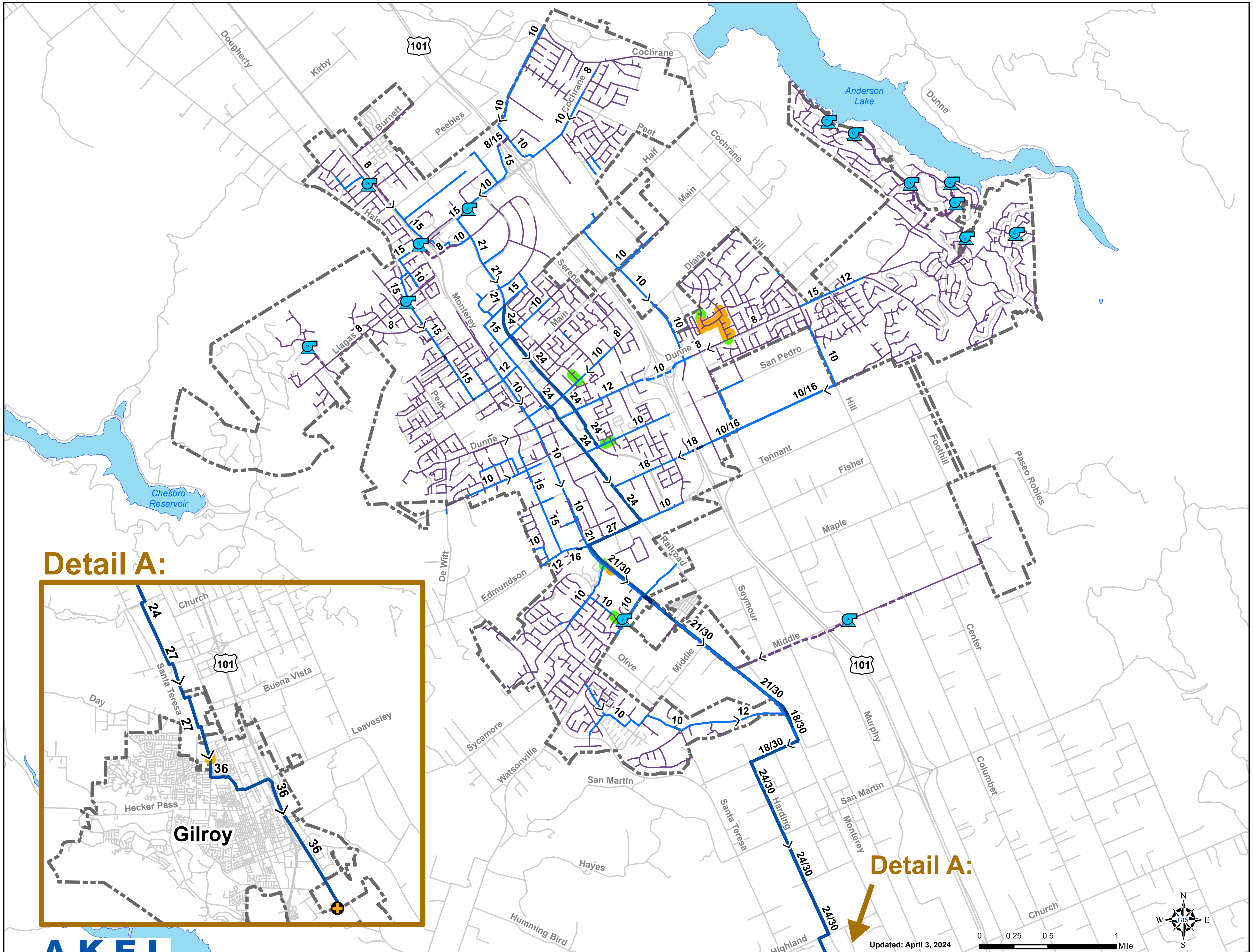
The hydraulic model results for each scenario are detailed in the following sections.

7.1.1 Scenario 1 - Existing PDWF

The capacity evaluation results under existing dry weather conditions are displayed on **Figure 7.1**. The hydraulic model indicates that the existing system is generally adequate to meet the City's design criteria as most pipes are less than 75 percent full. However, a few existing 8-inch diameter pipes tributary to the East Dunne Trunk exhibit a d/D higher than 92 percent.

7.1.2 Scenario 2 - Existing PWWF

The capacity evaluation results under existing wet weather conditions are documented on **Figure 7.2**. The hydraulic model indicates potential capacity deficiencies at several locations with surcharged flow conditions. These deficiencies can be attributed to the high volume of I&I observed in select basins (**Appendix A**). The lift station capacity evaluation is summarized on **Table 7.1** and does not indicate any system deficiencies.



Legend

Gravity Pipe Capacity

- d/D > 0.92 (> 92% Full)
- d/D = 0.75 to 0.92 (75% to 92% Full)

Existing System

- Lift Stations
- SCRWA WWTP

Gravity Pipes by Diamter

- 8" or Smaller
- 10" - 21"
- 24" or Larger

Force Mains / Siphons by Diameter

- 8" or Smaller
- 10" - 12"
- Greater than 14"

Other Features

- Roads
- Lakes
- City Limits

Figure 7.1
Existing PDWF
Capacity Evaluation
Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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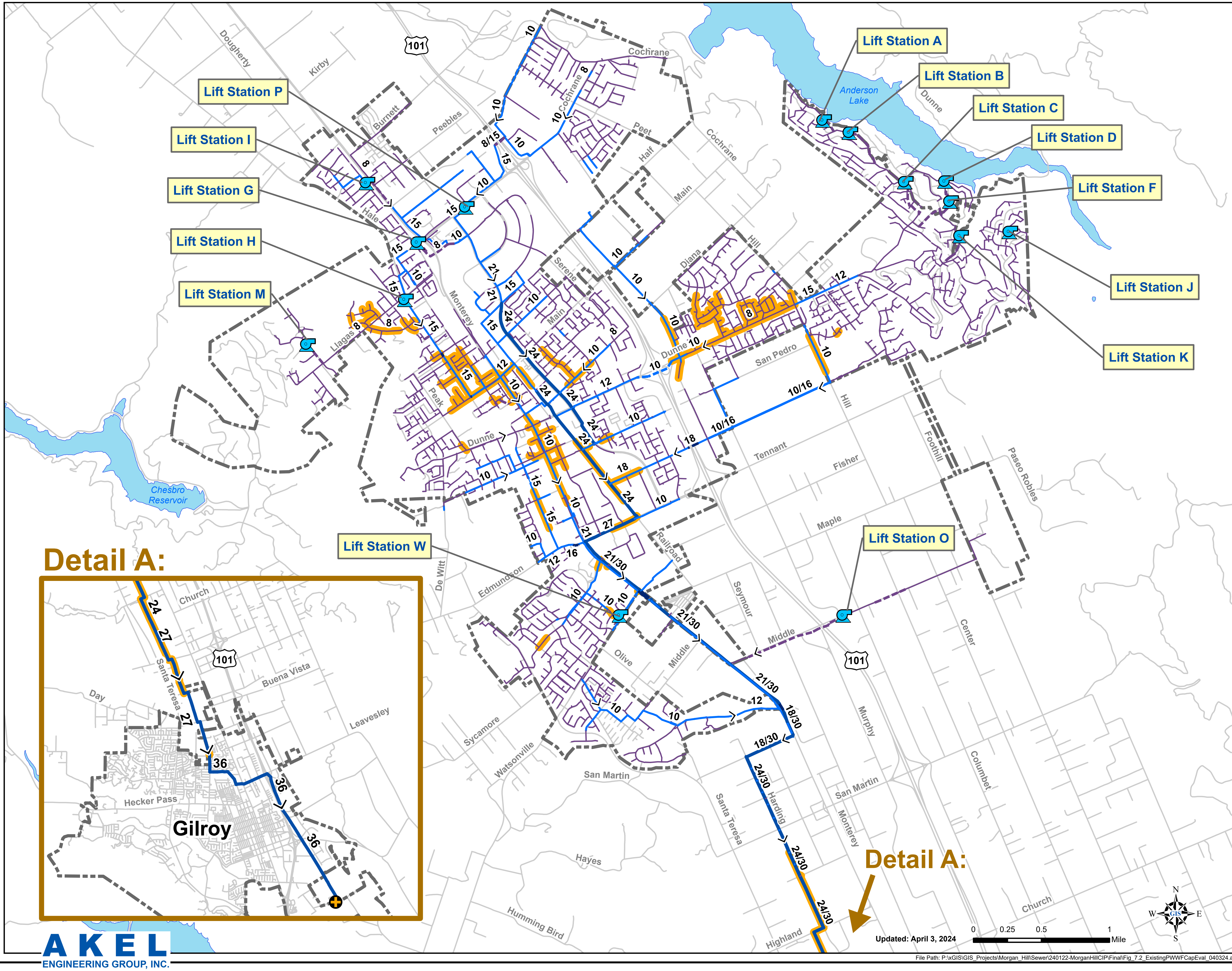

Legend

Gravity Pipe Capacity

d/D = 1 (100% Full)

Existing System

Lift Stations

SCRWA WWTP

Gravity Pipes by Diamter

8" or Smaller

10" - 21"

24" or Larger

Force Mains / Siphons by Diameter

8" or Smaller

10" - 12"

Greater than 14"

Other Features

Roads

Lakes

City Limits

Figure 7.2
Existing PWWF
Capacity Evaluation
Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

Table 7.1 Lift Station Capacity Evaluation

Wastewater Collection System Master Plan Update
City of Morgan Hill

Lift Station Information [1]			Existing Conditions [2]		Future Conditions [2,3]	
Name	Total Capacity (mgd)	Firm Capacity (mgd)	Peak Wet Weather Flow (mgd)	Surplus / Deficiency (mgd)	Peak Wet Weather Flow (mgd)	Surplus / Deficiency (mgd)
Lift Station A	1.5	0.7	0.1	+ 0.7	0.1	+ 0.7
Lift Station B	1.9	0.9	0.1	+ 0.8	0.1	+ 0.8
Lift Station C	1.5	0.7	0.3	+ 0.4	0.3	+ 0.4
Lift Station D	1.5	0.7	0.1	+ 0.7	0.1	+ 0.7
Lift Station F	1.5	0.7	0.1	+ 0.6	0.1	+ 0.6
Lift Station G	2.3	1.1	0.5	+ 0.6	0.7	+ 0.5
Lift Station H	1.5	0.7	0.3	+ 0.5	0.3	+ 0.5
Lift Station I	1.4	0.7	0.4	+ 0.3	0.5	+ 0.2
Lift Station J	1.6	0.8	0.03	+ 0.8	0.03	+ 0.8
Lift Station K	1.5	0.7	0.01	+ 0.7	0.01	+ 0.7
Lift Station M	1.4	0.7	0.3	+ 0.4	0.3	+ 0.4
Lift Station O	1.5	0.8	0.1	+ 0.7	0.1	+ 0.7
Lift Station P	1.3	0.7	0.02	+ 0.6	0.02	+ 0.6
Lift Station W	1.5	0.7	0.5	+ 0.2	0.7	0.0

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Notes:
1. Lift Station Information was obtained from the City's 2022 Sanitary Sewer System Management Plan.
2. Peak wet weather flows discharging into the lift stations are based on a 10-Year 24-Hour design storm event.
3. Future peak wet weather flows account for the buildout of the City's 2035 General Plan, and include a 20 percent reduction in I&I.

7.1.3 Scenario 3 - Future PDWF

The capacity evaluation results under projected dry weather conditions are shown on **Figure 7.3**. The hydraulic model results are similar to existing dry weather conditions, with potential deficiencies in the East Dunne Trunk tributary basin. A majority of the pipes are generally less than 75 percent full. This scenario includes two imminent hydraulic capacity projects that are currently at the design phase:

- **Project 1 - Relief Trunk.** This project proposes to construct 7 miles of new 36-inch diameter gravity pipes from the intersection of Harding Avenue and Highland Avenue to Highway 101 and Renz Lane in the City of Gilroy. This project includes a diversion structure with slide gates and a remote operating valve to route flows east into the existing Joint Trunk or west into the new Relief Trunk. This hydraulic capacity improvement will alleviate future system deficiencies, provide redundancy for maintenance in the Joint Trunk and operational flexibility for flow routing.

- **Project 2 - Condit Road Diversion.** This project proposes to construct a diversion structure at the intersection of Condit Road and East Dunne Avenue. The purpose of this project is to route future development flows south along Condit Road via 0.6 miles of new 12-inch diameter gravity pipes that connect into an existing 16-inch diameter pipe at the intersection of Condit Road and Barrett Avenue (Hill-Barrett Trunk). This hydraulic capacity improvement will alleviate current capacity issues in the existing 10-inch pipe that flows east under Highway 101 (East Dunne Trunk).

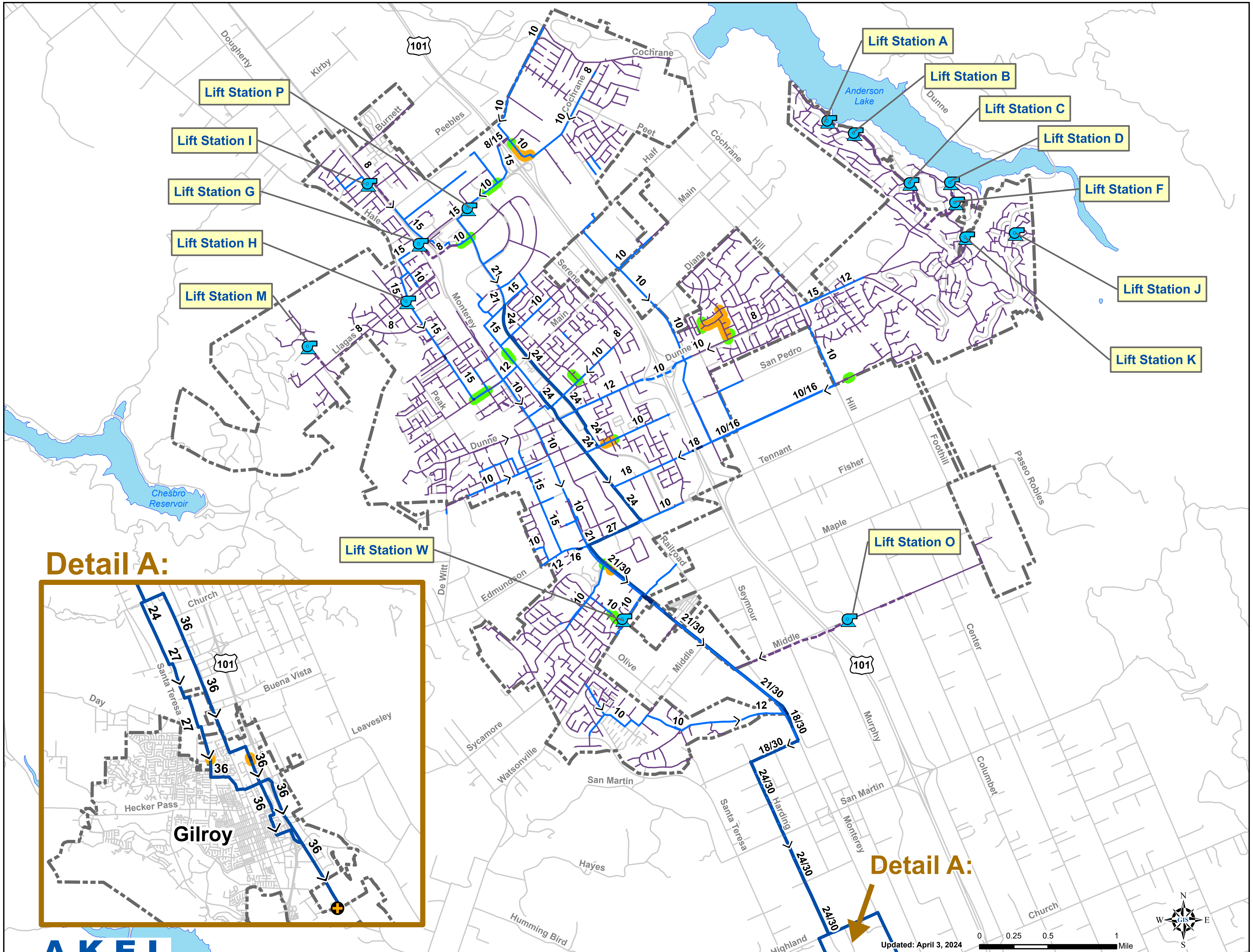
7.1.4 Scenario 4 - Future PWWF

The capacity evaluation results under projected wet weather conditions are documented on **Figure 7.4**. When compared to existing PWWF conditions, the future results indicate more surcharged flow conditions that can be attributed to future developments. It should be noted that this scenario includes two imminent hydraulic capacity projects (Relief Trunk and Condit Road Diversion), and a 20 percent reduction in the volume of I&I that enters the collection system.

Similar to existing PWWF conditions, the lift station capacity evaluation is summarized on **Table 7.1** and does not indicate any system deficiencies. However, it should be noted that Lift Station W located along Watsonville Road will reach its firm capacity at buildout.

7.2 HYDRAULIC CAPACITY IMPROVEMENTS

The hydraulic capacity improvements needed to mitigate existing and future system deficiencies are illustrated on **Figure 7.5**. These improvements were discussed with City staff and prioritized in the following order to accommodate growth envisioned in the City's 2035 General Plan.



Legend

Gravity Pipe Capacity

- d/D > 0.92 (> 92% Full)
- d/D = 0.75 to 0.92 (75% to 92% Full)

Wastewater System

- Lift Stations
- SCRWA WWTP

Gravity Pipes by Diamter

- 8" or Smaller
- 10" - 21"
- 24" or Larger

Force Mains / Siphons by Diameter

- 8" or Smaller
- 10" - 12"
- Greater than 14"

Other Features

- Roads
- Lakes
- City Limits

Figure 7.3
Future PDWF
Capacity Evaluation
Wastewater Collection System
Master Plan Update
City of Morgan Hill

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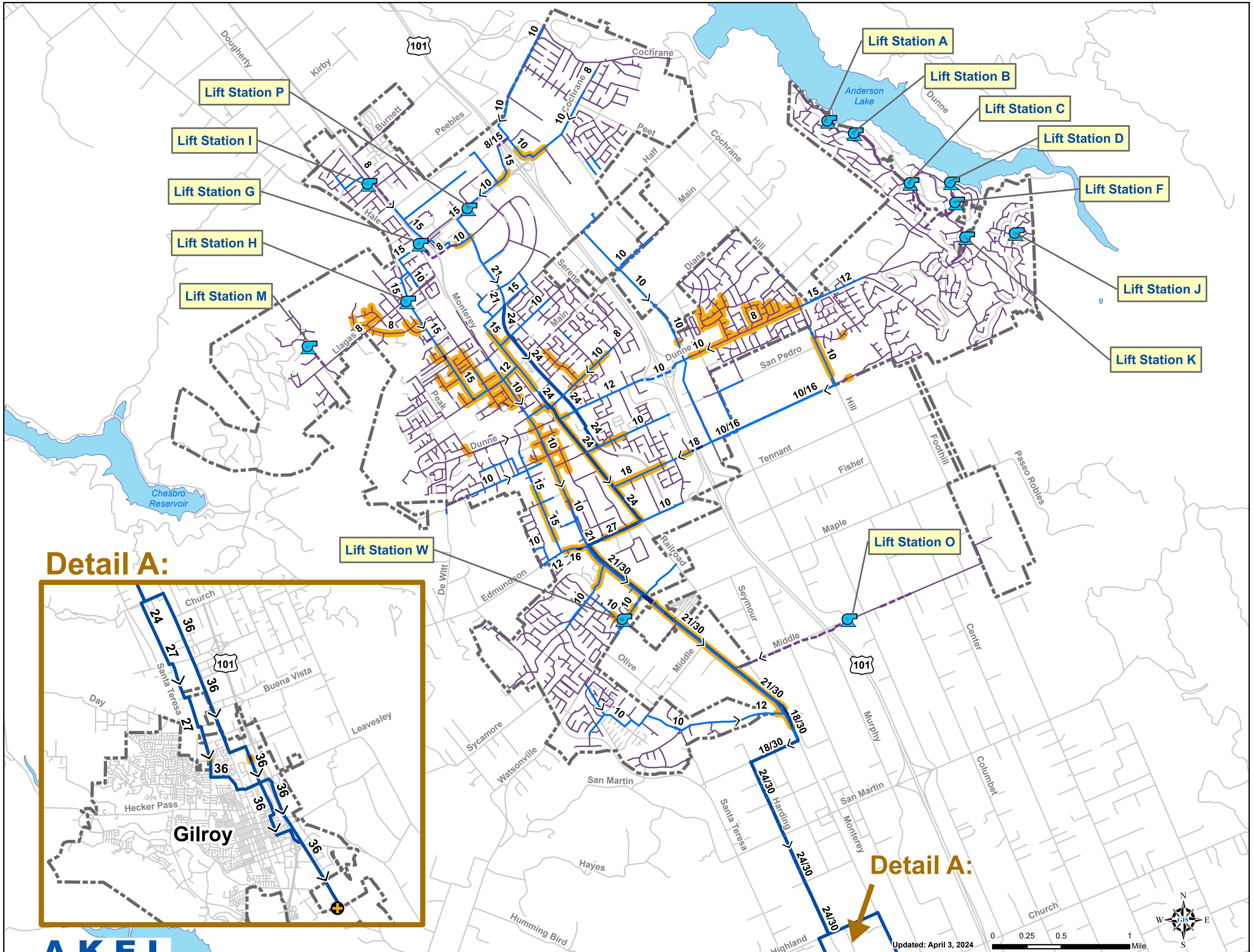

Legend

Gravity Pipe Capacity

- d/D = 1 (100% Full)

Wastewater System

- Lift Stations
- SCRWA WWTP

Gravity Pipes by Diamter

- 8" or Smaller
- 10" - 21"
- 24" or Larger

Force Mains / Siphons by Diameter

- 8" or Smaller
- 10" - 12"
- Greater than 14"

Other Features

- Roads
- Lakes
- City Limits

Figure 7.4
Future PWWF
Capacity Evaluation
Wastewater Collection System
Master Plan Update
City of Morgan Hill

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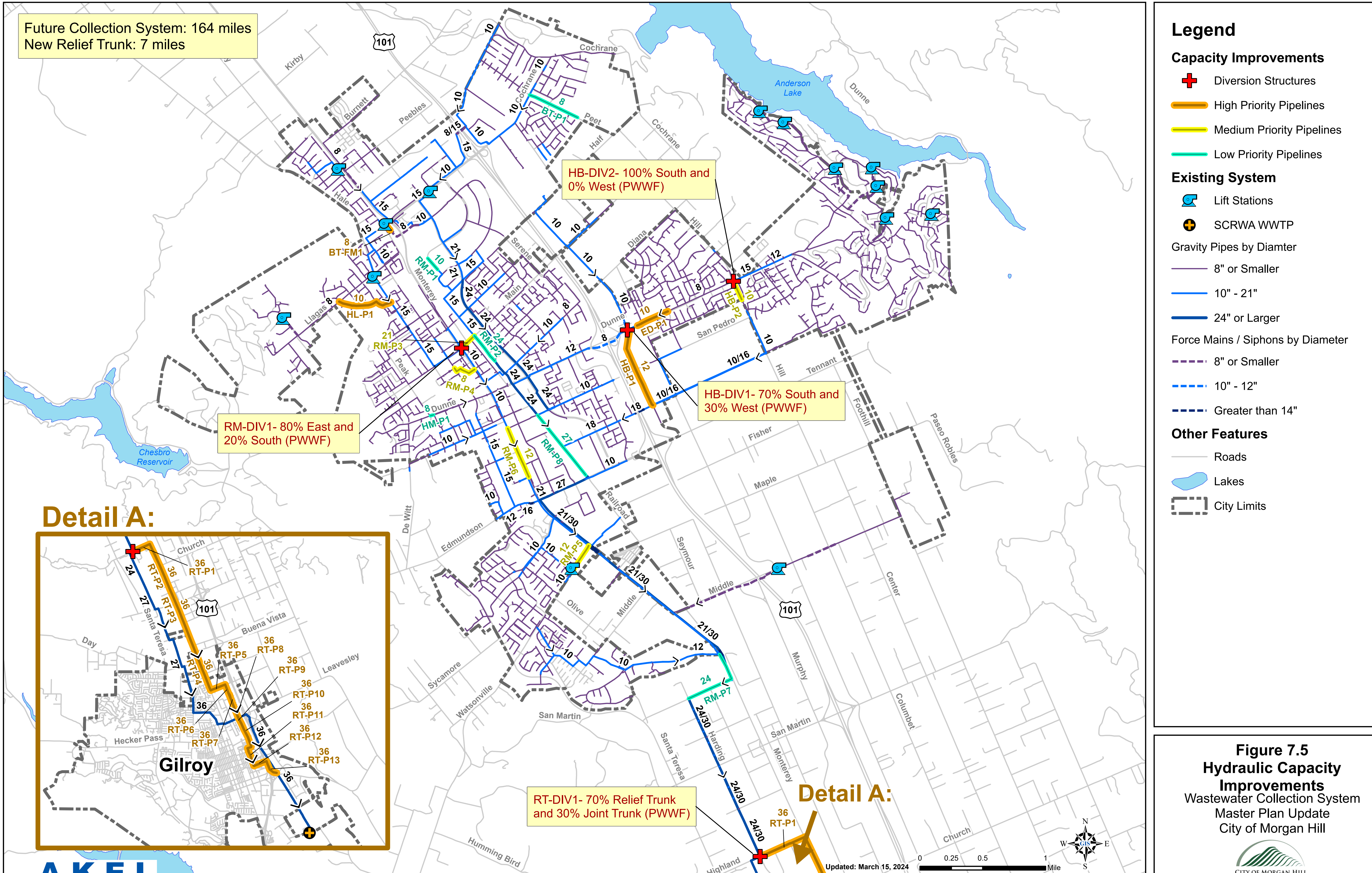

Figure 7.5
Hydraulic Capacity Improvements

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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High Priority (2024 to 2026)

- **Project BT-FM1** (Butterfield Trunk). Replace approximately 350 feet of existing 6-inch diameter force main with a new 8-inch main from Lift Station G to 340 feet west of Monterey Road. This project was triggered due to existing operational issues and will require temporary bypass pumping during the implementation phase.
- **Project HL-P1** (Hale-Llagas Trunk). Replace approximately 2,250 feet of existing 8-inch diameter gravity pipes with new 10-inch pipes along Llagas Creek Drive from Llagas Road to Hale Avenue. This project was identified in the previous master plan and triggered due to an existing deficiency.
- **Project ED-P1** (East Dunne Trunk). Replace approximately 1,600 feet of existing 8-inch diameter gravity pipes with new 10-inch pipes along East Dunne Avenue from Peppertree Drive to 300 feet east of Condit Road. This project was triggered due to an existing deficiency and will require traffic control during the implementation phase.
- **Projects HB-DIV1 and HB-P1** (Condit Road Diversion, Hill-Barrett Trunk).
    - Construct a new diversion manhole at the intersection of Condit Road and East Dunne Avenue to route flows south along Condit Road. An optimal flow split of 70% to the south and 30% to the west is recommended under future PWWF conditions.
    - Construct approximately 3,450 feet of new 12-inch diameter gravity pipes along Condit Road from East Dunne Avenue to Barrett Avenue.

  These projects are required to service future developments north of Condit Road and East Dunne Avenue. Additionally, it should be noted that these projects are currently at the design phase and captured in the future hydraulic model simulations.
- **Projects RT-DIV1 and RT-P1 to RT-P13** (Relief Trunk).
    - Construct a new diversion manhole at the intersection of Harding Avenue and Highland Avenue to optimize the flow split between the existing Joint Trunk and the new Relief Trunk. An optimal flow split of 70% to the east and 30% to the west is recommended under future PWWF conditions.
    - Construct approximately 36,850 feet of new 36-inch diameter gravity pipes along Highland Avenue, Monterey Road, Las Animas Avenue, Murray Avenue, Chestnut Street, East 7th Street and Renz Lane. The construction limits start at the intersection of Harding Avenue and Highland Avenue in the Village of San Martin and end at Highway 1010 and Renz Lane in the City Gilroy. Several segments along the construction area require 24-inch and 12-inch diameter siphons for creek crossings as well as twin 24-inch diameter pipes to resolve utility conflicts/meet vertical clearance requirements.

  These projects are required to facilitate growth envisioned in the City's 2035 General Plan and are currently at the design phase. These projects have been incorporated in the future hydraulic model simulations and were digitized from 90% Design Drawings received from the City in July 2023.

Medium Priority (2027 to 2030)

- **Projects HB-DIV2 and HB-P2** (Hill Road Diversion, Hill-Barrett Trunk).
  - Retrofit an existing diversion manhole at the intersection of Hill Road and East Dunne Avenue to divert all flows (100%) south along Hill Road under future PWWF conditions.
  - Replace approximately 850 feet of existing 8-inch diameter gravity pipes with 10-inch pipes along Hill Road from East Dunne Avenue to Sundance Drive.
- **Projects RM-DIV1 and RM-P3** (Main Avenue Diversion, Railroad-Monterey Trunk).
  - Retrofit an existing diversion manhole at the intersection of Main Avenue and Monterey Road to divert majority of the flows east along Main Avenue. An optimal flow split of 80% to the east and 20% to the south is recommended under future PWWF conditions.
  - Replace approximately 750 feet of existing 15-inch and 12-inch diameter gravity pipes with 21-inch pipes along Main Avenue from Monterey Road to Mason Lane.
- **Project RM-P4** (Railroad-Monterey Trunk). Replace approximately 1,050 feet of existing 6-inch diameter gravity pipes with 8-inch pipes along West 2nd Street and West 3rd Street from Del Monte Avenue to Monterey Road.
- **Project RM-P5** (Railroad-Monterey Trunk). Replace approximately 1,200 feet of existing 10-inch diameter gravity pipes with 12-inch pipes along Watsonville Road from 400 feet west of Calle Sueno to Monterey Road.
- **Project RM-P6** (Railroad-Monterey Trunk). Replace approximately 2,250 feet of existing 10-inch diameter gravity pipes with 12-inch pipes along Monterey Road from San Pedro Avenue to Edges Street.

Low Priority (2031 – 2035)

- **Project HM-P1** (Hale-Monterey Trunk). Replace approximately 150 feet of an existing 6-inch diameter gravity pipe with a 8-inch pipe along West Dunne Avenue from Peak Avenue to 150 feet east of Evergreen Drive.
- **Project RM-P1** (Railroad-Monterey Trunk). Construct approximately 650 feet of new 10-inch diameter gravity pipes 635 feet north of Digital Drive. This project will be triggered by future industrial growth (business park) north of Digital Drive.
- **Project RM-P2** (Railroad-Monterey Trunk). Replace approximately 1,400 feet of existing 15-inch diameter gravity pipes with 24-inch pipes along Mason Lane from Main Avenue to 150 feet north of East 4th Street.

Low Priority (Beyond 2035)

- **Project BT-P1** (Butterfield Trunk). Construct approximately 2,250 feet of new 8-inch diameter gravity pipes along Peet Road from 420 feet east of Avenida De Los Padres to Cochrane Road. This is a long-term low priority project recommended to tie-in a future growth area into the City's existing collection system.
- **Project RM-P7** (Railroad-Monterey Trunk). Replace approximately 3,150 feet of existing 18-inch diameter gravity pipes with 24-inch pipes along Monterey Road and California Avenue, from south of Llagas Creek to Harding Avenue. This is a long-term low priority project recommended to improve system performance.
- **Project RM-P8** (Railroad-Monterey Trunk). Replace approximately 3,250 feet of existing 24-inch diameter gravity pipes with 27-inch pipes along Railroad Avenue from San Pedro Avenue to 100 feet north of Tennant Avenue. This is a long-term low priority project recommended to improve system performance.

The recommended improvements span over 11.6 miles in length and consist of 4 diversion structures to optimize flow routing. It is assumed that any replacement pipes will be in the same alignment and at the same slope as the existing pipe.

Topographic and subsurface utility locate surveys are strongly recommended during the subsequent design phase to confirm existing conditions. Detailed project sheets for each improvement are provided in **Appendix D** to assist with the conceptual design phase.

CHAPTER 8 – REHABILITATION AND REPLACEMENT (R&R) PROGRAM

This chapter documents R&R improvements needed to address aging wastewater infrastructure.

8.1 OVERVIEW

Since the completion of the 2017 Wastewater Collection System Master Plan, the City of Morgan Hill has developed a comprehensive R&R program to repair/replace wastewater infrastructure based on condition and risk analyses.

8.2 R&R IMPROVEMENTS

The City's R&R Program is documented in the following recently completed studies:

- **2021 Sanitary Sewer System Asset Management Plan.** This plan defines appropriate Likelihood of Failure (LoF) and Consequence of Failure (CoF) criteria for wastewater facilities and includes a risk assessment of the collection system. A decision tree is used to recommend prioritized R&R improvements with capital cost estimates.
- **2021 Joint Trunk Pipeline Condition Assessment Report.** This report was completed in coordination with the City of Gilroy and documents condition defects in the existing Joint Trunk system. This report is also used to recommend prioritized rehabilitation improvements with capital cost estimates.

The pipeline and manhole R&R recommendations obtained from aforementioned studies are listed on **Table 8.1** and **Table 8.2**, respectively. The City currently maintains a detailed list of known structural deficiencies and continues to implement R&R improvements on an annual basis. Overall, the R&R program is supported by routine CCTV inspections, lift station condition assessments and annual capital projects that target high risk infrastructure.

It should be noted that the CCTV inspections follow the NASSCO rating system established for pipelines, manholes and laterals. Additionally, the City's Private Sewer Lateral Inspection Ordinance also requires residents to inspect old service laterals and repair deficiencies before sale or re-modeling of properties.

Table 8.1 Pipeline R&R Improvements

Wastewater Collection System Master Plan Update
City of Morgan Hill

R&R Priority	Recommended R&R Action						Total Linear Feet of Improvements
	Upsize - Open Trench (feet)	Upsize - Pipe Burst + Point Repairs (feet)	Replace - Open Trench (feet)	Replace - Pipe Burst + Point Repairs (feet)	Cured-in-Place Pipe Lining + Point Repairs (feet)	Point Repairs (feet)	(feet)
City of Morgan Hill R&R Program [1]							
Priority 1A	979	-	3,096	2,992	1,595	6,897	**15,559**
Priority 1B	239	90	89	124	1,506	3,860	**5,908**
Priority 2A	140	-	7,962	4,889	7,338	25,520	**45,849**
Priority 2B	204	-	332	-	1,030	10,358	**11,924**
Priority 3	-	2,903	517	70	1,649	1,100	**6,239**
Subtotal	**1,562**	**2,993**	**11,996**	**8,075**	**13,118**	**47,735**	**85,479**
Joint Trunk Rehabilitation [2]							
Priority 1					3,111	35	**3,146**
Priority 2					21,699		**21,699**
Subtotal					**24,810**	**35**	**24,845**

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Notes:

1. R&R action recommended in the City's 2021 Sanitary Sewer System Asset Management Plan.
2. Rehabilitation improvements recommended in the 2021 Joint Trunk Pipeline Condition Assessment Report, completed in coordination with the City of Gilroy.

Table 8.2 Manhole R&R Improvements

Wastewater Collection System Master Plan Update
City of Morgan Hill

Recommended R&R Action	No. of Manholes
City of Morgan Hill R&R Program [1]	
Replace Manhole (Priority 1A)	1
Replace Manhole Frames and Covers (Priority Valies)	17
Subtotal	**18**
Joint Trunk Rehabilitation [2]	
Raise Buried Manholes (Priority 2)	8
Line Manholes (Cementitious liner, Priority 2)	32
Subtotal	**40**

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Notes:

1. R&R action recommended in the City's 2021 Sanitary Sewer System Asset Management Plan.
2. Rehabilitation improvements recommended in the 2021 Joint Trunk Pipeline Condition Assessment Report, completed in coordination with the City of Gilroy.

CHAPTER 9 - CAPITAL IMPROVEMENT PROGRAM

This chapter presents a Capital Improvement Program with hydraulic capacity, rehabilitation, and Joint Trunk improvements. This chapter also presents the cost criteria and methodologies for developing capital improvement costs.

9.1 COST ESTIMATE ACCURACY

Cost estimates presented in the Capital Improvement Program were prepared for general master planning purposes and, where relevant, for further project evaluations. The final costs of a project will depend on several factors including the specific project scope of work, costs of labor and material, and market conditions during construction.

AACE International (Association for the Advancement of Cost Engineering, International) has defined five estimate classes for general construction based on the maturity level of the project scope definition. These estimate classes are extracted from the *AACE International Recommended Practice No. 56R-08, Cost Estimate Classification System – As Applied in Engineering, Procurement, and Construction for the Building and General Construction Industries (Rev. August 2020)* and briefly summarized as follows.

Estimate Class	Description	Data Availability and Percent Accuracy
Class 5	This classification is also known as an order of magnitude estimate and is generally intended for long-range capital planning and master plans. This estimate is not supported with detailed engineering data about the specific project, and its accuracy is dependent on historical data and cost indices.	The data is 0% to 2% complete and includes the location and proposed project. It is generally expected that this estimate would be accurate within -30 percent to +50 percent.
Class 4	This classification is also known as a schematic design or feasibility estimate and prepared based on limited information and used for project screening, determination of feasibility, concept evaluation, and preliminary budget approval.	The data is 1% to 15% complete and includes preliminary site plans, utilities, and a design criteria report. It is generally expected that this estimate would be accurate within -20 percent to +30 percent.

Class 3	This classification is also known as a budget or basic engineering phase estimate and is prepared to form the basis for budget authorization, appropriation, and/or funding.	The data is 10% to 40% complete and includes the site civil information. It is generally expected that this estimate would be accurate within -15 percent to +20 percent.
Class 2	This classification is also known as detailed or design development estimate and generally prepared to form a detailed contractor control baseline and used as a bid estimate to establish contract value.	The data is 30% to 70% complete and includes the complete design information. It is generally expected that this estimate would be accurate within -10 percent to +15 percent.
Class 1	This classification is also known as a final or pre-construction estimate and is prepared for discrete parts of the project and used by subcontractors for bids, or by owners for check estimates.	The data is 70% to 100% complete and includes the engineering and design documentation for the project and complete execution and commissioning plans. It is generally expected that this estimate would be accurate within -5 percent to +10 percent.

Costs developed in this study should be considered "Class 5" and have an expected accuracy range of **-30 percent** and **+50 percent**.

9.2 COST ESTIMATE METHODOLOGY

Cost estimates presented in this chapter are an opinion of probable construction cost developed from several sources including cost curves and local experience on other master planning projects. Where appropriate, costs were escalated to reflect the more current ENR (formerly Engineering News-Record) Construction Cost Index (CCI). Dating from the early 20th Century, ENR CCI is a cost estimating tool used by engineers to gauge the current cost for new construction.

This section documents the unit costs, the CCI, and markups to account for construction contingency and other project related costs.

9.2.1 Unit Costs

The unit cost estimates used to develop the Capital Improvement Program are summarized on Table 9.1. Gravity pipe and force main unit costs are based on length of pipe per chosen diameter. The infrastructure security cost was provided by City staff and covers general security equipment such as CCTV cameras and site fencing.

Table 9.1 Infrastructure Unit Costs

Wastewater Collection System Master Plan Update
City of Morgan Hill

Pipelines [1]	
Pipe Size (in)	**Cost** ($ / Linear Foot)
8	288
10	360
12	432
15	463
18	543
21	633
24	723
27	814
30	904
36	1,085
Force Mains [1]	
Pipe Size (in)	**Cost** ($ / Linear Foot)
8	174
10	217
Miscellaneous Improvements	
Diversion Manhole with Slide Gates / Weir [1]	$ 35,000 / Each
Infrastructure Security [2]	$ 500,000 / Every 5 Years
Master Plan Contingencies [3]	
Unknown Field Conditions	30%
Project Related Costs	30%

AKEL ENGINEERING GROUP, INC.

4/23/2024

Notes :

1. Unit Costs were based on typical industry trends and adjusted using a 20-city average ENR CCI of 13,532 from April 2024
2. Costs estimated based on discussions with the City staff.
3. Master plan contingencies established from typical industry trends.

These unit costs are intended for developing the Order of Magnitude estimate, and do not account for site specific conditions, labor, or material costs during the time of construction.

9.2.2 Construction Cost Index

Costs estimated in this study are escalated from the ENR CCI, which is widely used in the engineering and construction industries. The Capital Improvement Program costs were benchmarked using a 20-City Average ENR CCI of 13,532, reflecting a date of April 2024.

9.2.3 Construction Contingency Allowance

Knowledge about site-specific conditions for each proposed project is limited at the master planning stage; therefore, construction contingencies were used. The estimated construction costs in this master plan include a **30 percent** contingency allowance to account for unforeseen events and unknown field conditions, as indicated on **Table 9.1**.

9.2.4 Project Related Costs

The costs also account for project-related expenses such as engineering design, project administration (developer and City staff), construction management, and legal fees. The project related costs in this master plan were estimated by applying an additional **30 percent** to the estimated construction costs, as indicated on **Table 9.1**.

9.3 CAPITAL IMPROVEMENT PROGRAM

The Capital Improvement Program for the 2035 horizon consists of 11.6 miles of hydraulic capacity improvements (**Figure 7.5**) and 15.7 miles of rehabilitation improvements. The detailed program is summarized on **Table 9.2** with a uniquely coded project ID, description, implementation schedule, construction trigger and cost information. Detailed project sheets for hydraulic capacity improvements are included in **Appendix D** for subsequent design phases.

In total, the program implementation costs are estimated at approximately $138.1 million dollars, with $80.2 million attributed to existing customers and $57.9 million dollars attributed to future customers. A suggested 10-year expenditure budget is also provided on **Table 9.3**.

Table 9.2 Capital Improvement Program

Wastewater Collection System Master Plan Update

City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Customers ($)	Future Customers ($)
A. Hydraulic Capacity Improvements										Includes +30% Contingency	Includes +30% Contingency			1 EDU = 180 gpd [6]				
Butterfield Trunk															Cost Based on Proportional Average Wastewater Flows			
BT-P1	Gravity Pipe	Peet Rd	From approx. 420 ft e/o Avenida De Los Padres to Cochrane Rd	New	-	8	2,250	288	$648,000	$843,000	$1,096,000	Low	Beyond 2035	326 EDUs	0%	100%	$0	$1,096,000
BT-FM1	Force Main	Cochrane Road / Lift Station G	From Lift Station G to approx. 340 ft w/o Monterey Rd	Replace	6	8	350	174	$61,000	$80,000	$104,000	High	Imminent	Under Design	60%	40%	$62,400	$41,600
BT-FM1	Temporary Diversion	Cochrane Road / Lift Station G	Temporary pumped diversion to bypass Lift Station G during project BT-FM1	-	-	-	-	110,000	$110,000	$143,000	$186,000	High	Imminent	Under Design	60%	40%	$111,600	$74,400
						Butterfield Trunk Subtotal	**2,600**		**$819,000**	**$1,066,000**	**$1,386,000**						**$174,000**	**$1,212,000**
Hale-Llagas Trunk																		
HL-P1	Gravity Pipe	Llagas Creek Dr	From Llagas Rd to Hale Ave	Replace	8	10	2,250	360	$810,000	$1,053,000	$1,369,000	High	2024 - 2026	Existing Deficiency	80%	20%	$1,095,200	$273,800
						Hale-Llagas Trunk Subtotal	**2,250**		**$810,000**	**$1,053,000**	**$1,369,000**						**$1,095,200**	**$273,800**
East Dunne Trunk																		
ED-P1	Gravity Pipe	East Dunne Ave	From Peppertree Dr to 300 ft e/o of Condit Rd	Replace	8	10	1,600	360	$576,000	$749,000	$974,000	High	2024 - 2026	Existing Deficiency	100%	0%	$974,000	$0
ED-P1	Gravity Pipe	East Dunne Ave	Traffic Control Costs	-	-	-	-	50,000	$50,000	$65,000	$85,000	High	2024 - 2026	Existing Deficiency	100%	0%	$85,000	$0
						East Dunne Trunk Subtotal	**1,600**		**$626,000**	**$814,000**	**$1,059,000**						**$1,059,000**	**$0**
Hill-Barrett Trunk																		
HB-DIV1	Diversion Manhole	East Dunne Ave / Condit Rd	Route Flows 70% South along Condit Rd and 30% West along Dunne Ave	Replace	-	-	-	Condit Rd Diversion Project Cost Provided by the City			$60,000	High	Imminent	Under Design	25%	75%	$15,000	$45,000
HB-P1	Gravity Pipe	Condit Rd	From E Dunne Ave to Barrett Ave	New	-	12	3,450	Condit Rd Diversion Project Cost Provided by the City			$2,140,000	High	Imminent	Under Design	25%	75%	$535,000	$1,605,000
HB-DIV2	Diversion Manhole	East Dunne Ave / Hill Rd	Route Flows 100% South along Hill Rd	Replace	-	-	-	35,000	$35,000	$46,000	$60,000	Medium	2027 - 2030	103 EDUs	90%	10%	$54,000	$6,000
HB-P2	Gravity Pipe	Hill Rd	From East Dunne Ave to Sundance Dr	Replace	8	10	850	360	$306,000	$398,000	$518,000	Medium	2027 - 2030	103 EDUs	90%	10%	$466,200	$51,800
						Hill-Barrett Trunk Subtotal	**4,300**		**$341,000**	**$444,000**	**$2,778,000**						**$1,070,200**	**$1,707,800**
Hale-Monterey Trunk																		
HM-P1	Gravity Pipe	Peak Ave / W Dunne Ave	From Peak Ave to 150 ft e/o Evergreen Dr	Replace	6	8	150	288	$44,000	$58,000	$76,000	Low	2031 - 2035	112 EDUs	80%	20%	$60,800	$15,200
						Hale-Monterey Trunk Subtotal	**150**		**$44,000**	**$58,000**	**$76,000**						**$60,800**	**$15,200**
Railroad-Monterey Trunk																		
RM-P1	Gravity Pipe	North of Digital Dr	From 635 ft n/o Digital Drive to Digital Dr	New	-	10	650	360	$234,000	$305,000	$397,000	Low	2031 - 2035	114 EDUs	0%	100%	$0	$397,000
RM-P2	Gravity Pipe	Mason Ln	From East Main Ave to 150 ft n/o East 4th St	Replace	15	24	1,400	723	$1,013,000	$1,317,000	$1,713,000	Low	2031 - 2035	2264 EDUs	65%	35%	$1,113,450	$599,550
RM-DIV1	Diversion Manhole	East Main Ave / Monterey Rd	Route Flows 80% East along Main Ave and 20% South along Monterey Rd	Replace	-	-	-	35,000	$35,000	$46,000	$60,000	Medium	2027 - 2030	518 EDUs	80%	20%	$48,000	$12,000
RM-P3	Gravity Pipe	East Main Ave	From Monterey Rd to Mason Ln	Replace	15 and 12	21	750	633	$475,000	$618,000	$804,000	Medium	2027 - 2030	518 EDUs	80%	20%	$643,200	$160,800
RM-P4	Gravity Pipe and Siphon	West 2nd St / West 3rd St	From Del Monte Ave to Monterey Rd	Replace	6	8	1,050	288	$303,000	$394,000	$513,000	Medium	2027 - 2030	78 EDUs	40%	60%	$205,200	$307,800
RM-P5	Gravity Pipe and Siphon	Watsonville Rd	From 400 ft w/o Calle Sueno to Monterey Rd	Replace	10	12	1,200	432	$519,000	$675,000	$878,000	Medium	2027 - 2030	130 EDUs	90%	10%	$790,200	$87,800
RM-P6	Gravity Pipe	Monterey Rd	From San Pedro Ave to Edes St	Replace	10	12	2,250	432	$972,000	$1,264,000	$1,644,000	Medium	2027 - 2030	269 EDUs	50%	50%	$822,000	$822,000
RM-P7	Gravity Pipe	Monterey Rd / California Ave	From South of Llagas Creek to Harding Ave	Replace	18	24	3,150	723	$2,278,000	$2,962,000	$3,851,000	Low	Beyond 2035	4172 EDUs	70%	30%	$2,695,700	$1,155,300
RM-P8	Gravity Pipe	Railroad Ave	From San Pedro Ave to 100 ft n/o Tennant Ave	Replace	24	27	3,250	814	$2,646,000	$3,440,000	$4,472,000	Low	Beyond 2035	4922 EDUs	70%	30%	$3,130,400	$1,341,600
						Railroad-Monterey Trunk Subtotal	**13,700**		**$8,475,000**	**$11,021,000**	**$14,332,000**						**$9,448,150**	**$4,883,850**

Table 9.2 Capital Improvement Program

Wastewater Collection System Master Plan Update
City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Customers ($)	Future Customers ($)
Relief Trunk - Currently under Design															**25% Attributed to Existing Users and 75% to Future Users**			
RT-DIV1	Diversion Manhole	Highland Ave / Harding Ave	Route Flows 70% East into the Relief Trunk and 30% West into the existing Joint Trunk	New	-	-	-	35,000	$35,000	$46,000	$60,000	High	Imminent	Under Design	-	-	$15,000	$45,000
RT-P1	Gravity Pipe	Highland Ave	From Harding Ave to Monterey Rd	New	-	36	2,100	1,085	$2,279,000	$2,963,000	$3,852,000	High	Imminent	Under Design	-	-	$963,000	$2,889,000
RT-P2	Gravity Pipe	Monterey Rd	From Highland Ave to Masten Ave	New	-	36	7,550	1,085	$8,192,000	$10,650,000	$13,845,000	High	Imminent	Under Design	-	-	$3,461,250	$10,383,750
RT-P3	Gravity Pipe	Monterey Rd	From Masten Ave to Buena Vista Ave	New	-	36	5,650	1,085	$6,131,000	$7,971,000	$10,363,000	High	Imminent	Under Design	-	-	$2,590,750	$7,772,250
RT-P4	Gravity Pipe	Monterey Rd	From Buena Vista Ave to Las Animas Ave	New	-	36	4,650	1,085	$5,046,000	$6,560,000	$8,528,000	High	Imminent	Under Design	-	-	$2,132,000	$6,396,000
RT-P5	Gravity Pipe	Las Animas Ave	From Monterey Rd to Murray Ave	New	-	36	1,750	1,085	$1,899,000	$2,469,000	$3,210,000	High	Imminent	Under Design	-	-	$802,500	$2,407,500
RT-P6	Gravity Pipe	Murray Ave	From Las Animas Ave to 550 ft n/o of Kishimura Dr	New	-	36	1,100	1,085	$1,194,000	$1,553,000	$2,019,000	High	Imminent	Under Design	-	-	$504,750	$1,514,250
RT-P7	Siphon	Murray Ave	From 550 ft n/o of Kishimura Dr to Kishimura Dr	New	-	Twin 24 and 12 Siphon	1,700	723 and 432	$1,065,000	$1,385,000	$1,801,000	High	Imminent	Under Design	-	-	$450,250	$1,350,750
RT-P8	Gravity Pipe	Murray Ave	From Kishimura Dr to Leavesley Rd	New	-	36	2,200	1,085	$2,387,000	$3,104,000	$4,036,000	High	Imminent	Under Design	-	-	$1,009,000	$3,027,000
RT-P9	Gravity Pipe	Murray Ave	From 150 ft n/o Leavesley Rd to 150 ft s/o Leavesley Rd	New	-	Twin 24	600	723	$434,000	$565,000	$735,000	High	Imminent	Under Design	-	-	$183,750	$551,250
RT-P10	Gravity Pipe	Murray Ave	From 150 ft s/o Leavesley Rd to Chestnut St	New	-	36	3,550	1,085	$3,852,000	$5,008,000	$6,511,000	High	Imminent	Under Design	-	-	$1,627,750	$4,883,250
RT-P11	Gravity Pipe	Chestnut St	From Murray Ave to E 7th St	New	-	36	2,600	1,085	$2,821,000	$3,668,000	$4,769,000	High	Imminent	Under Design	-	-	$1,192,250	$3,576,750
RT-P12	Gravity Pipe	E 7th St	From Chestnut St to Renz Ln	New	-	36	1,500	1,085	$1,628,000	$2,117,000	$2,753,000	High	Imminent	Under Design	-	-	$688,250	$2,064,750
RT-P13	Gravity Pipe	Renz Ln	From E 7th St to Tie-in 250 ft n/o of Hwy 152	New	-	36	1,900	1,085	$2,062,000	$2,681,000	$3,486,000	High	Imminent	Under Design	-	-	$871,500	$2,614,500
						Relief Trunk Subtotal	**36,850**		**$39,025,000**	**$50,740,000**	**$65,968,000**						**$16,492,000**	**$49,476,000**
						A. Hydraulic Capacity Subtotal	**61,450**		**$50,140,000**	**$65,196,000**	**$86,968,000**						**$29,399,350**	**$57,568,650**

B. Rehabilitation and Miscellaneous Improvements

Project ID	Annual Pipeline and Manhole Rehabilitation Plan [7]	New/ Replace/ Repair	Existing Diameter	Total Length	Length w/o Capacity Projects [8]	Total Capital Cost Excluding Capacity Projects [9]	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows	Future Flows	Existing Customers	Future Customers
										Rehabilitation Attributed to Existing Users			
RR-2025	Complete Approx. 40% of Priority 1A Rehab Projects	Repair / Replace	Varies	6,154	5,975	$2,500,000	1A	2025	Defects	100%	0%	$2,500,000	$0
RR-2026	Complete Approx. 40% of Priority 1A Rehab Projects		Varies	6,154	5,975	$2,500,000	1A	2026	Defects	100%	0%	$2,500,000	$0
RR-2027	Complete Approx. 20% of Priority 1A, 100% of Priority 1B and 3% of Priority 2A Rehab Projects		Varies	10,689	9,881	$2,500,000	1A, 1B & 2A	2027	Defects	100%	0%	$2,500,000	$0
RR-2028	Complete Approx. 19% of Priority 2A Rehab Projects		Varies	8,935	8,745	$2,500,000	2A	2028	Defects	100%	0%	$2,500,000	$0
RR-2029	Complete Approx. 19% of Priority 2A Rehab Projects		Varies	8,935	8,745	$2,500,000	2A	2029	Defects	100%	0%	$2,500,000	$0
RR-2030	Complete Approx. 19% of Priority 2A Rehab Projects		Varies	8,935	8,745	$2,500,000	2A	2030	Defects	100%	0%	$2,500,000	$0
RR-2031	Complete Approx. 19% of Priority 2A Rehab Projects		Varies	8,935	8,745	$2,500,000	2A	2031	Defects	100%	0%	$2,500,000	$0
RR-2032	Complete Approx. 21% of Priority 2A and 14% of Priority 2B Rehab Projects		Varies	10,187	9,987	$2,500,000	2A & 2B	2032	Defects	100%	0%	$2,500,000	$0
RR-2033	Complete Approx. 86% of Priority 2B and 44% of Priority 3 Rehab Projects		Varies	13,081	12,871	$2,500,000	2B & 3	2033	Defects	100%	0%	$2,500,000	$0
RR-2034	Complete Approx. 56% of Priority 3 Rehab Projects		Varies	3,473	3,344	$2,334,000	3	2034	Defects	100%	0%	$2,334,000	$0
	Pipeline and Manhole 10-Year Rehabilitation Plan Subtotal				**83,012**	**$24,834,000**						**$24,834,000**	**$0**

Project ID	Lift Station Rehabilitation Plan [10]	Force Main Diameter	No. of Pumps	Total Capacity	Firm Capacity	Total Capital Cost	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows	Future Flows	Existing Customers	Future Customers
RR-LS1	Lift Station F (Wet well, pumps, electrical, control panels and slab) [11]	4	2	1012 gpm	506 gpm	$1,328,700	1A	Imminent	Varies	100%	0%	$1,328,700	$0
RR-LS2	Lift Station K (Wet well, pumps, electrical and slab) [11]	4	2	1030 gpm	515 gpm	$959,300	1A	Imminent	Varies	100%	0%	$959,300	$0

Table 9.2 Capital Improvement Program
Wastewater Collection System Master Plan Update
City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Customers ($)	Future Customers ($)
Lift Station Rehabilitation Plan [10]				Force Main Diameter	No. of Pumps	Total Capacity	Firm Capacity				Total Capital Cost							
RR-LS3	Lift Station D (Wet well, electrical, control panels, slab, roof, walls and lighting)			4	2	1012 gpm	506 gpm				$1,506,000	1B	2024 - 2027	Varies	100%	0%	$1,506,000	$0
RR-LS4	Lift Station D (Land Acquisition Cost for Site Access) [11]			Future Land Acquisition Costs for Site Access							$1,600,000	3	2032	Varies	100%	0%	$1,600,000	$0
RR-LS5	Lift Station A (Wet well, electrical, control panels, roof, lighting and generator)			4	2	1012 gpm	506 gpm				$1,506,000	1B	2027 - 2030	Varies	100%	0%	$1,506,000	$0
RR-LS6	Lift Station P (Wet well, roof, fencing and lighting)			4	2	916 gpm	458 gpm				$1,506,000	1B	2027 - 2030	Varies	100%	0%	$1,506,000	$0
RR-LS7	Lift Station B (Wet well and lighting)			6	2	1310 gpm	655 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS8	Lift Station C (Wet well and roof)			6	2	1012 gpm	506 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS9	Lift Station G (Wet well Only)			6 and 8	2	1568 gpm	784 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS10	Lift Station H (Roof only)			4	2	1030 gpm	515 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS11	Lift Station I (Wet well Only)			6	2	988 gpm	494 gpm				$728,000	2A	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS12	Lift Station M (Wet well, roof and lighting)			6	2	968 gpm	484 gpm				$1,506,000	2B	2031 - 2035	Varies	100%	0%	$1,506,000	$0
RR-LS13	Lift Station O (Roof Only)			6	2	1074 gpm	537 gpm				$728,000	2B	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS14	Lift Station J (Rehabilitation completed in 2018)			6	2	1108 gpm	554 gpm				$728,000	3	2031 - 2035	Varies	100%	0%	$728,000	$0
RR-LS15	Lift Station W (Rehabilitation completed in 2018)			6	2	1030 gpm	515 gpm				$728,000	3	2031 - 2035	Varies	100%	0%	$728,000	$0
						Lift Station Rehabilitation Subtotal	**13 Projects**				**$15,736,000**						**$15,736,000**	**$0**
Miscellaneous [10]					Purpose		Start Year				Total Capital Cost							
RR-M1	Wastewater Infrastructure Security Improvements ($500,000 Every 5 Years)			Infrastructure Security Upgrades			2024, 2029 and 2034				$1,500,000	3	2024 - 2035	Security Risk	100%	0%	$1,500,000	$0
RR-M2	Sanitary Sewer System Management Plan Updates ($50,000 Every 5 Years)			To Comply with Discharge Requirements			2027 and 2032				$100,000	3	2027 - 2035	Discharge Requirements	100%	0%	$100,000	$0
RR-M3	Wastewater Collection System Master Plan Updates ($250,000 Every 5 Years)			Identify Infrastructure Needs based on Growth			2029 and 2034				$500,000	3	2029 - 2035	General Plan Update	50%	50%	$250,000	$250,000
RR-M4	Wastewater Rate Study Updates ($50,000 Every 5 Years)			Identiy Funding Needs for Capital Improvements			2029 and 2034				$100,000	3	2029 - 2035	General Plan Update	50%	50%	$50,000	$50,000
						Miscellaneous Subtotal	**4 Projects**				**$2,200,000**						**$1,900,000**	**$300,000**
						B. Rehabilitation / Miscellaneous Subtotal	**83,012**				**$42,770,000**						**$42,531,000**	**$300,000**
C. Existing Joint Trunk Improvements										**Cost Based on 1992 JPA Capacity Allocation [13]**								
Pipeline and Manhole Rehabilitation [12]						No. of Manholes	Total Length		1992 JPA City of Morgan Hill Capacity / Cost Allocation	1992 JPA City of Gilroy Capacity / Cost Allocation	Total Capital Cost for City of Morgan Hill [14]				Rehabilitation Attributed to Existing Users			
RR-J1	Priority 1 Lining Manholes North of Highland Ave			Repair	Varies	6	-		100%	0%	$39,000	1	2027 - 2030	Defects	100%	0%	$39,000	$0
RR-J2	Priority 1 Lining Manholes / Raising Buried Manhole South of Fitzgerald Ave			Repair	Varies	3	-		57%	43%	$12,000	1	2027 - 2030	Defects	100%	0%	$12,000	$0
RR-J3	Priority 1 Lining Manhole North of Day Rd			Repair	Varies	1	-		47%	53%	$3,000	1	2027 - 2030	Defects	100%	0%	$3,000	$0
RR-J4	Priority 1 Pipe and Manhole Lining Between Highland Ave and Fitzgerald Ave			Repair	27	3	533		46%	54%	$136,000	1	2027 - 2030	Defects	100%	0%	$136,000	$0
RR-J5	Priority 1 Pipe and Manhole Lining / Open Cut Repair/ Raising Manholes South of Day Rd			Repair	36	27	2,612		50%	50%	$1,007,000	1	2027 - 2030	Defects	100%	0%	$1,007,000	$0
RR-J6	Priority 2 Pipe Lining Between Highland Ave and Fitzgerald Ave			Repair	36	-	1,072		46%	54%	$326,000	2	2031 - 2035	Defects	100%	0%	$326,000	$0
RR-J7	Priority 2 Pipe Lining / Open Cut Repair South of Day Rd			Repair	36	-	20,627		50%	50%	$6,812,000	2	2031 - 2035	Defects	100%	0%	$6,812,000	$0
						C. Joint Trunk Subtotal	**24,844**				**$8,335,000**						**$8,335,000**	**$0**

Table 9.2 Capital Improvement Program
Wastewater Collection System Master Plan Update
City of Morgan Hill

Improvement Description and Details								Opinion of Probable Construction Cost				Implementation			Capacity and Cost Allocation			
Project ID	Type of Improv.	Main Street	Construction Limits	New/ Replace/ Repair	Existing Diameter (inches)	Proposed Diameter (inches)	Total Length [1] (feet)	Unit Cost [2] ($/ feet)	Baseline Cost [3] ($)	Construction Cost [4] ($)	Total Capital Cost [5] ($)	Project Priority	Phasing Schedule	Construction Trigger	Existing Flows (%)	Future Flows (%)	Existing Customers ($)	Future Customers ($)
Total Capital Cost Estimates															Cost Distribution			
A. Hydraulic Capacity Improvements									$50,140,000	$65,196,000	$86,968,000	High, Medium and Low	2024 - 2035 and Beyond	Varies	34%	66%	$29,399,350	$57,568,650
B. Rehabilitation and Miscellaneous Improvements									-	-	$42,770,000	1A, 1B, 2A, 2B and 3	2024 - 2035	Varies	99%	1%	$42,531,000	$300,000
C. Existing Joint Trunk Improvements									-	-	$8,335,000	1 and 2	2027 - 2035	Defects	100%	0%	$8,335,000	$0
Total Capital Improvement Program Cost											$138,073,000						$80,265,350	$57,868,650

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Notes:

1. Total length was obtained from the hydraulic model and rounded up to the nearest 50 feet.
2. Unit Costs were based on typical industry trends and adjusted using a 20-city average ENR CCI of 13,532 from April 2024
3. Baseline costs were calculated by multiplying the pipeline length by the unit cost and rounding up to the nearest $1,000.
4. Construction costs were calculated by applying a 30% contingency to the baseline costs and rounding up to the nearest $1,000. These costs account for unknown field conditions and site-specific constraints.
5. Total capital costs were calculated by applying a 30% contingency to the construction costs and rounding up to the nearest $1,000. These costs account for project related expenses such as engineering, legal fees, contract administration and construction management.
6. According to the 2023 wastewater flow monitoring program and annual water billing records, a single-family dwelling unit generates an average wastewater flow of approximately 180 gallons per day (gpd).
7. Pipeline, manhole and O&M rehabilitation improvements were sourced from the City's 2021 Sanitary Sewer System Asset Management Plan (SSAMP).
8. Adjusted length and number of manholes were calculated by excluding rehabilitation projects that overlap with hydraulic capacity projects. A list of overlapping capacity and rehabilitation projects are as follows:
   a. Project RR-P1A (Priority 1A) length excludes 451 feet that overlaps with hydraulic capacity projects HL-P1 (310 feet) and RM-P4 (141 feet).
   b. Project RR-P1B (Priority 1B) length excludes 681 feet that overlaps with hydraulic capacity projects HL-P1 (483 feet) and RM-P4 (198 feet).
   c. Project RR-P2A (Priority 2A) length excludes 979 feet that overlaps with hydraulic capacity projects ED-P1 (439 feet) and RM-P6 (540 feet).
   d. Project RR-P2B (Priority 2B) length excludes 124 feet that overlaps with hydraulic capacity project ED-P1.
   e. Project RR-P3 (Priority 3) length excludes 231 feet that overlaps with hydraulic capacity projects HL-P1 (70 feet) and RM-P4 (161 feet).
   f. Project RR-M2 excludes 2 manholes that overlap with hydraulic capacity projects HL-P1 (1 manhole) and RM-P4 (1 manhole).
9. Pipeline and manhole rehabilitation capital cost estimates were obtained from the City's 2021 SSAMP with their compounded contingencies (30% costs and 30% engineering). These costs were also adjusted using a 20-city average ENR CCI of 13,532 from April 2024 and rounded up to the nearest $1,000. It should be noted that the costs are based on the adjusted length/manhole count, which excludes overlapping hydraulic capacity projects (See Note 8).
10. Lift station rehabilitation improvements, capital costs, and implementation schedule was provided by City staff on February 8, 2024.
11. Costs for Lift Stations F and K were provided by City staff on February 8, 2024. Additionally, the City also provided land acquisition costs for Lift Station D.
12. Joint Trunk rehabilitation improvements were obtained from the City's 2021 Joint Trunk Pipeline Condition Assessment Report.
13. Joint Trunk capacity allocations for the City of Gilroy and the City of Morgan Hill were obtained from the 1992 Joint Powers Agreement (JPA). These allocations were used to calculate the proportional capital cost for the City of Morgan Hill.
14. Joint Trunk cost estimates were obtained from the City's 2021 Joint Trunk Pipeline Condition Assessment Report with their total contingencies (45% total for priority 1 projects and 40% total for priority 2 projects). Subsequently, these costs were escalated using a 20-city average ENR CCI of 13,532 from April 2024 and rounded up to the nearest $1,000.

Table 9.3 Suggested 10-Year Expenditure Budget

Wastewater Collection System Master Plan Update

City of Morgan Hill

Phasing Schedule	Hydraulic Capacity Improvements ($)	Rehabilitation / Miscellaneous Improvements ($)	Existing Joint Trunk Improvements ($)	Total Capital Costs ($)
	A	B	C	A+B+C
Imminent / Under Design	$68,458,000	$2,288,000	-	$70,746,000
2024 - 2026	$2,428,000	$7,006,000	-	$9,434,000
2027 - 2030	$4,477,000	$13,862,000	$1,197,000	$19,536,000
2031 - 2035	$2,186,000	$19,614,000	$7,138,000	$28,938,000
Beyond 2035	$9,419,000	-	-	$9,419,000
Total	**$86,968,000**	**$42,770,000**	**$8,335,000**	**$138,073,000**

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APPENDICES

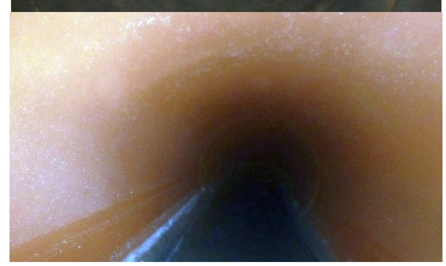
APPENDIX A

2023 Flow Monitoring and
Inflow/Infiltration Study (V&A)

City of Morgan Hill

Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study 2023





Prepared for: Akel Engineering Group, Inc.
7433 N. First Street, Suite 103
Fresno, CA. 93720

Draft Report Date: November 27, 2023

Prepared by: V&A consulting engineers

V&A Project No. 22-0442

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Abbreviations and Acronyms

Abbreviations/Acronyms

ADWF.. Average Dry Weather Flow
CO .... Carbon Monoxide
d .... Depth
D... Diameter
d/D .... Depth/Diameter Ratio
FT. .... Feet
FM .... Flow Monitor
fps .... Feet Per Second
GPD .... Gallons per Day
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 (Private but Publicly Available)
Q... Flow Rate
QAQC .... Quality Assurance Quality Control
RDI .... Rainfall-Dependent Infiltration
RG .... Rain Gauge
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 system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. Also refers to the ground surface area near and enclosed by pipelines. A basin may refer to the entire collection system upstream from a flow meter or exclude separately monitored basins upstream.
Depth/diameter (*d/D*) ratio	Depth of water in a pipe as a fraction of the pipe's diameter. A measure of 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

V&A Consulting Engineers (V&A) was retained by Akel Engineering Group, Inc. (AEG) to perform sanitary sewer flow monitoring and rainfall monitoring with I&I analysis within the City of Morgan Hill (City) collection system. Flow and rainfall monitoring were performed over the period of four weeks from January 9th to February 22nd, 2023. Open-channel flow monitoring was conducted at 10 scoped flow monitoring locations. In addition, data from one permanent flow site (FM 11) was also included in the report for better I&I analysis. The purpose of this work was to provide flow monitoring data with capacity analysis. The objectives of this flow monitoring project are listed below:

1. Establish the baseline sanitary sewer flows at the flow monitoring sites.
2. Establish the peak flow condition during rainfall events and indicate 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.

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 basins are correlated with the flow monitoring site ID's. The flow monitoring sites were selected and approved by AEG and the City. Information regarding the flow monitoring locations is listed in Table ES- 1 and illustrated in Figure ES-1. The site/basin flow schematic is presented in Figure ES- 2. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in Appendix A.

Basin and flow meter isolation calculations were estimated via the provided GIS attribute information. However, there were inconsistencies identified in the attribute information, specifically in the upstream and downstream manhole attributes or flow direction, which could change our initial assumptions. All preliminary flow isolation assumptions should be field verified, specifically at any split flow manhole locations. The split flow configurations and assumptions are discussed in more detail in Section 1.2.

Table ES-1: List of Monitoring Sites

Akel Site ID	V&A Monitoring Site	Manhole No.	Monitored Pipe	Pipe Dia. (in.)	Rim Elevation (ft)	Invert Elevation (ft)*	Location
Site 10	FM 01	J6-C.MH.004	W IN	11 .5	298	NM	Llaga Creek Access, off of Monterey Hwy
Site 1	FM 02	F6-D.MH.074	NE IN	15.5	371	NM	El Dunne Ave and Hill Rd
Site 7	FM 03	I5-A.MH.034	NW IN	21	333	NM	16099 Monterey Hwy
Site 8	FM 04	I5-A.MH.014	SW IN	17.5	332	NM	W Edmundson Ave and Monterey Hwy
Site 5	FM 05	I5-A.MH.008	NE IN	26.5	330	NM	49 Tennant Ave
Site 4	FM 06	G5-C.MH.055	NE IN	12	350	NM	339 E Dunne Ave
Site 2	FM 07	F4-D.MH.006	N IN	19.5	360	NM	18160 Butterfield Blvd
Site 3	FM 08	G4-D.MH.040	SW IN	15	350	NM	Monterey Hwy and W Main Ave
Site 6	FM 09	H5-C.MH.004	NE IN	17.5	338	NM	Railroad Ave and Barrett Ave
Site 9	FM 10	G4-A.MH.017	NW IN	15	354	NM	18052 Hale Ave
Site 11	FM 11[1]	13105	NW IN	29	267	NM	The field behind 12310 Santa Teresa Blvd

*NM = Not Measured nor in GIS data.

[1]Permanent Flow Monitoring Site

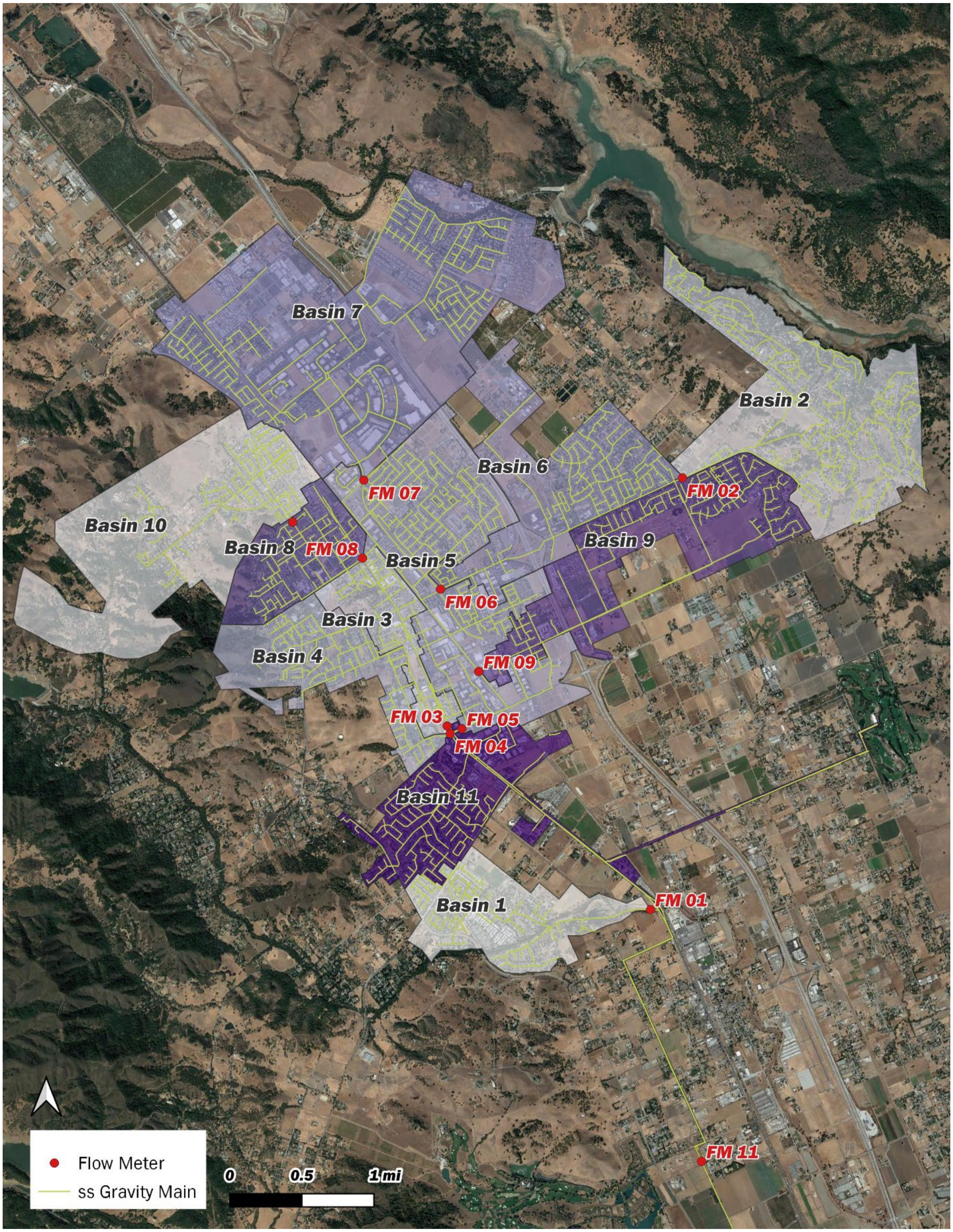

Figure ES-1. Map of Flow Monitoring Sites

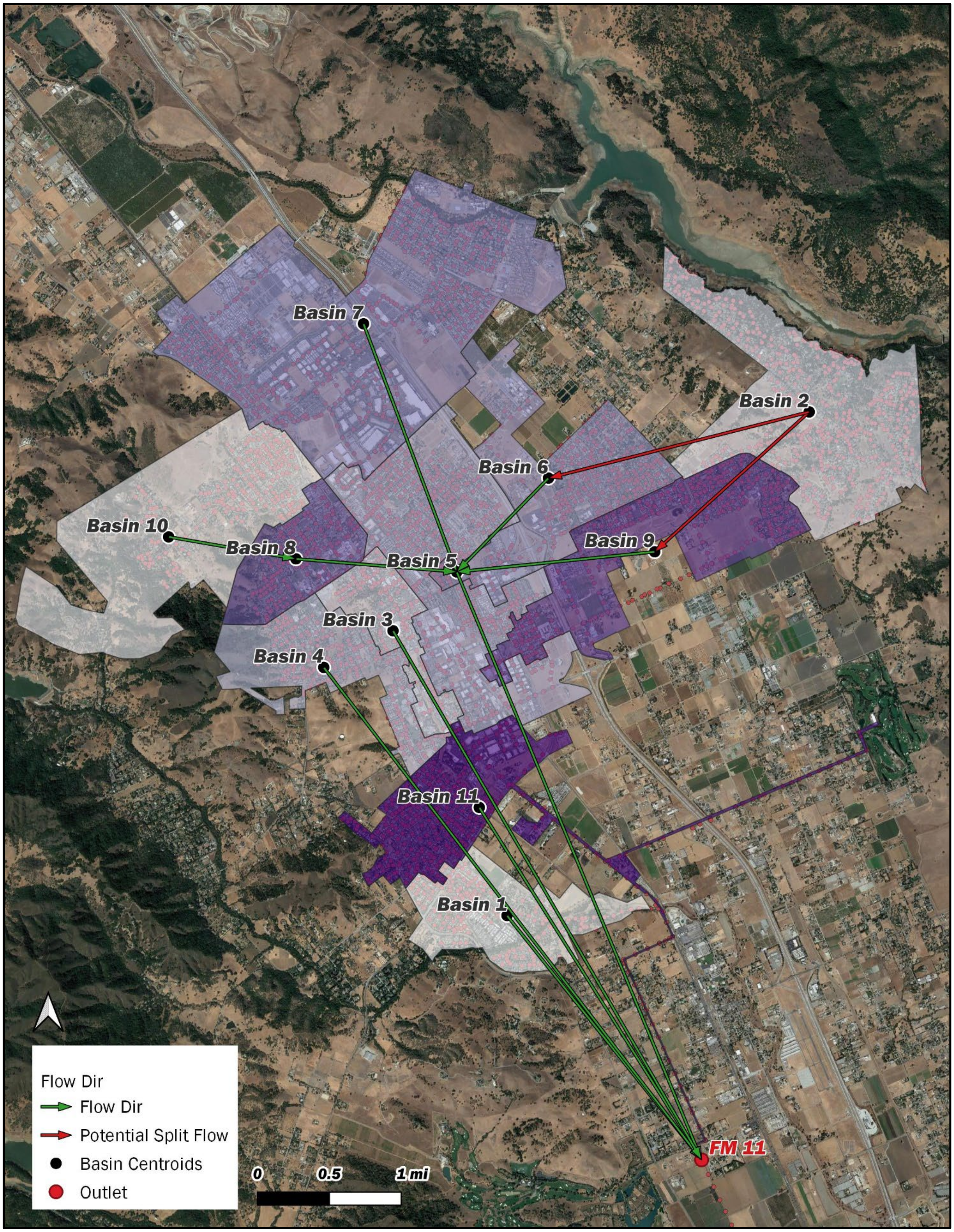

Figure ES-2. Basin Flow Schematic

Rainfall Monitoring

V&A retrieved rainfall data from 20 personal weather stations (PWS) located on Weather Underground (WU)[1]. Ultimately, selecting 8 sites that provided adequate coverage across the study area and quality data for future evaluation of wet-weather I/I responses at the flow monitoring sites. A triangulated average of 8.92 inches of rainfall was recorded over the monitoring period from January 9th, 2023 to February 22nd, 2023. The maximum total rainfall occurred at rain gauge NW with 11.41 inches recorded. The least amount of rainfall occurred at rain gauge NE with 8.01 inches recorded. Rain gauge IDs, installation locations, and total rainfall over the monitoring period are listed in Table ES-2. The 8 rain gauge sites were triangulated to the center of the study area (basins). The highest rainfall intensity measured was 0.62 inches/hour on January 9th, 2023. This event saw 2.16 inches of rainfall over 11 hours and has a return period of ~ 2 years based on the depth of rainfall and event duration. The largest rainfall event occurred on January 13th, 2023 where 4.30 inches of rainfall fell over 69 hours. This was estimated to be less than a 1-year storm event. Figure ES-3 illustrates the location of the 8 rain gauges in relation to the sewer sub-basins. Figure ES-4 shows the rainfall over the monitoring period along with three significant wet-weather events and their responses.

Table ES-2: Summary of Rainfall Data

Rain Gauge ID	Name	Source	X (Longitude)	Y (Latitude)	Total Rainfall (in)
NW	KCAMORGA209	WU	-121.670	37.148	11.41
NE	KCAMORGA250	WU	-121.613	37.159	8.01
W	KCAMORGA201	WU	-121.677	37.128	10.89
CW	KCAMORGA265	WU	-121.643	37.132	8.88
CE	KCAMORGA179	WU	-121.624	37.134	9.48
E	KCAMORGA182	WU	-121.590	37.143	8.66
SW	KCAMORGA213	WU	-121.651	37.115	10.01
S	KCAMORGA129	WU	-121.641	37.099	9.75

Monitored rainfall was plotted against the historical average rainfall. When this historical data is compared to the recorded rainfall, we see that cumulative precipitation was approximately 135% of historical precipitation.

[1] Weather Underground (wunderground.com) collects data from 180,000+ weather stations across the country, including Automated Surface Observation System (ASOS) at airports, (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.

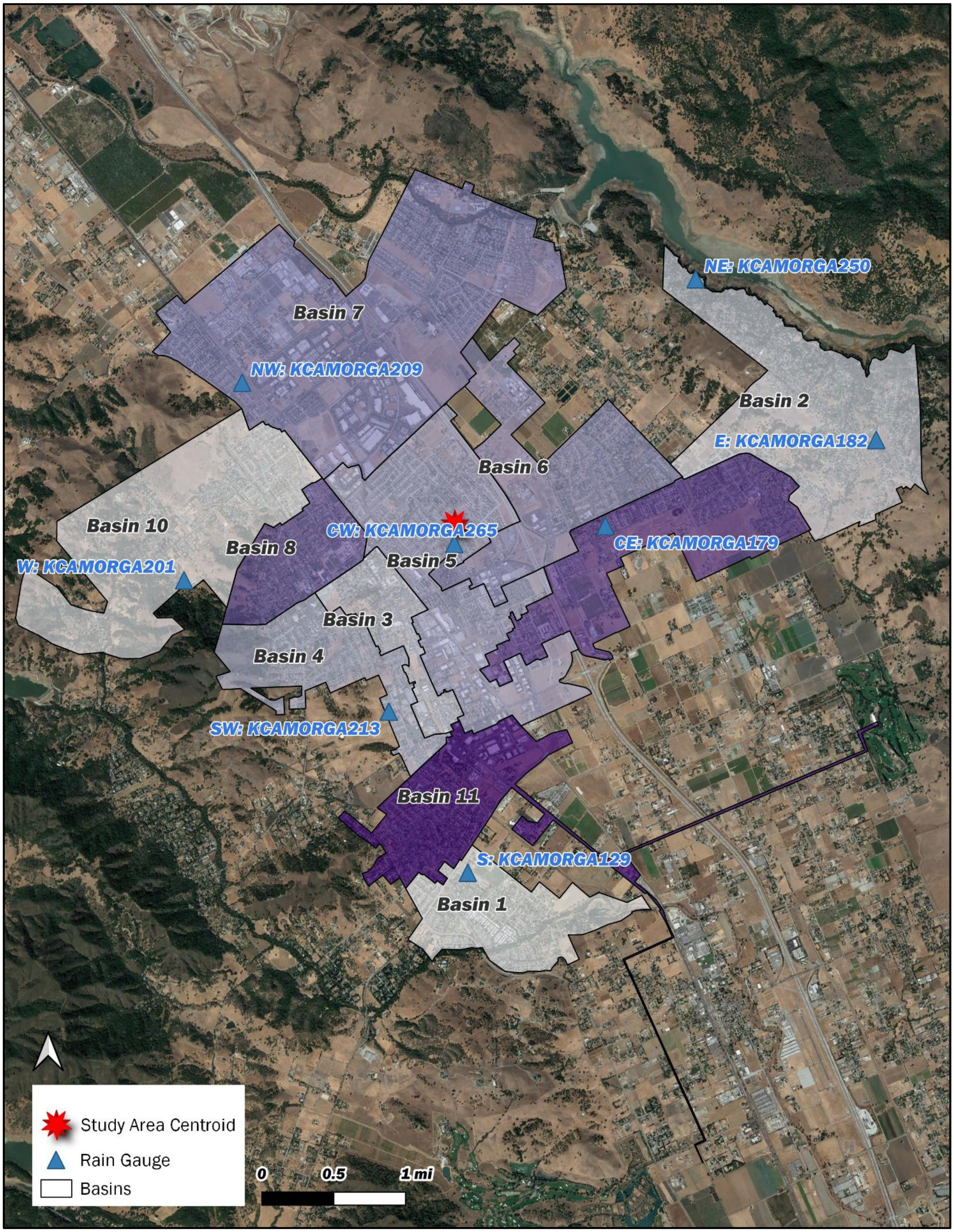

Figure ES-3. Map of Rain Gauge Sites

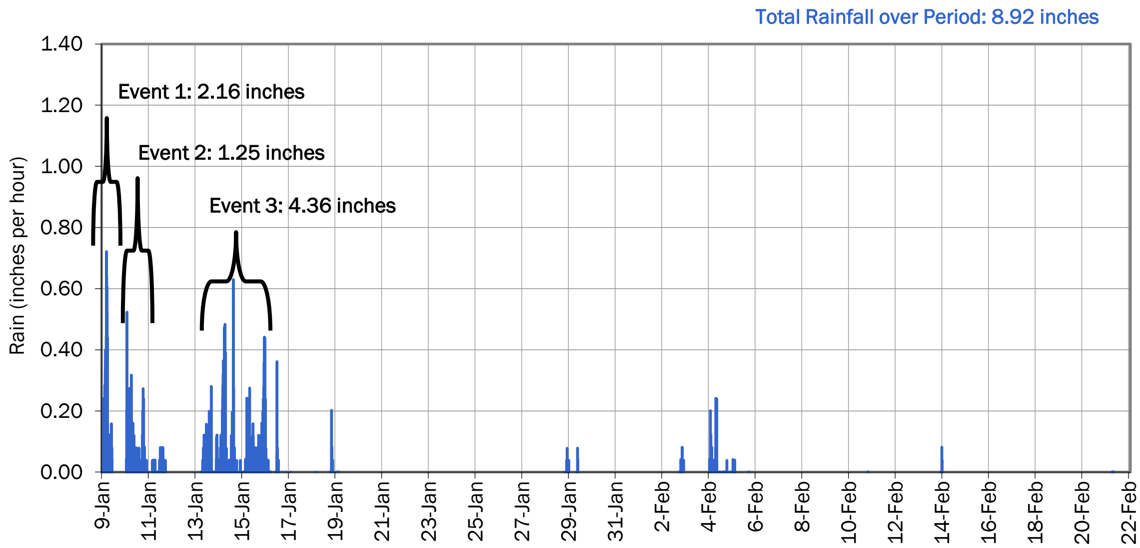

Figure ES-4. Rainfall Monitoring (triangulated to Center of Sub-Basins)

Site Flow Monitoring and Capacity Results

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 as 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:

- **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, PF > 7 is highlighted in RED[2]; 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 the d/D ratio vary from agency to agency but typically range between d/D ≤ 0.5 and d/D ≤ 0.75
  - For this report, d/D ratios > 0.75 are 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 ES-3 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 ES-5 and Figure ES-6 show bar graph summaries of the peaking factors and d/D ratios, respectively. Figure ES-7 shows the schematic diagram of the peak measured flows in each section with peak flow levels.

[2] 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.

The following capacity analysis results are noted:

- Dry weather
  - Site FM 03 had the lowest average d/D ratio of 0.11.
  - Site FM 08 had the highest average d/D ratio of 0.56.
- Peaking Factors
  - Site FM 03 was the only site with a PF's greater than 7:1. This site had a PF of 12:1.
  - The lowest PF was 3:1 at site FM 07.
  - The average site PF was 5:1.
- d/D Ratio:
  - d/D > 0.75: Two sites had d/D ratios greater than 0.75, site FM 08 and FM 09. Site FM 08 was the only site to surcharge.

Table ES- 3: 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 (FT)
FM 01	0.149	0.51	3.4	11.5	6.75	0.59	n/a
FM 02	0.192	0.73	3.8	15.5	6.78	0.44	n/a
FM 03	0.080	0.97	12.1	21	8.29	0.39	n/a
FM 04	0.241	1.36	5.6	17.5	11.51	0.66	n/a
FM 05	1.963	7.32	3.7	26.5	15.77	0.60	n/a
FM 06	0.142	0.85	6.0	12	6.82	0.57	n/a
FM 07	0.393	1.20	3.1	19.5	8.02	0.41	n/a
FM 08	0.429	2.21	5.2	15	29.81	1.99	1.23
FM 09	0.318	1.31	4.1	17.5	16.46	0.94	n/a
FM 10	0.183	0.96	5.2	15	9.24	0.62	n/a
FM 11	2.779	10.65	3.8	29	18.66	0.64	n/a

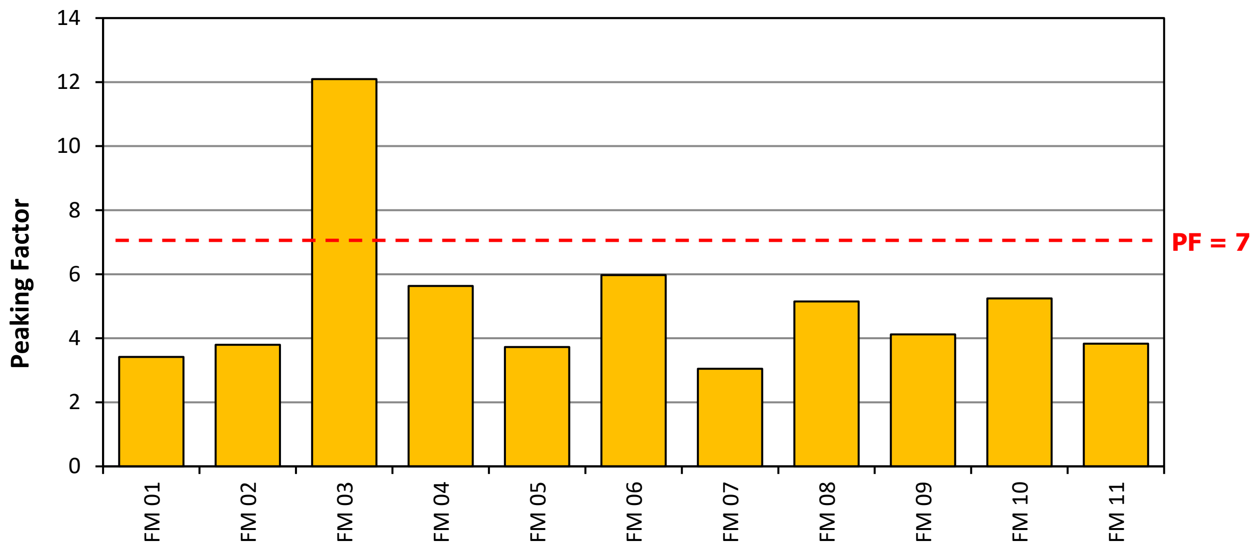

Figure ES-5. Peaking Factors

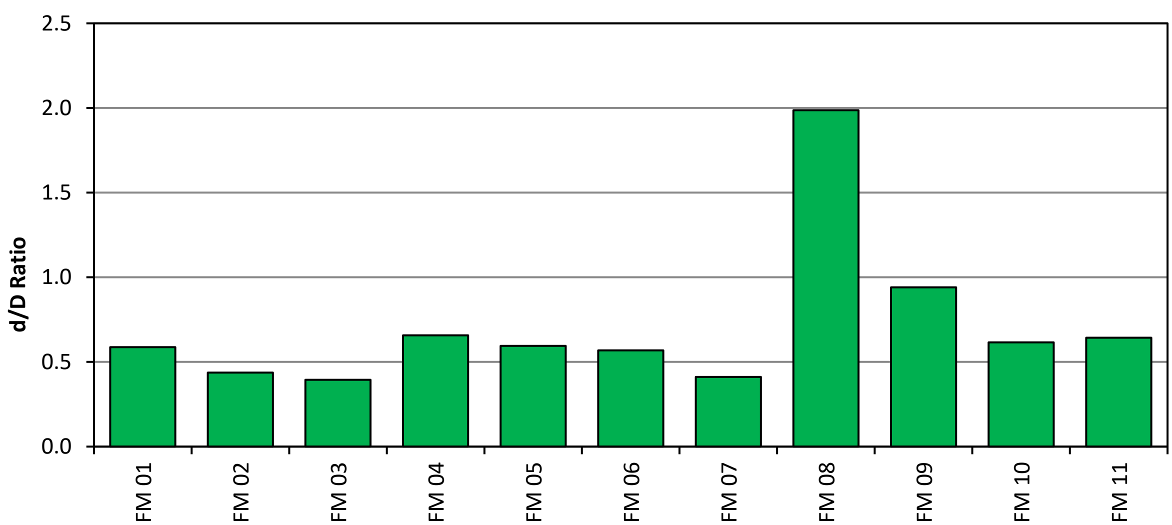

Figure ES-6. Capacity Summary: Max d/D Ratios

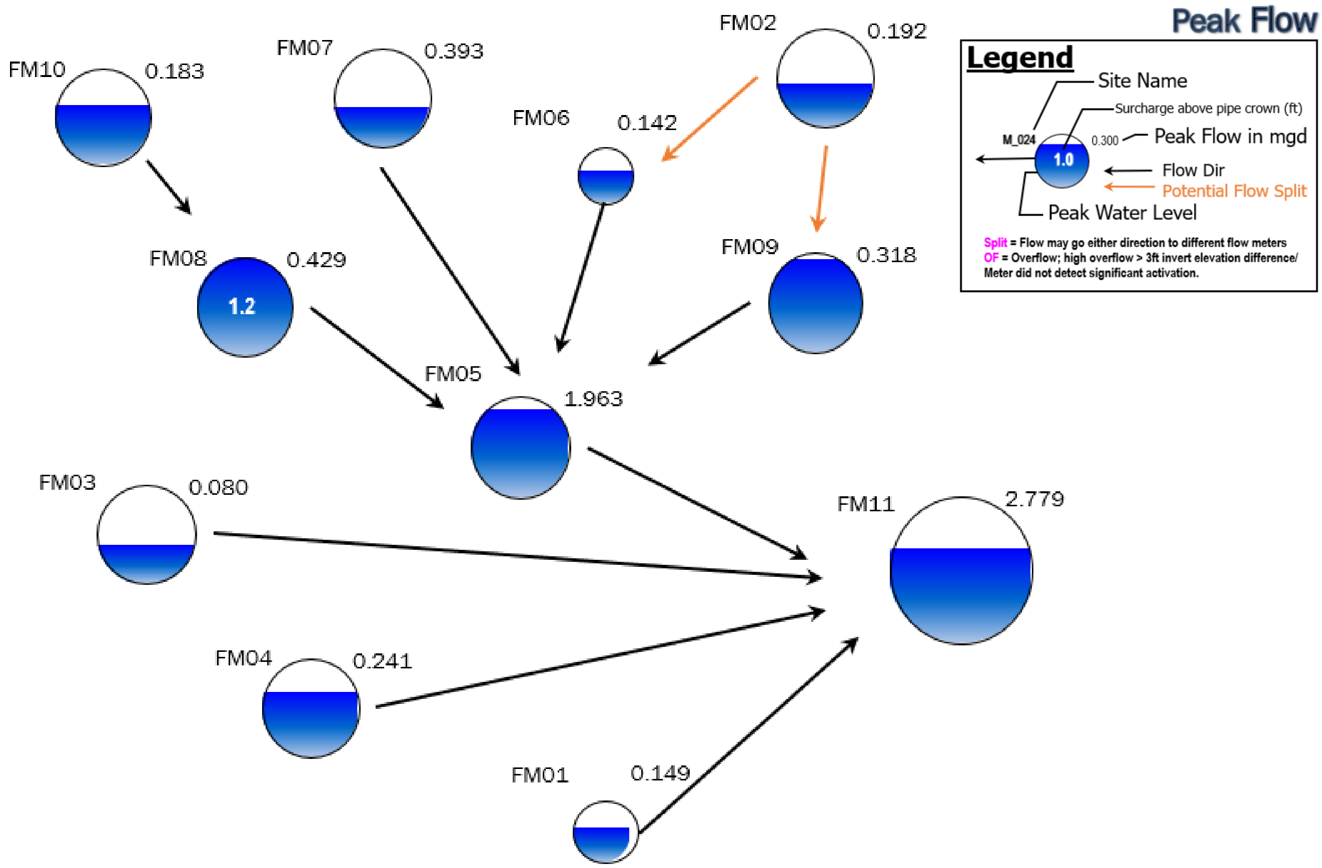

Figure ES-7. Peak Measured Flow (Flow Schematic)

Infiltration and Inflow Analysis

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. I/I analysis in this report will be conducted on a basin-by-basin basis. For this study subtraction of flows was required to isolate the drainage areas of some flow monitoring basins.

Basin and flow meter isolation calculations were estimated via the provided GIS attribute information. However, there were inconsistencies identified in the attribute information, specifically in the upstream and downstream manhole attributes (not populated), which could change initial assumptions. All preliminary flow isolation assumptions should be field verified, specifically at any split flow manhole locations. A key split flow configuration identified in the GIS is:

- Located directly downstream of site FM 02, at manhole F6-D.MH.012, is a split flow configuration where it appears flow from Basin 2 could go to either Basin 6 or 9 or both. In addition, another split flow configuration was identified at the adjacent manhole, F6-D.MH.007. Both of these manholes are located at the intersection of E. Dunne Ave. and Hill Rd.

The flow monitoring basins and basin isolation equations used to define the limits of the basin boundaries are listed in Table ES-4.

Table ES-4. Isolated Flow Monitoring Site / Basin Characteristics

Isolated Site / Basin	Flow Isolation Calculation	Isolated Acres	IDM
FM 01 / 1	$Q_{01} = Q_{01}$	487	66
FM 02 / 2	$Q_{02} = Q_{02}$	1,075	130
FM 03 / 3	$Q_{03} = Q_{03}$	266	69
FM 04 / 4	$Q_{04} = Q_{04}$	479	84
FM 05 / 5	$Q_{05} = Q_{05} - Q_{06} - Q_{07} - Q_{08} - Q_{09}$	1,039	253
FM 06 / 6	$Q_{06} = Q_{06} - Q_{02*}$	733	116
FM 07 / 7	$Q_{07} = Q_{07}$	1,982	244
FM 08 / 8	$Q_{08} = Q_{08} - Q_{10}$	343	65
FM 09 / 9	$Q_{09} = Q_{09} - Q_{02*}$	764	147
FM 10 / 10	$Q_{10} = Q_{10}$	1,150	84
FM 11 / 11	$Q_{11} = Q_{11} - Q_{01} - Q_{03} - Q_{04} - Q_{05}$	595	393

*Spilt flow assumptions presented prior to this table
Q = Flow Rate
IDM = Inch diameter mile

I/I results were taken from rainfall Event 3 and final rankings weighted 1/3 for each flow component (1/3 * Inflow, 1/3 *RDI, and 1/3 * combined). Any tie-breaks were broken by the highest inflow rank. Table ES-5 summarizes the I/I results for this study. The “Top 3” basin for each category has been shaded in RED. Please refer to the I/I Methods section for more information on inflow and infiltration analysis methods and ranking methods. Figure ES-8 shows the average ranking from the 3 criteria evaluated; inflow, RDI, and volume.

Table ES-5. Basin I/I Analysis Summary

Monitoring Basin	Basin Acreage	IDM	Weighted Inflow Rate (mgd)	RDI Rate (mgd)	Combined I/I (gallons)	Inflow Ranking	RDI Ranking	Combined I/I Ranking	Final Rank
01	487	66	0.184	0.099	382,781	10	7	9	10
02	1,075	130	0.503	0.210	950,283	6	6	7	6
03	266	69	0.850	0.167	914,018	1	4	1	1
04	479	84	0.945	0.367	1,625,053	3	1	3	3
05	1,039	253	0.883	0.325	1,332,788	7	8	8	8
06	733	116	0.635	0.057	491,409	4	10	10	9
07	1,982	244	0.717	0.072	505,433	9	11	11	11
08	343	65	0.916	0.282	1,283,624	2	3	2	2
09	764	147	0.304	0.067	1,434,726	8	9	5	7
10	1,150	84	0.672	0.235	1,195,428	5	5	6	4
11	595	393	-0.050	1.045	3,083,719	11	2	4	5

The following inflow/infiltration analysis results are noted:

- Inflow:
  - Basin 03 has the highest inflow per-ADWF, per-Acre, and ranked highest overall.
  - Basin 08 has the highest inflow per-IDM and ranks second highest overall.
  - Basin 04 was ranked the 3rd highest overall.
  - Basin 11 showed a slight loss in peak inflow. This is most likely due flow attenuation, as discussed in Section 2.6.
- Rainfall-Dependent Infiltration:
  - Basin 04 has the highest RDI per-IDM and highest calculated overall ranking.
  - Basin 11 has the highest RDI per-ADWF, RDI per-Acre, and ranked second highest overall.
- Combined I/I:
  - Basin 03 had the highest combined I/I per-ADWF, per-Acre, per-IDM and overall ranking.
  - Basins 08 and 04 ranked 2nd and 3rd respectively for combined I/I.
- Groundwater Infiltration:
  - The following basins had GWI rates higher than typical standards, indicating that 5 of 11 basins have elevated groundwater infiltration. See Section 3.3.5 full analysis. The 5 basins are listed below and noted in Figure 3-26.
    - FM 01, FM 05, FM 08, FM 09, FM 10

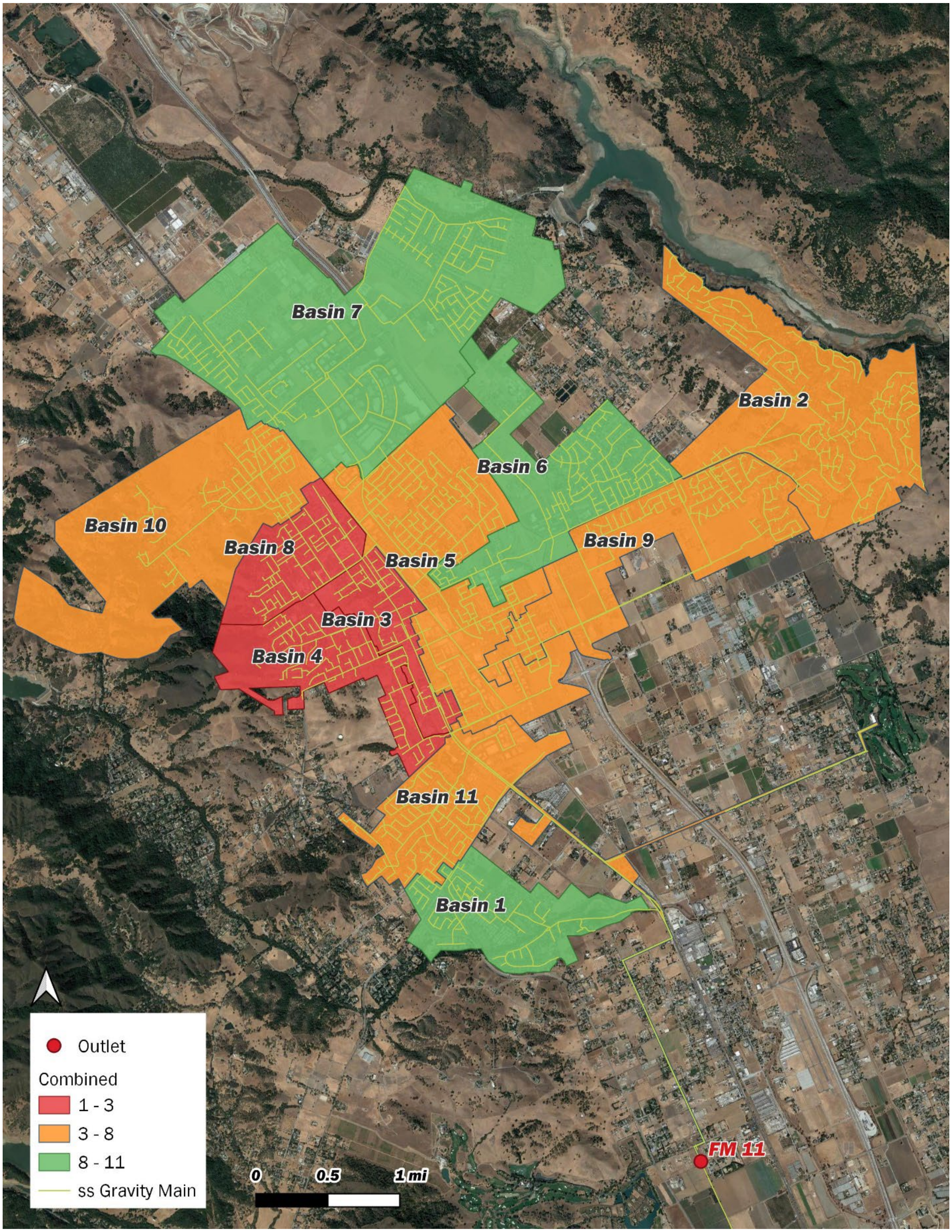

Figure ES-8 illustrates a temperature map summary of the I/I results.

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; however, the capacity deficiencies of the collection system may be of greater concern relative to the I/I response during peak wet weather events. The City may wish to have a model designed and/or a master plan study conducted to determine the overall needs of the City relative to I/I. Or simply: The study results can be used to update the master plan and compare with previous model assumptions and flow monitoring results.
2. **Verify Interconnections and Overflows:** understanding the interconnections and overflows can help with the master plan, basin isolation, and I/I analysis. There are a couple of split flows identified in the GIS at manholes F6-D.MH.012 and F6-D.MH.007 which should be field verified.
3. **Capacity Analysis:** Site FM 03 had a wet-to-dry weather flow peaking factor of over 12:1. In addition, sites FM 08 and FM 09 indicated capacity issues with site FM 09 reaching a d/D of 0.94 and site FM 08 surcharging 1.23-ft above the pipe crown. Additional investigation work is recommended in the collection system upstream of these sites to identify and remediation I/I sources.
4. **Determine I/I Reduction Program**: It is recommended that follow-up investigation work be conducted to identify sources of both I/I.
    a. If peak flows, sanitary sewer overflows and pipeline capacity issues are of greater concern, then priority can be given to investigate and reduce sources of inflow within the basins with the greatest inflow problems. The highest inflow occurs in Basins 03, 04, and 08.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the Basins with the greatest infiltration problems. The highest combined I/I occurs in Basins 03, 04, and 08, and the highest RDI occurs in Basins 04, 08, and 11. In addition, Basins 01, 05, 08, 09, and 10 show evidence of GWI.
5. **I/I Investigation Methods**: Potential I/I investigation methods include the following:
    a. Smoke testing.
    b. Manhole inspections
    c. CCTV
    d. Private building evaluations
    e. Nighttime flow isolation checks for high GWI
6. **I/I Reduction Cost Effective Analysis**: The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow/infiltration and systematically rehabilitating or replacing the faulty pipelines, or (2) continued treatment of the additional rainfall dependent I/I flow.

1 Introduction

1.1 Scope and Purpose

V&A Consulting Engineers (V&A) was retained by Akel Engineering Group, Inc. (AEG) to perform sanitary sewer flow monitoring and rainfall monitoring with I&I analysis within the City of Morgan Hill (City) collection system. Flow and rainfall monitoring were performed over the period of four weeks from January 9th to February 22nd, 2023. Open-channel flow monitoring was conducted at 10 scoped flow monitoring locations. In addition, data from one permanent flow site (FM 11) was also included in the report for better I&I analysis. The purpose of this work was to provide flow monitoring data with capacity analysis. The objectives of this flow monitoring project are listed below:

1. Establish the baseline sanitary sewer flows at the flow monitoring sites.
2. Establish the peak flow condition during rainfall events and indicate 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 is presented on a site-by-site basis. I/I analysis is presented on a basin-by-basin basis. The flow monitoring sites were selected and approved by AEG and the City. Information regarding the flow monitoring locations is listed in Table 1-1 and illustrated in Figure 1-1. Temporary flow monitoring sites are indicated in red. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in Appendix A.

Table 1-1. List of Monitoring Locations

Akel Site ID	Monitoring Site	Manhole No.	Monitored Pipe	Pipe Dia. (in.)	Rim Elevation (ft)	Invert Elevation (ft)*	Location
Site 10	FM 01	J6-C.MH.004	W IN	11 .5	298	NM	Llaga Creek Access, off of Monterey Hwy
Site 1	FM 02	F6-D.MH.074	NE IN	15.5	371	NM	El Dunne Ave and Hill Rd
Site 7	FM 03	I5-A.MH.034	NW IN	21	333	NM	16099 Monterey Hwy
Site 8	FM 04	I5-A.MH.014	SW IN	17.5	332	NM	W Edmundson Ave and Monterey Hwy
Site 5	FM 05	I5-A.MH.008	NE IN	26.5	330	NM	49 Tennant Ave
Site 4	FM 06	G5-C.MH.055	NE IN	12	350	NM	339 E Dunne Ave
Site 2	FM 07	F4-D.MH.006	N IN	19.5	360	NM	18160 Butterfield Blvd
Site 3	FM 08	G4-D.MH.040	SW IN	15	350	NM	Monterey Hwy and W Main Ave
Site 6	FM 09	H5-C.MH.004	NE IN	17.5	338	NM	Railroad Ave and Barrett Ave
Site 9	FM 10	G4-A.MH.017	NW IN	15	354	NM	18052 Hale Ave
Site 11	FM 11[1]	13105	NW IN	29	267	NM	The field behind 12310 Santa Teresa Blvd

*NM = Not Measured nor in GIS data.

[1]Permanent Flow Monitoring Site

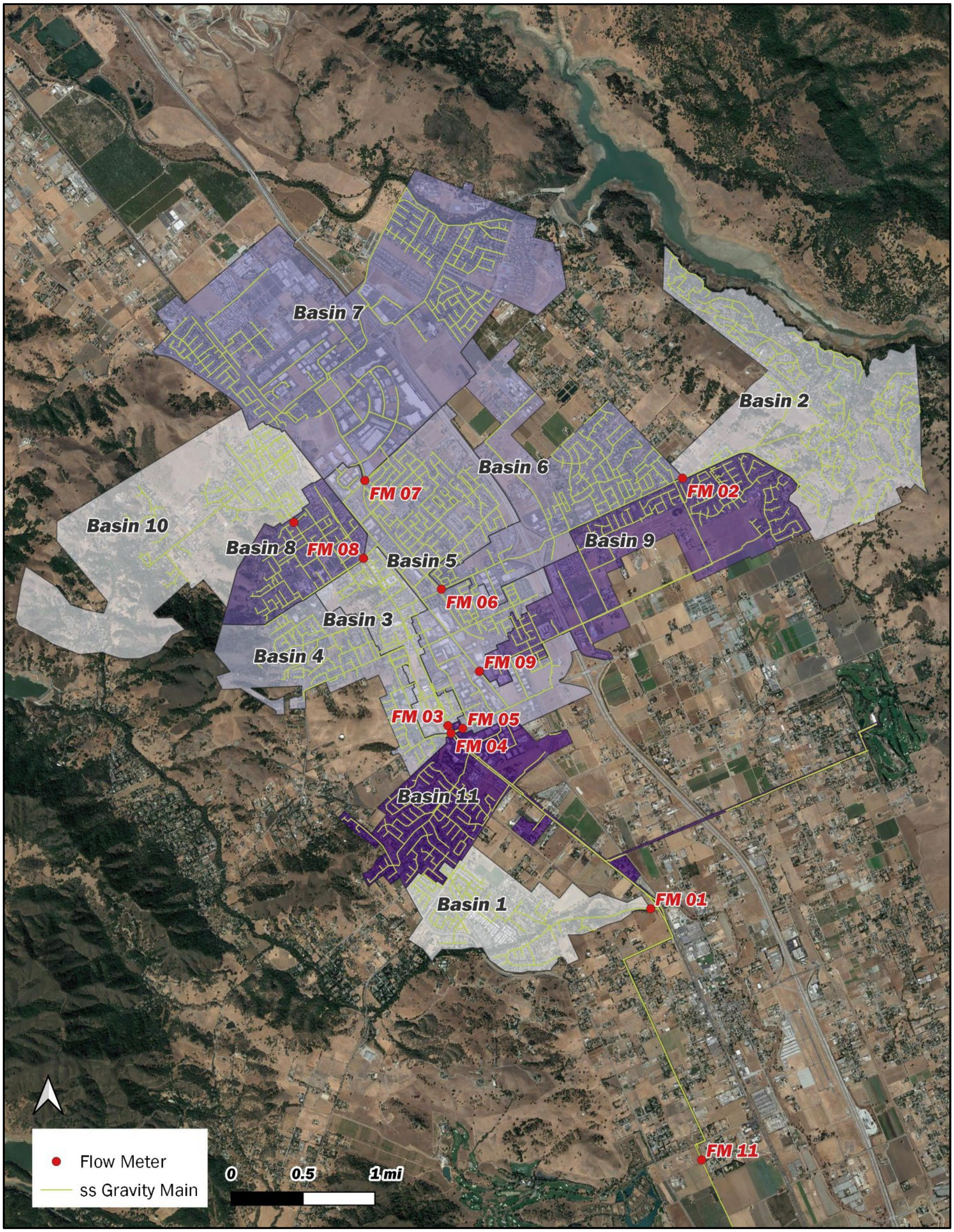

Figure 1-1. Map of Flow Monitoring Sites

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 and shown in Figure 1-2.

Basin and flow meter isolation calculations were estimated via the provided GIS attribute information. However, there were inconsistencies identified in the attribute information, specifically in the upstream and downstream manhole attributes (not populated), which could change initial assumptions. All preliminary flow isolation assumptions should be field verified, specifically at any split flow manhole locations. A key split-flow configuration identified in the GIS is:

- Located directly downstream of site FM 02, at manhole F6-D.MH.012, is a split flow configuration where it appears to flow from Basin 2 and could go to either Basin 6 or 9 or both. In addition, another split-flow configuration was identified at the adjacent manhole, F6-D.MH.007. Both of these manholes are located at the intersection of E. Dunne Ave. and Hill Rd.

