ONEWATER MORRO BAY

FINAL | October 2018

carollo

City of Morro Bay
OneWater Morro Bay Engineering and Planning

ONEWATER MORRO BAY PLAN

FINAL | October 2018

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Abbreviations

AC	acre
AFY	acre-feet per year
ADD	average day demand
ADWF	average dry weather flow
AWWA	American Water Works Association
BRC	Blue Ribbon Commission
BWRO	brackish water reverse osmosis
CAFR	Comprehensive Annual Financial Report
Carollo	Carollo Engineers, Inc.
CCWA	Central Coast Water Authority
CDP	Coastal Development Permit
CEQA	California Environmental Quality Act
CIP	Capital Improvement Plan or cast iron pipe
City	City of Morro Bay
cf	cubic feet
cfs	cubic feet per second
d/D	depth of flow/pipe's diameter
DCIA	Directly Connected Impervious Area
DDW	Division of Drinking Water
DPR	direct potable reuse
DWR	Department of Water Resources
EPA	Environmental Protection Agency
ETo	Evapotranspiration
F	Fahrenheit
ft	feet
ft/kft	feet per thousand feet
ft-msl	feet above sea level
ft/s	feet per second
FEMA	Federal Emergency Management Agency
FAAST	Hydraulic network computer modeling software
GAC	Granular Activated Carbon
GIS	Geographic Information System
gpcd	gallons per capita per day
gpd/ac	gallons per day per acre
gpm	gallons per minute
GWI	groundwater infiltration
HGL	Hydraulic Grade Line

hp	horsepower
I/I	infiltration and inflow
IPR	indirect potable reuse
kft	kilofoot
kWh	kilowatt-hour
LID	Low Impact Development
MCL	maximum contaminant level
MDD	maximum day demand
MG	million gallons
MSL	mean sea level
µg/L	micrograms per liter
mg/L	milligrams per liter
mgd	million gallons per day
MinDD	minimum day demand
MinMD	minimum month demand
MMD	maximum month demand
msl	mean sea level
NOAA	National Oceanic and Atmospheric Administration
NO_3	Nitrate
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
O&M	Operation and Maintenance
OMP&R	Operation and Maintenance, Power and Replacement
OneWater	OneWater Morro Bay Plan
PHD	peak hour demand
ppm	parts per million
PRV	pressure reducing valves
PS	pump station
psi	pounds per square inch
PWAB	Public Works Advisory Board
PWWF	peak wet weather flow
RO	reverse osmosis
ROW	Right of Way
RWQCB	Regional Water Quality Control Board
SCADA	supervisory control and data acquisition
SCS	Soil Conservation Service
SLOCFCWCD	San Luis Obispo County Flood Control and Water Conservation District
SWMM	Storm Water Management Model
SWP	State Water Project

TDS	total dissolved solids
TMDL	Total Maximum Daily Load
UF	ultrafiltration
UFW	unaccounted for water
UVAOP	ultra-violet light with advanced oxidation process
UWMP	Urban Water Management Plan
V&A	V&A Consulting Engineers
WaPUG	Wastewater Planning Users Group
WDF	water demand factor
WEF	Water Environment Federation
WRF	water reclamation facility
WWF	wet weather flow
WWTP	wastewater treatment plant
WTP	water treatment plant

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EXECUTIVE SUMMARY

This Executive Summary presents a brief background of City of Morro Bay (City) water supply, water distribution, wastewater collection, and storm drainage system. It also describes the need for this OneWater Morro Bay Plan (OneWater Plan) and the proposed improvements to mitigate system deficiencies. A summary of capital improvement project costs is also included.

ES.1 Introduction

The purpose of the OneWater Plan is to update the previous master plans, conduct an analysis of the City's water supplies, to identify capacity deficiencies in the water, sanitary sewer, and storm drainage systems, to develop feasible alternatives to correct these deficiencies, and to plan the infrastructure that will serve future development through the year 2040.

In March 2017, the City approved a professional service agreement with Carollo Engineers, Inc. (Carollo), to prepare the OneWater Plan. The professional services agreement included the following main tasks:

- Task 1 – Project Management and Meetings
- Task 2 – Data Collection and Background Review
- Task 3 – Planning Data Analysis
- Task 4 - Historic Water Demands and Wastewater Flows
- Task 5 - Water Supply Analysis
- Task 6 - Hydraulic and Hydrological Modeling.
- Task 7 - Flow Monitoring Data and I/I Mitigation Analysis
- Task 8 - Planning and Design Criteria
- Task 9 - Determine Existing System Capacities
- Task 10- Develop Improvements and Future Service Project Portfolios
- Task 11 - Develop Capital Improvement Program
- Task 12 - Financial Sustainability
- Task 13 - Project Deliverables
- Task 14 - Desalination Permitting Assistance

ES.2 Study Area

The City is located along the central coast of California in San Luis Obispo County, at the crossroads of Highway 1 and Highway 41, approximately 12 miles north of the city of San Luis Obispo, and approximately 6 miles south of the city of Cayucos. The City limits compromise roughly 5 square miles, and consist of varied topography ranging from steep mountain terrain to ocean basin.

The City is a small seaside town with strong historical roots in the fishing industry. Today, it is a thriving destination for visitors, offering natural beauty, outdoor recreation, a working

waterfront, a creative community, and a welcoming atmosphere. Approximately 20 percent of the City's residences are primarily used for vacation or seasonal uses.

The City, incorporated in 1964, provides water, sewer, and storm drainage services to its residents. Water services are provided to all residential, commercial, and industrial customers, and for fire protection services.

ES.3 Water, Sewer, and Storm Service Areas

Figure ES.1 illustrates the City's current water, wastewater, and storm drainage service area. The service area is the same for water, wastewater, and storm drainage.

The land use assumptions in this Master Plan were based on the projected future developments within the planning area from the City's planning document, Plan Morro Bay. Plan Morro Bay projects through the year 2040. Should future planning conditions change from the assumptions stated in the Master Plan (i.e. accelerated growth, more intense developments, etc.), revisions and adjustments to the Master Plan recommendations would be necessary.

ES.4 Planning Period

This OneWater Plan is intended to serve as a guiding document for the planning and implementation of water, sewer, and storm drainage system improvements to accommodate future growth within the City limits through the year 2040. The planning horizon is consistent with the City's planning document, Plan Morro Bay. The population and land use projects included in Plan Morro Bay are summarized in this chapter and set the foundation for the water demand projections and wastewater flow projections.

ES.5 Service Area Population

This section summarizes the City's historic and projected population within the system's service area. Population projection for the Cayucos community were not developed, as flow estimates for this areas were documented in separate planning documents for each agency and are considered separately.

Figure ES.2 shows the City's historic and projected population. Future growth in the City is limited by Measure F, a voter-approved growth management ordinance that limits the City to 12,200 residents. Plan Morro Bay has developed population projections based on this limitation, as summarized in Table ES.1. The projected population is estimated to increase by 1,435 residents within the planning period. The population estimates were developed, by source.

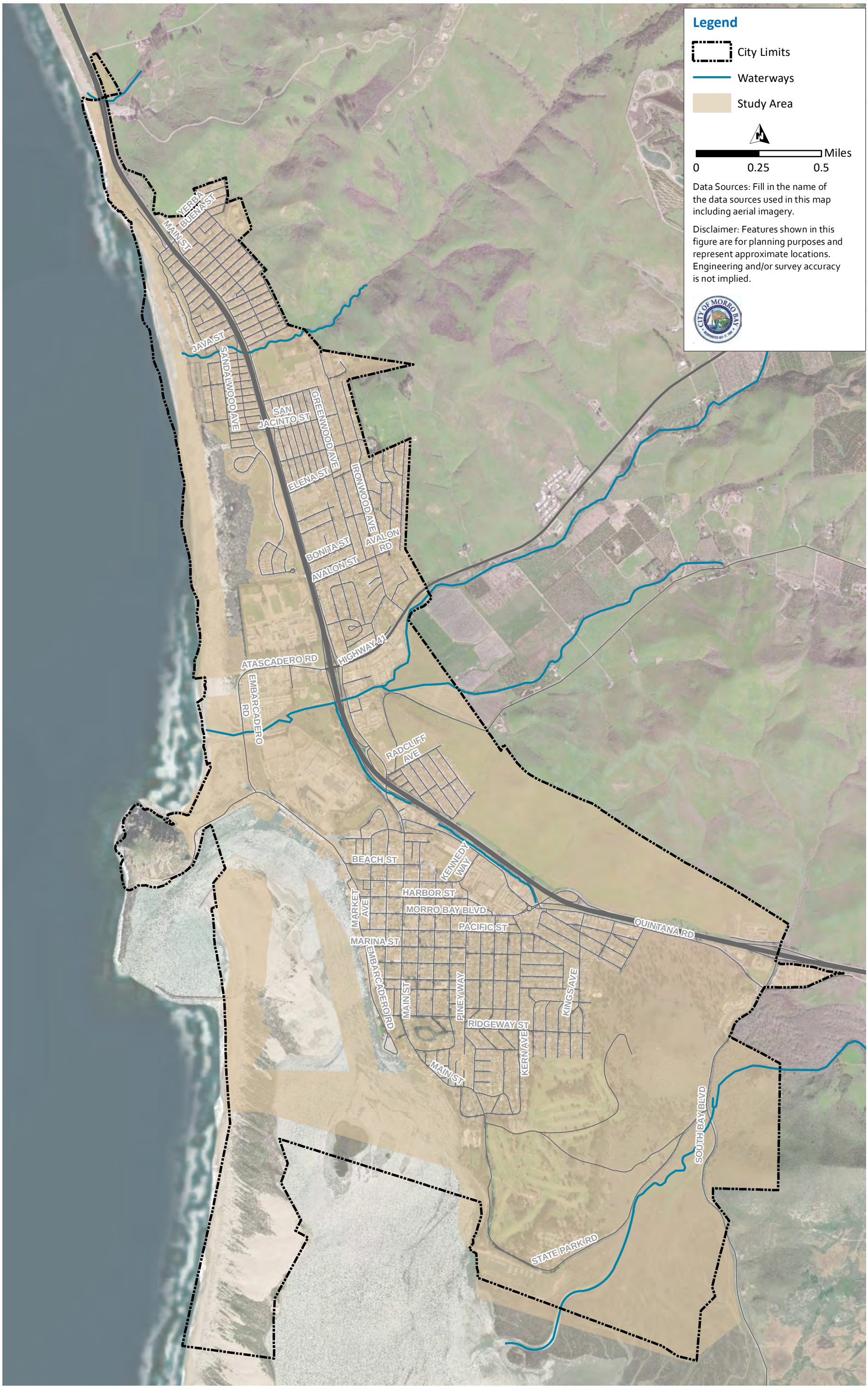

Figure ES.1 **Study Area Boundary**

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Table ES.1 Projected Population

Year	Population
2017	10,714
2020	10,901
2025	11,213
2030	11,525
2035	11,837
2040	12,149

Notes:
(1) Source: Plan Morro Bay

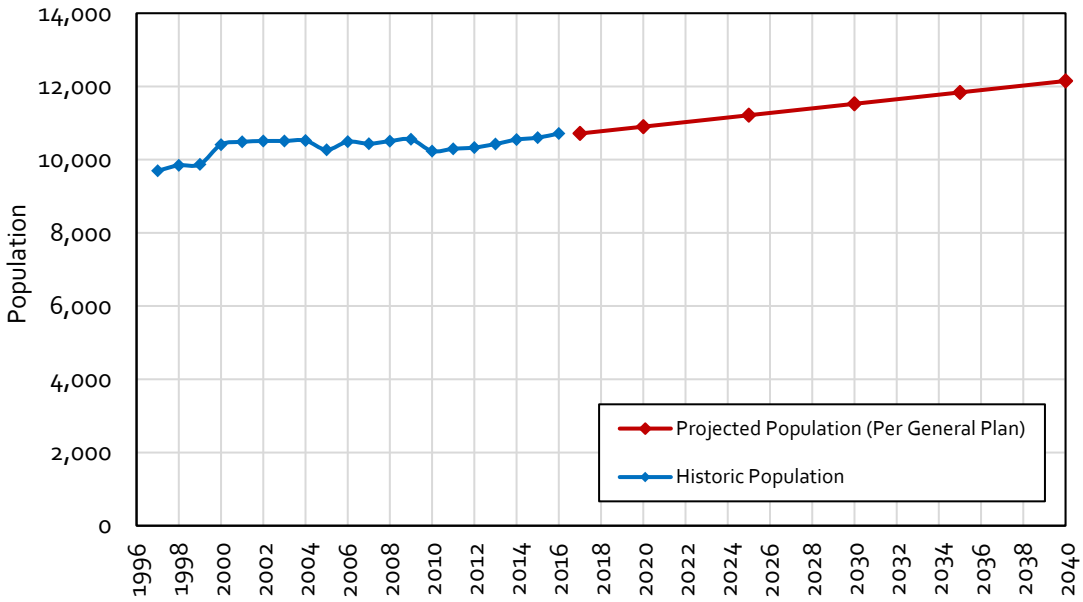


Figure ES.2 Historic and Projected Population

ES.6 Water Demands and Wastewater Flows

This section summarizes the historical, existing and future water demands and wastewater flows.

ES.6.1 Potable Water Demands

The planning horizon for this OneWater Plan is the year 2040, and according to the City's planning documents, modest growth is expected until then to comply with the population growth limits imposed by Measure F. The City's population is projected to increase to 12,149 people by year 2040.

Table ES.2 summarizes the projected demands through the year 2040. As shown in Table ES.2, the City's average day demand (ADD) and maximum day demand (MDD) are projected to approach 1.29 million gallons per day (mgd) and 2.09 mgd by 2040, respectively. The projected demands provided in Table ES.2 assume a per capita water demand of 106 gpcd, which is equal to the average per capita demand from 2010 to 2014. This is likely a conservative assumption,

because per capita waster demand is expected to decrease in the future due to increased conservation measures and more water efficient construction associated with new development. Because the purpose of the OneWater Plan is to size infrastructure needs for the distribution system, it is appropriate to include a level of conservatism when planning future transmission, distribution, and storage needs.

Table ES.2 Future Demand Projections through 2040

Year	Projected ADD (mgd)	Projected MDD(1) (mgd)
2017 (present)	1.14	1.85
2020	1.16	1.88
2025	1.19	1.93
2030	1.22	1.99
2035	1.25	2.04
2040	1.29	2.09

Notes:
(1) MDD/ADD peaking factor = 1.63

ES.6.2 Wastewater Flows

The existing average dry weather flow (ADWF) was estimated using the ADWF for the 2010 through 2014. For the purpose of OneWater, the existing ADWF is 0.88 mgd. This is the ADWF generated within the City, although the City of Cayucos contributes to the City's total wastewater generation.

Historical per-capita generation rates indicate a range between 56 and 164 gpcd. As previously stated, this OneWater Plan assumes a per capita flow of 82 gpcd for future flow projections. Table ES.3 summarizes projected wastewater flows through the year 2040. As shown in Table ES.3, the City's wastewater flow is projected to increase by 0.11 mgd by 2040.Heading 4

Table ES.3 Wastewater ADWF Projections

Year	Projected Population	Projected ADWF(1) (mgd)
2017	10,714	0.88
2020	10,901	0.89
2025	11,213	0.92
2030	11,525	0.94
2035	11,837	0.97
2040	12,149	0.99

Notes:
(1) Projected flow based on a per capita demand of 82 gpcd

The existing peak wet weather flow (PWWF) was derived throughout the system based on the hydraulic modeling results. This was accomplished by routing the 10-year, 24-hour design storm through the hydraulic model, which was calibrated to both dry weather and wet weather conditions. Similar to the existing PWWF, the future PWWFs were derived by routing a 10-year, 24 hour design storm through the hydraulic model.

Table ES.4 presents a summary of the existing and future ADWF and PWWF. As shown in Table ES.4, City's ADWF is projected to increase from 0.88 mgd to 0.99 mgd by 2040, whereas the PWWF is projected to increase from 7.90 mgd to about 8.12 mgd by 2040. Therefore, the City's PWWF to ADWF peaking factor is projected to decrease from roughly 9.0 to 8.2. This is typical, as newer sewers tend to have less RDII response than older areas of the system, primarily due to better construction methods. Furthermore, flow attenuation also tends to dampen out flow peaks as collection systems expand.

Table ES.4 Existing and Projected Wastewater Flows

Year	Projected ADWF[1] (mgd)	Projected PWWF[1][2][3] (mgd)	Peaking Factor (PWWF/ADWF)
2017	0.88	7.90[3]	9.0
2040	0.99	8.12	8.2

Notes:
(1) Does not include Cayucos influent flow.
(2) PWWF was estimated using a peak RDII of 1,000 gpd/ac.
(3) Assumes recommended existing improvement projects identified in Chapter 6 are completed.

ES.7 Water, Sewer, and Storm Drainage Systems and Hydraulic Models

This section summarizes the existing water distribution, wastewater collection, and storm drainage systems. It also summarizes the development of hydraulic models of each system.

ES.7.1 Existing Water, Sewer, and Storm Drainage Systems

The City's water, wastewater, and storm drainage systems consist of the following:

- Existing Water Supply and Distribution System. Morro Bay's water supply portfolio currently consists of four different sources. The City is currently in the process of developing a fifth water supply source by means of a new secondary wastewater treatment facility with the ability to produce recycled water for potable reuse. The supply portfolio includes:
  - California State Water Project (SWP) water conveyed by the Central Coast Water Authority (CCWA) and purchased from San Luis Obispo County
  - The Morro Groundwater Basin
  - The Chorro Groundwater Basin
  - Ocean Desalination
  - Water Reclamation Facility (future)

  The City's water system consists of approximately 72 miles of active water distribution pipelines up to 16-inches in diameter, ten treated water storage tanks, three booster pump stations, four Pressure Reducing Valve (PRV) stations, and six pressure zones.
- Existing Wastewater Collection System. The City's wastewater collection system consists of approximately 55 miles of active gravity sewer pipelines, 1.8 miles of force mains, and three pump stations that convey flow to the wastewater treatment plant (WWTP).
- Existing Storm Drainage System. The City operates a storm drainage collection system that consists of street drainage, inlets, gravity pipelines, open channels, one storm water detention basin, and outfalls to the Pacific Ocean.

ES.7.2 Water, Sewer, and Storm Drainage System Hydraulic Models

Three separate hydraulic models were utilized for the analysis of the water distribution, wastewater collection, and storm drainage systems. A summary of the hydraulic models updates is provided below. Additional details regarding the water and recycled water model updates, as well as the wastewater collection model development and calibration, are provided in Chapter 5 of this OneWater Plan.

- Water Distribution System Model. The City's water distribution system hydraulic model was developed constructed using InfoWater, developed by Innovyze. The water system hydraulic model was developed using the City's most recent Geographic Information Systems (GIS) database. Operational information, such as pump controls and PRV settings were input into the model based on information provided by the City. Allocation of water demands to appropriate nodes in the hydraulic model was accomplished by uses geocoded water billing records from the City. The City's water distribution system hydraulic model was calibrated using a three step process that included, a macro calibration, a fire flow test calibration, and an EPS calibration. The calibrated model was used for the distribution system analysis presented in this OneWater Plan.
- Wastewater Collection System Model. The City's wastewater collection system hydraulic model was developed constructed using InfoSWMM, developed by Innovyze. The wastewater collection system hydraulic model was developed using the City's most recent GIS database. Physical and operational data for the City's wastewater collection facilities was not available from the GIS data. This type of data, such as wet well dimensions, lift stations, and other special features, were input manually into the model based on available information. Dry weather wastewater flows were then allocated to the appropriate model junctions. The hydraulic model calibrated for dry- and wet-weather conditions using flow-monitoring data collected as part of this OneWater Plan. The calibrated model was used for the wastewater collection system analysis presented in this OneWater Plan.
- Storm Drainage Collection System Hydraulic and Hydrologic Model. The City's storm drainage system hydraulic and hydrologic model was developed constructed using InfoSWMM, developed by Innovyze. Hydrologic analysis of the City's storm drainage system was performed using the SWMM Runoff Block, which is included in the InfoSWMM modeling software. Physical and operational data for the City's storm drainage system facilities was not available from the GIS data. This type of data, such as outfall hydraulic grade assumption, detention basin information, open channel hydraulic parameters, and other special features, were input manually into the model based on available information. The storm drainage system model was used for the storm drainage system analysis presented in this OneWater Plan.

ES.8 Water, Wastewater, and Storm Drainage System Improvements

This section discusses the proposed projects that correct capacity deficiencies and serve future users.

ES.8.1 Water Distribution System Improvements

Following model calibration, a capacity analysis of the existing and future distribution system was performed. This section focuses on the capacity of the water distribution system only. A

detailed water supply evaluation is provided in the next section. Figure ES.3 provides a graphical illustration of the improvements recommended to address the capacity deficiencies. The following summarizes the improvements:

- Booster Pump Improvements
  - Elena Booster Pump Station (Project PWPS-1): To mitigate the existing capacity deficiency of Elena BPS under PHD conditions, it is recommended that the existing pumps be replaced. The existing pumps would be replaced with two 0.5 mgd booster pumps.
  - Kings Booster Pump Station (Project PWPS-2): To provide additional flow to Lower Kings zone under MDD plus fire flow conditions, it is recommended that a 2.16 mgd fire flow booster pump be added to Kings BPS.
- Storage Improvements
  - Nutmeg Tank (Project PWS-1): In order to provide additional storage for Nutmeg zone, it is recommended the City demolish the existing Nutmeg Tank and construct a new 1.0 MG tank.
  - Elena Tank (PWS-2): In order to provide more storage to the Elena zone, it is recommended the City demolish the two existing Elena Tanks and construct a new 0.15 MG tank.
- Fire Flow Improvements. A number of deficient fire flow nodes (residual pressures below 20 psi) were identified as part of the fire flow capacity analysis. To mitigate these deficiencies, recommendations for pipeline and PRV improvements were developed. These improvements generally consist of replacement of smaller diameter (≤6 inches) pipelines with larger (8- to 16-inch diameter pipelines. In total, 29,700 linear feet (5.6 miles) of fire flow improvements are recommended.
- Transmission/Velocity Improvements
  - Fill Line for Blanca Tanks (Project PWP-1): In order to properly fill the Blanca tanks under MDD conditions, it is recommended that approximately 2,350 feet of 4-inch diameter pipeline from the Blanca Tanks site to Vashon BPS be replaced with an 8-inch diameter pipeline.
  - Fill Line for Nutmeg Tank (Project PWP-2): To mitigate an existing headloss deficiency, it is recommended that 1,280 feet of 4-inch diameter pipeline from Elena BPS to the Nutmeg tank site be replaced with an 8-inch diameter pipeline.
  - Fill Line for Elena Tanks (Project PWP-3): It is recommended that approximately 1,910 feet of 10-inch diameter pipeline be added to serve as a feed line for the Elena tanks.
  - Parallel Line on Juniper Avenue (Project PWP-4): It is recommended that a parallel 8-inch diameter pipeline be constructed to convey water from Nutmeg pressure zone to Blanca pressure zone.
  - Pipeline on Sequoia Street (Project PWP-5): It is recommended that a parallel 8-inch diameter pipeline be constructed along Sequoia Street, between Dogwood Avenue and Main Street in order to supply flow from Nutmeg zone to Blanca zone.
- PRV Station Improvements
  - PRV on Juniper Avenue (Project PWV-1): It is recommended that a new PRV station be constructed to break Nutmeg zone and Blanca zone along the proposed parallel line along Juniper Avenue (Project PWP-4).

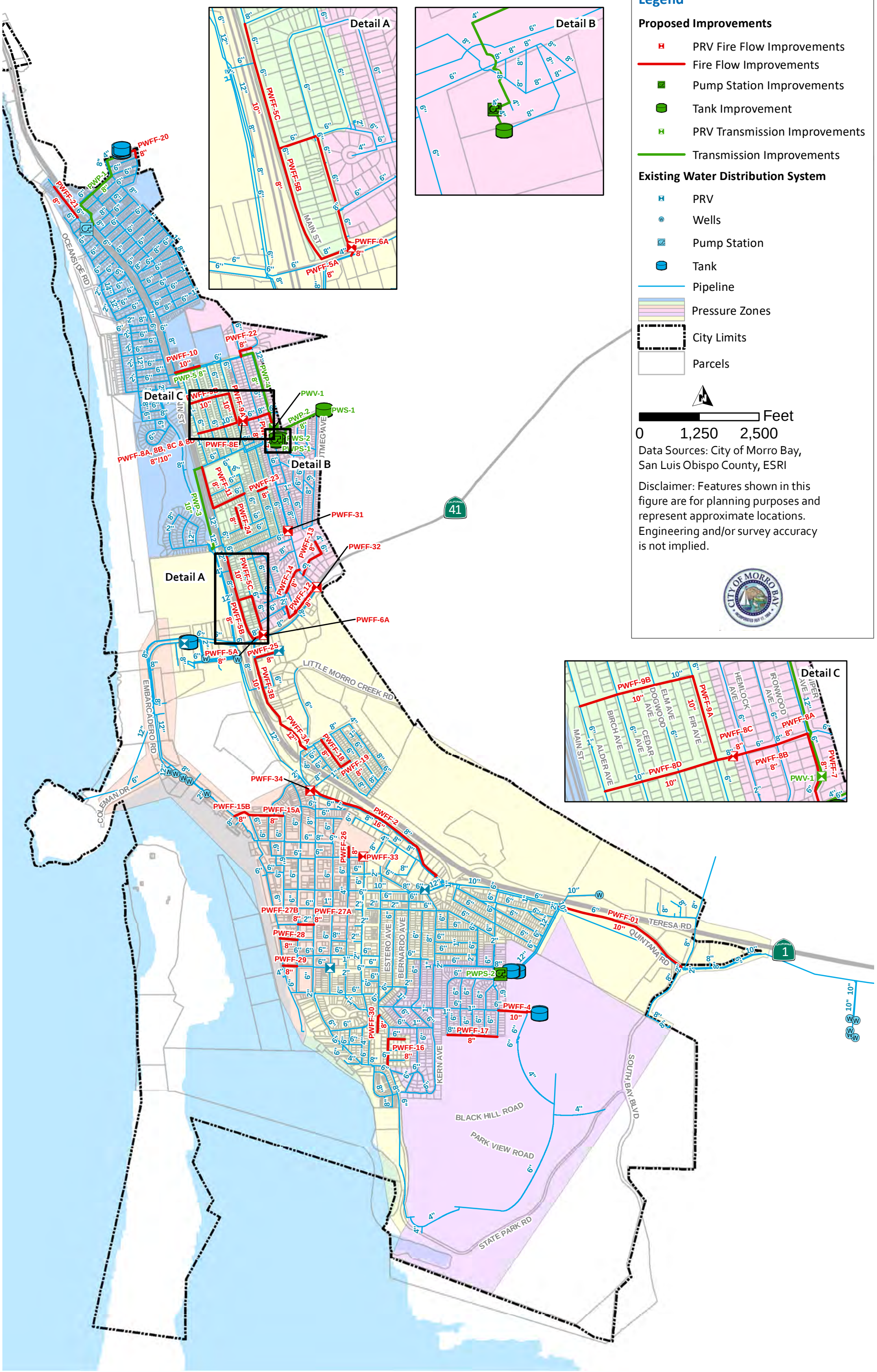
- Other Improvements
  - Water Master Plan Update (PWO-1): It is recommended that City update its water master plan every five years.
  - Re-Skin Desalination Plant Building (PWO-2): It is recommended that the existing Desalination building be re-skinned.
  - Control Upgrades at Desalination Plant (PWO-3): It is recommended that the controls at the Desalination plant be upgraded.
  - Add Screening at Desalination Plant (PWO-4): It is recommended that screening be added at the calcium carbonate tanks at the Desalination plane.
  - Chlorination Upgrades (PWO-5): This project includes chlorination upgrades at the Desalination plant and at the Kings tanks.
  - Desalination Plant Relocation (PWO-5): This project includes the relocation of the Desalination plant. The cost for this project is not defined in OneWater as there is not enough information to determine an accurate cost estimate. Cost for this project will be determined in the next master plan update.

ES.8.2 Wastewater Collection System Evaluation and Proposed Improvements

Following the dry and wet weather flow calibration, a capacity analysis of the existing and future collection system was performed. The capacity analysis entailed identifying areas in the sewer system where flow restrictions occur or where pipe capacity is insufficient to convey PWWFs. Sewers that lack sufficient capacity to convey PWWFs create bottlenecks in the collection system that can potentially cause sanitary sewer overflows (SSOs).

Figure ES.4 illustrates the recommended improvements to mitigate the collection system deficiencies. The following summarizes the improvements:

- Gravity Main Improvements
  - Gravity Main along Atascadero Road (Project WWGM-1): This project includes the replacement of approximately 1,000 feet of 18-inch diameter pipeline along Atascadero Road, between Main Street and Park Street. It is recommended that the existing pipeline be replaced with a 27-inch diameter pipeline.
  - Gravity Main along Main Street (Project WWGM-2): This project includes the replacement of approximately 2,900 feet 15-inch diameter pipeline along Main Street, between Atascadero Road and Las Vegas Street. It is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.
  - Gravity Main along Main Street (Project WWGM-3): This project includes the replacement of approximately 1,600 feet of 12-inch diameter pipeline along Main Street, between Las Vegas Street and San Joaquin Street. It is recommended that the existing pipeline be replaced with an 18-inch diameter pipeline.
  - Gravity Main along San Joaquin Street (Project WWGM-4A): This project includes the replacement of 6-inch diameter pipelines approximately 150 feet along San Joaquin Street, between Main Street and Alder Avenue. It is recommended that the existing pipeline be replaced with 12-inch diameter pipelines.



Legend

Proposed Improvements

- PRV Fire Flow Improvements
- Fire Flow Improvements
- Pump Station Improvements
- Tank Improvement
- PRV Transmission Improvements
- Transmission Improvements

Existing Water Distribution System

- PRV
- Wells
- Pump Station
- Tank
- Pipeline
- Pressure Zones
- City Limits
- Parcels

Feet

0 1,250 2,500

Data Sources: City of Morro Bay, San Luis Obispo County, ESRI

Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure ES.3 Water Distribution System Improvements

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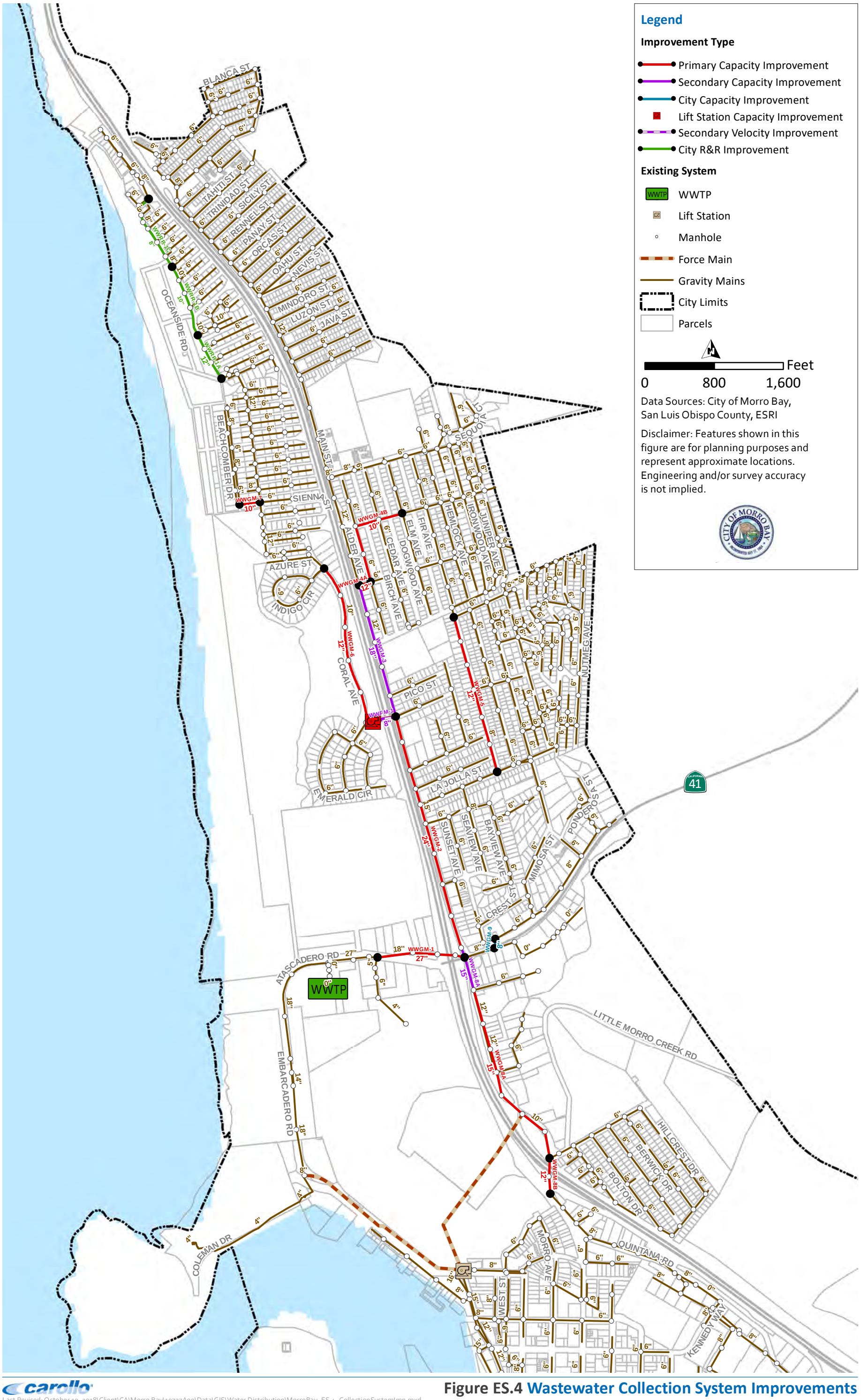

Figure ES.4 Wastewater Collection System Improvements

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  - Gravity Main along Alder Avenue, and San Jacinto Street (Project WWGM-4B): This project includes the replacement of 6-inch diameter pipelines approximately 650 feet along Alder Avenue, between San Joaquin Street and San Jacinto Street and 550 feet along San Jacinto Street, between Alder Avenue and Elm Avenue. It is recommended that the existing pipeline be replaced with 10-inch diameter pipelines.
  - Gravity Main along Greenwood Avenue (Project WWGM-5): This project includes the replacement approximately 1,850 feet 6-inch diameter pipeline along Greenwood Avenue, between Avalon Street and Elena Street. It is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
  - Gravity Main along Coral Avenue (Project WWGM-6): This project includes the replacement of approximately 1,900 feet of 10-inch diameter pipeline along Coral Avenue, between Lift Station 1 and Azure Street. It is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
  - Gravity Main along Sienna Street (Project WWGM-7): This project includes the replacement of approximately 250 feet 6-inch diameter pipeline along Sienna Street, between Sandalwood Avenue and Beachcomber Drive. It is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.
  - Gravity Main along Main Street (Project WWGM-8A): This project includes the replacement of approximately 2,600 feet of 10-inch and 12-inch diameter pipeline along Main Street, between Radcliff Avenue and Atascadero Road. It is recommended that the existing pipeline be replaced with 15-inch diameter pipelines.
  - Gravity Main along Main Street (Project WWGM-8B): This project includes the replacement of approximately 400 feet of 8-inch and 10-inch diameter pipeline along Main Street south of Radcliff Avenue. It is recommended that the existing pipeline be replaced with 12-inch diameter pipelines.
  - Gravity Main under Highway 41 (Project WWGM-9): City Staff indicated that there is a belly in the existing 6-inch diameter main that crosses Highway 41, which has created ongoing maintenance issues. It is recommended that the existing 6-inch diameter pipe crossing under Highway 41 be replaced with a new 8-inch diameter main. This pipeline would be installed in a 20-inch diameter casing.
- Lift Station Improvements
  - Lift Station 1 Replacement (WWLS-1): This project includes replacing Lift Station 1. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. It is recommended that the new lift station have a firm capacity of 1.10 mgd and a total capacity of 1.65 mgd.
  - Lift Station 1 Force Main (WWFM-1): The purpose of this project is to mitigate the high velocity (> 8 fps) that the existing force main experiences following WWLS-1 under existing conditions. It is recommended that an 8-inch diameter force main be constructed parallel to the existing 6-inch diameter force main.
- Rehabilitation and Replacement Projects
  - Gravity Main along Beachcomber Drive (Project WWRR-1A, 1B, and 1C): The area upstream of Lift Station 1 was shown to have the highest I/I rates in the City. The City suspects that a large contributor to this problem is the gravity main installed on Beachcomber Drive, which is known to contribute to I/I in the collection system. This

project includes the rehabilitation or replacement of approximately 570 feet of 12-inch diameter pipeline along Beachcomber Drive between Java Street and north of Mindoro Street, rehabilitation or replacement of approximately 960 feet of 10-inch diameter pipeline along Beachcomber Drive between north of Mindoro Street and north of Sicily Street, and rehabilitation or replacement of approximately 950 feet of 8-inch diameter pipeline along Beachcomber Drive between north of Sicily Lane and Toro Lane.

  - Pipe R&R Program Upstream of LS-1 (Project WWRR-2): This project includes the rehabilitation or replacement of pipelines that are upstream of Lift Station 1 and not already a recommended improvement project (e.g., Project WWRR-1A-1C). This equates to approximately 3,000 feet of pipeline projects between 2020 through 2028.
  - I/I Reduction Projects (Project WWRR-3): This project includes the rehabilitation or replacement of pipelines and structures that contribute significant amount of I/I. Inspection data will be used to prioritize projects and determine best rehabilitation method or replacement. An annual budget estimate of $100,000 per year has been assumed for rehabilitation and replacement through the year 2040.
  - Cap and Plug Replacement Upstream of Lift Station 1 (WWRR-4): This Project consists of replacing caps and/or plugs upstream of Lift Station 1 with manholes. The caps upstream of Lift Station 1 are known to contribute high amounts of inflow to the system. This project consists of replacing 31 caps with manholes between 2019 and 2020.
  - Cap and Plug Replacement Program (WWRR-5): This project consists of replacing the remaining cap and/or plug structures in the collection system with manholes. These projects do not include caps and plugs replaced upstream of Lift Station 1. This project consists of replacing 121 caps with manholes.
- Other Improvements
  - Collection System Master Plan Update (WWO-1): It is recommended that City update its collection system master plan every five years.
  - Flow Monitoring Program (WWO-2): It is recommended that a temporary flow monitoring program be conducted every five years in conjunction with the collection system master plan update.

ES.8.3 Storm Drainage System Evaluation and Proposed Improvements

When evaluating the adequacy of the City of Morro Bay storm drainage facilities, the City's storm drains were allowed to surcharge and provide additional storage capacity, thus reducing the number of storm drain improvements. For the 10 year design event, the maximum HGL was not allowed to exceed the ground level (i.e., no flooding permitted). The storm drainage criteria allowed flooding up to one foot above the gutter flow line in the 100-year design storm event.

The capacity analysis entailed identifying areas in the storm drainage system where flooding exceeded the maximum allowable flow depth (i.e., ground level for the 10-year event, 1-foot above the gutter for the 100-year event). Storm drains that lack sufficient capacity to convey runoff generated from the design storm could produce backwater effects in the drainage system and potentially cause flooding.

Figure ES.5 illustrates the recommended improvements to mitigate the storm drainage system deficiencies. This following provides a detailed description of each recommended improvement project:

- *Redirect a Portion of Runoff from Roundabout Area (SD-1A and 1B):* This project is recommended to redirect an area of approximately 86-acres that currently runs off to the roundabout area. It is recommended that approximately 125-feet of 27-inch and 2,600 feet of 30-inch diameter storm drains be constructed along Anchor Street from Kern Avenue to Main Street.
- Marina Street Storm Drain (SD-2A and 2B): There is a 6-inch bleeder line on Marina Street at Shasta, as well as some additional culverts on Marina Street that convey flow to downstream gutters. To improve system hydraulics in the area, it is recommended that approximately 370-feet of 12-inch and 600-feet of 15-inch diameter storm drain pipe be installed along Marina Street from Piney Way to Monterey Avenue.
- Harbor Street Storm Drain (SD-3): This project includes abandoning the existing 24-inch diameter pipeline along Main Street between Harbor and Main Street, and the addition of approximately 890 feet of 24-inch diameter pipeline along Harbor Street, between Main Street and Centennial Parkway. This project is recommended to mitigate flooding along Morro Bay Boulevard. The new pipeline will redirect flow to Harbor Street, which has enough capacity to convey the additional runoff.
- Roundabout Storm Drain Replacement (SD-4): This project, in conjunction with project SD-1A and 1B, will ease flooding in the roundabout area. This project includes the replacement of approximately 130 feet of 12-inch diameter pipelines along the north side of the roundabout at Morro Bay Boulevard and Quintana Road with an 18-inch diameter storm drain.
- Beach Street Storm Drain (SD-5): There is a 6-inch bleeder line on Beach Street near Morro Avenue that takes runoff from a 36-inch storm drain and discharges to a downstream gutter. To improve system hydraulics in this area, it is recommended that approximately 1,000 feet of 24-inch diameter pipeline along Beach Street, between Morro Avenue and Embarcadero Road be constructed.
- Bike Path Storm Drain (SD-6A and 6B): Willow Camp Creek crosses the City's bike path west of Main Street through a very small 1' x 3' box drain. This box drain floods routinely during wet weather, causing the bike path to be closed. This project would replace the box drain and extend a new storm drain line farther downstream. To mitigate the bike path flooding, a new 48-inch and 54-inch triple barrel storm drain is recommended. Approximately 3,500 feet of triple barrel storm drains are recommended.
- Main Street Storm Drains (SD-7, SD-8, and SD-31): These projects are required to mitigate flooding along Main Street upstream of Willow Camp Creek. The recommended improvements consist of replacing the existing storm drains in the area with 15-inch, 18-inch, 24-inch, and 27-inch diameter storm drains.
- Morro Creek Area Improvements (SD-9, SD-10, SD-24, SD-25, SD-27, and SD-28): These projects are needed to mitigate flooding in scattered areas in the Morro Creek drainage area. The existing storm drains in these areas will be replaced with 15-inch, 24-inch, and 36-inch diameter storm drains.
- Avalon and Hillview Street Area Storm Drains (SD-11, SD-12, SD-13, SD-21, SD-22, SD-23, SD-26, SD-29, SD-30, and SD-34): These projects are needed to mitigate flooding in

scattered areas in the near Avalon Street. The existing storm drains in these areas will be replaced with 12-inch, 24-inch, 30-inch, 36-inch, and 42-inch diameter storm drains.

- Sequoia Street/Sienna Street Storm Drains (SD-14, SD-17, and SD-18): The hydraulic model showed some flooding near Sienna Street/Sandlewood Avenue and Sandlewood Avenue/Bali Street, as well as the Main Street/Sequoia Street area. The existing storm drains in these areas will be replaced with 18-inch, 24-inch, 30-inch, and 36-inch diameter storm drains.
- Morro Strand Campground Storm Drain (SD-15): Orcas Creek discharges into a 16" by 42" box drain that flows underneath the Morro Strand Campground. The hydraulic model showed that the box drain does not have enough capacity to convey the 10-year design storm runoff. It is recommended that the existing box drain be replaced with a 48-inch diameter pipeline.
- Elena Street Storm Drains (SD-16, SD-19, and SD-20): These projects are recommended to mitigate simulated flooding upstream of Highway 1 on Elena Street. The existing storm drains in these areas will be replaced with 30-inch and 36-inch diameter storm drains.
- Surf Street and West Avenue Storm Drain (SD-32A and SD-32B): This project includes the replacement of approximately 360 feet of 12-inch and 18-inch diameter pipeline in the parking lot located west of the intersection of Surf Street and West Avenue with a new 18-inch and 24-inch pipeline.
- Main Street/Morro Bay Boulevard Storm Drain (SD-33): This project includes the replacement of approximately 140 feet of 24-inch diameter pipeline located east of the roundabout at Morro Bay Boulevard and Quintana Road with a 30-inch diameter pipeline.
- Main Street/Highway 1 Storm Drain (SD-35): This project includes the addition of approximately 270 feet of 15-inch diameter pipeline along Main Street, between Quintana Place and Cabrillo Highway. In this area, the flow travels over the street crown and crosses over Main Street. To fix this issue, it is recommended that this 15-inch diameter pipeline be added.
- Morro Bay Boulevard Off Ramp Storm Drain (SD-36): This project includes the replacement of approximately 150 feet of 24-inch diameter pipeline near the Morro Bay Boulevard off ramp that goes under Highway 1. This pipe has failed, and currently has no bottom. It is recommended that the existing failed pipeline be replaced with a 24-inch diameter pipeline.
- Rehabilitation and Replacement Projects (SDRR-1): This project includes the rehabilitation or replacement of failing storm drains. An annual budget estimate of $100,000 per year has been assumed for rehabilitation and replacement through the year 2040.
- Storm Drainage System Master Plan Update (SDO-1): It is recommended that City update its storm drain master plan every five years.

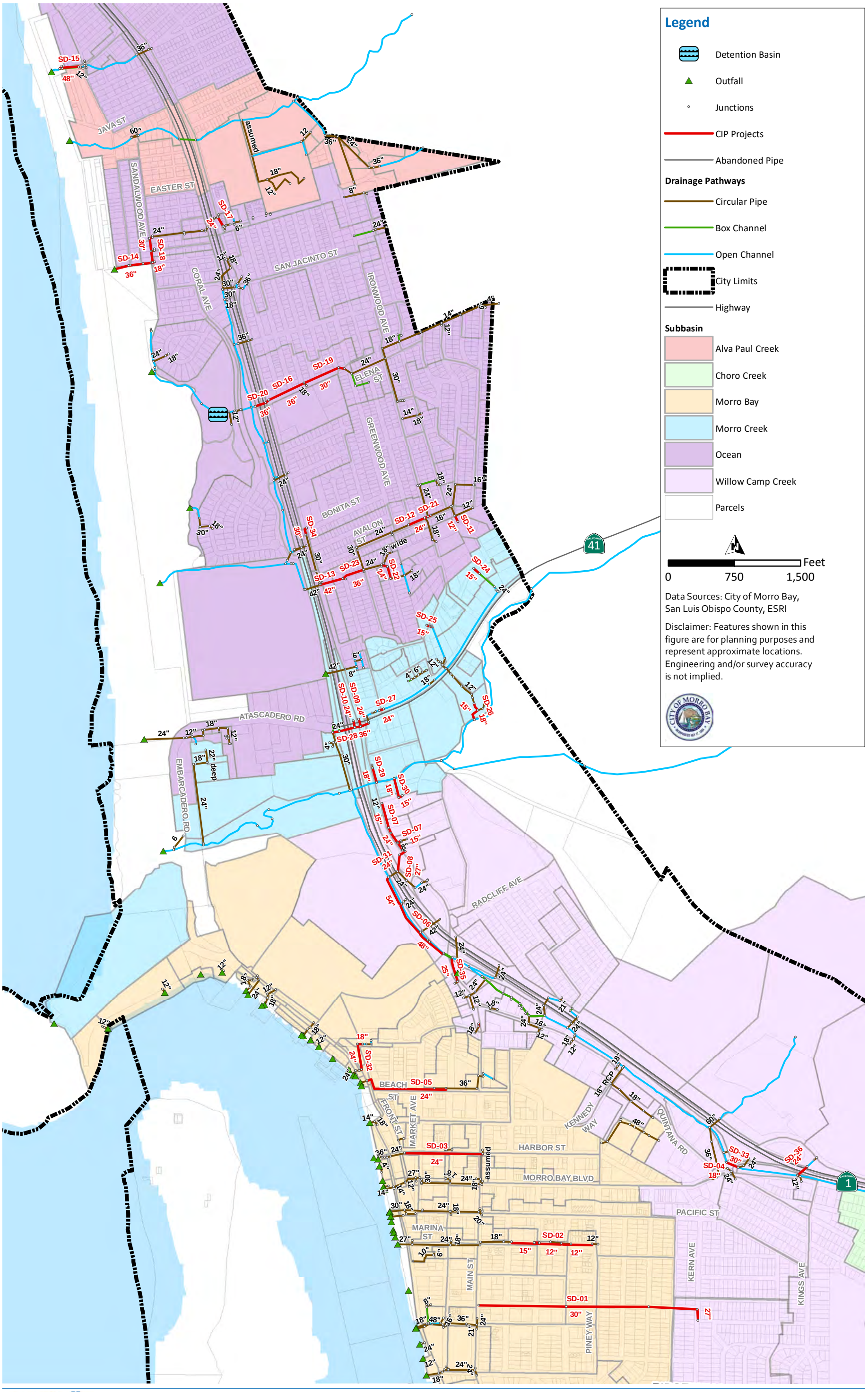

Figure ES.5 Storm Drain System Improvements

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ES.9 Water Supply Evaluation

The water supply evaluation alternative analysis for Morro Bay's potable water system presents six main water supply alternatives with sub-alternatives identified related to treatment. The alternatives presented incorporate both existing and proposed water supply production and treatment facilities operated by the City while including improvements to increase reliability and quality to the existing water supply. The goal of this analysis is to identify reliable, drought-tolerant supplies sufficient to provide for the City through 2050. Table ES.5 displays the water supply evaluation alternatives considered in this analysis. These alternatives are supplementary in unit cost and energy to the City's existing Water Reclamation Facility as well as the City's existing allocation of imported State Water. It was assumed that if a water supply alternative was not able to supply the City's 2050 demand, it was supplemented with imported State Water.

Table ES.5 Water Supply Alternatives

Project No.	Project Name
1	State Water Project
2A	Morro Well Field With Salinity Treatment
2B	Morro Well Field with Nitrate Treatment
2C	Morro & Chorro Well Fields with Salinity Treatment
2D	Morro & Chorro Well Fields with Nitrate Treatment
2E	Morro & Chorro Well Fields with Nitrate & Salinity Treatment
3A	Chorro Well Fields with Streamflow Augmentation
3B	Chorro Well Fields with Streamflow Augmentation & Nitrate Treatment
4A	Morro Well Field IPR by Groundwater Injection
4B	Morro Well Field IPR by Groundwater Injection with Salinity Treatment
4C	Morro Well Field IPR by Groundwater Injection with Nitrate Treatment
5	Ocean Desalination
6	Direct Potable Reuse

Each water supply alternative was evaluated with respect to criteria that reflected Morro Bay's city-wide interests. Table ES.6 displays the evaluation criteria used in this analysis. The evaluation considers nine separate criteria to assess how each water supply alternative compares to the existing system supply and operation as well as future infrastructure improvements. Every alternative was given a numerical score from 1 to 5 for each criteria with the sum of the evaluation criterion scores multiplied by a weighting factor was then used to rank alternatives to one another.

Table ES.6 Water Supply Evaluation Criteria

Evaluation Criteria	
Unit Cost	Drought Resiliency
Earthquake Resiliency	Flood Risk Mitigation
Local Supply Benefit	Energy Requirements
Constructability	Regulatory and Institutional Complexity
Quality Consistency	

Capital, energy, and O&M costs and scores for the individual evaluation criterion associated with the water supply alternatives are presented in Chapter 7. The findings of the water supply evaluation display that criteria including drought resiliency, earthquake resiliency, flood vulnerability, regulatory and institutional complexity and quality consistency remain constant between alternatives. The variability in scores is seen mainly in the unit cost, drought resiliency, local supply offset, energy requirements, and constructability criteria. It was found that, given the variability of water supply scenarios the City of Morro Bay may potentially face, Carollo recommends the City move forward with Alternative 4A. Alternative 4A would construct the advanced treatment facilities as part of the WRF project to facilitate IPR within the Morro groundwater basin and utilize the City's existing BWRO treatment facilities upon groundwater extraction. Additional implementation next steps are also included in Chapter 7 as a recommendation to strengthen City's existing water resources.

ES.9.1 Implementation and Next Steps

As a result of the recommendation to move forward with Alternative 4A to strengthen the City's water supply, the following implementation recommendations are being made for the City's other water resources:

ES.9.1.1 Indirect Potable Reuse

- In order to better understand treatment that may be required following extraction, the City should immediately move forward with additional hydrogeology work to inform the injection and extraction process.
- When State Water is available, the City should use IPR as a back-up supply and to help improve the quality in the Morro groundwater basin (i.e., flushing of nitrates).
- An IPR facility could offer the design flexibility to be modified to DPR operation in the future with the addition of new treatment processes (i.e., granular activated carbon vessels, engineered storage buffer, advanced monitoring, etc.) to the existing facility. DPR regulations are not anticipated from DDW until 2023, but the draft guidelines indicate that it will be preferred that a DPR facility is first permitted and operated as an IPR facility.

ES.9.1.2 State Water

- As the lowest cost supply source, the City should continue to maximize its use of State Water as available in the short term.
- Once the IPR process is operational, the City should discontinue purchase of the State Water "drought buffer" since the water injected into the Morro groundwater basin will serve as the City's back-up supply when State Water is not available. It is anticipated that the cost for drought buffer could increase by as much as $400/AF due to the impacts from the California Water Fix.
- The City should continue to monitor DWR's exchange policies and continue to engage with the San Luis Obispo County Flood Control and Water Conservation District to identify opportunities for generating revenue from unused State Water allocations.

ES.9.1.3 Ocean Desalination

- Since the desalination facility has reached the end of its useful life, the City should de-energize, pull pumps, pull wires, and secure the seawater wells. While it is not anticipated that the facility will be operated, removal of the wells could impact the City's existing desalination permit.

ES.9.1.4 Chorro Groundwater Basin

- The City spends approximately $40,000 per year for Chorro Creek stream flow monitoring and sampling and maintenance for the wells in the Chorro groundwater basin. Securing the water rights in the basin would also require significant capital investment from construction of a new treatment and pumping pipeline and permanent stream flow gauges.
- Since the Chorro groundwater basin has the lowest priority of any of the City's available water resources, Carollo recommends the City discontinue the existing activities at the Chorro well field.

ES.10 Capital Improvement Plan

This section presents the City's capital improvement plan (CIP), a summary of the capital costs, and a basic assessment of the possible financial impacts on the City. The CIP is based on the evaluation of the City's water distribution system, wastewater collection system and storm drainage system, as described in Chapter 6 of this OneWater Plan.

ES.10.1 20-Year CIP

The proposed capital improvements are prioritized based on their urgency to mitigate existing deficiencies and other factors. The capital improvements were phased into one of the following phases:

- *Near Term Projects Phase 1 (Year by Year, 2019-2023):* This phase includes projects that are targeted as the highest priority improvements.
- *Near Term Projects Phase 2 (2024-2028):* This phase generally includes medium priority improvements.
- *Long Term Projects (2029-2040):* This phase includes lower priority improvements.

The 20-year water distribution system CIP is summarized by phase and project type in Table ES.7. As shown in Table ES.7, out of the total $26.1 million in capital projects, $14.9 million are targeted for implementation in the Near Term Phase 1, and an additional $8.0 million are targeted for the Near Term Phase 2. The remaining $3.2 million of capital improvements has been included in the Long Term Phase (2029-2040). It should be noted that the potential cost of moving the water treatment plant is not included in Table ES.7. The costs associated with this project will be developed as part of a future study.

Table ES.7 and Figure ES.6 show the distribution of capital costs by project type. As shown on Figure ES.6, fire flow improvement projects and storage reservoir projects account for the largest portions of the capital improvement project costs at 50 percent and 17 percent, respectively. Other projects, transmission and distribution main projects, rehabilitation and replacement projects, and PRV stations account for roughly 12 percent, 11 percent, 9 percent, and 1 percent of the total CIP costs, respectively.

Table ES.7 20-Year Water System CIP Summary

Improvement Type	CIP Cost Estimate[(1),(2),(3),(4)] ($)	Project Cost Phasing ($)		
		Near Term Ph. 1 2019-23	Near Term Ph. 2 2024-28	Long Term 2029-40[(5)]
Capacity Improvements				
Fire Flow	$12,342,000	$3,293,000	$7,357,000	$1,692,000
Transmission and Distribution	$2,658,000	$2,658,000	-	-
Storage Reservoir	$4,352,000	$4,352,000	-	-
PRV Stations	$312,000	$312,000	-	-
Capacity Subtotal	$20,990,000	$11,941,000	$7,357,000	$1,692,000
Rehabilitation and Replacement Projects				
Rehabilitation and Replacement Projects	$2,200,000	$500,000	$500,000	$1,200,000
Other Projects				
Other Projects	$2,895,000	$2,445,000	$150,000	$300,000
Total CIP	**$26,085,000**	**$14,886,000**	**$8,007,000**	**$3,192,000**

Notes:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889
(2) Estimated Construction Cost includes a 30% contingency of the baseline construction cost.
(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.
(4) Total Mark-Up is 65.8% of the baseline construction costs.
(5) The cost associated with relocating the water treatment plant was assumed to be zero for the purposes of this Master Plan. Costs associated with this project will be determined in the future.
(6) Figure ES.6 20-Year Water System CIP by Project Type
(7) Figure ES.7 20-Year Water System CIP by Project Type

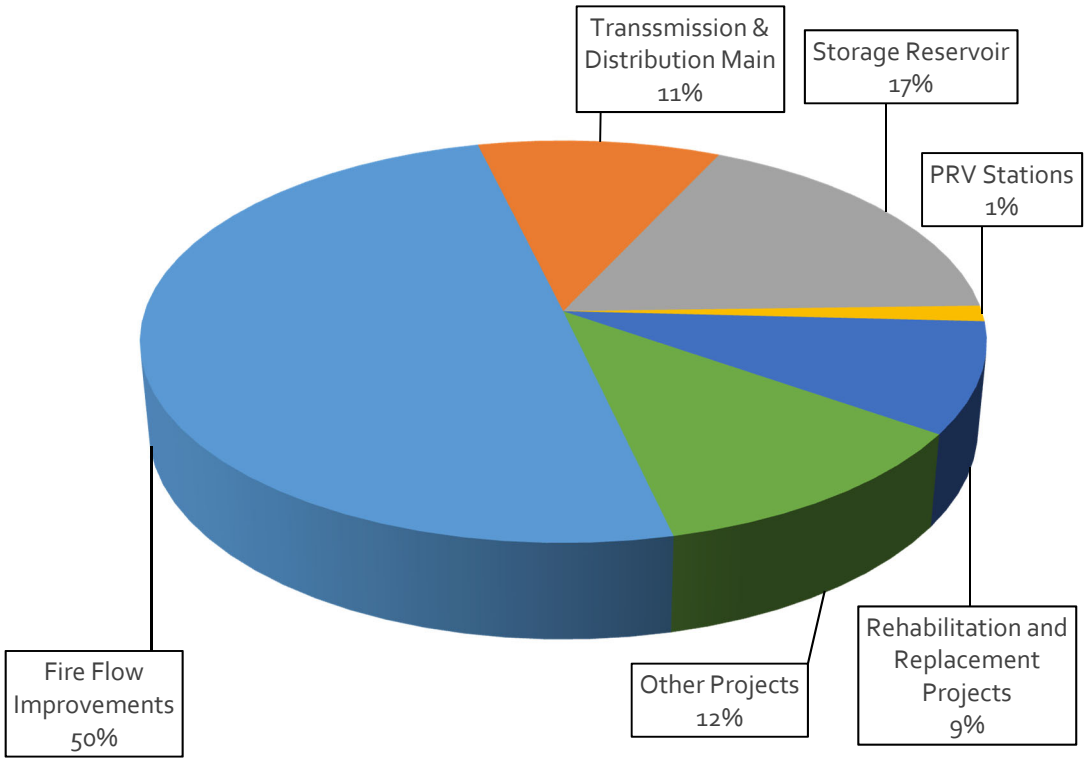

Figure ES.6 20-Year Water System CIP by Project Type

The 20-year wastewater collection system CIP is summarized by phase and project type in Table ES.8. As shown in Table ES.8, out of the total $18.2 million in capital projects, $7.8 million are targeted for implementation in the Near Term Phase 1, and an additional $4.7 million are targeted for the Near Term Phase 2. The remaining $5.7 million of capital improvements has been included in the Long Term Phase (2029-2040).

Table ES.8 and Figure ES.7 show the distribution of capital costs by project type. As shown on Figure ES.7, gravity main rehabilitation and replacement and gravity main capacity upgrades account for the largest portions of the capital improvement project costs at 46 percent and 27 percent, respectively. End cap replacement projects, lift stations, other projects, and force mains account for roughly 14 percent, 8 percent, 5 percent, and less than 1 percent of the total CIP costs, respectively.

Table ES.8 20-Year Wastewater Collection System CIP Summary

Improvement Type	CIP Cost Estimate[(1),(2),(3),(4)] ($)	Project Cost Phasing ($)		
		Near Term Ph. 1 2019-23	Near Term Ph. 2 2024-28	Long Term 2029-40[(5)]
Capacity Improvements				
Fire Flow	$4,994,000	$3,509,000	$933,000	$552,000
Transmission and Distribution	$1,439,000	-	-	$1,439,000
Storage Reservoir	$80,000	-	-	$80,000
Capacity Subtotal	$6,513,000	$3,509,000	$933,000	$2,071,000
Rehabilitation and Replacement Projects				
Rehabilitation and Replacement Projects	$8,298,000	$3,588,000	$3,510,000	$1,200,000
End Cap Replacement	$2,520,000	$514,000	-	$2,006,000
Rehabilitation and Replace Subtotal	$10,818,000	$4,102,000	$3,510,000	$3,206,000
Other Projects				
Other Projects	$840,000	$210,000	$210,000	$420,000
Total CIP	**$18,171,000**	**$7,821,000**	**$4,653,000**	**$5,697,000**

Notes:
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(4) Total Mark-Up is 65.8% of the baseline construction costs.

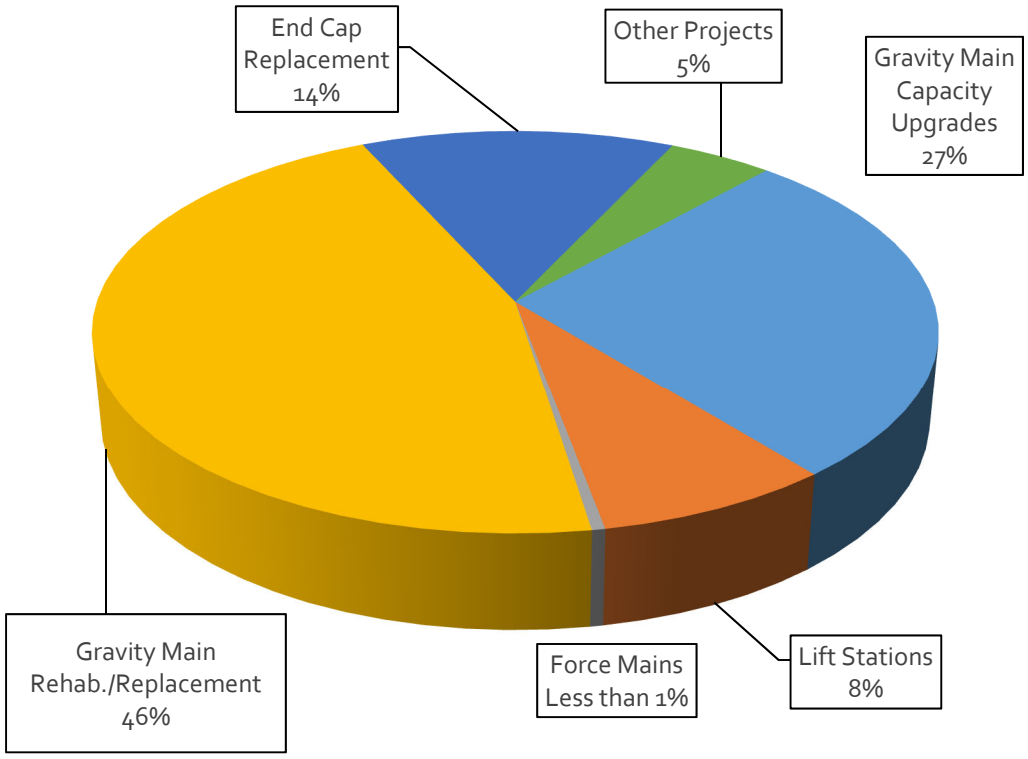

Figure ES.7 20-Year Wastewater Collection System CIP by Project Type

The 20-year storm drainage system CIP is summarized by phase and project type in Table ES.9. As shown in Table ES.9, out of the total $13.6 million in capital projects, $7.8 million are targeted for implementation in the Near Term Phase 1, and an additional $2.2 million are targeted for the Near Term Phase 2. The remaining $3.7 million of capital improvements has been included in the Long Term Phase (2029-2040).

Table ES.9 and Figure ES.8 show the distribution of capital costs by project type. As shown on Figure ES.8, projects needed to mitigate the 10-year design storm deficiencies account for the largest portion of the capital improvement project costs at 62 percent. Projects needed to mitigate the 100-year design storm, rehabilitation and replacement of failing storm drains, and other projects account for roughly 18 percent, 16 percent, and 4 percent of the total CIP costs, respectively.

Table ES.9 20-Year Storm Drainage System CIP Summary

Improvement Type	CIP Cost Estimate[(1),(2),(3),(4)] ($)	Project Cost Phasing ($)		
		Near Term Ph. 1 2019-23	Near Term Ph. 2 2024-28	Long Term 2029-40[(5)]
Projects to Mitigate 10-Year Storm Deficiencies	$8,405,000	$7,130,000	$1,275,000	$-
Projects to Mitigate 100-Year Storm Deficiencies	$2,442,000	$-	$253,000	$2,189,000
Rehabilitation and Replacement Projects	$2,200,000	$500,000	$500,000	$1,200,000
Other Projects	$600,000	$150,000	$150,000	$300,000
Total CIP	**$13,647,000**	**$7,780,000**	**$2,178,000**	**$3,689,000**

Notes:
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(4) Total Mark-Up is 65.8% of the baseline construction costs.

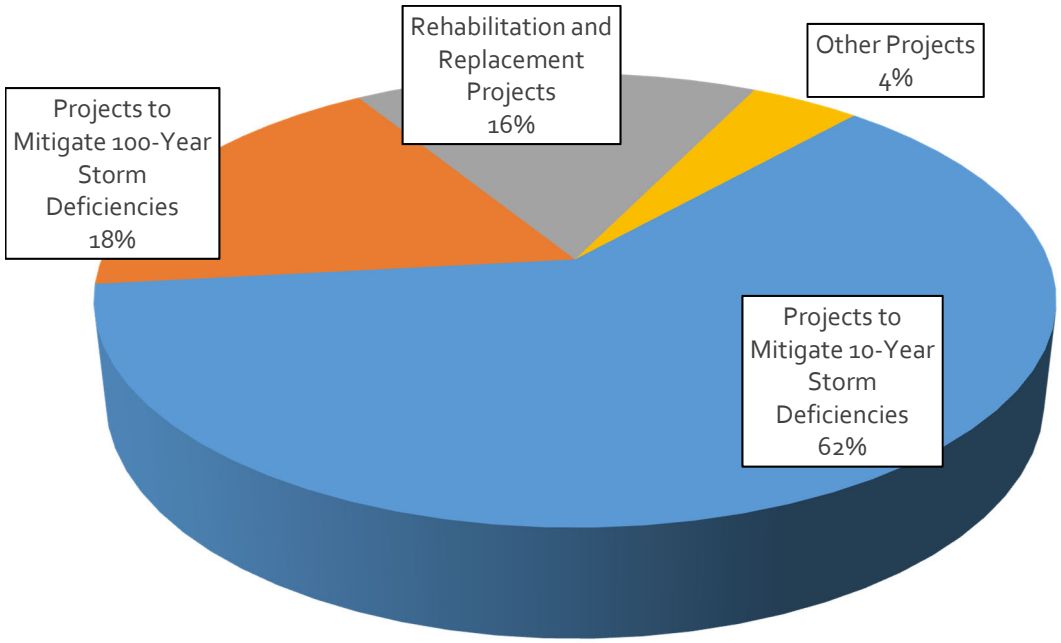

Figure ES.8 20-Year Storm Drainage System CIP by Project Type

It should be noted that the CIP phasing included in the 20-year CIP represents the preferred implementation schedule for the proposed improvements. Funding availability will limit the City's ability to implement the proposed projects. Carollo worked with the City's rate study consultant to develop a 5-year CIP (2019-2023) to develop an implementable rate schedule for

the water distribution system and wastewater collection systems. These 5-year CIPs are described in the next section.

ES.10.2 Revised 5-Year CIP

On April 23, 2018, the City appointed four citizens from Morro Bay to a Blue Ribbon Commission (BRC) to participate in the development of new rates for the Water Reclamation Facility (WRF) project. The BRC's mission statement was: "The WRF Blue Ribbon Commission is committed to independently evaluating the costs of the major components of the WRF program and water and sewer capital project needs and provide a recommendation to the City Manager for equitable and reasonable customer rates to pay for those needs."

Over the course of several months, the City, Carollo, BRC and the City's rate consultant held eight (8) meetings to review the costs for the WRF, the CIP projects, and current and future operations and maintenance costs to minimize the impacts of any new rates on the community. One of the BRC's primary activities was review of the 20-year CIPs described in the previous section. Table ES.10 and Table ES.11 show the revised 5-year CIPs for the water distribution and wastewater collection systems that were modified based on discussions with the BRC. These revised CIPs prioritize the most critical projects for the water distribution and wastewater collection systems such as the replacement of the Nutmeg Tank and the replacement of the gravity main on Main Street from San Joaquin Street to Atascadero Road, respectively. The findings of the BRC are summarized in their report dated June 21, 2018. It should be noted that while the 5-year CIPs have been revised, the overall cost and scope of the 20-year CIP is unchanged. These projects in the 5-year CIP were included in the City's rate study for the determination of water and sewer rates. As shown in Table ES.10 and Table ES.11, the City has included approximately 6.8 million dollars and 5.1 million dollars in the 5-year water and wastewater collection system CIPs, respectively.

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Table ES.10 5-Year Revised Water System CIP

Pump Station		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[1][2][3][4] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($)				
								Near-Term				
								2019	2020	2021	2022	2023
Capacity Improvements					$ 7,115,000	$ 7,060,000	$ 55,000	$ 904,000	$ 927,000	$ 1,224,000	$ 1,697,000	$ 1,326,000
Transsmission & Distribution Main		Diameter (in)	Diameter (in)	Length (ft)	$ 1,870,000	$ 1,833,000	$ 37,000	$ 904,000	$ 595,000	$ -	$ 371,000	$ -
PWP-1	Fill line for Blanca Tanks	4	8	2,210	$ 750,000	$ 735,000	$ 15,000	$ 750,000	$ -	$ -	$ -	$ -
PWP-2	Fill line for Nutmeg Tank	4	8	1,280	$ 371,000	$ 364,000	$ 7,000	$ -	$ -	$ -	$ 371,000	$ -
PWP-4	Parallel pipeline on Juniper Avenue	--	8	2,050	$ 595,000	$ 583,000	$ 12,000	$ -	$ 595,000	$ -	$ -	$ -
PWP-5	Pipeline on Sequoia Street	--	8	530	$ 154,000	$ 151,000	$ 3,000	$ 154,000	$ -	$ -	$ -	$ -
Pump Station		Capacity (mgd)	Capacity (mgd)	Length (ft)	$ 580,000	$ 568,000	$ 12,000	$ -	$ -	$ 580,000	$ -	$ -
PWPS-1	Elena Booster Pump Station Upgrade	0.44	0.50	--	$ 580,000	$ 568,000	$ 12,000	$ -	$ -	$ 580,000	$ -	$ -
Storage Reservoir		Capacity (MG)	Capacity (MG)	Length (ft)	$ 4,353,000	$ 4,353,000	$ -	$ -	$ 332,000	$ 332,000	$ 1,326,000	$ 1,326,000
PWS-1	Nutmeg Tank Upgrade	0.14	1	--	$ 3,316,000	$ 3,316,000	$ -	$ -	$ 332,000	$ 332,000	$ 1,326,000	$ 1,326,000
PRV Station		Diameter (in)	Diameter (in)	Length (ft)	$ 312,000	$ 306,000	$ 6,000	$ -	$ -	$ 312,000	$ -	$ -
PWV-1	PRV on Juniper Avenue	--	8	--	$ 312,000	$ 306,000	$ 6,000	$ -	$ -	$ 312,000	$ -	$ -
Rehabilitation and Replacement Projects					$ 500,000	$ 500,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000
PWRR-1	Pipeline R&R Program	1-16	> 6	--	$ 500,000	$ 500,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000
Other Projects					$ 210,000	$ 210,000	$ -	$ 60,000	$ -	$ -	$ -	$ 150,000
PWO-1	Water Master Plan Update	--	--	--	$ 150,000	$ 150,000	$ -	$ -	$ -	$ -	$ -	$ 150,000
PWO-2	Re-Skin Desalination Plant Building	--	--	--	$ 60,000	$ 60,000	$ -	$ 60,000	$ -	$ -	$ -	$ -
CIP Total					**$ 6,788,000**	**$ 7,770,000**	**$ 55,000**	**$ 1,064,000**	**$ 1,027,000**	**$ 1,324,000**	**$ 1,797,000**	**$ 1,576,000**
Annual Cost					**N/A**	**N/A**	**N/A**	**$ 1,064,000**	**$ 1,027,000**	**$ 1,324,000**	**$ 1,797,000**	**$ 1,576,000**

Notes:

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(4) Total Mark-Up is 65.8% of the baseline construction costs.

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Table ES.11 5-Year Revised Wastewater Collection System CIP

Project		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[(1)(2)(3)(4)] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($) Near-Term 2019	2020	2021	2022	2023
Capacity Related Improvements					$ 2,488,000	$ 2,242,000	$ 246,000	$ 530,000	$ -	$ 1,371,000	$ 587,000	$ -
Gravity Mains		Diameter (in)	Diameter (in)	Length (ft)	$ 2,488,000	$ 2,242,000	$ 246,000	$ 530,000	$ -	$ 1,371,000	$ 587,000	$ -
WWGM-1	Gravity Main along Atascadero Road	18	27	1,000	$ 530,000	$ 463,000	$ 67,000	$ 530,000	$ -	$ -	$ -	$ -
WWGM-2	Gravity Main along Main Street	15	24	2,900	$ 1,371,000	$ 1,194,000	$ 177,000	$ -	$ -	$ 1,371,000	$ -	$ -
WWGM-3	Gravity Main along Main Street	12	18	1,600	$ 544,000	$ 544,000	$ -	$ -	$ -	$ -	$ 544,000	$ -
WWGM-4A	Gravity Main along San Joaquin Street	6	12	150	$ 43,000	$ 41,000	$ 2,000	$ -	$ -	$ -	$ 43,000	$ -
Rehabilitation and Replacement Projects					$ 2,398,000	$ 2,398,000	$ -	$ 249,000	$ 945,000	$ -	$ 602,000	$ 602,000
Gravity Mains		Diameter (in)	Diameter (in)	Length (ft)	$ 1,884,000	$ 1,884,000	$ -	$ -	$ 680,000	$ -	$ 602,000	$ 602,000
WWRR-1A	Gravity Main Beachcomber Drive	12	12	570	$ 166,000	$ 166,000	$ -	$ -	$ 166,000	$ -	$ -	$ -
WWRR-1B	Gravity Main Beachcomber Drive	10	10	960	$ 262,000	$ 262,000	$ -	$ -	$ 262,000	$ -	$ -	$ -
WWRR-1C	Gravity Main Beachcomber Drive	8	8	950	$ 252,000	$ 252,000	$ -	$ -	$ 252,000	$ -	$ -	$ -
WWRR-2	Pipe R&R Program Upstream of LS-1	6-12	8-12	27,000	$ 1,204,000	$ 1,204,000	$ -	$ -	$ -	$ -	$ 602,000	$ 602,000
Structures		Type	Type	Quantity	$ 514,000	$ 514,000	$ -	$ 249,000	$ 265,000	$ -	$ -	$ -
WWRR-4	Cap Replacement Upstream of LS-1	Cap	Manhole	31	$ 514,000	$ 514,000	$ -	$ 249,000	$ 265,000	$ -	$ -	$ -
Other Projects					$ 210,000	$ 210,000	$ -	$ -	$ -	$ -	$ -	$ 210,000
WWO-1	Sewer Master Plan Update	--	--	--	$ 150,000	$ 150,000	$ -	$ -	$ -	$ -	$ -	$ 150,000
WWO-2	Flow Monitoring Program	--	4 weeks	15 FM	$ 60,000	$ 60,000	$ -	$ -	$ -	$ -	$ -	$ 60,000
CIP Total					**$ 5,096,000**	**$ 4,850,000**	**$ 246,000**	**$ 779,000**	**$ 945,000**	**$ 1,371,000**	**$ 1,189,000**	**$ 812,000**
Annual Cost					**N/A**	**N/A**	**N/A**	**$ 779,000**	**$ 945,000**	**$ 1,371,000**	**$ 1,189,000**	**$ 812,000**

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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(4) Total Mark-Up is 65.8% of the baseline construction costs.

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Chapter 1
INTRODUCTION

This Chapter presents a brief summary of the City of Morro Bay's (City's) need for the OneWater Morro Bay (OneWater) Plan, the objectives of OneWater, and a description and organization of the eight chapters that cover the water distribution system, wastewater collection system, and storm drainage system.

1.1 Background

The City is located along the central coast of California in San Luis Obispo County. The City is located at the crossroads of Highway 1 and Highway 41, approximately 12 miles north of the city of San Luis Obispo, and approximately 6 miles south of the City of Cayucos (Figure 1.1). The City limits compromise roughly 5 square miles, and consist of varied topography ranging from steep mountain terrain to ocean basin.

The City is a small seaside town with strong historical roots in the fishing industry. Today, it is a thriving destination for visitors, offering natural beauty, outdoor recreation, a working waterfront, a creative community, and a welcoming atmosphere. Approximately 20 percent of the City's residences are primarily used for vacation or seasonal uses.[1]

The City, incorporated in 1964, provides water, sewer, and storm drainage services to its residents. Water services are provided to all residential, commercial, and industrial customers, and for fire protection services.

1.2 Water, Sewer, and Storm Service Areas

Figure 1.2 illustrates the City's current water, wastewater, and storm drainage service area. The service area is the same for water, wastewater, and storm drainage.

The City's water system consists of approximately 72 miles of active water distribution pipelines up to 16-inches in diameter, ten treated water storage tanks, three booster pump stations, four Pressure Reducing Valve (PRV) stations, and six pressure zones. The City's wastewater collection system consists of approximately 55 miles of active gravity sewer pipelines, 1.8 miles of force mains, and three pump stations that convey flow to the wastewater treatment plant (WWTP). The City operates a storm drainage collection system that consists of street drainage, inlets, gravity pipelines, open channels, one storm water detention basin, and outfalls to the Pacific Ocean.

The land use assumptions in this Master Plan were based on the projected future developments within the planning area from the City's planning document, Plan Morro Bay. Plan Morro Bay projects through the year 2040. Should future planning conditions change from the assumptions stated in the Master Plan (i.e. accelerated growth, more intense developments, etc.), revisions and adjustments to the Master Plan recommendations would be necessary.

[1] City of Morro Bay Plan Morro Bay Public Draft, May 2018

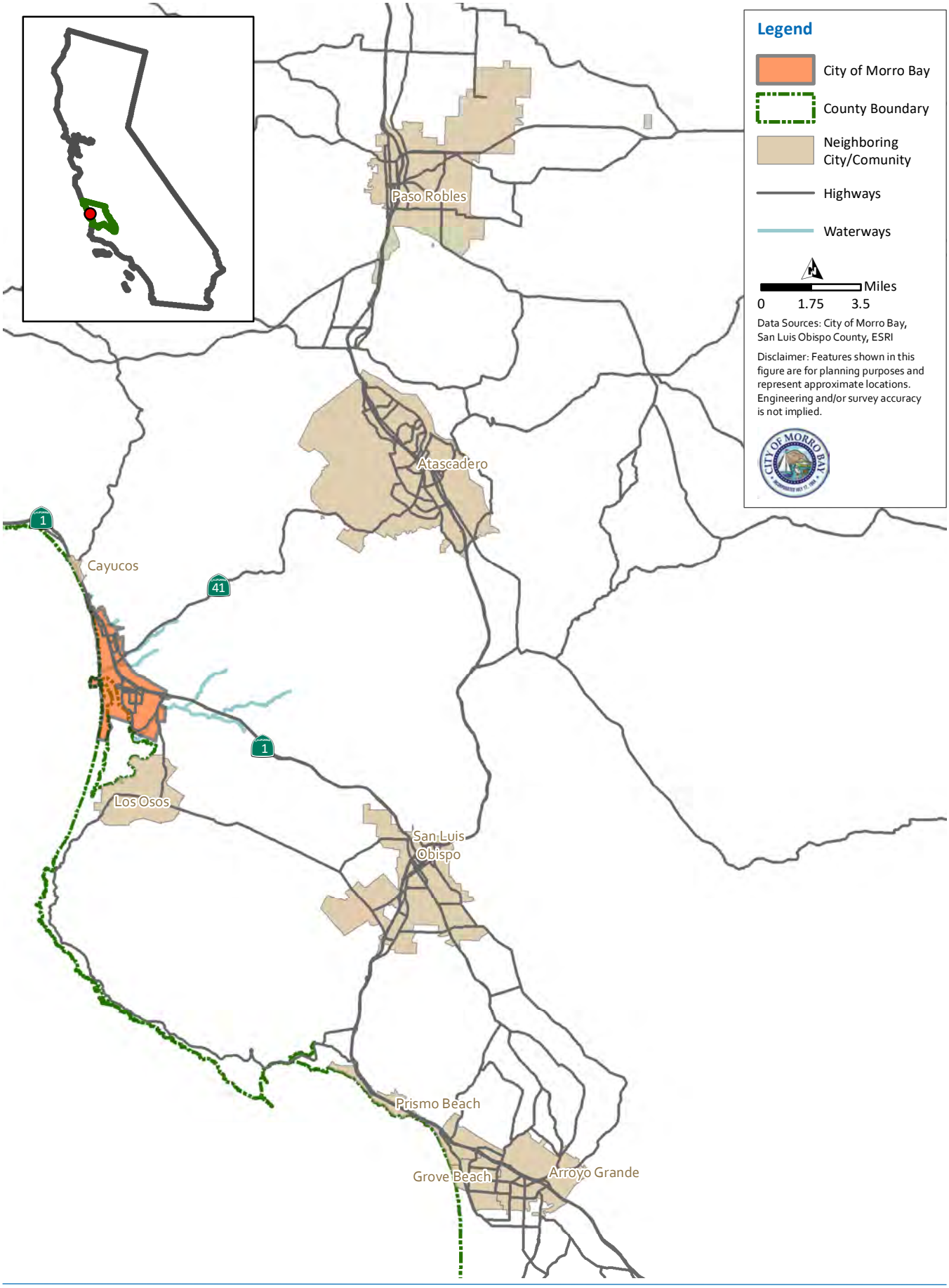

Figure 1.1 Regional Location Map

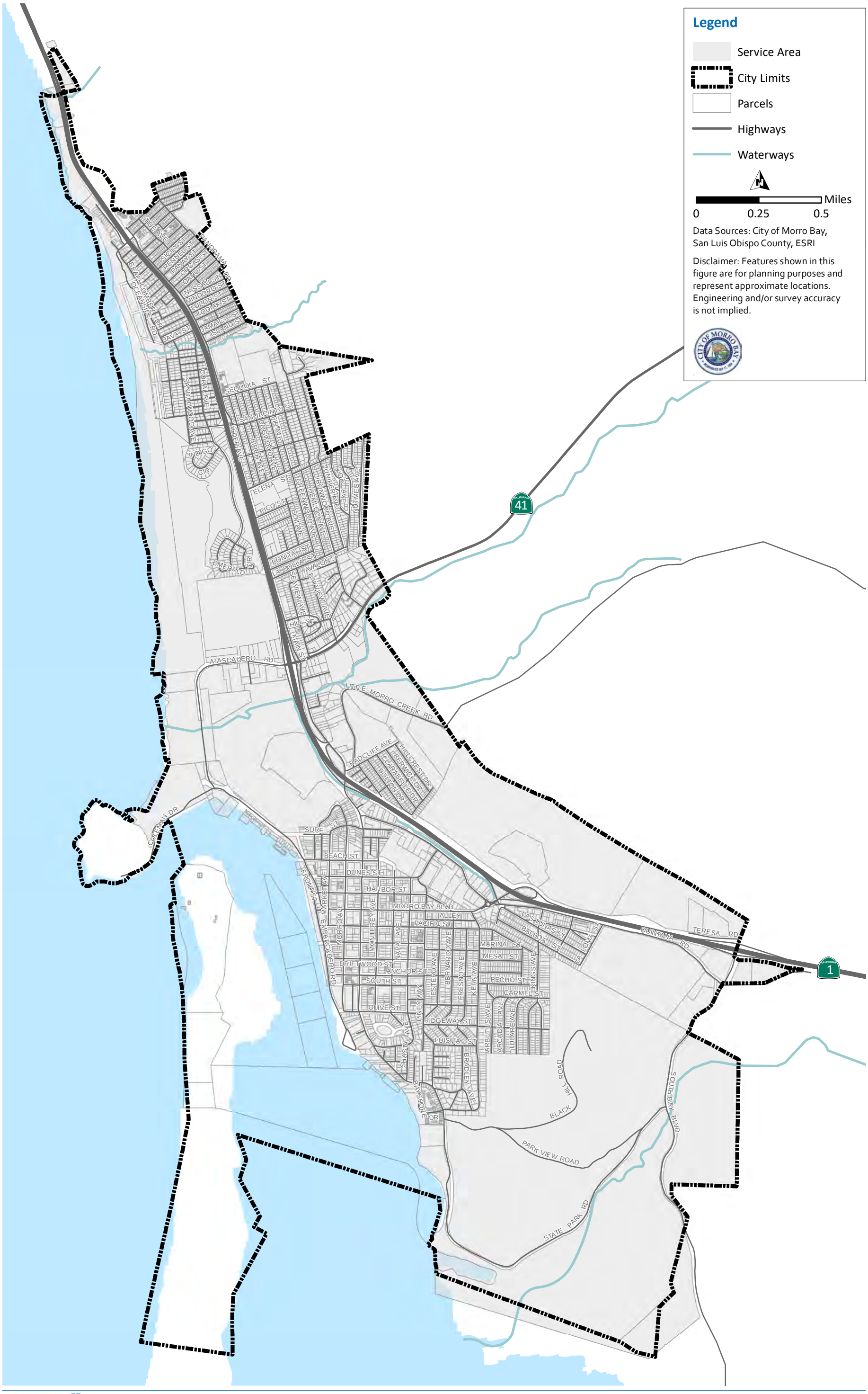

Figure 1.2 Service Area

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1.3 Previous Master Plans

This section describes the City's previous master plans for water, wastewater, and storm drainage.

1.3.1 Previous Water Master Plan

The City's previous Water Master Plan was developed in 1997 by Boyle Engineering Corporation. The objectives of the 1997 Water Master Plan was to review the existing distribution system and to provide the framework for meeting increasing water demands in accordance with the then current land use plan, zoning regulations, and other policy and planning documents. The 1997 Master Plan included the development of a water system hydraulic model, developed in FAAST hydraulic network computer modeling software.

1.3.2 Previous Wastewater Master Plan

The City's previous Sewer Collection System Master Plan Update was completed in May 2006 and developed by Wallace Group. The purpose of the 2006 Sewer Collection System Master Plan Update (2006 Sewer Master Plan) was to assist the City in prioritizing both present and future wastewater system needs and to set forth a mechanism for addressing those needs. The 2006 Sewer Master Plan included the development of a spreadsheet model that used Manning's Equation to evaluate the capacity of the City's trunk sewer system.

1.3.3 Previous Storm Drainage Master Plan

The City's previous Storm Drain Master plan was completed in August 1987 and was developed by John L. Wallace & Associates. The purpose of the 1987 Storm Drain Master Plan (1987 Storm Drain Plan) was to develop a plan to implement needed storm drainage improvements in an orderly and timely fashion. The 1987 Storm Drain Plan included the development of atlas maps that show the existing and proposed storm drainage facilities in the City, provided recommended improvements to properly convey storm runoff, and provided recommendations on how the City should implement the plan.

1.4 Study Purpose, Scope, and Authorization

The purpose of the OneWater Plan is to update the previous master plans, conduct an analysis of the City's water supplies, to identify capacity deficiencies in the water, sanitary sewer, and storm drainage systems, to develop feasible alternatives to correct these deficiencies, and to plan the infrastructure that will serve future development through the year 2040.

At the March 09 2017 City Council meeting (Resolution No. 19-17), a Carollo Engineers, Inc. (Carollo) was awarded a professional services contract to prepare the OneWater Plan. The professional services agreement included the following main tasks:

- Task 1 – Project Management and Meetings
- Task 2 – Data Collection and Background Review
- Task 3 – Planning Data Analysis
- Task 4 - Historic Water Demands and Wastewater Flows
- Task 5 - Water Supply Analysis
- Task 6 - Hydraulic and Hydrological Modeling.
- Task 7 - Flow Monitoring Data and I/I Mitigation Analysis

- Task 8 - Planning and Design Criteria
- Task 9 - Determine Existing System Capacities
- Task 10- Develop Improvements and Future Service Project Portfolios
- Task 11 - Develop Capital Improvement Program
- Task 12 - Financial Sustainability
- Task 13 - Project Deliverables
- Task 14 - Desalination Permitting Assistance

1.5 Report Organization

The OneWater Plan contains eight chapters, followed by appendices that provide supporting documentation for the information presented in the report. The chapters were developed using the work plan presented in Carollo's proposal dated February 24, 2017. The work plan is shown in Appendix A. The chapters are briefly described below:

- Chapter 1 - Introduction. This chapter presents the need for the OneWater Plan and the objectives of the study.
- Chapter 2 - Study Area Description. This chapter presents a description of the study area, defines the planning horizon for this study, and summarizes the land use classifications.
- Chapter 3 - Planning and Evaluation Criteria. This chapter presents the planning criteria that were used to evaluate the existing water distribution system, wastewater collection system and storm drainage system, and to size future improvements and expansions.
- Chapter 4 - Water Demands and Wastewater Flows. This chapter summarizes the City's historical water consumption and production records used to determine the daily, monthly, and seasonal fluctuations experienced by the water system. This chapter also defines the typical components of wastewater in a collection system as they pertain to this Master Plan. The flow monitoring data and results from the flow monitoring program are summarized and discussed.
- Chapter 5 – Water, Wastewater, and Storm Drainage System Facilities and Hydraulic Models. This chapter describes the development and calibration of the City's hydraulic models. A description of the City's water distribution system, wastewater collection system, and storm drainage systems is provided. In addition, an outline of the steps used to create and calibrate the models is provided.
- Chapter 6 - System Evaluation and Proposed Improvements. This chapter discusses the hydraulic evaluation of the water distribution system, wastewater collection system and storm drainage system, and the proposed projects that correct capacity deficiencies and serve future users. This chapter also describes the recommended infiltration and inflow (I/I) reduction plan.
- Chapter 7 - Water Supply Alternatives Analysis. This chapter summarizes the water supply evaluation alternative analysis for Morro Bay's domestic water system.
- Chapter 8 - Capital Improvement Plan. This chapter presents the capital improvement projects, a summary of the capital costs, and a basic assessment of the possible financial impacts on the City. This chapter presents the recommended capital improvement plan (CIP) for the City's water distribution system, wastewater collection system and storm drainage system, and a summary of the capital costs.

1.6 Acknowledgements

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

- Rob Livick, P.E.; Public Utilities Director
- Joe Mueller; Utilities Division Manager
- Damaris Hanson; Environmental Programs Manager
- Jennifer Callaway; Finance Director
- Jamie James; Water Systems Supervisor
- Dave Zevely; Collection System Supervisor

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

- Eric Casares, P.E.; Principal In Charge
- Ryan Orgill, P.E.; Project Engineer
- Lydia Holmes, P.E.; Water Supply Lead
- Inge Wiersema, P.E.; Regional Planning Lead
- Tim Loper, P.E.; Master Planning Lead
- Anthony Cemo, P.E.; Water Supply
- Danielle Orgill, P.E; Staff Engineer
- Mike Wetterau; Staff Engineer
- Grace Mitzel; Staff Engineer
- Riley Powers; GIS/Graphics
- Candice Padilla; Document Processing

1.7 Quality Management and Control

In preparation of this report, Carollo has implemented our internal Quality Management procedures focused on meeting high quality goals. The primary reviewer(s) for each chapter of the OneWater Plan.

Chapter	Title	Primary Author	Reviewer(s)
Chapter 1	Introduction	Ryan Orgill	Eric Casares
Chapter 2	Study Area Description	Ryan Orgill	Tim Loper Eric Casares
Chapter 3	Planning and Evaluation Criteria	Ryan Orgill Anthony Cemo	Eric Casares Lydia Holmes Tim Loper
Chapter 4	Water Demands and Wastewater Flows	Ryan Orgill	Eric Casares Tim Loper
Chapter 5	Water, Wastewater, and Storm Drainage System Facilities and Hydraulic Models	Ryan Orgill Anthony Cemo	Eric Casares Lydia Holmes Tim Loper
Chapter 6	System Evaluation and Proposed Improvements	Ryan Orgill	Eric Casares Tim Loper
Chapter 7	Water Supply Alternatives Analysis	Anthony Cemo	Eric Casares Lydia Holmes Inge Wiersema
Chapter 8	Capital Improvement Plan	Ryan Orgill	Eric Casares Tim Loper

In addition to Carollo's internal Quality Management procedures, we have worked closely with water and wastewater operations and maintenance staff throughout the development of the OneWater Plan. Specifically, the results of the water distribution, wastewater collection, and stormwater system models were reviewed with City staff to validate our findings. These findings ultimately formed the basis of noted deficiencies and projects that formed the basis of the capital improvement plans. In total, Carollo held the following three (3) workshops with the City focused on the results of the water, collection, and stormwater system model:

- Water Distribution - December 14, 2017
- Wastewater Collection - November 09, 2017
- Stormwater - May 03, 2018

1.8 Reference Material

The following documents were referenced in the preparation of this Master Plan:

- California, Department of Water Resources Bulletin 132-16
- California, Department of Water Resources, State Water Project Final Delivery Capability Report, 2016
- Central Coast Water Authority Fiscal Year Budgets
- Central Coast Water Authority Comprehensive Annual Financial Reports
- City of Morro Bay 2015 Urban Water Management Plan, MNS Engineers, June 2016
- City of Morro Bay Master Water Reclamation Plan, MKN & Associates, Draft, March 2017
- City of Morro Bay Plan Morro Bay Public Draft, May 2018
- City of Morro Bay Sewer Collection System Master Plan Update, Wallace Group, May 2006
- City of Morro Bay Storm Drain Master Plan, John L. Wallace & Associates, August 1987
- City of Morro Bay Stormwater Management and Guidance Manual for Low Impact Development & Post-Construction Requirements, March 2014 (Updated July 2017)
- City of Morro Bay Water Master Plan, Boyle Engineering Corporation, June 1997
- City of Morro Bay Water Reclamation Facility Master Plan, Black & Veatch, November 2016
- State Water Resources Control Board, Sediment Monitoring Report, Morro Bay Estuary Program, 2015
- The Metropolitan Water District of Southern California, California WaterFix Finance and Cost Allocation, 2017
- City of Morro Bay Environmentally Sensitive Habitat Area (ESHA) Analysis: 2050 Sea Level Rise Scenario
- City of Morro Bay Sea Level Rise Adaptation Strategy Report, January 2018

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Chapter 2

STUDY AREA, LAND USE, AND POPULATION

This chapter presents a discussion of the City of Morro Bay (City) planning area characteristics, the land use classifications, and historical and projected population trends.