Table 1-2. Isolated Flow Monitoring Site / Basin Characteristics

Isolated Site / Basin	Flow Isolation Calculation	Isolated Acres	IDM
FM 01 / 1	$Q_{01} = Q_{01}$	487	66
FM 02 / 2	$Q_{02} = Q_{02}$	1,075	130
FM 03 / 3	$Q_{03} = Q_{03}$	266	69
FM 04 / 4	$Q_{04} = Q_{04}$	479	84
FM 05 / 5	$Q_{05} = Q_{05} - Q_{06} - Q_{07} - Q_{08} - Q_{09}$	1,039	253
FM 06 / 6	$Q_{06} = Q_{06} - Q_{02*}$	733	116
FM 07 / 7	$Q_{07} = Q_{07}$	1,982	244
FM 08 / 8	$Q_{08} = Q_{08} - Q_{10}$	343	65
FM 09 / 9	$Q_{09} = Q_{09} - Q_{02*}$	764	147
FM 10 / 10	$Q_{10} = Q_{10}$	1,150	84
FM 11 / 11	$Q_{11} = Q_{11} - Q_{01} - Q_{03} - Q_{04} - Q_{05}$	595	393

*Spilt flow assumptions presented prior to this table
Q = Flow Rate
IDM = Inch diameter mile

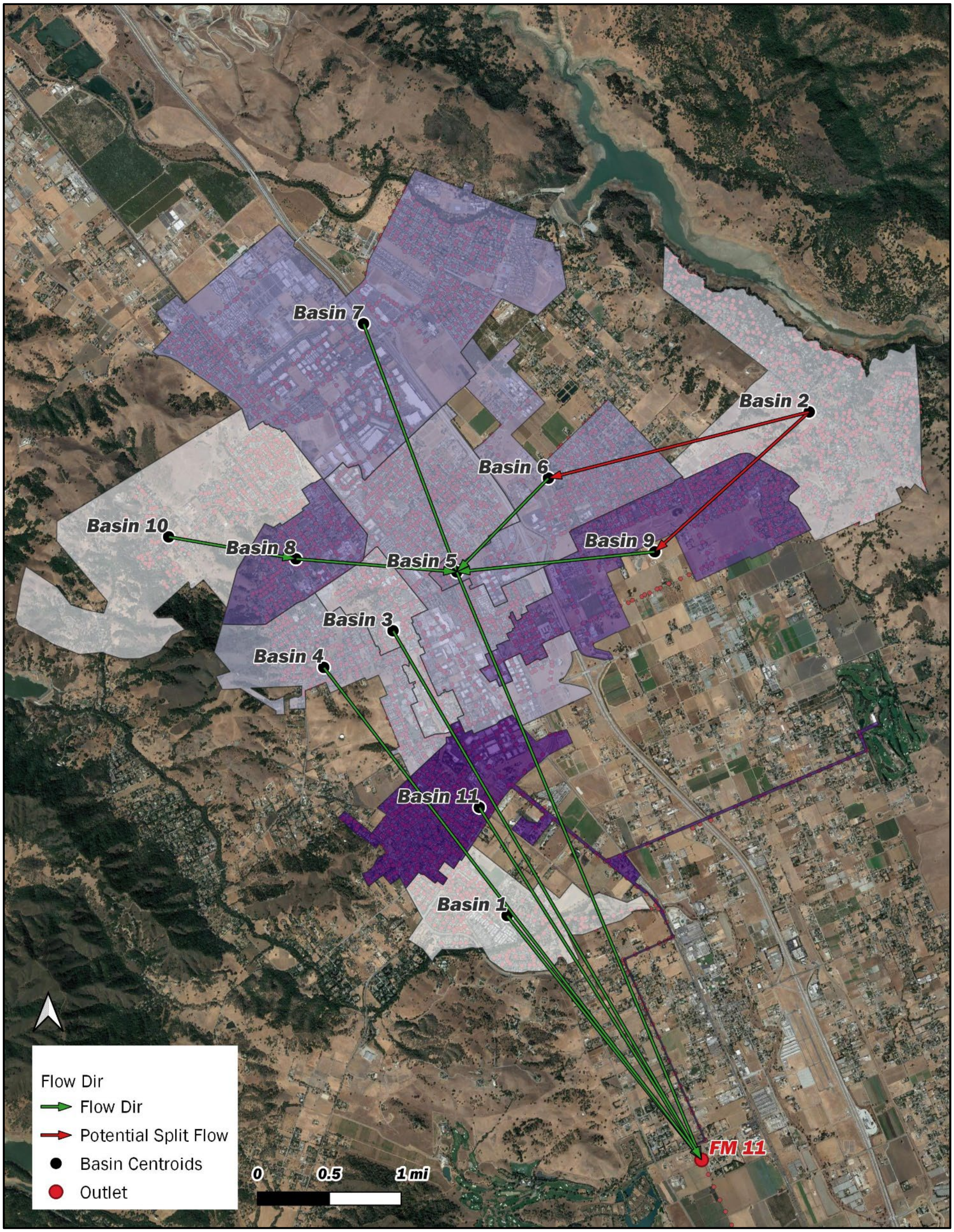

Figure 1-2. Map of Flow Monitoring Basins

1.3 Rainfall Monitoring Sites

V&A retrieved rainfall data from 20 personal weather stations (PWS) located around the City, ultimately selecting 8 sites that provided adequate coverage across the study area and quality data. These 8 PWS sites are listed in Table 1-3. Figure 1-3 shows the location of the PWS sites as well as the study area centroid utilized for rain gauge triangulation.

Table 1-3: List of Rain Gauge Locations

Rain Gauge ID	Name	Source	X (Longitude)	Y (Latitude)	Elev.
NW	KCAMORGA209	WU	-121.670	37.148	341
NE	KCAMORGA250	WU	-121.613	37.159	610
W	KCAMORGA201	WU	-121.677	37.128	436
CW	KCAMORGA265	WU	-121.643	37.132	348
CE	KCAMORGA179	WU	-121.624	37.134	351
E	KCAMORGA182	WU	-121.590	37.143	1,020
SW	KCAMORGA213	WU	-121.651	37.115	342
S	KCAMORGA129	WU	-121.641	37.099	333

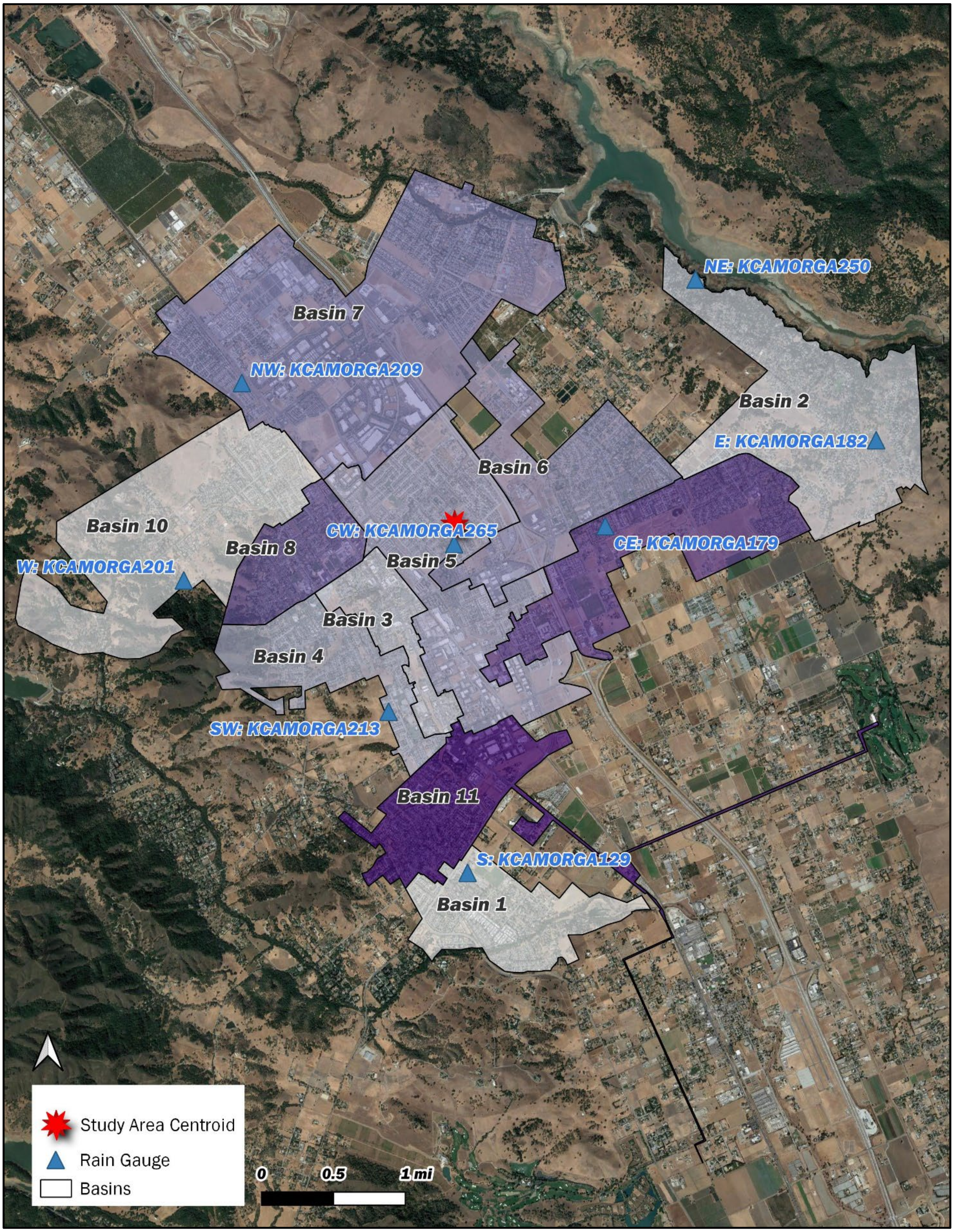

Figure 1-3: Map of Rain Gauges

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 typically 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 a safe work plan for the job at hand before entering.

Photo 2-1. Confined Space Entry

Photo 2-2. Typical Personal Four-Gas Monitor

2.2 Flow Meter Installation

V&A installed 11 area-velocity flow meters for temporary monitoring within the collection system using Sigma FL904 manufactured equipment. Sigma FL904 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, and 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.

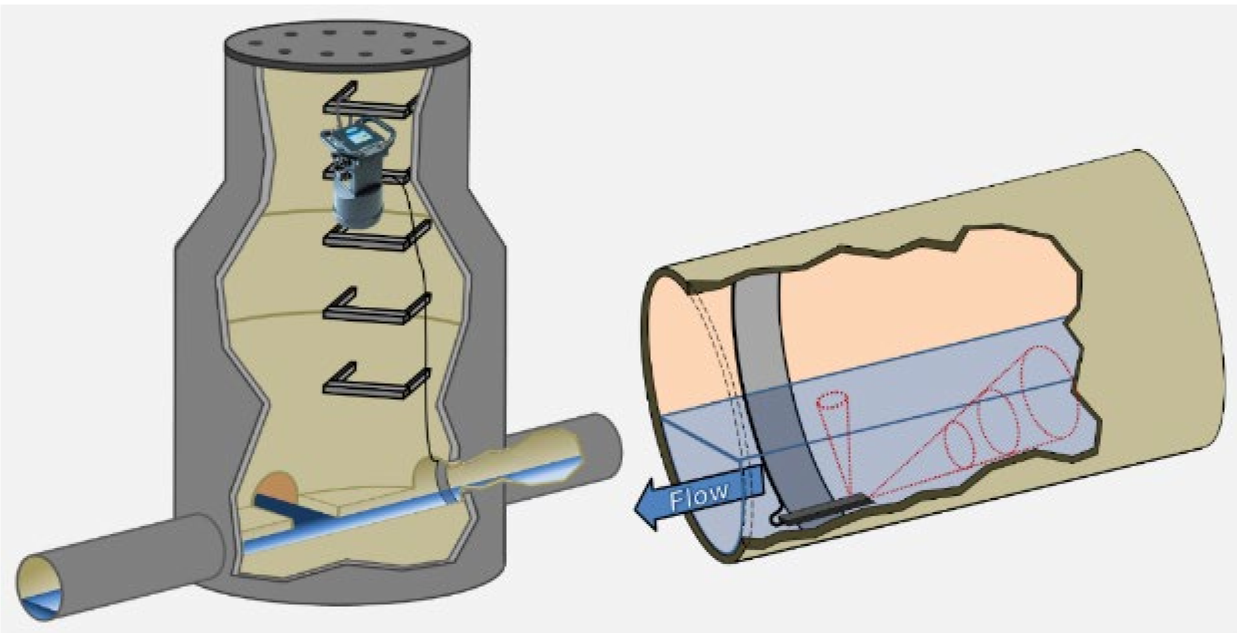

Figure 2-1. Typical Installation for Sigma 904 Flow Meter with Submerged Sensor

2.2.1 Installation Observations

During the installation of FM 08 (manhole G4-D.MH.040), the crew noticed an abandoned overflow line in the southeast (SE) quadrant of the manhole. It was initially thought, based on desktop analysis, that this was a split flow configuration. Our initial basin flow schematic was updated to reflect the change that basin 8 does not flow into basin 3. Photos 2-3 and Photo 2-4 show the top side view and the overflow pipe (SE effluent) view at manhole G4-D.MH.040

Photo 2-3. Manhole G4-D.MH.040 Topside View

Photo 2-4. Manhole G4-D.MH.040 SE Effluent Line (Capped)

2.3 Flow Calculation

Data retrieved from the flow meters were 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 pipes,

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

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

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 explained as a difference between a computed, estimated, or measured value and the generally accepted true or theoretically correct value. It can also be thought of as 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 is not performing to 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 and 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, the addition of flows at multiple monitoring sites is usually preferred over the subtraction of 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).

This concept is best demonstrated by the following example:

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%
- For flow addition, the final uncertainty is essentially a weighted average of the component uncertainties.

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%
- For flow subtraction, the final uncertainty will always be greater than the component uncertainties.
- 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%
- When subtracting similarly sized flow rates, the resulting uncertainties may not be manageable. 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 concerning 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 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 are often decreased (compared to Monday through Thursday). Similarly, flow patterns for Saturday and Sunday were also separated due to their potential for variations from the weekday hydrograph shape (timing) and magnitude. 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[3].

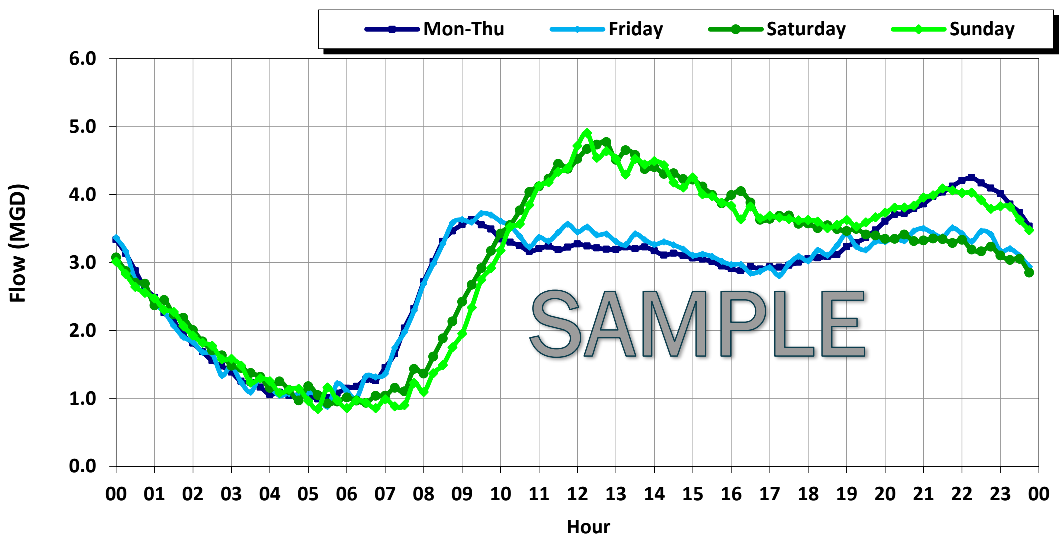

Figure 2-2. Sample ADWF Diurnal Flow Patterns

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

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

[3] 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 the reduction of the peak flow rate through the redistribution of the same volume of flow over a longer period of time. This occurs as a result of friction (resistance), internal storage, and diffusion along the sewer pipes. Fluids are constantly working towards equilibrium. For example, a volume of fluid poured into a static vessel with no outside turbulence will eventually stabilize to a static state, with a smooth fluid surface without peaks and valleys. Attenuation within a sanitary sewer collection system is based upon this concept. A flow profile with a strong peak will tend to stabilize towards equilibrium, as shown in Figure 2-3.

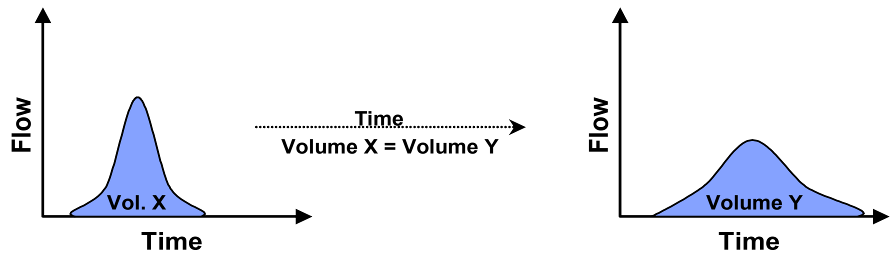

Figure 2-3. Attenuation Illustration

Within a sanitary sewer collection system, each individual basin will have a specific flow profile. As the flows from the basins combine within the trunk sewer lines, the peaks from each basin will 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

I/I consists of stormwater and groundwater that enters the sewer system through pipe defects and improper storm drainage connections and is defined as follows:

- **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 as a result of rainfall/stormwater.
- **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.

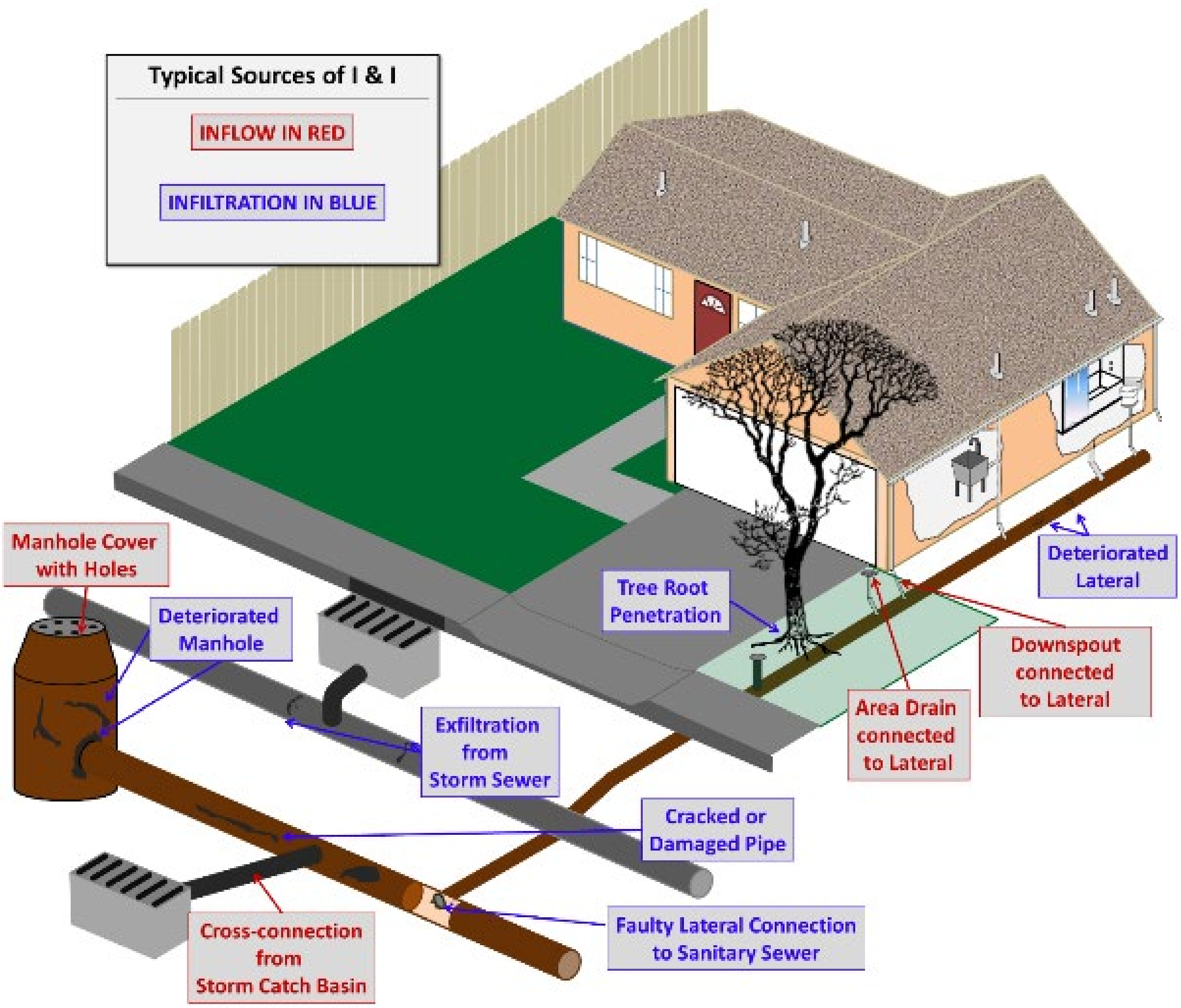

Figure 2-4. Typical Sources of Infiltration and Inflow

2.7.1 Infiltration Components

Infiltration can be further subdivided into components as follows:

- **Groundwater Infiltration (GWI)**: 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**: RDI 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 RDI 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 available capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Because the response and magnitude of inflow are tied closely to the intensity of the storm event, 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 that are billed strictly on flow volume).
  - Cost of Source Detection and Removal: Infiltration sources are usually harder to find and more expensive to correct than inflow sources. Infiltration sources include defects in deteriorated sewer pipes or manholes that may be widespread throughout a sanitary sewer system.

2.7.3 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 typically sustains for a period after rainfall has stopped and then gradually drops off as soils become less saturated and as groundwater levels recede to normal levels. Real-time flows 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.

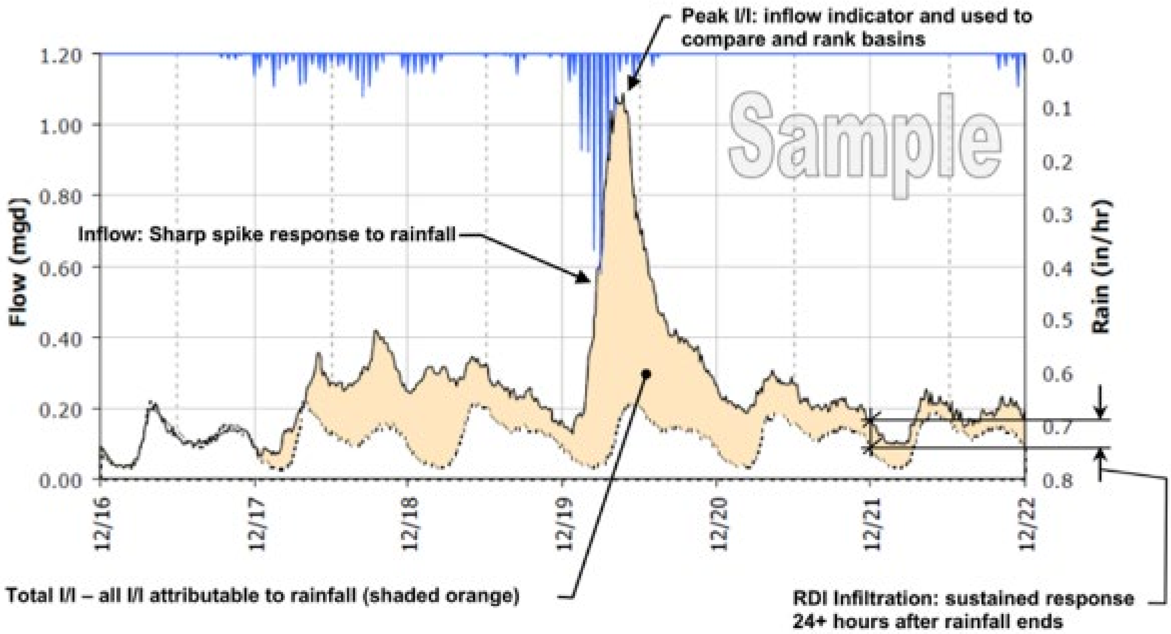

Figure 2-5. Sample Infiltration and Inflow Isolation Graph

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 a rainfall event. Peak I/I rates are used for inflow analysis.
- **Groundwater Infiltration**: 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 Analysis is conducted by looking at the infiltration rates at set periods after the conclusion of a storm event. Depending on the particular 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 RDI 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-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.
- **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.

The infiltration and inflow indicators were normalized by utilizing 41% of the per-IDM, 29% of the per-ADWF, and 30% of the per-ACRE calculations. The results will be shown in the following I/I analysis results sections.

3 Results and Analysis

3.1 Rainfall Monitoring

3.1.1 Rain Gauge Locations

V&A retrieved rainfall data from 20 personal weather stations (PWS) located on Weather Underground (WU). Ultimately, selecting 8 sites that provided adequate coverage across the study area and quality data for future evaluation of wet-weather I/I responses at the flow monitoring sites. Rain gauge IDs, installation locations, and total rainfall over the monitoring period are listed in Table 3-1. Figure 3-1 illustrates the location of the 8 rain gauges in relation to the sewer sub-basins.

Table 3-1. Summary of Rainfall Data

Rain Gauge ID	Name	Source	X (Longitude)	Y (Latitude)	Total Rainfall (in)
NW	KCAMORGA209	WU	-121.670	37.148	11.41
NE	KCAMORGA250	WU	-121.613	37.159	8.01
W	KCAMORGA201	WU	-121.677	37.128	10.89
CW	KCAMORGA265	WU	-121.643	37.132	8.88
CE	KCAMORGA179	WU	-121.624	37.134	9.48
E	KCAMORGA182	WU	-121.590	37.143	8.66
SW	KCAMORGA213	WU	-121.651	37.115	10.01
S	KCAMORGA129	WU	-121.641	37.099	9.75

3.1.2 Flow Study Rainfall Data

Multiple rainfall events elicited an I/I response during the flow monitoring period, as illustrated in Figure 3-2. Table 3-2 summarizes the rainfall from 3 significant wet-weather events during the monitoring period. Storm event dates/times listed in the table are from a triangulated average from the calculated centroid of all the sub-basins. The approximate centroid was calculated to be at 700 Juliann Way.

Figure 3-3 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[4] (triangulated to the centroid of the sub-basins) over the project duration. The cumulative precipitation (triangulated) was approximately 135% of historical precipitation averages over the specific duration of the flow monitoring. Only the maximum, minimum, and triangulated average rainfall totals are listed on the Figure to keep it from being congested.

[4] Historical data taken from the WRCC (Stations 045123 and 043417) in Los Gatos and Gilroy, CA: http://www.wrcc.dri.edu/summary/climsmnca.html

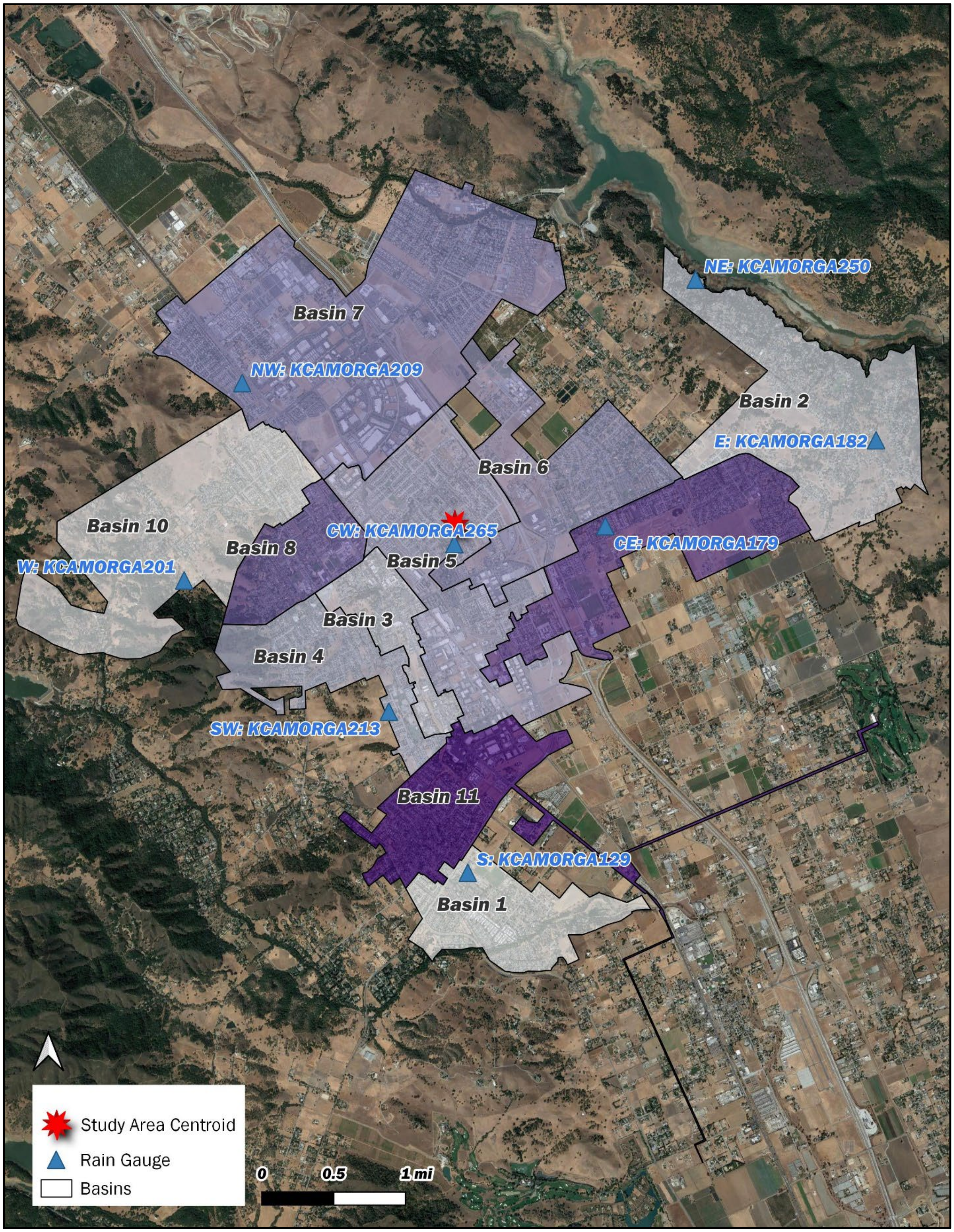

Figure 3-1. Location of Rain Gauges

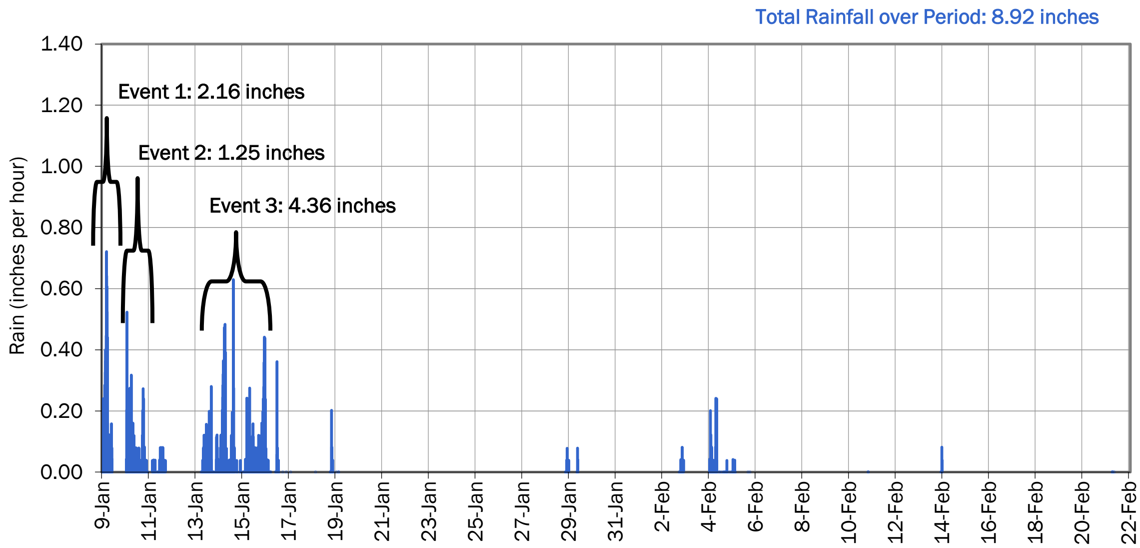

Figure 3-2. Rainfall Monitoring (Triangulated to Center of Sub-Basins)

Table 3-2. Summary of Significant Rainfall Data

Rainfall Event	Storm Start Date*	RG NW	RG NE	RG W	RG CW	RG CE	RG E	RG SW	RG S
Event 1	1/9/2023 0:00	2.97	1.91	2.80	2.16	2.33	2.11	2.63	2.73
Event 2	1/10/2023 2:00	1.35	0.99	1.30	1.26	1.40	1.24	1.28	0.83
Event 3	1/13/2023 10:00	5.54	3.72	5.22	4.36	4.31	4.01	4.79	4.83
Total over Monitoring Period:**		**11.41**	**8.01**	**10.88**	**8.88**	**9.48**	**8.66**	**10.00**	**9.74**

* Triangulated rainfall data/start times.
** Total rainfall may exceed rainfall for major events listed.

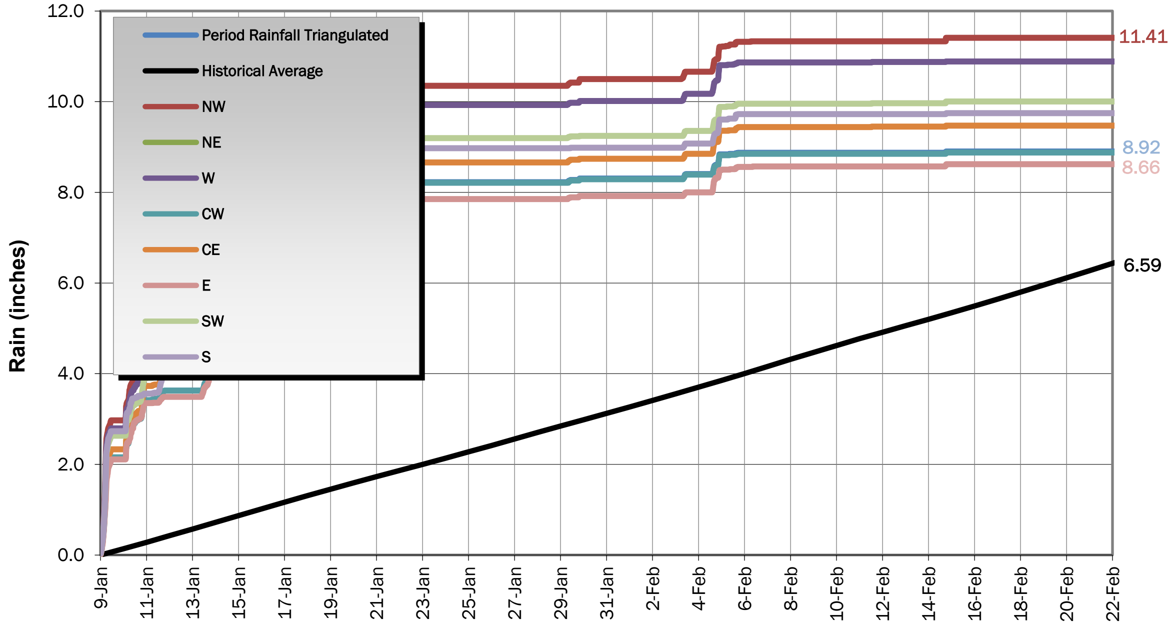

Figure 3-3. Rainfall Accumulation Plot

3.1.3 Regional Rainfall Event Classification

It is important to classify the relative size of a major storm event that occurs throughout a flow monitoring period[5]. Rainfall events are classified by intensity and duration. Based on historical data, frequency contour maps for storm events of a given intensity and duration have been developed by the NOAA for all areas within the continental United States (Figure 3-4)[7].

For example, the NOAA Rainfall Frequency Atlas[6] classifies a 10-year, 24-hour storm event at the Central rain gauge location as **4.33** inches. This means that in any given year, at this specific location, there is a 10% chance that **4.33** inches of rain will fall in any 24 hours.

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

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

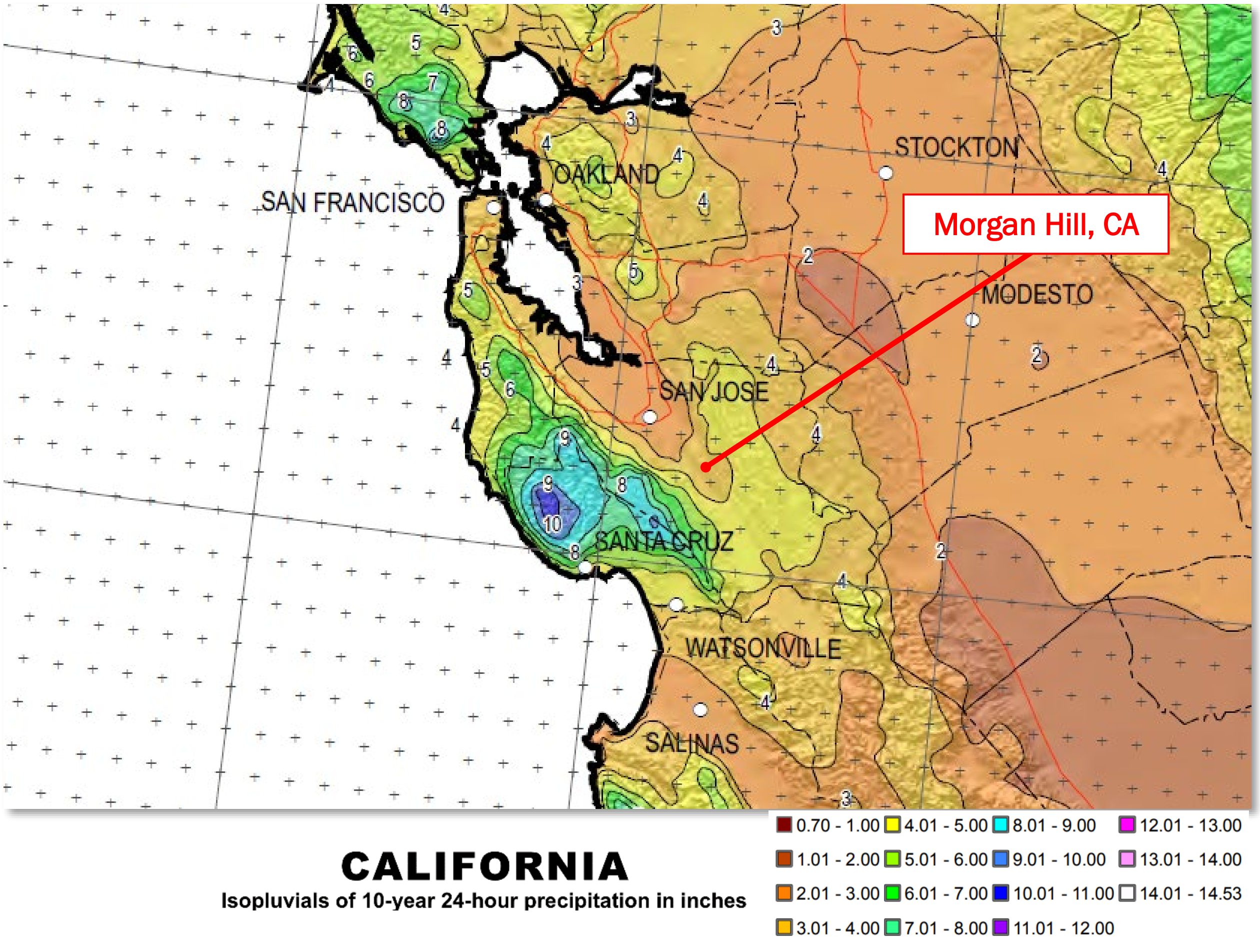

Figure 3-4. NOAA California 10-Yr 24-HR Precipitation Map

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.

Event 1 was classified as a 2-year event while Events 2 & 3 were classified as less than 1-year events, based upon total rainfall over the storm event duration. At peak intensity, Event 1 was approximately a 5-year, 3-hour event. Figure 3-5 shows the 60-min peak rainfall classification plot for the triangulated rain gauges. Figure 3-6 shows the 24-hour peak rainfall classification plot for the triangulated rain gauges.

Table 3-3. Rainfall Event Classification (Triangulated to Center of Sub-Basins)

Event	Storm Start Date	Duration (hrs)	Total Rainfall (in)	1-hr intensity (in/hr)	Return Period
1	1/9/2023 0:00	11	2.16	0.62	~ 2 - YR
2	1/10/2023 2:00	21.25	1.25	0.24	< 1 – YR
3	1/13/2023 10:00	69	4.3	0.39	< 1 – YR
Note: Only events > 0.50-inch listed					

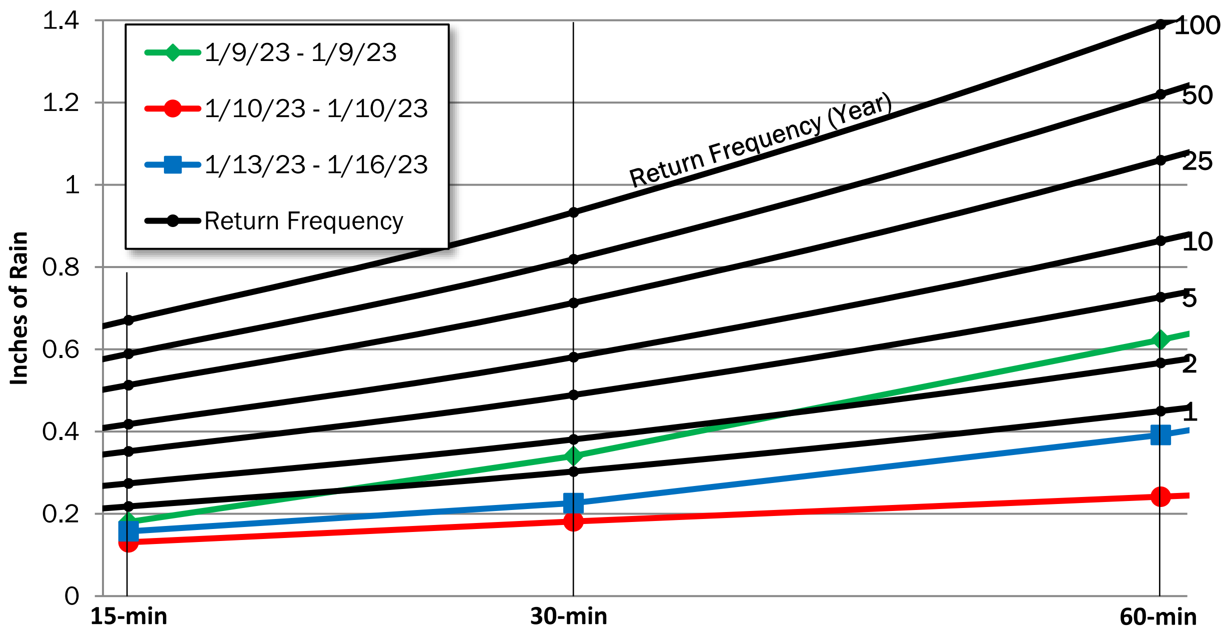

Figure 3-5. Rainfall Event Classification – 60-Min Period (Triangulated RG's)

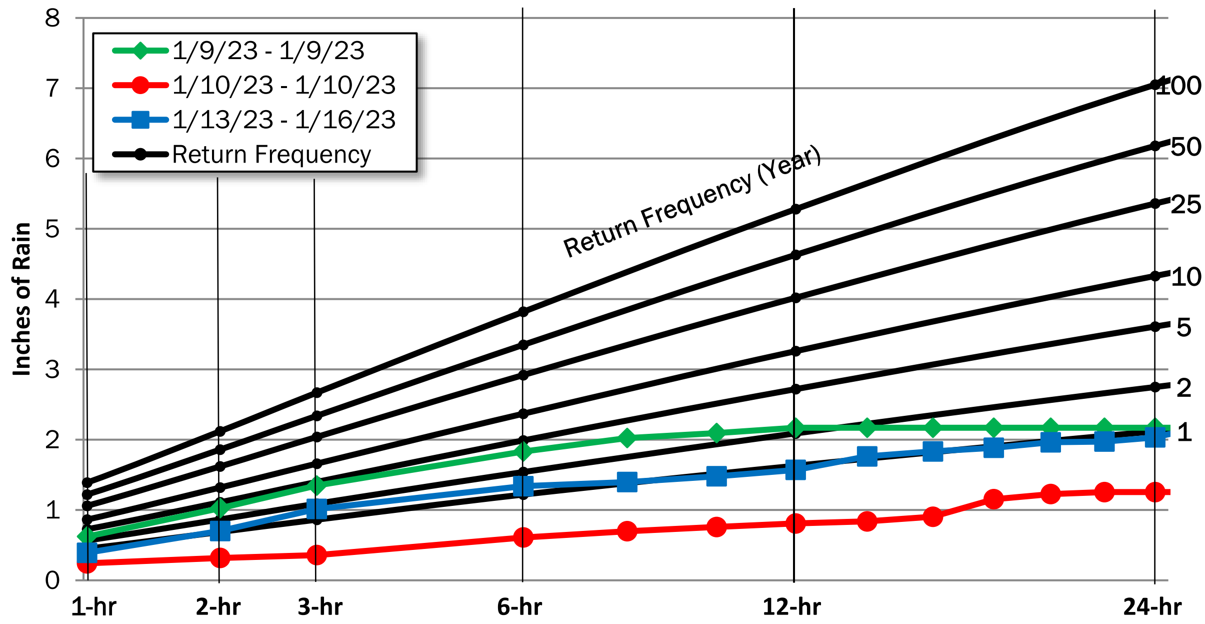

Figure 3-6. Rainfall Event Classification – 24-Hour Period (Triangulated RG's)

3.1.4 Rain Gauge Triangulation Distribution

The rainfall affecting the sanitary sewer collection system basins must be calculated based on the 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[7] is identified, and a weighted triangulated average is taken of the precipitation data from nearby rain gauge locations.

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 3-7 illustrates the IDW method with sample data. The rain gauge distribution, as calculated for each flow monitoring site, is shown in Table 3-4.

[7] 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

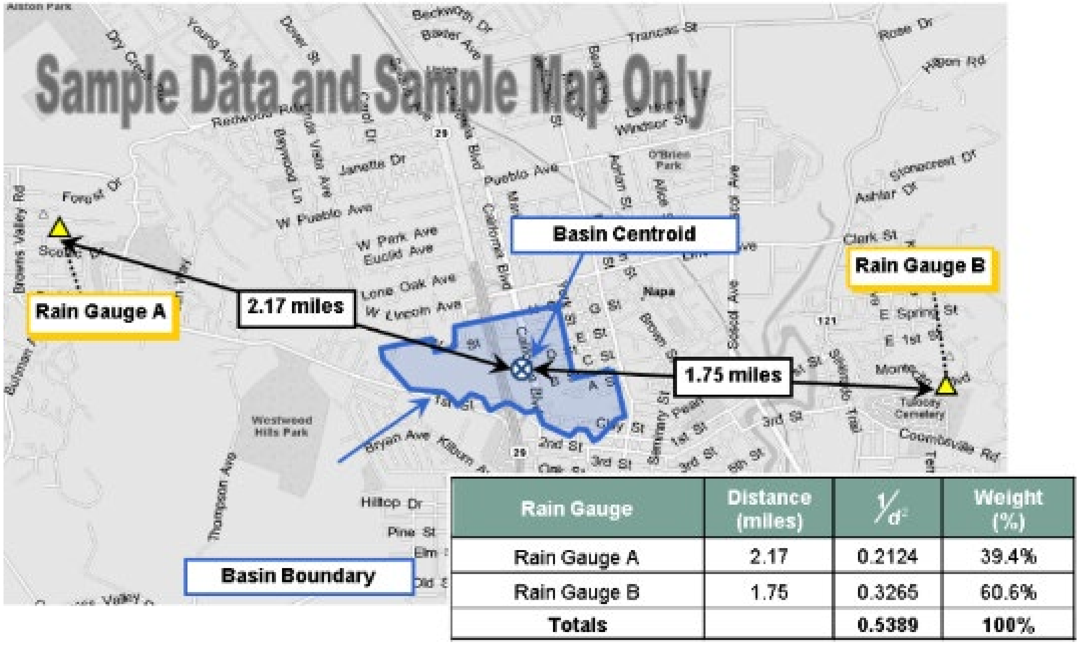

Figure 3-7. Rainfall Inverse Distance Weighting Method

Table 3-4. Rain Gauge Distribution per Monitoring Site

Monitoring Site	Rain Gauge								Total
	NW	NE	W	CW	CE	E	SW	S	
FM 01	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	5.7%	94.3%	100.0%
FM 02	0.0%	14.5%	0.0%	0.0%	38.7%	46.8%	0.0%	0.0%	100.0%
FM 03	0.4%	0.0%	0.8%	1.3%	0.0%	0.0%	95.5%	2.1%	100.0%
FM 04	0.0%	0.0%	15.0%	12.6%	0.0%	0.0%	65.9%	6.5%	100.0%
FM 05	16.9%	5.3%	23.2%	21.9%	22.9%	7.4%	2.4%	0.0%	100.0%
FM 06	3.2%	5.3%	0.0%	24.9%	59.4%	3.0%	4.1%	0.0%	100.0%
FM 07	54.1%	8.8%	10.0%	17.3%	9.9%	0.0%	0.0%	0.0%	100.0%
FM 08	8.3%	0.0%	80.4%	6.1%	0.0%	0.0%	5.2%	0.0%	100.0%
FM 09	0.0%	9.9%	0.0%	2.7%	57.2%	29.1%	1.1%	0.0%	100.0%
FM 10	6.4%	0.0%	88.8%	2.3%	0.0%	0.0%	2.4%	0.0%	100.0%
FM 11	13.6%	4.2%	19.4%	18.3%	18.4%	6.0%	9.7%	10.4%	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 3-5 summarizes the dry weather flow data measured for this study. ADWF curves for each site can be found in Appendix A. Figure 3-8 shows a flow schematic of the average daily flows and levels. The following ADWF analysis results are noted:

- Site FM 03 had the lowest average d/D ratio of 0.11.

- Site FM 08 had the highest average d/D ratio of 0.56.

Table 3-5. Dry Weather Flow

Monitored Site	Sediment (in.)	Average d/D Ratio	Mon-Thu ADWF (MGD)	Friday ADWF (MGD)	Saturday ADWF (MGD)	Sunday ADWF (MGD)	Overall ADWF (MGD)
FM 01	0	0.30	0.147	0.146	0.154	0.156	0.149
FM 02	0	0.25	0.188	0.192	0.194	0.207	0.192
FM 03	0	0.11	0.080	0.080	0.081	0.081	0.080
FM 04	0	0.32	0.240	0.241	0.243	0.246	0.241
FM 05	0	0.30	1.971	1.921	1.937	2.003	1.963
FM 06	0	0.27	0.141	0.138	0.145	0.149	0.142
FM 07	0	0.30	0.404	0.389	0.375	0.372	0.393
FM 08	0	0.56	0.422	0.430	0.433	0.451	0.429
FM 09	0	0.44	0.316	0.317	0.322	0.323	0.318
FM 10	0	0.31	0.182	0.180	0.186	0.187	0.183
FM 11	0	0.33	2.728	2.696	2.932	2.912	2.779

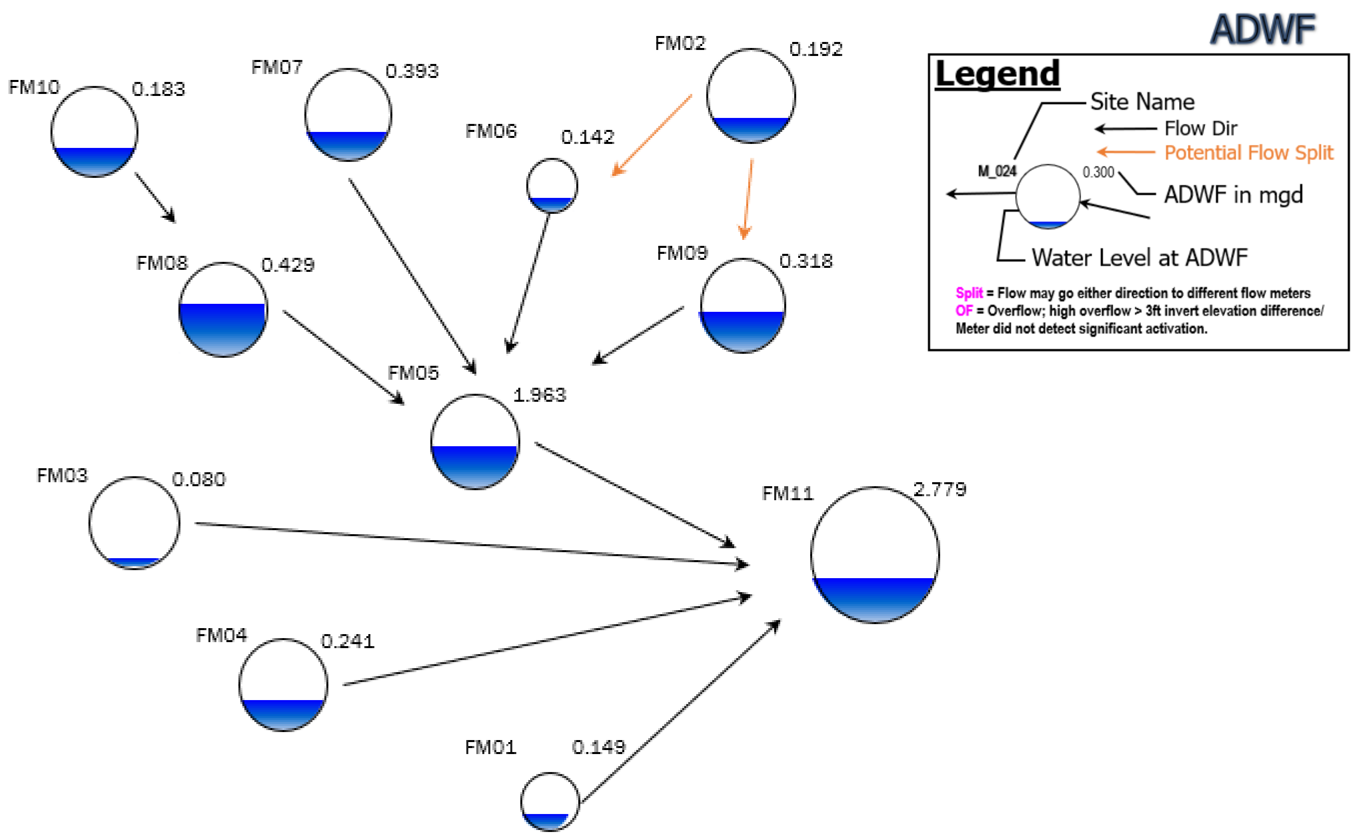

Figure 3-8. Average Dry Weather Flow (Flow Schematic)

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 as 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:

- **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, PF > 7 is highlighted in RED[8]; 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**: The d/D ratio for each site is computed based on the maximum depth of flow for the study. Standards for the d/D ratio vary from agency to agency but typically range between d/D ≤ 0.5 and d/D ≤ 0.75
  - For this report, d/D ratios > 0.75 are 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 3-6 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 3-9 and Figure 3-10 show bar graph summaries of the peaking factors and d/D ratios, respectively. Figure 3-11 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
  - Site FM 03 was the only site with a PF greater than 7:1. This site had a PF of 12:1.
  - The lowest PF was 3.1:1 at site FM 07.
  - The average site PF was 5:1.
- d/D Ratio:
  - d/D > 0.75: Two sites had d/D ratios greater than 0.75, site FM 08 and FM 09. Site FM 08 was the only site to surcharge.

[8] 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 3-6. 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 (FT)
FM 01	0.149	0.51	3.4	11.5	6.75	0.59	n/a
FM 02	0.192	0.73	3.8	15.5	6.78	0.44	n/a
FM 03	0.080	0.97	12.1	21	8.29	0.39	n/a
FM 04	0.241	1.36	5.6	17.5	11.51	0.66	n/a
FM 05	1.963	7.32	3.7	26.5	15.77	0.60	n/a
FM 06	0.142	0.85	6.0	12	6.82	0.57	n/a
FM 07	0.393	1.20	3.1	19.5	8.02	0.41	n/a
FM 08	0.429	2.21	5.2	15	29.81	1.99	1.23
FM 09	0.318	1.31	4.1	17.5	16.46	0.94	n/a
FM 10	0.183	0.96	5.2	15	9.24	0.62	n/a
FM 11	2.779	10.65	3.8	29	18.66	0.64	n/a

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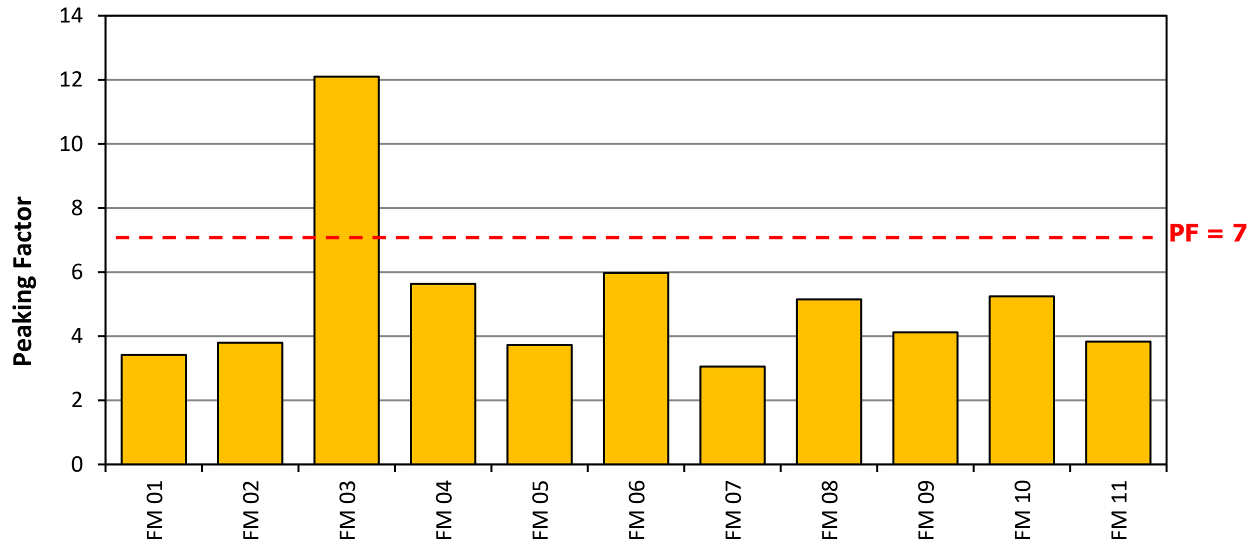

Figure 3-9. Peaking Factors

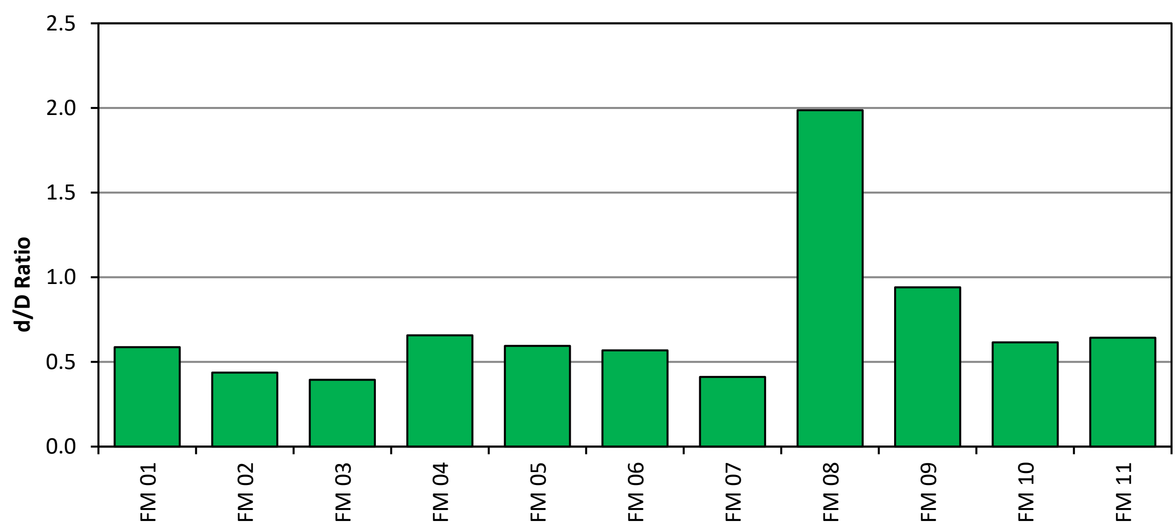

Figure 3-10. Capacity Summary: Max d/D Ratios

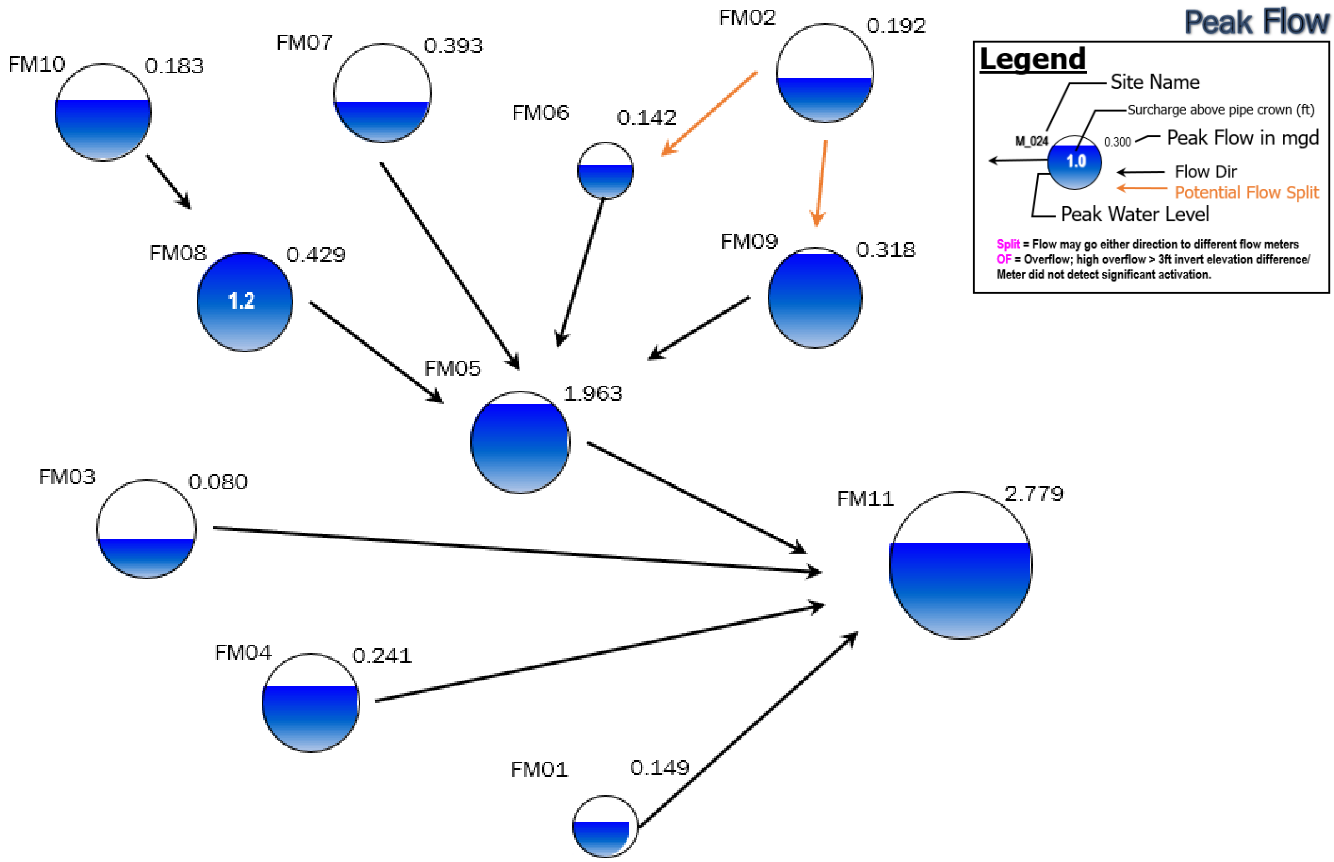

Figure 3-11. Peak Measured Flow (Flow Schematic)

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 3-12:

- **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 to 36 hours after the rainfall event has concluded, depending upon basin characteristics and types of I/I sources upstream.
- **Combined I/I**: the totalized volume (in gallons) of both inflow and RDI over the course of a rainfall event (shown below as the orange area).

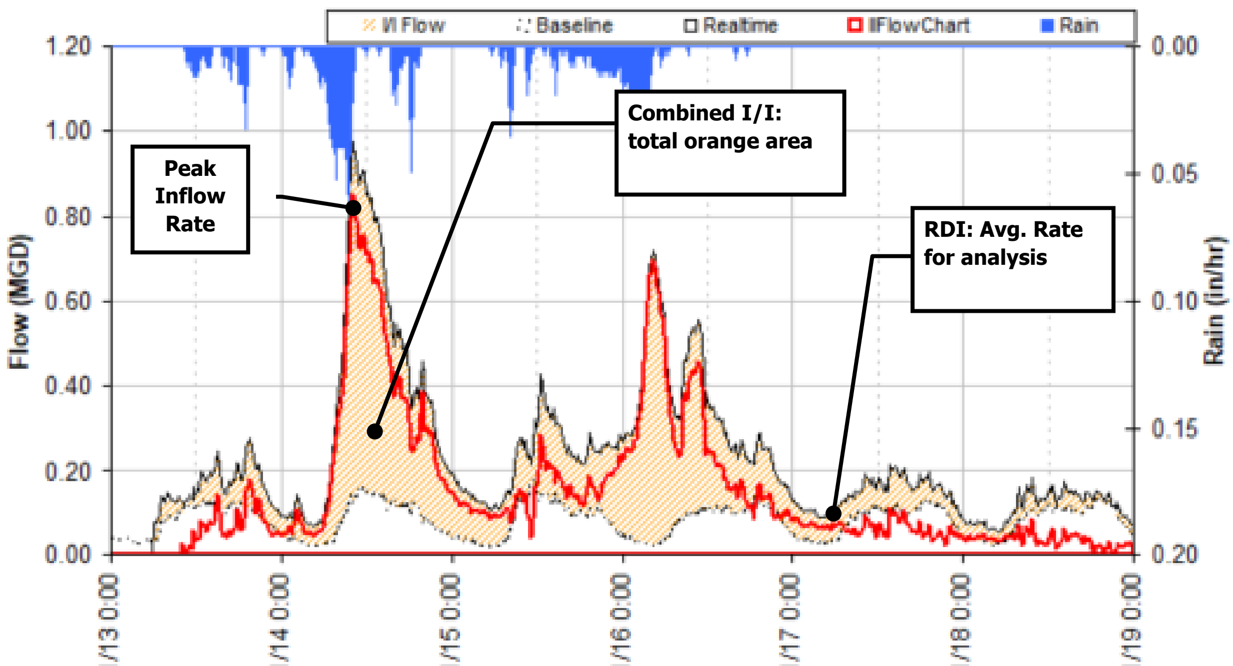

Figure 3-12. I/I Isolation, Site 3, Storm Event 3

I/I analysis was conducted on a basins-by-basin basis. To isolate the drainage areas of some flow monitoring basins, a subtraction of flow was required. Events utilized for analysis were presented previously in Table 3-3.

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.

Inflow results were taken from rainfall Event 3 and final rankings were weighted based on those described in Section 2.7.5. Table 3-7 summarizes the peak measured inflow analysis results for the relevant flow monitoring basins. Figures 3-13, 3-14, and 3-15 show the results of the inflow analysis. Results for Basin 11 were left off the bar graphs to not show the negative values. Figure 3-16 shows a temperature map summary of the inflow analysis results per basin. The “Top 3” basins have been shaded in RED. The following inflow results are noted:

- Basin 03 has the highest inflow per-ADWF, per-Acre, and ranked highest overall.
- Basin 08 has the highest inflow per-IDM and ranks second highest overall.
- Basin 04 was ranked the 3rd highest overall.
- Basin 11 showed a slight loss in peak inflow. This is most likely due to flow attenuation, as discussed in Section 2.6.

Table 3-7. Results and Rankings of Basin Inflow Analysis

Monitoring Basin	ADWF (mgd)	Basin Acreage	IDM	Basin Inflow Rate (mgd)	Inflow per-ADWF (ratio)	Inflow per-Acre (gpd/ACRE)	Inflow per IDM (gpd/IDM)	Final Inflow Ranking
01	0.15	487	66	0.184	1.2	378	2,789	10
02	0.19	1,075	130	0.503	2.6	467	3,865	6
03	0.08	266	69	0.850	10.6	3,194	12,314	1
04	0.24	479	84	0.945	3.9	1,973	11,250	3
05	0.68	1,039	253	0.883	1.3	850	3,490	7
06	0.14	733	116	0.635	4.5	866	5,474	4
07	0.39	1,982	244	0.717	1.8	362	2,939	9
08	0.25	343	65	0.916	3.7	2,671	14,092	2
09	0.13	764	147	0.304	2.4	398	2,069	8
10	0.18	1,150	84	0.672	3.7	584	7,996	5
11	0.34	595	393	-0.050	-0.1	-84	-127	11

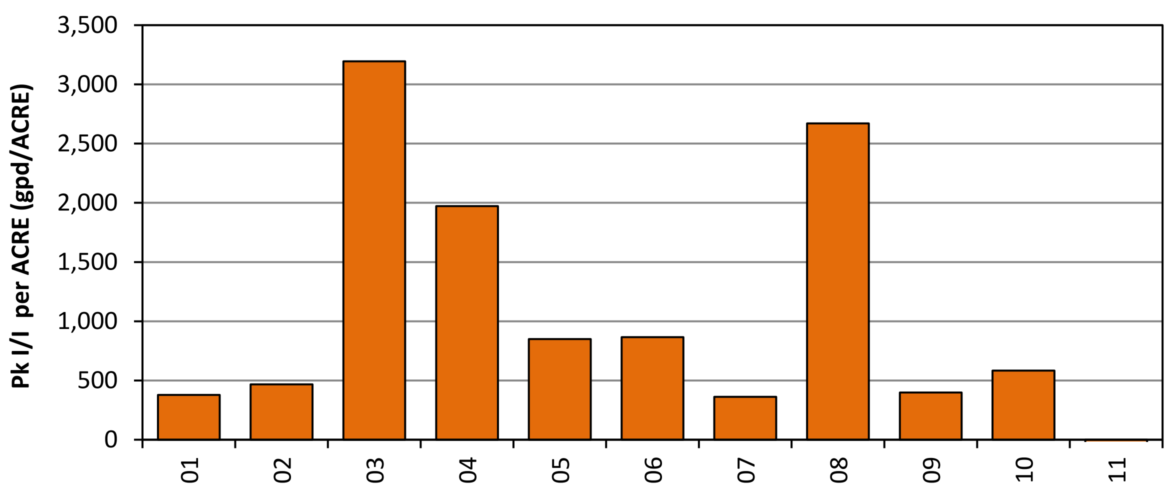

Figure 3-13: Averaged Peak I/I per Acre Analysis

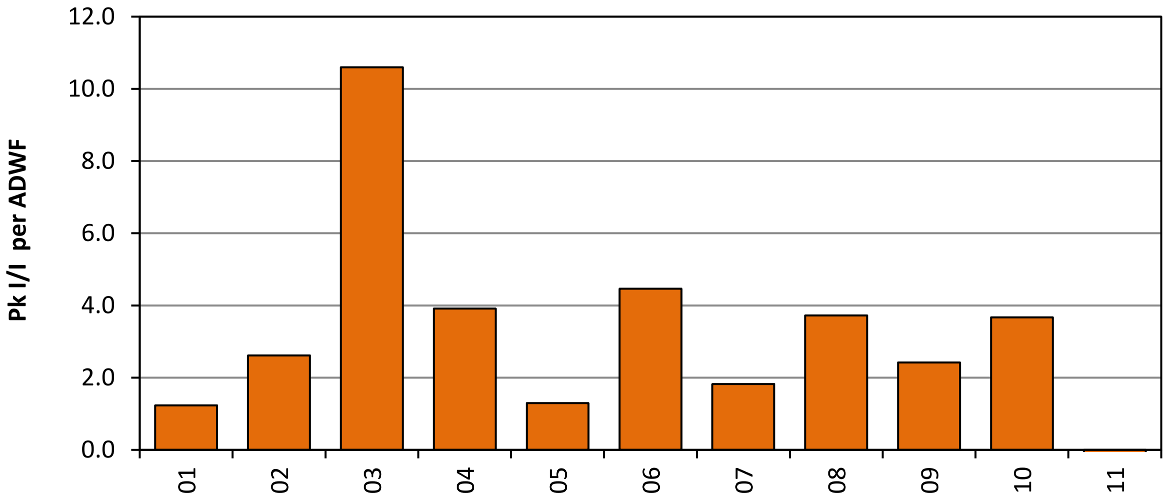

Figure 3-14: Averaged Peak I/I per ADWF Analysis

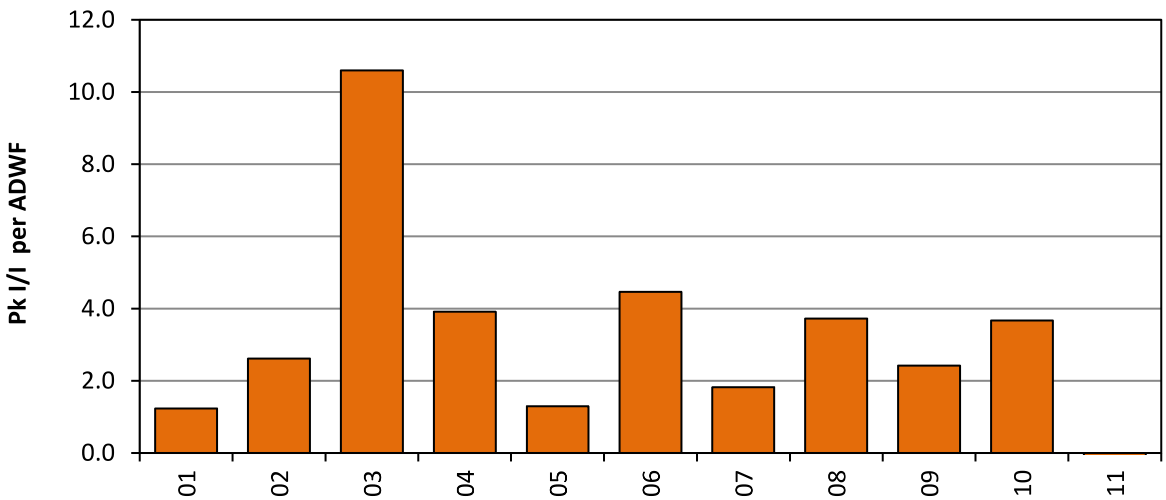

Figure 3-15. Averaged Peak I/I per IDM Analysis

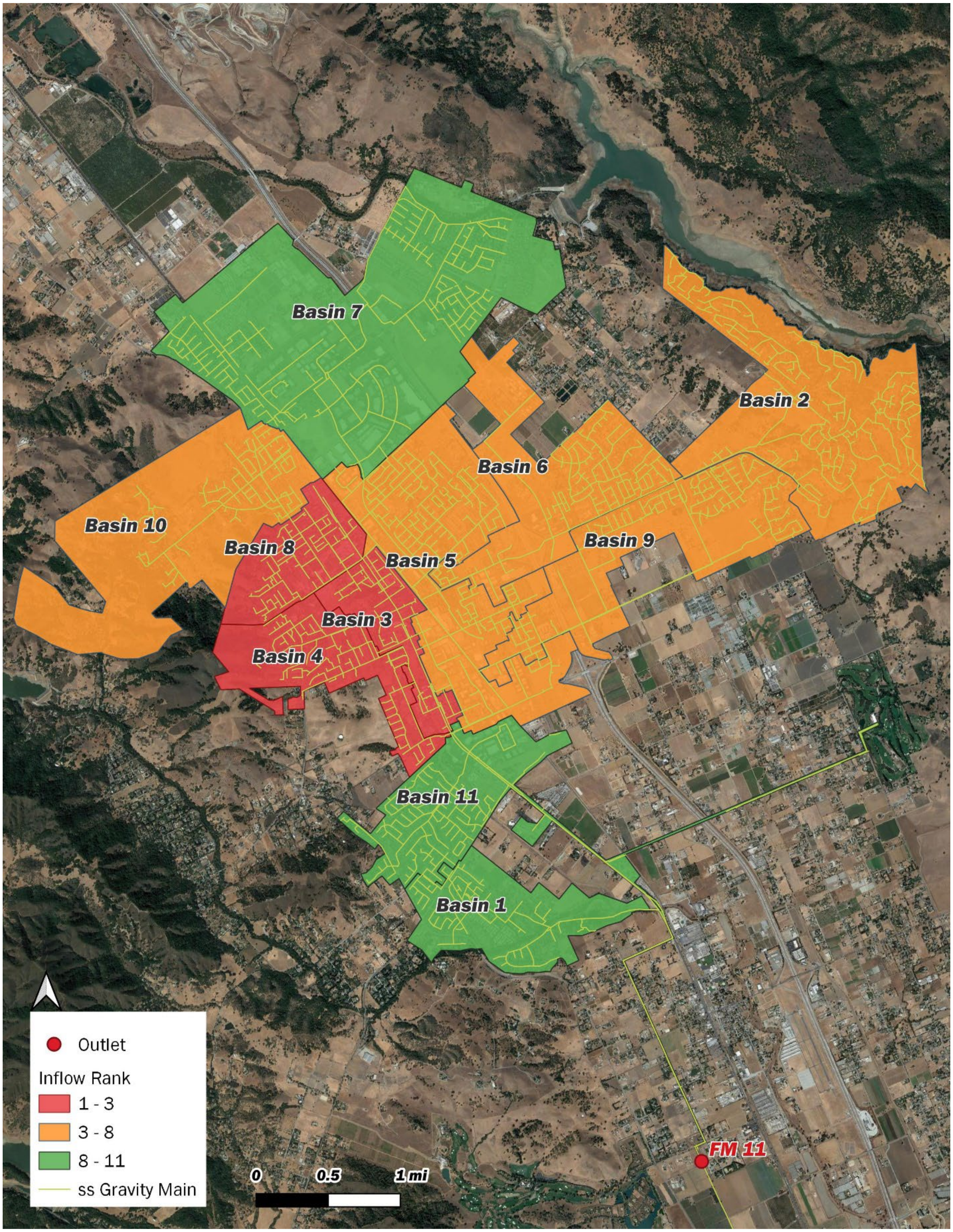

Figure 3-16. Temperature Map: Final Inflow Rankings

3.3.3 Rainfall Dependent Infiltration Results Summary

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. Increased flows into the sanitary sewer system are usually tied to groundwater levels 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).

For this study, the RDI rate used for comparative analysis was taken from rainfall Event 3, and final rankings were weighted based on those described in Section 2.7.5. Table 3-8 and Figures 3-17 through 3-19 summarize the captured RDI flow rates for the weighted Events. Figure 3-20 shows a temperature map. The “Top 3” basins for each category have been shaded in RED. The following RDI results are noted:

- Basin 04 has the highest RDI per-IDM and the highest calculated overall ranking.
- Basin 11 has the highest RDI per-ADWF, RDI per-Acre, and is ranked second highest overall.
- Basin 08 ranked 3rd highest overall according to RDI.

Table 3-8. Results and Rankings of Basin RDI Analysis

Monitoring Basin	ADWF (mgd)	Basin Acreage	IDM	Basin RDI Rate (mgd)	RDI per-ADWF (ratio)	RDI per-Acre (gpd/ACRE)	RDI per IDM (gpd/IDM)	Final RDI Ranking
01	0.149	487	66	0.099	0.7	203	1,495	7
02	0.192	1,075	130	0.210	1.1	196	1,617	6
03	0.080	266	69	0.167	2.1	630	2,427	4
04	0.241	479	84	0.367	1.5	766	4,370	1
05	0.681	1,039	253	0.325	0.5	313	1,284	8
06	0.142	733	116	0.057	0.4	77	487	10
07	0.393	1,982	244	0.072	0.2	36	294	11
08	0.246	343	65	0.282	1.1	823	4,340	3
09	0.126	764	147	0.067	0.5	88	456	9
10	0.183	1,150	84	0.235	1.3	204	2,798	5
11	0.345	595	393	1.045	3.0	1,756	2,659	2

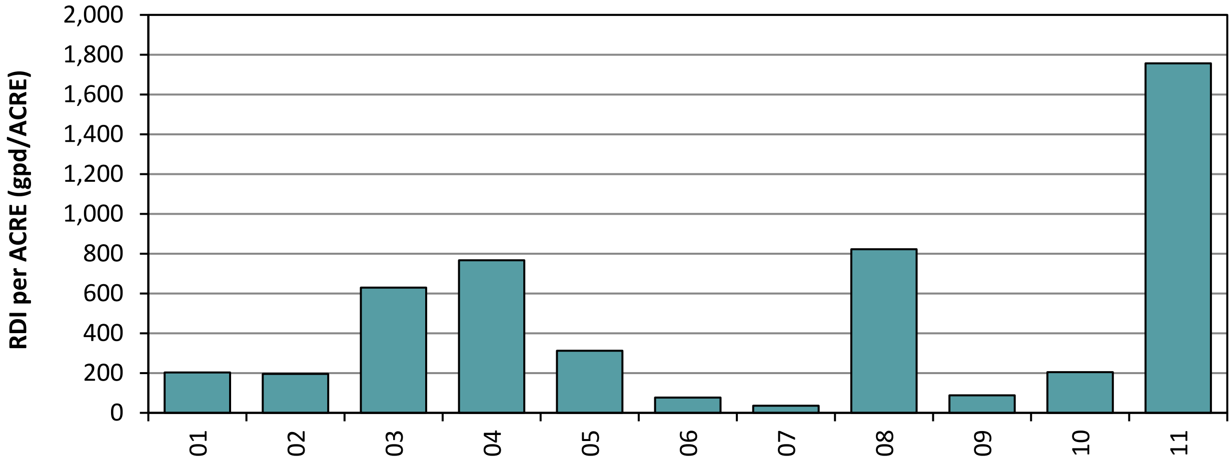

Figure 3-17: RDI per ACRE

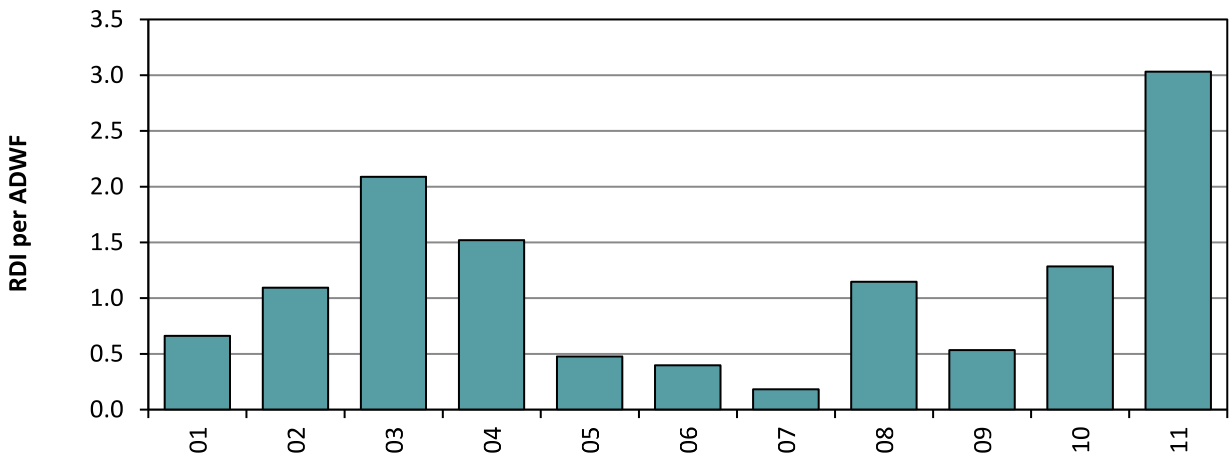

Figure 3-18: RDI per ADWF

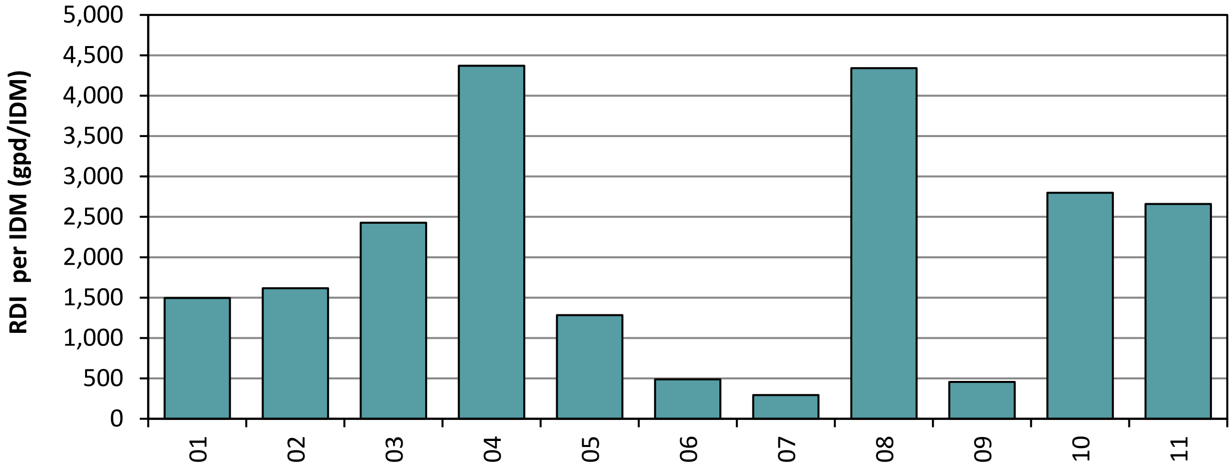

Figure 3-19: RDI per IDM

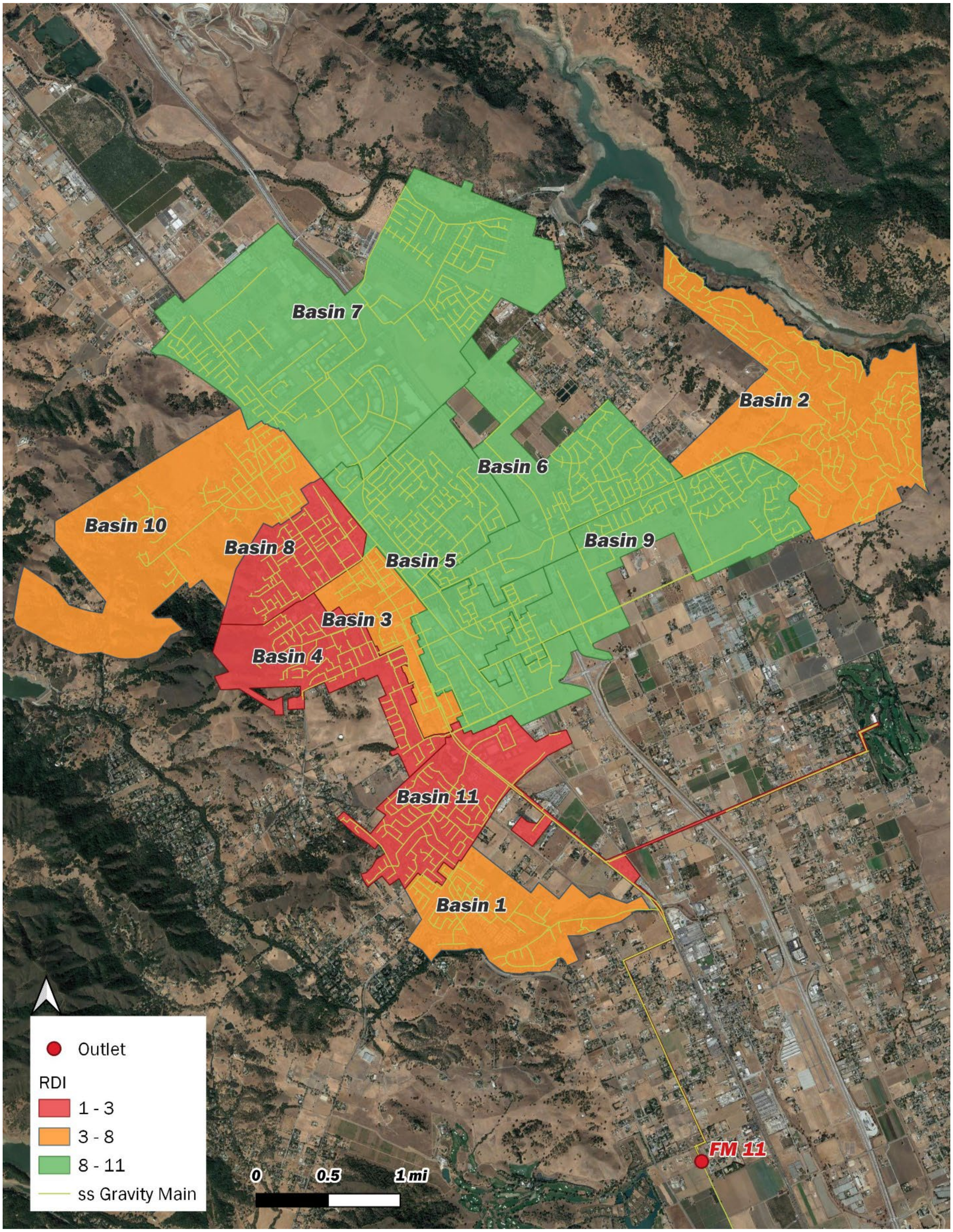

Figure 3-20. Temperature Map: Fina RDI Rankings

3.3.4 Combined I/I Results

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

Table 3-10 summarizes the combined I/I flow results for the three (3) selected events. The "Top 3" overall rankings for this analysis have been shaded in RED. Figures 3-20 through 3-22 show the bar graph results of the combined I/I analysis. A temperature map is shown in Figure 3-23.

The following combined I/I results are noted:

- Basin 03 had the highest combined I/I per-ADWF, per-Acre, per-IDM, and overall ranking.
- Basins 08 and 04 ranked 2nd and 3rd, respectively, for combined I/I.

Table 3-9. Basin Combined I/I Analysis Summary

Basin	ADWF (mgd)	Basin Acreag e	IDM	Combined I/I (gallons)	Combined I/I per ADWF (MGal/in/MGD	Combined I/I per Acre per inch-rain (R-Value)	Combined I/I per IDM (Gal/in/IDM)	Final Combined I/I Ranking
01	0.149	487	66	382,781	0.53	0.6%	1,209	9
02	0.192	1,075	130	950,283	1.22	0.8%	1,802	7
03	0.080	266	69	914,018	5.96	6.6%	6,921	1
04	0.241	479	84	1,625,053	1.41	2.6%	4,043	3
05	0.681	1,039	253	1,332,788	0.42	1.0%	1,131	8
06	0.142	733	116	491,409	0.80	0.6%	978	10
07	0.393	1,982	244	505,433	0.24	0.2%	389	11
08	0.246	343	65	1,283,624	1.50	4.0%	5,660	2
09	0.126	764	147	1,434,726	2.75	1.7%	2,350	5
10	0.183	1,150	84	1,195,428	1.26	0.7%	2,747	6
11	0.345	595	393	3,083,719	2.14	4.6%	1,875	4

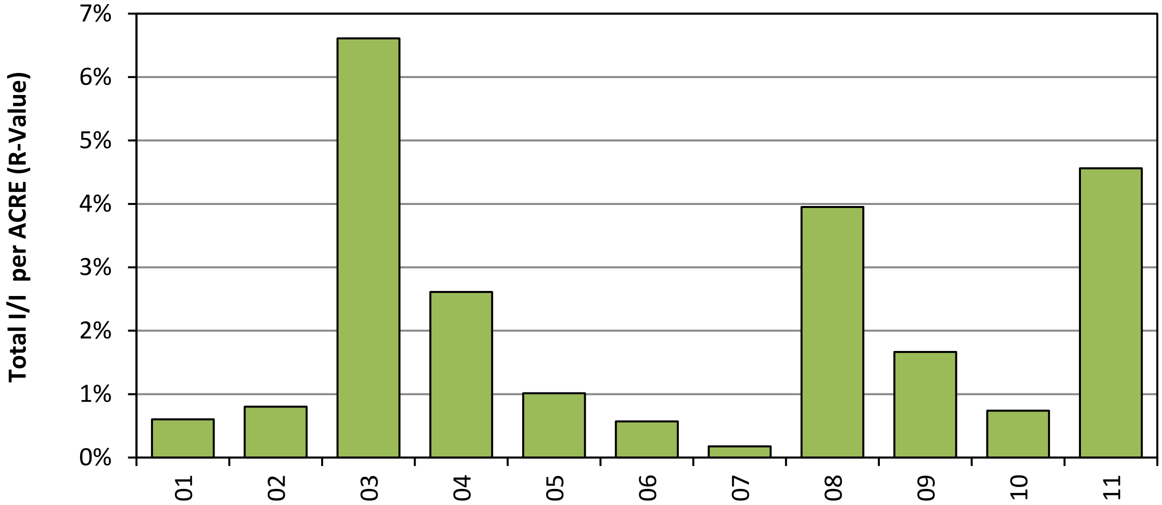

Figure 3-21: Combined I/I per ACRE

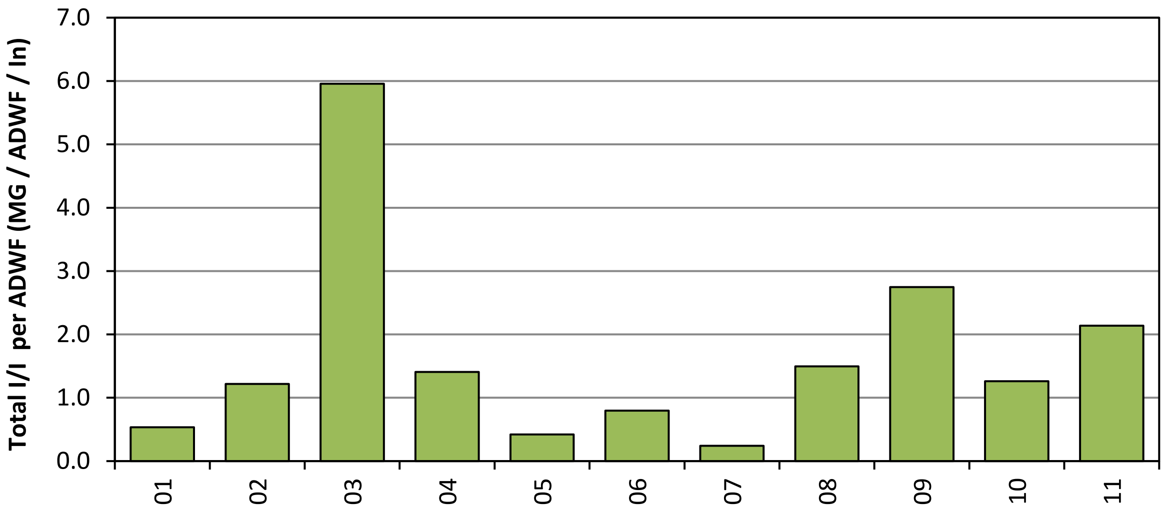

Figure 3-22: Combined I/I per ADWF

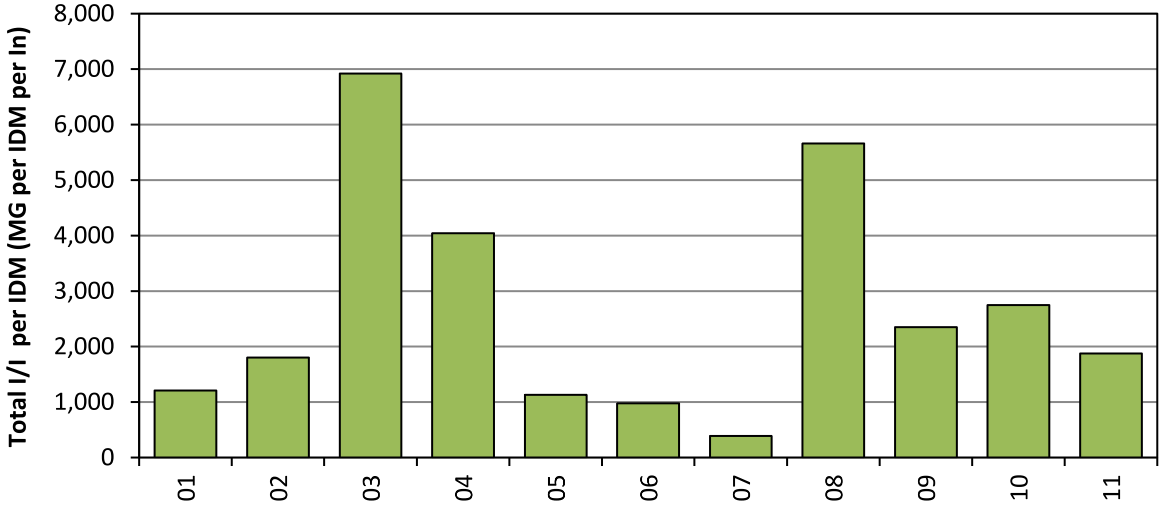

Figure 3-23: Combined I/I per IDM

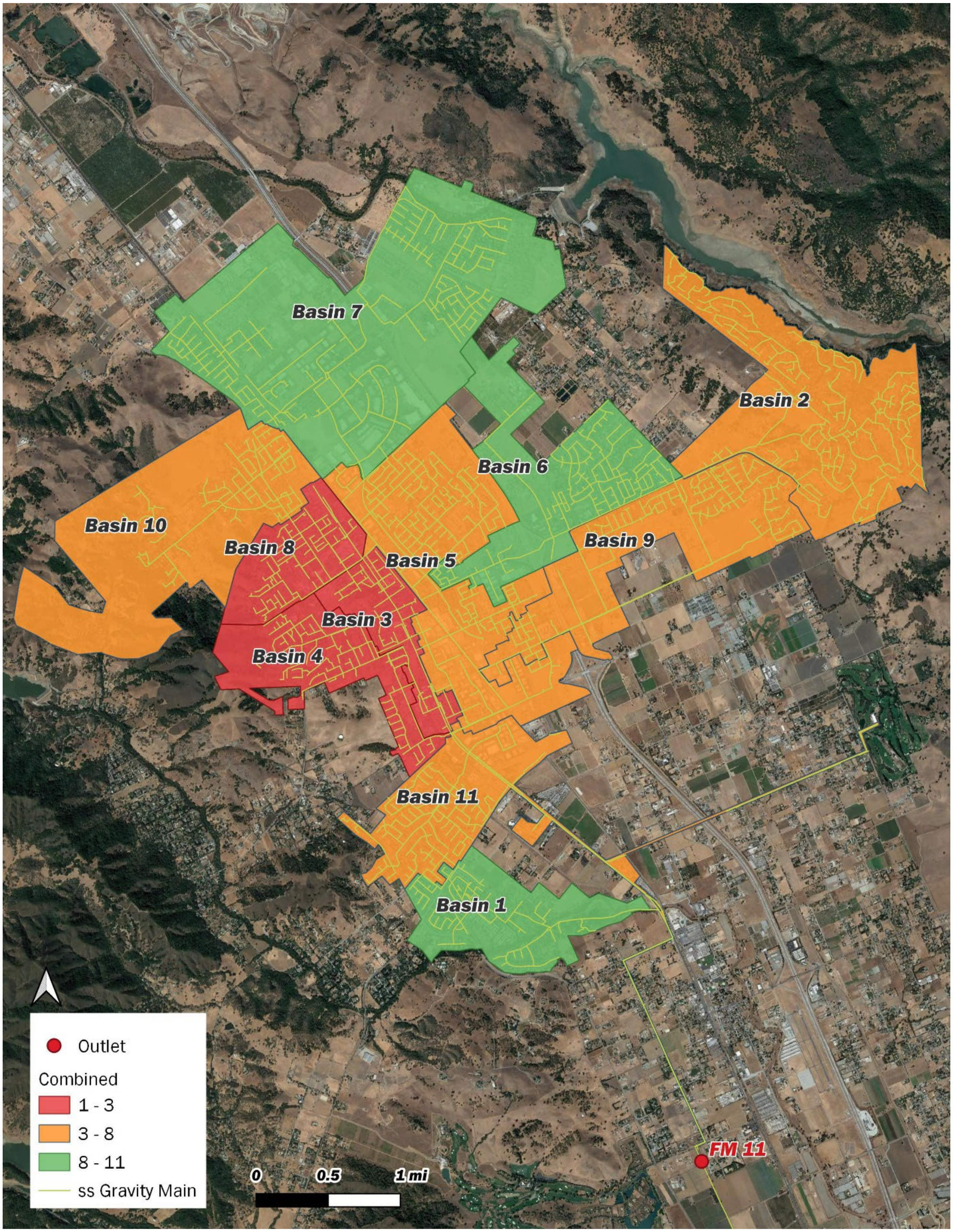

Figure 3-24. Temperature Map: Combined I/I Rankings

3.3.5 Groundwater Infiltration Results Summary

Dry weather (ADWF) 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[9]. Figure 3-24 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.

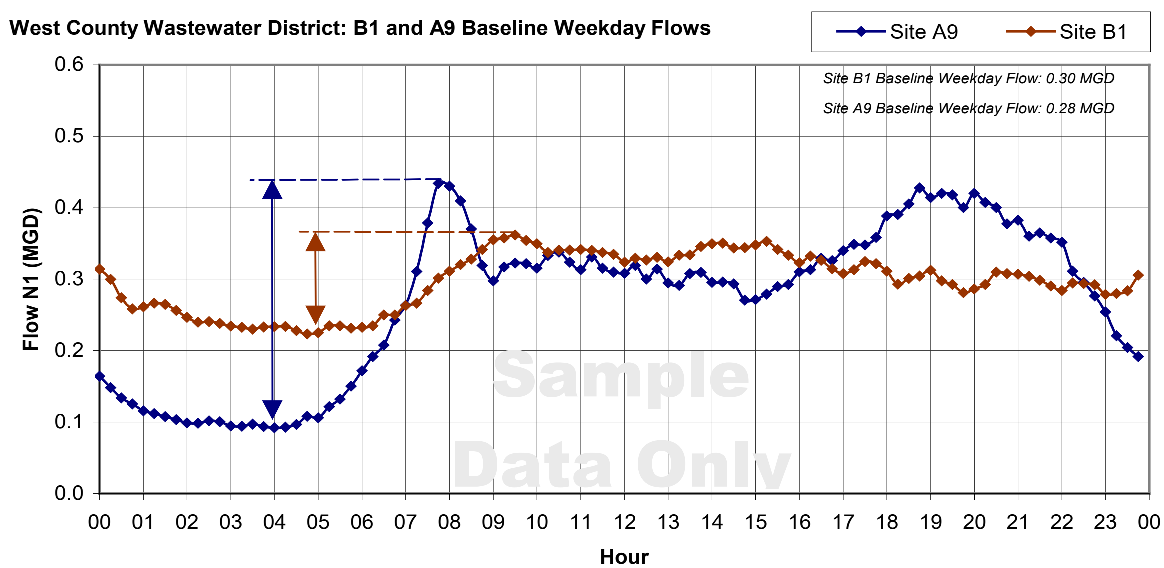

Figure 3-25. Groundwater Infiltration Sample Figure

It can be useful to compare the low-to-ADWF flow ratios for the flow monitoring sites. A site with abnormal ratios, and with 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 3-26 plots the low-to-ADWF flow ratios[10] 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). Figure 3-27 shows a color-coded map of the basins with rates of groundwater infiltration considerably above typical groundwater infiltration standards (as set forth by WEF).

WEF derived these ratios from residential sanitary sewer data. It is noted that the type of land use in each basin varies and there exists the possibility of excessive early-morning flows due to abnormal working hours in more commercial and industrial areas. This analysis is presented <u>for reference only</u>.

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[9] 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.

[10] 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).

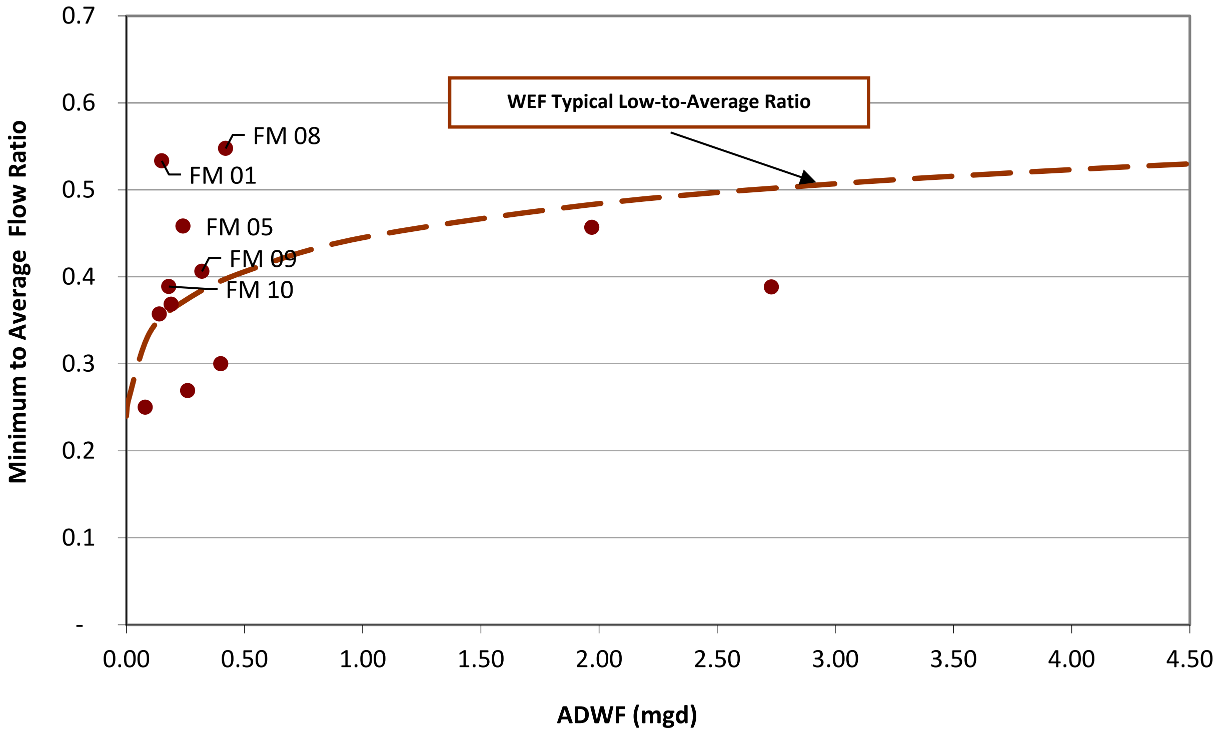


Figure 3-26. Minimum Flow Ratios vs ADWF[11]

The following GWI results are noted:

- The following basins had GWI rates higher-than typical standards, indicating that 5 of 11 basins have elevated groundwater infiltration. The five (5) basins are listed below and noted in the previous Figure 3-26.
  - FM 01
  - FM 05
  - FM 08
  - FM 09
  - FM 10

[11] 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.

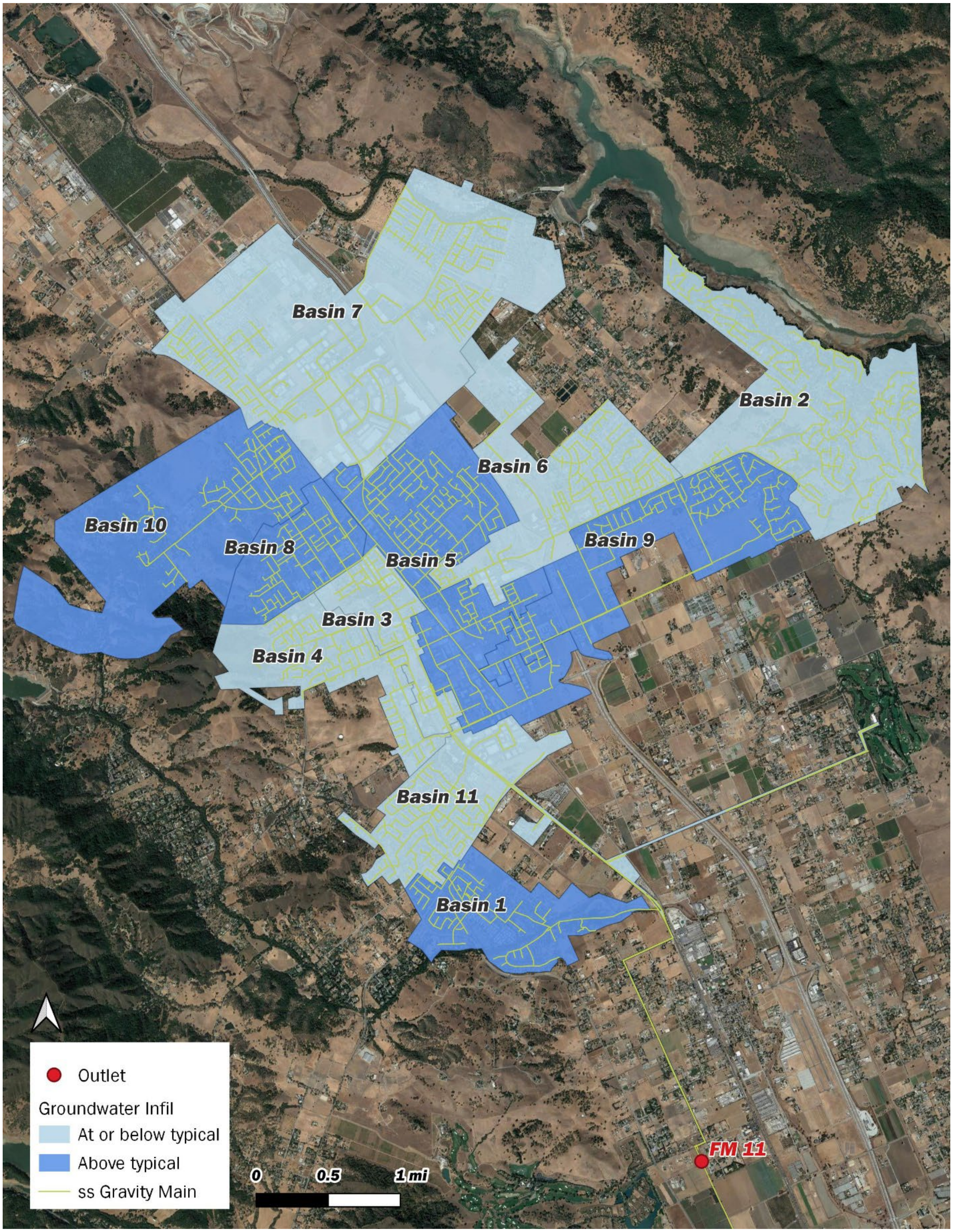


Figure 3-27. Basins with Groundwater Infiltration

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; however, the capacity deficiencies of the collection system may be of greater concern relative to the I/I response during peak wet weather events. The City may wish to have a model designed and/or a master plan study conducted to determine the overall needs of the City relative to I/I. Or simply, The study results can be used to update the master plan and compare with previous model assumptions and flow monitoring results.
2. **Verify Interconnections and Overflows:** understanding the interconnections and overflows can help with the master plan, basin isolation, and I/I analysis. There are a couple of split flows identified in the GIS at manholes F6-D.MH.012 and F6-D.MH.007, which should be field verified.
3. **Capacity Analysis:** Site FM 03 had a wet-to-dry weather flow peaking factor of over 12:1. In addition, sites FM 08 and FM 09 indicated capacity issues with site FM 09 reaching a d/D of 0.94 and site FM 08 surcharging 1.23 ft above the pipe crown. Additional investigation work is recommended in the collection system upstream of these sites to identify and remediate I/I sources.
4. **Determine I/I Reduction Program**: It is recommended that follow-up investigation work be conducted to identify sources of both I/I.
    a. If peak flows, sanitary sewer overflows and pipeline capacity issues are of greater concern, then priority can be given to investigate and reduce sources of inflow within the basins with the greatest inflow problems. The highest inflow occurs in Basins 03, 04, and 08.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the Basins with the greatest infiltration problems. The highest combined I/I occurs in Basins 03, 04, and 08, and the highest RDI occurs in Basins 04, 08, and 11. In addition, Basins 01, 05, 08, 09, and 10 show evidence of GWI.
5. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
    a. Smoke testing.
    b. Manhole inspections
    c. CCTV
    d. Private building evaluations
    e. Nighttime flow isolation checks for high GWI
6. **I/I Reduction Cost Effective Analysis**: The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow/infiltration and systematically rehabilitating or replacing the faulty pipelines, or (2) continued treatment of the additional rainfall dependent I/I flow.

Appendix A
Flow Monitoring Sites: Data, Graphs, Information

Monitoring Site: Site 1

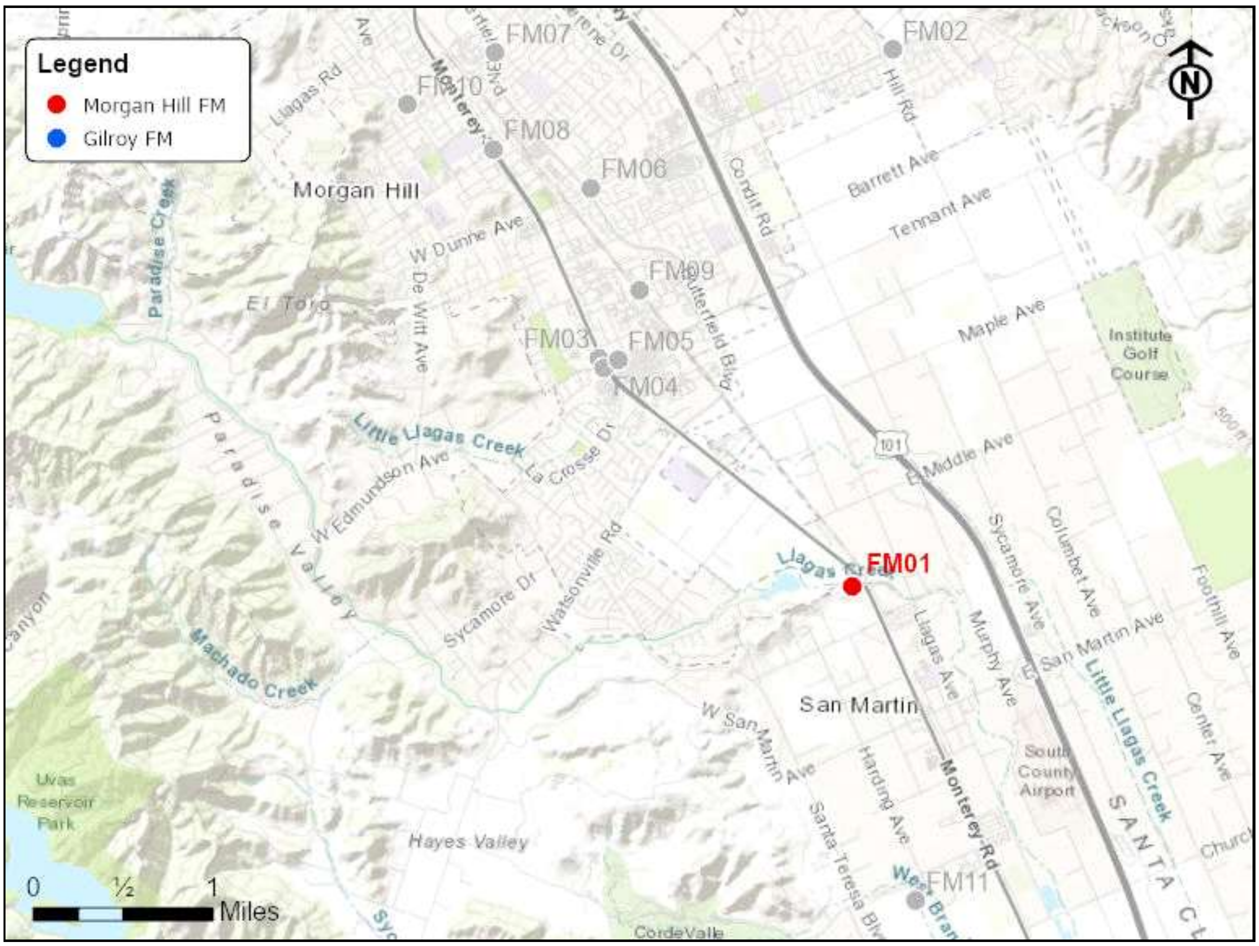
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: Llaga Creek Access, off of Monterey Hwy

Data Summary Report



Vicinity Map: Site 1

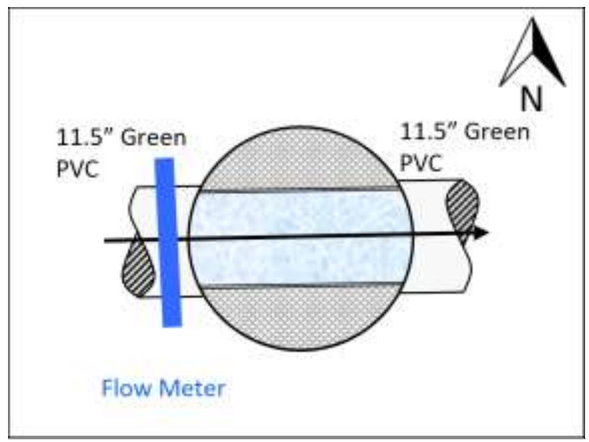
SITE 1

Site Information — MH ID: J6-C.MH.004

Location:	Llaga Creek Access, off of Monterey Hwy
Coordinates:	121.3737° W, 37.5435° N
Rim Elevation:	298 feet
Expected Pipe Diameter:	12 inches
Measured Pipe Diameter:	11.5 inches
ADWF:	0.149 mgd
Peak Measured Flow:	0.51 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch


Street View


Plan View

SITE 1

Additional Site Photos

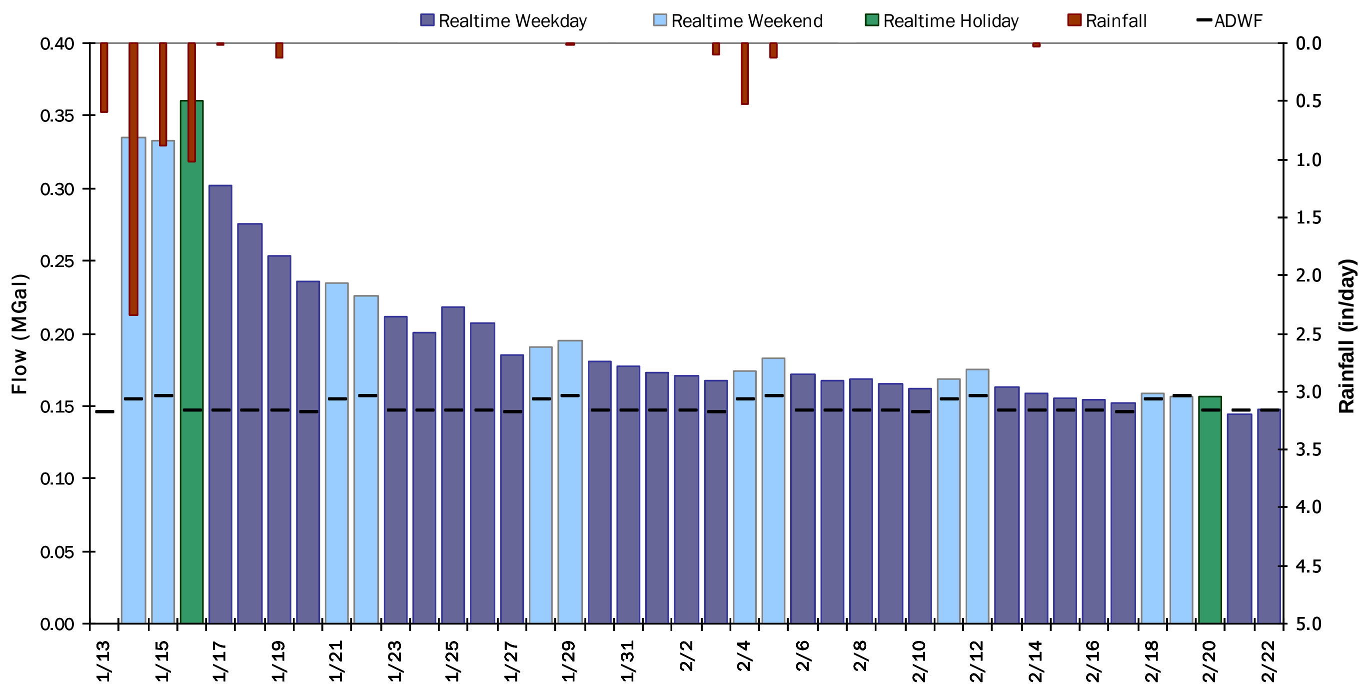
East Effluent Pipe

Monitored West Influent Pipe

SITE 1
Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.201 MGal   Peak Daily Flow: 0.360 MGal   Min Daily Flow: 0.145 MGal

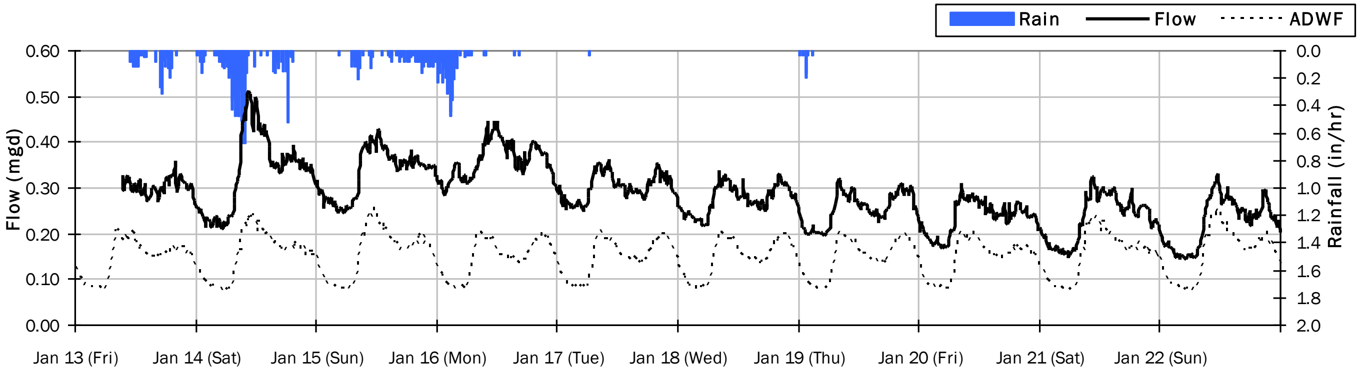
Total Rainfall: 5.75 inches

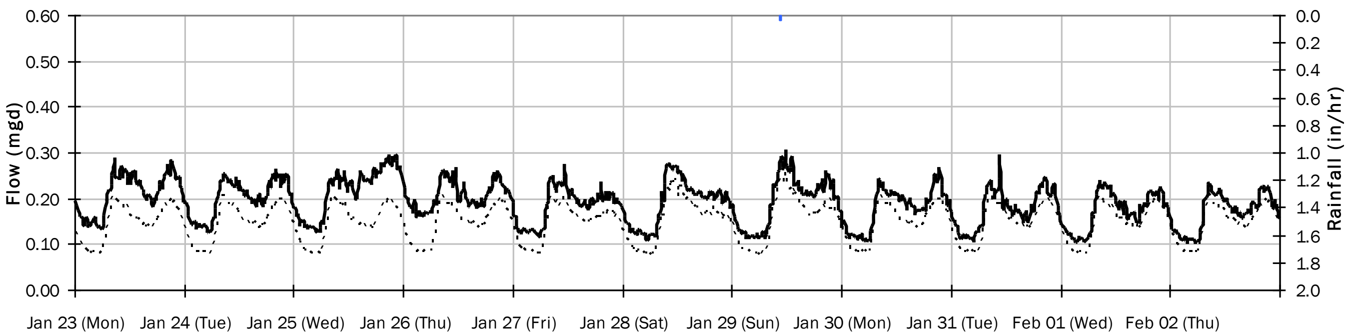


SITE 1

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 4.99 inches Period Avg Flow: 0.235 mgd Period Peak Flow: 0.514 mgd Period Min Flow: 0.101 mgd



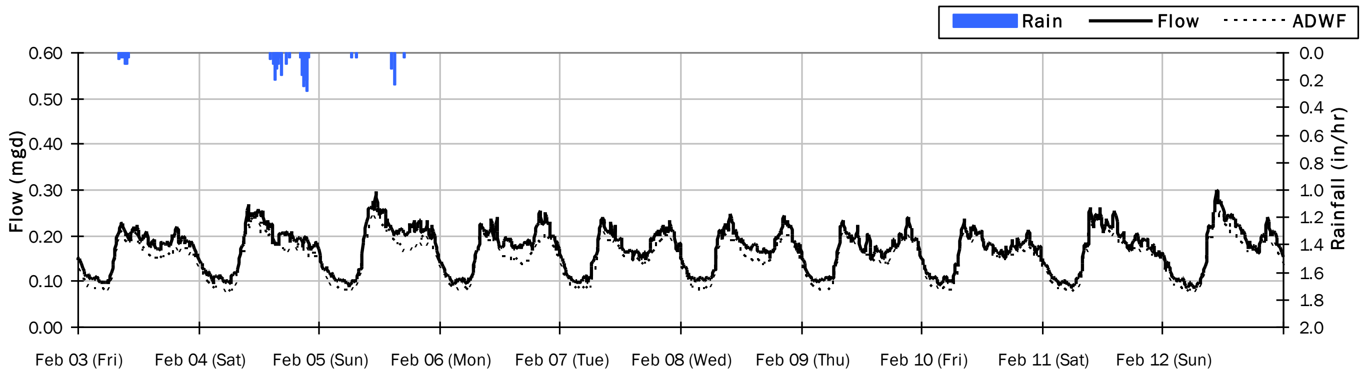


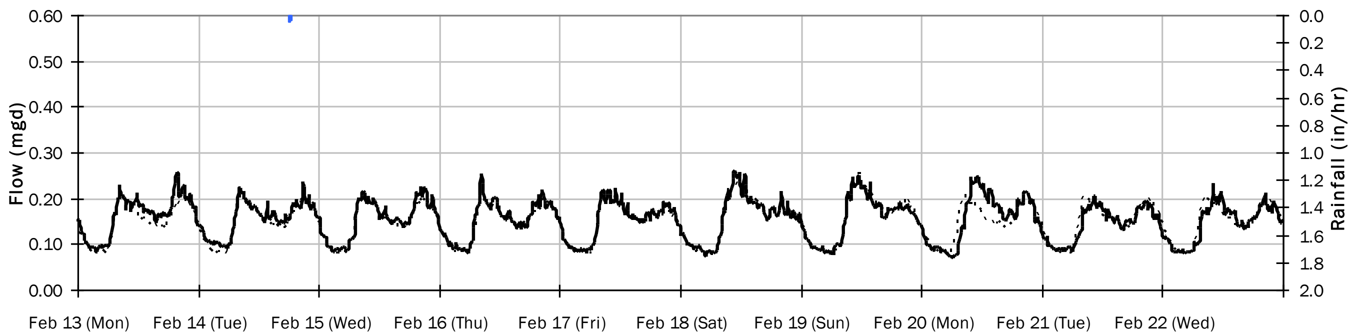
SITE 1

Flow Summary: 2/3/2023 to 2/22/2023

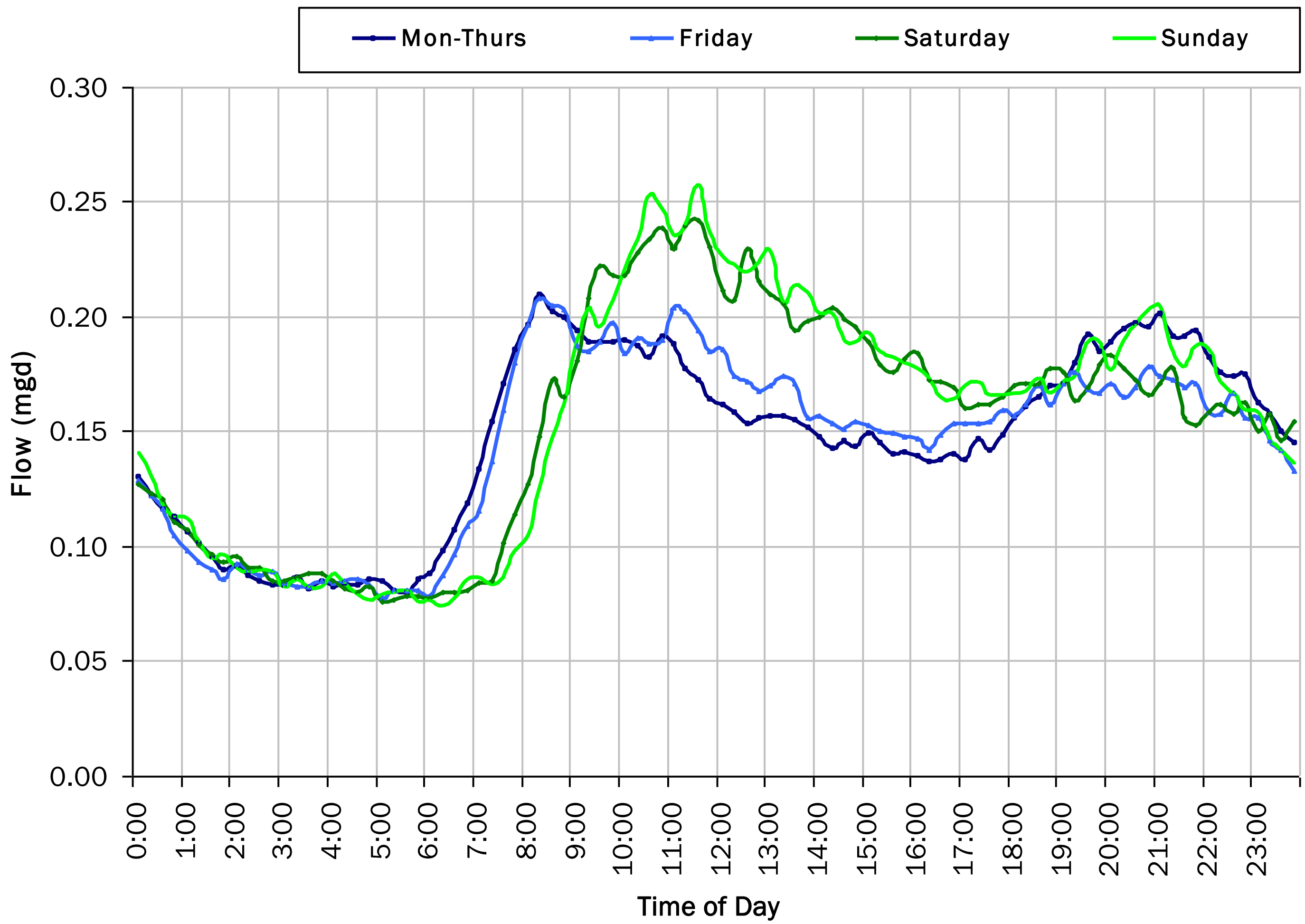
Period Rainfall: 0.76 inches

Period Avg Flow: 0.163 mgd Period Peak Flow: 0.300 mgd Period Min Flow: 0.071 mgd

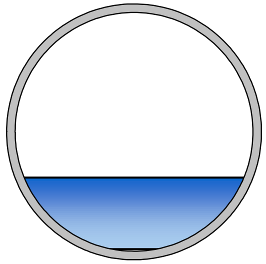




SITE 1
Average Dry Weather Flow Hydrographs

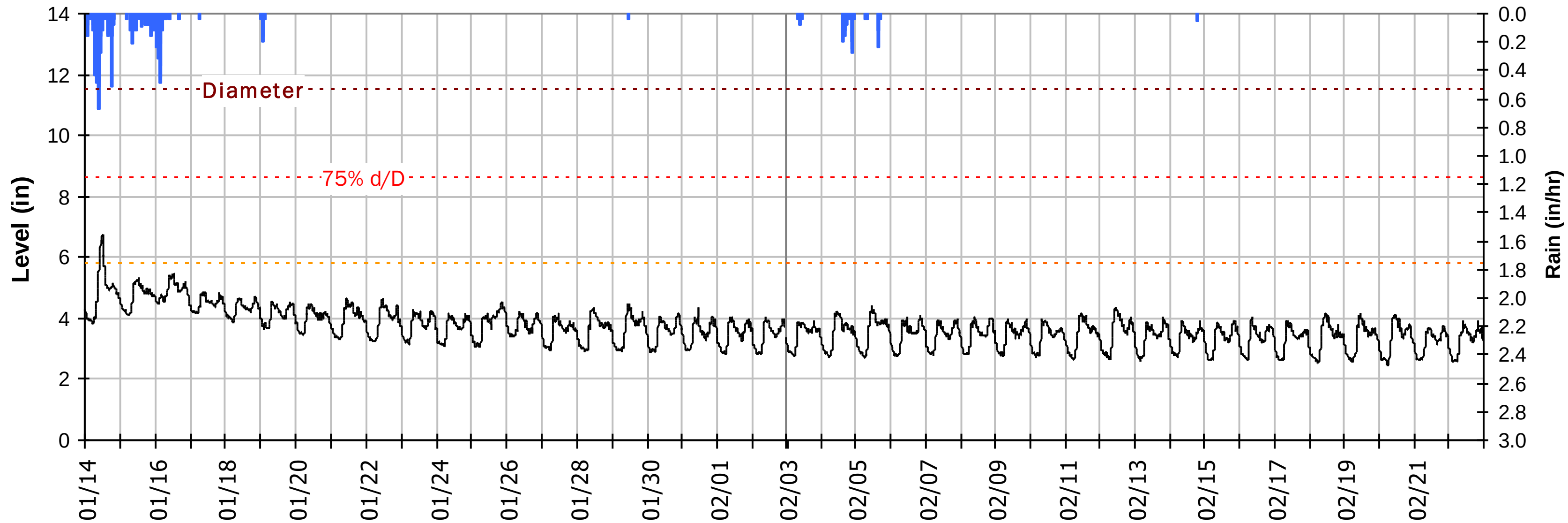


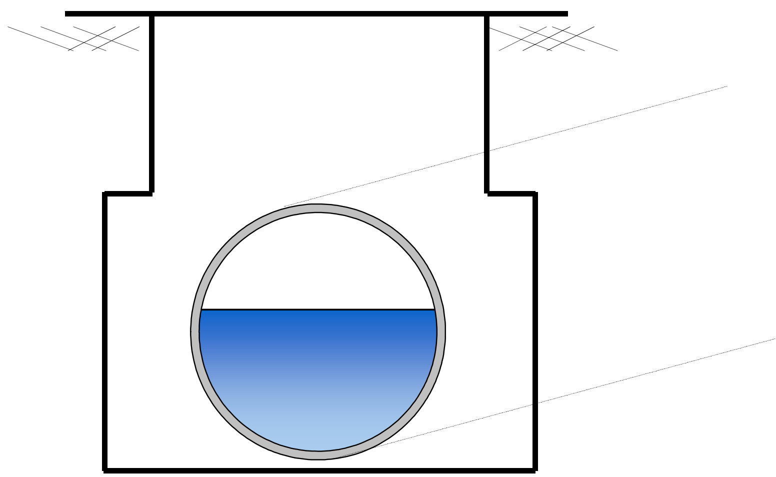
ADWF:
0.149 *mgd*



SITE 1
Site Capacity and Surcharge Summary

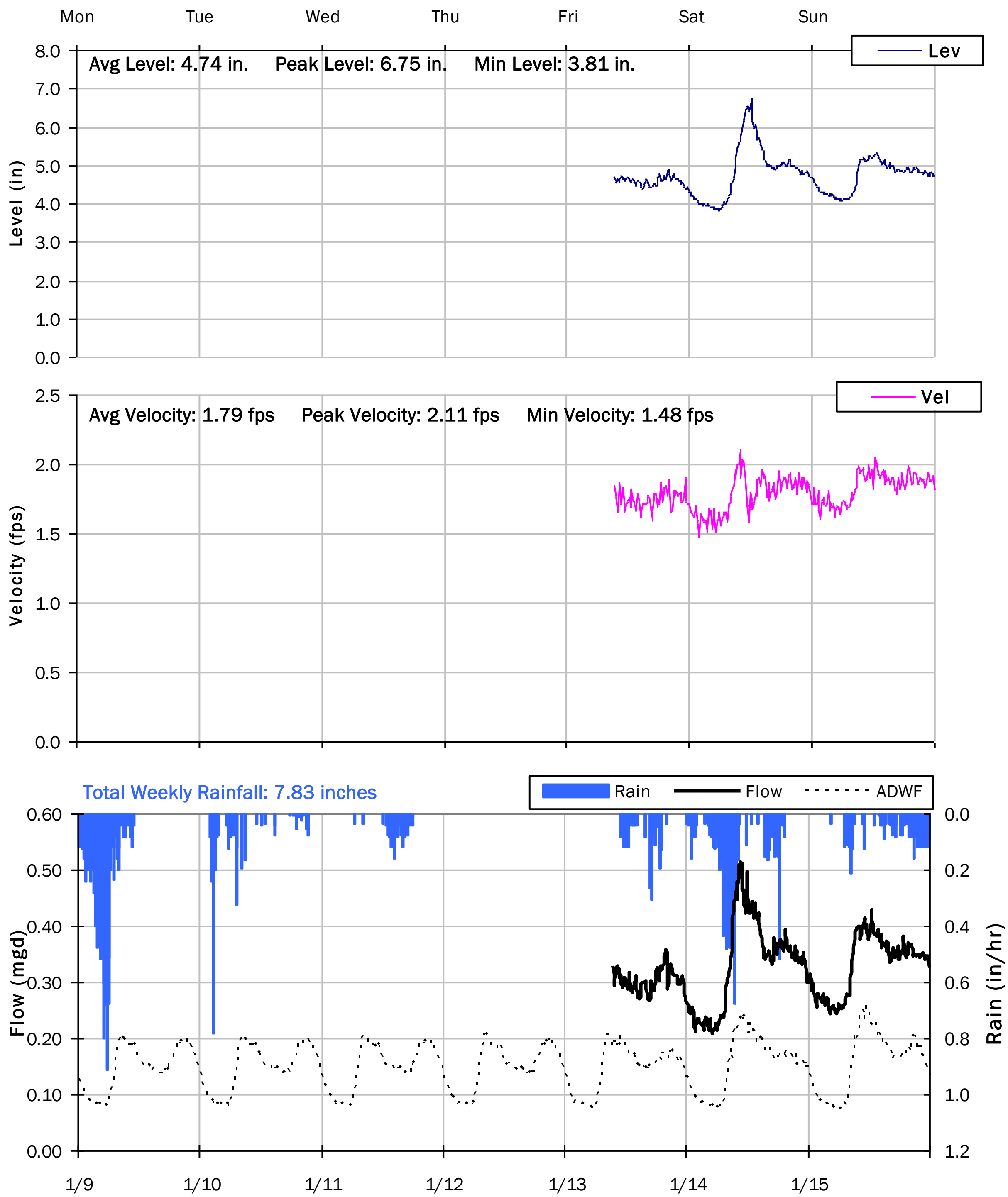
Realtime Flow Levels with Rainfall Data over Peak Level Period





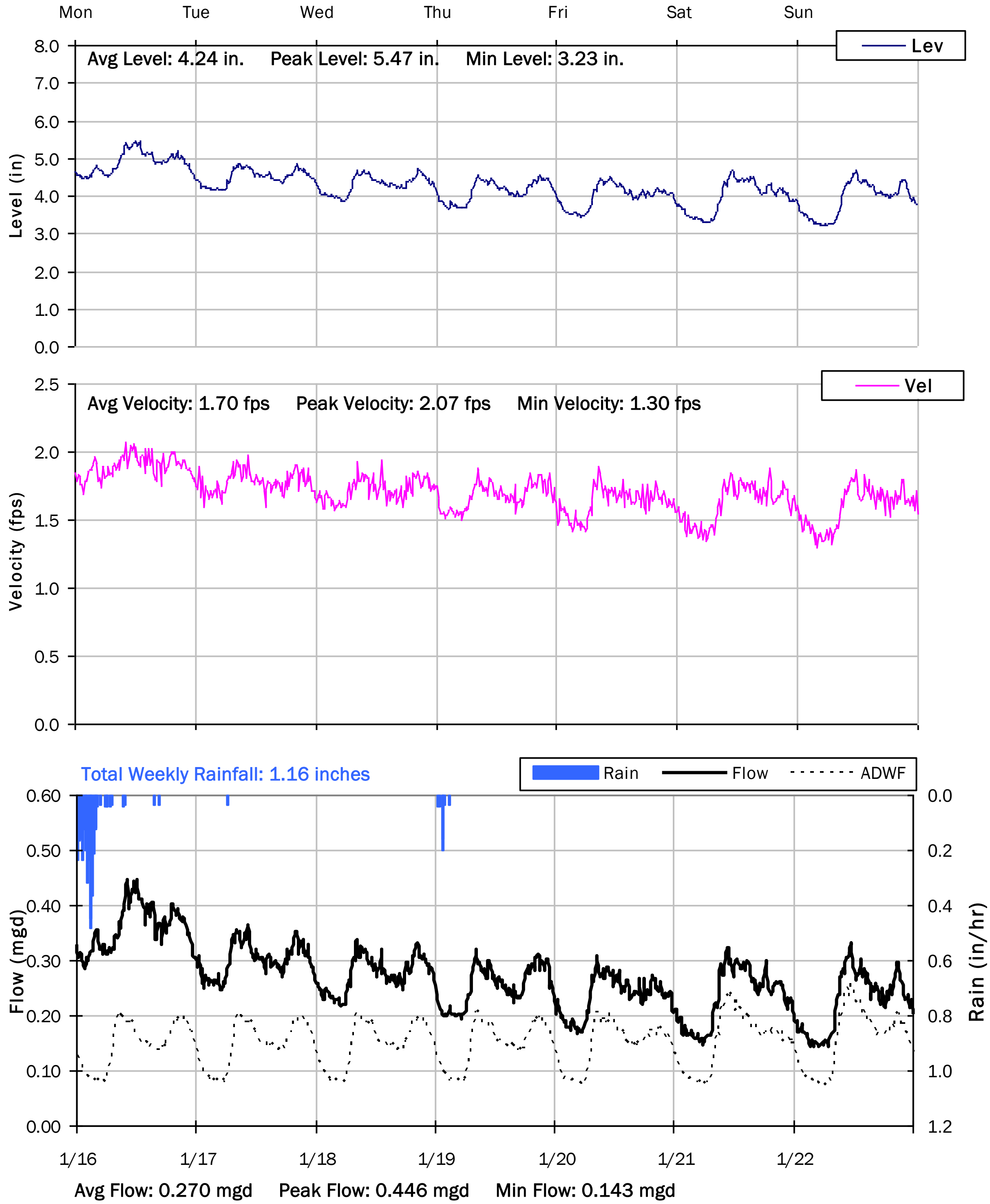
Pipe Diameter:	11.5	*inches*
Peak Measured Level:	6.75	*inches*
Peak d/D Ratio:	0.59	

SITE 1
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023

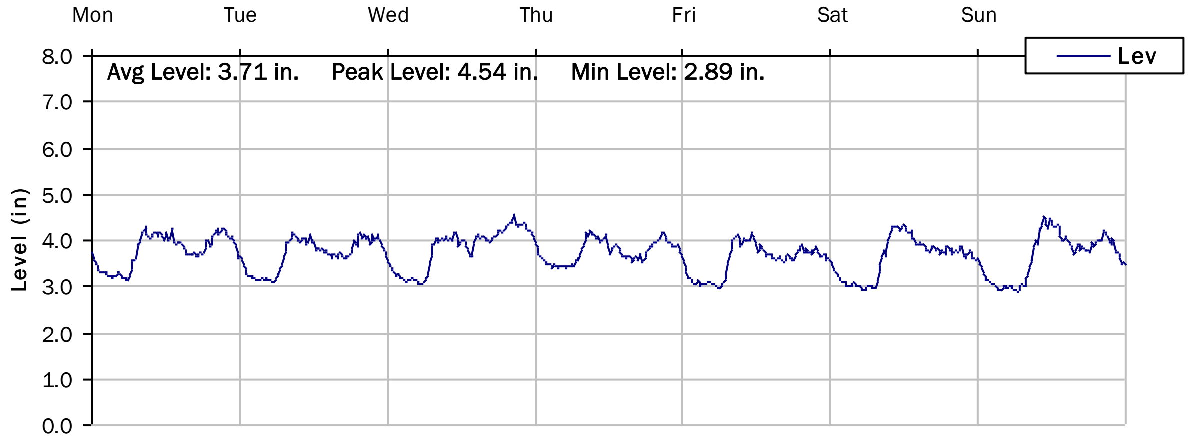


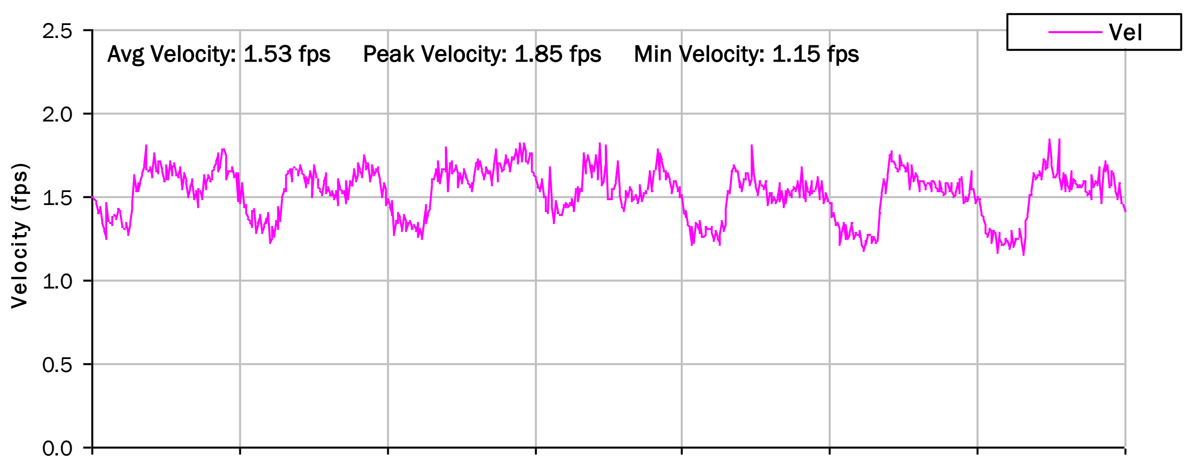
Avg Flow: 0.327 mgd   Peak Flow: 0.514 mgd   Min Flow: 0.209 mgd

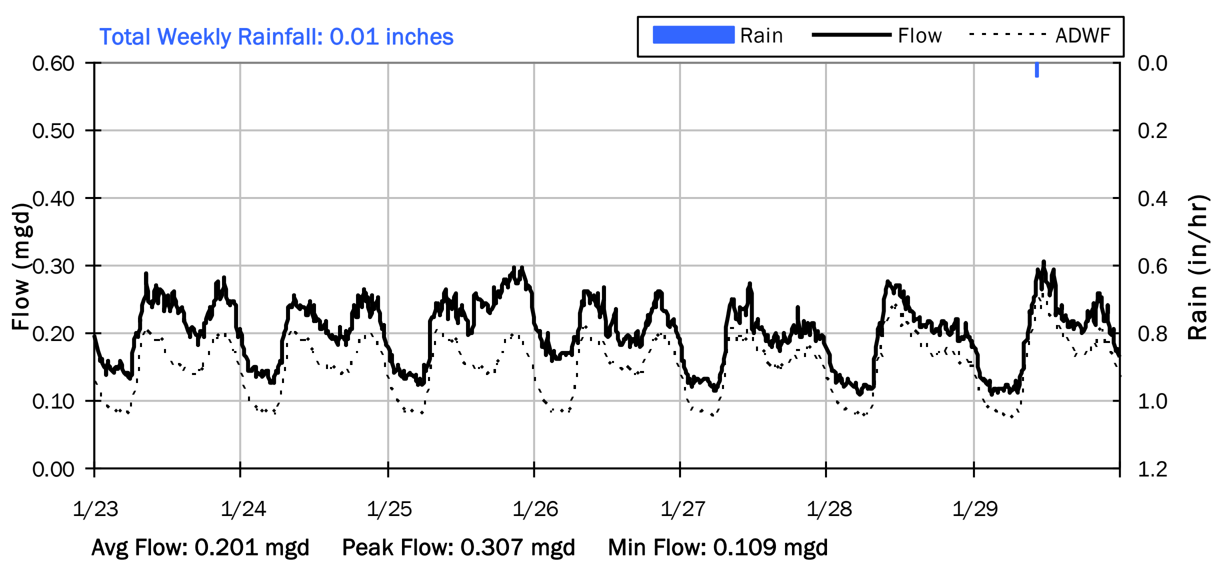
SITE 1
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023



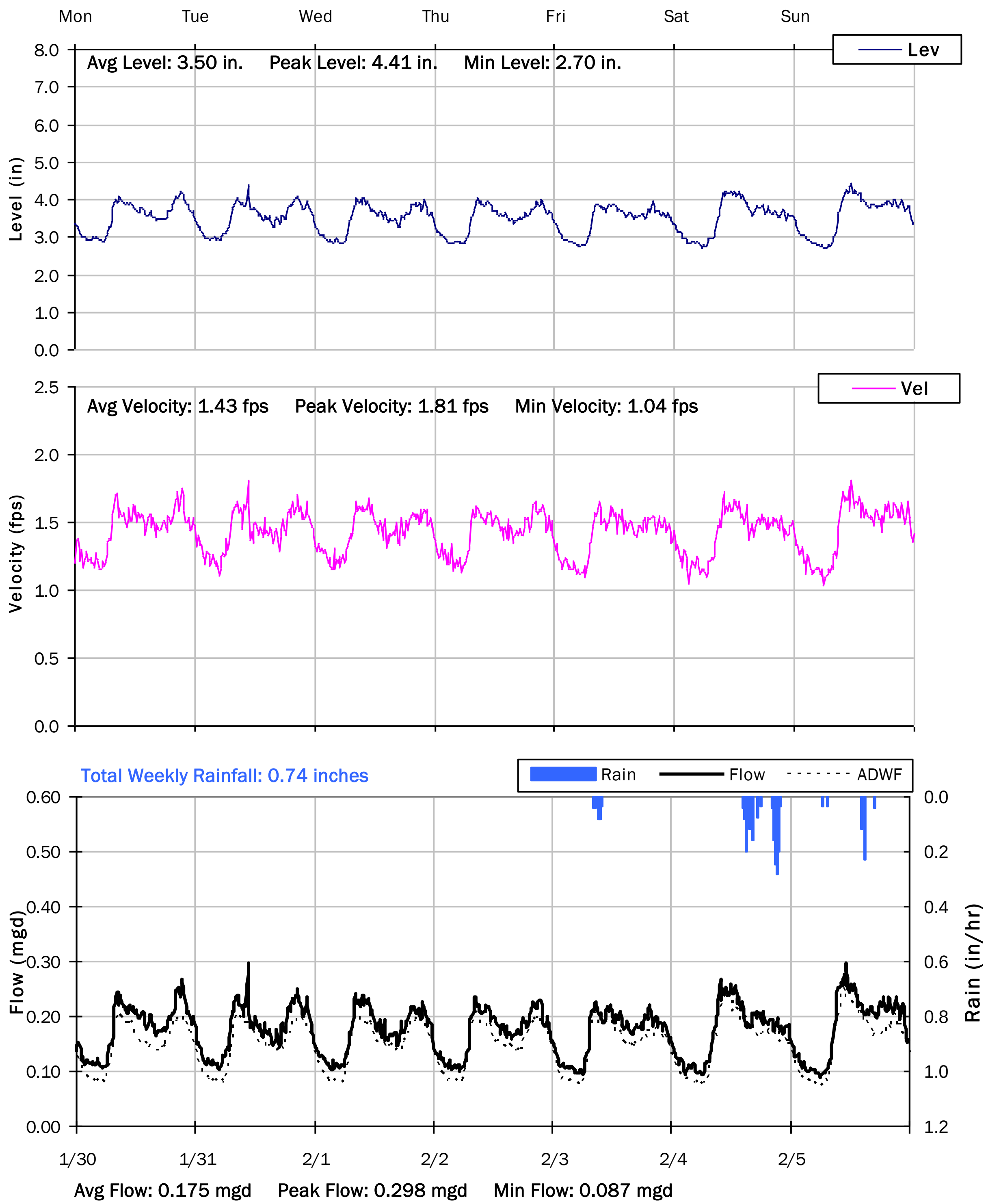
SITE 1
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023



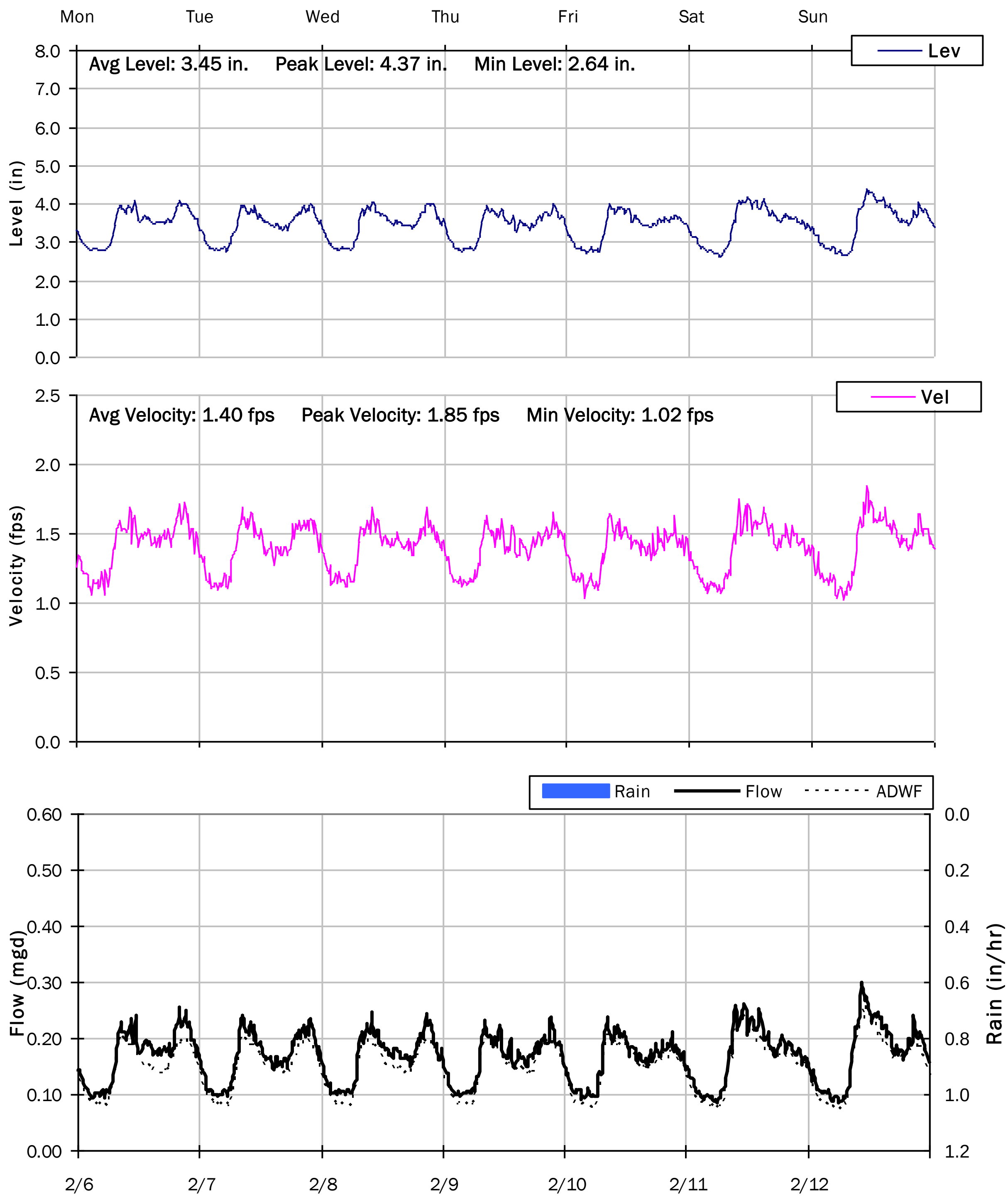




SITE 1
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023

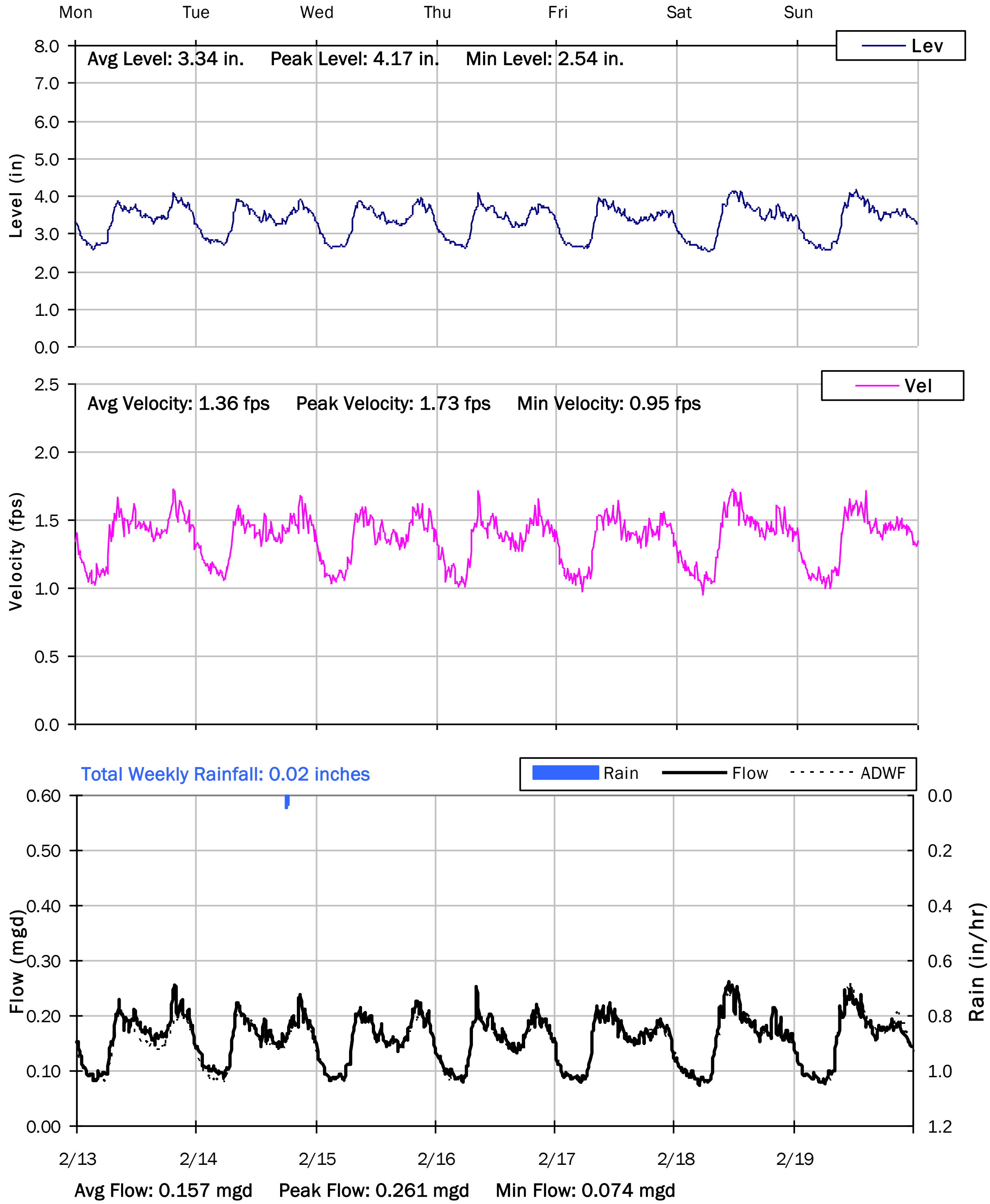


SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

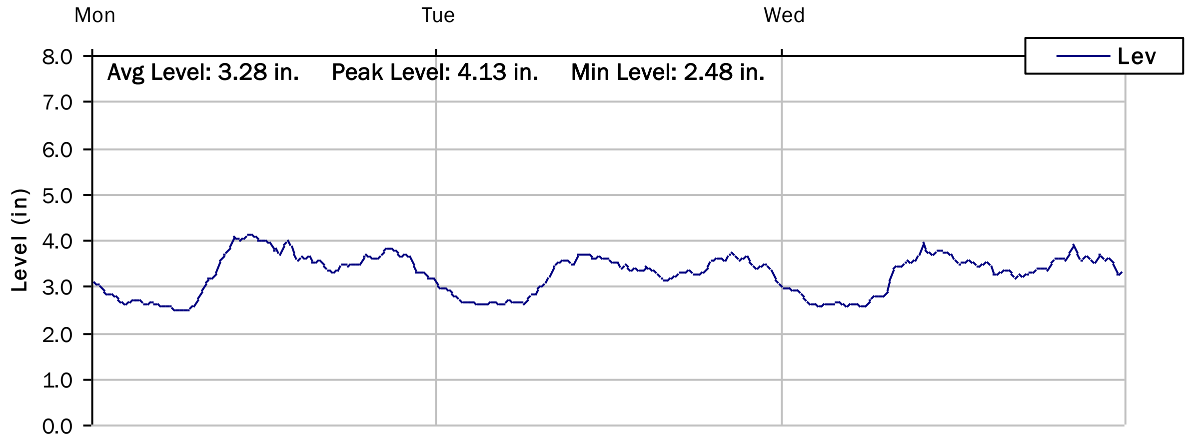


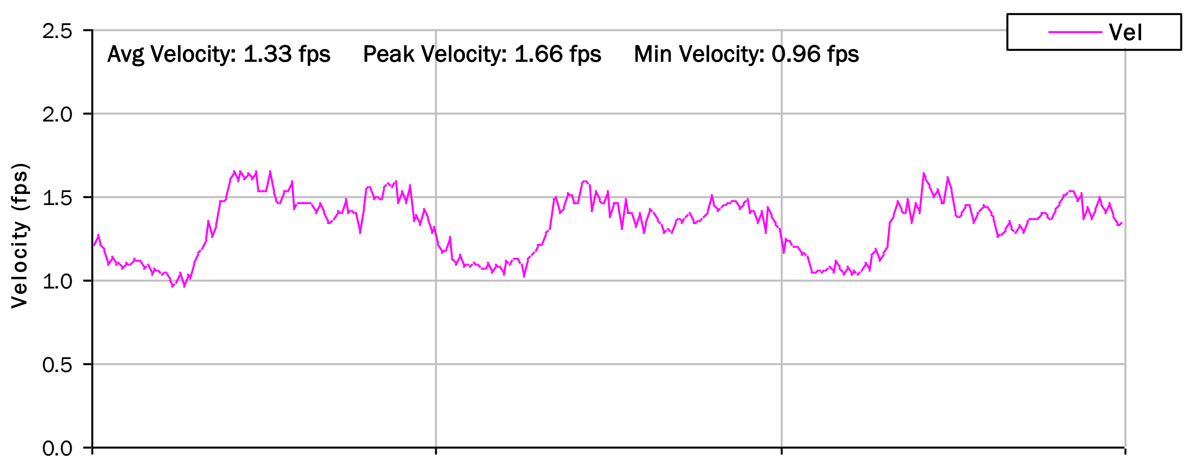
Avg Flow: 0.168 mgd Peak Flow: 0.300 mgd Min Flow: 0.084 mgd

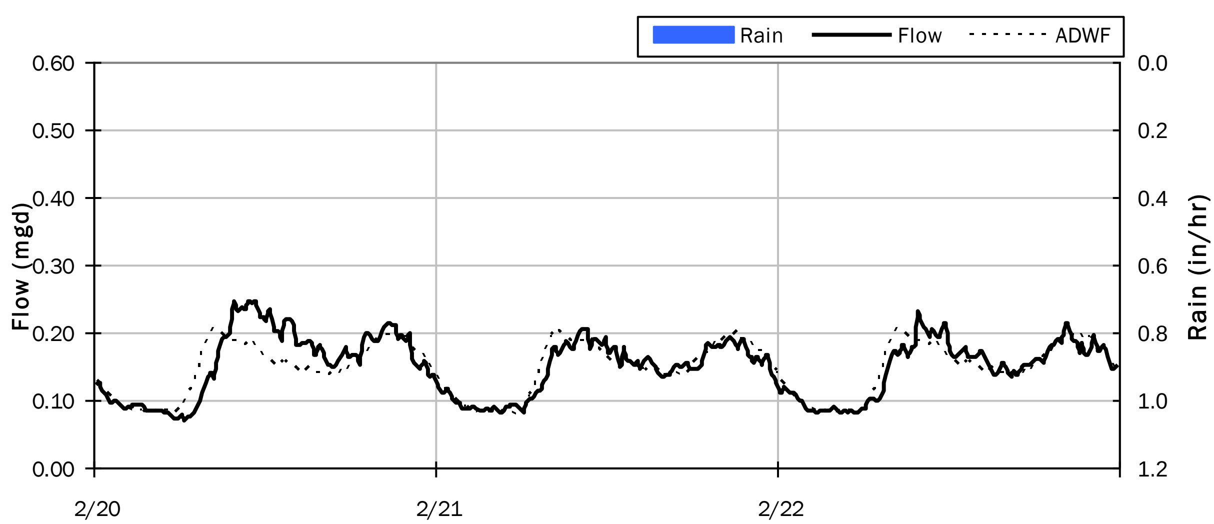
SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023







Avg Flow: 0.150 mgd Peak Flow: 0.248 mgd Min Flow: 0.071 mgd

Monitoring Site: Site 2

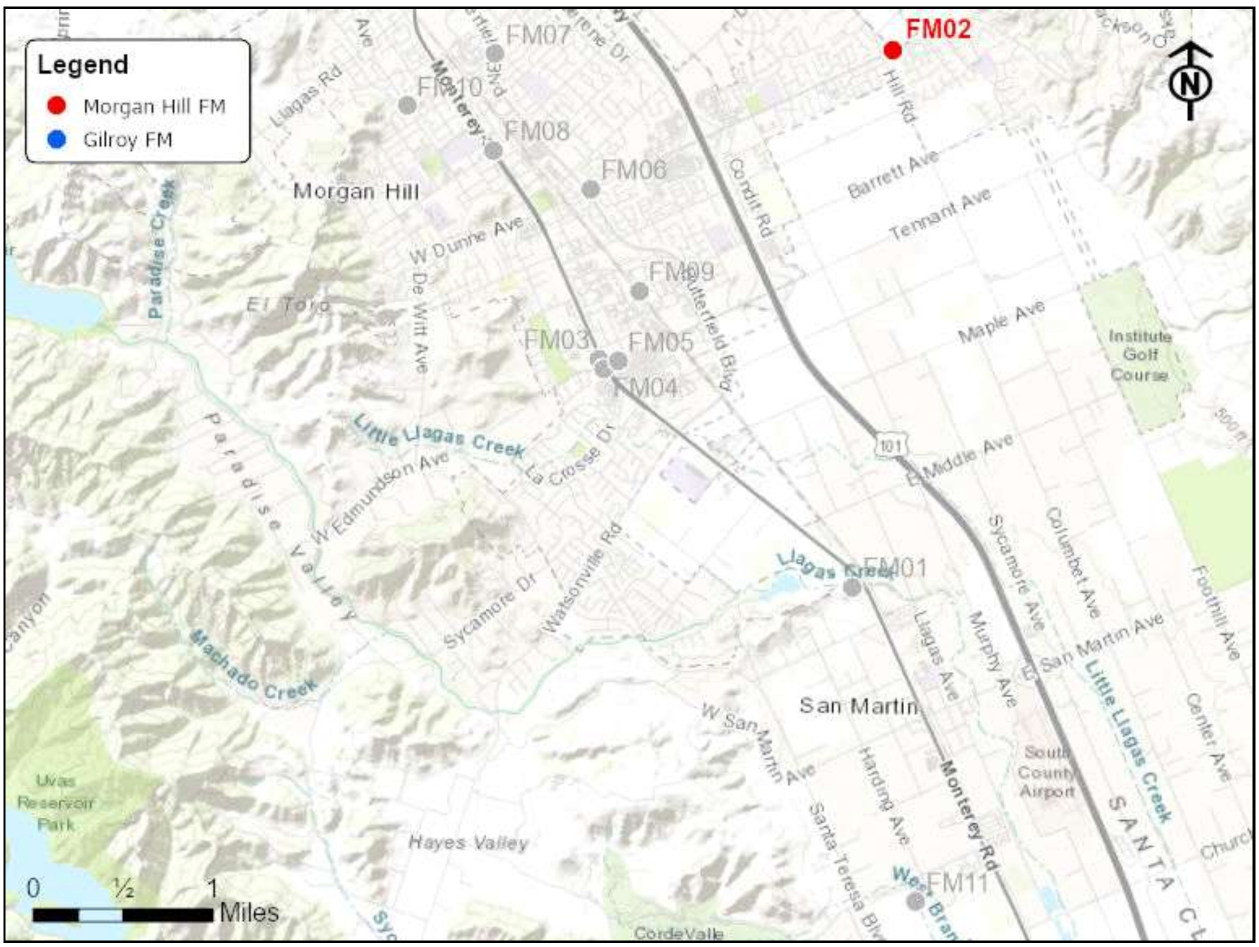
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: El Dunne Ave and Hill Rd

Data Summary Report



Vicinity Map: Site 2

SITE 2

Site Information — MH ID: F6-D.MH.074

Location: El Dunne Ave and Hill Rd

Coordinates: 121.3652° W, 37.8201° N

Rim Elevation: 371 feet

Expected Pipe Diameter: 15 inches

Measured Pipe Diameter: 15.5 inches

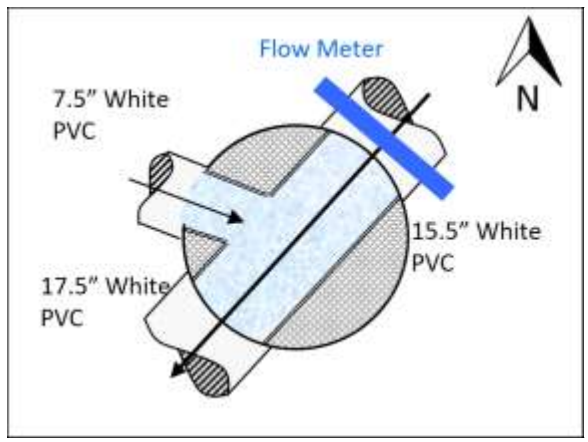
ADWF: 0.192 mgd

Peak Measured Flow: 0.73 mgd

Sediment: None

Satellite Map

Sanitary Map



Flow Sketch

Street View

Plan View

SITE 2

Additional Site Photos

Southwest Effluent Pipe

Monitored Southeast Influent Pipe

SITE 2

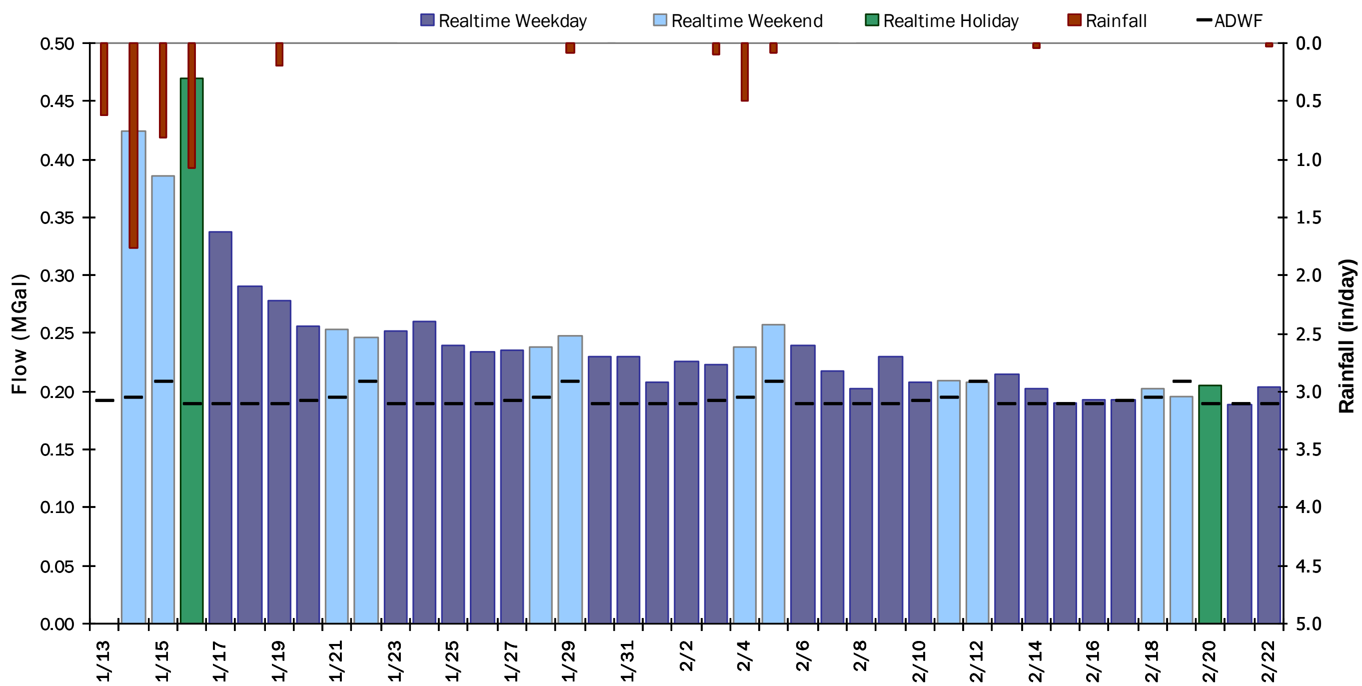
Additional Site Photos

Northeast Upper Influent Pipe

SITE 2
Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.245 MGal Peak Daily Flow: 0.470 MGal Min Daily Flow: 0.188 MGal

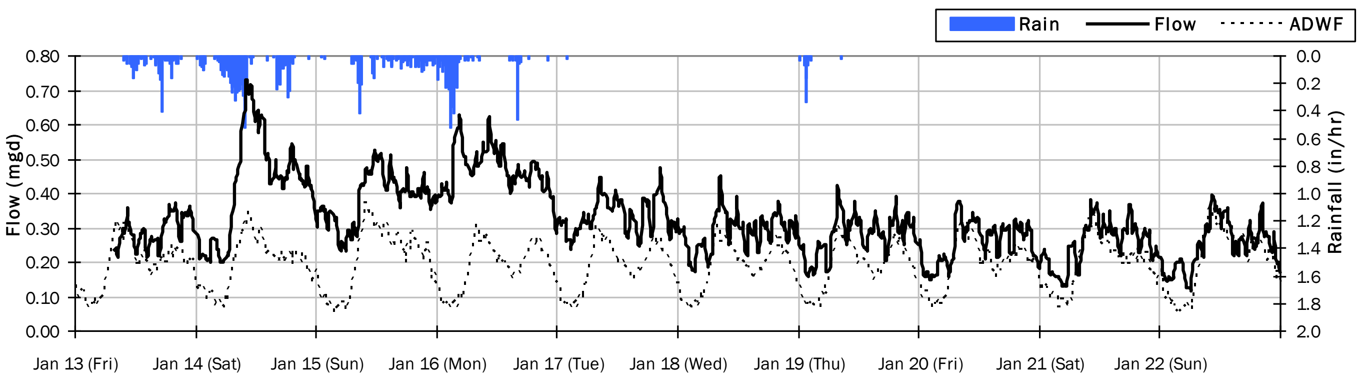
Total Rainfall: 5.29 inches

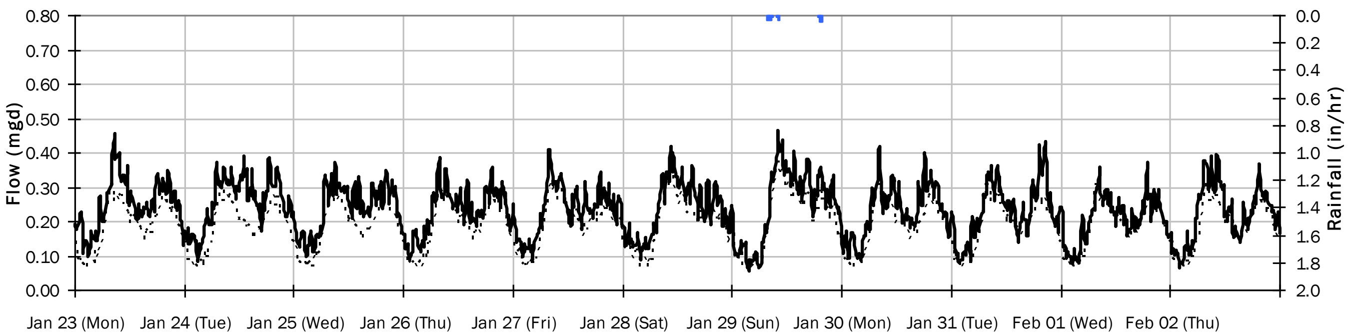


SITE 2

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 4.54 inches   Period Avg Flow: 0.278 mgd   Period Peak Flow: 0.729 mgd   Period Min Flow: 0.058 mgd

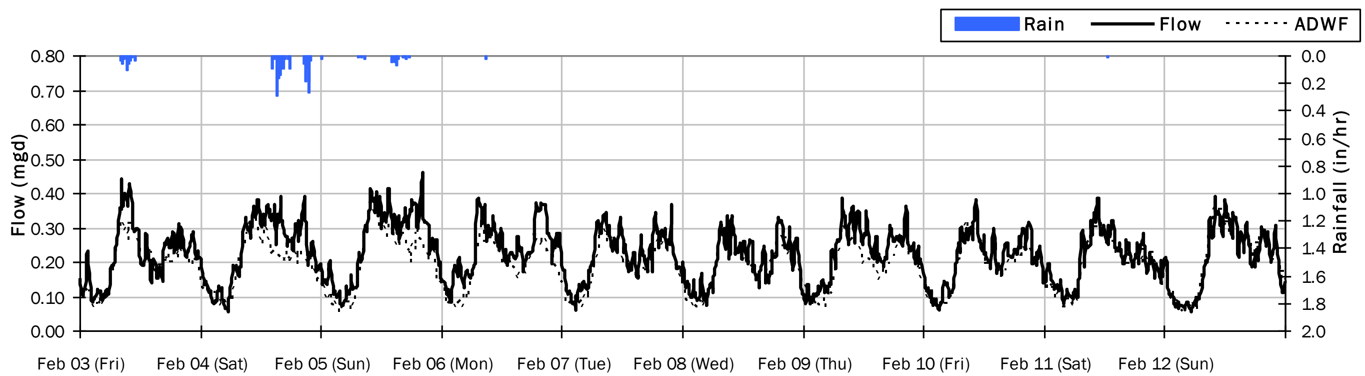


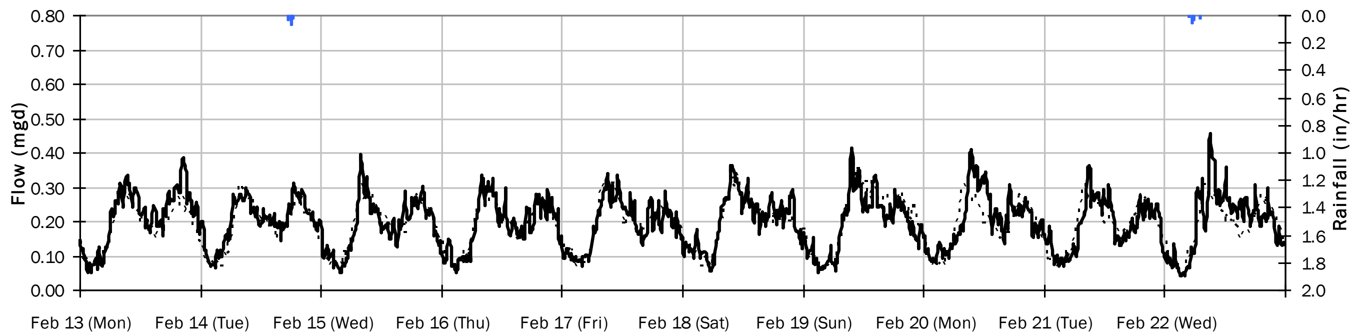


SITE 2

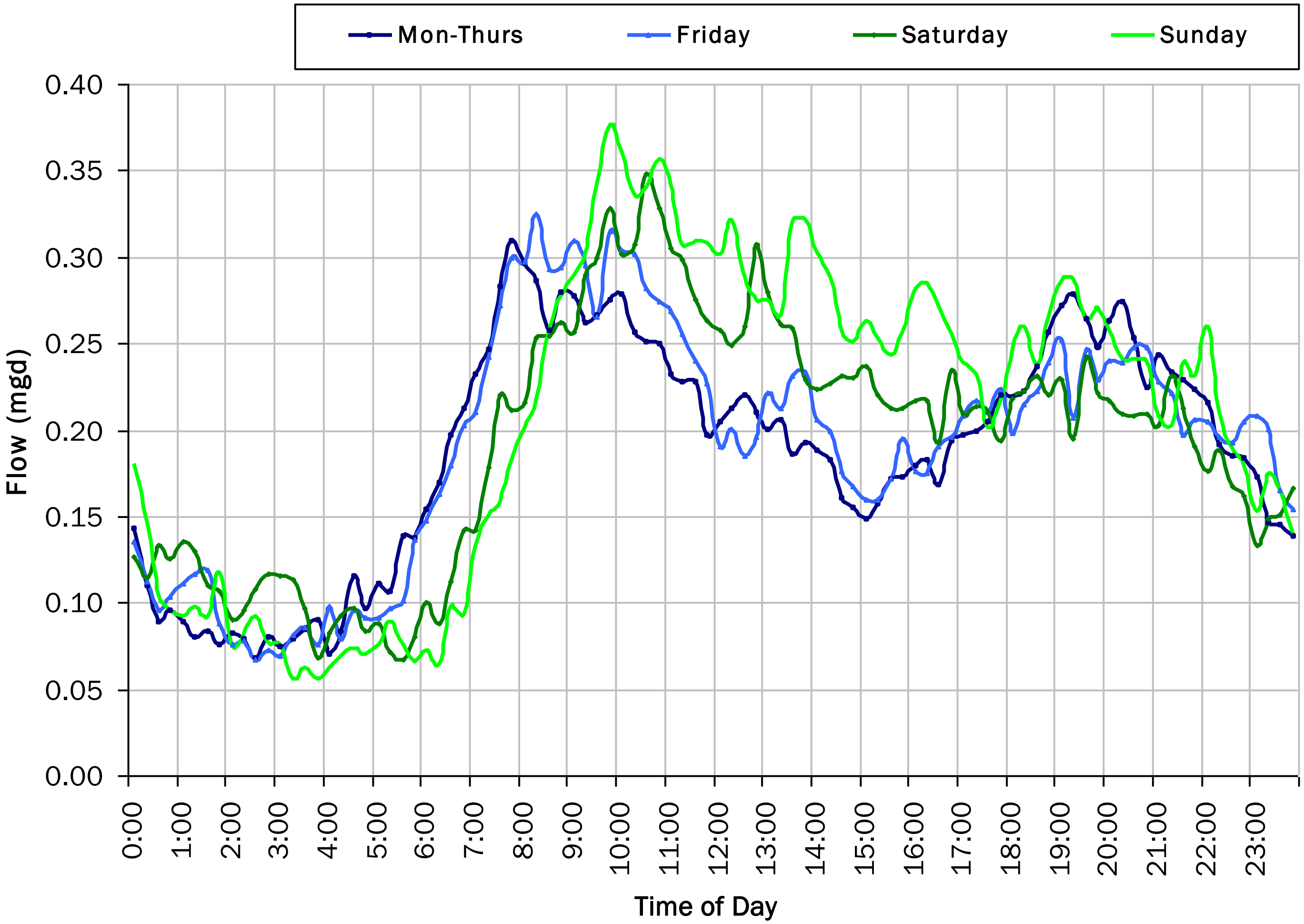
Flow Summary: 2/3/2023 to 2/22/2023

Period Rainfall: 0.75 inches   Period Avg Flow: 0.211 mgd   Period Peak Flow: 0.456 mgd   Period Min Flow: 0.040 mgd

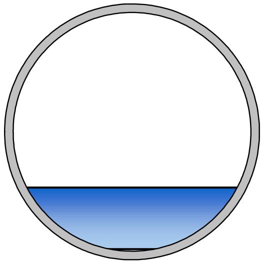




SITE 2
Average Dry Weather Flow Hydrographs

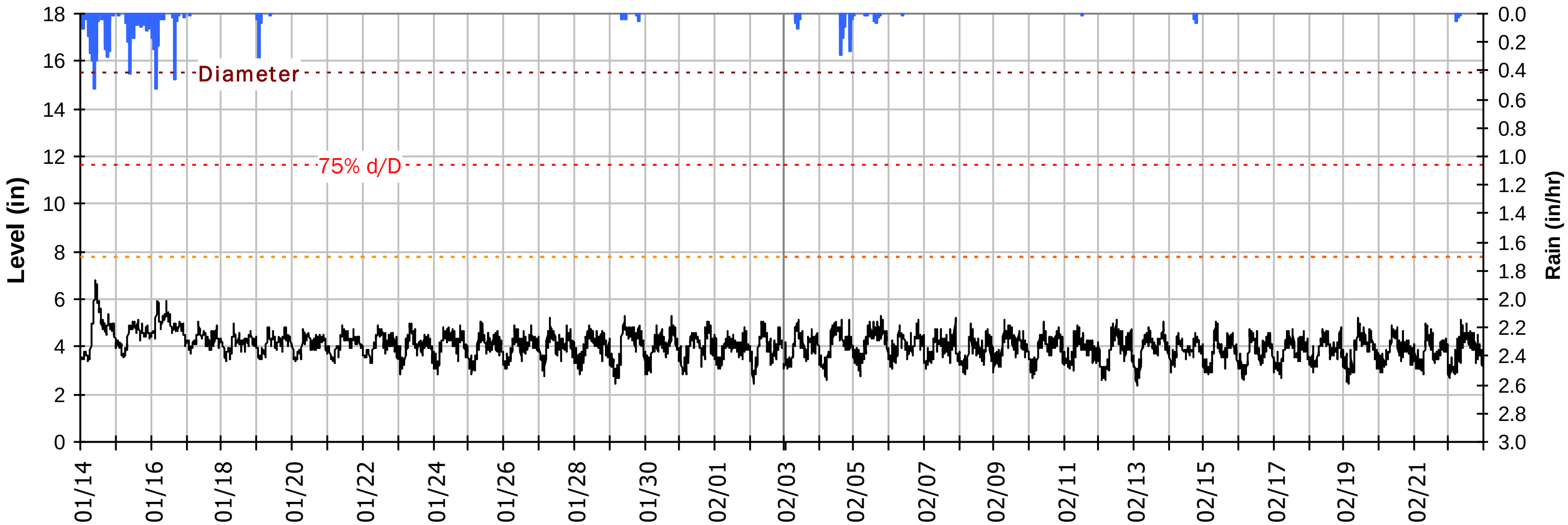


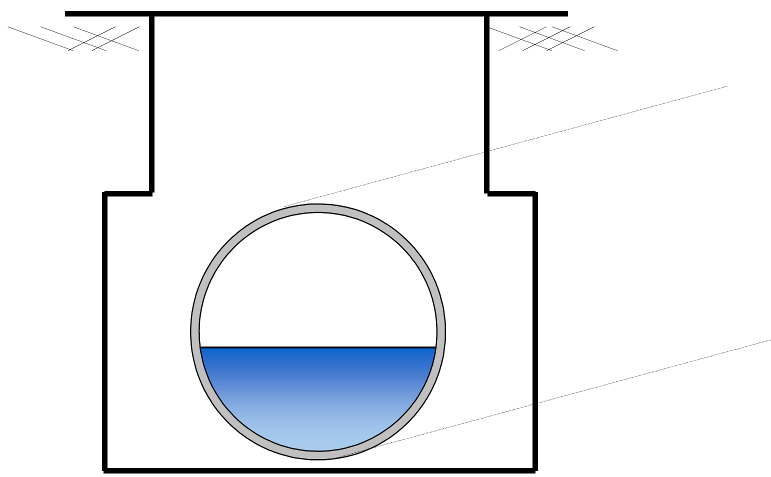
ADWF:
0.192 *mgd*



SITE 2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Peak Level Period



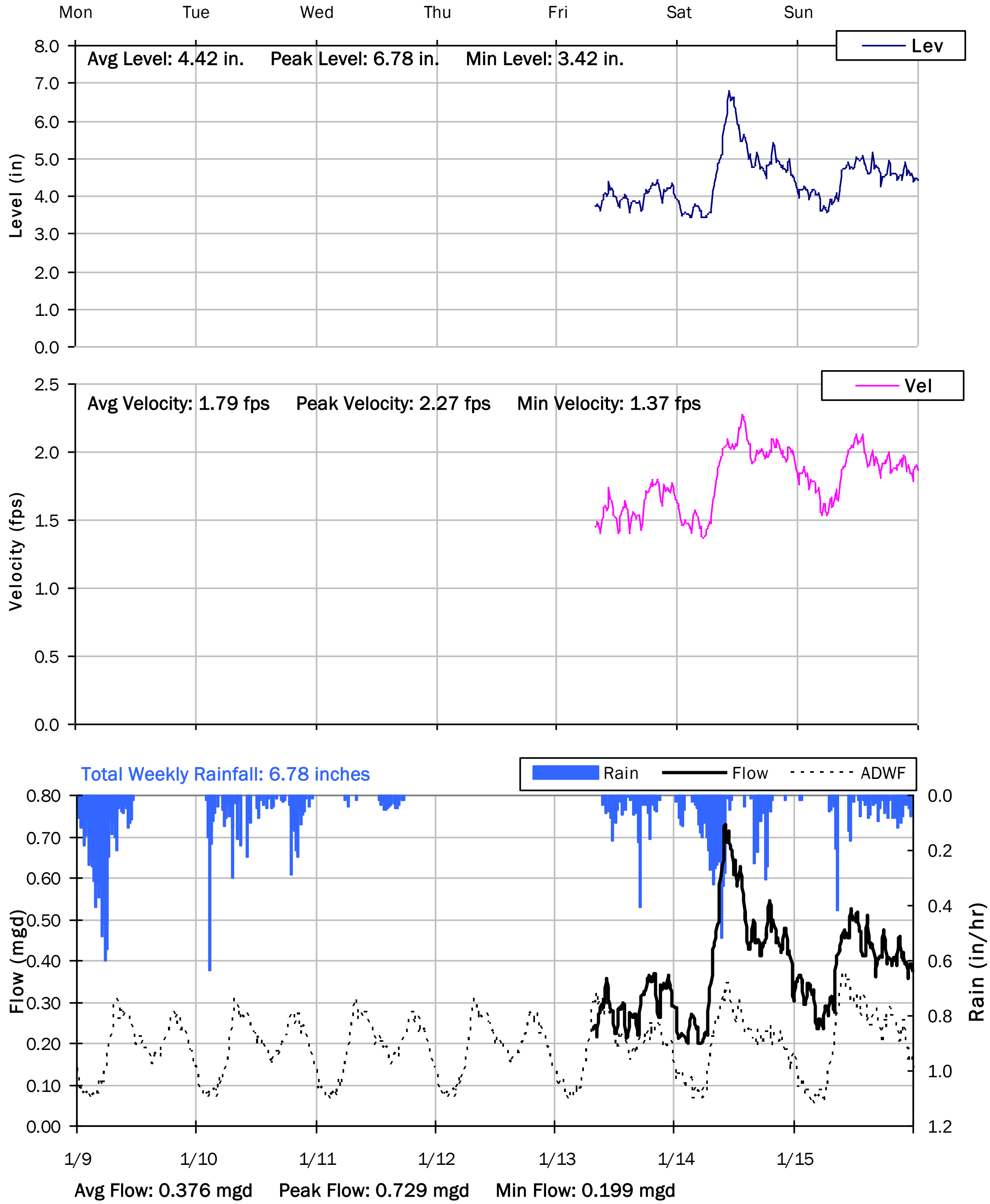


Pipe Diameter: 15.5 *inches*

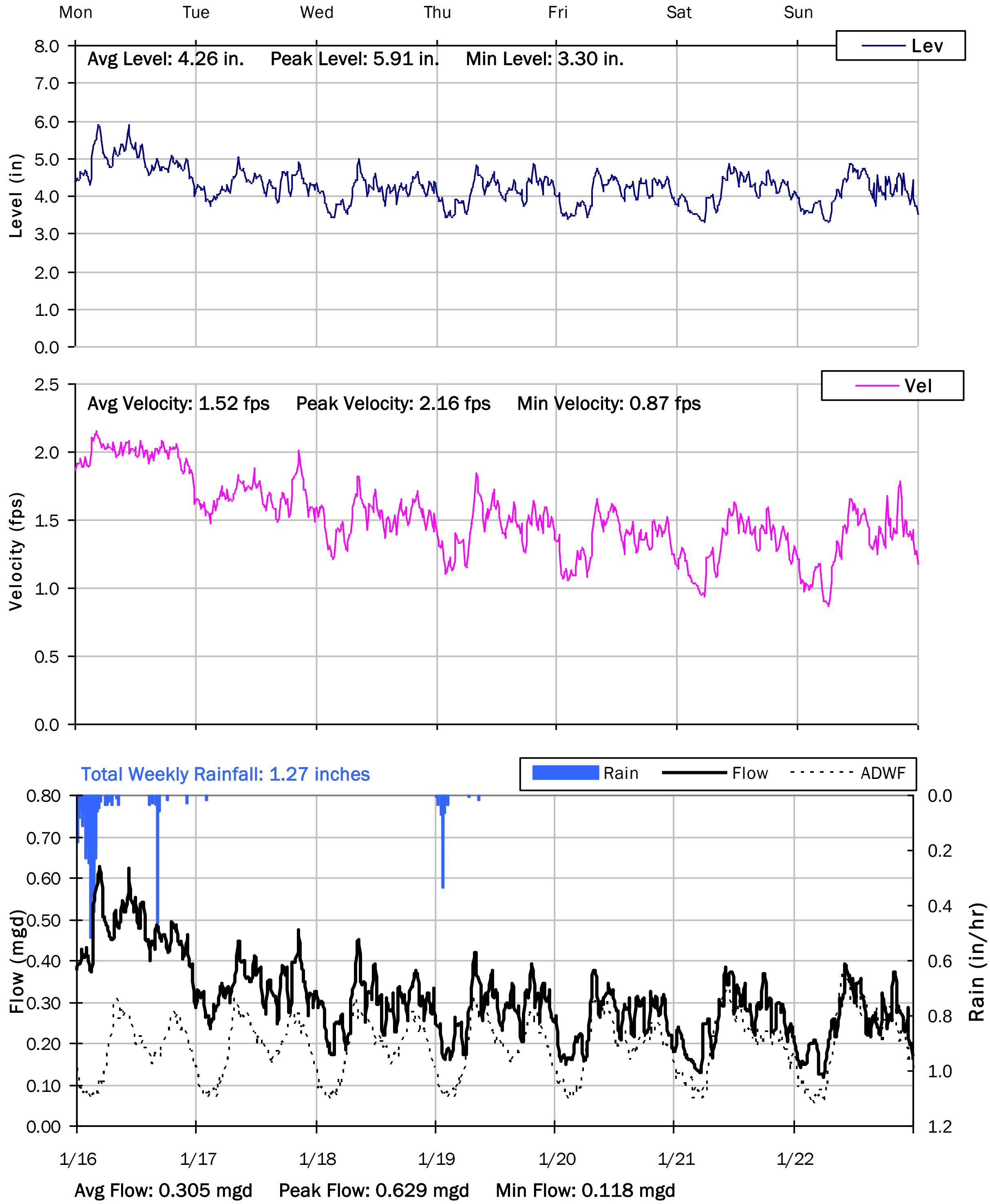
Peak Measured Level: 6.78 *inches*

Peak d/D Ratio: 0.44

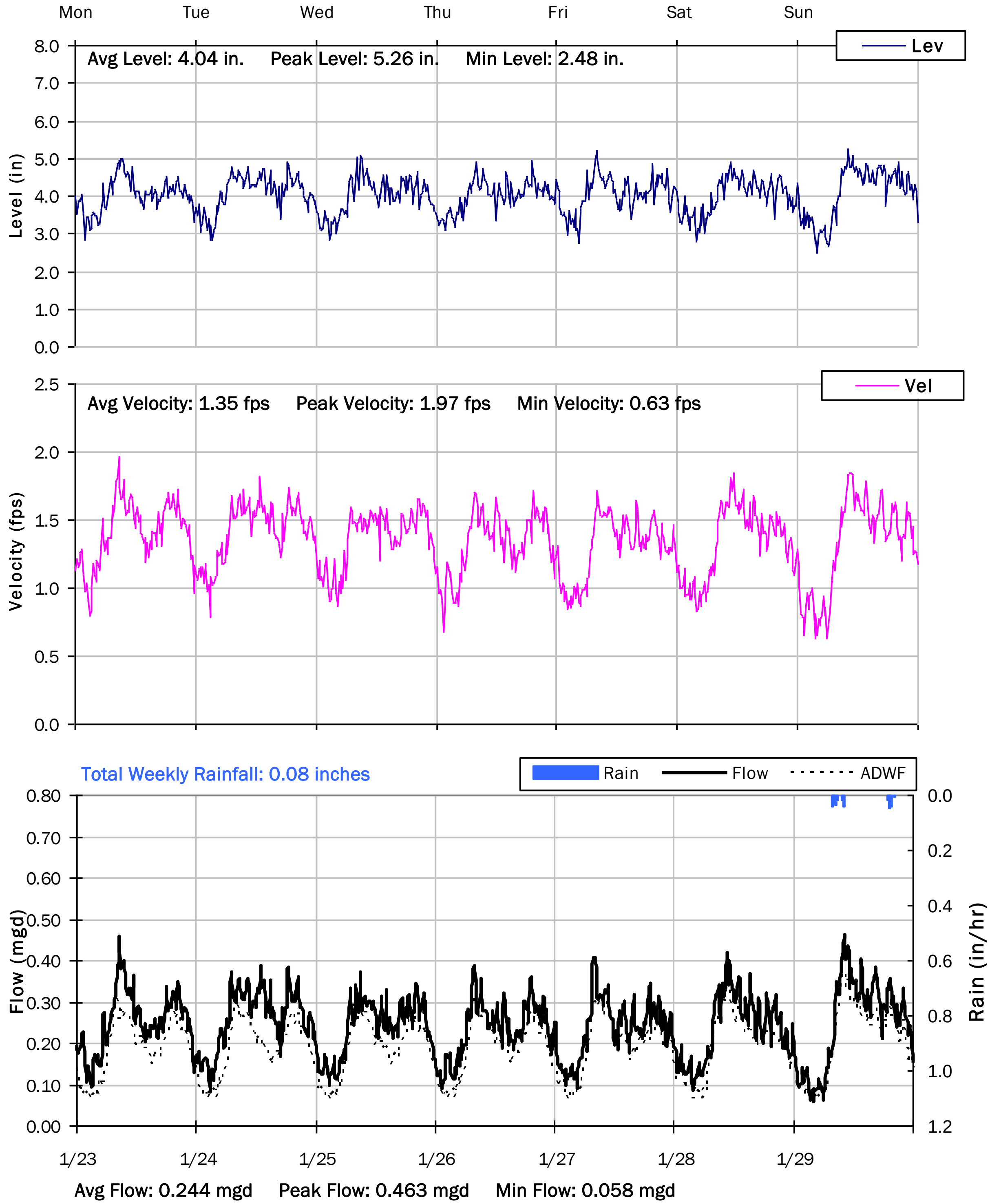
SITE 2
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023



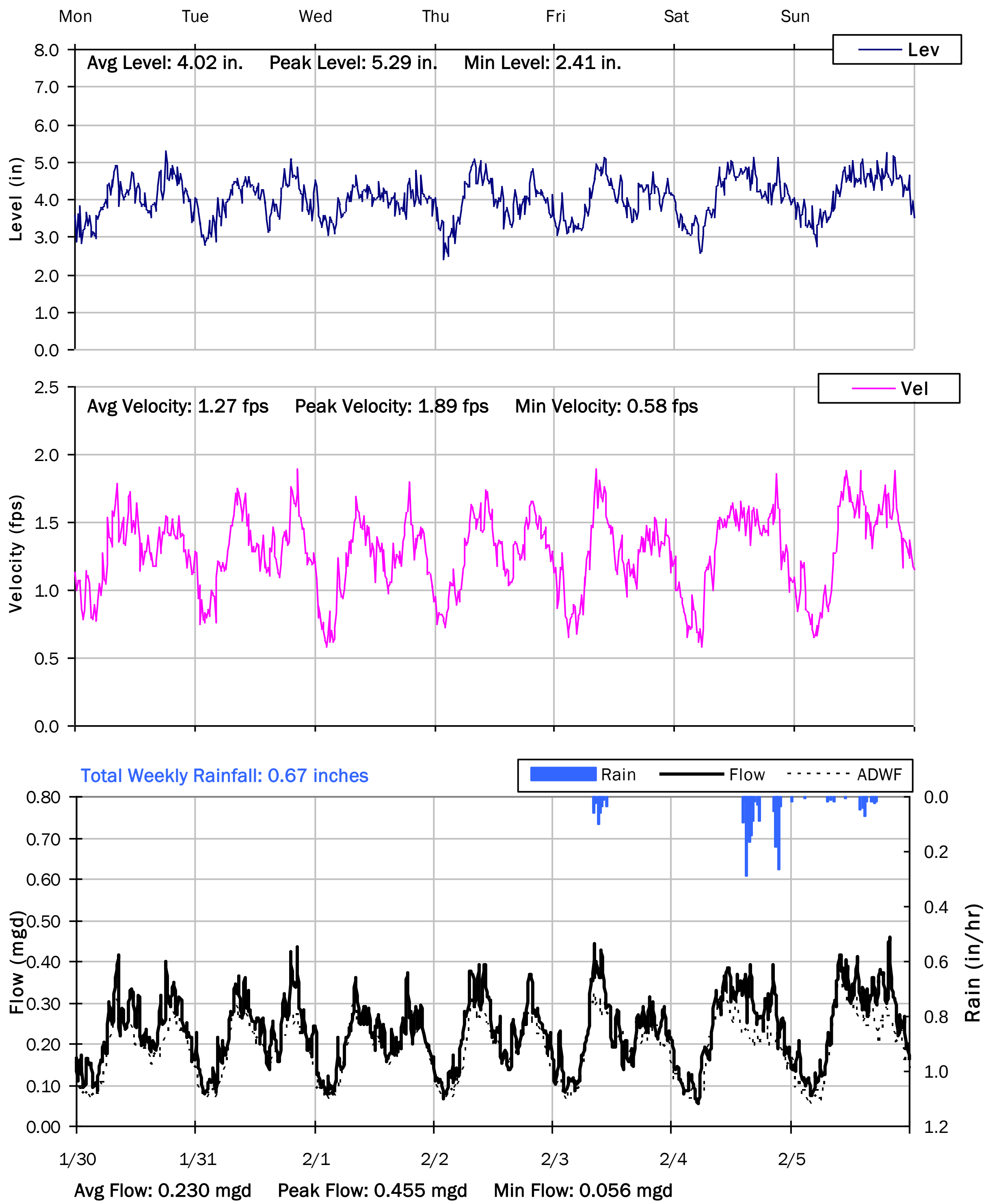
SITE 2
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023



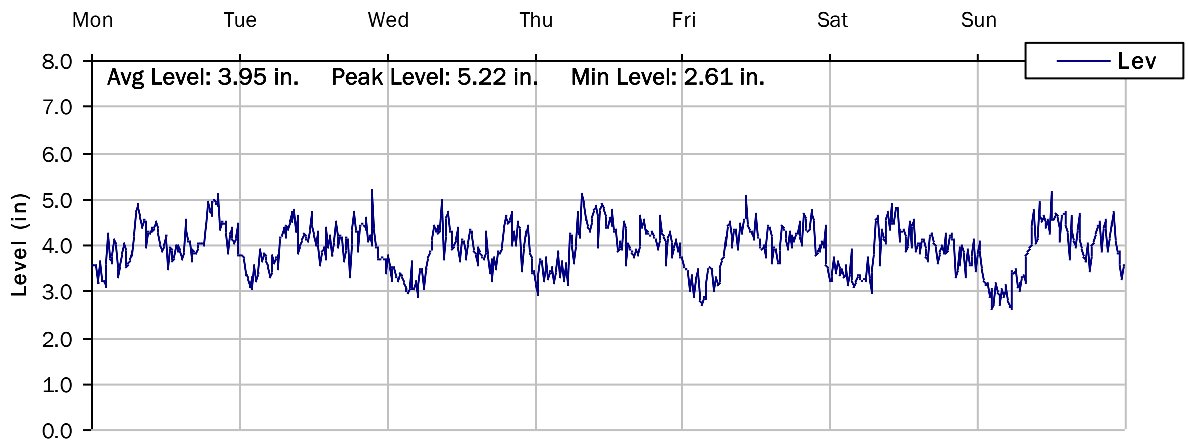
SITE 2
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023

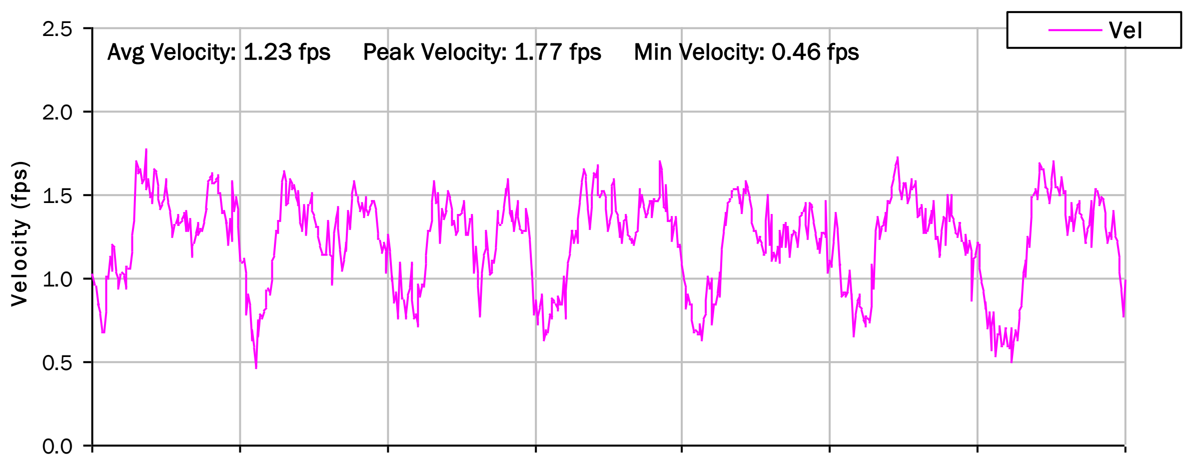


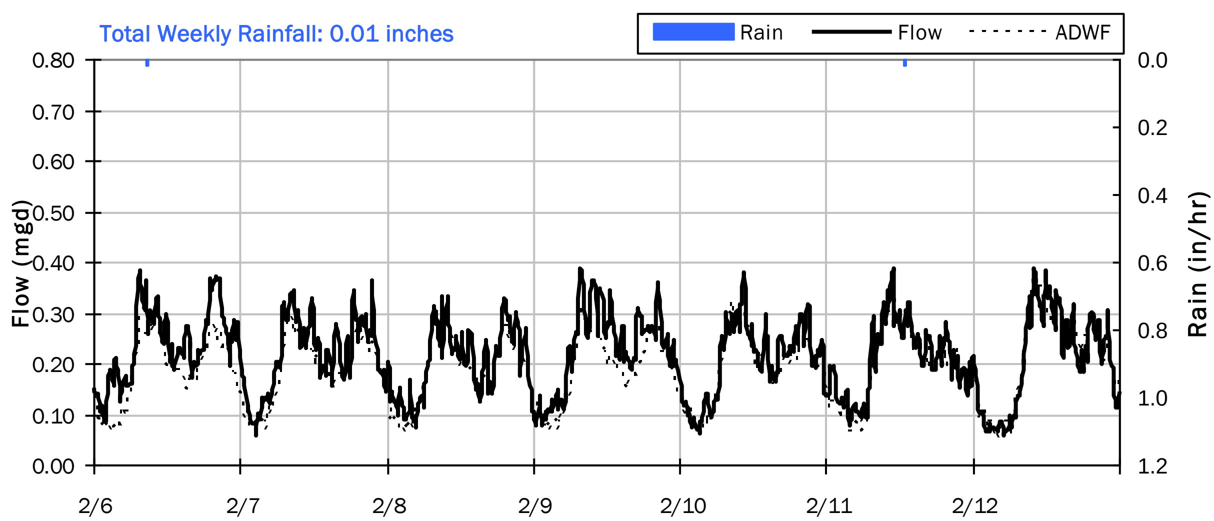
SITE 2
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023



SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

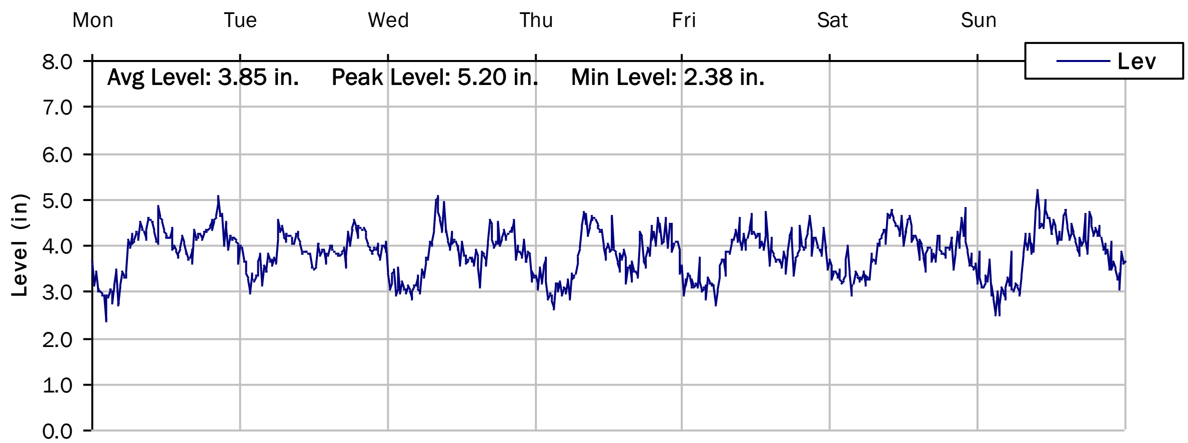


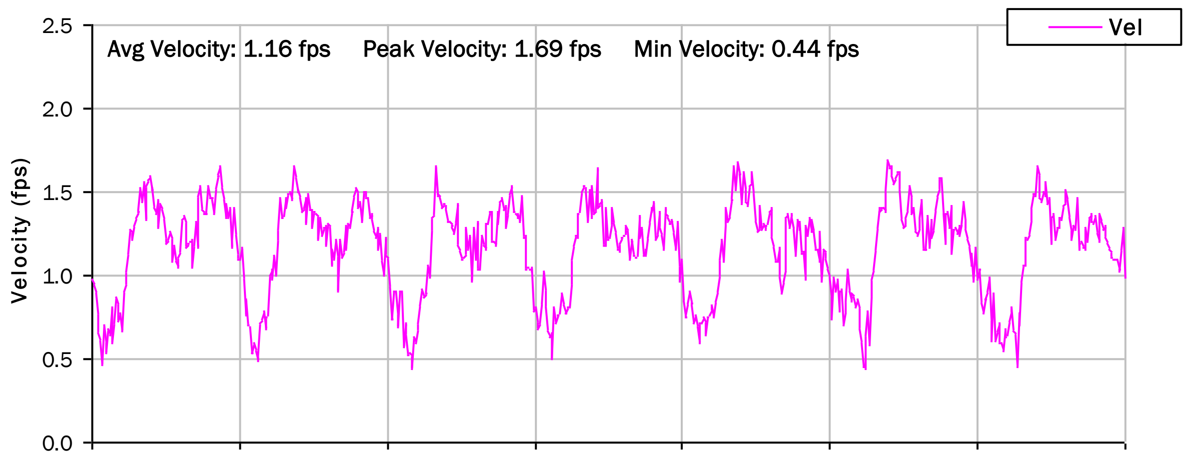


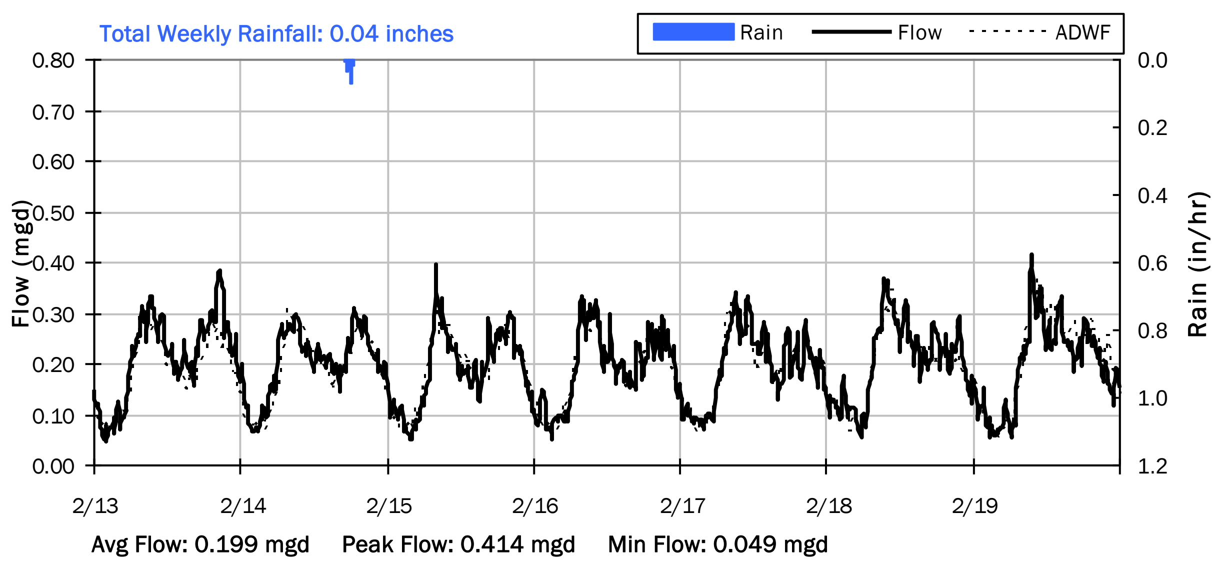


Avg Flow: 0.217 mgd Peak Flow: 0.387 mgd Min Flow: 0.057 mgd

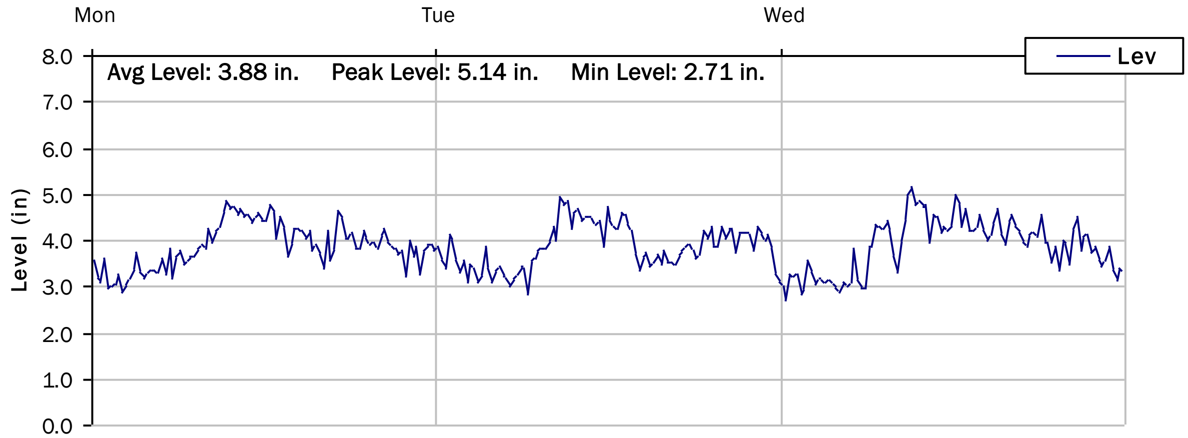
SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023

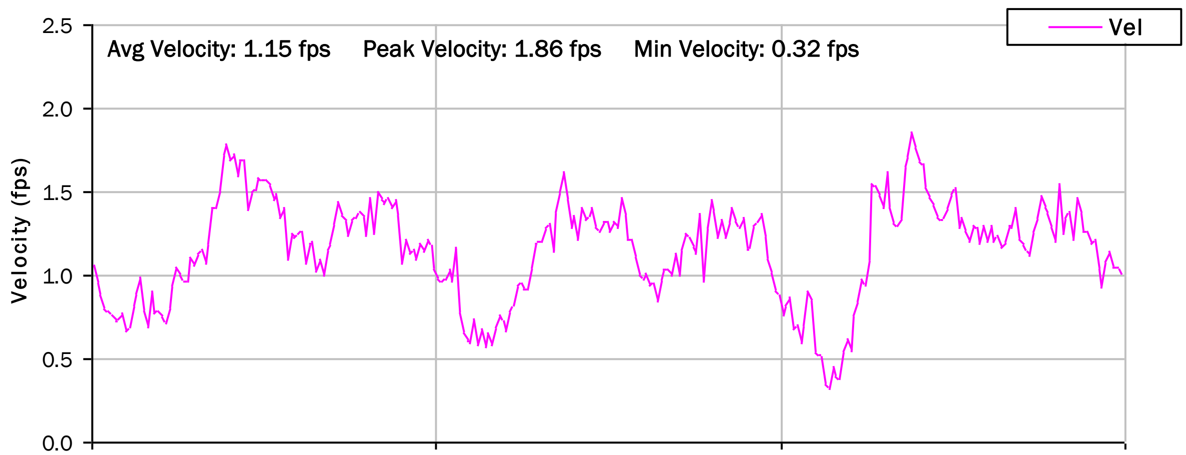


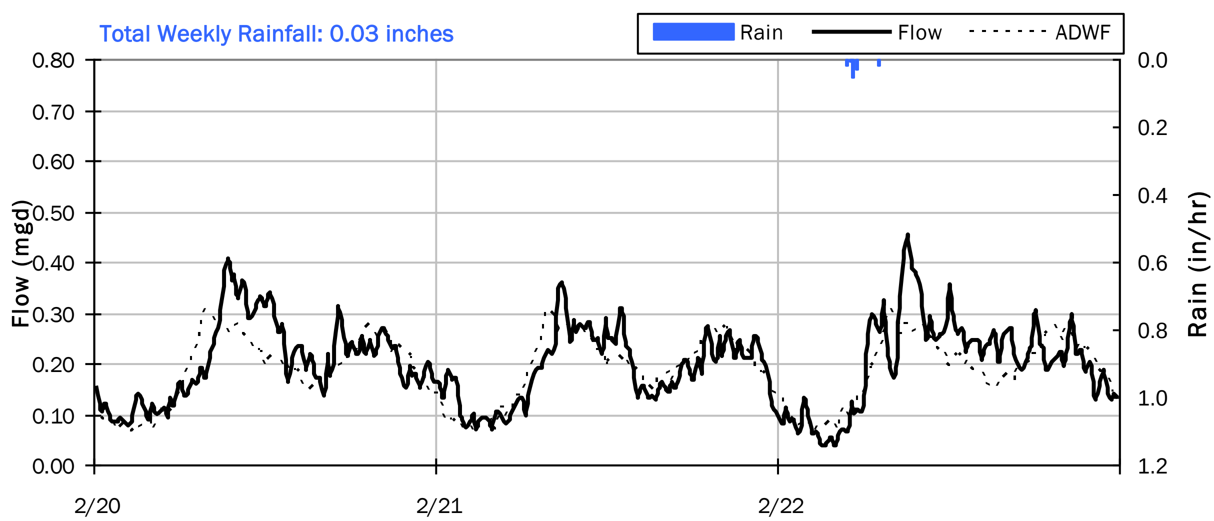




SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023







Avg Flow: 0.199 mgd   Peak Flow: 0.456 mgd   Min Flow: 0.040 mgd

Monitoring Site: Site 3

City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: 16099 Monterey Hwy

Data Summary Report

Vicinity Map: Site 3

SITE 3

Site Information MH ID: I5-A.MH.034

Location: 16099 Monterey Hwy

Coordinates: 121.3837° W, 37.6492° N

Rim Elevation: 333 feet

Expected Pipe Diameter: 21 inches

Measured Pipe Diameter: 21 inches

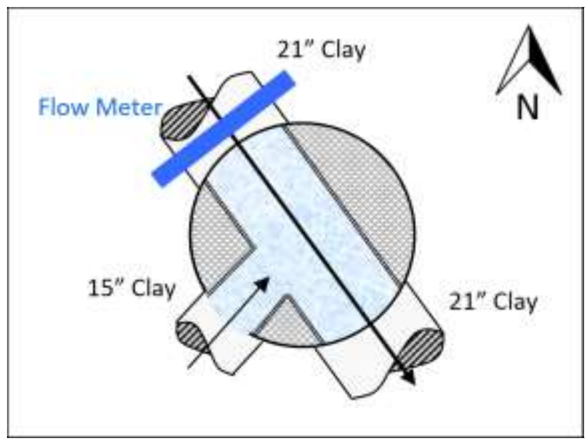
ADWF: 0.080 mgd

Peak Measured Flow: 0.97 mgd

Sediment: None

Satellite Map

Sanitary Map



Flow Sketch

Street View

Plan View

SITE 3

Additional Site Photos

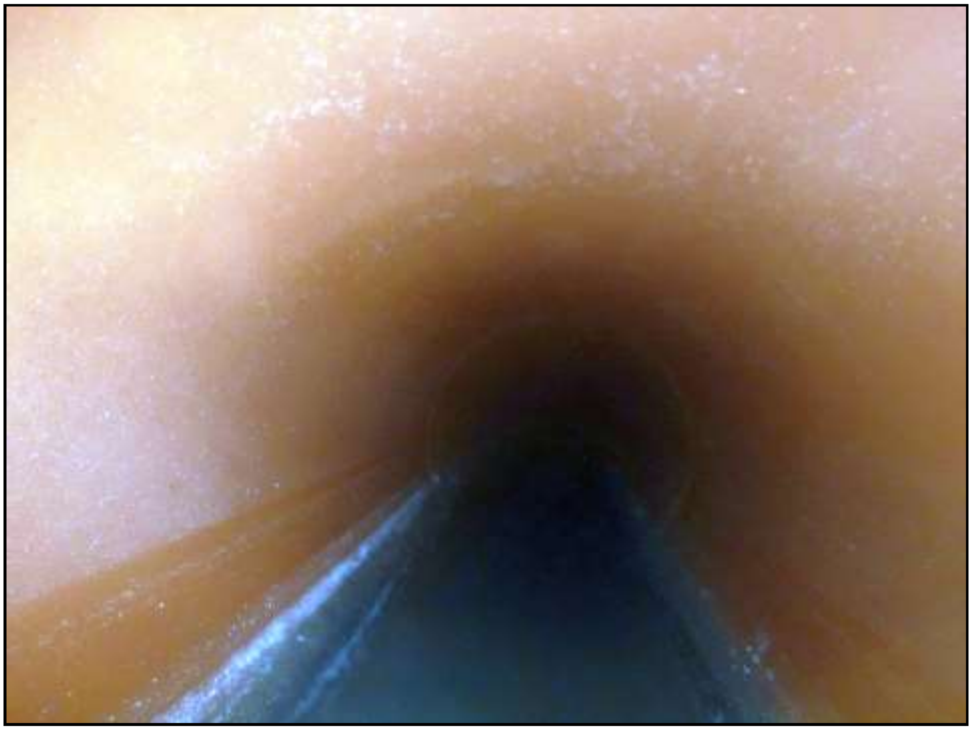
Southeast Effluent Pipe

Southeast Influent Pipe

SITE 3

Additional Site Photos

Monitored Northwest Influent Pipe



SITE 3

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.103 MGal   Peak Daily Flow: 0.409 MGal   Min Daily Flow: 0.036 MGal

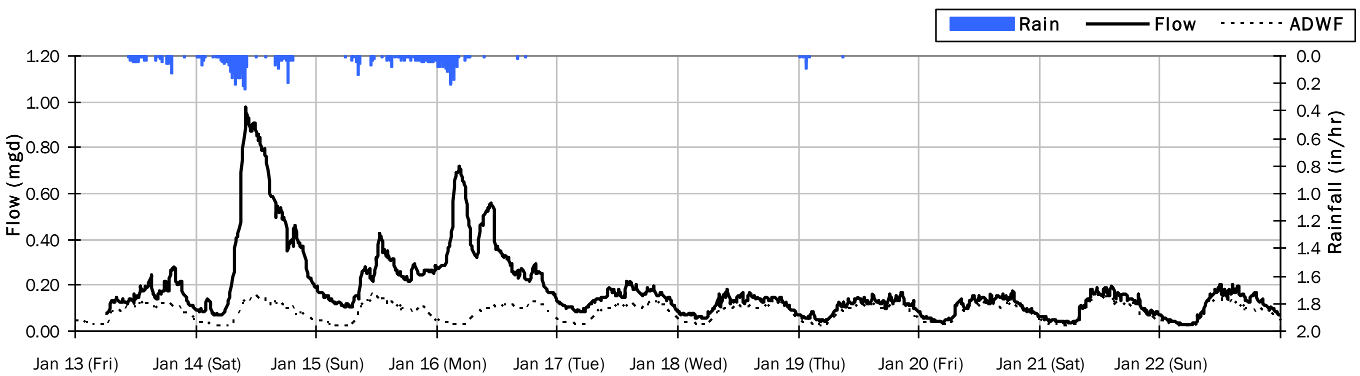
Total Rainfall: 2.30 inches

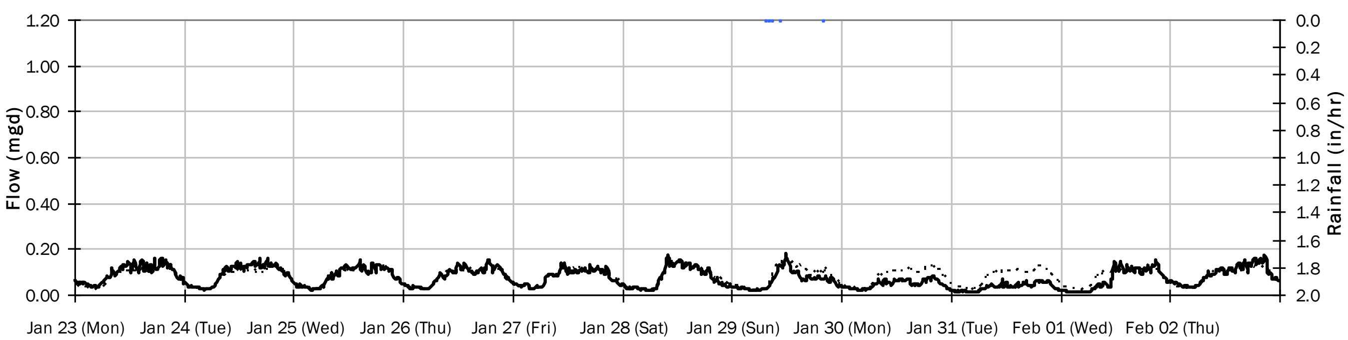
SITE 3

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 2.00 inches

Period Avg Flow: 0.126 mgd   Period Peak Flow: 0.974 mgd   Period Min Flow: 0.012 mgd

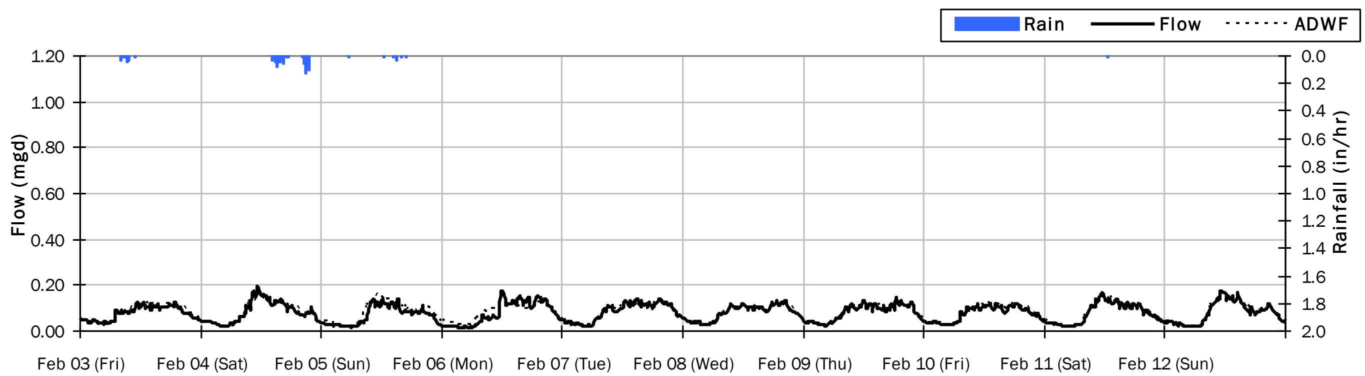


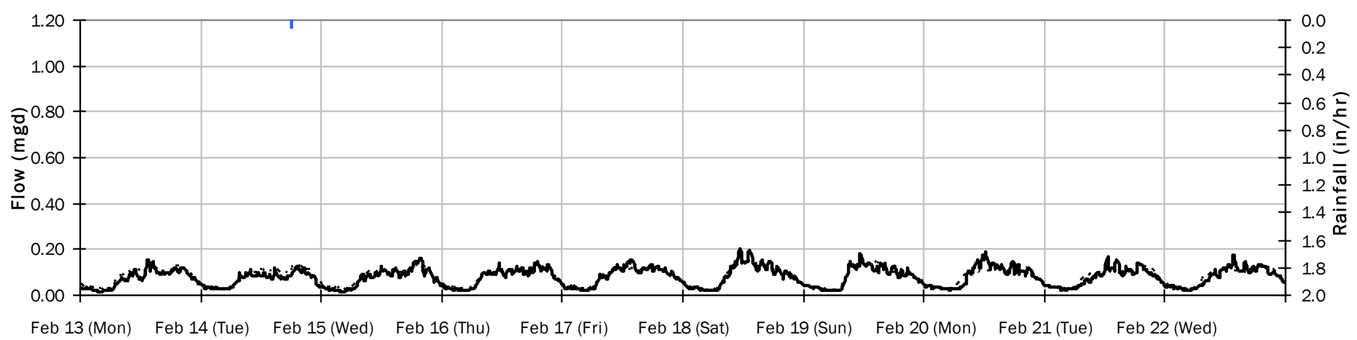


SITE 3

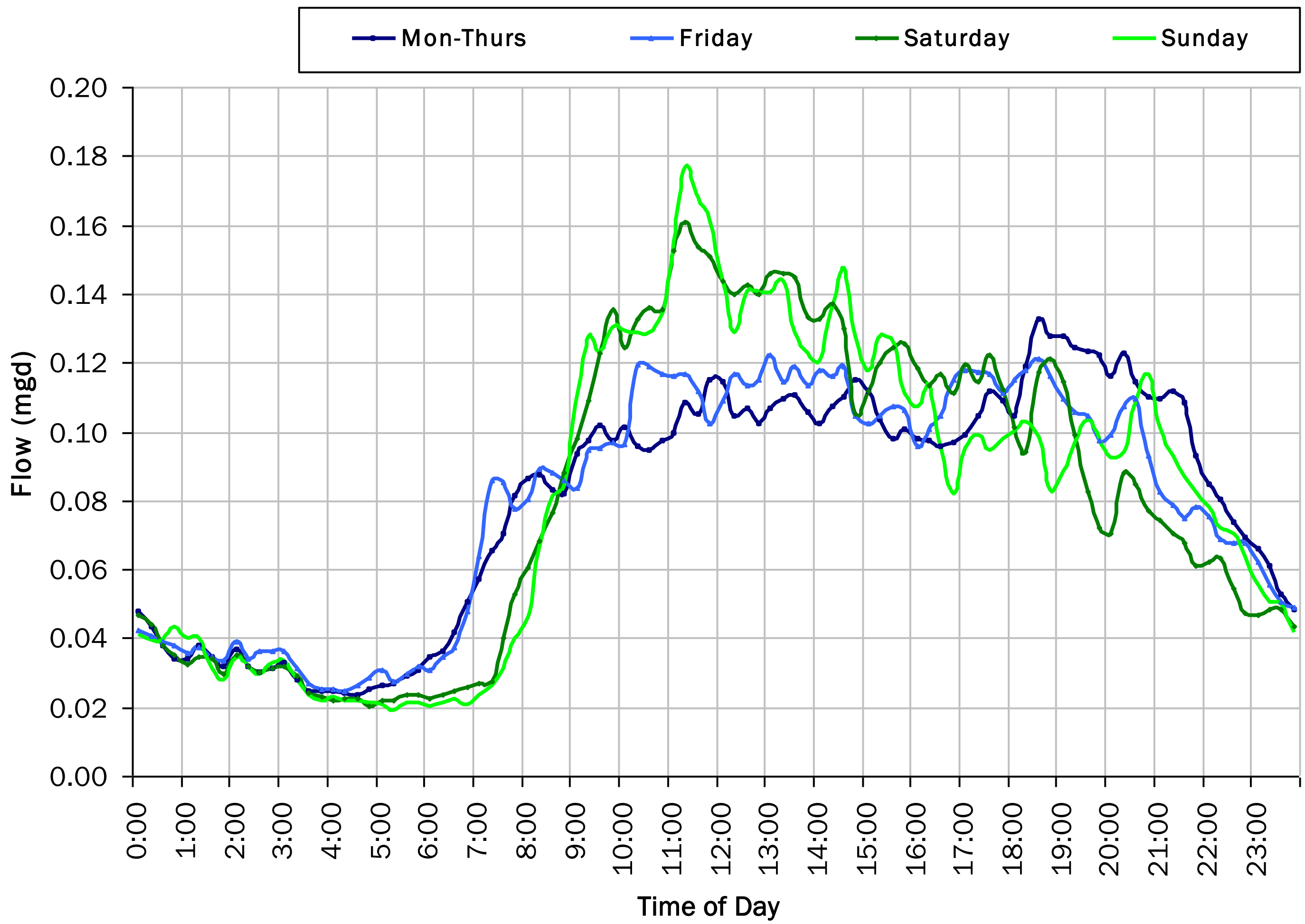
Flow Summary: 2/3/2023 to 2/22/2023

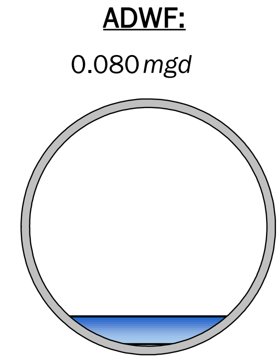
Period Rainfall: 0.30 inches   Period Avg Flow: 0.079 mgd   Period Peak Flow: 0.199 mgd   Period Min Flow: 0.012 mgd