2.1 Study Area

The study area for this OneWater Morro Bay (OneWater) Plan consists generally of the City limits. The City's planning area extends outside the current City limits, however, the land uses outside of the City limits generally consist of open space and agricultural uses which are not expected to connect to the City's utilities in the future. Figure 2.1 shows the study area boundary.

2.2 Planning Period

This OneWater Plan is intended to serve as a guiding document for the planning and implementation of water, sewer, and storm drainage system improvements to accommodate future growth within the City limits through the year 2040. The planning horizon is consistent with the City's planning document, Plan Morro Bay. The population and land use projects included in Plan Morro Bay are summarized in this chapter and set the foundation for the water demand projections and wastewater flow projections.

2.3 Climate and Topography

The City's service area is characterized as a warm-summer Mediterranean climate, with dry, warm summers and mild, wet winters. Table 2.1 summarizes the average maximum and average minimum monthly temperatures as well as the average monthly precipitation. The average high temperature stays fairly consistent throughout the year averaging 65 degrees Fahrenheit (degrees F) ranging between 62.3 degrees F and 69.2 degrees F. October is the City's hottest month, with an average high temperature of 69.2 degrees F and an average low temperature of 50.4 degrees F. February is the wettest month with an average 3.43 inches of precipitation. Approximately 91 percent of the annual rainfall occurs between November and April, with an average annual rainfall of 16.74 inches. Evapotranspiration (ETo) values, which serve as indicators of how much water is requires to maintain healthy agriculture and landscaping, range from 1.68-inches in February to 6.60-inches in July.

The study area terrain slopes from high elevation along the eastern border to the lower elevations along the western boarder where the City meets the Pacific Ocean. The average study area elevation is approximately 60-feet above mean sea level (MSL). The surface elevations within the study are range from approximately 0-feet above MSL to 423-feet above MSL.

Table 2.1 Study Area Climate

Month	Average Maximum Temperature[1] (degrees F)	Average Minimum Temperature[1] (degrees F)	Average Monthly Rainfall[1] (inches)	Monthly Average ETo[2] (inches)
January	62.3	42.9	3.29	1.76
February	62.9	44.1	3.43	1.68
March	63.2	44.7	2.95	4.26
April	64.1	45.4	1.21	4.56
May	63.6	47.8	0.34	5.16
June	64.6	50.3	0.07	6.35
July	66.0	52.5	0.03	6.60
August	66.8	53.2	0.06	5.18
September	68.2	52.6	0.28	4.91
October	69.2	50.4	0.72	4.81
November	66.3	46.7	1.69	2.72
December	62.4	42.6	2.66	1.79
Annual	**65.0**	**47.8**	**16.74**	**49.9**

Notes:
(1) Source: Western Regional Climate Center, Morro Bay Fire Department, CA (045866)
(2) Source: California Irrigation Management Information System (CIMIS) Station 160 Sab Luis Obispo West. Represents month-long average ETo from December 2016 through November 2017.

2.4 Land Use

Land use information is an integral component in defining the amount of wastewater generated within the service area. Land use in an area will impact the volume and characteristics of the wastewater generated. Therefore, adequately estimating water demands, and wastewater and storm drainage loads from various land use types is important in sizing and maintaining the water, wastewater, and storm drainage system facilities.

An important tool for determining land use projections is the City's Plan Morro Bay. The land use assumptions provided in Plan Morro Bay were used for the purpose of OneWater. Plan Morro Bay identifies 16 land use categories as listed below:

- Agriculture
- Commercial / Recreational Fishing
- Community Commercial
- District Commercial
- General (Light) Industrial
- High Density
- Low Density
- Mixed Uses (Harbor)
- Moderate Density
- Neighborhood Commercial
- Open Space / Recreation
- Public/Institutional
- Visitor Serving Commercial

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Figure 2.1 Study Area Boundary

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Descriptions of the categories are provided in Appendix B. The descriptions are excerpts from Plan Morro Bay. For the purpose of this OneWater Plan, the 16 land uses were grouped into similar land use types. Figure 2.2 shows the City's grouped land use map, as well as vacant parcels, as defined by Plan Morro Bay. Table 2.2 provides a summary, by land use, of the amount of developed and developable land within the study area. It should be noted that Table 2.2 includes a significant amount of Open Space & Agricultural land, a majority of which lies outside the City limits. Residential (rural density, low density, medium density, medium-high density, high density, and very high density) land use accounts for approximately 56 percent of the total existing developed acreage when Open Space and Agricultural is not considered.

As shown in Table 2.2, there are approximately 8,715 acres of developed land within the City limits (excluding right-of-ways such as streets, highways, and railroads). Of the 8,715 developed acres, 7,569.5 acres (86.9 percent) are classified as open space and agricultural, 642.4 acres (7.4 percent) are classified as residential, 277.9 acres (3.2 percent) are classified as commercial, and the remaining 225 acres (2.6 percent) are associated with industrial land, waterfront commercial/industrial land, public facilities/government land, or mixed use.

2.5 Service Area Population

This section summarizes the City's historic and projected population within the system's service area. Population projection for the Cayucos community were not developed, as flow estimates for this area were documented in separate planning documents for each agency and are considered separately.

Table 2.3 summarizes the City's historical population from 1997 to 2016. As shown in Table 2.3, the City's population has fluctuated from 1997 to 2016, but had an overall increase of approximately 10-percent. Historic population estimates for the City's service area were developed using the 2010 US Census Data for 2010, as well as the California Department of Finance for the years 1997 through 2016. Figure 2.3 shows the historic and projected population.

Future growth in the City is limited by Measure F, a voter-approved growth management ordinance that limits the City to 12,200 residents. Plan Morro Bay has developed population projections based on this limitation, as summarized in Table 2.4. The projected population is estimated to increase by 1,435 residents within the planning period.

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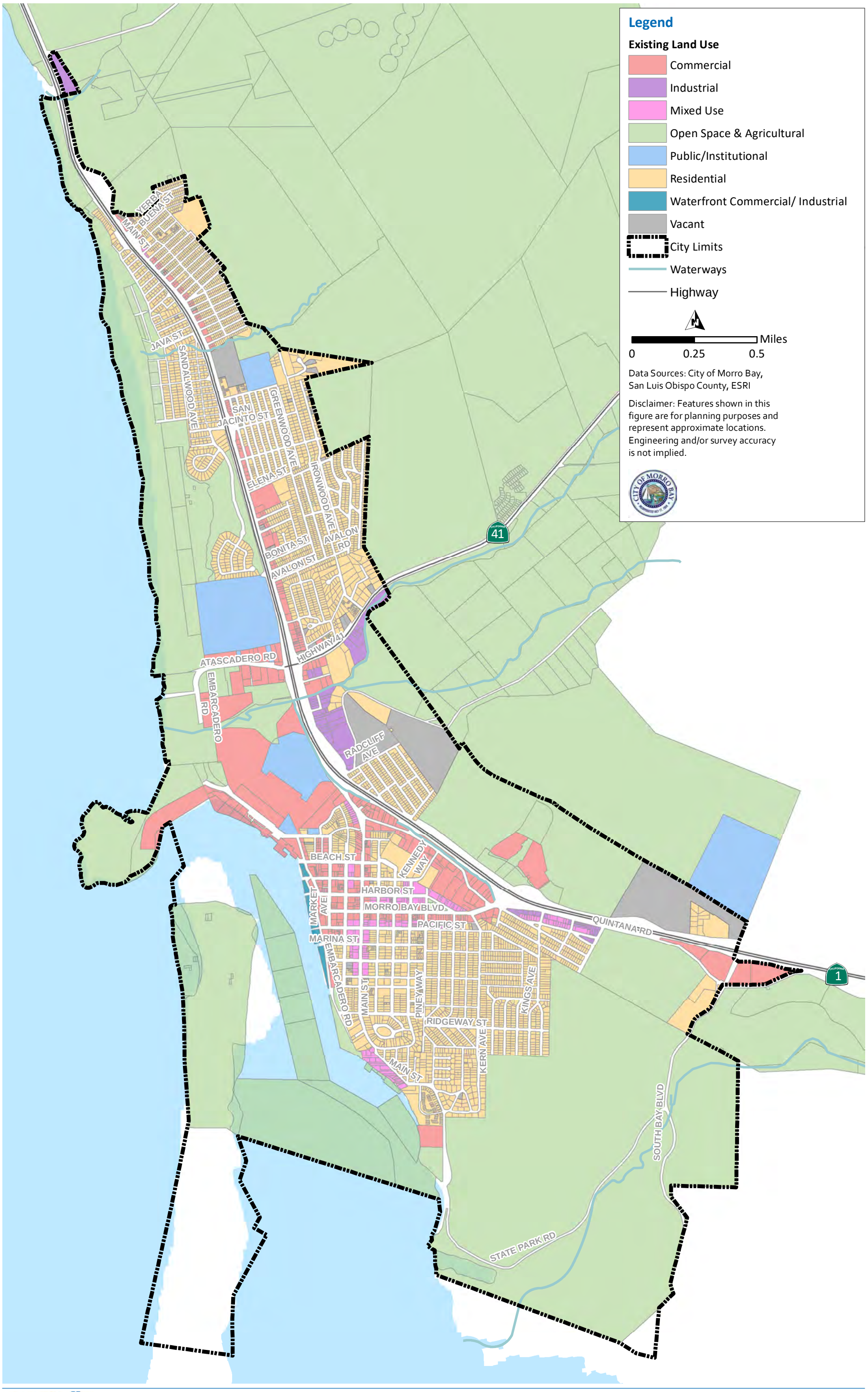

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Figure 2.2 General Plan Land Use

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Table 2.2 General Plan Land Use

Land Use Category	Area within the Water Service Area (acres)		
	Developed Land	Vacant Land	Total
Residential	642.4	110.0	752.4
Commercial	277.9	3.5	281.4
Industrial(1)	32.2	0.0	32.2
Waterfront Commercial/Industrial	6.3	0.0	6.3
Mixed Use	33.8	0.0	33.8
Open Space & Agricultural	7,569.5	0.0	7,569.5
Public/Institutional	152.6	0.0	152.6
Total(1)	**8,714.8**	**110.0**	**8,828.3**

Notes:
(1) Total excludes Coastal Dependent Industrial, which is not anticipated to connect to the system. Open Space & Agricultural land extends outside the City limits, but is not expected to contribute water demands or wastewater flows into the systems.

Table 2.3 Historical Population

Year	Population	Year	Population
1997	9,696	2007	10,436
1998	9,845	2008	10,506
1999	9,871	2009	10,555
2000	10,410	2010	10,234
2001	10,486	2011	10,296
2002	10,510	2012	10,331
2003	10,510	2013	10,427
2004	10,522	2014	10,547
2005	10,270	2015	10,601
2006	10,491	2016	10,714

Notes:
(1) Source: California Department of Finance

Table 2.4 Projected Population

Year	Population
2017	10,714
2020	10,901
2025	11,213
2030	11,525
2035	11,837
2040	12,149

Notes:
(1) Source: Plan Morro Bay

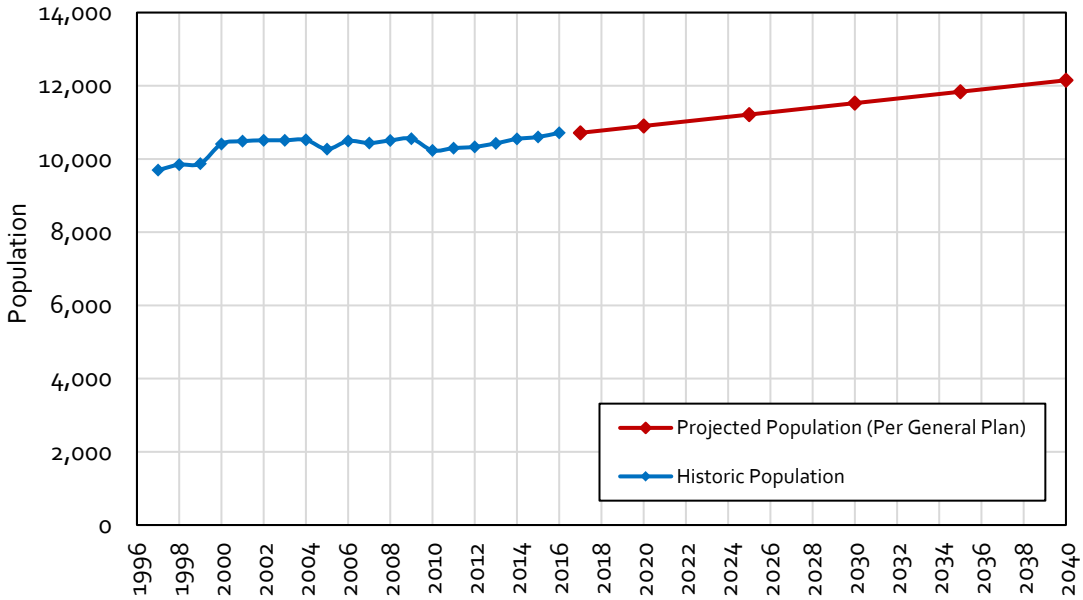

Figure 2.3 Historic and Projected Population

Chapter 3

PLANNING AND EVALUATION CRITERIA

This chapter presents the planning criteria that were used to evaluate the City of Morro Bay's (City's) existing water supply and distribution system, wastewater collection system, and storm drainage system, and to size future improvements and expansions.

3.1 Water System Planning and Evaluation Criteria

This section presents the planning criteria that were used to evaluate the City's water supply and distribution system. The developed criteria address the water supply capacity, storage capacity, acceptable service pressures, and distribution main performance.

3.1.1 Water Supply Criteria

The water supply evaluation analyzed different water supply alternatives that include the following supply sources:

1. Recycled water
2. California State Water Project water
3. Groundwater from the Morro and/or Chorro Wellfields
4. Stormwater
5. Ocean Desalination

The water supply sources listed above were each evaluated individually and in different combinations to compare their relative benefits and risks, and to determine which are the most effective and appropriate. Each alternative underwent an evaluation aimed to rank proposed projects with respect to criteria reflecting Morro Bay's interests. The evaluation considers nine separate criteria under the general economic, resilience, and implementation of each project to assess how each water supply alternative compares to the existing system supply and operation as well as future infrastructure improvements. Every alternative was given a numerical score from 1 to 5 for each criteria, with 5 being the best and 1 being the worst score. The sum of the evaluation criterion scores times a weighting factor can then be used to rank alternatives to allow comparison to the other alternatives. The City selected the water supply evaluation criteria below to assess each alternative relative to a benchmark. The following sections describe each evaluation criteria definition in detail and its respective scoring system used in the ranking analysis.

1. Unit Cost
2. Drought Resiliency
3. Earthquake Resiliency
4. Flood Risk Mitigation
5. Local Supply Benefit
6. Energy Requirements
7. Constructability
8. Regulatory and Institutional Complexity

9. Quality Consistency

3.1.1.1 Unit Cost

Unit costs is defined as the cost of water per unit volume of water produced. In this case, the unit cost is represented as a dollar per annual acre-foot of water, ($/AFY). The unit costs is the accumulation of all costs including amortized capital construction costs, annual energy costs, and annual O&M costs. Table 3.1 displays the definition of the unit cost Evaluation Criteria.

Table 3.1 Unit Cost Evaluation Criteria/Scoring Definitions

Rank Definition	From ($/AFY)	To ($/AFY)
5 - Very Low	$0	$500
4 – Low	$500	$1,000
3 – Average	$1,000	$2,000
2 – High	$2,000	$3,000
1 - Very High	$3,000	$4,000

3.1.1.2 Drought Resiliency

Drought resiliency is defined as the ratio between the yield of a water supply alternative during a dry year and a wet year. A dry year is defined as when yields are limited due to production and supply constraints. A wet year is defined as the maximum volume of allocated water, production, and supply are available for consumption. The scoring range of drought resilience is defined in Table 3.2.

Table 3.2 Drought Resiliency Evaluation Criteria/Scoring Definitions

Drought Resilience	Score Definition
5 - Very High Resilience	Greater than 75% Resilience
4 - High Resilience	> 50 to 75% Resilience
3 - Moderate Resilience	> 25 to ≤ 50% Resilience
2 - Low Resilience	> 10 to ≤ 25% Resilience
1 - Minimal/No Resilience	Less than 10% Resilience

3.1.1.3 Earthquake Resiliency

Earthquake resiliency is defined as the ability of a water supply alternative to withstand the effects of an earthquake and its ability to deliver water after a seismic event. Having localized project components reduces the risk of crossing geologic features such as faults and differential soils in comparison to multi-regional non-redundant projects which have high geologic exposure. The combination of these criteria define each category of earthquake resiliency as shown in Table 3.3.

Table 3.3 Earthquake Resiliency Evaluation Criteria/Scoring Definitions

Earthquake Resiliency	Score Definition
5 - High Resiliency	Project is highly unlikely to be damaged during an earthquake
	Water is likely to be delivered immediately (within 4 hours) after event
	Project is local and does not stretch across multiple geologic features
	Project is new construction up to modern standards
	Concept has significantly distributed water supply facilities and redundancy
3 – Moderate Resiliency	Project is unlikely to be damaged during an earthquake
	Water is likely to be delivered within 3 to 4 weeks after an earthquake
	Project is local and does not stretch across multiple geologic features
	Project is newer construction up to modern standards
	Concept has distributed water supply facilities and some redundancy
1 - Very Fragile	Project is highly likely to be damaged during an earthquake
	Project is likely to be out of service greater than 3 months after event
	Project spans across multiple geologic features
	Project was constructed over 20 years ago
	Concept includes long stretches of pipelines without redundancy

3.1.1.4 Flood Risk Mitigation

Flood Vulnerability is defined as the ability of a water supply alternative to withstand the effects of a flooding event, tsunami, and the effects of sea level rise. Comparing the layout of the water supply alternatives with respect to data provided by both the National Oceanic and Atmospheric Association (NOAA) and the Federal Emergency Management Agency (FEMA), each project was categorized by its location, effects to local flood elevations, and ability to convey onsite drainage. Additionally, the proposed sites of the water supply alternatives were compared with the results of the City of Morro Bay Environmentally Sensitive Habitat Area (ESHA) Analysis: 2050 Sea Level Rise Scenario report and the Sea Level Rise Adaptation Strategy Report. The following table displays the sea-level rise elevations from both reports.

It was found that none of the water supply alternatives are affected by the inundation, flood, dune, and bluff hazard zones as a result of sea-level rise. Sites located near the existing WTP are well above the sea-level rise elevations presented in Table 3.4. Sites located near the existing Kings Tank and the WRF project are also well above any potential sea-level rise threats. Additionally, the site near the existing WTP is also approximately 1,000 feet from the limits of flood and dune hazards and approximately 1,400 feet from the inundation limits presented in the sea-level rise studies prepared for the City. Table 3.5 displays the definitions of flood vulnerability evaluation criteria. Maps of sea level rise, tsunami inundation limits, and flood plain limits are included in Appendix C.

Table 3.4 Projected Sea-Level Rise Elevation

Projection Year	City of Morro Bay ESHA Analysis Report	Sea Level Rise Adaptation Strategy Report
2030	N/A	0.5
2050	0.9	0.9
2100	N/A	3.1

Table 3.5 Flood Vulnerability Evaluation Criteria/Scoring Definitions

Flood Vulnerability	Score Definition
5 - Minimal	Project is susceptible to local pooling from 2 percent event
	Project contains on site run-off storage w/ discharge, no pooling
	Project has private on site storm drainage system, with no hydraulic impacts
	Project is not susceptible to effects of sea level rise
	Project is not susceptible to tsunami threats
4 - Low	Project is susceptible to moderate local pooling from 2 percent event for < 24 hours
	Project decreases on-site flood pooling with storm water BMPs
	Project has surface conveyance system with minimal hydraulic impacts
3 - Average	Project is susceptible to moderate flooding from 2 percent event, outside of 1 percent plain
	Project susceptible to major pooling on site for > 24 hours
	No damage from 1 percent event
	Project is flood protected from sea level rise inundation
	Project is flood protected from tsunami wave threats
2 - High	Project is within 0.2 percent annual event flood plain
	Project does not affect FEMA base flood elevation
	Project is susceptible to damage from 0.2 percent event
1 - Extreme	Project is within 1 percent annual event flood plain
	Project affects FEMA base flood elevation, increase flood risk on local level
	Project susceptible to damage from 1.0 percent annual event
	Project is susceptible to damage from sea level rise inundation
	Project is susceptible to damage from tsunami waves

3.1.1.5 Local Supply Benefit

The local supply benefit is defined as the ability of a water supply alternative to provide the future design demand of 1,445 acre-feet per year. The score definition represents the percentage each water supply alternative offsets the dependence for imported water. For example, if a supply alternative can provide 30 percent of the future design demand, the imported offset is 30 percent and only 70 percent of total water supply is from State Water. Table 3.6 displays the score definition of the local supply benefit evaluation criteria.

Table 3.6 Local Supply Benefit Evaluation Criteria/Scoring Definitions

Local Supply Benefit	Score Definition
5 - Very High Offset	Greater than 75% offset
4 - High Offset	50 to 75% offset
3 - Average Offset	25 to 50% offset
2 - Low Offset	10 to 25% offset
1 - Minimal/No Offset	Less than 10% offset

3.1.1.6 Energy Requirements

The energy requirements criteria is defined as the amount of energy required to produce a unit volume of water. This is quantified as a kilowatt hour per acre-foot (kWh/AF). The energy requirements include the annual operation of treatment and conveyance equipment required to both treat and transport water to storage. Table 3.7 displays the energy requirements range for each score within the evaluation criteria.

Table 3.7 Energy Requirements Evaluation Criteria/Scoring Definitions

Ranking Definition	From (kWh/acre-ft)	To (kWh/acre-ft)
5 - Very Low Energy Impact	-	1,000
4 - Low Energy Impact	1,000	2,000
3 - Moderate Energy Impact	2,000	3,000
2 - High Energy Impact	3,000	4,000
1 - Very High Energy Impact	4,000	5,000

3.1.1.7 Constructability

The constructability evaluation criteria is defined as how complicated a supply alternative is to construct. Although a project may be considered in theory, complex construction efforts can significantly increase the cost and prove not to be economic. Constructability is defined as a project being localized (within or outside of the fence), length of pipeline, and ease of construction compared to a typical construction project (one of a kind versus standard practice). Table 3.8 displays the constructability evaluation criteria definitions.

Table 3.8 Constructability Evaluation Criteria/Scoring Definitions

Constructability	Constructability Description
5 - Very Straight-Forward	Nearly all project components are located inside the fence (of a city parcel or treatment facility)
	No underground utilities
	Project is expected to be much less complicated to construct compared to a typical project
4 - Straight-Forward	Most project components are located inside the fence and some outside the fence
	Less than 1 mile of underground utilities
	Project is expected to be less complicated to construct compared to a typical project
3 - Typical	Project components are located inside and outside the fence
	1 to 3 miles of underground utilities
	Project is expected to have typical construction concerns
2 - Difficult	Most project components are located outside the fence
	3 to 5 miles of underground utilities
	Project will be difficult to construct
1 - Very Difficult	Nearly all project components are located outside the fence
	Greater than 5 miles of underground utilities
	Project will be extremely difficult to construct

3.1.1.8 Regulatory and Institutional Complexity

The regulatory and institutional complexity evaluation criteria defines the ability for a project to be permitted and approved by governing agencies. These agencies have jurisdiction as to how a project is allowed to proceed in design, construction, and operation. The high complexity represents projects having a low chance of being permitted (e.g. one of a kind design or potential risk) whereas a low complexity represents a project undergoing standard industry permits and regulations. Table 3.9 represents the ranking definitions of this evaluation criteria.

Table 3.9 Regulatory and Institutional Complexity Evaluation Criteria/Scoring Definitions

Regulation and Permitting Complexity
5 - Low Complexity
3 - Average Complexity
1 - High Complexity

3.1.1.9 Quality Consistency

The quality consistency evaluation criteria represents the ability of a project to have a consistent quality of water during production. Water supply alternatives that have variable quality from low to high will be ranked as infrequently meeting quality regulations while supply alternatives that consistency exceed quality regulations will be ranked high. Table 3.10 displays the ranking definitions for quality consistency.

Table 3.10 Quality Consistency Evaluation Criteria/Scoring Definitions

Meets Water Quality Regulations
5 - Always Exceeds Quality Regulations
3 - Generally Meets Quality Regularly
1 - Infrequently Meets Quality Regulations

3.1.1.10 Morro Bay Evaluation Weighting Criteria

The evaluation criteria was weighted to incorporate the considerations of City Staff, City Council, and the Public Works Advisory Board (PWAB). The weighting factors were developed by providing the staff, PWAB and City Council with a survey (See Appendix D) to grade the evaluation criteria on a scale of 1 to 10, with 1 being least important and 10 being most important. The survey was filled out by 10 individuals at the City and this data was input into a table and normalized to develop an even representation of each contributor in calculating the weighted factors. The percentage of each criteria with respect to the combined sum results represents the weighting factor for each evaluation criteria. The evaluation criteria weighting was to ensure the criteria that was most valuable to the City was represented appropriately for each water supply alternative. This prevented less important criteria with higher scores to not misrepresent the City's considerations. As shown in Table 3.11, the applied weighting criteria enables the lower scores of important criteria to have a greater significance to the overall score of the alternative. Table 3.11 displays the weighting factors for each evaluation criteria.

Table 3.11 Evaluation Criteria Weighting Factors

Evaluation Criteria	Weight Factors
Unit Cost	1.22
Drought Resiliency	1.17
Earthquake Resiliency	1.00
Flood Vulnerability	1.10
Local Supply Benefit	1.17
Energy Requirements	0.98
Constructability	1.09
Regulatory and Institutional Complexity	1.03
Quality Consistency	1.24

3.1.2 Treated Water Storage Requirements

The principal function of storage is to provide a reserve supply of water for:

- Operational equalization
- Fire reserve; and
- Emergency needs

Operational equalization storage is directly related to the amount of water necessary to meet peak demands. The intent of operational equalization storage is to provide the difference in quantity between the customer's peak demands and the system's reliable available supply. The volume of water allocated for emergency uses is decided based on the historical record of

emergencies experienced, and on the amount of time which is expected to lapse before a hypothetical emergency can be corrected.

3.1.2.1 Operational Equalization Storage

This storage is the amount of desirable stored water in a system to regulate fluctuations in demand so that extreme variations will not be imposed on the source of supply. Operational equalization storage typically serves the peak demands exerted within the maximum day demand (MDD). With operational equalization storage, system pressures are improved and stabilized to better serve customers throughout the service area. Operational equalization storage is commonly estimated between 10 percent and 50 percent of the MDD.

The American Water Works Association (AWWA) M-32 states that operational storage is typically between 10 to 15 percent of the MDD for large systems, but could exceed 30 percent for small systems or arid climates.

An operational equalization storage equal to 25-percent of the MDD was used for the purposes of this OneWater Plan. This criterion is also consistent with the City's previous Water Master Plan.

3.1.2.2 Fire Storage

Fire storage is the amount required to meet the necessary fire flow demands. In general, the recommended fire storage volume is determined by multiplying the highest required fire flow by its corresponding duration. For municipalities with multiple pressure zones, such as the City, the recommended fire storage is determined based on the largest required fire flow volume per pressure zone (Chapter 5 shows a map of the City's pressure zones, as well as a hydraulic profile of the water distribution system for reference).

The recommended fire flows and durations used in this OneWater Plan are summarized in Section 3.1.6, and were developed based on input from City staff, including the City's Fire Marshall, and Carollo experience on similar projects. The maximum recommended fire flow and duration are 4,000 gallons per minute (gpm) for a duration of four hours. This provision equates to a storage requirement of 0.96 million gallons (MG).

3.1.2.3 Emergency Storage

This storage is the volume recommended to meet demands during emergency situations such as pipeline failures, major distribution main failures, pump failures, electrical power outages, or natural disasters. The amount of emergency storage included within a particular water distribution system is an owner option, based on an assessment of risk, the desired degree of system dependability, economic considerations, and water quality concerns. Emergency storage criteria are typically expressed as a multiplier of the MDD or the average day demand (ADD), and can range from 0 percent to 100 percent or more of the ADD or MDD. It was agreed that the recommended emergency storage for this OneWater Plan would be equal to 150 percent of the ADD which is approximately equal to 50 gallons per capita day (gpcd) for a 72 hour period (per the City's previous Water Master Plan).

3.1.2.4 Total Storage

The recommended minimum operational storage capacity for the City is equal to 25 percent of the MDD. The recommended fire storage capacity is determined by pressure zone, and is equal to the largest fire flow rate per pressure zone multiplied by the fire flow duration. For the City as

a whole, the largest required fire flow volume is equivalent to 0.96 MG. The recommended emergency storage is equal to 150 percent of the ADD.

3.1.3 Booster Pump Station Criteria

The required booster pump station capacity is sufficient capacity to supply MDD with the largest pump out of service.

3.1.4 Service Pressures

Pressures maintained within the distribution system vary depending on distribution system operations and pressure zone topography. It is essential that the water pressure in a consumer's residence or place of business be neither too high nor too low. Low pressures, below 40 pounds per square inch (psi), cause annoying flow reductions when more than one water-using appliance is used. High pressures may cause faucets to leak and valve seats to wear out quickly. Additionally, high service pressures usually result in wasted water and high water utility bills. It is recommended that the water pressures not exceed 80 psi at service connections, unless the service is provided with a pressure-reducing device.

The AWWA Manual on Distribution Network Analysis of Water Utilities (AWWA M-32), indicates that pressures between 30 psi and 90 psi are generally expected during the range of system water demands. For the purposes of this OneWater Plan, service pressures criteria were developed for various demand conditions, as summarized below.

- *Average Day Demand (ADD):* In general, it is recommended that pressures not exceed 120 psi during a typical ADD condition, and that service connections with pressures in excess of 80 psi be equipped with a pressure reducing device.
- *Peak Hour Demand (PHD):* In order to provide adequate service pressures, it is recommended that the City maintains a desirable service pressure of 40 psi during a typical PHD condition.
- *Maximum Day Demand (MDD) + Fire Flow:* This pressure criterion is related to fire flows and was devised to ensure adequate positive pressures during a fire. It is recommended that the City fire pressure criterion requires a minimum acceptable residual pressure of 20 psi at the connecting hydrant.

Chapter 6 includes maps showing service pressures throughout the distribution system.

3.1.5 Distribution Mains

Transmission mains are generally sized to carry the greater of:

- The PHD; or
- The MDD plus fire flow.

Other criteria related to the distribution piping include the maximum and minimum velocities and the maximum allowable friction losses.

High velocities may cause damage to the pipes and to their appurtenances. Normally, velocities of 10 feet per second (fps) (AWWA M-32), or higher, do not cause ill effects if they occur for a limited duration. It is normally good practice to limit pipe velocities to no more than 8 fps on a continuous basis.

New distribution/transmission system pipelines 12-inches in diameter or less should be sized for a maximum pipeline velocity of 5 feet per second (fps), while new distribution system pipelines 16-inch in diameter or more should be sized for a maximum pipeline velocity of 4 fps. Table 3.12 summarizes the pipeline velocity criteria.

Table 3.12 Pipeline Velocity Criteria

Description	Demand Condition	Value
Existing Pipelines	peak hour demand	8 ft/s
New Pipelines (≤ 12-inch diameter)	peak hour demand	5 ft/s
New Pipelines (≥ 16-inch diameter)	peak hour demand	4 ft/s

Provided that the maximum velocity criteria and the pressure criteria are not exceeded, high pipeline head loss by itself is not a controlling factor. However, it may be an indication that the pipe is nearing the limit of its carrying capacity, and may not have sufficient capacity to perform under stringent conditions. Good practice dictates monitoring pipes that have a head loss in excess of 10 feet per 1,000 feet (AWWA M-32). Table 3.13 summarizes the pipeline headloss criteria.

Table 3.13 Pipeline Headloss Criteria

Description	Demand Condition	Desirable Maximum Headloss
Existing Pipelines	peak hour demand	10ft/kft
New Pipelines	peak hour demand	5ft/kft

3.1.6 Fire Flow Criteria

Fire flows stress a water system in the area of the fire and often identify existing deficiencies. The deficiencies are generally associated with pipe sizes (diameter) or age (roughness) that results in high headloss and lower pressures. The fire flow criteria measures a system's ability to deliver a high rate of water while maintaining a minimum pressure.

To evaluate the effect of fire flows throughout the distribution system, large point demands are applied at fire hydrants. The fire flow demands are run concurrent with the maximum day demand. Simulating maximum day demand plus fire flows also demonstrates the performance of supply sources, booster pumps, and storage tanks operating under the upper limit high demand conditions.

The recommended fire flow criteria are summarized below by land use. These fire flow criteria were developed based on input from City staff, including the City's Fire Marshal, and Carollo experience on similar projects.

- *Single-Family Residential:* 1,500 gpm for a duration of two hours.
- *Multi-Family Residential:* 2,000 gpm for a duration of two hours.
- *Commercial:* 3,000 gpm for a duration of two hours.
- *Industrial:* 4,000 gpm for a duration of four hours.
- *Public/Institutional:* 4,000 gpm for a duration of four hours.

It should be noted that the recommended criteria flows are the minimum flows per land use type. Specific flow requirements for individual building sites may be higher depending on specific occupancy use, square footage, building height, and construction type used for the specific

building (based on the currently adopted California Fire Code requirements). This OneWater Plan assumes that all required fire flows in excess of 4,000 gpm would be met through private onsite water supplies or supplemental storage. This approach is consistent with industry standard practice.

3.2 Wastewater Collection System Planning and Evaluation Criteria

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

3.2.1 Gravity Sewers

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

3.2.1.1 Manning's Coefficient (n)

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

3.2.1.2 Peak Flow Depth Criteria

The primary criterion used to identify pipeline capacity deficiencies or to size new sewer improvements is the peak flow depth criteria. Based on Carollo's experience, City staff input, and industry standards, the criteria listed in Table 3.14 were used.

Table 3.14 Maximum Flow Depth Criteria

Pipe Diameter (inches)	Maximum Flow Depth Criteria (during Peak Wet Weather Flows)
Minimum Surcharge Depth Below Manhole Rim	
All Pipes	5-feet
Maximum Flow Depth to Pipe Diameter (d/D) Ratio for New Sewers	
For all Pipes	0.75

The following explains these criteria in more detail:

- *Flow Depth Criteria for Existing Sewers:* Maximum flow depth criteria for existing sanitary sewers are established based on a number of factors, including the acceptable risk tolerance of the utility, local standards and codes, and other factors. Using a conservative flow depth criteria when evaluating existing sewers may lead to unnecessary replacement of existing pipelines. Conversely, lenient flow depth criteria

could increase the risk of sanitary sewer overflows (SSOs). Ultimately, the maximum allowable flow depth criteria should be established to be as cost-effective as possible while at the same time reducing the risk of SSOs to the greatest extent possible. For the City, existing pipelines were flagged if the pipe surcharged within five-feet within the manhole rim.

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

- *Flow Depth Criteria for New Sewers:* When sizing new sewer pipelines, it is common practice to adopt different flow depth criteria for various pipe sizes. Design d/D ratios typically range from 0.5 to 0.92, with the lower values typically used for smaller pipes, which may experience flow peaks greater than design flow or blockages from debris, paper, or rags. The City's design standards, requires that pipes be sized to handle peak flows with pipes flowing three-quarters full (d/D = 0.75) for all pipe diameters. The recommended d/D criteria for sizing new trunk lines are summarized in Table 3.14.
- *Small Diameter Pipe:* The recommended minimum sewer main diameter is 8-inches. When a small diameter pipe needs to be replaced it is recommended that it be upsized to at least an 8-inches in diameter.

3.2.1.3 Design Velocities and Minimum Slopes

To minimize the settlement of sewage solids, it is industry standard to specify a minimum velocity of 2 fps be maintained. At this velocity, the sewer flow will provide self-cleaning of the pipe. Table 3.15 lists the recommended minimum slopes and their corresponding maximum flows for maintaining self-cleaning velocities (equal to or greater than 2 fps) when the pipe is flowing at its maximum design depth (d/D ratio). The City's design criteria require that 8-inch diameter gravity sewers must have a minimum slope of 0.0035.

Table 3.15 Minimum Slope for New Pipes

Pipe Diameter (inches)	Recommended Minimum Slope[1][2] (ft/ft)	Maximum d/D Ratio[1][3]	Calculated Maximum Flow at Maximum d/D[4] (mgd)
8	0.0035	0.75	0.42
10	0.0025	0.75	0.64
12	0.0020	0.75	0.94
15	0.0015	0.75	1.47
18	0.0012	0.75	2.14
21	0.0009	0.75	2.81
24	0.0008	0.75	3.78
27	0.0007	0.75	4.84
30	0.0005	0.75	5.42
36	0.0004	0.75	7.88
42	0.0003	0.75	10.29

Notes:
(1) Source: City of Morro Bay Standards and Specification.
(2) Recommended minimum slope to provide a minimum velocity of 2 fps.
(3) Pipes shall be sized to handle peak flows with pipes flowing 75 percent full.
(4) Assumes Manning number n = 0.013

3.2.1.4 Changes in Pipe Sizes

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the 80 percent depth point of both sewers at the same elevation. The City's design standards require that whenever a pipe changes sizes or an angle of 20 degrees or greater, the flow line of the pipe flowing into the manhole shall be a minimum of 0.17 feet above the flow line of the pipe flowing from the manhole. For planning purposes and designing new pipes, and in the absence of field data, sewer crowns are typically matched at the manholes.

3.2.2 Lift Station and Force Mains

Industry standard practice is to require that sewage lift stations have sufficient capacity to pump peak flows with the largest pump out of service (firm capacity). Force main piping should be sized to provide a minimum velocity of 3 fps at the design flow rate of the lift station and no more than 8 fps. For the determination of head loss, the Hazen-Williams equation is used with a C-factor of 110, which is typical for sewer system master planning purposes.

3.2.3 Design Storm for Sewer System Planning

Design storms are rainfall events used to analyze the performance of a collection system under extreme wet weather events. The City's design storm was routed through the collection system model to determine peak wet weather flows (PWWF), as described in Chapter 6. The first step in the development of the design storm is to define its recurrence interval and rainfall duration. The recurrence interval is based on the probability that a given rainfall event will occur or be exceeded in any given year. For example, a "100-year storm" means there is a 1 in 100 chance that a storm as large as or larger than this event will occur at a specific location in any year.

Duration is the length of time in which the rainfall occurs. It is industry standard in California to use the 10-year, 24-hour design storm for analyzing wastewater collection system performance during peak wet weather flow (PWWF) conditions. The 10-year, 24-hour design storm volume is 4.1 inches as document by NOAA Atlas 14[1].

Once the design storm recurrence interval, duration, and associated rainfall volume have been determined, the next step in defining the design storm is to distribute the total rainfall over duration of the storm. The Natural Resources Conservation Service (NRCS) Type 1 rainfall distribution was used. Figure 3.1 shows the 10-year, 24-hour design storm.

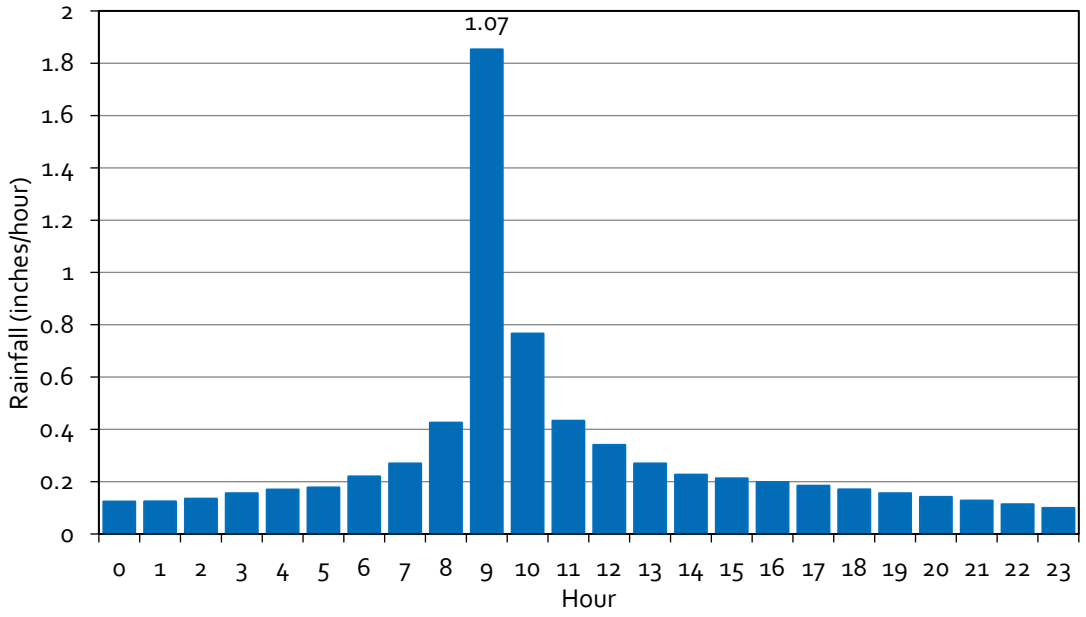

Figure 3.1 10-Year, 24-Hour Design Storm

3.3 Storm Drainage System Planning and Evaluation Criteria

The capacity of the City's storm drainage collection system was evaluated based on the planning criteria defined in this section. The criteria were established to assess the performance of the storm drainage system, as simulated by the computer hydraulic model, and for sizing future facilities. The criteria consisted of specific guidelines recommended by the City, State regulatory agencies, and other planning criteria developed by Carollo based on engineering judgment and past experience.

Precipitation characteristics, design storm duration and frequency, and impervious versus pervious surfaces were reviewed to perform the hydrologic analysis on the system. Facility deficiencies were identified through a comparison of the City's storm drainage system performance and the established planning criteria. Such a comparison defines the type, location, and extent of the facility deficiencies that should be corrected to maintain a storm drainage system with enough capacity to satisfy the selected storm conditions.

[1] https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

3.3.1 Hydrologic Criteria

This section describes the hydrologic characteristics of the City and the design storms that were used to estimate existing and future storm flows.

3.3.1.1 Precipitation Characteristics

The City has a Mediterranean climate with warm, dry summers and mild wet winters. Approximately 91 percent of the annual rainfall occurs between November and April, with an average annual rainfall of 16.74 inches.

3.3.1.2 Elements of the Design Storm

The capacity of storm drainage facilities depends on the selected level of protection to be provided by those facilities. The level of protection is often expressed in terms of the frequency, or return period, of the storm for which the facilities are to prevent damage or for which the facilities will safely pass the storm water flows. This storm is referred to as the design storm and is an idealized representation of a typical storm with a specified return period. Selection of the design storm can have a significant impact on the size and cost of required drainage facilities. There are three elements of a design storm: precipitation depth, duration, and frequency.

- *Precipitation Depth.* Precipitation depth is the amount of precipitation occurring during a specified storm duration. The depths of rainfall are statistical depths obtained by studying historical precipitation data to find the depth for each duration and for a particular frequency. Precipitation depth is usually expressed in inches.
- *Duration.* Duration is the specified length of storm time considered. When the design of storage facilities is involved, the duration should be sufficiently long so that the runoff and storage volumes return to near their level at the beginning of the simulation. Duration may be expressed in any time unit such as minutes, hours, or days.
- *Frequency.* Frequency is the number of occurrences of events with the specified precipitation depth and duration. It is expressed in terms of return period. In order to provide a reasonable level of flood protection, the statistical concept of return period or recurrence interval is utilized, which aids in assigning a probabilistic meaning to a precipitation event.

3.3.1.3 Design Storms for Storm Drainage System Planning

Two design storms were used for the evaluation of the City's storm drainage system and for sizing future storm drainage facilities. The 24-hour, 10-year event was used for evaluating storm conveyance facilities, while the 24-hour, 100-year event was used for evaluating the combined capacity of depression storage areas, streets, and pipes. The 10-year and 100-year recurrence intervals are standard selections in California because they provide a balance between level of service and affordability, and provide reasonable standards of care.

The City's design storms were developed using U.S. Department of Agriculture and NRCS Type I distribution with the 24-hour volumes defined by NOAA Atlas 14. Figure 3.2 illustrates the 24-hour design storms for the 10-year and 100-year recurrence intervals. The 10-year event has a total volume of 4.1 inches, whereas the 100-year event has a total volume of 7.1 inches.

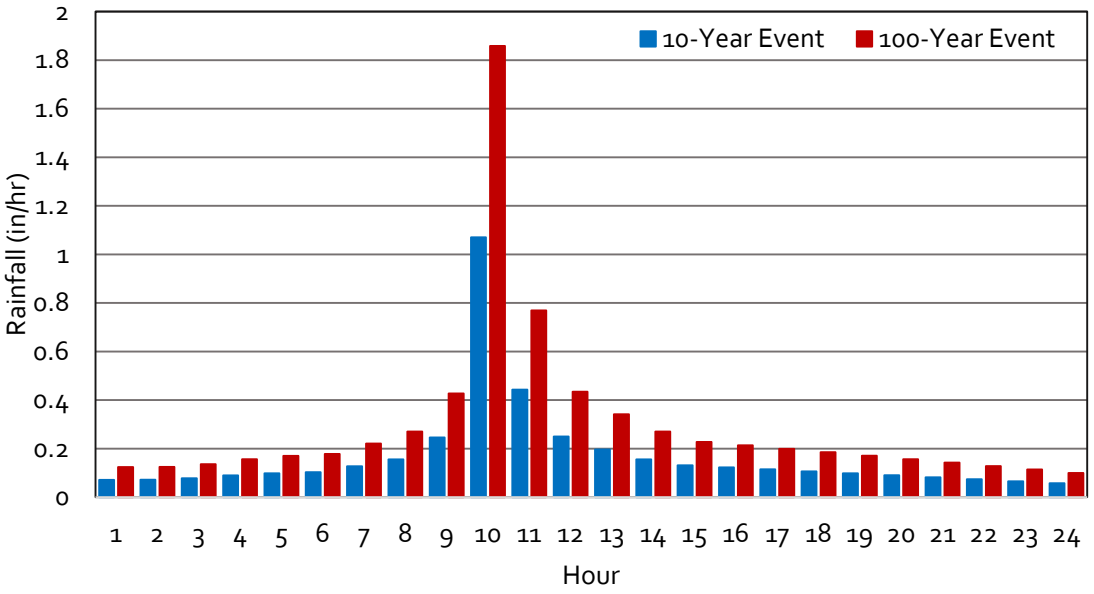

Figure 3.2 10-Year and 100-Year, 24-Hour Design Storms

3.3.1.4 Soil Characteristics

For storm water modeling, key factors relating land use to runoff are percent impervious and percent pervious area of the modeled area. The following sections describe the planning criteria and methods used to characterize the soil and runoff characteristics for the City.

- *Impervious Area.* Satellite imagery was used to determine land use imperviousness. Multispectral imagery allows for common band combinations, such as near infrared (bands 4, 3, and 2). Near infrared allows the user to identify vegetation, water bodies, and man-made features. Vegetation appear as shades of red, water as shades of blue or black, and urban areas as shades of blue-gray.

  An image processing model was developed whereby impervious and pervious surfaces were classified from the satellite imagery bands and then extracted based on user-defined variables. The classification method included samples throughout the study area encompassing all lands uses including multiple areas of vegetation, urban areas, and water. After the initial imager reclassification, the results were extracted and the surfaces were reclassified as either pervious or impervious cover. Figure 3.3 shows the pervious and impervious surfaces within the City's storm drainage system area.

  The analysis of the satellite imagery was used to determine that relative percent perviousness and imperviousness of the subcatchments in the hydraulic model. However, the hydraulic modeling software used for this study (described in Chapter 4) requires percent directly connected impervious area (DCIA) values to estimate the volume of runoff, rather than percent imperviousness.

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Figure 3.3 Impervious and Pervious Land Areas

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- *Directly Connected Impervious Area.* The basin proportion of directly connected or effective impervious area is related to land use, storm water drainage system configuration, and recurrence interval. If runoff from an impervious area flows directly into a concentrated flow path, i.e. into a gutter, it is considered directly connected. If it flows over a pervious area before becoming a concentrated flow, it is unconnected. Rainfall on impervious surfaces is not subject to losses by infiltration into the soil; the only losses in impervious areas are due to depression storage. All initial losses for impervious areas, typically 0.02 to 0.08 inches, were assumed to be satisfied by precipitation preceding the design storm.

  The imperviousness derived from the satellite imagery represents the total average impervious area in a subbasin. To convert average percent imperviousness to DCIA, two equations were used. The following equation (developed by the U.S. Department of Agriculture [USGS]) was used for total impervious areas between 10-percent and 50 percent:

  (1) $\%DCIA = 3.6 + 0.4*I$

  Where: I = percent total impervious area (between 10 percent and 50 percent)

  For all remaining subcatchments the following equation was used[2]:

  (2) $\%DCIA = 0.04*I^{1.7}$

  Where: I = percent total impervious area (<10 percent and >50 percent)

- *Non-Effective Percent Imperviousness.* In residential urban areas, either a portion of the pervious runoff area has no flow path to the drainage system, or the flow path is via groundwater drains, which effectively delays runoff until it does not contribute to the design hydrographs. These areas are typically backyards, swimming pools, dense shrub landscaping, and gardens.
- *Pervious Area Runoff and Infiltration Parameters.* The remaining runoff originates from pervious areas. There are several ways to estimate the volume and/or the rate of infiltration of water into a soil. Three excellent estimation methods are Green-Ampt, Soil Conservation Service (SCS) method, and Horton's method. All of these equations provide a relatively accurate assessment of the infiltration characteristics of the soil in question. Infiltration into the soil in pervious areas was estimated for each subbasin by the model using the Horton equation. Horton and Green-Ampt are widely used in SWMM, especially when using SWMM runoff module. The Green-Ampt method accounts for multiple variables that other methods, such as Horton, do not. The Green-Ampt method is a function of the soil solution head, porosity, hydraulic conductivity, and time. Some of these parameters are difficult to estimate.

On the other hand, the Horton equation is an empirical formula that states that infiltration starts at a given rate and decreases exponentially with time. After a period of time when the soil saturation level reaches a certain value, the rate of infiltration will become constant. Parameters for the Horton equation can be reasonably estimated from literature and USDA soil data.

[2] Sutherland, R.C. (Fall 1995). Methodology for Estimating the Effective Impervious Area of Urban Watersheds. Watershed Protection Techniques. Vol. 2, No. 1 (Fall 1995)

Because the Horton parameters vary depending on soil type, soil maps were examined to determine the soil type within each drainage area. Weighted average soil properties were determined for each SWMM model subbasin based on the amount of each hydrologic soil group in the subbasin, and typical soil properties for each group.

Four hydrologic soil groups are used. The soils are classified based on water intake at the end of long duration storms after prior wetting, an opportunity for swelling, and without the proactive effects of vegetation. The hydrologic soil groups are:

A. Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.
B. Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes the downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D. Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with high swelling potential, soils with permanent high water table, soils with claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

By determining the percentages of each hydrologic soil group within a subbasin, maximum and minimum infiltration rates were calculated. The constant decay rate for Horton infiltration analysis was set to 0.0015 per second.

Figure 3.4 shows the hydrologic soil groups within the City, which are based on data provided by the Natural Resources Conservation Service (NRCS). As shown in Figure 3.4, the dominant Hydrologic Soil Group with the study area is Hydrologic Soil Group A, with portions portion designated as Hydrologic Soil Groups C and D. Areas on Figure 3.4 without a soil type were assigned Hydrologic Soil Group A. Each soil group is associated with the typical infiltration soil properties as listed in Table 3.16.

- *Overland Flow.* The overland flow travel time is affected by the type of surface cover, and can be modeled using a different Manning's coefficient (n) for various surface types. For each SWMM unit, roughness coefficients were input into the model for both pervious and impervious surfaces. Typical roughness coefficients, based on the types of ground cover, are shown in Table 3.17.

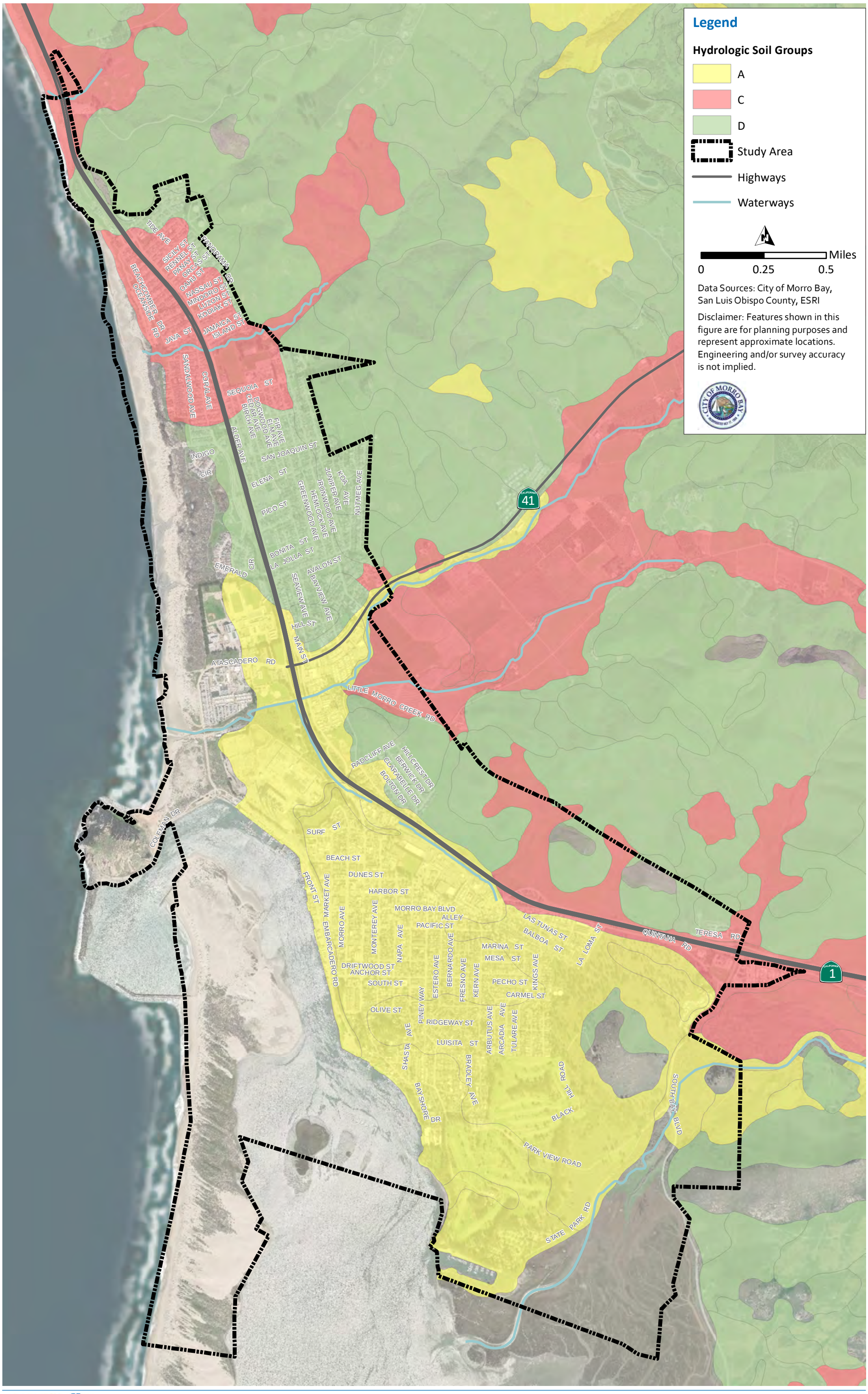

Figure 3.4 Hydrologic Soil Groups

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Table 3.16 Infiltration Rates for NRCS Hydrologic Soil Groups

Soil Group	Maximum Infiltration Rate (inches/hour)	Minimum Infiltration Rate (inches/hour)
A	2.0	0.065
B	1.5	0.050
C	1.0	0.035
D	0.5	0.20

Table 3.17 Parameters of Overland Flow

Surface	Overland Manning's n	Distance/Range (ft)
Pavement – smooth	0.02	50 – 200
Pavement – rough/cracked	0.05	50 – 200
Bare soil – newly graded areas	0.10	100 – 300
Range – heavily grazed	0.15	100 – 300
Turf – 1-2 in lawns/golf courses	0.20	100 – 300
Turf – 2-4 in parks/medians/pasture	0.30	200 – 500
Turf – 2-6 in natural grassland	0.40	200 – 500
Residential Landscaping	0.60	100 – 300
Few trees – natural grass undergrowth	0.50	300 – 600
Scattered trees – weed/shrub undergrowth	0.60	300 – 600
Numerous trees – dense undergrowth	0.80	300 – 600

- *Depression Storage in Pervious and Impervious Areas.* Depression storage is a volume that must be filled prior to the occurrence of runoff on pervious and impervious areas and is often used as a calibration parameter. Depression storage is input into the model as an average depth over the entire drainage area. Because this value is difficult to estimate, trial depression storage values were initially assumed for pervious and impervious portions of the City and then adjusted, where necessary, during hydraulic model development.
- *Ground Slope.* Ground slopes were determined using the City's elevation data and ArcView GIS. An average overland flow path slope is required for each hydraulic model subbasin. This value was automatically determined through intersection of subbasin areas with the Digital Elevation Model (DEM) provided by the City. The elevation grid was intersected with the subbasins and the slope of each grid cell within the subbasin was calculated. Using the number of cells within each subbasin, the average basin slope was calculated.

3.3.2 Hydraulic Criteria

This section describes the hydraulic criteria used to evaluate the City's storm drainage system.

3.3.2.1 Pipe Flow

The Manning coefficient 'n' is a friction coefficient and varies with respect to pipe material, smoothness of pipe and joints, and buildup of debris or other obstructions like root intrusion. For

storm drain pipes, the Manning coefficient typically ranges between 0.012 and 0.015 depending on material type. The Manning's n value for all storm drains was assumed to be 0.015 for the hydraulic analysis for most gravity mains, with the exception of corrugated metal pipes. These pipes were assigned a manning's roughness of 0.018, because according to the City, these pipes are generally in bad condition, This is a conservative estimate for Manning's n value, but is reasonable considering the age of some pipes in the drainage system.

3.3.2.2 Channel Flow

Manning's equation is used to derive travel time, velocity, flow, and width relationships for channels within the hydraulic model. The modeling software calculates ditch or channel travel time using entered values of slope, width, bank side slope, and Manning's n.

3.3.2.3 Surcharge Depth and Street Flooding

Storm drains are designed to surcharge under normal operation. It is common engineering practice to allow street curbs and gutters to act as storage and conveyance of storm water, similar to overland flow, for a given rainfall intensity and duration in order to protect adjacent properties from flooding. When evaluating the adequacy of the exiting conveyance facilities serving existing developments, City streets were allowed to flood and provide flow attenuation and storage capacity, thus avoiding cost-prohibitive improvements. Floodwaters were permitted to accumulate in streets up to 1-foot above the finished grade elevation for the 100-year storm. Floodwaters were generally not allowed to accumulate in the streets during the 10-year event, however in some cases the City's storm drain system is designed to convey water from an open channel or culvert through a downstream gutter. In these cases, floodwaters were not allowed to exceed the depth of the gutter.

3.3.2.4 Gutter Flow

Some areas of the City's storm drain system are designed to convey water from an open channel or culvert through a downstream gutter. Storage in streets/gutters was simulated in the model by adding a "conduit" in the model with the cross sectional shape of the roadway, which was estimated based on the best available information. These areas were allowed to flood or pond up to a depth of 1-foot during the 100-year event.

3.3.3 Low Impact Development Requirements

Urban runoff is a leading cause of pollution throughout the Central Coast region. As natural vegetated pervious ground cover is converted to impervious surfaces such as highways, streets, rooftops and parking lots, natural purification characteristics of pervious areas are lost. In addition, urban development creates new pollution sources leaving the developed urban area significantly greater in pollutant load as compared to the pre-development runoff.

Low Impact Development (LID) is an effective approach to managing stormwater to minimize the adverse effects of urbanization and development. LID strategies can achieve significant reductions in pollutant loading and runoff volumes as well as greatly enhanced groundwater recharge rates. The proper implementation of LID techniques results in greater benefits than single purpose stormwater and flood control infrastructure.

To protect downstream receiving water quality, downstream erosion potential, groundwater recharge potential, and stream habitat, the Central Coast Regional Water Quality Control Board

(RWQCB) adopted Resolution R3-2013-0032 requiring the City to implement Post-Construction stormwater management for new and redevelopment projects.

The City's Stormwater Management Guidance Manual for Low Impact Development & Post-Construction Requirements was initially published in March 2014, and was updated in July 2017. The City published two separate manuals. The "EZ Manual" is intended for single-family residential projects, with a net impervious area of less than 15,000 square feet. The "Main Manual" is intended for all other projects. Appendix E contains a copy of both the Main Manual as well as the EZ Manual. The primary objective of the Stormwater Management Manuals are to minimize the downstream impact of increased stormwater runoff that often occurs as the result of development or redevelopment projects. The requirements provided in the Stormwater Management Manuals are based on RWQCB Resolution R3-2013-0032 and its attachments.

The Stormwater Management Manuals include performance requirements that must be met by new/redevelopment projects in the City. The number of performance criteria that must be met depends on the type of project to be constructed. For example, a single-family residential project with less than 15,000 square feet is requirement to meet Performance Requirement No. 1, which includes the following:

- Limit disturbance of creeks and natural drainage features
- Minimize compaction of highly permeable soils
- Limit clearing and grading of native vegetation at the site to the minimum area needed to build the project, allow access, and provide fire protection
- Minimize impervious surfaces by concentrating improvements on the least sensitive areas of the site, while leaving the remaining land in a natural undisturbed state.
- Minimize stormwater runoff by implementing one or more of the following design measures:
  - Direct roof runoff into cisterns or rain barrels for reuse
  - Direct roof runoff onto vegetated areas safely away from building foundations and footings
  - Direct runoff from sidewalks, walkways, and/or patios onto vegetated areas safely away from building foundations and footings
  - Direct runoff from driveways and/or uncovered parking lots onto vegetated areas safely away from building foundations and footings
  - Construct bike lanes, driveways, uncovered parking lots, sidewalks, walkways, and patios with permeable surfaces

In total, there are five performance requirements that must be implemented depending on the type of project to be constructed. The City also allows for an alternate means of compliance for projects where the performance requirements are technically infeasible, including off-site source control measures or through the use of the Ten Percent Adjustment to Retention Requirement. Refer to Appendix E for more detail on the performance requirements and the alternate means of compliance.

Chapter 4

WATER DEMANDS AND WASTEWATER FLOWS

This chapter summarizes the historical, existing and future water demands and wastewater flows.

4.1 Water Demands

This section summarizes the City of Morro Bay's (City's) historical water consumption and production records used to determine the daily, monthly, and seasonal fluctuations experienced by the water system. Also summarized are the average Water Duty Factors (WDFs), peaking factors, and the projected demands through the year 2040.

4.1.1 Historical Water Demands

The City provided historical monthly production and consumption data for the years 1997 through 2016. Detailed monthly customer billing data was also provided for the years 2013 through 2016. The historical production and consumption data were evaluated to characterize the unique water use patterns of the City's customers.

Several key demand parameters were generated, including peaking factors, typical water use by customer class, per capita water demands, and unaccounted-for-water (UFW). The parameters were used as the basis for the existing demand estimates and demand projections.

4.1.1.1 Historical Water Production

Water production varies annually in response to customer water usage, which is correlated to weather, development, economic conditions, population, and conservation activities. The City's total annual production, as shown in Table 4.1, has been overall trending downward since 1997, from 501 million gallons (MG) in 1997 to 324 MG in 2016. Figure 4.1 illustrates the City's water production from 1997 to 2017. As shown on Figure 4.1, the City saw a significant drop in water production every year since 2014, which is attributable to water conservation associated with state mandates/extreme drought conditions.

4.1.1.2 Seasonal Demands and Peaking Factors

Peaking factors represent the seasonal and daily variations in water use, above or below the average day water demand. The various peaking conditions are either statistical concepts or numerical values established through a review of historical data and are, at times, adjusted to reflect a level of conservatism.

Peaking conditions that are of particular significance to hydraulic analysis of the water system include the average day demand (ADD), maximum day demand (MDD), and the peak hour demand (PHD). The ADD and maximum month demand (MMD) were developed based on the monthly production data provided by the City. The MDD and PHD were then developed based on typical peaking factors. Figure 4.1 shows the historical ADD and MMD from 1997 through 2016.

Table 4.1 Historical Water Production

Year	Total Production (MG)	Average Day Demand (mgd)	Maximum Month Demand (mgd)	MMD/ADD Peaking Factor
1997	501	1.37	1.82	1.33
1998	432	1.18	1.53	1.29
1999	454	1.24	1.55	1.24
2000	456	1.25	1.50	1.20
2001	459	1.26	1.54	1.22
2002	473	1.30	1.54	1.19
2003	465	1.28	1.57	1.23
2004	481	1.32	1.67	1.27
2005	444	1.22	1.46	1.20
2006	447	1.22	1.52	1.25
2007	460	1.26	1.53	1.22
2008	452	1.23	1.52	1.23
2009	447	1.23	1.51	1.24
2010	408	1.12	1.38	1.23
2011	405	1.11	1.33	1.20
2012	392	1.07	1.29	1.20
2013	415	1.14	1.36	1.20
2014	385	1.05	1.21	1.15
2015	354	0.97	1.07	1.11
2016	324	0.88	1.03	1.17

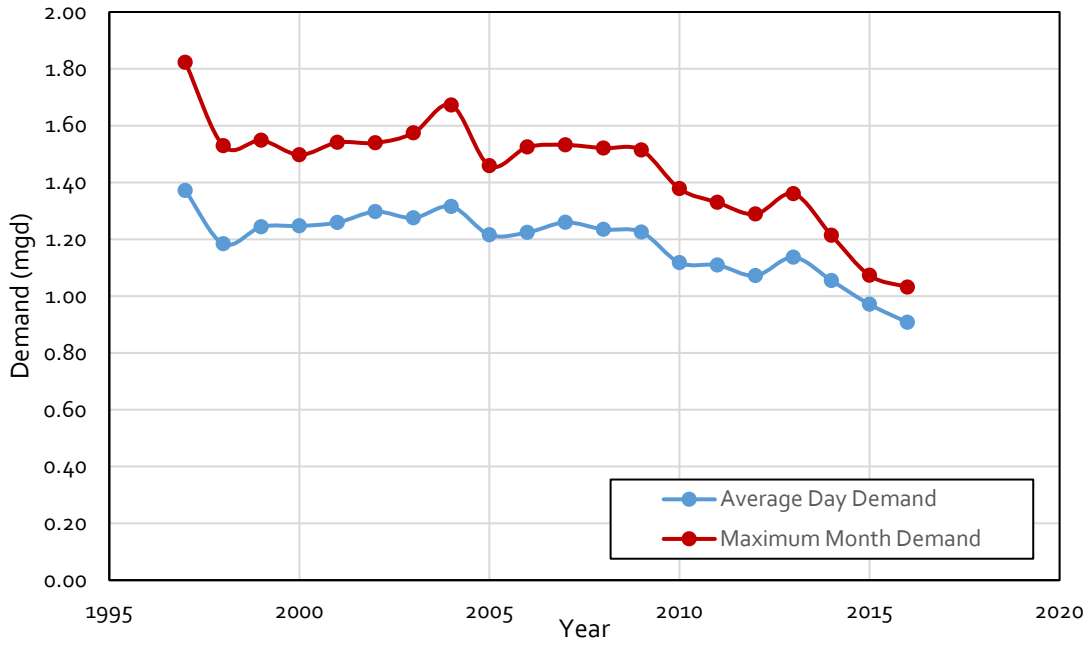


Figure 4.1 Historical Water Production

Average Day Demand

The ADD represents the daily average demand for the entire year. It is calculated by dividing the total water produced in any given year by the number of days per year. These values for the years 1997 through 2016 are presented in Table 4.1. As shown in this table, the ADD ranged from 1.37 million gallons per day (mgd) in 1997 and 0.91 mgd in 2016. The pre-drought (2010 through 2014) ADD was approximately 1.10 mgd. This value is more representative of the "existing" 2017 ADD than the 5-year average (2012 through 2016) because 2015 and 2016 have low ADD due to drought conservation measures.

Maximum Month Demand

The MMD is defined as the highest production in one month in a given year. The historical MMD and MMD/ADD peaking factors for years 1997 through 2016 are presented in Table 4.1. The MMD peaking factors ranged from 1.33 in 1997 to 1.14 in 2016. A MMD/ADD peaking factor of 1.25 was used to develop the existing and future MMDs for this OneWater Plan, which represents the highest MMD/ADD peaking factor seen in the system since 2006.

Maximum Day Demand

The MDD is an important demand condition, and is used to evaluate system supply, reservoir capacity, and pump station capacity. The MDD is defined as the highest production in one day in a given year, and usually occurs in the summer. Daily production/consumption data was not available, therefore, a MMD/MDD peaking factor of 1.3 was assumed. Based on the existing MMD peaking factor of 1.25, this yields an overall MDD/ADD peaking factor of 1.63.

Peak Hour Demand

The PHD is the highest water demand during any one-hour period of the year. A normal day typically experiences two peak demands, in the morning and then evening. The PHD is

expressed as a multiplier applied to the ADD. PHD simulates model high water use throughout the system and assist in identifying areas of the distribution system that experience low pressures.

The City's supervisory control and data acquisition (SCADA) data was used to develop a typical diurnal curve for the City's water system. The diurnal pattern was developed as part of the model calibration process, which is presented in Chapter 5. The calculated hourly diurnal peaking factor from this data was estimated to be 1.54.

The City's PHD/ADD peaking factor was developed by multiplying the MDD/ADD peaking factor of 1.63 by the typical daily diurnal peak of 1.54. This yields an estimated PHD/ADD peaking factor of 2.51.

4.1.1.3 Per Capita Water Demand

The per capita consumption rate is used for estimating the City's future water requirements, evaluating the adequacy of the supply source, and determining storage needs. The consumption rate, expressed in gallons per capita per day (gpcd), is applied to the projected population to yield future water requirements.

Historical City residential per capita water use is calculated by dividing the City's total production by the total population. Table 4.2 shows the historical water production and per capita consumption from 1997 to 2016. As shown in Table 4.2, the City's per capita demand ranged from 85 gpcd in 2016 to 141 gpcd in 1997, averaging 106 gpcd for 2010 through 2014 (pre-drought) and 87 gpcd for 2015 through 2016 (drought).

The City's 2015 Urban Water Management Plan (UWMP) includes information on per capita demand projections. The 2015 UWMP includes a projected per capita demand of 116 gpcd. The 5-year pre drought per capita demand average (2010-2014) was calculated to be 106 gpcd, as shown on Figure 4.2. As previously mentioned, the City's per capita demand has decreased significantly following the drought. The 2015 through 2016 per capita demand average was 88 gpcd. This OneWater Plan uses the 2010-2014 average per capita demand as the basis for demand projections. This assumption provides a level of conservatism over current per capita demands, but is more realistic than the 2015 UWMP per capita demand projection assumption of 116 gpcd.

4.1.1.4 Historical Metered Water Consumption

Historical metered water consumption data for the years 2013 through 2016 were obtained from the City's billing records and are presented in Table 4.3. As shown in Table 4.3, the City's total annual consumption in 2013 was 1.05 mgd. The City's metered consumption decreased significantly in 2014 through 2016. This is associated with the extreme drought conditions in California from 2014 to 2016.

Table 4.2 Historical Per Capita Water Demand

Year	Population	Average Day Demand (mgd)	Per Capita Demand (gpcd)
1997	9,696	1.37	141
1998	9,845	1.18	120
1999	9,871	1.24	126
2000	10,410	1.25	120
2001	10,486	1.26	120
2002	10,510	1.30	123
2003	10,510	1.28	121
2004	10,522	1.32	125
2005	10,270	1.22	118
2006	10,491	1.22	117
2007	10,436	1.26	121
2008	10,506	1.23	118
2009	10,555	1.23	116
2010	10,234	1.12	109
2011	10,296	1.11	108
2012	10,331	1.07	104
2013	10,427	1.14	109
2014	10,547	1.05	100
2015	10,601	0.97	92
2016	10,714	0.91	85

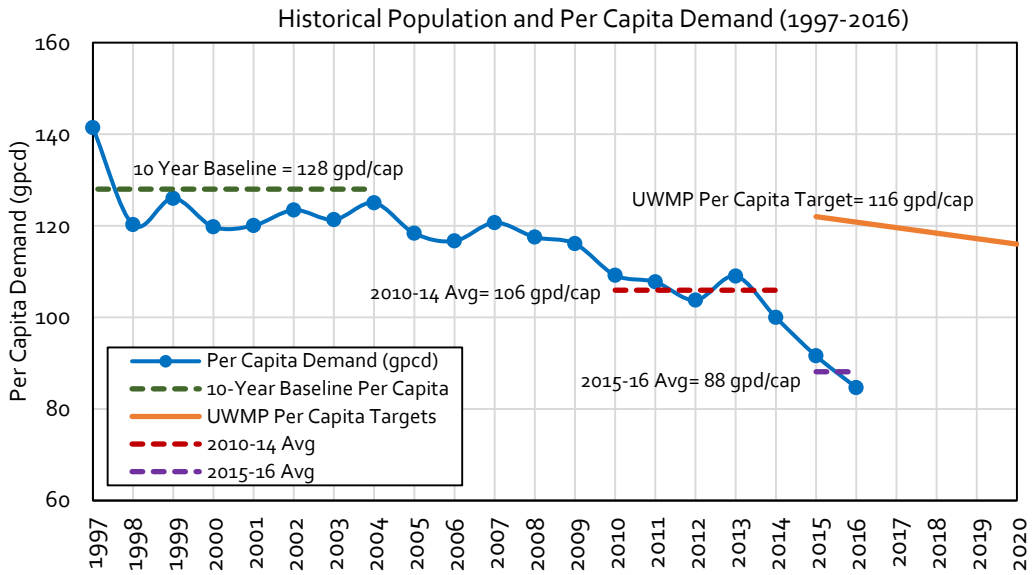


Figure 4.2 Historical Per Capita Water Demand

Table 4.3 Historical Metered Water Consumption

Year	Metered Consumption (mgd)
2013	1.05
2014	1.01
2015	0.93
2016	0.89

4.1.1.5 Unaccounted-For-Water

The difference between water production (supply) and consumption (billed to customers) is defined as unaccounted for water (UFW), or water loss. Water loss may be attributed to leaking pipes, unmetered or unauthorized water use, inaccurate meters, treatment losses, or other events causing water to be withdrawn from the system and not measured. Specific events that cause water loss can include hydrant flushing, street cleaning, system flushing, and firefighting. The City's estimated unaccounted for water is for the years 2013 through 2016 is summarized in Table 4.4.

Table 4.4 Unaccounted for Water

Year	Metered consumption (MG)	Total Production (MG)	Unaccounted for water	
			(MG)	(Percent of Production)
2013	386	415	28	6.8%
2014	370	385	14	3.8%
2015	338	354	16	4.6%
2016	332	324	8	2.4%

4.1.1.6 Water Demand Factors

A water demand factor (WDF) is defined as the estimated amount of water usage per area for a certain land use type. These factors are used to estimate the ADD for the potential development areas by multiplying the WDF with the total number of acres of each land us category. WDFs were developed as a part of this OneWater Plan to project demands for developable vacant land. WDFs are typically determined from a combination of geocoded billing records and land use information using special geographic information system (GIS) routines. The WDFs shown in Table 4.5 were developed by grouping the existing developed land use parcels by categories that correspond to the City's customer classes.

Table 4.5 Water Demand Factors

Grouped Land Use	Developed Acres	Water Demand Factor	Average Day Demand (mgd)
Residential	642.4	1,180	0.758
Commercial	277.9	700	0.195
Industrial	32.2	525	0.017
Waterfront Commercial/Industrial(1)	6.3	700	0.004
Mixed Use	33.8	1,000	0.034
Open Space & Agricultural	7,569.5	0	0
Public/Institutional	152.6	850	0.130

Notes:
(1) Total excludes Costal Dependent Industrial, which is not anticipated to connect to the system.

4.1.1.7 Existing Water Demands by Pressure Zone

In order to evaluate the capacity of the water system using the hydraulic model, demands associated with each individual connection were allocated into the hydraulic model to specific model nodes. The demands for specific model nodes were calculated by first geocoding the City's billing data using GIS techniques and then assigning the geocoded billing data into the hydraulic model. More information related to the allocation of water demands within the hydraulic model is provided in Chapter 5. The resulting demands by pressure zones are summarized in Table 4.6.

Table 4.6 Existing Demand by Pressure Zone

Pressure Zone	ADD (mgd)	MDD(1) (mgd)	Percent of Total Demand (%)
Blanca	0.18	0.29	15.6
Elena	0.14	0.24	12.7
Lower Kings	0.22	0.35	19.0
Nutmeg	0.11	0.17	9.3
Ridgeway	0.08	0.13	7.2
Upper Kings	0.41	0.67	36.1
Total	**1.136**	**1.846**	**100**

Notes:
(1) MDD/ADD peaking factor = 1.63

4.1.2 Water Demand Projections

The planning horizon for this OneWater Plan is the year 2040, and according to the City's planning documents, modest growth is expected until then to comply with the population growth limits imposed by Measure F. The City's population is projected to increase to 12,149 people by year 2040.

Table 4.7 summarizes the projected demands through the year 2040. As shown in Table 4.7, the City's ADD and MDD are projected to approach 1.29 mgd and 2.09 mgd by 2040, respectively. The projected demands provided in Table 4.7 assume a per capita water demand of 106 gpcd,

which is equal to the average per capita demand from 2010 to 2014. This is likely a conservative assumption, because per capita waster demand is expected to decrease in the future due to increased conservation measures and more water efficient construction associated with new development. Because the purpose of OneWater is to size infrastructure needs for the distribution system, it is appropriate to include a level of conservatism when planning future transmission, distribution, and storage needs. Table 4.8 summarizes the distribution of demands by pressure zone in 2040.

The City's 2015 Urban Water Management Plan (UWMP) identified a 2020 per capita demand target of 116 gpcd. Figure 4.3 shows the projected ADD based on the 2010-2014 per capita demand of 106 gpcd, a high range ADD projection based on the UWMP per capita target of 116 gpcd, as well as a low range demand projection based on the 2015-2016 per capita demand of 87 gpcd. This OneWater Plan assumes a per capita demand of 106 gpcd.

Table 4.7 Future Demand Projections through 2040

Year	Projected ADD (mgd)	Projected MDD[(1)] (mgd)
2017 (present)	1.14	1.85
2020	1.16	1.88
2025	1.19	1.93
2030	1.22	1.99
2035	1.25	2.04
2040	1.29	2.09

Notes:
(1) MDD/ADD peaking factor = 1.63

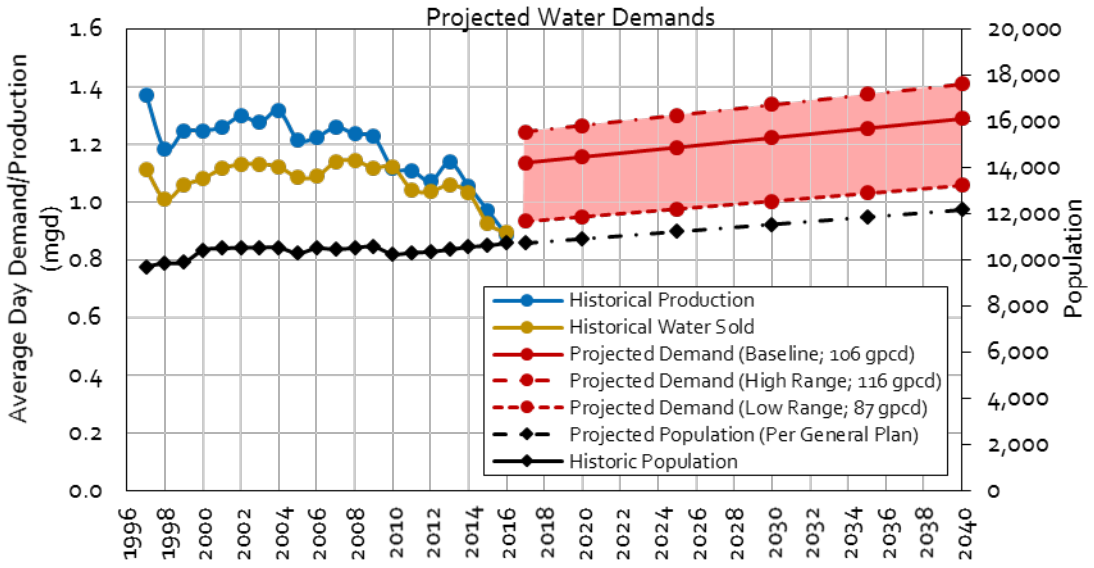


Figure 4.3 Projected Average Day Demand

Table 4.8 2040 Demand by Pressure Zone

Pressure Zone	ADD (mgd)	MDD[1] (mgd)	Percent of Total Demand (%)
Blanca	0.19	0.31	14.8%
Elena	0.15	0.24	11.4%
Lower Kings	0.23	0.37	17.6%
Nutmeg	0.13	0.21	10.0%
Ridgeway	0.08	0.13	6.2%
Upper Kings	0.51	0.84	40.0%
Total	**1.29**	**2.09**	**100.0%**

Notes:
(1) MDD/ADD peaking factor = 1.63

4.2 Wastewater Flows

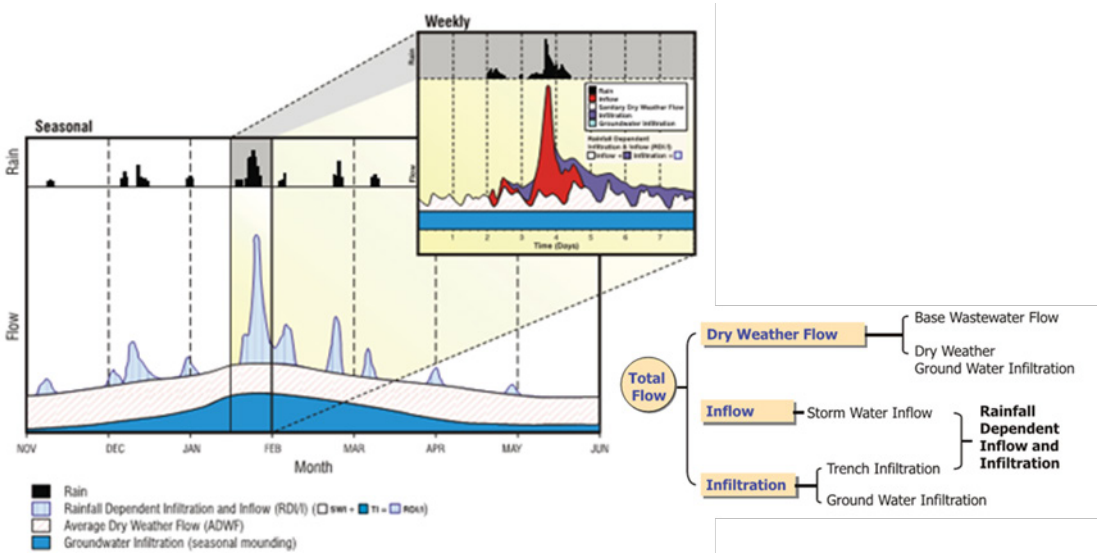
This section summarizes the existing and projected wastewater flows for the City's wastewater collection system. Historical wastewater flows, as well as flow data obtained as part of a temporary flow monitoring program are also presented.

4.2.1 Wastewater Flow Components

This section describes and provides definitions of commonly used terminology in the wastewater collection system analysis and evaluations conducted as part of this project. Wastewater flows vary according to the season. Dry weather flow (DWF) or base flow is flow generated by routine water usage in the residential, commercial, and industrial sectors of the collection system.

Another component of DWF is the contribution of dry weather groundwater infiltration (GWI) into the collection system. Dry weather GWI will enter the sewer system when the relative depth of the groundwater table is higher than the depth of the pipeline and when the susceptibility of the sanitary sewer pipe allows infiltration through defects such as cracks, misaligned joints, and broken pipelines. Figure 4.4 illustrates the various flow components of wastewater in general, which are described in detail in the following sections.

Wet weather flow (WWF) includes storm water inflow, trench infiltration, and GWI. The storm water inflow and trench infiltration comprise the WWF component termed infiltration/inflow (I/I). The response in the sewer system to rainfall is seen immediately (as with inflow) or within hours after the storm (as with infiltration).



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

Figure 4.4 Typical Wastewater Flow Components

- Base Wastewater Flow. The base wastewater flow (BWF) is the flow generated by the City's customers independent of wet weather influences. BWF is estimated by measuring flows during dry weather conditions. The flow has a diurnal pattern that varies depending on the type of use. Commercial and industrial patterns, though they vary depending on the type of use, typically have more consistent higher flows during business hours and lower flows at night. Furthermore, the diurnal flow pattern experienced during a weekend may vary from the diurnal flow experienced during a weekday.
- Average Annual Flow. The average annual flow (AAF) is the average flow that occurs on a daily basis throughout the year, including both periods of dry and wet weather conditions.
- Average Dry Weather Flow. The average dry weather flow (ADWF) is the average flow that occurs on a daily basis during the dry weather season. The ADWF includes the BWF generated by the City's residential, commercial, and industrial users, plus the dry weather GWI component.
- Peak Wet Weather Flow. Peak wet weather flow (PWWF) is the highest observed flow that occurs following a design storm event. Wet weather I/I cause flows in the collection system to increase. PWWF is typically used for designing sewers, lift stations, and some unit processes in a treatment plant. Therefore, the PWWF and the "Design Flow" are synonymous and will be used interchangeably throughout this report.
- Groundwater Infiltration. GWI is the result of extraneous water entering the sewer system through defects in pipes and manholes. GWI is related to the condition of the sewer pipes, manholes, and groundwater levels. GWI may occur throughout the year, although rates are typically higher in the late winter and early spring. Dry weather GWI (or base infiltration) cannot easily be separated from BWF by flow measurement techniques. Therefore, dry weather GWI is typically grouped with BWF.

- Rain Derived Infiltration and Inflow (RDII). Infiltration is defined as storm water flows that enter the sewer system by percolating through the soil and then through defects in pipelines, manholes, and joints. Examples of infiltration entry points are cracks in pipelines, misaligned joints, and root penetration. Inflow is defined as storm water that enters the sewer system via storm drain cross connections, leaky manhole covers, or cleanouts. Examples of inflow entry points are roof drain and downspout connections, leaky manhole covers, and illegal storm drain connections. Some of the most common sources of I/I are shown on Figure 4.5. The adverse effects of I/I entering the sewer system is that it increases both the flow volume and peak flows, as illustrated on Figure 4.6.

4.2.2 Wastewater Flows

This section summarizes the existing and projected wastewater flows for the City's wastewater collection system. Historical wastewater flows, as well as flow data obtained as part of a temporary flow monitoring program are also presented. Finally, future wastewater flow projections are presented.

4.2.2.1 Historical Wastewater Flows

Before discussing existing and future flows that were developed for this OneWater Plan, it is important to first take a look at historical flows and trends over the last several years. The City provided historical influent flow data at the wastewater treatment plant (WWTP) from 1997 through 2016. The data provided included monthly flow data from the City and Cayucos service area. For the purposes of this OneWater Plan, historical flows from the City were analyzed.

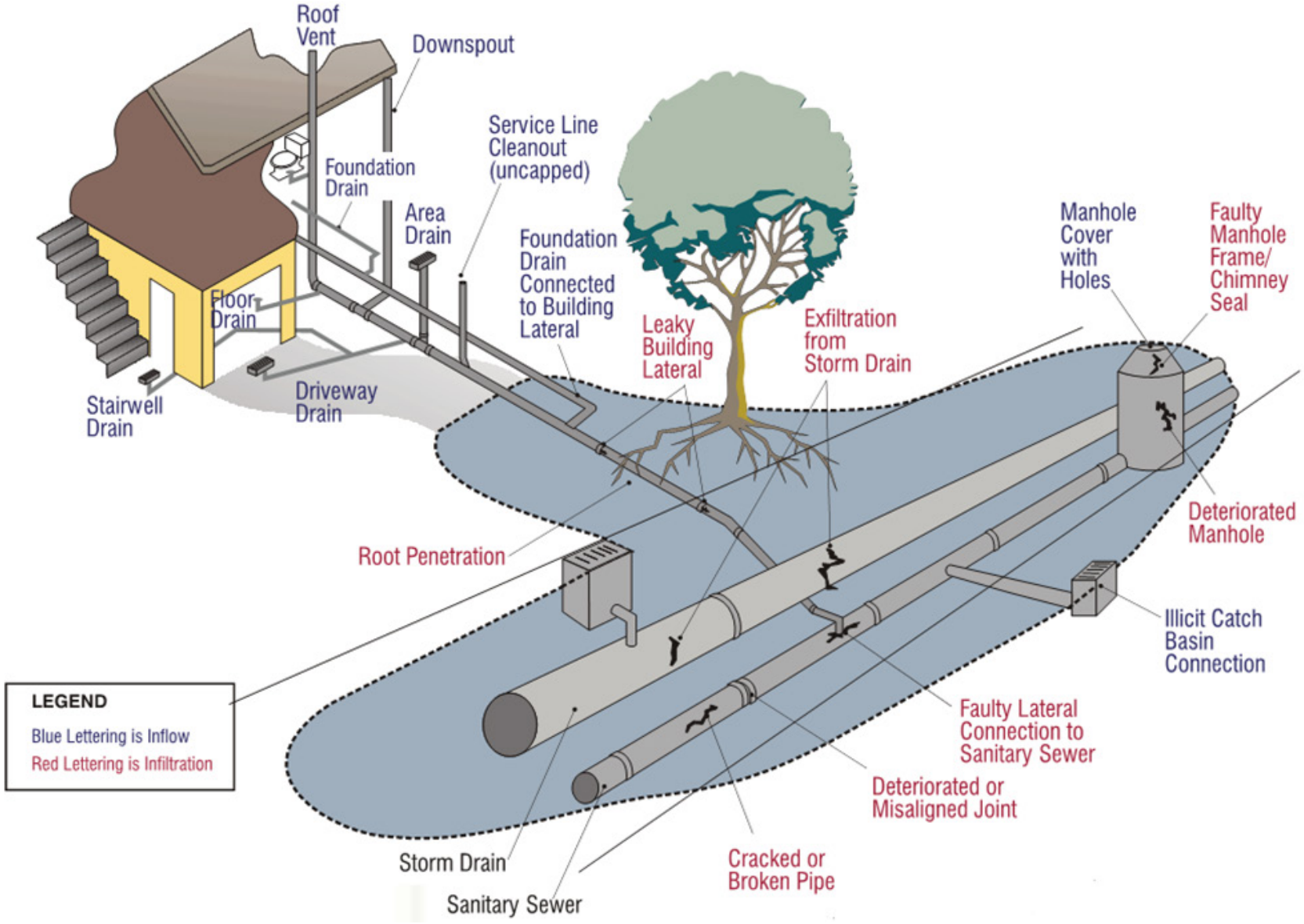


Figure 4.5 Typical Sources of Infiltration and Inflow

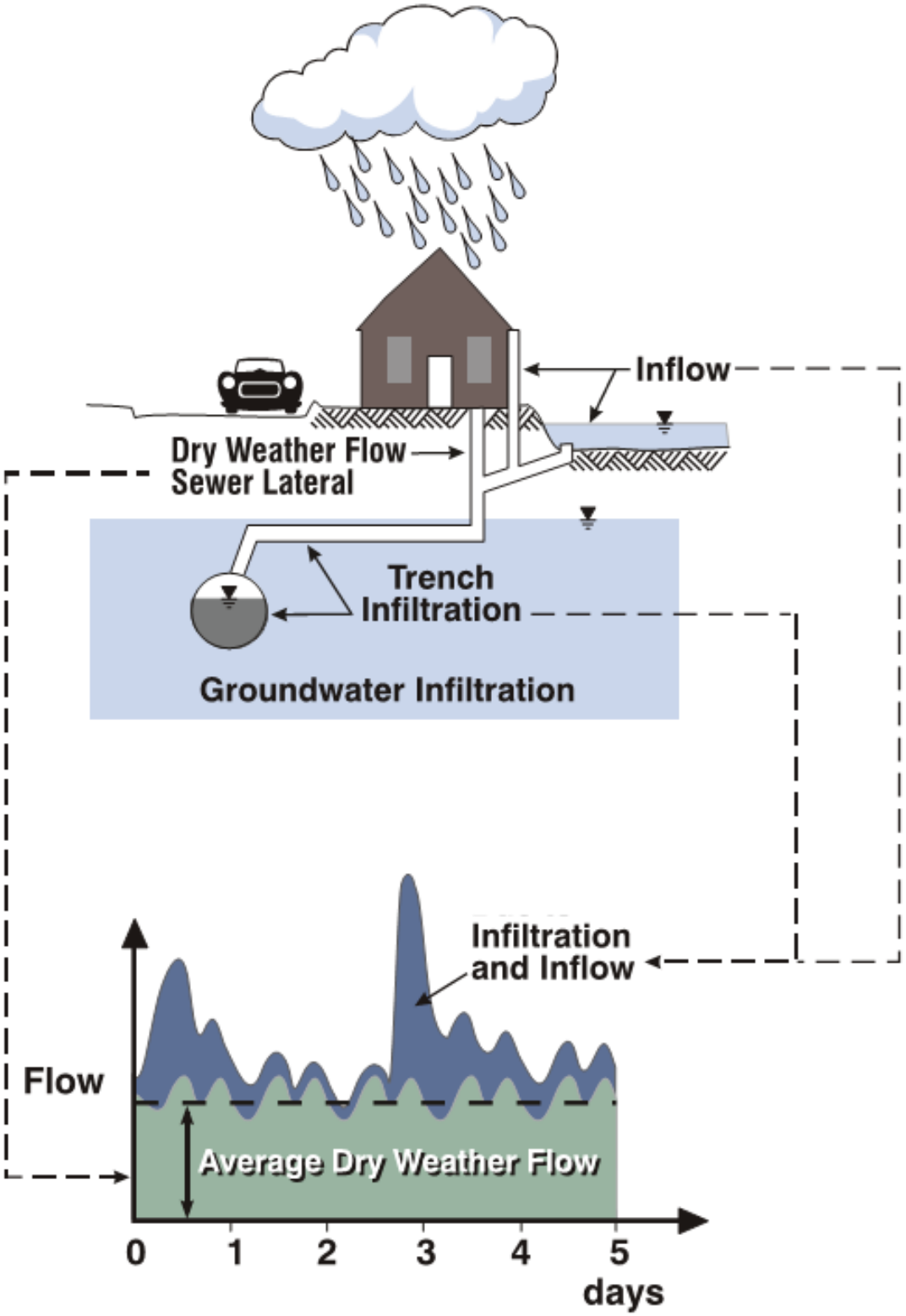


Figure 4.6 Typical Effects of Infiltration and Inflow

Figure 4.7 shows the historical ADWF and population for the City, while Table 4.9 summarizes the ADWF for the City. ADWF was calculated using the weighted average flow during the months May through September.