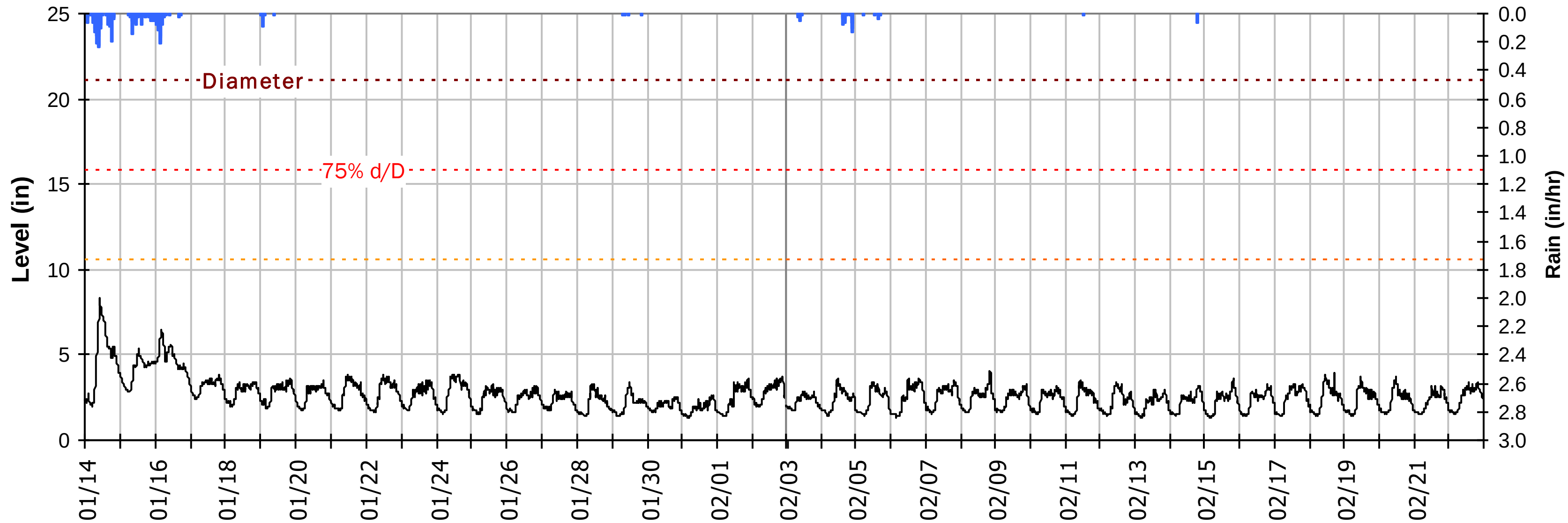
SITE 3
Average Dry Weather Flow Hydrographs

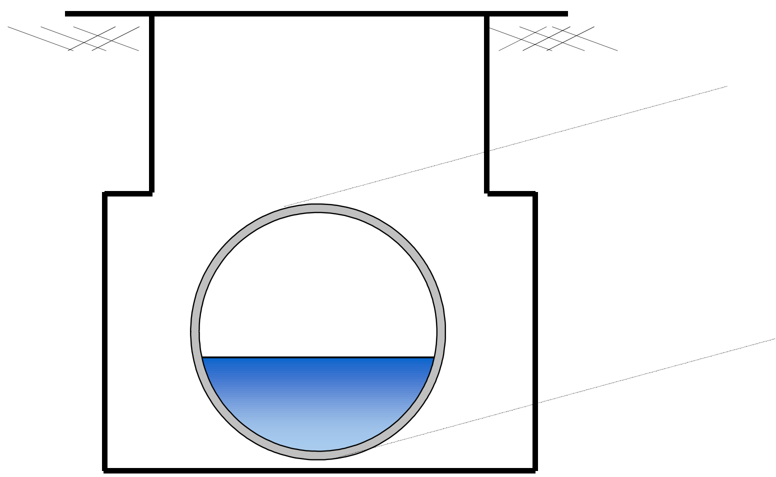




SITE 3
Site Capacity and Surcharge Summary

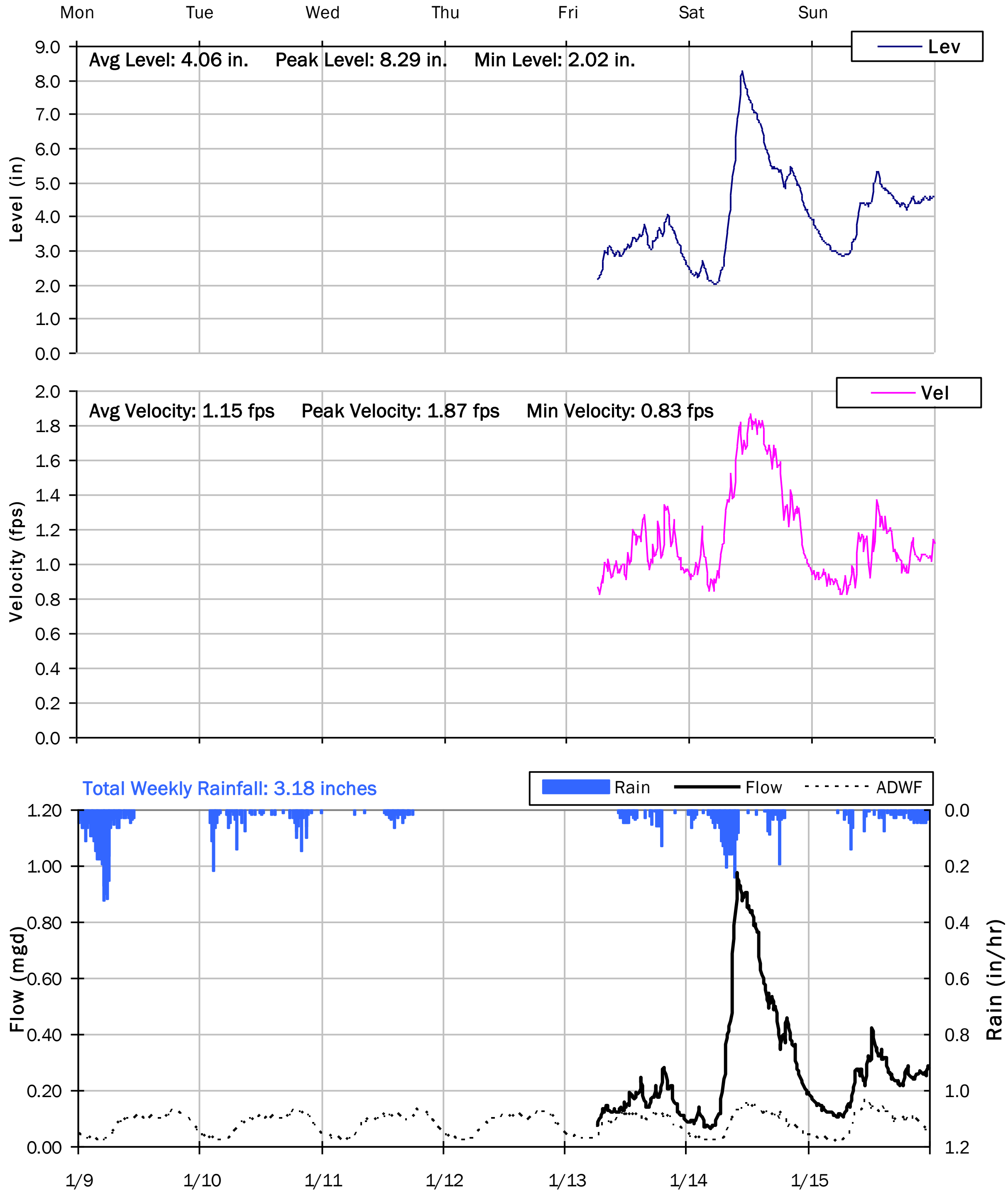
<u>Realtime Flow Levels with Rainfall Data over Peak Level Period</u>





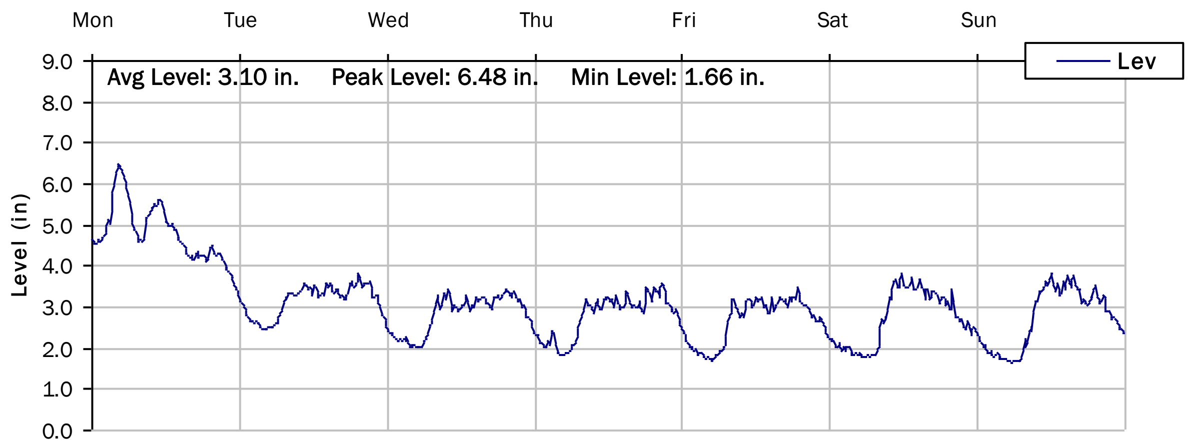
Pipe Diameter:	21	*inches*
Peak Measured Level:	8.29	*inches*
Peak d/D Ratio:	0.39	

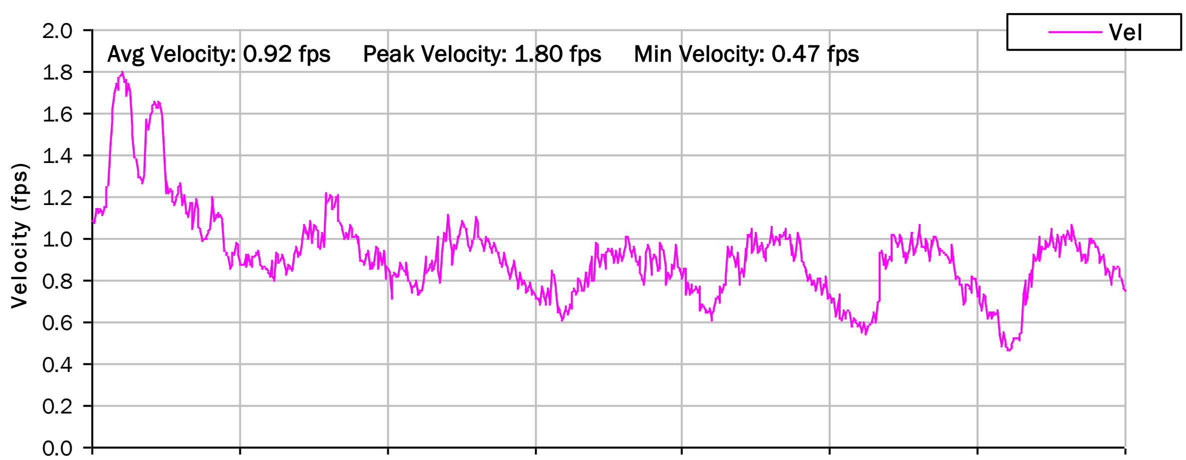
SITE 3
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023

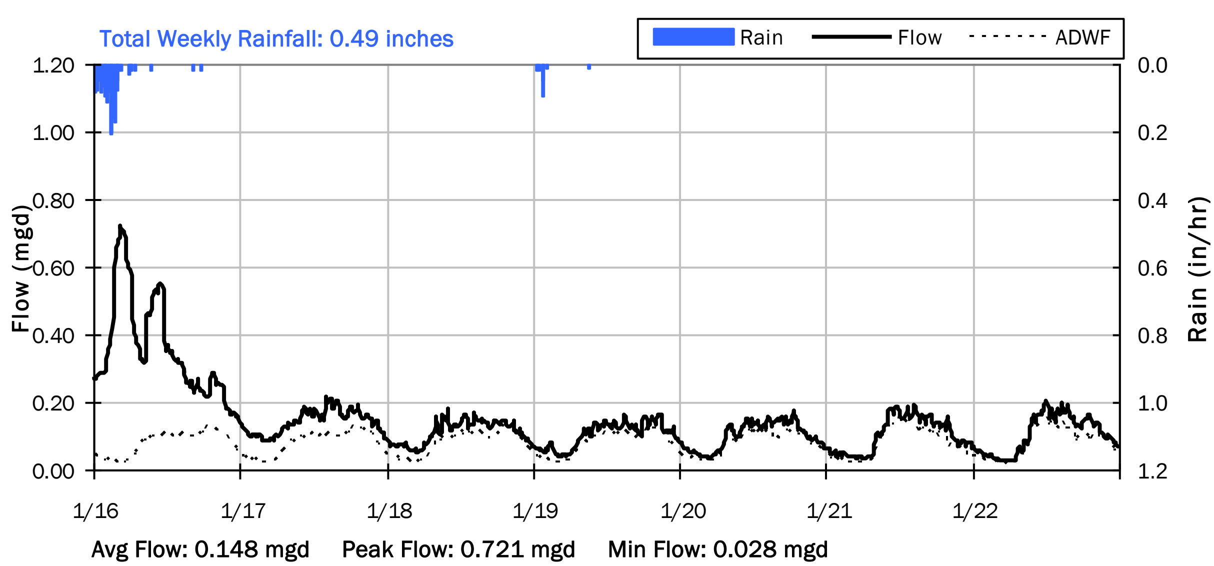


Avg Flow: 0.274 mgd Peak Flow: 0.974 mgd Min Flow: 0.067 mgd

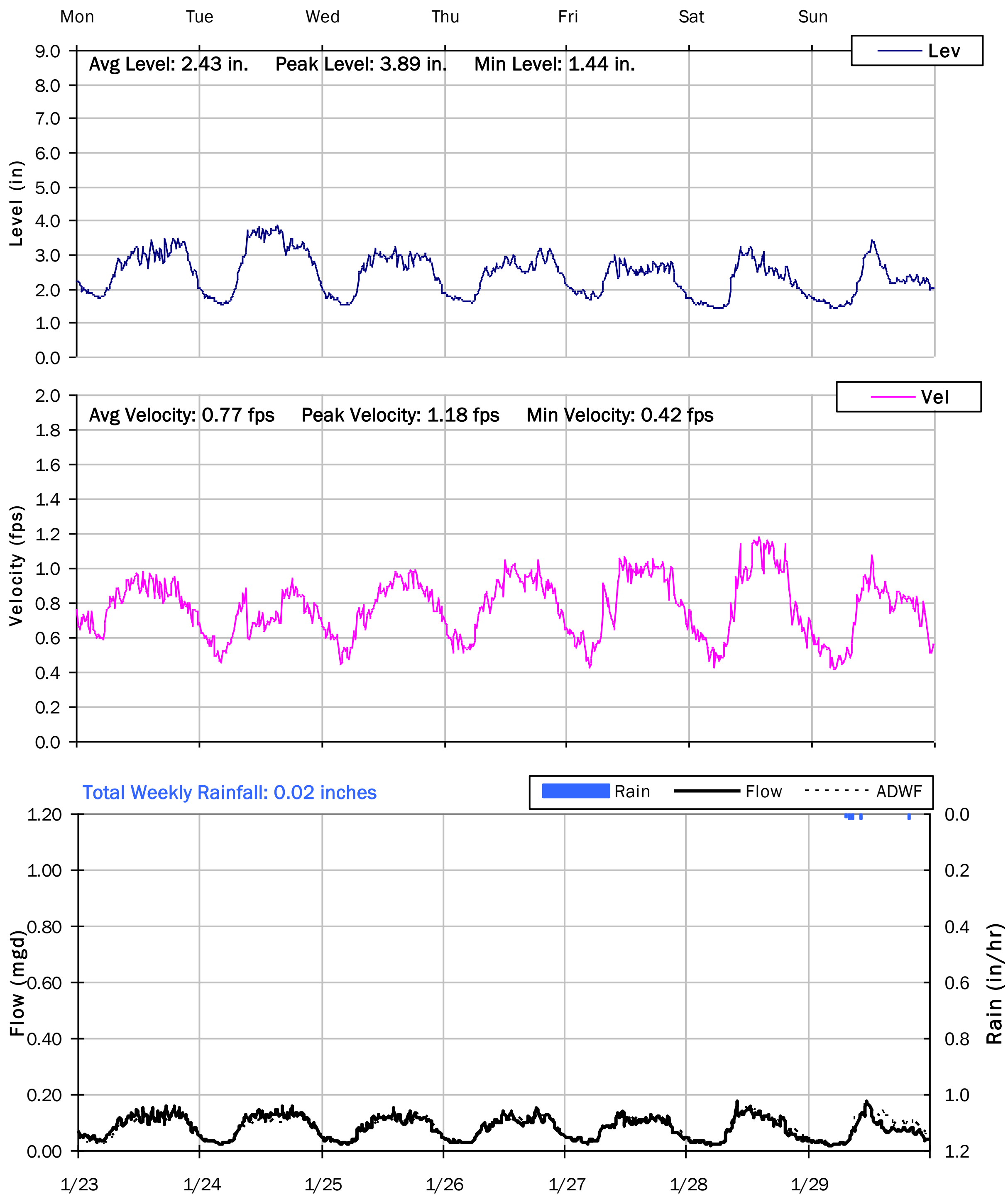
SITE 3
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023





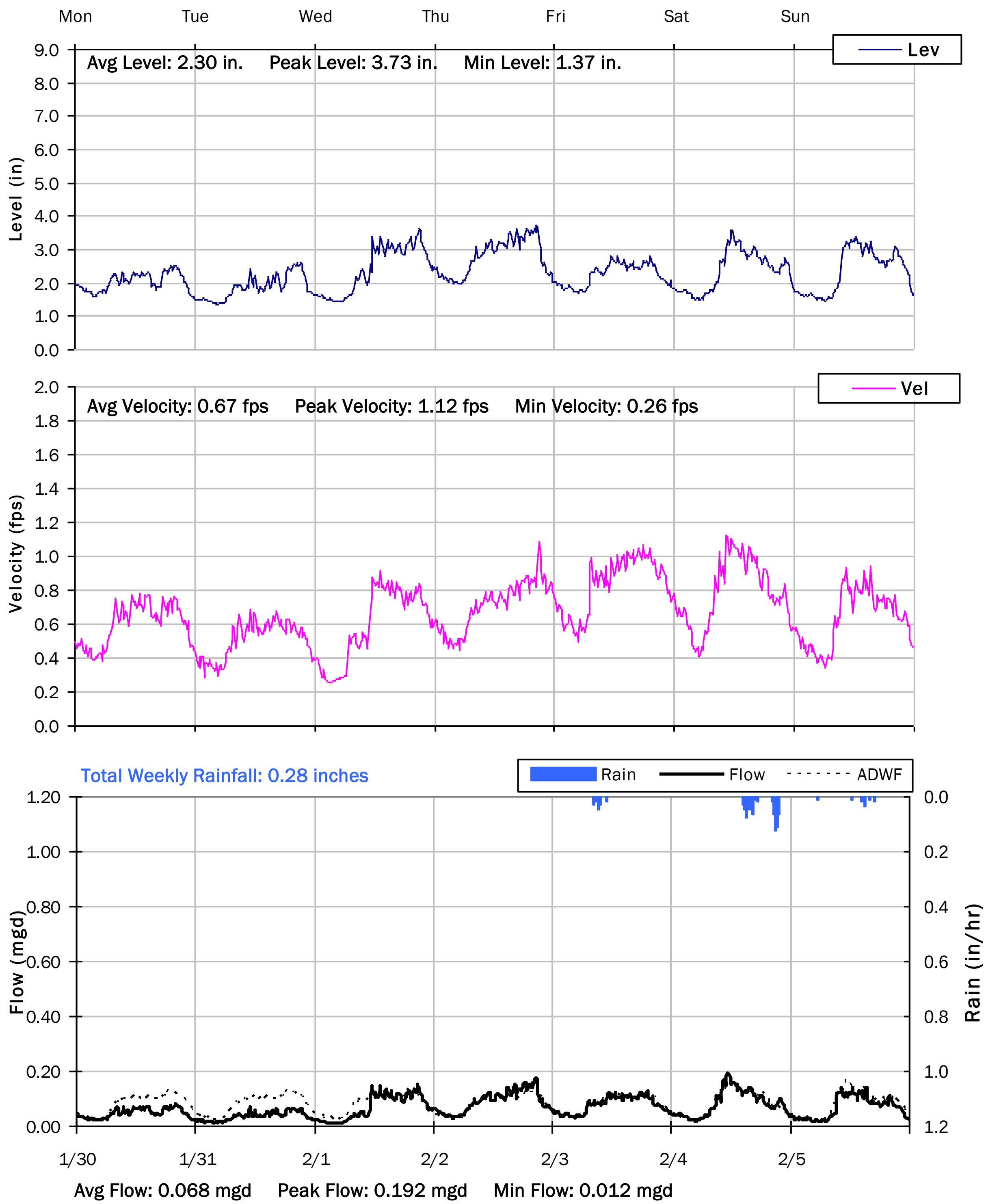


SITE 3
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023

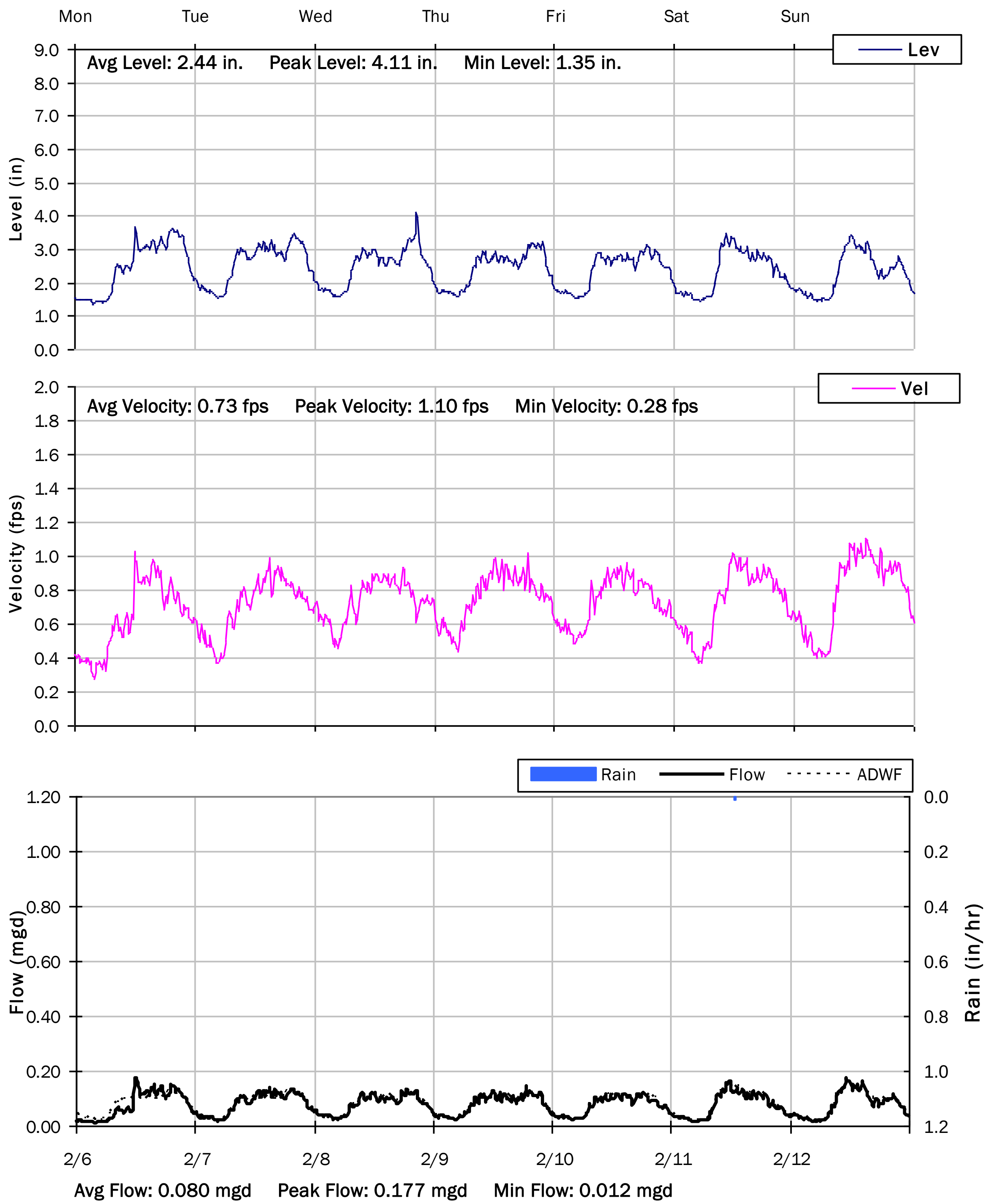


Avg Flow: 0.083 mgd Peak Flow: 0.179 mgd Min Flow: 0.019 mgd

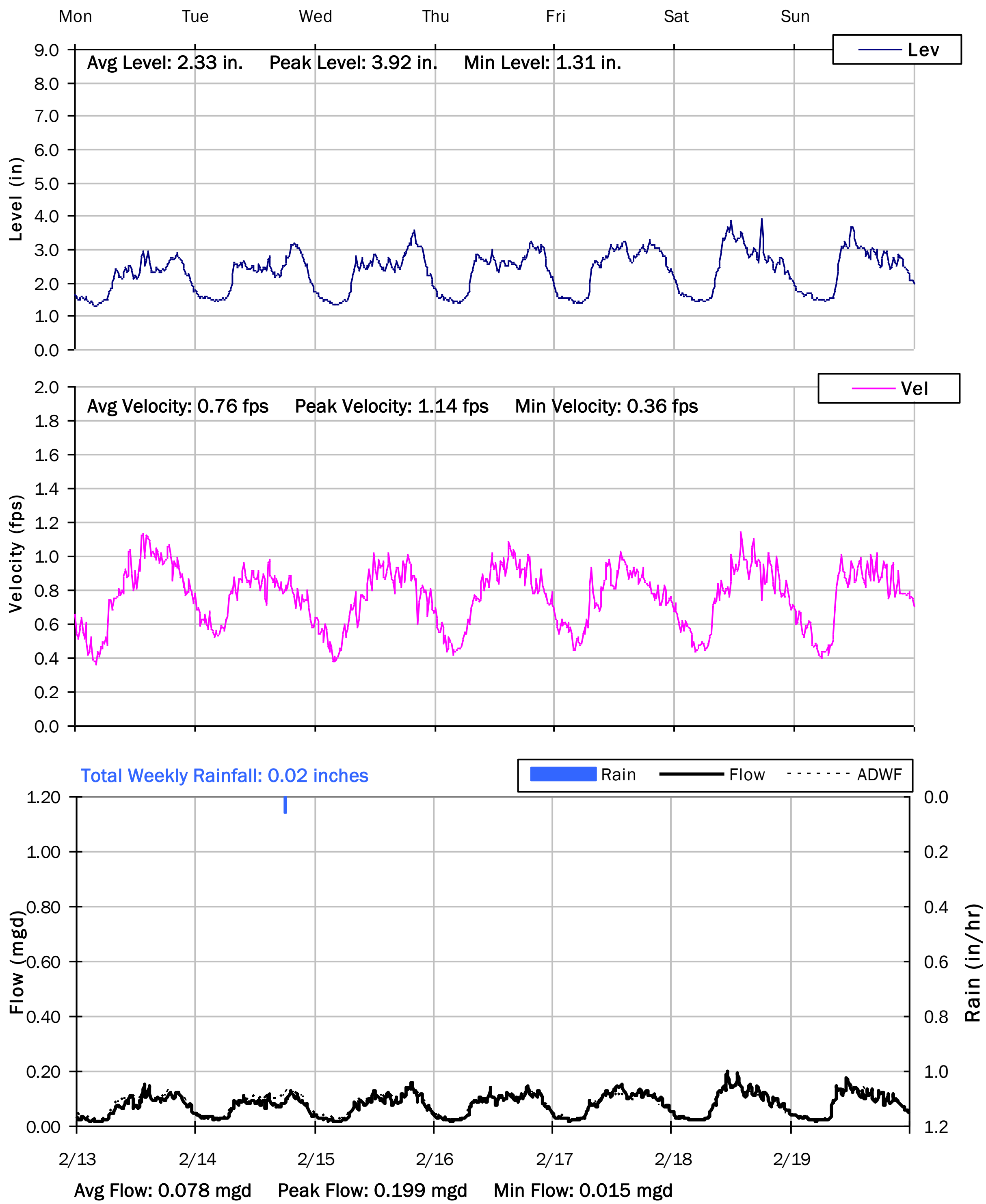
SITE 3
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023



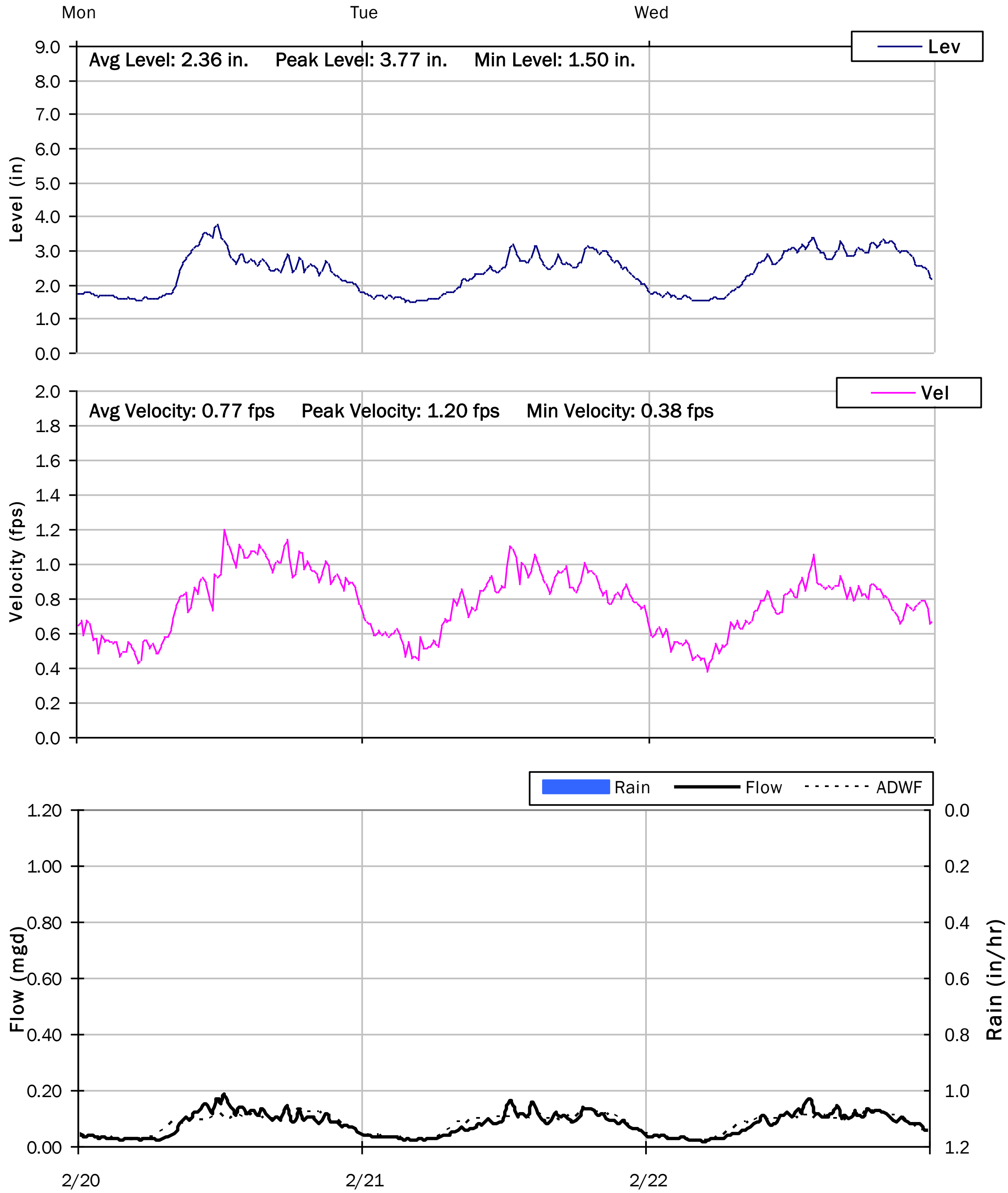
SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023



SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023



Avg Flow: 0.081 mgd Peak Flow: 0.186 mgd Min Flow: 0.020 mgd

Monitoring Site: Site 4

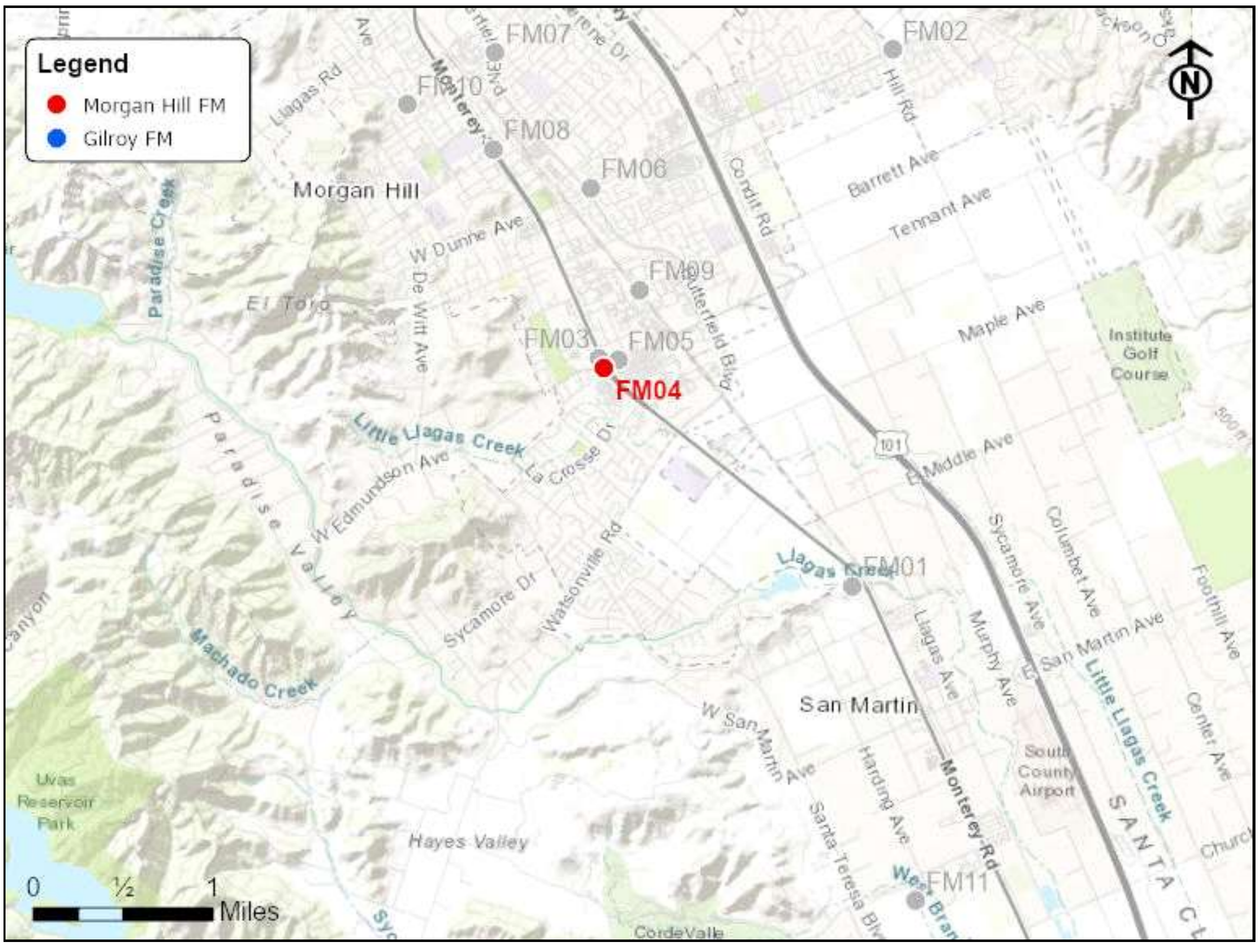
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: W Edmundson Ave and Monterey Hwy

Data Summary Report



Vicinity Map: Site 4

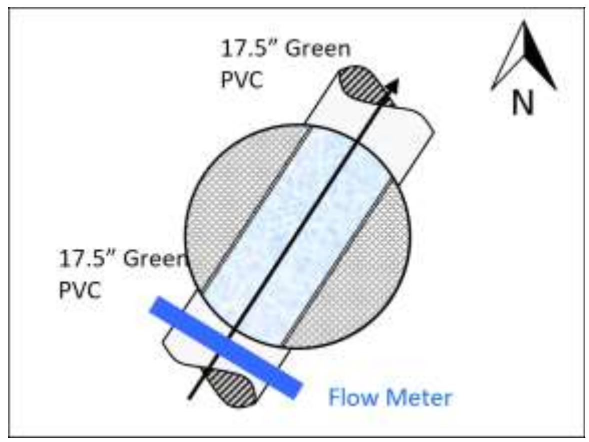
SITE 4

Site Information MH ID: I5-A.MH.014

Location:	W Edmundson Ave and Monterey Hwy
Coordinates:	121.3835° W, 37.6464° N
Rim Elevation:	332 feet
Expected Pipe Diameter:	15 inches
Measured Pipe Diameter:	17.5 inches
ADWF:	0.241 mgd
Peak Measured Flow:	1.36 mgd
Sediment:	None

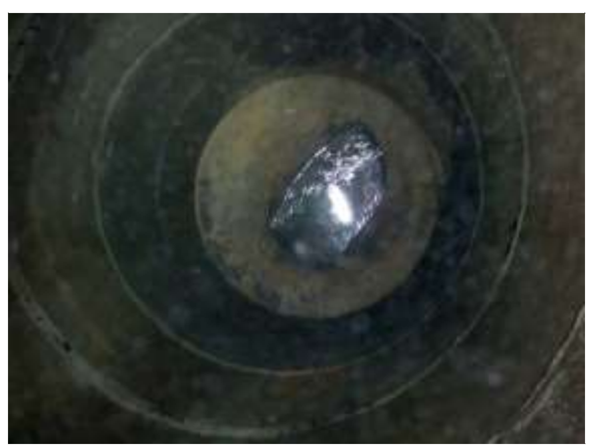
Satellite Map

Sanitary Map



Flow Sketch

Street View



Plan View

SITE 4

Additional Site Photos

Northeast Effluent Pipe

Monitored Southwest Influent Pipe

SITE 4

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.359 MGal Peak Daily Flow: 0.816 MGal Min Daily Flow: 0.234 MGal

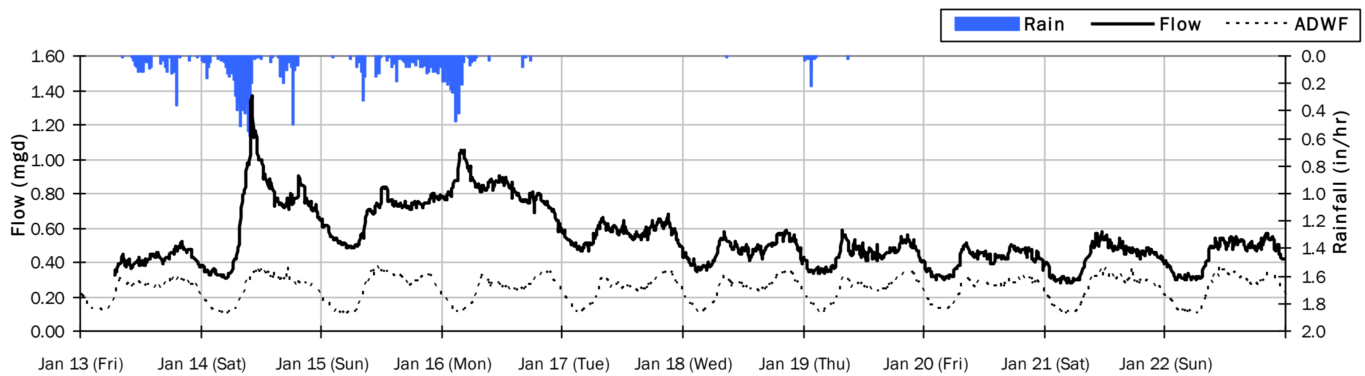
Total Rainfall: 5.76 inches

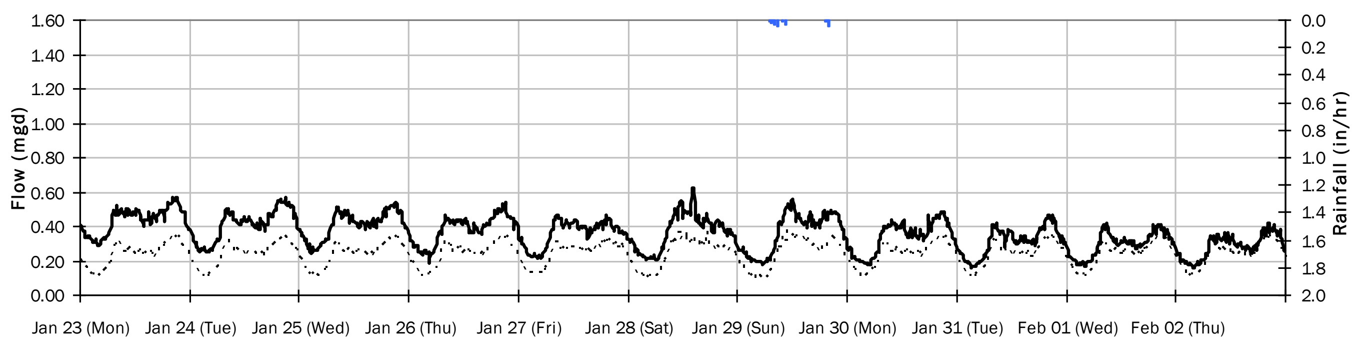
SITE 4

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 5.01 inches

Period Avg Flow: 0.444 mgd   Period Peak Flow: 1.357 mgd   Period Min Flow: 0.160 mgd



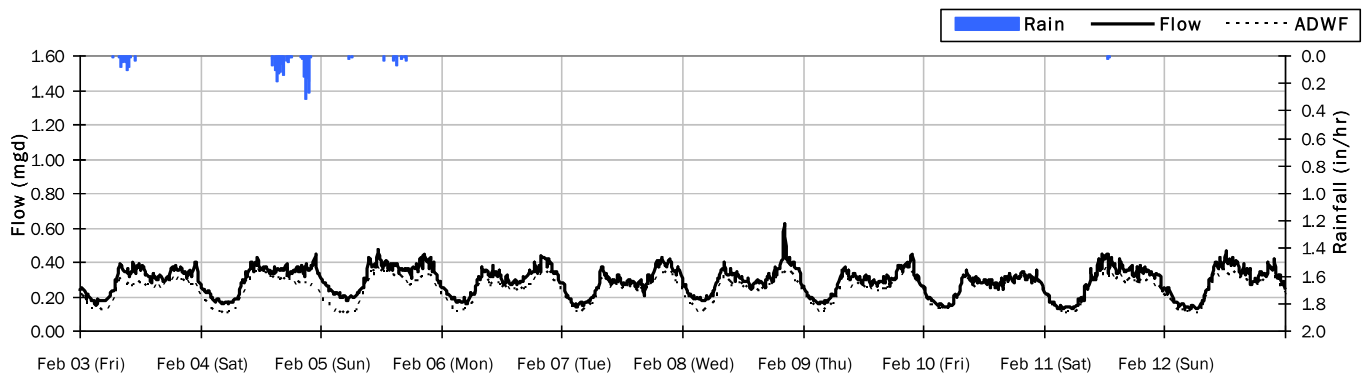


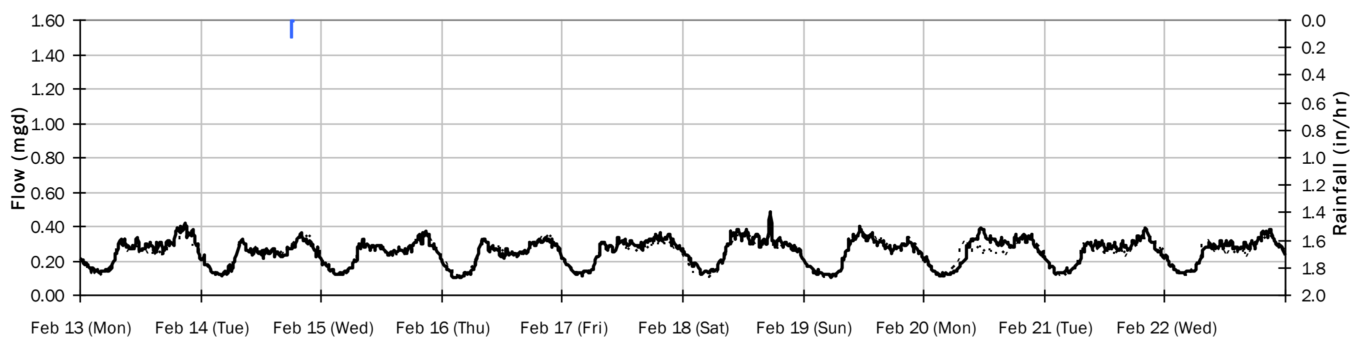
SITE 4

Flow Summary: 2/3/2023 to 2/22/2023

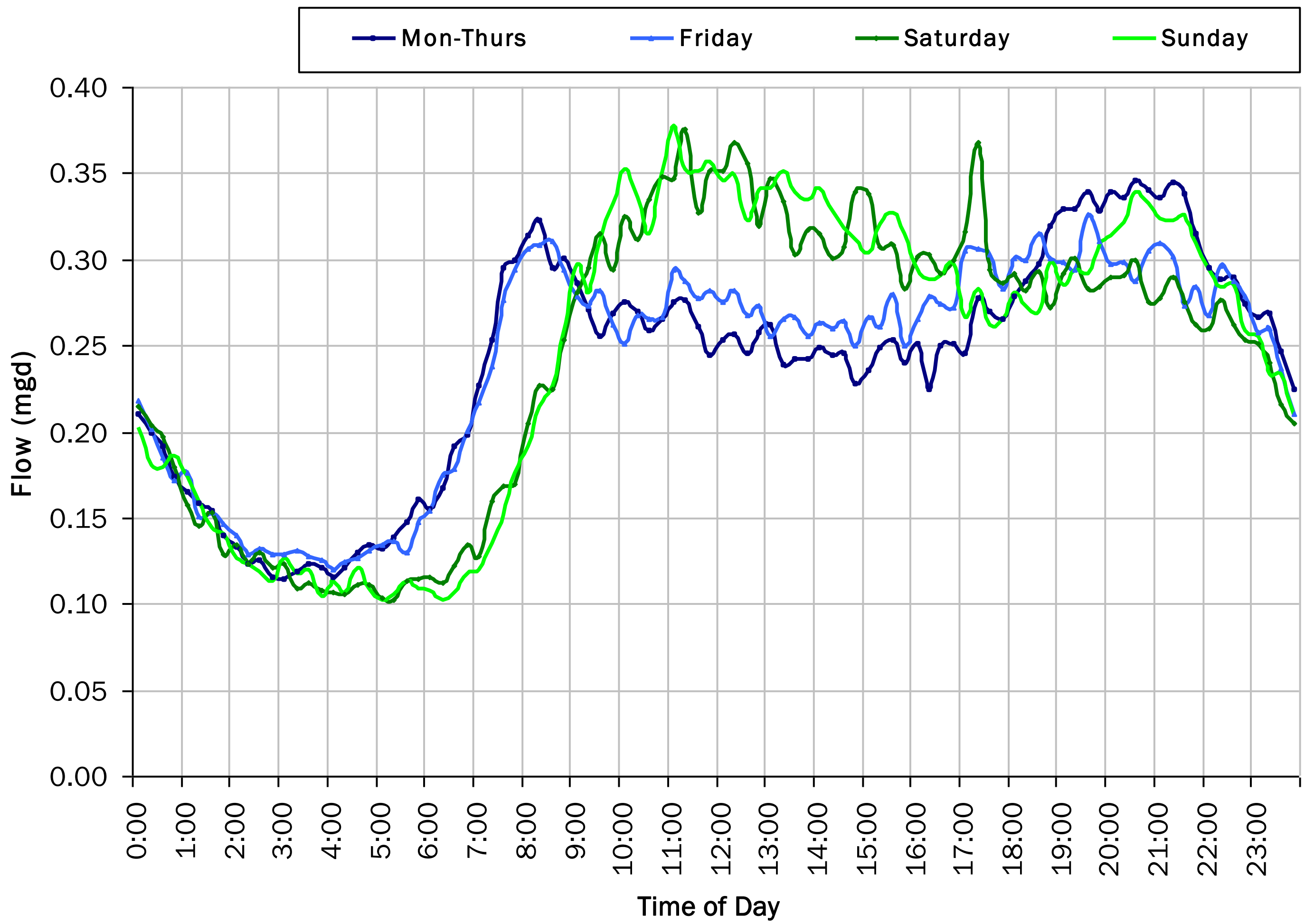
Period Rainfall: 0.76 inches

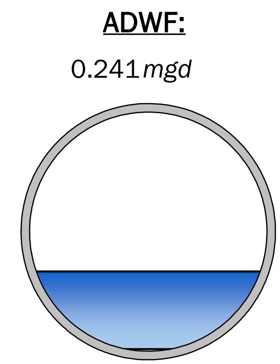
Period Avg Flow: 0.270 mgd   Period Peak Flow: 0.620 mgd   Period Min Flow: 0.103 mgd





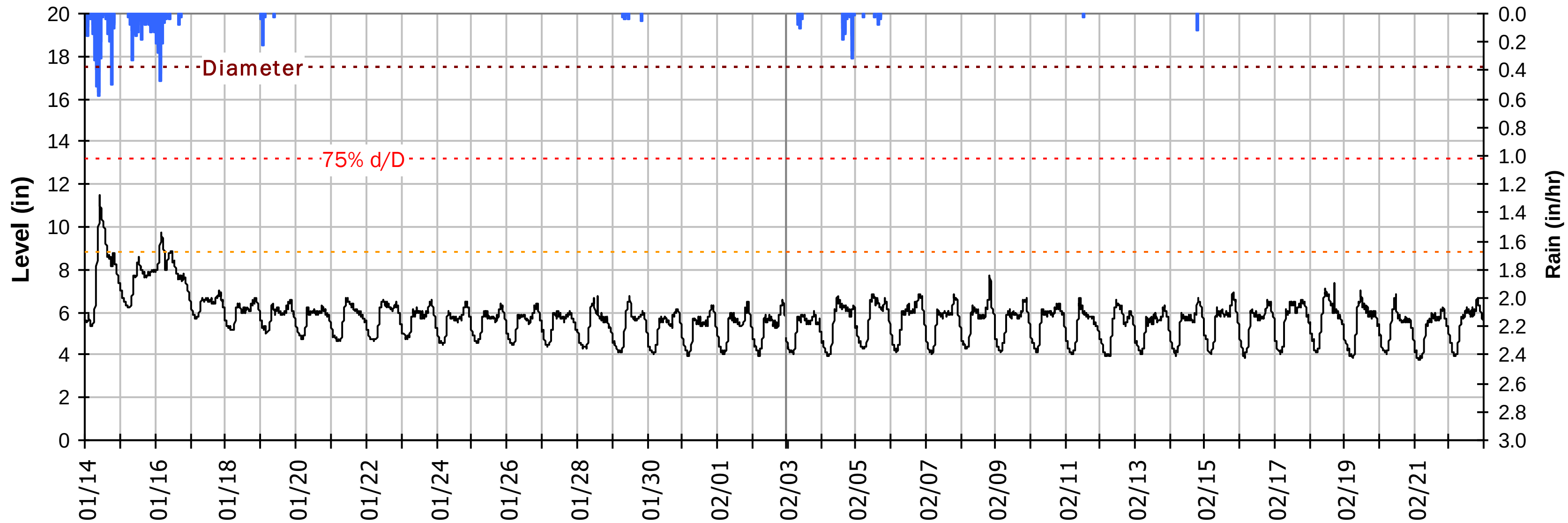
SITE 4
Average Dry Weather Flow Hydrographs

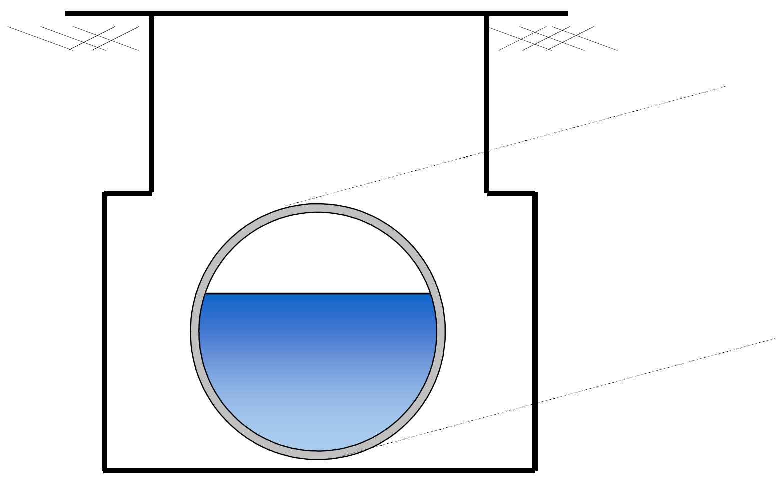




SITE 4
Site Capacity and Surcharge Summary

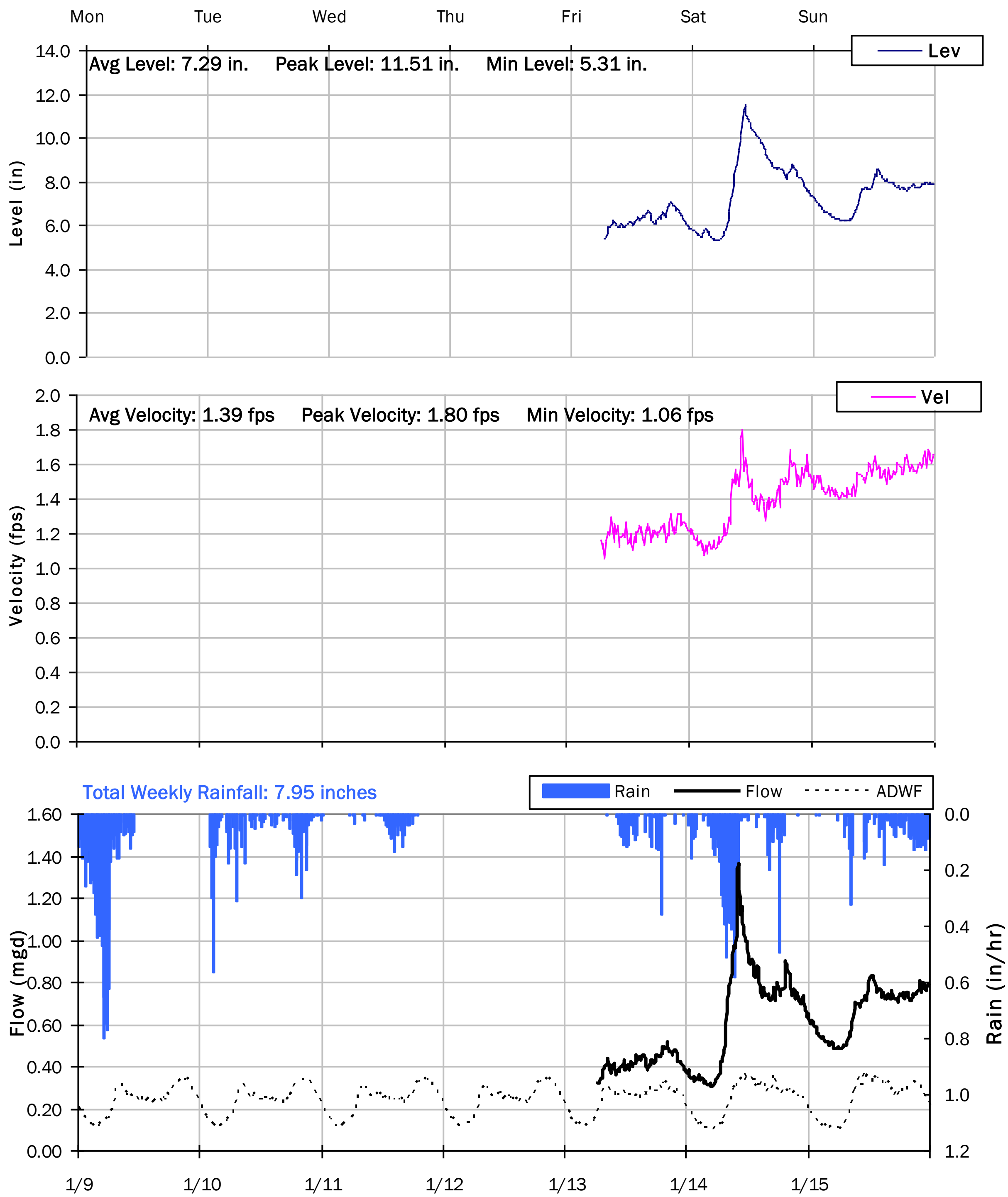
Realtime Flow Levels with Rainfall Data over Peak Level Period





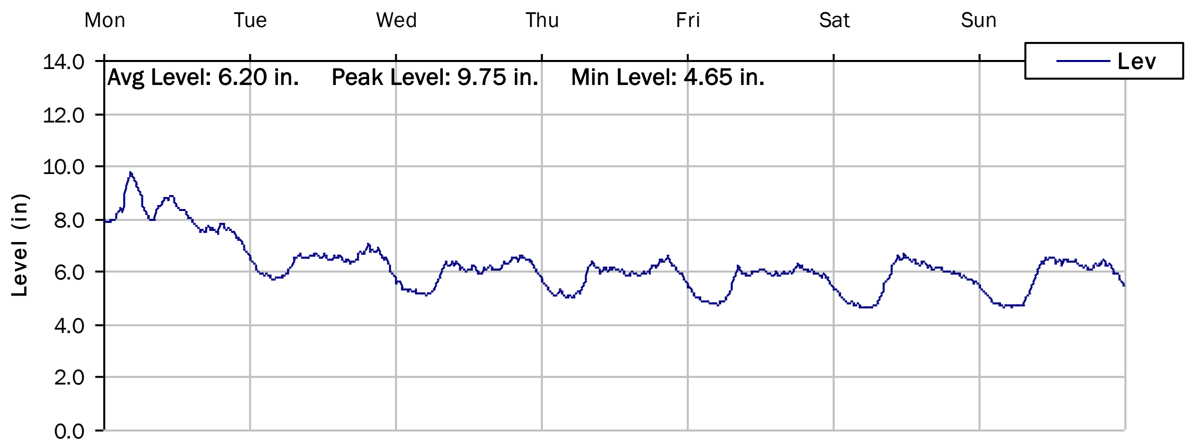
Pipe Diameter:	17.5	*inches*
Peak Measured Level:	11.5	*inches*
Peak d/D Ratio:	0.66	

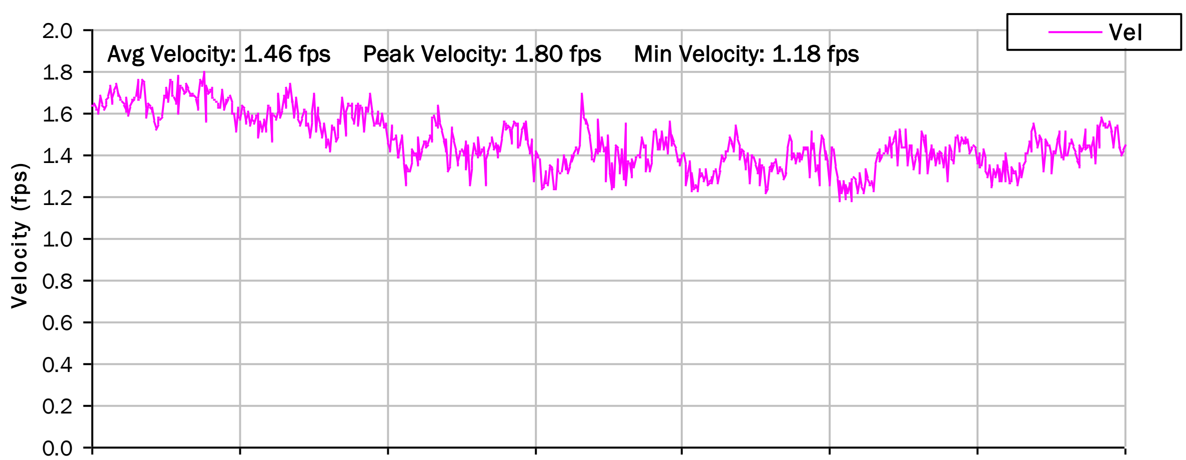
SITE 4
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023

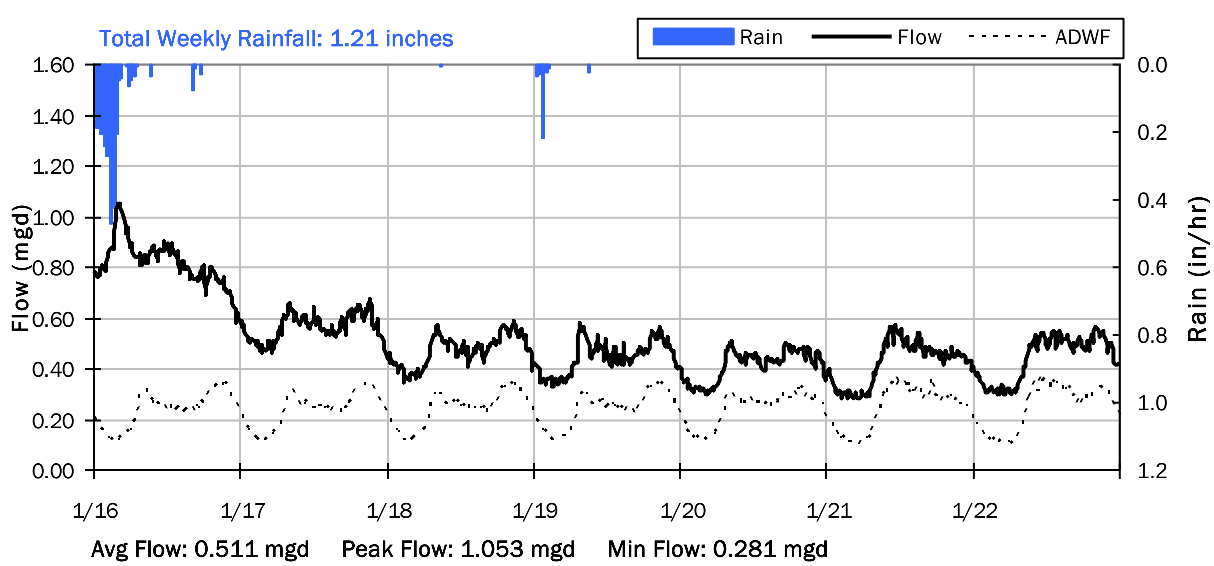


Avg Flow: 0.606 mgd Peak Flow: 1.357 mgd Min Flow: 0.308 mgd

SITE 4
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023



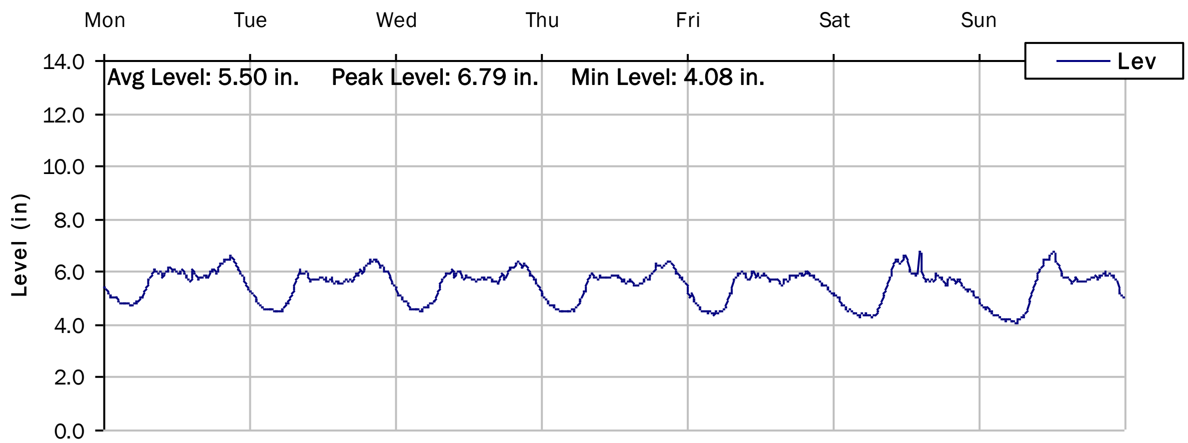


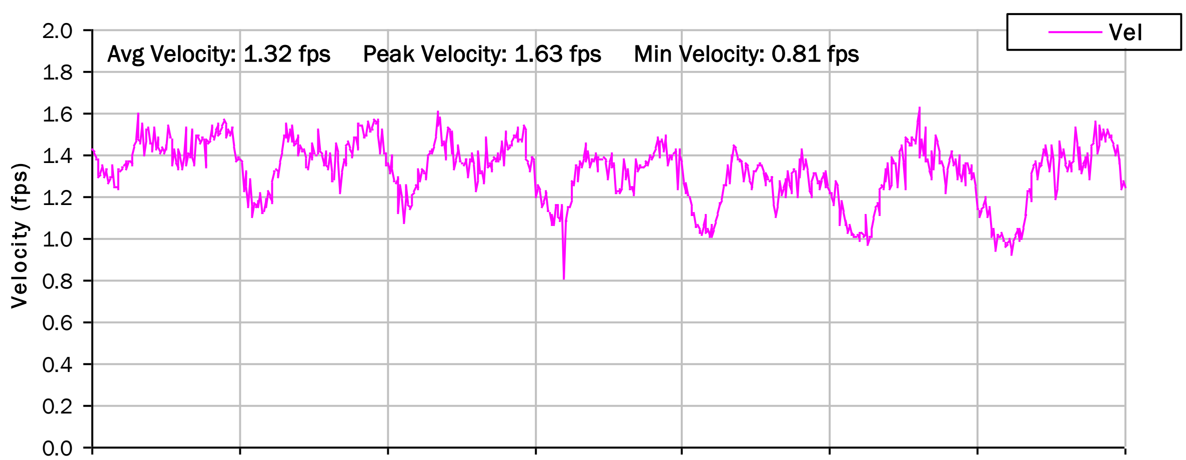


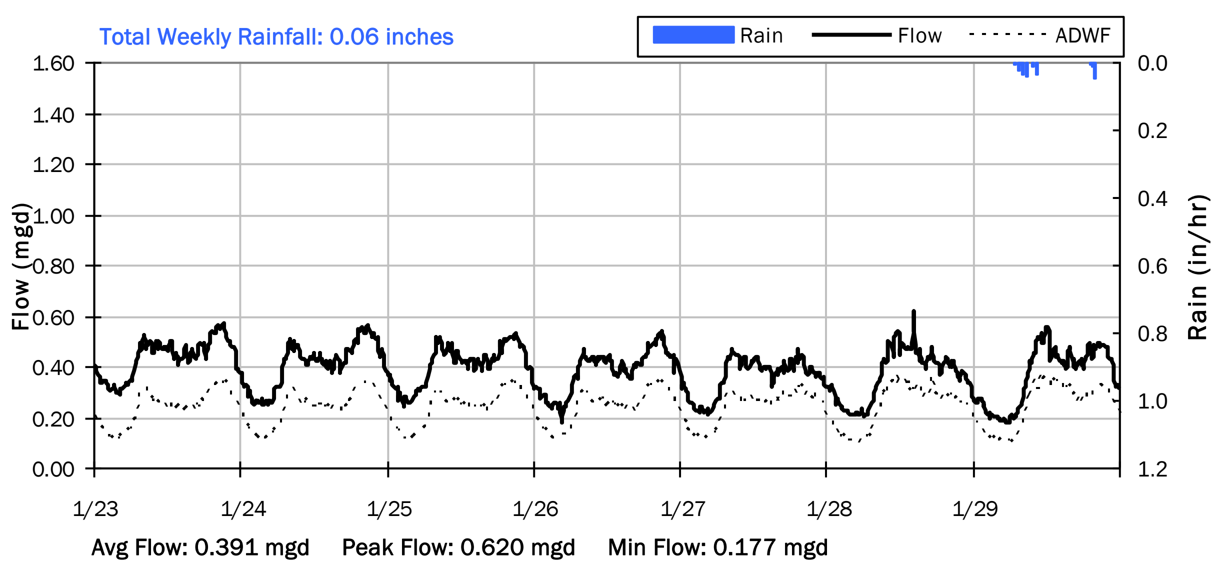
SITE 4

Weekly Level, Velocity and Flow Hydrographs

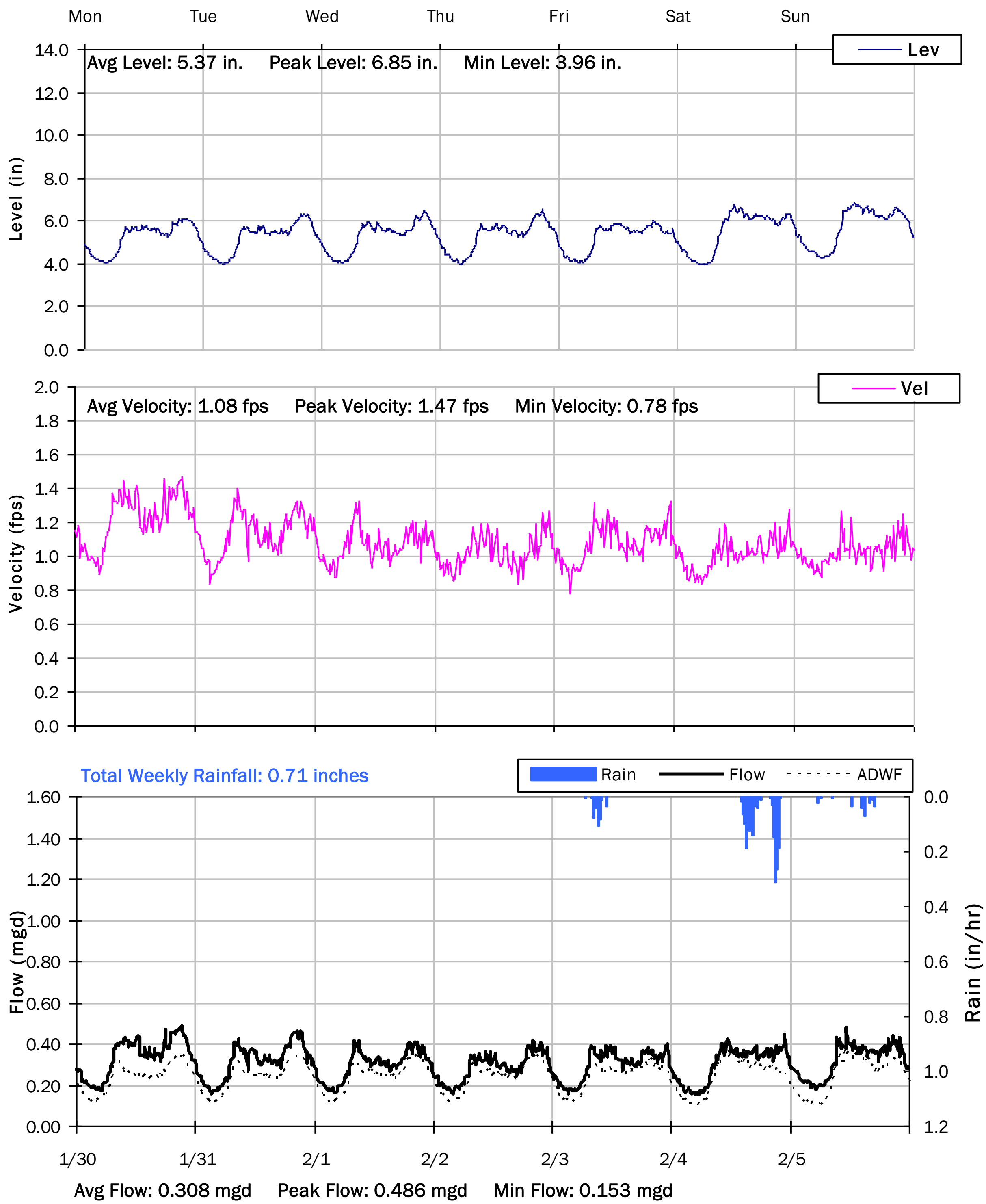
1/23/2023 to 1/30/2023



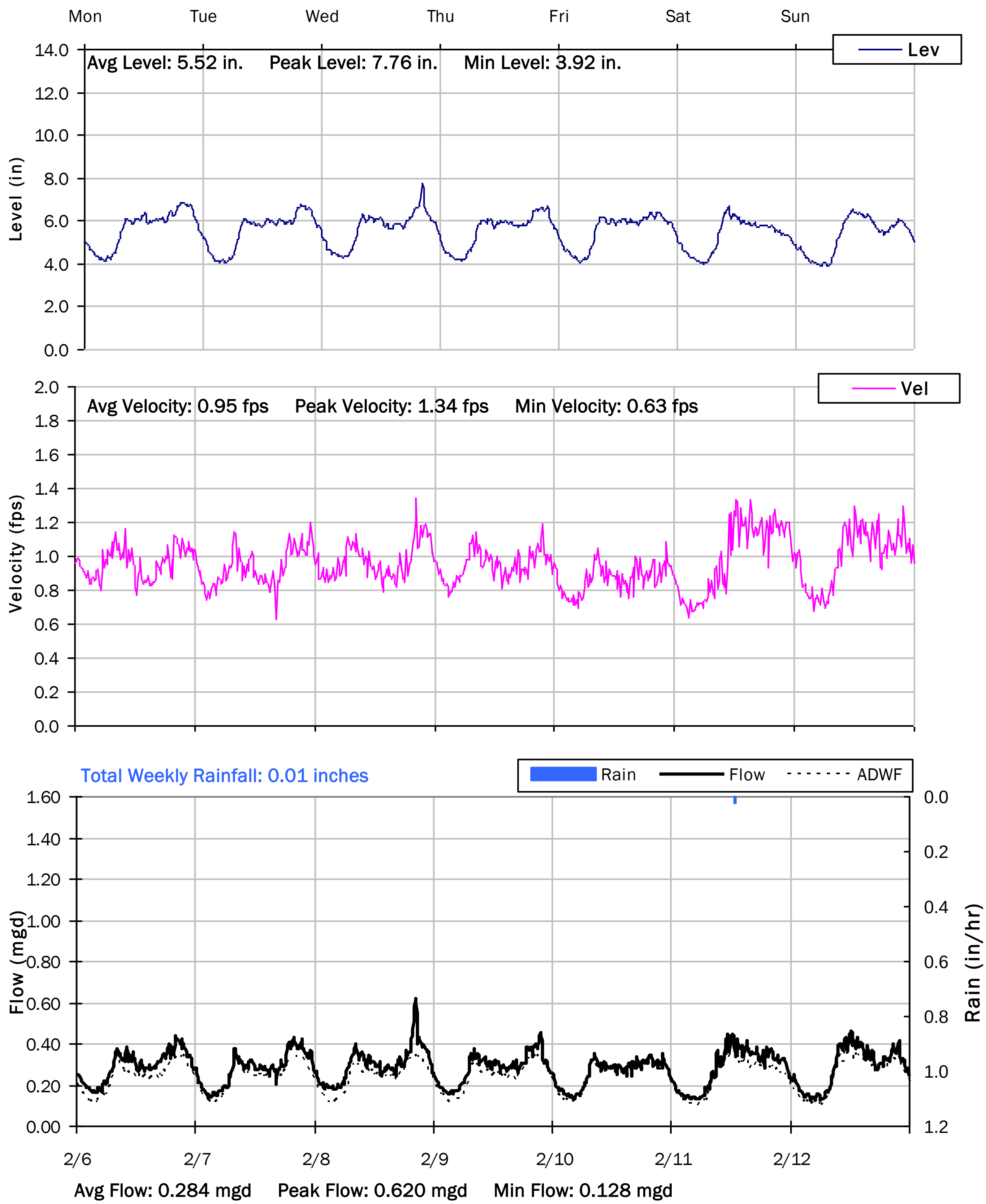




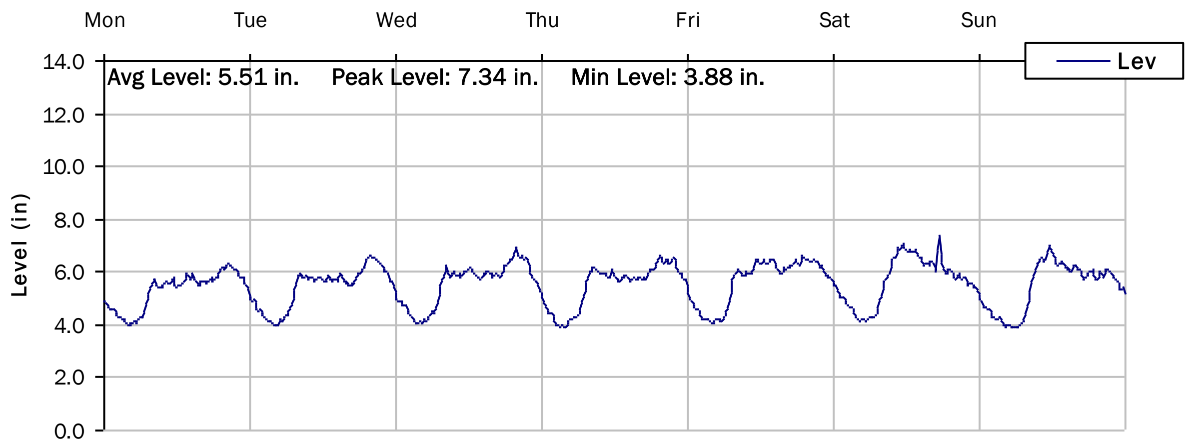
SITE 4
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023

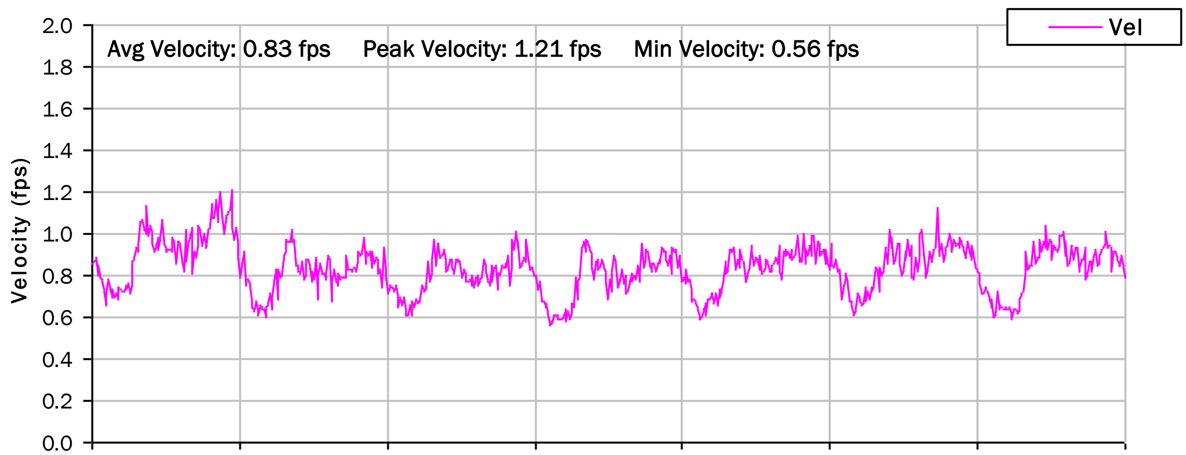


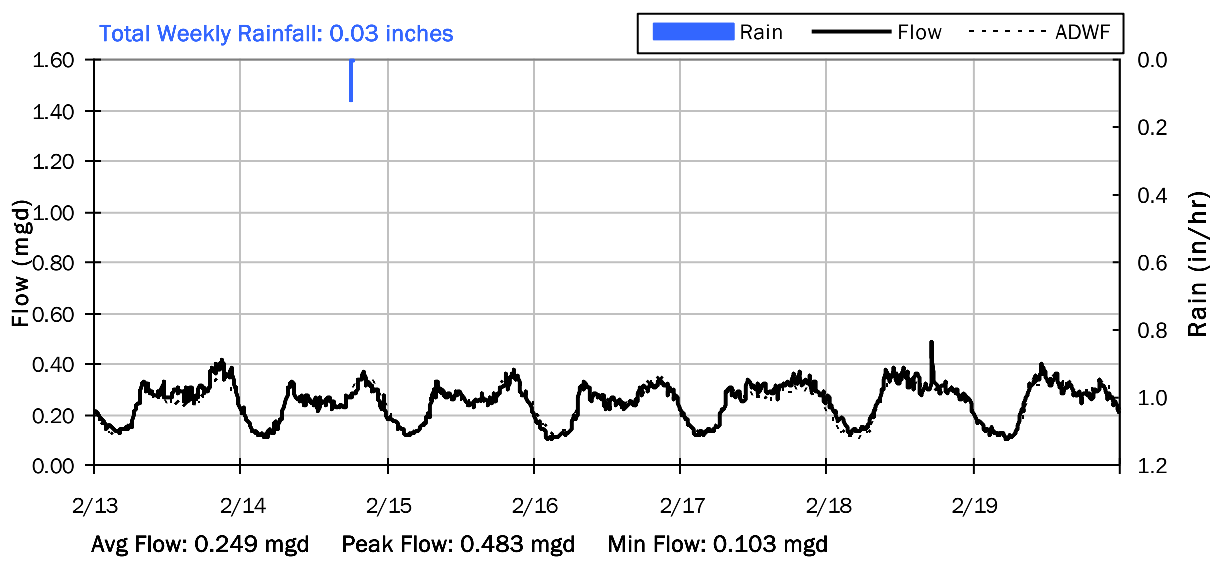
SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023



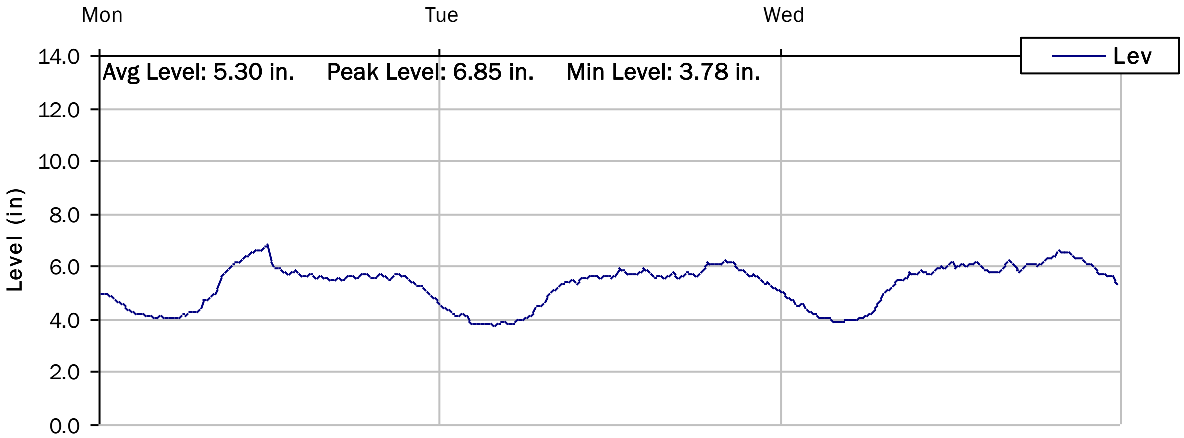
SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023

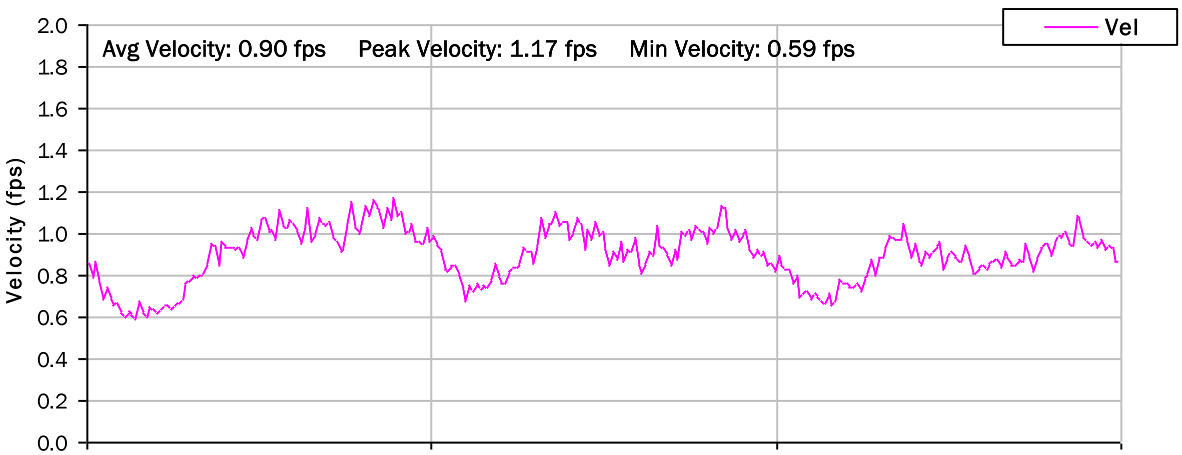


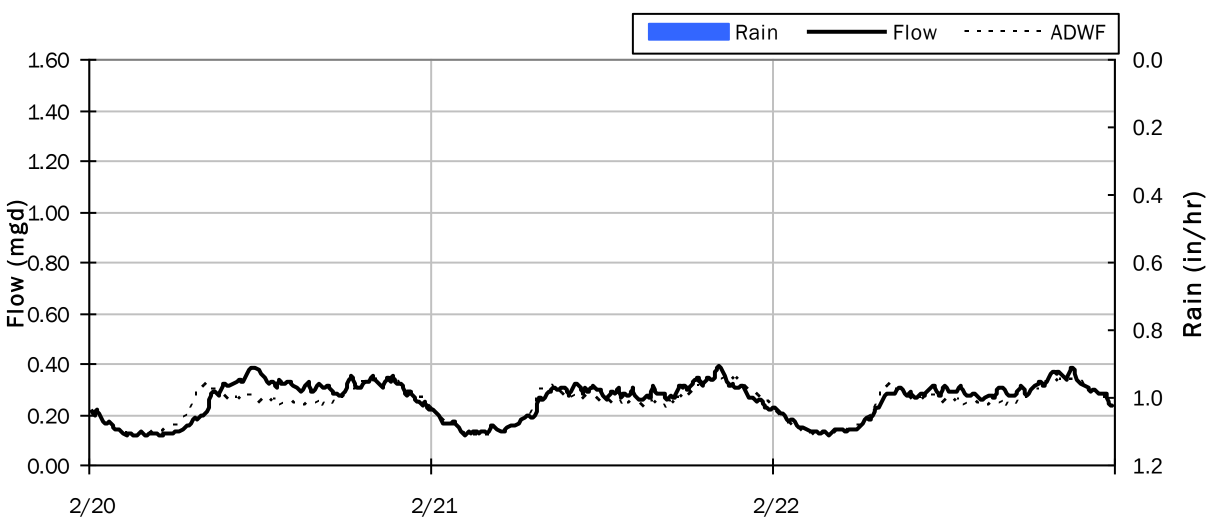




SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023







Avg Flow: 0.254 mgd Peak Flow: 0.388 mgd Min Flow: 0.115 mgd

Monitoring Site: Site 5

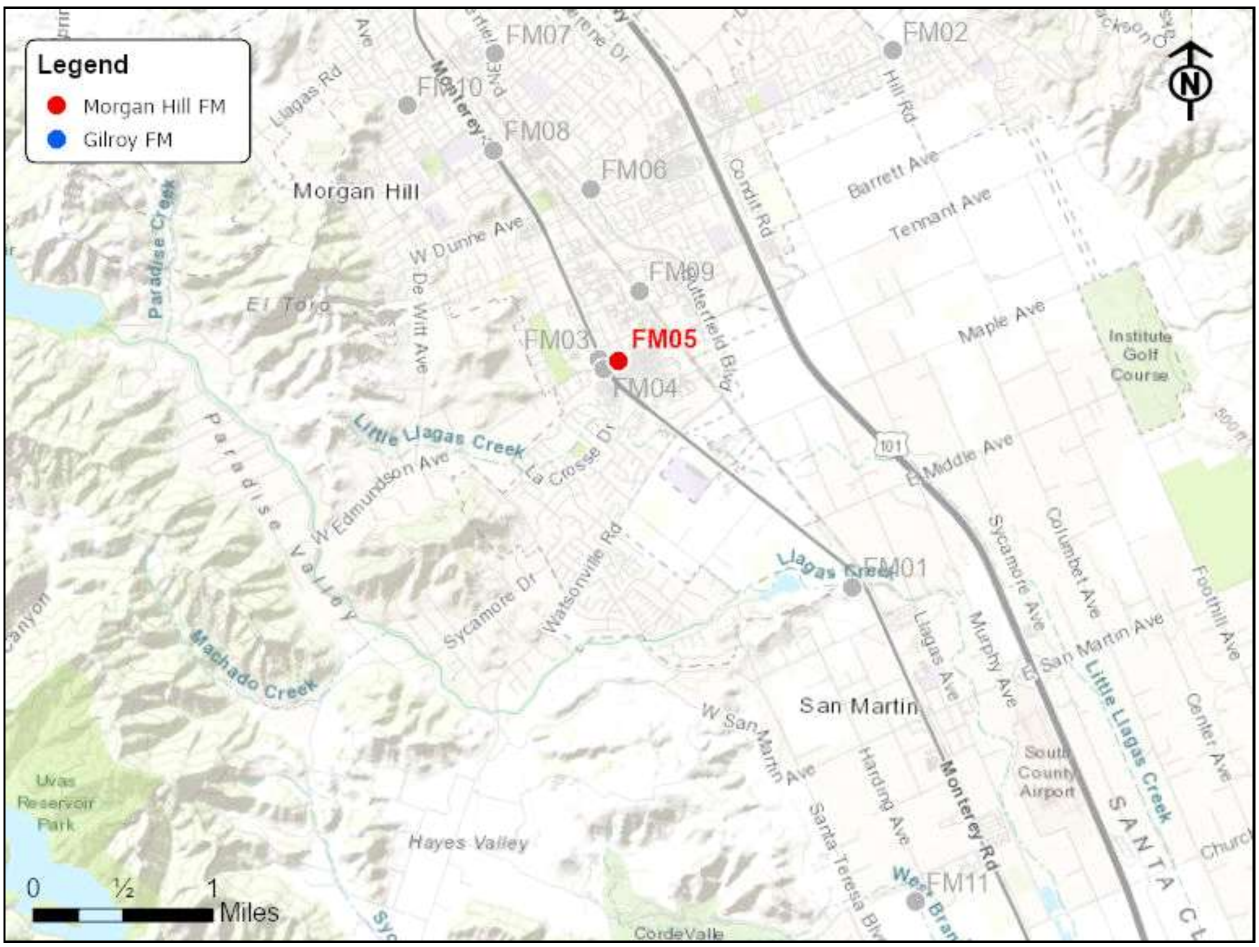
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: 49 Tennant Ave

Data Summary Report



Vicinity Map: Site 5

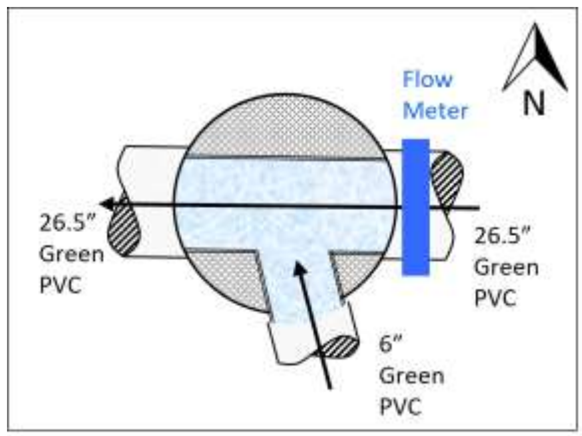
SITE 5

Site Information MH ID: I5-A.MH.007

Location:	49 Tennant Ave
Coordinates:	121.6417° W, 37.1135° N
Rim Elevation:	330 feet
Expected Pipe Diameter:	24 inches
Measured Pipe Diameter:	26.5 inches
ADWF:	1.964 mgd
Peak Measured Flow:	7.32 mgd
Sediment:	None

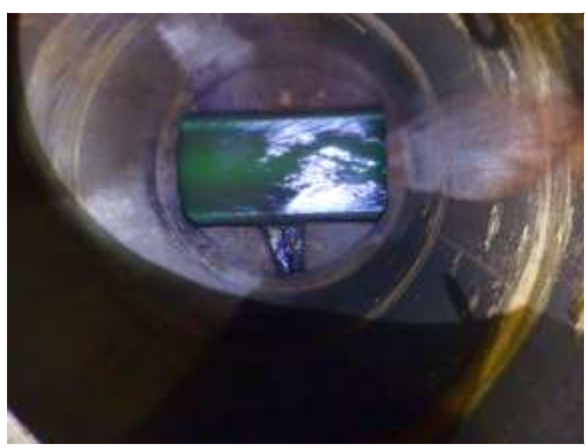
Satellite Map

Sanitary Map



Flow Sketch

Street View

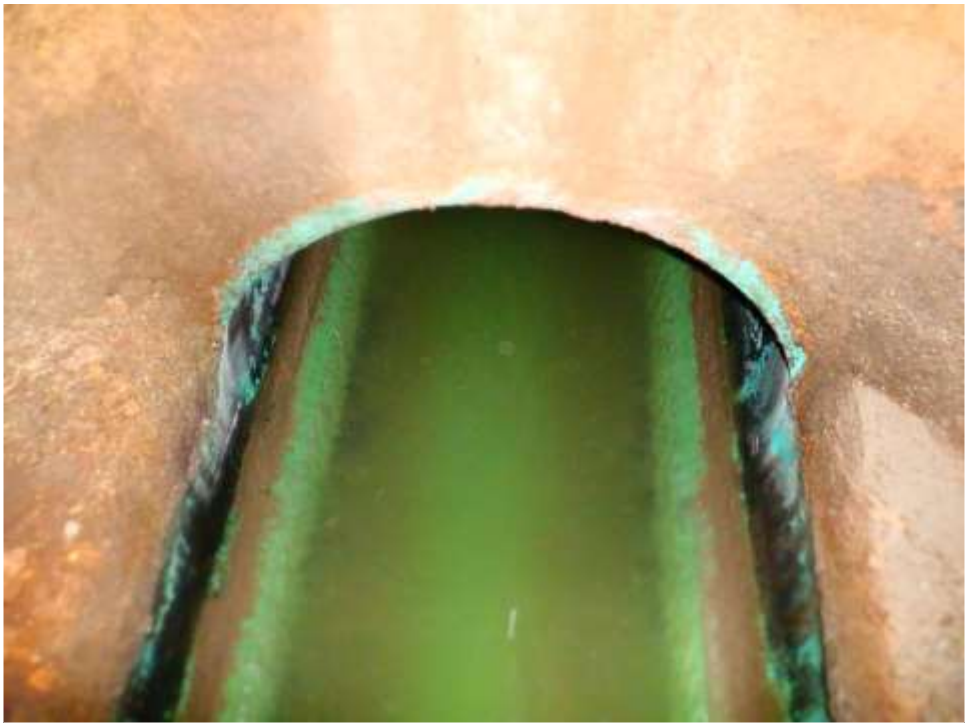


Plan View

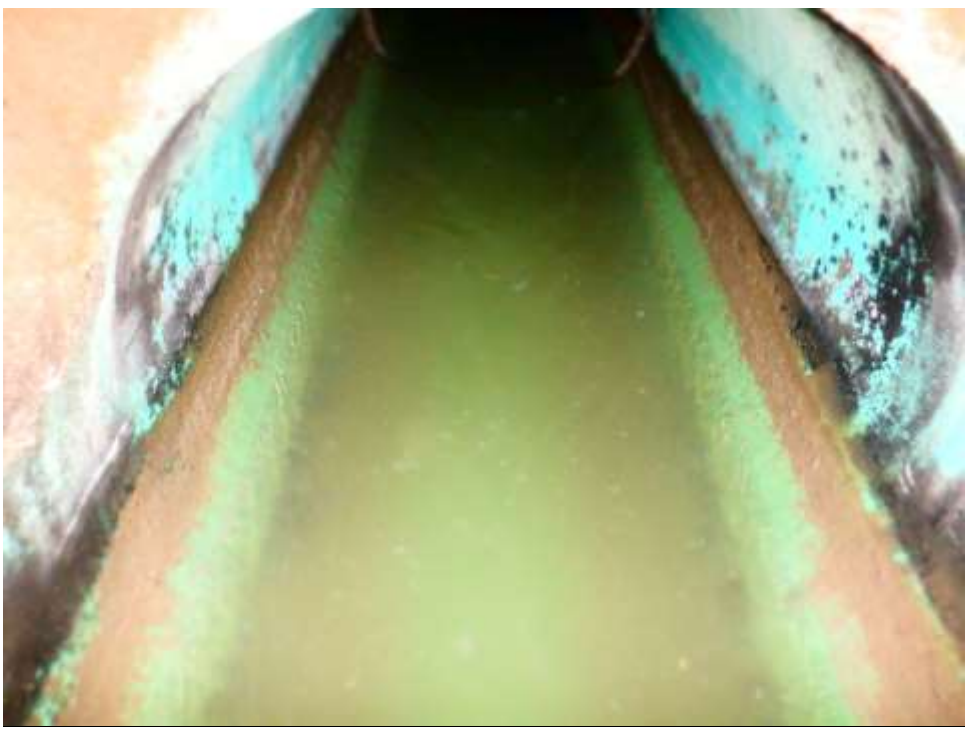
SITE 5

Additional Site Photos

Southwest Effluent Pipe



Monitored Northeast Influent Pipe



SITE 5

Additional Site Photos

South Influent Pipe

SITE 5

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 2.153 MGal **Peak Daily Flow: 3.923 MGal** **Min Daily Flow: 1.813 MGal**

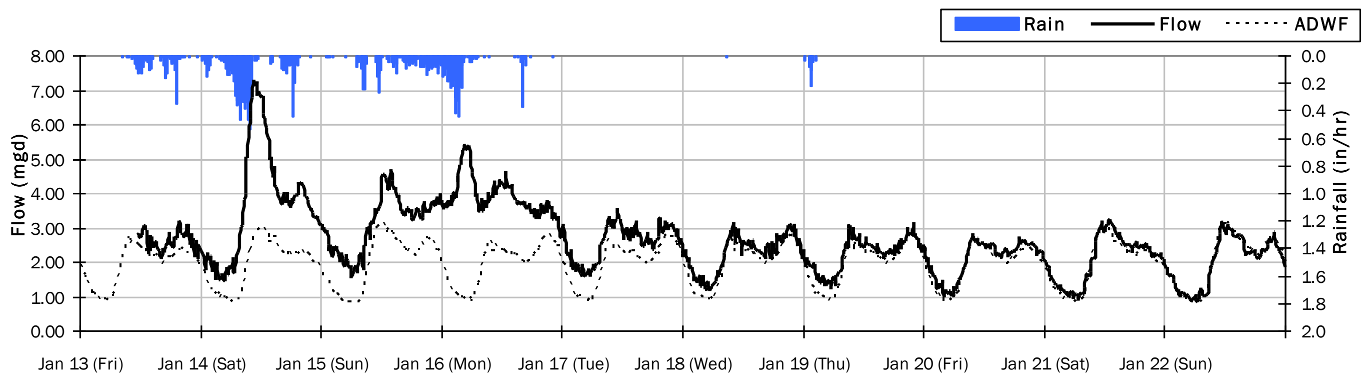
Total Rainfall: 5.75 inches

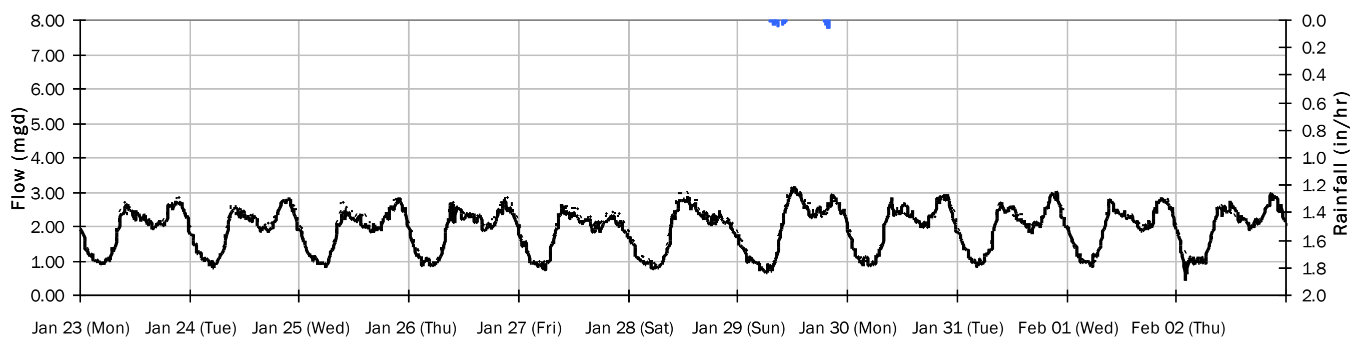
SITE 5

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 5.03 inches

Period Avg Flow: 2.269 mgd   Period Peak Flow: 7.321 mgd   Period Min Flow: 0.399 mgd

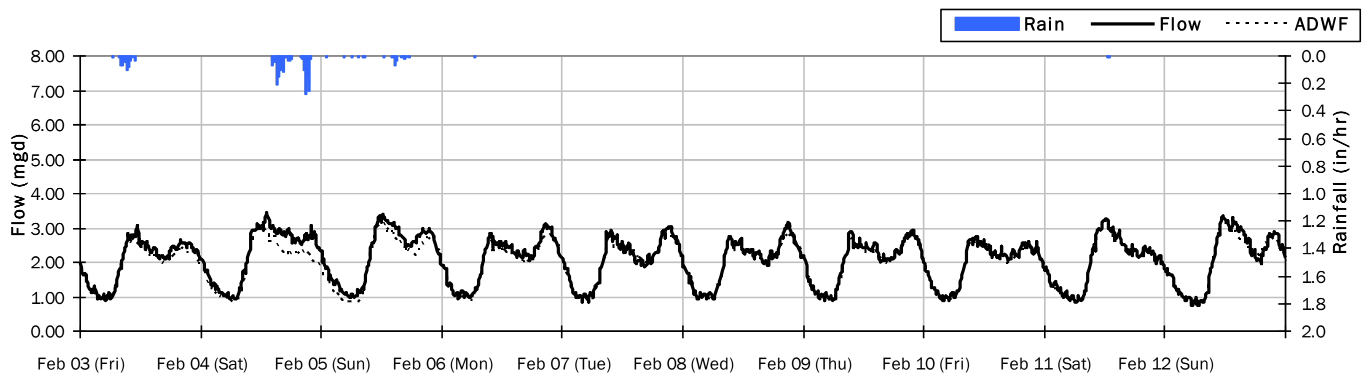


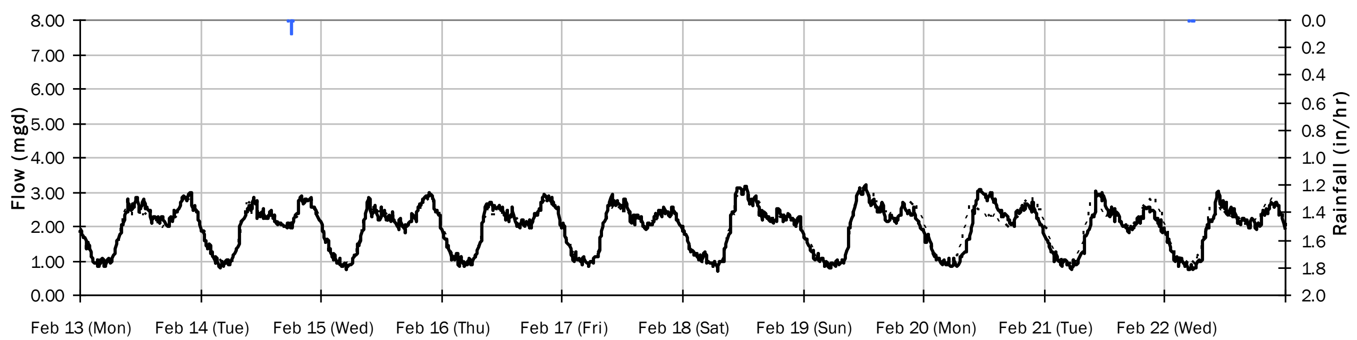


SITE 5
Flow Summary: 2/3/2023 to 2/22/2023

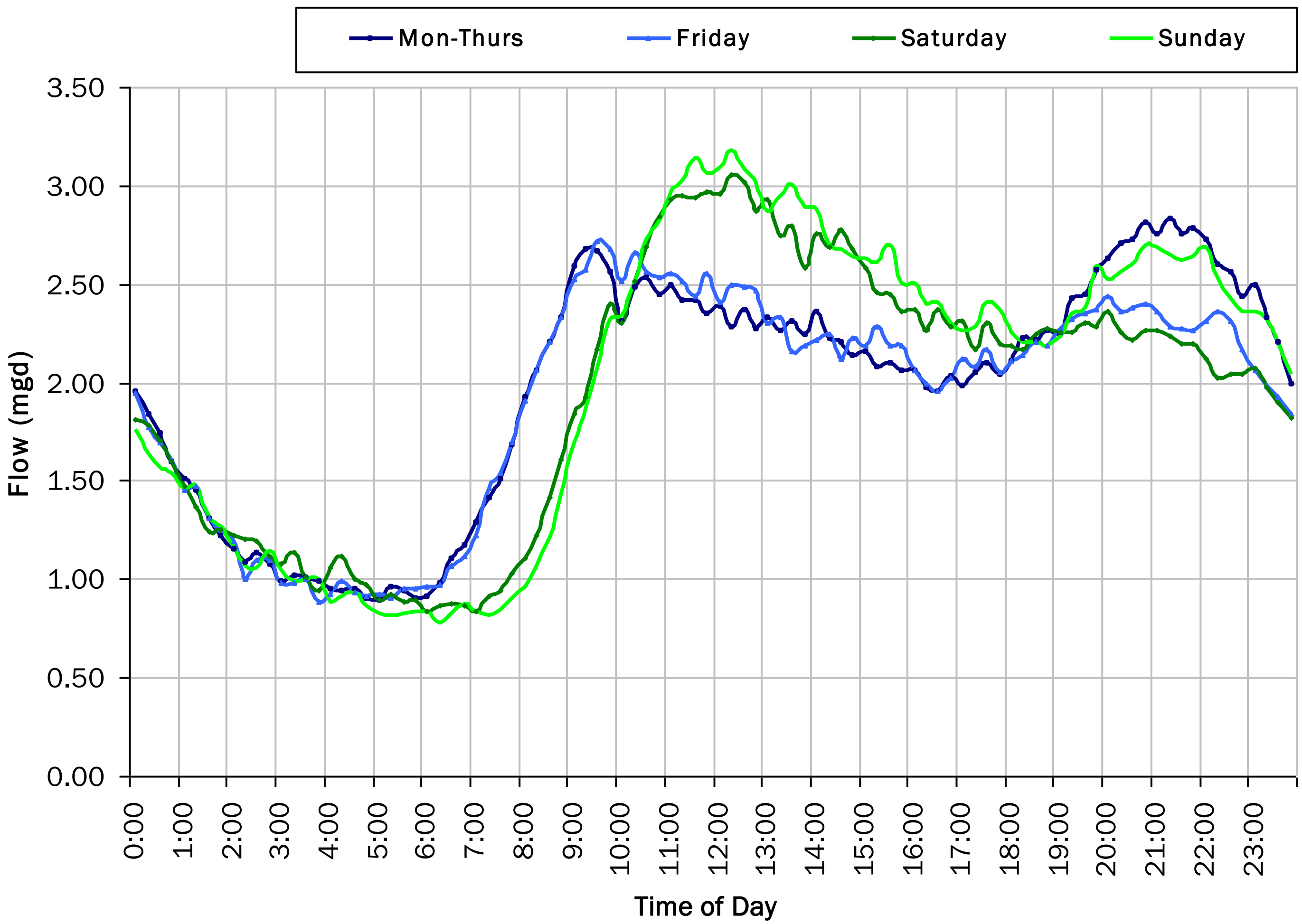
Period Rainfall: 0.77 inches

Period Avg Flow: 2.023 mgd   Period Peak Flow: 3.465 mgd   Period Min Flow: 0.702 mgd



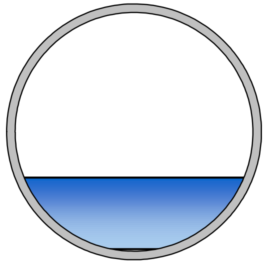


SITE 5
Average Dry Weather Flow Hydrographs



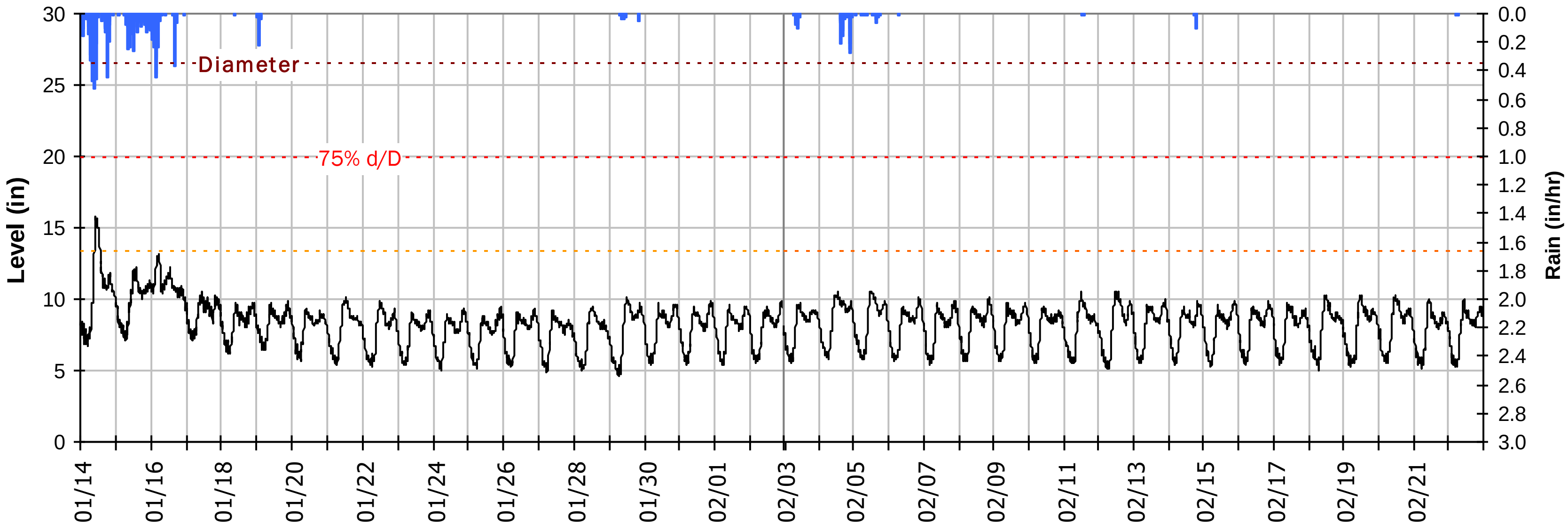
ADWF:

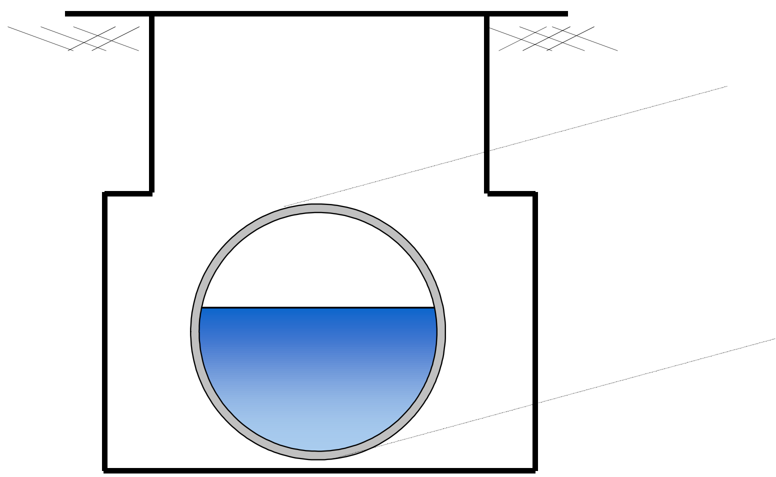
1.964 *mgd*



SITE 5
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Peak Level Period



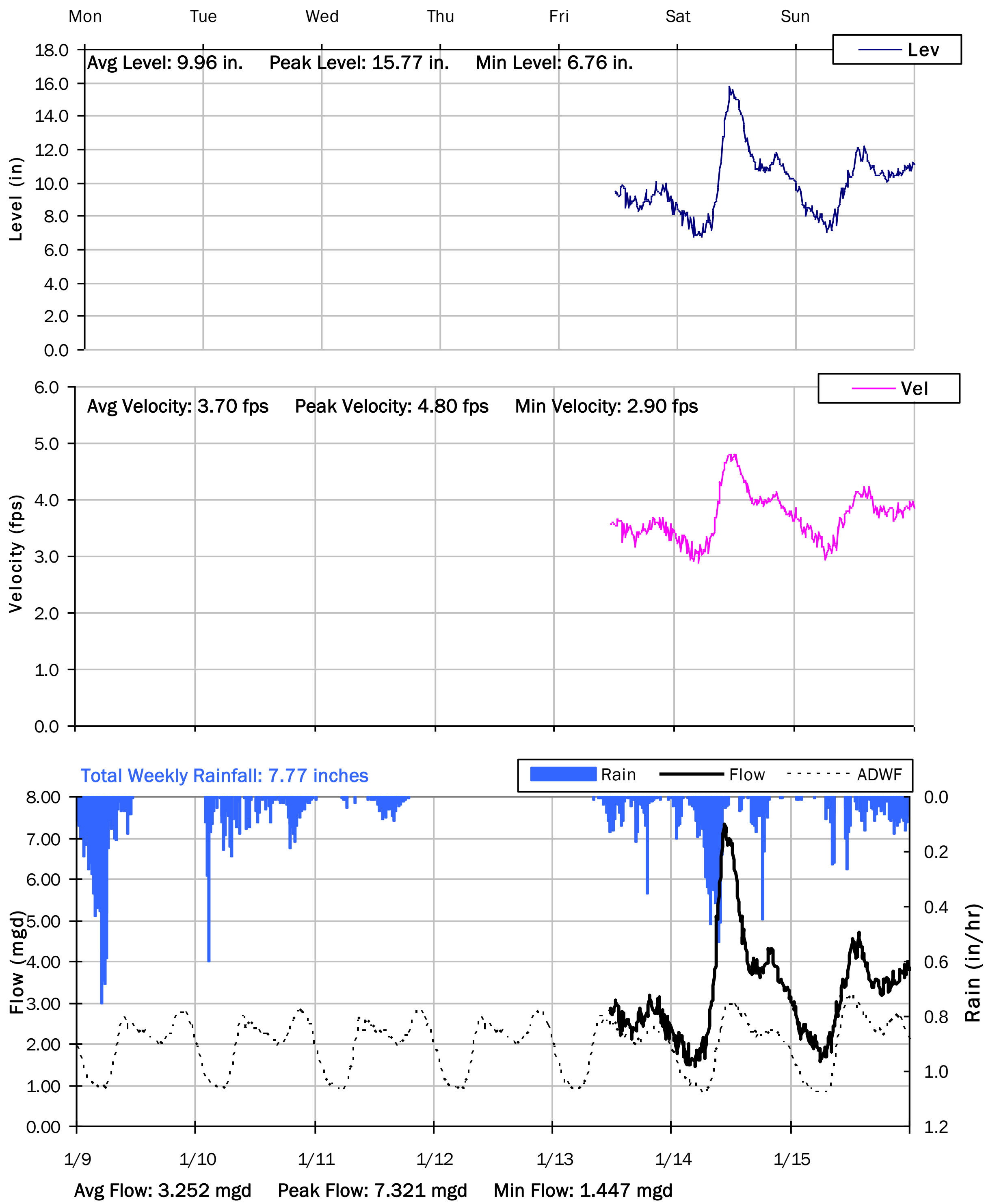


Pipe Diameter: 26.5 *inches*

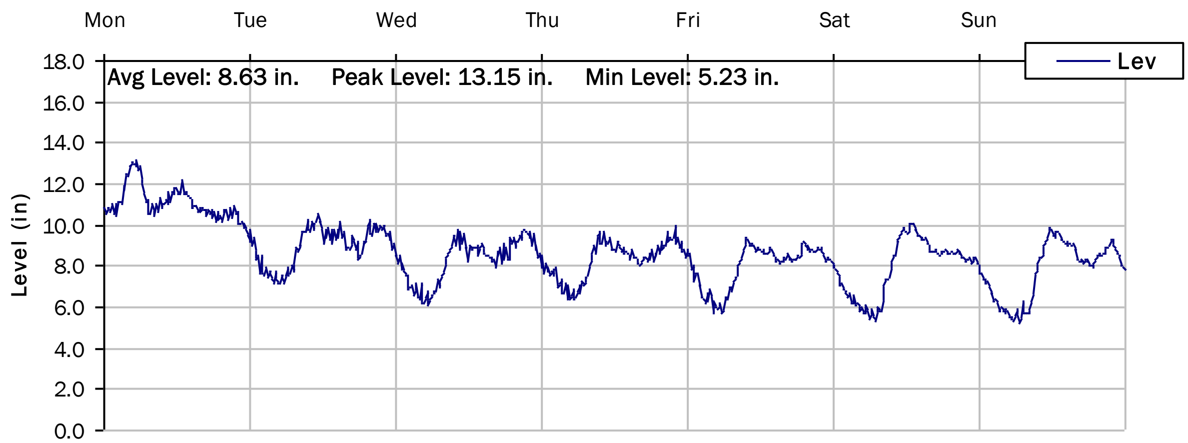
Peak Measured Level: 15.8 *inches*

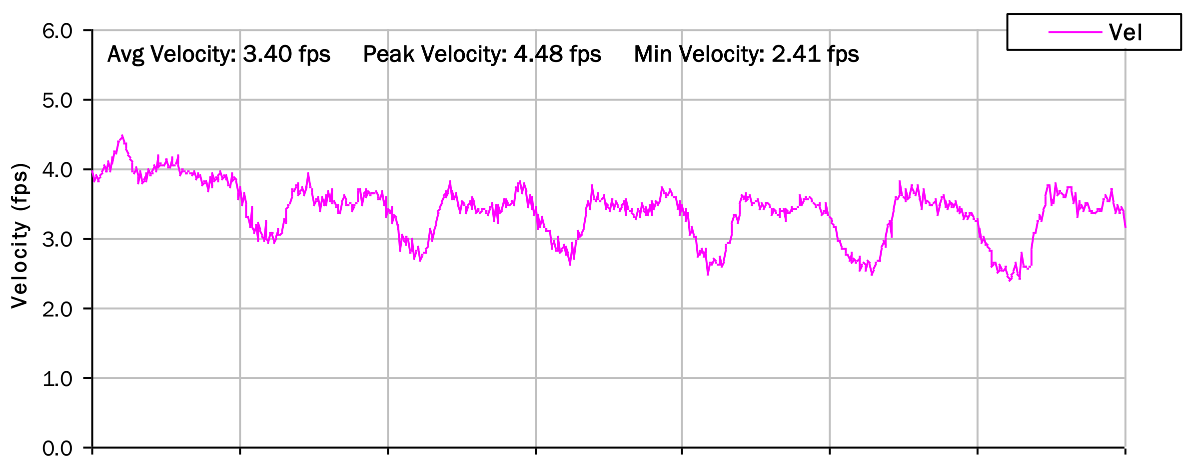
Peak d/D Ratio: 0.60

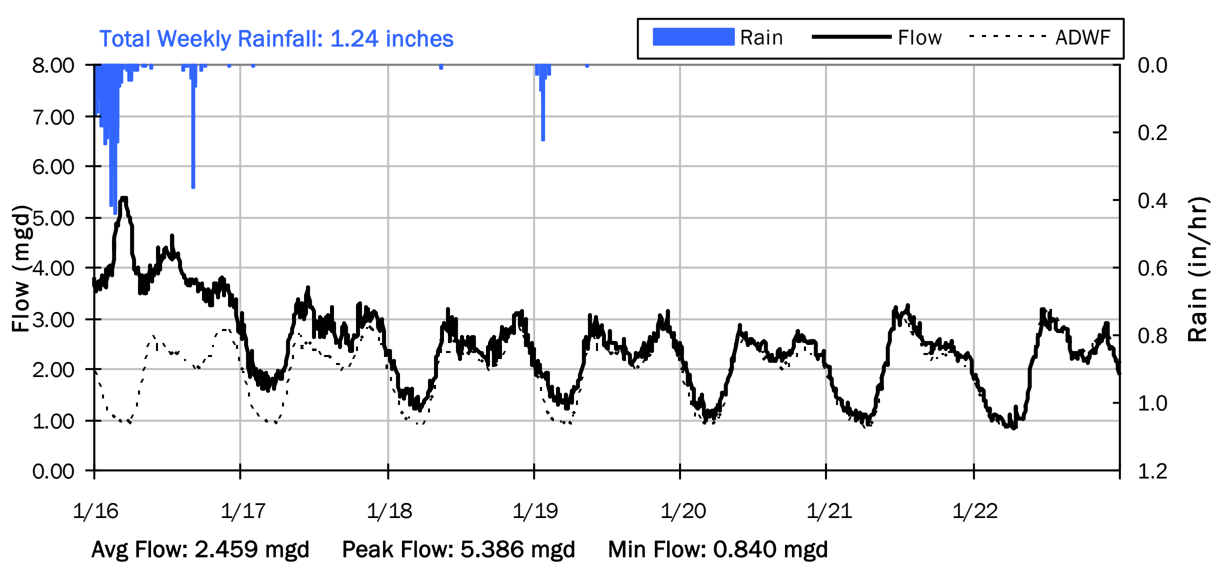
SITE 5
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023



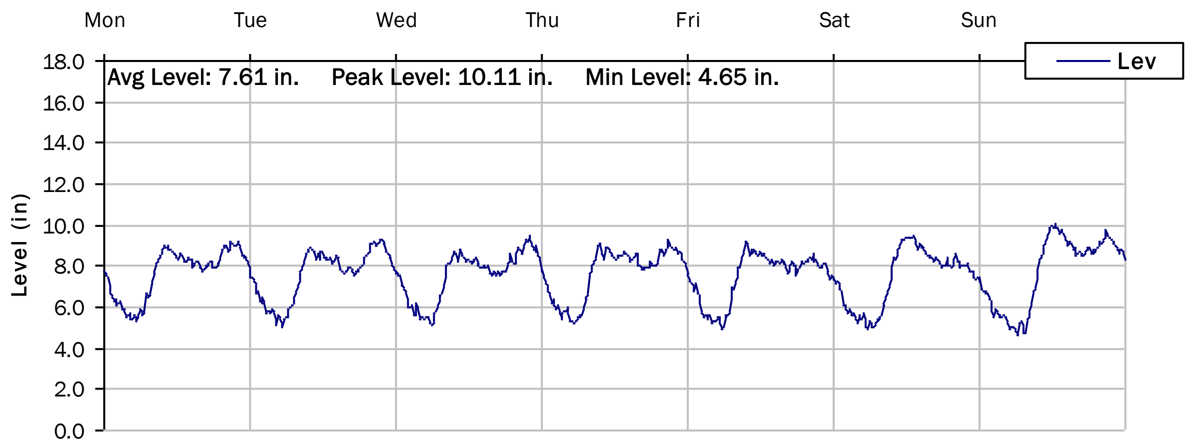
SITE 5
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023

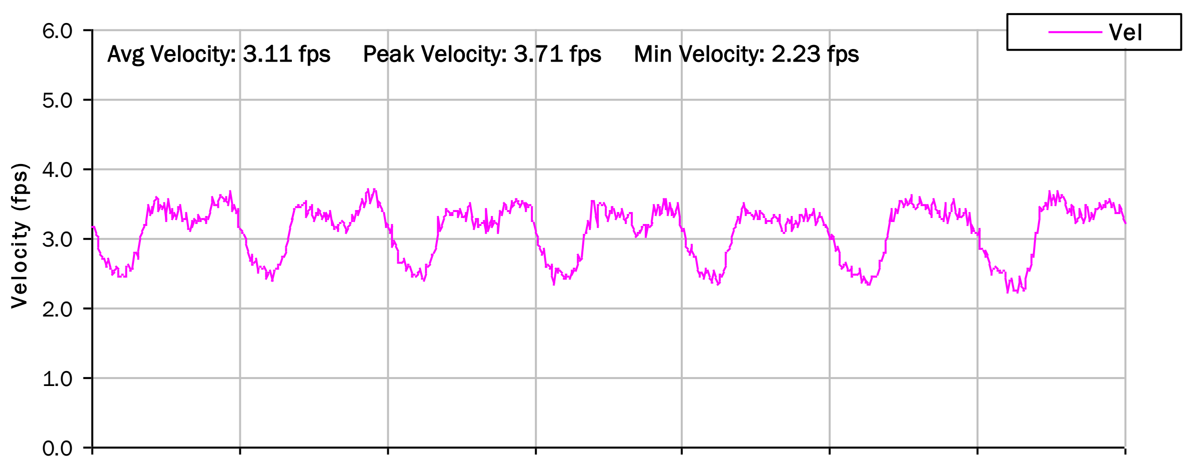


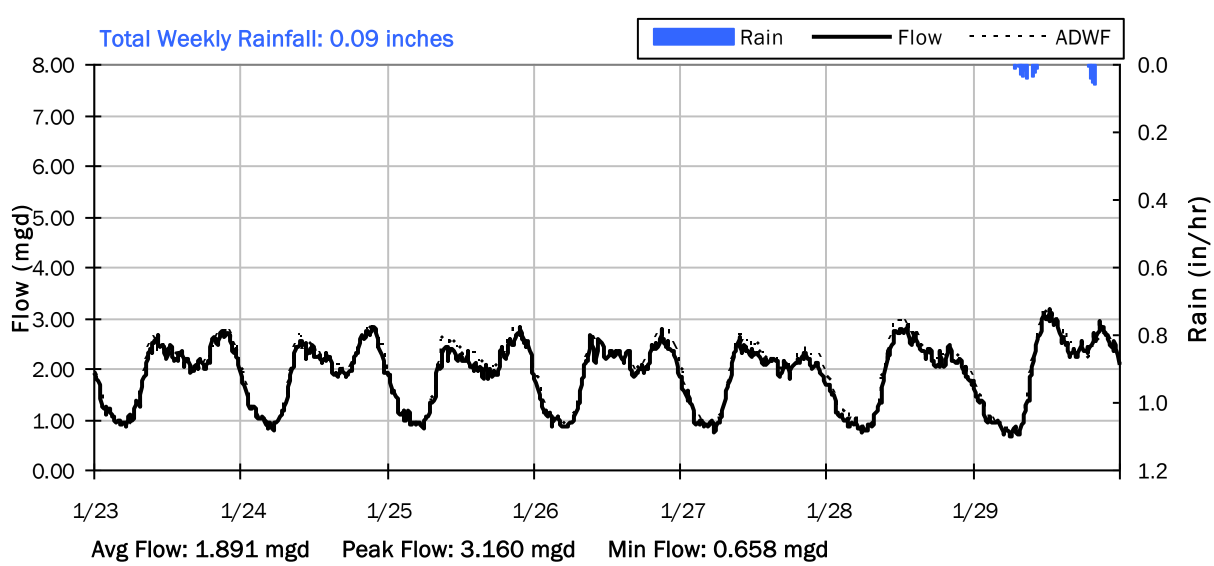




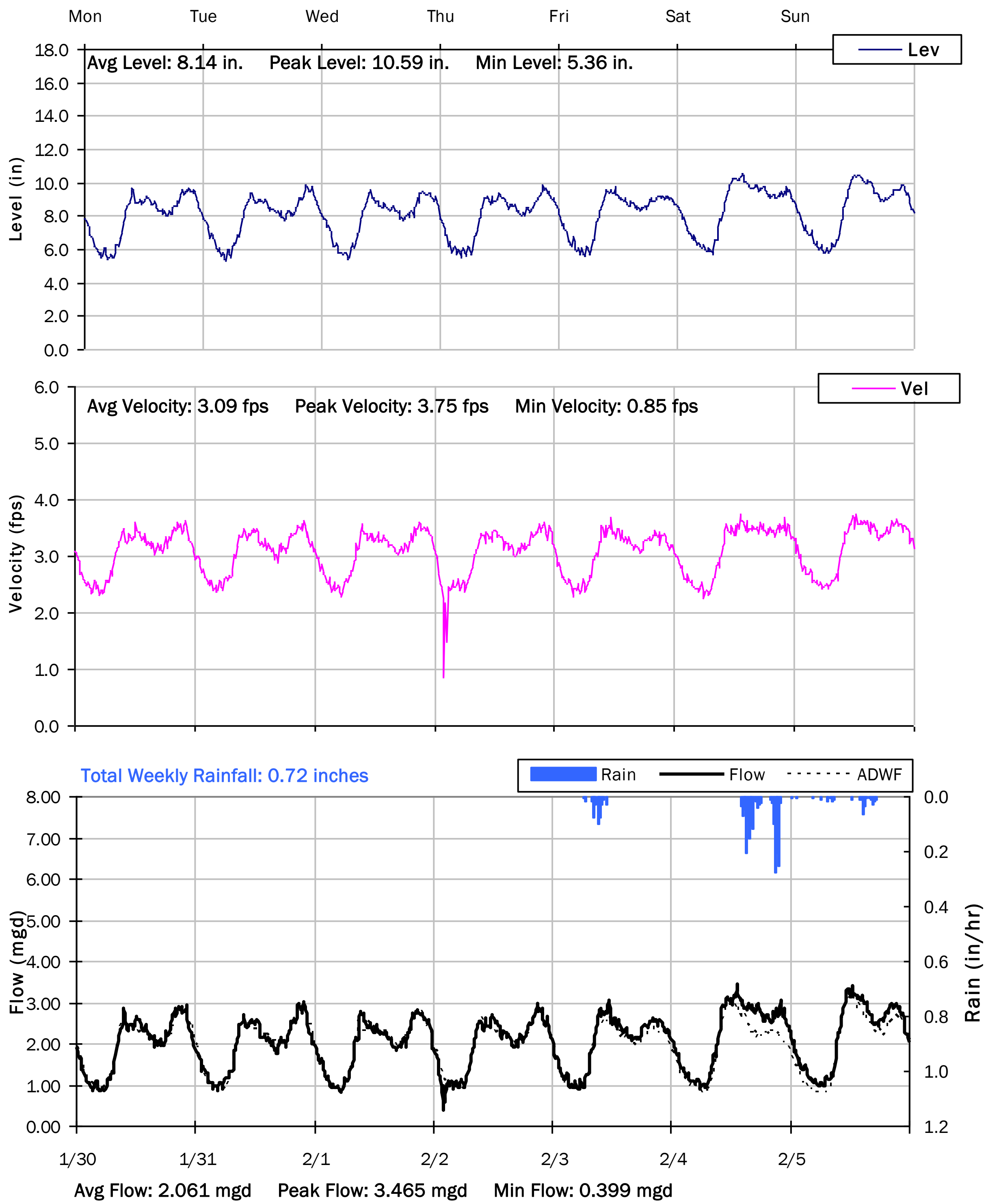
SITE 5
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023



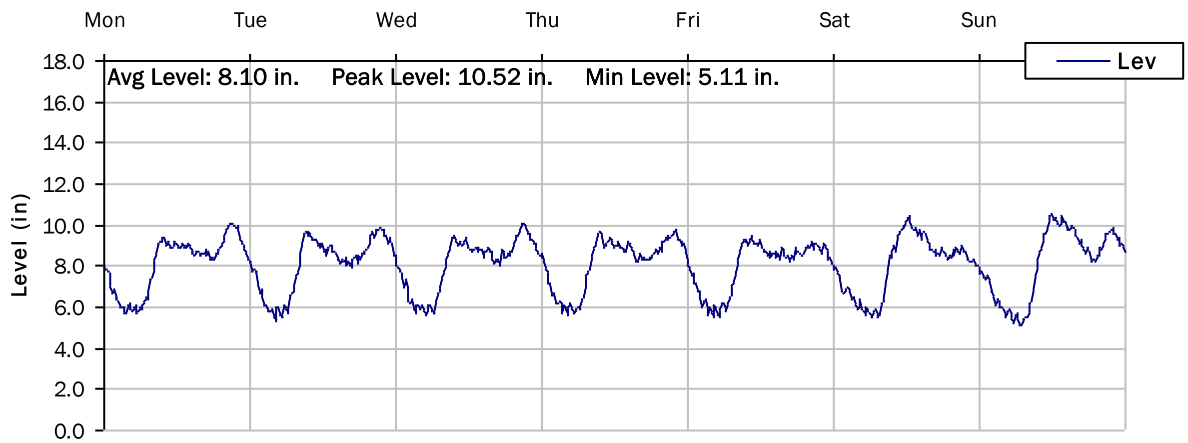


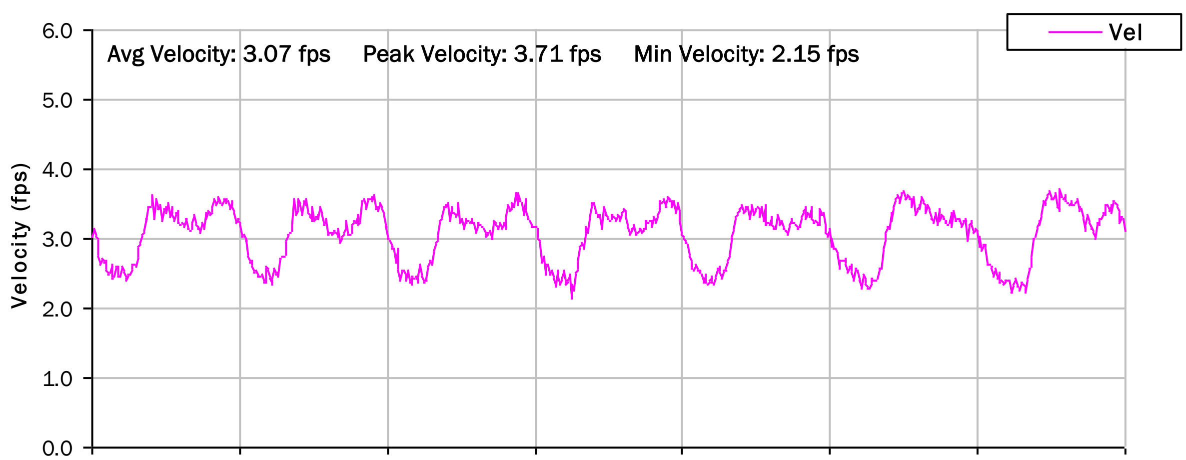


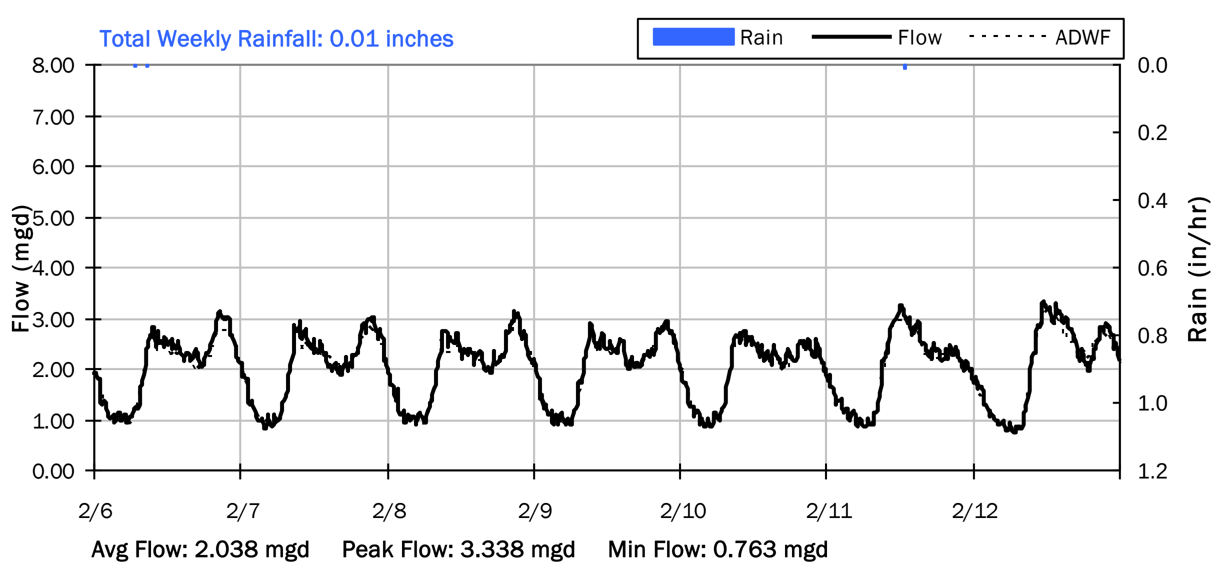
SITE 5
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023



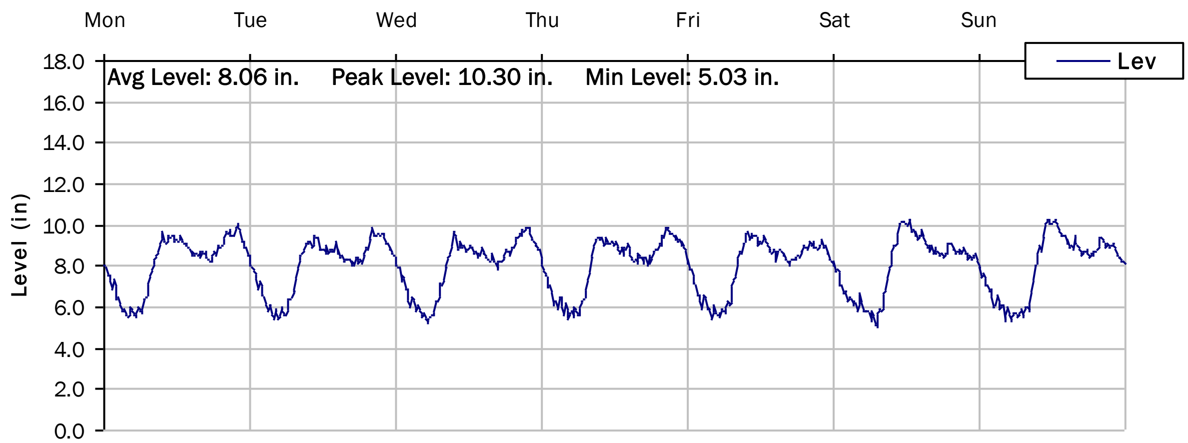
SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

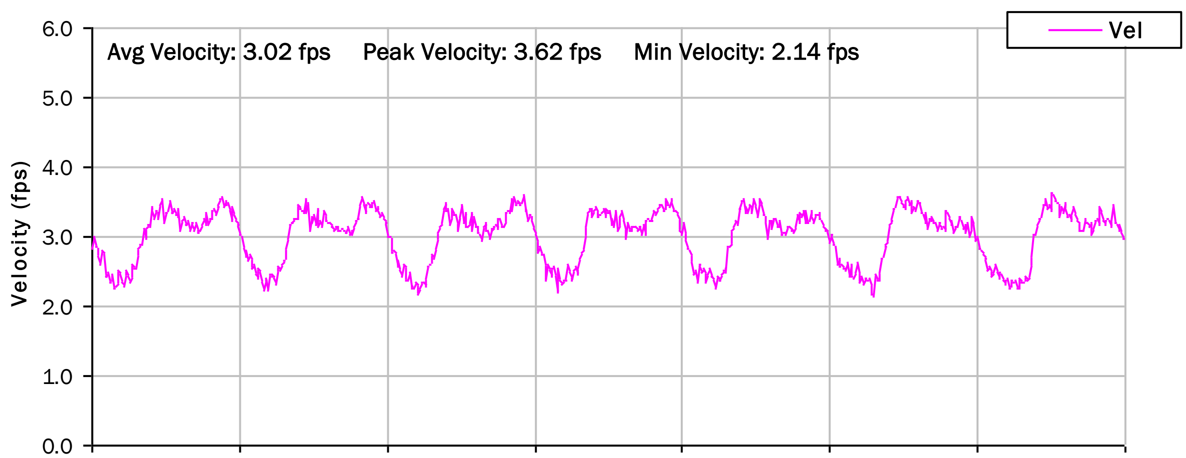


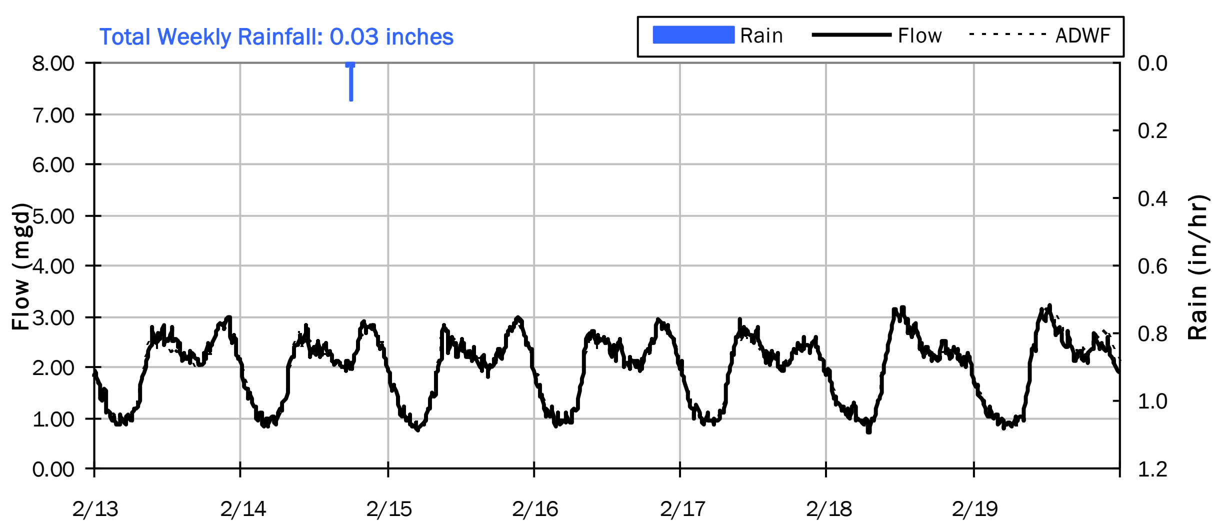




SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023

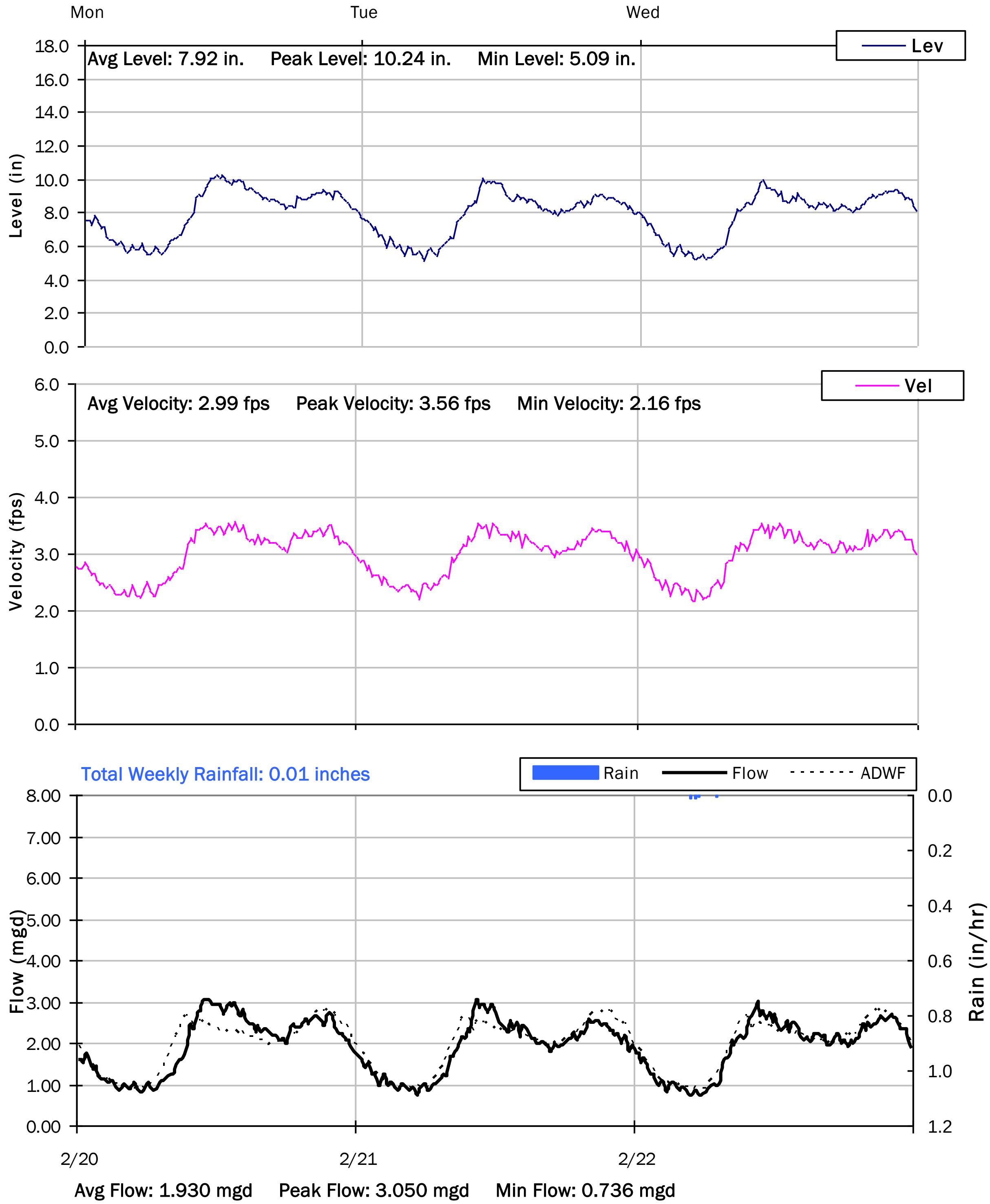






Avg Flow: 1.984 mgd Peak Flow: 3.215 mgd Min Flow: 0.702 mgd

SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023



Monitoring Site: Site 6

City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: 339 E Dunne Ave

Data Summary Report

Vicinity Map: Site 6

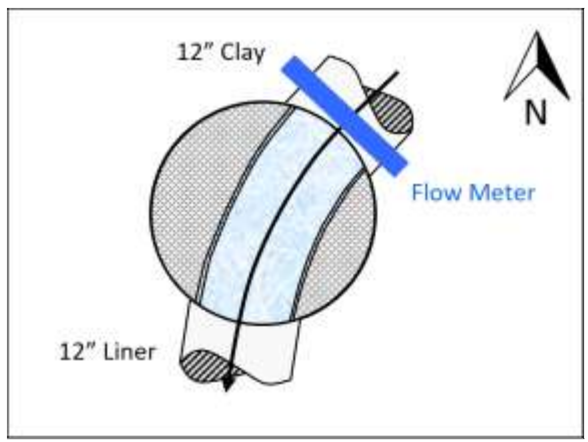
SITE 6

Site Information MH ID: G5-C.MH.055

Location:	339 E Dunne Ave
Coordinates:	121.3840° W, 37.7387° N
Rim Elevation:	350 feet
Expected Pipe Diameter:	12 inches
Measured Pipe Diameter:	12 inches
ADWF:	0.142 mgd
Peak Measured Flow:	0.85 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch

Street View


Plan View

SITE 6

Additional Site Photos

South Effluent Pipe

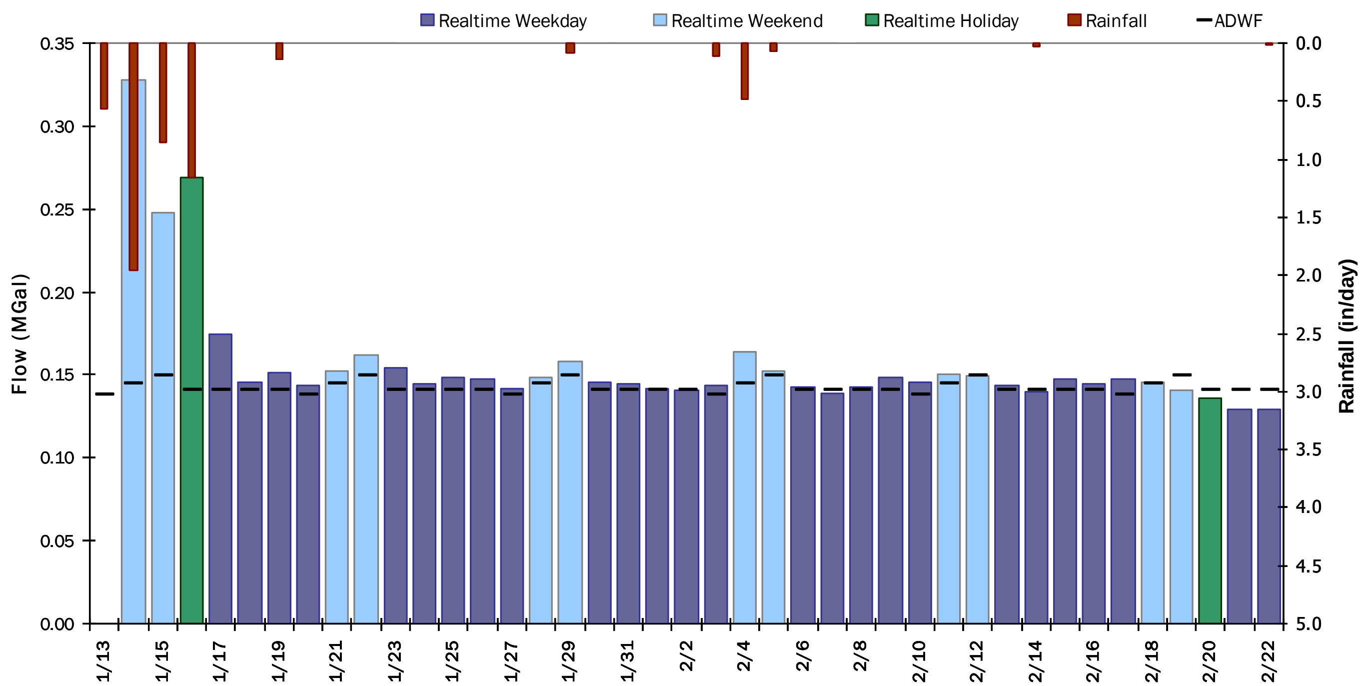
Monitored Northeast Influent Pipe

SITE 6

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.158 MGal Peak Daily Flow: 0.328 MGal Min Daily Flow: 0.129 MGal

Total Rainfall: 5.48 inches

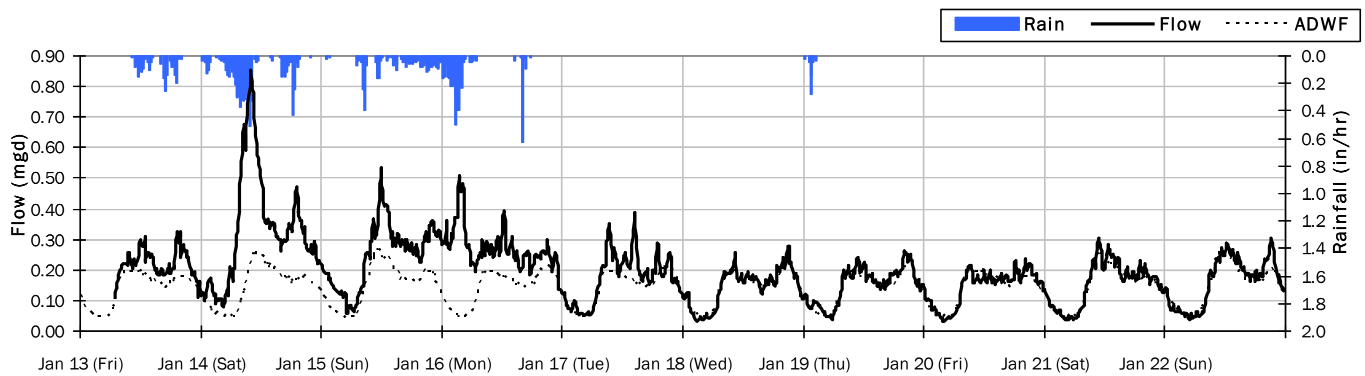


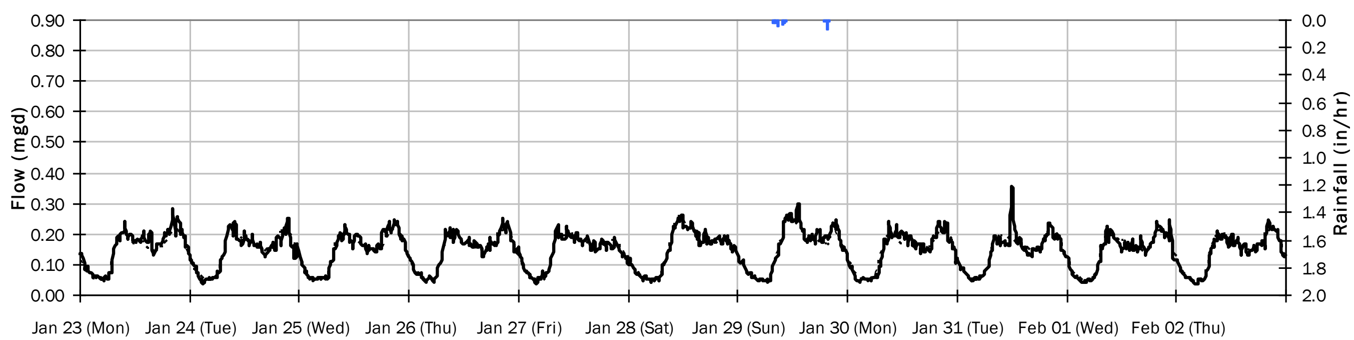
SITE 6

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 4.76 inches

Period Avg Flow: 0.171 mgd Period Peak Flow: 0.852 mgd Period Min Flow: 0.029 mgd



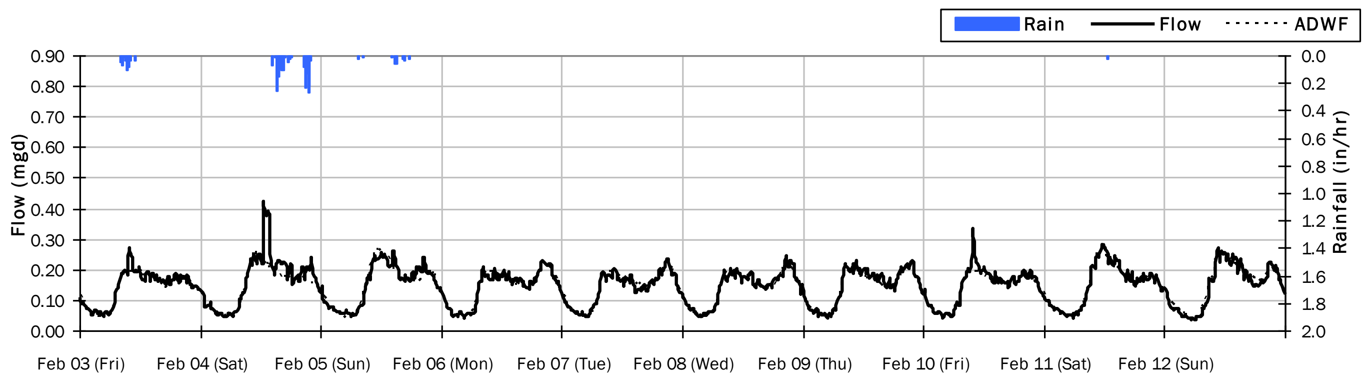


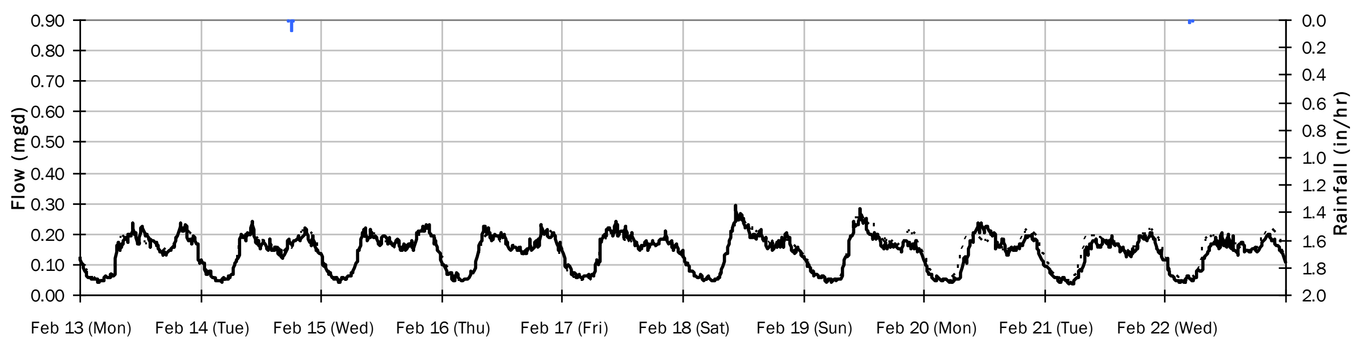
SITE 6

Flow Summary: 2/3/2023 to 2/22/2023

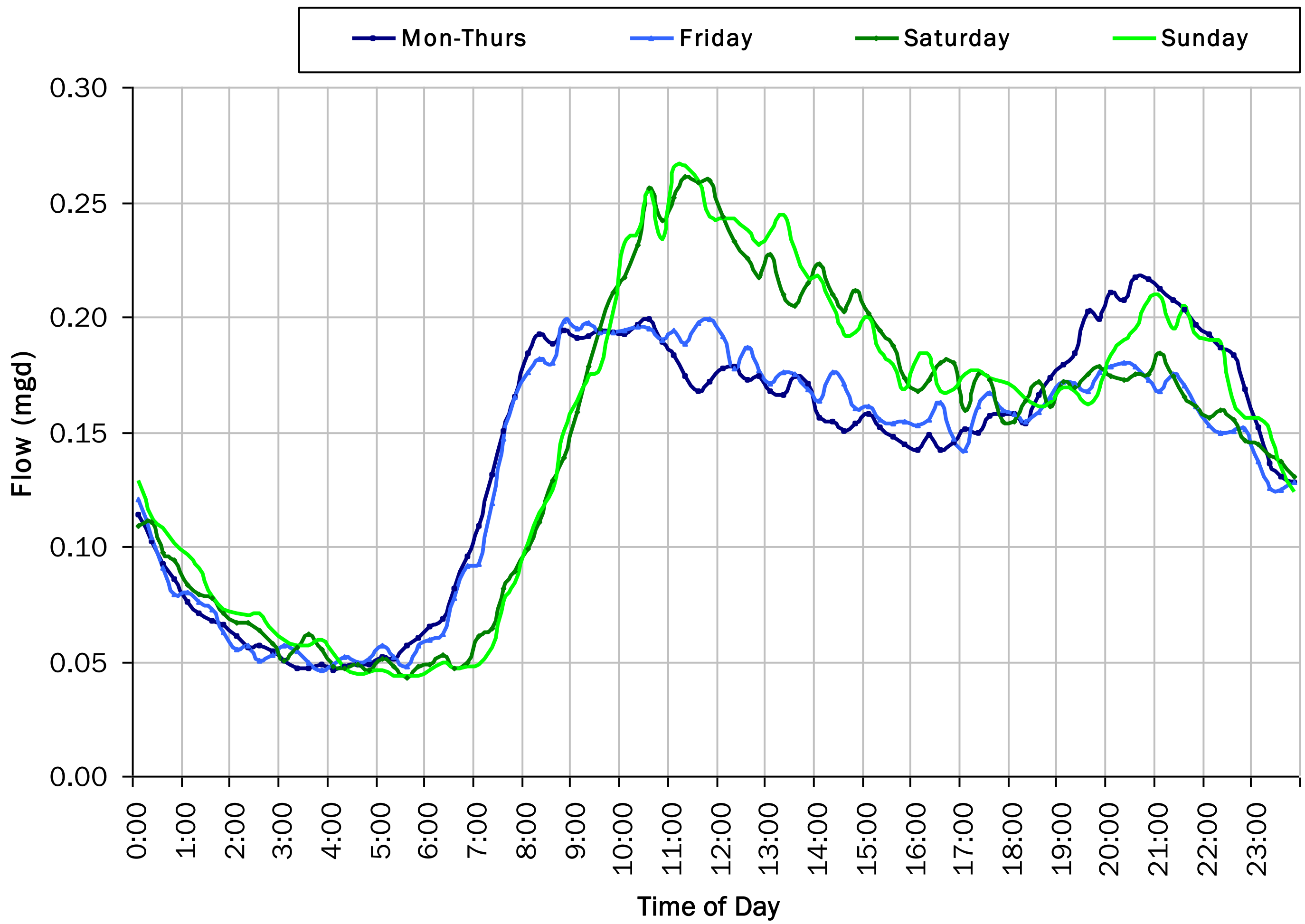
Period Rainfall: 0.71 inches

Period Avg Flow: 0.144 mgd   Period Peak Flow: 0.419 mgd   Period Min Flow: 0.038 mgd

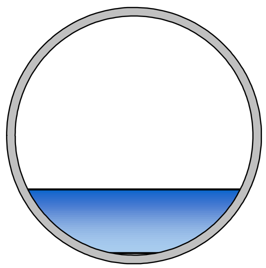




SITE 6
Average Dry Weather Flow Hydrographs

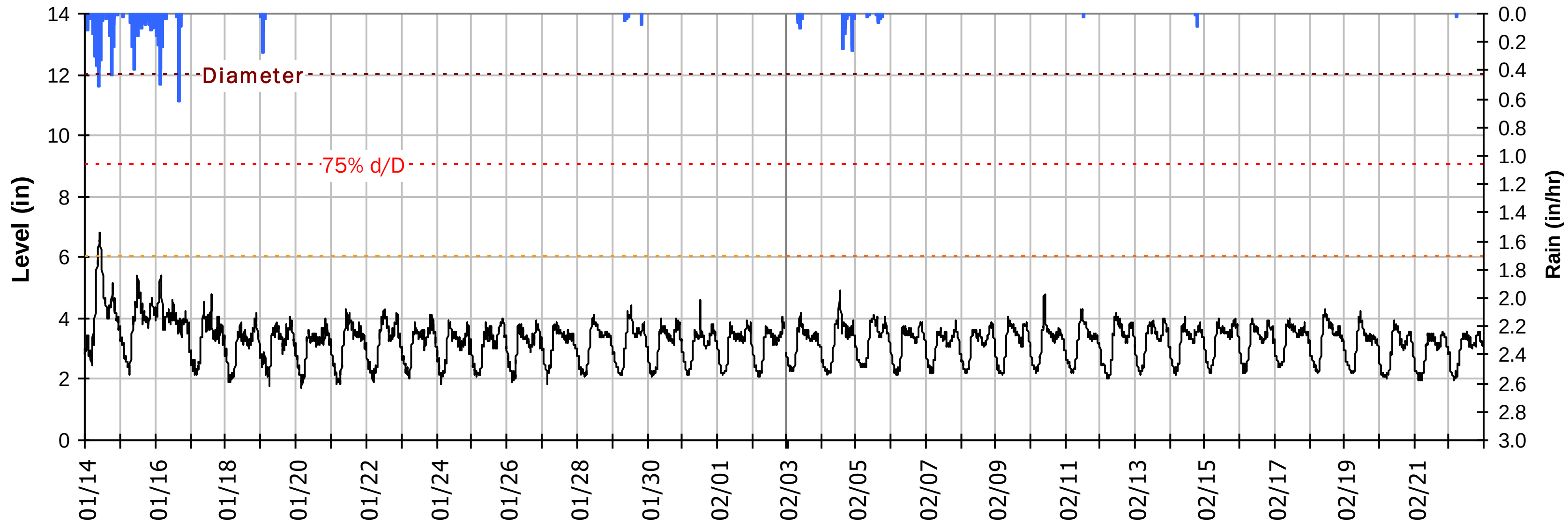


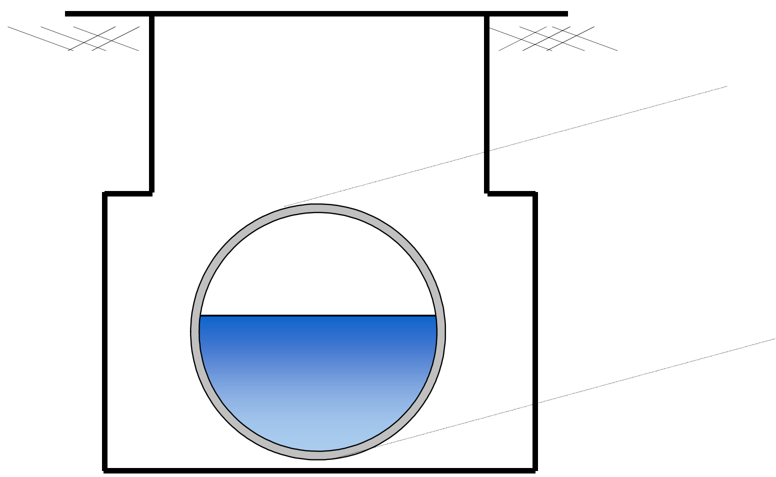
ADWF:
0.142 *mgd*



SITE 6
Site Capacity and Surcharge Summary

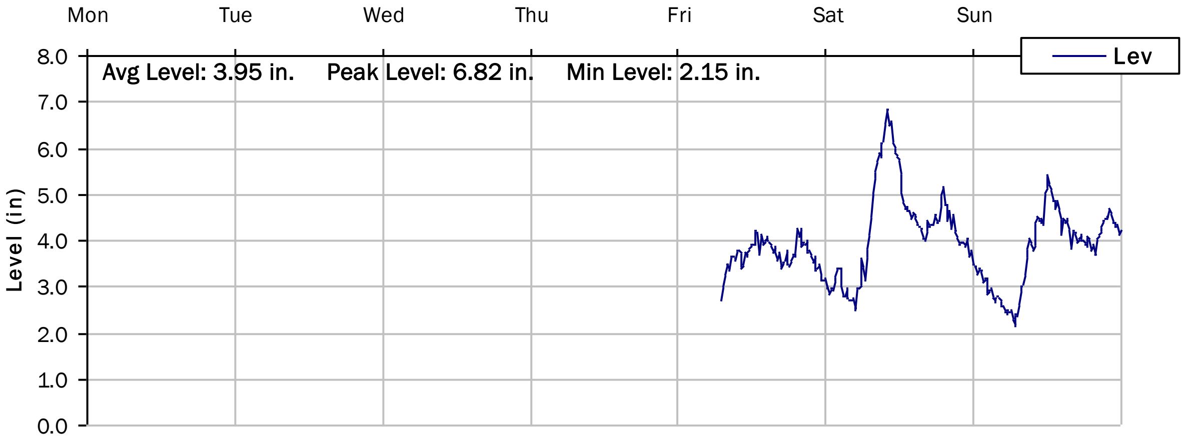
Realtime Flow Levels with Rainfall Data over Peak Level Period

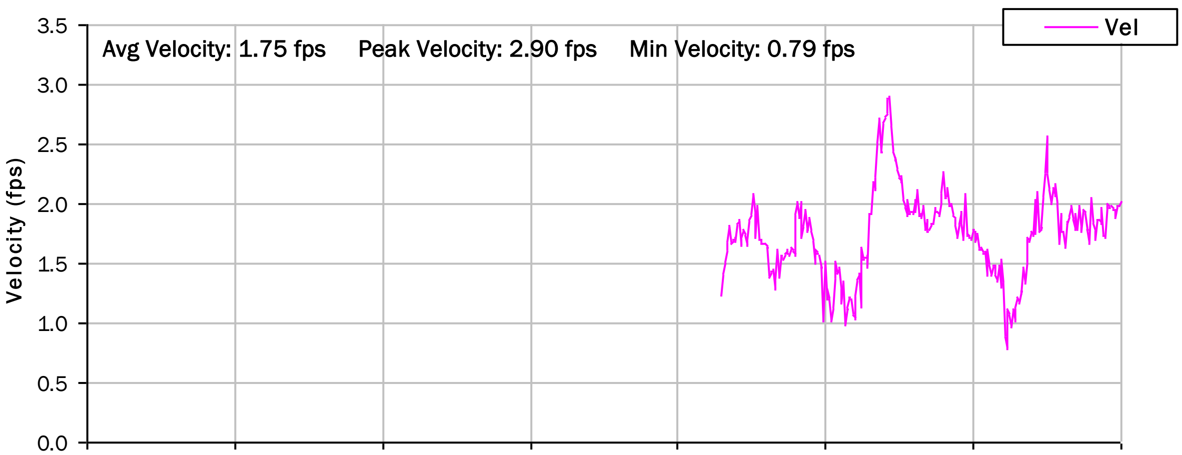


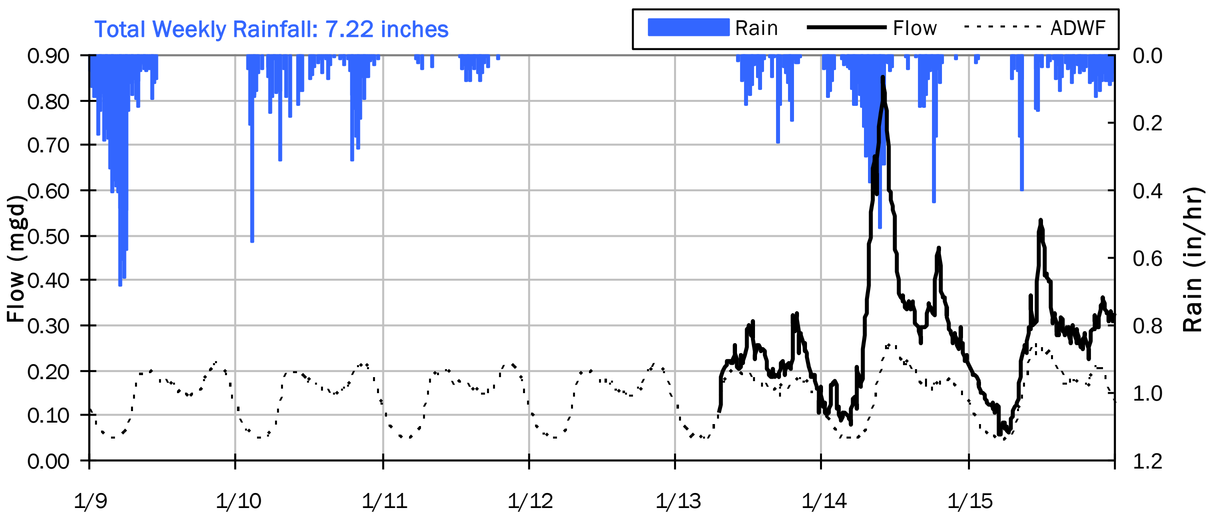


Pipe Diameter:	12	*inches*
Peak Measured Level:	6.82	*inches*
Peak d/D Ratio:	0.57	

SITE 6
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023

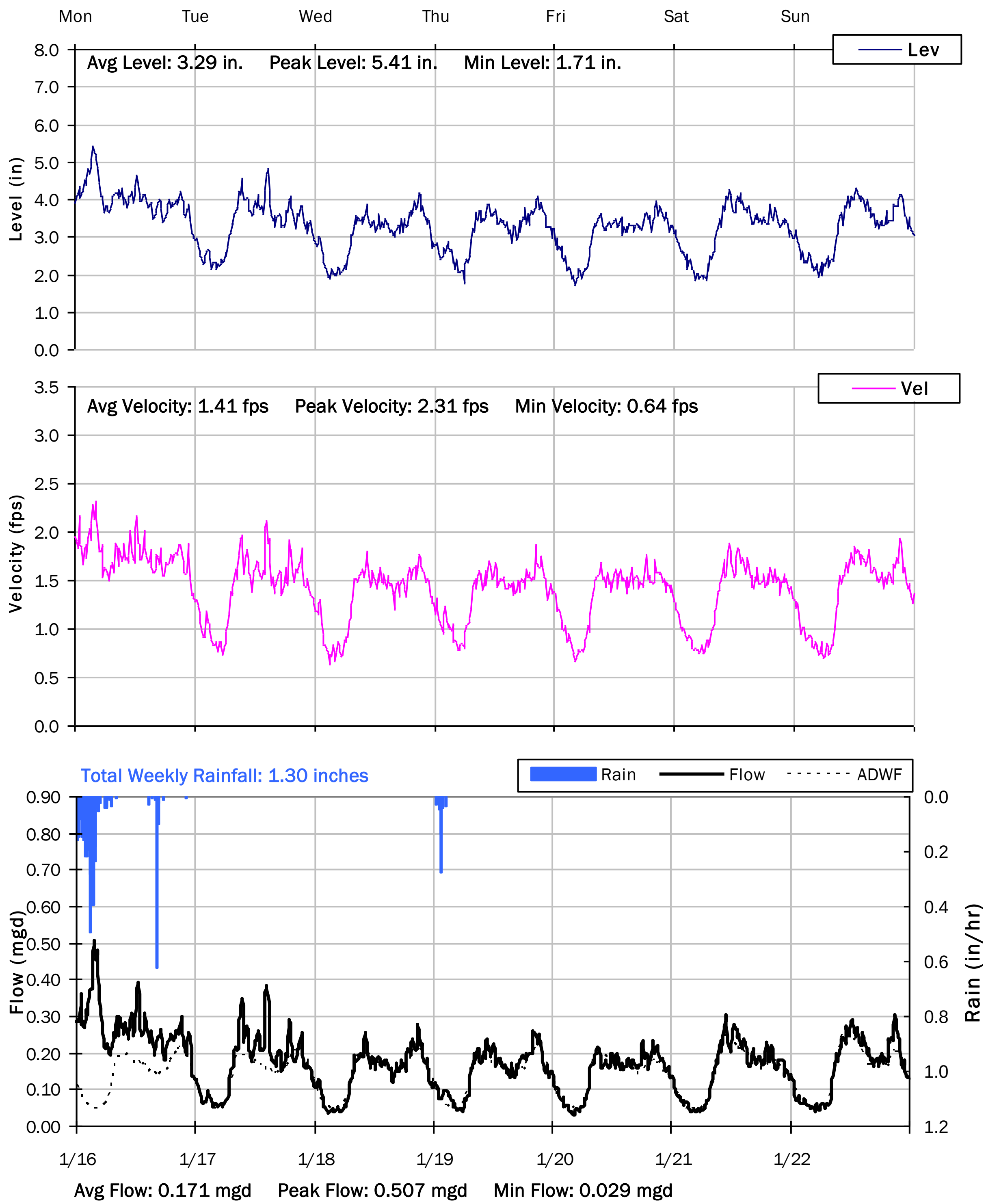




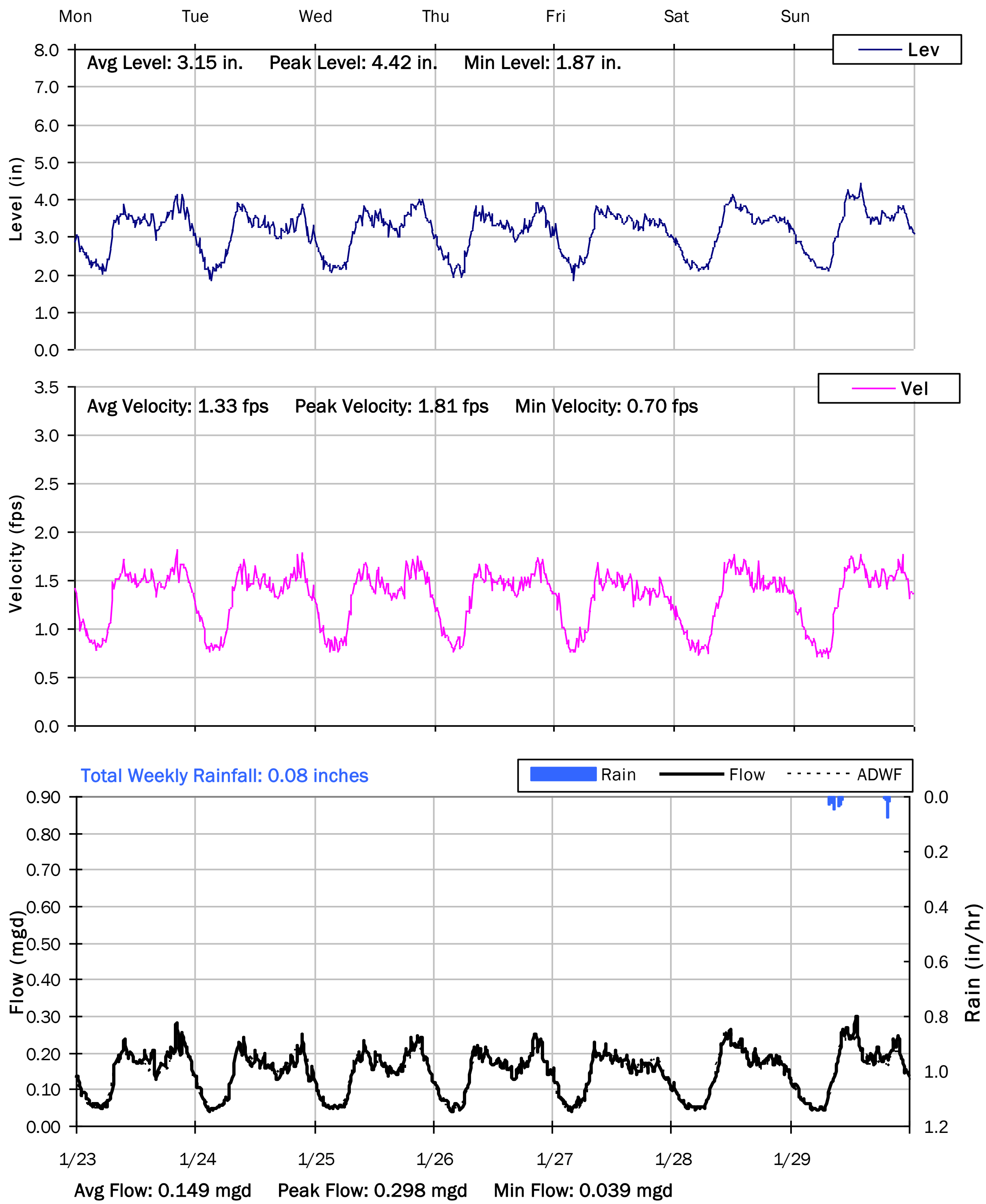


Avg Flow: 0.271 mgd Peak Flow: 0.852 mgd Min Flow: 0.058 mgd

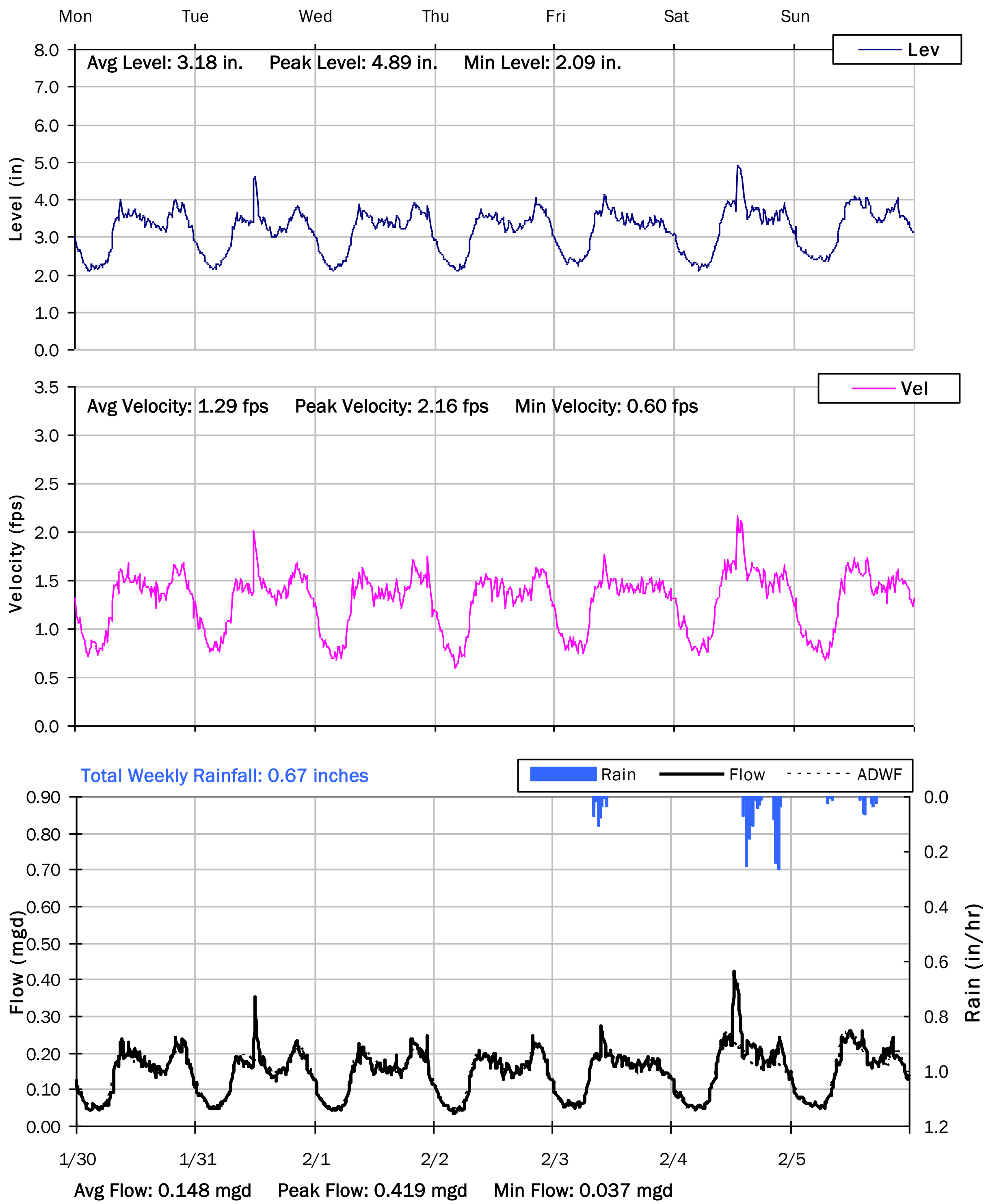
SITE 6
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023



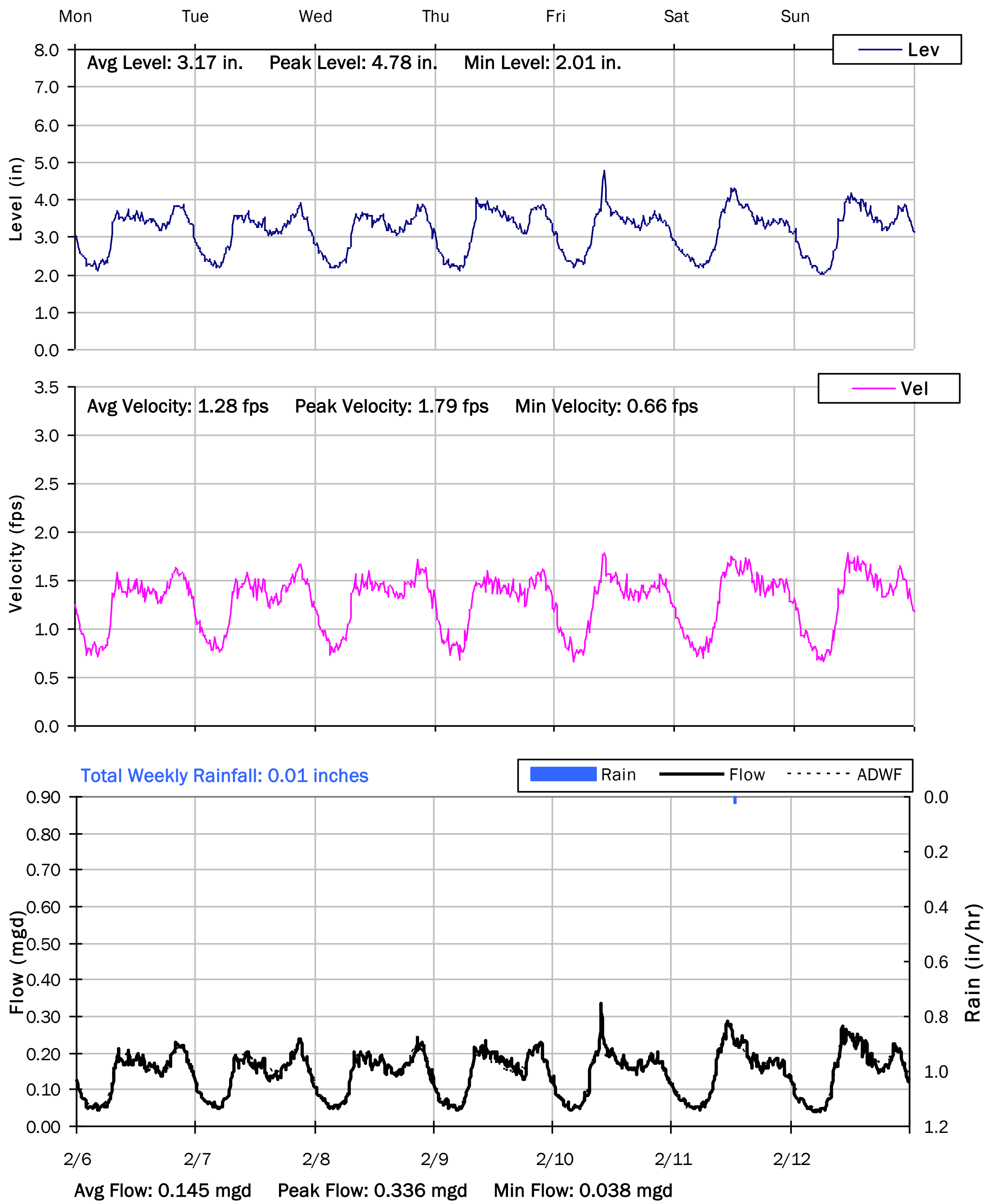
SITE 6
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023



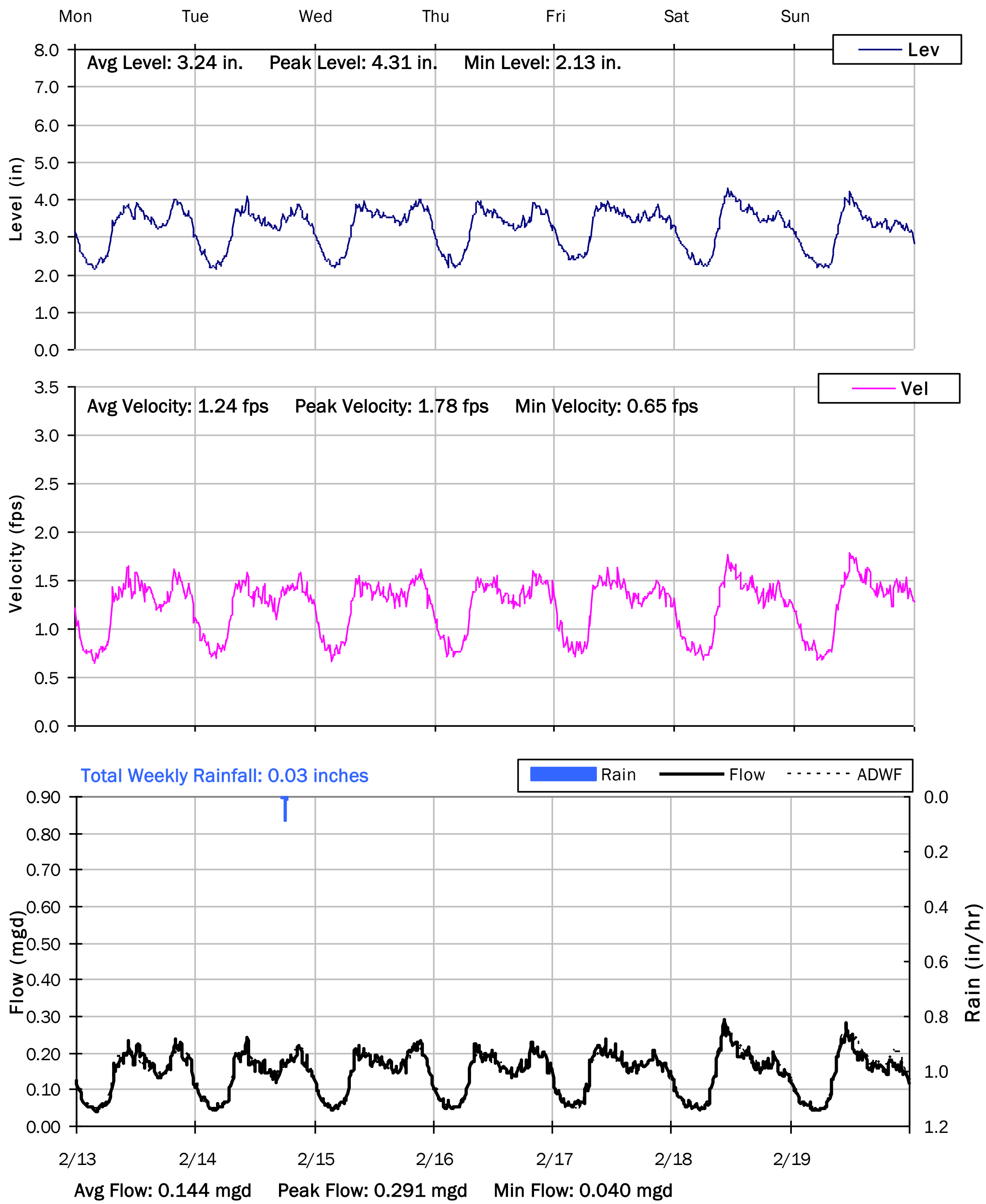
SITE 6
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023



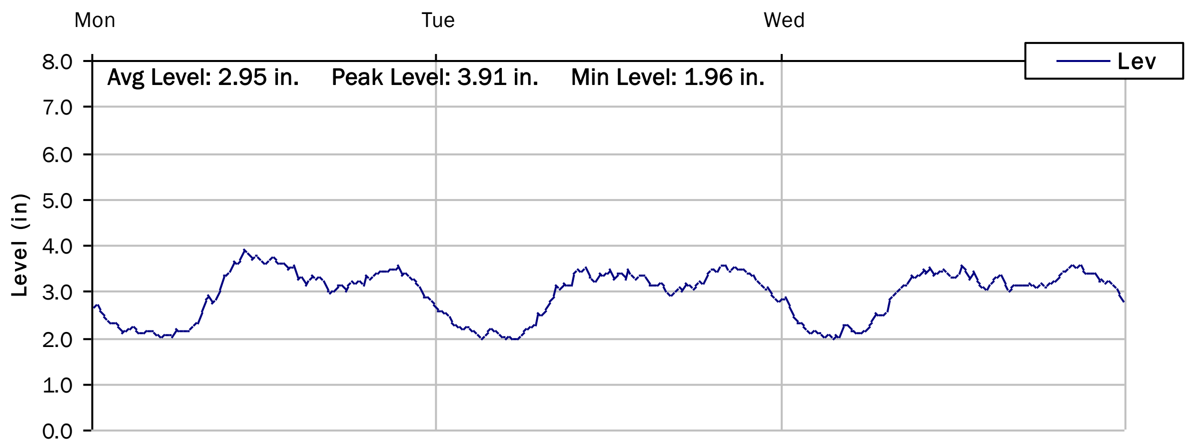
SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

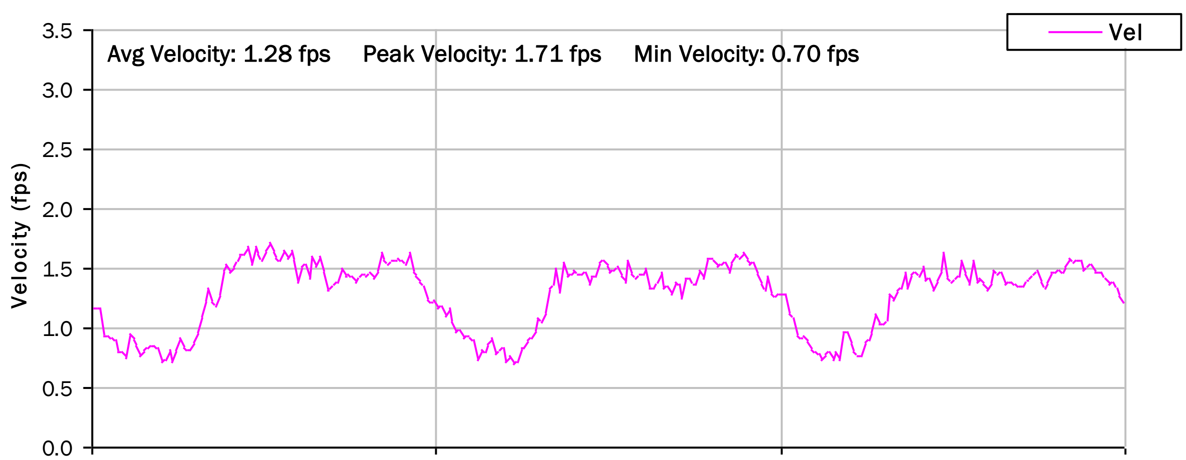


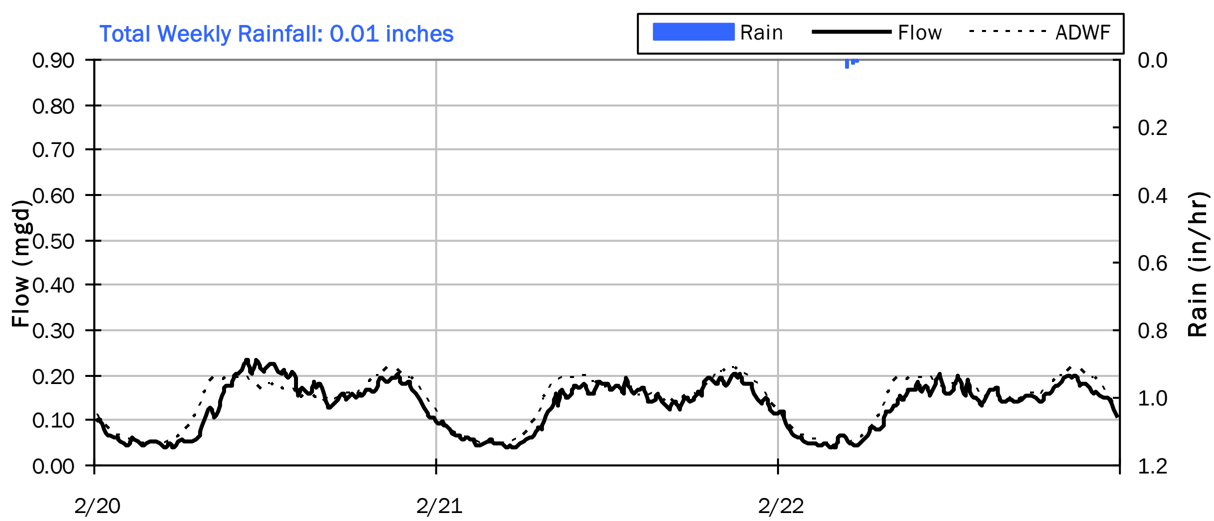
SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023







Avg Flow: 0.131 mgd   Peak Flow: 0.235 mgd   Min Flow: 0.038 mgd

Monitoring Site: Site 7

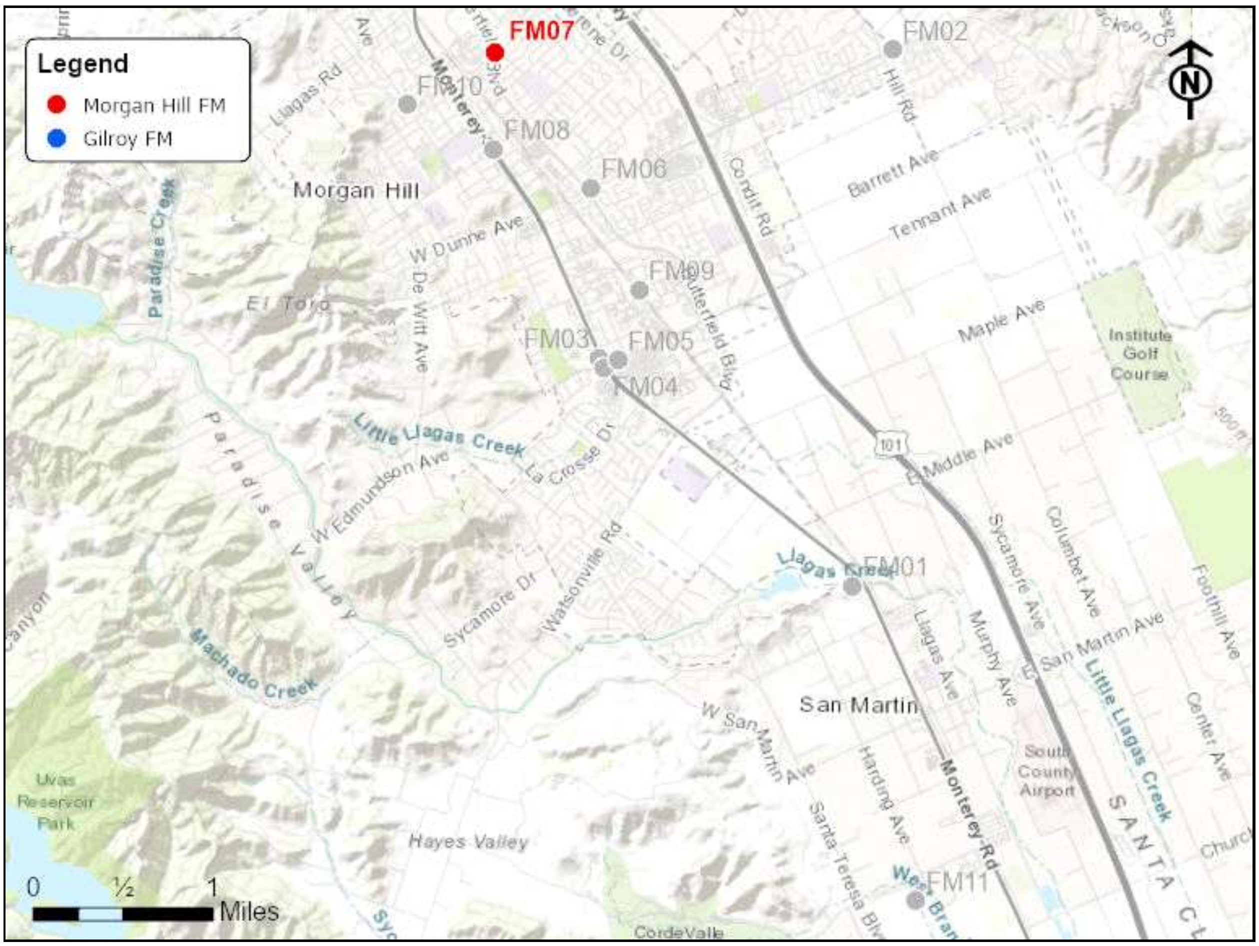
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: 18160 Butterfeild Blvd

Data Summary Report



Vicinity Map: Site 7

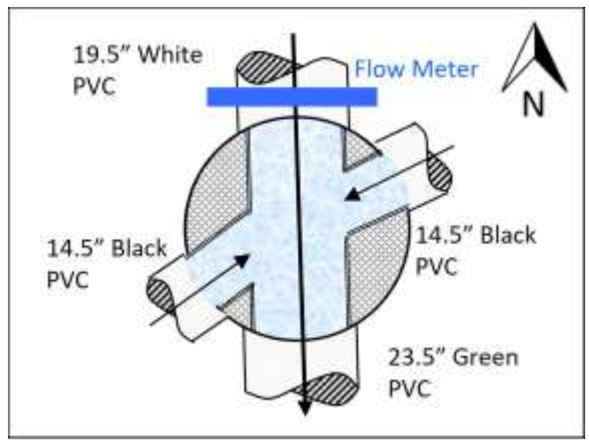
SITE 7

Site Information — MH ID: F4-D.MH.006

Location:	18160 Butterfeild Blvd
Coordinates:	121.3952° W, 37.8178° N
Rim Elevation:	360 feet
Expected Pipe Diameter:	12 inches
Measured Pipe Diameter:	19.5 inches
ADWF:	0.394 mgd
Peak Measured Flow:	1.20 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch

Street View

Plan View

SITE 7

Additional Site Photos

South Effluent Pipe

Southwest Influent Pipe

SITE 7

Additional Site Photos

Monitored North Influent Pipe

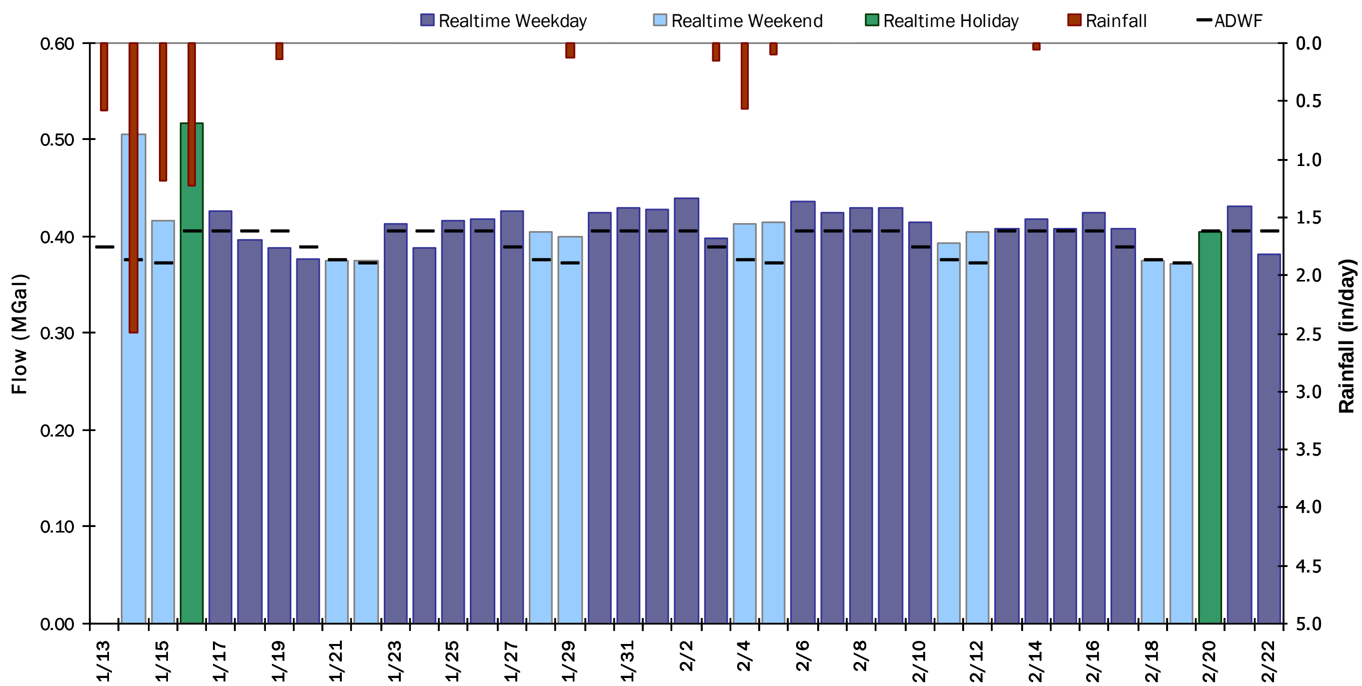
Northeast Influent Pipe

SITE 7

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.416 MGal Peak Daily Flow: 0.517 MGal Min Daily Flow: 0.371 MGal

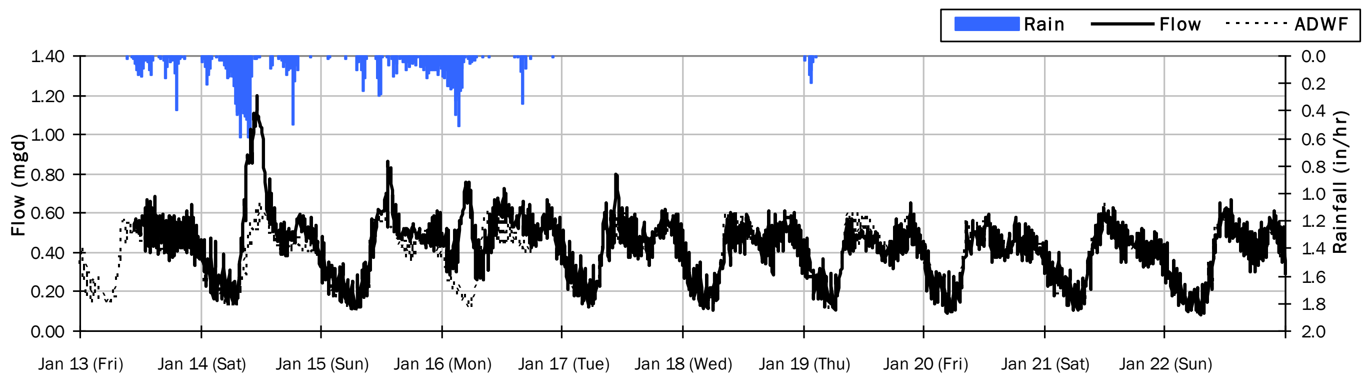
Total Rainfall: 6.62 inches

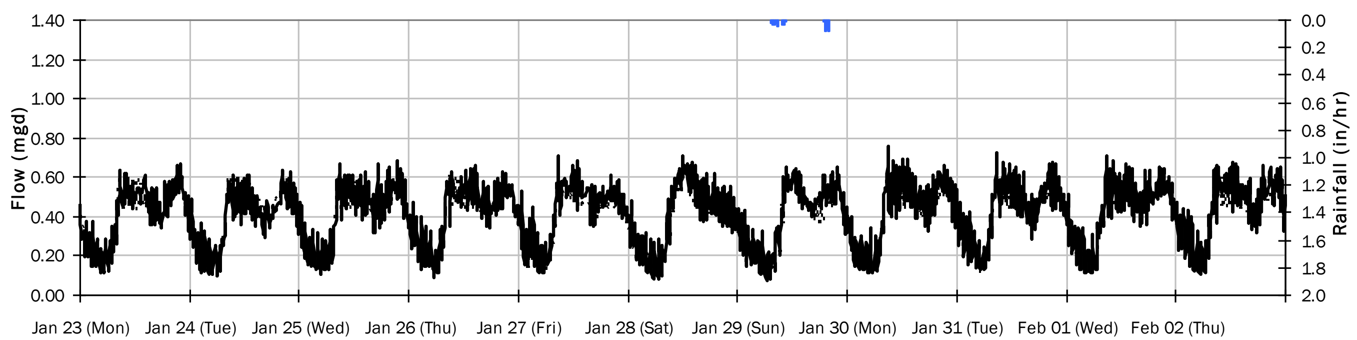


SITE 7

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 5.75 inches   Period Avg Flow: 0.421 mgd   Period Peak Flow: 1.199 mgd   Period Min Flow: 0.077 mgd

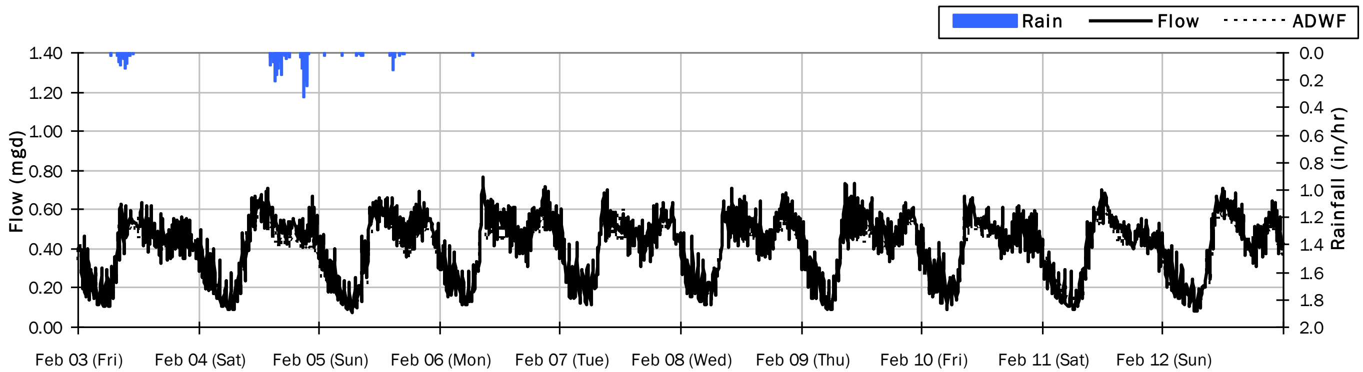


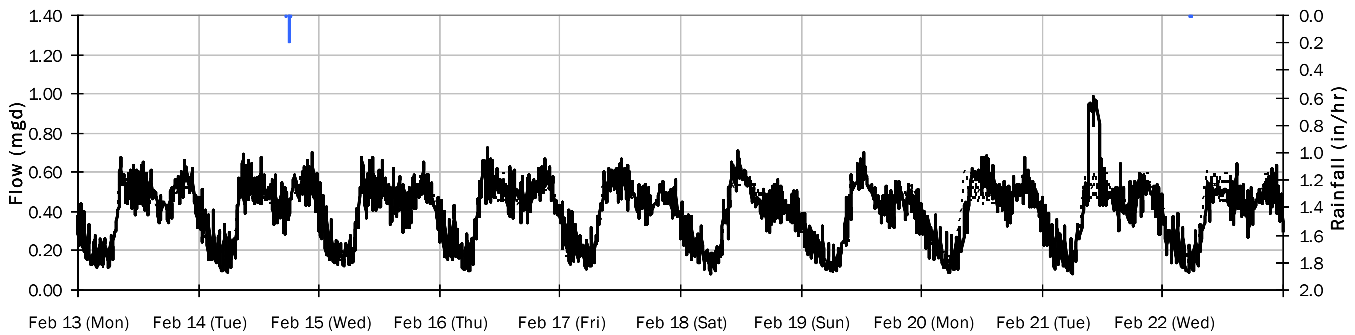


SITE 7

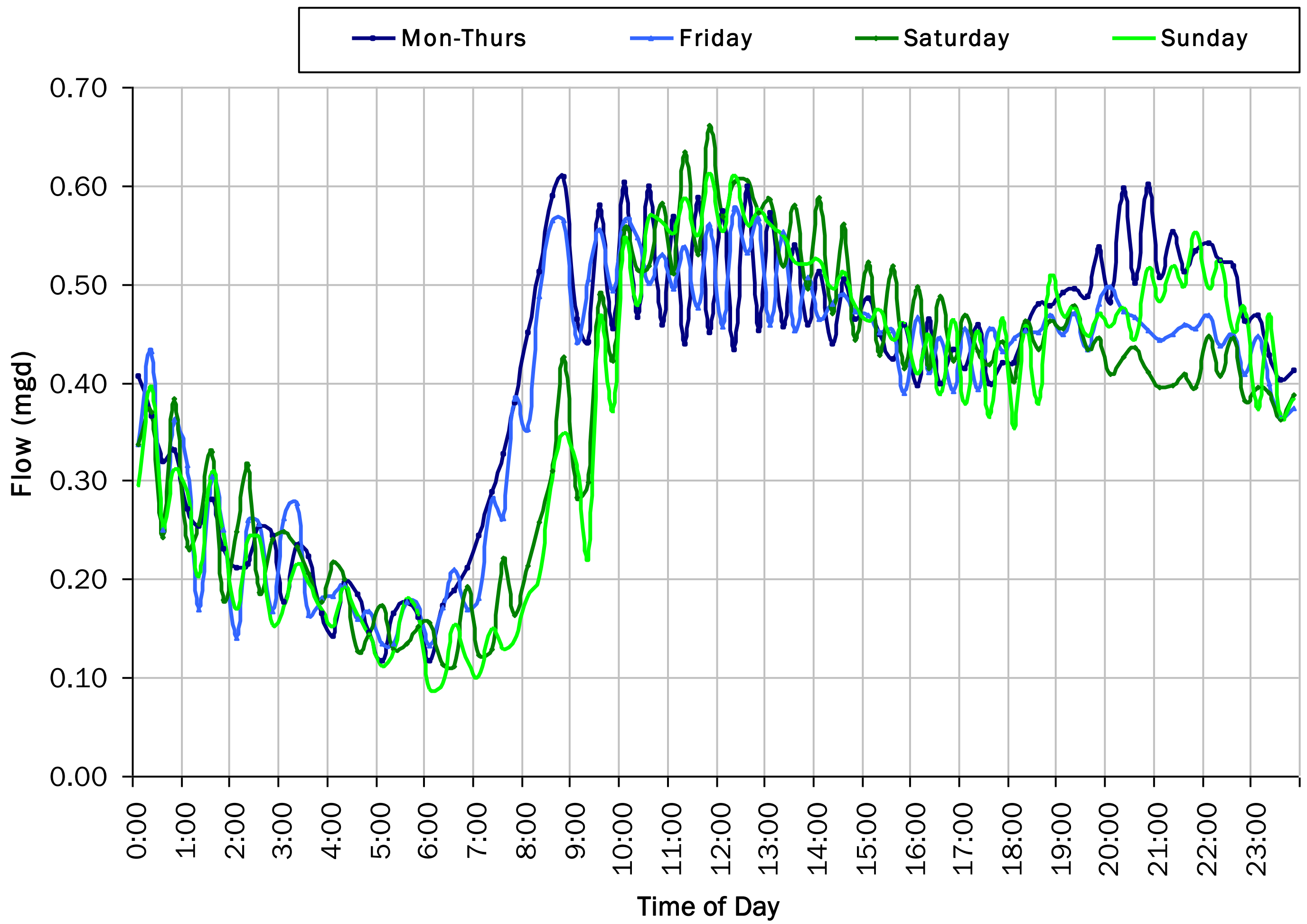
Flow Summary: 2/3/2023 to 2/22/2023

Period Rainfall: 0.89 inches   Period Avg Flow: 0.410 mgd   Period Peak Flow: 0.987 mgd   Period Min Flow: 0.076 mgd



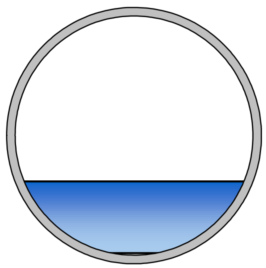


SITE 7
Average Dry Weather Flow Hydrographs



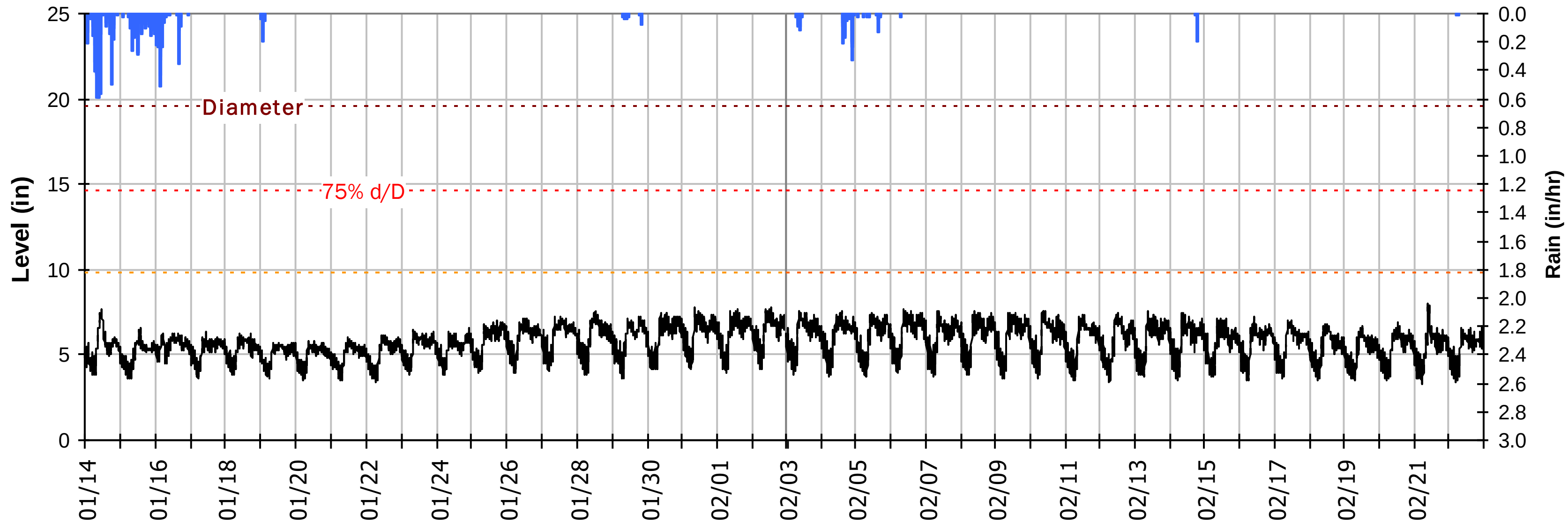
ADWF:

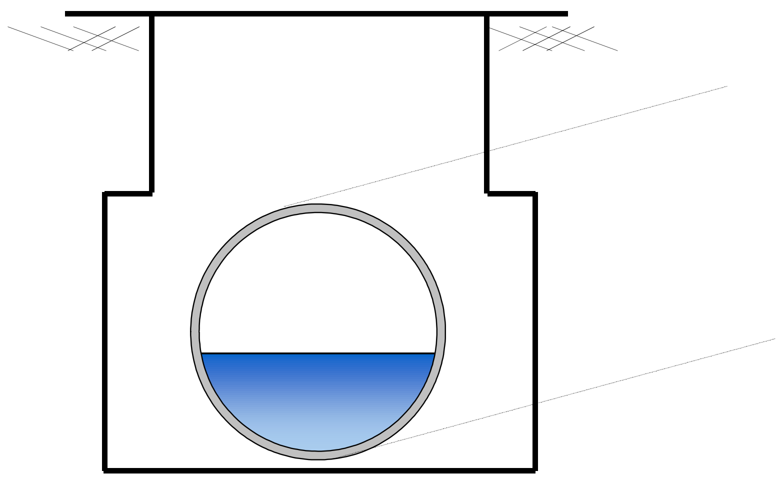
0.394 *mgd*



SITE 7
Site Capacity and Surcharge Summary

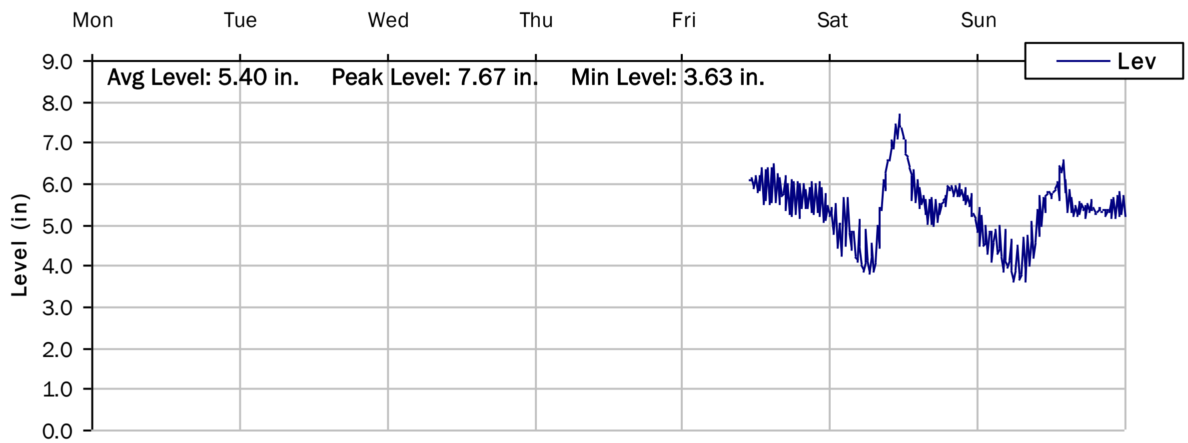
Realtime Flow Levels with Rainfall Data over Peak Level Period

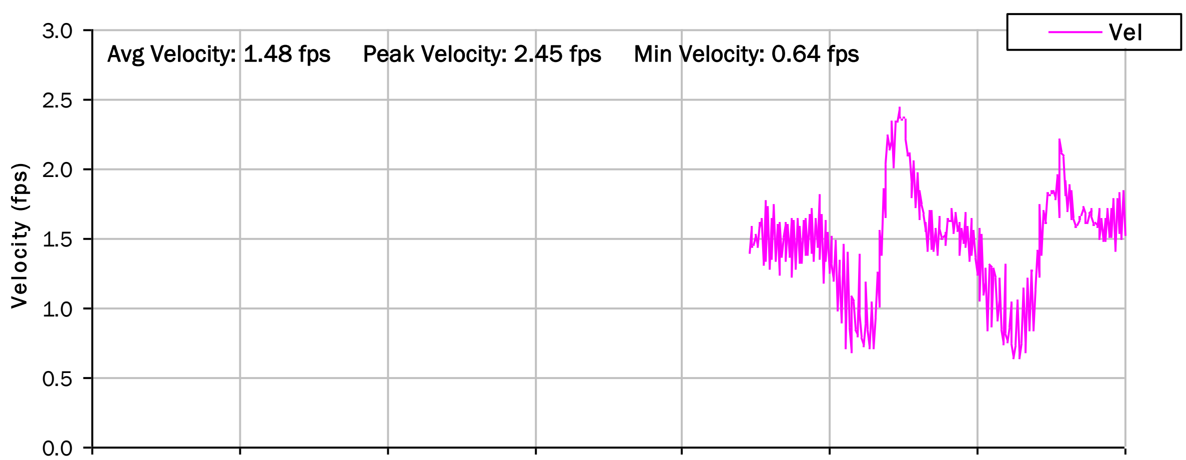


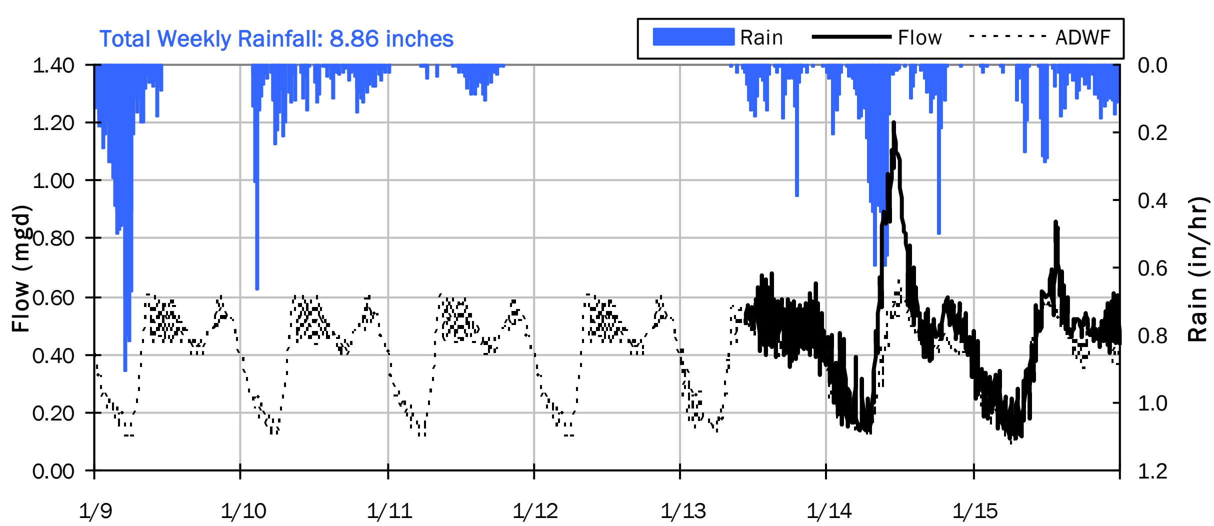


Pipe Diameter:	19.5	*inches*
Peak Measured Level:	8.02	*inches*
Peak d/D Ratio:	0.41	