Between 1997 and 2016, the ADWF from the City decreased by approximately 0.73 million gallons per day (mgd). From 2002 and 2015 the flows have remained relatively consistent on an ADWF basis. The 5-year (2010-14) ADWF is roughly 0.85 mgd, while the maximum ADWF during that period is 0.94 mgd. Due to recent drought conditions, the 2-year (2015-16) is roughly 0.69 mgd.

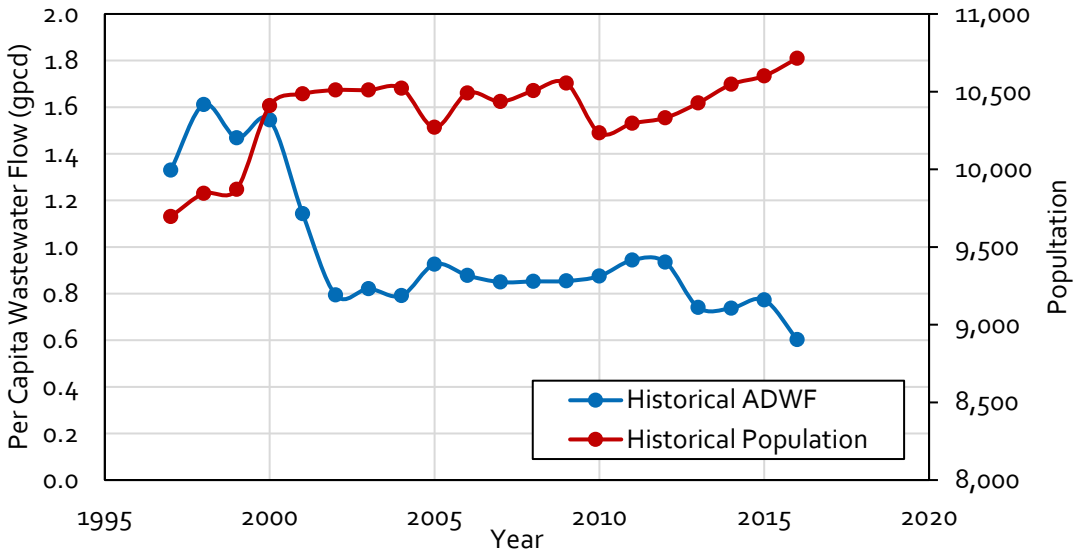


Figure 4.7 Historical City ADWF and Population

4.2.2.2 Per Capita Wastewater Flows

Table 4.9 summarizes the historical per capita wastewater generation within the City service area. As shown on Figure 4.8, the City's per capita wastewater flow has been decreasing since 1998, from 164 gallons per capita per day (gpcd) to 60 gpcd in 2016. The reduction in per capita flow can be attributed to increased conservation by the City customers, as well as additional conservation associated with recent drought conditions.

For planning purposes, it was assumed that a per capita flow of 82 gpcd would be used for future flow projections. The reason for this is that the 2015 and 2016 per capita flows may be artificially low due to state mandated water conservation. Therefore, the City may see a rebound in the per capita wastewater flow as drought conditions continue to ease.

Table 4.9 Historical Wastewater Flows at the WWTP

Year	City of Morro Bay Population[1]	Morro Bay ADWF[2][3] (mgd)	Per Capita Flow (gpcd)
1997	9,696	1.33	137
1998	9,845	1.61	164
1999	9,871	1.47	149
2000	10,410	1.55	148
2001	10,486	1.14	109
2002	10,510	0.79	76
2003	10,510	0.82	78
2004	10,522	0.79	75
2005	10,270	0.93	90
2006	10,491	0.88	84
2007	10,436	0.85	81
2008	10,506	0.85	81
2009	10,555	0.85	81
2010	10,234	0.87	85
2011	10,296	0.94	92
2012	10,331	0.94	91
2013	10,427	0.74	71
2014	10,547	0.74	70
2015	10,601	0.77	73
2016	10,714	0.60	56
2010-14 Avg.	**--**	**0.85**	**82**
2015-16 Avg.	**--**	**0.69**	**65**
2010-14 Max.	**--**	**0.94**	**92**

Notes:
(1) Source: California Department of Finance
(2) Source: City of Morro Bay Influent Data
(3) ADWF is defined as the weighted average flow during the months May through September

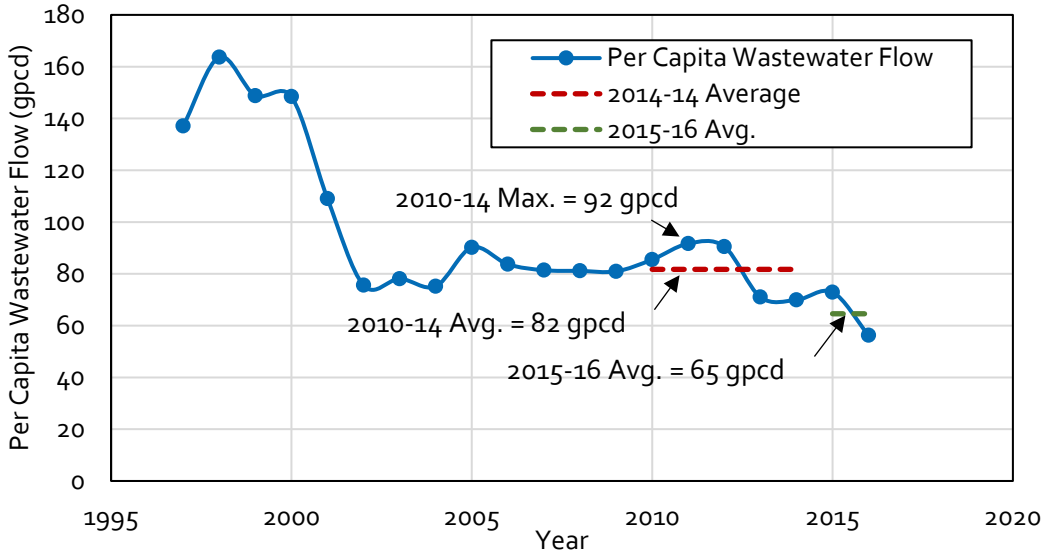


Figure 4.8 Historical Per Capita Flow

4.2.2.3 Existing Wastewater Flows

The existing ADWF was estimated using the ADWF for the 2010 through 2014. For the purpose of OneWater, the existing ADWF is 0.88 mgd. This is the ADWF generated within the City, although the City of Cayucos contributes to the City's total wastewater generation.

The estimated existing (2017) City sewer service area population is approximately 10,714. This equates to a per capita wastewater generation rate of about 82 gallons per capita per day (gpcd).

4.2.3 Temporary Flow Monitoring Program

This section describes the temporary flow monitoring program conducted as part of OneWater. The data and results from the flow monitoring program are summarized and discussed.

The City contracted with V&A Consulting Engineers (V&A) to conduct a temporary flow monitoring program within the City's wastewater sewer collection system. V&A prepared the stand-alone 2017 Sewer Flow Monitoring and Inflow/Infiltration Study, which is included in Appendix F.

Flow monitoring was performed over several periods from April 7, 2017 to July 13, 2017 at 15 flow monitoring sites. The City provided data from four City owned flow meters and the influent flume in order to provide data which reflects larger wet weather flows. The four City owned meters provided data from January 3, 2017 through February 27, 2017 and flume was from November 7, 2016 through July 18, 2017.

The temporary flow monitoring program helped develop design flow criteria, and correlate actual collection system flows to the hydraulic model predicted flows. Flow monitoring data was used to calibrate the wastewater collection system hydraulic model for dry weather and wet weather flow, and to help to identify areas of the system with the highest rates of I/I.

4.2.3.1 Flow Monitoring Sites and Tributary Areas

A total of 20 flow meters were used during the temporary flow monitoring program. 15 flow meters were installed by V&A at sites locations selected by Carollo. These flow meter sites were selected to best isolate and model the critical areas and subareas within the wastewater collection system. The other 5 sites included the City's 4 existing greyline meters, and the WWTP flume data. The flow monitoring sites are listed in Table 4.10 and shown on Figure 4.9. Figure 4.10 shows the basins used for the I/I analysis and Table 4.11 lists the V&A flow sites contained within these basins. Figure 4.11 provides a schematic illustration of the flow monitoring locations.

Table 4.10 Flow Monitoring Locations

Meter Name	Diameter (in)	Location
Site 1	11.75	Main St. ~225-feet south of Errol St.
Site 2	8	Atascadero Rd. ~85-feet east of Sunset Ave.
Site 3	15	Island between Main St. and Hwy 1 on-ramp, ~135-feet north of Atascadero Rd.
Site 4	17.5	Embarcadero, approximately 685-feet north of Colman Dr.
Site 5	8	371 Avalon St.
Site 6	13	Main St. approximately 290-feet south of Elena St.
Site 7	9.75	Coral Ave. approximately 630' north of Emerald Cir.
Site 8	13	Main St. approximately 400-feet south of Island St.
Site 9	12	125 Java St. (near intersection of Sandalwood Ave.)
Site 10	8	Main St. just north of intersection with Radcliff Ave.
Site 11	8	785 Quintana Rd.
Site 12	14.375	Parking lot of 1154 Embarcadero
Site 13	15.5	Sidewalk of 910 Embarcadero
Site 14	10	Intersection of Main St. and Marina St.
Site 15	10	301 Embarcadero approximately 90-feet north of Morro Bay Launching Facility
MB 1	12	Main St. approximately 225-feet south of Errol St.
MB 2	6	Atascadero Rd. approximately 85-feet east of Sunset Ave.
MB 3	15	Island between Main St. and Hwy 1 on-ramp, ~135-feet north of Atascadero Rd.
MB 4	18	Embarcadero approximately300-feet south of entrance to Morro Dunes mobile home park
Flume	27	WWTP 27-inch Pakmer-Bowlus Flume

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study.

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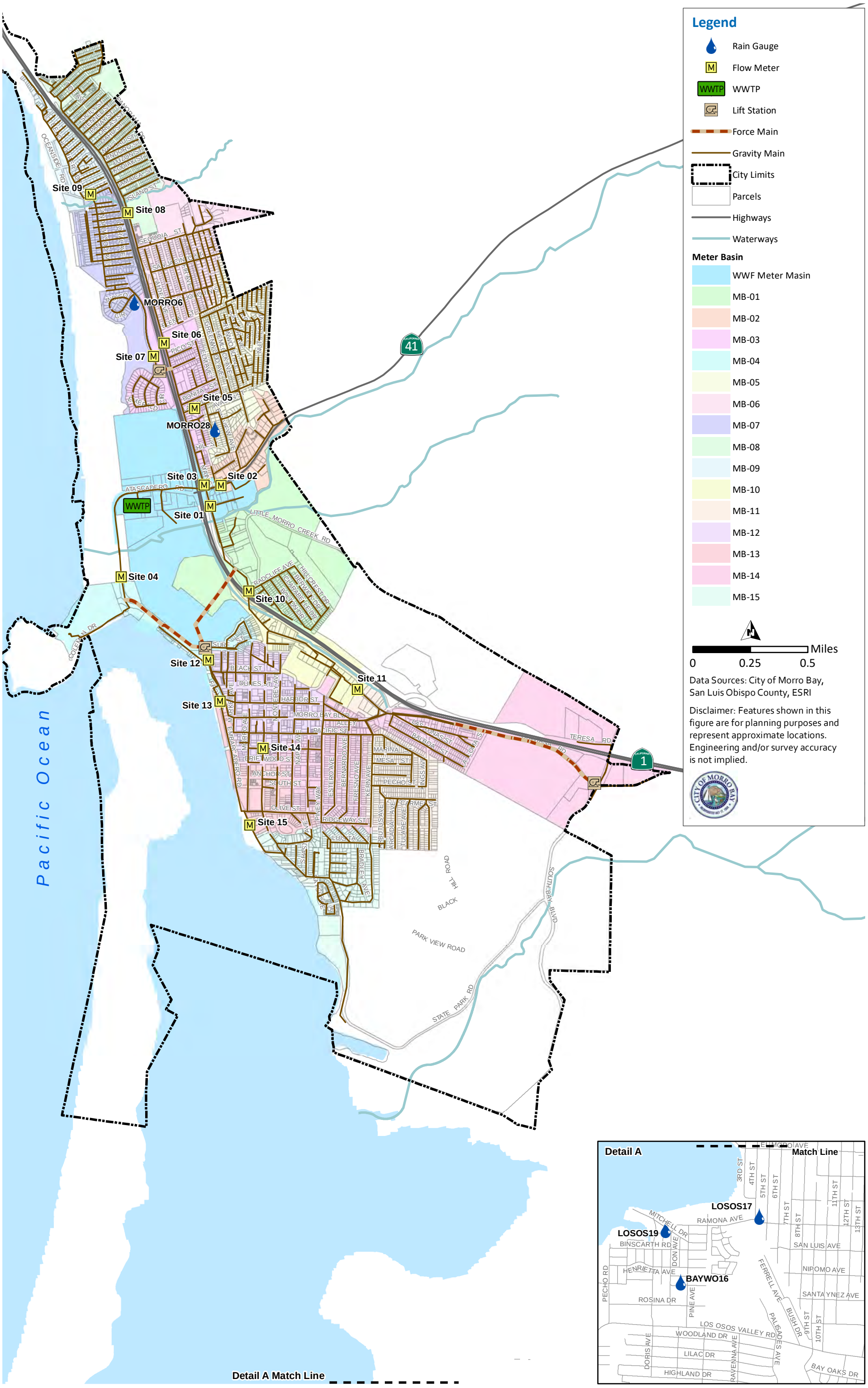

Figure 4.9 Temporary Flow Monitoring Locations

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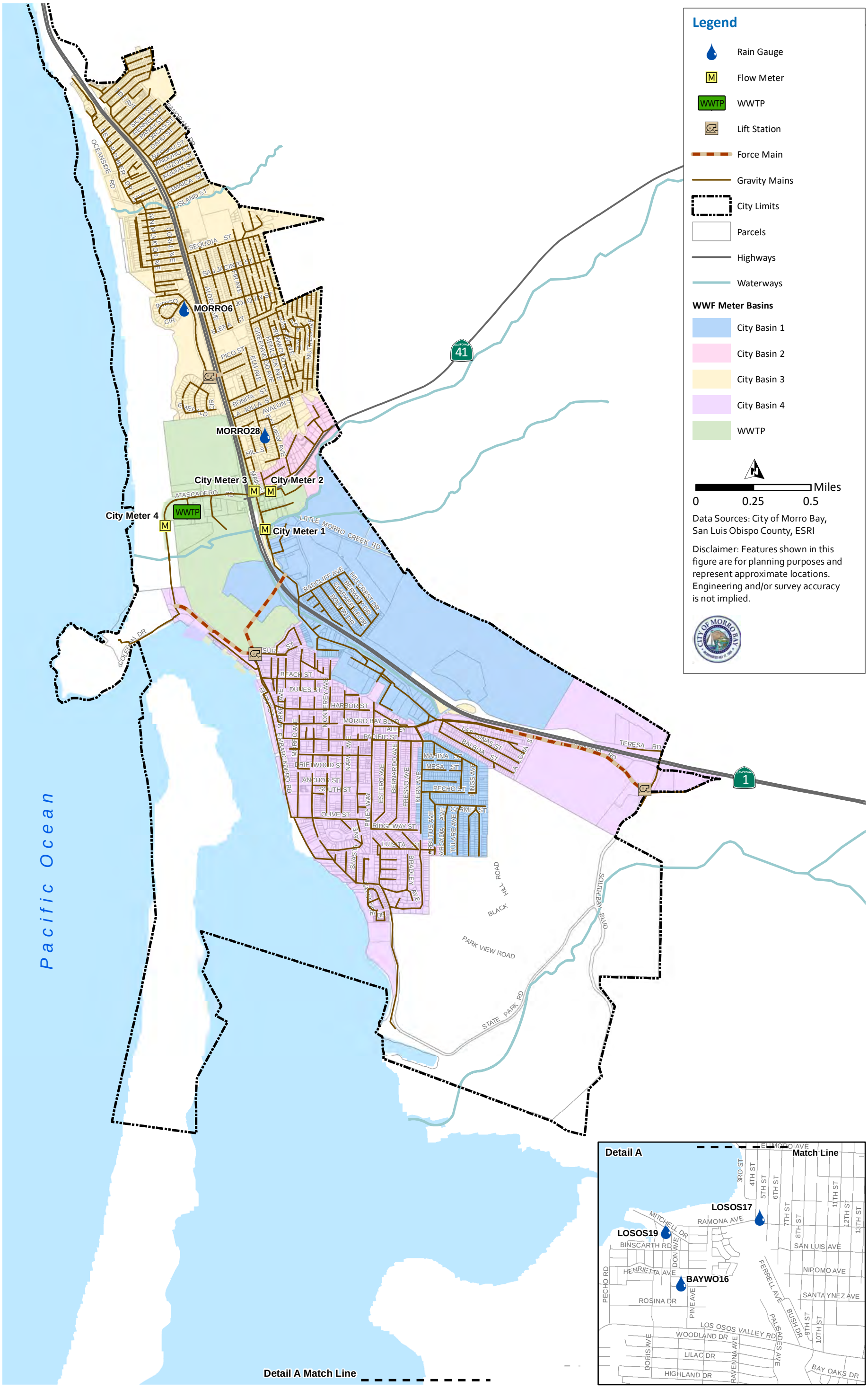

Figure 4.10 Flow Metering Basins for I/I Analysis

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Table 4.11 List of Basins Used in I/I Analysis

I/I Basin	V&A Sites
MB 1	Sites 1, 10, 11
MB 2	Site 2
MB 3	Sites 3, 5, 6, 7, 8, 9
MB 4	Sites 4, 12, 13, 14, 15

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study

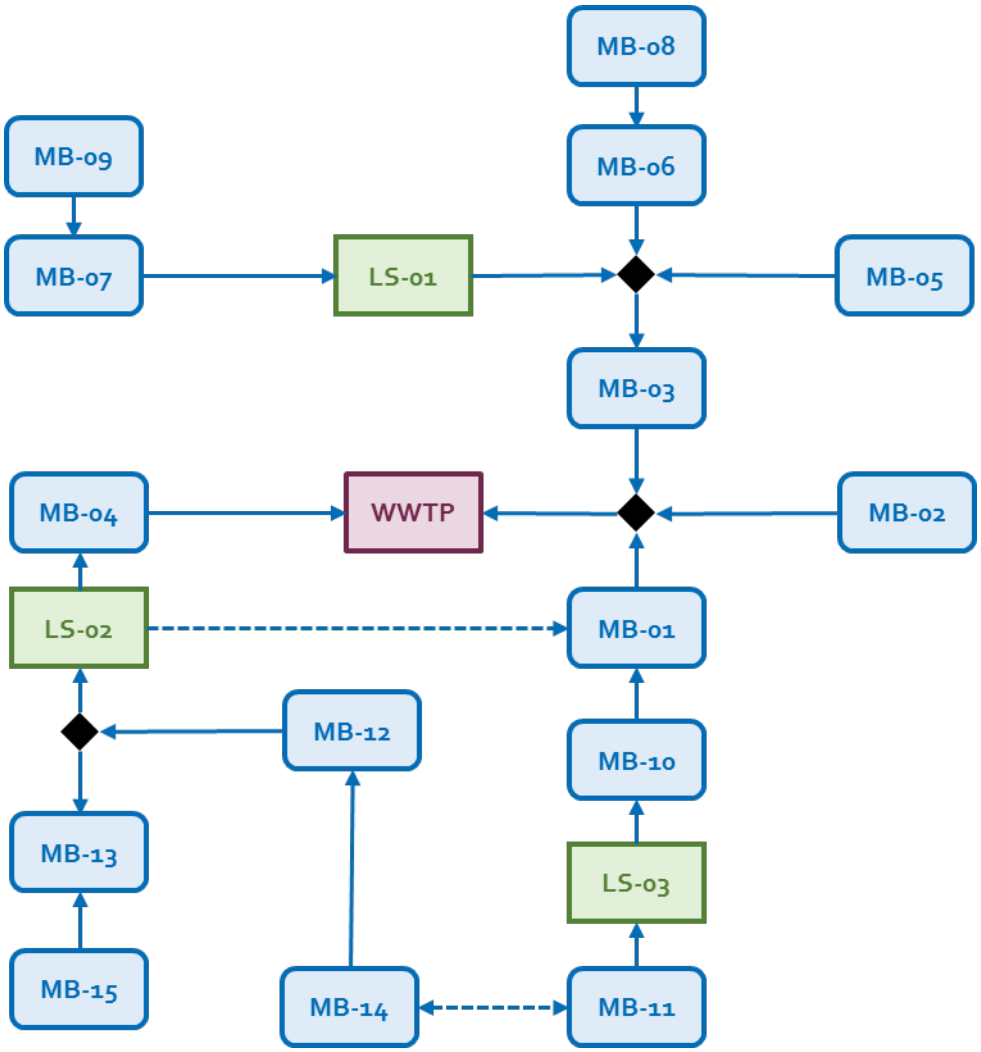

Figure 4.11 Temporary Flow Monitoring Schematic

4.2.3.2 Flow Meter Installation and Flow Calculation

V&A reviewed the hydraulics of the selected flow monitoring locations to select the optimal flow monitoring technology. For this project, Isco 2150 flowmeters were selected. Isco 2150 meters use submerged area/velocity (AV) sensors use a pressure transducer to collect depth readings and ultrasonic Doppler sensors on the probe to determine the average fluid velocity.

In order to ensure that each meter was accurate and calibrated, manual level and velocity measurements were taken by V&A when each meter was installed and again when they were removed. These manual measurements were compared to simultaneous level and velocity readings from the flowmeters. The pipe diameter was also verified, because the pipe diameter is needed to calculate flow rate in a pipe based on the velocity and level measurements. In addition, the depth of sediment, if any, was measured as this affects the cross sectional area of flow within a pipe. Figure 4.9 shows the location of the temporary flowmeters. V&A conducted an analysis of the data retrieved from each flowmeter, and made adjustments as needed for calibration based on the field measurements, and to account for any sediment build up. Flow data collected in 5 minute intervals throughout the flow monitoring period. The flow at each meter was then calculated at 15-minute intervals based on the continuity equation:

$Q = V \times A$

where,

Q = Pipeline flow rate, cfs.

V = Average velocity, ft/s.

A = Cross sectional flow area, ft^2.

4.2.3.3 Rain Gauges

V&A received rainfall data from five publically available rain gauges maintained by weather enthusiasts. Even though V&A had no control over these gauges, V&A performed a quality assurance, and quality control review of the data from the rain gauge to ensure their accuracy for the purposes of this study. Rainfall data was collected in 15-minute intervals throughout the flow monitoring period. Figure 4.9 shows the location of the rain gauges utilized for this study.

4.2.4 Temporary Flow Monitoring Program Results

This section summarizes the results of the flow monitoring program, including dry weather flow data, rainfall data, and wet weather flow data. Appendix F includes additional data summaries and other information associated with the remaining meter sites.

4.2.4.1 Dry Weather Flow Data

During the flow monitoring period, depth and velocity data were collected at each meter at 15 minute intervals. Carollo aggregated the 15-minute data to hourly data for use in the hydraulic model. Characteristic dry weather 24-hour diurnal flow patterns for each site were developed based on the hourly data. This hourly flow data was then used to calibrate the hydraulic model for the observed dry weather flows during the flow monitoring period. For this flow metering program V&A developed four average dry weather flow curves for each site location for Monday through Thursday, Friday, Saturday, and Sunday. Flows would often differ on Friday evenings compared to Monday through Thursday. Similarly, Saturday and Sunday flow patterns were unique during the evening. Hourly patterns for weekday and weekend flows vary and are separated to better understand dry weather flow. V&A used the data from days least affected by rainfall to estimate the weekday and weekend ADWF.

In addition, V&A provided estimates for the average weekday and weekend levels and velocities at each site, which are used in dry weather flow calibration. Figure 4.12 illustrates a typical variation of Monday through Thursday, Friday, Saturday, and Sunday flows in the City, which is

based on the data collection from flowmeter Site 14. Similar graphics associated with the remaining sites are included in Appendix F. Table 4.12 summarizes the dry weather flows at each meter.

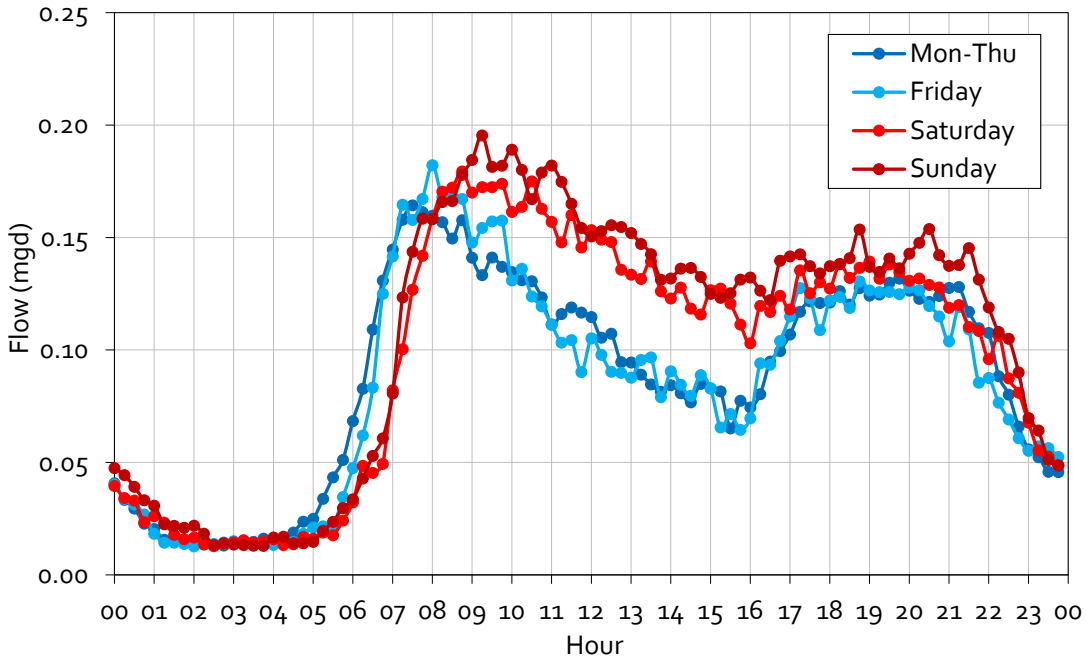

Figure 4.12 Typical Dry Weather Flow Variation (Site 14)

Table 4.12 Dry Weather Flow Summary

Meter Site[(1)(2)]	Monday-Thursday ADWF (mgd)	Friday ADWF (mgd)	Saturday ADWF (mgd)	Sunday ADWF (mgd)	Overall ADWF (mgd)
Site 1	0.18	0.19	0.19	0.18	0.18
Site 2	0.03	0.03	0.03	0.03	0.03
Site 3	0.28	0.30	0.32	0.32	0.30
Site 4	0.21	0.23	0.26	0.24	0.22
Site 5	0.09	0.09	0.10	0.11	0.09
Site 6	0.11	0.11	0.12	0.12	0.11
Site 7	0.04	0.04	0.04	0.05	0.04
Site 8	0.07	0.07	0.08	0.08	0.07
Site 9	0.02	0.02	0.02	0.02	0.02
Site 10	0.07	0.07	0.07	0.07	0.07
Site 11	0.06	0.06	0.07	0.07	0.06
Site 12	0.12	0.13	0.15	0.14	0.13
Site 13	0.09	0.09	0.11	0.10	0.09
Site 14	0.09	0.09	0.10	0.11	0.09
Site 15	0.04	0.05	0.05	0.05	0.05
MB 1	0.17	0.16	0.16	0.17	0.17
MB 2	0.03	0.03	0.03	0.03	0.03
MB 3	0.29	0.28	0.31	0.33	0.30
MB 4	0.21	0.23	0.26	0.25	0.22
Flume	0.89	0.91	0.98	1.00	0.92

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study
(2) Sites 1 through Site 15 April 7, 2017 through July 11, 2017. MB 1 through MB 4 measured data January 3, 2017 through February 27, 2017 and Morro Bay Flume data from November 7, 2016 through July 17, 2017.

Table 4.13 summarizes the peak measured flows, peak pipe diameter ratios (d/D ratios), and peaking factors per site during the flow monitoring period.

Table 4.13 Peak Measured Flows during the Flow Monitoring Program

Meter Site[1][2]	Diameter (inches)	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Peak d/D Ratio
Site 1	11.75	0.18	0.48	2.6	0.48
Site 2	8	0.03	0.08	3.1	0.32
Site 3	15	0.35	0.99	2.8	0.54
Site 4	17.5	0.22	0.67	2.5	0.48
Site 5	8	0.09	0.28	3.1	0.32
Site 6	13	0.11	0.37	3.0	0.33
Site 7	9.75	0.04	0.16	4.1	0.36
Site 8	13	0.07	0.21	3.1	0.56
Site 9	12	0.02	0.10	4.8	0.32
Site 10	8	0.07	0.32	4.7	0.57
Site 11	8	0.06	0.25	3.9	0.46
Site 12	14.375	0.15	0.41	2.8	0.54
Site 13	15.5	0.11	0.33	3.1	0.47
Site 14	10	0.09	0.32	3.5	0.25
Site 15	10	0.05	0.21	4.6	0.35
MB 1	12	0.17	0.93	5.6	1.17
MB 2	6	0.03	0.17	5.9	0.41
MB 3	15	0.36	2.64	7.3	1.61
MB 4	18	0.27	1.13	4.2	0.36
Flume	27	0.92	10.08	11.0	N/A

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study
(2) Sites 1 through Site 15 April 7, 2017 through July 11, 2017. MB 1 through MB 4 measured data January 3, 2017 through February 27, 2017 and Morro Bay Flume data from November 7, 2016 through July 17, 2017.

Holiday Flow

During the temporary flow monitoring program, Memorial Day and July 4th holidays were measured to provide the City with data about increases in flow due to tourists' population. Table 4.14 summarizes the percent difference of the holiday flows to ADWF.

Table 4.14 Summary of Holiday Flows

Meter Basin[1][2][3]	Memorial Day Holiday Weekend Difference (%)	July 4th Holiday Weekend Difference (%)
Basin 1	n/a	9%
Basin 2	7%	19%
Basin 3	n/a	-18%
Basin 5	23%	12%
Basin 6	n/a	34%
Basin 7	10%	-10%
Basin 8	14%	35%
Basin 9	20%	70%
Basin 10	n/a[4]	n/a[4]
Basin 11	16%	6%
Basin 12	25%	32%
Basin 13	15%	0%
Basin 14	10%	29%
Basin 15	2%	48%
Flume	7%	28%

Notes:

(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study

(2) Sites 1 through Site 15 April 7, 2017 through July 11, 2017. MB 1 through MB 4 measured data January 3, 2017 through February 27, 2017, and Morro Bay Flume data from November 7, 2016 through July 17, 2017.

(3) Flows are presented by basin, not by meter. Flows from upstream basins have been removed for the purposes of this analysis.

(4) There was not enough flow added into the system between Meter 11 and 10 to develop a meaningful comparison of holiday flows for Basin 10.

4.2.4.2 Rainfall Data

The rainfall data collected by V&A was used to correlate I/I response observed in the collection to specific storm recurrence intervals. Five rainfall events were captured during the monitoring period occurred. The rain gauges recorded between 2.04-inches and 3.77-inches during this period, depending on the event. Table 4.15 summarizes the duration and precipitation of each rainfall event.

Table 4.15 Rain Gauge Data

Rainfall Event	Duration	Morro6	Morro28	Losos17	Losos19	Baywo16	Avg.
Event 1	1/3/2017-1/6/2017	2.44	3.12	2.86	2.60	2.48	2.70
Event 2	1/7/2017-1/17/2017	3.42	3.29	2.50	2.33	2.04	2.72
Event 3	1/18/2017-1/25/2017	2.75	3.90	3.90	2.18	2.92	3.13
Event 4	2/5/2017-2/12/2017	2.23	2.60	2.48	2.44	2.33	2.42
Event 5	2/16/2017-2/22/2017	2.54	3.19	3.58	3.77	3.14	3.24
Total Over Monitoring Period		**15.53**	**19.22**	**17.99**	**16.22**	**15.76**	**16.95**

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study

4.2.4.3 Wet Weather Flow Data

The flow monitoring data was assessed to determine how the collection system responds to rain events. For wet weather flow (WWF), the four City owned meters were used. This was because there were several large rain events that occurred outside V&A's temporary flow metering program. Table 4.16 summarizes the measured flow data during the five rain events for each meter basin. The following should be noted about the wet weather flow data:

- WWTP Flume: The WWTP influent flume experienced a peak flow during rain event 1. V&A compared the data from the flume to the data from the four City owned flow meters. For this event, the flume data far exceeded the upstream measured data. This type excessive spike was not measured for any of the other strong rainfall events. Based on discussions with City staff, this flow spike was attributable to an abnormal discharge event from Cayucos.

Table 4.16 Measured Rain Event Flow Data Summary

Meter Basin	ADWF (mgd)	Event 1		Event 2		Event 3		Event 4		Event 5	
		Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Flow (mgd)	Peak Flow (mgd)
MB-1	0.17	0.27	0.91	0.29	0.73	0.32	0.93	0.32	0.71	0.36	0.87
MB-2	0.03	0.05	0.17	0.05	0.15	0.06	0.14	0.06	0.12	0.07	0.16
MB-3	0.30	0.71	2.54	0.63	2.26	0.76	2.37	0.79	1.71	0.76	1.97
MB-4	0.22	0.27	1.02	0.25	0.69	0.23	0.88	0.25	0.60	0.26	0.63
Flume	0.92	1.73	10.08	1.57	4.31	1.75	4.89	1.73	3.07	1.82	3.87

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study

Figure 4.13 shows an example of the wet weather response at flowmeter MB-3 during January 3, 2017 through January 9, 2017 rain events. The dashed line is the ADWF while the black line is the measured flow from the flow monitoring period. As shown on Figure 4.13, substantial amounts

of I/I do enter the system during wet weather events. Additional wet weather monitoring results for all meters can be found in the flow monitoring report prepared by V&A in Appendix F.

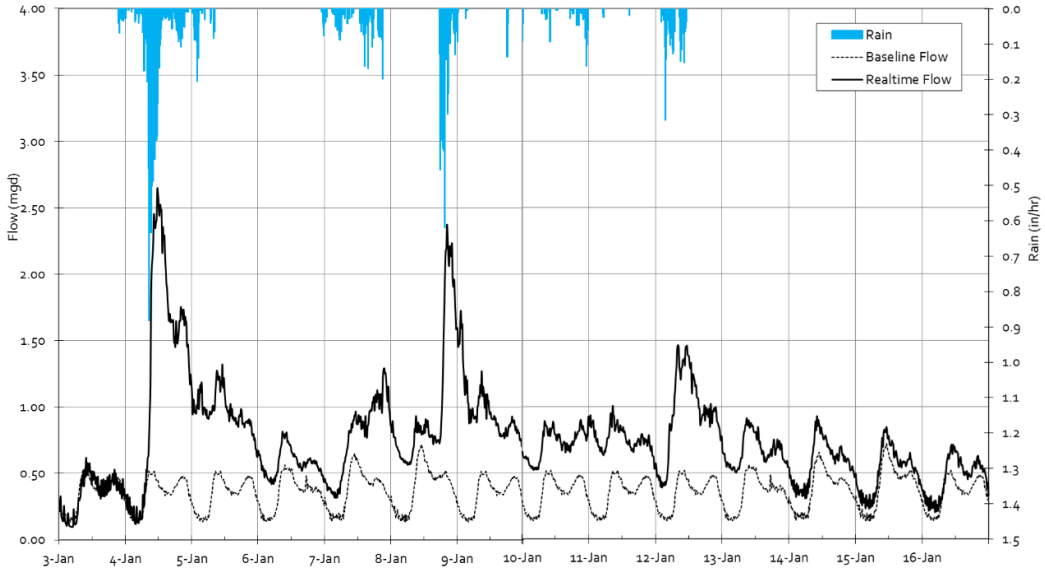

Figure 4.13 Example Wet Weather Flow Response (MB-3)

The following section summarizes the results from of V&A's Inflow/Infiltration Analysis as part of the Flow Monitoring Program.

4.2.4.4 Inflow and Infiltration Analysis

This section summarizes the findings from V&A's I/I analysis. V&A's I/I analysis used the data from four City owned flow meters because these meters captured five significant rain events. An I/I analysis was not performed on the WWTP flume because there is no substantive basin that could be isolated. The methods used to distinguish between I/I consisted of graphical identification, analysis metrics, and normalization methods. Detailed descriptions of these methods can be found in V&A's 2017 Sewer Flow Monitoring and Inflow/Infiltration Study which is attached in Appendix F.

Inflow

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

Table 4.17 summarizes the peak measured I/I flows and inflow analysis results. Peak I/I rates for all sites were measured during Rain Event 1 from January 3 through January 6, 2017, because this was the most intense and short term rain event. Meter basin MB-3 had the highest normalized inflow contribution, while meter basins MB-1 and MB-2 exhibited elevated levels of inflow.

Table 4.17 Inflow Analysis Summary

Meter Basin	ADWF (mgd)	Peak Inflow Rate (mgd)	Peak Inflow per ADWF	Peak Inflow per Acre (gpd/acre)	Inflow Rank(2)
MB-1	0.17	0.68	4.07	2,750	3
MB-2	0.03	0.14	4.87	2,130	2
MB-3	0.30	2.15	7.27	3,226	1
MB-4	0.22	0.70	3.12	1,018	4

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study.
(2) Ranking of 1 represents most inflow after normalization.

Rain Dependent Infiltration

Rain dependent infiltration (RDI) is water that enters the sanitary sewer system through defects in pipes, joints and manholes. RDI enters the system indirectly and is characterized by flows that increase gradually, are typically sustained for a period after rainfall has stopped, and then gradually drop off as soils become less saturated and as groundwater levels recede to normal. The major impact is the cost of pumping and treating the additional volume of water.

Table 4.18 summarizes the calculated RDI flow events for Rain Event 2 from January 7 through January 17, 2017. Meter basin MB-1 and MB-3 had the highest rates of infiltration.

Table 4.18 Infiltration Analysis Summary

Meter Basin	ADWF (mgd)	RDI Rate (mgd)	RDI per ADWF	Peak RDI per Acre (gpd/acre)	Infiltration Rank(2)
MB-1	0.17	0.13	0.75	509	1
MB-2	0.03	0.02	0.79	345	3
MB-3	0.30	0.26	0.87	386	2
MB-4	0.22	0.02	0.1	33	4

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study.
(2) Ranking 1 represents most infiltration after normalization.

Dry Weather Groundwater Infiltration

ADWF can have 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.

V&A concluded that MB-3 may have some evidence of dry weather GWI. Figure 4.14 plots the low-to-ADWF rations against the ADWF for the sites monitored during the flow monitoring program. The dotted line shows typical low to ADWF ratios according to Water Environment Federation (WEF) Manual of Practice No. 9, "Design and Construction of Sanitary and Rain Sewers." 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. Therefore, the WEF typical trend line's slope is closer to 1.0 as the ADWF increases, as shown in Figure 4.14.

The information on Figure 4.14 suggests that the City's dry weather groundwater infiltration is not excessive. This figure does not apply to wet weather groundwater infiltration, however.

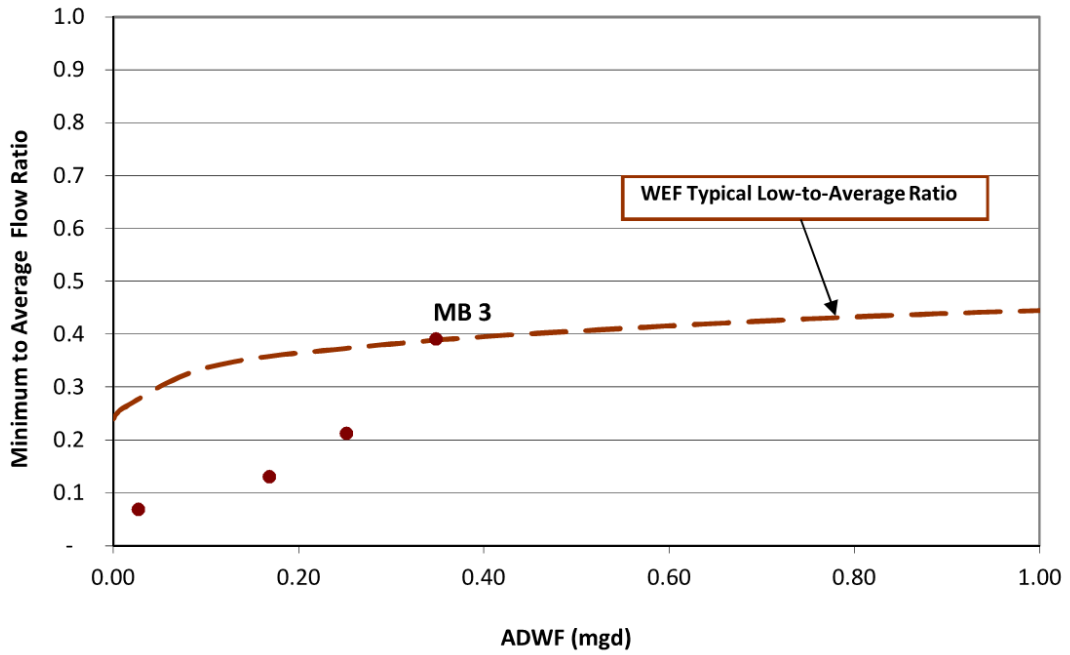

Figure 4.14 Minimum Flow Ratios vs. ADWF

Combined I/I

The combined I/I analysis considers the total volume of inflow and RDI over the source of a storm event. Table 4.19 summarizes the combined I/I results for the total I/I measured across the entirety of the flow monitoring period. Meter basin MB-03 had the highest combined I/I after normalization. Meter basins MB-1 and MB-2 had elevated levels of combined I/I.

Table 4.19 Total RDII Analysis Summary

Meter Basin	ADWF (mgd)	I/I (MG)	I/I per ADWF (per inch-rain)	R Value (%)	I/I per Acre (per inch-rain)	Rank(2)
MB-1	0.17	7.33	2.87	7.14	1,940	3
MB-2	0.03	1.29	2.90	4.67	1,268	2
MB-3	0.30	18.55	4.06	6.68	1,815	1
MB-4	0.22	1.22	0.36	0.43	116	4

Notes:
(1) Source: V&A 2017 Sewer Flow Monitoring and Inflow/Infiltration Study.
(2) Ranking of 1 represents most inflow after normalization.

4.2.5 Wastewater Flow Projections

Historical per-capita generation rates indicate a range between 56 and 164 gpcd. As previously stated, this OneWater Plan assumes a per capita flow of 82 gpcd for future flow projections. Table 4.20 summarizes projected wastewater flows through the year 2040 on population and

shown in Figure 4.15. As shown in Table 4.20, the City's wastewater flow is projected to increase by 0.11 mgd by 2040 using a per capita demand.

Table 4.20 Wastewater ADWF Projections

Year	Projected Population	Projected ADWF(1) (mgd)
2017	10,714	0.88
2020	10,901	0.89
2025	11,213	0.92
2030	11,525	0.94
2035	11,837	0.97
2040	12,149	0.99

Notes:
(1) Projected flow based on a per capita demand of 82 gpcd

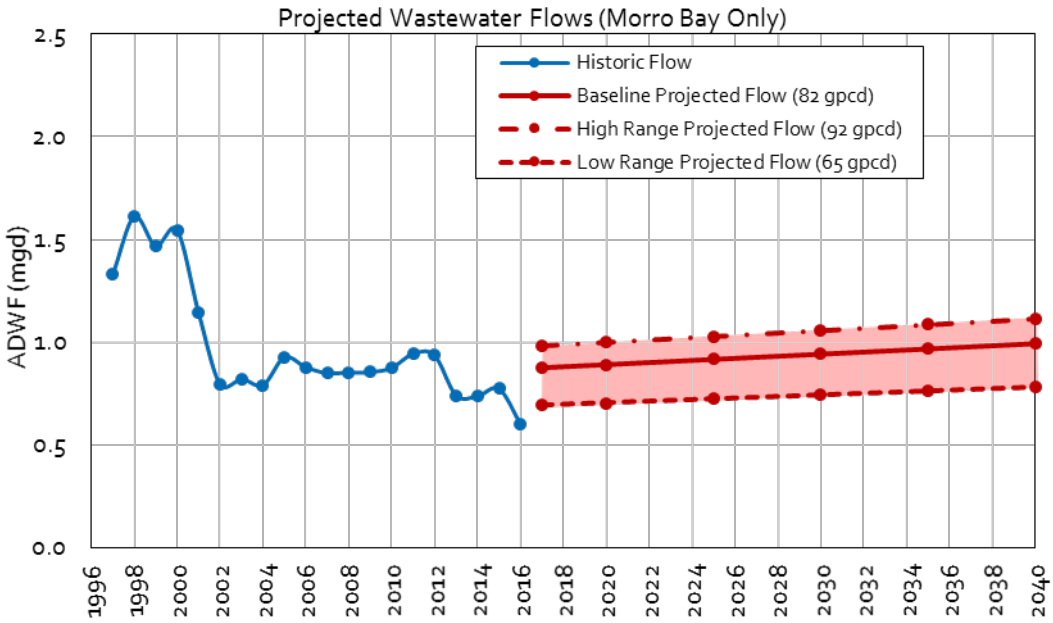

Figure 4.15 Wastewater Flow Projections

4.2.6 Existing and Projected Peak Wet Weather Flow

The existing PWWF was derived throughout the system based on the hydraulic modeling results. This was accomplished by routing the 10-year, 24-hour design storm through the hydraulic model, which was calibrated to both dry weather and wet weather conditions. Similar to the existing PWWF, the future PWWFs were derived by routing a 10-year, 24 hour design storm through the hydraulic model. Peak I/I rates for future growth within the existing City service area were developed based on a peak RDII rate of 1,000 gpd/ac.

Table 4.21 presents a summary of the existing and future ADWF and PWWF. As shown in Table 4.21, City's ADWF is projected to increase from 0.88 mgd to 0.99 mgd by 2040, whereas the PWWF is projected to increase from 7.90 mgd to about 8.12 mgd by 2040. Therefore, the City's PWWF to ADWF peaking factor is projected to decrease from roughly 9.0 to 8.2. This is typical, as newer sewers tend to have less RDII response than older areas of the system, primarily due to better construction methods. Furthermore, flow attenuation also tends to dampen out flow peaks as collection systems expand.

Table 4.21 Existing and Projected Wastewater Flows

Year	Projected ADWF[(1)] (mgd)	Projected PWWF[(1)(2)(3)] (mgd)	Peaking Factor (PWWF/ADWF)
2017	0.88	7.90[(3)]	9.0
2040	0.99	8.12	8.2

Notes:
(1) Does not include Cayucos influent flow.
(2) PWWF was estimated using a peak RDII of 1,000 gpd/ac.
(3) Assumes recommended existing improvement projects identified in Chapter 6 are completed.

Chapter 5

WATER, WASTEWATER, AND STORM DRAINAGE SYSTEM FACILITIES AND HYDRAULIC MODELS

This chapter summarizes the historical, existing and future water demands, and the existing and future wastewater flows.

5.1 Existing Water Supply and Distribution System Facilities

This section presents an overview of the City's existing water supply, water distribution system, storage, and pumping facilities. First, the City's water supply sources are described, followed by a description of the City's water distribution system and facilities.

5.1.1 Existing Water Supply

Morro Bay's water supply portfolio currently consists of four different sources. The City is currently in the process of developing a fifth water supply source by means of a new secondary wastewater treatment facility with the ability to produce recycled water for potable reuse. The supply portfolio includes:

1. California State Water Project (SWP) water conveyed by the Central Coast Water Authority (CCWA) and purchased from San Luis Obispo County
2. The Morro Groundwater Basin
3. The Chorro Groundwater Basin
4. Ocean Desalination
5. Water Reclamation Facility (future)

5.1.1.1 State Water Project.

The California SWP began construction in the 1960s as a primary means to deliver surface water for domestic and agricultural uses and to provide flood control, recreational areas, wildlife habitat improvements, and power generation throughout California. The entire network of SWP facilities include 30 dams, 20 reservoirs, 29 pumping plants, and 700 miles of aqueducts (both canals and pipelines). The primary source of SWP water is from precipitation and snowmelt within the Sierra Nevada Mountain range. The chain of reservoirs available to store Sierra runoff range reside hundreds of miles North of the California Delta and act as the main supply source of the California Aqueduct. Figure 5.1 shows the intricate network of SWP facilities with respect to Morro Bay.

Figure 5.1 California Water Project Flow Path to Morro Bay

State Water delivered to the City originates in both the Shasta-Trinity and Oroville reservoirs and is conveyed via the Sacramento and Feather Rivers to the Clifton Court Forebay of the Sacramento-San Joaquin Delta. From the Clifton Court Forebay, water flows by the California

Aqueduct and is lifted through a series of pumping plants which divert state water to the Coastal Branch turnout at the end of reach 8D, approximately 12 miles south of Kettleman City. Water is pumped by the Las Perillas, Badger Hill, Devil's Den, Bluestone, and Polonio Pass pumping plants a combined normal static head of 1,744 feet to the Polonio Pass Water Treatment Plant located outside of Shandon.

The Polonio Pass water treatment facility is owned and operated by the Central Coast Water Association (CCWA). CCWA is a public entity formed under Central Coast public agencies to manage water resources infrastructure with respect to DWR facilities for beneficial use to the surrounding regions. From Polonio Pass, CCWA facilities serve as the primary transmission facilities of State Water to San Luis Obispo and Santa Barbara Counties. Of the 4,830 acre-feet of Table A water contracted to San Luis Obispo County, 1,313 acre-feet is allocated to the City of Morro Bay. The Chorro Valley transmission pipeline delivers treated water to the City of Morro Bay off the Coastal Branch Pipeline that brings treated water south and west from the Polonio Pass Treatment Plant.

The County of San Luis Obispo has a long term contract for Table A water of 25,000 acre-feet per year displayed in Table B-4 of DWRs Bulletin 132. Costs incurred by the SWP are a proportion of the Table A allotments, therefore San Luis Obispo's share of the construction capital costs are a function of their Table A volume. For future costs projections, it is highly unlikely that a full 25,000 acre-feet per year would be allocated to San Luis Obispo County, Bulletin 132 Table B-5A uses a delivered quantity of 13,237 acre-feet as a conservative delivered volume to calculate future variable OMP&R charges. On average, allocations have been 77 percent in the last 10 years. San Luis Obispo County water contractors only have a combined contracted delivery of 4,830 acre feet per year consequently resulting in an unrealistic variable OMP&R displayed in Bulletin 132. Though total Table A allocations have been reduced, the last 10 years show deliveries for Morro Bay have been on average 75 percent of their contracted allocation from CCWA. The remaining 25 percent of water demand has been supplemented by groundwater. The City has the ability to take unused State Water allocation and store it behind San Luis Reservoir as carryover water for use during the same year or the following year. Since most of the State Water costs are fixed costs (i.e., the City pays the majority of costs regardless of how allocation it takes) it is prudent to pay the small variable costs for the additional allocation and take it as carryover water. This is the City's typical practice.

During SWP low allocation years Morro Bay relies on its 174 percent drought buffer with San Luis Obispo County Flood Control and Water Conservation District (SLOCFCWCD). This ensures the City receives their entire allocation of 1,313 acre-feet per year (AFY) when the SWP can deliver at least 36 percent of contracted water to its recipients. The City purchases drought buffer water which can be stored for use at a later date as carryover water. However, the storage from carryover water is no longer available if it interferes with storage of State Water for project needs in San Luis Reservoir. Once this occurs, the carry over water is converted to Article 21 water and made available to all SWP contractors causing Morro Bay to lose its stored carryover water.

In the event the City has unused State Water, the City may transfer Table-A water to another contractor or sell back unused allocation to the DWR turn-back pool. Currently, the City's opportunity to participate in water transfers is minimal since the City is maximizing its Table-A allocation to meet City-wide demands. Water sold to the turn-back pool is purchased by DWR or

other SWP contractors at a very low price in comparison to the original price purchased by Morro Bay. In the future, if the City moves forward with a water supply option, Morro Bay will have the opportunity to exchange unused Table-A allocation with other State Water contractors or nearby agencies. Currently, San Luis Obispo County has initiated feasibility studies to operate a groundwater banking facility in the Paso Robles Groundwater Basin and is currently interested in securing additional capacity within the State Water Project Coastal Branch and the Polonio Pass Treatment Facility. This opportunity presents the City with the ability to maintain their existing Table-A allocation as a back-up water supply while simultaneously exchanging Table-A allocation for their benefit.

5.1.1.2 Morro Groundwater Basin

The Morro Groundwater Basin is the primary source of groundwater for the City. The Morro Well Field is located west of Highway 1 near Morro Creek. The groundwater basin is approximately 1,200 acres and is bounded by the Pacific Ocean on the West and the coastal range mountains to the east. The groundwater basin is sourced by surface water percolation from Morro Creek flowing into Morro Bay from the east. DWR Bulletin 118 and well reports indicate a basin thickness of around 60 feet with well depths ranging from 60 to 80 feet. The City has 8 wells within the Morro Groundwater Basin.

The permitted yield by the Water Quality Control Board for Morro well field is 581 AFY annually and 1.2 cubic feet per second (cfs) instantaneously. The water quality of Morro Basin has been of concern in regards to high concentrations of Nitrate (NO_3) and Salinity (TDS). Recent reports indicate upstream fertilizer runoff by agricultural land use has been found to be the leading cause of nitrate contamination in the groundwater basin leading to concentrations as high as 220 mg/L. During drought years, the low groundwater elevation leads to potential seawater intrusion issues threatening the basin groundwater quality. Studies have found that TDS concentrations have ranged from 400 to 1000 mg/L in the Morro Basin. Prior to public distribution, the City treats groundwater yield from the Morro well field for both nitrate and TDS at the Brackish Water Treatment Facility and then finished water is sent to the Kings Tank.

The Morro well field has been used 9 out of the last 10 years due to drought conditions leading to SWP cutbacks. Average use of the well field has been 104 AFY in the years used.

5.1.1.3 Chorro Groundwater Basin

The Chorro Groundwater Basin is located southeast of Morro Bay and serves as the secondary source of groundwater to the City. The basin is approximately 3,200 acres and runs parallel to Highway one and ultimately drains into Morro Bay State Marine Reserve. The Chorro groundwater basin is sourced by Chorro Creek and receives treated wastewater effluent from the California Men's Colony wastewater treatment plant.

The State Water Quality Control Board permitted yield of Chorro groundwater basin is 1,142.5 AFY with an instantaneous yield of 3.2 cfs. Pumping groundwater is available at the Chorro well field only when the recorded stream flow in the adjacent Chorro Creek is above 1.2 cfs. Studies show that pumping of the Chorro wells has a direct interference on Chorro Creek Discharge. Additionally, streamflow records from San Luis Obispo County displays time spans of Chorro Creek discharge below 1.2 cfs ranging from a few days up to 3 months.

The City has 6 wells in the Chorro groundwater basin with capacity of 1142.5 AFY. The City has had concerns regarding high concentrations of nitrates within the Chorro groundwater basin.

There is currently no means of treatment of the Chorro wells prior to being sent to the Kings Tank, therefore currently these well are disconnected from the drinking water distribution system. Nitrate concentrations range from 25 mg/L to 100 mg/L and have been found to be attributed to nitrate based fertilizer runoff from upstream agricultural land use.

5.1.1.4 Ocean Desalination

Morro Bay has operated a desalination facility that in the past has provided a portion of the City's water supply since the 1990s. Currently, the facility is in a state of much needed repair and is not in use. A condition assessment of the desalination facility was performed and can be found in Appendix G. The City originally constructed the facility in response to drought conditions and its initial emergency coastal development permits (CDP) allowed it to operate for only two years before requiring a new CDP to continue operation. Source water for the facility is obtained from several nearby brackish water wells and five seawater wells located along the Harbor Walk Pathway. Operating the desalination train of the brackish water treatment facility produces an instantaneous treatment capacity of 400 gallons per minute (gpm) and a permitted annual production capacity of 645 AFY. The waste product from the desalination process is a salinity brine with concentrations ranging from 45,000 to 47,000 parts per million (ppm) TDS. The desalination facility outfall for the brine is the now-retired Morro Bay Power Plant outlet formerly used to discharge power plant cooling water.

5.1.1.5 Water Reclamation Facility (Future)

The City's existing secondary wastewater treatment facility is to be relocated as part of the proposed Water Reclamation Facility (WRF) project. The location of the WRF will be east of Highway 1 along South Bay Boulevard. The WRF intends to provide secondary treatment for the wastewater effluent from the existing primary treatment facility, and may produce purified recycled water through advanced treatment for potable reuse to augment and diversify the City's existing water supply portfolio.

5.1.1.6 WTP

The City's WTP was originally constructed in 1922 to treat 400 gpm of seawater from five wells along the Morro Bay Harbor. The WTP was expanded in 2009 with the installation of Brackish Water Reverse Osmosis trains to treat brackish groundwater from the Morro Wells. In 2012, the WTP Improvement Plan was completed. This improvement allowed the plant to deliver up to 1,350 gpm of product water to the City's distribution system. The WTP has experienced significant operating issues and is currently not operable and was not used in 2016. Currently, the City receives water from the State Water Project.

5.1.2 Distribution System Pressure Zones

Water distribution systems that have varied topography are typically divided into different hydraulic regions, known as pressure zones. The purpose of pressure zones is to maintain adequate pressures throughout the distribution system in spite of the varying topography. A Hydraulic Grade Line (HGL) is established for each pressure zone and the high water levels in reservoirs are set to maintain these HGLs. The City's service area ranges in elevation from approximately sea level to about 420 feet above sea level.

The City's water distribution system consists of 6 pressure zones. The characteristics of the City's pressure zones are described in Table 5.1. Figure 5.2 shows the existing water distribution

system. Figure 5.3 shows the City's pressure zone boundaries and distribution system pipelines color coded by pressure zone. Figure 5.4 is a hydraulic profile of the distribution system.

Water from the State Water Project is supplied to the Kings Tanks which convey water to the distribution system. Lower areas of the City's distribution system receive water through several pressure reducing valves (PRV) stations.

Table 5.1 Pressure Zone Summary

Pressure Zone Name	Nominal HGL (ft)	Reservoirs	Supply Sources to Pressure Zone		
			Booster Pump Stations	PRV Stations	Supply Sources
Blanca	210	Blanca (4)	Vashon	--	--
Elena	153	Elena (2)	--	Atascadero Road, Errol Street	--
Nutmeg	411	Nutmeg	Elena	--	--
Upper Kings	316	Upper Kings (2)	--	--	State Water
Ridgeway	452	Black Hill	Upper Kings	--	--
Lower Kings		--	--	South St & Monterey Ave, MB Blvd at PD Yard	--

5.1.2.1 Transmission and Distribution System

The City's distribution system consists of approximately 72 miles of pipeline ranging from 1-inch to 16-inches in diameter. This section describes the distribution of pipelines by diameter, material, age, and pressure zone. Figure 5.2 shows a map of the existing distribution system with pipe diameters and alignments.

Table 5.2 provides a breakdown of the water distribution system by diameter, excluding laterals and the desalination line. This information is also presented graphically in Figure 5.5. As shown in Table 5.2 and Figure 5.5, roughly 82 percent of the City's distribution system is 8-inch in diameter or smaller. About 17 percent of the system is 10-inch or 12-inch in diameter, and one percent of the system is 14-inch in diameter and larger.

Table 5.2 Existing Water Distribution System Pipeline Summary, by Diameter

Diameter (inches)	Length (1)(2) (feet)	Length (miles)	Percent of System
<6	18,500	3.5	5
6	215,600	40.8	57
8	76,100	14.4	20
10	34,000	6.4	9
12	32,000	6.1	8
14	2,500	0.5	1
16	200	0	0
Total	**378,000**	**71.8**	**100**

Notes:
(1) Source: City of Morro Bay, GIS database.
(2) Does not include laterals or the desalination line.

5.1.2.2 Storage Tanks

Water distribution systems rely on stored water to help equalize daily fluctuations between supply and demand, to supply sufficient water for firefighting, and to meet demands during an emergency or an unplanned outage of a major source of supply.

The City's water system has ten active treated water reservoirs at five different sites totaling 3.25 Million Gallons (MG). The locations of the City's existing reservoirs are shown on Figure 5.2, while detailed information for the treated water reservoirs is summarized in Table 5.3.

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Blanca Tanks
Vashon Booster Station
Nutmeg Tank
Elena Tank & Booster Station
Desalination Plant & PRV
Errol PRV
Morro Bay Blvd at PD Yard PRV
South Street & Monterey PRV
SWP Turnout Kings Tank & Booster Pump
Ridgeway Tank

BEACHCOMBER DR
JAVA ST
EASTER ST
SANDALWOOD AVE
CORAL AVE
SAN JACINTO ST
GREENWOOD AVE
ELENA ST
BONITA ST
AVALON ST
AVALON RD
TASCADERO RD
EMBARCADERO RD
RADCLIFF AVE
QUINTANA RD
KENNEDY WAY
BEACH ST
FRONT ST
MARKET AVE
HARBOR ST
MORRO BAY BLVD
PACIFIC ST
MARINA ST
MAIN ST
PINEY WAY
KINGS AVE
RIDGEWAY ST
CYPRESS AVE
KERN AVE
STATE PARK RD
SOUTH BAY BLVD
41
1

Legend

- PRV
- Wells
- Pump Stations
- Tank

Water Main by Diameter

- 6 inches and smaller
- 8 - 10 inches
- 12 inches and greater
- City Limits
- Parcels
- Waterways
- Highway

Miles
0 0.25 0.5

Data Sources: City of Morro Bay, San Luis Obispo County, ESRI

Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure 5.2 Existing Water Distribution System

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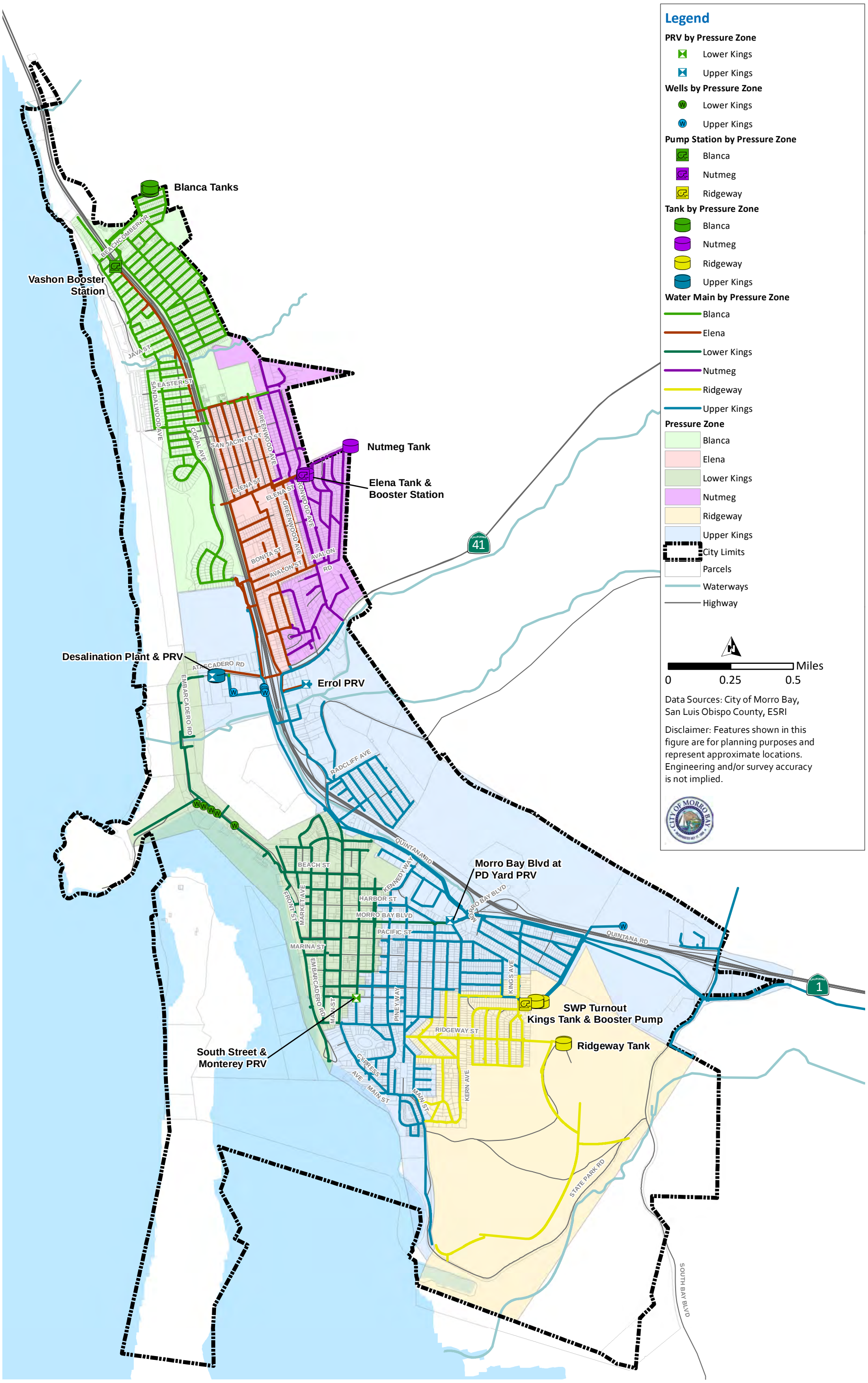

Figure 5.3 Existing Water Distribution System by Pressure Zone

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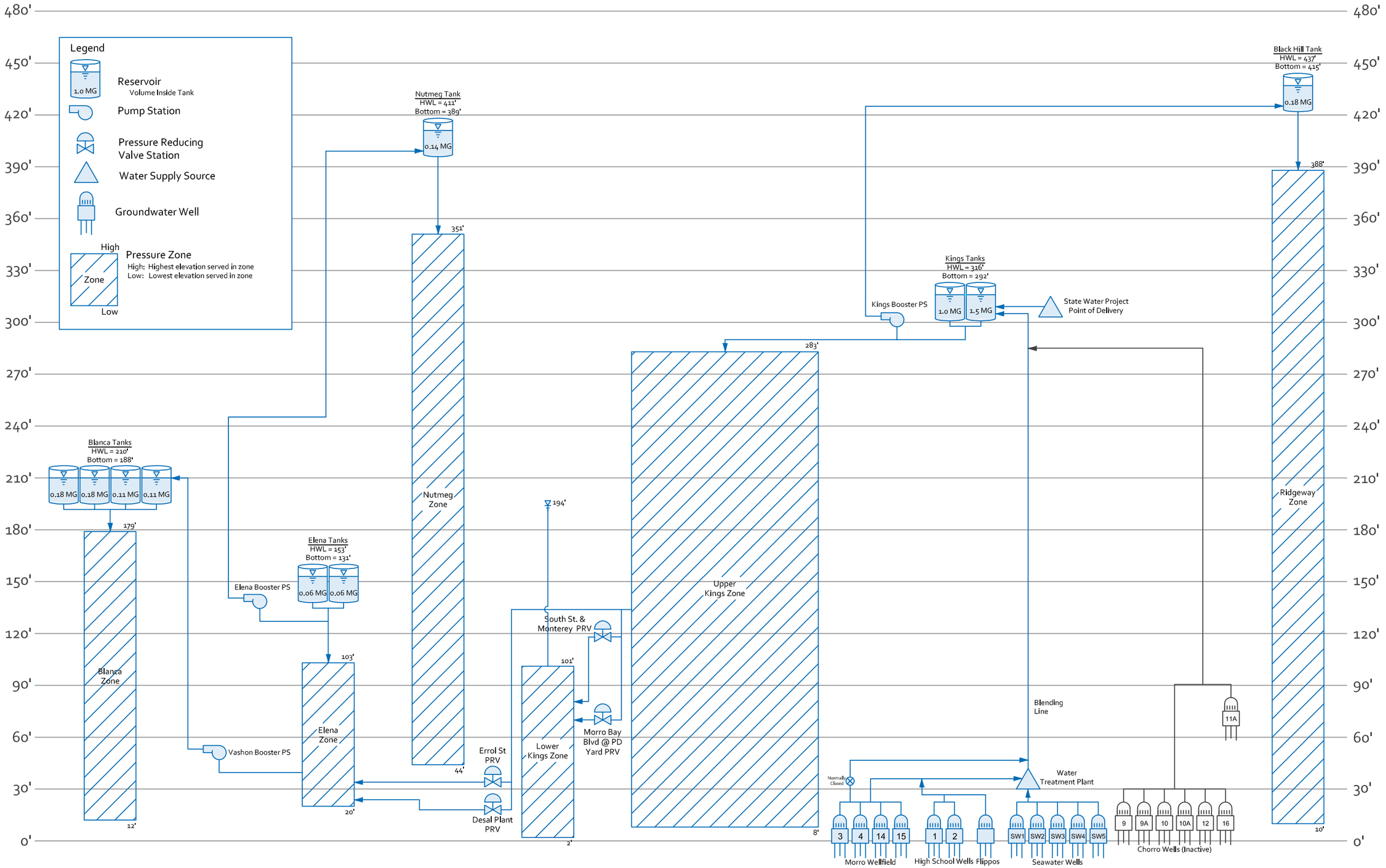

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Figure 5.4 Existing System Hydraulic Profile

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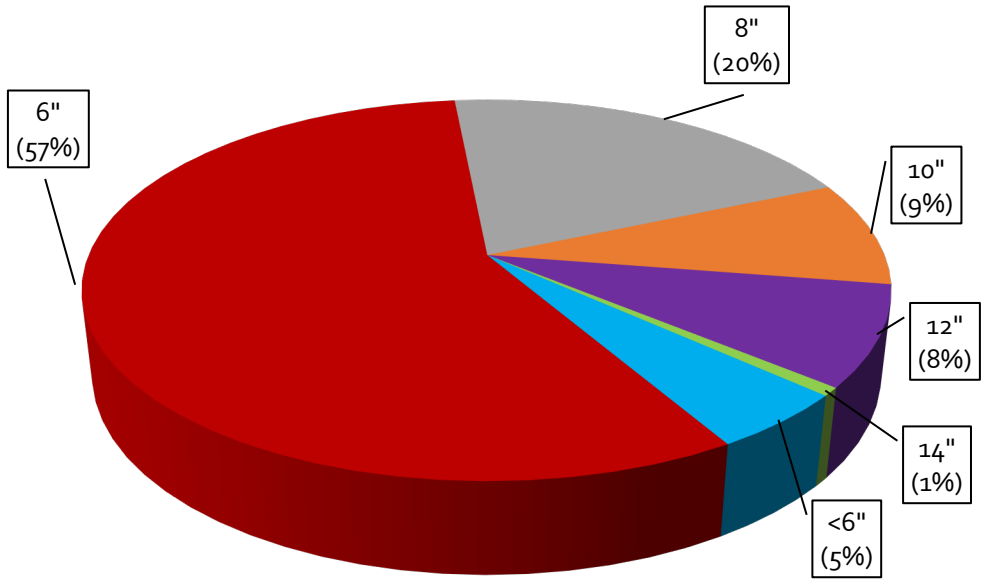

Figure 5.5 Existing Water Distribution System Pipeline Summary, by Diameter

Table 5.3 Existing Storage Tank Summary

Name	Number of Tanks	Volume (MG)	Diameter (ft)	Height (ft)	High Water Line (ft)	Pressure Zone
Blanca Tanks	2	0.18	37.3	22	210	Blanca
	2	0.11	29.2	22	210	Blanca
Elena Tanks	2	0.06	21.6	22	153	Elena
Nutmeg Tank	1	0.14	32.9	22	411	Nutmeg
Upper Kings Tanks	1	1.00	84.2	24	316	Upper Kings
	1	1.50	103.1	24	316	Upper Kings
Black Hill Tank	1	0.18	37.3	22	452	Ridgeway

5.1.2.3 Booster Pump Stations

There are three booster pump stations located throughout the distribution system. A summary of the existing pump stations is located in Table 5.4. The City's Vashon BPS which consists of one 15 HP pump is used to fill the Blanca Tanks, but is unable to fill the tanks during peak demands. During peak demand, the City manually partially opens a gate valve from the Nutmeg Zone to maintain the levels in the Blanca Tanks.

Table 5.4 Existing Pump Station Summary

Name	Pump Number	Power (HP)	Pump Capacity (gpm)	Pump Elevation (ft)	From	To
Vashon BPS[1]	1	15	150	47	Elena	Blanca
Elena BPS	1	40	280	131	Elena	Nutmeg
	2	40	280	131		
Kings Avenue BPS	1	20	250	290	Upper Kings	Ridgeway
	2	20	250	290		

Notes:
(1) The Vashon BPS consists of a single pump. City staff reports that the pump does not have sufficient capacity to fill the Blanca tanks during high demand periods. A gate valve is partially opened to help fill the tanks during high demand periods.

5.1.2.4 PRV Stations

PRV stations allow distribution systems to transfer water from upper pressure zones to lower pressure zones without exceeding the allowable pressures in the lower zones or completely draining the pressure out of the higher zone. The water is transferred through a valve that reduces the pressure to a specified pressure setting, while maintaining the pressure in the upper pressure zones. PRV stations will often include several valves of various types, generally configured to use the smaller valves first and the larger valves when the smaller valve's flow capacity is not sufficient for the intended function.

The City's distribution system includes 4 active PRV stations. The locations of these PRV stations are shown on Figure 5.2, while detailed information for each PRV is summarized in Table 5.5. It should be noted that specific customers in higher pressure areas of the City may be equipped with individual home site PRVs. Any such valves are not indicated on Figure 5.2 or in Table 5.5, because individual home site PRVs are owned and maintained by the individual homeowner.

Table 5.5 Existing PRV Station Summary

Name	Valve Size (in)	Elevation (ft)	Upstream Zone	Downstream Zone	Pressure Setting (psi)
Atascadero Road (Desal Plant)	6	86.0	Upper Kings	Elena	72
MB Blvd at PD Yard	10	148.0	Upper Kings	Lower Kings	20
South St & Monterey Ave.	6	~70	Upper Kings	Lower Kings	50
Errol Street	8	~34	Upper Kings	Elena	60

Two of the City's PRVs include operational controls. The controls are described below:

- *Errol Street:* This PRV station includes on/off controls based on the level in the Elena Tanks, and is used to fill the Elena Tanks, as well as provide water service to the Elena Zone. The PRV controls are set to maintain the level in the Elena Tanks between 18.5 and 19.75 feet.

- *Atascadero Road (WTP):* The on/off controls for this PRV station are also based on the levels in the Elena Tanks. This PRV station serves as a backup to the Errol Street PRV and is set to turn on if the level in the Elena Tanks drops below 18.5 feet.

5.1.3 Existing Wastewater Collection System

The City's collection system consist of gravity sewers, lift stations, and force mains that collect and convey wastewater to the wastewater treatment plant (WWTP), at 160 Atascadero Road. The City's wastewater collection system has approximately 800 manholes, three lift stations, 9,600-feet of force mains, and 55-miles of gravity sewer pipes according to the City's GIS database. Figure 5.6 presents the City's existing collection system.

The collection system conveys wastewater primarily by gravity flow. In addition to customers within the service area, the City also serves flows from Cayucos service area. Flows from Cayucos are pumped to a manhole immediately upstream of the City's WWTP influent flume.

5.1.3.1 Gravity Collection System

The City's existing sanitary sewer collection system is comprised of 55-miles of gravity sewer from 4-inches to 27-inches in diameter, and approximately 800 manholes. Table 5.6 and Figure 5.7 present a summary by diameter of the known gravity sewers in the collection system. As shown in Table 5.6, roughly three quarters of the collection system (73.4-percent) is 6-inches in diameter. Table 5.7 summarizes the total length of gravity sewers in the collection system by pipe material.

Table 5.6 Collection System Gravity Pipeline Summary

Pipe Diameter (inches)	Length[(1)(2)] (miles)	Percent of System (by Length)
4	0.7	1.3
6	40.5	73.5
8	6.7	12.2
10	2.4	4.4
12	2.4	4.4
14	0.0	0.0
15	1.0	1.8
16	0.0	0.0
18	0.7	1.3
27	0.1	0.2
Unknown	0.6	1.1
Total	**55.2**	--

Notes:
(1) Source: City of Morro Bay's GIS database.
(2) Total lengths include only pipes owned by the City and are active.

As shown in Table 5.6, approximately 87 percent of the system is 8-inches in diameter and smaller, with the majority (73 percent) being 6-inches in diameter. Table 5.7 summarizes the gravity collections by pipe material. As shown in Table 5.7, the majority of the collection system (88 percent) consists of Vitrified Clay Pipe (VCP) and approximately 10 percent of the collection system consists of Polyvinyl Chloride (PVC) pipe.

Table 5.7 Collection System Gravity Pipeline Material Summary

Pipe Material	Length (miles)	Percent of Total System (by Length)
Asbestos Concrete	0.4	0.7
Ductile Iron	0.3	0.6
Polyvinyl Chloride	5.5	9.9
Vitrified Clay	48.7	88.3
Unknown	0.3	0.5

Notes:
(1) Source: City of Morro Bay's GIS database.
(2) Total lengths include only pipes owned by the City and are active.

5.1.3.2 Lift Stations and Force Mains

The City operates and maintains three active wastewater lift stations throughout the City (excluding private and abandoned lift stations). Figure 5.6 shows the location of each lift station. Table 5.8 summarizes the available data for each of the City's active lift stations. Table 5.9 summarizes the available data for force mains associated with their respective lift station.

Table 5.8 Lift Station Summary

Lift Station	Street Address	No. of Pumps	Capacity (per pump) (gpm)	Capacity (per pump) (mgd)	Total Capacity (mgd)
1	Coral Ave. near Emerald Cir.	2	340	0.49	0.98
2	Embarcadero at Maritime Museum Parking Lot	3	420	0.60	1.60(1)
3	Quintana Rd. and S. Bay Blvd.	2	250	0.36	0.72

Notes:
(1) City staff reports that the maximum capacity with three pumps running at this lift station is 1.60 mgd.

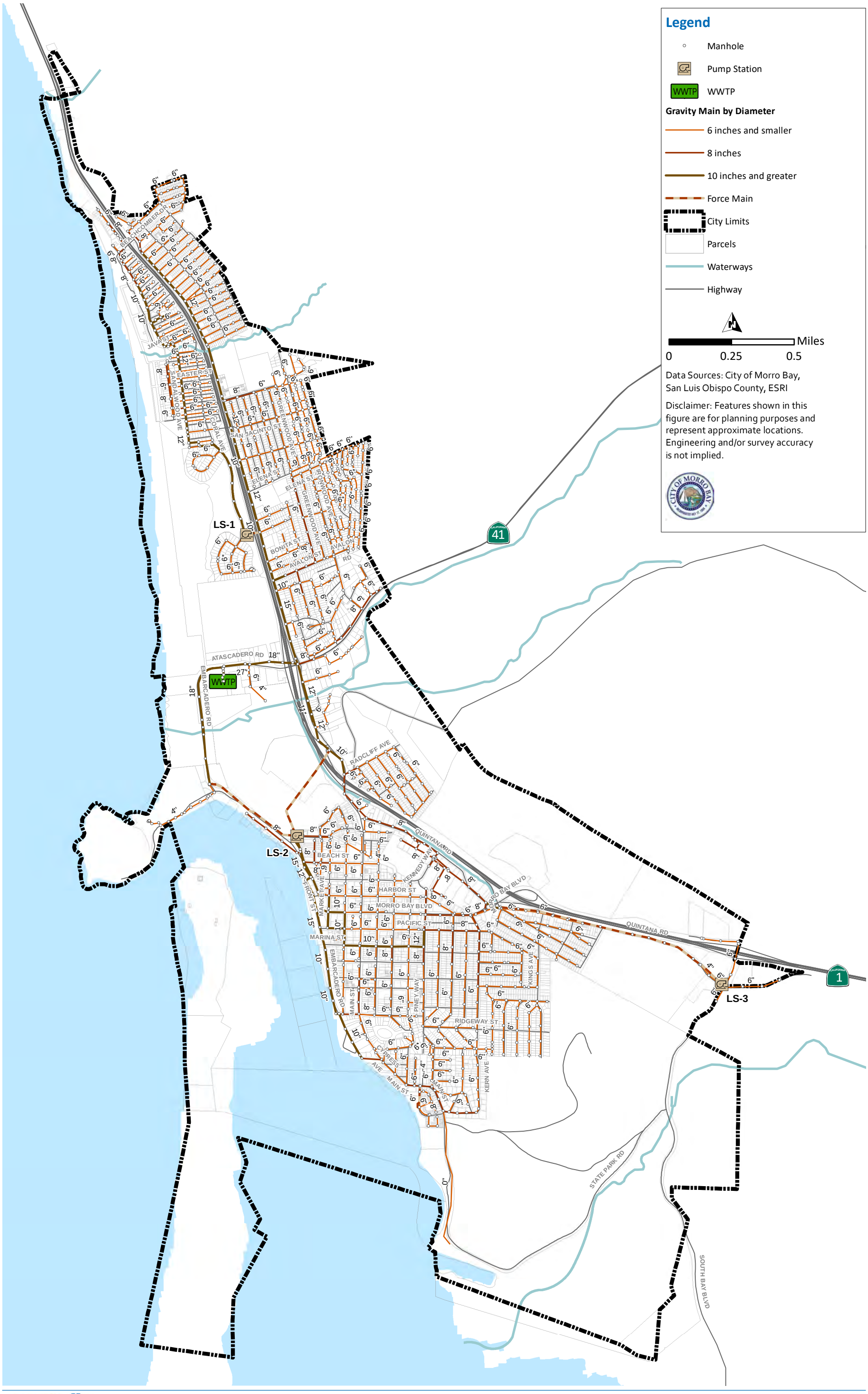

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Figure 5.6 Existing Wastewater Collection System

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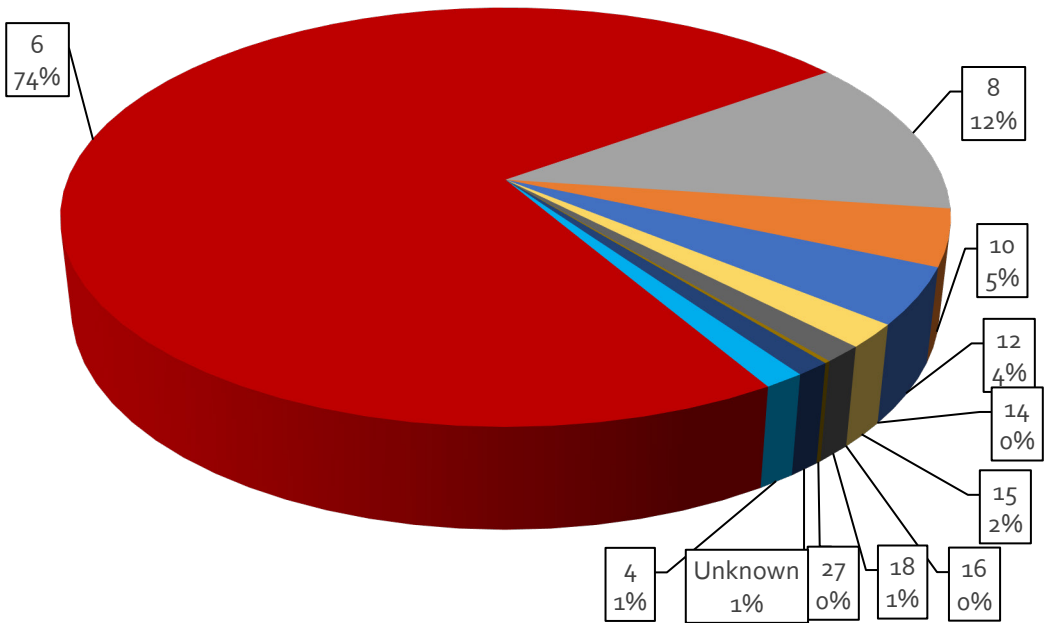

Figure 5.7 Existing Collection System Pipeline Summary, by Diameter

Table 5.9 Force Main Summary by Lift Station

Lift Station	Pipe Material[1]	Diameter[1] (inches)	Length[1] (ft)
Lift Station 1	DIP	6	275
Lift Station 2	DIP VCP	8 12	2,130 2,180
Lift Station 3	PVC	8	4,990

Notes:
(1) Source: City of Morro Bay's GIS database.

5.1.4 Existing Storm Drainage System

The City's existing storm drainage system collects and conveys surface water runoff from developed and undeveloped areas throughout the City and ultimately discharges the runoff into the Pacific Ocean.

Figure 5.8 shows the existing storm drainage system, including storm drain diameters, outfall locations, open channels, and storm basins. In total, there are approximately 10.2 miles of circular gravity storm drains and 1.0 miles of box shaped storm drain lines. Table 5.10 and Figure 5.9 summarize the total length of gravity storm drain lines by diameter. The City also contains approximately 11.6 miles of open drainage channels and streams that convey storm water through the City and to the Pacific Ocean.

Table 5.10 Storm Drain Gravity Pipeline Summary

Pipe Diameter (inches)	Length[(1)(2)] (miles)	Percent of System (by Length)
8 and Smaller	0.5	4.5
10-14	1.4	12.8
15-18	2.5	22.7
20-26	3.4	30.7
27-30	0.9	7.9
36-42	0.9	7.8
48 and Larger	0.5	4.7
Box Shaped Drains	1.0	8.7
Total	**11.2**	**100.0**

Notes:
(1) Source: City of Morro Bay's GIS database.

5.1.4.1 Existing Storm Drainage Areas

The City's storm drainage system has been broken down into six major drainage areas for the purposes of this master plan, shown on Figure 5.10. Each drainage area is described below:

- Alva Paul Creek Drainage Area. The Alva Paul Creek Drainage Area is located in the northern portion of the City, and is roughly bounded by Hatteras Street, Caster Street and Sequoia Street on the south, Java Street on the north, the Pacific Ocean on the west, and open space on the east. This drainage area is approximately 82 acres, but also includes runoff from the open space area east of the City that is tributary to Alva Paul Creek. Runoff within this drainage area is routed through a network of storm drains to Alva Paul Creek before it is discharged into the ocean.
- Chorro Creek Drainage Area. The Chorro Creek Drainage Area is a mostly rural undeveloped area on the eastern edge of the City. It is roughly bounded by the Morro Bay Golf Course on the west, Morro Bay State Park on the east, Morro Bay on the south, and open space on the north. This drainage area is approximately 1,300 acres, but also includes runoff from the open space that is tributary to Chorro Creek. Runoff within this drainage area is routed through a network of storm drains to Chorro Creek before it is discharged into Morro Bay.
- Morro Bay Drainage Area. The Morro Bay Drainage Area drains the central portion of the City's limits. It is roughly bounded by Morro Bay on the west, Chorro Creek on the south, Kern and Fresno Avenue on the east, and Willow Camp Creek on the north. This drainage area is approximately 750 acres. Runoff within this drainage area is routed through isolated storm drain networks that discharge into Morro Bay.

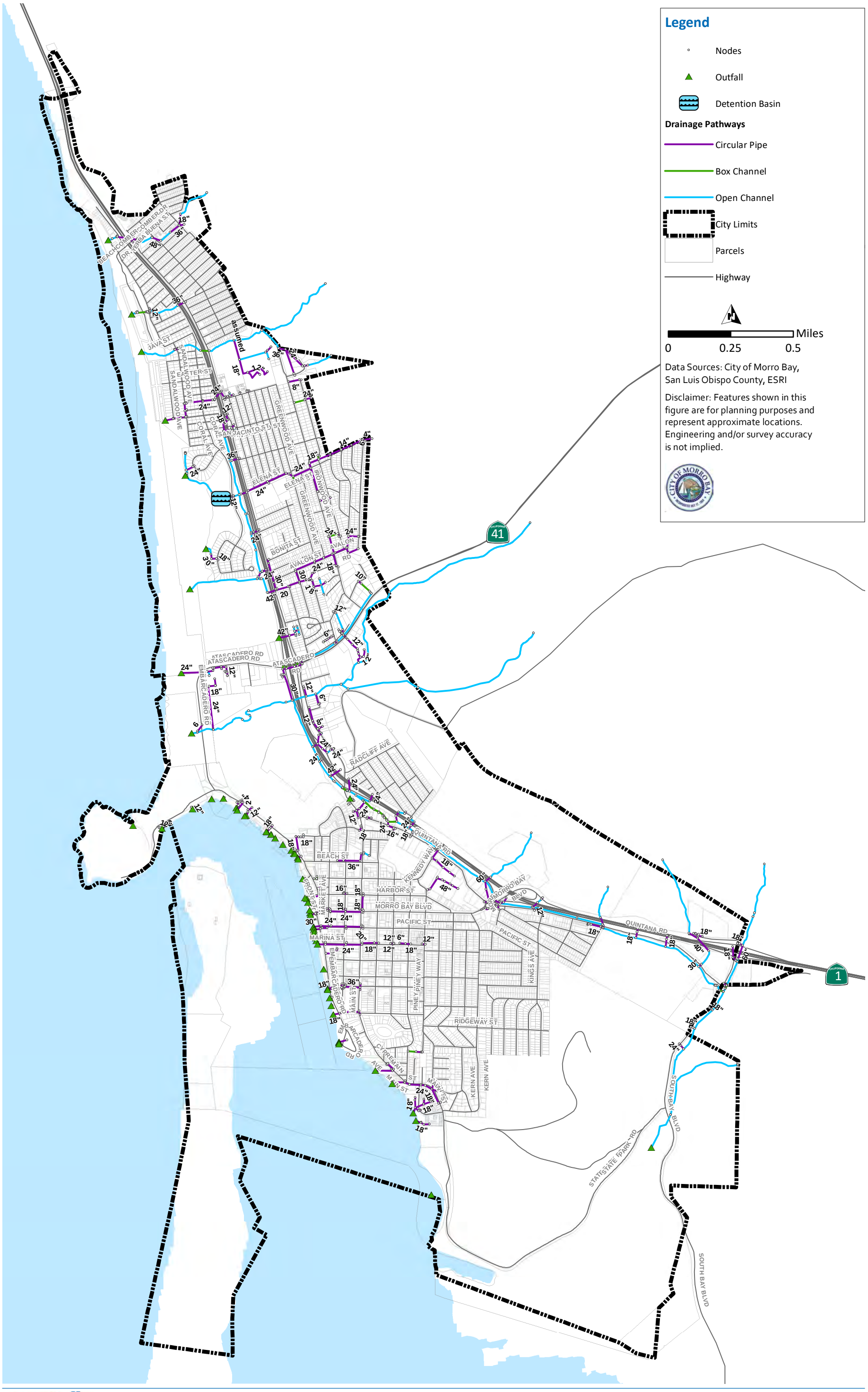

Figure 5.8 Existing Storm Drainage System

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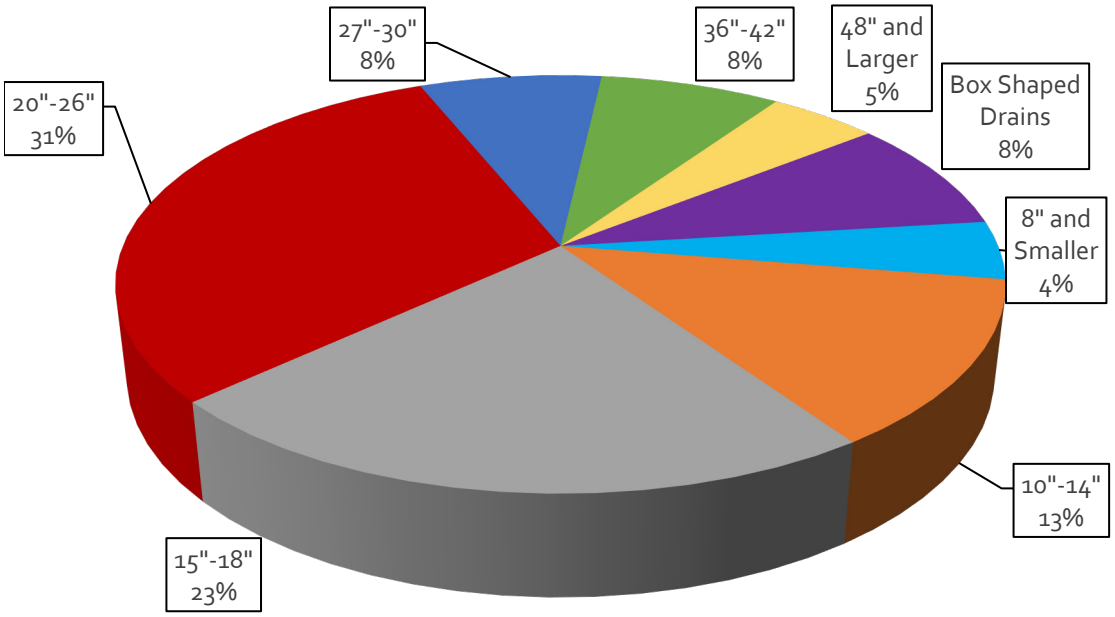

Figure 5.9 Existing Storm Drain Pipeline Summary, by Diameter

- Morro Creek Drainage Area. The Morro Creek Drainage Area is a relatively small area in the central portion of the City, near Highway 41/Atascadero Road. It is roughly bounded by the power plant to the south, Atascadero Road, Hill Street and Ponderosa Street to the north, Morro Creek to the south, and the City limits to the east. This drainage area is approximately 110 acres, but also includes runoff from the open space tributary to Morro Creek and Little Morro Creek. Runoff from Willow Camp Creek is also conveyed through Morro Creek. Runoff within this drainage area is routed through a network of storm drains to Morro Creek before it is discharged into the ocean.
- Ocean Drainage Area. The Ocean drainage basin consists of the majority of the north area of the City. It is roughly bounded by the City limits to the north, the ocean to the west, Morro Creek Drainage Area to the south, and open space to the east. This drainage area is approximately 590 acres, but also includes runoff from the open space upstream of the City limits. Runoff from the Ocean Drainage Basin is conveyed through a series isolated drainage systems out to the ocean.
- Willow Camp Creek Drainage Area. The Willow Camp Creek Drainage Area drains the north-central area of the City. It is roughly bounded by open space to the north, Chorro Creek Drainage Area to the east, Kern/Fresno Avenue, and Willow Camp Creek on the west, and Morro Creek to the north. This drainage area is approximately 640. Runoff from Willow Camp Creek Drainage Area is collected through a series of storm drains to Willow Camp Creek, which discharges into Morro Creek.