SITE 7
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023

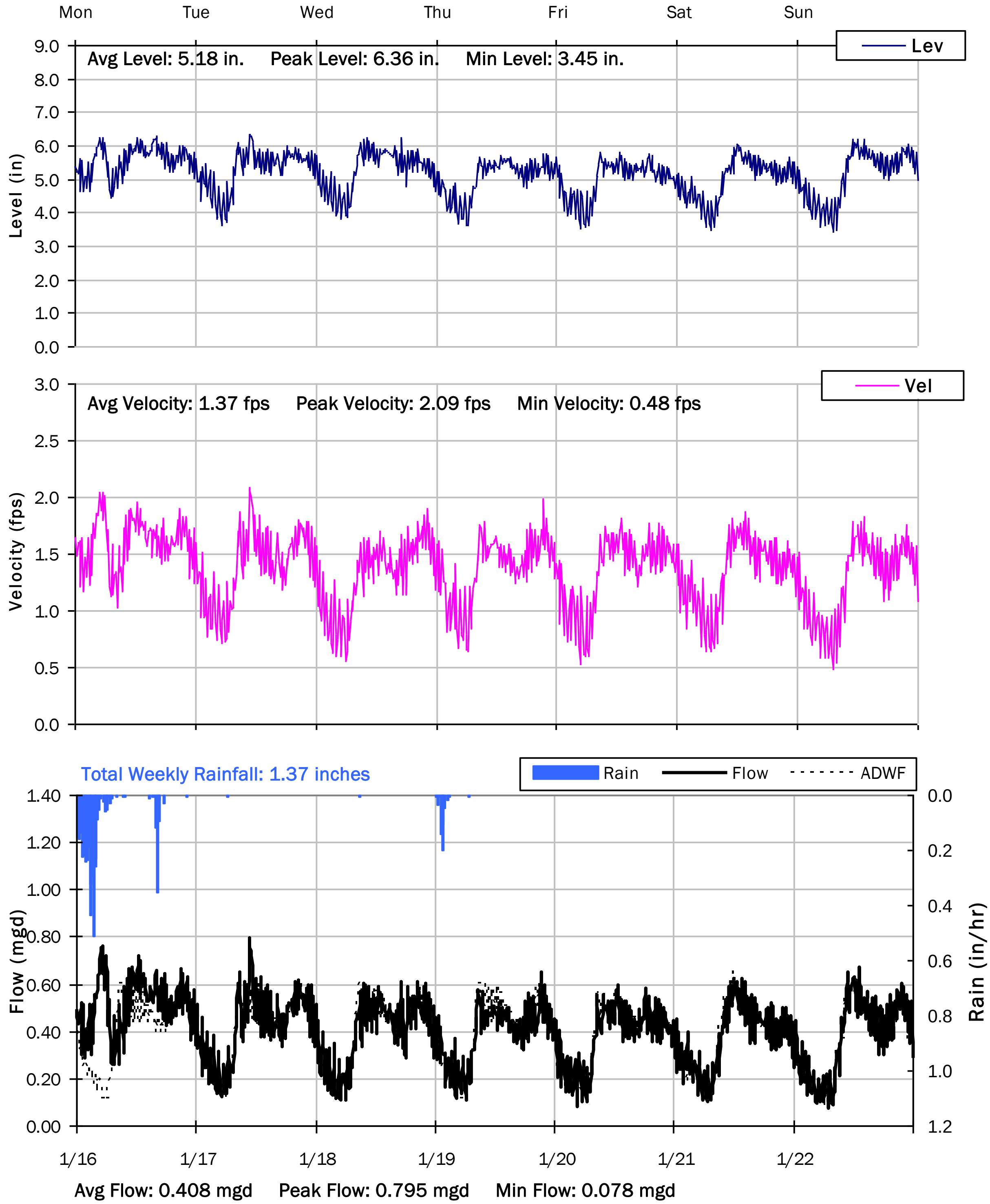






Avg Flow: 0.470 mgd
Peak Flow: 1.199 mgd
Min Flow: 0.111 mgd

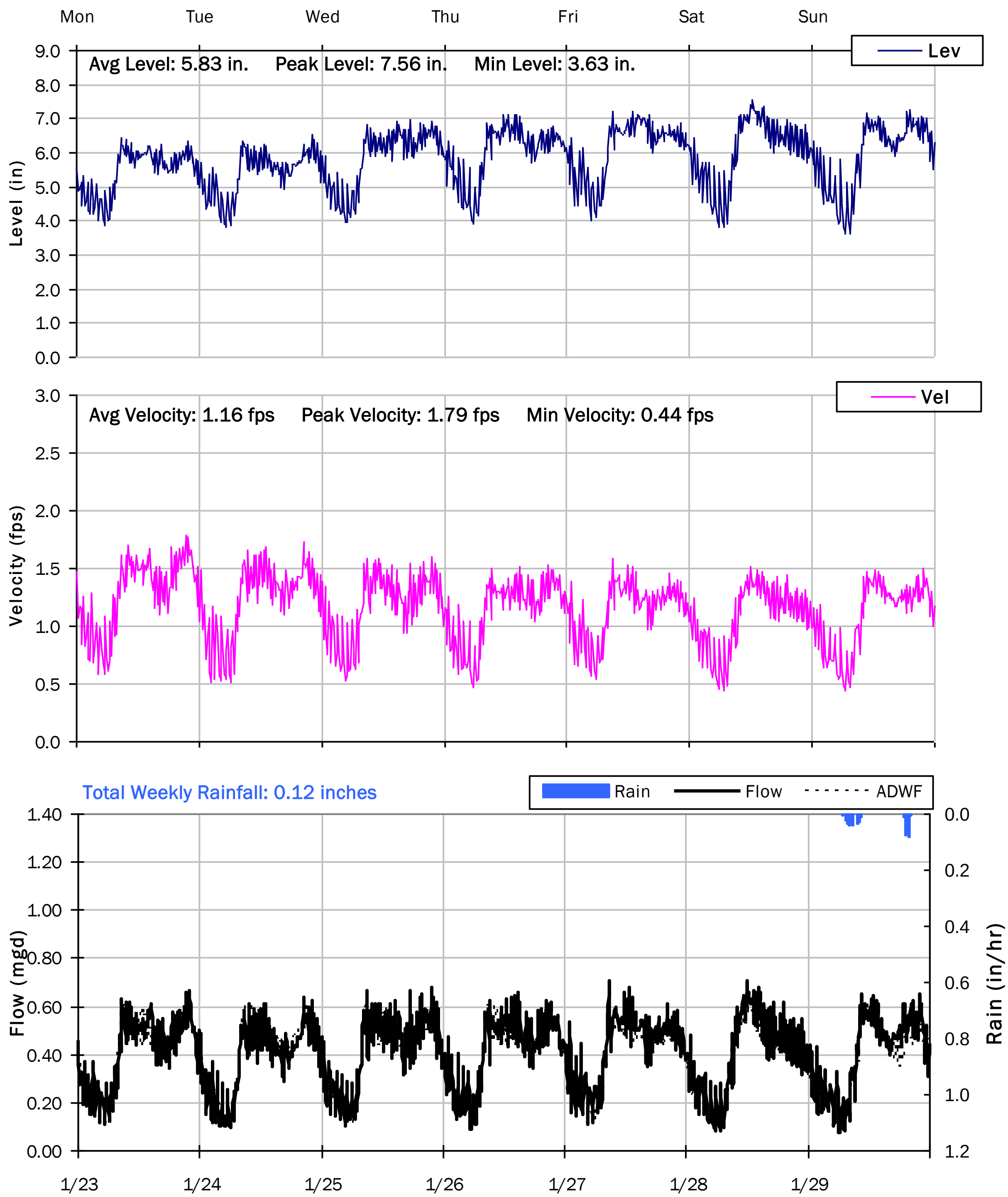
SITE 7
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023



SITE 7

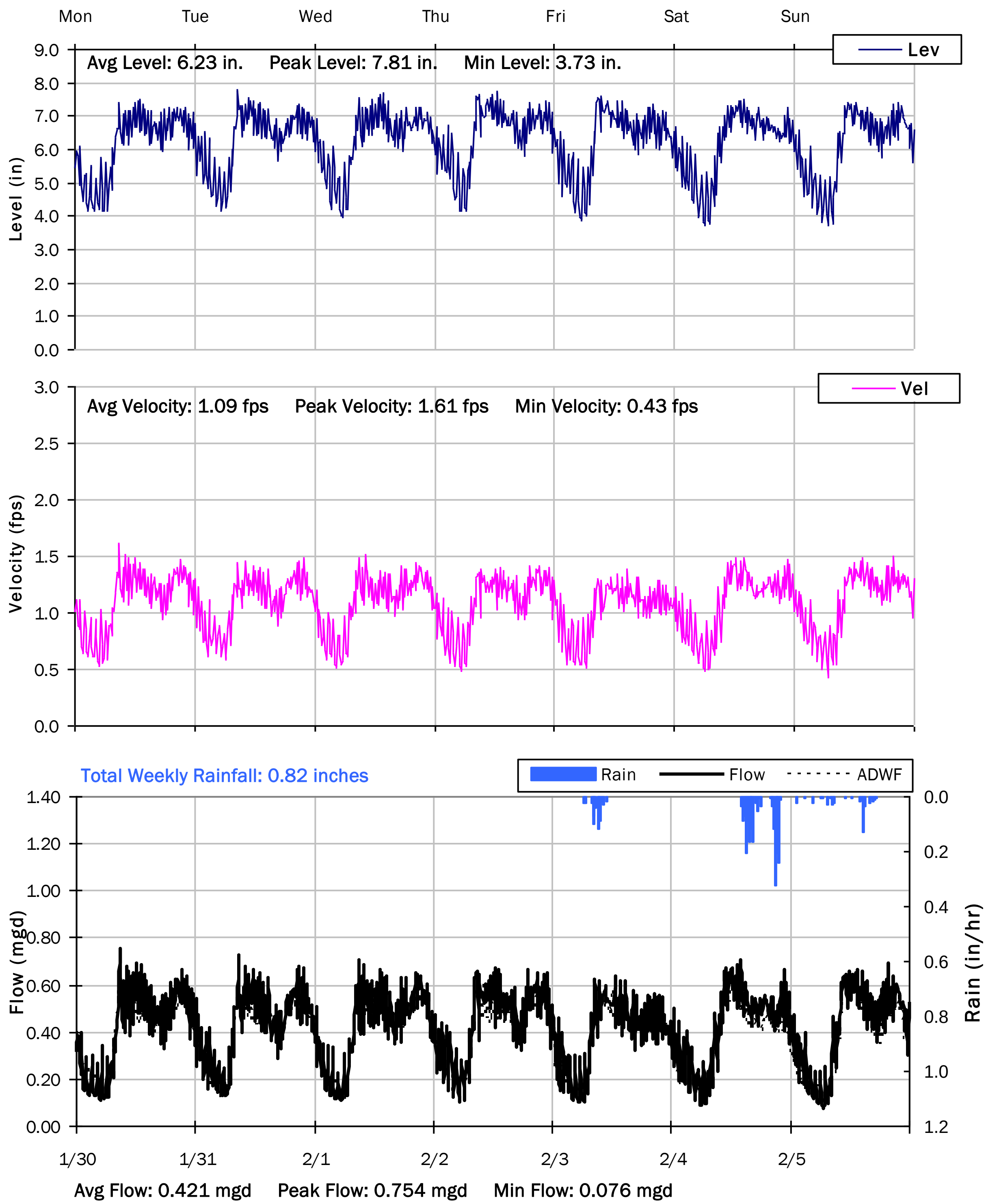
Weekly Level, Velocity and Flow Hydrographs

1/23/2023 to 1/30/2023

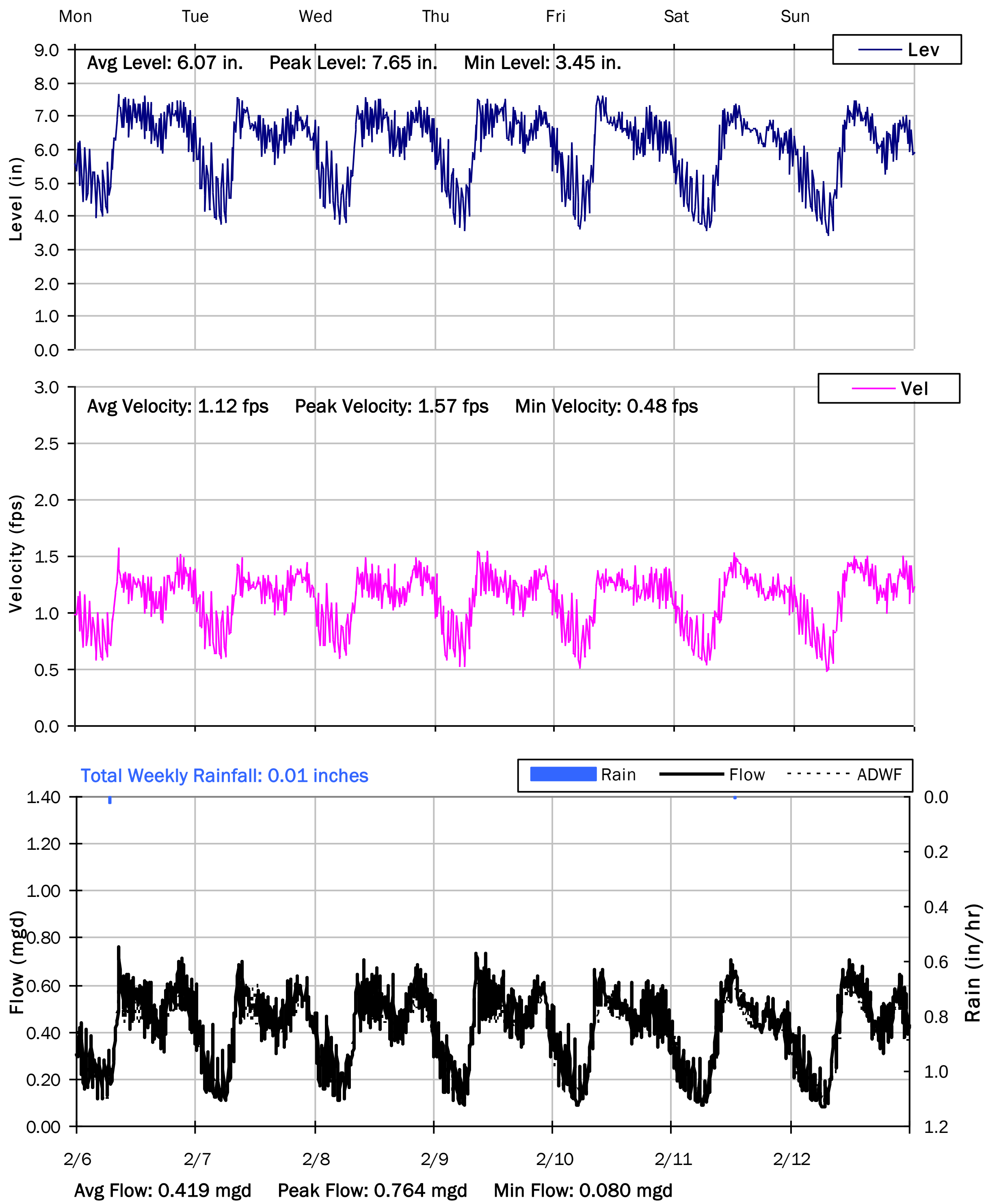


Avg Flow: 0.409 mgd   Peak Flow: 0.709 mgd   Min Flow: 0.077 mgd

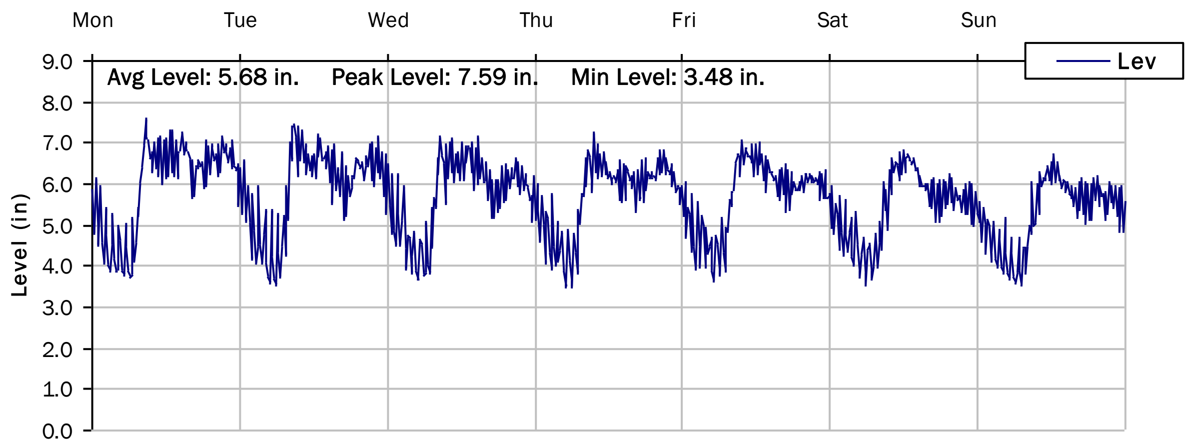
SITE 7
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023

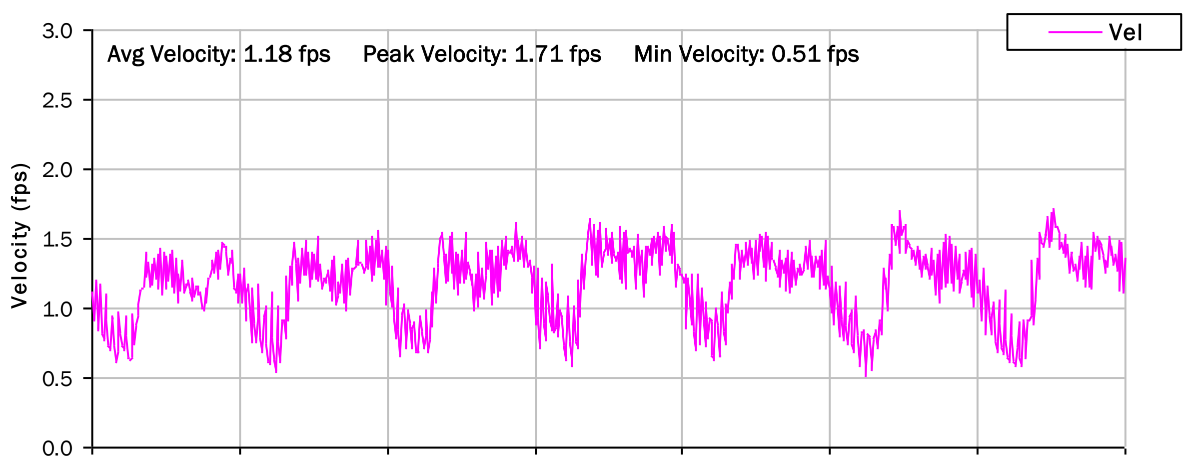


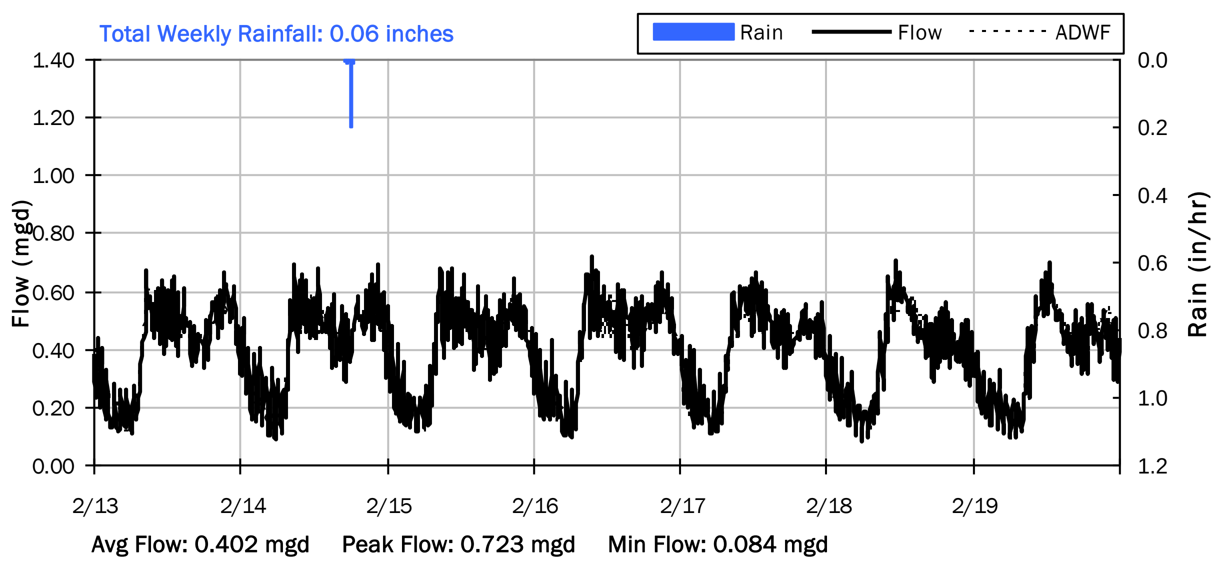
SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023



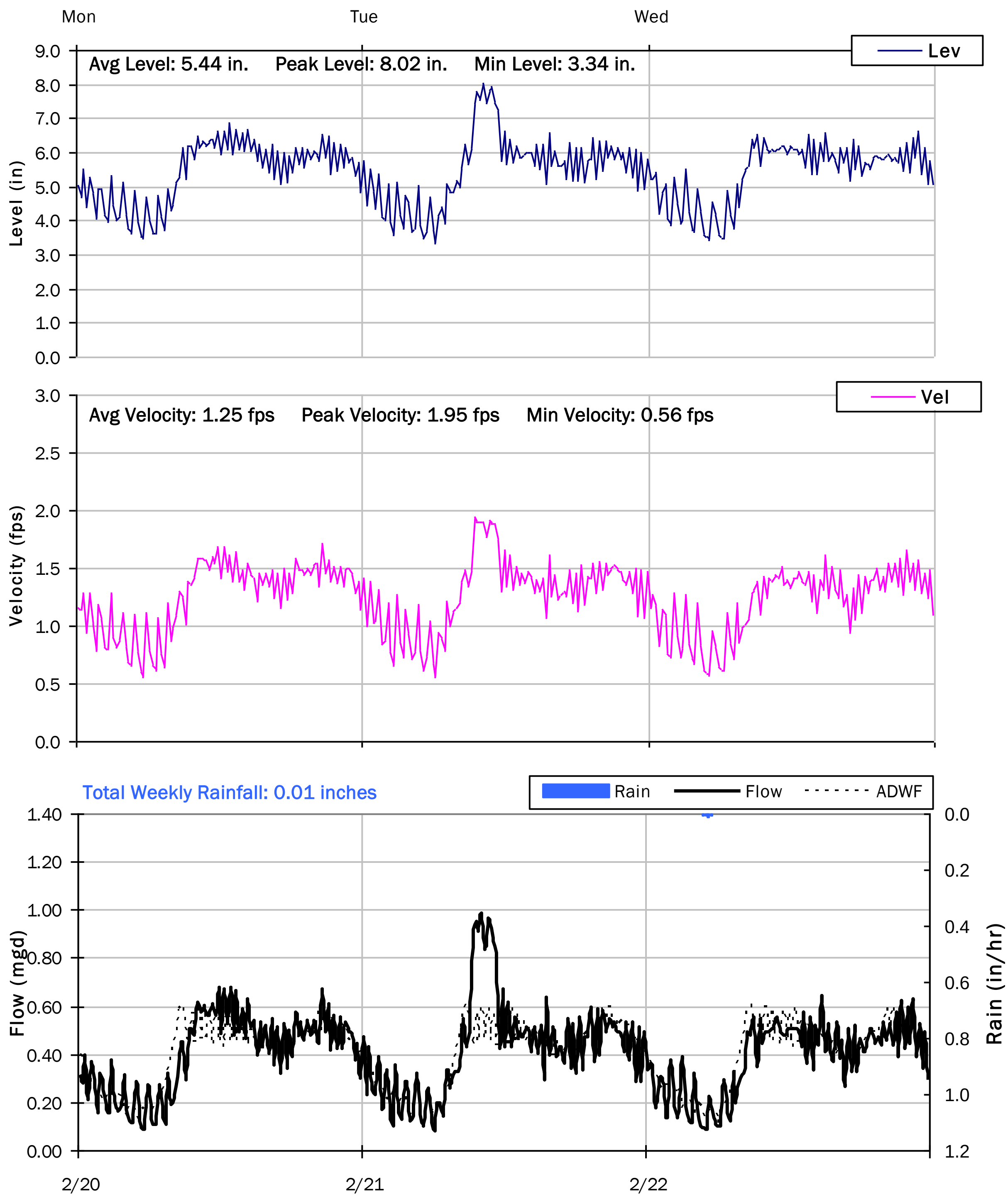
SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023







SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023



Avg Flow: 0.406 mgd   Peak Flow: 0.987 mgd   Min Flow: 0.085 mgd

Monitoring Site: Site 8

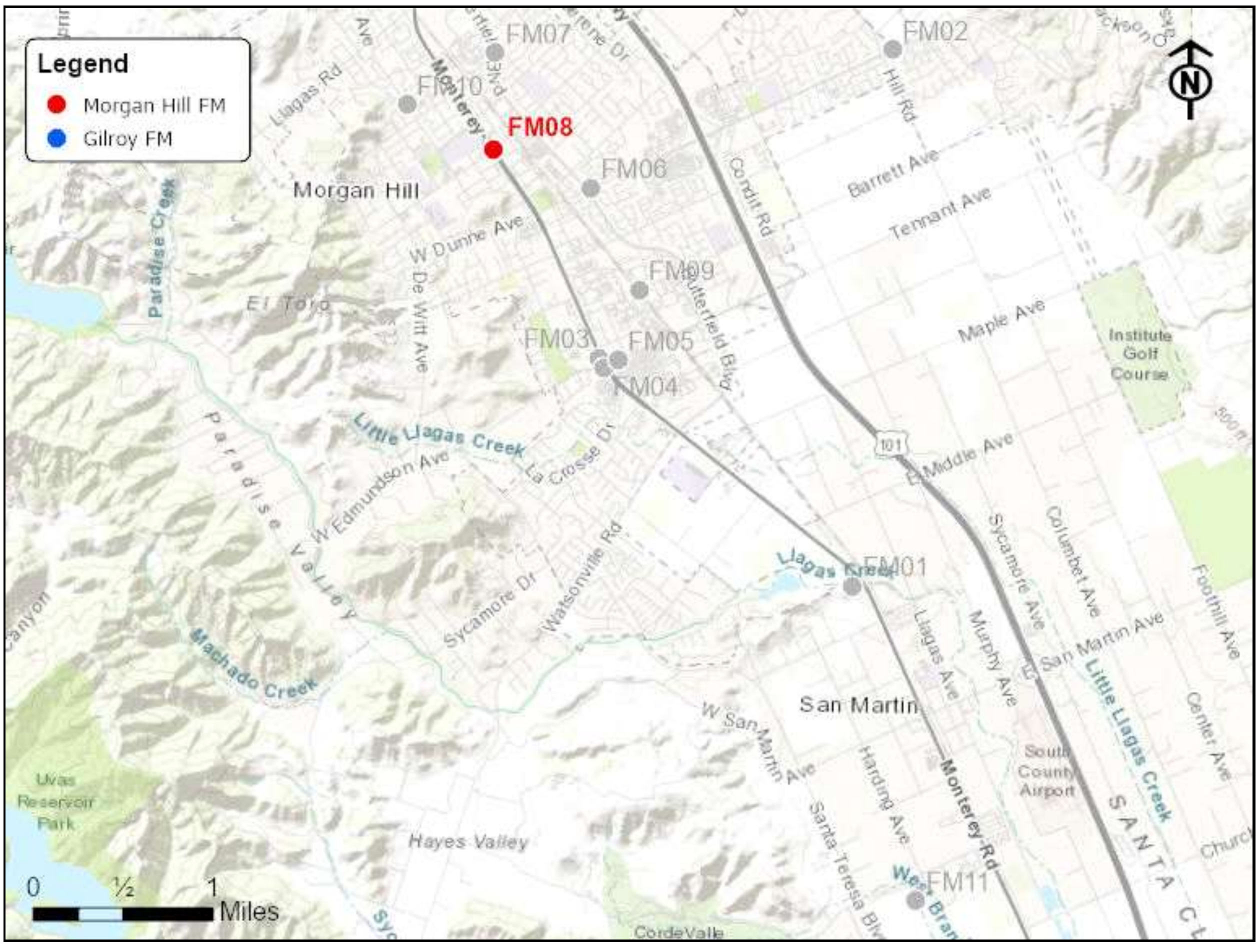
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: Monterey Hwy and W Main Ave

Data Summary Report



Vicinity Map: Site 8

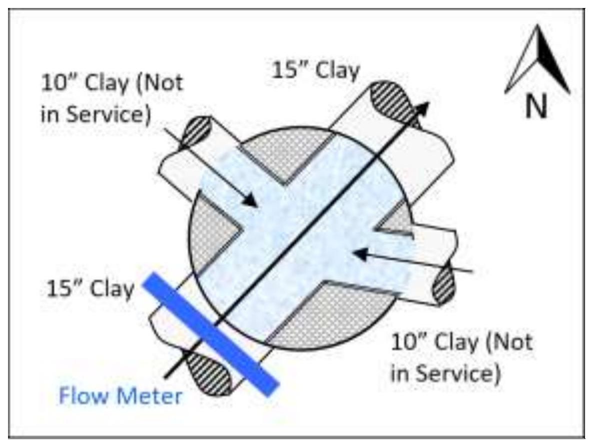
SITE 8

Site Information MH ID: G4-D.MH.040

Location:	Monterey Hwy and W Main Ave
Coordinates:	121.3916° W, 37.7496° N
Rim Elevation:	350 feet
Expected Pipe Diameter:	15 inches
Measured Pipe Diameter:	15 inches
ADWF:	0.429 mgd
Peak Measured Flow:	2.21 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch

Street View

Plan View

SITE 8

Additional Site Photos

Northeast Effluent Pipe

East Influent Pipe

SITE 8

Additional Site Photos

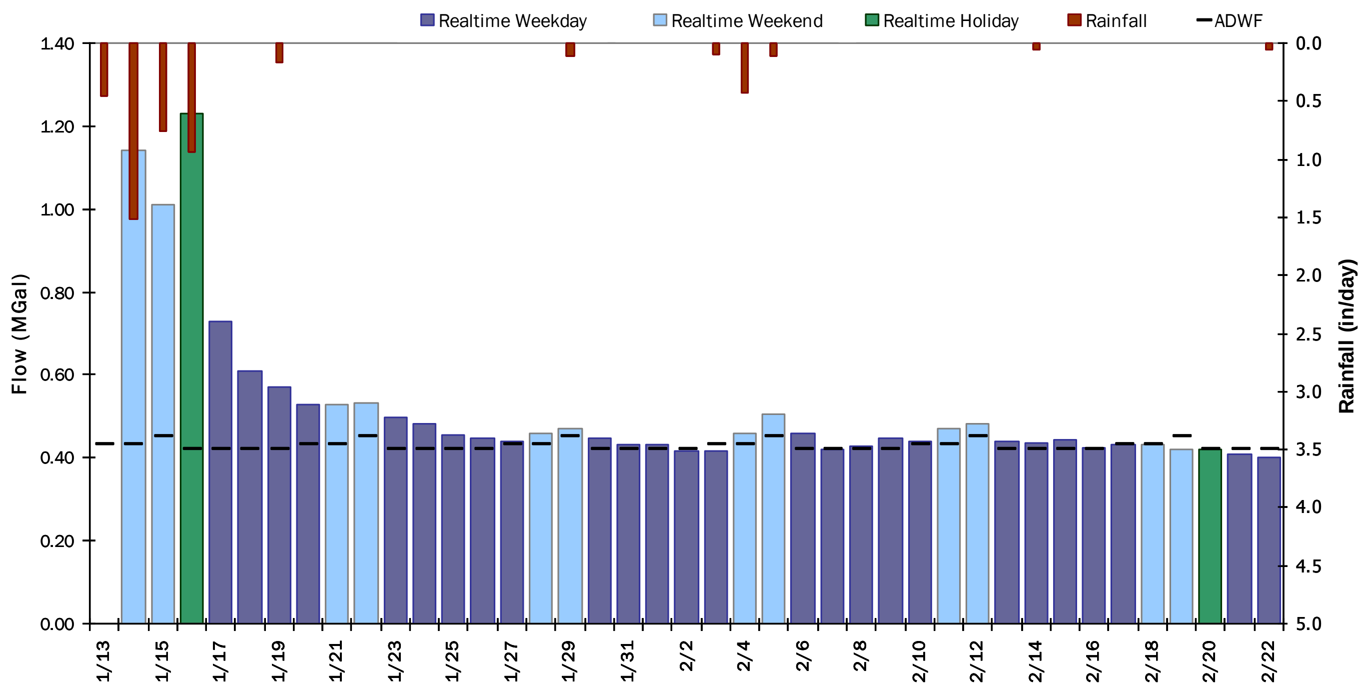
Monitored Southwest Influent Pipe

Northwest Influent Pipe

SITE 8
Period Flow Summary: Daily Flow Totals

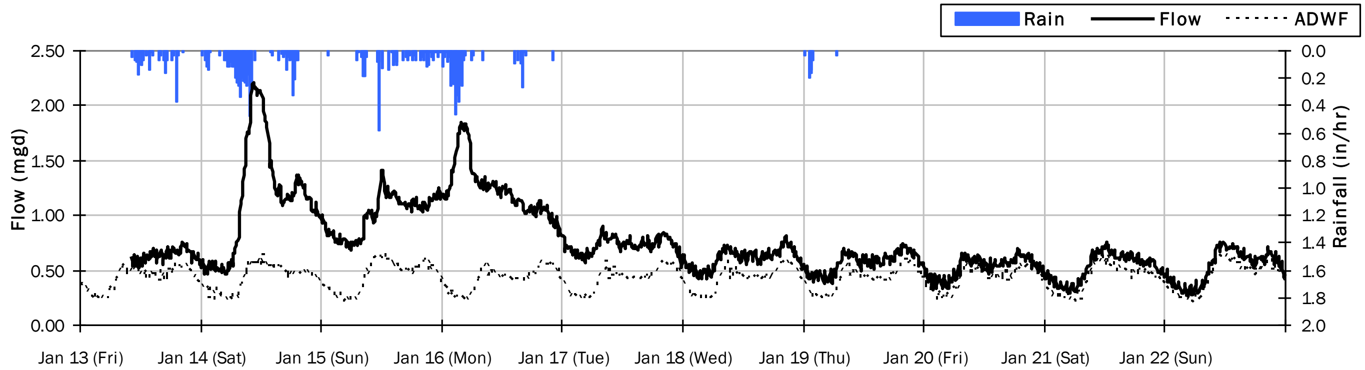
Avg Daily Flow: 0.519 MGal   Peak Daily Flow: 1.230 MGal   Min Daily Flow: 0.401 MGal

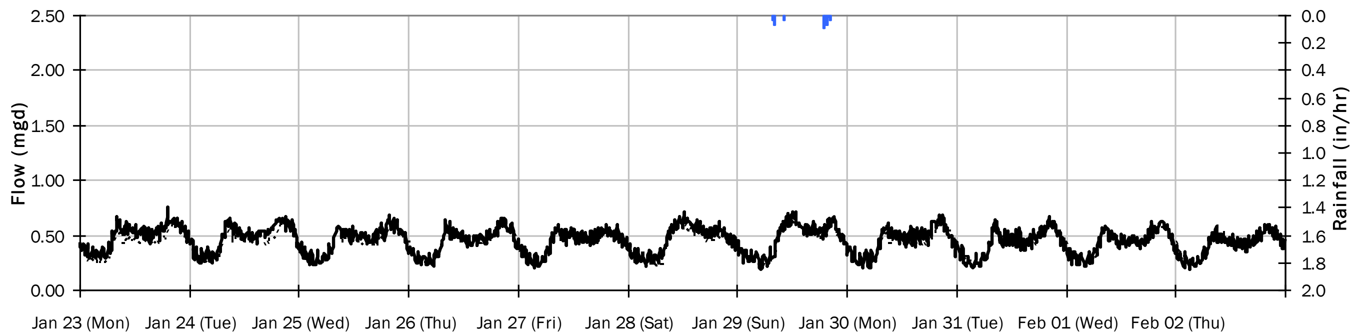
Total Rainfall: 4.67 inches



SITE 8
Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 3.94 inches   Period Avg Flow: 0.594 mgd   Period Peak Flow: 2.205 mgd   Period Min Flow: 0.190 mgd

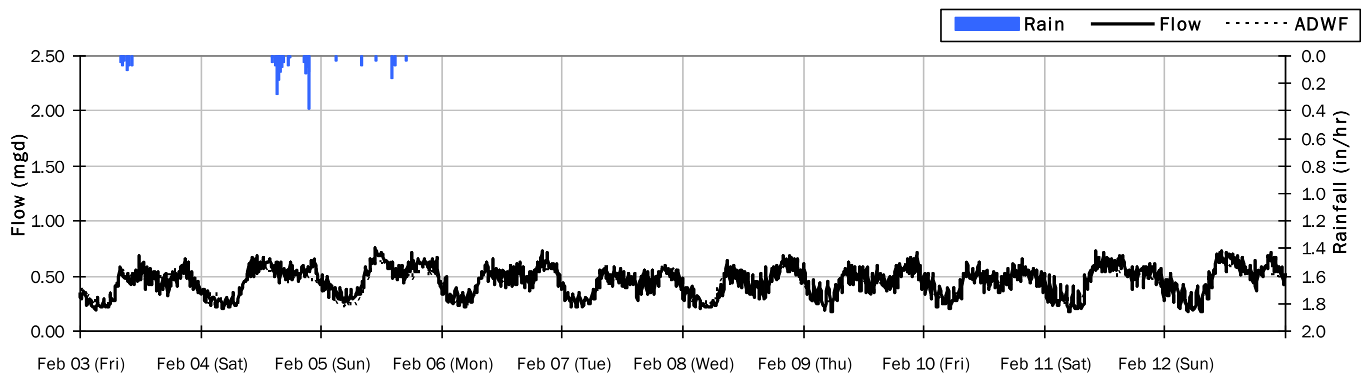


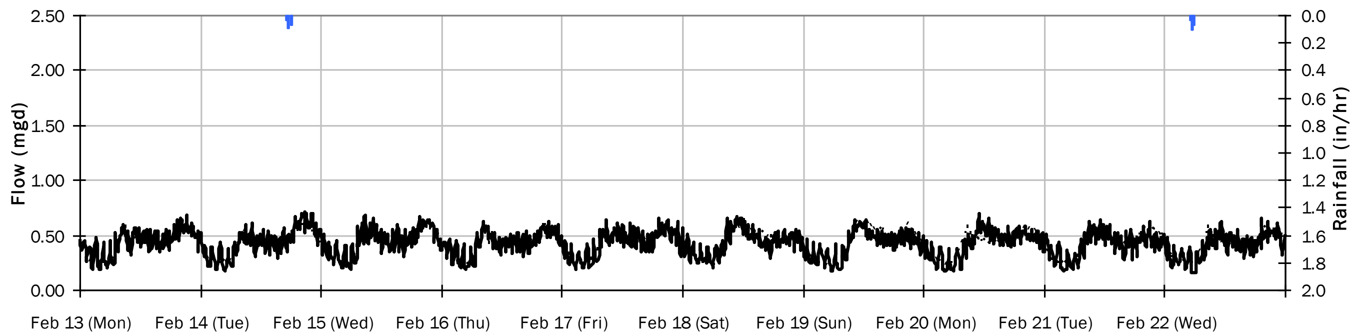


SITE 8

Flow Summary: 2/3/2023 to 2/22/2023

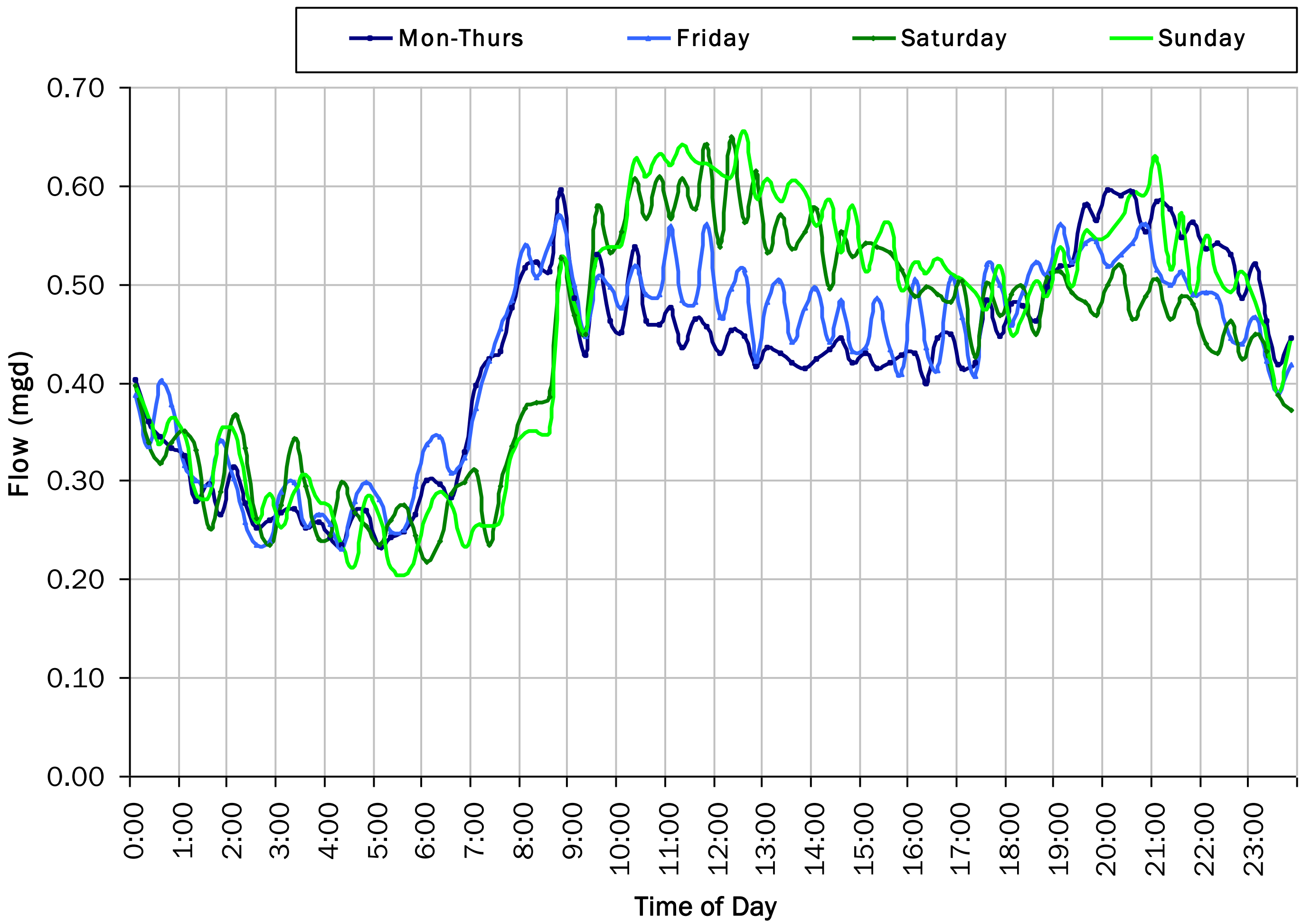
Period Rainfall: 0.73 inches | Period Avg Flow: 0.439 mgd | Period Peak Flow: 0.761 mgd | Period Min Flow: 0.153 mgd





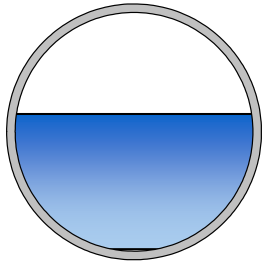
SITE 8

Average Dry Weather Flow Hydrographs



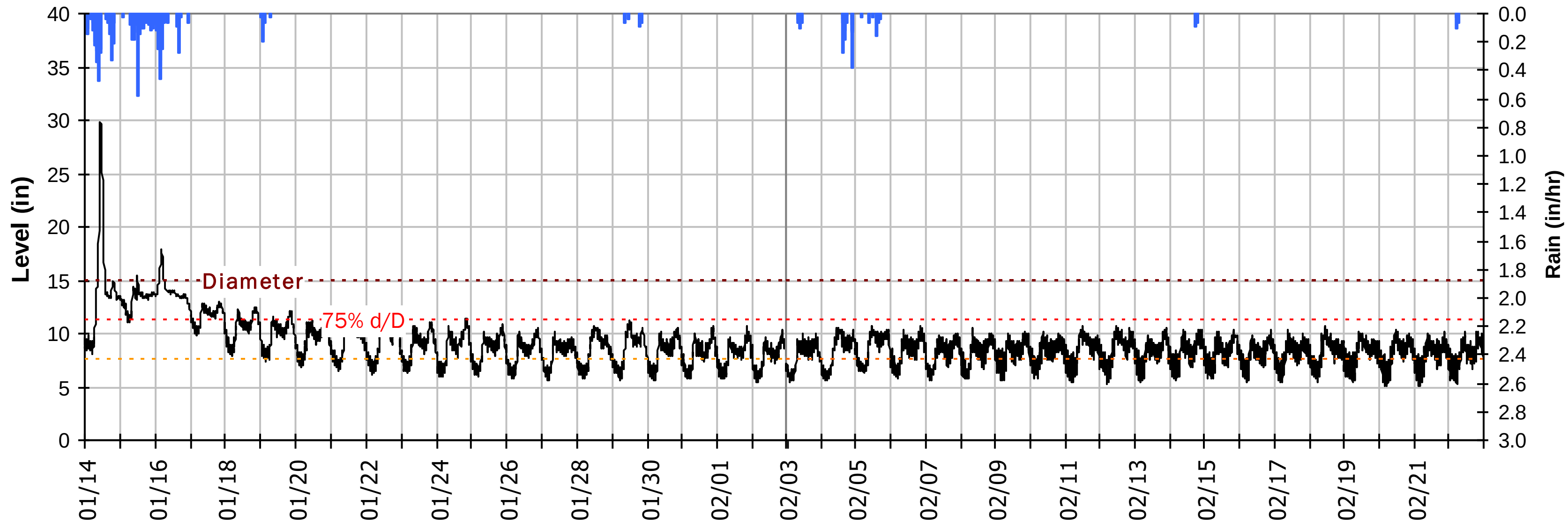
ADWF:

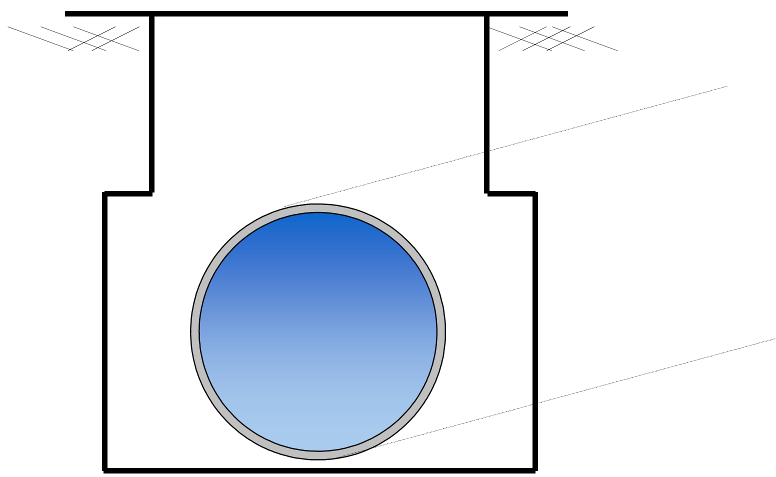
0.429 *mgd*



SITE 8
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Peak Level Period

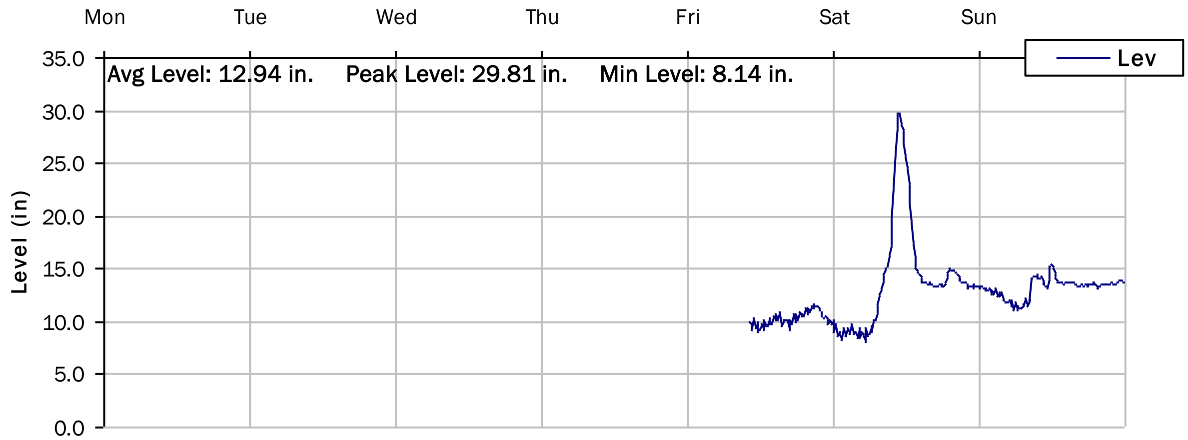


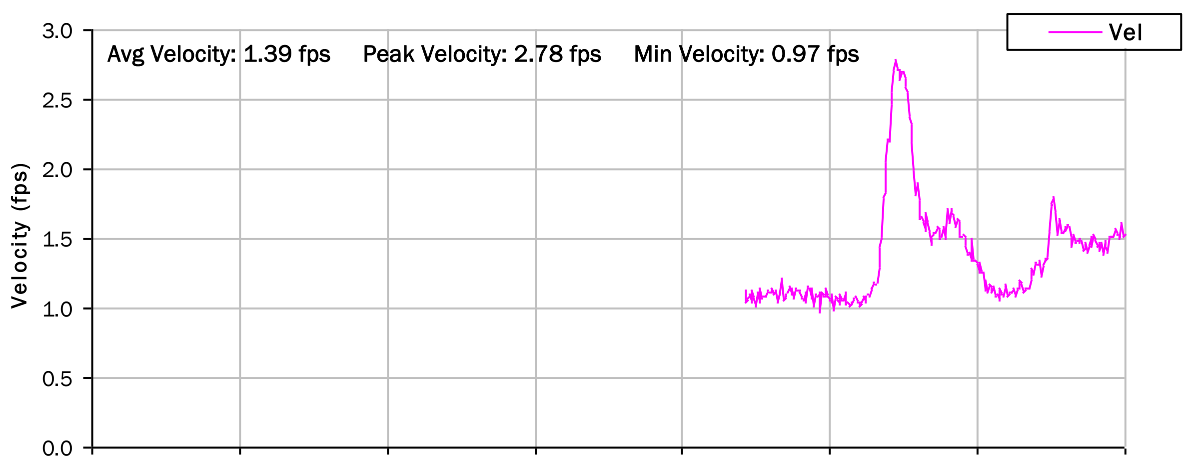


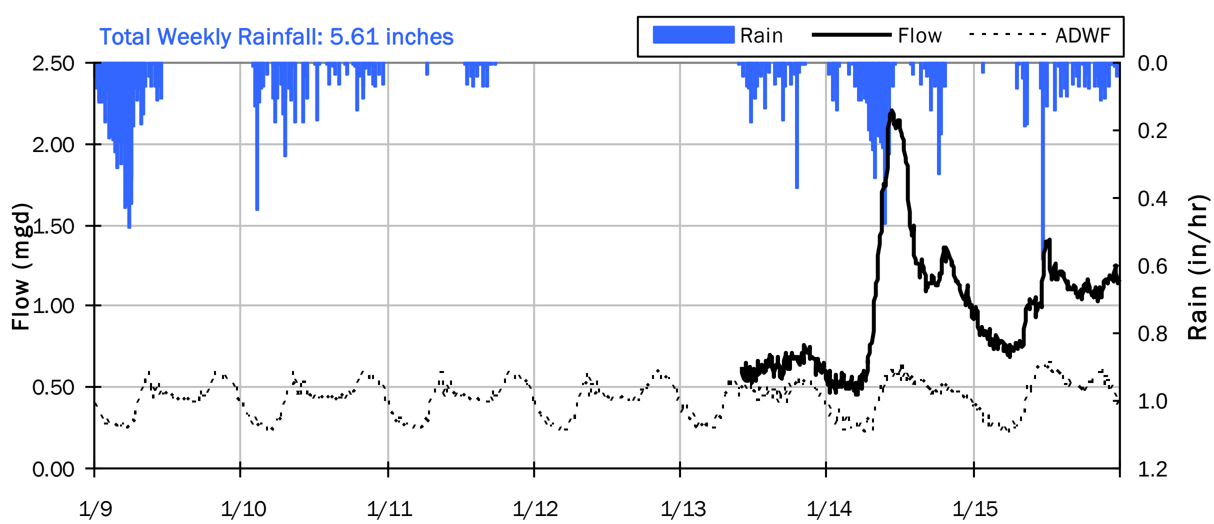
Pipe Diameter:	15	*inches*
Peak Measured Level:	29.8	*inches*
Peak d/D Ratio:	1.99	

Surcharged 14.8 inches over crown

SITE 8
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023

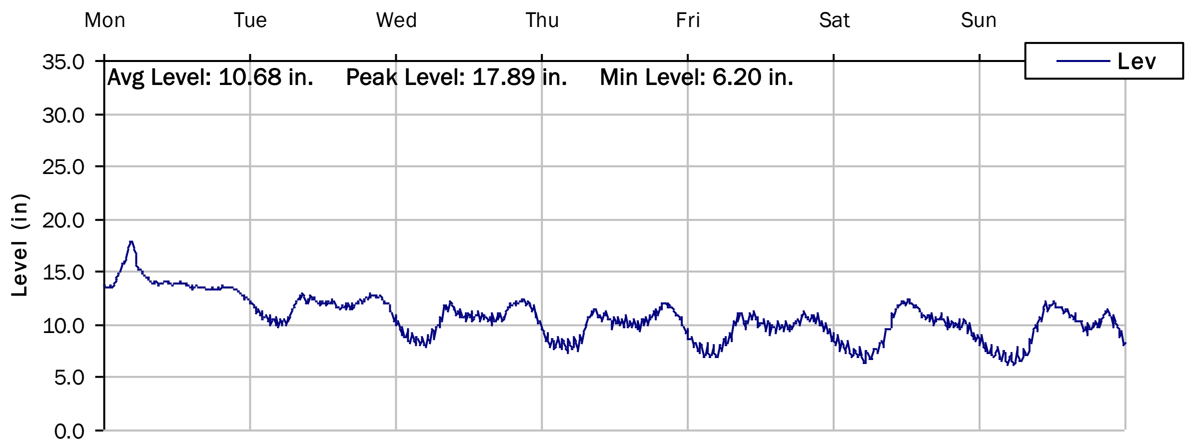


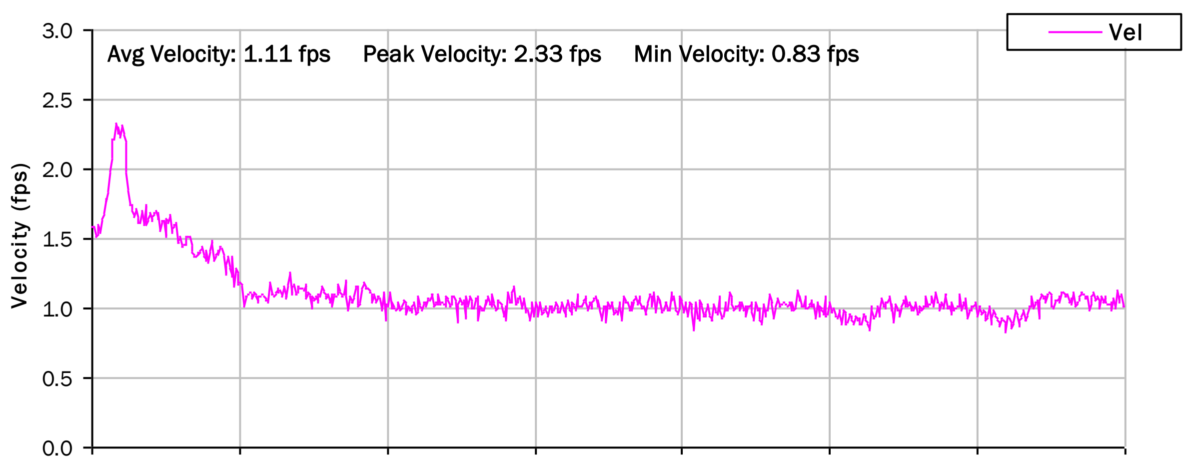


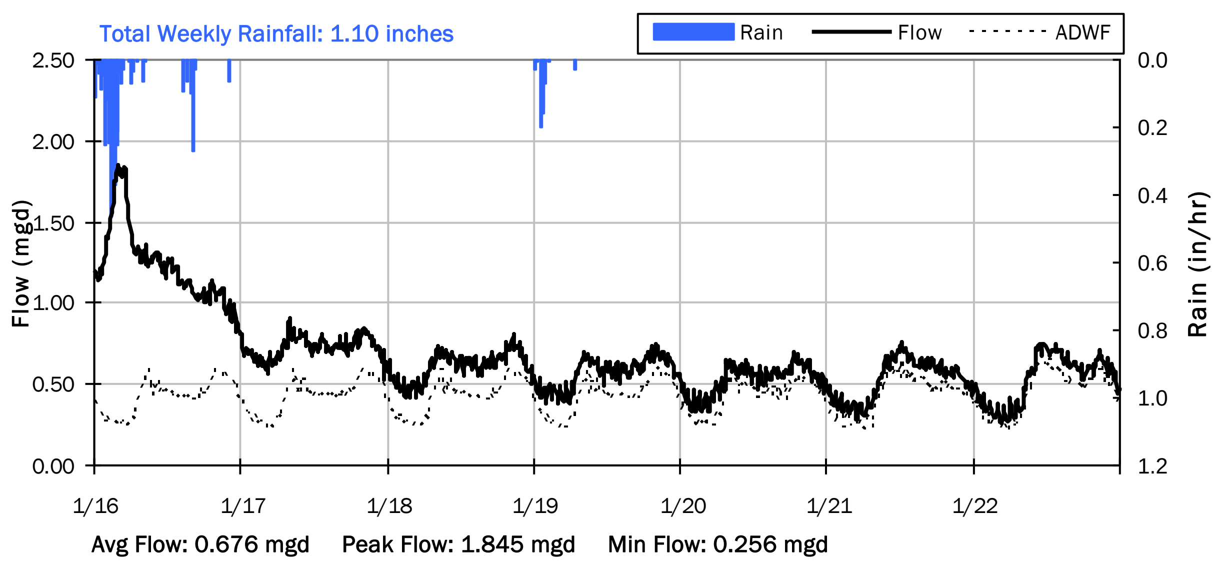


Avg Flow: 0.976 mgd Peak Flow: 2.205 mgd Min Flow: 0.458 mgd

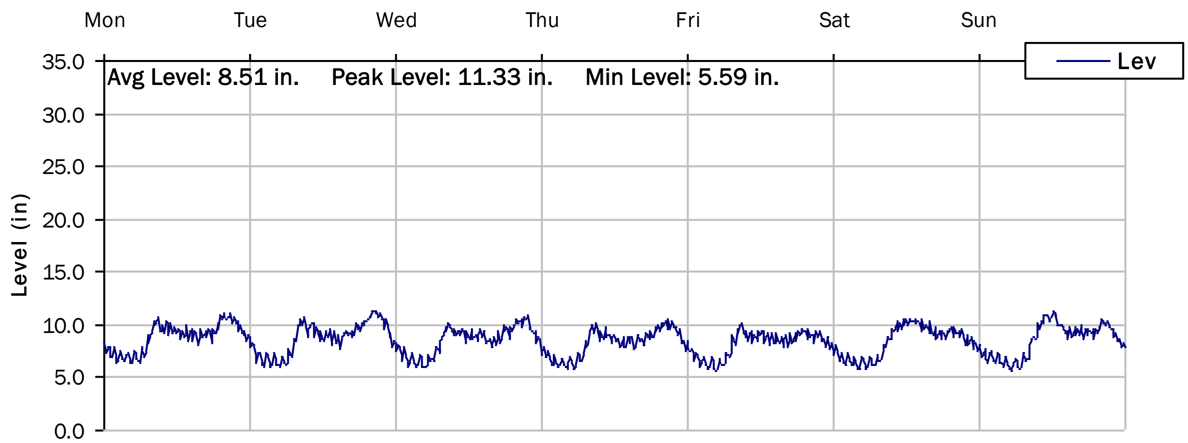
SITE 8
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023

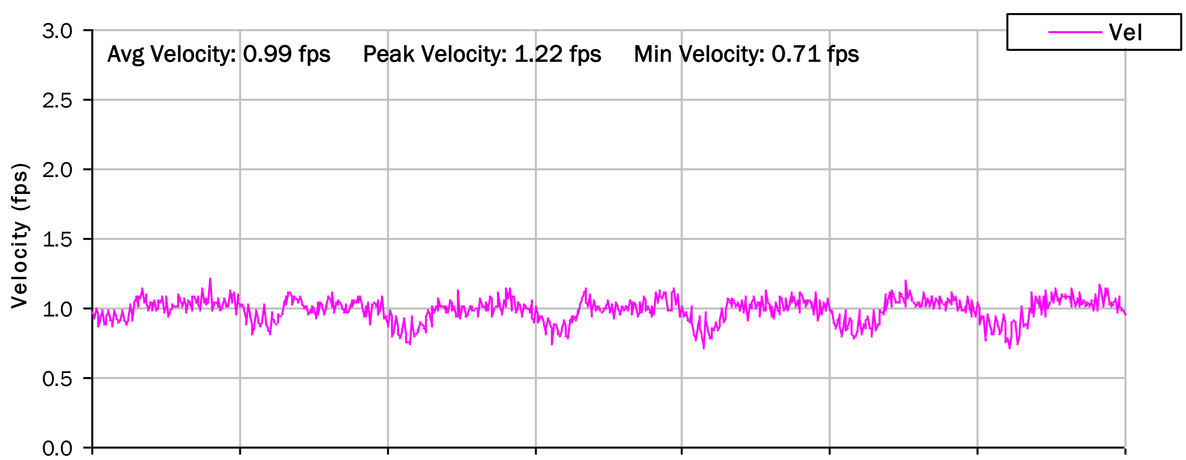


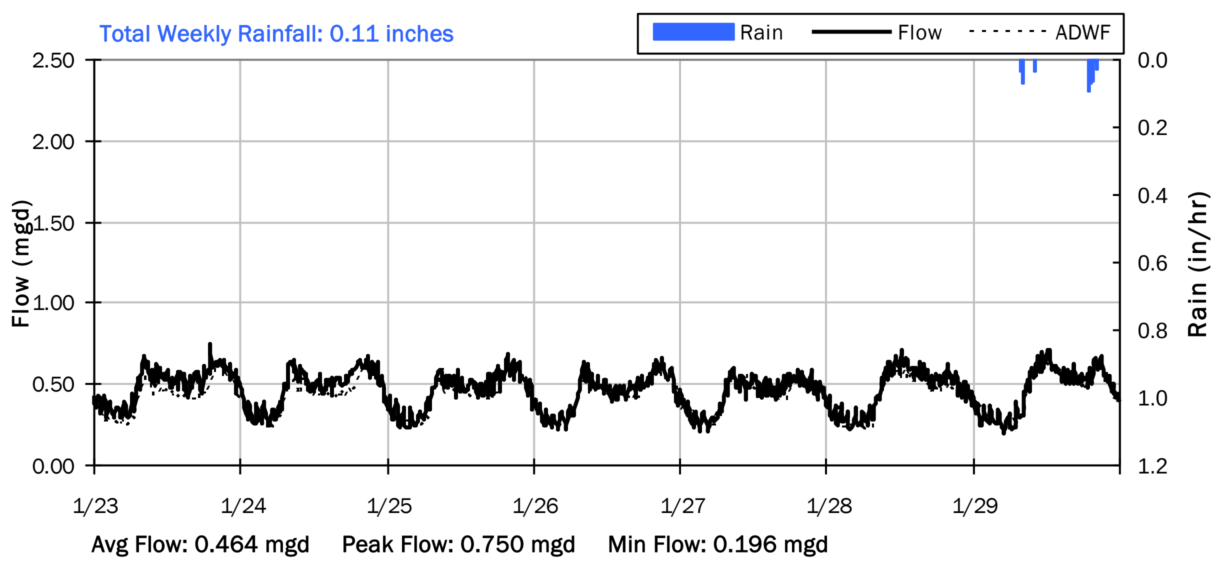




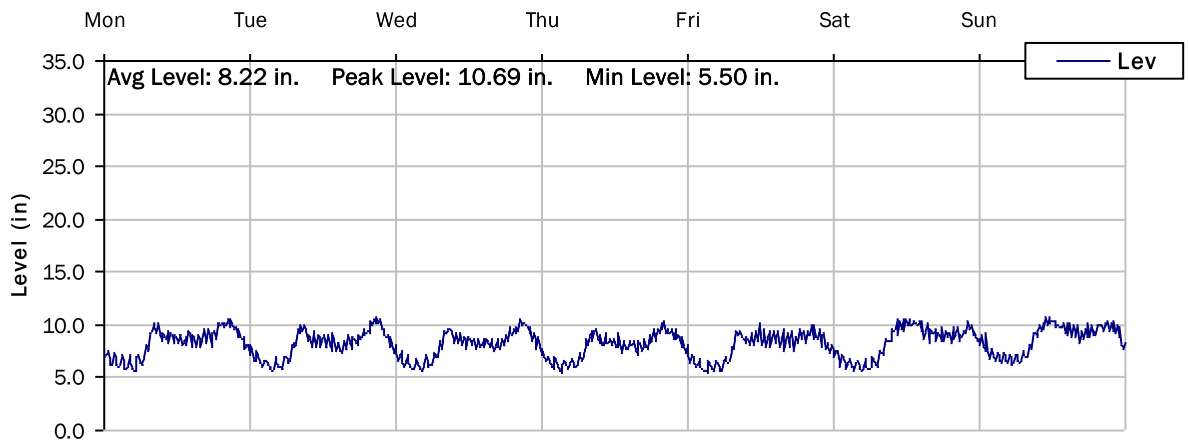
SITE 8
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023

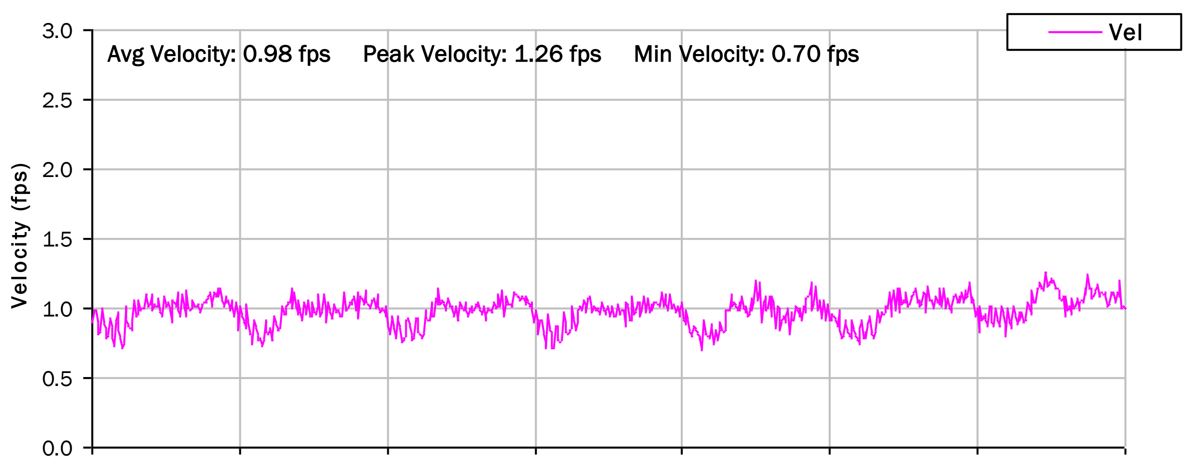


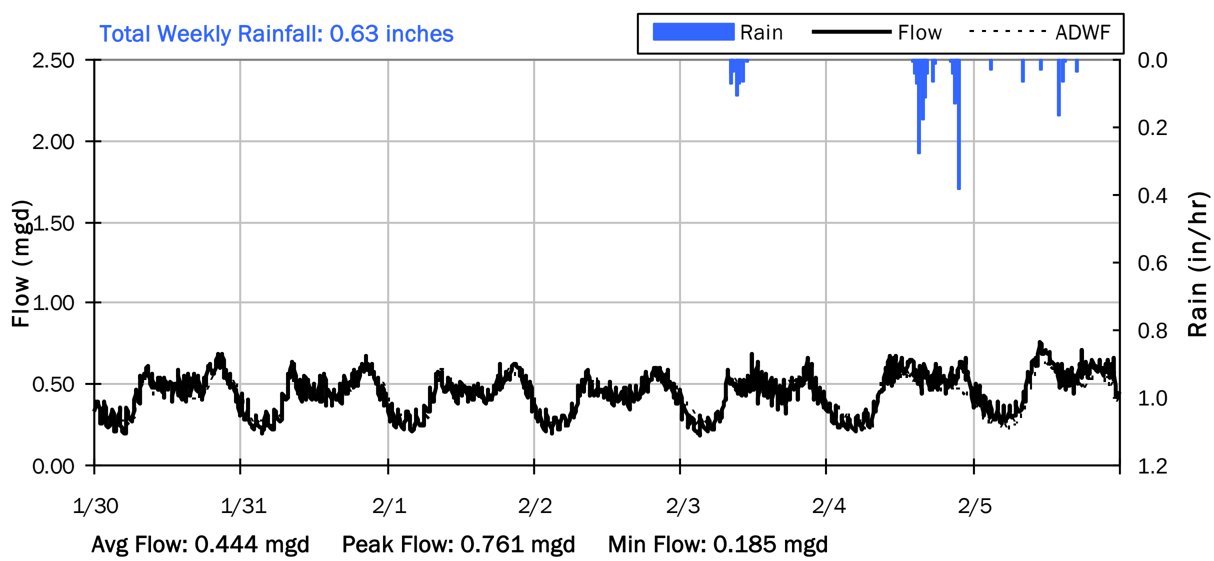




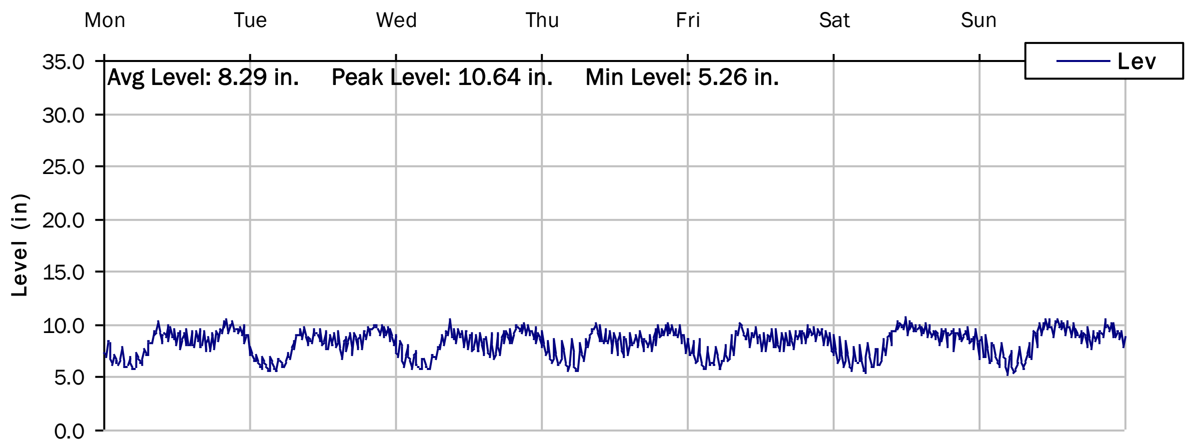
SITE 8
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023

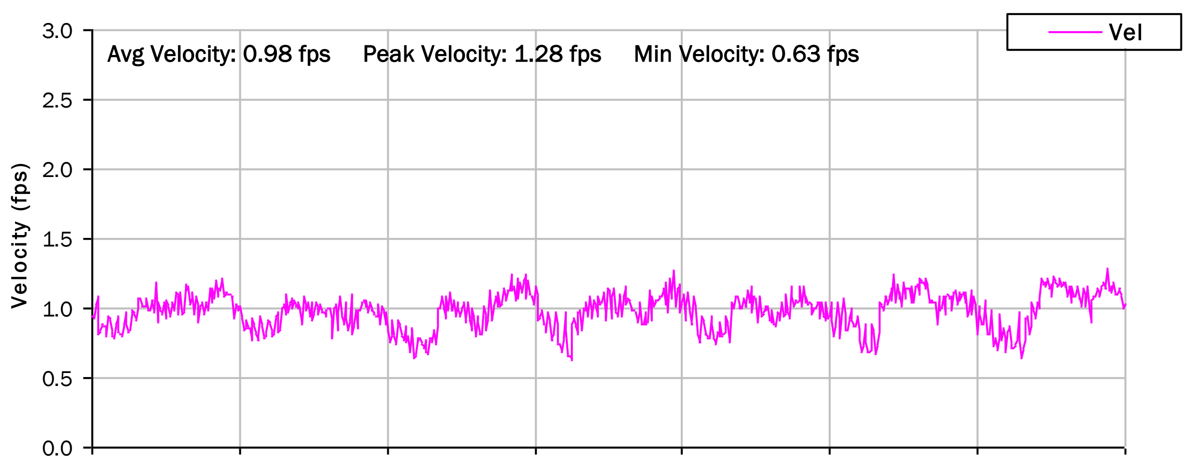


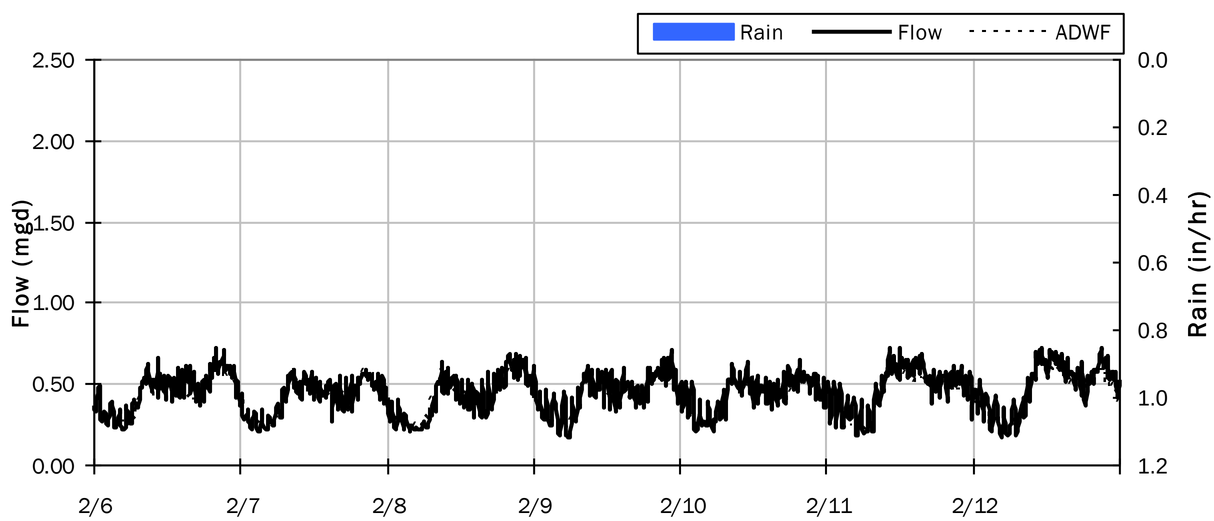




SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

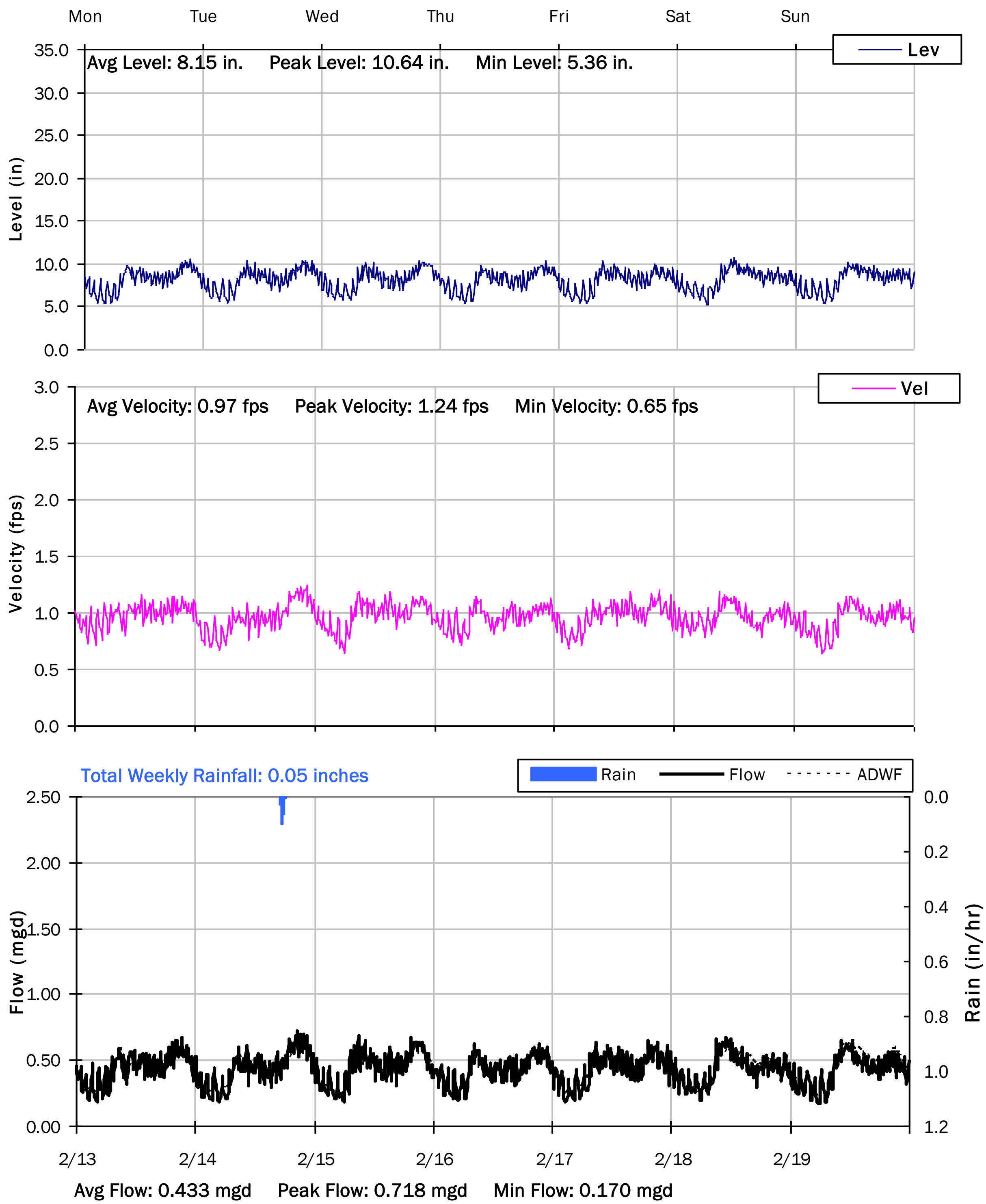




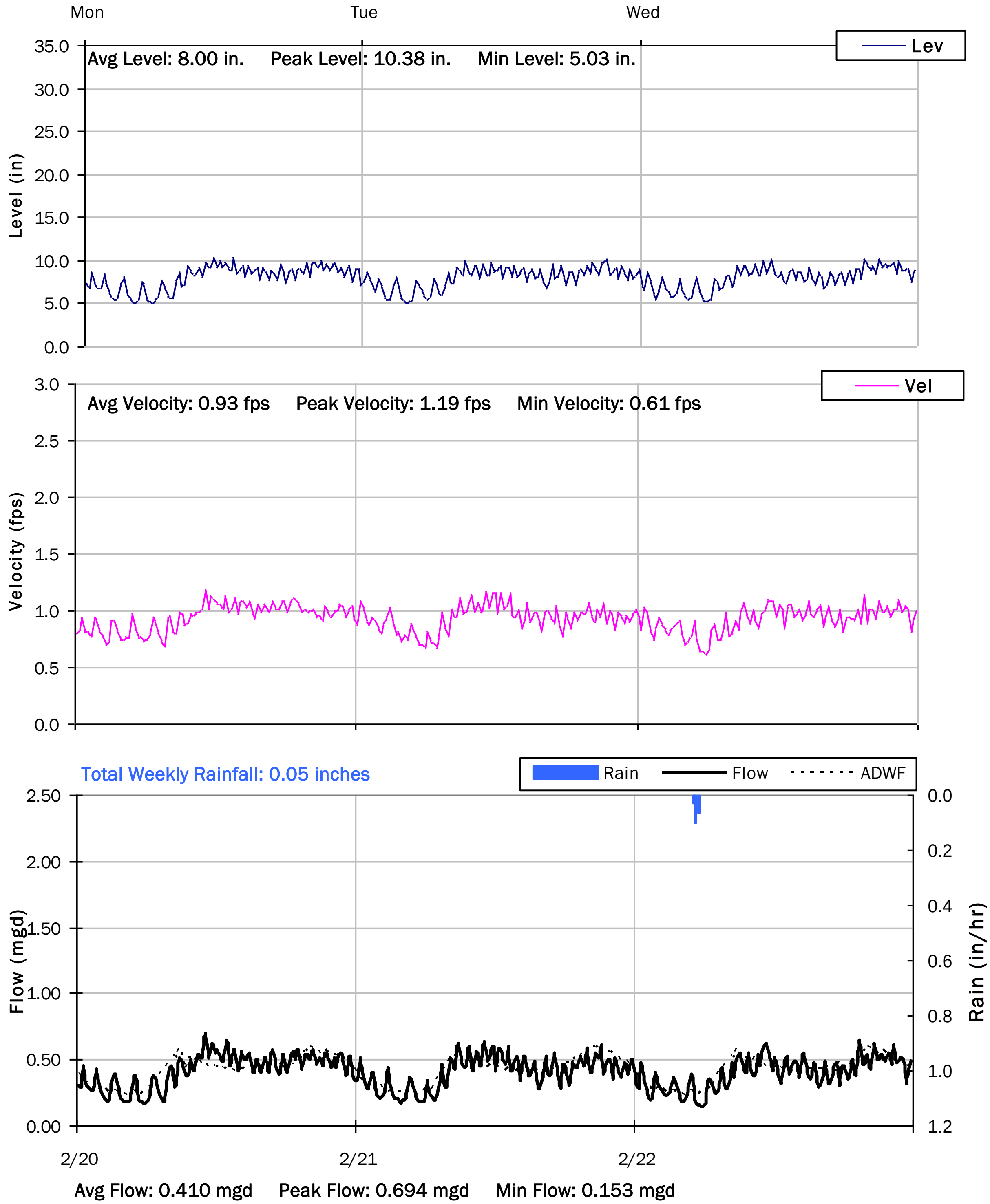


Avg Flow: 0.449 mgd   Peak Flow: 0.723 mgd   Min Flow: 0.170 mgd

SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023



Monitoring Site: Site 9

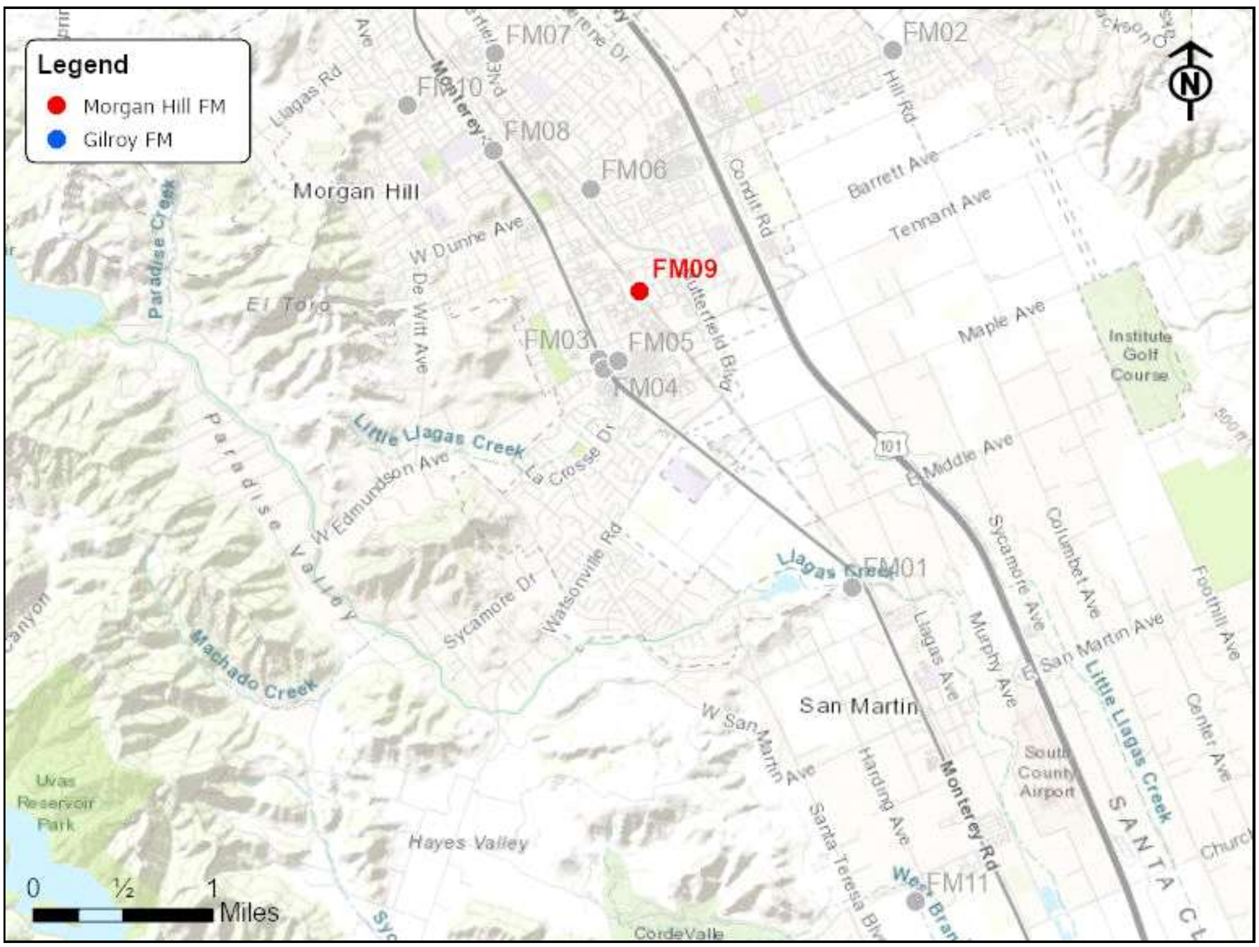
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: Railroad Ave and Barrett Ave

Data Summary Report



Vicinity Map: Site 9

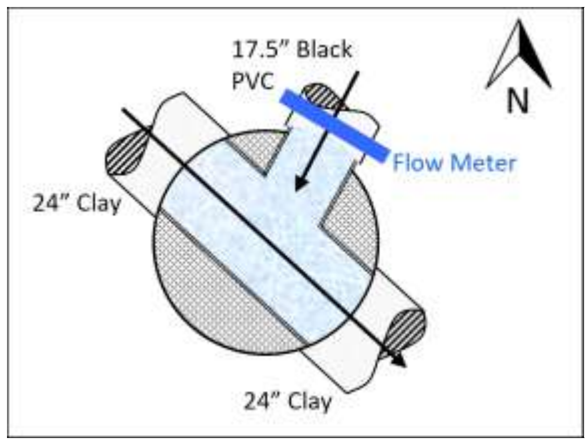
SITE 9

Site Information MH ID: H5-C.MH.004

Location:	Railroad Ave and Barrett Ave
Coordinates:	121.3823° W, 37.7903° N
Rim Elevation:	338 feet
Expected Pipe Diameter:	12 inches
Measured Pipe Diameter:	17.5 inches
ADWF:	0.318 mgd
Peak Measured Flow:	1.31 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch

Street View

Plan View

SITE 9

Additional Site Photos

Northeast Effluent Pipe

Northwest Influent Pipe

SITE 9

Additional Site Photos

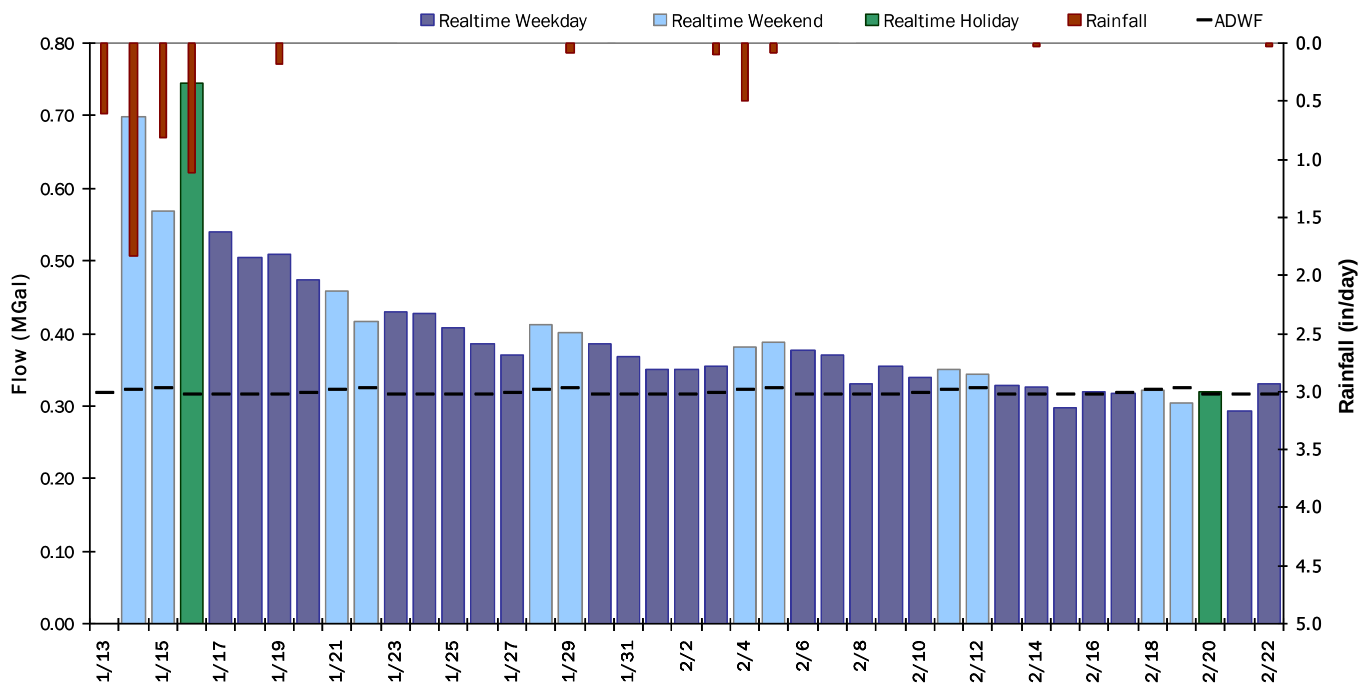
Monitored Northeast Influent Pipe

SITE 9

Period Flow Summary: Daily Flow Totals

Avg Daily Flow: 0.402 MGal Peak Daily Flow: 0.745 MGal Min Daily Flow: 0.293 MGal

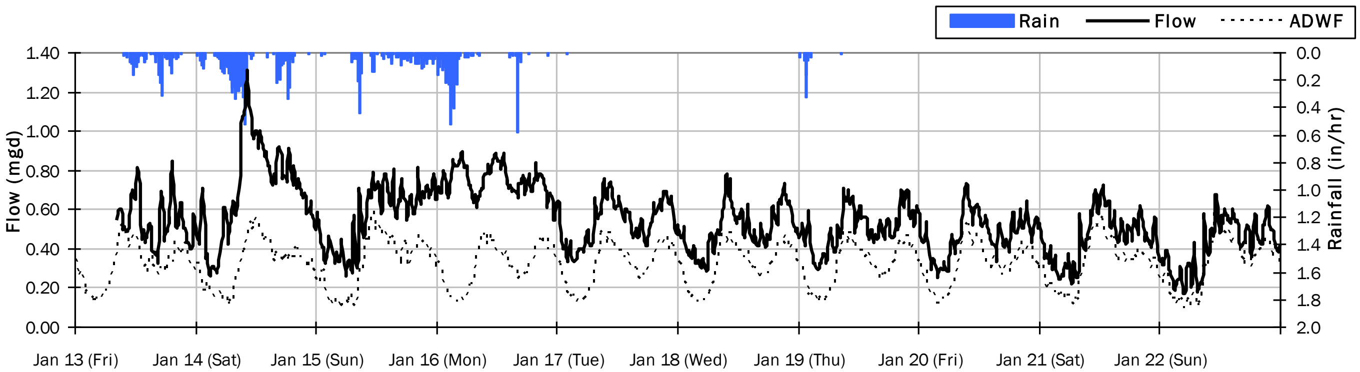
Total Rainfall: 5.37 inches

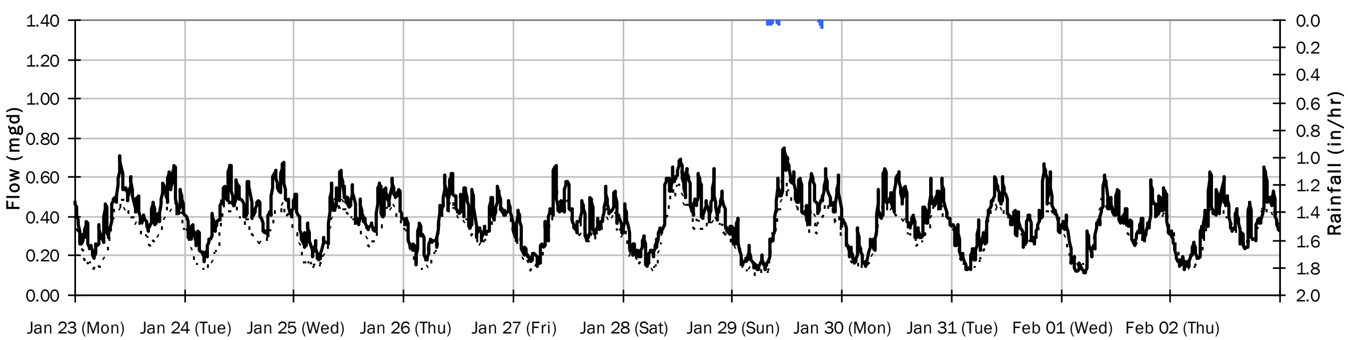


SITE 9

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 4.63 inches Period Avg Flow: 0.463 mgd Period Peak Flow: 1.311 mgd Period Min Flow: 0.115 mgd



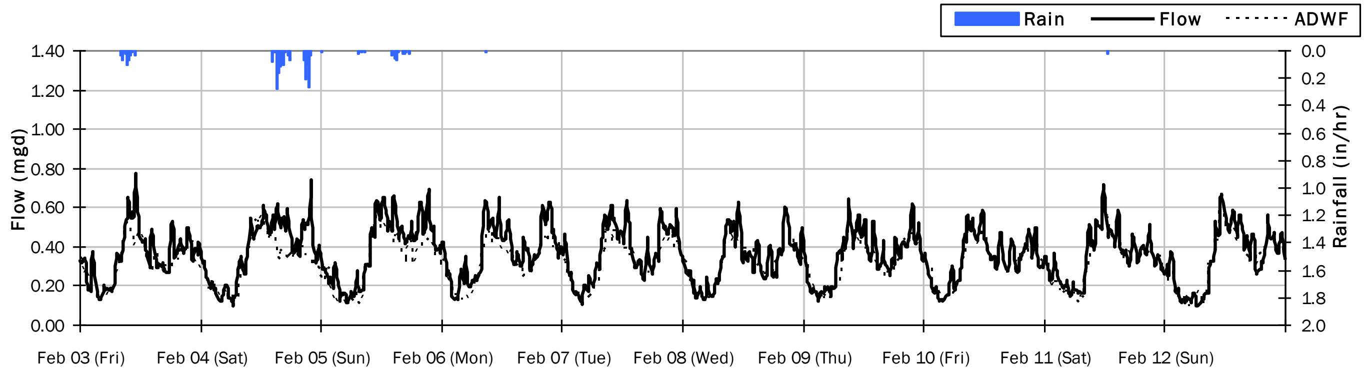


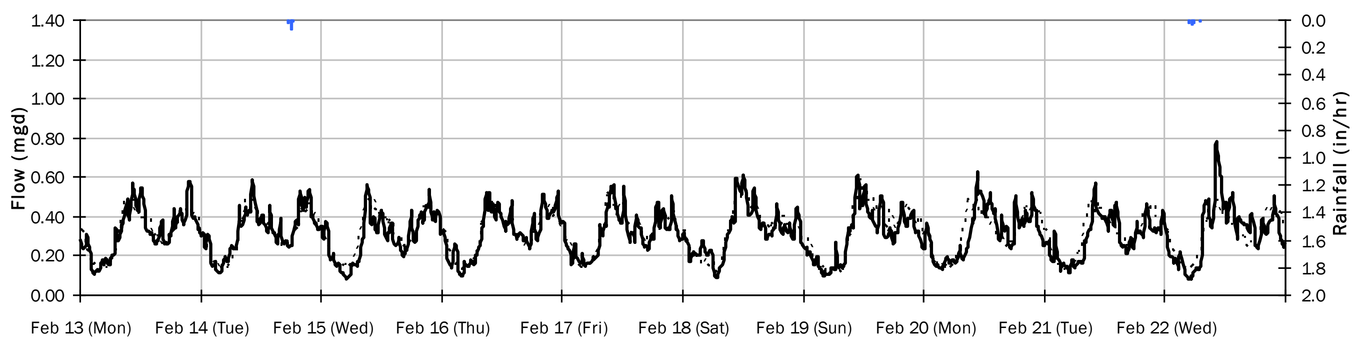
SITE 9

Flow Summary: 2/3/2023 to 2/22/2023

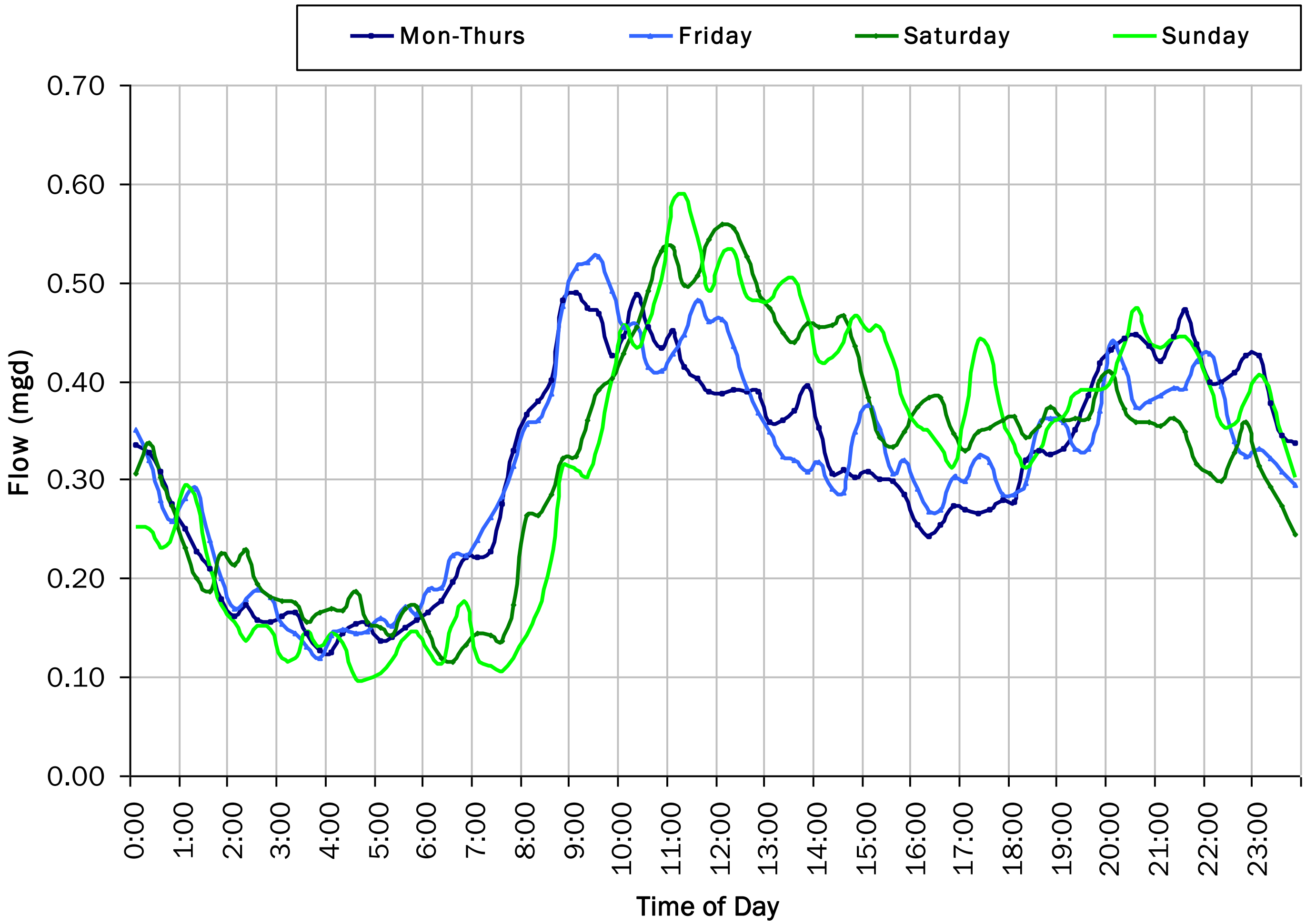
Period Rainfall: 0.74 inches

Period Avg Flow: 0.337 mgd Period Peak Flow: 0.782 mgd Period Min Flow: 0.079 mgd



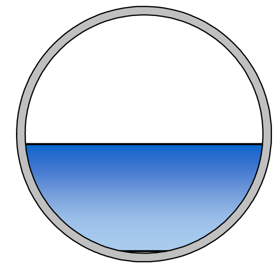


SITE 9
Average Dry Weather Flow Hydrographs



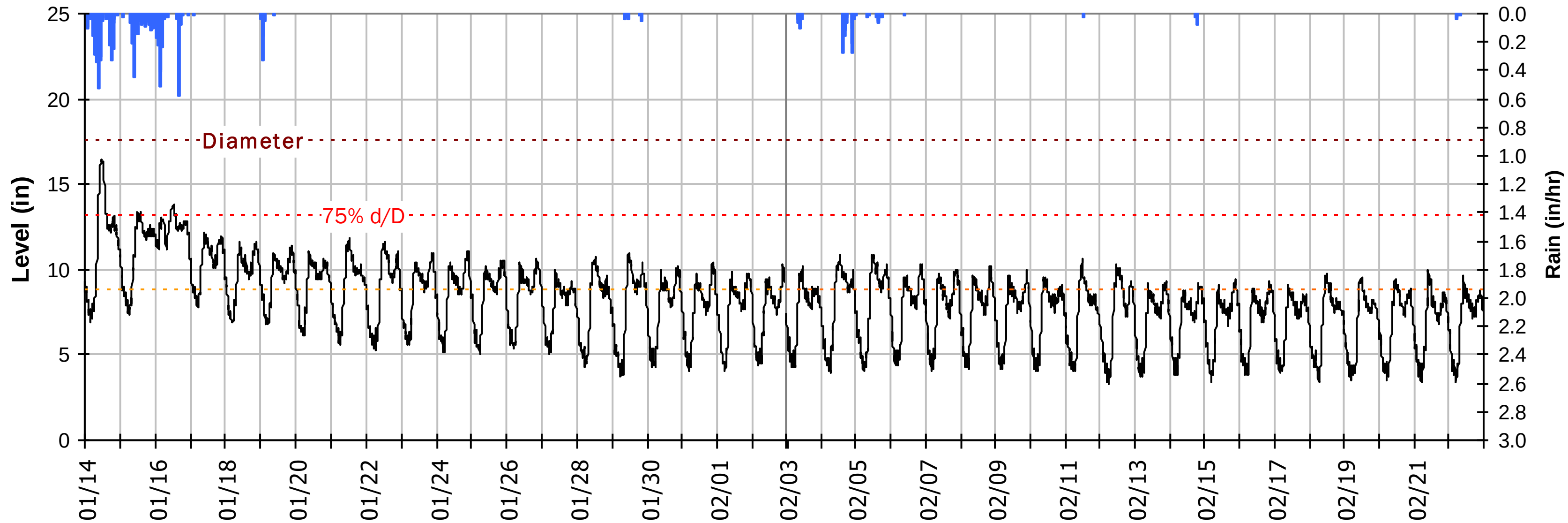
ADWF:

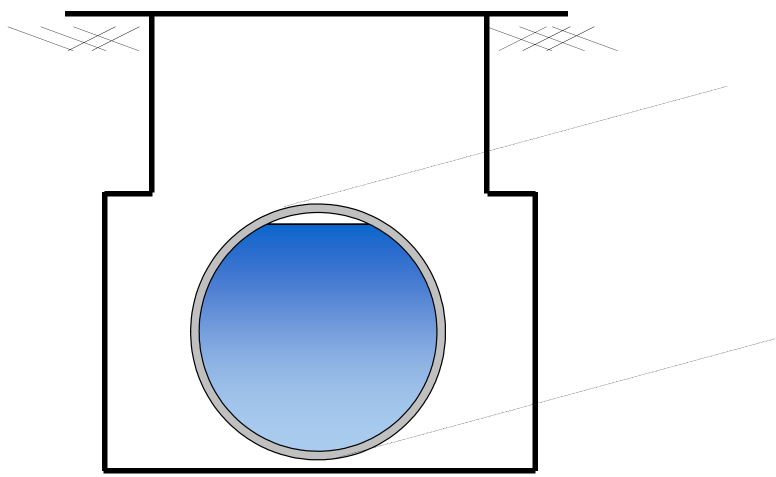
0.318 *mgd*



SITE 9
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Peak Level Period



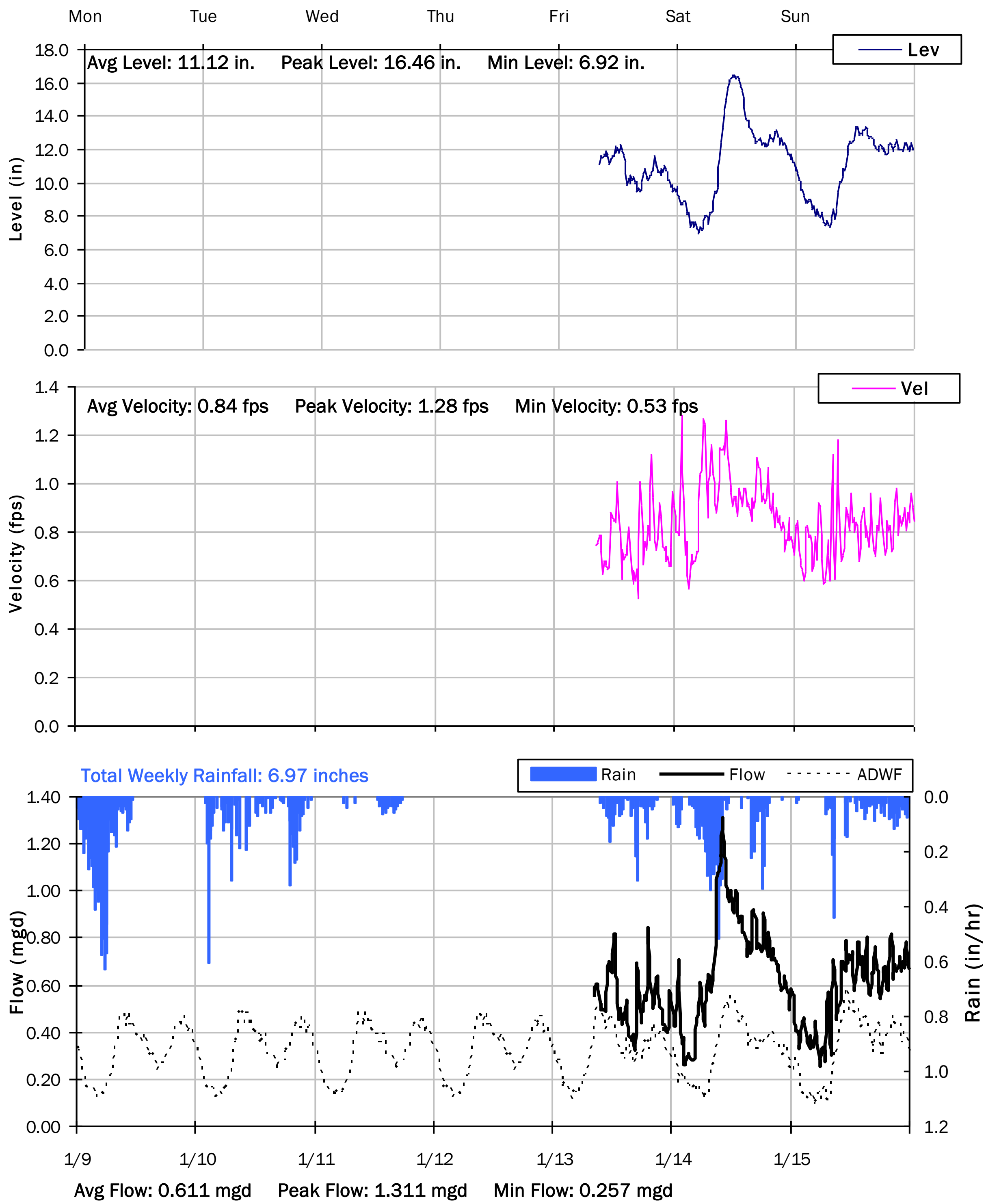


Pipe Diameter: 17.5 *inches*

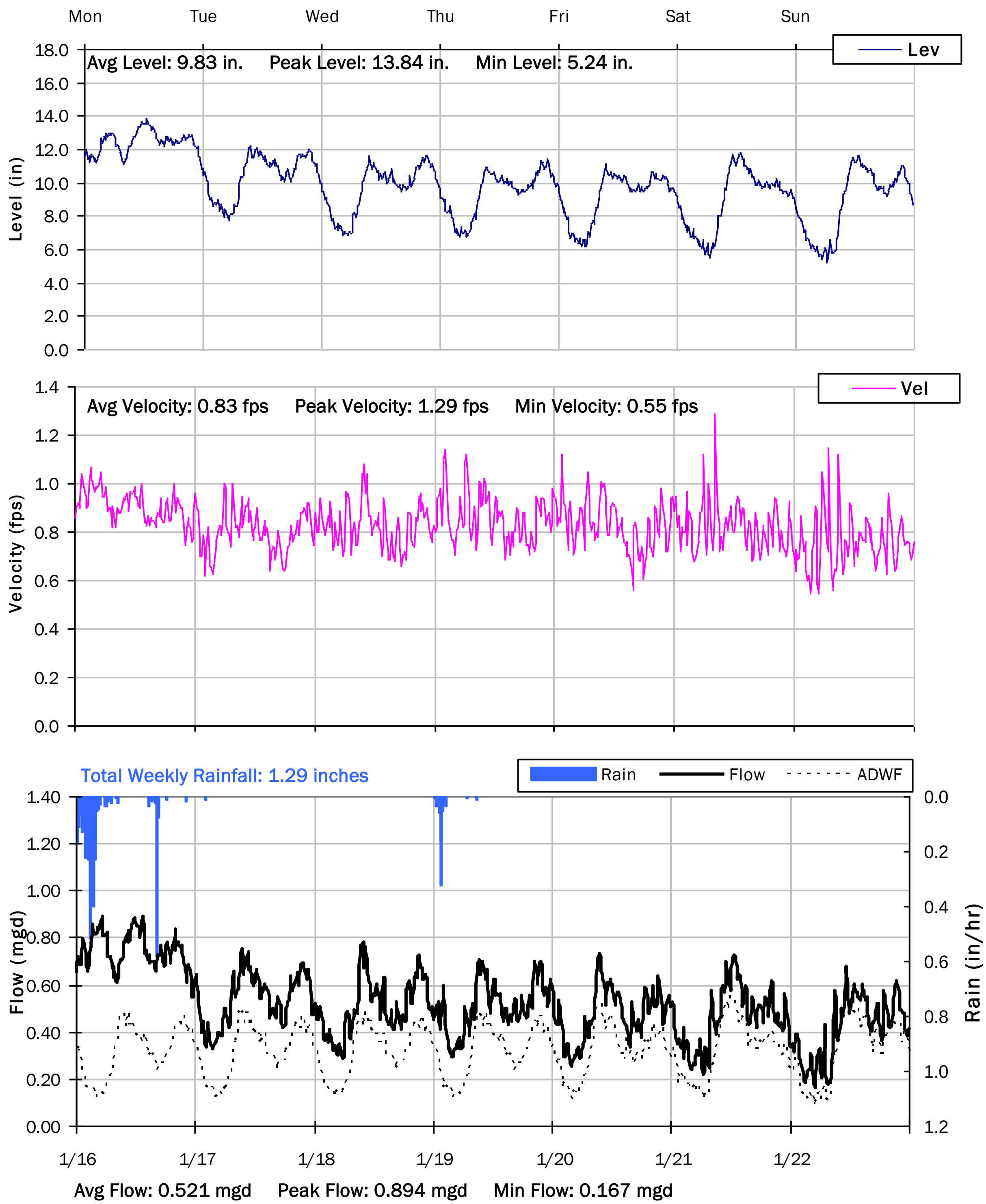
Peak Measured Level: 16.5 *inches*

Peak d/D Ratio: 0.94

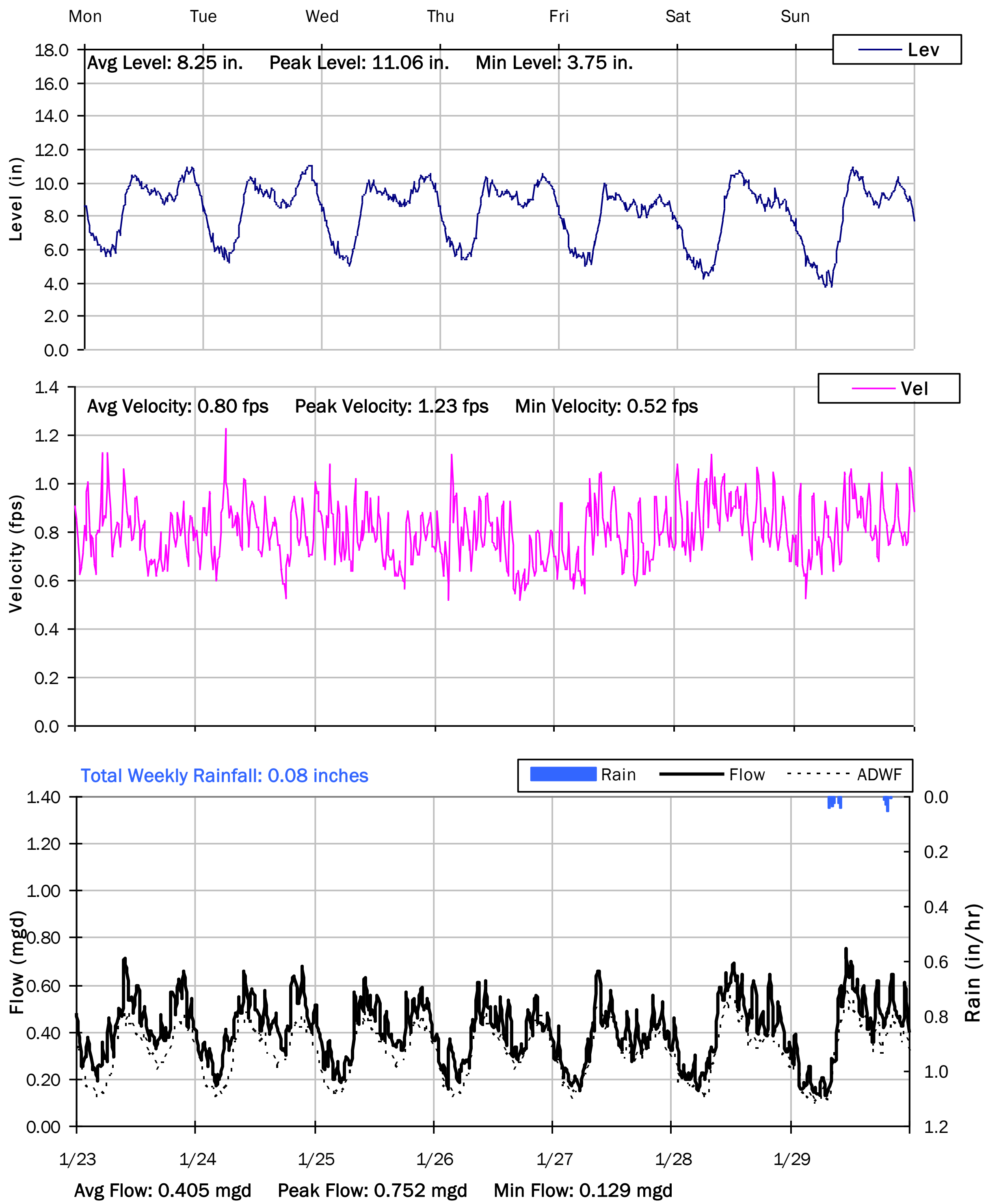
SITE 9
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023