5.1.4.2 Storm Drain Detention Basins

The City operates one detention basin within the storm drainage system. The Cloisters pond was constructed in 1996 and is located within the Cloisters Community Park (see Figure 5.8). The pond detains runoff from the storm drains that convey runoff adjacent to Elena Street. It also

serves as a wetland. Runoff that enters the basin eventually flows over a weir and is discharged into the ocean.

5.1.4.3 Existing Discharge Locations (Outfall Locations)

The City currently discharges storm runoff into the Pacific Ocean at approximately 61 locations throughout the City (see Figure 5.8). Conveyance systems to the outfalls are comprised of a combination of pipelines and open channels. Several large open space areas on the coastline drain directly to the ocean and do not require conveyance infrastructure.

5.2 Hydraulic Model Development

This section describes the development of new hydraulic models of the water distribution, wastewater collection, and storm drainage systems.

5.2.1 Selected Hydraulic Modeling Software

Two hydraulic modeling software packages were utilized as part of the OneWater Plan, as described below:

- Selected water system modeling software. A new hydraulic model of the water distribution system was constructed as part of this OneWater Plan, which provided the City an opportunity to examine the software available on the market today. It was agreed that the City's hydraulic model would be constructed using InfoWater, developed by Innovyze. InfoWater is a comprehensive hydraulic and dynamic water quality modeling software application that utilizes the EPANET computational engine, which is widely used throughout the industry. InfoWater is run directly within the ArcGIS environment, and therefore offers an enhanced GUI and a variety of additional features and functionality.
- Selected sewer and storm drainage system modeling software. It was agreed that InfoSWMM, by Innovyze, would be used to assemble the City's hydraulic model. InfoSWMM is a fully dynamic, geospatial wastewater and stormwater modeling and management software application, which is built to run within the ESRI ArcGIS software platform. The hydraulic modeling engine for the InfoSWMM software package uses the Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM), which is widely used throughout the world for planning, analysis, and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems. InfoSWMM routes flows through the model using the Dynamic Wave method, which solves the complete Saint Venant, one dimensional equations of fluid flow.

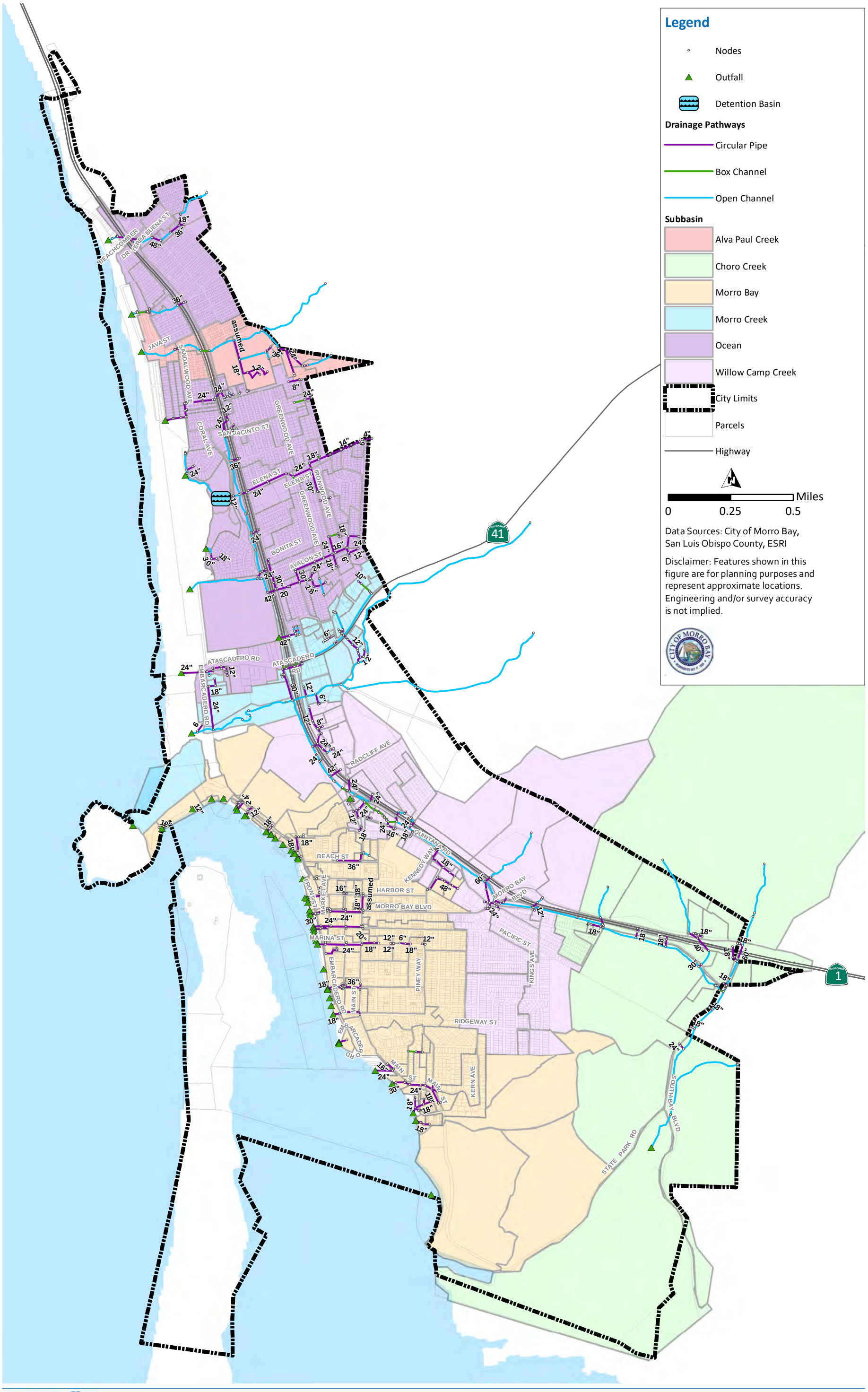

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Figure 5.10 Existing Storm Drainage System Subbasins

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5.2.2 Elements of a Hydraulic/Hydrologic Model

The following sections provide an overview of the various elements of the hydraulic models for water, wastewater, and storm drainage systems.

5.2.2.1 Elements of the Water Distribution System Hydraulic Model

The following provides an overview of the elements of a hydraulic water model and the required input parameters associated with each:

- *Junctions.* Locations where pipe sizes change, pipelines intersect, or where water demands are applied is represented by junctions in the hydraulic model. Required inputs for junctions include service elevation and water demands.
- *Pipes.* Water mains are represented as pipes in the hydraulic model. Input parameters for pipes include length, roughness (Hazen Williams C factor), diameter, and whether or not the pipe is a check valve (i.e., does not allow reverse flow).
- *Tanks.*
  - Cylindrical and Variable Area Tanks: Water tanks are included in the hydraulic model as either cylindrical tanks or variable area tanks, depending on the complexity of the tank geometry. Required input parameters for cylindrical tanks include bottom elevation, maximum level, initial level, and diameter. Required input parameters for variable area tanks include bottom elevation, maximum level, initial level, and a curve that varies the cross sectional area of the tank depending on the tank level (developed as appropriate based on As-built drawings).
  - Fixed Head Reservoirs: For water distribution system modeling, fixed head reservoirs are used to represent a water source with a constant HGL. Typically, fixed head reservoirs are used to represent water sources, such as groundwater or other sources of water.
- *Pumps.* Pumps are included in the hydraulic model as links. Input parameters for pumps include pump curves and operational controls.
- *Valves.* Certain types of valves, such as altitude valves and pressure reducing valves, are represented explicitly as valves in the hydraulic model. Required input parameters for valves include diameter, operational controls, and other settings or headloss curves depending on the type of valve.
- *Demands.* Water demands are applied at specific junctions in the hydraulic model. Up to ten different demands can be assigned at a particular junction.

5.2.2.2 Elements of the Collection System Hydraulic Model

The following provides an overview of the elements of a hydraulic wastewater model and the required input parameters associated with each:

- *Junctions:* Sewer manholes, cleanouts, as well as other locations where pipe sizes change or where pipelines intersect are represented by junctions in the hydraulic model. Required inputs for junctions include rim elevation, invert elevation, and surcharge depth (used to represent pressurized systems). Junctions are also used to represent locations where flows are split or diverted between two or more downstream links.
- *Pipes:* Gravity sewers and force mains are represented as pipes in the hydraulic model. Input parameters for pipes include length, friction factor (e.g., Manning's n for gravity

mains, Hazen Williams C for force mains), invert elevations, diameter, and whether or not the pipe is a force main.

- *Storage Nodes:* For sewer system modeling, storage nodes typically are used to represent lift station wet wells (although other storage basins, etc. can be modeled as storage nodes). Input parameters for storage nodes include invert elevation, wet well depth, and wet well cross section.
- *Pumps:* Pumps are included in the hydraulic model as links. Input parameters for pumps include pump curves and operational controls.
- *Outfalls:* Outfalls represent areas where flow leaves the system. For sewer system modeling, an outfall typically represents the connection to the influent pump station or headworks of a wastewater treatment plant.
- *Rain Gauges:* Rain gauges are input into the hydraulic model to simulate historical or theoretical hourly rainfall events.
- *Inflows:* The following are the three types of wastewater flow sources that can be injected into individual model junctions (and storage nodes):
  - External: External inflows can represent any number of flows into the collection system, such as metered flow data or groundwater inflow. External inflows are applied to a specific model junction by applying a baseline flow value and a pattern that varies the flow by hour, day, or month of the year.
  - Dry Weather: Dry weather inflows simulate base sanitary wastewater flows and represent the average flow. The dry weather flows can be multiplied by up to four patterns that vary the flow by month, day, hour, and day of the week (e.g., weekday or weekend). The dry weather diurnal patterns are adjusted during the dry weather calibration process.
  - Rainfall Derived Infiltration and Inflow (RDII): RDII flows are applied in the model by assigning a unit hydrograph and a corresponding tributary area to a given junction. The unit hydrographs consists of several parameters that are used to adjust the volume of RDII that enters the system at a given location. These parameters are adjusted during the wet weather calibration process.

5.2.2.3 Elements of the Storm Drainage Hydrologic Model

The following provides an overview of the elements of a hydrologic storm drainage model and the required input parameters associated with each:

- *Junctions:* Storm drain manholes, inlets, as well as other locations where pipe sizes change or where pipelines intersect are represented by junctions in the hydraulic model. Required inputs for junctions include rim elevation, invert elevation, and surcharge depth. Junctions are also used to represent locations where flows are split or diverted between two or more downstream links.
- *Conduits:* Gravity storm drains and open channels are represented as conduits in the hydraulic model. Input parameters for conduits include length, friction factor (e.g., Manning's n for gravity mains), invert elevations, and cross section information (diameter or dimensions).
- *Storage Nodes:* For storm drainage system modeling, storage nodes typically are used to represent detention or retention basins (although other basins, etc. can be modeled as storage nodes). Input parameters for storage nodes include invert elevation, basin depth, and cross section information.

- *Outfalls:* Outfalls represent areas where flow leaves the system. For storm drainage system modeling, an outfall typically represents a discharge to the ocean or some other water body.
- *Rain Gauges:* Rain gauges are input into the hydraulic model to simulate design storm rainfall events.
- *Subcatchments:* Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point (i.e., a modeled junction). Required input parameters for subcatchments include area, associated rain gauge, percent impervious, average subcatchment width, slope, soil types, and overland flow parameters.

5.2.3 Hydraulic Model Development

This section summarizes the steps used to develop the water distribution, wastewater collection, and storm drainage system hydraulic models.

5.2.3.1 Water System Hydraulic Model Development

The City's water system hydraulic model combines information on the physical and operational characteristics of the distribution system, and performs calculations to solve a series of mathematical equations to simulate flows in pipes. The model development process consisted of six steps, as described below:

- *Step 1:* The City's GIS database files for the distribution system was imported into the model using the "GIS Exchange" feature functionality in InfoWater.
- *Step 2:* The City's storage tanks and pump stations were then imported into the model using the "GIS Exchange" feature, and operational information, such as pump controls and PRV settings were input into the model manually based on information provided by the City.
- *Step 3:* Junctions, or areas where two pipelines meet in the model, are required at every pipe intersection and dead end, as well as other areas in the model where demands are applied. Junctions were added into the model using InfoWater's "Append Nodes" feature.
- *Step 4:* Elevations were applied to each modeled junction using the City's ground elevation contour file and the "Elevation Extractor" tool in InfoWater.
- *Step 5:* InfoWater includes several connectivity tools that are used to verify that each pipeline in the model is connected properly. The model flagged questionable pipelines and facilities, which were reviewed and corrected, if necessary.
- *Step 6:* The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run duration, time steps, reporting parameters, output units, and other technical parameters. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

Diurnal Pattern Development

As a part of the calibration process, the City provided flow data for the City's State Water Project supply, and tank level data. This data was used to establish a daily diurnal demand pattern by balancing the total inflow into the water distribution system and the change in storage. The

diurnal pattern is shown on Figure 5.11, and was applied into the hydraulic model for use in 24 hour model simulations.

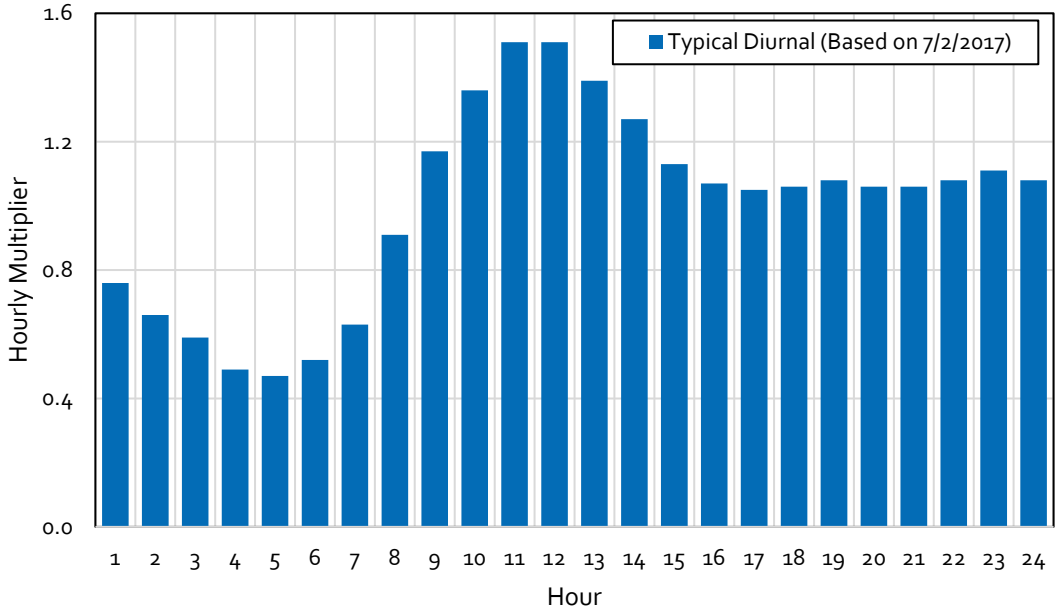

Figure 5.11 Water System Diurnal Pattern

Water Demand Allocation

Allocation of water demands to appropriate nodes in the hydraulic model was accomplished in several steps using the City's water billing records. The City provided a MS Excel based database with monthly water consumption, by customer, for the years 2013 through 2016. The City's 2013 billing data was used as the basis for the model demand allocation.

The City's 2013 water billing records were manipulated in excel to calculate the total average day consumption for each City customer. This billing data was then "geocoded" using GIS techniques to develop a point shapefile with the spatial location of each billing record with the annual consumption. This shapefile was used along with InfoWater's "Demand Allocator" tool to allocate the system demand into the model. Once the geocoded billing data was allocated into the hydraulic model, the model demands were "scaled up" to match the existing average day demand (ADD), thereby accounting for the additional demand associated with unaccounted-for-water (UFW). Finally, diurnal patterns were input into the model (see Figure 5.11) to account for the temporal change in demands throughout the day. Allocation of future water demands followed a similar procedure as the allocation of existing water demands.

5.2.3.2 Wastewater Collection System Hydraulic Model Development

The City's hydraulic model combines information on the physical and operational characteristics of the wastewater collection system, and performs calculations to solve a series of mathematical equations to simulate flows in pipes. The model construction process consisted of five steps, as described below:

- *Step 1:* The City's GIS shapefiles for the sewer collection system were obtained. Recent projects not in their GIS data were added based on as-built drawings provided by the City.
- *Step 2:* The GIS data was reviewed and formatted to allow easy import into the InfoSWMM® modeling platform.
- *Step 3:* The GIS shapefiles were imported into the hydraulic model. Physical and operational data for the City's wastewater collection facilities was not available from the GIS data. This type of data, such as wet well dimensions, lift stations, and other special features, were input manually into the model based on available information. In addition, pipelines and junctions with missing inverts or invert discrepancies were reviewed and manually input or modified based on the City's as-built records, and engineering judgment. For instance, pipelines with missing invert elevations were estimated using a constant slope based on upstream and downstream pipelines with invert elevations. Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the model data was input correctly and that the flow direction and size of the modeled pipelines were logical.
- *Step 4:* Dry weather wastewater flows were then allocated to the appropriate model junctions. These flows were scaled up or down, as necessary, to match the dry weather flows recorded during the flow monitoring period.
- *Step 5:* The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run dates, time steps, reporting parameters, output units, and flow routing method. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

Wastewater Load Allocation

Determining the quantity of average dry weather flows (ADWF) generated by a municipality and how they are distributed throughout the collection system is a critical component of the hydraulic modeling process.

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

- *Step 1:* The City's service area was broken up into individual loading polygons. In an all pipe model, such as the City's hydraulic model, a loading polygon will usually encompass an area the size of a few lots. Each loading polygon represents the geographic area that contributes flows into a single model node (i.e., manhole). Loading polygons were developed using GIS, based on the City's parcel and sewer pipeline shapefiles.
- *Step 2:* One approach for estimating the existing ADWF associated with each loading polygon is based on land use designations, flow coefficients, and land use area. In reality, the wastewater generation rates of each existing customer will vary from an average flow coefficient (significantly in some cases). For this reason, water billing records can be considered as an alternative to the land use based load allocation method for existing dry weather flows. For this project, water consumption billing records/sewer

billing records by parcel were available. Thus, the billing records were used. Billing records were assigned point loads in GIS. Loading polygons were developed based on the parcels that discharge into certain manholes. Using InfoSWMM's "Load Allocator" tool point loads, and the loading polygons calculated the loads in the model. The land use method was used for future 2040 loads. Wastewater flow coefficients were developed by balancing typical wastewater flow coefficients with the flow monitoring data. Connected parcels were assigned a land use in GIS, according to the One Morro Bay. Loading polygons were developed based on the parcels that discharge into certain manholes. Using InfoSWMM's "Load Allocator" tool flow coefficients, connected parcels, and the loading polygons calculated the ADWF loads in the model.

- *Step 3:* Once the existing water demand/land use based loads were allocated into the model, they were adjusted as needed during model calibration to closely match the dry weather flows recorded during the flow monitoring program. During the system evaluation existing ADWF were scaled according to the existing ADWF developed based on AWDF.

5.2.3.3 Storm Drainage Hydraulic Model Development

The City's storm drainage system hydraulic model combines information on the hydraulic and hydrologic characteristics of the drainage system, and performs calculations to solve a series of mathematical equations to simulate flows in pipes.

The model development process consisted of the development of the both the hydrologic and hydraulic portions of the model.

Hydrologic Model

Hydrologic analysis of the City's storm drainage system was performed using the SWMM Runoff Block, which is included in the InfoSWMM modeling software. The SWMM Runoff Block was designed to simulate the surface water runoff response of a drainage basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. The Runoff Block was used to simulate the quantity of storm water runoff that flows overland in each subbasin during a particular storm event.

In the SWMM Runoff Block, each model component represents a specific aspect of the rainfall-runoff processes occurring in a portion of the watershed. A component may represent the runoff occurring in a subbasin, the routing of flows down a drainage channel, or the routing of flows through a detention basin. The model operates by reading an input data file that contains the parameters describing each component of the drainage basin, along with information describing how the various components work together to form the drainage basin. The result of the modeling process was a tabulation of flow hydrographs at desired locations within the study area.

The Runoff Block output data was generated by the model based on the input parameters detailed below. Parameters describing the various components of the model are based on land use, soils, vegetation, drainage channels, and topography. For example, the land use in a subbasin will determine the percent of that subbasin that is impervious. These values, along with others describing additional components of the subbasin, are placed in a computer input data file that is read by the SWMM Runoff computer model and used as a basis for computation of the rainfall-runoff processes in the subbasin.

- *Design Hydrographs.* Design hydrographs were determined using the SWMM Runoff Module of InfoSWMM, which is incorporated in the Morro Bay storm water model. The 10-year and 100-year, 24 hour storms were used in the analysis. The design storms are shown in Chapter 3 for reference.
- *SWMM Hydrologic Unit (Subcatchment).* Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The City was divided up into 405 individual subcatchments and the appropriate outlet point was defined. The area and boundary of each subcatchment was determined with the use of development plans, available topographic data, and field observations to determine the drainage path. Hydrologic characteristics for each subcatchment were input based on the information presented in Chapter 3.

Hydraulic Model

The InfoSWMM hydraulic model was used to simulate the hydraulic conditions in the City's storm drainage system. The computer hydraulic model was used to analyze the storm drainage system, to identify deficiencies, and to propose system improvements.

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

- *Step 1:* The City's GIS shapefiles for the storm drainage system were obtained. Recent projects not in their GIS data were added based on as-built drawings provided by the City.
- *Step 2:* The GIS data was reviewed and formatted to allow easy import into the InfoSWMM® modeling platform.
- *Step 3:* The GIS shapefiles were imported into the hydraulic model. The City's storm drainage system GIS database included limited pipeline invert information. Reasonable assumptions were developed and input for all pipelines missing invert data.
- *Step 4:* Open channel data was input into the model. Cross sectional information for open channels and streams was typically not available. Cross sectional information for these facilities were estimated based on aerial/street view photography. Open channels were typically simulated as trapezoidal cross sections. Channel inverts were typically estimated using the City's topographic data (major creek inverts were available from FEMA's San Luis Obispo County 2012 Flood Insurance Study).
- *Step 5:* Outfalls were added to the model to represent discharges into the ocean. Hydraulic grade line stages included for major creeks were input based on FEMA 2012 Flood Insurance Study Data. Other outfalls were assumed to be "Free Discharges"
- *Step 6:* Street drainage was added to the model as needed to connect bleeder storm drains to a downstream inlet. The cross sectional areas assumptions for street drainage were assumed based on the typical roadway dimensions (e.g., 1 foot deep and 20 feet wide). Ponded areas were also included in model as appropriate.
- *Step 7:* Modeling data for the Cloisters pond was input based on information provided by the as-built drawings for this facility.
- *Step 8:* The subcatchment areas were input into the model.
- *Step 9:* Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the model data was input correctly and that the flow direction and size of the modeled facilities were logical.
- *Step 10:* The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run dates, time steps, reporting

parameters, output units, and flow routing method. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

5.3 Hydraulic Model Calibration

This section summarizes the model calibration process.

5.3.1 Water Hydraulic Model Calibration

This section summarizes overall methodology employed to calibrate the City's water system hydraulic model and the calibration results, including a detailed description of each of the major components of the model calibration process.

5.3.1.1 Model Calibration Data Collection

Carollo coordinated closely with City staff regarding the supervisory control and data acquisition (SCADA) and field data needs that were required to calibrate the hydraulic model. The required calibration data included site maps for specific fire flow test locations, pressure logger locations, and included a list of the SCADA data needs, durations, time intervals, and units. This section summarizes the data collection process that was conducted

- *SCADA and Historical Fire Flow Data Gathering:* Field testing and data gathering for model calibration took place for June 1 – July 6, 2017. For the EPS data gathering period, Carollo coordinated with City staff to obtain 1-minute data for all of the major SCADA points within the water distribution system, including flow data and reservoir levels. The location of major facilities in the system where SCADA data were available is shown on Figure 5.12. This data was primarily used to generate the City's diurnal pattern and for EPS model calibration, but it was also used to identify boundary conditions for the fire flow calibration. The data was primarily used to complete additional fire flow calibrations across the system.
- *Temporary Pressure Logger Installation:* In addition to the data obtained from the City's SCADA system from the major system facilities, Carollo also provided temporary pressure loggers to City staff that were attached to hydrants within the City's distribution system. The data obtained from the temporary pressure loggers consisted of 5-minute pressure data for the duration of the EPS data gathering periods. Figure 5.12 shows the hydrant locations where the temporary pressure loggers were installed. The pressure logger distribution in late June and early July 2017 was selected to get a good representation of system pressures throughout the City.
- *Fire Flow Field Testing:* Carollo selected ten fire flow testing sites, which are shown on Figure 5.13 to conduct additional fire flow field testing. The tests were conducted on July 7, 2017 at each of the ten selected sites.

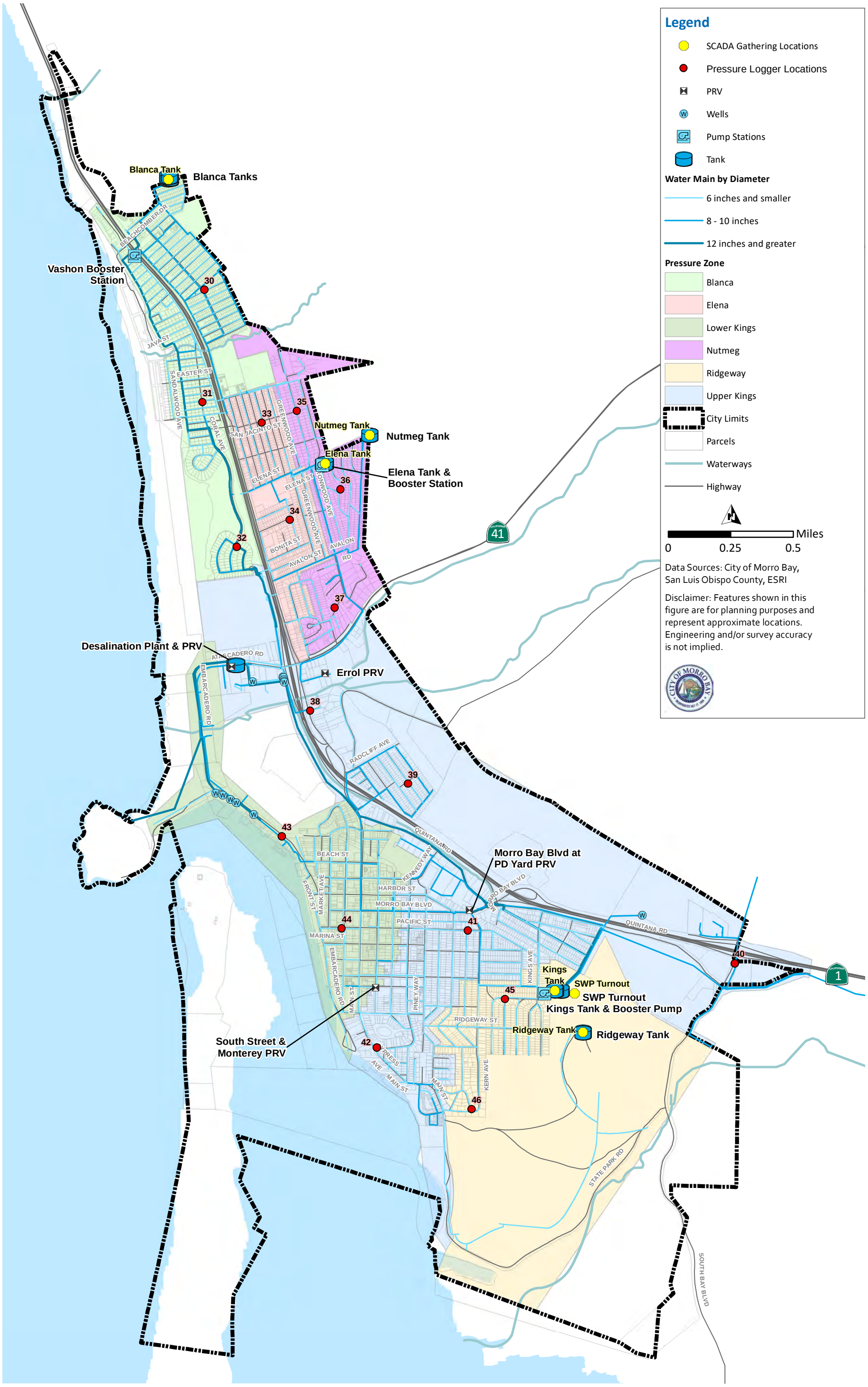

Figure 5.12 SCADA Gathering Locations and Pressure Logger Locations

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Blanca Tanks
FF TEST SITE 1
Vashon Booster Station
FF TEST SITE 2
Nutmeg Tank
Elena Tank & Booster Station
FF TEST SITE 3
FF TEST SITE 4
Desalination Plant & PRV
Errol PRV
FF TEST SITE 5
Morro Bay Blvd at PD Yard PRV
FF TEST SITE 10
FF TEST SITE 7
FF TEST SITE 6
FF TEST SITE 8
SWP Turnout
Kings Tank & Booster Pump
Ridgeway Tank
South Street & Monterey PRV
FF TEST SITE 9
BEACHCOMBER DR
JAVA ST
EASTER ST
SANDALWOOD AVE
CORAL AVE
SAN JACINTO ST
GREENWOOD AVE
ELENA ST
AVALON ST
AVALON RD
BONITA ST
EMBARCADERO RD
ATASCADERO RD
RADCLIFF AVE
QUINTANA RD
HARBOR ST
MORRO BAY BLVD
PACIFIC ST
MARINA ST
FRONT ST
MARKET AVE
KENNEDY WAY
MORRO BAY BLVD
KINGS AVE
PINEY WAY
RIDGEWAY ST
MAIN ST
KERN AVE
STATE PARK RD
SOUTH BAY BLVD
41
1

Legend

Test Hydrant Types
- F-1
- P-1
- P-2
- PRV
- Wells
- Pump Stations
- Tank

Water Main by Diameter
- 6 inches and smaller
- 8 - 10 inches
- 12 inches and greater

Pressure Zone
- Blanca
- Elena
- Lower Kings
- Nutmeg
- Ridgeway
- Upper Kings

- City Limits
- Parcels
- Waterways
- Highway

Miles
0 0.25 0.5

Data Sources: City of Morro Bay, San Luis Obispo County, ESRI

Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure 5.13 Fire Flow Test Locations

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5.3.1.2 Model Calibration Methodology and Results

The purpose of a water system hydraulic model is to estimate, or predict, how the water distribution system will respond under a given set of conditions. One way to test the accuracy of the hydraulic model is to create a set of known conditions in the water system and then compare the results observed in the field against the results of the hydraulic model simulation using the same conditions. Flow tests conducted in the field on the water system can yield a profound tool in verifying data used in the hydraulic model and a greater understanding of how the water system operates.

Field testing can indicate errors in the data used to develop the hydraulic model, or show that a condition might exist in the field not otherwise known. Valves, which are reported as being open, might actually be closed (or vice versa), an obstruction could exist in a pipeline, or pressure settings for a PRV may be slightly different than noted. Field testing can also correct erroneous model data such as incorrect pipe diameters or connections.

Data obtained from the field tests can be used to determine appropriate roughness coefficients for each pipeline, as roughness coefficients can vary with age and pipe material. Other parameters can also be adjusted to generate a calibrated model.

The calibration process for the City's water distribution system hydraulic model consisted of three parts, a macro calibration, a fire flow test calibration, and an EPS calibration.

Macro Calibration

Initially, the model was run under existing demand conditions and necessary adjustments were made to produce reasonable system pressures. Such adjustments include modifications of pipeline connectivity, operational controls, ground elevations, and facility characteristics.

The macro calibration process involves several steps to ensure that the model produces reasonable results:

- *Transmission Main Connectivity.* Using the connectivity features of the modeling software, the connectivity of the transmission mains within the distribution system was verified. Problems found using the connectivity locators were reviewed to determine whether adjustments were needed to the connectivity of the model. Output reports of pipe flow characteristics, such as headloss (feet per thousand feet [ft/kft]) and velocity (feet per second [fps]) were also used to locate problem areas where additional adjustments may be necessary.
- *System Pressures.* The macro calibration compared the model output to the typical pressures observed within the distribution system in pounds per square inch (psi). This process was used to locate major errors in model creation, elevations, or connectivity, as well as changes that reflect how operational controls of the system should be implemented in the model.
- *Facility Characteristics.* Hydraulic model results were compared to data provided by the City to verify that facility attributes entered into the model, such as the physical characteristics of the tanks and PRV stations, produced results comparable to what the City experiences.

Extended Period Simulation Calibration

The purpose of extended period simulation calibration is to demonstrate the hydraulic model's ability to accurately mimic the real world operations of the distribution system facilities, including pressure fluctuations, tank fill/drain cycles, PRV station operation, and pump station operations. The primary varied parameters for this calibration were operational controls and pipeline roughness coefficients, although other parameters were also adjusted as calibration results were generated.

The first step in the EPS calibration process is the selection of a calibration day. During the EPS calibration period, it was determined that Wednesday July 2, 2017 was the most appropriate calibration day. July 2, 2017 was chosen because it represented a typical higher demand day and because there were no unusual flow spikes or dips in the system-wide diurnal.

The calculated daily demand for the calibration day was estimated to be 1.37 mgd (948 gallons per minute [gpm]) for July 2, 2017. Therefore, the demands that were allocated in the hydraulic model were scaled to match a total demand of 1.37 mgd for the purposes of the EPS calibration scenario.

The EPS calibration compared model simulated pump station flows, tank levels, and PRV station status (if available) to the field measured data. In addition, model simulated pressures at the pressure logger locations were compared to the actual field pressures recorded during the calibration day.

A sample comparison of model results to observed field conditions for the Kings East Tank Level and Pressure Logger 32 for the EPS calibration period is shown on Figure 5.14 and Figure 5.15, respectively. Similar model results for the remaining facilities are presented in Appendix H.

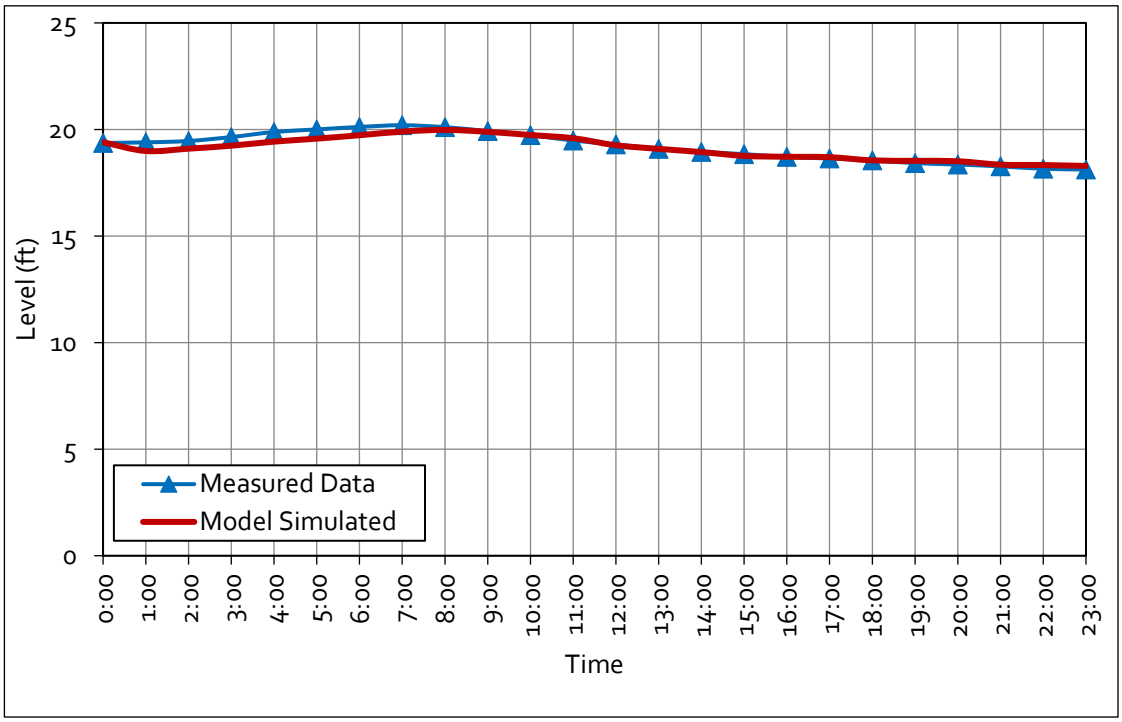

Figure 5.14 Example EPS Calibration Result – Kings East Tank Level

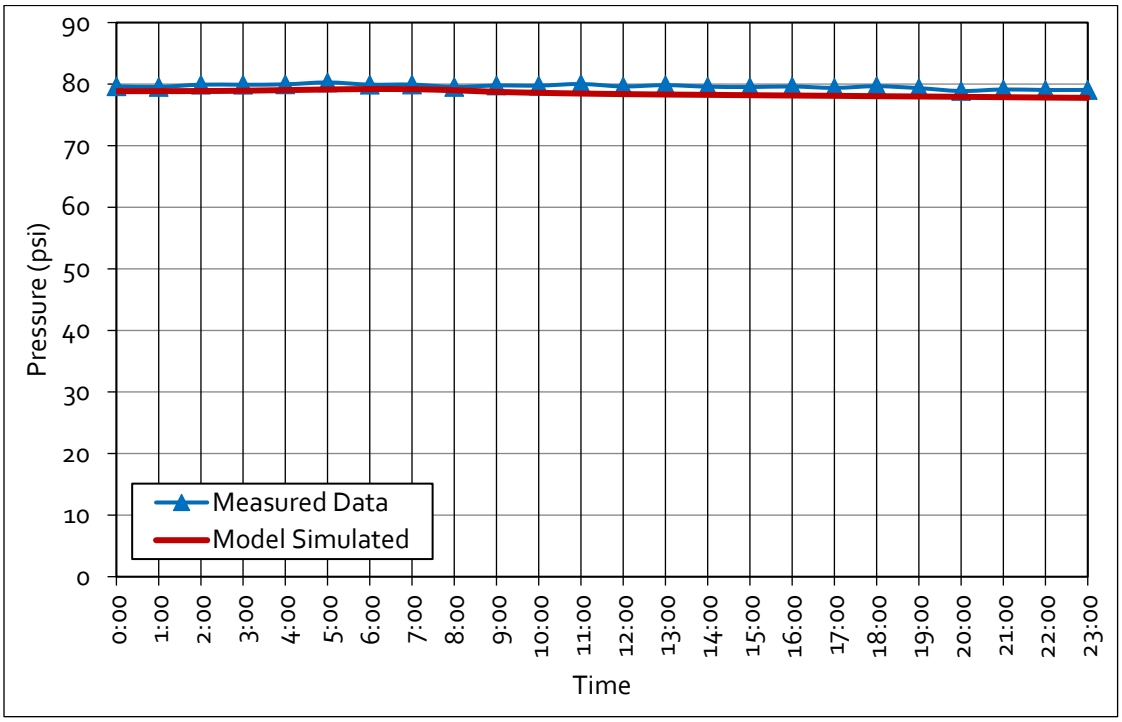

Figure 5.15 Example EPS Calibration Result – Pressure Logger 32

Overall, the trends seen in the field data are well predicted by the model, with the exception of a few pressure loggers. Some notable items from the EPS model calibration effort include:

- *Pressure Loggers 38 and 39:* The model was unable to replicate the measured pressures at these locations. It is recommended that pressure loggers be re-deployed at a later date to identify the cause of the issue.

Fire Flow Test Calibration

The purpose of fire flow calibration is demonstrate the model's ability to accurately replicate the performance of the distribution system of extreme demand conditions, such as when fire hydrants are being operated. The primary varied parameter for this calibration is pipeline roughness coefficient, although other parameters can also be adjusted as calibration results are generated.

Hazen-Williams roughness coefficients, or C-factors, have industry accepted value ranges based on pipeline material, diameter, and age. Characteristics specific to the City water distribution system such as water quality, temperature, construction methodologies, material suppliers, and other factors may result in roughness coefficients that differ from the average of the industry accepted ranges. Fire flow calibration refines the initial estimation of the value of roughness coefficients that best indicate the conditions of the City's distribution system.

During average day flows, roughness coefficients have a relatively small effect on the operation of the distribution system. However, as the flows increase in the system on higher demand days, velocity within pipelines increase and roughness coefficients contribute more to overall system headloss. Fire flow tests artificially create high demand events to generate more headloss, allowing a better estimation of the pipeline roughness coefficients.

Fire flow tests stress the distribution system by creating a differential between the HGL at the point of hydrant flow and the system HGL at neighboring hydrants. This HGL differential increases the effect of the roughness coefficients on system losses and allows adjustments to the model to match model pressures to field pressures within an acceptable tolerance. As the model is adjusted to match system pressures, roughness coefficients should be adjusted only within a tolerance of industry accepted roughness coefficient ranges. If a model is unable to match the calibration results without leaving the acceptable range of roughness coefficient values for a given pipeline material and age, there may be cause for further investigation of a previously unknown field condition. Examples of such conditions, which typically arise during hydraulic model calibration, include closed valves, partially closed or malfunctioning valves, extreme corrosion within pipelines, connectivity, and diameter errors in GIS layers or record drawings, and diurnal patterns of large water users.

A separate hydraulic model scenario was created for each flow test for both the static and the dynamic, or flowing, condition. The flow observed at each fire flow hydrant was assigned as a demand to the model node at the location of the hydrant. Residual pressures were then read at each hydrant location while the hydrant was flowing. Model results were considered acceptable if they fall within a 10 percent tolerance or a 10 psi value. Table 5.11 shows a summary of the July 2017 fire test model calibration results.

As shown in Table 5.11, the comparison of model results to observed field data are good. There are a few notable items from the fire flow calibration results:

- *Fire Flow Test 3:* The hydraulic model was not able to simulate the static pressure measured by City staff. It appears that the static pressure recorded at this location is incorrect.
- *Fire Flow Test 10:* The hydraulic model was not able to simulate the static pressure measured by City staff. The static pressures fluctuated significantly during the test, and therefore the simulated pressures are acceptable.

5.3.1.3 Hydraulic Model Calibration Summary

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

Based on the results presented in this chapter, it can be concluded that the model is calibrated to steady state and extended period conditions. The model provides an accurate representation of the City's distribution system and system operations to a level suitable for OneWater and for the City's future hydraulic modeling needs.

Table 5.11 Fire Flow Calibration Results (July 17, 2017)

Test Site	Test Location (Pressure Zone)	Time	Hydrant Type	Field Measured Data			Model Simulated Data		Percent Difference		Pressure Drop			Notes
				Hydrant Flow (gpm)	Hydrant Pressure (psi)		Hydrant Pressure (psi)				Measured (psi)	Modeled (psi)	Difference	
					Static	Residual	Static	Residual	Static	Residual				
1	Tide Avenue and Oahu St (Blanca)	8:00	Flowing	850	68	60	6839	--	1.3	--	--	--	--	
			Pressure 1	--	71	66	71.5	67.1	0.7	1.7	5	4.4	-1	
			Pressure 2	--	51	43	50.9	46.5	-0.2	8.1	8	4.4	-4	
2	Juniper Ave South of Sequoia St. (Nutmeg)	8:16	Flowing	1,127	104	60	104.1	--	0.1	--	--	--	--	
			Pressure 1	--	104	70	108.1	67.9	3.9	-3.0	34	40.2	6	
			Pressure 2	--	116	82	115.3	93.1	-0.6	1.3	34	32.2	-2	
3	Elm Ave and Bonita St (Elena	8:30	Flowing	820	40	30	41.1	--	2.8	--	--	--	--	
			Pressure 1	--	46	41	45	40.9	-2.2	-0.2	5	4.1	-1	
			Pressure 2	--	48	30	35.1	31.6	**-26.9**	5.3	18	3.5	-15	
4	Rockview St east of Sunset Rd (Nutmeg)	8:42	Flowing	1,000	142	58	142.6	--	0.4	--	--	--	--	
			Pressure 1	--	142	68	144.7	68.7	1.9	1.0	74	76	2	Hydrant P-2 in Elena Zone (no residual drop)
			Pressure 2	--	41	41	44.7	44.7	9.0	9.0	0	0	0	
5	Morro Ave south of Scott St (Lower Kings)	8:57	Flowing	700	58	48	53	--	-8.6	--	--	--	--	
			Pressure 1	--	54	48	54	46.4	0.0	-3.3	6	7.6	2	
			Pressure 2	--	60	51	61.5	54.1	2.5	3.1	9	7.4	-2	
6	Embarcadero Rd (Lower Kings)	~9:10	Flowing	930	86	52	83.3	--	-3.1	--	--	--	--	
			Pressure 1	--	81	60	81.5	64.9	0.6	8.2	21	16.6	-4	
			Pressure 2	--	81	59	83.5	64.8	3.1	9.8	22	18.7	-3	
7	Estero Ave south of Pacific St (Upper Kings)	9:27	Flowing	1,020	70	60	73.1	--	4.4	--	--	--	--	
			Pressure 1	--	76	66	75.6	67.7	-0.5	8.2	21	16.6	-4	
			Pressure 2	--	63	57	68.4	57.3	-0.9	9.8	22	18.7	-3	
8	Arcadia Ave and Ridgeway St (Ridgeway)	9:40	Flowing	700	73	48	71	--	-2.7	--	--	--	--	Pressures off due to discrepancy from EPA calibration
			Pressure 1	--	72	52	72.8	53.4	1.1	2.7	20	19.4	-1	
			Pressure 2	--	59	40	60.6	42.6	2.7	6.5	19	18	-1	
9	Main St south of Walnut St (Upper Kings)	9:53	Flowing	1,300	118	88	114.3	--	-3.1	--	--	--	--	
			Pressure 1	--	115	92	114.6	87.4	-0.3	-5.0	23	27.2	4	
			Pressure 2	--	115	84	111.5	84.9	-3.0	1.1	31	26.6	-4	
10	Teresa Rd west of South Bay Blvd (Upper Kings)	10:14	Flowing	720	126	56	99.6	--	**-21.0**	--	--	--	--	Static pressure fluctuated significantly during test
			Pressure 1	--	102	36	101.2	33.8	-0.8	-6.1	66	67.4	1	
			Pressure 2	--	100	37	99.8	33.7	-0.2	-8.9	63	66.1	3	

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5.3.2 Wastewater Hydraulic Model Calibration

Hydraulic model calibration is a crucial component of the hydraulic modeling effort. Calibrating the model to match data collected during the flow monitoring program ensures the most accurate results possible. The calibration process consists of calibrating to both dry and wet weather conditions.

For this project, dry weather flow (DWF) monitoring was conducted at V&A Consulting Engineer's (V&A) 15 temporary metering sites from April 7, 2017 to July 13, 2017. The DWF calibration ensures an accurate depiction of base wastewater flow generated within the study area. The wet weather flow (WWF) calibration consists of calibrating the hydraulic model to a specific storm event or events to accurately simulate the peak and volume of infiltration/inflow (I/I) into the sewer system. For this project, WWF calibration used the four City owned meters. The WWF calibration consisted of five storm events from January 3, 2017 through February 22, 2017. The amount of I/I is essentially the difference between the WWF and DWF components.

5.3.2.1 Calibration Standards

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

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

5.3.2.2 Dry Weather Flow Calibration

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

- Divide the system into areas tributary to each flowmeter. The first step in the calibration process was to divide the City into flowmeter tributary areas, one for each flowmeter site. A map showing the locations of each flow monitoring site and their associated tributary area are provided in Chapter 4 along with a schematic of the flowmeters.
- Define flow volumes within each area. The next step was to define the flow volumes within each area, which was accomplished in the flow allocation step.
- Create diurnal patterns to match the temporal distribution of flow. A diurnal curve is a pattern of hourly multipliers that are applied to the base flow to simulate the variation in flow that occurs throughout the day. Two diurnal curves were developed for each flow monitoring tributary area, one representing weekday flow and one representing weekend flow. The diurnal patterns were initially developed based on the flow monitoring data and adjusted as part of the calibration process until the model simulated flows matched the field measured flows as closely as possible. Figure 5.16 shows the calibrated weekday and weekend diurnal pattern for the area tributary to Site 11. Additional diurnal patterns were developed for all flowmeter tributaries. These diurnal patterns are found on the DWF calibration sheets that are included in Appendix I.

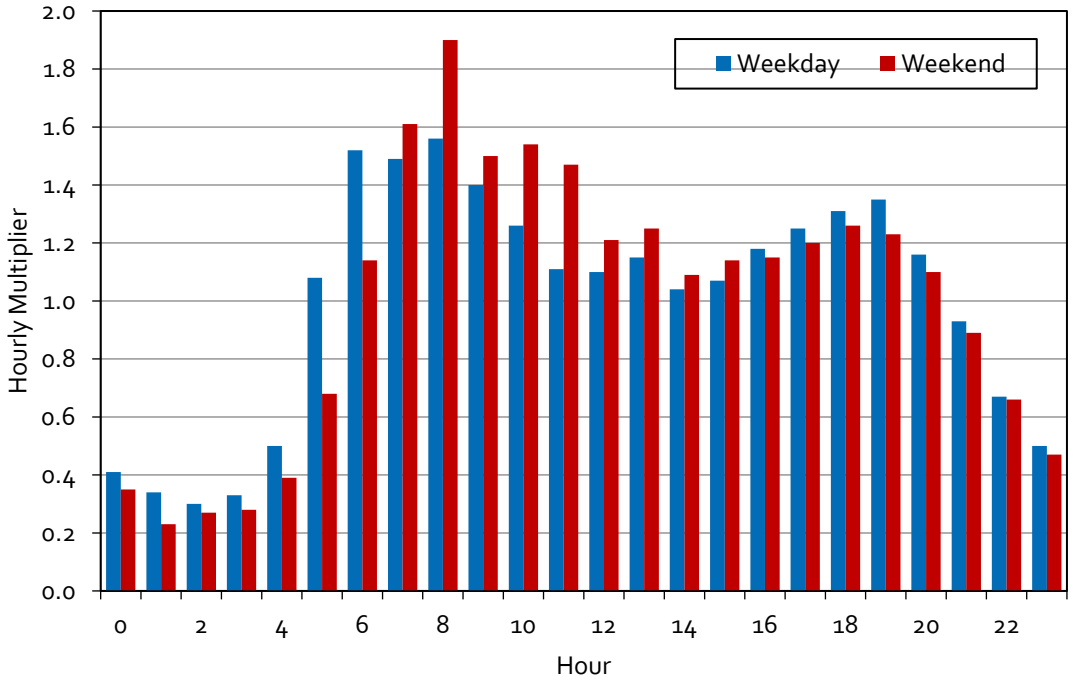


Figure 5.16 Example Weekday and Weekend ADWF Diurnal Patterns (Site 11)

- Adjust model variables to match field measured velocity and flow depths. Once the model simulated flows acceptably matched the field measured flows, the model simulated velocity and flow depth were compared to the field measured velocity and flow depth. Adjustments were made to various model parameters until the modeled and measured velocity and depth closely matched one another. The primary varied

parameters for this process are pipeline roughness (Manning's n) and sediment buildup in the pipe, although other parameters can also be adjusted as calibration results are generated.

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

Figure 5.17 is an example DWF calibration sheet for flowmeter Site 13. Calibration sheets provide plots and tables that compare model results and the field measured flow, velocity, and level for during the calibration period. Appendix I contains detailed DWF calibrations sheets for all meter locations. As shown on Appendix I, a majority of model simulated average flows for weekday and weekend DWF were all within 10 percent. The few meter locations fell outside the calibration standard and are discussed below:

- Greyline Meters. The available dry weather flow monitoring data from the Greyline meters did not correspond with the time frames included in the temporary flow monitoring program, therefore, the measured dry weather flows at the Greyline meters were not exactly the same as the temporary flow meters. In general, the temporary flow monitoring data was assumed to be more accurate. The Greyline meter calibration was shown to verify that the modeled flows from the flow monitoring program are reasonable, however, it was not possible to calibrate both the temporary flow meters and the Greyline meters to within 10-percent. Therefore, the model was calibrated to more closely match the temporary meter data.

With a few exceptions, the hydraulic model met the established dry weather calibration standards. This sites that did not meet the established calibration standards require further investigation to determine the cause for the discrepancy. Overall, the model accurately simulates DWF, and the sites that did not had little impact on the model's overall accuracy. For these reasons the mode is considered calibrated for DWF conditions.

5.3.2.3 Wet Weather Flow Calibration

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

- Identify calibration rainfall events. For this project, the WWF calibration process consists of running model simulations of historic rainfall. The goal of any WWF calibration is to capture and characterize a system's response to a significant rainfall event, preferably

during wet antecedent moisture conditions. Between January 3, 2017 and February 26, 2017, five storm events were selected for the model calibration. The storm events are as follows:

- Storm Event 1: January 3, 2017 through January 6, 2017
- Storm Event 2: January 7, 2017 through January 17, 2017
- Storm Event 3: January 18, 2017 through January 25, 2017
- Storm Event 4: February 5, 2017 through February 12, 2017
- Storm Event 5: February 16, 2017 through February 22,2017

- Define RDII tributary areas. For the WWF calibration, RDII flows are superimposed on top of the DWF. The model calculates RDII by assigning "RDII Inflows" to each node in the model. RDII inflows consist of both a unit hydrograph and the total area that is tributary to the model node. The RDII tributary areas were calculated in GIS using the loading polygons. The tributary area provides a means to transform hourly rainfall depth from the rainfall hyetographs into a rainfall volume. The rainfall volume is transformed into actual RDII flows using the unit hydrograph, as described in the next step.
- Create I/I parameter database and modify to match field measured flows. The main step in the WWF calibration process involved creating a custom unit hydrograph for the City service area using the "RTK Method," which is widely used in collection system master planning. Using the RTK Method, the RDII unit hydrograph is the summation of three separate triangular hydrographs (short term, medium term, and long term), which are each defined by three parameters: R, T, and K. R represents the fraction of rainfall over the sewer basin that enters the collection system; T represents the time to peak of the Therefore, there are a total of nine separate variables associated with a unit hydrograph. Figure 5.18 shows the shape of an example unit hydrograph.

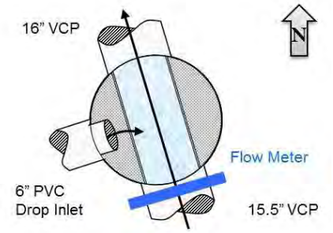
The hydrograph utilizes the R-values (percent of rainfall that enters the collection system) calculated for each basin to simulate I/I. The nine variables in each unit hydrograph were initially set based on engineering judgment and then adjusted until the model simulated flows (both peak flows and average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

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

Flow Monitoring Site 13, Dry Weather Flow Calibration

Location: Sidewalk of 910 Embarcadero
Pipeline Diameter: 15.5"
City Manhole ID: 12.11
Model Pipe ID: 1125

Flow Monitor Location



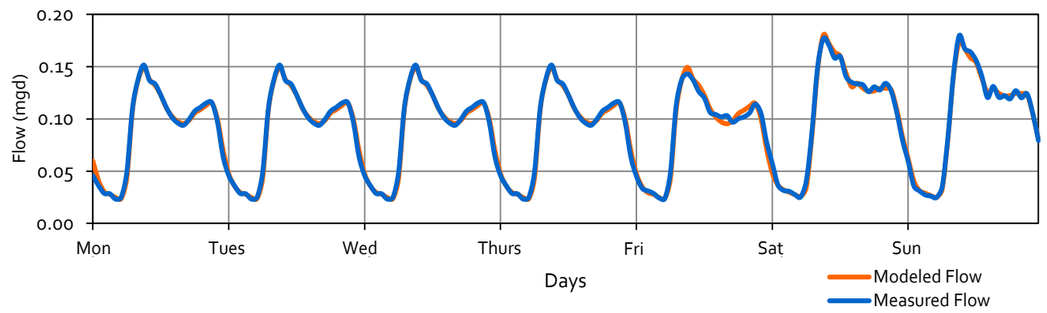
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Tues.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Wed.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Thur.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Fri.	0.09	0.14	4.5	0.47	0.09	0.15	4.7	0.47	0.1%	4.3%	3.0%	0.6%
Sat.	0.11	0.18	4.7	0.51	0.10	0.18	4.9	0.50	-0.6%	1.8%	3.8%	-1.9%
Sun.	0.10	0.18	4.7	0.49	0.10	0.17	4.9	0.49	-0.1%	-3.3%	4.6%	0.9%
Summary												
Weekday	0.09	--	4.6	0.47	0.09	--	4.7	0.47	0.0%	--	1.7%	1.6%
Weekend	0.10	--	4.7	0.50	0.10	--	4.9	0.50	-0.3%	--	4.2%	-0.5%
ADWF[4]	0.09	--	4.6	0.47	0.09	--	4.7	0.48	-0.1%	--	2.5%	0.9%

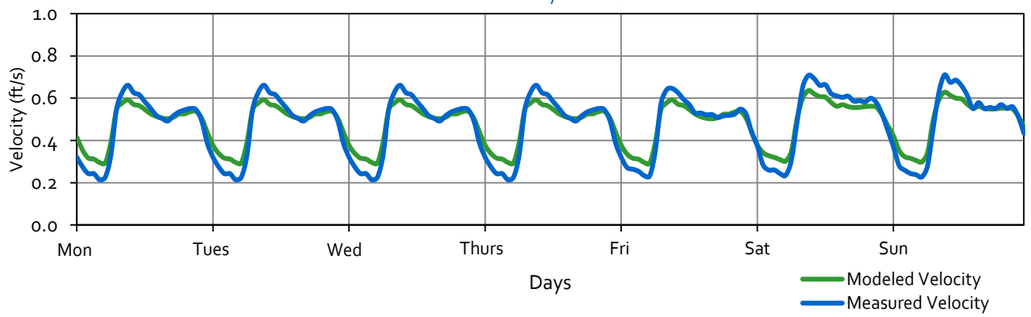
Notes:

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

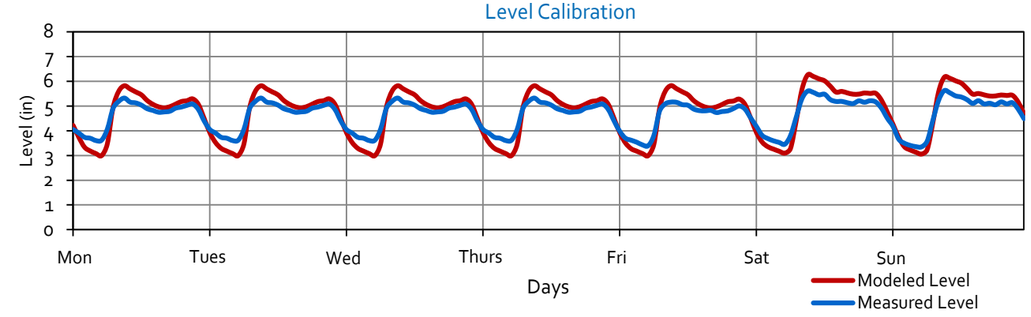
Flow Calibration



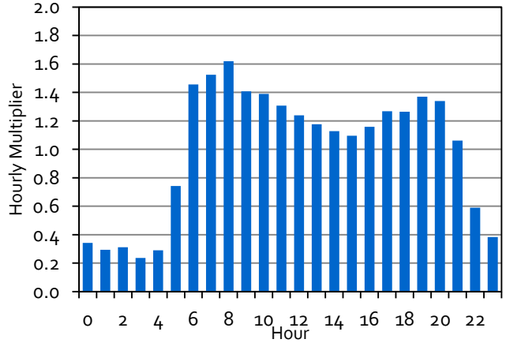
Velocity Calibration



Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern

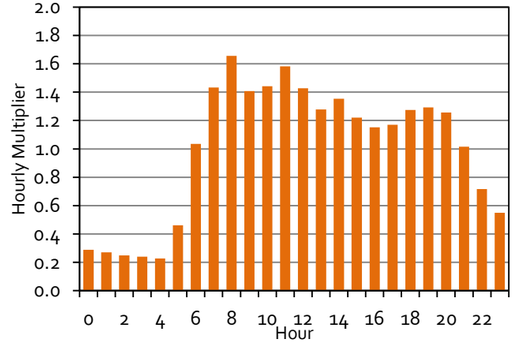


Figure 5.17 Example DWF Calibration Sheet (Site 13)

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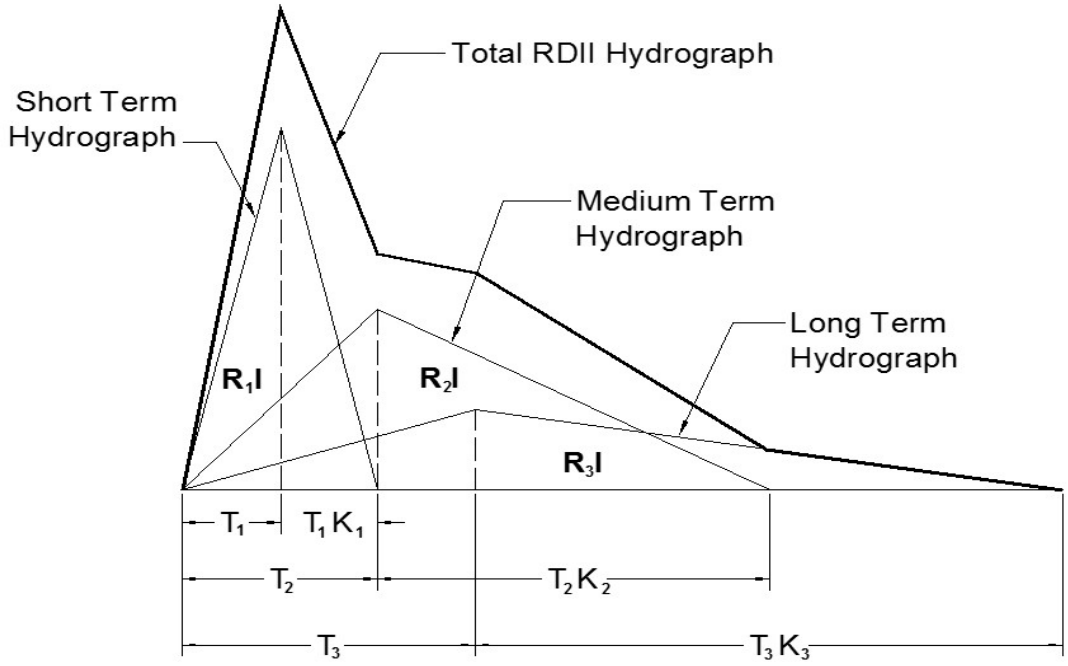

Figure 5.18 Example RDII Unit Hydrograph

Figure 5.19 is an example WWF calibration sheet for Meter Basin MB-3. The WWF calibration sheets show figures comparing the measured data and model results for flow, velocity, and level in response to rainfall. The WWF calibrations sheets are included in Appendix J. There is good correlation between the model simulated flows and the flows that were measured at each meter location with the exception of the following:

- Lift Station 2. The model simulated flows at Lift Station 2 were generally higher than the field measured flows. The flow meter at Lift Station 2 may not be accurate. It is recommended that the City calibrate this meter to determine if it is accurately measuring flow.

With a few exceptions, the hydraulic model met the established wet weather calibration standards. Overall, however the model accurately simulated the effects of wet weather events, as was considered calibrated and ready to use for capacity analysis purposes.

5.3.2.4 Collection System Hydraulic Model Calibration Summary

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

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

5.3.3 Storm Drainage System Model Verification

The reasonableness of the model results and the hydraulic grade line profiles were evaluated during the initial model runs. This was accomplished by comparing areas of flooding predicted by the model with observations offered by the City. Areas around the City that experience flooding were confirmed by the model results. Following the verification process, the model was used for the existing and future storm drainage system analysis.

MB-3 Wet Weather Calibration
Location: Island between Main St. and Hwy 1 on-ramp, ~135-feet north of Atascadero Rd.
Pipeline diameter: 15"
City Manhole ID: 5.25A
Model Pipe ID: 675
Silt Level at Site: 0"

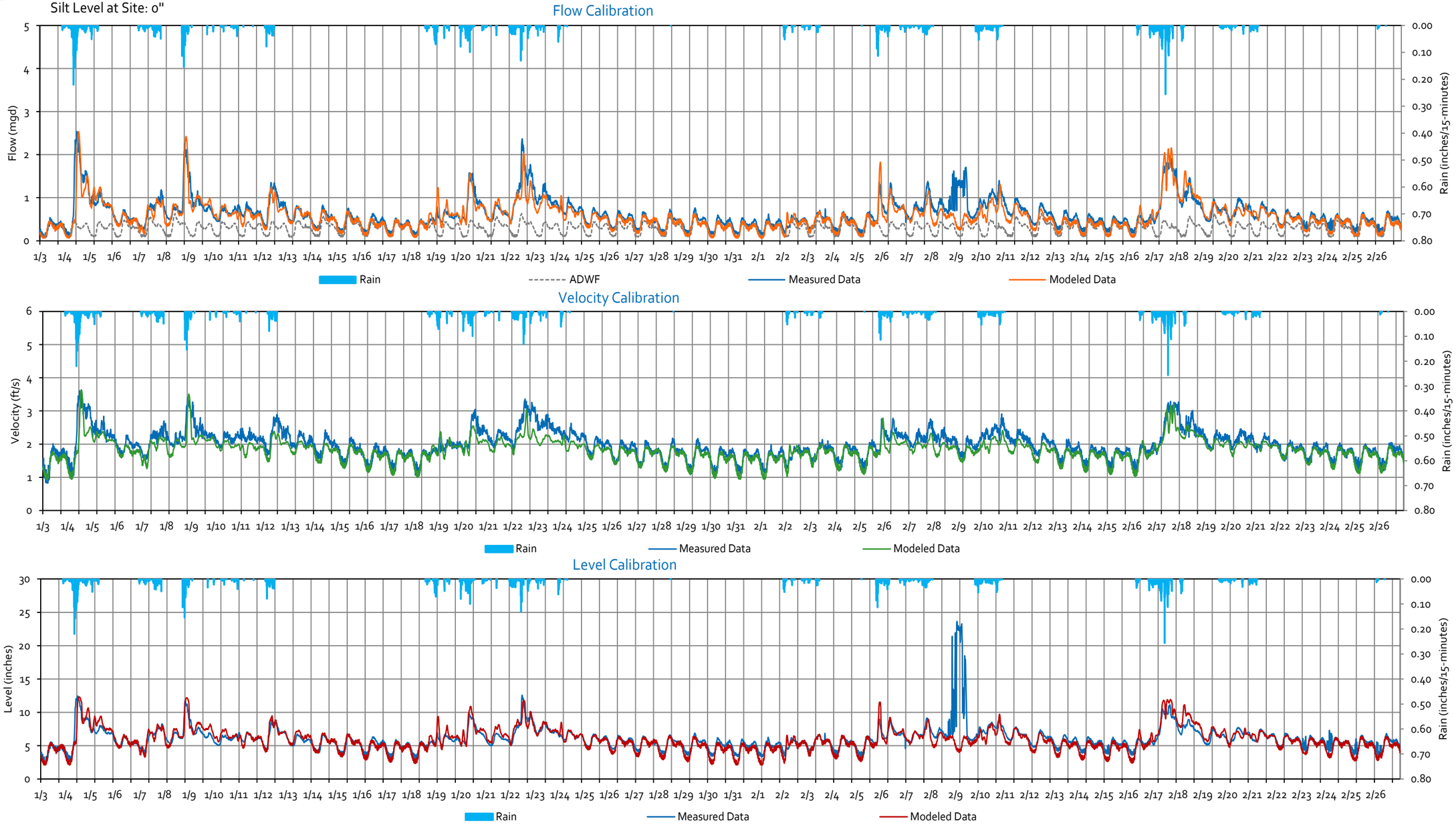

Figure 5.19 Example Wet Weather Flow Calibration Sheet (MB-3)

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Chapter 6

SYSTEM EVALUATION AND PROPOSED IMPROVEMENTS

This chapter discusses the hydraulic evaluation of the City of Morro Bay (City) water distribution system, wastewater collection system and storm drainage system, and the proposed projects that correct capacity deficiencies and serve future users.

6.1 Water Distribution System Capacity Evaluation

Following the model calibration, which is summarized in Chapter 5, a capacity analysis of the existing and future distribution system was performed. This section focuses on the capacity of the water distribution system only. A detailed water supply evaluation is provided in Chapter 7.

6.1.1 Booster Pumping Capacity Evaluation

The booster pump capacity evaluation under existing and future demand conditions was performed by comparing the available water supplies to the projected water demands. As noted in Chapter 3, this study recommends that the City maintain a firm pumping capacity equal to the maximum day demand (MDD). Demands in excess of the MDD will be met through storage.

There are three booster pump stations (BPS) located throughout the distribution system. As shown in Table 6.1, Elena BPS and Vashon BPS do not have sufficient capacity to meet existing or 2040 MDD. Kings BPS has sufficient capacity to supply Ridgeway Zone under 2040 MDD.

Table 6.1 Booster Pump Station Capacity Evaluation

Booster Station	Pumping Capacity (mgd)		Pressure Zone	Ex. MDD(1) (mgd)	2040 MDD (mgd)	Pumping Surplus/(Deficit) (mgd)	
	Total	Firm				Ex.	2040
Vashon BPS	0.36	0.00	Blanca	0.29	0.31	(0.29)	(0.31)
Elena BPS	0.81	0.40	Elena/Blanca	0.52	0.55	(0.12)	(0.14)
Kings BPS	0.72	0.36	Ridgeway	0.13	0.13	0.23	0.23

Notes:
(1) MDD/ADD Peaking Factor = 1.63

6.1.2 Storage Capacity Evaluation

The City currently has 10 storage tanks at five tank sites with a total of 3.25 million gallons (MG) of water storage. These reservoirs provide the City with operational equalization storage to meet peak hour demand (PHD), fire flow storage and emergency storage.

Treated water storage capacity criteria are defined in Chapter 3. The required operational storage is equal to 25 percent of MDD. The required fire flow storage is equal to the largest fire

flow demand multiplied by the duration. The required emergency storage is equal to 150 percent of the average day demand (ADD).

In systems with multiple pressure zones, the storage evaluation is performed by pressure zone, meaning that each zone should be capable of providing the total required storage for that zone. In the case of the City, water is conveyed between pressure zones by PRV stations, and therefore multiple zones were grouped for the purposes of evaluating existing and future storage capacity.

Table 6.2 summarizes the results of the existing system storage capacity evaluation. As shown in Table 6.2, there is currently a City-wide storage capacity surplus of approximately 0.40 MG (assuming a required emergency storage of 150 percent of the ADD). However, several analysis zones do not have sufficient storage capacity. For example, as shown in Table 6.2, Nutmeg zone has a 0.42 MG storage deficiency.

The City plans to replace the existing 0.14 MG Nutmeg Tank with a new 1.0 MG tank. When the new tank is installed, the City plans to abandon the Vashon Booster Pump Station and supply water from the Nutmeg Tank down to the Blanca Pressure Zone.

In addition, it was noted that the existing Elena tanks are in poor condition. For this reason, several options were considered to replace or abandon the existing 0.12 MG Elena Tanks. Ultimately, it was determined that replacing the existing Elena Tanks with a new larger tank was the preferred alternative.

Table 6.3 summarizes the existing storage capacity analysis, including the following:

- Replace the existing 0.14 MG Nutmeg Tank with a new 1.0 MG tank
- Replace the existing 0.12 MG Elena Tanks with a new 0.15 MG tank

Table 6.4 summarizes the 2040 storage tank analysis with the replacement tanks identified above. As shown in Table 6.3 and 6.4, the two replacement tanks will provide enough storage capacity to meet the projected 2040 water demands.

6.1.3 Distribution System Hydraulic Analysis

This section presents results of system pressure analysis, fire flow analysis, and pipeline velocity analysis for the City's water distribution system. Recommendations to address identified deficiencies are presented in Section 6.4.2.

6.1.3.1 System Pressure Evaluation

In accordance with the criteria summarized in Chapter 3, system pressure analyses were performed using the hydraulic model for ADD and peak hour demand (PHD) conditions. This section summarizes the results of the analysis for the existing and future demand conditions.

For the ADD and PHD demand conditions, the hydraulic model was used to identify service nodes within the distribution system with pressures that violate the established pressure criteria. For the ADD conditions, the City's maximum desirable pressure is 120 pounds per square inch (psi). Under PHD conditions, the minimum pressure criterion in 40 psi.

Table 6.2 Existing Storage Evaluation

Storage Analysis Zone	Pressure Zone(s)	Ex. Demand (mgd)		Required Storage (MG)				Existing Storage (MG)	Abandoned Storage[5] (MG)	Proposed Storage[6] (MG)	Storage Surplus/(Deficit) (MG)
		ADD	MDD[1]	Operational[2]	Emergency[3]	Fire Flow[4]	Total				
A	Upper Kings, Lower Kings	0.626	1.017	0.25	0.94	0.96	2.15	2.50	0.00	0.00	0.35
B	Upper Kings, Lower Kings, Elena	0.770	1.252	0.31	1.16	0.96	2.43	2.62	0.00	0.00	0.19
C	Nutmeg Zone	0.106	0.172	0.04	0.16	0.36	0.56	0.14	0.00	0.00	(0.42)
D	Blanca Zone	0.178	0.289	0.07	0.27	0.96	1.30	0.58	0.00	0.00	(0.72)
E	Ridgeway Zone	0.082	0.133	0.03	0.12	0.18	0.34	0.18	0.00	0.00	(0.16)
Citywide	All Pressure Zones	1.136	1.846	0.46	1.70	0.96	3.12	3.52	0.00	0.00	0.40

Notes:
1. MDD/ADD Peaking Factor = 1.63
2. Operational Storage = 25% x MDD
3. Emergency Storage = 150% of the ADD
4. Fire Flow = Maximum Fire Flow Rate Per Zone x Duration
5. Amount of storage volume assumed to be abandoned as part of this analysis scenario
6. Amount of storage volume assumed to be added as part of this analysis scenario.

Table 6.3 Existing Storage Evaluation with New Elena and Nutmeg Tanks

Storage Analysis Zone	Pressure Zone(s)	Ex. Demand (mgd)		Required Storage (MG)				Existing Storage (MG)	Abandoned Storage[5] (MG)	Proposed Storage[6] (MG)	Storage Surplus/(Deficit) (MG)
		ADD	MDD	Operational	Emergency	Fire Flow	Total				
A	Upper Kings, Lower Kings	0.626	1.017	0.25	0.94	0.96	2.15	2.50	0.00	0.00	0.35
B	Upper Kings, Lower Kings, Elena	0.770	1.252	0.31	1.16	0.96	2.43	2.62	0.12	0.15	0.22
A/E	Upper Kings, Lower Kings, Ridgeway[7]	0.708	1.150	0.29	1.06	0.96	2.31	2.68	0.00	0.00	0.37
B/E	Upper Kings, Lower Kings, Elena, Ridgeway	0.852	1.385	0.35	1.28	0.96	2.58	2.80	0.12	0.15	0.25
C	Nutmeg	0.106	0.172	0.04	0.16	0.36	0.56	0.14	0.14	1.00	0.44
C/D	Nutmeg, Blanca	0.284	0.461	0.12	0.43	0.96	1.50	0.86	0.14	1.00	0.22
Citywide	All Pressure Zones	1.136	1.846	0.46	1.70	0.96	3.12	3.52	0.26	1.15	1.29

Notes:
1. MDD/ADD Peaking Factor = 1.63
2. Operational Storage = 25% x MDD
3. Emergency Storage = 150% of the ADD
4. Fire Flow = Maximum Fire Flow Rate Per Zone x Duration
5. Amount of storage volume assumed to be abandoned as part of this analysis scenario
6. Amount of storage volume assumed to be added as part of this analysis scenario.
7. Ridgeway Zone borrowing storage from Upper Kings Zone.

Table 6.4 2040Storage Evaluation with New Elena and Nutmeg Tanks

Storage Analysis Zone	Pressure Zone(s)	Fut. Demand (mgd)		Required Storage (MG)				Available Storage (MG)	Abandoned Storage[5] (MG)	Proposed Storage[6] (MG)	Storage Surplus/(Deficit) (MG)
		ADD	MDD	Operational	Emergency	Fire Flow	Total				
A	Upper Kings, Lower Kings	0.739	1.201	0.30	1.11	0.96	2.37	2.50	0.00	0.00	0.13
B	Upper Kings, Lower Kings, Elena	0.886	1.440	0.36	1.33	0.96	2.65	2.62	0.12	0.15	0.00
A/E	Upper Kings, Lower Kings, Ridgeway[7]	0.821	1.333	0.33	1.23	0.96	2.52	2.68	0.00	0.00	0.16
B/E	Upper Kings, Lower Kings, Elena, Ridegway	0.968	1.573	0.39	1.45	0.96	2.80	2.80	0.12	0.15	0.03
C	Nutmeg	0.111	0.180	0.05	0.17	0.36	0.57	0.14	0.14	1.00	0.43
C/D	Nutmeg, Blanca	0.300	0.488	0.12	0.45	0.96	1.53	0.72	0.14	1.00	0.05
Citywide	All Pressure Zones	1.268	2.061	0.52	1.90	0.96	3.38	3.52	0.26	1.15	1.03

Notes:
1. MDD/ADD Peaking Factor = 1.63
2. Operational Storage = 25% x MDD
3. Emergency Storage = 150% of the ADD
4. Fire Flow = Maximum Fire Flow Rate Per Zone x Duration
5. Amount of storage volume assumed to be abandoned as part of this analysis scenario
6. Amount of storage volume assumed to be added as part of this analysis scenario.
7. Ridgeway Zone borrowing storage from Upper Kings Zone.

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ADD Maximum Pressure Analysis

Figure 6.1 shows the maximum modeled system pressures under existing ADD conditions. As shown on Figure 6.1, a large portion of the maximum pressures in the system are less than 100 psi. Nodes with model simulated pressures in excess of 120 psi are shown in red on Figure 6.1. These nodes account for approximately 15 percent of the nodes in the system.

No improvement projects were identified to address high system pressures. However, it should be noted that per current code requirements, it is recommended that any customers with a system pressure in excess of 80 psi be equipped with an individual service connection pressure reducing valve (PRV), if not already equipped.

PHD Minimum Pressure Analysis

Figure 6.2 shows the results of the PHD pressure analysis under existing demand conditions. The analysis showed that only 3.3 percent of low pressures ranged from 30 psi to 40 psi and 9.8 percent 30 psi under PHD conditions.

The areas that were identified as having pressures below 40 psi were due to high elevations in isolated portions of existing pressure zones. In many cases, there is not much that can be done to address these low pressure conditions, except for modifying the hydraulic grade line (HGL) of the pressure zone or installing small booster pump stations to serve the affected portions. The City had not reported any pressure complaints from the areas shown on Figure 6.2, therefore no improvement projects were recommended to meet the minimum pressure criteria of 40 psi.

6.1.3.2 Fire Flow Analysis

The hydraulic model performs a steady state simulation at each node and reports the residual pressure at the node to evaluate impacts of the fire flow demands on the system. Nodes with less than 20 psi of residual pressure were considered deficient.

Fire flow demands were simulated at all model nodes within the existing system that are associated with a fire hydrant. This process excluded model nodes without an associated fire hydrant (such as dead end pipes without fire hydrants and nodes near tank sites).

The fire flow simulations were performed using the hydraulic models built in automated fire flow simulator. This eliminates the need to assign individual fire flow demands to all nodes, which would be extremely time consuming. Fire flow demands were assigned in the model by land use according to the fire flow demand criteria summarized in Chapter 3. Figure 6.3 shows the fire flow demand types that were assigned to each modeled system hydrant. The fire flow demands ranged from 1,500 gpm to 4,000 gpm.

The existing system fire flow analysis showed that 139 hydrants (approximately 18 percent of the system) had a residual pressure less than 20 psi under MDD plus fire flow conditions. For deficient hydrants with multiple hydrants in close proximity, it is possible to split the fire flow demand with another hydrant. The fire flow analysis was repeated for deficient hydrants by using multiple hydrants. Distributing the fire flow over two adjacent hydrants typically results in a lower pressure drop compared with the use of one hydrant. Using this approach, the number of deficient was reduced to 127 hydrants. Figure 6.4 depicts hydrants that meet the evaluation criteria, deficient hydrants that were mitigated through splitting fire flows with another hydrant, and deficient hydrants. Individual improvements to address these deficiencies are presented later in Section 6.4.2.

6.1.3.3 Pipeline Velocity Analysis

Pipeline velocity analyses were performed based on the criteria provided in Chapter 3 using the hydraulic model for PHD conditions. As documented in Chapter 3, the pipeline velocity analysis was performed to identify pipelines within the distribution system with velocities that exceed 8 feet per second (fps) under PHD conditions. Under existing PHD conditions, six pipelines were identified that exceeded the pipeline velocity criteria. By year 2040, no additional pipelines were identified.

The pipelines that exceeded the established maximum velocity criteria are shown on Figure 6.5. The proposed improvement projects to mitigate these deficiencies are discussed later in this chapter.

6.2 Wastewater Collection System Capacity Evaluation

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

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

6.2.1 Gravity Collection System Evaluation

For the existing sewer collection system, the PWWF was routed through the hydraulic model. In accordance with the established flow depth criteria for existing sewers, manholes where the maximum hydraulic grade line (HGL) is within 5-feet of the manhole rim.

Note that the pipelines that surcharge within 5-feet of the manhole rim are not necessarily deficient. In some cases, a surcharged condition within a given pipeline is due to backwater effects created by a downstream bottleneck (i.e. upstream surcharging is caused by downstream pipeline deficiencies). An illustration of backwater effects is shown on Figure 6.6. For this reason, the hydraulic model was used to identify the pipeline segments that are capacity deficient (i.e., not subject to backwater conditions).

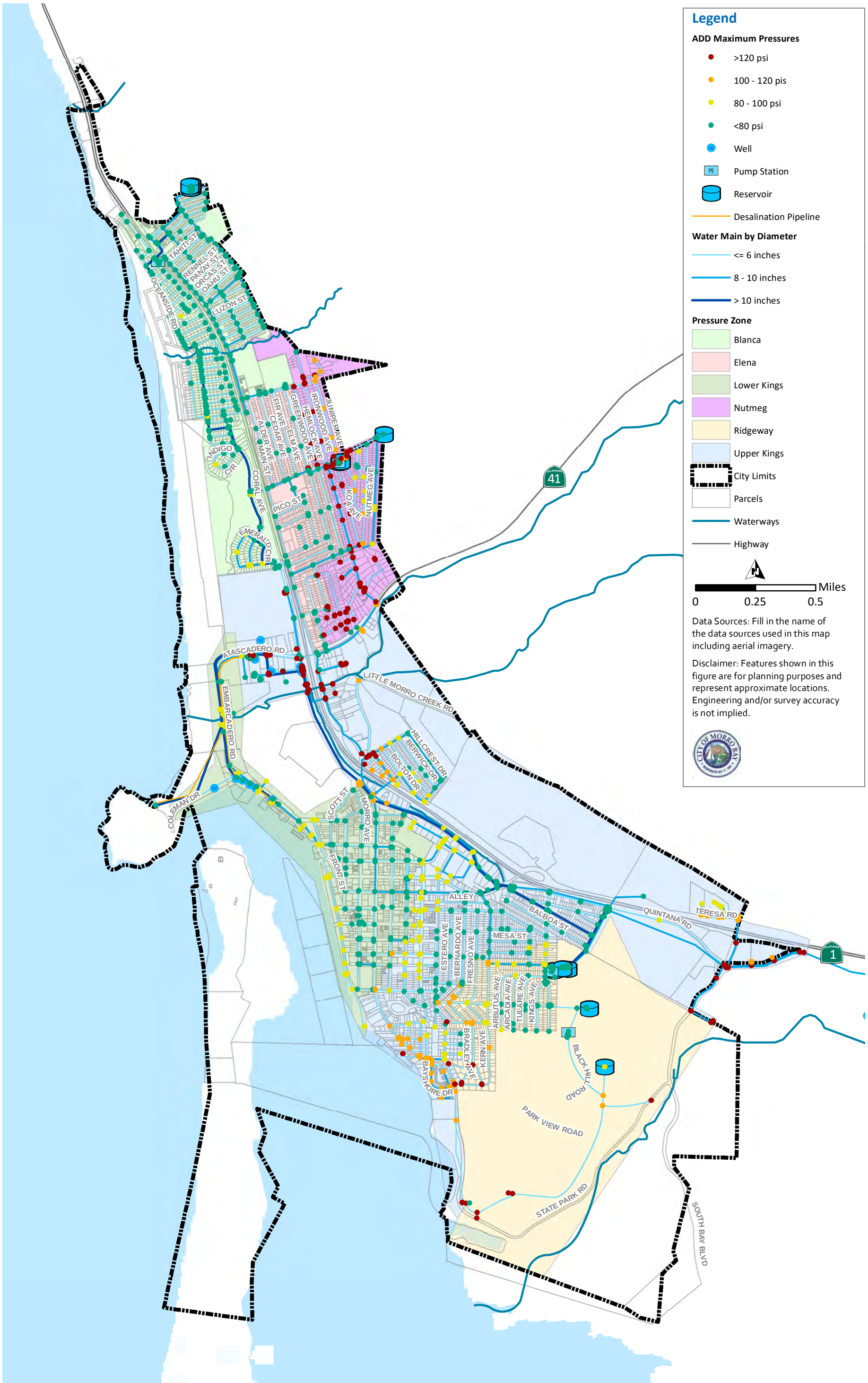

Figure 6.1 Existing Average Day Demand Maximum Pressures

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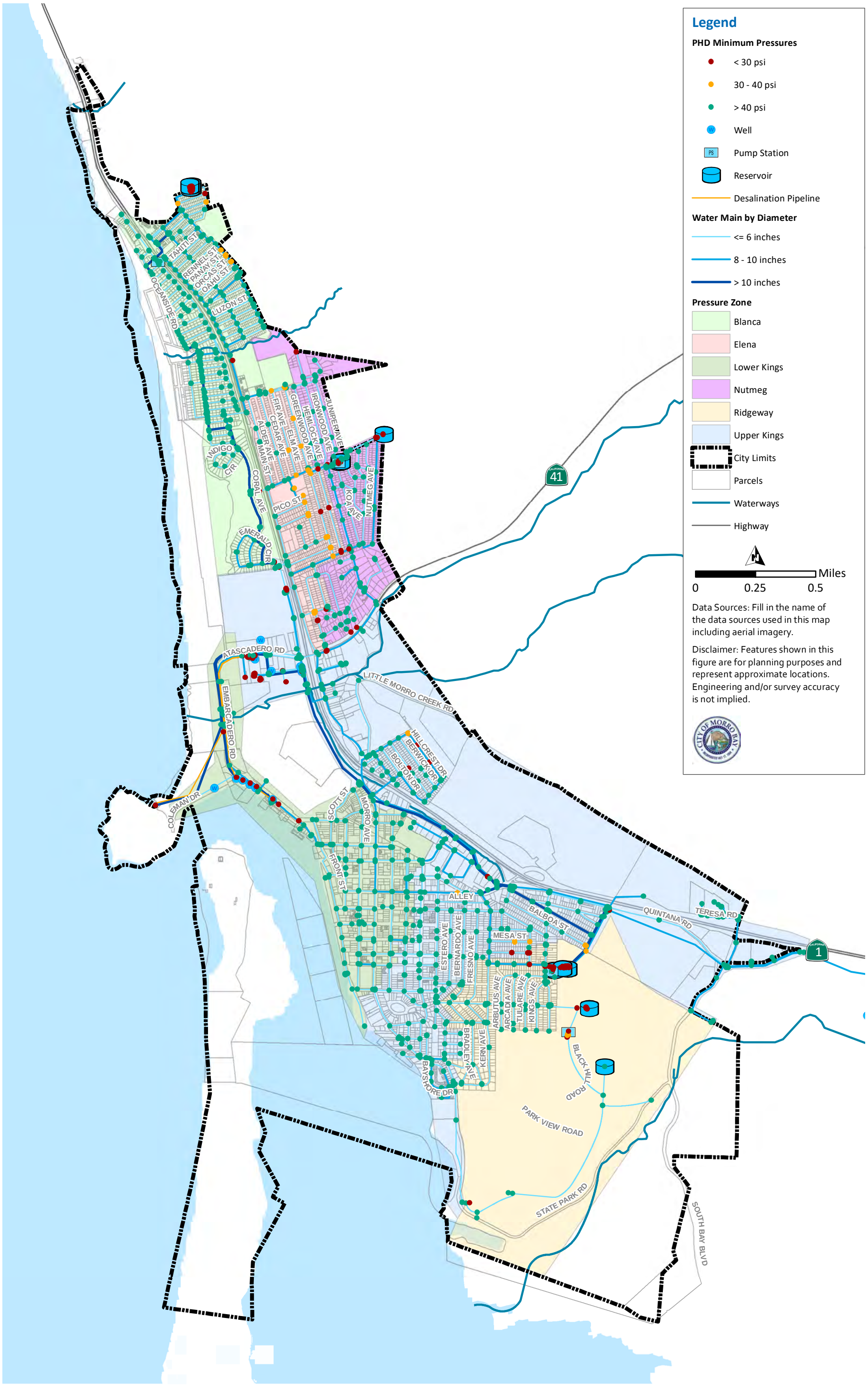

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Figure 6.2 Existing Peak Hour Demand Minimum Pressures

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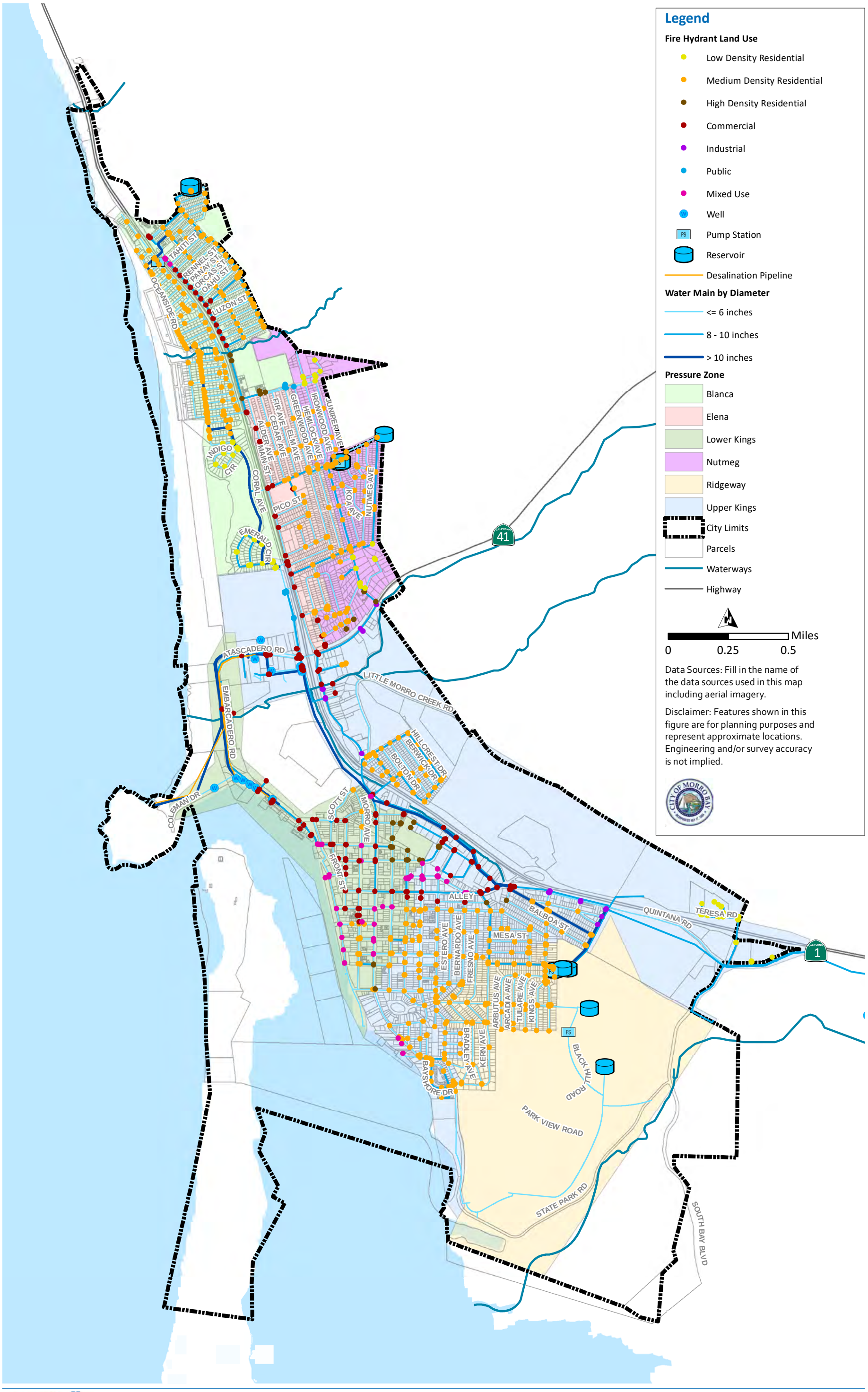

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Figure 6.3 Existing Fire Hydrant Land Use

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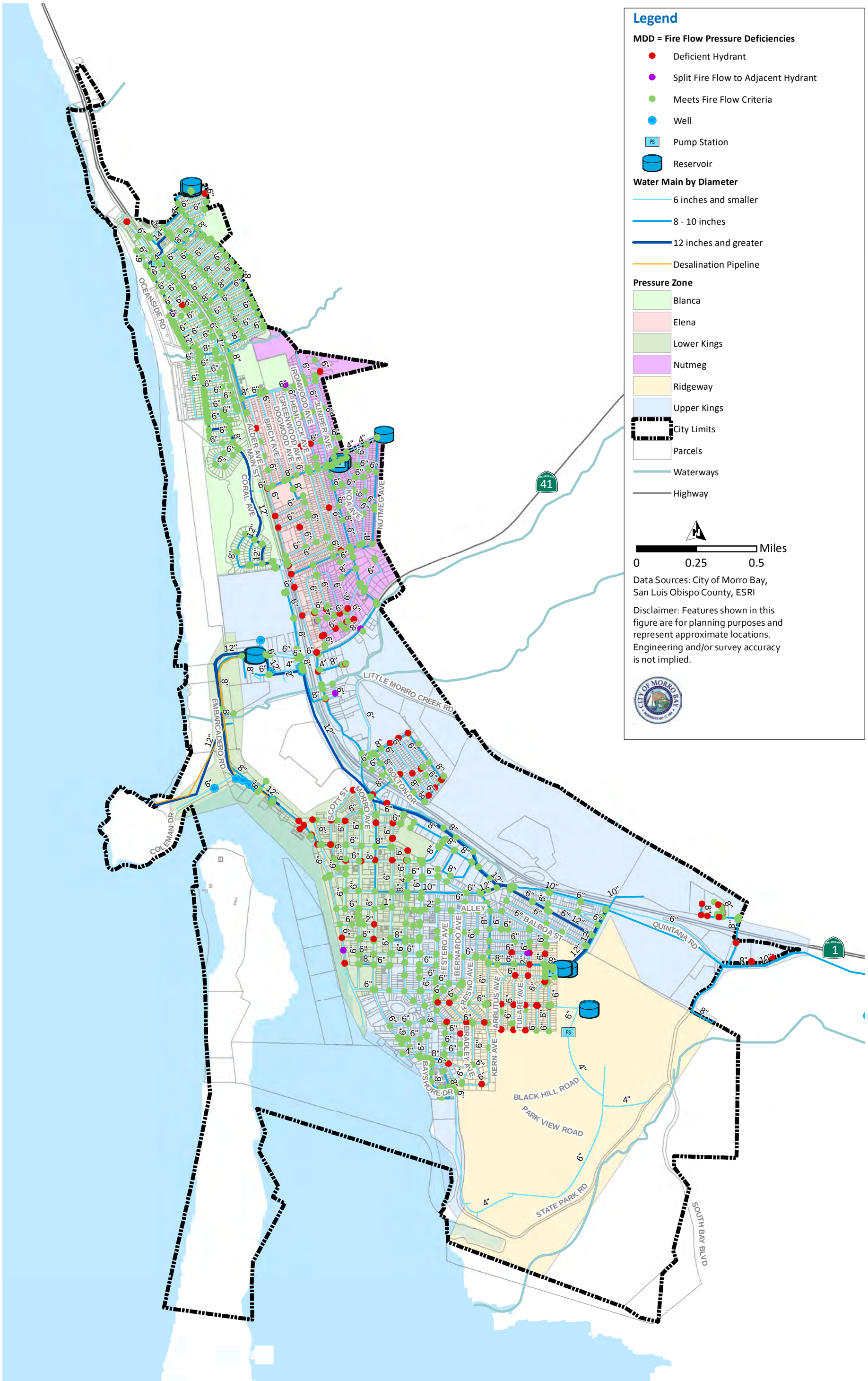

Figure 6.4 Existing Maximum Day Demand Plus Fire Flow Analysis Results

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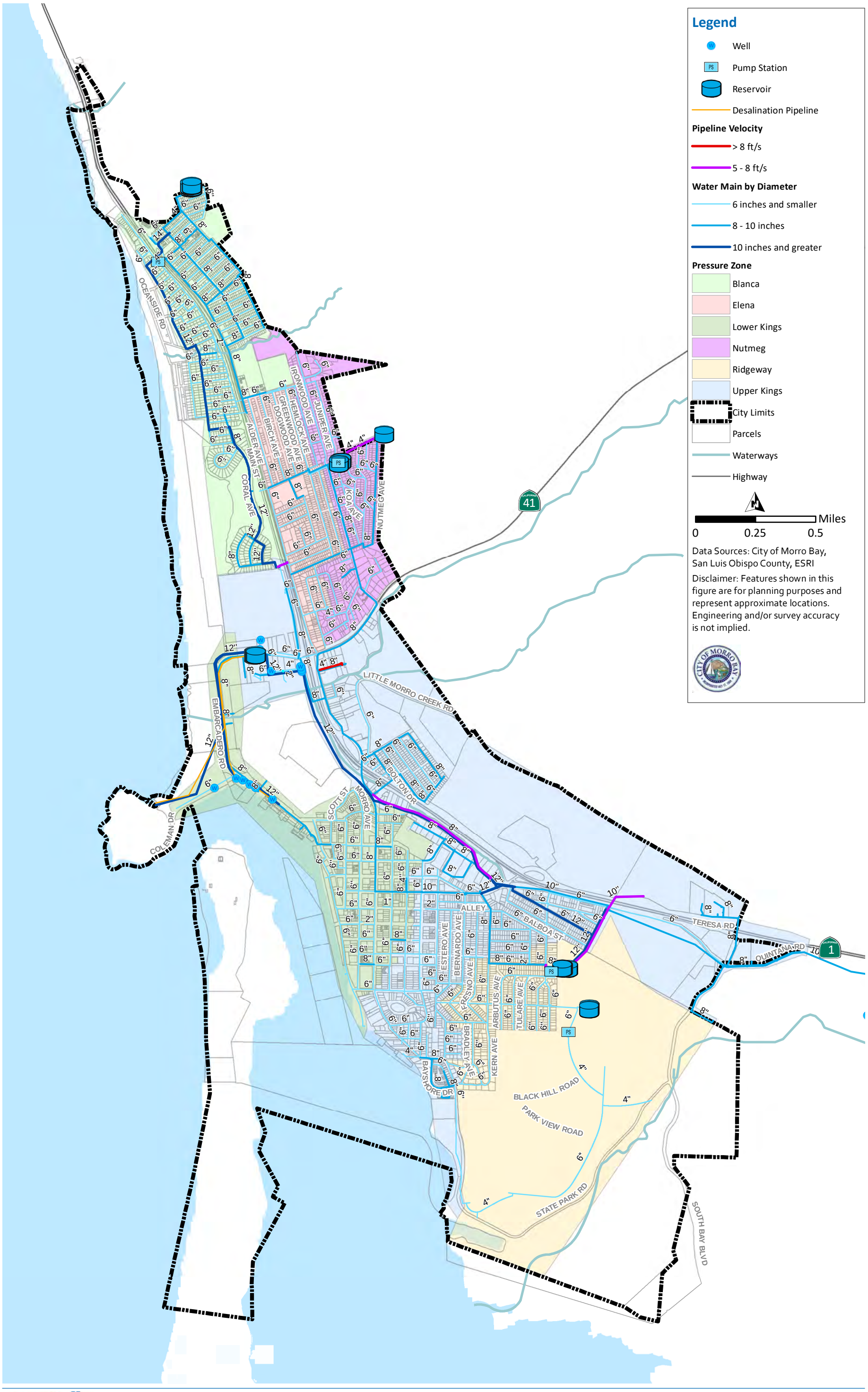

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Figure 6.5 Peak Hour Demand Maximum Velocities

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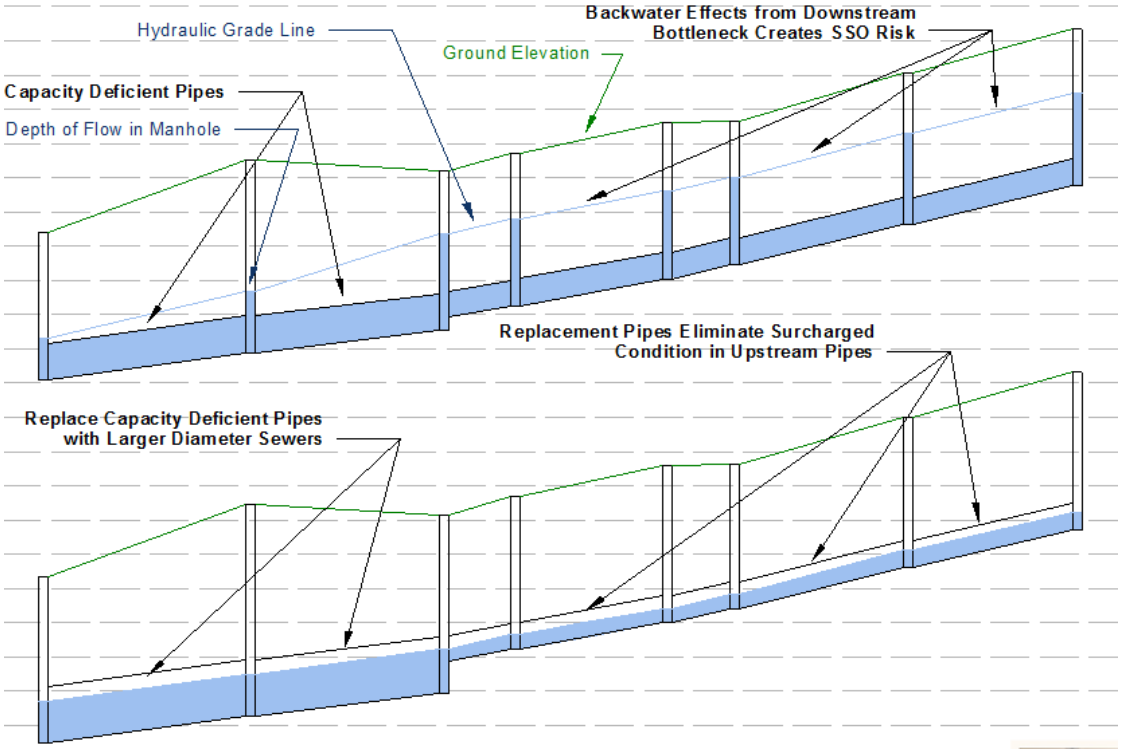

Figure 6.6 Sample Illustration of Backwater Effects in a Sewer

In general, the City's collection system has sufficient capacity to convey current PWWFs without exceeding the established flow depth criterion. However, there were areas of the collection system that surcharged within 5-feet of the manhole rim. These are shown on Figure 6.7 in red. Replacing a capacity limited (bottleneck) sewer will allow for higher peak flows to be carried to downstream sewers. In some cases, this increase in peak flow overwhelms the downstream sewers, which creates additional deficiencies. These additional "secondary" deficiencies are shown on Figure 6.7 in purple.

Following the completion of the existing system analysis, improvement projects and alternatives were identified in order to mitigate existing system pipeline capacity deficiencies. The recommended improvement projects are discussed in greater detail in Section 6.4.3. In accordance with the established planning criteria, new sewer pipelines were sized such that the maximum d/D does not exceed the new pipe design criteria summarized in Chapter 3 under PWWF and future 2040 conditions.

The analysis of the future system was performed in a manner similar to the existing system analysis. The purpose of the future system evaluation is to verify that the existing system improvements were appropriately sized to convey future PWWFs, and to identify the locations of sewers that are adequately sized to convey existing PWWFs, but cannot convey future 2040 PWWFs.

Future 2040, the City's wastewater flows are expected to increase by 0.22 mgd, under PWWF conditions. As such, there were no additional areas in the existing collection system that were identified as being capacity deficient under 2040 PWWF conditions.

6.2.2 Lift Station and Force Main

The City's hydraulic model includes three lift stations that are associated with the City's hydraulic model. In accordance with the established planning criteria, the City's existing lift stations were evaluated to determine if they have sufficient capacity to convey the maximum projected inflow from the 10-year 24 hour design storm under existing and buildout conditions. Lift stations with an influent peak wet weather flow (PWWF) above firm pumping capacity were flagged as deficient.

Table 6.5 summarizes the modeled lift stations capacity evaluation. It was determined that the City's Lift Station 1 and Lift Station 2 have an existing capacity deficiency. The severity of Lift Station 1's deficiency is significant enough to warrant a capacity upgrade. The high amount of inflow/infiltration during rain events was confirmed with lift station flow monitoring data used during the wet weather hydraulic model calibration.

It was determined that Lift Station 2 had an existing PWWF 0.02 mgd greater than the existing firm capacity and future 2040 PWWF 0.09 mgd greater than existing firm capacity existing conditions and 0.09 mgd under future. Because these deficiencies are only slightly greater than the lift station capacity, no capacity improvements are recommended for existing or future 2040 flow conditions. However, it is recommended that flow monitoring data be used to monitor and confirm lift station capacity and performance. Not all lift station deficiencies will result from an improvement project if considered minor.

Table 6.5 Lift Station Capacity Analysis

Lift Station	No. of Pumps	Capacity per Pump[(1)] (mgd)	Existing Firm Capacity[(2)] (mgd)	Existing PWWF [(3)] (mgd)	Existing Balance (mgd)	2040 PWWF[(4)] (mgd)	2040 Balance (mgd)
Lift Station 1	2	0.49	0.49	0.93	-0.44	1.07	-0.58
Lift Station 2	4	0.60	1.20	1.22	-0.02	1.29	-0.09
Lift Station 3	2	0.36	0.36	0.13	0.23	0.16	0.20

Notes:
(1) Source:
(2) Firm Capacity is defined as the lift station capacity with the largest pump not operational.
(3) Existing PWWF is based on the hydraulic model's maximum flow into the wet well during the 10-year 24-hour design storm under existing conditions.
(4) 2040 PWWF is based on the hydraulic model's maximum flow into the wet well during the 10-year 24-hour design storm under 2040 conditions.

In accordance with the established planning criteria, the City's existing force mains were evaluated to determine if the velocity in the force main fell within the range of 3 feet per second (fps) and 8 fps. After Lift Station 1 capacity improvement, it was determined that the 6-inch diameter force main exceeded the maximum allowable velocity. No other force main deficiencies were simulated in the hydraulic model.

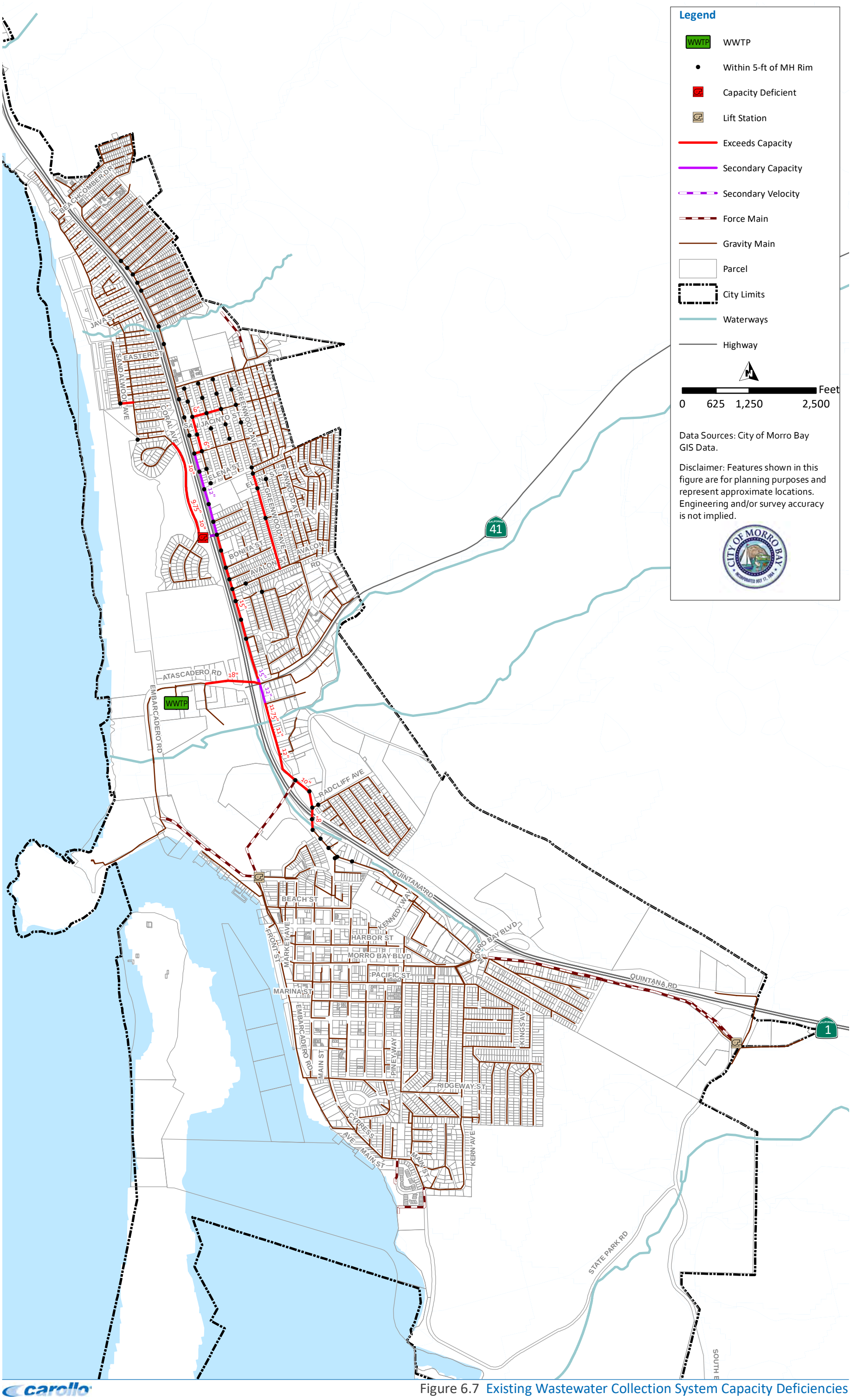

Figure 6.7 Existing Wastewater Collection System Capacity Deficiencies

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6.3 Storm Drainage Capacity Evaluation

When evaluating the adequacy of the City's storm drainage facilities, the City's storm drains were allowed to surcharge and provide additional storage capacity, thus reducing the number of storm drain improvements. For the 10 year design event, the maximum HGL was not allowed to exceed the ground level (i.e., no flooding permitted). The storm drainage criteria allowed flooding up to one foot above the gutter flow line in the 100-year design storm event.

The capacity analysis entailed identifying areas in the storm drainage system where flooding exceeded the maximum allowable flow depth (i.e., ground level for the 10-year event, 1 foot above the gutter for the 100-year event). Storm drains that lack sufficient capacity to convey runoff generated from the design storm could produce backwater effects in the drainage system and potentially cause flooding. This section discusses the possible locations of existing and future flooding caused by these deficiencies. When an increase to capacity is required, it was assumed that storm drains would be replaced with a larger diameter pipeline.

Figure 6.8 shows the locations of areas that exceeded the maximum allowable flood depth for the 10-year and 100-year storm events. As shown on Figure 6.8, the storm drainage system generally has enough capacity to convey the design storm runoff in some locations, the existing storm drains lack sufficient capacity to convey the 10-year or 100-year design runoff while meeting the allowable flooding criterion. Notable areas of flooding shown in the capacity evaluation include the following:

- Orcas Creek. The City recently completed a stream improvement on Orcas Creek near Beachcomber. The creek discharges into a 16 inch by 42 inch box drain that flows underneath the Morro Strand Campground. The hydraulic model showed that the box drain does not have enough capacity to convey the 10-year design storm runoff. As the box drain floods, runoff would flow overland across the campground out to the beach and then to the ocean.
- Sequoia Street and Sienna Street. The hydraulic model showed some flooding during the 10-year design storm near Sienna Street/Sandalwood Avenue, and Sandalwood Avenue/Bali Street. Under the 100-year event, this area as well as the Main Street/Sequoia Street area were identified as being flooded.
- Elena Street Upstream of Highway 1. The hydraulic model predicted flooding upstream of Highway 1 on Elena Street during the 10-year and 100-year events.
- Morro Creek/Willow Camp Creek Area. The hydraulic model predicted flooding in scattered portions of this area during the 10-year and 100-year design storms. There are two main areas of concern within this area, as described below:
  - Roundabout Area: The roundabout area has been an area of persistent flooding in the City. Some regrading work and interim fixes have been implemented by City staff to improve drainage in the area, but flooding still occurs during large rainfall events.
  - Bike Path Near Main Street: Willow Camp Creek crosses the City's bike path west of Main Street through a very small 1 ft x 3 ft box drain. This box drain floods routinely during wet weather, causing the bike path to be closed.
- Beach Street/Harbor Street/Morro Bay Boulevard. There is a 6-inch bleeder line on Beach Street near Morro Avenue that takes runoff from a 36-inch storm drain and discharges to a downstream gutter. To improve system hydraulics in the area, a new

gravity storm drain could be installed to replace the bleeder line. The hydraulic model showed flooding during the 10-year and 100-year storms on Morro Bay Boulevard between Main Street and the Embarcadero.

- Marina Street and Shasta Avenue. Similar to Beach Street, there is a 6-inch bleeder line on Marina Street at Shasta, as well as some additional culverts on Marina Street that convey flow to downstream gutters. A new gravity storm drain could be installed to replace the bleeder line and improve system hydraulics in the area.

6.4 Recommended Improvements

This section summarizes the improvements recommended for the water distribution system, wastewater collection system, and storm drainage system. Detailed improvement sheets for water distribution, wastewater collection, and storm drainage improvements can be found in Appendix K, Appendix L and Appendix M, respectively.

6.4.1 Existing Versus Future Improvements

An existing deficiency is one where the existing facility's capacity is insufficient to meet the planning criteria (e.g. pipeline upgrades required to meet fire flow criteria) for existing users. If a project was proposed to exclusively correct an existing deficiency, then existing users would be assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs.

Future growth will trigger the construction of new facilities to support this growth (e.g., new distribution system pipelines to serve vacant areas within the City service area). If a specific project is needed to serve future growth exclusively, the future users will be assigned 100 percent of the future project's benefit and 100 percent of the costs.

In some cases, such as a proposed storage tank, projects are needed to mitigate existing deficiencies and to accommodate future growth. Where a project is needed to mitigate existing deficiencies and serve future growth, the future user benefit was determined based on the additional capacity necessary to serve future growth. All of the improvements for the water distribution, collection system, and storm drainage system were triggered by an existing capacity deficiency and are primarily attributable to existing users.

6.4.2 Water Distribution System Improvements

Figure 6.9 provides a graphical illustration of the improvements recommended to address the capacity deficiencies identified in Section capacity deficiencies. This section provides a detailed description of each recommended water distribution system improvement project.

6.4.2.1 Booster Pump Improvements

The following supply improvements are recommended for the water distribution system:

- *Elena Booster Pump Station (Project PWPS-1):* To mitigate the existing capacity deficiency of Elena BPS under PHD conditions, it is recommended that the existing pumps be replaced. The existing pumps would be replaced with two 0.5 mgd booster pumps.
- *Kings Booster Pump Station (Project PWPS-2)*: To provide additional flow to Lower Kings zone under MDD plus fire flow conditions, it is recommended that a 2.16 mgd fire flow booster pump be added to Kings BPS.

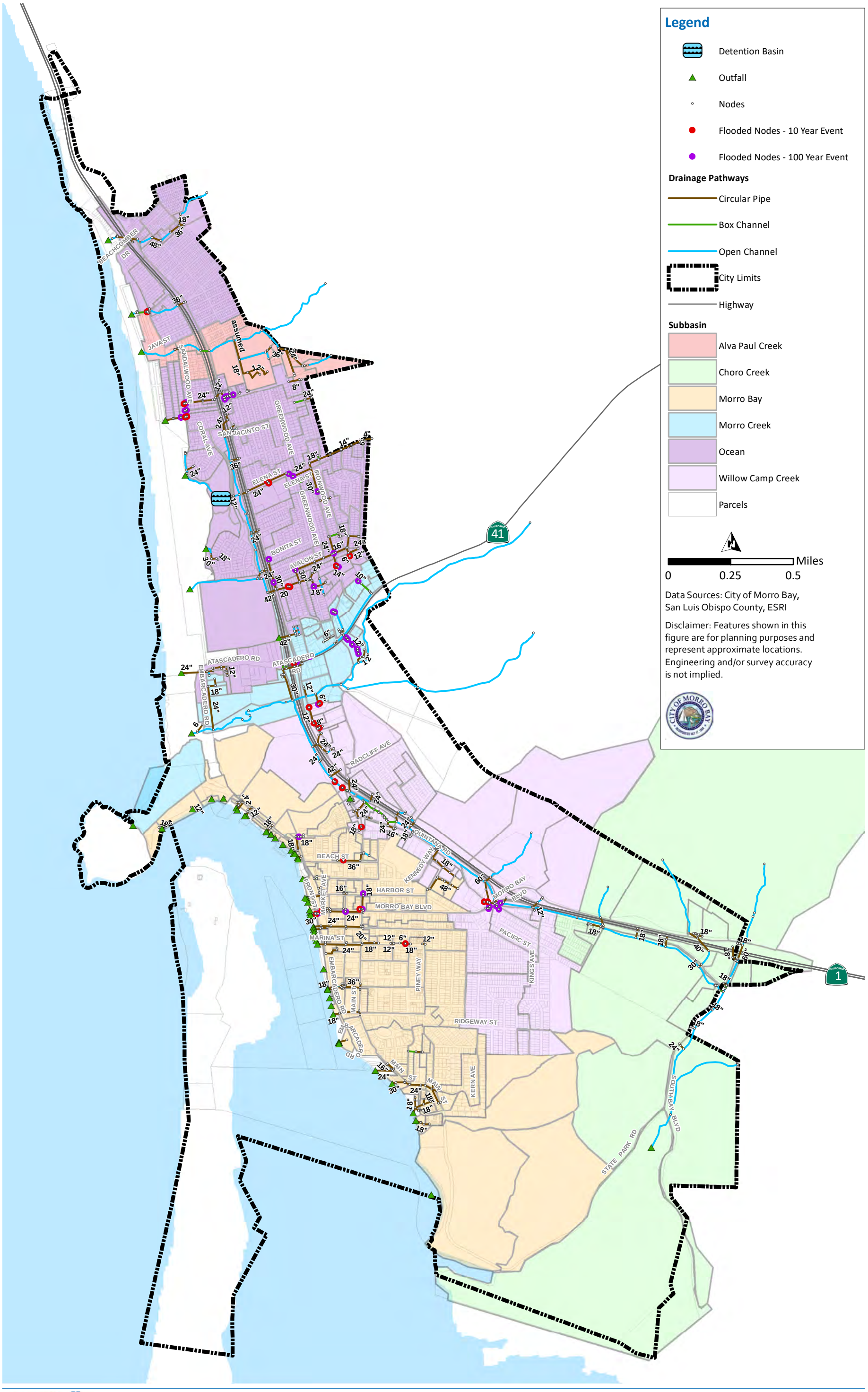

Figure 6.8 Storm Drain System Capacity Deficiencies

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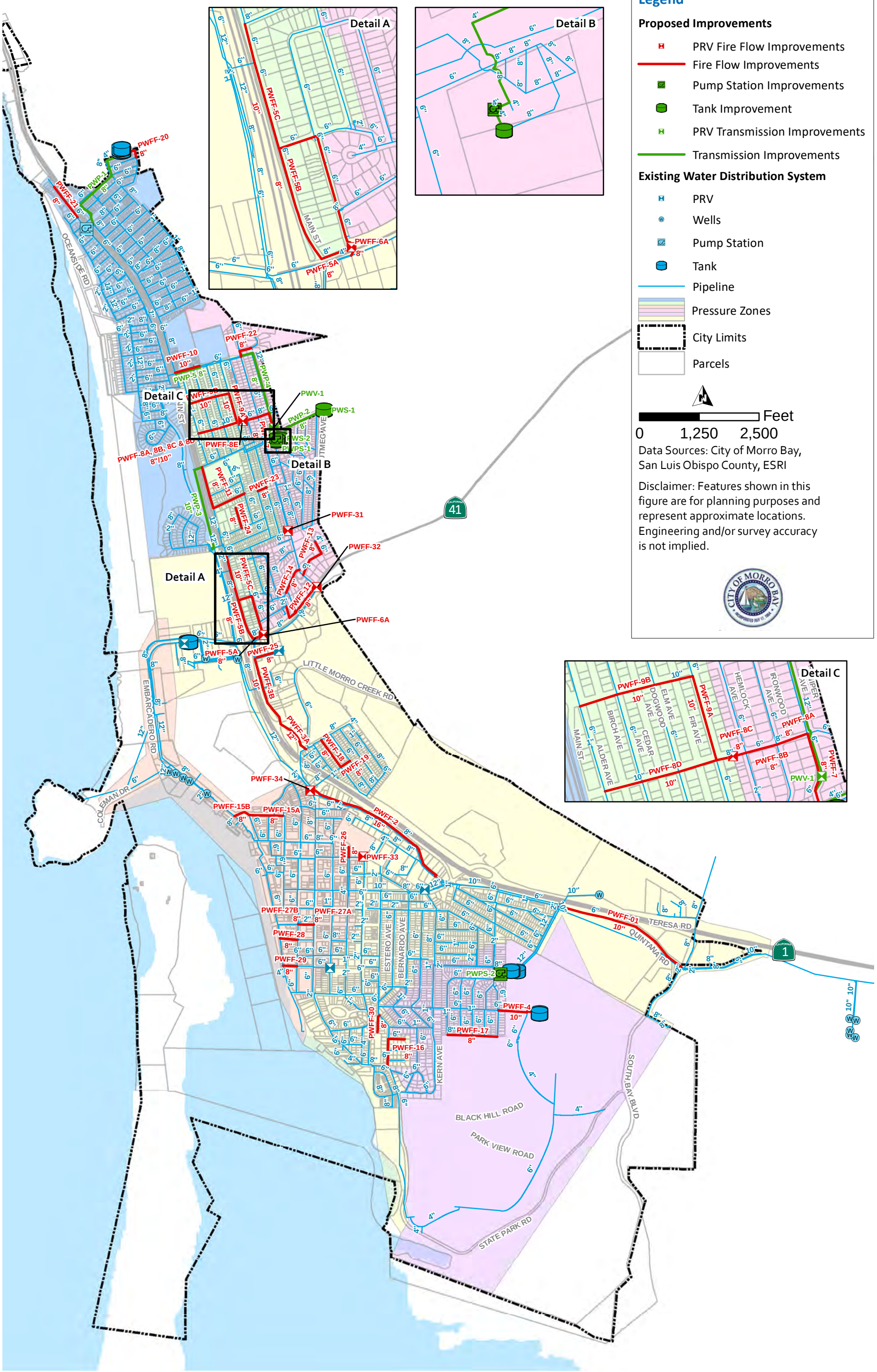

Legend

Proposed Improvements

- PRV Fire Flow Improvements
- Fire Flow Improvements
- Pump Station Improvements
- Tank Improvement
- PRV Transmission Improvements
- Transmission Improvements

Existing Water Distribution System

- PRV
- Wells
- Pump Station
- Tank
- Pipeline
- Pressure Zones
- City Limits
- Parcels

Feet

0 1,250 2,500

Data Sources: City of Morro Bay, San Luis Obispo County, ESRI

Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure 6.9 Water Distribution System Improvements

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6.4.2.2 Storage Improvements

As discussed in Section 6.1, the City will need to construct additional storage to meet the existing and 2040 demand requirements. A total of two replacement tanks are recommended.

Similar to the City's current storage tanks, each tank will be located at ground level. The locations of the proposed tanks were identified based on input from City staff. Below is a summary of the location and purpose of each of the proposed tanks.

- *Nutmeg Tank (Project PWS-1):* In order to provide additional storage for Nutmeg zone, it is recommended the City demolish the existing Nutmeg Tank and construct a new 1.0 MG tank.
- *Elena Tank (PWS-2):* In order to provide more storage to the Elena zone, it is recommended the City demolish the two existing Elena Tanks and construct a new 0.15 MG tank.

6.4.2.3 Fire Flow Improvements

As discussed previously, a number of deficient fire flow nodes (residual pressures below 20 psi) were identified as part of the fire flow capacity analysis. To mitigate these deficiencies, recommendations for pipeline and PRV improvements were developed. These improvements generally consist of replacement of smaller diameter (≤6 inches) pipelines with larger (8- to 16-inch diameter pipelines. In total, 29,700 linear feet (5.6 miles) of fire flow improvements are recommended. Table 6.6 provides additional detail related to each fire flow improvement.

6.4.2.4 Transmission/Velocity Improvements

As discussed in Section 6.1, six pipelines were identified that exceeded the pipeline velocity criteria under existing PHD conditions. Several other transmission system improvements are recommended to either improve system operations or to increase system reliability. These projects are described below:

- *Fill Line for Blanca Tanks (Project PWP-1):* In order to properly fill the Blanca tanks under MDD conditions, it is recommended that approximately 2,350 feet of 4-inch diameter pipeline from the Blanca Tanks site to Vashon BPS be replaced with an 8-inch diameter pipeline.
- *Fill Line for Nutmeg Tank (Project PWP-2):* To mitigate an existing headloss deficiency, it is recommended that 1,280 feet of 4-inch diameter pipeline from Elena BPS to the Nutmeg tank site be replaced with an 8-inch diameter pipeline.
- *Fill Line for Elena Tanks (Project PWP-3):* It is recommended that approximately 1,910 feet of 10-inch diameter pipeline be added to serve as a feed line for the Elena tanks.
- *Parallel Line on Juniper Avenue (Project PWP-4):* It is recommended that a parallel 8-inch diameter pipeline be constructed to convey water from Nutmeg pressure zone to Blanca pressure zone.
- *Pipeline on Sequoia Street (Project PWP-5):* It is recommended that a parallel 8-inch diameter pipeline be constructed along Sequoia Street, between Dogwood Avenue and Main Street in order to supply flow from Nutmeg zone to Blanca zone.

Table 6.6 Recommended Fire Flow Improvements

Improv. ID	Location	Extent	Diam. (inches)	Length (feet)
PWFF-1	Quintana Rd	South Bay Blvd. to La Loma St.	10	3,000
PWFF-2	Quintana Rd	Morro Bay Blvd. to Quintana Pl.	16	3,300
PWFF-3A	Main St.	Radcliff Ave. to Preston Ln.	12	1,650
PWFF-3B	Main St.	Preston Ln. to Errol St.	10	600
PWFF-4	Ridgeway St.	Black Hill Tank to Fairview Ave.	10	830
PWFF-5A	Highway 41 and Main St.	Sunset Ave. to Main St.	8	270
PWFF-5B	Highway 41 and Main St.	Highway 41 to Hill St.	8	870
PWFF-5C	Highway 41 and Main St.	Hill St. to Avalon St.	10	950
PWFF-6A	Highway 41 and Main St.	Highway 41 to Atascadero Rd.	8/20	270
PWFF-6B	Upper Kings Zone	PRV from Upper Kings to Elena Zone	8	--
PWFF-7	Juniper Ave.	Elena St. to San Joaquin St.	8	570
PWFF-8A	San Joaquin St.	Juniper Ave. to Ironwood Ave.	8	240
PWFF-8B	San Joaquin St.	Ironwood Ave. to Hemlock Ave.	8	230
PWFF-8C	San Joaquin St.	Hemlock Ave. to Greenwood Ave.	8	100
PWFF-8D	San Joaquin St.	Hemlock Ave. to Alder Ave.	10	970
PWFF-8E	San Joaquin St.	PRV between Hemlock Ave. and Greenwood Ave.	8	--
PWFF-9A	Greenwood Ave.	San Joaquin St. to San Jacinto St.	10	640
PWFF-9B	San Jacinto St.	Greenwood Ave. to Alder Ave.	10	850
PWFF-10	Sequoia St.	Dogwood Ave. to Main St.	10	540
PWFF-11	Las Vegas St. and Main St.	Elm Ave. to Main St. and Las Vegas St. to Elena St.	8	1,610
PWFF-12	Highway 41 and Hill St.	Ironwood Ave. to Hill St. and Rockview St along Hill St.	8	1,290
PWFF-13	Ponderosa St.	entire length of Ponderosa St.	8	910
PWFF-14	Mimosa St.	Ironwood Ave. to alley northwest of Hill St.	8	580
PWFF-15A	Surf St.	Market St. to west of West St.	8	640
PWFF-15B	Surf St.	Embarcadero Rd. to west of West St.	8	420

Table 6.7 Recommended Fire Flow Improvements (continued)

Improv. ID	Location	Extent	Diam. (inches)	Length (feet)
PWFF-16	Piney Way and Alta Ct.	Main St. to Alta Ct. and along entire length of Alta Ct.	8	910
PWFF-17	Fairview Ave.	Fairview Ave. to Arbutus Ave.	8	1,050
PWFF-18	Bolton Dr.	Norwich Ave. to Radclidd Ave.	8	700
PWFF-19	Norwich Ave.	Clarabell Dr. to Bolton Dr.	8	220
PWFF-20	Blanca St. and Tuscan Ave.	Blanca Tanks to Tuscon Ave. and Blanca St. to Dawson St.	8	450
PWFF-21	Toro Ln.	northwest of Yerba Buena St.	8	800
PWFF-22	Sequoia Ct. and Ironwood Ave.	Sequoia St. to Sequoia Ct. and Ironwood Ave. to east	8	385
PWFF-23	Paula St.	Hemlock Ave. to Greenwood Ave.	8	200
PWFF-24	Reno Ct.	entire length of Reno Ct.	8	450
PWFF-25	Errol St.	entire length of Errol St.	8	545
PWFF-26	Napa Ave.	north of Dunes St.	8	210
PWFF-27A	Marina St.	Main St. to Morro Ave.	8	220
PWFF-27B	Marina St.	Morro Ave. to Main St.	8	115
PWFF-28	Driftwood St.	Morro Ave. to Embarcadero Rd.	8	370
PWFF-29	South St.	Morro Ave. to Embarcadero Rd.	8	300
PWFF-30	Piney Ln.	entire length of Piney Ln.	8	370
PWFF-31	Avalon St. and Ironwood Ave.	PRV at intersection of Avalon St. and Ironwood Ave.	8	--
PWFF-32	Highway 41 and Ironwood Ave.	PRV at intersection of Highway 41 and Ironwood Ave.	8	--
PWFF-33	Dunes St. and Shasta Ave.	PRV at intersection of Dunes St. and Shasta Ave.	8	--
PWFF-34	Main St. and Quintana Pl.	PRV at intersection of Main St. and Quintana Pl.	8	--
PWFF-35	Sunset Rd.	Highway 41 to Hill St. and Sunset Rd. to Main St.	8	2,000

6.4.2.5 PRV Station Improvements

PRV Station improvement projects were developed to mitigate system deficiencies and improve reliability. The recommended PRV station improvement is described below:

- *PRV on Juniper Avenue (Project PWV-1):* It is recommended that a new PRV station be constructed to break Nutmeg zone and Blanca zone along the proposed parallel line along Juniper Avenue (Project PWP-4).

6.4.2.6 Other Improvements

A variety of other projects were identified in addition to the distribution system improvements. The recommended improvements are described below:

- *Water Master Plan Update (PWO-1):* It is recommended that City update its water master plan every five years.
- *Re-Skin Desalination Plant Building (PWO-2):* It is recommended that the existing Desalination building be re-skinned.
- *Control Upgrades at Desalination Plant (PWO-3):* It is recommended that the controls at the Desalination plant be upgraded.
- *Add Screening at Desalination Plant (PWO-4):* It is recommended that screening be added at the calcium carbonate tanks at the Desalination plane.
- *Chlorination Upgrades (PWO-5):* This project includes chlorination upgrades at the Desalination plant and at the Kings tanks.
- *Desalination Plant Relocation (PWO-5):* This project includes the relocation of the Desalination plant. The cost for this project is not defined in OneWater as there is not enough information to determine an accurate cost estimate. Cost for this project will be determined in the next master plan update.

6.4.3 Wastewater Collection System Improvements

Figure 6.10 illustrates the recommended improvements to mitigate the collection system deficiencies. This section provides a detailed description of each recommended wastewater collection system improvement project.

6.4.3.1 Gravity Main Improvements

The following gravity main improvements are recommended for the collection system:

- *Gravity Main along Atascadero Road (Project WWGM-1):* This project includes the replacement of approximately 1,000 feet of 18-inch diameter pipeline along Atascadero Road, between Main Street and Park Street. The flow levels of the gravity sewer causes upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 27-inch diameter pipeline.
- *Gravity Main along Main Street (Project WWGM-2):* This project includes the replacement of approximately 2,900 feet 15-inch diameter pipeline along Main Street, between Atascadero Road and Las Vegas Street. The flow levels within the gravity sewer cause the existing pipeline to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.
- *Gravity Main along Main Street (Project WWGM-3):* This project includes the replacement of approximately 1,600 feet of 12-inch diameter pipeline along Main Street, between Las Vegas Street and San Joaquin Street. The flow levels within the gravity sewer cause upstream surcharge within 5 feet of the rim during PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with an 18-inch diameter pipeline.

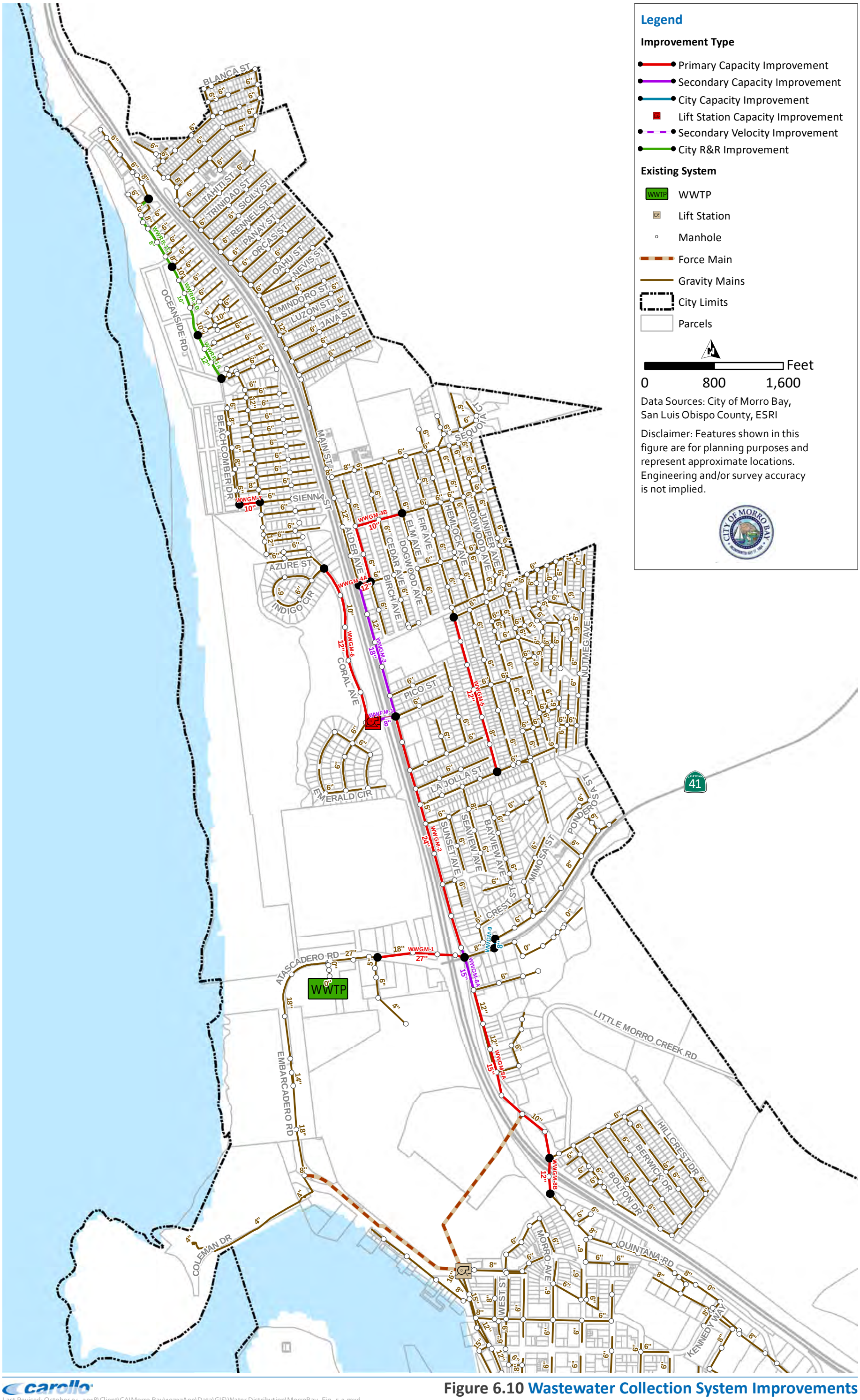

Figure 6.10 Wastewater Collection System Improvements

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- *Gravity Main along San Joaquin Street (Project WWGM-4A):* This project includes the replacement of 6-inch diameter pipelines approximately 150 feet along San Joaquin Street, between Main Street and Alder Avenue. The flow levels within the gravity sewer cause the existing pipeline to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 12-inch diameter pipelines.
- *Gravity Main along Alder Avenue, and San Jacinto Street (Project WWGM-4B):* This project includes the replacement of 6-inch diameter pipelines approximately 650 feet along Alder Avenue, between San Joaquin Street and San Jacinto Street and 550 feet along San Jacinto Street, between Alder Avenue and Elm Avenue. The flow levels within the gravity sewer cause the existing pipeline to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 10-inch diameter pipelines.
- *Gravity Main along Greenwood Avenue (Project WWGM-5):* This project includes the replacement approximately 1,850 feet 6-inch diameter pipeline along Greenwood Avenue, between Avalon Street and Elena Street. The flow levels within the gravity sewers cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
- *Gravity Main along Coral Avenue (Project WWGM-6):* This project includes the replacement of approximately 1,900 feet of 10-inch diameter pipeline along Coral Avenue, between Lift Station 1 and Azure Street. The flow levels in the gravity sewer cause upstream manholes to surcharge within 5 feet of the rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
- *Gravity Main along Sienna Street (Project WWGM-7):* This project includes the replacement of approximately 250 feet 6-inch diameter pipeline along Sienna Street, between Sandalwood Avenue and Beachcomber Drive. The flow levels within the gravity sewer causes upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.
- *Gravity Main along Main Street (Project WWGM-8A):* This project includes the replacement of approximately 2,600 feet of 10-inch and 12-inch diameter pipeline along Main Street, between Radcliff Avenue and Atascadero Road. The flow levels within the gravity sewer cause the existing pipeline to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 15-inch diameter pipelines.
- *Gravity Main along Main Street (Project WWGM-8B):* This project includes the replacement of approximately 400 feet of 8-inch and 10-inch diameter pipeline along Main Street south of Radcliff Avenue. The flow levels within the gravity sewer cause the existing pipeline to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 12-inch diameter pipelines.

- *Gravity Main under Highway 41 (Project WWGM-9):* City Staff indicated that there is a belly in the existing 6-inch diameter main that crosses Highway 41, which has created ongoing maintenance issues. It is recommended that the existing 6-inch diameter pipe crossing under Highway 41 be replaced with a new 8-inch diameter main. This pipeline would be installed in a 20-inch diameter casing.

6.4.3.2 Lift Station Improvements

The following lift station improvements are recommended for the existing collection system:

- *Lift Station 1 Replacement (WWLS-1):* This project includes replacing Lift Station 1. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. To mitigate the risk of a SSO occurring during PWWF conditions it will be sized to handle future PWWF. It is recommended that the new lift station have a firm capacity of 1.10 mgd and a total capacity of 1.65 mgd.
- *Lift Station 1 Force Main (WWFM-1):* The purpose of this project is to mitigate the high velocity (> 8 fps) that the existing force main experiences following WWLS-1 under existing conditions. It is recommended that an 8-inch diameter force main be constructed parallel to the existing 6-inch diameter force main.

6.4.3.3 Rehabilitation and Replacement Projects

Historically, high amounts of inflow and infiltration (I/I) enters the City's wastewater collection systems during wet weather events. The temporary flow monitoring conducted by V&A Consulting Engineers (V&A) confirmed that the City's sewer system experiences high amounts rainfall derived inflow and infiltration (RDII). The results of the flow monitory program are summarized in Chapter 4. High rates of I/I can lead to sanitary sewer overflows (SSOs), basement backups, hydraulic overloading at lift stations and the wastewater treatment plant (WWTP), and higher operational costs to pump and treat the additional flows.

To mitigate the negative impacts of I/I on the City's system, and to rehabilitate aging infrastructure, the following projects are recommended:

- *Gravity Main along Beachcomber Drive (Project WWRR-1A, 1B, and 1C):* The area upstream of Lift Station 1 was shown to have the highest I/I rates in the City. The City suspects that a large contributor to this problem is the gravity main installed on Beachcomber Drive, which is known to contribute to I/I in the collection system. This project includes the rehabilitation or replacement of approximately 570 feet of 12-inch diameter pipeline along Beachcomber Drive between Java Street and north of Mindoro Street, rehabilitation or replacement of approximately 960 feet of 10-inch diameter pipeline along Beachcomber Drive between north of Mindoro Street and north of Sicily Street, and rehabilitation or replacement of approximately 950 feet of 8-inch diameter pipeline along Beachcomber Drive between north of Sicily Lane and Toro Lane.
- *Pipe R&R Program Upstream of LS-1 (Project WWRR-2):* This project includes the rehabilitation or replacement of pipelines that are upstream of Lift Station 1 and not already a recommended improvement project (e.g., Project WWRR-1A-1C). This equates to approximately 3,000 feet of pipeline projects between 2020 through 2028. The City should use existing inspection data to prioritize pipelines and choose the best rehabilitation or replacement method. Pipelines less than 8-inches in diameter be replaced by a minimum pipeline diameter of 8-inches.

- *I/I Reduction Projects (Project WWRR-3):* This project includes the rehabilitation or replacement of pipelines and structures that contribute significant amount of I/I. These projects will be identified during the I/I Reduction Program's inspection phases. Inspection data will be used to prioritize projects and determine best rehabilitation method or replacement. An annual budget estimate of $100,000 per year has been assumed for rehabilitation and replacement through the year 2040.
- *Cap and Plug Replacement Upstream of Lift Station 1 (WWRR-4):* This Project consists of replacing caps and/or plugs upstream of Lift Station 1 with manhole. The caps upstream of Lift Station 1 are known to contribute high amounts of inflow to the system. This project consists of replacing 31 caps with manholes between 2019 and 2020.
- *Cap and Plug Replacement Program (WWRR-5):* This project consists of replacing the remaining cap and/or plug structures in the collection system with manholes. These projects do not include caps and plugs replaced upstream of Lift Station 1. This project consists of replacing 121 caps with manholes.

6.4.3.4 Other Improvements

A variety of other projects were identified in addition to the distribution system improvements. The recommended improvements are described below:

- *Collection System Master Plan Update (WWO-1):* It is recommended that City update its collection system master plan every five years.
- *Flow Monitoring Program (WWO-2):* It is recommended that a temporary flow monitoring program be conducted every five years in conjunction with the collection system master plan update.

6.4.4 Storm Drainage System Improvements

Figure 6.11 illustrates the recommended improvements to mitigate the storm drainage system deficiencies. The following provides a detailed description of each recommended wastewater collection system improvement project:

- *Redirect a Portion of Runoff from Roundabout Area (SD-1A and 1B):* This project is recommended to redirect an area of approximately 86-acres that currently runs off to the roundabout area. It is recommended that approximately 125-feet of 27-inch and 2,600 feet of 30-inch diameter storm drains be constructed along Anchor Street from Kern Avenue to Main Street. This project is needed to accommodate the 10-year and 100-year design storms.
- Marina Street Storm Drain (SD-2A and 2B): There is a 6-inch bleeder line on Marina Street at Shasta, as well as some additional culverts on Marina Street that convey flow to downstream gutters. To improve system hydraulics in the area, it is recommended that approximately 370-feet of 12-inch and 600-feet of 15-inch diameter storm drain pipe be installed along Marina Street from Piney Way to Monterey Avenue. This project is needed to accommodate the 10-year and 100-year design storms.
- Harbor Street Storm Drain (SD-3): This project includes abandoning the existing 24-inch diameter pipeline along Main Street between Harbor and Main Street, and the addition of approximately 890 feet of 24-inch diameter pipeline along Harbor Street, between Main Street and Centennial Parkway. This project is recommended to mitigate flooding along Morro Bay Boulevard. The new pipeline will redirect flow to Harbor Street, which

has enough capacity to convey the additional runoff. This project is needed to accommodate the 10-year and 100-year design storms.

- Roundabout Storm Drain Replacement (SD-4): This project, in conjunction with project SD-1A and 1B, will ease flooding in the roundabout area. This project includes the replacement of approximately 130 feet of 12-inch diameter pipelines along the north side of the roundabout at Morro Bay Boulevard and Quintana Road with an 18-inch diameter storm drain. This project is needed to accommodate the 10-year and 100-year design storms.
- Beach Street Storm Drain (SD-5): There is a 6-inch bleeder line on Beach Street near Morro Avenue that takes runoff from a 36-inch storm drain and discharges to a downstream gutter. To improve system hydraulics in this area, it is recommended that approximately 1,000 feet of 24-inch diameter pipeline along Beach Street, between Morro Avenue and Embarcadero Road be constructed. This project is needed to accommodate the 10-year and 100-year design storms.
- Bike Path Storm Drain (SD-6A and 6B): Willow Camp Creek crosses the City's bike path west of Main Street through a very small 1 foot x 3 foot box drain. This box drain floods routinely during wet weather, causing the bike path to be closed. This project would replace the box drain and extend a new storm drain line farther downstream. To mitigate the bike path flooding, a new 48-inch and 54-inch triple barrel storm drain is recommended. Approximately 3,500 feet of triple barrel storm drains are recommended. This project is needed to accommodate the 10-year and 100-year design storms.
- Main Street Storm Drains (SD-7, SD-8, and SD-31): These projects are required to mitigate flooding along Main Street upstream of Willow Camp Creek. The recommended improvements consist of replacing the existing storm drains in the area with 15-inch, 18-inch, 24-inch, and 27-inch diameter storm drains. Projects SD-7 and SD-8 are needed to accommodate the 10-year and 100-year design storms. Project SD-31 is only needed to accommodate the 100-year design storm.
- Morro Creek Area Improvements (SD-9, SD-10, SD-24, SD-25, SD-27, and SD-28): These projects are needed to mitigate flooding in scattered areas in the Morro Creek drainage area. The existing storm drains in these areas will be replaced with 15-inch, 24-inch, and 36-inch diameter storm drains. Projects SD-9 and SD-10 are needed to accommodate the 10-year and 100-year design storms. Projects SD-25, SD-25, Sd-27, and SD-28 are only needed to accommodate the 100-year design storm.
- Avalon and Hillview Street Area Storm Drains (SD-11, SD-12, SD-13, SD-21, SD-22, SD-23, SD-26, SD-29, SD-30, and SD-34): These projects are needed to mitigate flooding in scattered areas in the near Avalon Street. The existing storm drains in these areas will be replaced with 12-inch, 24-inch, 30-inch, 36-inch, and 42-inch diameter storm drains. Projects SD-11, SD-12, and SD-13 are needed to accommodate the 10-year and 100-year design storms. Projects SD-21, SD-22, SD-23, SD-26, SD-29, SD-30, and SD-34 are only needed to accommodate the 100-year design storm.

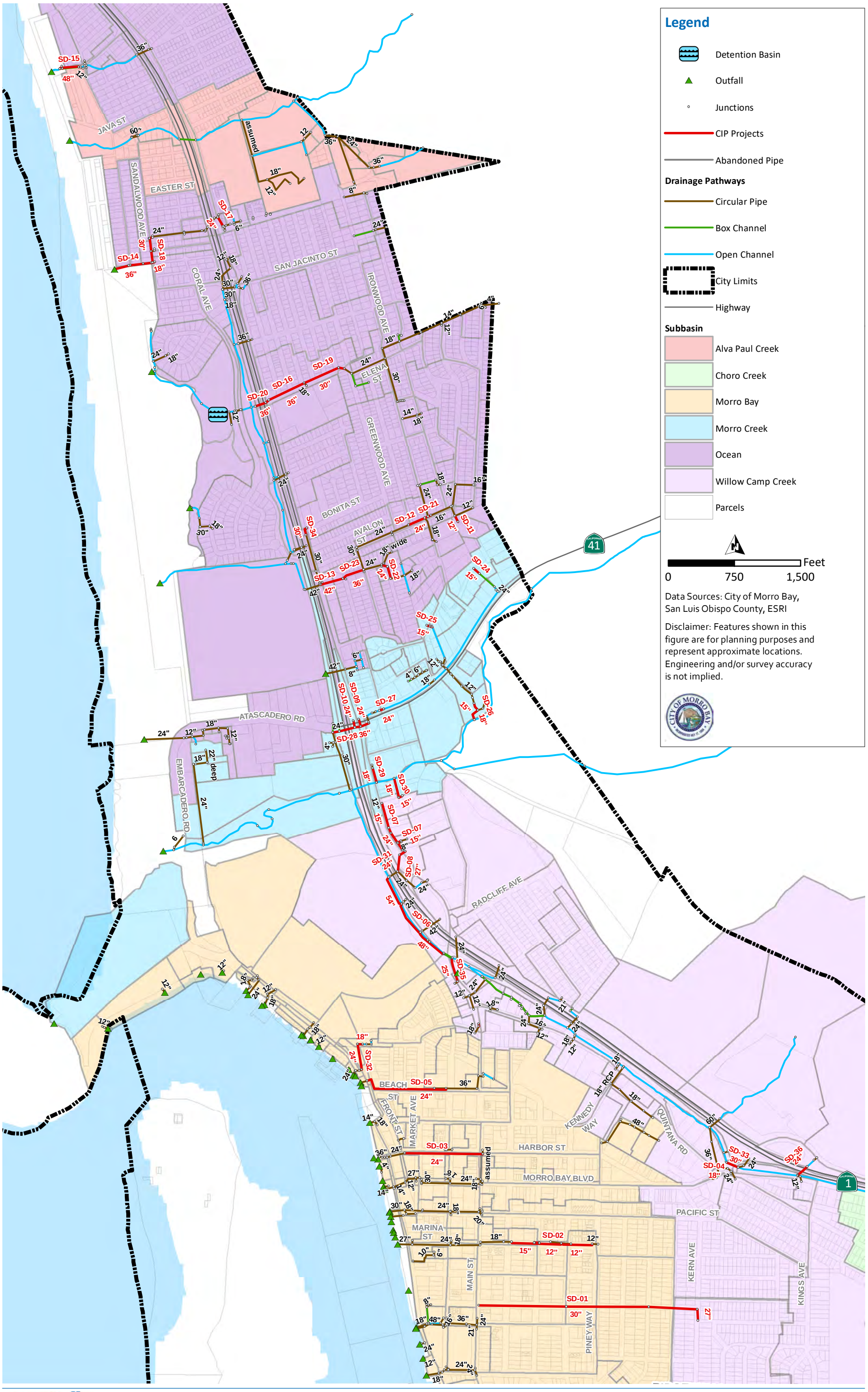

Figure 6.11 Storm Drain System Improvements

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- Sequoia Street/Sienna Street Storm Drains (SD-14, SD-17, and SD-18): The hydraulic model showed some flooding near Sienna Street/Sandalwood Avenue and Sandalwood Avenue/Bali Street, as well as the Main Street/Sequoia Street area. The existing storm drains in these areas will be replaced with 18-inch, 24-inch, 30-inch, and 36-inch diameter storm drains. Project SD-14 is needed to accommodate the 10-year and 100-year design storms. Projects SD-17 and SD-18 are only needed to accommodate the 100-year design storm.
- Morro Strand Campground Storm Drain (SD-15): Orcas Creek discharges into a 16 inch by 42 inch box drain that flows underneath the Morro Strand Campground. The hydraulic model showed that the box drain does not have enough capacity to convey the 10-year design storm runoff. It is recommended that the existing box drain be replaced with a 48-inch diameter pipeline. This project is needed to accommodate the 10-year and 100-year design storms.
- Elena Street Storm Drains (SD-16, SD-19, and SD-20): These projects are recommended to mitigate simulated flooding upstream of Highway 1 on Elena Street. The existing storm drains in these areas will be replaced with 30-inch and 36-inch diameter storm drains. Project SD-16 is needed to accommodate the 10-year and 100-year design storms. Projects SD-19 and SD-20 are only needed to accommodate the 100-year design storm.
- Surf Street and West Avenue Storm Drain (SD-32A and SD-32B): This project includes the replacement of approximately 360 feet of 12-inch and 18-inch diameter pipeline in the parking lot located west of the intersection of Surf Street and West Avenue with a new 18-inch and 24-inch pipeline. This project is needed to accommodate the 100-year design storm only.
- Main Street/Morro Bay Boulevard Storm Drain (SD-33): This project includes the replacement of approximately 140 feet of 24-inch diameter pipeline located east of the roundabout at Morro Bay Boulevard and Quintana Road with a 30-inch diameter pipeline. This project is needed to accommodate the 100-year design storm only.
- Main Street/Highway 1 Storm Drain (SD-35): This project includes the addition of approximately 270 feet of 15-inch diameter pipeline along Main Street, between Quintana Place and Cabrillo Highway. In this area, the flow travels over the street crown and crosses over Main Street. To fix this issue, it is recommended that this 15-inch diameter pipeline be added. This project is needed to accommodate the 10-year and 100-year design storms.
- Morro Bay Boulevard Off Ramp Storm Drain (SD-36): This project includes the replacement of approximately 150 feet of 24-inch diameter pipeline near the Morro Bay Boulevard off ramp that goes under Highway 1. This pipe has failed, and currently has no bottom. It is recommended that the existing failed pipeline be replaced with a 24-inch diameter pipeline. This project is needed to accommodate the 10-year and 100-year design storms.
- Rehabilitation and Replacement Projects (SDRR-1): This project includes the rehabilitation or replacement of failing storm drains. An annual budget estimate of $100,000 per year has been assumed for rehabilitation and replacement through the year 2040.

- Storm Drainage System Master Plan Update (SDO-1): It is recommended that City update its storm drain master plan every five years.

Chapter 7
WATER SUPPLY ALTERNATIVES ANALYSIS

This chapter summarizes the water supply evaluation alternative analysis for Morro Bay's potable water system. The goal of this analysis is to identify reliable, drought-tolerant supplies sufficient to provide for the City of Morro Bay (City) through 2050. As identified in Chapter 4, the 2040 demands are expected to be 1,445 acre-feet per year (AFY).

7.1 Water Supply Alternatives Development

The following sections describe in detail the water supply alternatives developed and presented to the City. There were six main alternatives considered for the City with sub-alternatives identified related to treatment. The alternatives are presented in Table 7.1. Each water supply alternative was developed keeping in mind the City's future water reclamation facility (WRF) project which must provide up to a minimum of secondary wastewater treatment. The capital improvement cost for the WRF is not included in the final unit cost for each alternative. Alternatives that utilize the WRF project's secondary treated effluent as source water will include costs for the advanced treatment as necessary. These alternatives display the supplementary unit cost in addition to the City's existing capital financing for the WRF project. It is assumed all alternatives that do not completely meet the future average day demand will be supplemented by imported water from the California State Water Project (SWP) surface water deliveries as shown in Figure 7.1. Since the City's allocation for SWP cannot meet the total future water demand, Alternative 1 is supplemented the extra supply needed with groundwater.

Table 7.1 Supply Source Percentages of Water Supply Alternatives

Project No.	Project Name
1	State Water Project
2A	Morro Well Field With Salinity Treatment
2B	Morro Well Field with Nitrate Treatment
2C	Morro & Chorro Well Fields with Salinity Treatment
2D	Morro & Chorro Well Fields with Nitrate Treatment
2E	Morro & Chorro Well Fields with Nitrate & Salinity Treatment
3A	Chorro Well Fields with Streamflow Augmentation
3B	Chorro Well Fields with Streamflow Augmentation & Nitrate Treatment
4A	Morro Well Field IPR by Groundwater Injection
4B	Morro Well Field IPR by Groundwater Injection with Salinity Treatment
4C	Morro Well Field IPR by Groundwater Injection with Nitrate Treatment
5	Ocean Desalination
6	Direct Potable Reuse

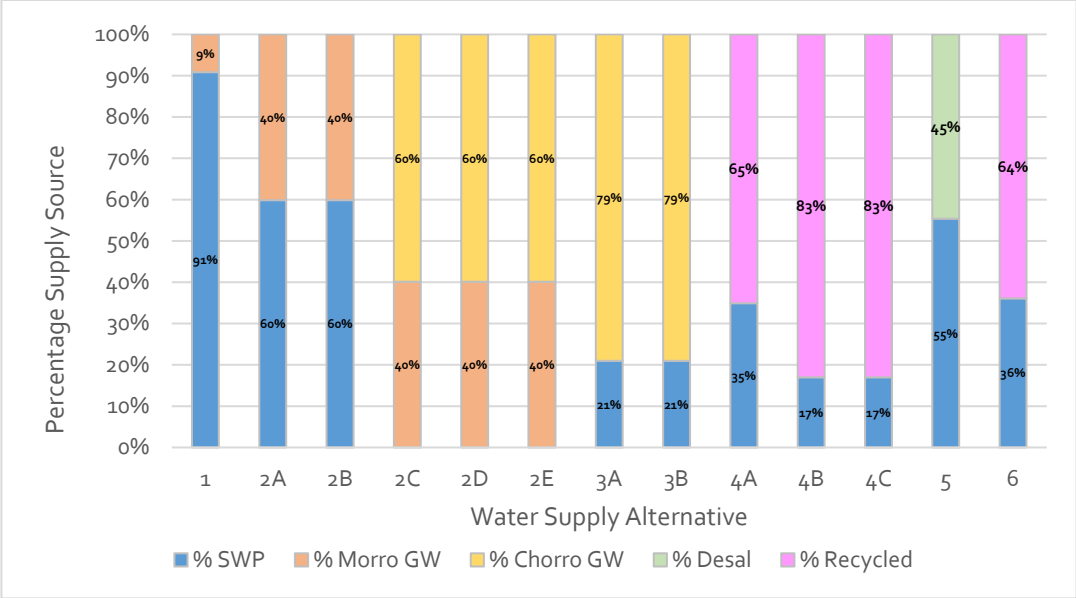

Figure 7.1 Water Supply Source Percentages of Average Day Demand

7.1.1 Preliminary Siting of Water Supply Alternatives

The preliminary siting for any treatment facilities associated with the individual water supply alternatives in this analysis considered the location of the existing brackish water reverse osmosis (BWRO), sea water desalination facility, and associated infrastructure as well as the proposed location of the WRF at the South Bay Boulevard site. For this analysis, site locations remained consistent for each alternative so that a relative cost analysis could be completed between them. Any facilities located adjacent to the City's existing water treatment facilities could be located anywhere along the pipeline alignment between the Morro wells and Kings Tank with relatively minimal cost impacts. The location used in this analysis was selected to minimize the differences between supply alternatives since this site, as discussed in Section 3.1.1.4, is outside of the Federal Emergency Management Agency (FEMA) flood hazard zone and is not affected by coastal hazards created by sea-level rise. This site is also near the existing water treatment facilities which minimize the necessary improvements to the existing water distribution infrastructure. If the City were to move forward with a project that included relocation of any of the water treatment facilities, then an extensive siting study would be needed that would consider numerous factors including consistency with the City's Local Coastal Plan.

7.1.2 Alternative 1: Purchased Water from California State Water Project

7.1.2.1 Description of Alternative 1

The SWP delivers imported surface water to the City of Morro Bay. State Water delivered to the City of Morro Bay originates in Northern California and is conveyed through a series of canals, pipes, and pumping plants to the Polonio Pass Treatment Facility owned and operated by the Central Coast Water Authority (CCWA). From Polonio Pass, the CCWA facilities serve as the primary conveyance of State Water to San Luis Obispo and Santa Barbara Counties. Of the 4,830 acre-feet (ac-ft) of Table A water contracted to San Luis Obispo County, 1,313 ac-ft is allocated to the City of Morro Bay. The City and other SWP users in San Luis Obispo County have

also purchased "drought buffer" whereby they have excess Table-A supply, but have not purchased pipeline capacity. This in effect means that even in times of reduced deliveries, the City has a more reliable supply. The location and major project components of Alternative 1 are shown in Figure 7.2.

7.1.2.2 Supply Yield and Availability

Historically, Morro Bay has received about 80 percent of their domestic water supply from imported State Water. During periods of drought, the State Water is not always available to its contractors, as demonstrated in 2015 when SWP deliveries dropped to 20 percent. California Department of Water Resources (DWR) State Water Project Reliability Delivery Reports estimates the SWP Table A delivery allocations meeting the full requests of its contractors will be approximately 77 percent over the next two years. Morro Bay's base Table A contractual allocation does not meet the projected future average day demand without a supplemental source to the City. Currently, the City supplements imported state water supply with groundwater from the Morro well field treated by the BWRO facility.

7.1.2.3 Implementation Considerations

The infrastructure necessary to convey State Water to the City of Morro Bay is already in place. Capital improvement projects, retrofits, or maintenance measures are completed annually to maintain the system's ability to continue deliveries to State Water contractors. All costs associated with maintenance, retrofit, and operations is included in the annual DWR charges to the City.

7.1.2.4 Water Quality Considerations

All imported State Water for the City is fully treated at the Polonio Pass Water Treatment Facility. Water quality reports of State Water and treated State Water are available from CCWA. Records show instances where the quality of imported State Water has had fluctuating total dissolved solids (TDS) concentrations from year to year. Nitrate concentrations have remained relatively low throughout the implementation of the State Water deliveries.

Figure 7.2 California State Water Project Delivery to Morro Bay

7.1.2.5 Costs

This section describes in detail the costs the City of Morro Bay pays to import State Water from the California Aqueduct. All data used in this cost analysis is available to the public in the most recent State of California Bulletin 132-16 Management of the California State Water Project and CCWA's CAFR and Fiscal Year budgets. Figure 7.3 summarizes the general cost components that comprise the total cost of water delivered from the California SWP to Morro Bay.

State Water Cost Analysis Assumptions

The cost analysis performed for the OneWater Plan by Carollo projects State Water costs through the year 2050. Being that DWR financial data is only available to 2035 and CCWA to 2027 several assumptions were included to complete the study.

DWR:

- Where costs remain constant to the year 2035, the value of that financial component was carried through the year 2050. For example, the Delta Water Charge remains constant at $1,721,914 from 2017 to 2035. This annual charge was continually projected to 2050.
- Where a series of costs fluctuated on an annual basis to the year 2035, the trend of each series was applied to the year 2050 to extend the projected cost. For example, from 2017 to 2035, Variable Operation and Maintenance, Power and Replacement (OMP&R) has increased by 1 percent on an annual basis. This trend was used to project the Variable OMP&R cost to 2050.
- For net present value calculation, a discount rate of 2 percent was used.
- The construction cost of the SWP will be paid off in 2035.

CCWA:

- CCWA fixed and variable costs trends between 2017 and 2027 were projected to the year 2050.
- San Luis Obispo County's allocation of State water from 2028 to 2050 is assumed to be 57 percent which follows the projected allocation from 2019 to 2027.
- The CCWA bond debt payment was refinanced in 2017.
- The CCWA bond debt payment will be paid off in 2022.
- Drought buffer allocations are not accounted for in this analysis
- Administrative fees are not included in this analysis

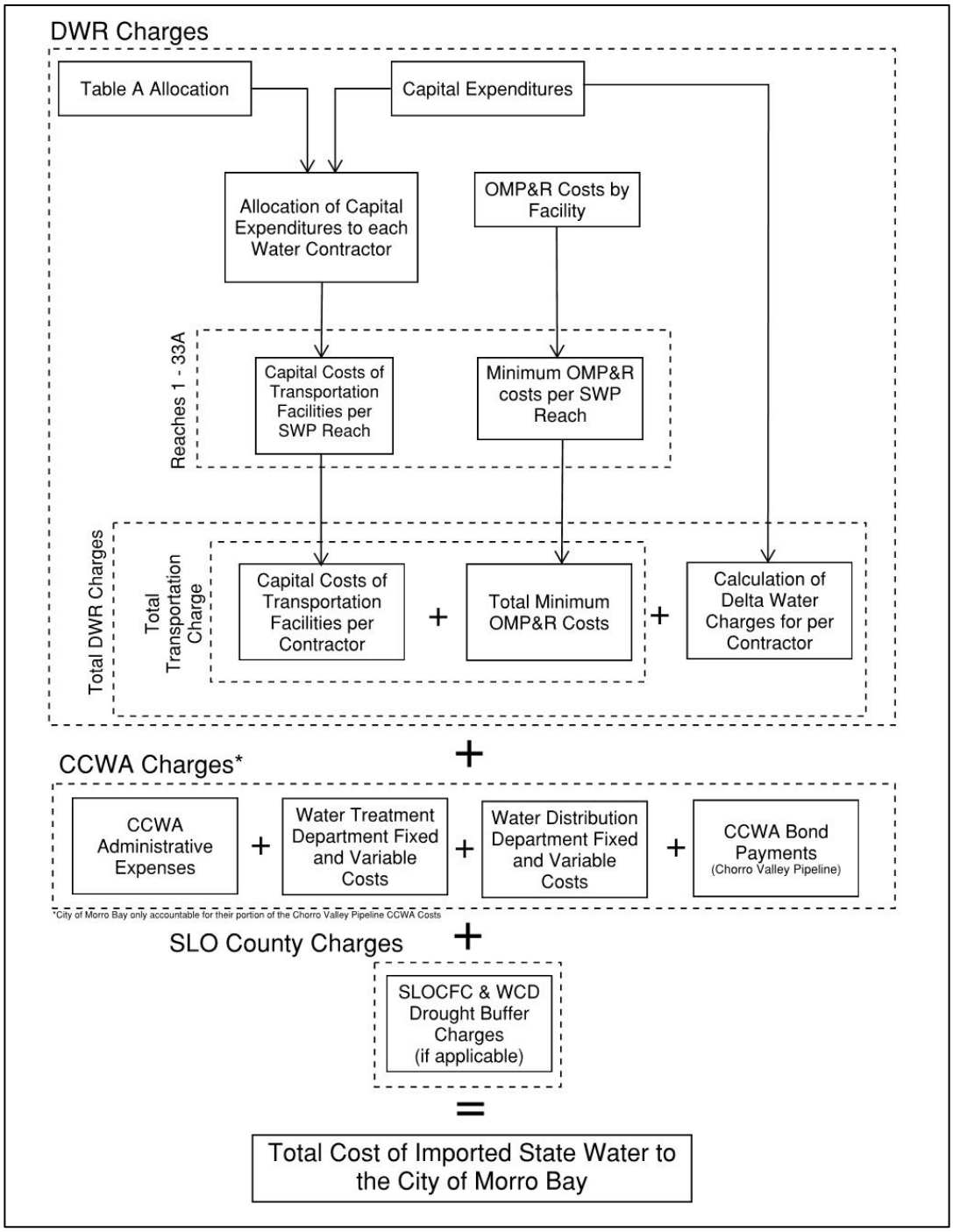

Figure 7.3 Cost Components of DWR Charges of State Water

California Department of Water Resources Costs

The County of San Luis Obispo has a long term contract with DWR for 25,000 ac-ft per year of Table A water as displayed in Bulletin 132, Table B-4. Costs incurred by the SWP are a proportion of the Table A allotments, therefore San Luis Obispo's share of the construction costs are a

function of their Table A volume. For future costs projections, it is highly unlikely, both hydraulically and by demand, that a full 25,000 ac-ft per year would be allocated to San Luis Obispo County. Bulletin 132 Table B-5A uses a delivered quantity of 13,237 ac-ft as a conservative delivered volume to calculate future variable OMP&R charges. Table B-17 of the Bulletin 132 Appendix B displays the Unit Variable OMP&R charge accumulated by reach of the SWP calculated from deliveries made and anticipated to water contractors. San Luis Obispo County only receives 4,830 AFY maximum based on contractual limitations. DWR costs were calculated and adjusted using this reduced allocation volume as a realistic unit cost estimate. With the adjusted allocation of Table A water to San Luis Obispo County, the Total Variable OMP&R cost can be recalculated to represent a realistic estimate. The recalculated Variable OMP&R cost is included in the total transportation charge throughout the cost analysis. Figure 7.4 displays the total DWR water charge including the Total Transportation Charge and the Delta Water charge projected to 2050. The significant drop at 2035 represents the retirement of the SWP capital construction annual payment. The high cost outlier in 2016 displays the extremely low allocation as a result of peak drought conditions in the year 2015. This unit cost represents the DWR component to deliver water to San Luis Obispo County via SWP.

Figure 7.4 Adjusted DWR Unit Cost of State Water

Central Coast Water Authority (CCWA) Costs

The City has a 27 percent proportion of the entire allocation within San Luis Obispo County. The CCWA budget associates both fixed and variable components proportionate to the deliveries contracted to project participants. San Luis Obispo County participants are separated into three regions, Shandon, Chorro Valley, and Lopez. Each represents a separate turnout constructed to serve various groups in contract with San Luis Obispo County Flood Control and Water Conservation District (SLOCFCWCD) and CCWA to receive state water. Future costs follow trends set by published projections within CCWA's fiscal year budgets and it is assumed that the CCWA bond repayment for both the Polonio Pass Treatment Facility and the Chorro Valley Pipeline will be retired in 2022. Figure 7.5 displays the projected unit costs for the CCWA component of water. The significant decrease in cost is a result of the CCWA bond debt

retirement in 2022. Additionally, the linear trend between 2022 and 2050 follows the assumed escalation in costs CCWA projected in their annual budget report.

Figure 7.5 CCWA Unit Cost of State Water

California WaterFix Costs

The State of California continually upgrades the SWP operation procedures and infrastructure to improve efficiency in delivering water. With increasing demands for water from growing populations and climate change, the reliability of the SWP is essential in securing water deliveries. The California WaterFix project proposes to increase reliability to the SWP by bypassing the Sacramento-San Joaquin Delta with underground transmission tunnels in order to protect delicate wildlife ecosystems, update California's out of date State Water system, and improve delivery reliability to downstream users. Figure 7.6 displays a map of the California WaterFix project with respect to the Sacramento-San Joaquin Delta.

The financial planning of the California WaterFix project implements the total cost of the project after the original SWP capital construction costs have been retired. Similar to the SWP, construction costs will be distributed in proportion to each water contractor per Table A allocations. Planning documents assume construction is to be substantially complete in 2032 at a total cost of $16.7 billion dollars. Chapter 8 of the Bay Delta Conservation Plan 2013 and Metropolitan Water's "California WaterFix Finance and Cost Allocation" estimate total Operation and Maintenance (O&M) costs in 2013 dollars to be $38.1 million and $44.1 million in 2017 dollars. Additionally, it was assumed mitigation was included in the operating costs which results in a total annual O&M costs of approximately $64.4 million. Table 7.2 displays the cost breakdown for the San Luis Obispo County portion of the California WaterFix. San Luis Obispo County is 0.616 percent of the total Table A allocation. Therefore the annual cost of the total project can be proportioned by this percentage to reflect San Luis Obispo's repayment proportion.

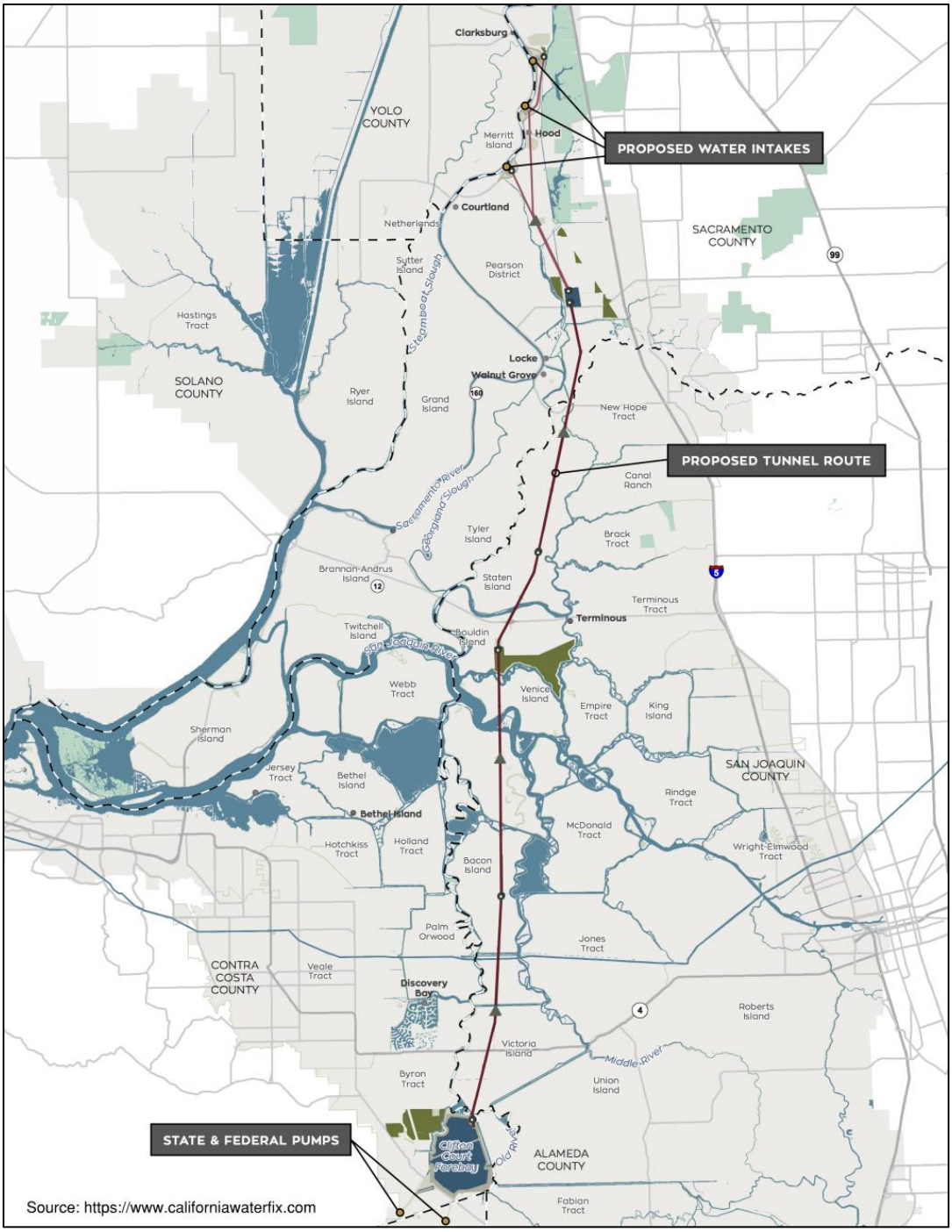

Figure 7.6 Overview of California WaterFix

The years prior to operation of the facility, capital repayments are to steadily grow and increase total costs of State Water until construction is complete in 2032. This represents increased payment and distribution of bonds as construction costs increase until substantial completion is reached. Using a growth rate of 3 percent over 15 years, unit costs start at $12.90 per acre foot in 2017 dollars, and with a 6 percent interest rate, ultimately reach $239.85 per acre foot in 2032. The repayment of the growing value represents the ramp-up dollar value San Luis Obispo

County is responsible for. Once construction is complete, the full financing repayment unit cost can be applied to the total unit cost of water delivery to each project participant. The fixed capital costs repayment remains constant over the finance period of the project. Inflation increases the dollar value of O&M over the analysis period to 2050. Figure 7.7 displays the increase in unit cost of imported State Water as a result of the California WaterFix. Similar to the SWP capital construction finance estimates, San Luis Obispo County's unit cost is dependent on their Table A allocation of 25,000 ac-ft.

Table 7.2 California WaterFix Finance Components

Component	Amount
Assumed Total Capital Cost:	$16,700,000,000
Assumed Total Annual O&M Cost:	$64,400,000
SLO Co. Table A (AFY):	25,000
SLO Co. SWP Cost Share:	0.616 percent
Total SLO Co. Cost Share:	$56,579,600
Financing Rate:	6 percent
Financing Term (Years):	40
SLO. Co. Annual Financing Cost:	$3,760,367.12
SLO. Co. Annual O&M Cost:	$218,187.20
Total SLO Co. Annual Cost:	$3,978,554.32

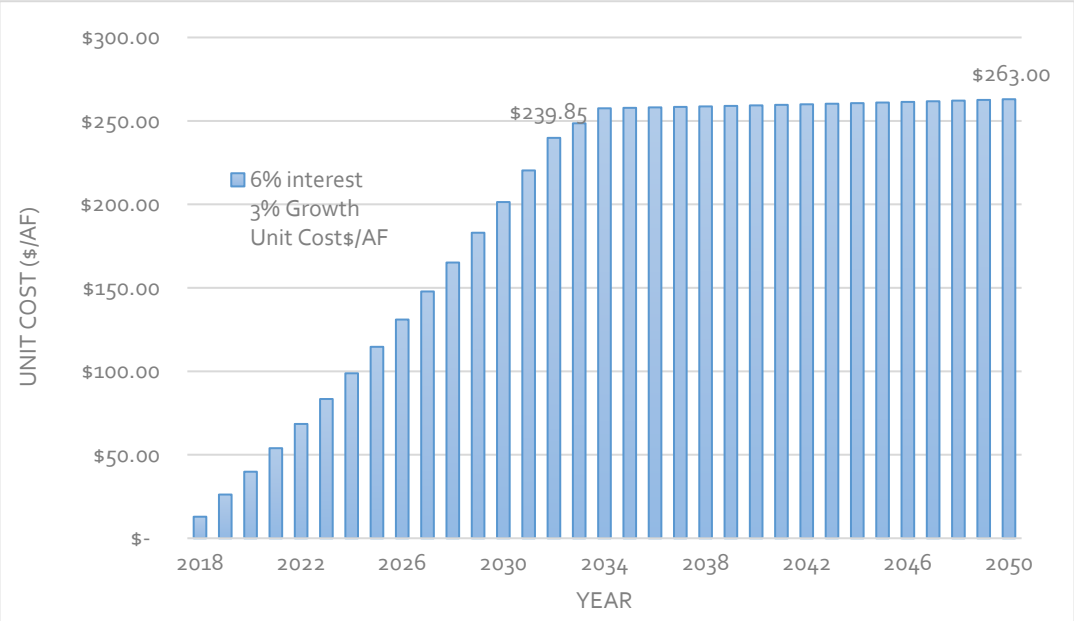

Figure 7.7 California WaterFix Unit Cost Component of State Water

Combined Net Present Value of California State Water

Combining the unit costs from DWR, CCWA, and WaterFix results in an aggregate unit cost for State Water. Figure 7.8 displays the relationship between projected costs as displayed in DWR's Bulletin 132 and the adjusted projected costs based on a realistic allocation of 4,830 ac-ft projected to the year 2050

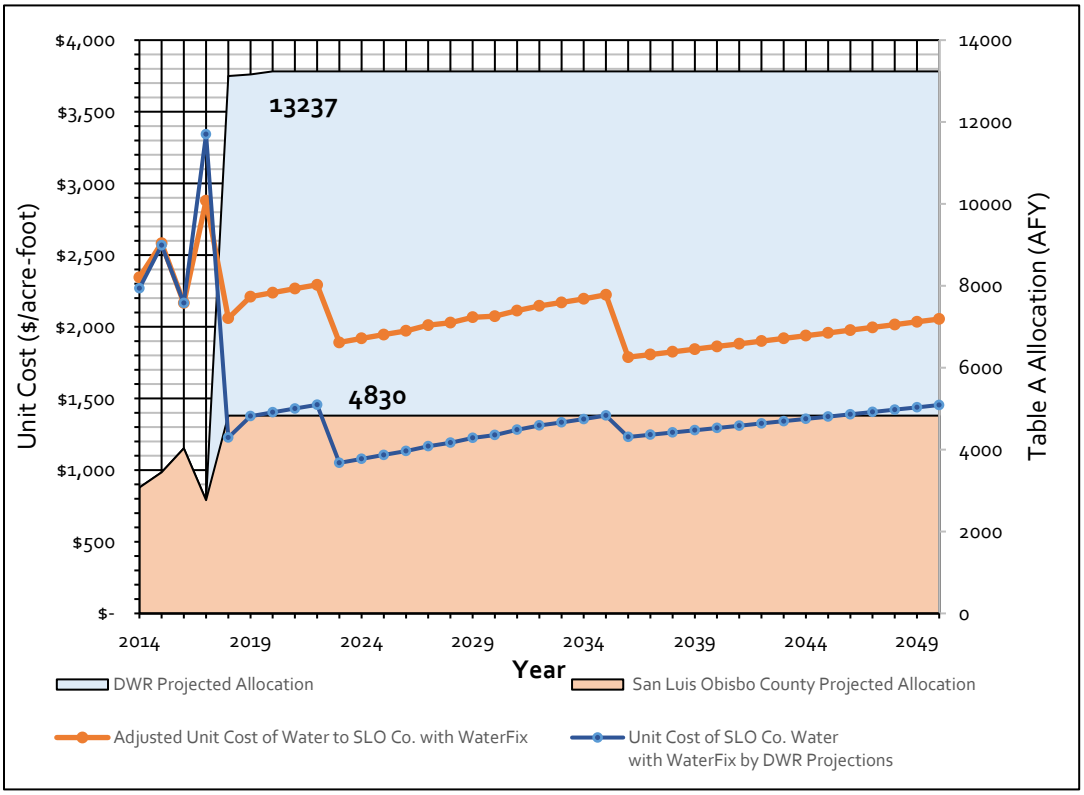

Figure 7.8 Cost of State Water versus Adjusted and Projected Allocations

The low allocation in 2016 results in a high unit costs in comparison to the normal allocations projected up to 2050. In 2022, the first decrease in unit cost is a result of the CCWA debt retirement while the second drop at year 2035 is from the retirement of the DWR capital construction cost for the State Water Project. The escalation in costs displayed between 2022 and 2035 represent the combination of both the CCWA cost projections and the California WaterFix ramp-up financial implementation. The latter increase between 2036 and 2050 includes the CCWA future unit cost as well as the inflation of the transportation O&M and the capital construction cost of the California WaterFix. The adjusted unit cost of State Water is greater than DWR's projected cost since the delivered Table A allocations of 4,830 AFY is much less than the contractual volume of 25,000 AFY. Table 7.3 displays the net present value calculation of the projected financing cost of imported State Water.

Table 7.3 Imported State Water Net Present Value

Component	Amount
Total Adjusted Allocation (ac-ft)	172,705
Total Adjusted Cost by DWR Projections	$369,895,044
Discount Rate	2 percent
Projected Year	2050
Net Present Unit Cost Per Adjusted Allocation	$1,558

The adjusted long-term net present value of State Water was approximately $1,558 per ac-ft in current day dollars. This value represents the cost of importing State Water to the City of Morro Bay over the course of the analysis period to 2050.

A separate analysis was performed that assessed the unit cost based on varying percentages of Table-A water allocated to Morro Bay. Figure 7.9 displays the fluctuation of unit costs with respect to Table-A allocation percentage. Shown in Figure 7.9, a 100 percent Table-A allocation results in a unit costs of $1,558 per ac-ft as calculated in Table 7.3 above. Given a reduced allocation, as shown ranging from 10 to 75 percent, the unit cost significantly increases due to the high fixed cost of State Water.

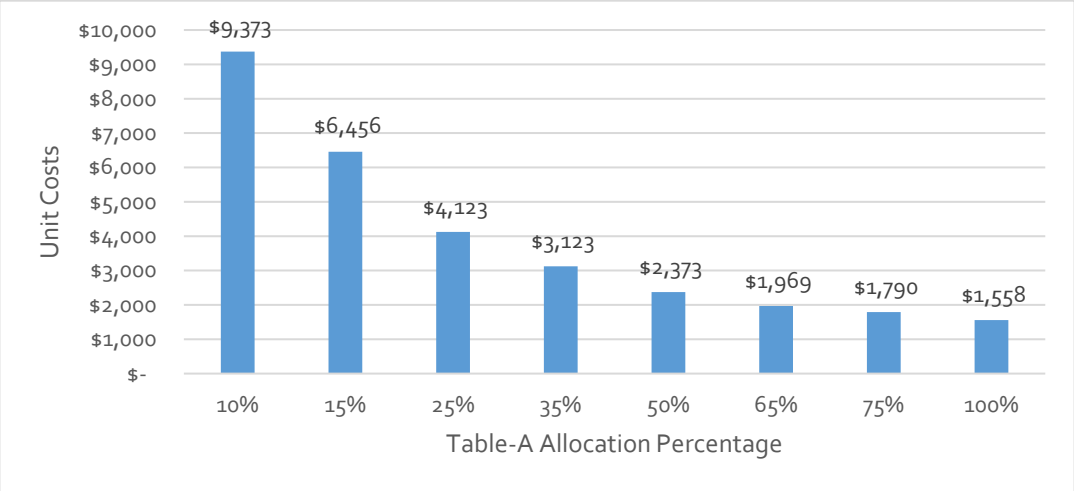

Figure 7.9 State Water Unit Cost versus Table-A Allocation Percentage

7.1.3 Alternative 2A: Morro Well Field with Salinity Treatment

7.1.3.1 Description of Alternative 2A

Alternative 2A (Figure 7.10) specifically treats groundwater extracted from the Morro Well field at a new brackish water reverse osmosis (BWRO) facility. The Morro Well Field has historically experienced seawater intrusion from over extraction and close proximity to the ocean. The existing water treatment facility has been in operation since the 1990s and is currently located in a flood zone and tsunami run-up area. For this alternative, it has been assumed that the existing facility will be completely replaced by the proposed BWRO facility at a new location and the waste product (brine) will be sent directly into the existing ocean outfall as shown in Figure 7.8. Alternative 2A would be exclusively fed by the Morro Well Field and once treated, finished water

will be sent by a new booster pump station to the Kings Tank prior to distribution. The process flow diagram for Alternative 2A is shown in Figure 7.11.