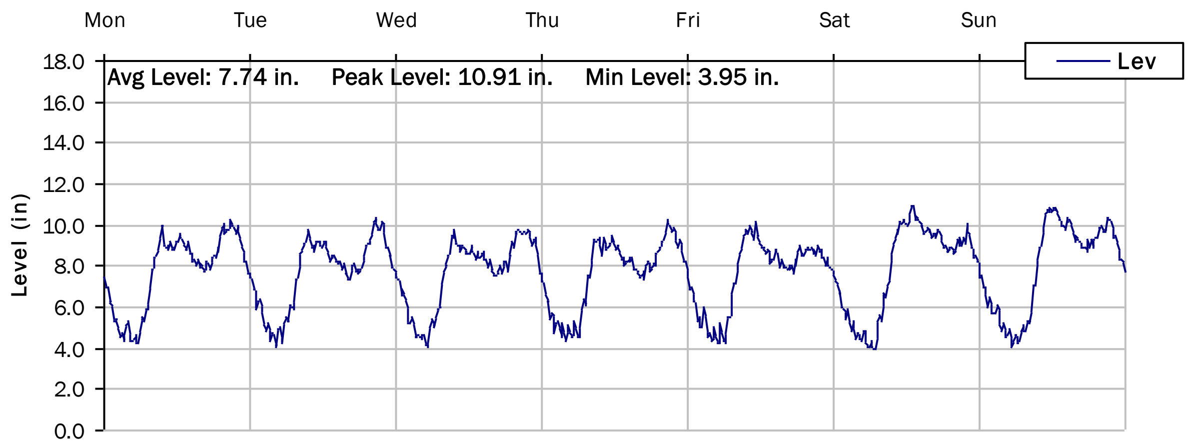
SITE 9
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023

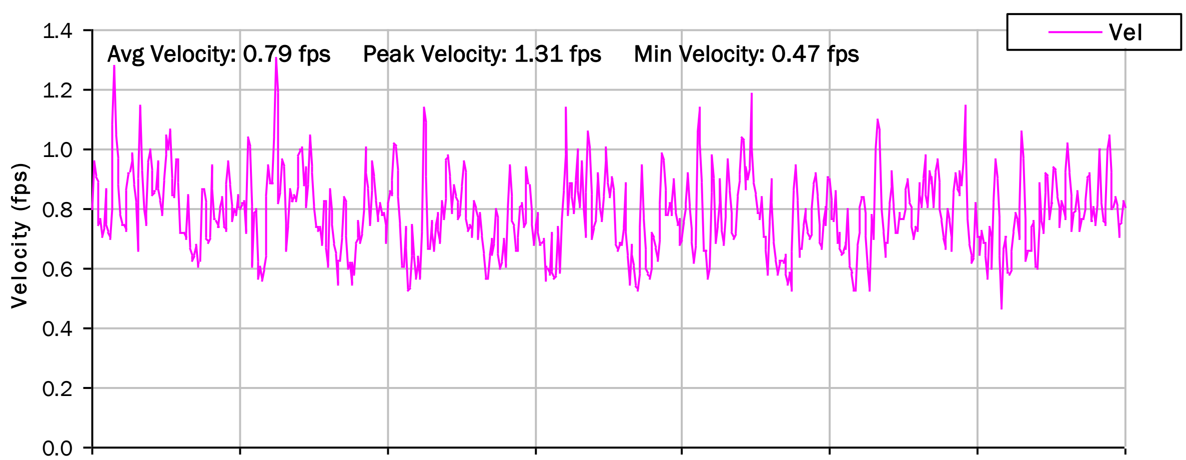


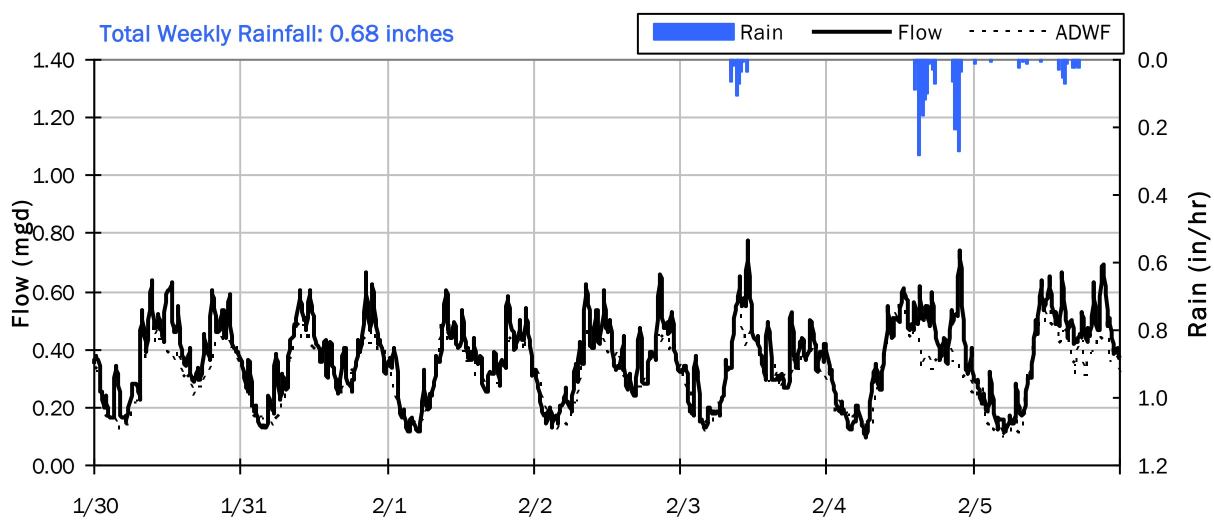
SITE 9
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023



SITE 9
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023

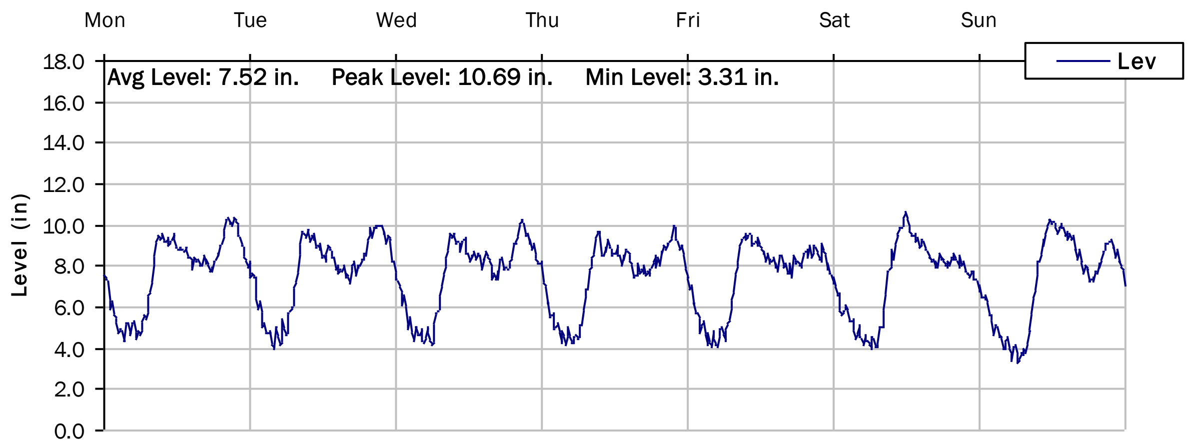


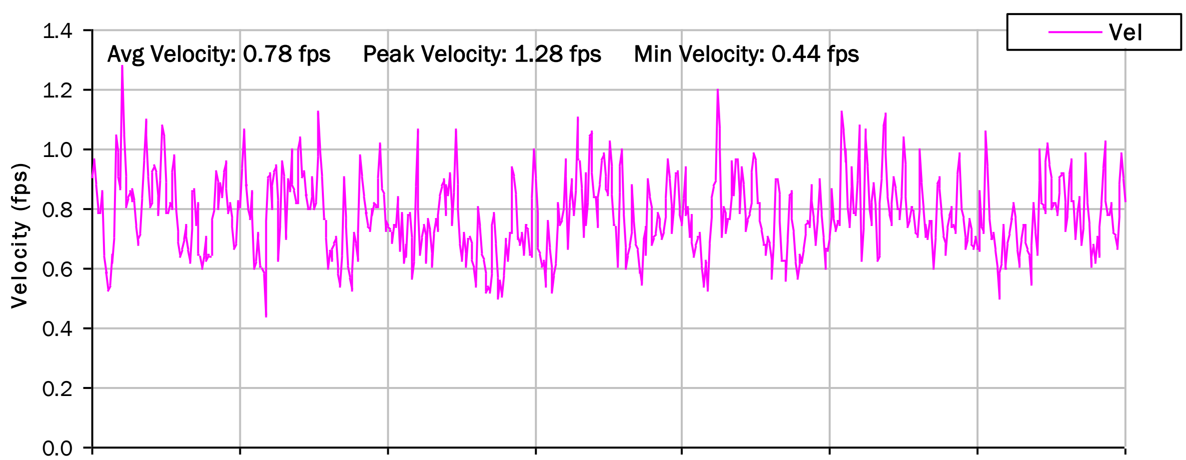


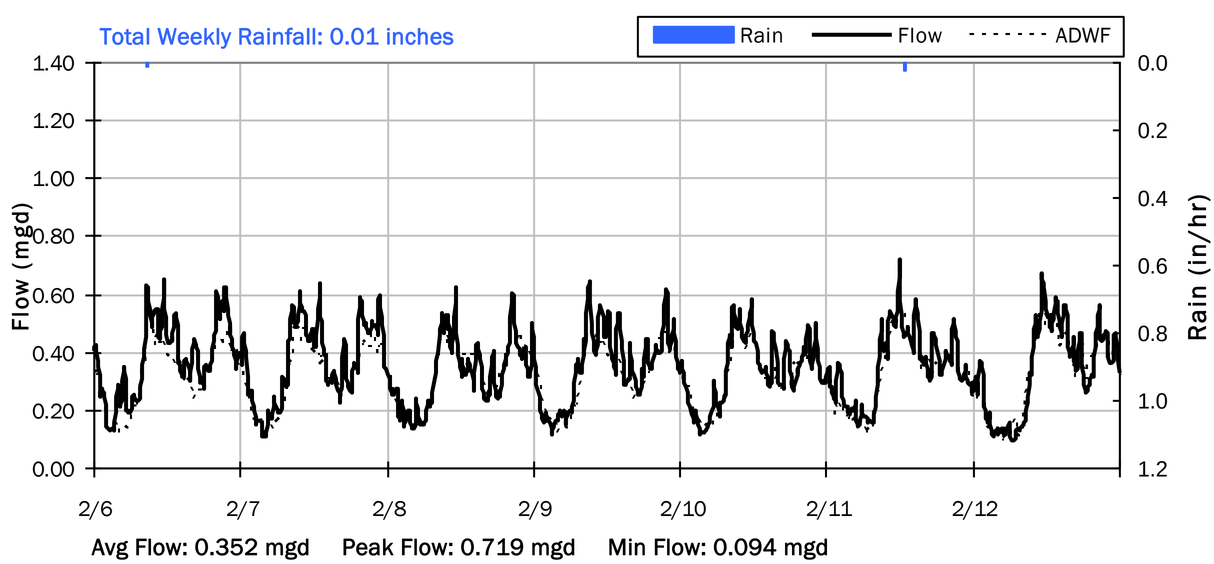


Avg Flow: 0.369 mgd Peak Flow: 0.773 mgd Min Flow: 0.099 mgd

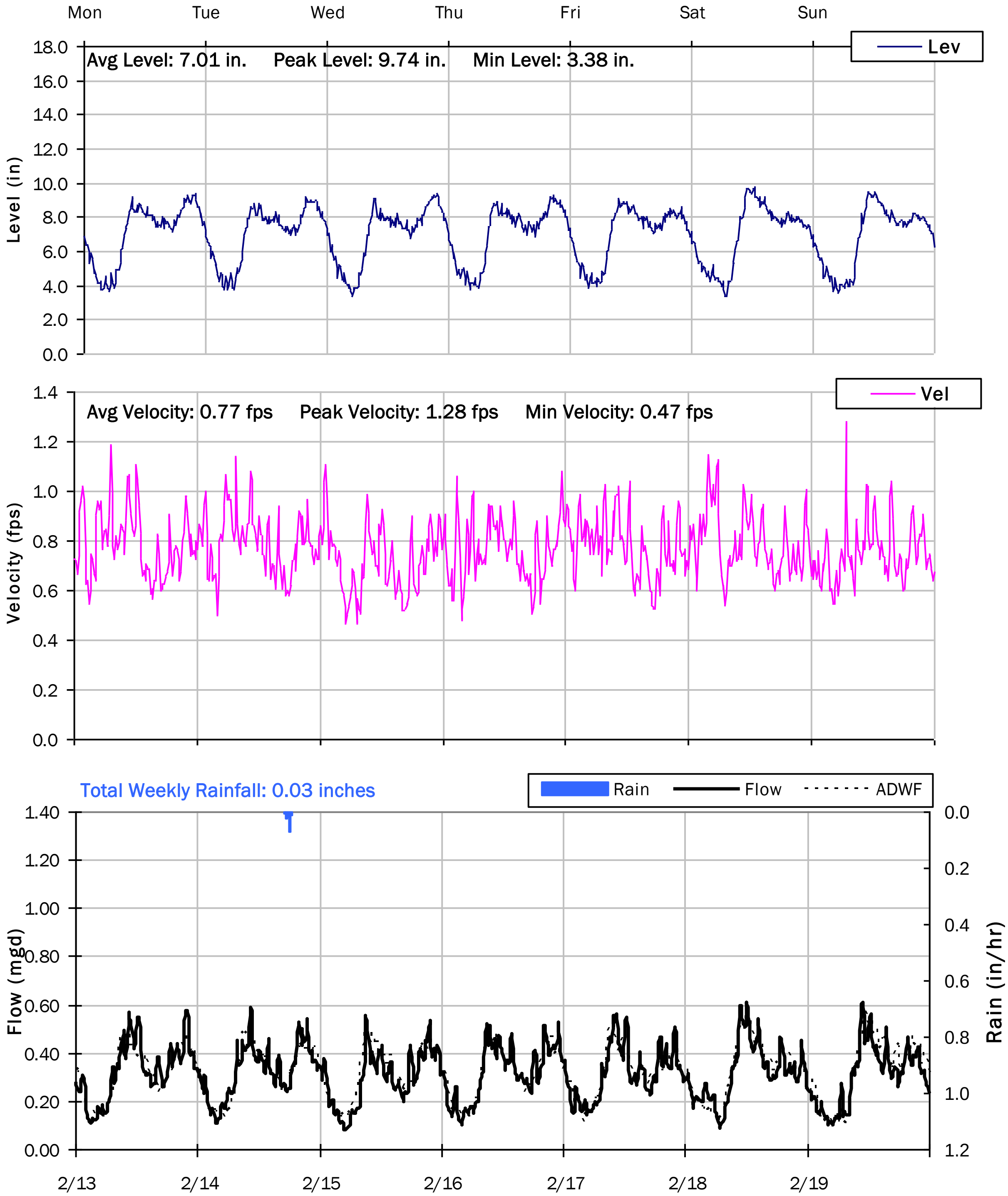
SITE 9
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023





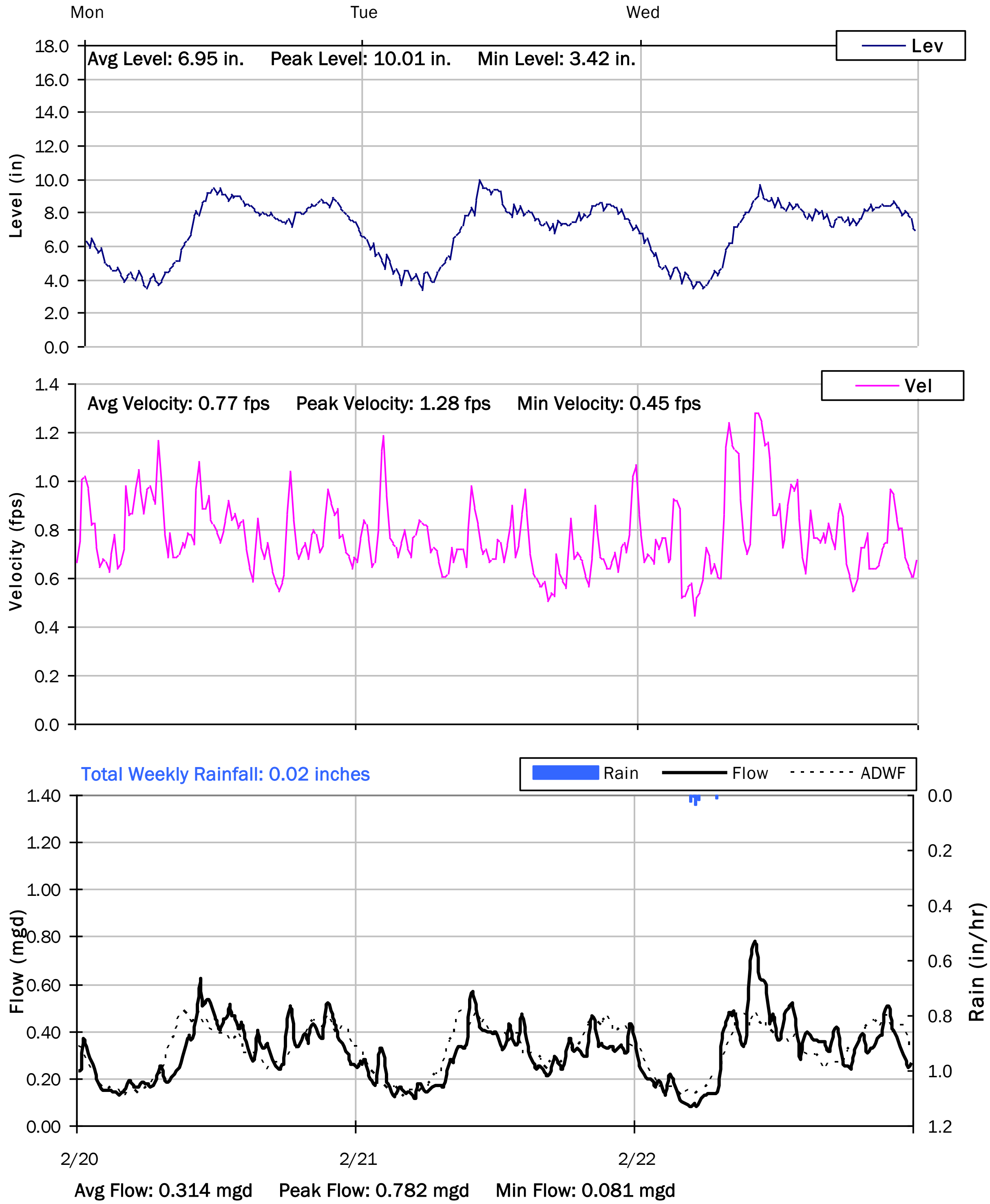


SITE 9
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



Avg Flow: 0.316 mgd Peak Flow: 0.611 mgd Min Flow: 0.079 mgd

SITE 9
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023



Monitoring Site: Site 10

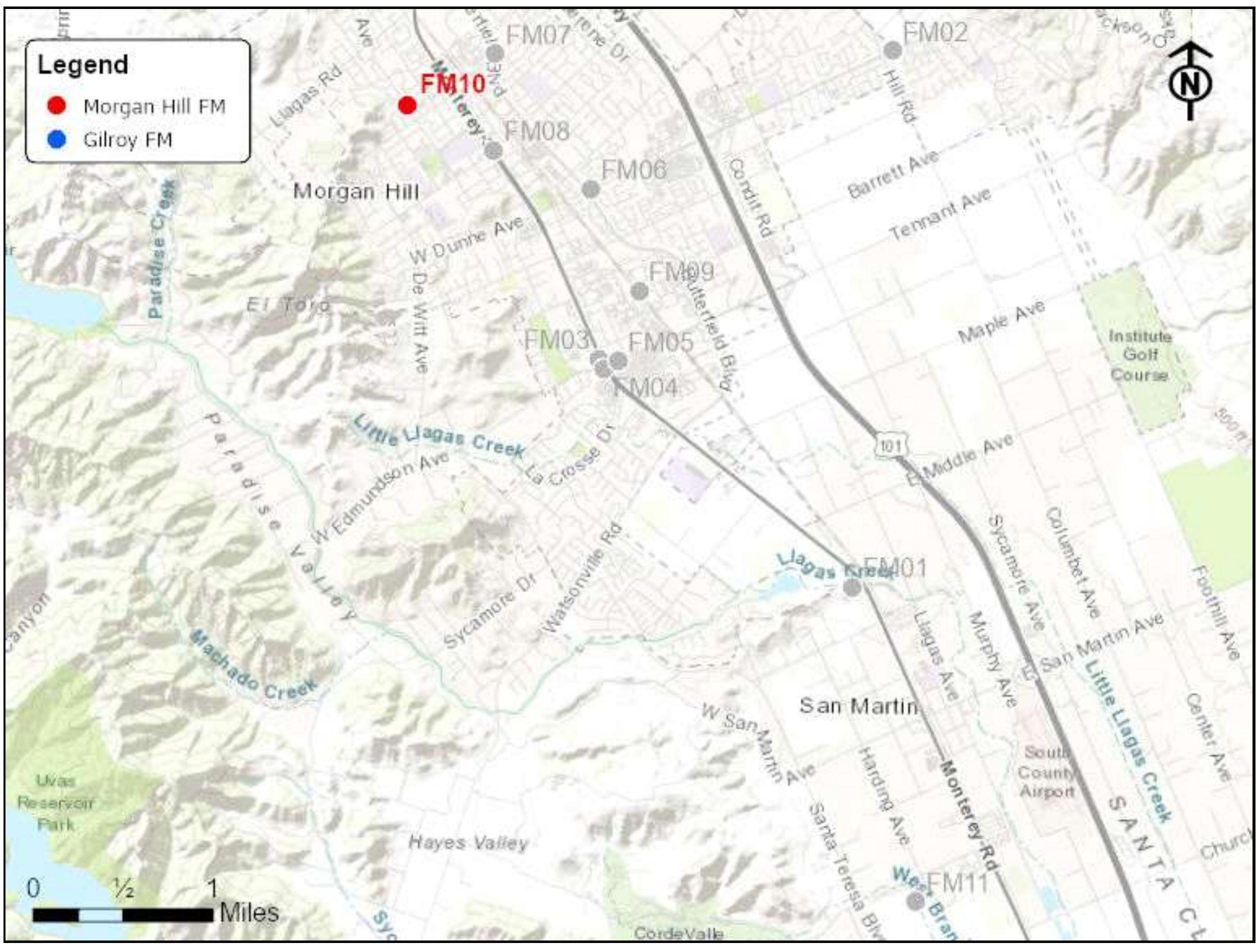
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: 18052 Hale Ave

Data Summary Report



Vicinity Map: Site 10

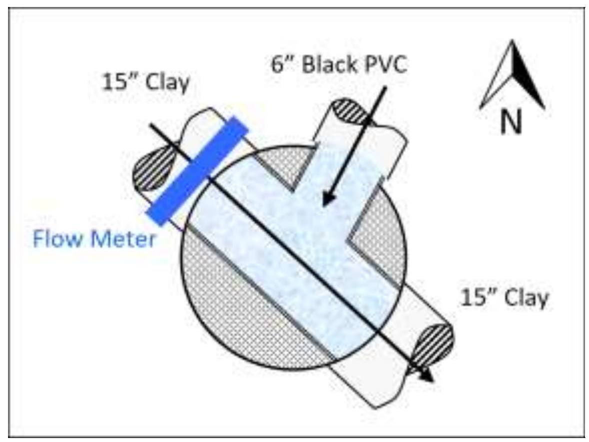
SITE 10

Site Information MH ID: G4-A.MH.017

Location:	18052 Hale Ave
Coordinates:	121.3948° W, 37.5227° N
Rim Elevation:	354 feet
Expected Pipe Diameter:	15 inches
Measured Pipe Diameter:	15 inches
ADWF:	0.183 mgd
Peak Measured Flow:	0.96 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch


Street View


Plan View

SITE 10

Additional Site Photos

Southwest Effluent Pipe

Monitored Northwest influent Pipe

SITE 10

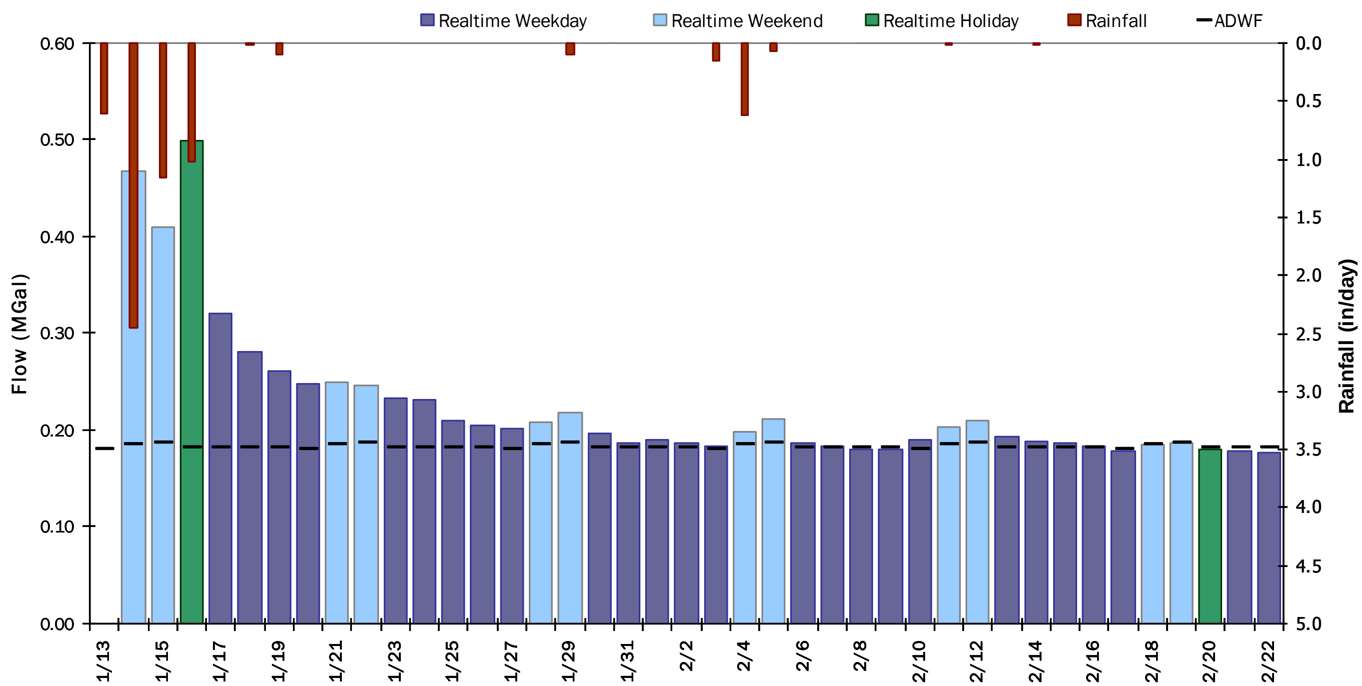
Additional Site Photos

Northeast Influent Pipe

SITE 10
Period Flow Summary: Daily Flow Totals

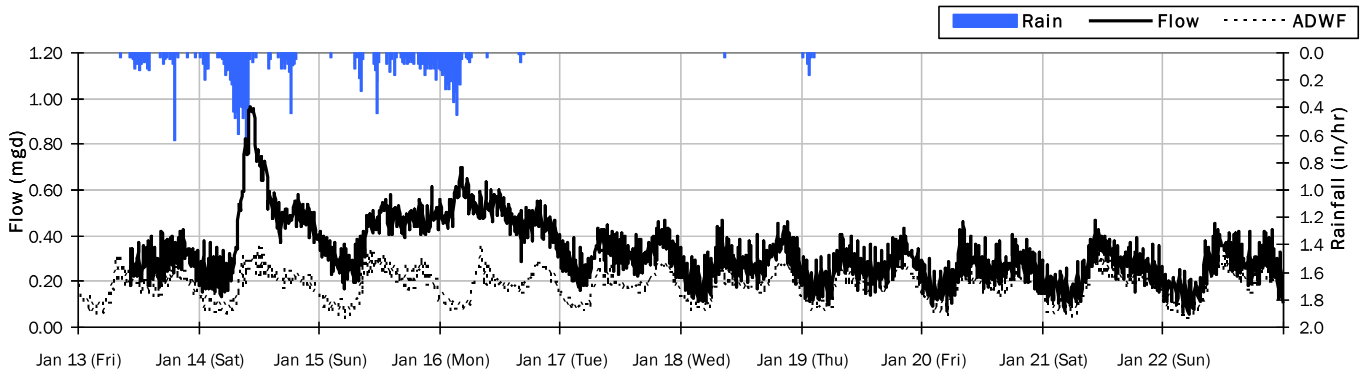
Avg Daily Flow: 0.227 MGal Peak Daily Flow: 0.499 MGal Min Daily Flow: 0.176 MGal

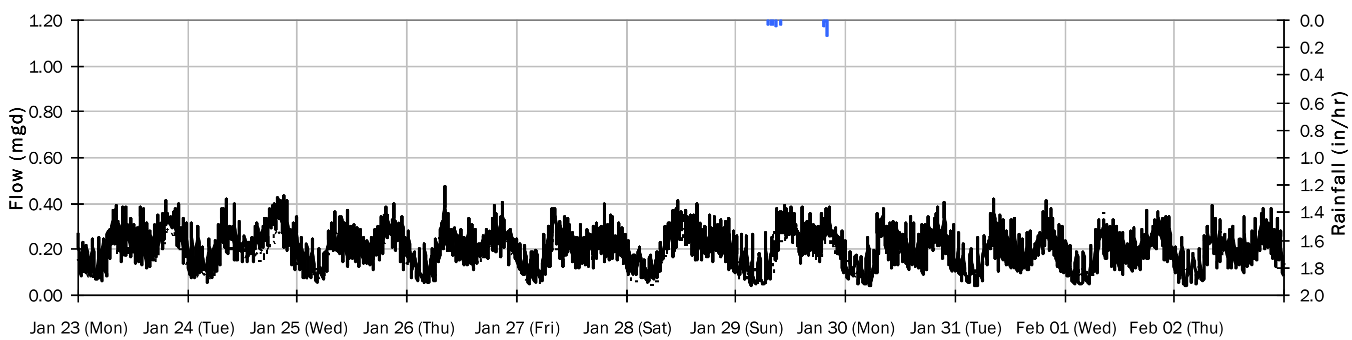
Total Rainfall: 6.27 inches



SITE 10
Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 5.43 inches Period Avg Flow: 0.264 mgd Period Peak Flow: 0.962 mgd Period Min Flow: 0.039 mgd



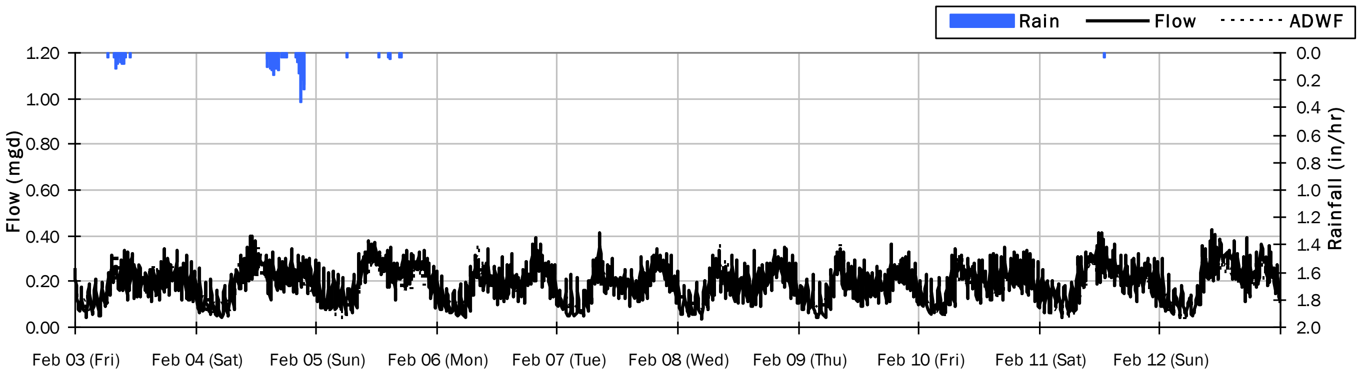


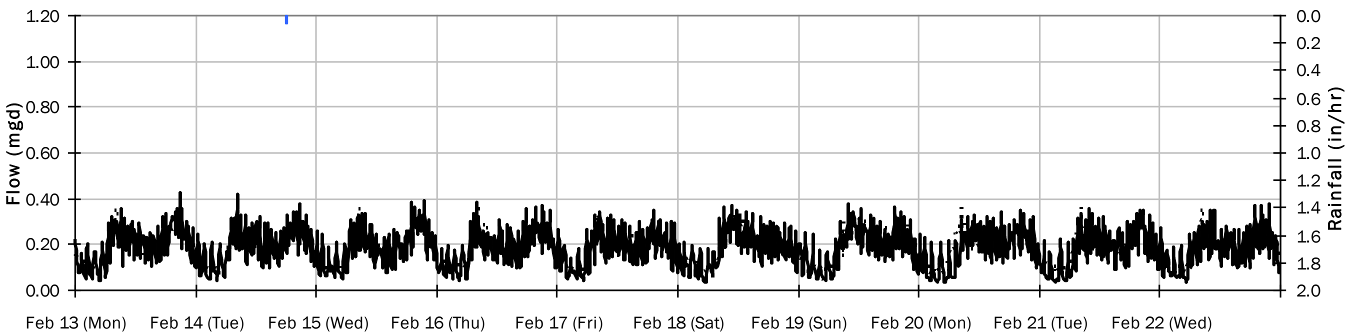
SITE 10

Flow Summary: 2/3/2023 to 2/22/2023

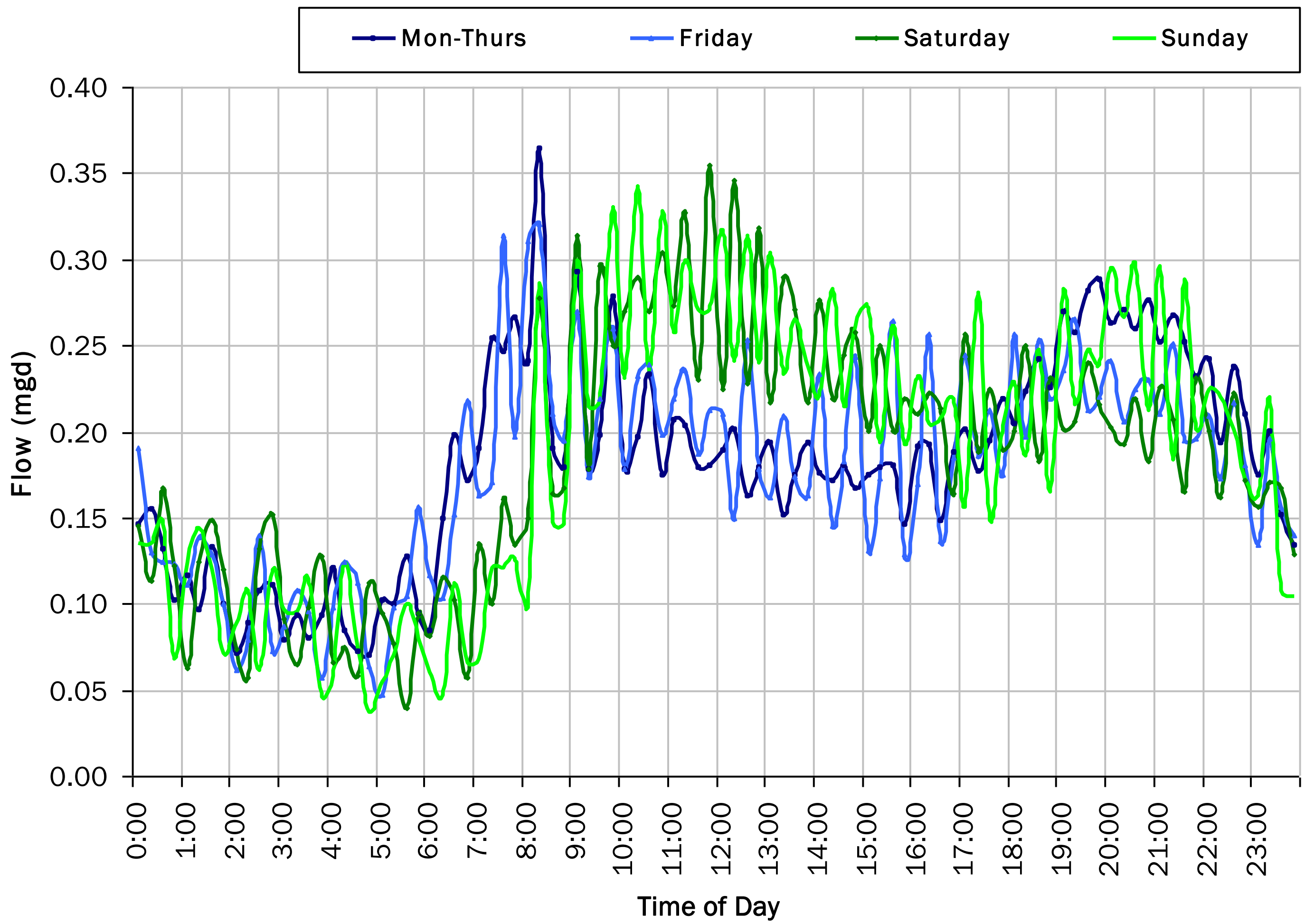
Period Rainfall: 0.86 inches

Period Avg Flow: 0.188 mgd   Period Peak Flow: 0.427 mgd   Period Min Flow: 0.032 mgd



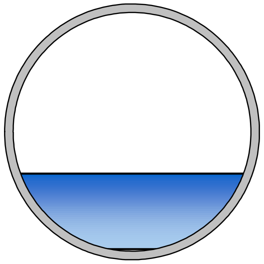


SITE 10
Average Dry Weather Flow Hydrographs



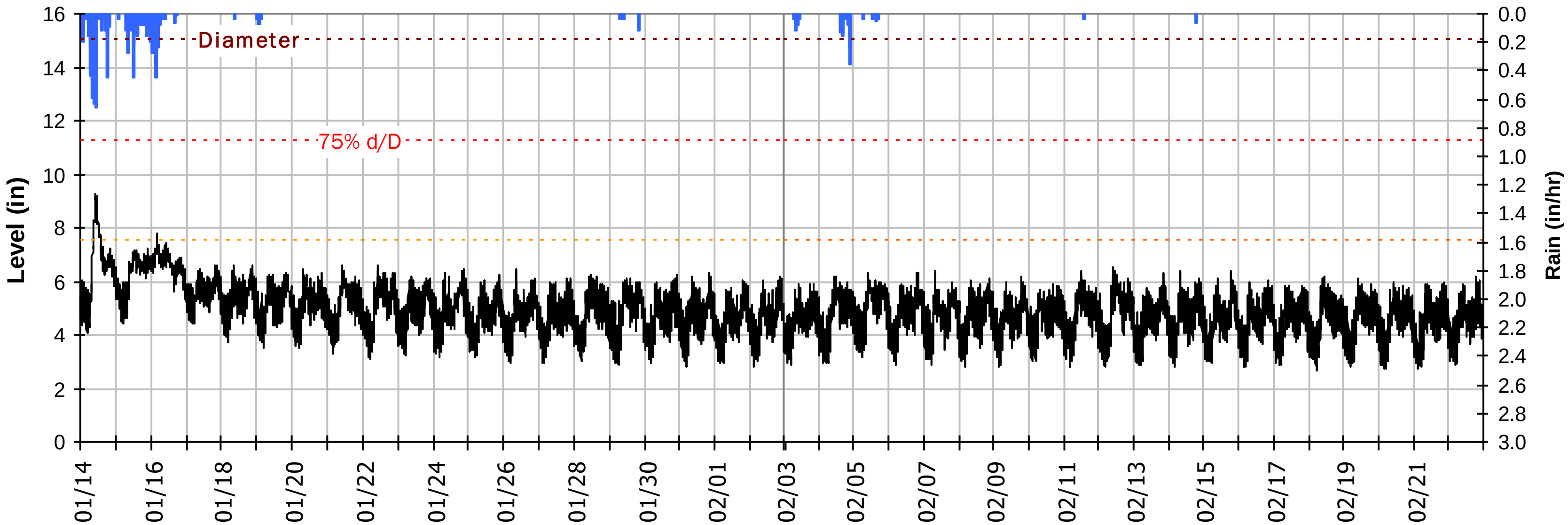
ADWF:

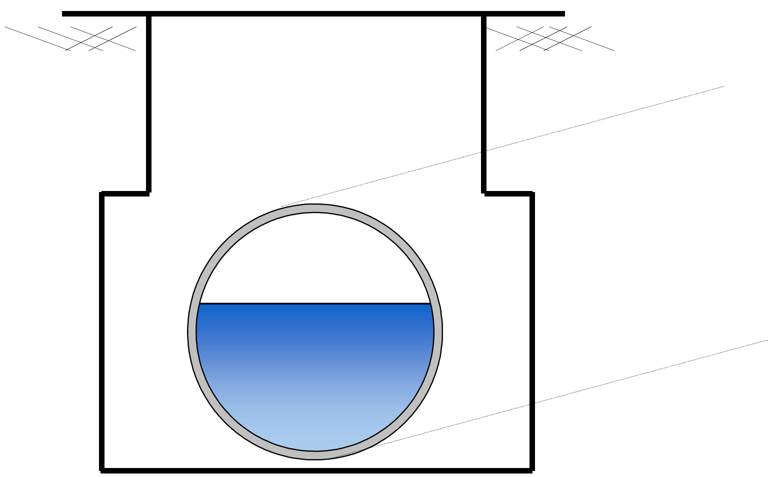
0.183 *mgd*



SITE 10
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Peak Level Period



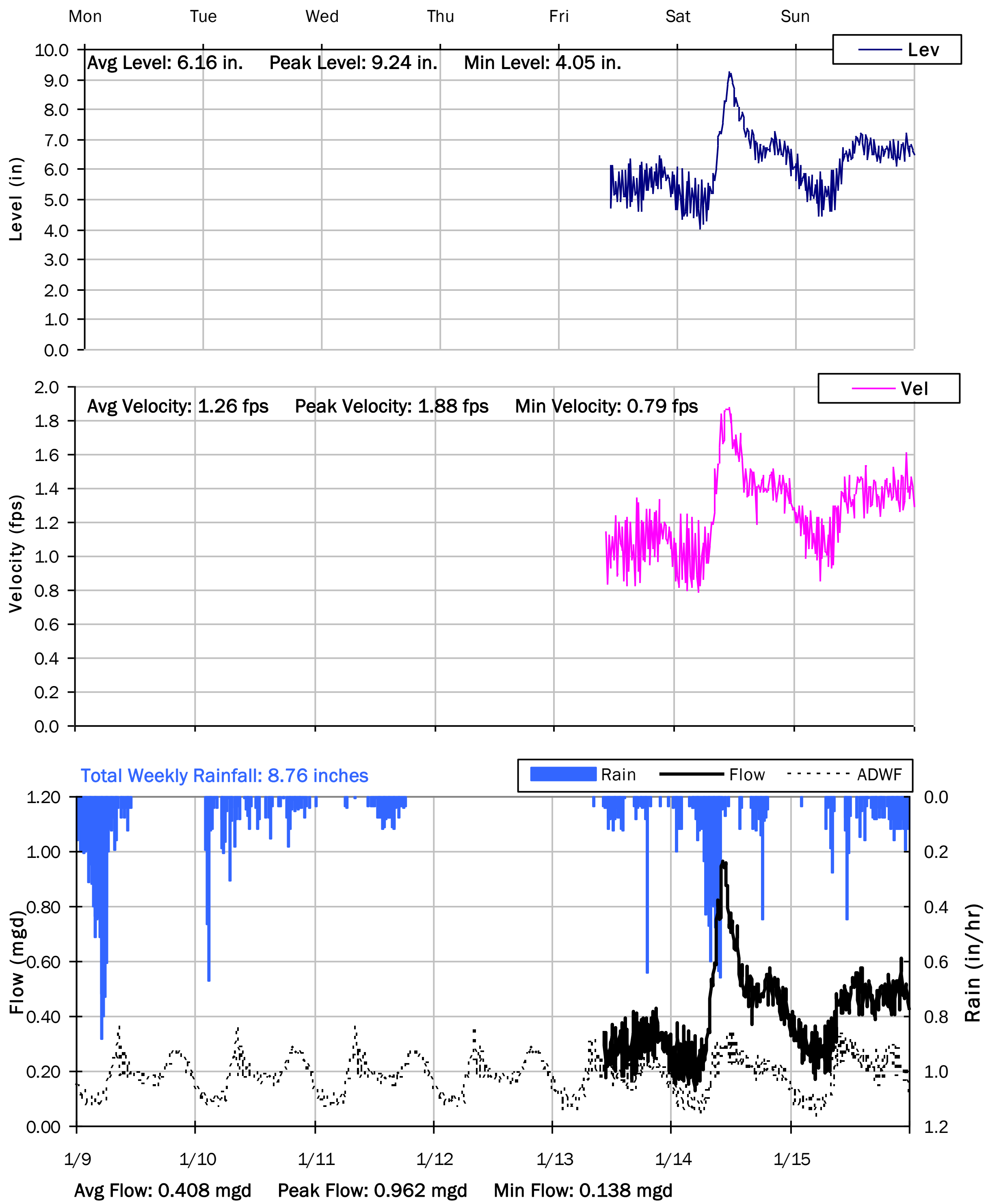


Pipe Diameter: 15 *inches*

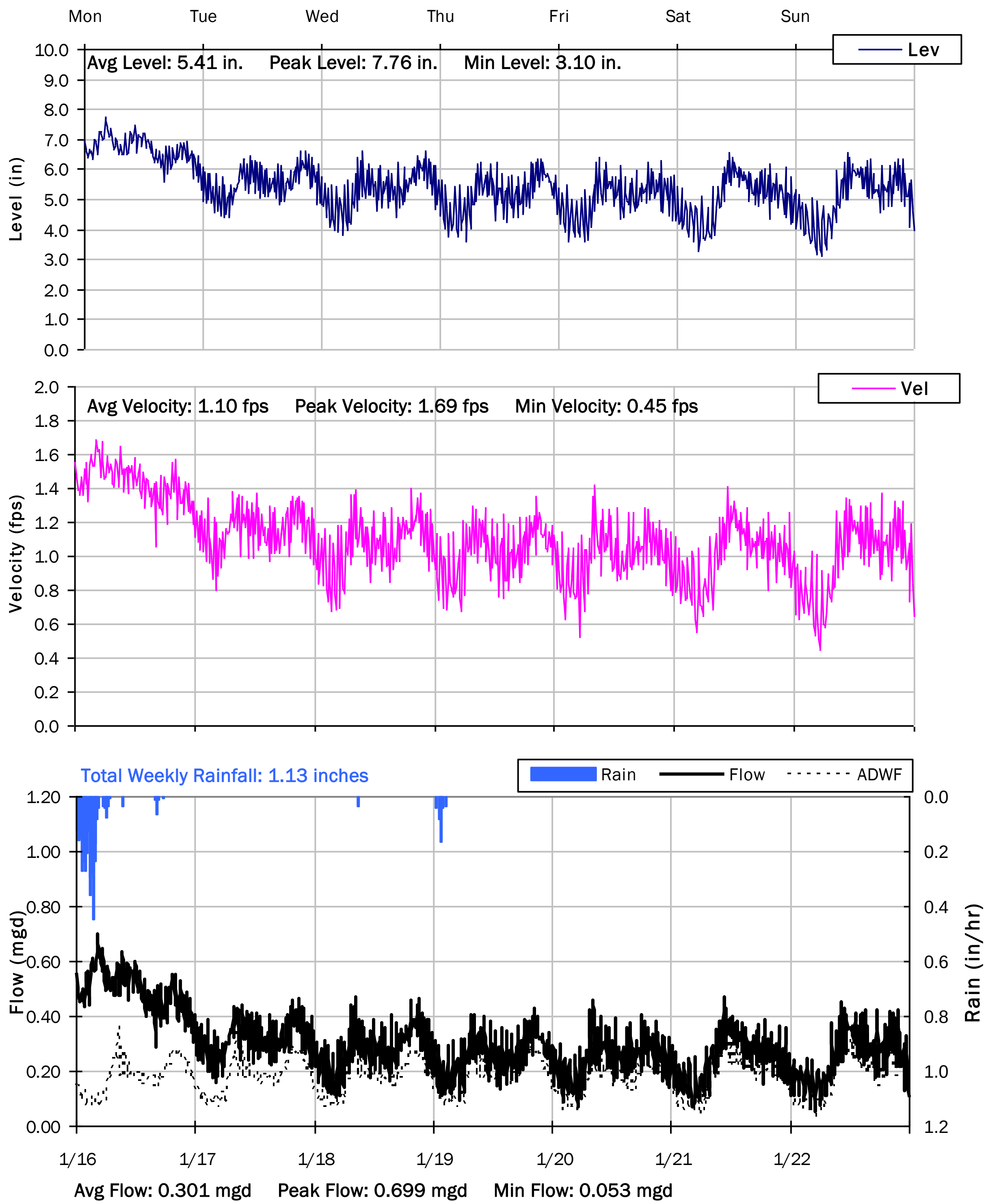
Peak Measured Level: 9.24 *inches*

Peak d/D Ratio: 0.62

SITE 10
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023



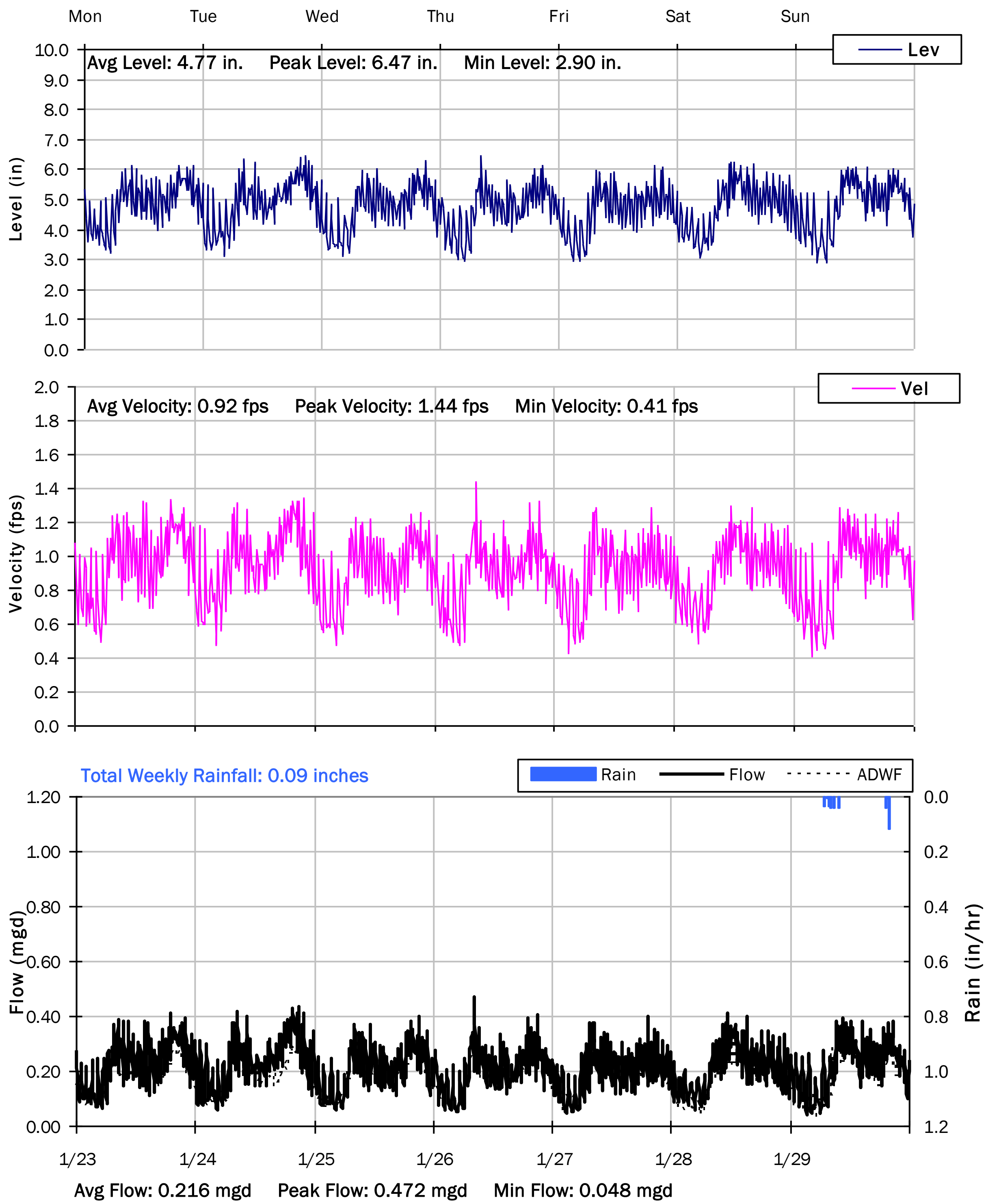
SITE 10
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023



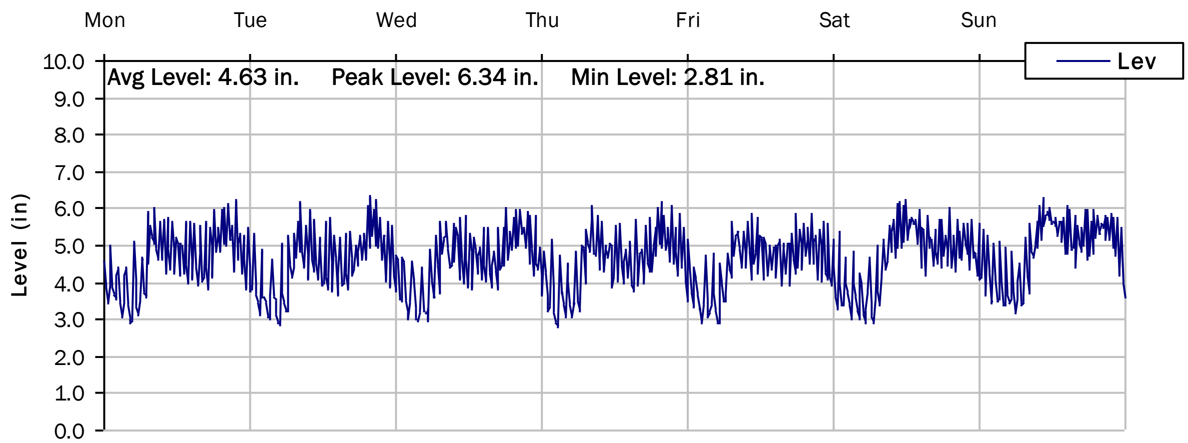
SITE 10

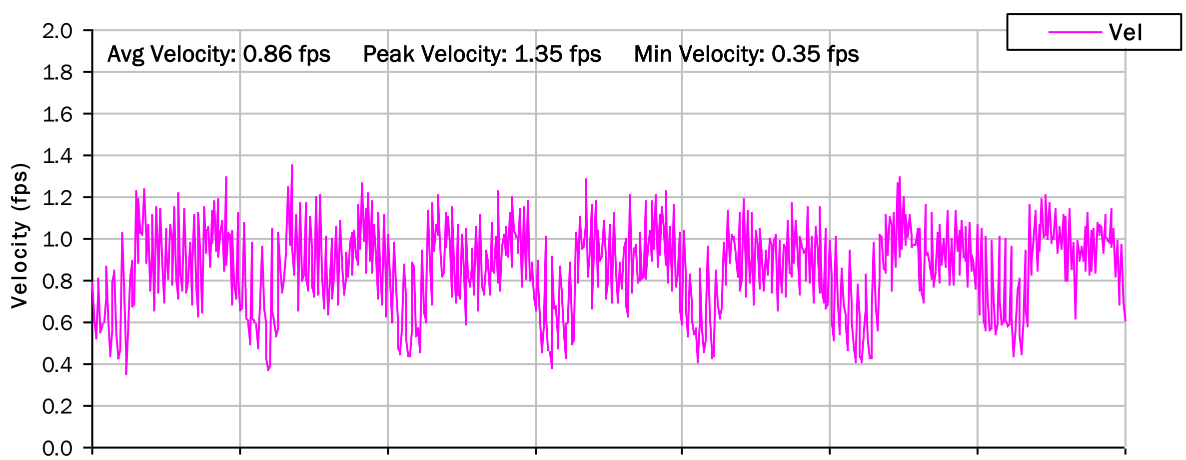
Weekly Level, Velocity and Flow Hydrographs

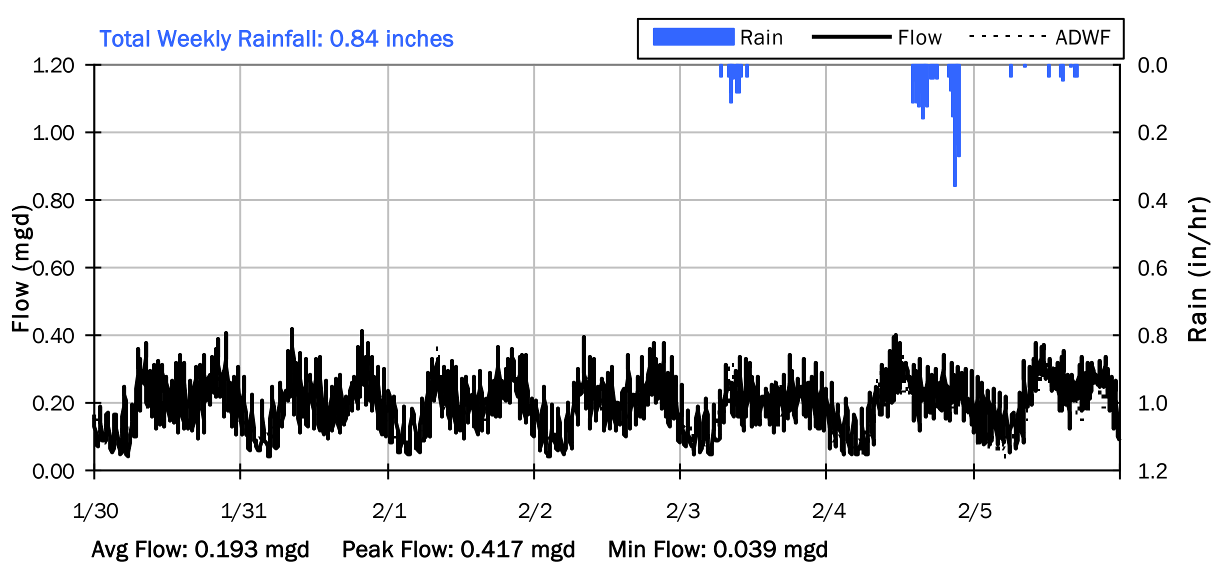
1/23/2023 to 1/30/2023



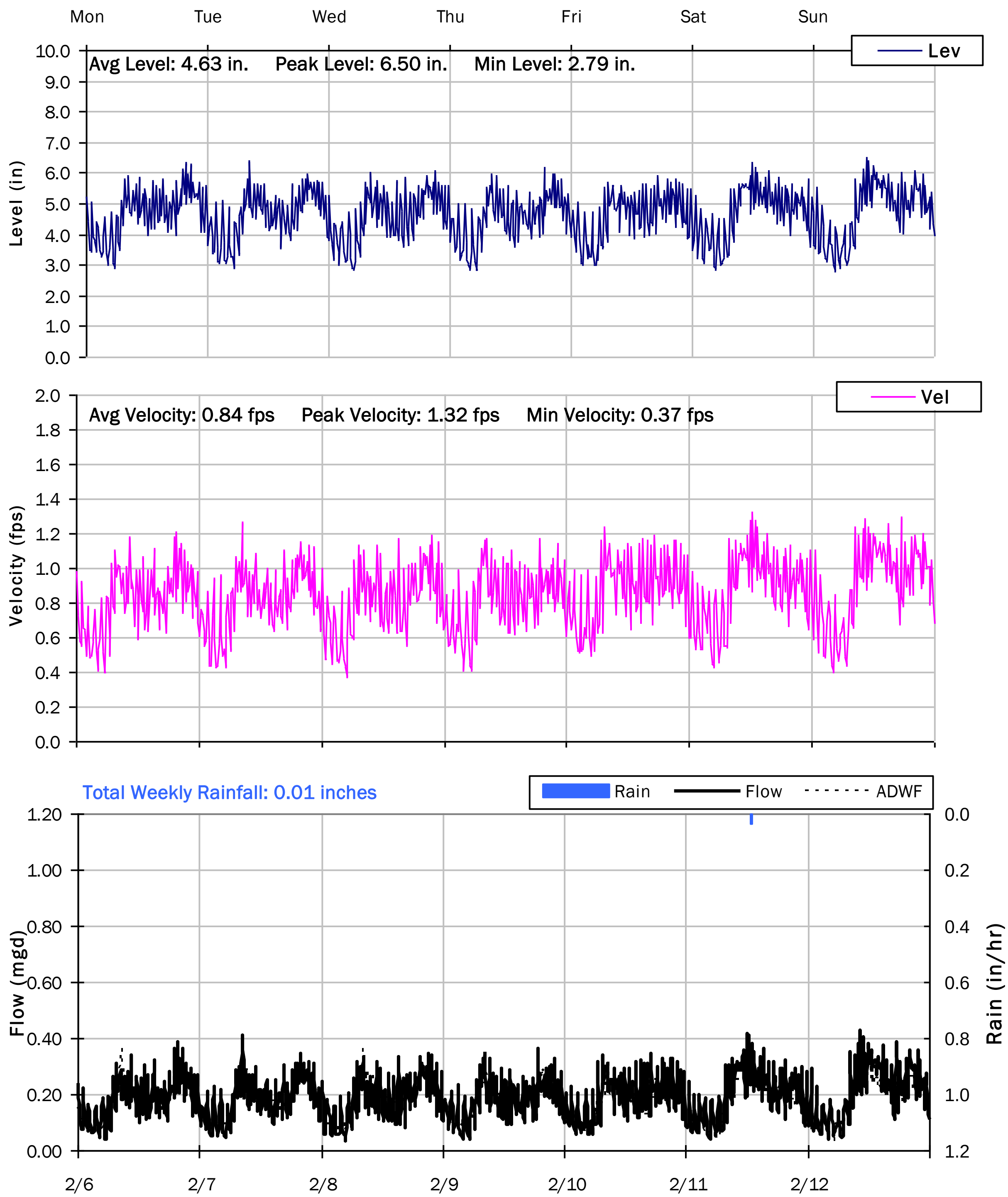
SITE 10
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023





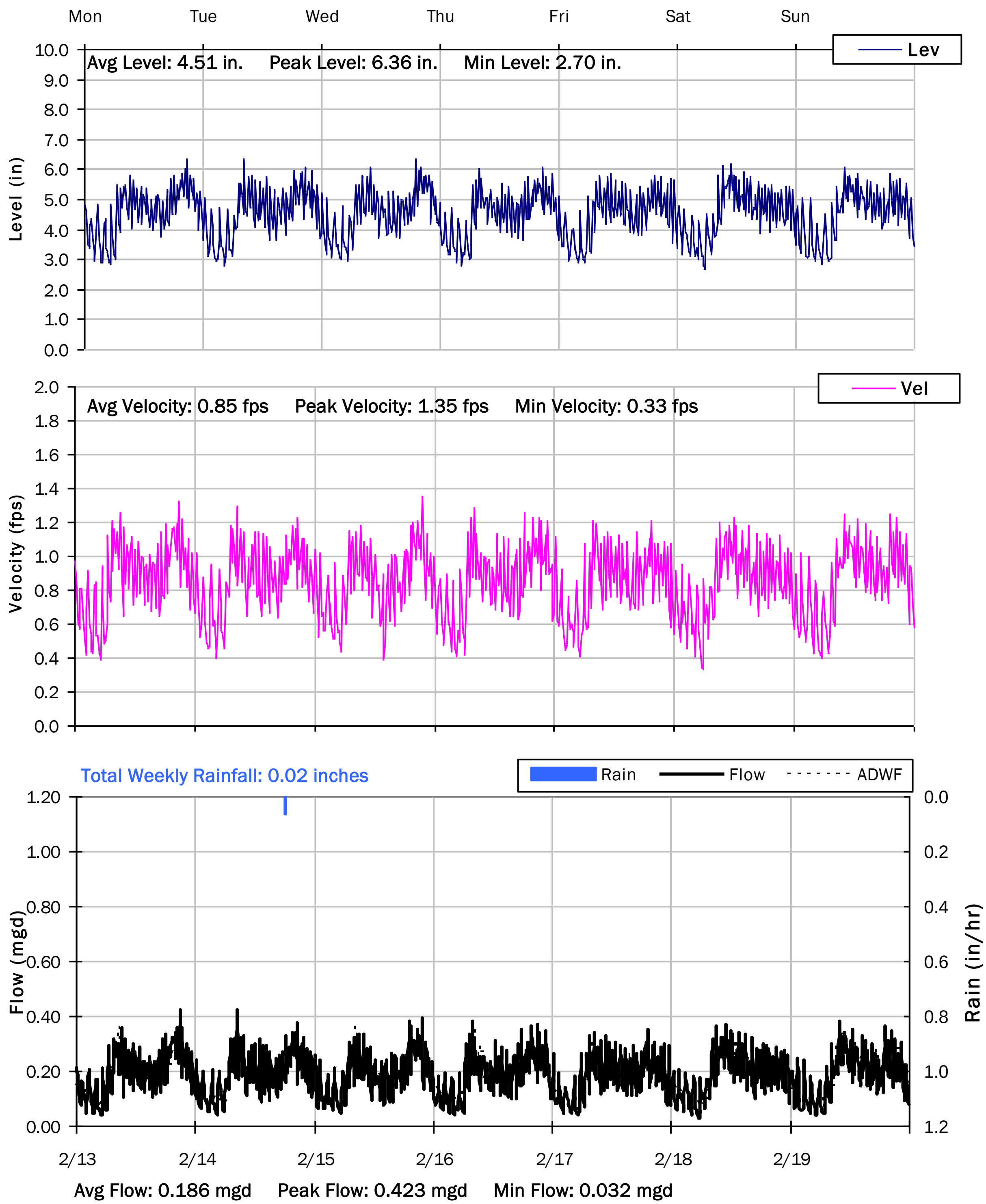


SITE 10
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

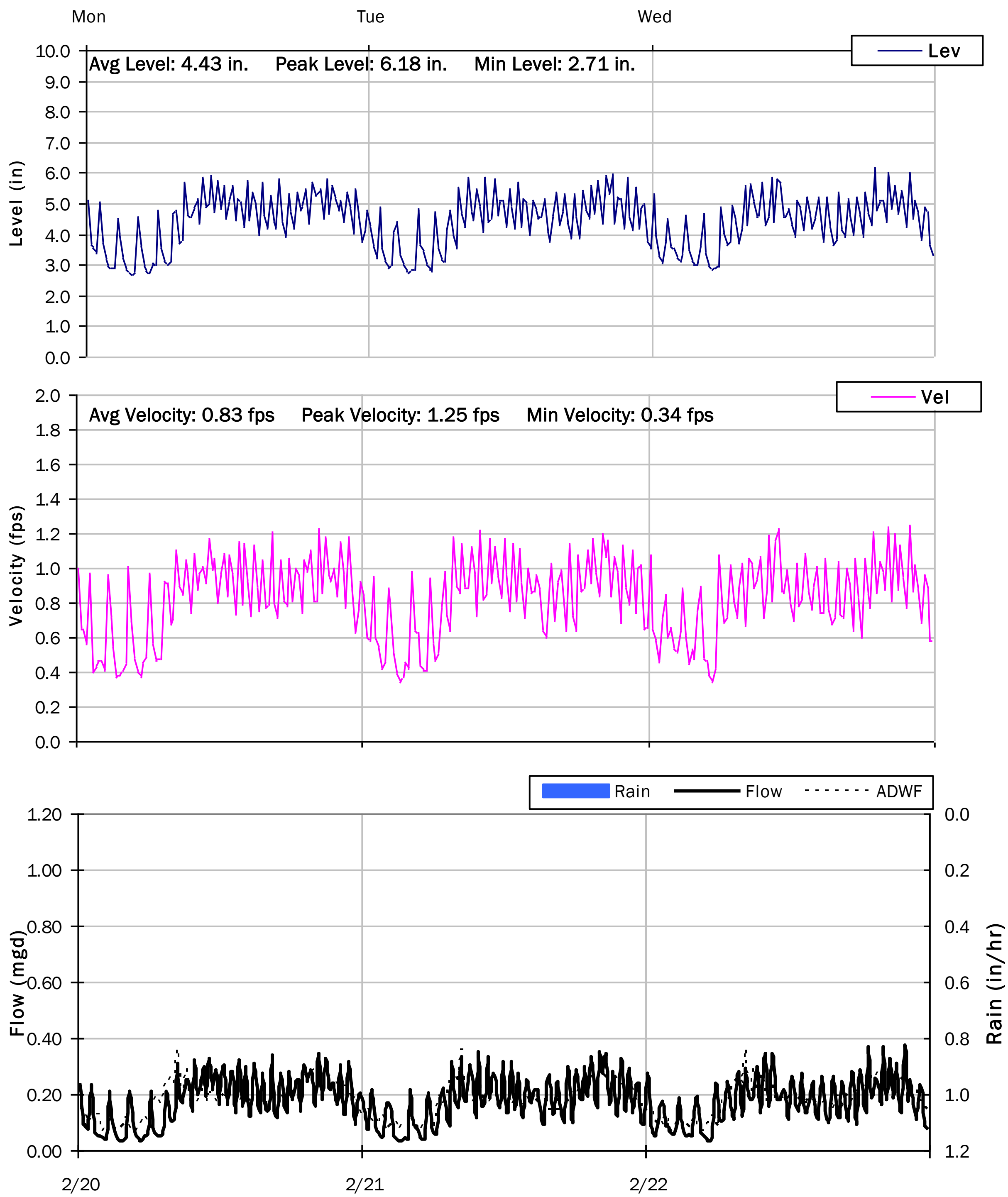


Avg Flow: 0.191 mgd Peak Flow: 0.427 mgd Min Flow: 0.038 mgd

SITE 10
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



SITE 10
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023



Avg Flow: 0.178 mgd Peak Flow: 0.375 mgd Min Flow: 0.035 mgd

Monitoring Site: Site 11

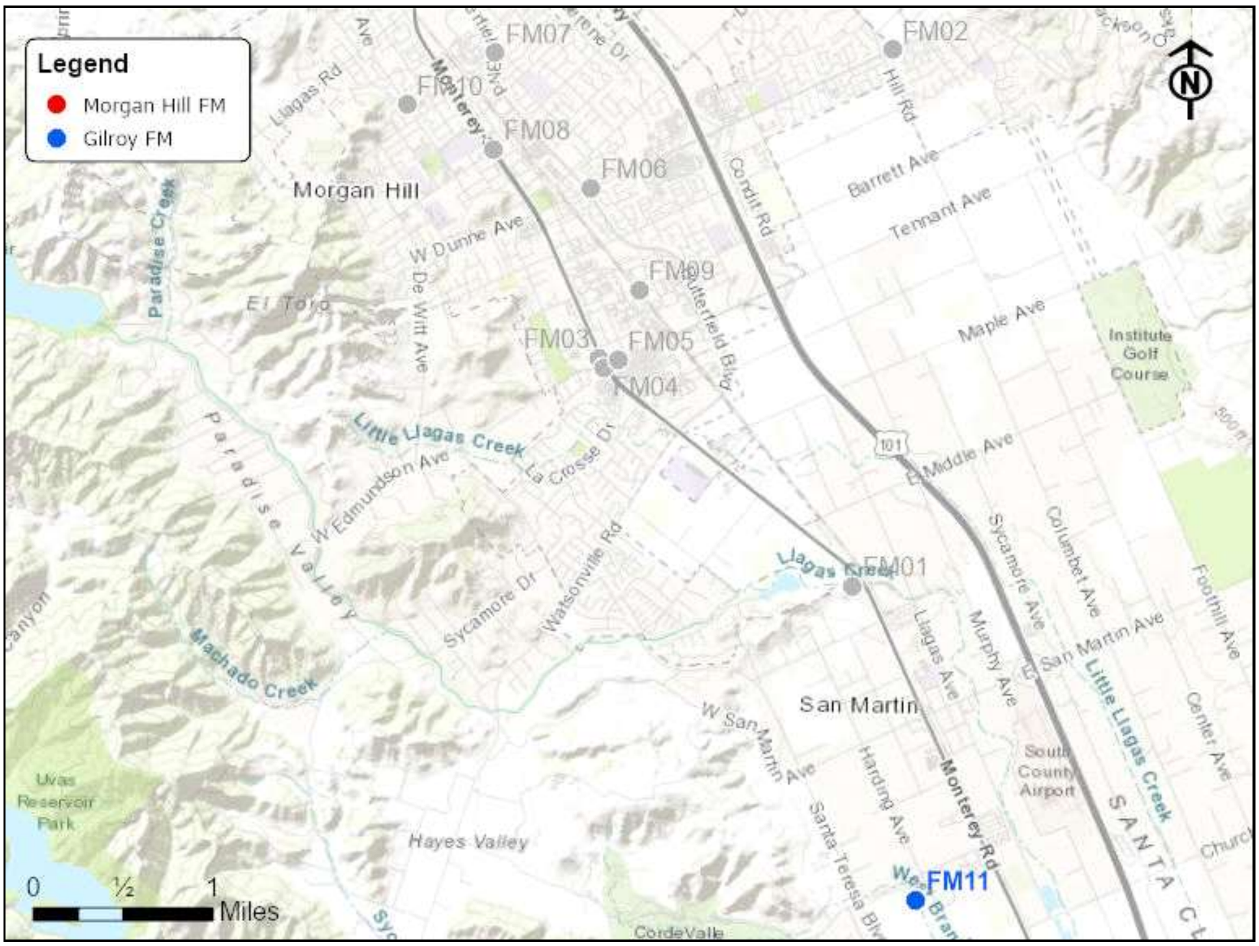
City of Morgan Hill, California

Sanitary Sewer Flow Monitoring

January 13, 2023 - February 22, 2023

Location: Field behind 12310 Santa Teresa Blvd

Data Summary Report



Vicinity Map: Site 11

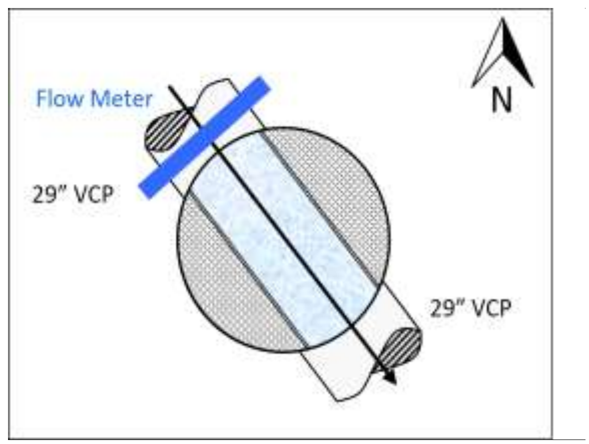
SITE 11

Site Information MH ID: MH.051

Location:	Field behind 12310 Santa Teresa Blvd
Coordinates:	-37.0702° W, 37.0702° N
Rim Elevation:	267 feet
Expected Pipe Diameter:	29 inches
Measured Pipe Diameter:	29 inches
ADWF:	2.775 mgd
Peak Measured Flow:	10.65 mgd
Sediment:	None

Satellite Map

Sanitary Map



Flow Sketch


Street View


Plan View

SITE 11

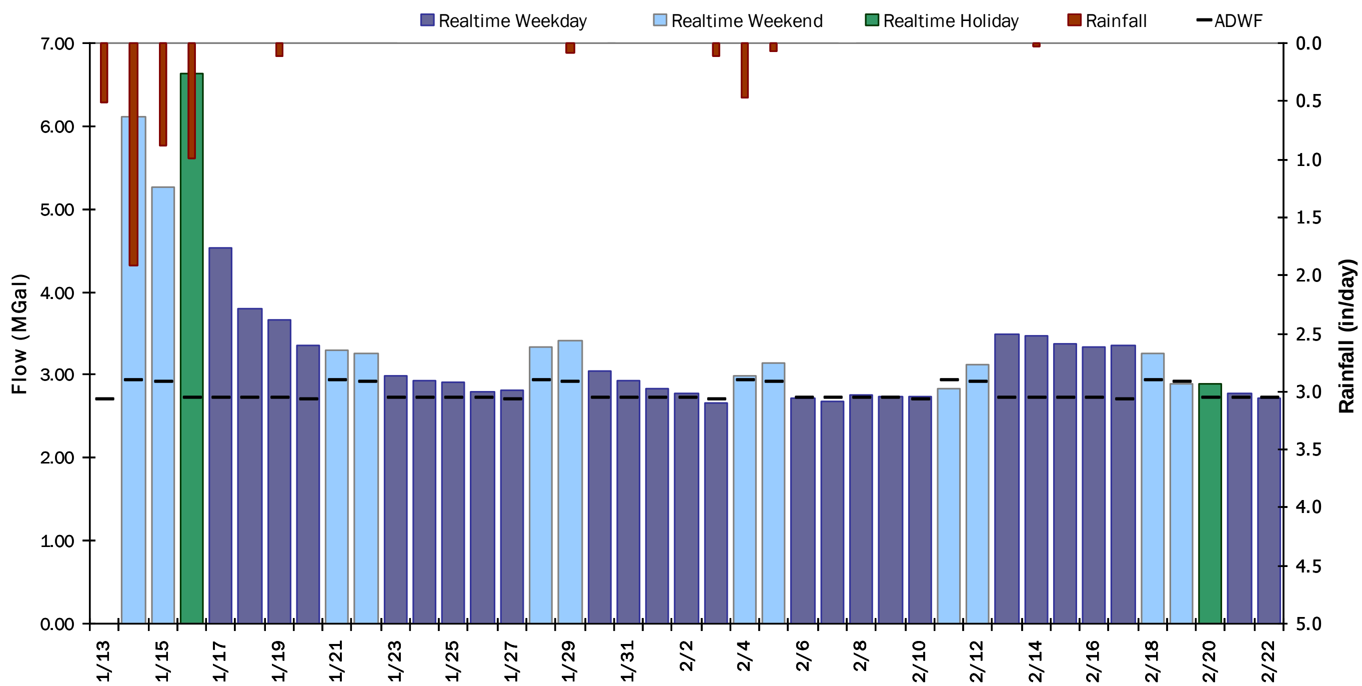
Additional Site Photos

Southeast Effluent Pipe

Monitored Northwest influent Pipe

SITE 11

Period Flow Summary: Daily Flow Totals

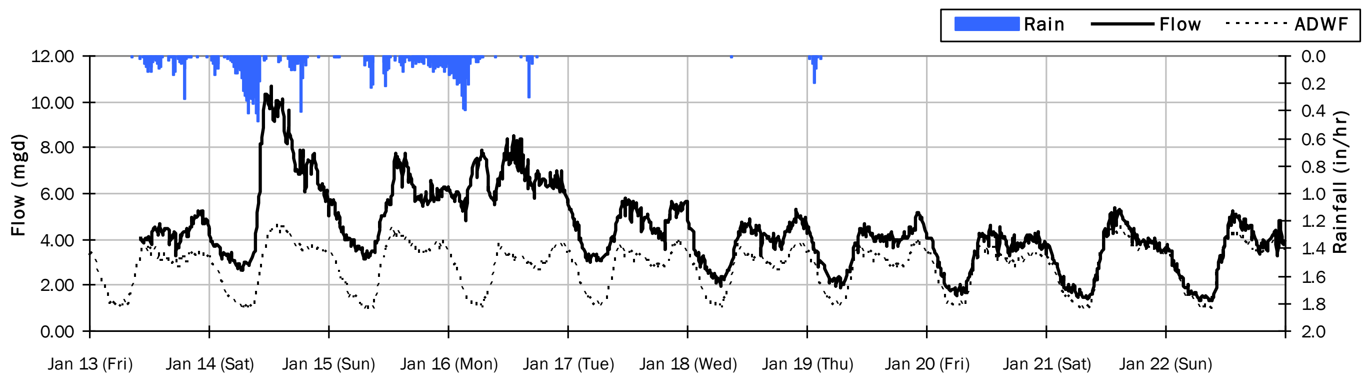


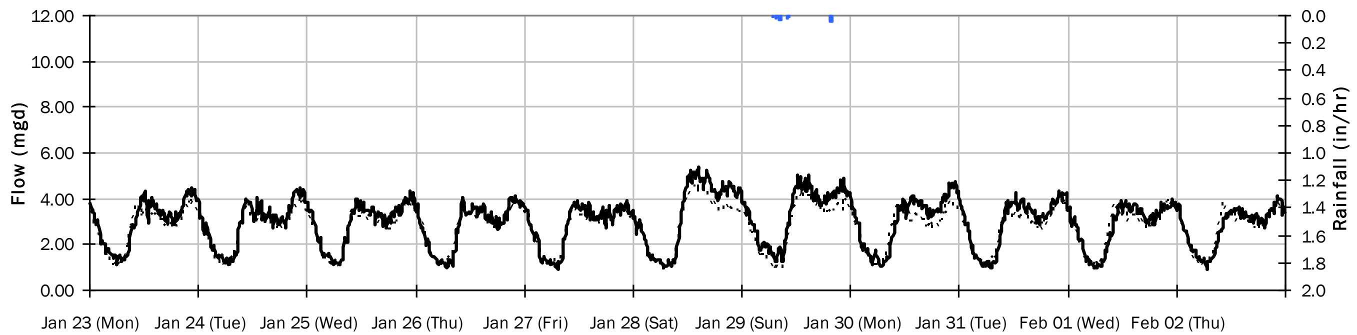
SITE 11

Flow Summary: 1/13/2023 to 2/2/2023

Period Rainfall: 4.51 inches

Period Avg Flow: 3.656 mgd Period Peak Flow: 10.651 mgd Period Min Flow: 0.909 mgd

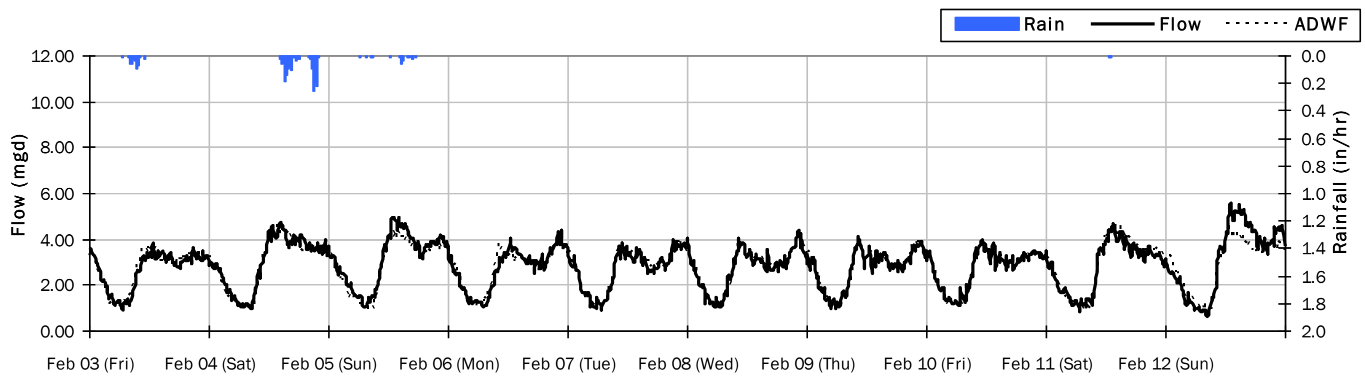


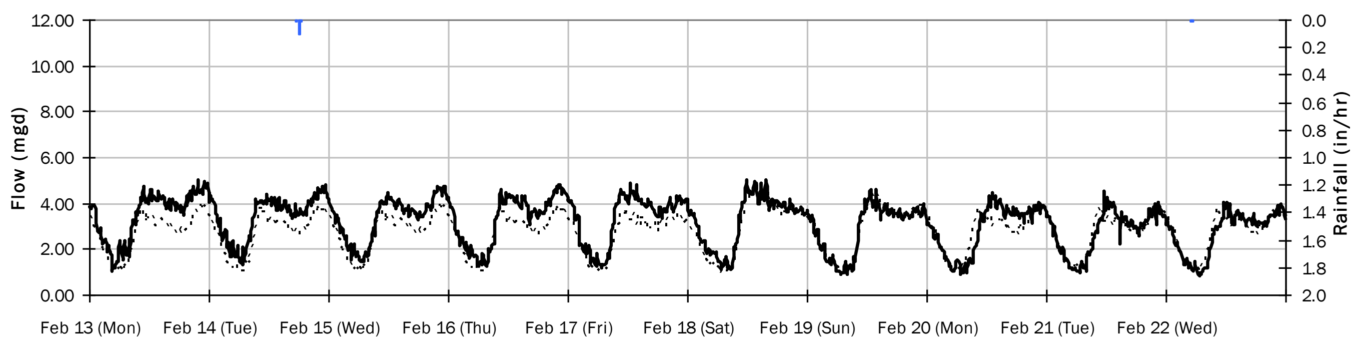


SITE 11

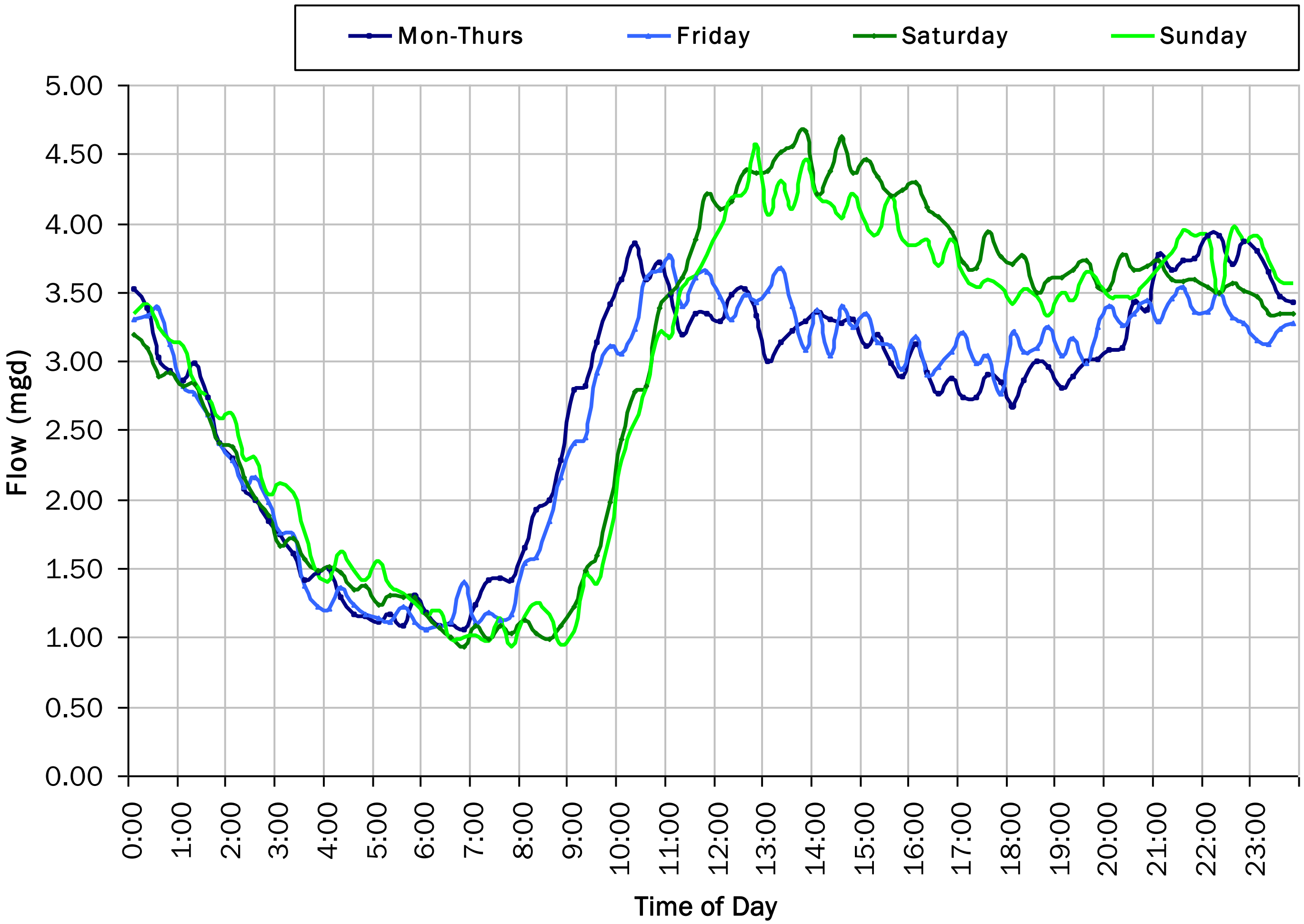
Flow Summary: 2/3/2023 to 2/22/2023

Period Rainfall: 0.69 inches   Period Avg Flow: 3.000 mgd   Period Peak Flow: 5.554 mgd   Period Min Flow: 0.612 mgd

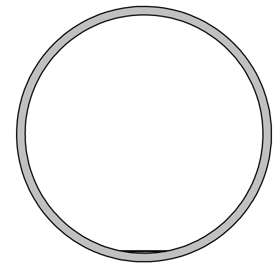




SITE 11
Average Dry Weather Flow Hydrographs

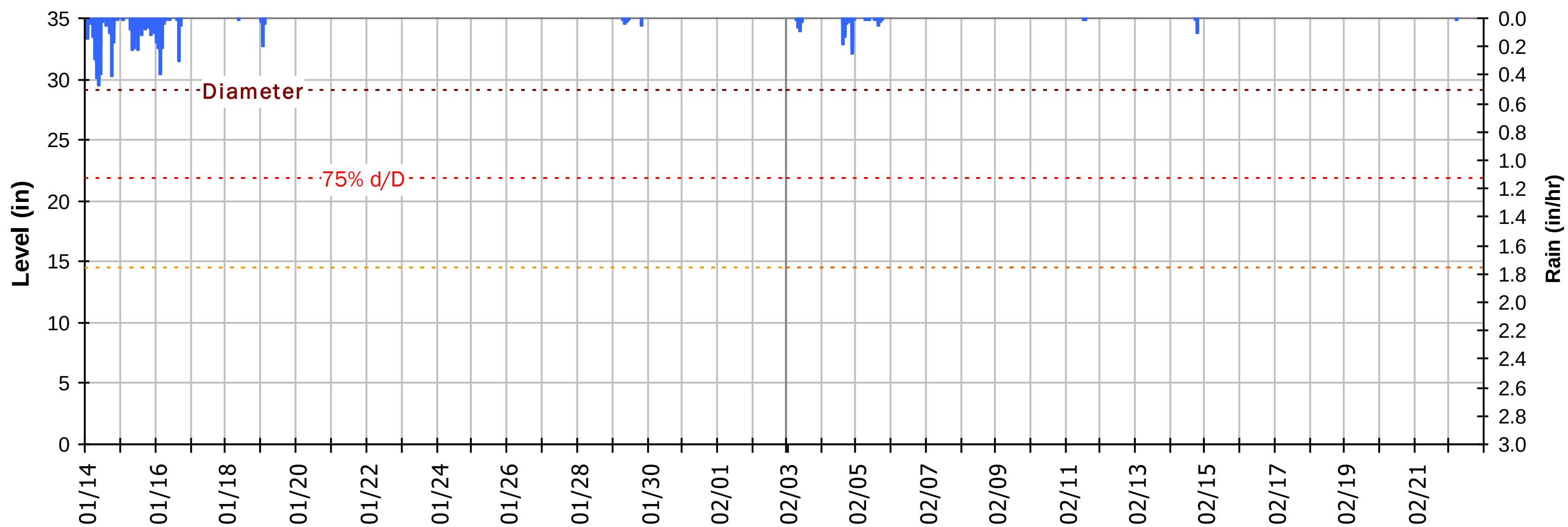


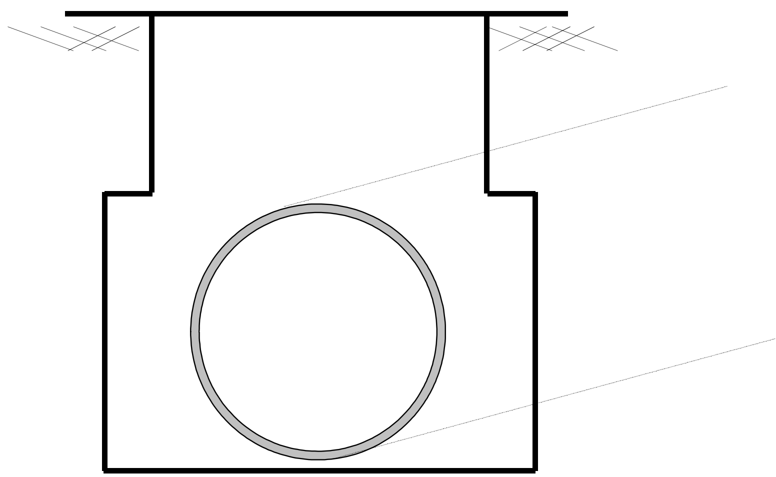
ADWF:
2.775 *mgd*



SITE 11
Site Capacity and Surcharge Summary

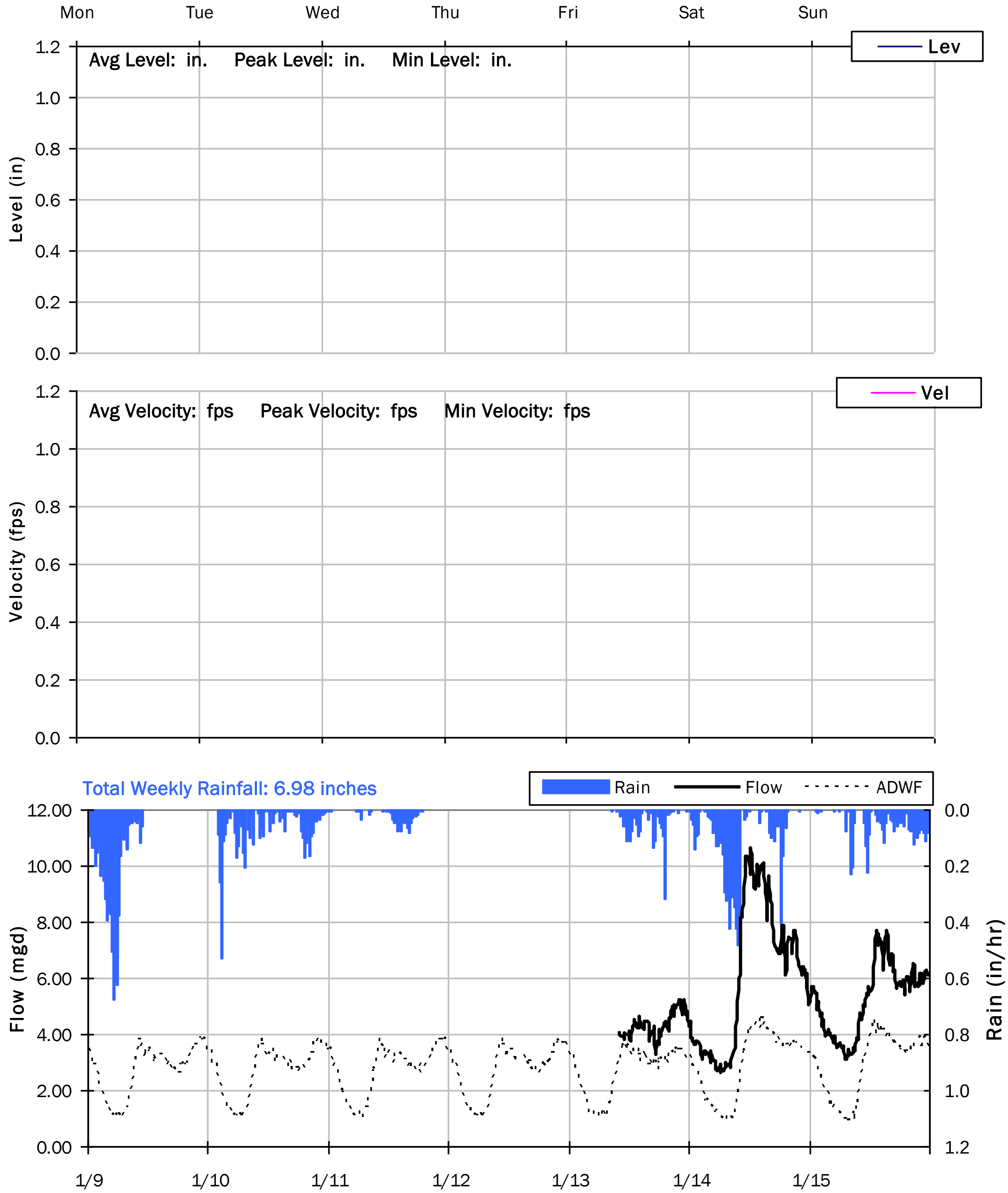
Realtime Flow Levels with Rainfall Data over Peak Level Period





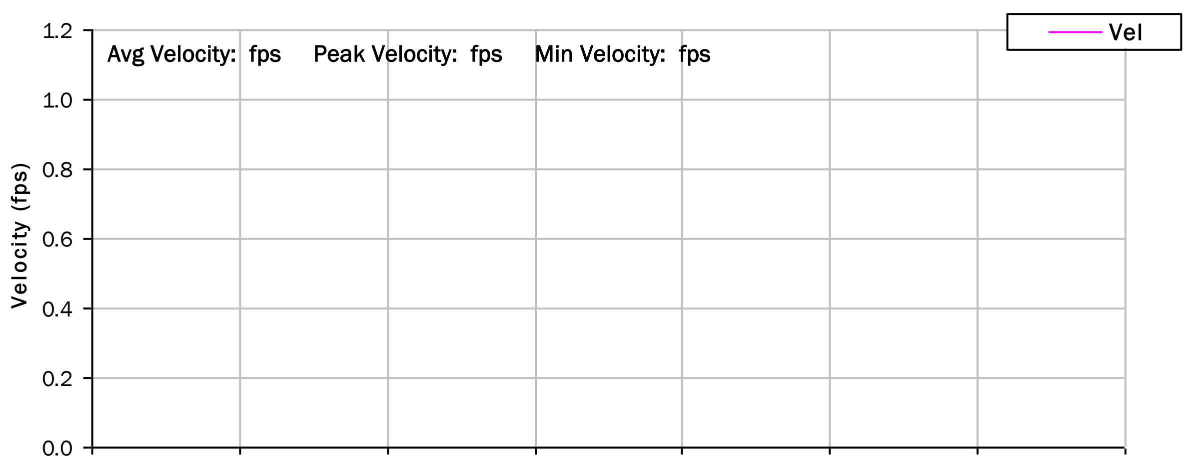
Pipe Diameter:	29	*inches*
Peak Measured Level:		*inches*
Peak d/D Ratio:		

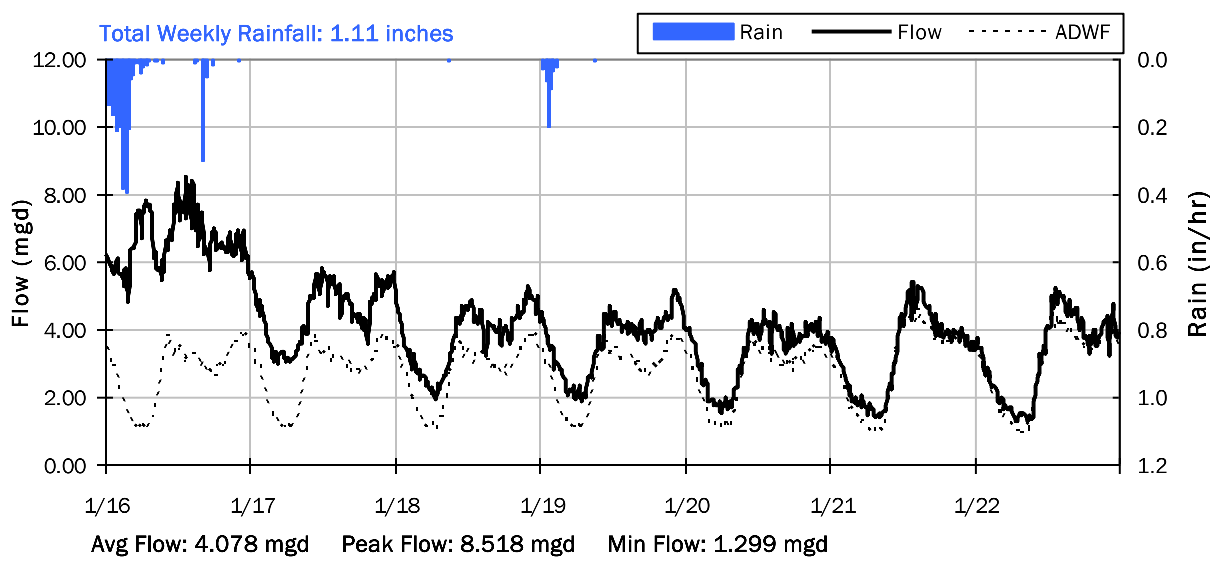
SITE 11
Weekly Level, Velocity and Flow Hydrographs
1/9/2023 to 1/16/2023



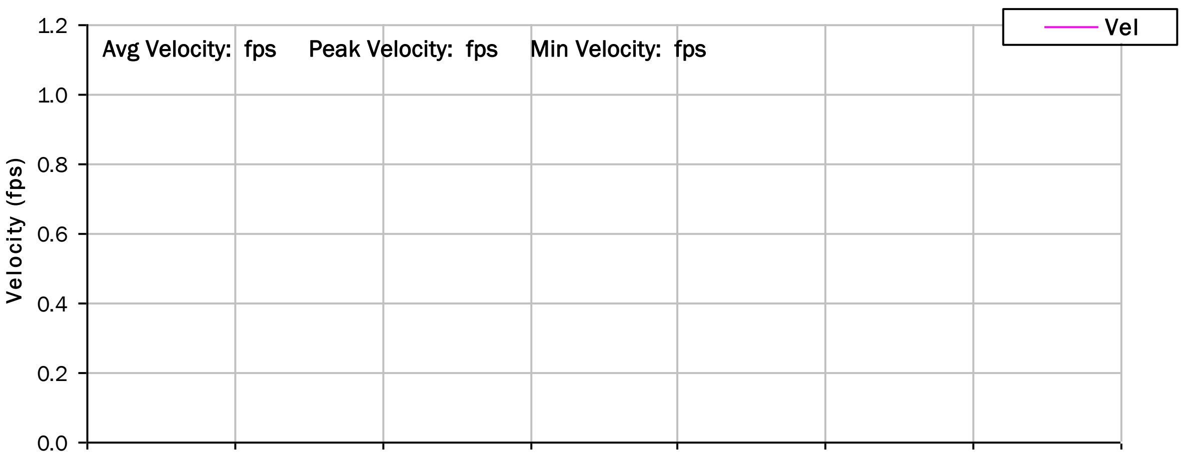
Avg Flow: 5.387 mgd Peak Flow: 10.651 mgd Min Flow: 2.674 mgd

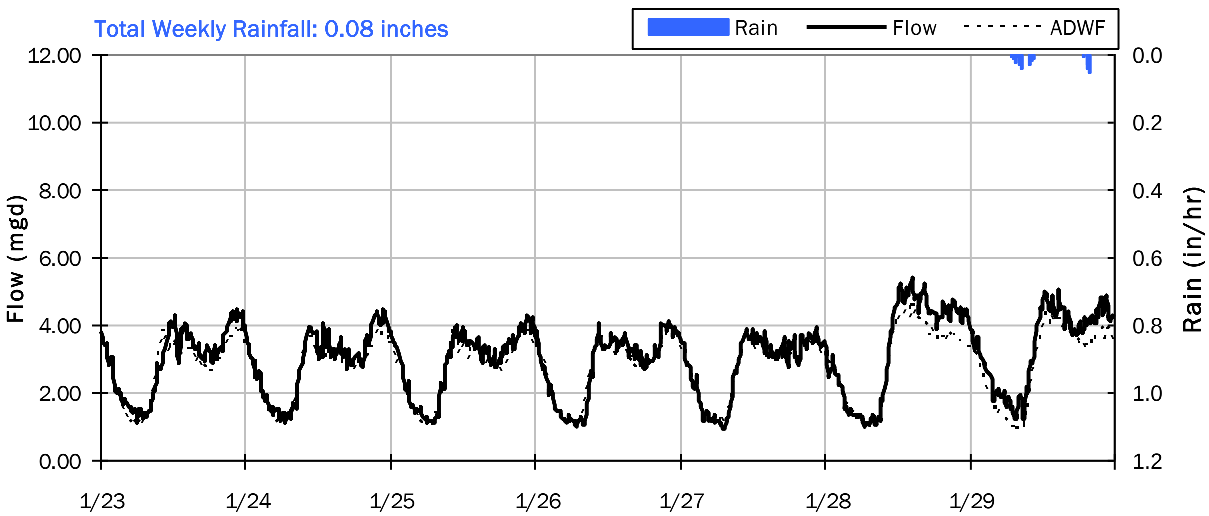
SITE 11
Weekly Level, Velocity and Flow Hydrographs
1/16/2023 to 1/23/2023





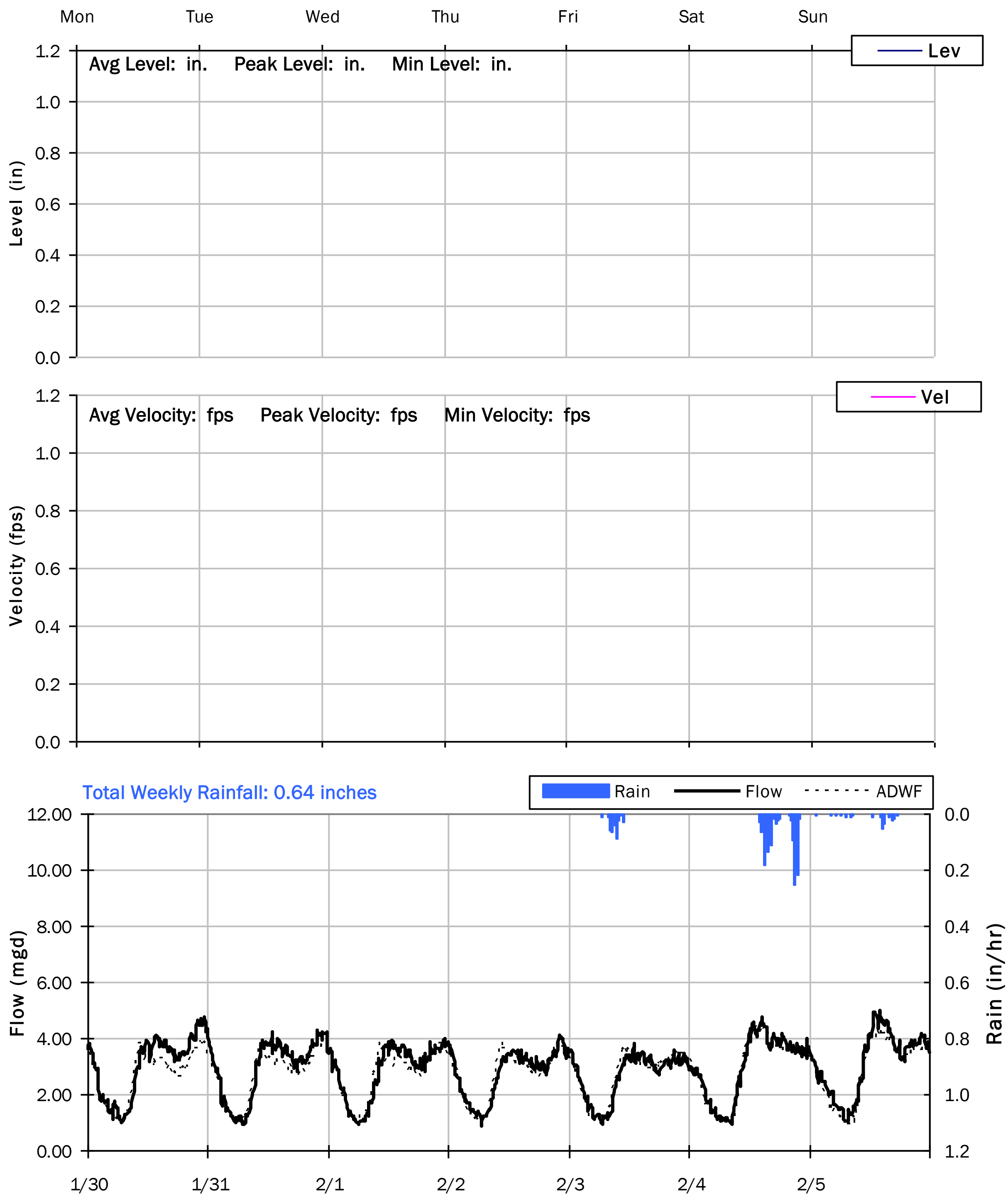
SITE 11
Weekly Level, Velocity and Flow Hydrographs
1/23/2023 to 1/30/2023





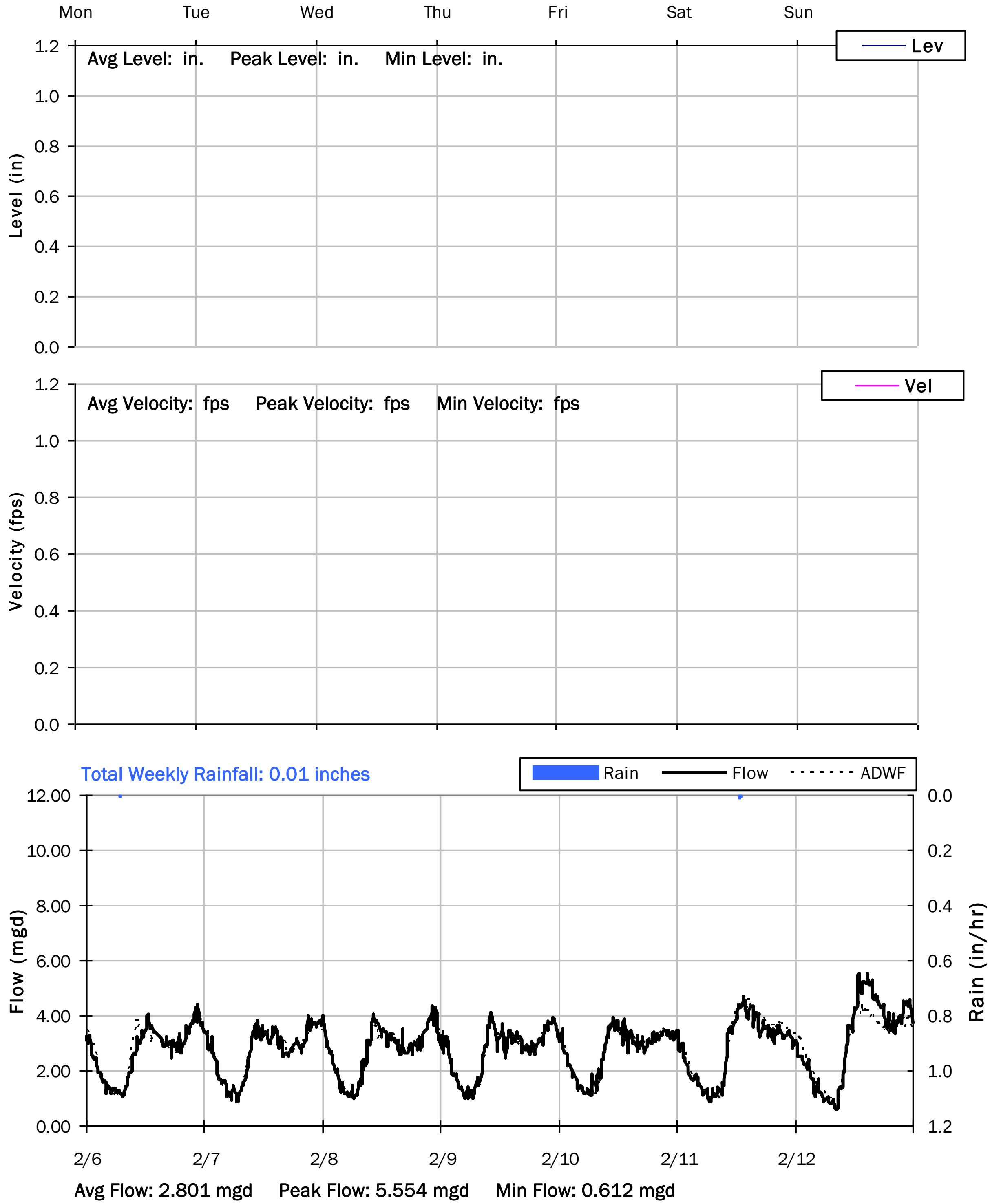
Avg Flow: 3.027 mgd Peak Flow: 5.406 mgd Min Flow: 0.925 mgd

SITE 11
Weekly Level, Velocity and Flow Hydrographs
1/30/2023 to 2/6/2023

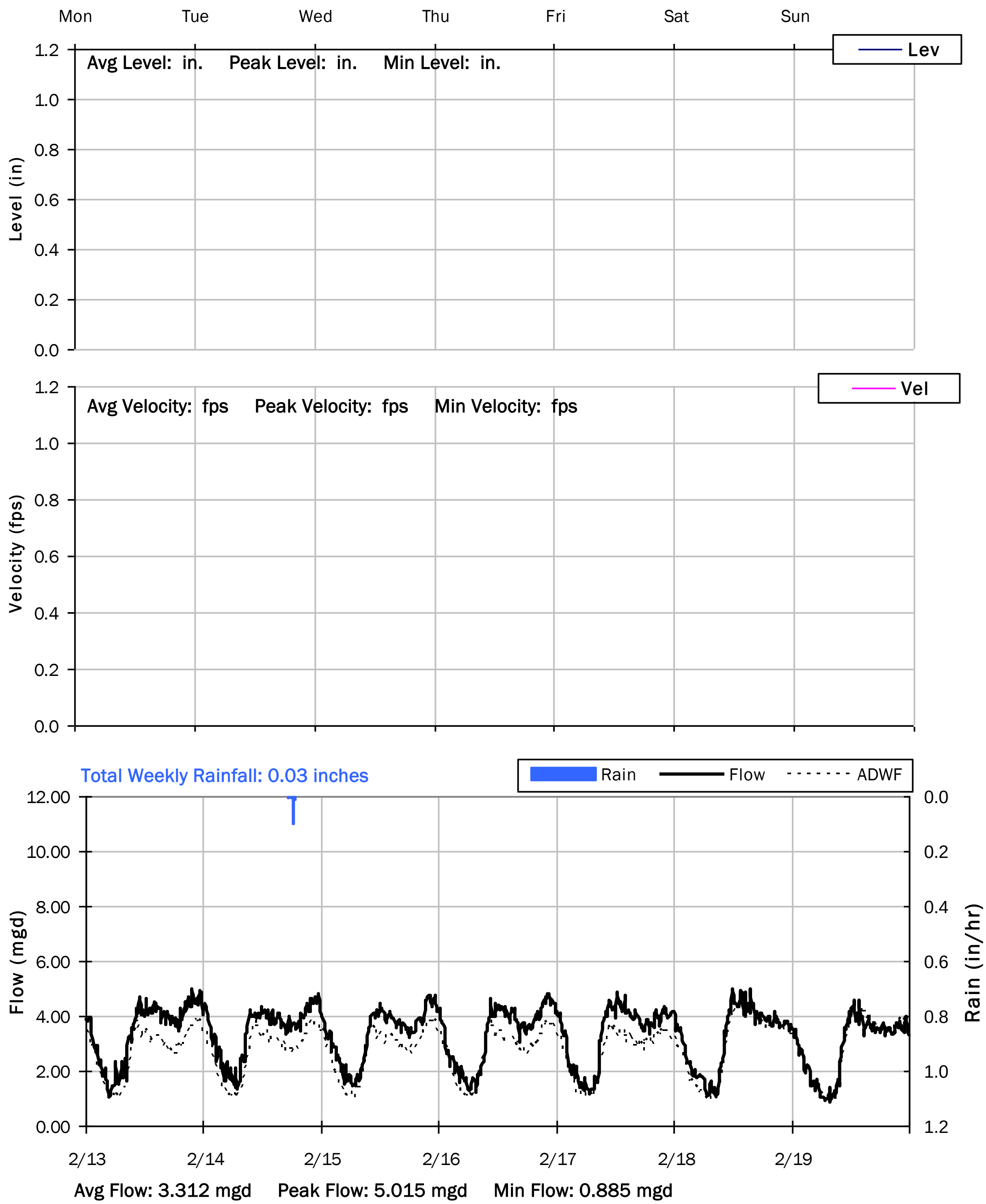


Avg Flow: 2.914 mgd Peak Flow: 4.980 mgd Min Flow: 0.909 mgd

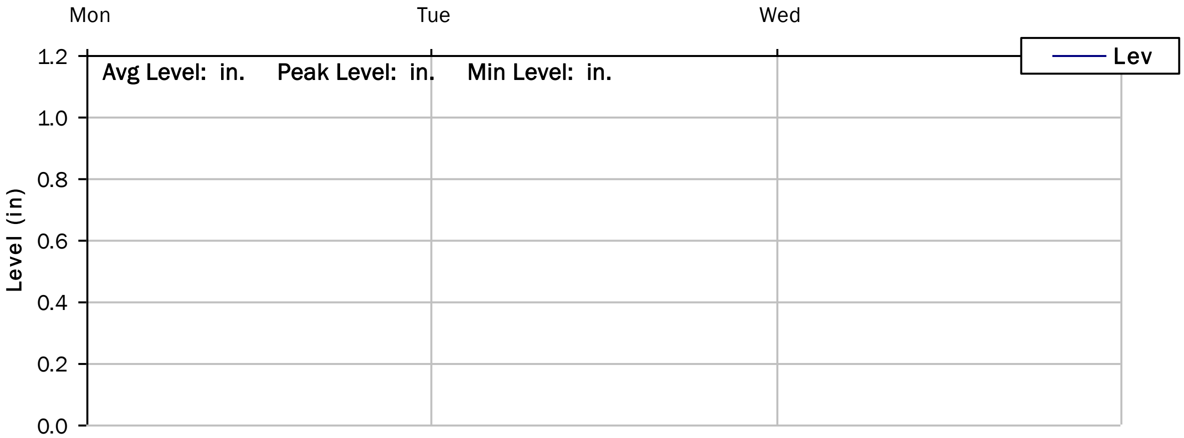
SITE 11
Weekly Level, Velocity and Flow Hydrographs
2/6/2023 to 2/13/2023