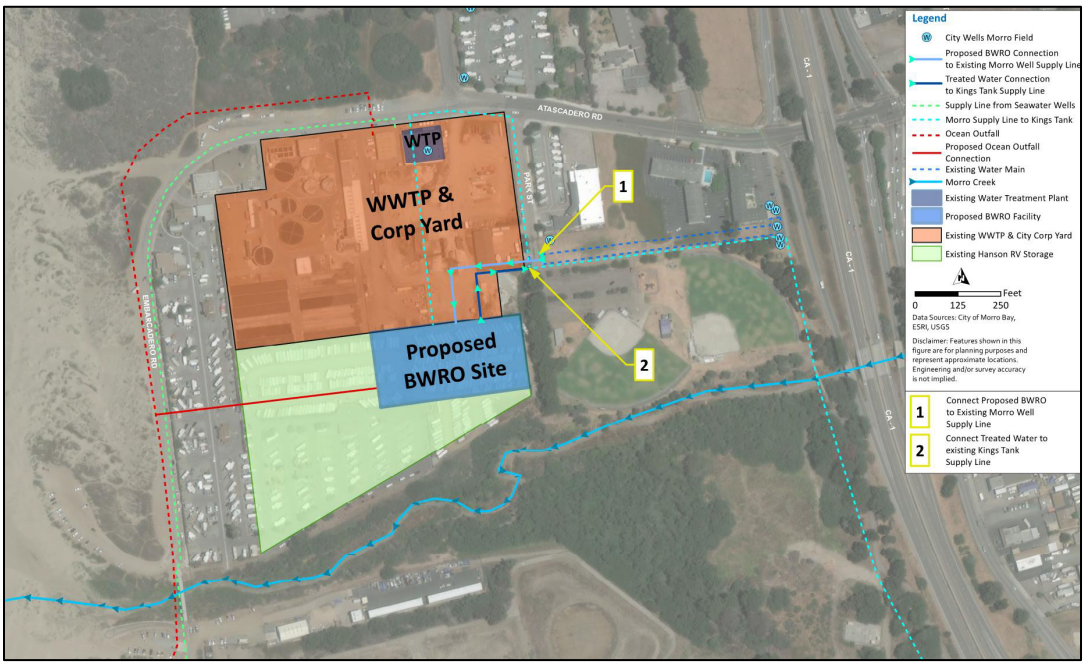

Figure 7.10 Overview of Alternative 2A

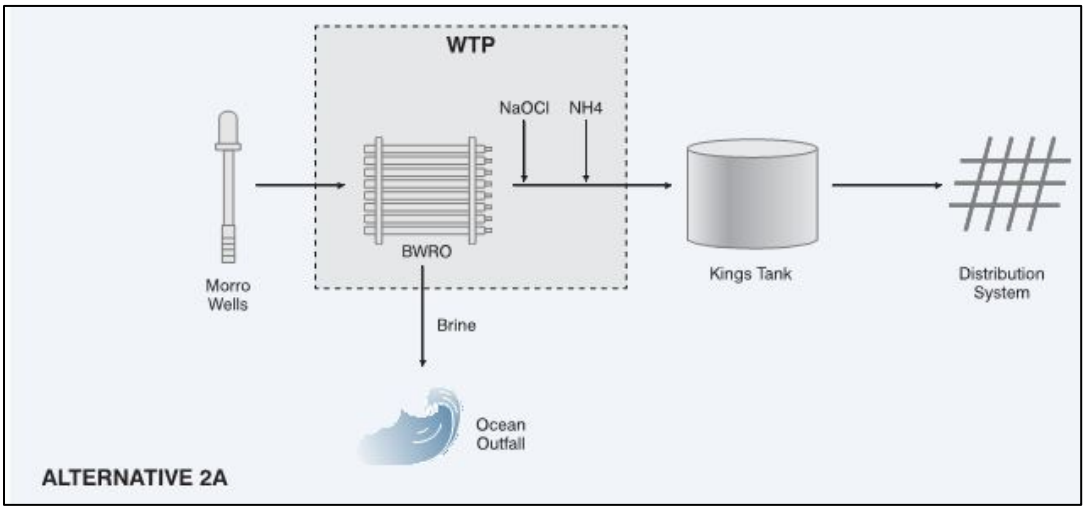

Figure 7.11 Alternative 2A Process Flow Diagram

7.1.3.2 Supply Yield and Availability

The total yield of Alternative 2A is equivalent to the total permitted Morro Well Field extraction volume of 581 AFY. The BWRO facility is preliminarily estimated to provide treated capacity to match the Morro Well field annual extraction. To meet the total demand for the City, Alternative 2A will be supplemented with imported State Water.

7.1.3.3 Implementation Considerations and Details

The proposed BWRO facility must be located outside of the 100-year flood plain as defined by the FEMA Firm Map, (Appendix C). Special Conditions of the Coastal Development Permit acknowledge that operation of the existing BWRO facility is at risk to flooding hazards. Significant improvements or replacement of the BWRO would require the facility to be relocated to mitigate threats of coastal hazards. Additionally, in comparison to National Oceanic and Atmospheric Administration (NOAA) sea-level rise mapping studies, the City of Morro Bay 2050 Sea Level Rise Scenario report, and the Sea Level Rise Adaptation Strategy Report the proposed location of Alternative 2A will not be affected by an increase in the sea-level elevation. Both the elevation and the vicinity of the proposed site are outside of areas with potential sea level rise hazards. As mentioned in Section 7.1.1, treated groundwater must be supplemented with imported State Water to meet the future demand.

7.1.3.4 Water Quality Considerations

As mentioned in Section 5.1.2.2, TDS concentrations from the extracted Morro Well Field have ranged from 400 to 1,000 mg/L and are required to be reduced prior to distribution. Fluctuating TDS levels may occur depending on seasonal drought conditions. The new BWRO facility would produce drinking water with TDS levels between 100 and 150 mg/l. A combination of BWRO treatment and blending with imported State Water (see Section 7.1.1.4) will reduce salinity concentrations in the City's water supply prior to storage and distribution.

7.1.3.5 Costs of Alternative 2A

The capital cost for Alternative 2A is estimated to be approximately $7.059 million, which includes the construction of treatment and conveyance facilities and soft costs. The amortized annual cost, which includes financed capital costs, annual O&M, and energy costs is approximately $898,500 per year. The energy consumption for Alternative 2A is approximately 3,340 kilowatt-hour per acre-foot (kWh)/ac-ft. The total annual costs per ac-ft of production yield results in a unit cost of $1,546 per ac-ft. Table 7.4 lists the financial components of Alternative 2A.

Table 7.4 Costs of Alternative 2A

Component	Cost
BWRO Facility	$5,450,000
Pump Station	$826,000
12-Inch Potable Water Pipeline	$540,000
12-Inch Brine Line	$243,000
Subtotal	**$7,059,000**
Amortized Capital Costs (30 years)	$445,558
Annual O&M Costs	$161,940
Annual Energy Costs	$291,000
Total Annual Costs	**$898,498.00**
Energy Consumption	3,339 kWh/ac-ft
Net Yield	581 AFY
Unit Cost	**$ 1,546 per AFY**

7.1.4 Alternative 2B: Morro Well Field with Nitrate Treatment

7.1.4.1 Description of Alternative 2B

Alternative 2B (Figure 7.12) includes treating extracted groundwater from the Morro Well Field at a new nitrate treatment facility. Nitrate concentrations in the Morro Well Field have exceeded the drinking water primary maximum contaminant level (MCL) as a result of agricultural runoff. This option assumes that TDS is not the primary concern for the Morro Well Field and high nitrates are the leading constituent. From the new treatment facility, finished water will be pumped by a new booster pump station to the existing Kings Tank prior to distribution. Raw water will be sourced exclusively from the Morro Wells and eventually combine with imported State Water at the Kings Tank. The process flow diagram for Alternative 2B is shown in Figure 7.13.

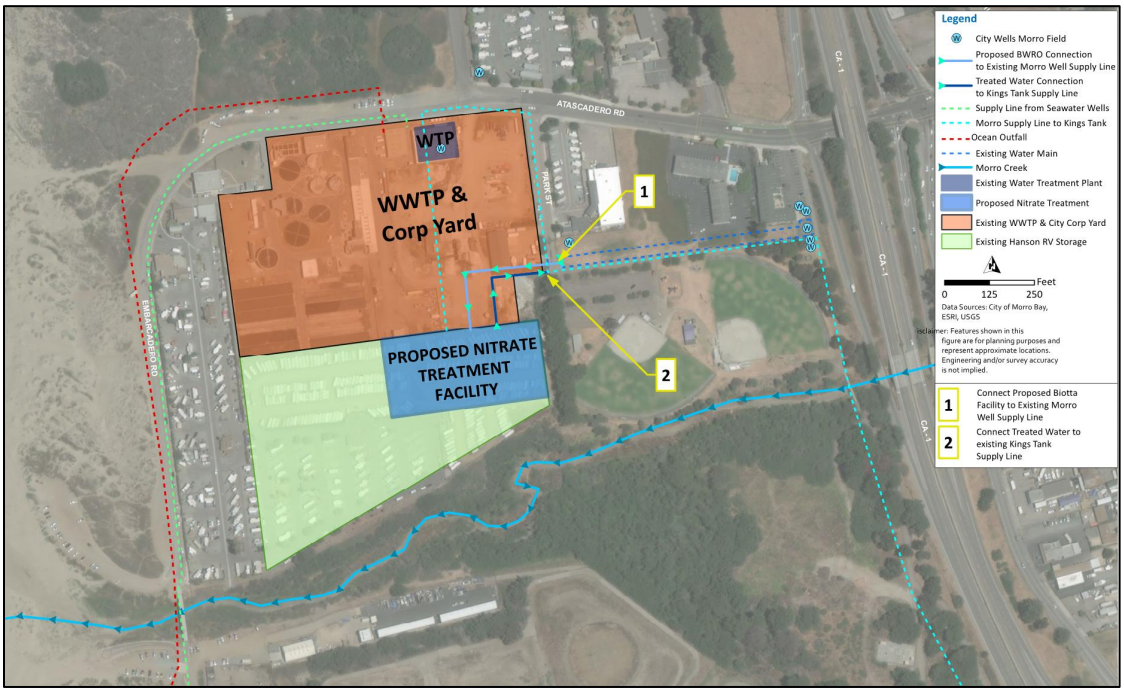

Figure 7.12 Overview of Alternative 2B

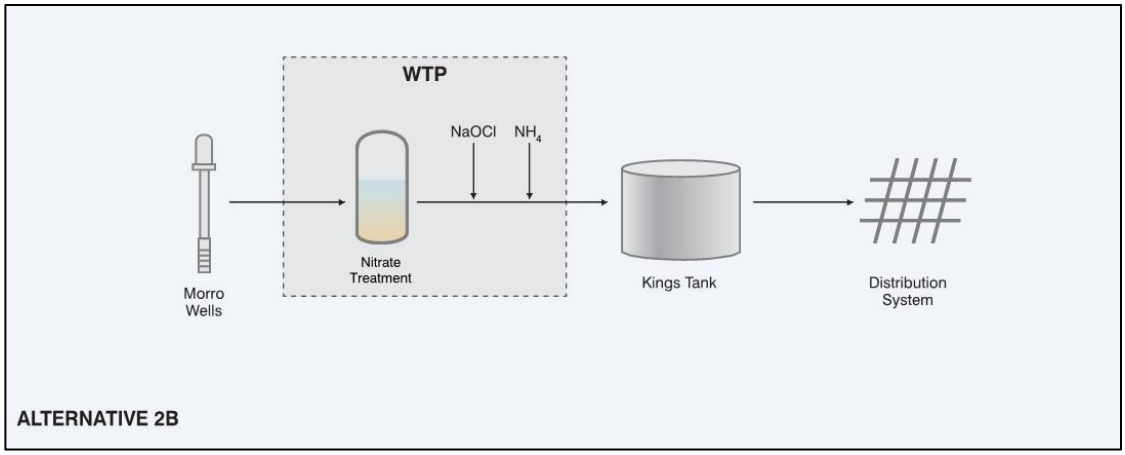

Figure 7.13 Alternative 2B Process Flow Diagram

7.1.4.2 Supply Yield and Availability

The total yield for Alternative 2B is comparable to Alternative 2A. The production of the nitrate treatment facility is limited to the annual permitted capacity of the Morro Well Fields of 581 AFY.

7.1.4.3 Implementation Considerations

The proposed nitrate treatment facility must be located outside of the 100-year flood plain as defined by the FEMA Firm Map, (Appendix C). Special Conditions of the Coastal Development Permit acknowledge that operation of the existing BWRO facility is at risk to flooding hazards and must be relocated to mitigate coastal flood threats. Additionally, the proposed new treatment facility will not be affected by possible sea-level rise limits as shown in Appendix C and in comparison to the City of Morro Bay 2050 Sea Level Rise Scenario report and the Sea-Level Rise Adaptation Strategy Report. The elevation and the vicinity of the proposed site are located outside potential sea level rise hazards. The capacity of the Morro Wells alone do not meet the future demand for the City. As mentioned in Section 7.1.1, treated groundwater must be supplemented with imported State Water to meet the City's future design demand for this alternative.

7.1.4.4 Water Quality Considerations

Nitrate levels in raw Morro Well Field groundwater have been recorded at concentrations up to 220 mg/L. The proposed facility will reduce the nitrate concentrations below the primary MCL and send finished water directly to the Kings Tank Prior to distribution. Blending operations will be determined upon operation of the facility and preferences to the City of Morro Bay. This alternative assumes that nitrate concentrations exceed drinking water primary MCL limits and therefore requires treatment following extraction. Alternative 2B does not provide means for TDS removal and relies on SWP blending in the Kings Tank to lower TDS, if necessary, to meet secondary MCLs.

7.1.4.5 Costs of Alternative 2B

The capital cost for Alternative 2B is estimated to be approximately $6.136 million, which includes construction of treatment and conveyance facilities and soft costs. The amortized annual cost includes financed capital, annual O&M and energy cost is approximately $718,000 per year. The energy consumption for Alternative 2A is about 2,249 kWh/ac-ft. The total annual

costs and production yield result in a unit cost $1,236 per ac-ft. Table 7.5 lists the financial components of Alternative 2B.

Table 7.5 Costs of Alternative 2B

Component	Cost
Nitrate Treatment Facility	$4,500,000
Booster Pump Station	$826,000
12-inch Potable Water Pipeline	$810,000
Subtotal	**$6,136,000**
Amortized Capital Costs (30 years)	$385,238
Annual O&M Costs	$136,800
Annual Energy Costs	$196,000
Total Annual Costs	**$718,038**
Energy Consumption	2,249 kWh/ac-ft
Net Yield	581 AFY
Unit Cost	**$1,236/AFY**

7.1.5 Alternative 2C: Morro and Chorro Well Fields with Salinity Treatment

7.1.5.1 Description of Alternative 2C

Alternative 2C (Figure 7.14) includes treating extracted groundwater from both the Morro and the Chorro groundwater basins at a centralized salinity treatment facility similar to Alternative 2A. A transmission pipeline and booster pump station will be required to convey extracted raw groundwater from the Chorro wells to the new treatment facility. Finished water will be sent via booster pump station to the Kings Tank prior to distribution. The process flow diagram for Alternative 2C is shown in Figure 7.15.

Figure 7.14 Overview of Alternative 2C

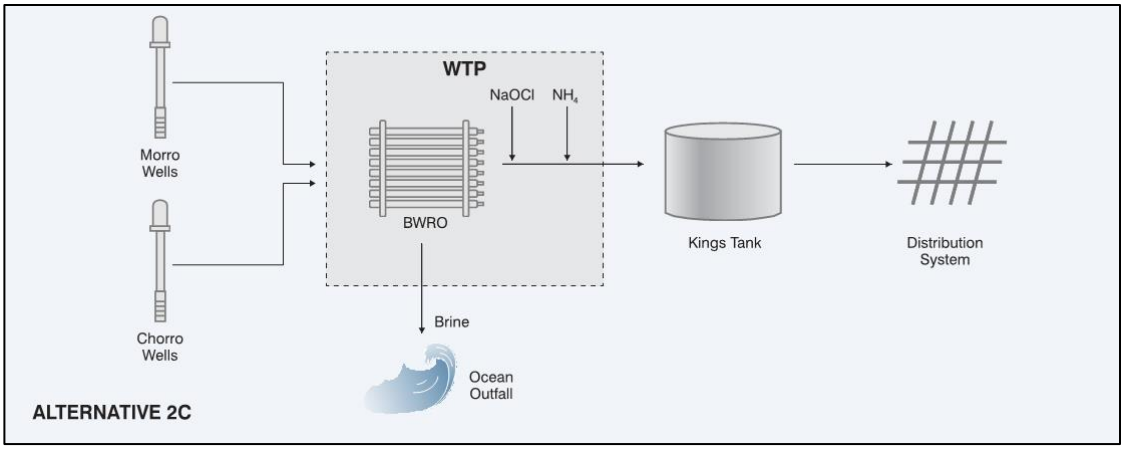

Figure 7.15 Alternative 2C Process Flow Diagram

7.1.5.2 Supply Yield and Availability

The Morro and Chorro groundwater basins have permitted yields of 581 and 1,142.5 ac-ft per year, respectively. The groundwater extraction permit for the Chorro well field states that extraction is only permitted when the adjacent Chorro Creek discharge is at least 1.4 cubic feet per second (cfs). Therefore, the combined groundwater yield of 1,723.5 AFY is only available when the Chorro Creek discharge is above the permit limit. Records show that during drought conditions, Chorro Creek can be below the 1.4 cfs threshold for periods of up to 3 months. During creek low flow periods, the availability of supply is limited to only the Morro Well Field for this alternative. Morro Bay has appropriative rights to both groundwater basins, meaning extraction volumes would follow that of riparian right owners. Even with an allocation set by the state groundwater permit, Morro Bay's appropriative water right to the Morro groundwater basin can potentially limit yield to below the permitted allocation. For the combined treatment of yields from both the Morro and Chorro well fields, facilities must be designed to treat the well capacity of both basins combined and have the range to treat water from only Morro if conditions dictate.

7.1.5.3 Implementation Considerations

Similar to Alternative's 2A and 2B, the proposed treatment facility must be located outside of the 100-year flood plain as defined by the FEMA Firm Map, (Appendix C). Additionally, the proposed (new) treatment facility will not be affected by possible sea-level rise limits as shown in Appendix C and in comparison to the City of Morro Bay 2050 Sea Level Rise Scenario report and the Sea Level Rise Adaptation Strategy Report. The Chorro wells require electrical and instrumentation improvements as well as continuous monitoring of the Chorro Creek streamflow rate for well operation to be within the limits of the permit.

7.1.5.4 Water Quality Considerations

Both the Morro and Chorro groundwater basins have water quality records showing high concentrations of nitrates and TDS. Alternative 2C assumes that TDS is the leading constituent exceeding drinking water standards. The combination of groundwater monitoring, treatment, and finished blending will need to be continually monitored and adjusted to maintain water quality levels to meet drinking water standards.

7.1.5.5 Costs of Alternative 2C

The capital costs for Alternative 2C is approximately $27.5 million. This cost includes a combined capacity salinity treatment facility, conveyance costs, and existing infrastructure improvements. The annual costs include the amortized capital costs, annual O&M and energy costs, totaling approximately $3.012 million per year. Alternative 2C energy consumption is estimated to be 3,191 kWh per ac-ft. The resulting unit costs of water from the estimated total yield capacity of both well fields is $1,748 per acre foot. Additionally, if Chorro Creek streamflow is less than 1.4 cfs, the Chorro well raw water pumping station will be offline and the treatment facility would run at a reduced capacity. Under these low streamflow conditions, the decreasing plant energy consumption creates a lower energy use and cost, but a higher unit cost of production at $4,519/ac-ft. Table 7.6 lists the financial components of Alternative 2C.

Table 7.6 Costs of Alternative 2C

Component	Chorro Online	Chorro Offline
BWRO Treatment Facility	$16,170,000	$16,170,000
Well E&I and Infrastructure Updates	$3,000,000	$3,000,000
Booster Pump Stations	$2,970,000	$2,970,000
12-Inch Brine Line	$243,000	$243,000
12-Inch Water Pipeline	$540,000	$540,000
10-inch Water Pipeline	$4,050,000	$4,050,000
8-inch Water Pipeline	$486,000	$486,000
Subtotal	**$27,459,000**	**$27,459,000**
Amortized Capital Costs (30 years)	$1,680,655	$1,680,655
Annual O&M Costs	$506,660	$497,8600
Annual Energy Costs	$825,000	$447,000
Total Annual Costs	**$3,012,315**	**$2,625,515**
Energy Consumption	3,191 kWh/ac-ft	5,129 kWh/ac-ft
Net Yield	1,723.5 AFY	581 AFY
Unit Cost	**$1,748/AFY**	**$4,519/AFY**

7.1.6 Alternative 2D: Morro and Chorro Well Fields with Nitrate Treatment

7.1.6.1 Description of Alternative 2D

Alternative 2D (Figure 7.16) includes treating extracted groundwater from both the Morro and the Chorro groundwater basins at a centralized nitrate treatment facility similar to Alternative 2B. This facility would be located at the Kings Tank site as shown in Figure 7.16. A transmission pipeline will be required to convey extracted raw groundwater from the Chorro wells to the new treatment facility. Finished water will be stored in the Kings Tank for storage prior to distribution. The process flow diagram of Alternative 2D is shown in Figure 7.17. This alternative assumes that TDS removal is not required.

7.1.6.2 Supply Yield and Availability

The Morro and Chorro groundwater basins have permitted yields of 581 and 1,142.5 ac-ft per year respectively with a combined yield of 1,723.5 AFY. Limitations of the groundwater

extraction permit for the Chorro well field state extraction is only allowed when the adjacent Chorro Creek discharge is at least 1.4 cfs. The proposed treatment facilities in Alternative 2D must be designed to treat the well capacity of both basins combined and have the range to treat water from only Morro if conditions require.

Figure 7.16 Overview of Alternative 2D

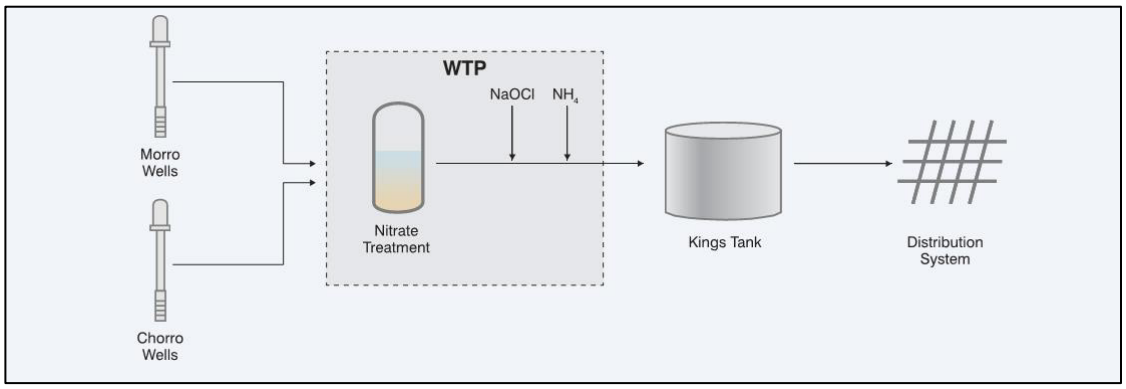

Figure 7.17 Alternative 2D Process Flow Diagram

7.1.6.3 Implementation Considerations

The proposed location for the Alternative 2D nitrate treatment facility is at the Kings Tank site, which is outside the area of coastal hazards. The Chorro groundwater basin is regulated by the streamflow within Chorro Creek. Constant monitoring of the flow rate will be required to adhere to the limitations set forth by the well permit. Alternative 2D will supply nearby private land owners along Quintana and Chorro Creek Road that currently receive drinking water directly from the Chorro Well supply line with treated drinking water supplied from the Kings Tank. This

will require an additional pipeline to be constructed parallel to the Chorro Well supply line to the Kings Tank.

7.1.6.4 Water Quality Considerations

Alternative 2D assumes nitrates exceedances are the dominant contaminant in the groundwater. As discussed in Section 7.1.4.4, both the Morro and Chorro well fields have high levels of nitrate and TDS. This centralized nitrate facility will be able to only treat the nitrate constituents within the raw groundwater. TDS levels, if present, are likely be reduced with a blending strategy or alternative treatment methods.

7.1.6.5 Costs of Alternative 2D

The capital costs for Alternative 2D is approximately $17.512 million. This cost includes a combined groundwater capacity nitrate treatment facility, conveyance costs, and existing infrastructure improvements. The annual costs include the amortized capital costs, annual O&M and energy costs totaling $2.041 million per year. Alternative 2D energy consumption is estimated to be 2,340 kWh per ac-ft. The resulting unit costs of water from the estimated total yield capacity of both well fields is $1,184 per acre foot. If the Chorro well field is offline due to Chorro Creek streamflow restrictions, the unit cost of Alternative 2D is $3,016. Table 7.7 lists the cost components of Alternative 2D.

Table 7.7 Costs of Alternative 2D

Component	Chorro Online	Chorro Offline
Groundwater Nitrate Treatment System	$12,000,000	$12,000,000
Well E&I and Infrastructure Updates	$3,000,000	$3,000,000
Booster Pump Stations	$1,486,000	$1,486,000
12-inch Water Pipeline	$540,000	$540,000
8-inch Water Pipeline	$486,000	$486,000
Subtotal	**$17,512,000**	**$17,512,000**
Amortized Capital Costs (30 years)	$1,087,749	$1,087,749
Annual O&M Costs	$348,640	$348,640
Annual Energy Costs	$605,000	$316,000
Total Annual Costs	**$2,041,389**	**$1,752,389**
Energy Consumption	2,340 kWh/ac-ft	3,625.93 kWh/ac-ft
Net Yield	1,723.5 AFY	581 AFY
Unit Cost	**$1,184 per AFY**	**$3,016 per AFY**

7.1.7 Alternative 2E: Morro and Chorro Well Fields with Nitrate and Salinity Treatment

7.1.7.1 Description of Alternative 2E

Alternative 2E (Figure 7.18) includes treating extracted groundwater from both the Morro and the Chorro groundwater basins at individual salinity and nitrate treatment facilities, respectively. The salinity treatment facility will be located in the same location as Alternative 2A, and the nitrate facility will be located at the Kings Tank site, similar to Alternative 2D. Figure 7.11 shows the facilities associated with Alternative 2E. A transmission pipeline and pump station will be required to convey extracted raw groundwater from the Chorro wells to the new treatment

facility whereas the majority of the conveyance for the Morro Wells will utilize existing infrastructure. All treated groundwater will be sent to the Kings Tank prior to distribution. The process flow diagram of Alternative 2E is shown in Figure 7.19.

Figure 7.18 Overview of Alternative 2E

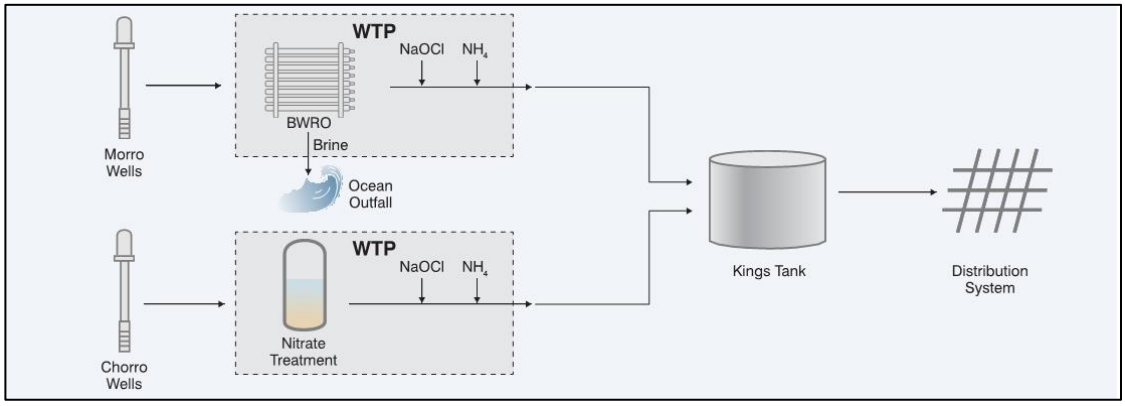

Figure 7.19 Alternative 2E Process Flow Diagram

7.1.7.2 Supply Yield and Availability

The Morro and Chorro groundwater basins have permitted yields of 581 and 1,142.5 ac-ft per year respectively with a combined yield of 1,723.5 AFY. Limitations of the groundwater extraction permit for the Chorro well field state extraction is only allowed when the adjacent Chorro Creek discharge is at least 1.4 cfs. As shown in the process flow diagram (Figure 7.19), the Chorro well field is solely treated by the nitrate facility. If Chorro well field is offline due to low flows in Chorro Creek, the proposed nitrate facility will be non-operational as well.

7.1.7.3 Implementation Considerations

The location of the BWRO treatment facility will fall within similar design constraints as Alternatives 2A through Alternatives 2D. In order to mitigate coastal hazards, the BWRO facility must be relocated from its current location. With the relocation of these facilities, no impacts are seen by potential sea-level rise. Since there are two treatment facilities proposed, the nitrate treatment plant will be located at the Kings Tank site. Alternative 2E will also supply nearby private land owners along Quintana and Chorro Creek Road similar to Alternative 2D.

7.1.7.4 Water Quality Considerations

Alternative 2E assumes nitrates and TDS concentrations exceed primary and secondary MCLs in the Morro and Chorro basins, respectively. As mentioned in Sections 5.1.2.2 and 5.1.2.3, the Morro ground water basin has historically had high levels of both nitrates and TDS while the Chorro has shown high concentrations of nitrates. The ability to fully treat both groundwater basins also allows for the flexibility of blending the sources together to meet drinking water standards and supply the City's total water demand.

7.1.7.5 Costs of Alternative 2E

The capital costs for Alternative 2E is approximately $22.875 million. This cost includes a combined capacity nitrate and salinity treatment facility, conveyance costs, and existing infrastructure improvements. The annual costs include the amortized capital costs, annual O&M and energy costs is $2.356 million per year. Alternative 2E energy consumption is estimated to be 2,355 kWh per ac-ft. The resulting unit costs of water from the estimated total yield capacity of both well fields is $1,472 per acre foot. If the Chorro well field is offline due to Chorro Creek streamflow restrictions, the unit cost of Alternative 2E is $4,056. Table 7.8 lists the financial components of Alternative 2E.

Table 7.8 Costs of Alternative 2E

Component	Chorro Online	Chorro Offline
Groundwater Nitrate Treatment System	$12,000,000	$12,000,000
Groundwater Salinity Treatment System	$5,450,000	$5,450,000
Well E&I and Infrastructure Updates	$3,000,000	$3,000,000
Booster Pump Stations	$1,156,000	$1,156,000
12-inch Brine Pipeline	$243,000	$243,000
12-inch Water Pipeline	$540,000	$540,000
8-inch Water Pipeline	$486,000	$486,000
Subtotal	**$22,875,000**	**$22,875,000**
Amortized Capital Costs (30 years)	$1,436,358	$1,436,358
Annual O&M Costs	$491,180	$491,180
Annual Energy Costs	$609,000	$429,000
Total Annual Costs	**$2,536,538**	**$2,356,538**
Energy Consumption	2,355 kWh/ac-ft	4,923 kWh/ac-ft
Net Yield	1,723.5 AFY	581 AFY
Unit Cost	**$1,472/AFY**	**$4,056/AFY**

7.1.8 Alternative 3A: Chorro Well Field Streamflow Augmentation

7.1.8.1 General Description

Alternative 3A (Figure 7.20) consists of utilizing treated water from the WRF project's secondary effluent as streamflow augmentation to satisfy the discharge limitations stated in the Chorro well field groundwater permit. The treated wastewater water undergoes filtration and disinfection meeting recycled water standards (i.e. Title 22) at the WRF site and is discharged directly to Chorro Creek. Approximately half of the total effluent flow will pass through RO at the WRF to reduce TDS below the TMDL for Chorro Creek. The discharge locations are specific to the Chorro groundwater basin permit requirements. Therefore, a new permanent streamflow gauge is required to be installed downstream of the Chorro well field. Having the appropriate flow within Chorro Creek allows for normal use of the Chorro Well field. Extracted groundwater will be conveyed directly to the Kings Tank prior to distribution. The process flow diagram of Alternative 3A can is shown in Figure 7.21.

Figure 7.20 Overview of Alternative 3A

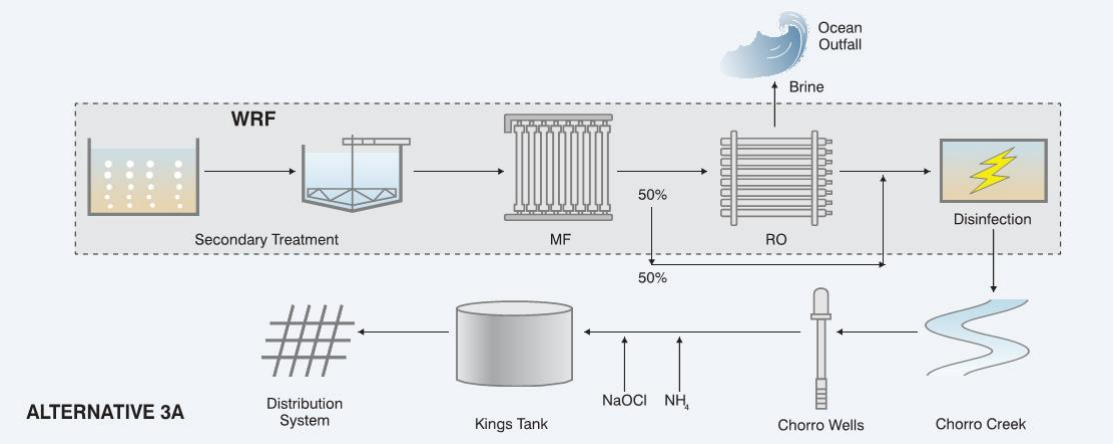

Figure 7.21 Alternative 3A Process Flow Diagram

7.1.8.2 Supply Yield and Availability

The streamflow limitations as defined in the Chorro groundwater permit limit access to groundwater pumping and will be governed by the total discharge in Chorro Creek. Having Chorro Creek discharge augmented to meet the required discharge requirements of Chorro Creek, yields a total volume of 1,142.5 acre foot per year to the City from the Chorro well field. Utilizing only the Chorro well field as the domestic supply, groundwater yields are required to be supplemented with imported State Water.

7.1.8.3 Implementation Considerations

Currently the Chorro well field is off line due to the stream flow limitations and high nitrate concentrations, and electrical and mechanical conditions of the wells. Prior to utilizing the Chorro groundwater basin as a supply source, the Chorro well field will need significant electrical and mechanical rehabilitation due to the age of the infrastructure. Additionally, a new streamflow gauge will need to be installed downstream of the Chorro Well field in order to continuously monitor any draw down or direct affects groundwater pumping may have on Chorro Creek.

7.1.8.4 Water Quality Considerations

The Chorro groundwater basin has historically contained nitrate concentrations in excess of the primary MCL from agricultural fertilizers. As mentioned in Section 5.1.2.3, nitrate levels have ranged from 25 to 100 mg/L. Currently, there is no treatment post extraction, therefore nitrate levels will need to be monitored continually during groundwater pumping to effectively blend this source with State Water or treated groundwater. The treated effluent from the WRF will improve the water quality in the Chorro groundwater basin. However, additional groundwater modeling is needed.

7.1.8.5 Costs of Alternative 3A

The capital costs for Alternative 3A is approximately $20.038 million. This cost includes the tertiary and RO treatment facility (any costs in addition to the secondary treatment facility), pumping and conveyance costs, and existing infrastructure improvements. The annual costs include the amortized capital costs, annual O&M and energy costs which total $1.9 million per year. As mentioned, these costs are in addition to the existing WRF project construction costs.

Alternative 3A energy consumption is estimated to be 2,346 kWh per ac-ft. The resulting unit costs of water from the estimated total yield capacity of the Chorro well fields is $1,663 per acre foot. Table 7.9 lists the financial components of Alternative 3A.

Table 7.9 Costs of Alternative 3A

Component	Cost
Purified Water Treatment Facility	$9,869,000
Well Mechanical and Electrical and Infrastructure Improvements	$3,000,000
Storage Tanks	$938,000
Booster Pump Stations	$660,000
Jack and Bore	$788,000
Streamflow Gauge	$58,000
12-inch Purified Water Pipeline	$4,455,000
8-inch Water Pipeline	$270,000
Subtotal	**$20,038,000**
Amortized Capital Costs (30 years)	$1,210,965
Annual O&M Costs	$286,470
Annual Energy Costs	$402,000
Total Annual Costs	**$1,899,435**
Energy Consumption	2,346 kWh/ac-ft
Net Yield	1,142 AFY
Unit Cost	**$1,663/AFY**

7.1.9 Alternative 3B: Chorro Well Field Streamflow Augmentation with Nitrate Treatment

7.1.9.1 Description of Alternative 3B

Alternative 3B (Figure 7.22) consists of utilizing recycled water produced from the WRF project secondary effluent as streamflow augmentation to satisfy the streamflow limitations in Chorro Creek similar to Alternative 3A. Extracted groundwater will be conveyed directly to a new nitrate treatment facility prior to the Kings Tank for storage. The process flow diagram for Alternative 3B is shown in Figure 7.23. While Alternative 3A assumed that the drinking water standards for nitrate could be met without post treatment, Alternative 3B assumes that water extracted at the Chorro Well Field would be treated prior to reaching the Kings Tank.

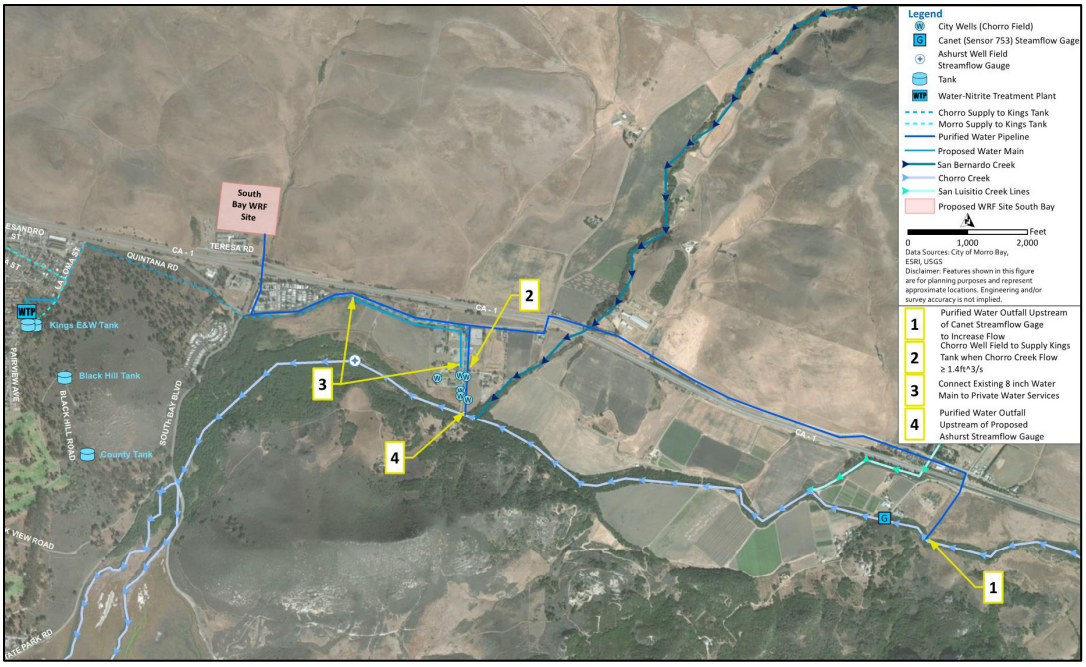

Figure 7.22 Overview of Alternative 3B

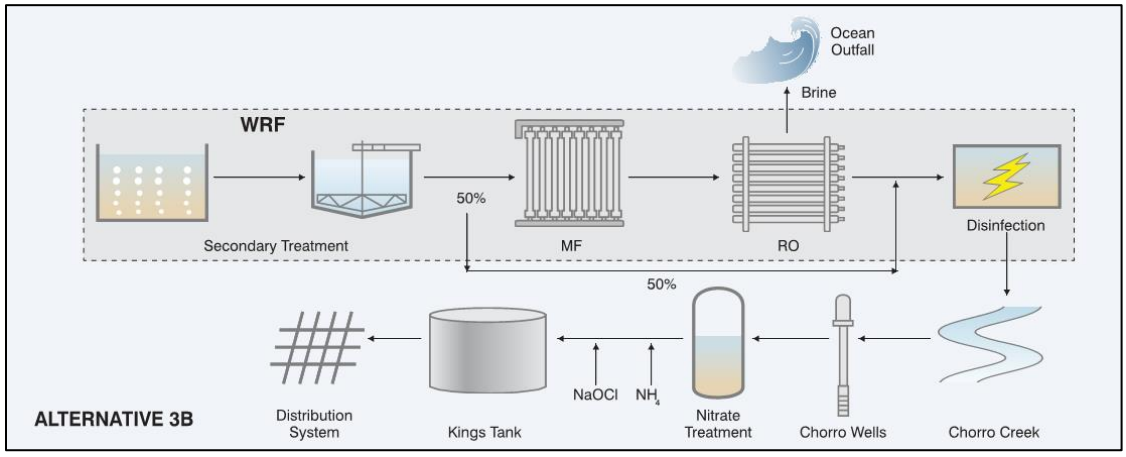

Figure 7.23 Alternative 3B Process Flow Diagram

7.1.9.2 Supply Yield and Availability

The streamflow limitations as defined in the Chorro groundwater permit limit access to groundwater pumping and will be governed by the total discharge in Chorro Creek. Having Chorro creek discharge augmented to meet the required discharge requirements of Chorro Creek, yields a total volume of 1,142.5 AFY to the City from the Chorro well field. Utilizing only the Chorro well field as the domestic supply, groundwater yields are required to be supplemented with imported State Water.

7.1.9.3 Implementation Considerations

Currently the Chorro well field is off line due to the stream flow limitations and high nitrate concentrations as stated in Section 7.1.7.2. Alternative 3B will also supply nearby private land

owners along Quintana and Chorro Creek Road that currently receive drinking water directly from the Chorro Well supply line with treated drinking water supplied from the Kings Tank. This will require an additional pipeline to be constructed parallel to the Chorro well supply line to the Kings tank. The proposed site of the nitrate treatment facility is outside of FEMA flood zones and coastal hazards area.

7.1.9.4 Water Quality Considerations

As mentioned in Section 5.1.2.3, nitrate levels have ranged from 25 to 100 mg/L in the Chorro groundwater basin. The proposed nitrate treatment facility will be downstream of the Chorro well field in order to treat and reduce nitrate levels from the extracted groundwater. Finished water will be stored in the Kings Tank prior to distribution. Similar to Alternative 3A, an additional 8-inch pipeline will be extended to properties adjacent to the Chorro well field to provide treated drinking water.

7.1.9.5 Costs of Alternative 3B

The capital cost for Alternative 3B is approximately $25.924 million. This cost includes the advanced treatment and nitrate treatment facilities, pumping and conveyance costs, and existing infrastructure updates. The annual costs include the amortized capital costs, annual O&M and energy costs which total approximately $2.578 million per year. Alternative 3B energy consumption is estimated to be 3,245 kWh per ac-ft. The resulting unit costs of water from the estimated total capacity of the Chorro well field is $2,258 per acre foot. Table 7.10 lists the financial components for Alternative 3B.

Table 7.10 Costs of Alternative 3B

Component	Cost
Purified Water Treatment Facility	$9,869,000
Groundwater Nitrate Treatment Facility	$5,400,000
Well Mechanical and Electrical and Infrastructure Improvements	$3,000,000
Storage Tanks	$938,000
Booster Pump Stations	$660,000
Jack and Bore	$788,000
Streamflow Gauge	$58,000
12-inch Purified Water Pipeline	$4,455,000
8-inch Water Pipeline	$756,000
Subtotal	**$25,924,000**
Amortized Capital Costs (30 years)	$1,588,865
Annual O&M Costs	$433,710
Annual Energy Costs	$556,000
Total Annual Costs	**$2,578,575**
Energy Consumption	3,245 kWh/ac-ft
Net Yield	1,142 AFY
Unit Cost	**$2,258/AFY**

7.1.10 Alternative 4A: Morro Well Field IPR by Groundwater Injection

7.1.10.1 Description of Alternative 4A

Alternative 4A (Figure 7.24) consists of utilizing purified water produced by the WRF project for injection into the Morro Well Field for Indirect Potable Reuse (IPR). The purified water at the WRF will undergo complete advanced treatment consisting of ultrafiltration (UF), reverse osmosis (RO) and ultra-violet light with advanced oxidation process (UVAOP). The injection wells will be located upstream of the existing Morro groundwater wells and east of the Highway 1 Bridge and will ensure the appropriate retention times prior to extraction to meet State potable water reuse regulations. Once extracted from the existing Morro Wells, the water will undergo the City of Morro Bay's existing treatment and distribution operating procedures as necessary. The process flow diagram for Alterantive 4A is shown in Figure 7.25.

Figure 7.24 Overview of Alternative 4A

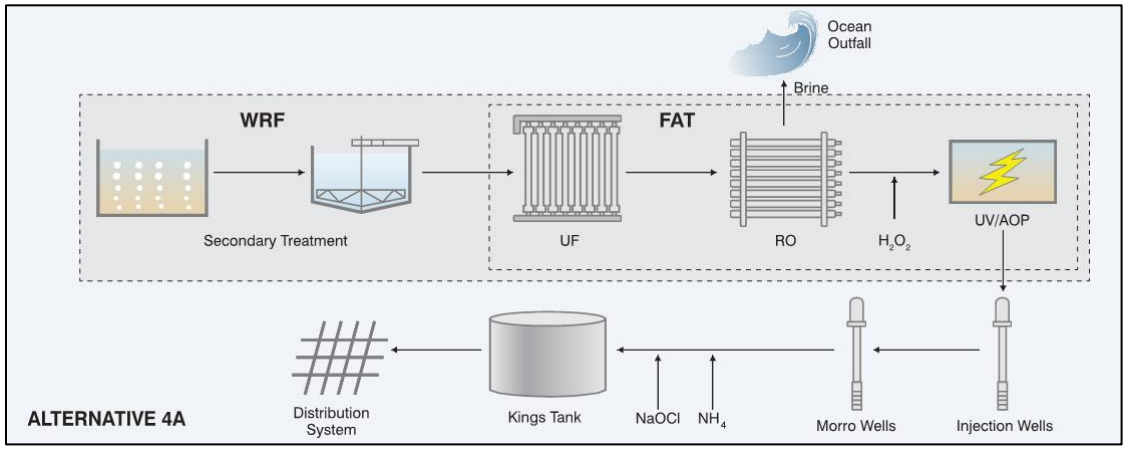

Figure 7.25 Alternative 4A Process Flow Diagram

7.1.10.2 Supply Yield and Availability

The supply yield of Alternative 4A is limited by the allowed volume of water extracted from the Morro Well Field. A preliminary IPR study was performed by GSI Water Solutions and their findings suggested that a maximum extraction volume of 943 AFY would limit sea water intrusion effects on the Morro groundwater basin. To meet the City's future demand, Alternative 4A yield, would be supplemented by imported State Water.

7.1.10.3 Implementation Considerations

The groundwater injection wells would be located east of the Morro well field near Morro Creek. Once groundwater is extracted current water distribution operations would be implemented as necessary. The injection wells and existing Morro wells fall within the Morro Creek 100-year flood plain. There are also no coastal hazards as displayed in the NOAA sea-level rise model results and the City of Morro Bay 2050 Sea Level Rise Scenario report and the Sea-Level Rise Adaptation Strategy Report. The existing WTP and Morro Well Field are not within the limits of sea level rise hazards as shown in both studies. A transmission pipeline will need to be constructed parallel to Highway 1 and will require ROW acquisition for the necessary easements.

7.1.10.4 Water Quality Considerations

As mentioned in Alternative 2A through 2E, the Morro well field is vulnerable to seawater intrusion. Section 5.1.2.2 discusses that the Morro well field has historically had TDS concentrations ranging from 220 to 1000 mg/L. Alternative 4A does not include a proposed treatment facility following groundwater extraction, therefore it is advised TDS levels should be closely monitored for exceedances and any TDS treatment will be performed at the existing BWRO treatment facility.

7.1.10.5 Costs of Alternative 4A

The capital costs for Alternative 4A is approximately $26.405 million. This cost includes the advanced treatment facility, pumping and conveyance costs, and injection wells. The annual costs include the amortized capital costs, annual O&M and energy costs totaling $2.259 million per year. Alternative 4A energy consumption is estimated to be 1,912 kWh per ac-ft. The resulting unit costs of water from the estimated total capacity is $2,396 per acre foot. Table 7.11 lists the financial components of Alternative 4A.

Table 7.11 Cost of Alternative 4A

Component	Cost
Advanced Treatment Facility	$9,065,000
Injection Wells	$12,300,000
Booster Pump Station	$990,000
12-Inch Purified Water Pipeline	$4,050,000
Subtotal	**$26,405,000**
Amortized Capital Costs (30 years)	$1,542,986
Annual O&M Costs	$445,920
Annual Energy Costs	$270,500
Total Annual Costs	**$2,259,406**
Energy Consumption	1,912 kWh/ac-ft
Net Yield	943 ac-ft
Unit Cost	**$2,396/AFY**

7.1.11 Alternative 4B: Morro Well Field IPR by Groundwater Injection with Salinity Treatment

7.1.11.1 Description of Alternative 4B

Alternative 4B (Figure 7.26) includes the same infrastructure as Alternative 4A with the exception of a new salinity treatment facility to treat extracted groundwater from the Morro groundwater basin. In the past, Morro groundwater basin has experienced seawater intrusion issues therefore increasing TDS levels in the extracted groundwater. The new salinity treatment facility will be located with the same layout and infrastructure as Alternative 2A. The process flow diagram for Alternative 4A is shown in Figure 7.27.

Figure 7.26 Overview of Alternative 4B

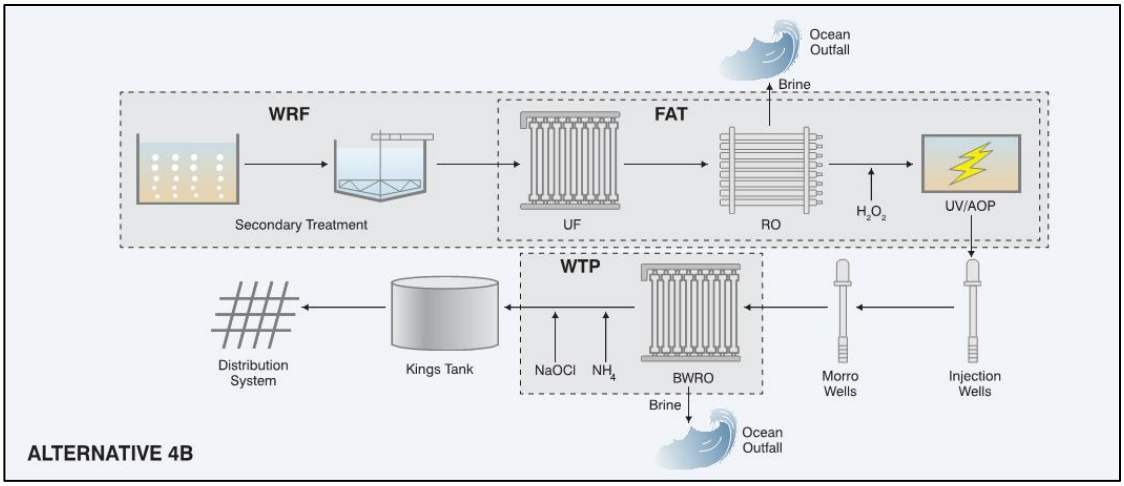

Figure 7.27 Alternative 4B Process Flow Diagram

7.1.11.2 Supply Yield and Availability

The supply yield of Alternative 4B is limited by the allowed volume of water extracted from the Morro Well Field. A preliminary IPR study was performed by GSI Water Solutions and their findings suggested that a maximum extraction volume of 1,200 AFY would cause sea water intrusion effects on the Morro groundwater basin, but with the proposed salinity treatment facility, TDS levels caused by increased seawater intrusion can be eliminated with the use of BWRO. It must be noted that effects of seawater instruction must be continually monitored to avoid deterioration of the Morro groundwater basin. To meet the City's future demand, Alternative 4B yield would be supplemented by imported State Water.

7.1.11.3 Implementation Considerations

Alternative 4B implementation will undergo the same constraints as listed for Alternative 4A with the addition of relocating the existing BWRO facility to its new location. New infrastructure outside of the treatment footprint will be required in order to connect to the existing Morro well supply line. The new BWRO facility is similar to Alternative 2A in that it is outside of potential sea level rise hazard boundaries.

7.1.11.4 Water Quality Considerations

In comparison to Alternative 4A, greater TDS levels as a result of increased pumping efforts will require additional salinity treatment. With the new treatment facility, mitigating TDS exceedances for drinking water will be achieved to meet water quality standards.

7.1.11.5 Costs of Alternative 4B

The capital costs for Alternative 4B is approximately $37.651 million. This cost includes the cost of the advanced treatment facility, pumping and conveyance costs, and injection well infrastructure and a post extraction salinity treatment facility. The annual costs include the amortized capital costs, annual (operation and maintenance) O&M and energy costs which total $3.605 million per year. Alternative 4B energy consumption is estimated to be 3,252 kWh per ac-ft. The resulting unit costs of water from the estimated total capacity is $3,005 per acre foot. Table 7.12 lists the financial components of Alternative 4B.

Table 7.12 Costs of Alternative 4B

Component	Cost
Advanced Treatment Facility	$9,065,000
Groundwater Salinity Treatment Facility	$11,246,000
Injection Wells	$12,300,000
Booster Pump Station	$990,000
12-Inch Purified Water Pipeline	$4,050,000
Subtotal	**$37,651,000**
Amortized Capital Costs (30 years)	$2,274,554
Annual O&M Costs	$745,800
Annual Energy Costs	$585,500
Total Annual Costs	**$3,605,854**
Energy Consumption	3,252 kWh/ac-ft
Net Yield	1,200 ac-ft
Unit Cost	**$3,005/AFY**

7.1.12 Alternative 4C: Morro Well Field IPR by Groundwater Injection with Nitrate Treatment

7.1.12.1 Description of Alternative 4C

Alternative 4C (Figure 7.28) includes the same infrastructure as Alternative 4A and 4B with the exception of a new nitrate treatment facility to treat extracted groundwater from the Morro groundwater basin. Historically, agricultural runoff upstream of Morro Creek has increased nitrate concentrations in the groundwater. Exceeding nitrate concentrations within the groundwater requires treatment post extraction from the Morro wells. The process flow diagram for Alternative 4C is shown in Figure 7.29.

Figure 7.28 Overview of Alternative 4C

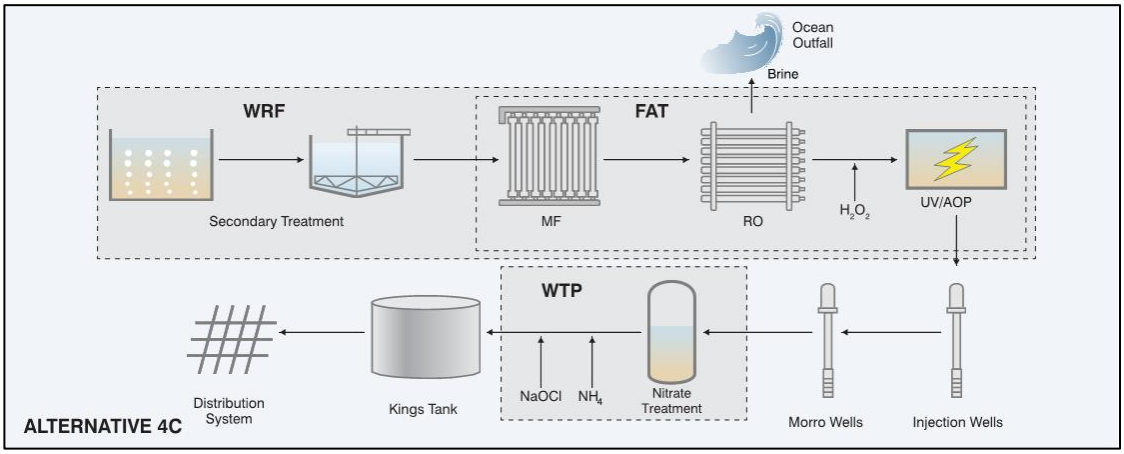

Figure 7.29 Alternative 4C Process Flow Diagram

7.1.12.2 Supply Yield and Availability

The supply yield of Alternative 4C is limited similarly to Alternative 4A. Since the nitrate treatment facility is not designed to treat high salinity levels, the seawater intrusion affects from increased pumping will need to be closely monitored in order to prevent seawater intrusion. To meet the City's future demand, Alternative 4C yield, would be supplemented by imported State Water.

7.1.12.3 Implementation Considerations

Similar to Alternative 4A and 4B, the same implementation constraints are present in the construction of Alternative 4C.

7.1.12.4 Water Quality Considerations

As mentioned in Alternative 2A through 2E, the Morro well field is constrained and susceptible to high levels of nitrate due to upstream agricultural land use. The proposed treatment facility will be able to remove nitrate to meet primary MCL drinking water standards. Groundwater pumping must be monitored to prevent significant increases in seawater intrusion to limit TDS concentrations.

7.1.12.5 Costs of Alternative 4C

The capital cost for Alternative 4C is approximately $31.805 million. This cost includes the advanced treatment facility, pumping and conveyance costs, and injection wells and a post extraction nitrate treatment plant. The annual costs include the amortized capital as well as annual O&M and energy costs totaling $2.881 million per year. Alternative 4C energy consumption is estimated to be 2,810 kWh per ac-ft. The resulting unit costs of water from the estimated total capacity is $3,056 per acre foot. Table 7.13 lists the financial components of Alternative 4C.

Table 7.13 Costs of Alternative 4C

Component	Cost
Advanced Treatment Facility	$9,065,000
Groundwater Nitrate Treatment Facility	$5,400,000
Injection Wells	$12,300,000
Booster Pump Station	$990,000
12-Inch Purified Water Pipeline	$4,050,000
Subtotal	**$31,805,000**
Amortized Capital Costs (30 years)	$1,894,264
Annual O&M Costs	$589,920
Annual Energy Costs	$397,500
Total Annual Costs	**$2,881,684**
Energy Consumption	2,810 kWh/ac-ft
Net Yield	943 ac-ft
Unit Cost	**$3,056/AFY**

7.1.13 Alternative 5: Ocean Desalination

7.1.13.1 Description of Alternative 5

Alternative 5 (Figure 7.30) consists of a new Ocean Desalination treatment facility. The existing facility has been used intermittently as an emergency water source during periods of drought. The original Coastal Development Permit (CDP) limited emergency operation of the desalination facility to only two years, with the opportunity to renew for normal use. The CDP was never renewed, but the facility was operated during water supply shortages. Due to the age and condition of the original desalination facility, Alternative 5 completely replaces the existing facility. To mitigate exposure from the 100-year flood zone, Alternative 5 will be located near the existing RV parking lot. This facility will be supplied by the existing seawater wells which will all be rehabilitated prior to operation. The treated seawater will be conveyed to the Kings Tank prior to distribution. The Regional Water Quality Control Board (RWQCB) has indicated that

continued use of a desalination facility would require the City to dispose of brine using an alternative outfall. One option is the outfall jointly owned by the City and Cayucos Sanitary District (CSD). This alternative does not include the cost for a new outfall. The process flow diagram for Alternative 5 is shown in Figure 7.31.

Figure 7.30 Overview of Alternative 5

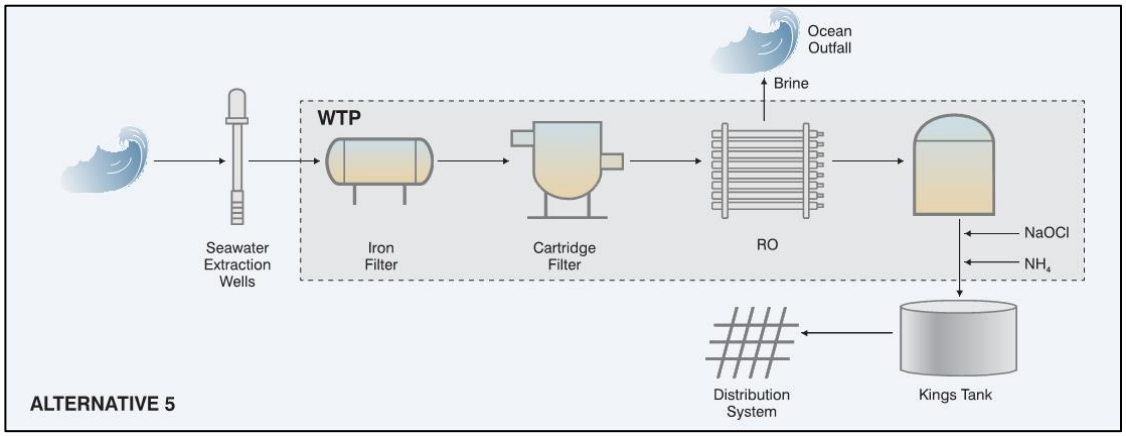

Figure 7.31 Alternative 5 Process Flow Diagram

7.1.13.2 Supply Yield and Availability

The City's existing sweater desalination facility has an instantaneous capacity of 400 gallons per minute (gpm) (645 AFY). Alternative 5 assumes the new facility will have the same capacity of the existing desalination plant to be within the existing permit guidelines. To meet the projected demands for the City, it would be necessary for either groundwater or imported State Water to supplement the proposed desalinated supply. The capacity of the desalination facility is

governed by the RWQCB. Therefore an increase in capacity would require a new permit for the facility and a new mechanism for brine disposal.

7.1.13.3 Implementation Considerations

The proposed desalination facility must be located outside of the 100-year flood plain as defined by the FEMA Firm Map, (Appendix C). The existing RV parking facility shown in Figure 7.30 is outside of the 100-year flood plain and limits the amount of new piping required to connect the desalination facility to both the Kings Tank and the existing seawater supply line. There are also no coastal hazards as displayed in the NOAA sea-level rise model results and the City of Morro Bay 2050 Sea Level Rise Scenario report and the Sea Level Rise Adaptation Strategy Report. Additionally, the existing seawater wells must be rehabilitated. Previous studies shown the wells to have varied performance during operation.

7.1.13.4 Water Quality Considerations

As mentioned previously, implementation of this alternative would require the use of a different outfall. The RWQCB has indicated that the existing outfall is not acceptable to serve a facility that would be used on a routine basis.

7.1.13.5 Costs of Alternative 5

The capital cost for Alternative 5 is approximately $16.564 million. This cost includes the cost of the desalination treatment facility, conveyance costs, and infrastructure updates. The annual costs include the amortized capital costs, annual O&M and energy cost which totals $1.89 million per year. Alternative 5 energy consumption is estimated to be 5,343 kWh per ac-ft. The resulting unit costs of water from the estimated total capacity is $2,932 per acre foot. Table 7.14 lists the financial components of Alternative 5.

Table 7.14 Costs of Alternative 5

Component	Cost
Ocean Desalination Treatment Facility	$10,368,000
Iron Removal Pretreatment Facility	$432,000
Well Mechanical and Electrical and Infrastructure Improvements	$4,500,000
Booster Pump Station	$83,000
Storage Tank	$938,000
12-inch Brine Pipeline	$243,000
Subtotal	**$16,564,000**
Amortized Capital Costs (30 years)	$1,074,453
Annual O&M Costs	$299,370
Annual Energy Costs	$517,000
Total Annual Costs	**$1,890,823**
Energy Consumption	5,343 kWh/ac-ft
Net Yield	645 ac-ft
Unit Cost	**$2,932/AFY**

7.1.14 Alternative 6: Direct Potable Reuse

7.1.14.1 General Description

Alternative 6 (Figure 7.32) consists of utilizing the current WRF project secondary effluent coupled with advanced treatment monitoring and engineered storage for direct potable reuse (DPR). DPR includes the full advanced treatment train similar to Alternatives 4A through 4C followed by back end microfiltration, granular activated carbon vessels for polishing, advanced monitoring, and engineered water storage. From the on-site storage tanks, the treated water will be conveyed to the Kings Tank prior to distribution. The process flow diagram of Alternative 6 is shown in Figure 7.33.

Figure 7.32 Overview of Alternative 6

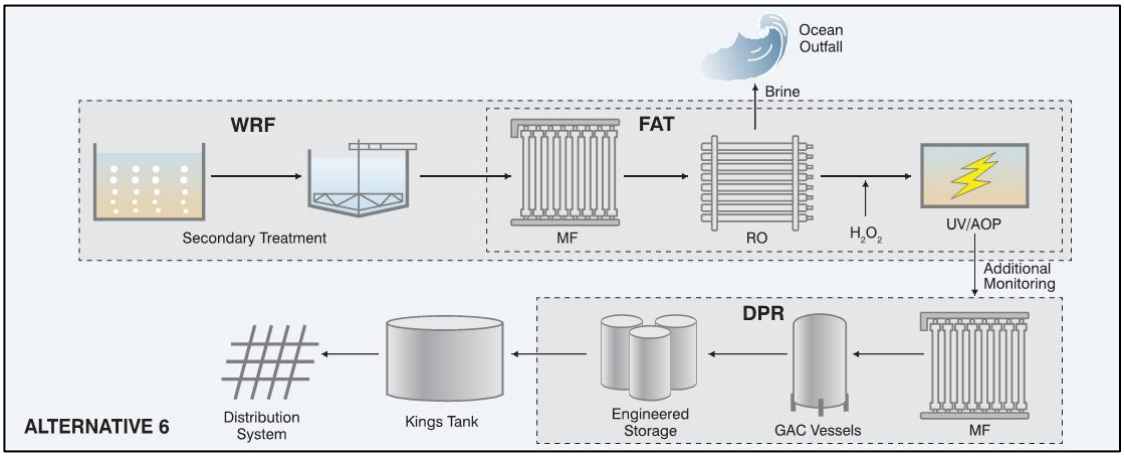

Figure 7.33 Alternative 6 Process Flow Diagram

7.1.14.2 Supply Yield and Availability

Unlike the other alternatives described in this report, this alternative does not rely on the City's ground or surface water resources. The capacity of Alternative 6 is dependent upon the treatment capacity of the WRF facility. The WRF facility is designed for a capacity of 0.97 million gallons per day (mgd) and with an 85 percent recovery for advanced treatment applied the resulting capacity is approximately 0.82 mgd (923 AFY). To meet the projected demands for the City, it would be necessary for either groundwater or imported State Water to supplement the proposed supply.

7.1.14.3 Implementation Considerations

The DPR facility would be constructed onsite as part of the WRF project. Extensive permitting will be required in order to meet anticipated standards for direct potable reuse. Currently the State does not have regulations for DPR, although they are working on developing them for certain applications by 2023. Additionally site planning efforts will be needed to accommodate the proposed DPR facility on the new WRF site.

7.1.14.4 Water Quality Considerations

The DPR facility will treat effluent from the WRF to anticipated direct potable reuse standards. To meet the intent of the draft DPR guidelines released by the State in 2018, additional on-line monitoring to ensure compliance with pathogen removal requirements as well as an engineered storage buffer would be provided prior to distribution.

7.1.14.5 Costs of Alternative 6

The capital cost for Alternative 6 is approximately $20.022 million. This cost includes the DPR advanced treatment facility with additional monitoring, conveyance and pumping costs, and engineered storage. The annual costs include the amortized capital costs over 30 years as well as annual O&M and energy costs which total around $2.039 million per year. Alternative 6 energy consumption is estimated to be 3,515 kWh per ac-ft. The resulting unit costs of water from the estimated total capacity is $2,210 per acre foot. Table 7.15 lists the financial components of Alternative 6.

Table 7.15 Cost of Alternative 6

Component	Cost
Advanced Treatment Facility	$14,732,000
DPR Additional Monitoring	$150,000
Granular Activated Carbon Vessels	$371,000
Booster Pump Station	$1,735,000
Water Storage Tanks	$1,407,000
12-inch Water Pipeline	$1,627,000
Subtotal	**$20,022,000**
Amortized Capital Costs (30 years)	$1,084,461
Annual O&M Costs	$467,955
Annual Energy Costs	$487,000
Total Annual Costs	**$2,039,416**
Energy Consumption	3,515 kWh/ac-ft
Net Yield	923 ac-ft
Unit Cost	**$2,010/AFY**

7.2 Summary of Unit Costs for Water Supply Alternatives

The summary of the unit costs for each water supply alternative is shown in Table 7.16. As discussed in the water supply alternative descriptions, for a given alternative, if the proposed water production capacity does not meet the City's future demand then the supply will be supplemented by imported State Water. As shown in Table 7.16, since State Water represents a percentage of the future demand portfolio for the City (see Figure 7.1), the weighted unit cost represents the unit cost the City would pay for water per water supply alternative.

Table 7.16 Water Supply Alternative Unit Cost Summary

Alternative	Unit Cost of Local Supply	Unit Cost of State Water	Percent of Local Supply	Percent of SWP Supplement	Weighted Unit Cost
1	$1,546	$1,558	9	91	$1,557
2A	$1,546	$1,558	40	60	$1,553
2B	$1,236	$1,558	40	60	$1,428
2C(1)	$1,748	$ -	100	0	$1,748
2D(1)	$1,184	$ -	100	0	$1,184
2E(1)	$1,472	$ -	100	0	$1,472
3A	$1,663	$1,558	79	21	$1,641
3B	$2,258	$1,558	79	21	$2,111
4A	$2,396	$1,558	65	35	$2,103
4B	$3,005	$1,558	83	17	$2,759
4C	$3,056	$1,558	83	17	$2,801
5	$2,932	$1,558	45	55	$2,171
6	$2,209	$1,558	64	36	$1,974
2C(1)	$4,519	$1,558	40	60	$2,749
2D(1)	$3,016	$1,558	40	60	$2,144
2E(1)	$4,056	$1,558	40	60	$2,562

Notes:
(1) If Chorro groundwater basin is unavailable, then Morro groundwater must be supplemented with State Water.

7.3 Evaluation of Water Supply Alternatives

As discussed in Section 3.1.1, each water supply alternative underwent an evaluation with respect to the selected criteria to develop an overall score. The nine scoring criteria listed in Chapter 3 were weighted based on input from the City staff and City leadership (see Appendix D). Two separate ranking sets were developed to compare all of the water supply alternatives. Given the probability that without augmentation Chorro Creek has the potential to limit the use of the Chorro well field, therefore the difference in scores between the two flow scenarios for Alternatives 2C, 2D, and 2E were necessary to compare with the other water supply alternatives. The first set, as shown in Table 7.17, summarizes the resulting scores assuming that the Chorro well field is fully operational in Alternatives 2C, 2D, and 2E, therefore resulting in a lower unit cost and energy requirement and a higher net yield, drought resiliency, and local supply offset. The second set, as shown in Table 7.18, summarizes the resulting scores assuming the Chorro well field is non-operational due to low flows in Chorro Creek. This results in a significantly higher unit cost, energy requirement, lower net yield, drought resiliency, and local supply offset therefore decreasing the overall score of Alternative 2C, 2D, and 2E.

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Table 7.17 Water Supply Alternatives Scoring Results

Alternative		1	2A	2B	2C	2D	2E	3A	3B	4A	4B	4C	5	6
Quick Description		State Water	Morro BWRO	Morro NO_3	Morro + Chorro BWRO	Morro + Chorro NO_3	Morro (BWRO) + Chorro (NO_3)	Chorro Augmentation	Chorro NO_3 Augmentation	WRF Morro IPR	WRF Morro IPR + BWRO	WRF Morro IPR + NO_3	Desalination	DPR
Net Yield (AFY) (*wet yield)*		1,313	581	581	1,724	1,724	1,724	1,142	1,142	943	1,200	943	645	923
Energy Requirements (kWh/ac-ft)		2,899	3,339	2,249	3,191	2,340	2,356	2,347	3,246	1,912	3,253	2,810	5,344	3,516
Unit Cost		$1,558	$1,546	$1,236	$1,748	$1,184	$1,472	$1,663	$2,258	$2,396	$3,005	$3,056	$2,932	$2,208.50
Drought Yield (AFY)		540	581	581	1,150	1,150	1,150	1,142	1,142	943	1,200	943	645	923
Drought Resiliency		41%	100%	100%	67%	67%	67%	100%	100%	100%	100%	100%	100%	100%
Local Supply Offset (% of Future Demand Met by Local Supply of 1445 AFY)		0%	40%	40%	100%	100%	100%	79%	79%	65%	83%	65%	45%	64%
SCORES														
Alternatives	Weighting	1	2A	2B	2C	**2D**	**2E**	3A	3B	4A	4B	4C	5	6
Unit Cost	1.22	3.7	3.7	3.7	3.7	3.7	3.7	3.7	2.4	2.4	1.2	1.2	2.4	2.4
Drought Resiliency	1.17	3.5	5.9	5.9	4.7	4.7	4.7	5.9	5.9	5.9	5.9	5.9	5.9	5.9
Earthquake Resiliency	1.00	1.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Flood Vulnerability	1.10	5.5	2.2	2.2	4.4	4.4	4.4	5.5	5.5	4.4	4.4	4.4	2.2	5.5
Local Supply Offset	1.17	1.2	3.5	3.5	5.9	5.9	5.9	5.9	5.9	4.7	5.9	4.7	3.5	4.7
Energy Req.	0.98	2.9	2.0	2.9	2.0	2.9	2.9	2.9	2.0	3.9	2.0	2.9	1.0	2.0
Constructability	1.09	5.5	4.4	4.4	3.3	4.4	4.4	2.2	2.2	3.3	3.3	3.3	4.4	3.3
Regulatory and Institutional Complexity	1.03	5.2	3.1	3.1	3.1	3.1	3.1	1.0	1.0	3.1	3.1	3.1	1.0	1.0
Quality Consistency	1.24	6.2	6.2	6.2	6.2	6.2	6.2	1.2	6.2	1.2	6.2	6.2	6.2	6.2
Total Score		34.6	35.8	36.8	38.1	**40.2**	**40.2**	33.3	36.0	33.9	36.8	36.7	31.6	35.9

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Table 7.18 Water Supply Alternatives Scoring Results (Chorro Wellfield Non-Operational)

Alternative	1	2A	2B	2C	2D	2E	3A	3B	4A	4B	4C	5	6
Quick Description	State Water	Morro BWRO	Morro NO_3	Morro + Chorro BWRO	Morro + Chorro NO_3	Morro (BWRO) + Chorro (NO_3)	Chorro Augment	Chorro NO_3 Augment	WRF Morro IPR	WRF Morro IPR + BWRO	WRF Morro IPR + NO_3	Desalination	DPR
Net Yield (AFY) (*wet yield)*	1,313	581	581	1724	1724	1724	1,142	1,142	943	1,200	943	645	923
Energy Requirements (kWh/ac-ft)	2,899	3,339	2,249	3,110	3,626	4,923	2,347	3,246	1,912	3,253	2,810	5,344	3516
Unit Cost	$1,558	$1,546	$1,236	$4,517	$3,016	$4,056	$1,663	$2,258	$2,396	$3,005	$3,056	$2,932	$2,208.50
Drought Yield (AFY)	540	581	581	581	581	581	1,142	1,142	943	1,200	943	645	923
Drought Resiliency	41%	100%	100%	34%	34%	34%	100%	100%	100%	100%	100%	100%	100%
Local Supply Offset (% of Future Demand Met by Local Supply of 1445 AFY)	0%	40%	40%	40%	40%	40%	79%	79%	65%	83%	65%	45%	64%
SCORES													
Alternatives	Weighting	1	2A	2B	2C	2D	2E	3A	3B	4A	**4B**	4C	5	6
Unit Cost	1.22	3.7	3.7	3.7	1.22	1.2	1.2	3.7	2.4	2.4	1.2	1.2	2.4	2.4
Drought Resiliency	1.17	3.5	5.9	5.9	3.5	3.5	3.5	5.9	5.9	5.9	5.9	5.9	5.9	5.9
Earthquake Resiliency	1.00	1.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Flood Vulnerability	1.10	5.5	2.2	2.2	4.4	4.4	4.4	5.5	5.5	4.4	4.4	4.4	2.2	5.5
Local Supply Offset	1.17	1.2	3.5	3.5	3.5	3.5	3.5	5.9	5.9	4.7	5.9	4.7	3.5	4.7
Energy Req.	0.98	2.9	2.0	2.9	2.0	2.0	1.0	2.9	2.0	3.9	2.0	2.9	1.0	2.0
Constructability	1.09	5.5	4.4	4.4	3.3	4.4	4.4	2.2	2.2	3.3	3.3	3.3	4.4	3.3
Regulatory and Institutional Complexity	1.03	5.2	3.1	3.1	3.1	3.1	3.1	1.0	1.0	3.1	3.1	3.1	1.0	1.0
Quality Consistency	1.24	6.2	6.2	6.2	6.2	6.2	6.2	1.2	6.2	1.2	6.2	6.2	6.2	6.2
Total Score		34.6	35.8	36.8	32.2	33.3	32.3	33.3	36.0	33.9	**36.8**	36.7	31.6	35.9

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7.4 Analysis of Water Supply Evaluation

The results of the water supply evaluation are summarized in Tables 7.17 and 7.18.

Criteria including drought resiliency, earthquake resiliency, flood vulnerability, regulatory and institutional complexity, and quality consistency remain relatively consistent between the alternatives. All of the alternatives were developed to achieve a drought resiliency with the exception of Alternatives 2C through 2E as shown by their low drought resiliency scores on Table 7.18. The supply alternatives would be designed to mitigate any potential damage from natural hazards. It is assumed modern design and construction techniques would be used to prevent damage from seismic events. All alternatives are considered to be local in comparison to the State Water Project infrastructure which spans across most of California. Flood damage prevention would be completed by either local onsite storm drainage systems and locating future alternative sites outside of area of coastal hazards. In areas where potential flood hazards existed, scores were reduced. For example, all of the existing Morro wells are within the 100-year flood boundary resulting in a lower flood vulnerability score. For water quality consistency, alternatives that include a new treatment facility received a higher score. For example, Alternatives 3A and 4A do not provide new treatment facilities and represent only a supply augmentation to Morro Bay's current domestic source resulting in a reduced quality score.

The variability in scores is seen mainly in the unit cost, drought resiliency, local supply offset, energy requirements, and constructability criteria. While there are a few alternatives with low unit costs, their score is reduced by either local supply offset or quality results. For example, Alternatives 2A and 2B both have the highest unit cost score, but the lowest local supply offset score. In comparison, lower unit cost scores as shown in Alternatives 4A through 4C display highest local supply offset scores. Alternatives 4A, 4B, 4C, 5, and 6 have the highest unit cost, but with the exception of Alternative 4A, these alternatives also have the highest quality consistency scores. High scores in local supply and quality do not provide a direct relationship to increasing an alternatives overall score. As displayed in Table 7.18, when Chorro well field is offline due to low creek flows, Alternatives 2C, 2D, and 2E have the highest unit costs with a high energy requirement and low local supply offset.

Alternatives 3A and 3B eliminate any circumstance of the Chorro well field being restricted by Chorro Creek, but the regulatory and institutional complexity of these alternatives reduce their respective scores due to permitting challenges and potential environmental considerations. Alternative 3A and 3B do show a promising yield for a lower unit cost, but require high levels of treatment, energy, and regulatory approval for an unreliable supply alternative. Alternatives 4A, 4B, and 4C are comparable to Alternatives 3A and 3B in providing a local supply and is similar in cost, but overall are reliable and have lower institutional and regulatory complexity.

Energy costs were reduced by limiting new pumping stations required for conveyance. Alternatives requiring long transmission mains (see previous sections) utilize booster pump stations that significantly increase energy costs and reduce scores. Alternatives relying on existing treatment facilities or no treatment (Alternatives 3A and 4A) have higher scores for energy requirements, but the lowest scores for quality consistency due to the variability of the existing groundwater basin water quality. The remaining alternatives have lower scores for energy use with nitrate treatment requiring the least energy, followed by BWRO (salinity), and

ultimately ocean desalination having the highest energy requirement therefore the lowest evaluation score.

The scores for constructability, regulatory and institutional complexity varied significantly for each alternative. Alternatives 2A, 2B, 2C, 2D, and 2E utilize straight-forward treatment technologies and have a regulatory pathway that is similar to the City's existing BWRO facility. In comparison, Alternatives 3A and 3B may be very difficult to permit and a new National Pollutant Discharge Elimination System (NPDES) permit for Chorro Creek. While these alternatives include discharge to Chorro Creek, the RWQCB may view these alternatives as recharge and could require full advanced treatment similar to Alternative 4A, 4B, and 4C. While Alternatives 4A, 4B, and 4C require full advanced treatment, the permitting effort required for potable reuse is well understood and becoming more commonplace with the RWQCB and Division of Drinking Water (DDW). Alternatives 5 assumes that the City would utilize its existing desalination permit and would not be difficult to modify that permit to allow for more regular, continuous operation. Alternative 6 represents the most difficult permitting of any of the alternatives developed. As mentioned previously, Carollo has developed the treatment train based on our interpretation of the draft DPR guidelines. However, final DPR requirements are not anticipated from DDW until 2023. The draft DPR guidelines indicate that it may be difficult to permit a DPR facility for a smaller agency like the City, and would likely prefer to permit a DPR project that was first permitted and operated as IPR.

7.5 Summary of Findings

The water supply evaluation provided a comprehensive assessment of various water supplies available to the City of Morro Bay. The potential to augment stream flow in Chorro Creek (Alternative 3A and 3B) originally looked promising, however the significant regulatory challenges associated with recharge and the nitrate Total Maximum Daily Load (TMDL) are reflected in the scores summarized in Table 7.18. Without stream flow augmentation, heavy reliance on groundwater is complicated by periods of drought that would make the Chorro well field unavailable to the City during the times when it is most needed. While the groundwater treatment alternatives have some of the lowest unit costs, the overall scoring is tempered by their resiliency to drought. This is reflected in the scores for Alternatives 2C, 2D, and 2E. With the impending WRF project, IPR represents the best option for supplementing the City's water supply portfolio. Of the IPR alternatives, Alternative 4B ranks the highest due to the scoring for water quality and limiting the City's reliance on imported State Water. While Alternative 4B has a higher unit cost than Alternative 4A, the construction of a new BWRO facility to treat the water extracted by the Morro wells improves the scoring for water quality. While the Morro groundwater basin currently has nitrate concentrations that slightly exceed the nitrate primary MCL, preliminary hydrogeology work for the WRF project has indicated that injection and more consistent pumping for the Morro groundwater basin could lower the nitrate concentrations below drinking water standards. Other options such as blending with State Water could be used if the water extracted from the Morro groundwater basin with IPR still exceeds drinking water standards. At this time, Carollo recommends the City move forward with Alternative 4A and construct the advanced treatment facilities as part of the WRF project to facilitate IPR. It is not recommended that the City construct a new nitrate treatment or BWRO facility to replace its existing WTP at this time. While additional hydrogeological work planned as part of the WRF project will help inform the anticipated, long-term water quality for the water extracted from the Morro groundwater basin, the City has the option to utilize the existing BWRO facility if needed

in the short-term. If once the existing WTP has reached the end of its useful life and the City has determined that either salinity or nitrate removal is needed to meet drinking water standards, it is recommended that a new treatment facility be constructed outside of the area of coastal hazards. This staged approach minimizes the City's initial capital expenditure by avoiding the construction of additional facilities before it is confirmed that they are needed. Carollo recommends expediting the hydrogeological work necessary to better understand the fate and transport of nitrates and TDS in the Morro groundwater basin with the injection and extraction associated with Alternative 4.

7.6 Implementation and Next Steps

As a result of the recommendation to move forward with Alternative 4A to strengthen the City's water supply, the following implementation recommendations are being made for the City's other water resources:

7.6.1 Indirect Potable Reuse

- In order to better understand treatment that may be required following extraction, the City should immediately move forward with additional hydrogeology work to inform the injection and extraction process.
- When State Water is available, the City should use IPR as a back-up supply and to help improve the quality in the Morro groundwater basin (i.e., flushing of nitrates).
- An IPR facility could offer the design flexibility to be modified to DPR operation in the future with the addition of new treatment processes (i.e., granular activated carbon vessels, engineered storage buffer, advanced monitoring, etc.) to the existing facility. DPR regulations are not anticipated from DDW until 2023, but the draft guidelines indicate that it will be preferred that a DPR facility is first permitted and operated as an IPR facility.

7.6.2 State Water

- As the lowest cost supply source, the City should continue to maximize its use of State Water as available in the short term.
- Once the IPR process is operational, the City should discontinue purchase of the State Water "drought buffer" since the water injected into the Morro groundwater basin will serve as the City's back-up supply when State Water is not available. It is anticipated that the cost for drought buffer could increase by as much as $400/ac-ft due to the impacts from the California Water Fix.
- The City should continue to monitor DWR's exchange policies and continue to engage with the San Luis Obispo County Flood Control and Water Conservation District to identify opportunities for generating revenue from unused State Water allocations.

7.6.3 Ocean Desalination

- Since the desalination facility has reached the end of its useful life, the City should de-energize, pull pumps, pull wires, and secure the seawater wells. While it is not anticipated that the facility will be operated, removal of the wells could impact the City's existing desalination permit.

7.6.4 Chorro Groundwater Basin

- The City spends approximately $40,000 per year for Chorro Creek stream flow monitoring and sampling and maintenance for the wells in the Chorro groundwater basin. Securing the water rights in the basin would also require significant capital investment from construction of a new treatment and pumping pipeline and permanent stream flow gauges.
- Since the Chorro groundwater basin has the lowest priority of any of the City's available water resources, Carollo recommends the City discontinue the existing activities at the Chorro well field.

Chapter 8
CAPITAL IMPROVEMENT PLAN

This chapter presents the City of Morro Bay (City) capital improvement plan (CIP), a summary of the capital costs, and a basic assessment of the possible financial impacts on the City. This chapter is organized to assist the City in making financial decisions. The CIP is based on the evaluation of the City's water distribution system, wastewater collection system and storm drainage system, as described in Chapter 6.

8.1 Project Prioritization

As discussed in Chapter 6, the capital projects identified will allow the City to provide reliable service to its customers through the year 2040. The improvement projects were prioritized based on the following factors:

- Water distribution system projects
  - Implementing storage, pumping, and transmission improvements
  - Addressing fire flow deficiencies
  - Planning for ongoing rehabilitation and replacement of aging water mains
- Wastewater collection system improvements
  - Reducing the risk on sanitary sewer overflows (SSOs) in the collection system
  - Implementing projects to reduce the rates of infiltration and inflow (I/I) in the collection system
- Storm drainage system improvements
  - Mitigating flooding during the 10-year design storm
  - Mitigating additional flooding during the 100-year design storm

Based on these factors, each project was assigned an implementation year. Critical projects were phased in the earlier phases (years) of the 20-year CIP. Less critical projects were phased into later phases of the 20-year CIP.

8.2 Capital Improvement Project Costs

The capacity upgrades and other system capital improvements set the foundation of the City's water distribution system, wastewater collection system, and storm drainage system CIP's. The cost estimates presented in this study are opinions developed from bid tabulations, cost curves, information obtained from previous studies, and Carollo Engineers, Inc. (Carollo) experience on other projects. The costs are based on an *Engineering News Record* Construction Cost Index (ENR CCI) 20-City Average of 10,889 (February 2018).

8.3 Cost Estimating Accuracy

The cost estimates presented in the CIP's have been prepared for general master planning purposes and for guidance in project evaluation and implementation. Final costs of a project will depend on actual labor and materials costs, competitive market conditions, final project scope,

implementation schedule, and other variable factors such as preliminary alignment generation, investigation of alternative routings, and detailed utility and topography surveys.

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

8.4 Construction Unit Costs

The construction costs are representative of water distribution system facilities, sewer collection system facilities, and storm drainage facilities under normal construction conditions and schedules. Costs have been estimated for public works construction.

8.4.1 Pipeline Unit Costs

This section summarizes the unit costs for water distribution, wastewater collection, and storm drainage system pipelines. All of the unit costs presented in this section include pipeline costs, excavation, and other appurtenances (e.g., valves, manholes, etc.)

8.4.1.1 Water Distribution System Pipelines

Water distribution system pipeline improvements range in size from 8-inches to 16-inches in diameter for this OneWater Plan. Pipeline unit costs for relevant sized upgrades are shown in Table 8.1. The unit costs are for "typical" field conditions with construction in stable soil.

Table 8.1 Water Pipeline Unit Costs

Pipe Size (inches)	Replacement Unit Construction Cost [(1)] ($/linear foot)
8	175
10	220
12	225
14	300
16	300

Notes:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889

8.4.1.2 Wastewater Collection System Pipelines

Sewer pipeline improvements range in size from 8-inches to 27-inches in diameter in this study. Unit costs for the construction of pipelines and appurtenances (e.g., manholes) are shown in Table 8.2. The construction cost estimates are based upon these unit costs. The unit costs are for "typical" field conditions with construction in stable soil at a depth ranging between 10 feet to 15 feet.

Table 8.2 Sewer Pipeline Unit Costs

Pipe Size (inches)	Replacement Unit Construction Cost (1) ($/linear foot)
Gravity Main Unit Costs	
10	165
12	175
14	185
15	190
16	190
18	205
20	250
21	260
24	285
27	320
Force Main Unit Costs	
8	160

Notes:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889

8.4.1.3 Storm Drainage System Pipelines

Storm drain improvements range in size from 10-inches to 54-inches in diameter in this study. Unit costs for the construction of pipelines and appurtenances (e.g., manholes, inlets, etc.) are shown in Table 8.3. The construction cost estimates are based upon these unit costs. The unit costs are for "typical" field conditions with construction in stable soil at a depth ranging between 10 feet to 15 feet.

Table 8.3 Storm Drain Pipeline Unit Costs

Pipe Size (inches)	Replacement Unit Construction Cost [(1)] ($/linear foot)
10	165
12	175
14	185
15	190
16	190
18	205
20	250
21	260
24	285
27	320
30	355
33	395
36	440
39	550
42	550
48	600
54	640

Notes:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889

8.4.2 Storage Tank, Booster Pump, and PRV Station Unit Costs

This OneWater Plan includes tank, pump station, and pressure reducing valve (PRV) improvement projects. The costs for these facilities were developed based on the unit costs shown in Table 8.4, Table 8.5, and Table 8.6, respectively.

Table 8.4 Unit Construction Costs - Reservoir Storage

Type (MG)	Unit Construction Cost(1) ($/gallon)
<1	$2.50
1 to 3	$2.00
3 to 5	$1.75
5 to 10	$1.50

Note:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889

Table 8.5 Unit Construction Costs - Pump Stations

Station Size (HP)	Unit Construction Cost[1] ($/HP)
100 hp and smaller	$5,000
150 to 500 hp	$3,000

Note:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889

Table 8.6 Unit Construction Costs - Pressure Reducing Valves

Type	Unit Construction Cost[1] ($/PRV)
Small (1-2 valves <8")	$94,000
Medium (2-3 valves 8" and up)	$187,500
Large (3-4 valves 12" and up)	$281,000

Note:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889

8.4.3 Lift Station Unit Cost

Lift station costs were developed on the basis of pumping capacity. Lift station improvements will increase the firm pumping capacity, which is the capacity with the largest pump out of service. The lift station construction cost curve shown in Figure 8.1 was derived from past costs for projects of similar size in California.

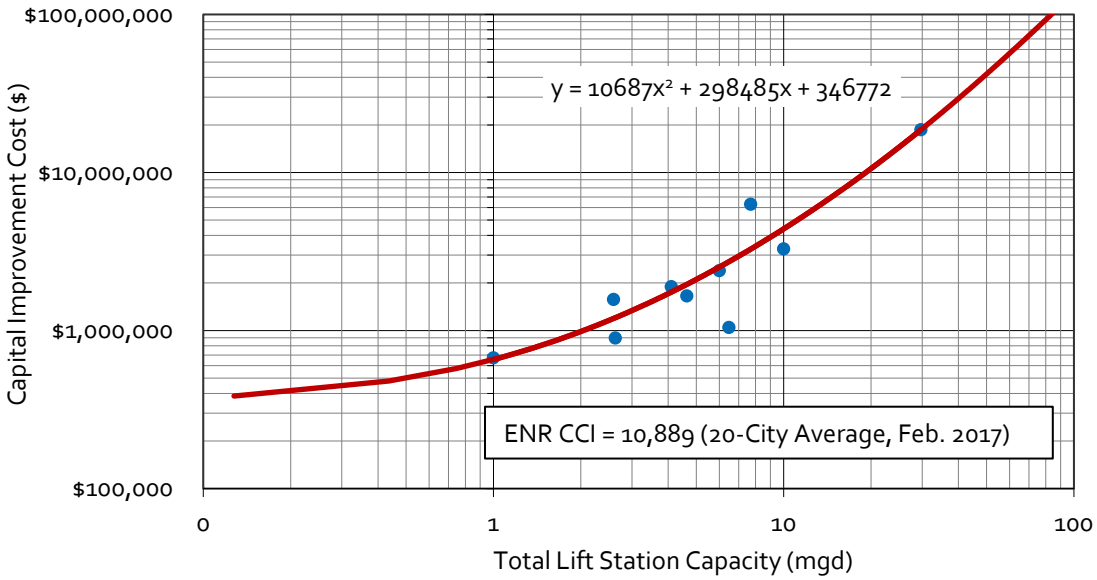

Figure 8.1 Lift Station Cost versus Capacity Curve

8.5 Project Costs and Contingencies

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

8.5.1 Baseline Construction Cost

Baseline Construction Cost is the total estimated construction cost, in dollars, of the proposed improvements for pipelines, storage tanks, booster pump stations, PRVs, lift stations, and force mains. Baseline Construction Costs were developed using the following criteria:

- *Pipelines/Force Mains:* Calculated by multiplying the estimated length by the unit cost.
- *Storage Tanks:* Calculated by multiplying the tank volume by the unit cost.
- *Booster Pump Stations/Lift Stations:* Calculated on a case-by-case basis depending on the type of work that is required.
- *PRV Stations:* Calculated based on the information presented in Table 8.6.