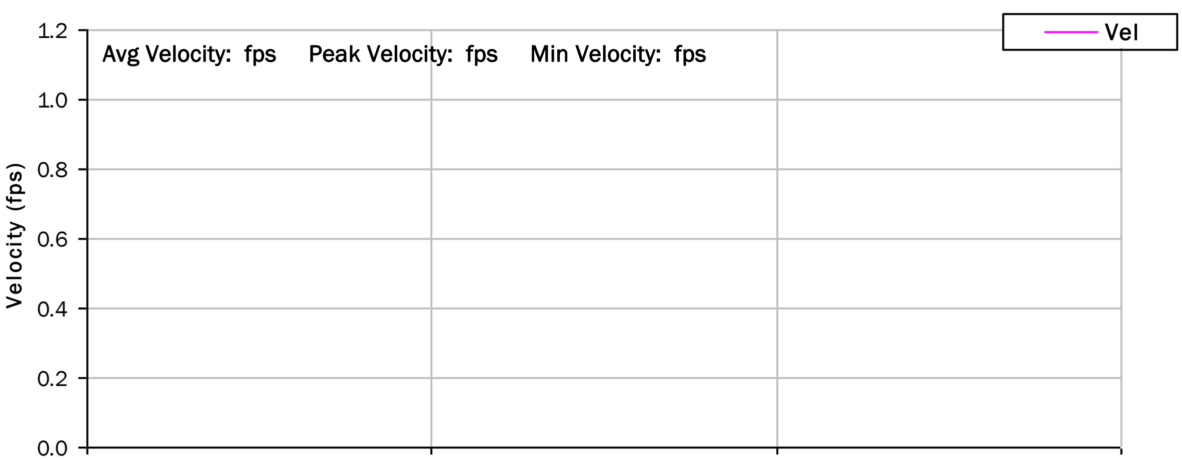


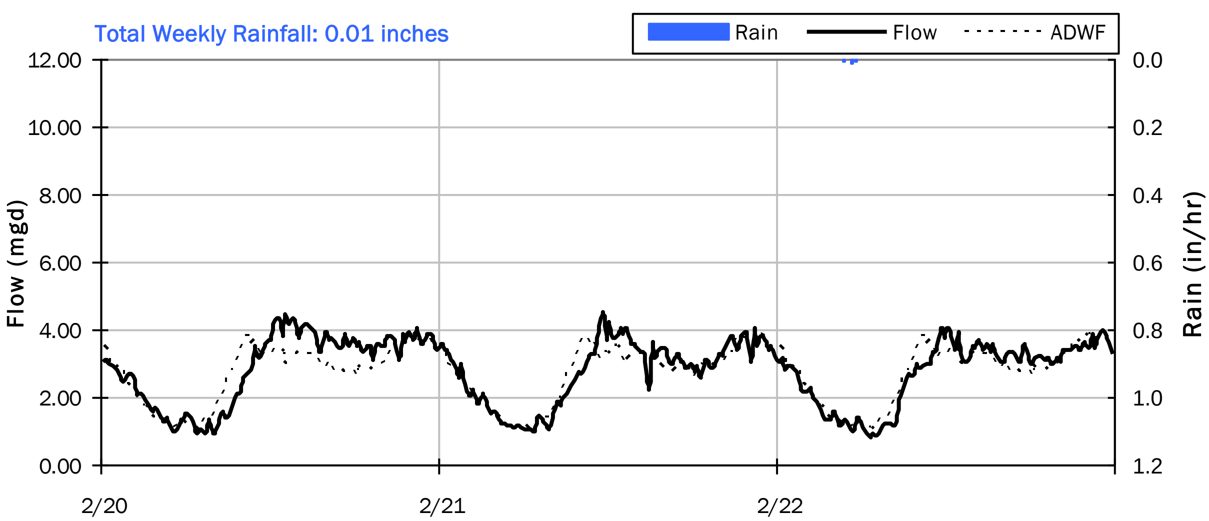
SITE 11
Weekly Level, Velocity and Flow Hydrographs
2/13/2023 to 2/20/2023



SITE 11
Weekly Level, Velocity and Flow Hydrographs
2/20/2023 to 2/23/2023

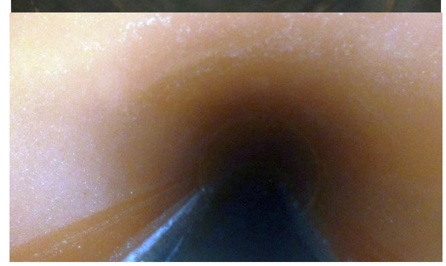






Avg Flow: 2.798 mgd   Peak Flow: 4.544 mgd   Min Flow: 0.818 mgd

V&A Project No. 22-0442



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

APPENDIX B

Design Storm Development

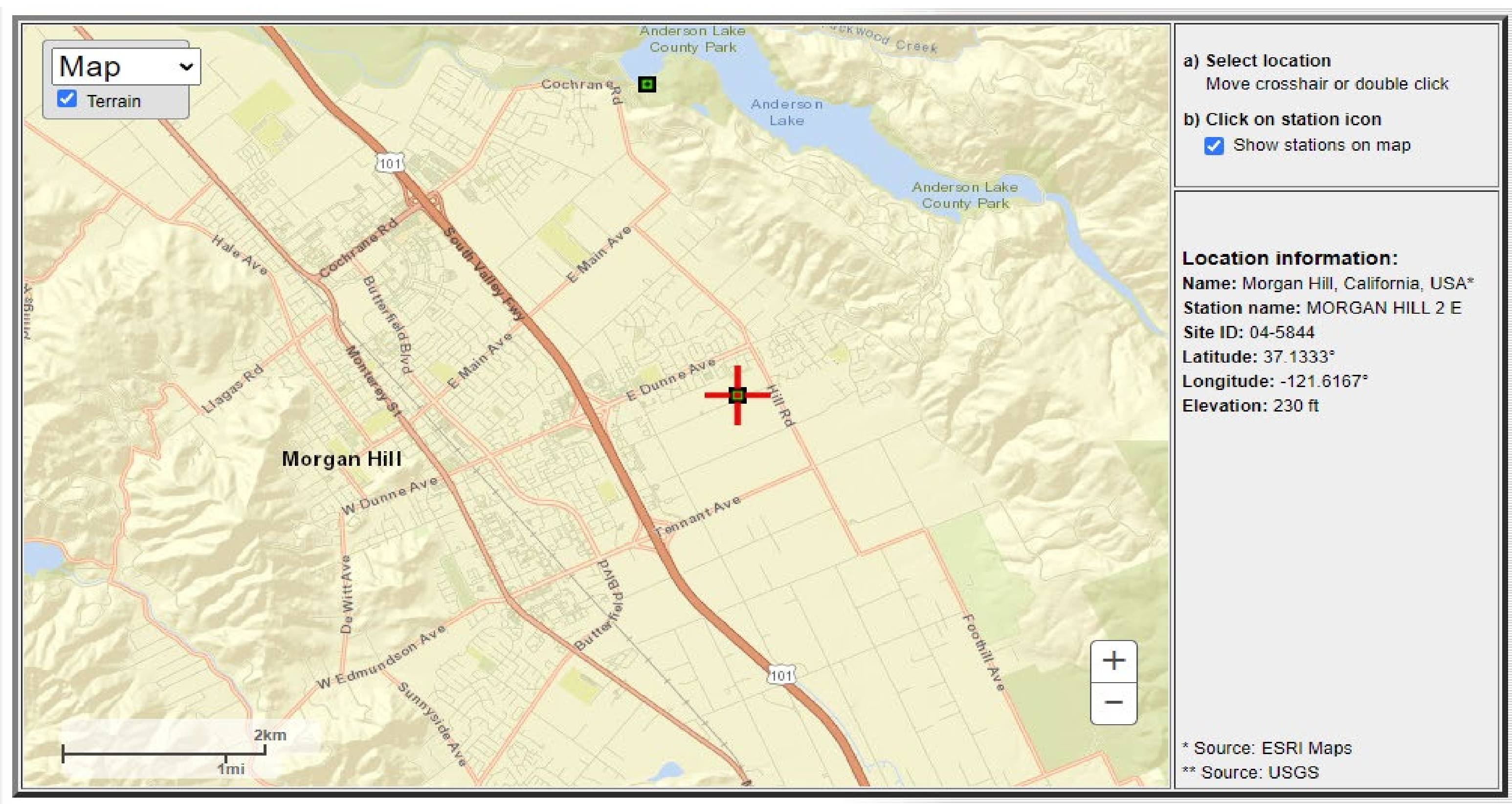
NOAA Atlas 14 Point Precipitation Frequency Data Server

1. Access NOAA Atlas 14, Volume 6, Version 2, Point Precipitation Frequency Data Server

NOAA Atlas 14 Volume 6:	https://hdsc.nws.noaa.gov/hdsc/pfds/

2. Select Weather Station

Location:	**City of Morgan Hill**
Station Name:	**Morgan Hill 2 E**
Site ID:	**04-5844**
Latitude:	**37.1333°**
Longitude:	**-121.6167°**
Elevation:	**230 feet**



3. Download Rainfall Intensity for Varying Return Period Intervals

Data Type:	**Precipitation Intensity**
Units:	**English**
Time Series Type:	**Partial Duration**

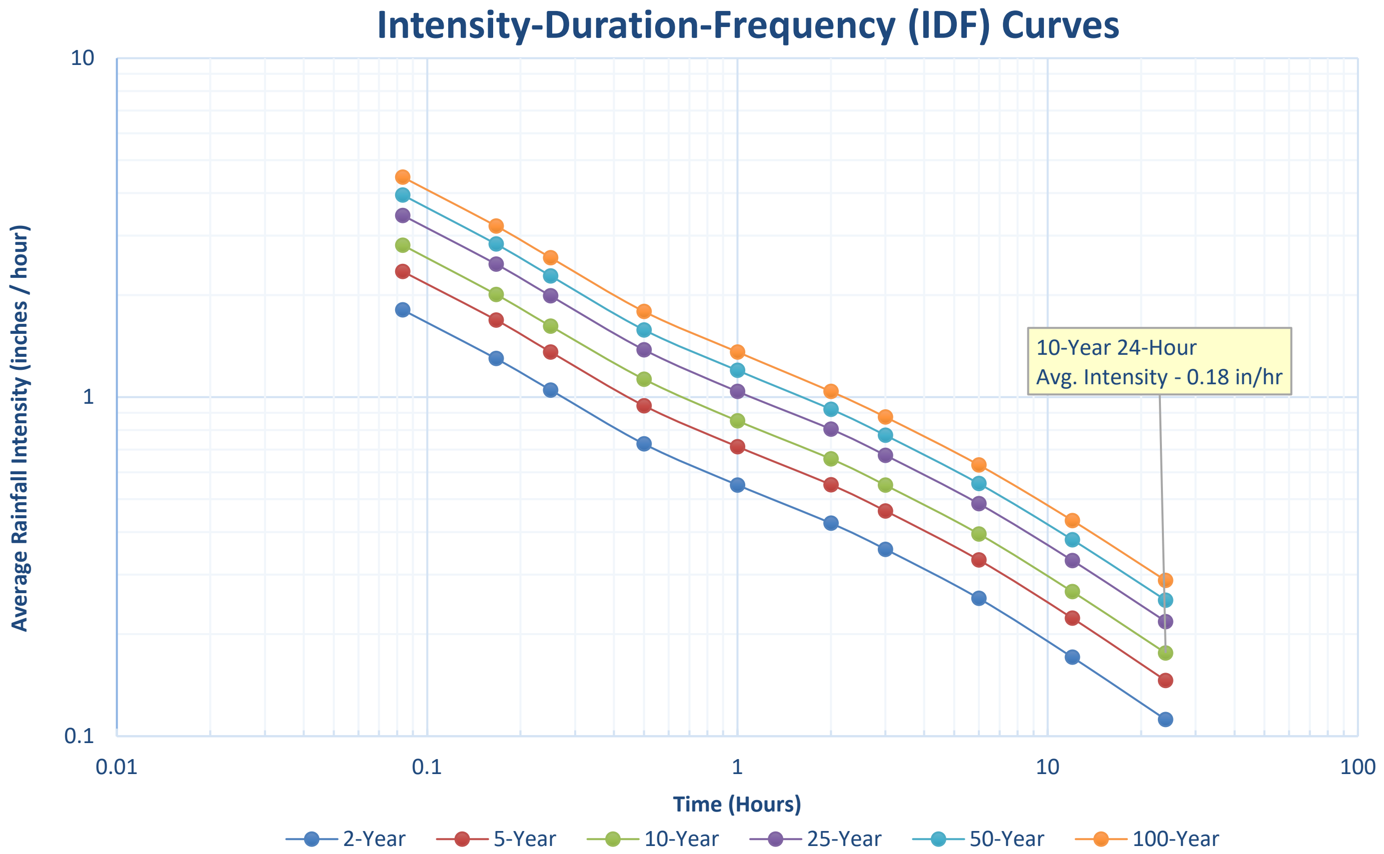
4. Plot IDF and DDF Curves

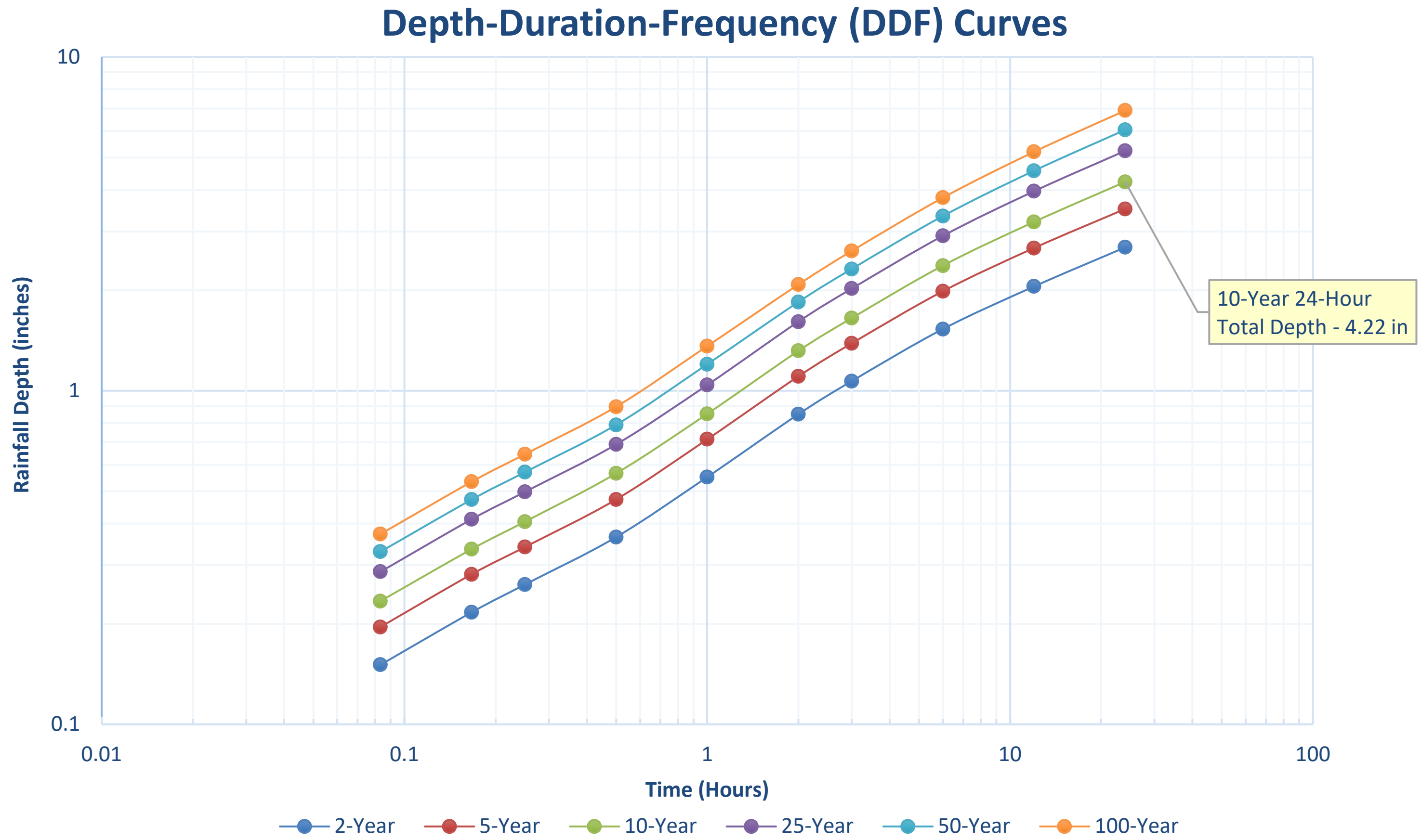
Average Rainfall Intensity in Inches Per Hour - Intensity Duration Frequency (IDF) Curves

Storm Duration / Return Period	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
5-min:	1.81	2.35	2.81	3.44	3.95	4.46
10-min:	1.3	1.69	2.01	2.47	2.83	3.2
15-min:	1.05	1.36	1.62	1.99	2.28	2.58
30-min:	0.728	0.944	1.13	1.38	1.58	1.79
60-min:	0.551	0.715	0.852	1.04	1.2	1.36
2-hr:	0.425	0.552	0.658	0.805	0.922	1.04
3-hr:	0.356	0.462	0.55	0.674	0.772	0.874
6-hr:	0.255	0.331	0.395	0.485	0.556	0.631
12-hr:	0.171	0.223	0.267	0.33	0.38	0.433
24-hr:	0.112	0.146	0.176	0.218	0.252	0.288

Total Rainfall Depth in Inches - Depth Duration Frequency (DDF) Curves

Storm Duration / Return Period	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
5-min:	0.151	0.196	0.234	0.287	0.329	0.372
10-min:	0.217	0.282	0.335	0.412	0.472	0.533
15-min:	0.263	0.340	0.405	0.498	0.570	0.645
30-min:	0.364	0.472	0.565	0.690	0.790	0.895
60-min:	0.551	0.715	0.852	1.040	1.200	1.360
2-hr:	0.850	1.104	1.316	1.610	1.844	2.080
3-hr:	1.068	1.386	1.650	2.022	2.316	2.622
6-hr:	1.530	1.986	2.370	2.910	3.336	3.786
12-hr:	2.052	2.676	3.204	3.960	4.560	5.196
24-hr:	2.688	3.504	4.224	5.232	6.048	6.912





6. Select Appropriate Rainfall Distribution

Rainfall Distribution: **Distributon from the City's 2017 Infrastructure Master Plans**

Time	**Fraction of Total Rainfall**
(hours)	% Incremental
0:00	2.05%
1:00	2.23%
2:00	2.30%
3:00	2.42%
4:00	2.61%
5:00	2.82%
6:00	3.07%
7:00	3.38%
8:00	3.91%
9:00	4.62%
10:00	6.21%
11:00	18.59%
12:00	11.05%
13:00	5.83%
14:00	4.47%
15:00	3.82%
16:00	3.32%
17:00	3.04%
18:00	2.76%
19:00	2.58%
20:00	2.42%
21:00	2.27%
22:00	2.20%
23:00	2.05%
Sum	**100.00%**

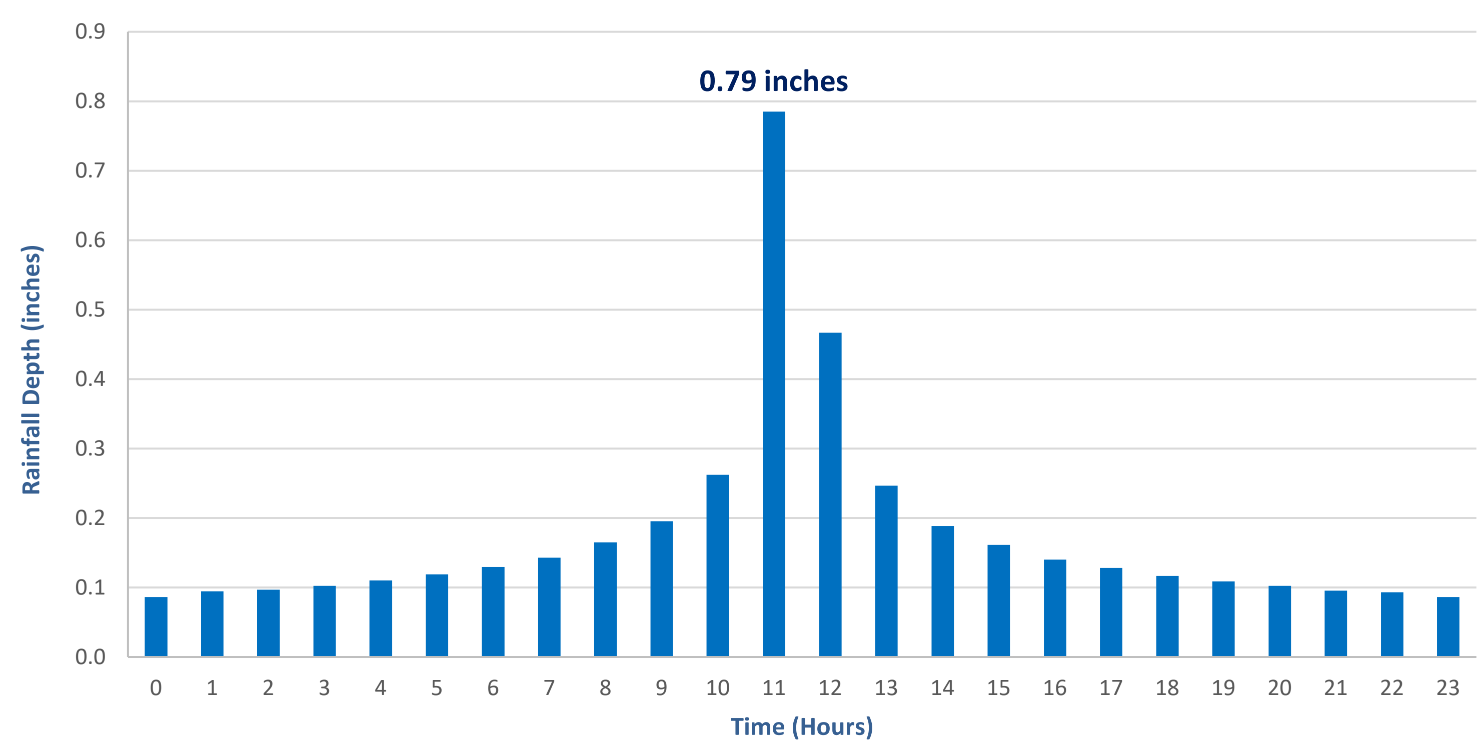
7. Develop Design Storms for Capacity Evaluation

Data Source for Rainfall Volume/Depth: **NOAA Atlas 14, Volume 6, Version 2, Morgan Hill Weather Station**

Data Source for Rainfall Distribution: **2017 Infrastructure Master Plans**

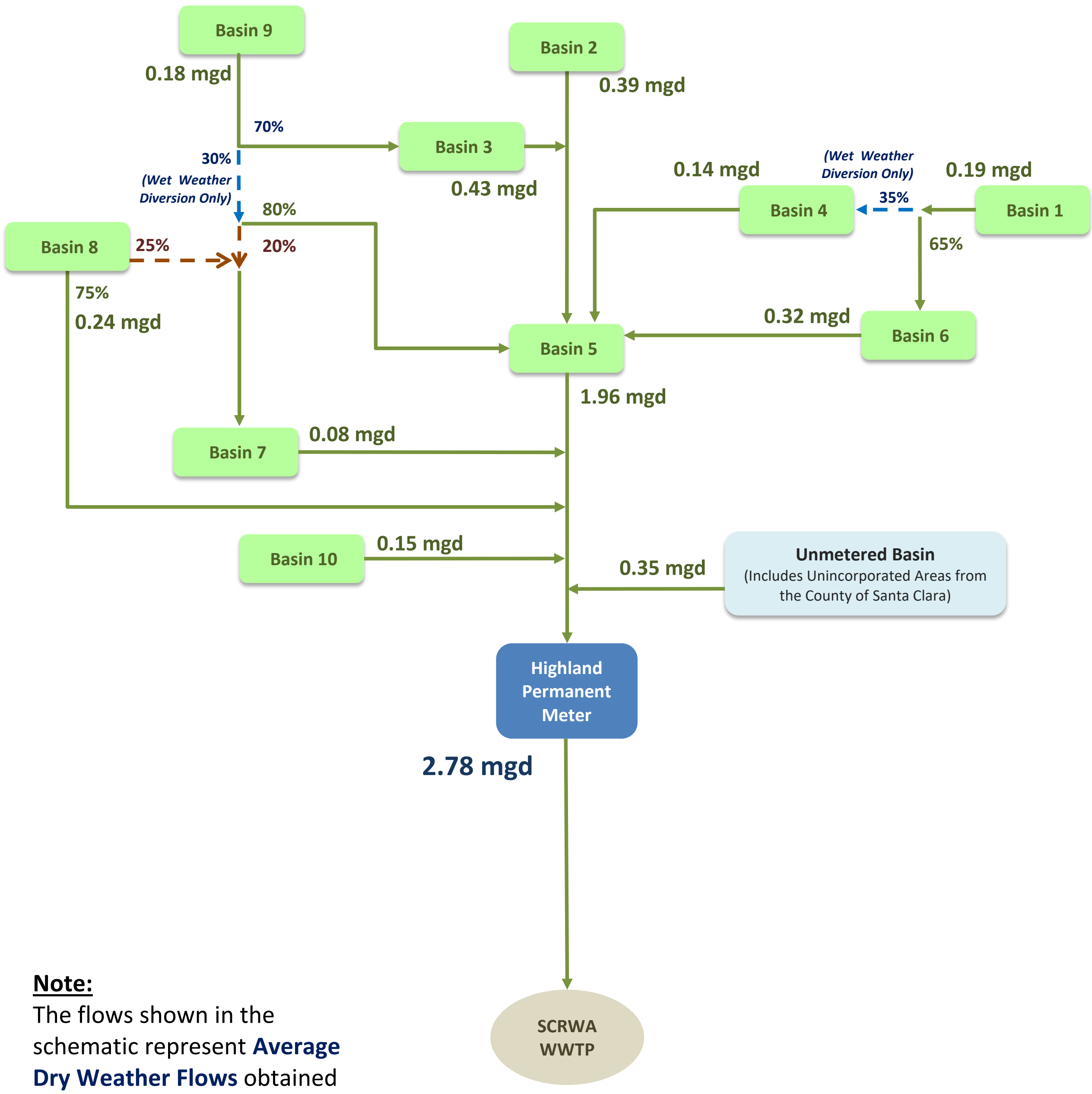
Time (hours)	Rainfall in	Intensity in/hr
0:00	0.0865	0.09
1:00	0.0944	0.09
2:00	0.0970	0.10
3:00	0.1022	0.10
4:00	0.1101	0.11
5:00	0.1192	0.12
6:00	0.1297	0.13
7:00	0.1428	0.14
8:00	0.1652	0.17
9:00	0.1953	0.20
10:00	0.2622	0.26
11:00	0.7851	0.79
12:00	0.4666	0.47
13:00	0.2464	0.25
14:00	0.1887	0.19
15:00	0.1612	0.16
16:00	0.1402	0.14
17:00	0.1284	0.13
18:00	0.1166	0.12
19:00	0.1088	0.11
20:00	0.1022	0.10
21:00	0.0957	0.10
22:00	0.0931	0.09
23:00	0.0865	0.09
Total	**4.22**	
	Peak Intensity	**0.79**

8. Design Storm for Capacity Evaluation



APPENDIX C

Hydraulic Model Calibration



Note:

The flows shown in the schematic represent **Average Dry Weather Flows** obtained from the 2023 Flow Monitoring Program.

LEGEND

- Temporary Flow Meter Basin
- Permanent Flow Meter Basin
- SCRWA WWTP
- Unmetered Basin
- Pipelines
- Active Diversion
- Overflow Diversion (Only Active During Wet Weather Conditions)

4/19/2024

Exhibit C
Flow Meter Schematic
Wastewater Collection System Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

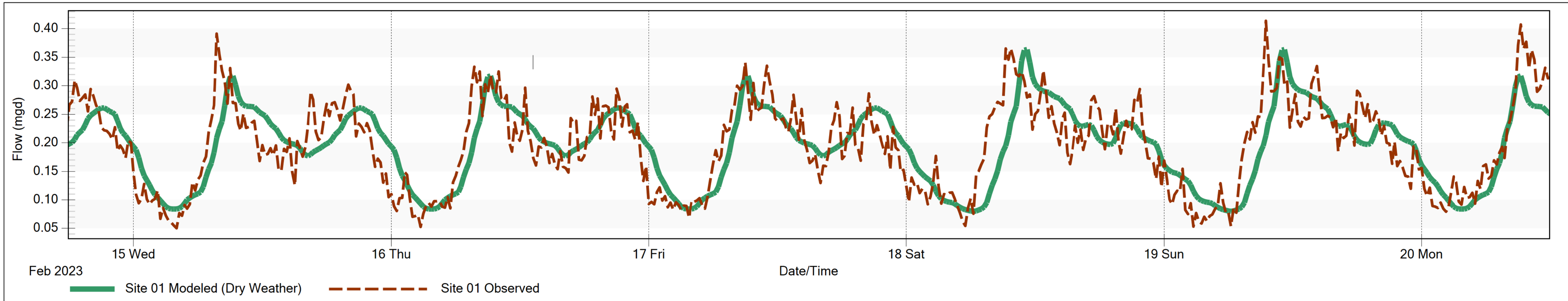
AKEL ENGINEERING GROUP, INC.

Table C1 Flow Calibration Results

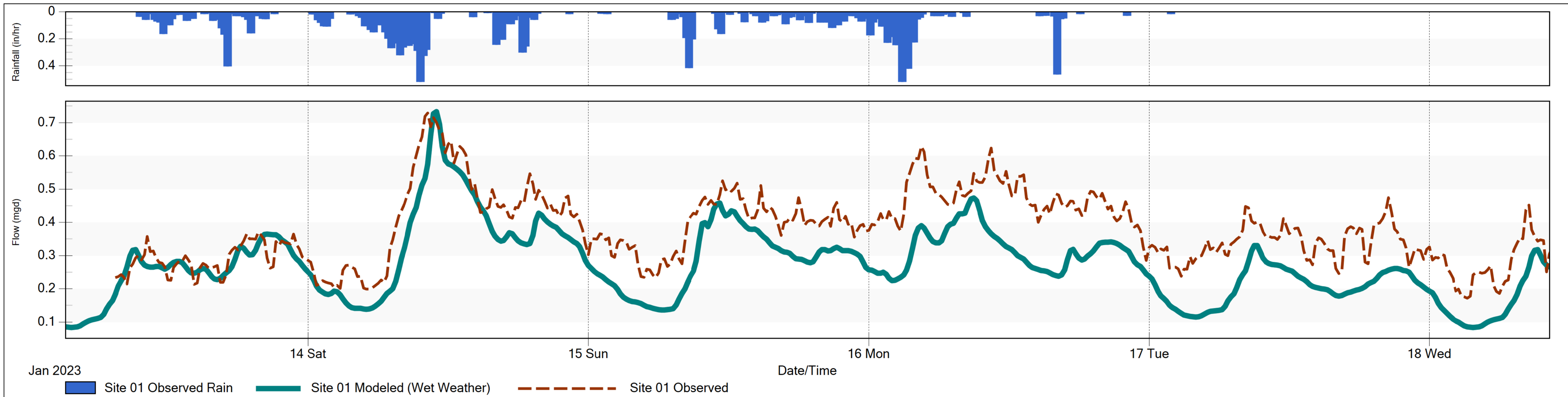
Wastewater Collection System Master Plan Update
City of Morgan Hill

Site ID	Figure No.	Peak Dry Weather Flow						Peak Wet Weather Flow		
		Weekday (Monday - Friday)			Weekend (Saturday - Sunday)			January 14 to 16, 2023 Rainfall Event		
	Appendix C	Modeled (mgd)	Observed (mgd)	% Diff. (%)	Modeled (mgd)	Observed (mgd)	% Diff. (%)	Modeled (mgd)	Observed (mgd)	% Diff. (%)
Temporary Flow Monitors										
Site 1	C1	0.318	0.325	**2%**	0.364	0.377	**4%**	0.733	0.730	**0%**
Site 2	C2	0.569	0.610	**7%**	0.641	0.662	**3%**	1.162	1.200	**3%**
Site 3	C3	0.612	0.596	**3%**	0.653	0.656	**0%**	2.246	2.210	**2%**
Site 4	C4	0.264	0.217	**19%**	0.306	0.266	**14%**	0.871	0.850	**2%**
Site 5	C5	2.792	2.834	**1%**	3.301	3.182	**4%**	7.353	7.320	**0%**
Site 6	C6	0.582	0.526	**10%**	0.661	0.591	**11%**	1.319	1.310	**1%**
Site 7	C7	0.145	0.133	**8%**	0.176	0.177	**1%**	0.951	0.970	**2%**
Site 8	C8	0.361	0.346	**4%**	0.391	0.378	**3%**	1.419	1.360	**4%**
Site 9	C9	0.350	0.365	**4%**	0.348	0.355	**2%**	0.952	0.960	**1%**
Site 10	C10	0.217	0.210	**3%**	0.250	0.258	**3%**	0.532	0.510	**4%**
Permanent Highland Avenue Flow Monitor (Joint Trunk)										
Site 11	C11	4.071	3.910	**4%**	4.860	4.676	**4%**	10.330	10.650	**3%**

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 0.730 and 0.733 mgd, respectively.

April 3, 2024

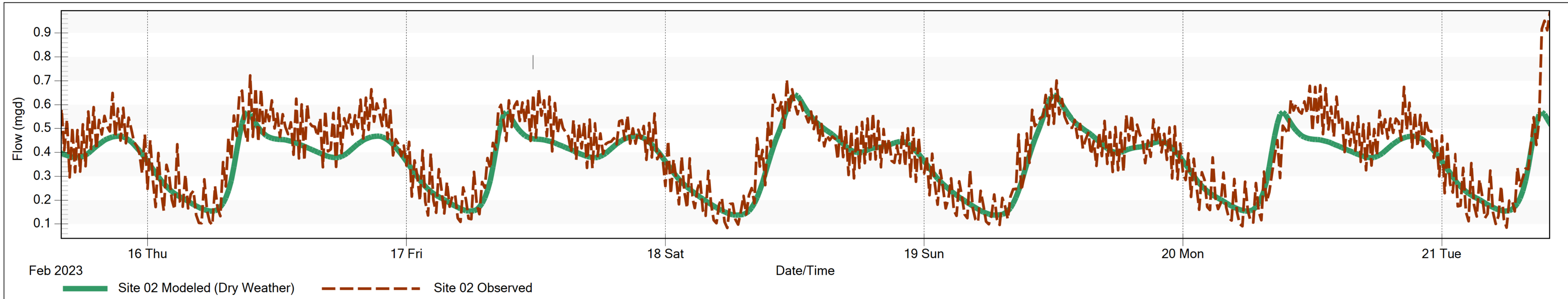
Figure C1

Site 1 Flow Calibration Results

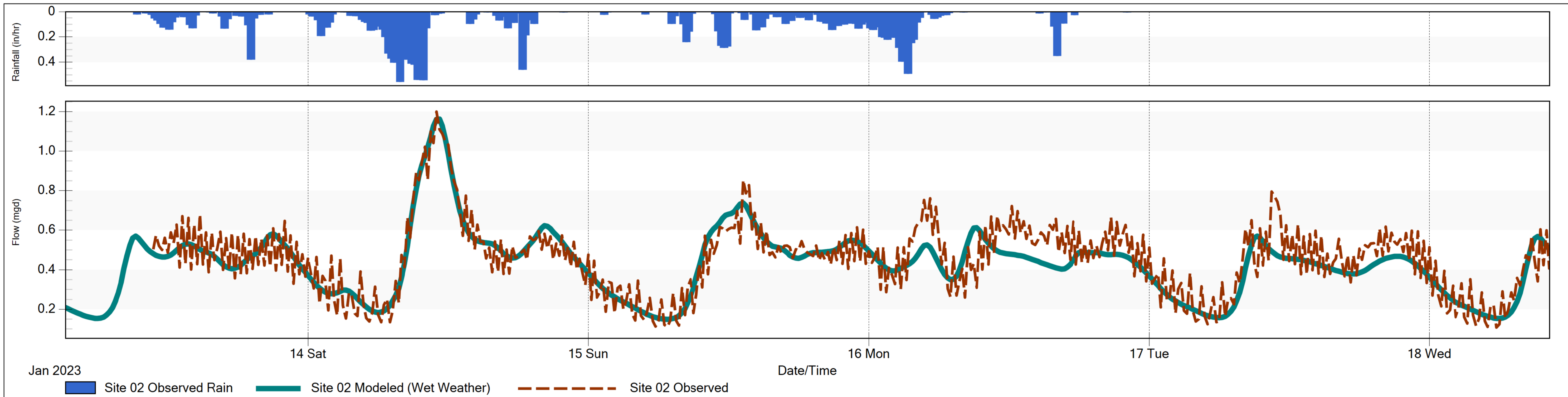
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 1.200 and 1.162 mgd, respectively.

April 3, 2024

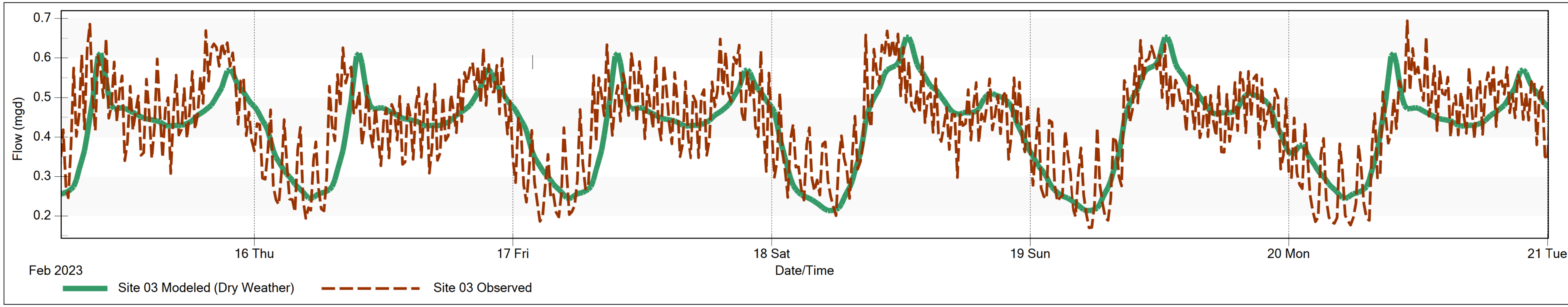
Figure C2

Site 2 Flow Calibration Results

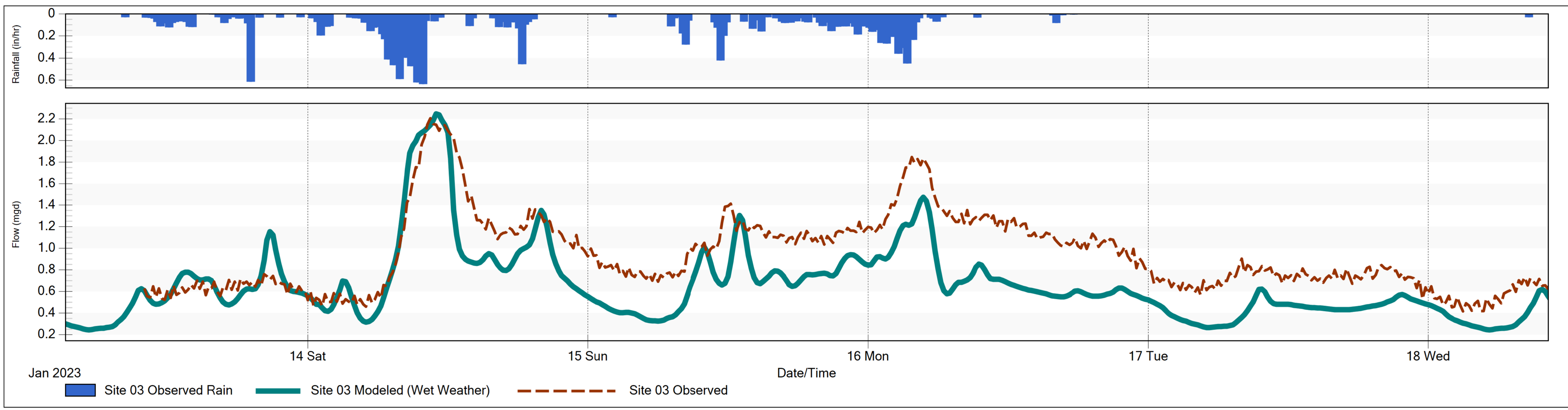
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 2.210 and 2.246 mgd, respectively.

April 3, 2024

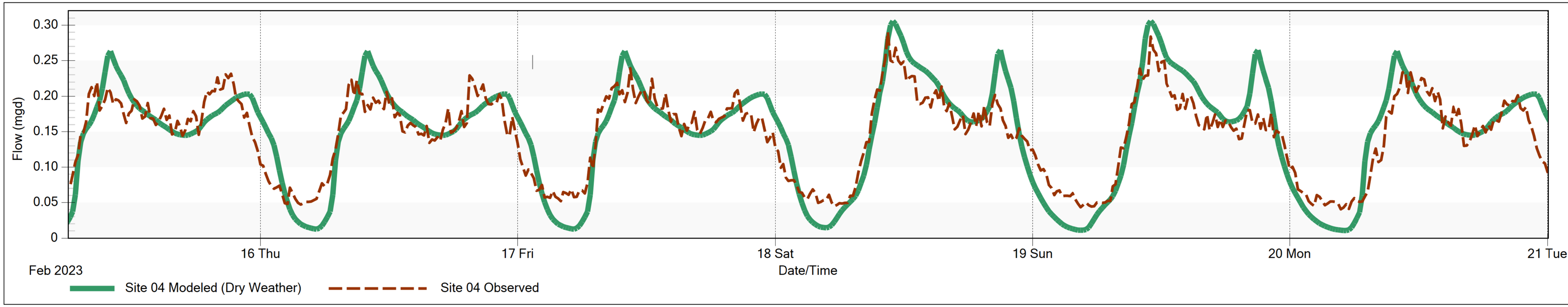
Figure C3

Site 3 Flow Calibration Results

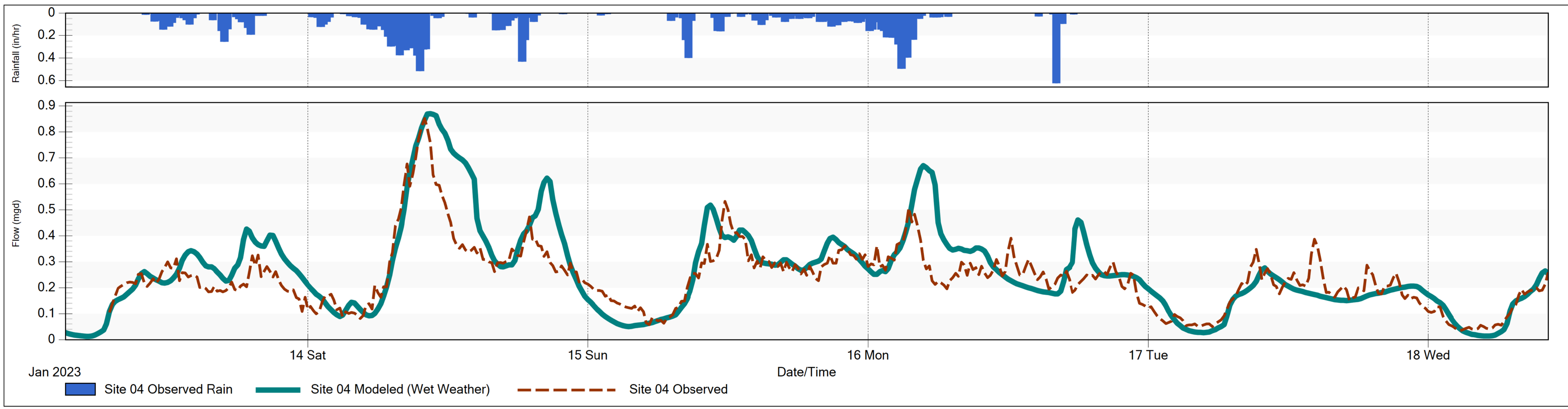
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 0.850 and 0.871 mgd, respectively.

April 3, 2024

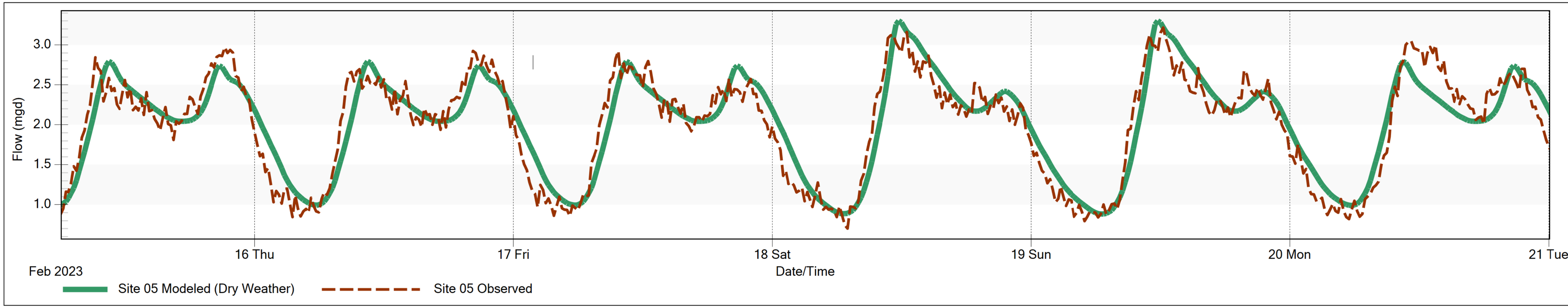
Figure C4

Site 4 Flow Calibration Results

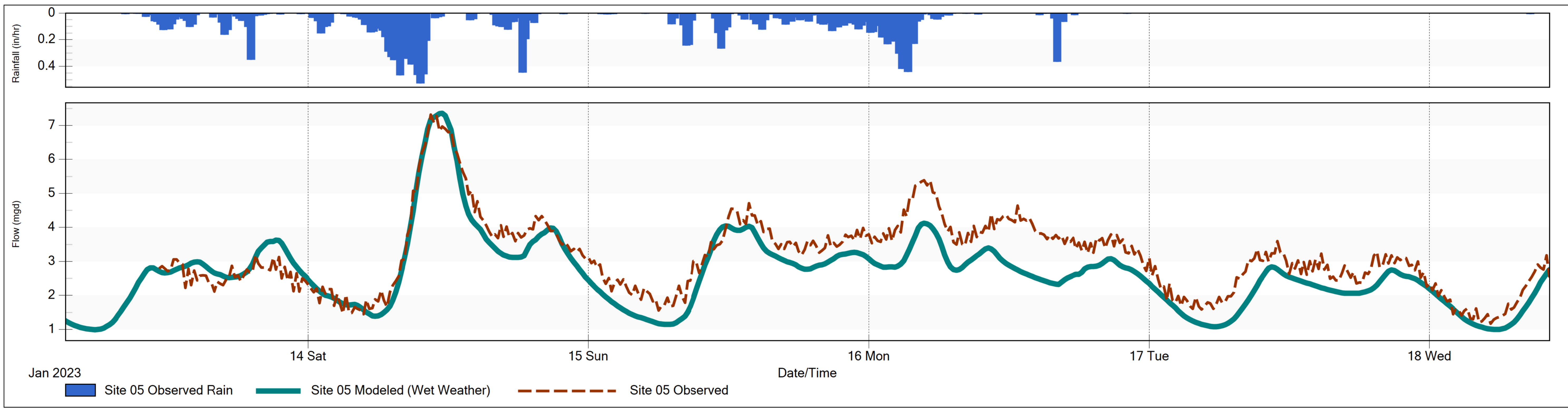
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 7.320 and 7.353 mgd, respectively.

April 3, 2024

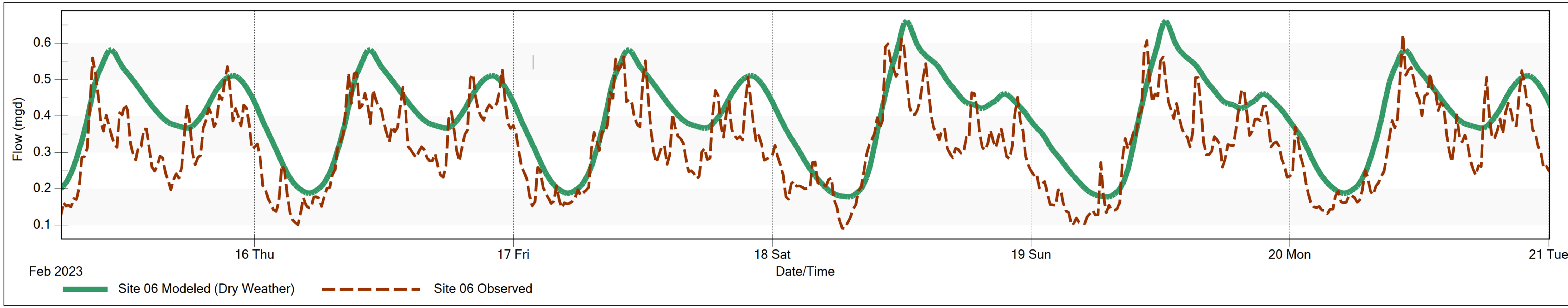
Figure C5

Site 5 Flow Calibration Results

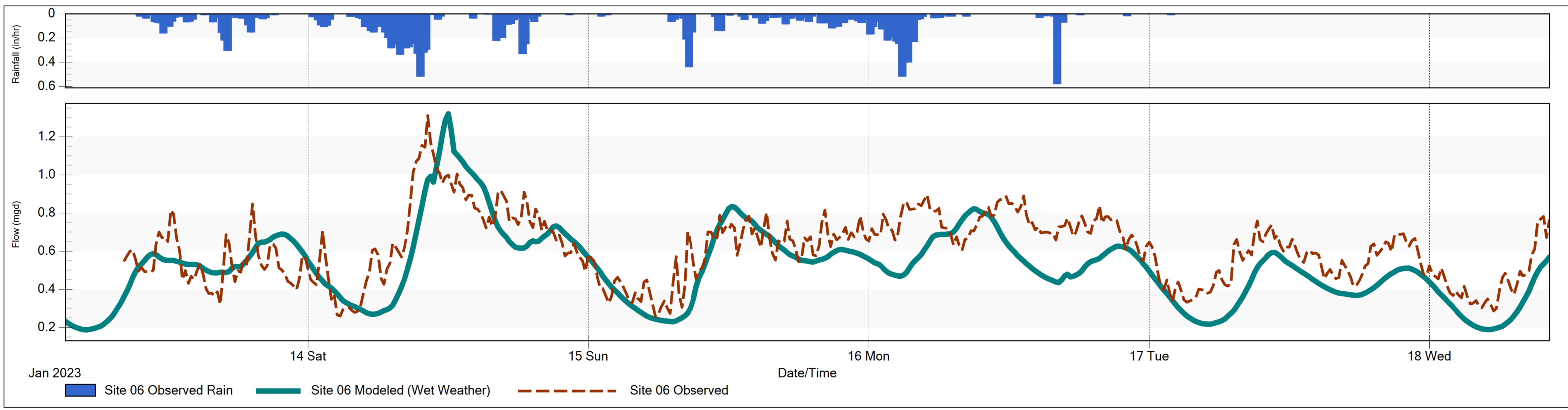
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 1.310 and 1.319 mgd, respectively.

April 3, 2024

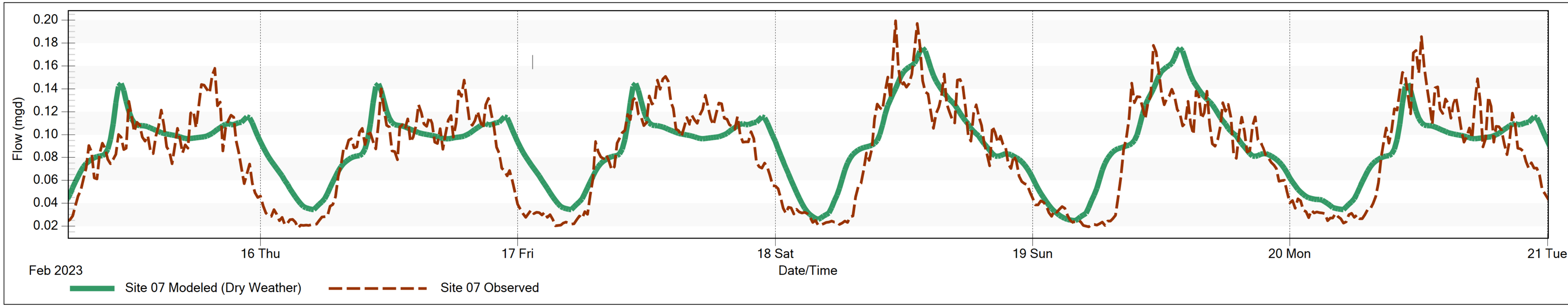
Figure C6

Site 6 Flow Calibration Results

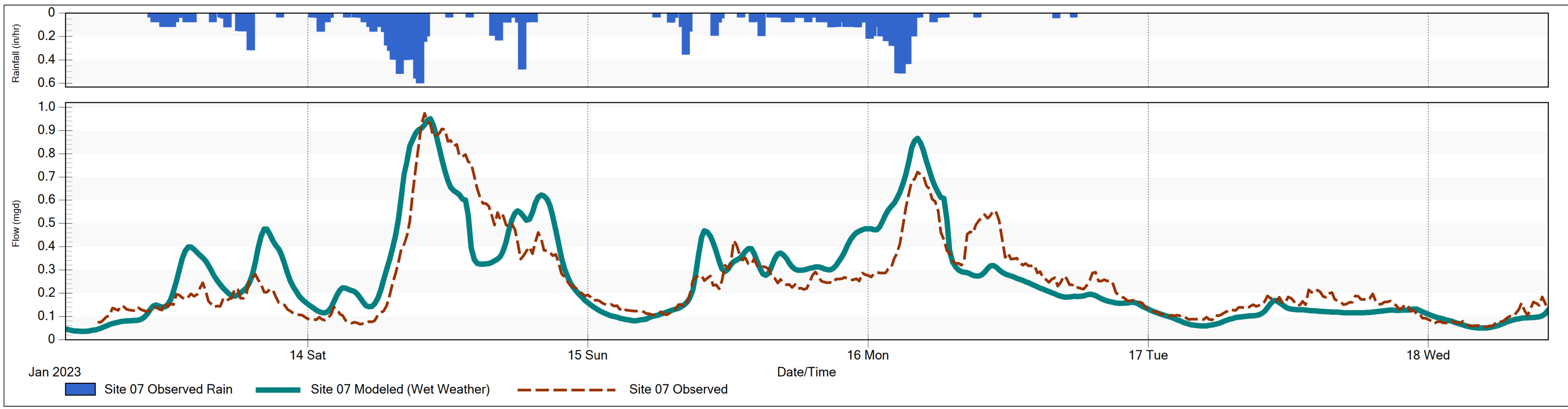
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 0.970 and 0.951 mgd, respectively.

April 3, 2024

Figure C7

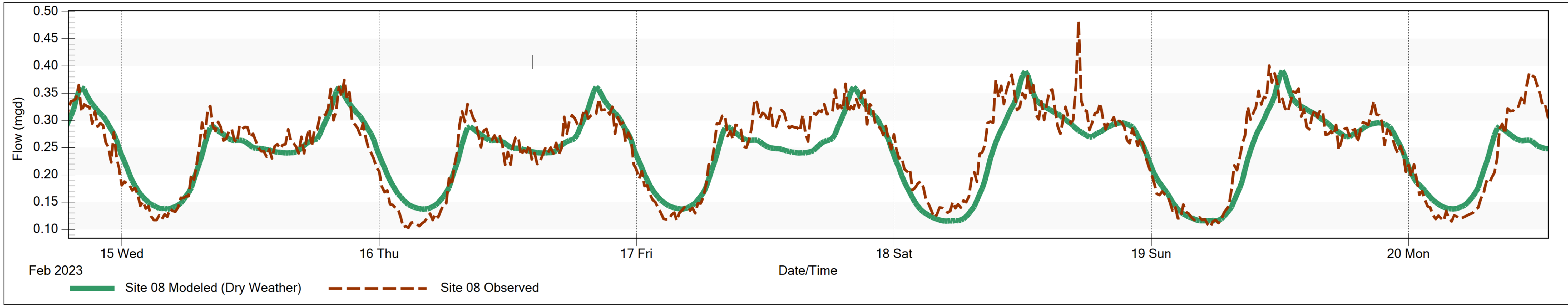
Site 7 Flow Calibration Results

Wastewater Collection System

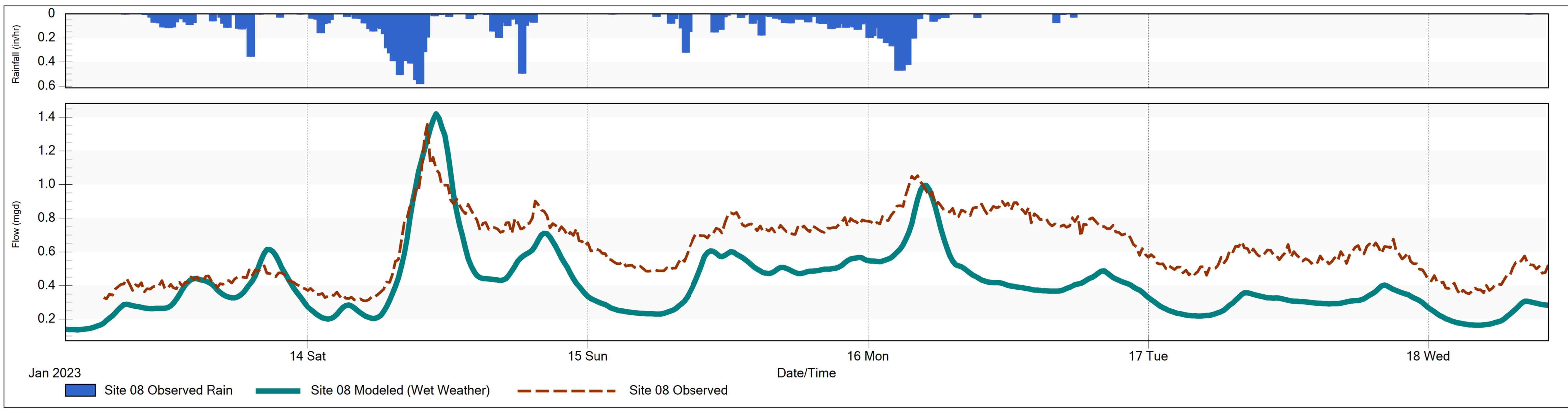
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 1.360 and 1.419 mgd, respectively.

April 3, 2024

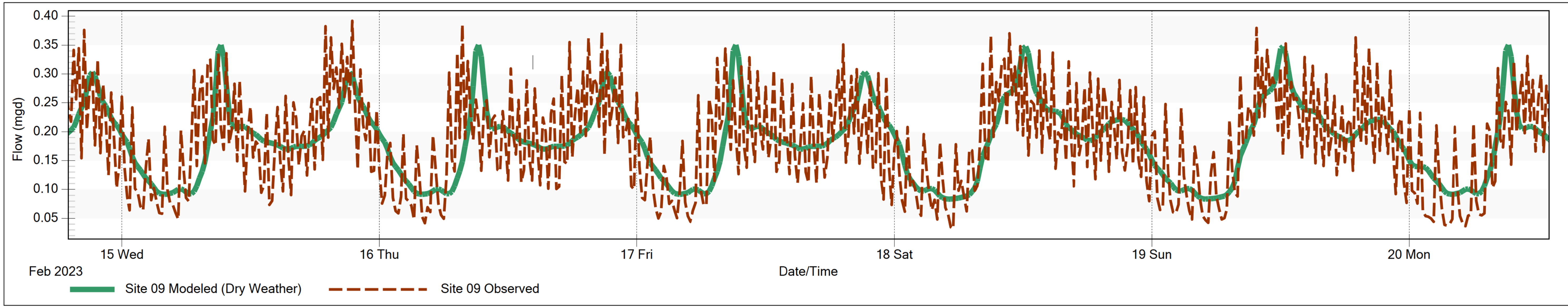
Figure C8

Site 8 Flow Calibration Results

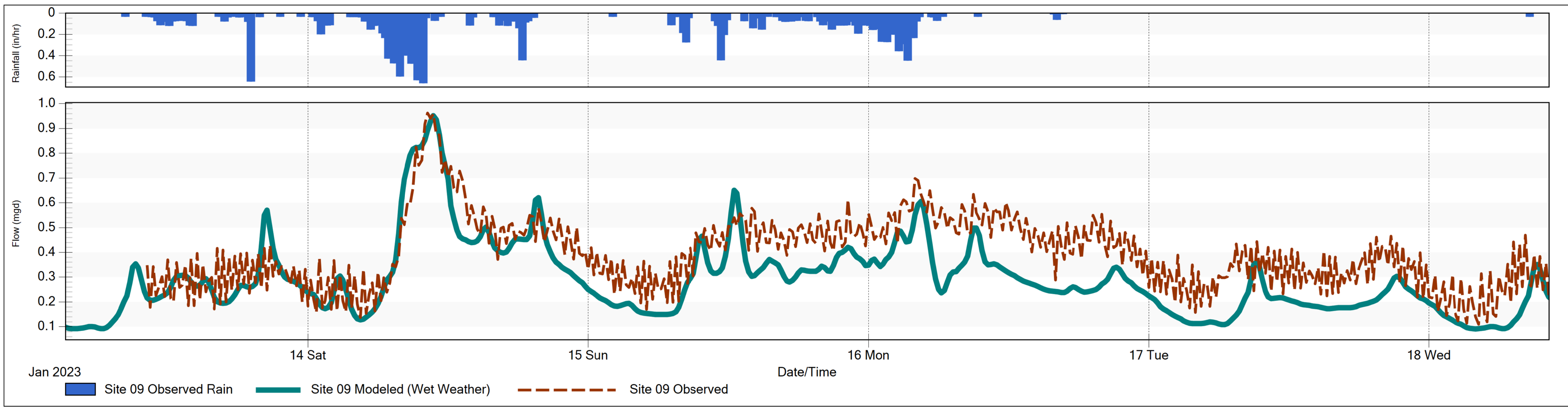
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 0.960 and 0.952 mgd, respectively.

April 3, 2024

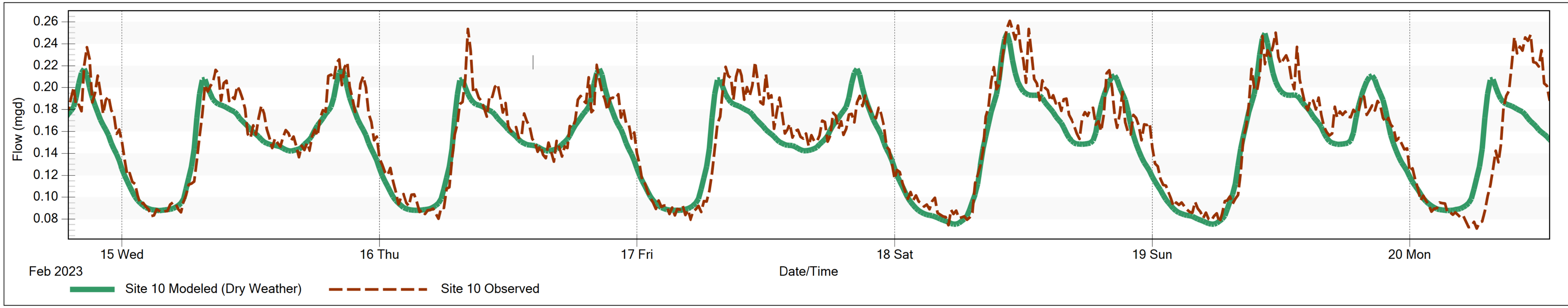
Figure C9

Site 9 Flow Calibration Results

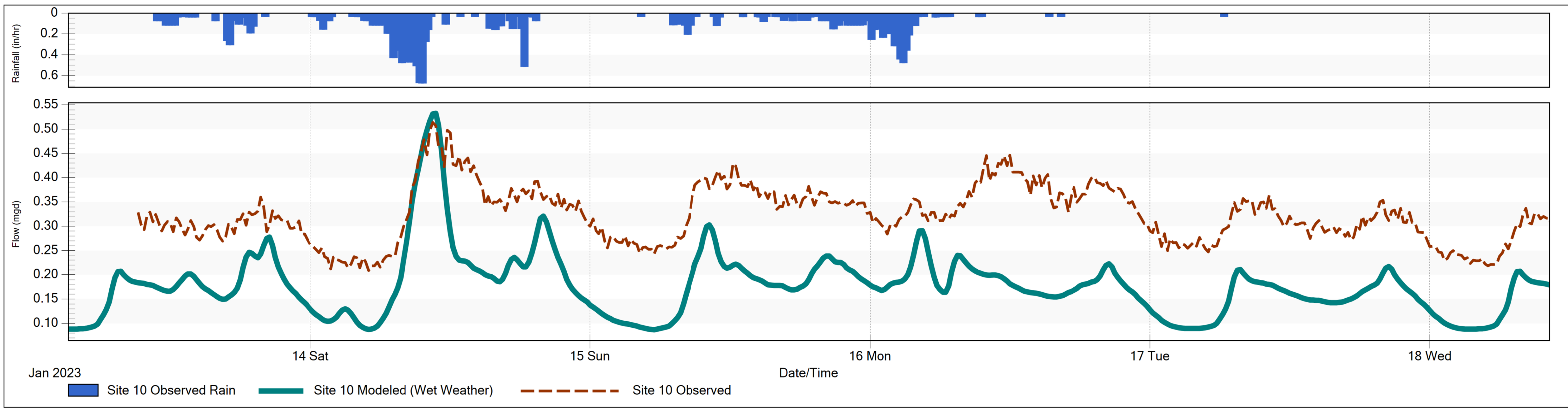
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 0.510 and 0.532 mgd, respectively.

April 3, 2024

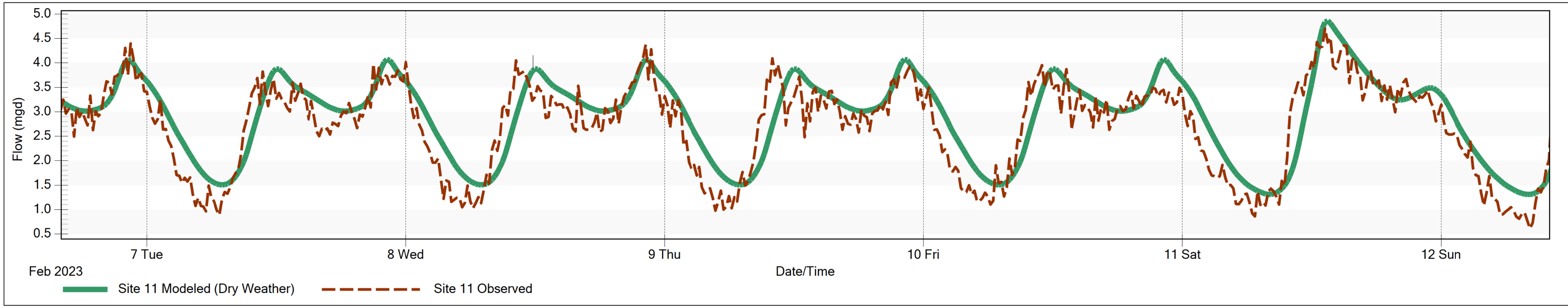
Figure C10

Site 10 Flow Calibration Results

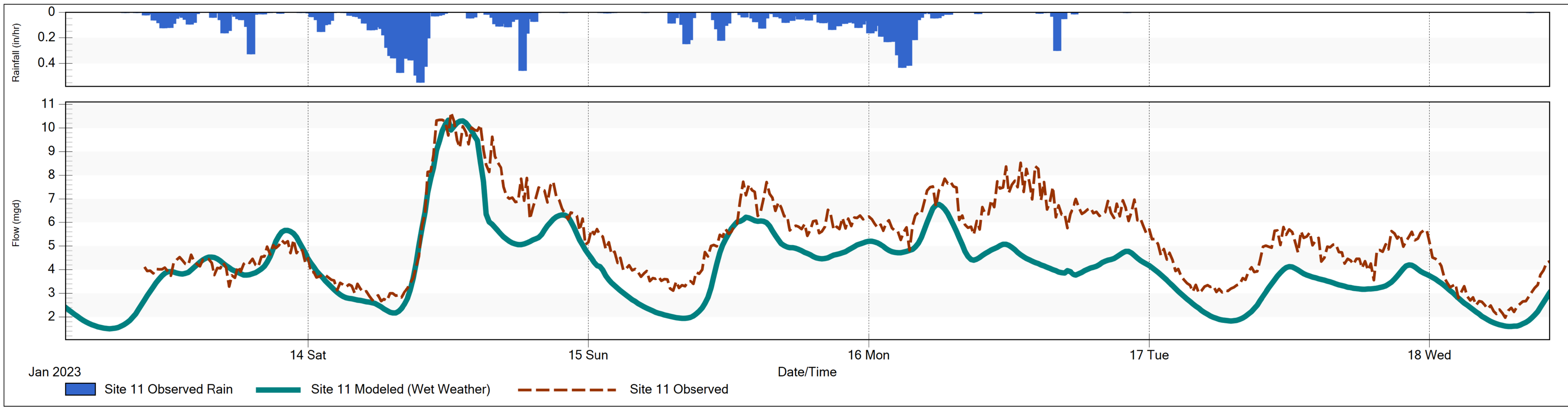
Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

Peak Dry Weather Flow Calibration



Peak Wet Weather Flow Calibration



Notes:

1. The observed and modeled datasets are shown in 15-minute intervals.
2. The observed and modeled peak wet weather flow for the January 14-16, 2023 storm event are 10.650 and 10.330 mgd, respectively.
3. This is a permanent flow meter site is located in the Joint Trunk system near the intersection of Harding Avenue and Highland Avenue.
4. This site measures the total wastewater flows generated by the City of Morgan Hill. Additionally, flows from this site were also validated and confirmed using another temporary flow meter.

April 3, 2024

Figure C11

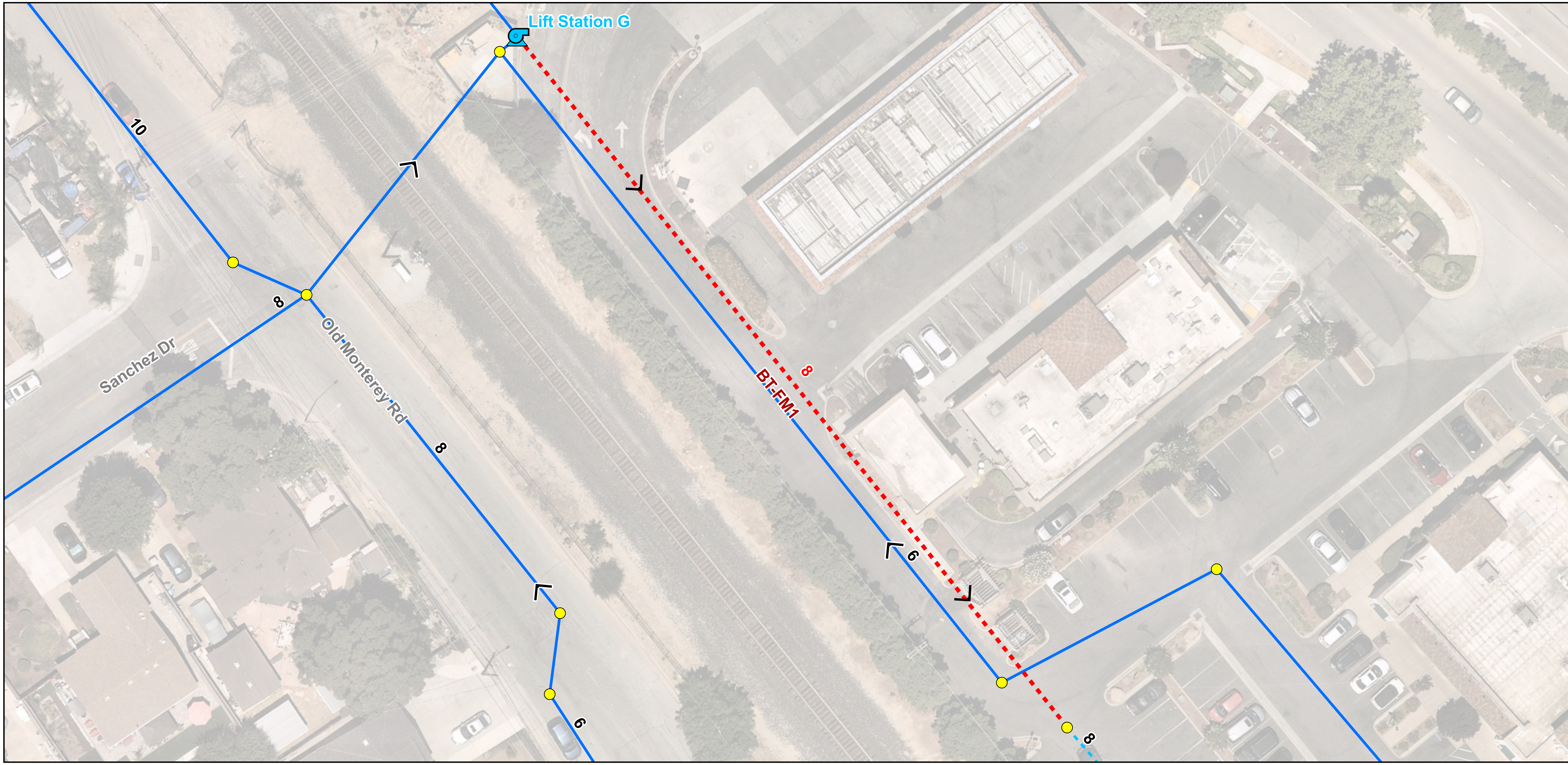
Site 11 Flow Calibration Results

Wastewater Collection System
Master Plan Update

CITY OF MORGAN HILL

APPENDIX D

Hydraulic Capacity Improvements –
Project Sheets



Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**BT-FM1**
Project Priority:	**High**
Implementation Schedule:	**2024 - 2026**
Description:	Replace approximately 350 feet of existing 6-inch diameter force main with a new 8-inch main from Lift Station G to 340 feet west of Monterey Road. This project was triggered due to existing operational issues and will require temporary bypass pumping during the implementation phase.

N W E S

Updated: April 16, 2024

0 12.5 25 50 Feet

Figure D1
Project: BT-FM1

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx

Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**BT-P1**
Project Priority:	**Low**
Implementation Schedule:	**Beyond 2035**
Description:	Construct approximately 2,250 feet of new 8-inch diameter gravity pipes along Peet Road from 420 feet east of Avenida De Los Padres to Cochrane Road. This is a long-term low priority project recommended to tie-in a future growth area into the City's existing collection system.

Updated: April 16, 2024

0 50 100 200 Feet

N S E W

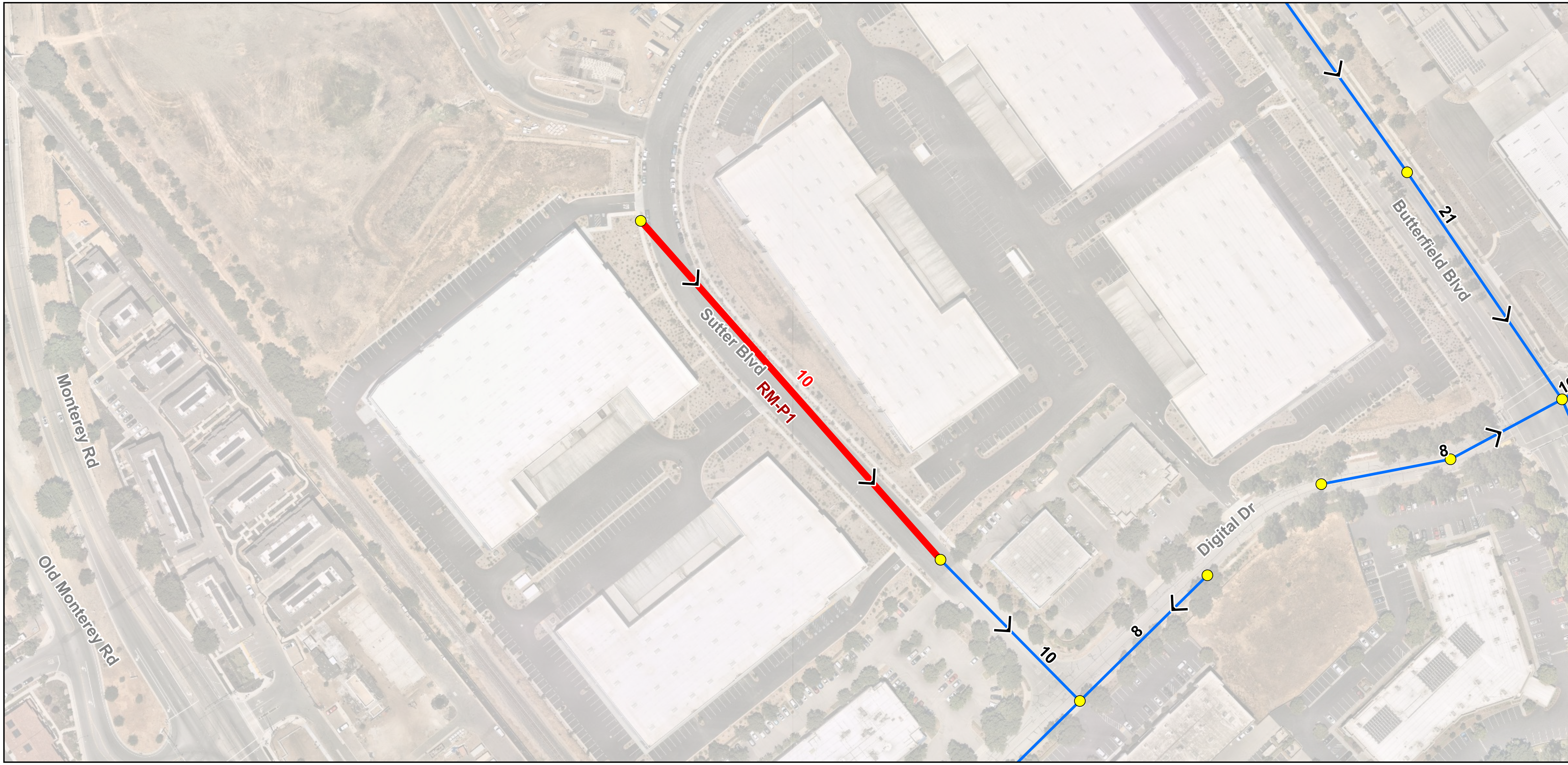
File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx

AKEL ENGINEERING GROUP, INC.

Figure D2
Project: BT-P1

Wastewater Collection System Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL



Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**RM-P1**
Project Priority:	**Low**
Implementation Schedule:	**2031 - 2035**
Description:	Construct approximately 650 feet of new 10-inch diameter gravity pipes 635 feet north of Digital Drive. This project will be triggered by future industrial growth (business park) north of Digital Drive.

Updated: April 16, 2024

0 50 100 200 Feet

N W E S

Figure D3
Project: RM-P1

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx

Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**HL-P1**
Project Priority:	**High**
Implementation Schedule:	**2024 - 2026**
Description:	Replace approximately 2,250 feet of existing 8-inch diameter gravity pipes with new 10-inch pipes along Llagas Creek Drive from Llagas Road to Hale Avenue. This project was identified in the previous master plan and triggered due to an existing deficiency.

N W E S

0 55 110 220 Feet

Updated: April 16, 2024

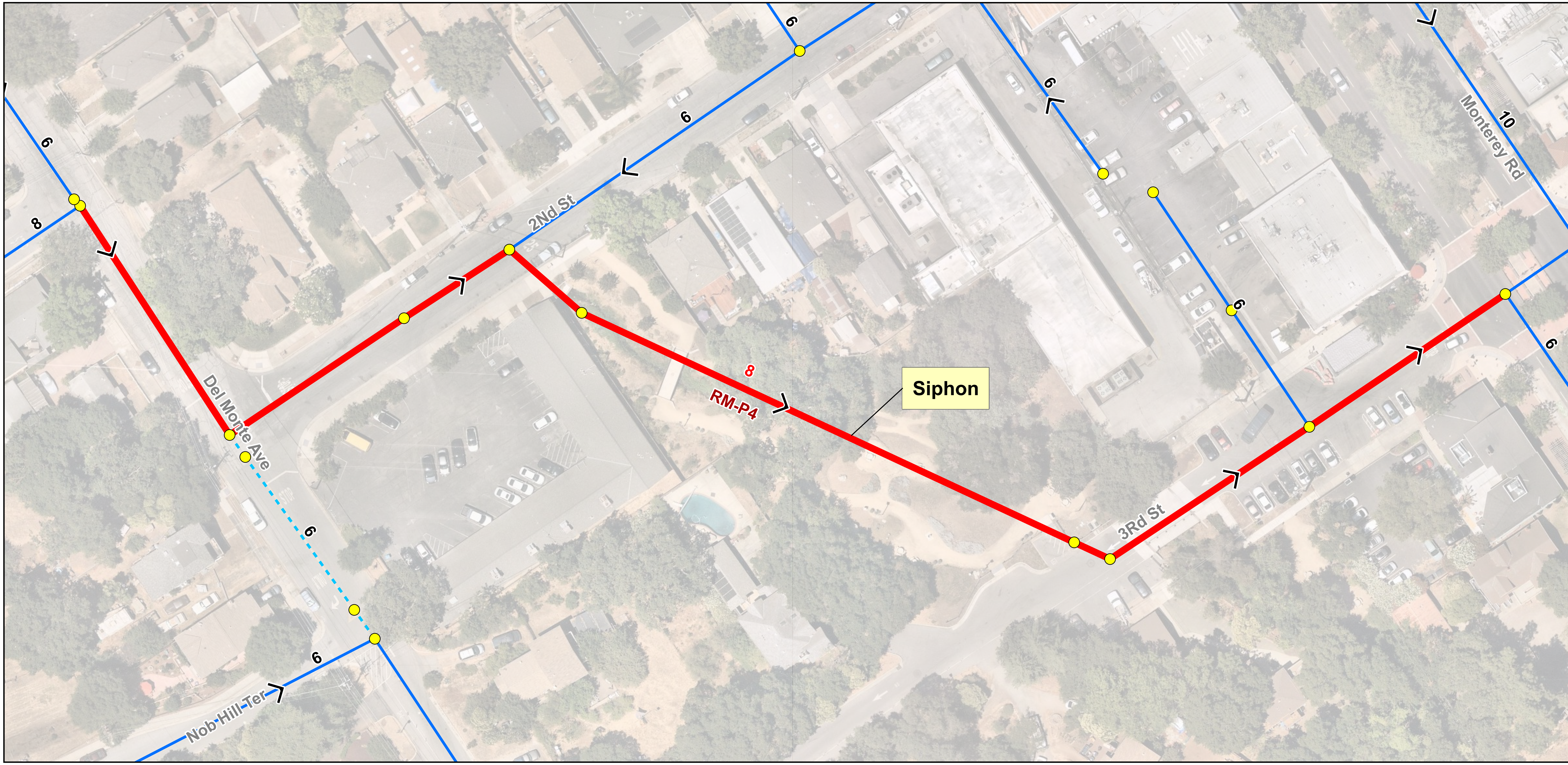
Figure D4
Project: HL-P1

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx



Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**RM-P4**
Project Priority:	**Medium**
Implementation Schedule:	**2027 - 2030**
Description:	Replace approximately 1,050 feet of existing 6-inch diameter gravity pipes with 8-inch pipes along West 2nd Street and West 3rd Street from Del Monte Avenue to Monterey Road. This project includes a siphon from West 2nd Street to West 3rd Street.

Updated: April 16, 2024

0 20 40 80 Feet

N W E S

Figure D5
Project: RM-P4

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx

Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus
- Other CIP Projects

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**RM-DIV1 and RM-P3**
Project Priority:	**Medium**
Implementation Schedule:	**2027 - 2030**
Description:	RM-DIV1 - Retrofit an existing diversion manhole at the intersection of Main Avenue and Monterey Road to divert majority of the flows east along Main Avenue. RM-P3 - Replace approximately 750 feet of existing 15-inch and 12-inch diameter gravity pipes with 21-inch pipes along Main Avenue from Monterey Road to Mason Lane.

Updated: April 16, 2024

0 25 50 100 Feet

N W E S

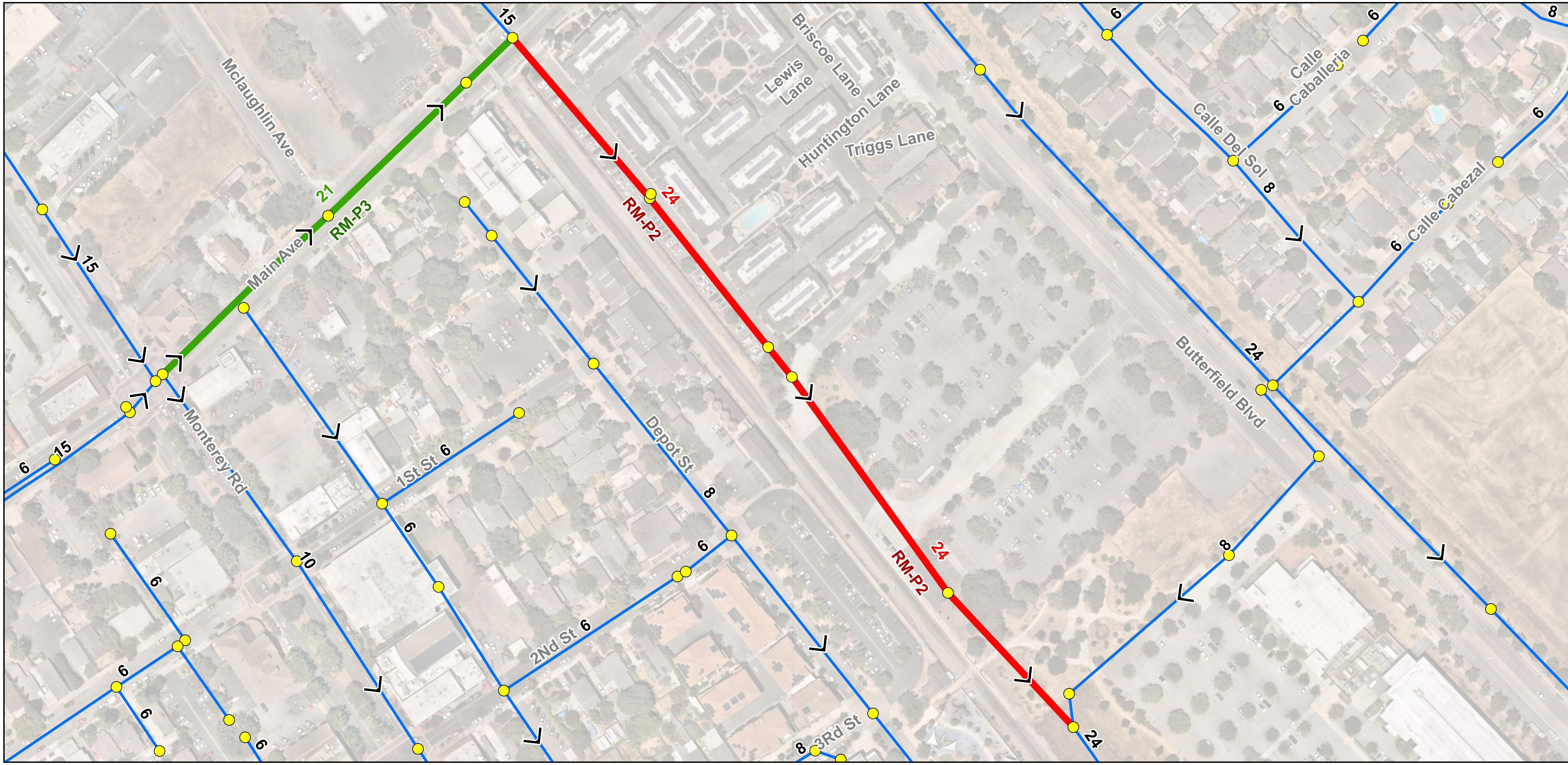
Figure D6
Project: RM-DIV1 and RM-P3

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx



Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus
- Other CIP Projects

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**RM-P2**
Project Priority:	**Low**
Implementation Schedule:	**2031 - 2035**
Description:	Replace approximately 1,400 feet of existing 15-inch diameter gravity pipes with 24-inch pipes along Mason Lane from Main Avenue to 150 feet north of East 4th Street. This project will be triggered by future growth.

Updated: April 16, 2024

0 55 110 220 Feet

N S E W

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx

Figure D7
Project: RM-P2

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**HB-DIV2 and HB-P2**
Project Priority:	**Medium**
Implementation Schedule:	**2027 - 2030**
Description:	HB-DIV2 - Retrofit an existing diversion manhole at the intersection of Hill Road and East Dunne avenue to divert all flows south along Hill Road. HB-P2 - Replace approximately 850 feet of existing 8-inch diameter gravity pipes with 10-inch pipes along Hill Road from East Dunne Avenue to Sundance Drive.

N W E S

0 40 80 160 Feet

Updated: April 16, 2024

Figure D8
Project: HB-DIV2 and HB-P2

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus
- Other CIP Projects
- Pipes to be Abandoned

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

AKEL ENGINEERING GROUP, INC.

Project ID:	**HB-DIV1 and HB-P1**
Project Priority:	**High**
Implementation Schedule:	**2024 - 2026**
Description:	HB-DIV1 - Construct a new diversion manhole at the intersection of Condit Road and East Dunne Avenue to route flows south along Condit Road. HB-P1 - Construct approximately 3,450 feet of new 12-inch diameter gravity pipes along Condit Road from East Dunne Avenue to Barrett Avenue.

Updated: April 16, 2024

0 170 340 680 Feet

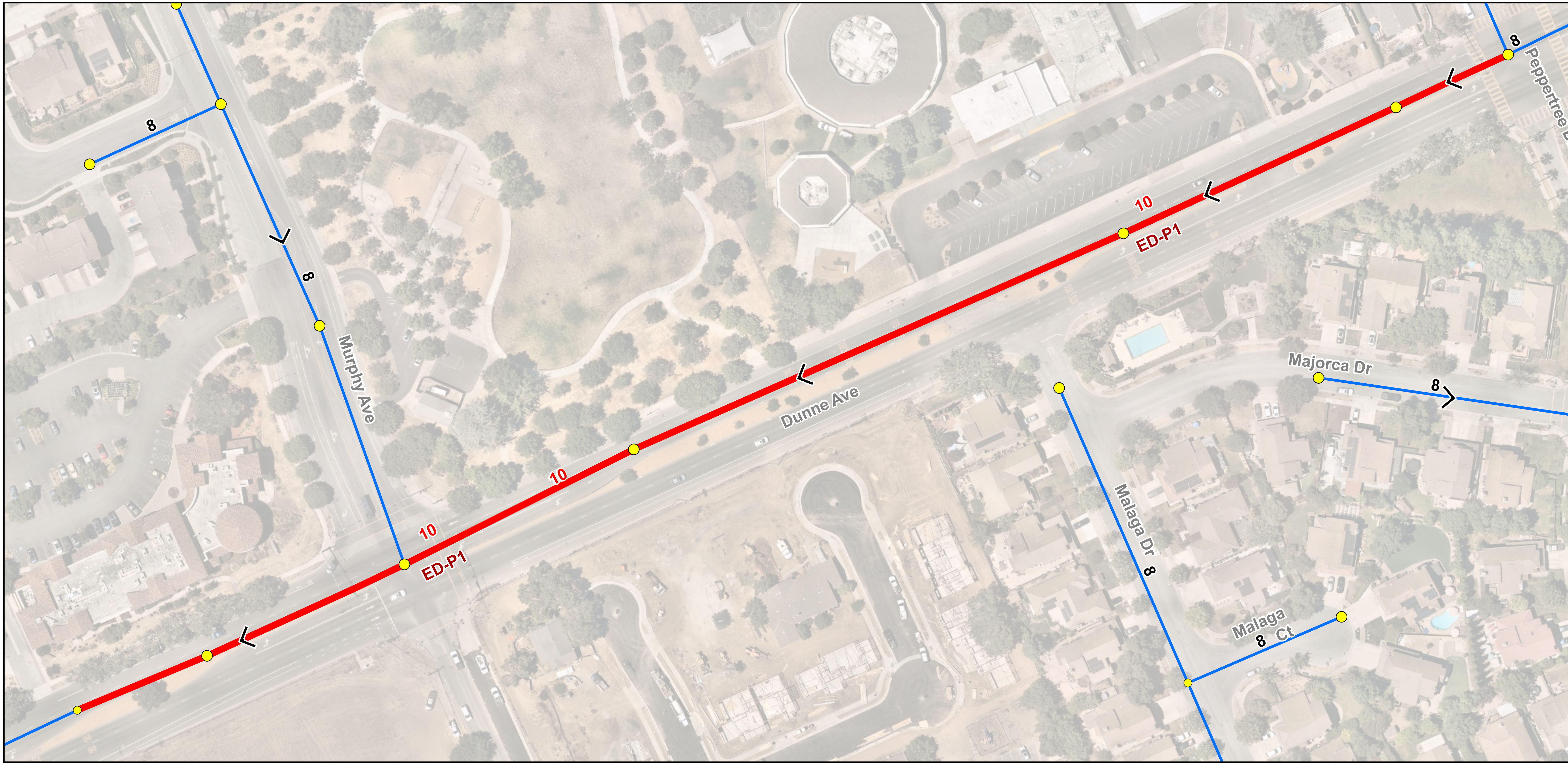
N E S W

Figure D9
Project: HB-DIV1 and HB-P1

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx



Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**ED-P1**
Project Priority:	**High**
Implementation Schedule:	**2024 - 2026**
Description:	Replace approximately 1,600 feet of existing 8-inch diameter gravity pipes with new 10-inch pipes along East Dunne Avenue from Peppertree Drive to 300 feet east of Condit Road. This project was triggered due to an existing deficiency and will require traffic control during the implementation phase.

Updated: April 16, 2024

N W E S

0 35 70 140 Feet

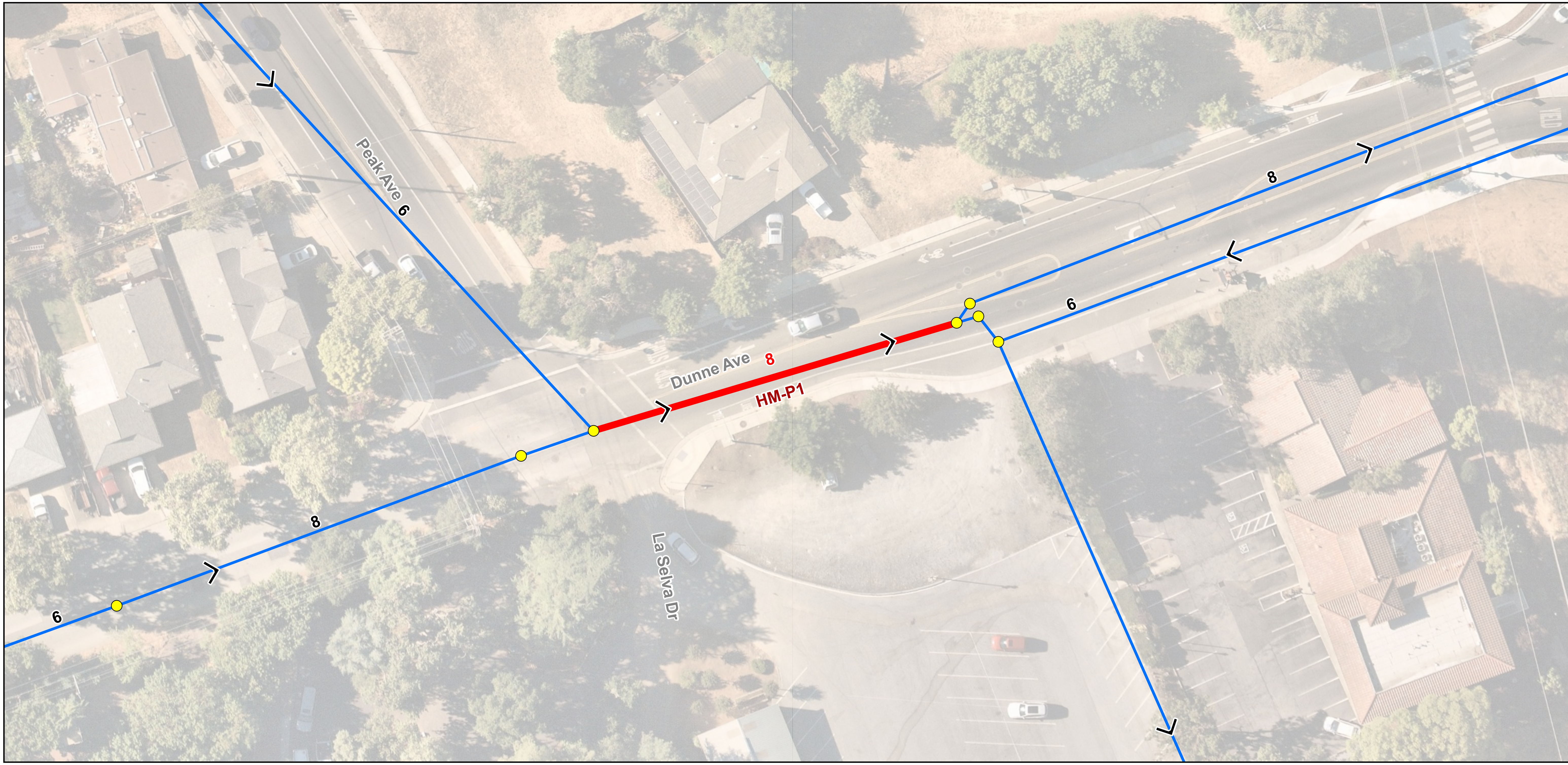
Figure D10
Project: ED-P1

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx



Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**HM-P1**
Project Priority:	**Low**
Implementation Schedule:	**2031 - 2035**
Description:	Replace approximately 150 feet of an existing 6-inch diameter gravity pipe with a 8-inch pipe along West Dunne Avenue from Peak Avenue to 150 feet east of Evergreen Drive.

Updated: April 16, 2024

0 12.5 25 50 Feet

N W E S

Figure D11
Project: HM-P1

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx

Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus
- Other CIP Projects

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**RM-P6**
Project Priority:	**Medium**
Implementation Schedule:	**2027 - 2030**
Description:	Replace approximately 2,250 feet of existing 10-inch diameter gravity pipes with 12-inch pipes along Monterey Road from San Pedro Avenue to Edes Street.

Updated: April 16, 2024

0 110 220 440 Feet

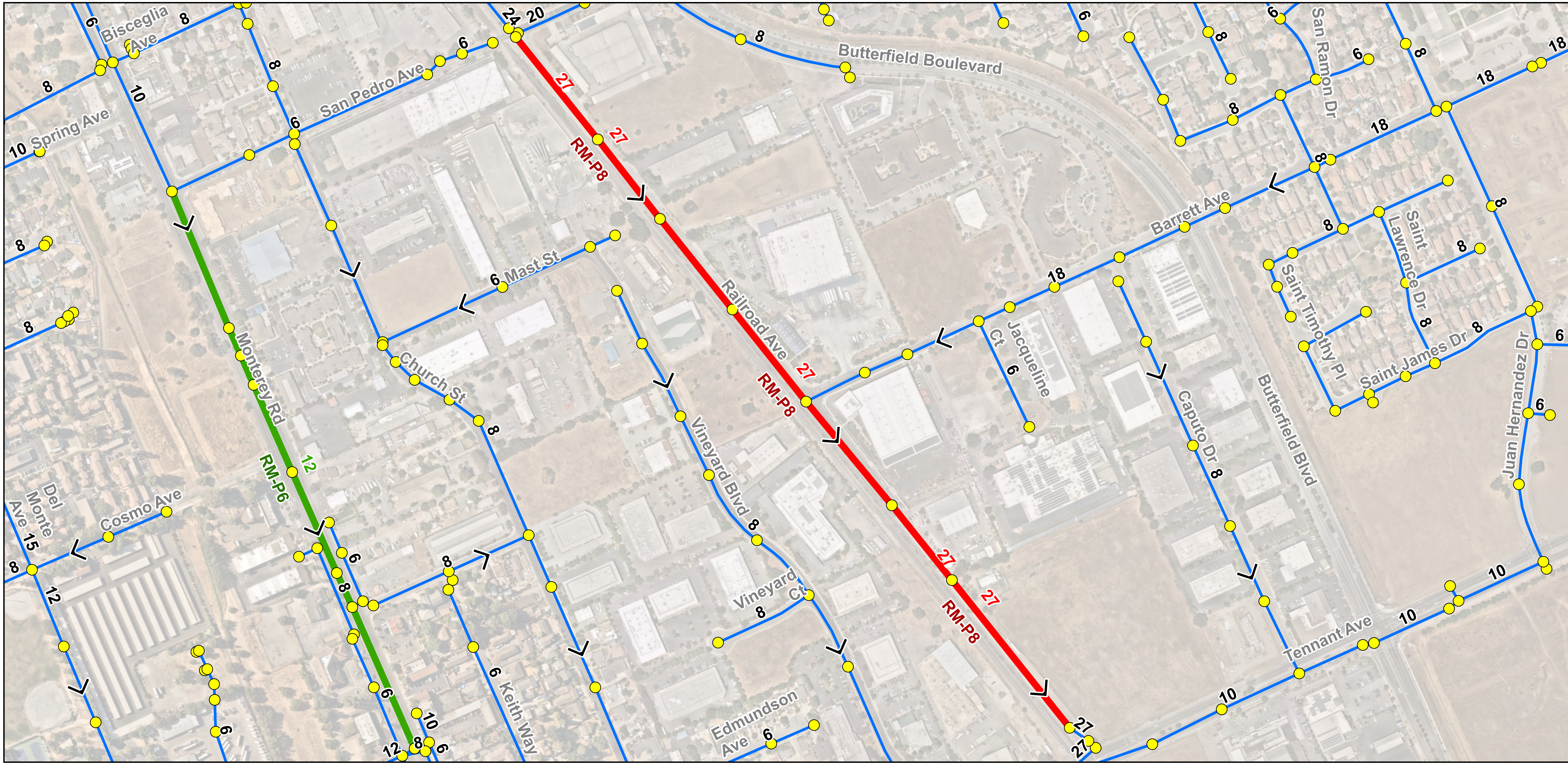
Figure D12
Project: RM-P6

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\240122-MorganHillCIP\Details\MH_CIP_Details_041524.aprx



Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus
- Other CIP Projects

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**RM-P8**
Project Priority:	**Low**
Implementation Schedule:	**Beyond 2035**
Description:	Replace approximately 3,250 feet of existing 24-inch diameter gravity pipes with 27-inch pipes along Railroad Avenue from San Pedro Avenue to 100 feet north of Tennant Avenue. This is a long-term low priority project recommended to improve system performance.

Updated: April 16, 2024

0 137.5 275 550 Feet

N E S W

Figure D13
Project: RM-P8

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

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Legend

Capacity Improvements
- Diversion Structures
- CIP Project Focus

Existing System
- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other
- Lakes
- City Limits

Project ID:	**RM-P5**
Project Priority:	**Medium**
Implementation Schedule:	**2027 - 2030**
Description:	Replace approximately 1,200 feet of existing 10-inch diameter gravity pipes with 12-inch pipes along Watsonville Road from 400 feet west of Calle Sueno to Monterey Road.

Updated: April 16, 2024

0 70 140 280 Feet

N W E S

Figure D14
Project: RM-P5

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

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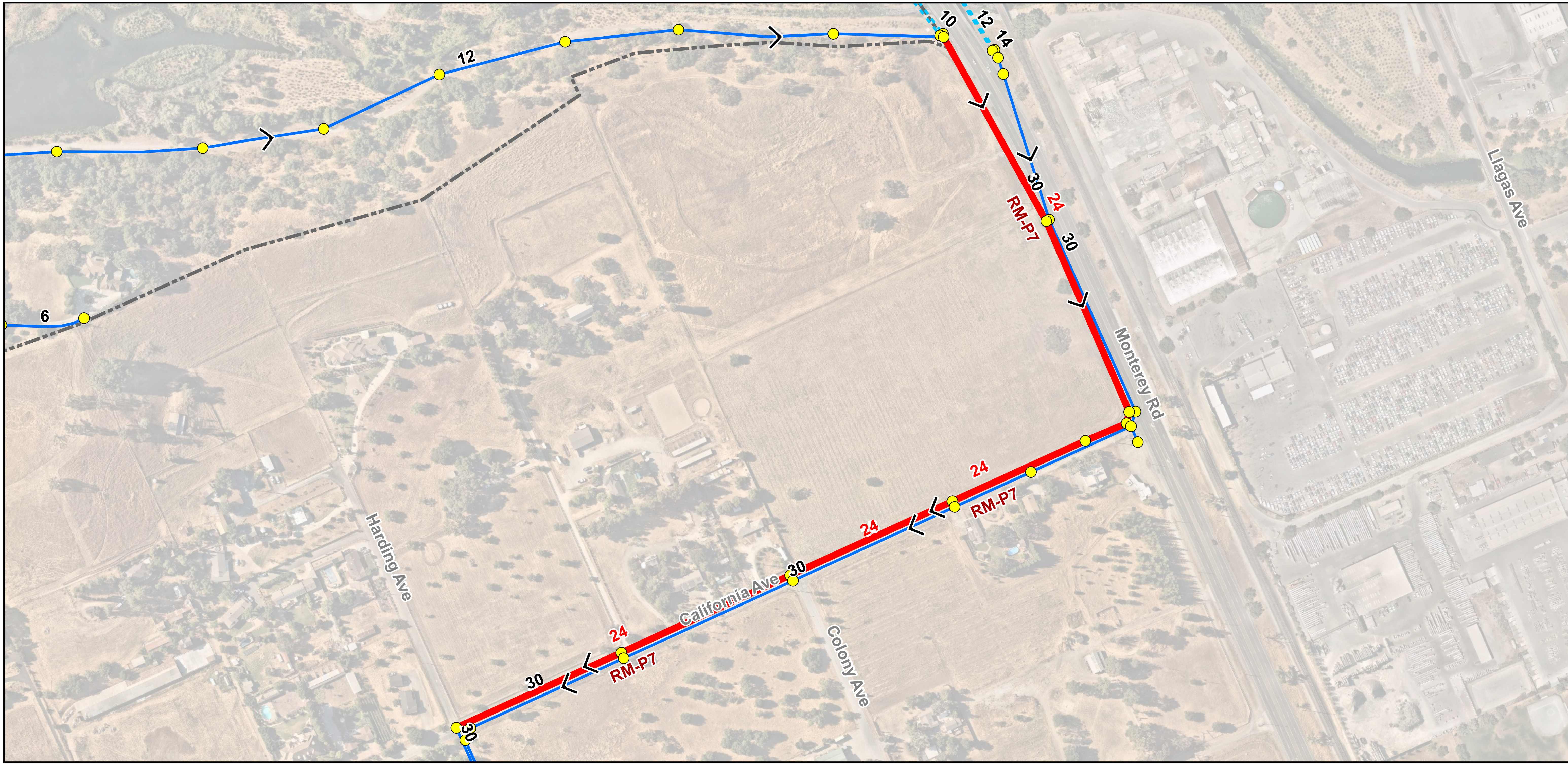

Legend

Capacity Improvements

- Diversion Structures
- CIP Project Focus

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	**RM-P7**
Project Priority:	**Low**
Implementation Schedule:	**Beyond 2035**
Description:	Replace approximately 3,150 feet of existing 18-inch diameter gravity pipes with 24-inch pipes along Monterey Road and California Avenue, from south of Llagas Creek to Harding Avenue. This is a long-term low priority project recommended to improve system performance.

Updated: April 16, 2024

0 100 200 400 Feet

N W E S

Figure D15
Project: RM-P7

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

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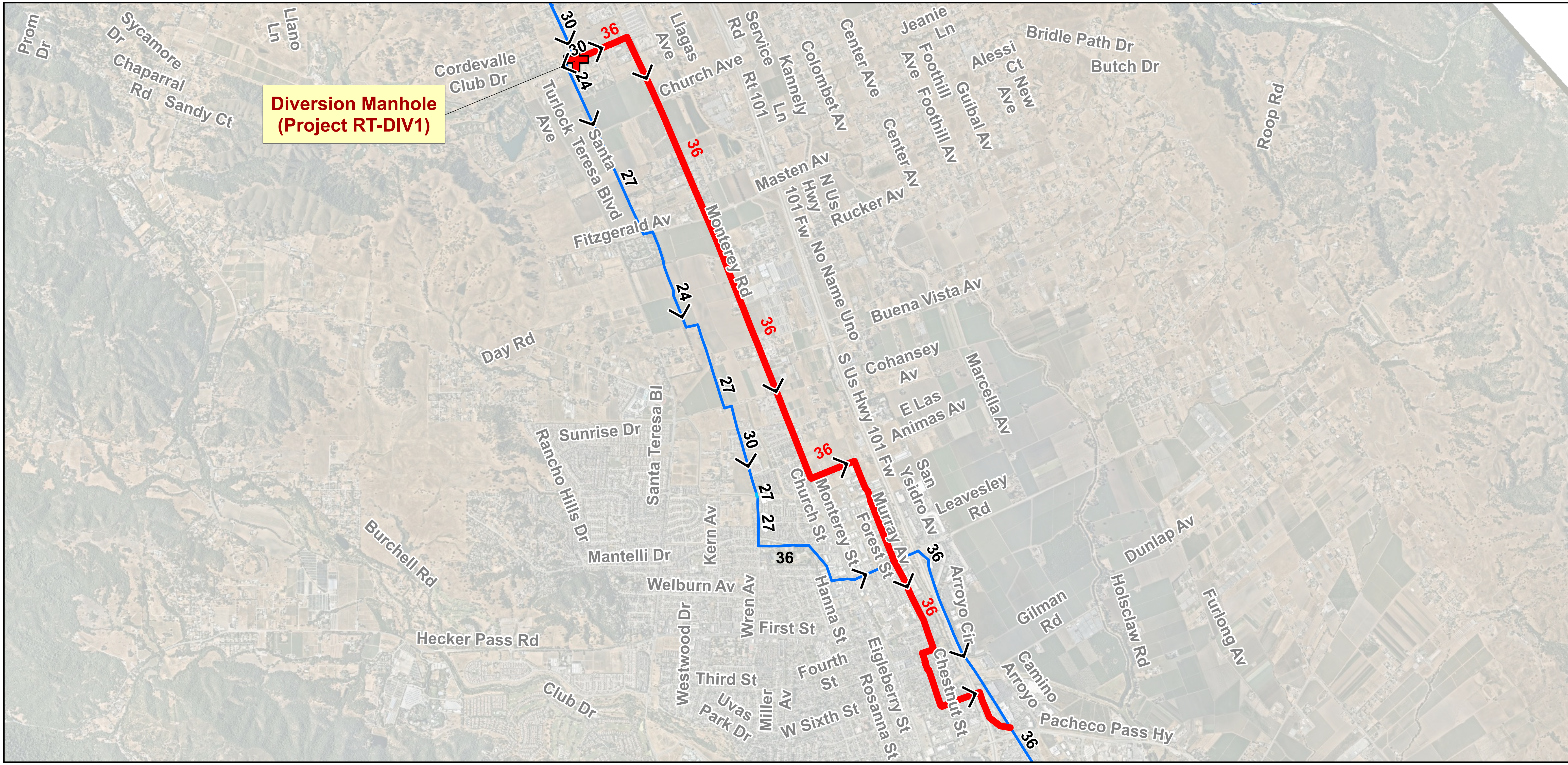

Legend

Capacity Improvements

- Diversion Structures
- Proposed Relief Trunk (90% Design)

Existing System

- Lift Stations
- SCRWA WWTP
- Manholes
- Gravity Mains
- Force Mains / Siphons

Other

- Lakes
- City Limits

Project ID:	RT-DIV1 and RT-P1 to RT-P13
Project Priority:	High
Implementation Schedule:	2024 - 2026
Description:	Construct 7 miles of new 36-inch diameter gravity pipes from the intersection of Harding Avenue and Highland Avenue to Highway 101 and Renz Lane in the City of Gilroy. This project includes a diversion structure with slide gates to route flows east intothe existing Joint Trunk or west into the new Relief Trunk. This hydraulic capacity improvement will alleviate future system deficiencies, provide redundancy for maintenance in the Joint Trunk, and operational flexibility for flow routing.

Updated: April 16, 2024

0 1,250 2,500 5,000 Feet

N S E W

Figure D16
Project: RT-DIV1 and RT-P1 to RT-P13

Wastewater Collection System
Master Plan Update
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

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