8.5.2 Estimated Construction Cost

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

Since knowledge about site-specific conditions of each proposed project is limited at the master planning stage, a 30 percent contingency was applied to the Baseline Construction Cost to account for unforeseen events and unknown conditions. A 30 percent contingency was used to account for unknown sit conditions such as unforeseen conditions, environmental mitigations, and other unknowns is typical for master planning projects.

8.5.3 Capital Improvement Cost

Other project construction contingency costs include costs associated with project engineering, construction phase professional services, and project administration. Engineering services associated with new facilities include preliminary investigation and reports, Right of Way (ROW) acquisition, foundation explorations, preparation of drawings and specifications during construction, surveying and staking, sampling of testing material, and start-up services. Construction phase professional services cover items such as construction management, engineering services, materials testing, and inspection during construction. Finally, there are project administration costs, which cover items such as legal fees, environmental/California Environmental Quality Act (CEQA) compliance requirements, financing expenses, administrative costs, and interest during construction.

The cost of these items can vary, but for the purpose of this study, it is assumed that the other project contingency costs will equal approximately 27.5 percent of the Estimated Construction Cost.

As shown in the following simple calculation of the Capital Improvement Cost, the total cost of all project construction contingencies (construction, engineering services, construction management, and project administration) is 65.8 percent of the Baseline Construction Cost. Note that contingencies were not applied to land acquisition costs. Calculation of the 65.8 percent is the overall mark-up on the Baseline Construction Cost to arrive at the Capital Improvement Cost. It is not an additional contingency.

Example:

Baseline Construction Cost	**$1,000,000**
Construction Contingency (30%)	$300,000
Estimated Construction Cost	**$1,300,000**
Engineering Cost (10%)	$130,000
Construction Management (10%)	$130,000
Project Administration (7.5%)	$97,500
Capital Improvement Cost	**$1,657,500**

A summary of the capital project costs for water, wastewater, and storm drainage are presented in Table 8.7, Table 8.8, and Table 8.9, respectively. These tables identify the projects, provide a brief description of each project, identify facility sizes (e.g. pipe diameter and length), and provide capital improvement costs. The columns used in Table 8.7, Table 8.8, and Table 8.9 refer to the following:

- *ID:* Assigned number that corresponds to the Proposed Improvement Table. This is an alphanumeric number that starts with one letter indicating the type of improvement (PWFF = Fire Flow Related; PWP = Potable Water Transmission Related; PWPS = Water Pump Station Related; PWS = Water Storage Tank Related; PWV = Pressure Reducing Valve; WWGM = Sewer Gravity Main; WWLS = Sewer Lift Station; WWFM = Sewer Force Main; WWRR = Sewer Rehabilitation/Replacement; SD = Storm Drain) and continues with a number
- *Types of Improvements:* Pipelines, storage tanks, booster pumps, pressure reducing valves, lift stations, force mains, rehabilitation and repair.
- *Description/Street:* Street in which the improvement is proposed.
- *Existing Size/Type:* This is the size of the existing pipeline/facility. It represents the diameter of the existing pipelines (in inches) or the size of the facility (e.g., storage tanks in MG, size of booster stations in gallons per minute (gpm), etc.).
- *Proposed Size/Type:* This is the size of the proposed improvement. It represents the diameter of the proposed pipeline (in inches) or the size of the proposed facility (e.g., storage tanks in MG, size of booster stations in gpm, etc.).
- *Replace/New:* Indicates whether the proposed improvement is a replacement pipeline, parallel pipeline, or a new facility.
- *Length:* Estimated length of the proposed improvement (in feet), if applicable. It should be noted that the length estimates do not account for re-routing the alignments to avoid unknown conditions.
- *Preliminary Project Schedule:* This is an estimated improvement project start year.

The tables also show the probable phase in which the projects would be implemented. The implementation timeframe was based on the priority of each project to correct existing deficiencies or to serve future users.

8.6 20-Year CIP

The proposed capital improvements are prioritized based on their urgency to mitigate existing deficiencies and other factors. The capital improvements were phased into one of the following phases:

- *Near Term Projects Phase 1 (Year by Year, 2019-2023):* This phase includes projects that are targeted as the highest priority improvements.
- *Near Term Projects Phase 2 (2024-2028):* This phase generally includes medium priority improvements.
- *Long Term Projects (2029-2040):* This phase includes lower priority improvements.

Each project is itemized by phase in Table 8.7 through Table 8.9. The following sections summarize the 20-year CIPs for the water distribution, sewer collection, and storm drainage systems.

It should be noted that the CIP phasing included in the 20-year CIP, and summarized in Table 8.7 through Table 8.9 is based on the project prioritization factors described in Section 8.1, and represents the preferred implementation schedule for the proposed improvements. Funding availability will limit the City's ability to implement the proposed projects according to the implementation schedule included in Table 8.7 through Table 8.9 and described in this section. Carollo worked with the City's rate study consultant to develop a 5-year CIP (2019-2023) to develop an implementable rate schedule for the water distribution system and wastewater collection systems. These 5-year CIPs are described in more detail in Section 8.7.

8.6.1 Water Distribution System 20-Year CIP Summary

The 20-year water distribution system CIP is summarized by phase and project type in Table 8.10. As shown in Table 8.10, out of the total $26.1 million in capital projects, $14.9 million are targeted for implementation in the Near Term Phase 1, and an additional $8.0 million are targeted for the Near Term Phase 2. The remaining $3.2 million of capital improvements has been included in the Long Term Phase (2029-2040). It should be noted that the potential cost of moving the water treatment plant is not included in Table 8.10. The costs associated with this project will be developed as part of a future study.

Table 8.10 and Figure 8.2 show the distribution of capital costs by project type. As shown on Figure 8.2, fire flow improvement projects and storage reservoir projects account for the largest portions of the capital improvement project costs at 50 percent and 17 percent, respectively. Other projects, transmission and distribution main projects, rehabilitation and replacement projects, and PRV stations account for roughly 12 percent, 11 percent, 9 percent, and 1 percent of the total CIP costs, respectively.

Table 8.7 20-Year Water Distribution System CIP

Pump Station		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[1][2][3][4] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($)						
								Near-Term						Long-Term
								2019	2020	2021	2022	2023	2024-2028	2029-2040
Capacity Improvements					$ 20,990,000	$ 20,806,000	$ 184,000	$ 1,094,000	$ 5,170,000	$ 2,467,000	$ 1,464,000	$ 1,746,000	$ 7,357,000	$ 1,692,000
Fire Flow Improvements		Diameter (in)	Diameter (in)	Length (ft)	$ 12,342,000	$ 12,342,000	$ -	$ 1,094,000	$ -	$ 523,000	$ 676,000	$ 1,000,000	$ 7,357,000	$ 1,692,000
PWFF-1	Pipeline along Quintana Road	8	10	3,000	$ 1,094,000	$ 1,094,000	$ -	$ 1,094,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-2	Pipeline along Quintana Road	8	16	3,300	$ 1,641,000	$ 1,641,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,641,000	$ -
PWFF-3A	Pipeline along Main Street	8	12	1,650	$ 615,000	$ 615,000	$ -	$ -	$ -	$ -	$ -	$ 615,000	$ -	$ -
PWFF-3B	Pipeline along Main Street	8	10	600	$ 219,000	$ 219,000	$ -	$ -	$ -	$ -	$ -	$ 219,000	$ -	$ -
PWFF-4	Pipeline along Ridgeway Street	6	10	830	$ 303,000	$ 303,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 303,000
PWFF-5A	Pipeline along Highway 41 and Main Street	--	8	270	$ 78,000	$ 78,000	$ -	$ -	$ -	$ -	$ 78,000	$ -	$ -	$ -
PWFF-5B	Pipeline along Highway 41 and Main Street	6	8	870	$ 252,000	$ 252,000	$ -	$ -	$ -	$ -	$ 252,000	$ -	$ -	$ -
PWFF-5C	Pipeline along Highway 41 and Main Street	6	10	950	$ 346,000	$ 346,000	$ -	$ -	$ -	$ -	$ 346,000	$ -	$ -	$ -
PWFF-6A	Pipeline along Highway 41 and Main Street	--	8/20	270	$ 211,000	$ 211,000	$ -	$ -	$ -	$ 211,000	$ -	$ -	$ -	
PWFF-6B	PRV from Upper Kings Zone to Elena Zone	--	8	--	$ 312,000	$ 312,000	$ -	$ -	$ -	$ 312,000	$ -	$ -	$ -	
PWFF-7	Pipeline along Juniper Avenue	6	8	570	$ 166,000	$ 166,000	$ -	$ -	$ -	$ -	$ -	$ 166,000	$ -	
PWFF-8A	Pipeline along San Joaquin Street	--	8	240	$ 70,000	$ 70,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 70,000	
PWFF-8B	Pipeline along San Joaquin Street	6	8	230	$ 66,000	$ 66,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 66,000	$ -
PWFF-8C	Pipeline along San Joaquin Street	--	8	100	$ 30,000	$ 30,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 30,000	$ -
PWFF-8D	Pipeline along San Joaquin Street	--	10	970	$ 353,000	$ 353,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 353,000	$ -
PWFF-8E	PRV on San Joaquin Street	--	8	--	$ 312,000	$ 312,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 312,000	$ -
PWFF-9A	Pipeline along Greenwood Avenue and San Jacinto Street	6	10	640	$ 234,000	$ 234,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 234,000	$ -
PWFF-9B	Pipeline along Greenwood Avenue and San Jacinto Street	--	10	850	$ 310,000	$ 310,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 310,000	$ -
PWFF-10	Pipeline along Sequoia Street	8	10	540	$ 197,000	$ 197,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 197,000	$ -
PWFF-11	Pipeline along Las Vegas Street and Main Street	6	8	1,610	$ 467,000	$ 467,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 467,000
PWFF-12	Pipeline along Highway 41 and Hill Street	--	8	1,290	$ 375,000	$ 375,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 375,000
PWFF-13	Pipeline along Ponderosa Street	6	8	910	$ 264,000	$ 264,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 264,000	$ -
PWFF-14	Pipeline along Mimosa Street	6	8	580	$ 169,000	$ 169,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 169,000	$ -
PWFF-15A	Pipeline along Surf Street	6	8	640	$ 186,000	$ 186,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 186,000	$ -
PWFF-15B	Pipeline along Surf Street	--	8	420	$ 123,000	$ 123,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 123,000	$ -
PWFF-16	Pipeline along Piney Way and Alta Court	6	8	910	$ 264,000	$ 264,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 264,000	$ -
PWFF-17	Pipeline from Kings Avenue to Arbutus Avenue	--	8	1,050	$ 305,000	$ 305,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 305,000
PWFF-18	Pipeline along Bolton Drive	6	8	700	$ 204,000	$ 204,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 204,000	$ -
PWFF-19	Pipeline along Norwich Avenue	--	8	220	$ 65,000	$ 65,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 65,000	$ -
PWFF-20	Pipeline along Blanca Street and Tuscan Avenue	6	8	450	$ 131,000	$ 131,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 131,000	$ -
PWFF-21	Pipeline along Toro Lane	6	8	800	$ 232,000	$ 232,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 232,000	$ -
PWFF-22	Pipeline along Sequoia Court and Ironwood Avenue	6	8	385	$ 111,000	$ 111,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 111,000
PWFF-23	Pipeline along Paula Street	--	8	200	$ 58,000	$ 58,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 58,000	$ -
PWFF-24	Pipeline along Reno Court	6	8	450	$ 131,000	$ 131,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 131,000
PWFF-25	Pipeline along Errol Street	4	8	545	$ 157,000	$ 157,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 157,000	$ -
PWFF-26	Pipeline along Napa Avenue	6	8	210	$ 61,000	$ 61,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 61,000	$ -
PWFF-27A	Pipeline along Marina Street	2	8	220	$ 65,000	$ 65,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 65,000	$ -
PWFF-27B	Pipeline along Marina Street	--	8	115	$ 33,000	$ 33,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 33,000	$ -
PWFF-28	Pipeline along Driftwood Street	--	8	370	$ 108,000	$ 108,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 108,000	$ -
PWFF-29	Pipeline along South Street	--	8	300	$ 88,000	$ 88,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 88,000	$ -
PWFF-30	Pipeline along Piney Lane	6	8	370	$ 108,000	$ 108,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 108,000	$ -
PWFF-31	PRV on Avalon Street and Ironwood Avenue	--	8	--	$ 312,000	$ 312,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 312,000	$ -
PWFF-32	PRV on Highway 41 and Ironwood Avenue	--	8	--	$ 312,000	$ 312,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 312,000	$ -
PWFF-33	PRV on Dunes Street and Shasta Avenue	--	8	--	$ 312,000	$ 312,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 312,000	$ -
PWFF-34	PRV on Main Street and Quintana Place	--	8	--	$ 312,000	$ 312,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 312,000	$ -

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Table 8.7 20-Year Water Distribution System CIP (continued)

Pump Station		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[(1)(2)(3)(4)] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($)						
								Near-Term						Long-Term
								2019	2020	2021	2022	2023	2024-2028	2029-2040
PWFF-35	Pipeline along Sunset Road	6	8	--	$ 580,000	$ 580,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 580,000	$ -
Transmission & Distribution Main		Diameter (in)	Diameter (in)	Length (ft)	$ 2,658,000	$ 2,558,000	$ 100,000	$ -	$ 1,275,000	$ 595,000	$ 788,000	$ -	$ -	$ -
PWP-1	Fill line for Blanca Tanks	4	8	2,210	$ 750,000	$ 735,000	$ 15,000	$ -	$ 750,000	$ -	$ -	$ -	$ -	$ -
PWP-2	Fill line for Nutmeg Tank	4	8	1,280	$ 371,000	$ 364,000	$ 7,000	$ -	$ 371,000	$ -	$ -	$ -	$ -	$ -
PWP-3	Fill line for Elena Tanks	--	10	1,730	$ 788,000	$ 725,000	$ 63,000	$ -	$ -	$ -	$ 788,000	$ -	$ -	$ -
PWP-4	Parallel pipeline on Juniper Avenue	--	8	2,050	$ 595,000	$ 583,000	$ 12,000	$ -	$ -	$ 595,000	$ -	$ -	$ -	$ -
PWP-5	Pipeline on Sequoia Street	--	8	530	$ 154,000	$ 151,000	$ 3,000	$ -	$ 154,000	$ -	$ -	$ -	$ -	$ -
Pump Station		Capacity (mgd)	Capacity (mgd)	Length (ft)	$ 1,326,000	$ 1,314,000	$ 12,000	$ -	$ 580,000	$ -	$ -	$ 746,000	$ -	$ -
PWPS-1	Elena Booster Pump Station Upgrade	0.44	0.50	--	$ 580,000	$ 568,000	$ 12,000	$ -	$ 580,000	$ -	$ -	$ -	$ -	$ -
PWPS-2	Kings Booster Pump Station Upgrade	--	2.16	--	$ 746,000	$ 746,000	$ -	$ -	$ -	$ -	$ -	$ 746,000	$ -	$ -
Storage Reservoir		Capacity (MG)	Capacity (MG)	Length (ft)	$ 4,352,000	$ 4,286,000	$ 66,000	$ -	$ 3,315,000	$ 1,037,000	$ -	$ -	$ -	$ -
PWS-1	Nutmeg Tank Upgrade	0.14	1	--	$ 3,315,000	$ 3,249,000	$ 66,000	$ -	$ 3,315,000	$ -	$ -	$ -	$ -	$ -
PWS-2	Elena Tank Upgrade	0.12	0.15	--	$ 1,037,000	$ 1,037,000	$ -	$ -	$ -	$ 1,037,000	$ -	$ -	$ -	$ -
PRV Station		Diameter (in)	Diameter (in)	Length (ft)	$ 312,000	$ 306,000	$ 6,000	$ -	$ -	$ 312,000	$ -	$ -	$ -	$ -
PWV-1	PRV on Juniper Avenue	--	8	--	$ 312,000	$ 306,000	$ 6,000	$ -	$ -	$ 312,000	$ -	$ -	$ -	$ -
Rehabilitation and Replacement Projects					$ 2,200,000	$ 2,200,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 500,000	$ 1,200,000
PWRR-1	Pipeline R&R Program	1-16	> 6	--	$ 2,200,000	$ 2,200,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 500,000	$ 1,200,000
Other Projects					$ 2,895,000	$ 2,895,000	$ -	$ 77,000	$ 746,000	$ 1,472,000	$ -	$ 150,000	$ 150,000	$ 300,000
PWO-1	Water Master Plan Update	--	--	--	$ 600,000	$ 600,000	$ -	$ -	$ -	$ -	$ -	$ 150,000	$ 150,000	$ 300,000
PWO-2	Re-Skin Desalination Plant Building	--	--	--	$ 60,000	$ 60,000	$ -	$ 60,000	$ -	$ -	$ -	$ -	$ -	$ -
PWO-3	Control Upgrades at Desalination Plant	--	--	--	$ 1,472,000	$ 1,472,000	$ -	$ -	$ -	$ 1,472,000	$ -	$ -	$ -	$ -
PWO-4	Add Screening at Desalination Plant	--	--	--	$ 17,000	$ 17,000	$ -	$ 17,000	$ -	$ -	$ -	$ -	$ -	$ -
PWO-5	Chlorination Upgrades	--	--	--	$ 746,000	$ 746,000	$ -	$ -	$ 746,000	$ -	$ -	$ -	$ -	$ -
PWO-6	Desalination Plant Relocation	--			$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
CIP Total					**$ 26,085,000**	**$ 25,901,000**	**$ 184,000**	**$ 1,271,000**	**$ 6,016,000**	**$ 4,039,000**	**$ 1,564,000**	**$ 1,996,000**	**$ 8,007,000**	**$ 3,192,000**
Annual Cost					**N/A**	**N/A**	**N/A**	**$ 1,271,000**	**$ 6,016,000**	**$ 4,039,000**	**$ 1,564,000**	**$ 1,996,000**	**$ 1,601,400**	**$ 266,000**

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

(4) Total Mark-Up is 65.8% of the baseline construction costs.

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Table 8.8 20-Year Wastewater Collection System CIP

Project		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[1][2][3][4] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($)						
								Near-Term						Long-Term
								2019	2020	2021	2022	2023	2024-2028	2029-2040
Capacity Related Improvements					$ 6,513,000	$ 6,149,000	$ 364,000	$ 530,000	$ 1,371,000	$ 905,000	$ 660,000	$ 43,000	$ 933,000	$ 2,071,000
Gravity Mains		Diameter (in)	Diameter (in)	Length (ft)	$ 4,994,000	$ 4,630,000	$ 364,000	$ 530,000	$ 1,371,000	$ 905,000	$ 660,000	$ 43,000	$ 933,000	$ 552,000
WWGM-1	Gravity Main along Atascadero Road	18	27	1,000	$ 530,000	$ 463,000	$ 67,000	$ 530,000	$ -	$ -	$ -	$ -	$ -	$ -
WWGM-2	Gravity Main along Main Street	15	24	2,900	$ 1,371,000	$ 1,194,000	$ 177,000	$ -	$ 1,371,000	$ -	$ -	$ -	$ -	$ -
WWGM-3	Gravity Main along Main Street	12	18	1,600	$ 544,000	$ 544,000	$ -	$ -	$ -	$ -	$ 544,000	$ -	$ -	$ -
WWGM-4A	Gravity Main along San Joaquin Street	6	12	150	$ 43,000	$ 41,000	$ 2,000	$ -	$ -	$ -	$ -	$ 43,000	$ -	$ -
WWGM-4B	Gravity Main along Alder Avenue, and San Jacinto Street	6	10	1,200	$ 328,000	$ 301,000	$ 27,000	$ -	$ -	$ -	$ -	$ -	$ 328,000	$ -
WWGM-5	Gravity Main along Greenwood Avenue	6	12	1,850	$ 537,000	$ 537,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 537,000	$ -
WWGM-6	Gravity Main along Coral Avenue	10	12	1,900	$ 552,000	$ 552,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 552,000
WWGM-7	Gravity Main along Sienna Street	6	10	250	$ 68,000	$ 68,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 68,000	$ -
WWGM-8A	Gravity Main along Main Street	10 & 12	15	2,600	$ 819,000	$ 728,000	$ 91,000	$ -	$ -	$ 819,000	$ -	$ -	$ -	$ -
WWGM-8B	Gravity Main along Main Street	8 & 10	12	400	$ 116,000	$ 116,000	$ -	$ -	$ -	$ -	$ 116,000	$ -	$ -	$ -
WWGM-9	Gravity Main under Highway 41	6	8/20	110	$ 86,000	$ 86,000	$ -	$ -	$ -	$ 86,000	$ -	$ -	$ -	$ -
Lift Stations		Capacity (mgd)	Capacity (mgd)		$ 1,439,000	$ 1,439,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,439,000
WWLS-1	Lift Station 1 Replacement	0.98	1.65	N/A	$ 1,439,000	$ 1,439,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,439,000
Force Main		Diameter (in)	Diameter (in)	Length (ft)	$ 80,000	$ 80,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 80,000
WWFM-1	Lift Station 1 Force Main	6	8	300	$ 80,000	$ 80,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 80,000
Rehabilitation and Replacement Projects					$ 10,818,000	$ 10,818,000	$ -	$ 1,029,000	$ 967,000	$ 702,000	$ 702,000	$ 702,000	$ 3,510,000	$ 3,206,000
Gravity Mains		Diameter (in)	Diameter (in)	Length (ft)	$ 8,298,000	$ 8,298,000	$ -	$ 780,000	$ 702,000	$ 702,000	$ 702,000	$ 702,000	$ 3,510,000	$ 1,200,000
WWRR-1A	Gravity Main Beachcomber Drive	12	12	570	$ 166,000	$ 166,000	$ -	$ 166,000	$ -	$ -	$ -	$ -	$ -	$ -
WWRR-1B	Gravity Main Beachcomber Drive	10	10	960	$ 262,000	$ 262,000	$ -	$ 262,000	$ -	$ -	$ -	$ -	$ -	$ -
WWRR-1C	Gravity Main Beachcomber Drive	8	8	950	$ 252,000	$ 252,000	$ -	$ 252,000	$ -	$ -	$ -	$ -	$ -	$ -
WWRR-2	Pipe R&R Program Upstream of LS-1	6-12	8-12	27,000	$ 5,418,000	$ 5,418,000	$ -	$ -	$ 602,000	$ 602,000	$ 602,000	$ 602,000	$ 3,010,000	$ -
WWRR-3	I/I Reduction Projects	4-18	> 6	Varies	$ 2,200,000	$ 2,200,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 500,000	$ 1,200,000
Structures		Type	Type	Quantity	$ 2,520,000	$ 2,520,000	$ -	$ 249,000	$ 265,000	$ -	$ -	$ -	$ -	$ 2,006,000
WWRR-4	Cap Replacement Upstream of LS-1	Cap	Manhole	31	$ 514,000	$ 514,000	$ -	$ 249,000	$ 265,000	$ -	$ -	$ -	$ -	$ -
WWRR-5	Cap Replacement Program	Cap	Manhole	121	$ 2,006,000	$ 2,006,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,006,000
Other Projects					$ 840,000	$ 840,000	$ -	$ -	$ -	$ -	$ -	$ 210,000	$ 210,000	$ 420,000
WWO-1	Sewer Master Plan Update	--	--	--	$ 600,000	$ 600,000	$ -	$ -	$ -	$ -	$ -	$ 150,000	$ 150,000	$ 300,000
WWO-2	Flow Monitoring Program	--	4 weeks	15 FM	$ 240,000	$ 240,000	$ -	$ -	$ -	$ -	$ -	$ 60,000	$ 60,000	$ 120,000
CIP Total					**$ 18,171,000**	**$ 17,807,000**	**$ 364,000**	**$ 1,559,000**	**$ 2,338,000**	**$ 1,607,000**	**$ 1,362,000**	**$ 955,000**	**$ 4,653,000**	**$ 5,697,000**
Annual Cost					**N/A**	**N/A**	**N/A**	**$ 1,559,000**	**$ 2,338,000**	**$ 1,607,000**	**$ 1,362,000**	**$ 955,000**	**$ 931,000**	**$ 475,000**

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

(4) Total Mark-Up is 65.8% of the baseline construction costs.

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Table 8.9 20-Year Storm Drainage System CIP

Project		10-Year/100-Year Improvement	Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[1][2][3][4] ($)	CIP Phasing ($)						
							Near-Term						Long-Term
							2019	2020	2021	2022	2023	2024-2028	2029-2040
Capacity Related Improvements						$ 10,847,000	$ 1,596,000	$ 3,617,000	$ 517,000	$ 684,000	$ 716,000	$ 1,528,000	$ 2,189,000
Gravity Mains			Diameter (in)	Diameter (in)	Length (ft)	$ 10,847,000	$ 1,596,000	$ 3,617,000	$ 517,000	$ 684,000	$ 716,000	$ 1,528,000	$ 2,189,000
SD-01A	Pipeline along Kern Street and Anchor Street	10-Year	--	27	125	$ 66,000	$ 66,000	$ -	$ -	$ -	$ -	$ -	$ -
SD-01B	Pipeline along Kern Street and Anchor Street	10-Year	--	30	2,600	$ 1,530,000	$ 1,530,000	$ -	$ -	$ -	$ -	$ -	$ -
SD-02A	Pipeline along Marina Street	10-Year	--	12	365	$ 106,000	$ -	$ -	$ -	$ -	$ 106,000	$ -	$ -
SD-02B	Pipeline along Marina Street	10-Year	--	15	600	$ 189,000	$ -	$ -	$ -	$ -	$ 189,000	$ -	$ -
SD-03	Pipeline along Harbor Street	10-Year	--	24	890	$ 421,000	$ -	$ -	$ -	$ -	$ 421,000	$ -	$ -
SD-04	Pipeline at Morro Bay Boulevard and Quintana Road	10-Year	12	18	130	$ 45,000	$ -	$ -	$ 45,000	$ -	$ -	$ -	$ -
SD-05	Pipeline along Beach Street	10-Year	--	24	1,000	$ 472,000	$ -	$ -	$ 472,000	$ -	$ -	$ -	$ -
SD-06A	Pipeline along Cabrillo Highway	10-Year	--	48	1,140	$ 1,134,000	$ -	$ 1,134,000	$ -	$ -	$ -	$ -	$ -
SD-06B	Pipeline along Cabrillo Highway	10-Year	--	54	2,340	$ 2,483,000	$ -	$ 2,483,000	$ -	$ -	$ -	$ -	$ -
SD-07A	Pipeline along Main Street	10-Year	12	15	360	$ 113,000	$ -	$ -	$ -	$ 113,000	$ -	$ -	$ -
SD-07B	Pipeline along Main Street	10-Year	12	24	230	$ 109,000	$ -	$ -	$ -	$ 109,000	$ -	$ -	$ -
SD-07C	Pipeline along Main Street	10-Year	12	15	50	$ 17,000	$ -	$ -	$ -	$ 17,000	$ -	$ -	$ -
SD-08	Pipeline between Main Street and Cabrillo Highway	10-Year	18	27	190	$ 101,000	$ -	$ -	$ -	$ -	$ -	$ 101,000	$ -
SD-09	Pipeline at Main Street and Atascadero Road	10-Year	--	24	85	$ 40,000	$ -	$ -	$ -	$ -	$ -	$ 40,000	$ -
SD-10	Pipeline at Cabrillo Highway and Atascadero Road	10-Year	--	24	80	$ 38,000	$ -	$ -	$ -	$ -	$ -	$ 38,000	$ -
SD-11	Pipeline along Laurel Street	10-Year	6	12	80	$ 23,000	$ -	$ -	$ -	$ 23,000	$ -	$ -	$ -
SD-12	Pipeline along Avalon Street	10-Year	16	24	200	$ 94,000	$ -	$ -	$ -	$ 94,000	$ -	$ -	$ -
SD-13	Pipeline between Sunset Avenue and Main Street	10-Year	20	42	260	$ 237,000	$ -	$ -	$ -	$ -	$ -	$ 237,000	$ -
SD-14	Pipeline along Sienna Street	10-Year	24	36	450	$ 328,000	$ -	$ -	$ -	$ -	$ -	$ 328,000	$ -
SD-15	Pipeline perpendicular to Oceanside Drive and Driftwood Avenue	10-Year	--	48	240	$ 239,000	$ -	$ -	$ -	$ -	$ -	$ 239,000	$ -
SD-16	Pipeline along Elena Street	10-Year	24	36	450	$ 328,000	$ -	$ -	$ -	$ 328,000	$ -	$ -	$ -
SD-17	Pipeline along Main Street	100-Year	18	24	120	$ 56,000	$ -	$ -	$ -	$ -	$ -	$ 56,000	$ -
SD-18A	Pipeline along Sandalwood Avenue and Sienna Street	100-Year	24	30	320	$ 189,000	$ -	$ -	$ -	$ -	$ -	$ 189,000	$ -
SD-18B	Pipeline along Sandalwood Avenue and Sienna Street	100-Year	12	18	25	$ 8,000	$ -	$ -	$ -	$ -	$ -	$ 8,000	$ -
SD-19	Pipeline along Elena Street	100-Year	24	30	500	$ 295,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 295,000
SD-20	Pipeline along Elena Street (under Cabrillo Highway)	100-Year	24	36	360	$ 597,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 597,000
SD-21	Pipeline at Avalon Street and Ironwood Avenue	100-Year	16	24	80	$ 38,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 38,000
SD-22	Pipeline along Bayview Avenue	100-Year	18	24	210	$ 99,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 99,000
SD-23	Pipeline along Hillview Street	100-Year	30	36	260	$ 189,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 189,000
SD-24	Pipeline near Ponderosa Street	100-Year	10	15	90	$ 28,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 28,000
SD-25	Pipeline near Mimosa Street	100-Year	12	15	105	$ 33,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 33,000
SD-26A	Pipeline near Miner's Ace Hardware and Morro Creek	100-Year	12	15	90	$ 28,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 28,000
SD-26B	Pipeline near Miner's Ace Hardware and Morro Creek	100-Year	12	18	150	$ 51,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 51,000
SD-27	Pipeline at Sunset Avenue and Atascadero Road	100-Year	18	24	60	$ 28,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 28,000
SD-28	Pipeline along Atascadero Road (underpass)	100-Year	24	36	430	$ 313,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 313,000
SD-29	Pipeline along Main Street	100-Year	12	18	170	$ 58,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 58,000
SD-30A	Pipeline along Preston Lane	100-Year	6	15	40	$ 13,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 13,000
SD-30B	Pipeline along Preston Lane	100-Year	6	18	220	$ 75,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 75,000

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Table 8.9 20-Year Storm Drainage System CIP (continued)

Project		10-Year/100-Year Improvement	Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[1][2][3][4] ($)	CIP Phasing ($)						
							Near-Term						Long-Term
							2019	2020	2021	2022	2023	2024-2028	2029-2040
SD-31	Pipeline along Main Street (near Main Street on ramp)	100-Year	18	24	80	$ 38,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 38,000
SD-32A	Pipeline near parking lot west of Surf Street and West Avenue	100-Year	12	18	60	$ 20,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 20,000
SD-32B	Pipeline near parking lot west of Surf Street and West Avenue	100-Year	18	24	300	$ 143,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 143,000
SD-33	Pipeline along Main Street	100-Year	24	30	140	$ 83,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 83,000
SD-34	Pipeline at Bonita Street and Main Street	100-Year	24	30	100	$ 60,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 60,000
SD-35	Pipeline along Main Street	10-Year	--	15	270	$ 85,000	$ -	$ -	$ -	$ -	$ -	$ 85,000	$ -
SD-36	Pipeline near Morro Bay Boulevard off ramp	10-Year	24	24	150	$ 207,000	$ -	$ -	$ -	$ -	$ -	$ 207,000	$ -
Rehabilitation and Replacement Projects						$ 2,200,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 500,000	$ 1,200,000
SDRR-1	Pipeline R&R Program	--	Varies	>6	Varies	$ 2,200,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 500,000	$ 1,200,000
Other Projects						$ 600,000	$ -	$ -	$ -	$ -	$ 150,000	$ 150,000	$ 300,000
SDO-1	Storm Drain Master Plan Update	--	--	--	--	$ 600,000	$ -	$ -	$ -	$ -	$ 150,000	$ 150,000	$ 300,000
CIP Total						**$ 13,647,000**	**$ 1,696,000**	**$ 3,717,000**	**$ 617,000**	**$ 784,000**	**$ 966,000**	**$ 2,178,000**	**$ 3,689,000**
Annual Cost						**N/A**	**$ 1,696,000**	**$ 3,717,000**	**$ 617,000**	**$ 784,000**	**$ 966,000**	**$ 436,000**	**$ 307,000**

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

(4) Total Mark-Up is 65.8% of the baseline construction costs.

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Table 8.10 20-Year Water System CIP Summary

Improvement Type	CIP Cost Estimate[(1),(2),(3),(4)] ($)	Project Cost Phasing ($)		
		Near Term Ph. 1 2019-23	Near Term Ph. 2 2024-28	Long Term 2029-40[(5)]
Capacity Improvements				
Fire Flow	$12,342,000	$3,293,000	$7,357,000	$1,692,000
Transmission and Distribution	$2,658,000	$2,658,000	$-	$-
Storage Reservoir	$4,352,000	$4,352,000	$-	$-
PRV Stations	$312,000	$312,000	$-	$-
Capacity Subtotal	$20,990,000	$11,941,000	$7,357,000	$1,692,000
Rehabilitation and Replacement Projects				
Rehabilitation and Replacement Projects	$2,200,000	$500,000	$500,000	$1,200,000
Other Projects				
Other Projects	$2,895,000	$2,445,000	$150,000	$300,000
Total CIP	**$26,085,000**	**$14,886,000**	**$8,007,000**	**$3,192,000**

Notes:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889
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(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.
(4) Total Mark-Up is 65.8% of the baseline construction costs.
(5) The cost associated with relocating the water treatment plant was assumed to be zero for the purposes of this Master Plan. Costs associated with this project will be determined in the future.

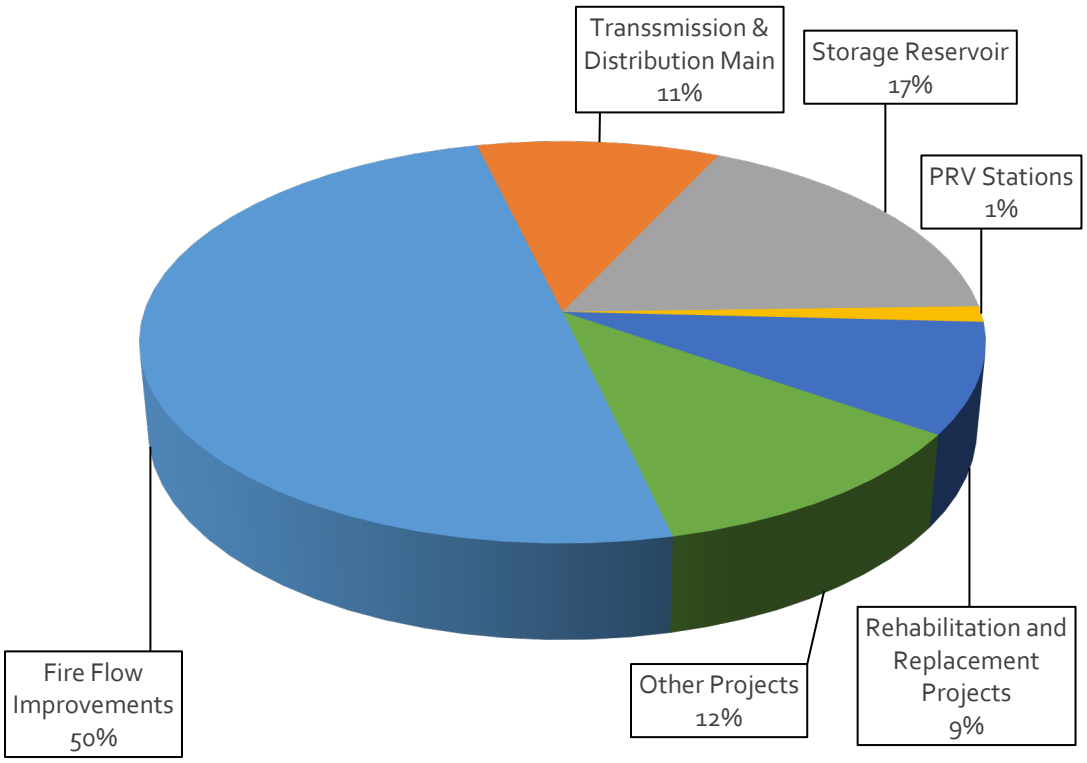

Figure 8.2 20-Year Water System CIP by Project Type

8.6.2 Wastewater Collection System 20-Year CIP Summary

The 20-year wastewater collection system CIP is summarized by phase and project type in Table 8.11. As shown in Table 8.11, out of the total $18.2 million in capital projects, $7.8 million are targeted for implementation in the Near Term Phase 1, and an additional $4.7 million are targeted for the Near Term Phase 2. The remaining $5.7 million of capital improvements has been included in the Long Term Phase (2029-2040).

Table 8.11 and Figure 8.3 show the distribution of capital costs by project type. As shown on Figure 8.3, gravity main rehabilitation and replacement and gravity main capacity upgrades account for the largest portions of the capital improvement project costs at 46 percent and 27 percent, respectively. End cap replacement projects, lift stations, other projects, and force mains account for roughly 14 percent, 8 percent, 5 percent, and less than 1 percent of the total CIP costs, respectively.

Table 8.11 20-Year Wastewater Collection System CIP Summary

Improvement Type	CIP Cost Estimate[(1),(2),(3),(4)] ($)	Project Cost Phasing ($)		
		Near Term Ph. 1 2019-23	Near Term Ph. 2 2024-28	Long Term 2029-40[(5)]
Capacity Improvements				
Fire Flow	$4,994,000	$3,509,000	$933,000	$552,000
Transmission and Distribution	$1,439,000	$-	$-	$1,439,000
Storage Reservoir	$80,000	$-	$-	$80,000
Capacity Subtotal	$6,513,000	$3,509,000	$933,000	$2,071,000
Rehabilitation and Replacement Projects				
Rehabilitation and Replacement Projects	$8,298,000	$3,588,000	$3,510,000	$1,200,000
End Cap Replacement	$2,520,000	$514,000	$-	$2,006,000
Rehab./Replace. Subtotal	$10,818,000	$4,102,000	$3,510,000	$3,206,000
Other Projects				
Other Projects	$840,000	$210,000	$210,000	$420,000
Total CIP	**$18,171,000**	**$7,821,000**	**$4,653,000**	**$5,697,000**

Notes:
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(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.
(4) Total Mark-Up is 65.8% of the baseline construction costs.

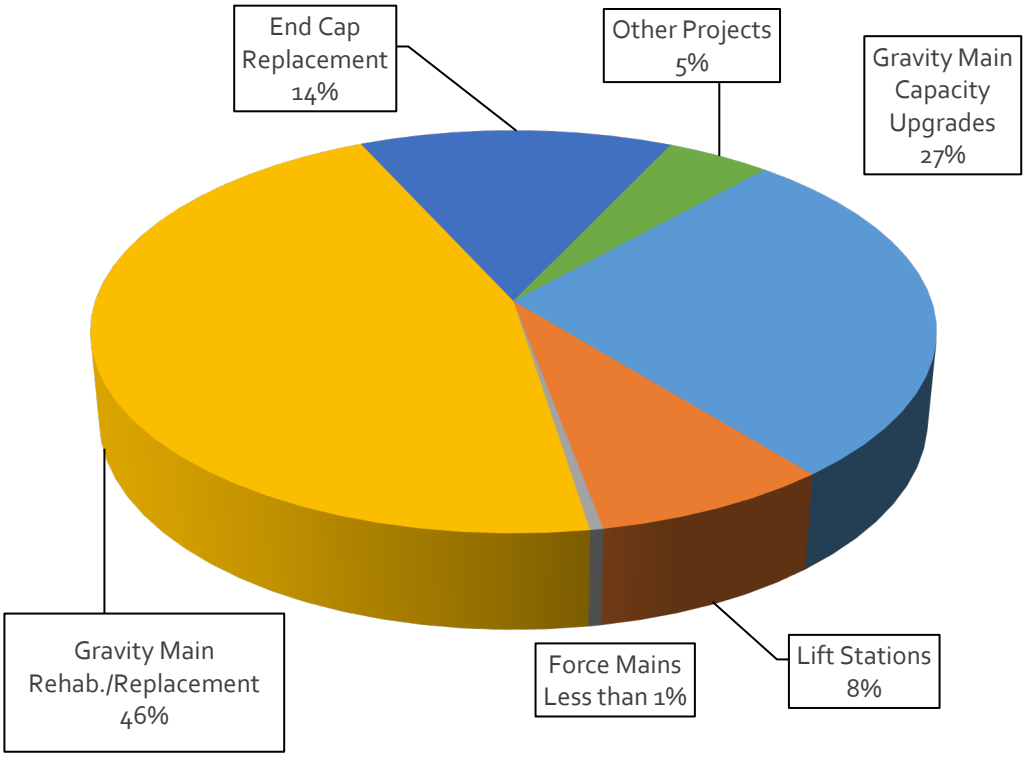

Figure 8.3 20-Year Wastewater Collection System CIP by Project Type

8.6.3 Storm Drainage System 20-Year CIP Summary

The 20-year storm drainage system CIP is summarized by phase and project type in Table 8.12. As shown in Table 8.12, out of the total $13.6 million in capital projects, $7.8 million are targeted for implementation in the Near Term Phase 1, and an additional $2.2 million are targeted for the Near Term Phase 2. The remaining $3.7 million of capital improvements has been included in the Long Term Phase (2029-2040).

Table 8.12 and Figure 8.3 show the distribution of capital costs by project type. As shown on Figure 8.3, projects needed to mitigate the 10-year design storm deficiencies account for the largest portion of the capital improvement project costs at 62 percent. Projects needed to mitigate the 100-year design storm, rehabilitation and replacement of failing storm drains, and other projects account for roughly 18 percent, 16 percent, and 4 percent of the total CIP costs, respectively.

Table 8.12 20-Year Storm Drainage System CIP Summary

Improvement Type	CIP Cost Estimate[(1),(2),(3),(4)] ($)	Project Cost Phasing ($)		
		Near Term Ph. 1 2019-23	Near Term Ph. 2 2024-28	Long Term 2029-40[(5)]
Projects to Mitigate 10-Year Storm Deficiencies	$8,405,000	$7,130,000	$1,275,000	$-
Projects to Mitigate 100-Year Storm Deficiencies	$2,442,000	$-	$253,000	$2,189,000
Rehabilitation and Replacement Projects	$2,200,000	$500,000	$500,000	$1,200,000
Other Projects	$600,000	$150,000	$150,000	$300,000
Total CIP	**$13,647,000**	**$7,780,000**	**$2,178,000**	**$3,689,000**

Notes:
(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889
(2) Estimated Construction Cost includes a 30% contingency of the baseline construction cost.
(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.
(4) Total Mark-Up is 65.8% of the baseline construction costs.

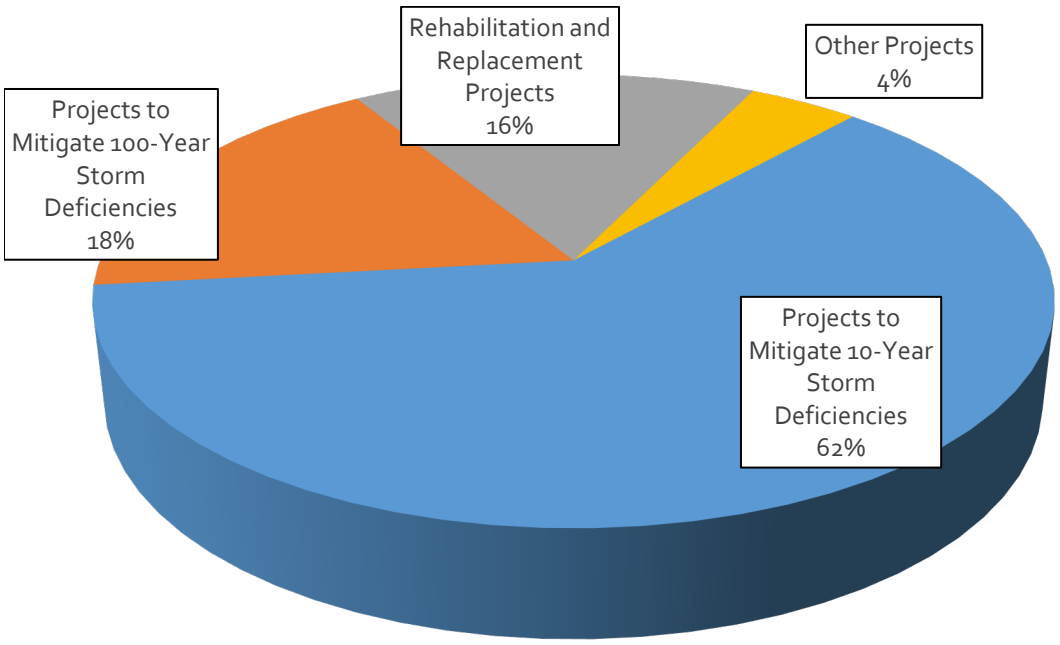

Figure 8.4 20-Year Storm Drainage System CIP by Project Type

8.7 Revised 5-Year CIP

On April 23, 2018, the City appointed four citizens from Morro Bay to a Blue Ribbon Commission (BRC) to participate in the development of new rates for the Water Reclamation Facility (WRF) project. The BRC's mission statement was: "The WRF Blue Ribbon Commission is committed to independently evaluating the costs of the major components of the WRF program and water and sewer capital project needs and provide a recommendation to the City Manager for equitable and reasonable customer rates to pay for those needs."

Over the course of several months, the City, Carollo, BRC and the City's rate consultant held eight (8) meetings to review the costs for the WRF, the CIP projects, and current and future operations and maintenance costs to minimize the impacts of any new rates on the community. One of the BRC's primary activities was review of the 20-year CIPs described in Section 8.6. Table 8.13 and Table 8.14 show the revised 5-year CIPs for the water distribution and wastewater collection systems that were modified based on discussions with the BRC. These revised CIPs prioritize the most critical projects for the water distribution and wastewater collection systems such as the replacement of the Nutmeg Tank and the replacement of the gravity main on Main Street from San Joaquin Street to Atascadero Road, respectively. The findings of the BRC are summarized in their report dated June 21, 2018. It should be noted that while the 5-year CIPs have been revised, the overall cost and scope of the 20-year CIP is unchanged. These projects in the 5-year CIP were included in the City's rate study for the determination of water and sewer rates. As shown in Table 8.13 and Table 8.14, the City has included approximately 6.8 million dollars and 5.1 million dollars in the 5-year water and wastewater collection system CIPs, respectively.

Table 8.13 5-Year Revised Water System CIP

Pump Station		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[(1)(2)(3)(4)] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($)				
								Near-Term				
								2019	2020	2021	2022	2023
Capacity Improvements					$ 7,115,000	$ 7,060,000	$ 55,000	$ 904,000	$ 927,000	$ 1,224,000	$ 1,697,000	$ 1,326,000
Transsmission & Distribution Main		Diameter (in)	Diameter (in)	Length (ft)	$ 1,870,000	$ 1,833,000	$ 37,000	$ 904,000	$ 595,000	$ -	$ 371,000	$ -
PWP-1	Fill line for Blanca Tanks	4	8	2,210	$ 750,000	$ 735,000	$ 15,000	$ 750,000	$ -	$ -	$ -	$ -
PWP-2	Fill line for Nutmeg Tank	4	8	1,280	$ 371,000	$ 364,000	$ 7,000	$ -	$ -	$ -	$ 371,000	$ -
PWP-4	Parallel pipeline on Juniper Avenue	--	8	2,050	$ 595,000	$ 583,000	$ 12,000	$ -	$ 595,000	$ -	$ -	$ -
PWP-5	Pipeline on Sequoia Street	--	8	530	$ 154,000	$ 151,000	$ 3,000	$ 154,000	$ -	$ -	$ -	$ -
Pump Station		Capacity (mgd)	Capacity (mgd)	Length (ft)	$ 580,000	$ 568,000	$ 12,000	$ -	$ -	$ 580,000	$ -	$ -
PWPS-1	Elena Booster Pump Station Upgrade	0.44	0.50	--	$ 580,000	$ 568,000	$ 12,000	$ -	$ -	$ 580,000	$ -	$ -
Storage Reservoir		Capacity (MG)	Capacity (MG)	Length (ft)	$ 4,353,000	$ 4,353,000	$ -	$ -	$ 332,000	$ 332,000	$ 1,326,000	$ 1,326,000
PWS-1	Nutmeg Tank Upgrade	0.14	1	--	$ 3,316,000	$ 3,316,000	$ -	$ -	$ 332,000	$ 332,000	$ 1,326,000	$ 1,326,000
PRV Station		Diameter (in)	Diameter (in)	Length (ft)	$ 312,000	$ 306,000	$ 6,000	$ -	$ -	$ 312,000	$ -	$ -
PWV-1	PRV on Juniper Avenue	--	8	--	$ 312,000	$ 306,000	$ 6,000	$ -	$ -	$ 312,000	$ -	$ -
Rehabilitation and Replacement Projects					$ 500,000	$ 500,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000
PWRR-1	Pipeline R&R Program	1-16	> 6	--	$ 500,000	$ 500,000	$ -	$ 100,000	$ 100,000	$ 100,000	$ 100,000	$ 100,000
Other Projects					$ 210,000	$ 210,000	$ -	$ 60,000	$ -	$ -	$ -	$ 150,000
PWO-1	Water Master Plan Update	--	--	--	$ 150,000	$ 150,000	$ -	$ -	$ -	$ -	$ -	$ 150,000
PWO-2	Re-Skin Desalination Plant Building	--	--	--	$ 60,000	$ 60,000	$ -	$ 60,000	$ -	$ -	$ -	$ -
CIP Total					**$ 6,788,000**	**$ 7,770,000**	**$ 55,000**	**$ 1,064,000**	**$ 1,027,000**	**$ 1,324,000**	**$ 1,797,000**	**$ 1,576,000**
Annual Cost					**N/A**	**N/A**	**N/A**	**$ 1,064,000**	**$ 1,027,000**	**$ 1,324,000**	**$ 1,797,000**	**$ 1,576,000**

Notes:

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(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

(4) Total Mark-Up is 65.8% of the baseline construction costs.

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Table 8.14 5-Year Revised Wastewater Collection System CIP

Project		Existing Size/Type	Proposed Size/Type	Proposed Amount	CIP Cost Estimate[(1)(2)(3)(4)] ($)	Existing User Cost ($)	Future User Cost ($)	CIP Phasing ($) Near-Term 2019	2020	2021	2022	2023
Capacity Related Improvements					$ 2,488,000	$ 2,242,000	$ 246,000	$ 530,000	$ -	$ 1,371,000	$ 587,000	$ -
Gravity Mains		Diameter (in)	Diameter (in)	Length (ft)	$ 2,488,000	$ 2,242,000	$ 246,000	$ 530,000	$ -	$ 1,371,000	$ 587,000	$ -
WWGM-1	Gravity Main along Atascadero Road	18	27	1,000	$ 530,000	$ 463,000	$ 67,000	$ 530,000	$ -	$ -	$ -	$ -
WWGM-2	Gravity Main along Main Street	15	24	2,900	$ 1,371,000	$ 1,194,000	$ 177,000	$ -	$ -	$ 1,371,000	$ -	$ -
WWGM-3	Gravity Main along Main Street	12	18	1,600	$ 544,000	$ 544,000	$ -	$ -	$ -	$ -	$ 544,000	$ -
WWGM-4A	Gravity Main along San Joaquin Street	6	12	150	$ 43,000	$ 41,000	$ 2,000	$ -	$ -	$ -	$ 43,000	$ -
Rehabilitation and Replacement Projects					$ 2,398,000	$ 2,398,000	$ -	$ 249,000	$ 945,000	$ -	$ 602,000	$ 602,000
Gravity Mains		Diameter (in)	Diameter (in)	Length (ft)	$ 1,884,000	$ 1,884,000	$ -	$ -	$ 680,000	$ -	$ 602,000	$ 602,000
WWRR-1A	Gravity Main Beachcomber Drive	12	12	570	$ 166,000	$ 166,000	$ -	$ -	$ 166,000	$ -	$ -	$ -
WWRR-1B	Gravity Main Beachcomber Drive	10	10	960	$ 262,000	$ 262,000	$ -	$ -	$ 262,000	$ -	$ -	$ -
WWRR-1C	Gravity Main Beachcomber Drive	8	8	950	$ 252,000	$ 252,000	$ -	$ -	$ 252,000	$ -	$ -	$ -
WWRR-2	Pipe R&R Program Upstream of LS-1	6-12	8-12	27,000	$ 1,204,000	$ 1,204,000	$ -	$ -	$ -	$ -	$ 602,000	$ 602,000
Structures		Type	Type	Quantity	$ 514,000	$ 514,000	$ -	$ 249,000	$ 265,000	$ -	$ -	$ -
WWRR-4	Cap Replacement Upstream of LS-1	Cap	Manhole	31	$ 514,000	$ 514,000	$ -	$ 249,000	$ 265,000	$ -	$ -	$ -
Other Projects					$ 210,000	$ 210,000	$ -	$ -	$ -	$ -	$ -	$ 210,000
WWO-1	Sewer Master Plan Update	--	--	--	$ 150,000	$ 150,000	$ -	$ -	$ -	$ -	$ -	$ 150,000
WWO-2	Flow Monitoring Program	--	4 weeks	15 FM	$ 60,000	$ 60,000	$ -	$ -	$ -	$ -	$ -	$ 60,000
CIP Total					**$ 5,096,000**	**$ 4,850,000**	**$ 246,000**	**$ 779,000**	**$ 945,000**	**$ 1,371,000**	**$ 1,189,000**	**$ 812,000**
Annual Cost					**N/A**	**N/A**	**N/A**	**$ 779,000**	**$ 945,000**	**$ 1,371,000**	**$ 1,189,000**	**$ 812,000**

Notes:

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(4) Total Mark-Up is 65.8% of the baseline construction costs.

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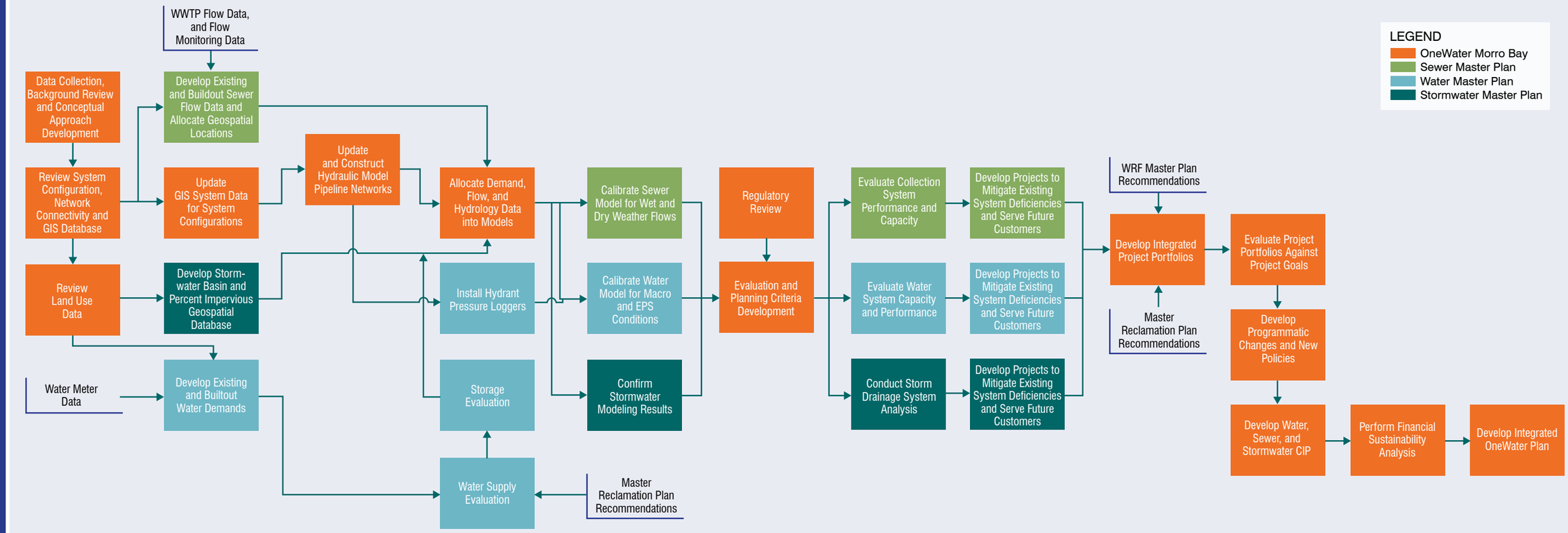
Appendix A

ONEWATER MORRO BAY WORK PLAN

Work Plan

Project Tasks



Project Meetings and Deliverables

Kick-off Meeting

- Project Management Plan
- Kick-off Meeting Agenda and Meeting Minutes
- Action Item List and Decision Log
- Data Collection Matrix

Meet with City GIS, and Planning Staff

- Summarize Existing Wastewater Flows
- Summarize Water Production, Consumption, and Demands
- Develop System Configuration Maps for City Review
- Develop Storm Drain Basin Map for Review

Model Calibration and Criteria Development Workshop

- Summary of Model Calibration Results
- Draft Criteria Summary Sheets
- Workshop Presentation Materials
- Meeting Agenda, Minutes, Action Item List, and Decision Log

Meet with City GIS, and Planning Staff

- Summary of System Deficiencies
- Proposed Mitigation Projects
- Draft System Integration Concepts
- Workshop Presentation Materials
- Meeting Agenda, Minutes, Action Item List, and Decision Log

Proposed Draft Project Portfolios Workshop

- Proposed Project Portfolios
- Workshop Presentation Materials
- Meeting Agenda, Minutes, Action Item List, and Decision Log

Public, PWAB, and City Council Presentations and Workshops

- Recommended Project Portfolios
- Results of Financial Sustainability Analysis
- Workshop Presentation Materials
- Meeting Agenda, Minutes, Action Item List, and Decision Log

Draft OneWater Workshop

- Draft OneWater Plan
- Comment Response Log
- Workshop Presentation Materials
- Meeting Agenda, Minutes, Action Item List, and Decision Log

Coordination with WRF Master Plan and Master Reclamation Plan Consultant Teams.

V:\Client86\MorroBay\Prop\OneWater\Indd\03-WorkProgram.indd

Appendix B

GENERAL PLAN LAND USE DESCRIPTIONS

Land Use Designations

Land use designations are applied to every parcel within the planning area; however, the City can only regulate land uses located within the city limits. *Plan Morro Bay* establishes 19 designations—17 primary land use designations and 2 overlay designations—that govern land uses within the planning area; see Table LU-1. These designations apply density and intensity requirements, use characteristics, development standards, and land use policies to individual parcels. As most of the planning area is already developed and maintained in good condition, the designations generally correspond to the pattern of existing uses.

Four land use designations accommodate solely residential development in Morro Bay. The designations encompass a wide variety of densities and housing types, ranging from lower-density, primarily detached single-family residences in neighborhoods, to some medium and higher-density, mostly attached housing in various places in the city.

Five land use designations accommodate commercial development in Morro Bay and one designation is a mixed commercial/industrial use designation. The businesses and other organizations located in these designations provide jobs, services, and goods, contributing to economic vitality and shaping the physical environment. These commercial-focused designations are distinguished by location and the customers the uses are intended to serve. Neighborhood-serving commercial uses are located in one- or two-story stand-alone buildings or small centers near residential neighborhoods. Community commercial uses occupy properties in and at the edge of downtown which are almost all developed as retail and office uses with few vacant parcels. District commercial uses include more auto-oriented and service-oriented commercial uses distributed through various areas of the city. Visitor-serving uses are located near primary tourist destinations, including the Embarcadero and Highway 1. Commercial/recreational fishing uses are located in the Measure D area.

The Waterfront Commercial/Industrial designation allows a mix of visitor-serving commercial uses and harbor-related industrial uses on the west side of the Embarcadero on state tidelands trust land.

Three land use designations accommodate industrial development in Morro Bay to ensure there are opportunities for both light industrial uses and uses that transition between light industrial and other uses; such uses might include live/work, harbor-related, or those supporting new employment opportunities that accompany emerging technologies and the redevelopment of transitioning industrial areas. In addition, a land use designation is provided for coastal-dependent uses, including energy, harbor-related uses, and other coastal priority uses.

One land use designation accommodates mixed-use development in Morro Bay. The designation is intended to provide for a wide variety of commercial uses that are developed along with higher-density residential uses. This use is focused at the edge of downtown and in locations where downtown transitions to residential areas. It is also located adjacent to the light industrial area behind Quintana Road near the southeastern edge of the city.

Three land use designations accommodate agriculture, resource conservation, parks, and recreation in Morro Bay. They include Agriculture, Harbor/Navigational Ways, and Open Space/Recreation. These designations make up the largest percentage of land in the planning area. They cover 7,500 acres (86 percent) of the planning area.

One Public/Institutional land use designation accommodates a wide variety of publicly owned facilities and community-serving uses.

Two overlay designations are identified on the Land Use Map: the Mixed Use Residential Overlay and the Environmentally Sensitive Habitat Areas (ESHA) Overlay. The Mixed Use Residential Overlay designation provides additional development criteria to supplement the underlying or base land use designation. The ESHA overlay provides information about where requirements related to ESHA are implemented in the city. The ESHA overlay is for information purposes, and ESHA may be located in areas not shown in the overlay once on-the-ground mapping is conducted for a specific site. In addition, because ESHA changes over time, the overlay shown in this plan will become less accurate over the life of the plan. To that end, the electronic version of the overlay will be updated throughout the life of the plan to reflect changes to ESHA. The overlay designations are identified as a cross-hatch on top of the base land use designation on the Land Use Map.

Table LU-1 identifies the land use designations, land use characteristics associated with each designation, and the land use density/development intensity allowed within each designation.

Table LU-1:
Land Use Designations

Designation	Description	Density/Intensity
Low Density	Detached single-family homes and some group housing uses.	0-4.0 du/ac
Moderate Density	Detached or attached single-family homes and some group housing uses.	4.1-7.0 du/ac
Medium Density	Detached or attached single-family homes, townhomes, duplexes, apartments, condominiums, and some group housing uses.	7.1-15.0 du/ac
High Density	Multifamily housing, including apartments, townhomes, condominiums, and some group housing uses. Single-family homes are allowed where the sites' characteristics, such as size or topography, would preclude multi-family development.	15.1-27.0 du/ac
Community Commercial	Community-oriented uses including retail stores, restaurants, professional and medical offices, and personal services. Residential uses are allowed both above and behind commercial uses with discretionary approval.	1.25 FAR for nonresidential component 15.1-27.0 du/ac for residential component
District Commercial	Retail, commercial, and service uses that meet local and regional demand. This designation is intended for larger-scale development that is appropriate in an auto-oriented environment.	0.5 FAR
Neighborhood Commercial	Smaller-scale commercial uses that provide for the daily needs and services of nearby residents. Residential uses are allowed both above and behind commercial uses with discretionary approval.	1.0 FAR for nonresidential component 4.1-15.0 du/ac for residential component
Visitor-Serving Commercial	Visitor-oriented services and uses located at easily accessible locations and tourist destinations within the coastal zone. In general, ground-floor development should be reserved for retail shops, restaurants and bars, and visitor accommodations, with the upper floors reserved for additional visitor accommodations and offices.	1.25 FAR

Designation	Description	Density/Intensity
Commercial/ Recreational Fishing	Implements Measure D, which protects the tidelands area between Beach Street and Target Rock. Development and use permits are limited to fishing activities only.	0.5 FAR
Waterfront Commercial/ Industrial	A mixture of visitor-serving commercial uses and harbor-dependent land uses located in the coastal zone.	1.25 FAR
General (Light) Industrial	Light industry uses which are generally not compatible with residential or most commercial uses. Existing residential buildings are permitted and are considered conforming.	0.5 FAR
Coastal-Dependent Industrial	Uses within the coastal zone which must be located near the coast to function, and are thereby given priority pursuant to the California Coastal Act.	0.65 FAR
Mixed Use	Implemented by the zoning code, any combination of commercial uses; offices; attached single-family housing, multiple-family housing, and live-work units; institutional uses; cultural facilities; developments including an open space component; visitor-serving uses; and/or civic facilities. Mixing of these uses may occur in a vertical and/or horizontal orientation. Mixed-use development is required within the constraints of parcel size, context/adjacent uses, and access to transportation.	1.0 FAR for nonresidential component 15.1-27.0 du/ac for residential component
Public/ Institutional	Facilities which serve the public, including government buildings and service facilities; or quasi-public facilities such as hospitals and cultural or civic resources.	0.5
Harbor/Navigational Ways	Areas of the city covered by seawater and used for boating, fishing, and visitor-serving uses.	N/A
Open Space/Recreation	Areas of improved and unimproved park facilities, open space areas, natural resource areas, and outdoor recreation.	N/A
Agriculture	Land for cultivating crops and raising animals.	N/A

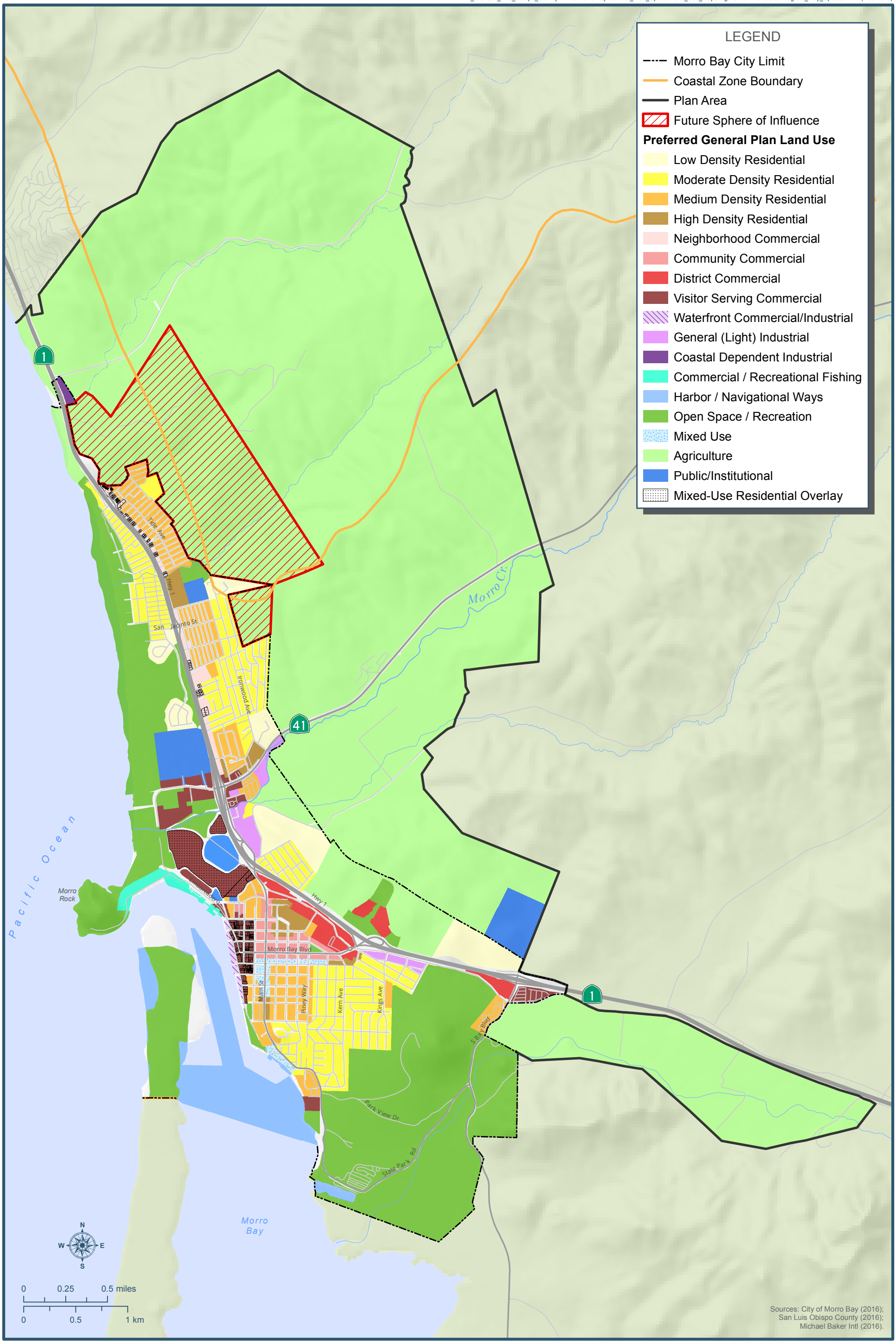
Designation	Description	Density/Intensity
Overlays		
Mixed-Use Residential	For Visitor-Serving Commercial, the overlay allows residential uses in addition to the base allowed uses. For Neighborhood Commercial, the overlay allows residential on the whole lot.	Same as underlying base designation for nonresidential component When paired with Neighborhood Commercial, residential density is the same as for the base designation When paired with Visitor-serving Commercial, 15.1-27.0 du/ac for residential component
Environmentally Sensitive Habitat Areas	Protected areas within the coastal zone which serve as habitat for rare or especially valuable plant or animal life that could be easily disturbed or degraded by human activity.	N/A

Land Use Diagram

LCP

The Land Use Diagram (Figure LU-3) illustrates how *Plan Morro Bay* goals and policies translate into on-the-ground land uses by showing the distribution of the land use designations described above in correlation to the street network and natural landscapes.

T:_GIS\San_Luis_Obispo_County\MXDs\Morro Bay\General_Plan_Update\Land_Use_Maps\Figure LU-4 Land Use Diagram_May_Updates.mxd (4/30/2018)



Plan MORRO BAY

FIGURE LU-3

Land Use Diagram

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DEVELOPMENT CAPACITY

LCP

Table LU-2 identifies the development capacity associated with the planned distribution of land uses described in this element and summarizes the land use distribution and the resulting residential and nonresidential levels of development that can be expected from implementation of land use policies established by *Plan Morro Bay*. As the density and intensity standards for each land use designation are applied to future development projects and land use decisions, properties will gradually transition from one use to another, and land uses and intensities will gradually shift to align with the intent of this Land Use Element.

Table LU-2:
Plan Morro Bay Development Capacity

Land Use Designation	Acres (approximate)	Total Estimated Dwelling Units (2040)	Total Estimated Households	Population (2040)	Non residential Square Feet (2040)
Residential	753.4	6,573		10,870	—
Commercial	307.6	565		934	8,819,081
Industrial	41.5	—		—	893,006
Waterfront Commercial/ Industrial	6.3	—		—	220,869
Mixed Use	17.6	141		233	607,984
Open Space & Agriculture	1,547.4				—
Public/Institutional	336.9	—		—	371,651
Outside City in Future SOI and Plan Area					
Open Space & Agriculture	6,079.9	15		25	—
Public/Institutional	56.6				
Total (2040)	9,147.2*	7,295	5,792	12,062	10,912,591
Existing (2016) Totals	9,147.2*	6,414	5,063	10,714	2,613,654
Change, 2017-2040	—	881	729	1,348	8,298,937

Notes:

**Totals may not add up due to rounding*

Plan Morro Bay does not directly specify a maximum population for Morro Bay. Any growth (including any potential expansion of the SOI) in Morro Bay also must be consistent with Measure F, a voter-approved growth management ordinance that limits the city to 12,200 residents. To exceed this number, Morro Bay must secure additional water resources and a majority of voters must elect to remove the limit. *Plan Morro Bay* policy proposes to undertake a process to either affirm, amend, or repeal Measure F at the point where the City's population reaches 11,700.

The maximum possible number of residential units is determined by the different maximum densities allowed for each land use designation and the amount of land area within that designation. However, this maximum number of units is unlikely to be reached because every residential parcel in Morro Bay would need to be developed to its maximum potential. Because most of the planning area is built out and existing buildings are generally in good condition, these changes will primarily occur in the Dynegy and Wastewater Treatment Plant Areas, as well as parts of downtown and the adjacent Highway 1 corridor, North Main, and Highway 41. Forecasting assumptions are used to determine the realistic expected number of residential units that Morro Bay will have when all of the parcels that are reasonably expected to redevelop have done so, subject to voter-approved growth management initiatives.

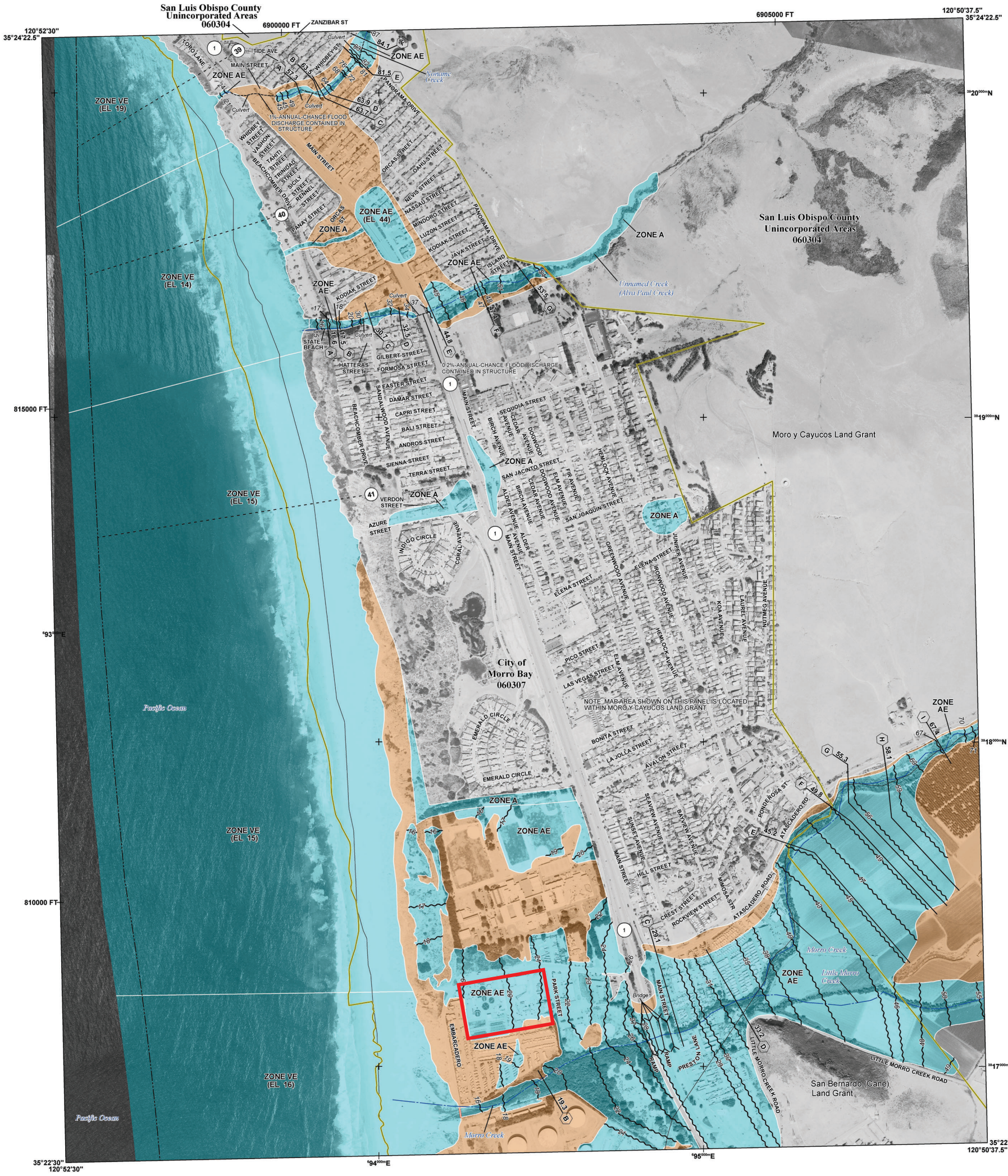
KEY ISSUES

Diverse Mix of Land Uses

Morro Bay is physically defined as a community by a land use pattern, design elements, and overall community form. The balance of uses and ways in which they are arranged dramatically affect the character, sustainability, health, and economy of the city. The uses present in the city and their locations can ensure equitable access to the coast, visitor amenities and services, and housing, employment, retail and services, education, and recreation for residents and employees. Having a diverse mix of land uses also allows for a varied economy that promotes local businesses and a strong jobs/housing balance, and is able to adapt to changing markets and economic disruptions. In these ways, the form of the community directly contributes to Morro Bay's sustainability, resiliency, success, and the well-being of residents and visitors.

Appendix C

FEMA FLOOD HAZARD, NOAA SEA-LEVEL RISE, AND TSUNAMI INUNDATION LIMITS



FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT

HTTP://MSC.FEMA.GOV

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A,V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee See Notes *Zone X*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Accredited or Provisionally Accredited Levee, Dike, or Floodwall
- Non-accredited Levee, Dike, or Floodwall

OTHER FEATURES
- E 18.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation (BFE)
- 8 Coastal Transect
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary

NOTES TO USERS

For information and questions about this Flood Insurance Rate Map (FIRM), available products associated with this FIRM, including historic versions, the current map date for each FIRM panel, how to order products, or the National Flood Insurance Program (NFIP) in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Flood Map Service Center website at http://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Flood Map Service Center at the number listed above.

For community and countywide map dates refer to the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Base map information shown on this FIRM was derived from Coastal California LiDAR and Digital Imagery dated 2011. USDA NAIP 2010 imagery is used in areas not covered by the Coastal California digital imagery.

SCALE

N

Map Projection:

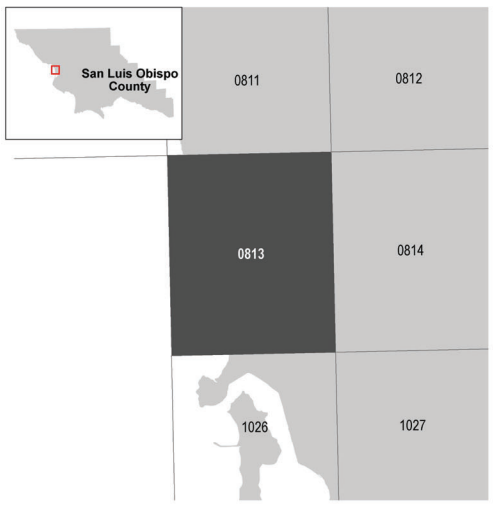
Universal Transverse Mercator Zone 10N; North American Datum 1983; Western Hemisphere; Vertical Datum: NAVD 88

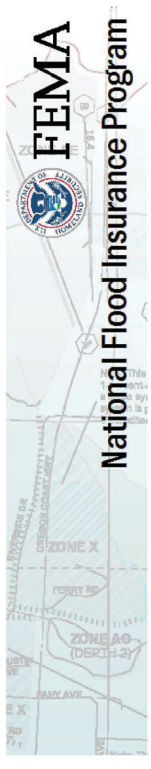
1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 125 250 500 Meters

PANEL LOCATOR





NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

SAN LUIS OBISPO COUNTY, CALIFORNIA

and Incorporated Areas

PANEL 813 OF 2050

FEMA

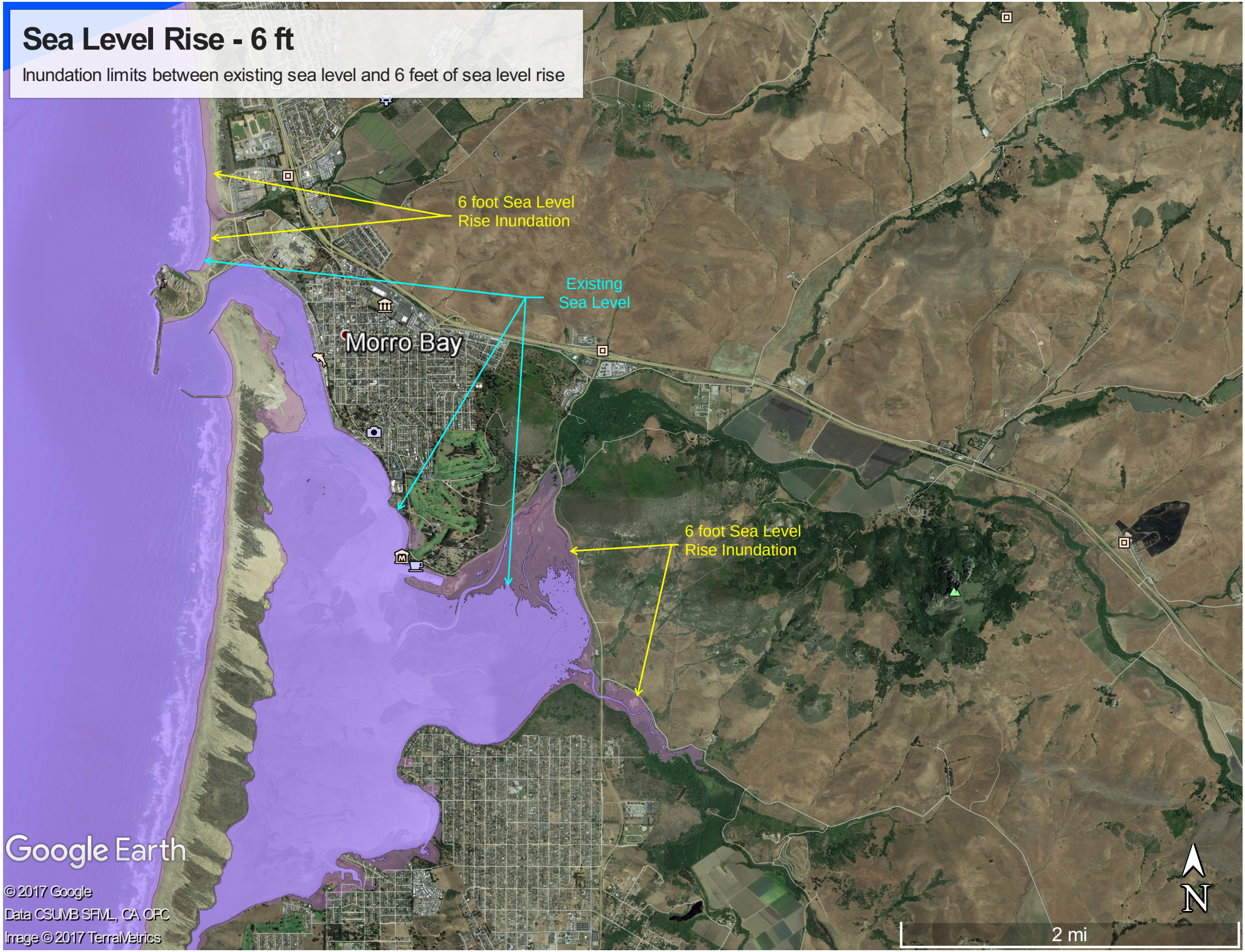
Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
MORRO BAY, CITY OF	060307	0813	H
SAN LUIS OBISPO COUNTY	060304	0813	H

VERSION NUMBER 2.3.2.2

MAP NUMBER 06079C0813H

MAP REVISED May 16, 2017


Sea Level Rise - 6 ft
Inundation limits between existing sea level and 6 feet of sea level rise
6 foot Sea Level Rise Inundation
Existing Sea Level
Morro Bay
6 foot Sea Level Rise Inundation
Google Earth
© 2017 Google
Data CSUMB SFML, CA OPC
Image © 2017 TerraMetrics
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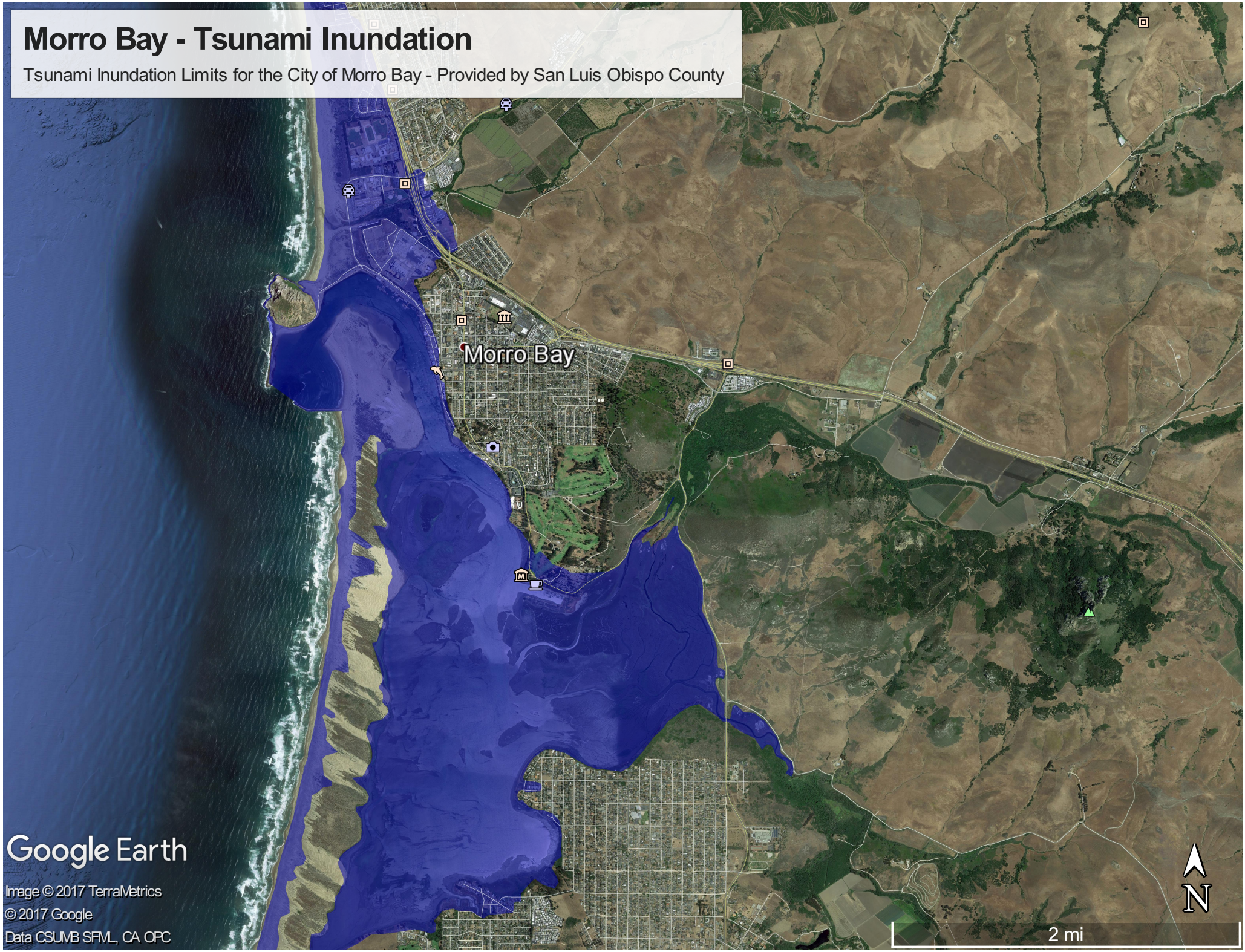
2 mi

Morro Bay - Tsunami Inundation

Tsunami Inundation Limits for the City of Morro Bay - Provided by San Luis Obispo County



Appendix D

WATER SUPPLY EVALUATION SURVEY

February 13, 2018

Subject: OneWater Plan Water Supply Evaluation Criteria Ranking Input

Mayor, Members of the City Council, Members of the Public Works Advisory Board, and City Staff:

With the overarching goal of ensuring the adequacy of its underground utilities and long-term water security, the City of Morro Bay is currently undertaking the OneWater Plan. "One water" is an integrated planning and implementation approach to managing finite water resources for long-term resilience and reliability. This new philosophy of master planning is being utilized by other municipalities and agencies in California and was endorsed by the City Council in 2016.

The City's OneWater Plan will consist of four main components including:

- Water Master Planning
- Collection System Master Planning
- Storm Water Master Planning
- Water Supply Evaluation

The City began work on the OneWater Plan in June 2017 and presented an overview and status update to the Public Works Advisory Board and City Council in November 2017 and January 2018, respectively. A major focus of these presentations was the water supply evaluation component of the OneWater Plan. In response to the City Council's 2016 goal of diversifying the City's water supply portfolio and decreasing its reliance on State Water, the water supply evaluation will identify viable water supply options for the City and evaluate those criteria using a host of quantitative (total life-cycle cost) and qualitative criteria. The water supply evaluation component of the OneWater Plan will ultimately define the City's long-term water future.

At this time, the City has identified nine water supply alternatives, a total of nine qualitative evaluation criteria, and performed an initial ranking of those criteria. In response to feed back received at the January 23, 2018 City Council Special Meeting, City staff and the consultant team are reevaluating the original approach used to rank the criteria. To promote collaboration and ensure the opinions of key stakeholders are represented in the evaluation, we are soliciting input from the public, in the form of the Public Works Advisory Committee, and City Council that will help establish the ranking and weighting of the nine qualitative criteria.

In order to facilitate receipt of this input, we are asking members of the Public Works Advisory Board and City Council to take part in this short survey. The following pages include a description of the nine qualitative criteria selected by the consultant team and City staff. Following each description, we ask that individuals give each criteria a score of 1 to 10 (1 being low importance and 10 being high importance) by circling the appropriate number. After all the scores have been received, they will be combined with the scores from City staff and tabulated by the consultant team. The scores will then be used to establish a weighting for each criteria. The details and results of this weighting exercise and the results of the water supply evaluation will be presented to the Public Works Advisory Board and City Council in March 2018 and April 2018, respectively.

The OneWater Plan represents a significant shift in the way the City plans and manages its infrastructure needs and will ensure the community of Morro Bay has a resilient water supply to promote the long-term prosperity of its residents. If you have any questions about the specifics of the survey or other components of the OneWater Plan, please do not hesitate to contact either Rob Livick or Eric Casares.

Sincerely,

Rob Livick, P.E./P.L.S.
Public Works Director
City of Morro Bay
rlivick@morrobayca.gov
Phone: (805) 772-6569

Eric Casares, P.E.
OneWater Plan Project Manager
Carollo Engineers
ecasares@carollo.com
Phone: (559) 436-6616

RL:EC:asw

Enclosures: Evaluation Criteria Description and Scoring Sheets

Table 1

Criteria	**Unit Cost**
Description	Unit cost of water supply for a project is calculated as: $$Unit\ Cost = \frac{Ammortized\ Capital\ Cost + Annual\ O\&M\ Cost}{Total\ Annual\ Yield}$$ Unit cost includes both amortized capital cost and operation and maintenance (O&M) costs utilizing typical inflation rates, interest rates, and life expectancies to the calculation. As a ratio to the project yield, the unit costs is a representation of the cost of water production per each project alternative.

Unit Cost Criteria Range

Definition	From	To
Very Low	$ -	$ 500.00
Low	$ 500.00	$ 1,000.00
Average	$ 1,000.00	$ 2,000.00
High	$ 2,000.00	$ 3,000.00
Very High	$ 3,000.00	$ 4,000.00

From a scale of 1 to 10, how important is Unit Cost as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Table 2

Criteria	**Drought Resiliency**
Description	Drought resiliency is defined as the project's ability to provide water for consumption during drought years. This value is calculated as the ratio between project yields from a normal and drought year. $$Drought\ Resiliency\ Ratio = \frac{Project\ Yield\ During\ Drought\ Year}{Project\ Yield\ During\ Normal\ Year}$$

Drought Resiliency Criteria Range

Drought Resiliency	Range
Very High Resilience	Greater than 75% Resilience
High Resilience	> 50 to 75% Resilience
Moderate Resilience	> 25 to ≤ 50% Resilience
Low Resilience	> 10 to ≤ 25% Resilience
Minimal/No Resilience	Less than 10% Resilience

From a scale of 1 to 10, how important is Drought Resiliency as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Table 3

Criteria	**Earthquake Resiliency**
Description	Earthquake resiliency is defined as the ability of the project to withstand earthquakes while maintaining the ability to deliver water after a major earthquake. The earthquake resilience evaluation criteria takes into consideration the immediate response to an earthquake, duration the project would be offline if succumed to damage, project redundancy, construction methods, and proximity to major active geological features.

Earthquake Resiliency Criteria Range

Earthquake Resiliency	Resilience Criteria
High Resiliency	Project is highly unlikely to be damaged during an earthquake Water is likely to be delivered within 4 hours after an earthquake Project is local and does not stretch across multiple geologic features Project is new construction up to modern standards Project has significant distributed supply facilities and redundancy
Moderate Resiliency	Project is unlikely to be damaged during an earthquake Water is likely to be delivered within 3 to 4 weeks after an earthquake Project is local and does not stretch across multiple geologic features Project is newer construction up to modern standards Concept has distributed water supply facilities and some redundancy
Very Fragile	Project is highly likely to be damaged during an earthquake Project is likely to be out of service ≥ 3 months after earthquake Project spans across multiple geologic features Project was constructed over 20 years ago Concept includes long stretches of pipelines without redundancy

From a scale of 1 to 10, how important is Earthquake Resiliency as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Table 4

Criteria	**Flood Vulnerability**
Description	Flood vulnerability is defined as the ability of the project to withstand the affects of multiple flooding events, sea level rise, and tsunami run up. Vulnerability from annual flooding events is categorized as potential damage to the project resulting from a significant event. The risk of potential damage outlined by the limits of flood inundation provided by FEMA display various storm event inundantion boudaries which differentiate levels of vulnerability ranging from minimal to extreme. Projects suseptible to tsunami run up fall within the California Department of Conservation tsunami limits. Sea level rise limits were defined by a 6-foot sea level rise model provided by NOAA Office for Coastal Management.

Flood Vulnerability Criteria Range

Flood Vulnerability	Vulnerability Criteria
Minimal	Suseptible to local pooling from 2% event Contains on site run-off storage w/ discharge, no pooling Contains private on site storm drainage, no hydraulic impacts Not suseptible to effects of sea level rise Not suseptible to tsunami threats
Low	Suseptible to moderate local pooling from 2% event for < 24 hours Decreases on-site flood pooling with storm water BMPs Surface conveyance system with minimal hydraulic impacts
Average	Suseptible to moderate flooding from 2% event, outside of 1% plain Suseptible to major pooling on site for > 24 hours No damage from 1% event Flood protected from sea level rise inundation Flood protected from tsunami wave threats
High	Within 0.2% annual event flood plain Does not affect FEMA base flood elevation Suseptible to damage from 0.2% event
Extreme	Within 1% annual event flood plain Affects FEMA BFE, increase flood risk on local level Suseptible to damage from 1.0% annual event Suseptible to damange from sea level rise inundation Suseptible to damange from tsunami waves

From a scale of 1 to 10, how important is Flood Vulnerability as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Table 5

Criteria	**Local Supply Benefit**
Description	Local supply benefit is defined as the ability of a project to provide the City of Morro Bay's future water demand with only local supply water, not including any imported water. This criteria is calculated as follows: $$Local\ Supply\ Benefit = \frac{Annual\ Project\ Yield\ (normal\ year)}{City\ Future\ Demand}$$

Local Supply Benefit Criteria Range

Local Supply Benefit	Range
Very High Benefit	Greater than 75% offset
High Benefit	50 to 75% offset
Average Benefit	25 to 50% offset
Low Benefit	10 to 25% offset
Minimal/No Benefit	Less than 10% offset

From a scale of 1 to 10, how important is Local Supply Benefit as an project evaluation criteria?

Please Circle One:

Less					**Moderate**				**More**
1	2	3	4	5	6	7	8	9	10

Table 6

Criteria	**Energy Requirements**
Description	Energy requirements are defined as the amount of power used per unit of water processed. Measured in kilowatt-hour per acre-foot. The total annual energy consumption per unit of supply is the metric for greenhouse gas emissions and associated climate change impacts.

Energy Requirements Criteria Range

Energy Impact	From (kWh/ac-ft)	To (kWh/ac-foot)
Very Low Impact	-	1,000
Low Impact	1,000	2,000
Moderate Impact	2,000	3,000
High Impact	3,000	4,000
Very High Impact	4,000	5,000

From a scale of 1 to 10, how important is Energy Requirements as an project evaluation criteria?

Please Circle One:

Less					**Moderate**				**More**
1	2	3	4	5	6	7	8	9	10

Table 7

Criteria	**Constructability**
Description	Constructability is evaluated based on the the complexity in constructing the project. Projects with multiple locations (outside the fence), dangerous or limited access or special construciton requirements are termed as difficult. Projects with local locations (inside the fence), no access limitations, and standard construction are defined as straight forward.

Constructability Criteria Range

Constructability	Constructability Criteria
Very Straight-Forward	Nearly all project components are located inside the fence No underground utilities Expected to be significantly less complicated to construct
Straight-Forward	Project omponents are located both inside and outside the fence Less than 1 miles of underground utilities Expected to be less complicated to construct
Typical	Project components are located both inside and outside the fence 1 to 3 miles of underground utilities Project is expected to have typical construction concerns
Difficult	Most project components are located outside the fence 3 to 5 miles of underground utilities Project will be difficult to construct
Very Difficult	Nearly all project components are located outside the fence Greater than 5 miles of underground utilities Project will be extremely difficult to construct

From a scale of 1 to 10, how important is Constructability as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Table 8

Criteria	**Regulatory and Institutional Complexity**
Description	Regulatory and institutional complexity is defined as how difficult a project would be to permit or meet an agency requirements. Projects range from low to high complexity categorizing their ability to be permittable based on environmental, drinking water, construction, and a wide range of various regulatory standards.

Regulatory and Institutional Complexity Criteria Range

Definition
Low Complexity
Averge Complexity
High Complexity

From a scale of 1 to 10, how important is Regulatory and Institutional Complexity as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Table 9

Criteria	**Quality Consistency**
Description	Quality consistency represents the water quality of the produced water from each project. Quality is the standard set by drinking water regulations to be categorized safe for consumption. The levels of meeting water quality criteria are classified as the project yield always meeting regulations to infrequently meeting regulations on the high and low side respectively. What this criteria translates to for alternatives is that those alternatives that have final treatment processes (which comes with additional costs) after contact with groundwater score highly, while those alternatives that rely on blending highly treated water with groundwater, which may be variable in quality, score lower due to the risk of lower quality.

Quality Consistency Criteria Range

Risk Level	Water Quality Criteria
Low Risk	Low Risk of Exceeding Water Quality Regulations
Moderate Risk	Some risk of occasional water quality variability that will potentially require monitoring or blending with other sources to comply with drinking water regulations
More Risk	More Risk of water quality variability that will require treatment or significant blending with other sources to comply with drinking water regulations.

From a scale of 1 to 10, how important is Quality Consistency as an project evaluation criteria?

Please Circle One:

Less				**Moderate**					**More**
1	2	3	4	5	6	7	8	9	10

Appendix E

STORMWATER MANAGEMENT GUIDANCE MANUAL FOR LOW IMPACT DEVELOPMENT AND POST-CONSTRUCTION REQUIRMENTS

STORMWATER MANAGEMENT GUIDANCE MANUAL FOR LOW IMPACT DEVELOPMENT & POST-CONSTRUCTION REQUIREMENTS

MAIN MANUAL

March 6, 2014
(Updated: July 1, 2017)

This Guidance Manual is a supplement to the City of Morro Bay Department of Public Works Standard Drawings and Specifications

City of Morro Bay, California

APPROVED: _____ 23 August 2017

Rob Livick, PE/PLS, Director of Public Works/City Engineer — Date

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Introduction

Post-Construction Stormwater Management Performance Requirements

The primary objective of these Post-Construction Stormwater Management Performance Requirements is to minimize the downstream impact of increased stormwater runoff that often occurs as the result of development or redevelopment projects. The Post-Construction Requirements emphasize protecting and, where degraded, restoring key watershed processes to create and sustain healthy watersheds. Maintenance and restoration of watershed processes is necessary to protect water quality and beneficial uses.

The intention of this Guidance Manual is to provide developers a tool to both determine the specific requirements for a given project and to plan and design the project so that those requirements are met as efficiently as possible. The requirements in this Manual are based on Resolution R3-2013-0032 and Attachments adopted July 12, 2013 by the Central Coast Regional Water Quality Control Board. In the event of conflict, the requirements of Resolution R3-2013-0032 shall take precedence over those contained in this Guidance Manual. Attachment 1 of the Resolution, which contains the referenced requirements, can be viewed or downloaded at: http://www.waterboards.ca.gov/centralcoast/water_issues/programs/stormwater/docs/lid/hydromod_lid_docs/2013_0032_attach1_post_construction_requirements.pdf

Definitions Related to Post-Construction Requirements

Bioretention – A Stormwater Control Measure designed to retain stormwater runoff using vegetated depressions and soils engineered to collect, store, treat, and infiltrate runoff. Bioretention designs do not include underdrains.

Biotreatment or Biofiltration Treatment –A Stormwater Control Measure designed to detain stormwater runoff, filter stormwater through soil media and plant roots, and release the treated stormwater runoff to the storm drain system. Biotreatment systems include an underdrain.

Discretionary Approval – A project approval which requires the exercise of judgment or deliberation when the MS4 decides to approve or disapprove a particular activity, as distinguished from situations where the MS4 merely has to determine whether there has been conformity with applicable statutes, ordinances, or regulations.

Dispersion – The practice of routing stormwater runoff from impervious areas, such as rooftops, walkways, and patios, onto the surface of adjacent pervious areas. Stormwater runoff is dispersed via splash block, dispersion trench, or sheet flow and soaks into the ground as it moves slowly across the surface of the pervious area.

Drainage Management Area (DMAs) – Following the low impact development principle of managing stormwater through small-scale, decentralized measures, DMAs are designated individual drainage areas within a Regulated Project that typically follow grade breaks and roof ridge lines and account for each surface type (e.g., landscaping, pervious paving, or roofs). Stormwater Control Measures for runoff reduction and structural facilities are designed for each DMA.

Equivalent Impervious Surface Area – is equal to *Impervious Tributary Surface Area* (ft^2) + *Pervious Tributary Surface Area* (ft^2), where *Impervious Tributary Surface Area* is defined as the sum of all of the site's conventional impervious surfaces, and *Pervious Tributary Surface Area* is defined as the sum of all of the site's pervious surfaces, corrected by a factor equal to the surface's runoff coefficient.

Evapotranspiration (ET) – The loss of water to the atmosphere by the combined processes of evaporation (from soil and plant surfaces) and transpiration (from plant tissues).

Flow-Through Water Quality Treatment Systems – Stormwater Control Measures that are designed to treat stormwater through filtration and/or settling. Flow-through systems do not provide significant retention or detention benefits for stormwater volume control.

Gross Impervious Area – Impervious surfaces that are created or replaced by the project. Manufactured permeable surfaces (pervious paving, gapped paving stones, etc.) may be considered as a pervious surface and are considered on a case by case basis. Do not include the surface area of decks with gaps that allow runoff to drain to permeable surfaces below. Gross Impervious Area is used in the initial determination of performance requirements.

Groundwater Basins – Groundwater basin areas defined by the California Department of Water Resources (DWR) and used in the Central Coast Water Board Joint Effort for Hydromodification Control to identify groundwater receiving-water issues and areas where recharge is a key watershed process. DWR based identification of the groundwater basins on the presence and areal extent of unconsolidated alluvial soils identified on a 1:250,000 scale from geologic maps provided by the California Department of Conservation, Division of Mines and Geology. DWR then further evaluated identified groundwater basin areas through review of relevant geologic and hydrogeologic reports, well completion reports, court-determined adjudicated basin boundaries, and contact with local agencies to refine the basin boundaries.

Impervious Surface – A hard, non-vegetated surface area that prevents or significantly limits the entry of water into the soil mantle, as would occur under natural conditions prior to development. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or storage areas, concrete or asphalt paving, oiled, macadam or other surfaces which similarly impede the natural infiltration of stormwater. Open, uncovered retention/detention facilities shall not be considered as impervious surfaces for purposes of determining whether the thresholds for application of Performance Requirements are exceeded. However, for modeling purposes, open, uncovered facilities that retain/detain water (e.g., retention ponds, pools) shall be considered impervious surfaces. There are three methods of calculating impervious surface area, depending on the context of the calculation. For more details, see ***Net Impervious Area, Gross Impervious Area, and Equivalent Impervious Area*** definitions.

Land recycling – The reuse of abandoned, vacant, or underused properties for redevelopment or repurposing

Landscaped Areas – Areas of soil and vegetation not including any impervious surfaces of ancillary features such as impervious patios, BBQ areas, and pools.

Large River – A river draining 200 square miles or more.

Low Impact Development (LID) – A stormwater and land use management strategy that strives to mimic pre-disturbance hydrologic processes of infiltration, filtration, storage, evaporation, and transpiration by emphasizing conservation, use of on-site natural features, site planning, and distributed stormwater management practices that are integrated into a project design.

Ministerial Approval – A project approval which involves little or no personal judgment by the MS4 as to the wisdom or manner of carrying out the project and only involves the use of fixed standards or objective measurements.

Native Vegetation – Vegetation comprised of plant species indigenous to the Central Coast Region and which reasonably could have been expected to naturally occur on the site.

Net Impervious Area – The sum of new and replaced post-project impervious areas, minus any reduction in total imperviousness from the pre-project to post-project condition: *Net Impervious Area =*

(New and Replaced Impervious Area) – (Reduced Impervious Area Credit), where *Reduced Impervious Area Credit* is the total pre-project to post-project reduction in impervious area, if any.

New Development – Land disturbing activities that include the construction or installation of buildings, roads, driveways and other impervious surfaces. Development projects with pre-existing impervious surfaces are not considered New Development.

Percentile Rainfall Event (e.g., 85th and 95th) – A percentile rainfall event represents a rainfall amount which a certain percent of all rainfall events for the period of record do not exceed. For example, the 95th percentile rainfall event is defined as the measured rainfall depth accumulated over a 24-hour period, for the period of record, which ranks as the 95th percentile rainfall depth based on the range of all daily event occurrences during this period.

Permeable or Pervious Surface – A surface that allows varying amounts of stormwater to infiltrate into the ground. Examples include pasture, native vegetation areas, landscape areas, and permeable pavements designed to infiltrate.

Pre-Project – Stormwater runoff conditions that exist onsite immediately before development activities occur. This definition is not intended to be interpreted as that period before any human-induced land activities occurred. This definition pertains to redevelopment as well as initial development.

Project Site or Project Area – The area defined by the legal boundaries of a parcel or parcels of land within which the new development or redevelopment takes place and is subject to these Post-Construction Stormwater Management Requirements.

Rainwater Harvest – Capture and storage of rainwater or stormwater runoff for later use, such as irrigation (without runoff), domestic use (e.g. toilets), or storage for fire suppression.

Receiving Waters – Bodies of water, surface water systems or groundwater that receive surface water runoff through a point source, sheet flow or infiltration.

Redevelopment – On a site that has already been developed, construction or installation of a building or other structure subject to the Permittee's planning and building authority including: 1) the creation or addition of impervious surfaces; 2) the expansion of a building footprint or addition or replacement of a structure; or 3) structural development including construction, installation or expansion of a building or other structure. It does not include routine road maintenance, nor does it include emergency construction activities required to immediately protect public health and safety.

Replaced Impervious Surface – The removal of existing impervious surfaces down to bare soil, base course or foundation slab and replacement with new impervious surface. Replacement of impervious surfaces that are part of routine maintenance activities are not considered replaced impervious surfaces.

Self-Retaining Areas – (also called "zero discharge" areas), are designed to retain some amount of rainfall (by ponding and infiltration and/or evapotranspiration) without producing stormwater runoff. Self-Retaining Areas may include graded depressions with landscaping or pervious pavement.

Self-Treating Areas – are a portion of a Regulated Project in which infiltration, evapotranspiration and other natural processes remove pollutants from stormwater. The self-treating areas may include conserved natural open areas and areas of native landscaping. The self-treating area only treats the rain falling on itself and does not receive stormwater runoff from other areas.

Routine Road Maintenance – includes pothole and square cut patching; overlaying existing asphalt or concrete pavement with asphalt or concrete without expanding the area of coverage; shoulder grading; reshaping/regrading drainage systems; crack sealing; resurfacing with in-kind material without expanding the road prism or altering the original line and grade and/or hydraulic capacity of the road.

Single-Family Residence – The building of one single new house or the addition and/or replacement of impervious surface associated with one single existing house, which is not part of a larger plan of development.

Stormwater Control Measures (SCM) – Stormwater management measures integrated into project designs that emphasize protection of watershed processes through replication of pre-development runoff patterns (rate, volume, duration). Physical control measures include, but are not limited to, bioretention/rain gardens, permeable pavements, roof downspout controls, dispersion, soil quality and depth, minimal excavation foundations, vegetated roofs, and water use. Design control measures include but are not limited to conserving and protecting the function of existing natural areas, maintaining or creating riparian buffers, using onsite natural drainage features, directing runoff from impervious surfaces toward pervious areas, and distributing physical control measures to maximize infiltration, filtration, storage, evaporation, and transpiration of stormwater before it becomes runoff.

Stormwater Control Plan – A plan, developed by the Regulated Project applicant, detailing how the project will achieve the applicable Post-Construction Stormwater Management Requirements (for both onsite and offsite systems). A preliminary Stormwater Control Plan is required for Planning Permits and the final version is required prior to issuance of a Building Permit. See Appendix K for required contents.

Tributary Area – The entire project area except for undisturbed areas of planted areas with native vegetation that do not receive runoff from other areas and impervious surface areas that discharge to infiltration areas that will not produce runoff or create nuisance ponding. The Drainage Management Areas are smaller Tributary Areas that cumulatively make up the Tributary Area of the entire site.

Does My Project Need to Meet Post-Construction Performance Requirements?

Projects subject to these Post-Construction Performance Requirements include all New Development or Redevelopment projects that create and/or replace ≥15,000 square feet of impervious surface (collectively over the entire project site). In general, the larger the impervious surface created or replaced, the more rigorous the requirements become. However, the requirements are also dependent on project type and location. Consequently, these three elements (gross impervious area, project type, and project location) need to be determined and quantified as a first step in the process.

1. **Gross Impervious Area**: Gross Impervious Area is the total of newly created and replaced impervious surfaces. Existing impervious surfaces that are within the project site but are not being replaced are not included in this total. Impervious surfaces are any hard, non-vegetated surface areas that prevent or significantly limit the entry of water into the soil. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or storage areas, concrete or asphalt paving, oiled, macadam or other surfaces which similarly impede the natural infiltration of stormwater. Manufactured permeable surfaces (pervious paving, gapped paving stones, etc.) may be considered as pervious surfaces and are considered on a case by case basis. Surface areas of decks with gaps that allow runoff to drain to permeable surfaces below are not considered imperious areas. For redevelopment projects, both new and replaced impervious surfaces are included. If the Gross Impervious Area is less than 15,000 square feet, the project is exempt from requirements.
2. **Type of Project**: A list of types of projects that are exempt for all stormwater requirements is detailed below, also see Appendix A. If not exempt:
    a. Is the project a new development or redevelopment project? Projects are classified as redevelopment if the project replaces or adds to existing impervious surfaces. Projects located on land with no existing impervious surfaces are considered new development.
    b. Does the project involve the construction or reconstruction of one or more detached single family residences (SFR)?
3. **Location**: The City is divided into several Watershed Management Zones. The map in Appendix B shows the boundaries of these zones. A Google Earth overlay is available on the City website for more precise project location.

The Performance Requirement Determination Form in Appendix C is provided to document the results of the above assessment. It shall be completed and filed with the Planning permit application. If the project is exempt, no further documentation is required. If not exempt, a calculation of the Net Impervious Area is required.

PROJECTS EXEMPT FROM STORMWATER REQUIREMENTS

Project that are exempt from the Post-Construction Performance are as follows (check any box on the list and no further action is required):

- ☐ Road and Parking Lot maintenance:
  - Road surface repair including slurry sealing, fog sealing, and pothole and square cut patching
  - Overlaying existing asphalt or concrete pavement with asphalt or concrete without expanding the area of coverage
  - Shoulder grading
  - Cleaning, repairing, maintaining, reshaping, or re-grading drainage systems
  - Crack sealing
  - Resurfacing with in-kind material without expanding the road or parking lot
  - Practices to maintain original line and grade, hydraulic capacity, and overall footprint of the road or parking lot
  - Repair or reconstruction of the road because of slope failures, natural disasters, acts of God or other man-made disaster
- ☐ Sidewalk and bicycle path or lane projects, where no other impervious surfaces are created or replaced, built to direct stormwater runoff to adjacent vegetated areas
- ☐ Trails and pathways, where no other impervious surfaces are replaced or created, and built to direct stormwater runoff to adjacent vegetated areas
- ☐ Underground utility projects that replace the ground surface with in-kind material or materials with similar runoff characteristics
- ☐ Curb and gutter improvement or replacement projects that are not part of any additional creation or replacement of impervious surface area (e.g., sidewalks, roadway)
- ☐ Second-story additions that do not increase the building footprint
- ☐ Roof repair or replacement
- ☐ Raised (not built directly on the ground) decks, stairs, or walkways designed with spaces to allow for water drainage
- ☐ Photovoltaic systems installed on/over existing roof or other impervious surfaces, and panels located over pervious surfaces with well-maintained grass or vegetated groundcover, or panel arrays with a buffer strip at the most down gradient row of panels
- ☐ Temporary structures (in place for less than six months)
- ☐ Electrical and utility vaults, sewer and water lift stations, backflows and other utility devices
- ☐ Above-ground fuel storage tanks and fuel farms with spill containment system

Net Impervious Area Calculation

Net Impervious Area is the Gross Impervious Area minus any reduction in total imperviousness from the pre-project to post-project condition: Net Impervious Area = (Gross Impervious Area) - (Reduced Impervious Area Credit), where Reduced Impervious Area Credit is the total pre-project to post-project reduction in impervious area, if any. The result of this calculation is used to determine if a project is subject to PR.2 or in the case of an SFR project, subject to PR.3 requirements.

Examples of Calculating **Net Impervious Area**

Example 1:

The project is a property that is an existing commercial shopping center with 100,000 sf of impervious surface, including buildings, parking lot, etc. The new project will redevelop the site and have a total impervious area of 85,000 sf.
The **Reduced Imperious Area Credit** is 100,000 – 85,000 = 15,000 sf.
The **Net Impervious Area** is 85,000 – 15, 000 = 70,000 sf.
The **Net Impervious Area** is 70,000 sf which is greater than 5,000 sf .
The project is subject to PR.2.
Design the Water Quality Treatment measures to treat the water runoff from the 85,000 sf of impervious area.

Example 2:

The project is an existing commercial shopping center with 100,000 sf of impervious surface. The new project will redevelop the entire site leaving a gross impervious area of 52,000 sf. (they've added a lot of landscaping and used green roofs to reduce the impervious area by 48,000 sf).
The **Reduced Imperious Area Credit** is 100,000 – 52,000 = 48,000 sf.
The **Net Impervious Area** is 52,000 – 48,000 = 4,000 sf.
The **Net Impervious Area** is 4,000 sf which is less than 5,000 sf.
The project is **NOT** subject to PR.2 (though it may be subject to other PRs).

Once the Net Impervious Area Calculation is determined, use the following flow charts to determine the Post-Construction Performance Requirements for the project. Complete the Performance Requirement Determination Form (Appendix C) once the Flow Charts have been used to determine Performance Requirements.

Flow Charts

Performance Requirements Flow Chart
for non-exempt projects

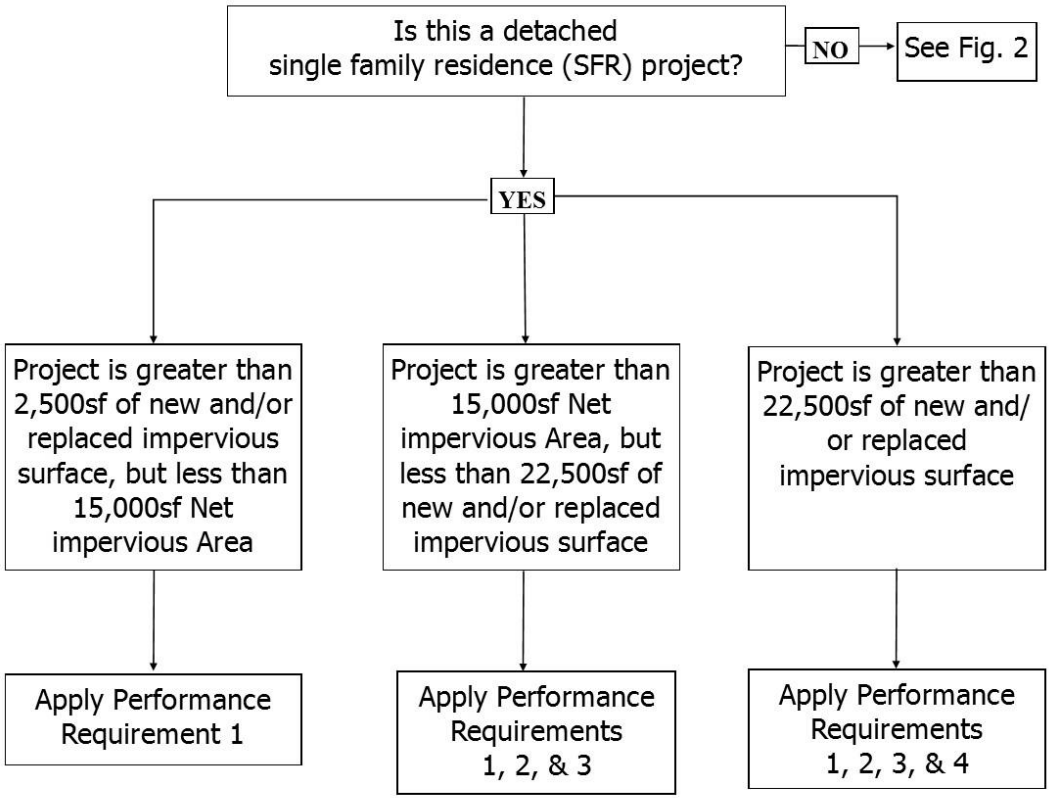


Figure 1

Performance Requirements Flow Chart
for non-exempt projects
(other than SFR projects)

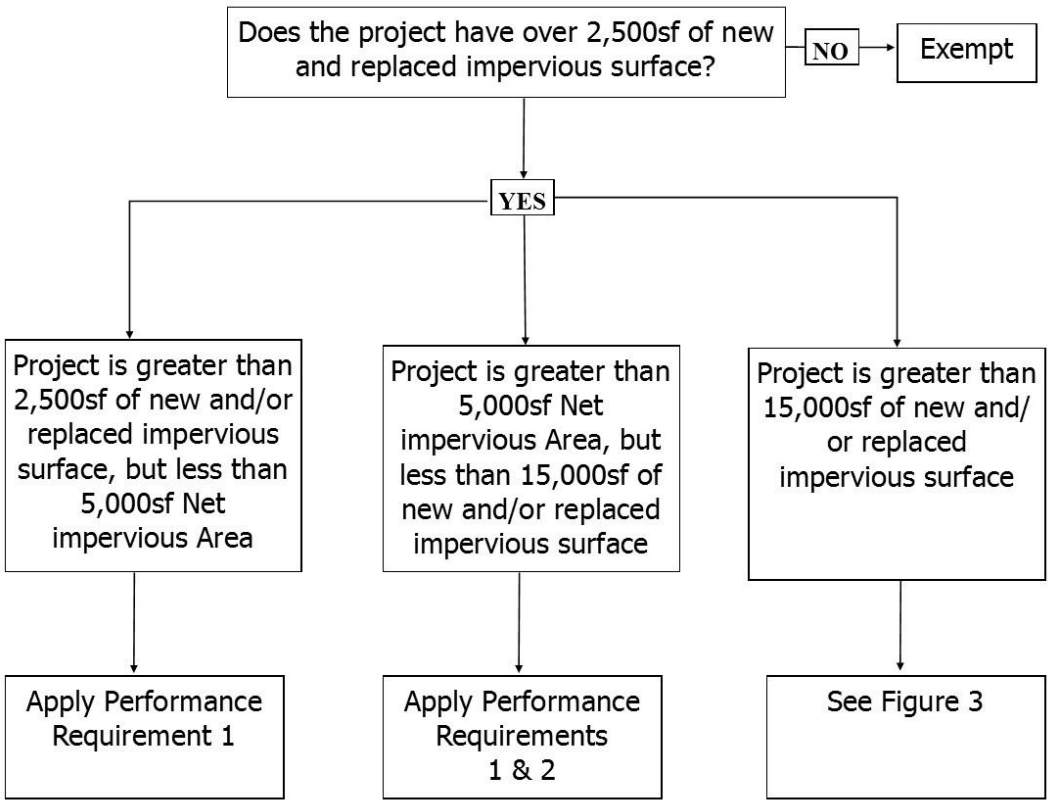


Figure 2

Performance Requirements Flow Chart for projects greater than 15,000sf of new and replaced impervious surface (other than SFR projects)

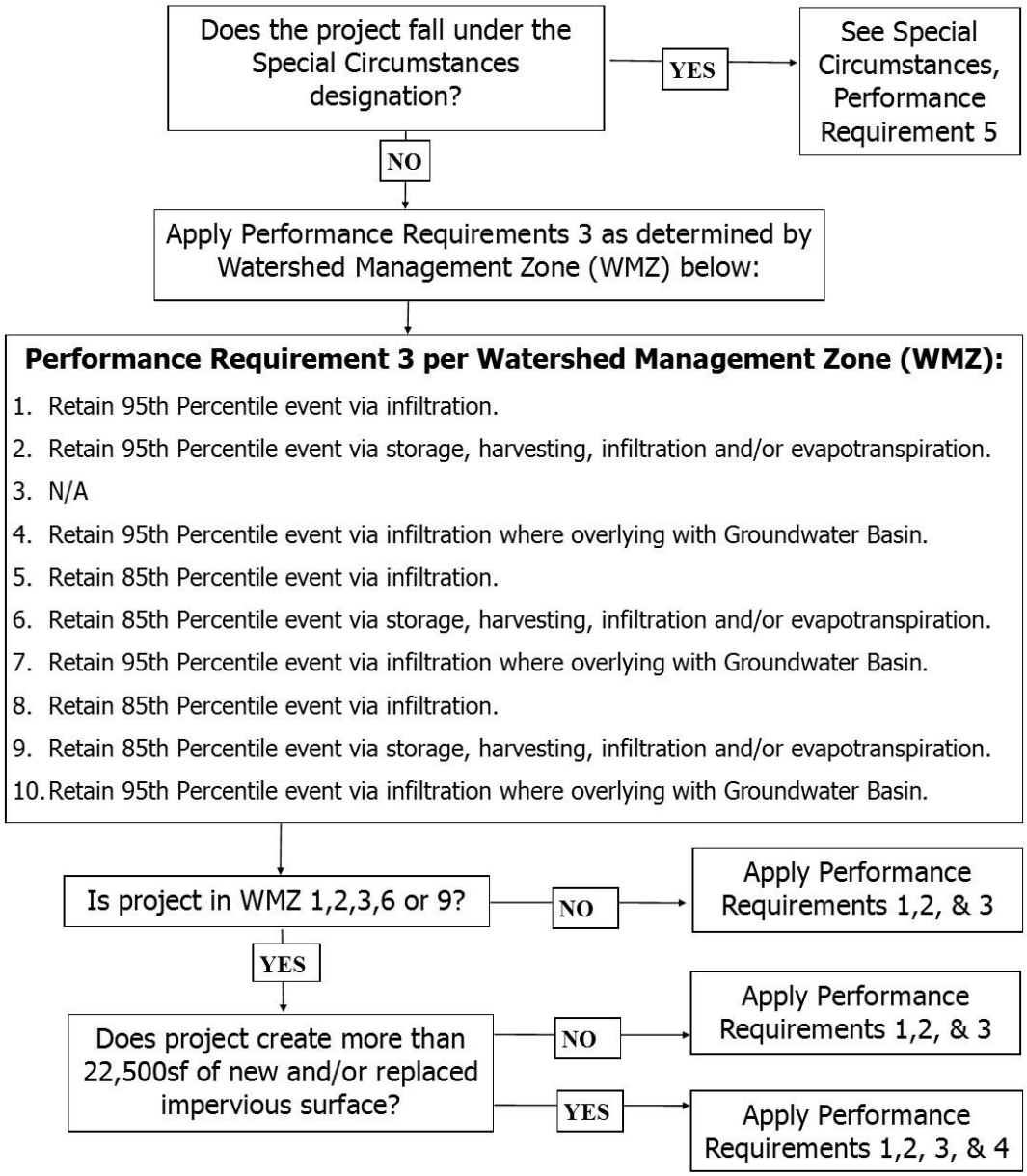


Figure 3

Section 1

Performance Requirement No. 1

Site Design and Runoff Reduction

Projects subject to Performance Requirement No. 1 (PR.1) are:

Projects that create and/or replace ≥ 2,500 square feet of impervious surface (collectively over the entire project site), including detached single-family homes.

The Project Engineer shall submit a stamped and signed copy of the Performance Requirement No.1 Certification, as included on the following page; certifying Low Impact Development design strategies are included in the project design. Each strategy that has been incorporated into the design should be initialed by the project engineer, or marked NA if not applicable.

PERFORMANCE REQUIREMENT NO. 1 CERTIFICATION		
	LOW IMPACT DEVELOPMENT (LID) DESIGN STRATEGY	INCORPORATED
1.	Limit disturbance of creeks and natural drainage features.	
2.	Minimize compaction of highly permeable soils.	
3.	Limit clearing and grading of native vegetation at the site to the minimum area needed to build the project, allow access, and provide fire protection.	
4.	Minimize impervious surfaces by concentrating improvements on the least sensitive areas of the site, while leaving the remaining land in a natural undisturbed state.	
5.	Minimize stormwater runoff by implementing one or more of the following design measures:	
	a) Direct roof runoff into cisterns or rain barrels for reuse.	
	b) Direct roof runoff onto vegetated areas safely away from building foundations and footings.	
	c) Direct runoff from sidewalks, walkways, and/or patios onto vegetated areas safely away from building foundations and footings.	
	d) Direct runoff from driveways and/or uncovered parking lots onto vegetated areas safely away from building foundations and footings.	
	e) Construct bike lanes, driveways, uncovered parking lots, sidewalks, walkways, and patios with permeable surfaces.	

I, _____, acting as the Project Engineer for _____ project, located at _____, hereby state that the Site Design and Runoff Reductions design strategies initialed above have been incorporated into the design of the project.

_____ _____

Signature Date

Section 2

Performance Requirement No. 2

Water Quality Treatment

Projects subject to Performance Requirement No. 2 (PR.2) are:

Projects with ≥ 5,000 square feet of Net Impervious Area, except detached single-family homes.

Detached single-family homes ≥ 15,000 square feet of Net Impervious Area.

Projects subject to PR.2 are also subject to PR.1. Applicant is required to submit PERFORMANCE REQUIREMENT NO.1 CERTIFICATION.

PR.2 requires the applicant to provide Water Quality Treatment design measures to reduce pollutant loads and concentrations using physical, biological, and chemical removal.

A Stormwater Control Plan is required and must follow the outline provided in Appendix K. The Project Engineer shall identify which of the on-site water quality treatment measures on the following page is included in the design. The on-site water quality treatment measures are listed in order of preference. The plans shall clearly identify the type, location, and size of all on-site water quality treatment measures. Initial each on-site water quality treatment measure and note the page of the plans that shows the location and size, that has been incorporated into the design or mark NA if not applicable. Submit a stamped and signed copy of the PR.2 Certification together with the Stormwater Control Plan.

PERFORMANCE REQUIREMENT NO. 2 CERTIFICATION		
	ON-SITE WATER QUALITY TREATMENT MEASURES	INCORPORATED
1.	Low Impact Development (LID) Treatment Systems designed to retain stormwater runoff generated by the 85th percentile 24-hour storm (see Appendix I). Stormwater Control Measures Implemented (check all that apply, design documentation is required): ☐ Harvesting and Use, ☐ Infiltration, ☐ Evapotranspiration	
2.	Biofiltration Treatment Systems – with the following design parameters: (1) Maximum surface loading rate appropriate to prevent erosion, scour and channeling within the biofiltration treatment system itself and equal to 5 inches per hour, based on the flow of runoff produced from a rain event equal to or at least 0.2 inches per hour intensity	
	(2) Follow Central Coast LIDI Bioretention Design Guidance for other parameters. If site conditions warrant, an underdrain with discharge to a storm drainage facility is allowed.	
3.	Non-Retention Based Treatment Systems – designed to meet at least one of the following hydraulic sizing criteria:	
	(a) Volume Hydraulic Design Basis – Treatment systems whose primary mode of action depends on volume capacity shall be designed to treat stormwater runoff equal to the volume of runoff generated by the 85th percentile 24-hour storm event (see Appendix I)	
	(b) Flow Hydraulic Design Basis – Treatment systems whose primary mode of action depends on flow capacity shall be sized to treat the flow of runoff resulting from a rain event equal to at least 0.2 inches per hour intensity.	
4.	Stormwater Control Plan is required – see Appendix K	

I, _____, acting as the Project Engineer for _____ project, located at _____, hereby state that the On-Site Water Quality Treatment Measures initialed above have been incorporated into the design of the project.

_____ _____

Signature Date

Section 3

Performance Requirement No. 3

Runoff Retention

Projects subject to Performance Requirement No. 3 (PR.3) are:

Single Family Residential Projects that create and/or replace ≥15,000 square feet of Net Impervious Area and other projects which create and/or replace ≥15,000 square feet of Gross Impervious Area in Watershed Management Zones (WMZ) 1, 2, 4*, 5, 6, 7* 8, 9, and 10*

(* Applicable only to those areas that overlay designated Groundwater Basins.)

PR. 3 requires the applicant to manage Runoff Retention from the site.

Adjustment to PR.3 Requirements for Redevelopment Projects – Where the Project includes replaced impervious surface, the following adjustment applies. This adjustment is accounted for in the Tributary Area calculation in Appendix E:

i) The total amount of replaced impervious surface shall be multiplied by 0.5 when calculating the volume of runoff subject to PR.3 requirements.

The Project Engineer shall certify that the Runoff Retention requirements were included in the design either onsite or through an Alternative Compliance agreement.

Compliance with PR.3:

- Identify WMZ and treatment requirements
- LID Site Assessment Check List
- LID Site Design Measures Certification
- A Stormwater Control Plan is required (see Appendix K) and shall include discrete Drainage Management Areas (DMAs), structural Stormwater Control Measures (SCM), hydraulic sizing calculations, and off-site mitigation.

Design Rainfall Events & Treatment Requirement for WMZs

WMZ	Treatment Options & Design Rainfall	Check Applicable WMZs
WMZ 1	Via Infiltration, prevent offsite discharge from events up to the 95th percentile 24-hour rainfall event as determined from local rainfall data.	
WMZ 2	Via storage, rainwater harvesting, infiltration, and/or evapotranspiration, prevent offsite discharge from events up to the 95th percentile 24-hour rainfall event as determined from local rainfall data.	
WMZ 3	Not Applicable	
WM 4*, 7* & 10*	Via Infiltration, prevent offsite discharge from events up to the 95th percentile 24-hour rainfall event as determined from local rainfall data.	
WMZ 9	Via storage, rainwater harvesting, infiltration, and/or evapotranspiration, prevent offsite discharge from events up to the 85th percentile 24-hour rainfall event as determined from local rainfall data.	
* Applicable only to those areas that overlay designated Groundwater Basins		

LID Site Assessment Checklist

	Included
1. Site topography	
2. Hydrologic features including contiguous natural areas, wetlands, watercourses, seeps, or springs	
3. Depth to seasonal high groundwater	
4. Locations of groundwater wells used for drinking water	
5. Depth to an impervious layer such as bedrock	
6. Presence of unique geology (e.g., karst)	
7. Geotechnical hazards	
8. Documented soil and/or groundwater contamination	
9. Soil types and hydrologic soil groups	
10. Vegetative cover/trees	
11. Run-on characteristics (source and estimated runoff from offsite which discharges to the project area)	
12. Existing drainage infrastructure for the site and nearby areas including the location of municipal storm drains	
13. Structures including retaining walls	
14. Utilities	
15. Easements	
16. Covenants	
17. Zoning/Land Use	
18. Setbacks	
19. Open space requirements	
20. Other pertinent overlay(s)	

LID Site Design Measures

In addition to site design measures listed in PR1, the Project Engineer shall certify the Project design optimizes the use of the following design measures. Initial each runoff retention measure that has been incorporated and optimized into the design or mark NA if not applicable

PERFORMANCE REQUIREMENT NO. 3 CERTIFICATION OF LID SITE DESIGN MEASURES		
	DESIGN MEASURE	INCORPORATED/OPTIMIZED
1.	Defining the development envelope, identifying the protected areas, and identifying areas that are most suitable for development and areas to be left undisturbed	
2.	Identifying conserved natural areas, including existing trees, other vegetation, and soils (shown on the plans)	
3.	Limit the overall impervious footprint of the project	
4.	Design of streets, sidewalks, or parking lot aisles to the minimum widths necessary, provided that public safety or mobility uses are not compromised	
5.	Set back development from creeks, wetlands, and riparian habitats	
6.	Design conforms the site layout along natural landforms	
7.	Design avoids excessive grading and disturbance of vegetation and soils	

I, _____, acting as the Project Engineer for _____ project, located at _____, hereby state that LID Site Design Measures initialed above have been incorporated into the design of the project.

_____ _____

Signature Date

Drainage Management Areas (DMAs)

The site shall be designed to provide for decentralized stormwater management with discrete DMAs.

(1) Provide a map of the entire project site showing the discrete DMAs and detailing the drainage for the rainfall event specified.

(2) Drainage from each DMA shall be accounted for using the following measures:

(a) Undisturbed or areas planted with native vegetation that do not receive runoff from other areas are self-treating and no additional stormwater management is required.

(b) Runoff from impervious surfaces, generated by the rainfall events identified for PR.3, may be directed to undisturbed or natural landscaped areas. Where the design demonstrates that this runoff will be infiltrated and will not produce runoff to the storm drain system, or a surface receiving waterbody, or create nuisance ponding, then no additional stormwater management is required for these impervious surfaces.

(c) Runoff that cannot be captured by the above methods must be managed by a structural stormwater control measure as described below.

Structural Stormwater Control Measures (SCM)

SCMs shall optimize retention and result in optimal protection and restoration of watershed processes. SCMs are typically small-scale, decentralized facilities designed to infiltrate, evapotranspirate, filter, or capture and use stormwater. SCMs shall be sized and designed as described in Appendix E.

Off-Site Mitigation

Off-site mitigation of full Retention Volume is not required where technical infeasibility, as described in the Alternate Compliance section (Section 7), limits on-site compliance with the Runoff Retention Performance Requirement AND ten percent of a project's Equivalent Impervious Surface Area [1] has been dedicated to retention-based Stormwater Control Measures. (NOTE: PR.2, the Water Quality Treatment Performance Requirement is NOT subject to this adjustment. Mitigation to achieve full compliance with the Water Quality Treatment Performance Requirement is required on- or off-site.)

[1] Calculate Equivalent Impervious Surface Area using guidance in Appendix F

- Use the Appendix F instructions to calculate the ten percent adjustment for applying the Runoff Retention Performance Requirement.

- Use the Appendix G instructions to calculate the Off-Site retention requirements when a Project subject to the Runoff Retention Performance Requirement cannot allocate the full ten percent of the project site's Equivalent Impervious Surface Area to retention-based Stormwater Control Measures.

Section 4

Performance Requirement No. 4

Peak Management

Projects subject to Performance Requirement No. 4 (PR.4) are:

- Projects that create and/or replace ≥22,500 square feet of gross impervious surface (collectively over the entire project site); and are In Watershed Management Zones 1, 2, 3, 6, or 9

PR. 4 requires the applicant to manage post-development peak stormwater runoff from the site.

The Project Engineer shall provide a Stormwater Control Plan that includes a Hydrology Analysis demonstrating that post-development stormwater runoff peak flows discharged from the site do not exceed pre-project peak flows for the 2- through 10- year storm events. If detention storage is required, use the same design method described in the following Section 5.

Section 5

Performance Requirement No. 5

Special Circumstances

Special Circumstances as defined by the Central Coast Regional Water Quality Control Board:

Projects that are subject to Special Circumstances based on certain site and/or receiving water conditions may be exempt from Runoff Retention and/or Peak Management Performance Requirements where those Performance Requirements would be ineffective to maintain or restore beneficial uses of receiving waters.

Special Circumstances are defined as projects that discharge stormwater to the following:

- Highly Altered Channels
- Intermediate Flow Control Facility
- Historic Lake and Wetland

Projects subject to Special Circumstances must still comply with the Water Quality Treatment Performance Requirements if so required.

Section 6

Alternative Compliance

Technical Infeasibility

When on-site compliance is demonstrated to be technically infeasible, Water Quality Treatment (PR.2), Runoff Retention (PR.3), and Peak Management Performance Requirements (PR.4) may be achieved through off-site SCMs or through use of the Ten Percent Adjustment to Retention Requirement (Appendix F).

An application for approval of Alternative Compliance based on technical infeasibility shall include a site-specific hydrologic analysis conducted and endorsed by a registered professional engineer, geologist, architect, and/or landscape architect, demonstrating that compliance with the applicable numeric Post-Construction Stormwater Management Requirements is technically infeasible.

Technical Infeasibility may be caused by site conditions, including:

i) Depth to seasonal high groundwater limits infiltration and/or prevents construction of subgrade stormwater control measures[2]
ii) Depth to an impervious layer such as bedrock limits infiltration
iii) Sites where soil types significantly limit infiltration
iv) Sites where pollutant mobilization in the soil or groundwater is a documented concern
v) Space constraints (e.g., infill projects, some redevelopment projects, high density development)
vi) Geotechnical hazards
vii) Stormwater Control Measures located within 100 feet of a groundwater well used for drinking water
viii) Incompatibility with surrounding drainage system (e.g., project drains to an existing stormwater collection system whose elevation or location precludes connection to a properly functioning treatment or flow control facility)

See Appendix H for methods of determining feasibility of infiltration measures.

[2] According to the CASQA Frequently Asked Questions about LID, "some MS4 permits and BMP guidance manuals require anywhere from 3-10 feet of separation from the groundwater level for infiltration practices. This distance depends on the soil type, pollutants of concern, and groundwater use. In some cases, however, where there may be groundwater or soil contamination, LID infiltrative practices may be restricted completely. (p. 7 in https://www.casqa.org/Portals/0/LID/CA_LID_FAQ_06-28-2011.pdf)

If the Technical Infeasibility analysis is approved, the use of the Ten Percent Adjustment to Retention Requirement method of compliance may be used (Appendix F). For off-site mitigation, a plan detailing the project(s) that will provide off-site mitigation shall be submitted. The proposed off-site projects may be existing facilities and/or prospective projects that are as effective in maintaining watershed processes as implementation of the applicable Post-Construction Stormwater Requirements on-site. The description shall include:

a) The location of the proposed off-site project(s), which must be within the same watershed as the Regulated Project. Alternative Compliance project sites located outside the watershed may be approved by the Central Coast Water Board Executive Officer.
b) A schedule for completion of offsite mitigation project(s), where the off-site mitigation project(s) has not been constructed.

Watershed or Regional Management Plan

The City of Morro Bay does not have an approved Watershed or Regional Management Plan**.**

Urban Sustainability Area

Projects located within an approved Urban Sustainability Area (USA) may apply for Alternative Compliance for numeric Runoff Retention and Peak Management Performance Requirements without demonstrating technical infeasibility; however the City of Morro Bay has not established an Urban Sustainability Area.

Application for approval of an Urban Sustainability Area may be made to the Central Coast Regional Water Quality Control Board. The Urban Sustainability Area may only encompass redevelopment in high density urban centers that are pedestrian-oriented and/or transit-oriented development projects intended to promote infill of existing urban areas. The USA proposal must include, at minimum:

i) A definition and delineation of the USA for high-density infill and redevelopment for which area-wide approval for Alternative Compliance is sought.
ii) Information and analysis that supports the intention to balance water quality protection with the needs for adequate housing, population growth, public transportation, land recycling, and urban revitalization.
iii) Demonstration that implementation of Alternative Compliance for Regulated Projects in the USA will meet or exceed the on-site requirements for Runoff Retention and Peak Management. The proposal must include quantitative analysis used to evaluate off-site compliance. Identification of specific off-site projects is not necessary for approval of the USA designation.

Projects in a USA may meet Water Quality Treatment Performance Requirements off-site only when:

i) It has been demonstrated that on-site water quality treatment is Technically Infeasible; AND
ii) The proposed off-site project(s) have been demonstrated to comply with the Water Quality Treatment Performance Requirements.

Off-Site Compliance Project(s) Requirements

Location of Alternative Compliance Project(s) – The location of the proposed off-site project(s) must be within the same watershed as the Project. Alternative Compliance project sites located outside the watershed may be approved by the Central Coast Water Board Executive Officer.

Timing and Funding Requirements for Alternative Compliance Projects – A schedule for the completion of off-site mitigation projects, including milestone dates to identify funding, design, and construction of the off-site projects shall be submitted with the application for Alternative Compliance.

a) Complete the project(s) as soon as practicable and no longer than four years from the date of the certificate of occupancy for the project for which off-site mitigation is required, unless a longer period is otherwise authorized by the Central Coast Water Board Executive Officer.
b) The timeline for completion of the off-site mitigation project may be extended, up to five years with prior Central Coast Water Board Executive Officer approval. Central Coast Water Board Executive Officer approval will be granted contingent upon a demonstration of good faith efforts to implement an Alternative Compliance project, such as having funds encumbered and applying for the appropriate regulatory permits.
c) Off-site mitigation projects on public property shall be fully funded by the applicants.
d) Off-site mitigation projects on private property shall include all documentation necessary to provide legal authority to use the property for the mitigation and shall include project bonding.

Section 7

Maintenance and Reporting

An Operation and Maintenance Plan (O&M) is required for all projects that utilize Structural Control Measures (SCMs) to satisfy Performance Requirements 2, 3, and/or 4. A maintenance program is essential to ensure that the stormwater facilities continue to function as designed to maintain water quality and prevent possible flooding and property damage.

A proper maintenance plan must include:

- Site map of all SCMs requiring O&M practices to function as designed
- Procedures are provided for each structural control measure including, but not limited to, LID facilities, retention/detention basins, and proprietorship devices
- Short and long term maintenance requirements
- Estimated cost for maintenance

Appendix J has templates to aid in the development of the O&M Plan.

The SWCP and O&M plan shall be prepared under the direction of a professional civil engineer registered in the State of California. The plans shall be stamped, signed and include a certifying statement indicating that all stormwater BMPs have been designed to meet the City's stormwater quality requirements.

Applicants of regulated projects subject to Performance Requirements 2, 3, or 4 are required to demonstrate compliance with these requirements on an annual basis.

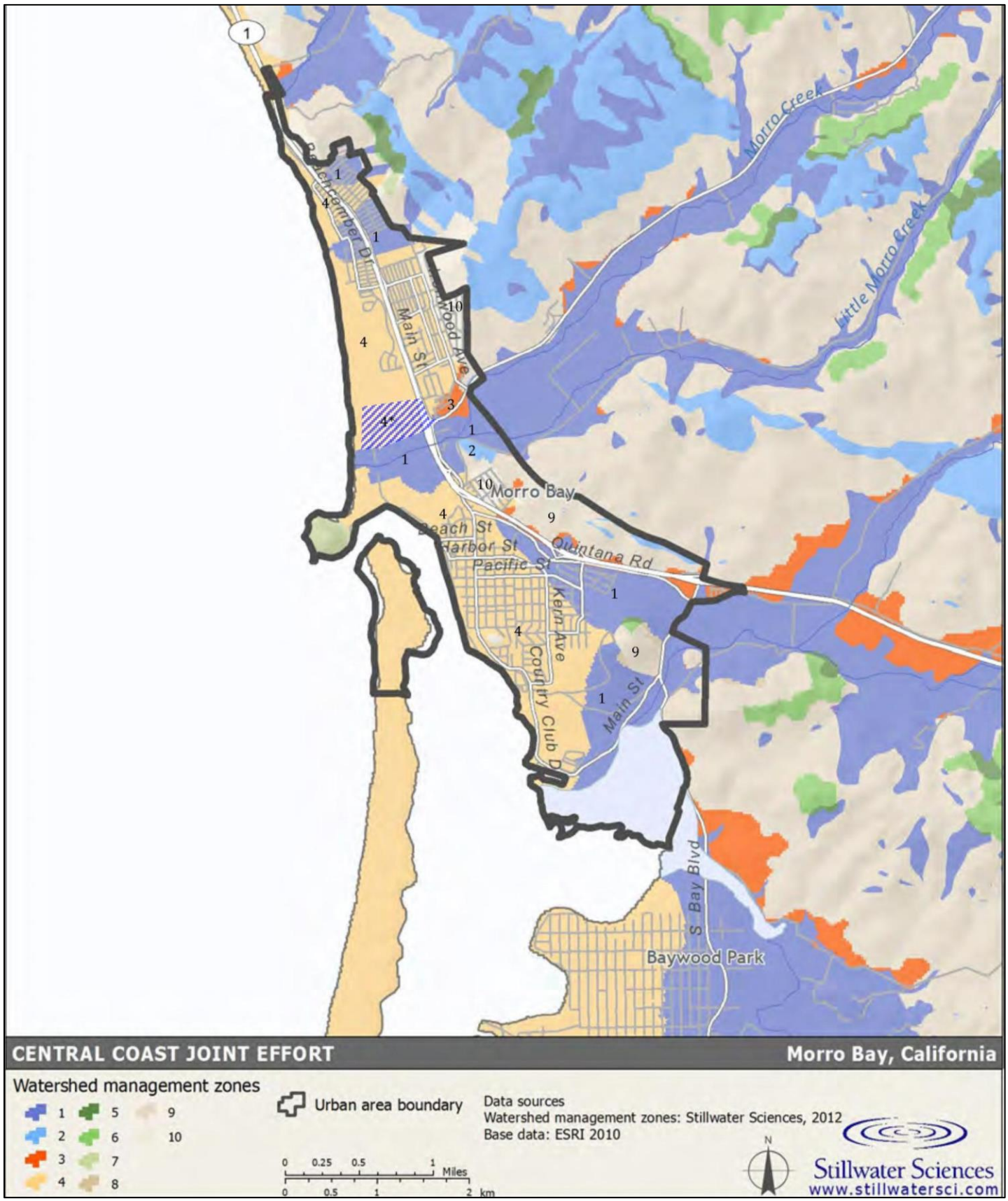
APPENDIX A

PROJECTS EXEMPT FROM STORMWATER REQUIREMENTS

Project that are exempt from the Post-Construction Performance are as follows (check any box on the list and no further action is required):

- ☐ Road and Parking Lot maintenance:
  - Road surface repair including slurry sealing, fog sealing, and pothole and square cut patching
  - Overlaying existing asphalt or concrete pavement with asphalt or concrete without expanding the area of coverage
  - Shoulder grading
  - Cleaning, repairing, maintaining, reshaping, or re-grading drainage systems
  - Crack sealing
  - Resurfacing with in-kind material without expanding the road or parking lot
  - Practices to maintain original line and grade, hydraulic capacity, and overall footprint of the road or parking lot
  - Repair or reconstruction of the road because of slope failures, natural disasters, acts of God or other man-made disaster
- ☐ Sidewalk and bicycle path or lane projects, where no other impervious surfaces are created or replaced, built to direct stormwater runoff to adjacent vegetated areas
- ☐ Trails and pathways, where no other impervious surfaces are replaced or created, and built to direct stormwater runoff to adjacent vegetated areas
- ☐ Underground utility projects that replace the ground surface with in-kind material or materials with similar runoff characteristics
- ☐ Curb and gutter improvement or replacement projects that are not part of any additional creation or replacement of impervious surface area (e.g., sidewalks, roadway)
- ☐ Second-story additions that do not increase the building footprint
- ☐ Roof repair and replacement
- ☐ Raised (not built directly on the ground) decks, stairs, or walkways designed with spaces to allow for water drainage
- ☐ Photovoltaic systems installed on/over existing roof or other impervious surfaces, and panels located over pervious surfaces with well-maintained grass or vegetated groundcover, or panel arrays with a buffer strip at the most down gradient row of panels
- ☐ Temporary structures (in place for less than six months)
- ☐ Electrical and utility vaults, sewer and water lift stations, backflows and other utility devices
- ☐ Above-ground fuel storage tanks and fuel farms with spill containment system

APPENDIX B
WATERSHED MANAGEMENT ZONES



APPENDIX C

PERFORMANCE REQUIREMENT DETERMINATION FORM

The following form shall be completed for all development and redevelopment projects. Projects that are exempt from performance requirements are required to complete Sections 1 & 2 only.

Section 1: General Information	
Project name	
Project Address	
Assessor's Parcel Number(s)	
Name of Applicant	
Applicant email address:	
Applicant phone:	
Project Type (e.g. single-family residential, commercial, etc.)	
Section 2: Area Information	
Total Project Area	
Total Existing impervious surface area	
Proposed Gross Impervious Area (list only the surface areas that are being created or replaced)	
a. Rooftops	
b. Driveways	
c. Patios	
d. Parking Lots	
e. Other	
Total Gross Impervious Area	
If Gross Impervious Area <2,500 ft^2, write "EXEMPT". Otherwise continue to Sec. 3	
Section 3: PR Determination	
Watershed Management Zone (App. B)	
Net Impervious Area (from page 10)	
Performance Requirements (from Flow Charts)	

APPENDIX D

Definitions Related to Post-Construction Requirements

Bioretention – A Stormwater Control Measure designed to retain stormwater runoff using vegetated depressions and soils engineered to collect, store, treat, and infiltrate runoff. Bioretention designs do not include underdrains.

Biotreatment or Biofiltration Treatment –A Stormwater Control Measure designed to detain stormwater runoff, filter stormwater through soil media and plant roots, and release the treated stormwater runoff to the storm drain system. Biotreatment systems include an underdrain.

Discretionary Approval – A project approval which requires the exercise of judgment or deliberation when the MS4 decides to approve or disapprove a particular activity, as distinguished from situations where the MS4 merely has to determine whether there has been conformity with applicable statutes, ordinances, or regulations.

Dispersion – The practice of routing stormwater runoff from impervious areas, such as rooftops, walkways, and patios, onto the surface of adjacent pervious areas. Stormwater runoff is dispersed via splash block, dispersion trench, or sheet flow and soaks into the ground as it moves slowly across the surface of the pervious area.

Drainage Management Area (DMAs) – Following the low impact development principle of managing stormwater through small-scale, decentralized measures, DMAs are designated individual drainage areas within a Regulated Project that typically follow grade breaks and roof ridge lines and account for each surface type (e.g., landscaping, pervious paving, or roofs). Stormwater Control Measures for runoff reduction and structural facilities are designed for each DMA.

Equivalent Impervious Surface Area – is equal to *Impervious Tributary Surface Area* (ft^2) + *Pervious Tributary Surface Area* (ft^2), where *Impervious Tributary Surface Area* is defined as the sum of all of the site's conventional impervious surfaces, and *Pervious Tributary Surface Area* is defined as the sum of all of the site's pervious surfaces, corrected by a factor equal to the surface's runoff coefficient.

Evapotranspiration (ET) – The loss of water to the atmosphere by the combined processes of evaporation (from soil and plant surfaces) and transpiration (from plant tissues).

Flow-Through Water Quality Treatment Systems – Stormwater Control Measures that are designed to treat stormwater through filtration and/or settling. Flow-through systems do not provide significant retention or detention benefits for stormwater volume control.

Gross Impervious Area – Impervious surfaces that are created or replaced by the project. Manufactured permeable surfaces (pervious paving, gapped paving stones, etc.) may be considered as a pervious surface and are considered on a case by case basis. If sidewalks or new pavement in the City right of way is planned or required by code, these surfaces shall also be included in the total. Do not include the

surface area of decks with gaps that allow runoff to drain to permeable surfaces below. Gross Impervious Area is used in the initial determination of performance requirements.

Groundwater Basins – Groundwater basin areas defined by the California Department of Water Resources (DWR) and used in the Central Coast Water Board Joint Effort for Hydromodification Control to identify groundwater receiving-water issues and areas where recharge is a key watershed process. DWR based identification of the groundwater basins on the presence and areal extent of unconsolidated alluvial soils identified on a 1:250,000 scale from geologic maps provided by the California Department of Conservation, Division of Mines and Geology. DWR then further evaluated identified groundwater basin areas through review of relevant geologic and hydrogeologic reports, well completion reports, court-determined adjudicated basin boundaries, and contact with local agencies to refine the basin boundaries.

Impervious Surface – A hard, non-vegetated surface area that prevents or significantly limits the entry of water into the soil mantle, as would occur under natural conditions prior to development. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or storage areas, concrete or asphalt paving, oiled, macadam or other surfaces which similarly impede the natural infiltration of stormwater. Open, uncovered retention/detention facilities shall not be considered as impervious surfaces for purposes of determining whether the thresholds for application of Performance Requirements are exceeded. However, for modeling purposes, open, uncovered facilities that retain/detain water (e.g., retention ponds, pools) shall be considered impervious surfaces. There are three methods of calculating impervious surface area, depending on the context of the calculation. For more details, see ***Net Impervious Area, Gross Impervious Area, and Equivalent Impervious Area*** definitions.

Land recycling – The reuse of abandoned, vacant, or underused properties for redevelopment or repurposing

Landscaped Areas – Areas of soil and vegetation not including any impervious surfaces of ancillary features such as impervious patios, BBQ areas, and pools.

Large River – A river draining 200 square miles or more.

Low Impact Development (LID) – A stormwater and land use management strategy that strives to mimic pre-disturbance hydrologic processes of infiltration, filtration, storage, evaporation, and transpiration by emphasizing conservation, use of on-site natural features, site planning, and distributed stormwater management practices that are integrated into a project design.

Ministerial Approval – A project approval which involves little or no personal judgment by the MS4 as to the wisdom or manner of carrying out the project and only involves the use of fixed standards or objective measurements.

Native Vegetation – Vegetation comprised of plant species indigenous to the Central Coast Region and which reasonably could have been expected to naturally occur on the site.

Net Impervious Area – The sum of new and replaced post-project impervious areas, minus any reduction in total imperviousness from the pre-project to post-project condition: *Net Impervious Area = (New and Replaced Impervious Area) – (Reduced Impervious Area Credit),* where *Reduced Impervious Area Credit* is the total pre-project to post-project reduction in impervious area, if any.

New Development – Land disturbing activities that include the construction or installation of buildings, roads, driveways and other impervious surfaces. Development projects with pre-existing impervious surfaces are not considered New Development.

Percentile Rainfall Event (e.g., 85th and 95th) – A percentile rainfall event represents a rainfall amount which a certain percent of all rainfall events for the period of record do not exceed. For example, the 95th percentile rainfall event is defined as the measured rainfall depth accumulated over a 24-hour period, for the period of record, which ranks as the 95th percentile rainfall depth based on the range of all daily event occurrences during this period.

Permeable or Pervious Surface – A surface that allows varying amounts of stormwater to infiltrate into the ground. Examples include pasture, native vegetation areas, landscape areas, and permeable pavements designed to infiltrate.

Pre-Project – Stormwater runoff conditions that exist onsite immediately before development activities occur. This definition is not intended to be interpreted as that period before any human-induced land activities occurred. This definition pertains to redevelopment as well as initial development.

Project Site – The area defined by the legal boundaries of a parcel or parcels of land within which the new development or redevelopment takes place and is subject to these Post-Construction Stormwater Management Requirements.

Rainwater Harvest – Capture and storage of rainwater or stormwater runoff for later use, such as irrigation (without runoff), domestic use (e.g. toilets), or storage for fire suppression.

Receiving Waters – Bodies of water, surface water systems or groundwater that receive surface water runoff through a point source, sheet flow or infiltration.

Redevelopment – On a site that has already been developed, construction or installation of a building or other structure subject to the Permittee's planning and building authority including: 1) the creation or addition of impervious surfaces; 2) the expansion of a building footprint or addition or replacement of a structure; or 3) structural development including construction, installation or expansion of a building or other structure. It does not include routine road maintenance, nor does it include emergency construction activities required to immediately protect public health and safety.

Replaced Impervious Surface – The removal of existing impervious surfaces down to bare soil or base course, and replacement with new impervious surface. Replacement of impervious surfaces that are part of routine road maintenance activities are not considered replaced impervious surfaces.

Self-Retaining Areas – (also called "zero discharge" areas), are designed to retain some amount of rainfall (by ponding and infiltration and/or evapotranspiration) without producing stormwater runoff. Self-Retaining Areas may include graded depressions with landscaping or pervious pavement.

Self-Treating Areas – are a portion of a Regulated Project in which infiltration, evapotranspiration and other natural processes remove pollutants from stormwater. The self-treating areas may include conserved natural open areas and areas of native landscaping. The self-treating area only treats the rain falling on itself and does not receive stormwater runoff from other areas.

Routine Road Maintenance – includes pothole and square cut patching; overlaying existing asphalt or concrete pavement with asphalt or concrete without expanding the area of coverage; shoulder grading; reshaping/regrading drainage systems; crack sealing; resurfacing with in-kind material without expanding the road prism or altering the original line and grade and/or hydraulic capacity of the road.

Single-Family Residence – The building of one single new house or the addition and/or replacement of impervious surface associated with one single existing house, which is not part of a larger plan of development.

Stormwater Control Measures – Stormwater management measures integrated into project designs that emphasize protection of watershed processes through replication of pre-development runoff patterns (rate, volume, duration). Physical control measures include, but are not limited to, bioretention/rain gardens, permeable pavements, roof downspout controls, dispersion, soil quality and depth, minimal excavation foundations, vegetated roofs, and water use. Design control measures include but are not limited to conserving and protecting the function of existing natural areas, maintaining or creating riparian buffers, using onsite natural drainage features, directing runoff from impervious surfaces toward pervious areas, and distributing physical control measures to maximize infiltration, filtration, storage, evaporation, and transpiration of stormwater before it becomes runoff.

Stormwater Control Plan – A plan, developed by the Regulated Project applicant, detailing how the project will achieve the applicable Post-Construction Stormwater Management Requirements (for both onsite and offsite systems).

Tributary Area – The entire project area except for undisturbed areas of planted areas with native vegetation that do not receive runoff from other areas and impervious surface areas that discharge to infiltration areas that will not produce runoff or create nuisance ponding. The Drainage Management Areas are smaller Tributary Areas that cumulatively make up the Tributary Area of the entire site.

APPENDIX E

Hydrologic Analysis and Stormwater Control Measure Sizing Guidance

Project site conditions will influence the ability to comply with the Water Quality Treatment and Runoff Retention Performance Requirements. This Appendix provides the acceptable Stormwater Control Measure (SCM) sizing methodology to evaluate runoff characteristics. This guidance provides a simple event-based approach and a runoff routing approach. Both of these approaches are based on sizing for a single-event and avoid the necessity of using calibrated, continuous simulation modeling. The project applicant may use a locally/regionally calibrated continuous simulation-based model to improve hydrologic analysis and SCM sizing.

1) Determination of Retention Tributary Area

Determining the Retention Tributary Area is the basis for calculating the runoff volumes subject to Performance Requirement Number 3. Retention Tributary Area should be calculated for each individual Drainage Management Area to facilitate the design of SCMs for each Drainage Management Area. The generic equation below illustrates how various portions of the site are addressed when determining the Retention Tributary Area. The Retention Tributary Area calculation must also account for the adjustments for Redevelopment Projects subject to Performance Requirement No. 3.

a) Compute the Retention Tributary Area, using the equation:

Retention Tributary Area = (Entire Project Area) – (Undisturbed or Planted Areas) – (Impervious Surface Areas that Discharge to Infiltrating Areas)***

* As defined under Drainage Management Areas 2.a (in Section 3)

** As defined under Drainage Management Areas 2.b. (in Section 3)

a) Adjustments for Redevelopment Project Retention Tributary Area – Where the Regulated Project includes replaced impervious surface, the following Retention Tributary Area adjustments apply:

i) Redevelopment Projects outside an approved Urban Sustainability Area, as described in the Alternative Compliance Section. – The total amount of replaced impervious surface area shall be multiplied by 0.5 when calculating the Retention Tributary Area.

ii) Redevelopment Projects located within an approved Urban Sustainability Area, as described in the Alternative Compliance Section. – The replaced impervious surface areas may be subtracted from the Retention Tributary Area. The total amount of runoff volume to be retained from replaced impervious surfaces shall be equivalent to the pre-project runoff volume retained.

2) Determination of Retention Volume

a) Based on the Regulated Project's Watershed Management Zone, determine the Regulated Project's Runoff Retention Requirement (e.g., Retain 95th Percentile 24-hour Rainfall Event, or, Retain 85th Percentile 24-hour Rainfall Event).

b) Determine the 85th or 95th percentile 24-hour rainfall event (Appendix I)

c) Compute the Runoff Coefficient[3] "C" for the area tributary to the SCMs, using the equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04$$

Where "i" is the fraction of the tributary area that is impervious[4]

d) Compute Retention Volume:

Retention Volume for 95th Percentile 24-hr Rainfall Depth = C x Rainfall Depth$_{95th}$ x Retention Tributary Area

or,

Retention Volume for 85th Percentile 24-hr Rainfall Depth = C x Rainfall Depth$_{85th}$ x Retention Tributary Area

All rainfall directly incident to each SCM must be considered in determining runoff, including: tributary landscaping, impervious areas, pervious pavements, and bioretention features.

Note: For redevelopment projects located within an approved Urban Sustainability Area the total amount of runoff volume to be retained from replaced impervious surfaces shall be equivalent to the pre-project runoff volume retained.

3) Structural Stormwater Control Measure Sizing

The Project engineer shall use structural SCMs that optimize retention and result in optimal protection and restoration of watershed processes, such as Structural Control Measures associated with small-scale, decentralized facilities designed to infiltrate, evapotranspirate, filter, or capture and use stormwater, to address the volumes calculated in 2 (above). Where the Project is within a Watershed Management Zone where infiltration is required, SCM designs that optimize infiltration of the entire Retention Volume is required, to minimize the potential need for off-site mitigation. Various resources provide design guidance for fully infiltrative SCMs including:

- The Contra Costa C.3 Manual
- The City of Santa Barbara LID BMP Manual
- The City of San Diego LID Design Manual, July 2011
- Central Coast LID Initiative Bioretention Design Guidance

[3] As set forth in WEF Manual of Practice No. 23/ASCE Manual of Practice No. 87, (1998), pages 175-178 and based on the translation of rainfall to runoff using a runoff regression equation developed using two years of data from more than 60 urban watersheds nationwide.

[4] As defined in Post-Construction Requirements Attachment D.

a) Calculate SCM Capture Volume – Calculate the required SCM Capture Volume, associated with the Regulated Project's Runoff Retention Requirement, by one of the following methods:

Method 1: Simple Method

SCM Capture Volume = Retention Volume for 95th Percentile 24-hr Rainfall Depth

or,

SCM Capture Volume = Retention Volume for 85th Percentile 24-hr Rainfall Depth

Method 2: Routing Method

Use a hydrograph analysis[5] to determine the SCM Capture Volume needed to retain the Retention Volume for 95th or 85th Percentile 24-hr Rainfall Depth calculated in 2 (above). The SCM Capture Volume shall be based on both the rate of flow from tributary areas into the SCM, and the rate of flow out of the SCM through infiltration into the underlying soil during the rain event. When conducting the hydrograph analysis, adhere to the criteria included in Table 1. The SCM shall be designed such that a single 95th or 85th Percentile 24-hr Rainfall Event will not overflow the SCM.

If the Retention Volume cannot infiltrate within 48-hours, a multiplier of 1.20 shall be applied to the SCM Capture Volume calculated through the routing method.

TABLE 1: Routing Method Criteria

Parameter	Criteria
Hydrograph Analysis Method	National Resources Conservation Service
Pond Routing Method	Storage-indication, unless otherwise justified to be more correct based on site and storage conditions.
Infiltration Rate	Underlying soil saturated infiltration rate, as indicated by on-site testing.
Rainfall Distribution	National Resources Conservation Service Type I[6] or based on local rainfall data
Time of Concentration	No less than 10 minutes
Time Increment	0.10 hour, unless otherwise justified to be more correct based on rainfall distribution

[5] HydroCAD is an example of a commonly used and widely accepted program for performing hydrograph analyses and design of stormwater infrastructure. HydroCAD is based on U.S. Department of Agriculture Soil Conservation Service's (now Natural Resources Conservation Service) TR-55: Urban Hydrology for Small Watersheds.

[6] The National Resources Conservation Service developed standard 24-hour rainfall distributions for hydrograph analyses. These rainfall distributions were intended to represent intensities associated with shorter duration storms, ranging from durations of 30 minutes to 12 hours. The National Resources Conservation Service Type 1 storm applies to the California West Coast, including the Central Coast Region. The Type 1 rainfall distribution was derived using National Oceanic Atmospheric Administration Atlas 2 rainfall statistics for the 1-year through 100-year storm.

b) Demonstration of Compliance – Project engineer shall demonstrate that site SCMs: a) will infiltrate and/or evapotranspirate the Retention Volume or, b) will provide sufficient Capture Volume to retain the Retention Volume. Any outlet (i.e., underdrain) installed in a structural SCM shall be installed above the elevation of any portion of the structural SCM dedicated to Retention Volume storage.

Compliance with Water Quality Treatment Performance Requirement – Projects that propose to use the retention-based structural Stormwater Control Measures, shall also meet the Water Quality Treatment Performance Requirement, and demonstrate in the Stormwater Control Plan, that the Water Quality Treatment Performance Requirement is being fully met.

APPENDIX F

Ten Percent Adjustment to Retention Requirement – Calculation Instructions

Where technical infeasibility, as described in the Alternative Compliance, prevents full on-site compliance with the Runoff Retention Performance Requirement, on-site retention of the full Retention Volume per Section PR.3 is not required and the Project is required to dedicate no less than ten percent of the Project's Equivalent Impervious Surface Area to retention-based Stormwater Control Measures. The Water Quality Treatment Performance Requirement is not subject to this adjustment, i.e.,

Calculating Ten Percent of a Project's Equivalent Impervious Surface Area

The area of the project that must be dedicated to structural SCMs to waive off-site compliance with the Runoff Retention Requirement is equal to ten percent of the project's Equivalent Impervious Surface Area, defined as:

Equivalent Impervious Surface Area (ft^2) = (Impervious Tributary Surface Area (ft^2) + (Pervious Tributary Surface Area (ft^2))

Impervious Tributary Surface Area is defined as the sum of all of the site's conventional impervious surfaces. When calculating Impervious Tributary Area:

- Do include: concrete, asphalt, conventional roofs, metal structures and similar surfaces
- Do not include: green roofs

Pervious Tributary Surface Area is defined as the sum of all of the site's pervious surfaces, corrected by a factor equal to the surface's runoff coefficient. When calculating Pervious Tributary Surface Area:

- Do include surfaces such as: unit pavers on sand; managed turf[7]; disturbed soils; and conventional landscaped areas (see Table 1 for correction factors).
  Example:
  Project Site includes 500 ft^2 of unit pavers on sand.
  Pervious Tributary Surface Area = 500 ft^2 x C = 50 ft^2
  Where C = Correction Factor for unit pavers, 0.1, from Table 1.

- Do not include: Infiltration SCM surfaces (e.g., SCMs designed to specific performance objectives for retention/infiltration) including, bioretention cells, bioswales; natural and

[7] Managed Turf includes turf areas intended to be mowed and maintained as turf within residential, commercial, industrial, and institutional settings.

undisturbed landscape areas, or landscape areas compliant with the Model Water Efficient Landscape Ordinance (California Code of Regulations, Title 23. Waters, Division 2. Department of Water Resources, Chapter 2.7.), or a local ordinance at least as effective as the Model Water Efficient Landscape Ordinance.

TABLE 1: Correction Factors[8] for Use in Calculating Equivalent Impervious Surface Area

Pervious Surface	Correction Factor
Disturbed Soils/Managed Turf (dependent on original Hydrologic Soil Group)	A: 0.15 B: 0.20 C: 0.22 D: 0.25
Pervious Concrete	0.60
Cobbles	0.60
Pervious Asphalt	0.55
Natural Stone (without grout)	0.25
Turf Block	0.15
Brick (without grout)	0.13
Unit Pavers on Sand	0.10
Crushed Aggregate	0.10
Grass	0.10

[8] Factors are based on runoff coefficients selected from different sources: Turf and Disturbed Soils from *Technical Memorandum: The Runoff Reduction Method.* Center for Watershed Protection & Chesapeake Stormwater Network. p.13, April 18, 2008.

http://town.plympton.ma.us/pdf/land/scheuler_runoff_reduction_method_techMemo.pdf. All other correction factors from *C.3 Stormwater Handbook, Santa Clara Valley Urban Runoff Pollution Prevention Program, Appendix F*, p. F-9., May 2004.

http://www.sanjoseca.gov/planning/stormwater/pdfs/appendices_files/Appendix_F_Final.pdf

APPENDIX G

Calculating Off-Site Retention Requirements When Less Than 10 Percent of the Project Site Equivalent Impervious Surface Area is Allocated to Retention-Based Structural Stormwater Control Measures

The following instructions demonstrate how to determine the Off-Site Retention Requirements when a Project subject to the Runoff Retention Performance Requirement, cannot allocate the full 10% of the project site's Equivalent Impervious Surface Area[9] to retention-based Stormwater Control Measures (SCMs).

STEP A. Potential Off-Site Mitigation Retention Volume

First calculate the Potential Off-Site Mitigation Retention Volume, which represents the additional volume of runoff that would have been retained on-site, had the full 10% of Equivalent Impervious Surface Area been dedicated to retention-based SCMs.

Equation A:

Potential Off-Site Mitigation Retention Volume = (the portion of the 10% Equivalent Impervious Area not allocated on-site) X (the On-Site Retention Feasibility Factor)

Where:

- *The portion of the 10% Equivalent Impervious Surface Area not allocated on-site* is that portion not allocated to on-site structural retention-based SCMs. For example, if 10% of Equivalent Impervious Surface Area is 1,000 ft^2 and only 8% (800 ft^2) is allocated to retention-based SCMs, the remaining 2% (200 ft^2) is the value inserted in the equation.

- *The On-Site Retention Feasibility Factor* is the ratio of Design Retention Volume[10] managed on-site (ft^3), to actual area (ft^2) allocated to structural SCMs. This establishes the site's retained volume**:**area ratio, expressed as cubic feet of retained runoff volume per square foot of area. For example, if a project is able to infiltrate 3,500 ft^3 of runoff over an 800-ft^2 area, this ratio of 3,500**:**800, or 4.38, is the On-Site Retention Feasibility Factor.

STEP B. Actual Off-Site Mitigation Retention Volume

Next, determine the Actual Off-Site Mitigation Retention Volume, which may be less than the Potential Off-Site Mitigation Retention Volume. The Actual Off-Site Mitigation Retention Volume is the lesser of the volume calculated in Equation A, and the remaining portion of the Design Retention Volume,

[9] Calculate Equivalent Impervious Surface Area using guidance in Post-Construction Requirements Attachment F

[10] Calculate Design Retention Volume using guidance in Post-Construction Requirements Attachment E, or equivalent method. Final Design Retention Volumes should reflect the applicant's demonstrated effort to use non-structural design measures to reduce the amount of runoff (e.g., reduction of impervious surfaces) as required by the Post-Construction Requirements' LID Development Standards (Section B.4.d).

calculated per Appendix E, not controlled on-site. There are two possible outcomes when the Runoff Retention Performance Requirement is not met on-site and less than 10% of the site's Equivalent Impervious Surface Area is allocated to retention-based SCMs:

- Potential Off-Site Mitigation Retention Volume is the Actual Off-Site Mitigation Retention Volume
- Remaining Design Retention Volume represents Actual Off-Site Design Retention Mitigation Volume

APPENDIX H

Soil Infiltration Assessment

Introduction and Purpose

This document provides guidance for conducting a Soil Infiltration Assessment to support the use of shallow or deep infiltration based stormwater control measures (SCMs), such as low impact development. This guidance is intended to provide a universal starting point for assessment of the infiltration characteristics of each project site and provide useful data in a cost-effective manner. Consideration and discussion of the application of these guidelines among the jurisdiction, the design professional and the geotechnical engineer is encouraged. They should be modified using sound engineering and geologic judgment to accommodate the unique characteristics of each project as they relate to each unique site.

The guidelines walk the user through a step-wise process from an Initial Site Assessment to a level of soil/geotechnical methodology appropriate for the site. The concept is to obtain information to:

1. Assess the general potential within the site for infiltration based SCMs
2. Provide a preliminary methodology to obtain soil infiltration data while balancing the need for data with the cost of acquiring the data.
3. Provide an extended or more comprehensive soil/geotechnical methodology where the results from the preliminary methodology as well as other site considerations warrant a more thorough soil analysis to facilitate better SCM design.

Note: Throughout this document the term “boring” is used for the purpose of observing the soil profile. However, except as indicated otherwise, an “excavation” may be substituted for the same purpose. Similarly, the term “drill” is the term used as the means of creating the boring. Except as otherwise indicated, it is meant to be synonymous with “excavating” or “digging” of an excavation. The two methods are meant to be interchangeable.

THESE METHODS DO NOT ADDRESS HEALTH OR SAFETY ASPECTS ASSOCIATED WITH THEIR USE. HEALTH AND SAFETY OF PERSONEL CONDUCTING THE METHODOLOGIES AND OF PEDESTRIANS, PASSERS-BY, SITE OWNERS OR TENANTS, ETC. SHOULD BE CONSIDERED. IT IS THE RESPONSIBILITY OF THE USER TO COMPLY WITH ALL APPLICABLE HEALTH AND SAFETY LAWS, REGULATIONS, POLICIES AND PROCEDURES, AND TO ENSURE THAT THE METHODOLOGIES ARE USED SAFELY.

The methodologies are guidelines only for the means of assessing the infiltration rates. Aspects related to permits, disposal of soil cuttings and samples, backfill, compaction, site restoration, etc. are not addressed. It is incumbent on the user to follow all laws, regulations, policies and procedures in decommissioning the borings.

Step 1: Initial Site Assessment

Initial Site Assessment is encouraged early in the design of post-construction SCMs. Infiltration SCMs may be required to comply with State post-construction stormwater control requirements. Various characteristics of a site may limit or preclude the use of infiltration SCMs including soil and geotechnical constraints. Early in the project planning phase, the Project Applicant should identify all site characteristics that may influence (both positively and negatively), the ability of the site to infiltrate stormwater. The list below relates to soil and geotechnical feasibility only and the Project Applicant is encouraged review the full list of possible infeasibility constraints as provided by the municipality.

Initial Site Assessment related to infiltration potential should include, but is not limited to:

- Slope / topography of parcel
- Descending slopes nearby
- Protected Vegetation (endangered species, heritage oaks, etc.)
- Springs, seeps
- Bedrock outcrops
- Soil types from USDA Soil Charts, local geologic and geotechnical knowledge, etc.
- Area(s) available for infiltration
- Nearby wells
- Soil of groundwater contamination
- Other geotechnical constraints that may impact public safety or property

Step 2: Interpretation of Initial Site Assessment

If the Initial Site Assessment indicates that there is documentation of characteristics that entirely preclude the use of shallow or deep infiltration based SCMs, go to **Step 2A**. Examples of such characteristics might be unstable slopes throughout the site; high groundwater, shallow impervious bedrock throughout the site, etc. Note: poor soils do not necessarily preclude the use of infiltration based BMPs but may limit the amount of infiltration.

If the Initial Site Assessment indicates that site characteristics do not preclude the use of infiltration based SCMs, go to **Step 2B**.

Step 2A: Omit use of infiltration-based SCMs, Infiltration analysis complete.

When site conditions entirely preclude the use of infiltration-based SCMs, the Project Applicant will need to contact the municipal representative responsible for the project to determine any required documentation of the infiltration infeasibility and the adjusted post-construction requirements for the project.

Step 2B: Conduct Quick Infiltration Testing

If Initial Site Assessment indicates that use of shallow infiltration-based SCMs (e.g. vegetated swales, bioswales, bioretention facilities, shallow infiltration basins, etc.) may be feasible, a "Shallow Quick Infiltration Test" may provide information to refine shallow SCM siting within the project and associated sizing calculations. See **Attachment 1** for Shallow Quick Infiltration Test methodology.

If Initial Site Assessment indicates that use of deep SCMs (e.g. seepage pits, deep infiltration basins, etc.) may be feasible, a "Deep Quick Infiltration Test" may provide information to refine deep SCM siting within the project and associated sizing calculations. See **Attachment 2** for Deep Quick Infiltration Test methodology.

Step 2C: Interpretation of Quick Infiltration Test Results

If results of the "quick" test (shallow or deep) are 5 inches/hour or slower (moderate to poor soils), then no further data are needed and soil infiltration assessment is complete. Design of SCMs should be based upon the data acquired, as modified by appropriate factors (i.e. factors for size and scale of the SCM, anticipated maintenance, initial and final silt loading, etc.)

Similarly, if results of the Quick Infiltration Testing (shallow or deep) indicate good soils (infiltration rates faster than 5 inches/hour), AND no further data are considered to be necessary for the SCM design, soil infiltration assessment is complete. Design of SCMs should be based upon the data acquired, as modified by appropriate factors (i.e. factors for size and scale of the SCM, anticipated maintenance, initial and final silt loading, etc.).

If results of the Quick Infiltration Testing (shallow or deep) indicate good soils (i.e. infiltration rates faster than 5 inches/hour), AND other considerations may necessitate more soil data, then "Extended Infiltration Testing" should be conducted. See **Attachment 3** for Extended Infiltration Testing methodology.

ATTACHMENT 1

Shallow Quick Infiltration Testing Methodology

1. For small sites with limited areas for infiltration-based SCMs, drill 1 profile boring and 2 infiltration test borings in each potential SCM area.

2. For acreage and unconstrained sites:

   - Up to 5 acres: drill 1 profile boring and 2 infiltration test borings per acre potentially usable for SCMs.
   - Over 5 acres: drill 1 profile boring and 2 infiltration test borings per geologic unit that may be usable for SCMs, with 2 to 4 infiltration test borings associated with each profile boring.

3. Profile borings should be 6" to 12" diameter. Where the planned SCMs will be constructed near the site's existing grade, borings should be 10' to 15' deep. If significant cuts will be necessary to install the SCMs, the borings should extend 5' to 10' below the invert of the planned SCM. The boring cuttings should be observed and the soils in the borings sampled as necessary to allow accurate logging. Where excavations are utilized to determine the profile, they should be no wider than necessary to facilitate logging of the strata with the same level of detail as for borings.

4. All soil strata should be identified on the logs as to USCS classification, consistency, presence of moisture or free water, color, impermeable and permeable zones, and any other characteristics that may be pertinent to infiltration potential. All logs should include the boring identification, date of drilling, auger type and diameter, sampling methods, and surface elevation (known or assumed).

5. Infiltration test borings should also be 6" to 12" diameter. They should be of depths such that the zone tested will range from about the elevation of SCM invert, to about 2' below the elevation of the invert.

6. Infiltration test excavations should be dug by any means to approximately the elevation of the *top* of the planned SCM. From the elevation of the top of the planned SCM to 2' below the elevation of the *invert* of the SCM, a hand auger or hand shovel should be used to excavate the actual test zone. Preferably, the test zone should be 6" to 12" in diameter; if conditions mandate a larger diameter, it should be as close to 12" as is practicable.

7. A perforated pipe, of a diameter that will facilitate the taking of the test measurements should be placed in each test boring or in the test zone of each test excavation.

8. The annulus between each perforated pipe and the boring sidewall should be filled with fine gravel.

9. A suitable elevation datum should be established from which each measurement can be taken. The elevation of the datum relative to the elevation of the top of the SCM should be noted.

10. Using a hose equipped with a water meter, a graduated water tank, or other suitable means of measuring water volume, add water to the approximate elevation of *top* of the planned SCM and maintain the head for 30 minutes.

11. At the end of the 30-minute period, shut off water and record volume of water that entered the test boring.

12. As the water level falls, measure from the datum to the water level at suitable intervals. Measurements should be to the degree of precision practicable (usually 1/8-inch or 0.01 foot) for a period of 2 hours. Depending upon the rate of fall, intervals between measurements may need to be from 1 minute to 30 minutes. Intervals should be as uniform as is practicable, however, as the water level falls and the head is reduced, the infiltration rate may decrease and the measurement intervals may need to be incrementally lengthened.

13. If a test boring runs dry within 2-hour measurement period, refill the boring and continue measuring the falling head to end of original 2-hour period. If it runs dry again, refill and continue measurements to the end of the original 2-hour period. If it runs dry a third time, do not refill, the testing of that boring is complete.

14. If the fall recorded in any test boring is less than 6” in 2 hours, continue taking measurements for an additional 2 hours (4 hours total).

15. See **Attachment 4** for a discussion of how to report the test results.

ATTACHEMENT 2

Deep Quick Infiltration Testing Methodology

1. For small sites with limited areas for infiltration-based SCMs, drill 2 profile / test borings in each potential deep SCM area.

2. For acreage and unconstrained sites:
   - Up to 5 acres: drill 3 profile / test borings per acre potentially usable for SCMs.
   - Over 5 acres: drill 4 profile / test borings per geologic unit that may be usable for SCMs.

3. Profile / test borings should be 6" to 12" diameter. The borings should extend 5' to 10' below the bottom of the planned SCM. The boring cuttings should be observed and the soils in the borings sampled as necessary to allow accurate logging. Use of excavations for deep testing is probably not practical.

4. All soil strata should be identified on the logs as to USCS classification, consistency, presence of moisture or free water, color, permeable and impermeable zones, and any other characteristics that may be pertinent to infiltration potential. All logs should include the boring identification, date of drilling, auger type and diameter, sampling methods, and surface elevation (known or assumed).

5. A perforated pipe, of a diameter that will facilitate the taking of test measurements should be placed in each profile / test boring.

6. The annulus between each perforated pipe and the boring sidewall should be filled with fine gravel.

7. A suitable elevation datum should be established from which each measurement can be taken. The elevation of the datum relative to the elevation of the top of the SCM should be noted.

8. Using a garden hose equipped with a water meter, a graduated water tank, or other suitable means of measuring water volume, add water to approximate elevation of *top* of the planned SCM and maintain the head for 30 minutes.

9. At the end of the 30-minute period, shut off water and record volume of water that entered the test boring.

10. As the water level falls, measure from the datum to the water level at suitable intervals. Measurements should be to the degree of precision practicable

(usually 1/8-inch or 0.01 foot) for a period of 2 hours. Depending upon the rate of fall, intervals between measurements may need to be from 1 minute to 30 minutes. Intervals should be as uniform as is practicable, however, as the water level falls and the head is reduced, the infiltration rate may decrease and the reading intervals may need to be incrementally lengthened.

11. If a test boring runs dry within the 2-hour measurement period, refill the boring and continue measuring the falling head to end of original 2-hour period. If it runs dry again, refill and continue measurements to the end of the original 2-hour period. If it runs dry a third time, do not refill, the testing of that boring is complete.

12. If the fall recorded in any test boring is less than 6” in 2 hours, discontinue testing as deep infiltration is not practical.

13. See **Attachment 4** for a discussion of how to report the test results.

ATTACHMENT 3

Extended Test Methodology

The following “extended” methodology is intended to provide more comprehensive soil/geotechnical information where the results from the Initial Site Assessment and/or Quick methodology, as well as other site and design considerations warrant a more thorough soil analysis to facilitate better SCM design.

1. Extended test methodology for *deep* SCMs is too complex an issue to be adequately addressed in these guidelines. Test locations, depths, methods, etc. should be discussed among the jurisdiction, the design professional and the geotechnical engineer and a consensus reached as to the appropriate means of securing the data required for design of the deep SCMs on the specific site.

2. For *shallow* extended testing, locations, depths, continuity of subsurface conditions, etc. should be discussed among the jurisdiction, the design professional and the geotechnical engineer. Consideration should be given to drilling and testing at least twice as many test borings as recommended under Quick Testing.

3. Extended shallow test methodology should be essentially the same as Steps 3 through 14 under Quick Testing, except for the following:
    a. Consideration should be given to presoaking the test borings for up to 24 hours prior to commencing testing.
    b. Measurements for extended testing should continue for 4 hours or more, regardless of infiltration rates.
    c. The 30-minute constant head period may be excluded if adequate constant head data were obtained during Quick Testing.

4. See **Attachment 4** for a discussion of how to report the test results.

ATTACHMENT 4

Reporting of Test Results

1. Reporting of test results, whether quick or extended, shallow or deep, should contain essentially the same information.

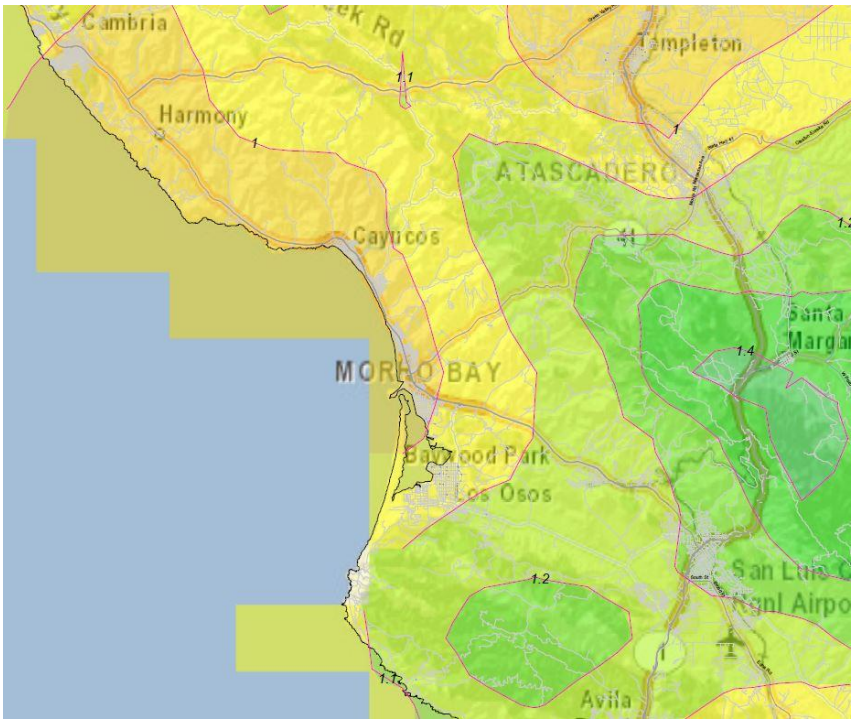
2. For each test boring, tabulate the test data showing:
   a. Test identification
   b. Date drilled
   c. Date tested
   d. Test boring diameter
   e. Perforated pipe diameter
   f. Test boring depth
   g. Stratum present in the test zone
   h. Elevation of top of SCM (known or assumed)
   i. Elevation of invert of SCM (known or assumed)
   j. Test duration
   k. Volume introduced between commencement of filling and the end of the 30-minute constant head period, typically in units of cubic feet
   l. Head during initial 30-minute period
   m. Time of the first falling head measurement and depth to the water surface
   n. Time of each subsequent measurement and depth to the water surface
   o. Intervals between measurements
   p. Incremental drop between measurements
   q. Infiltration rate between measurements, typically in units of inches per hour

3. Provide a map showing the approximate locations of all profile and test borings, as well as property lines, landmarks, planned improvements and SCM locations (if known), and other pertinent features that will help the user better understand the boring and testing program.

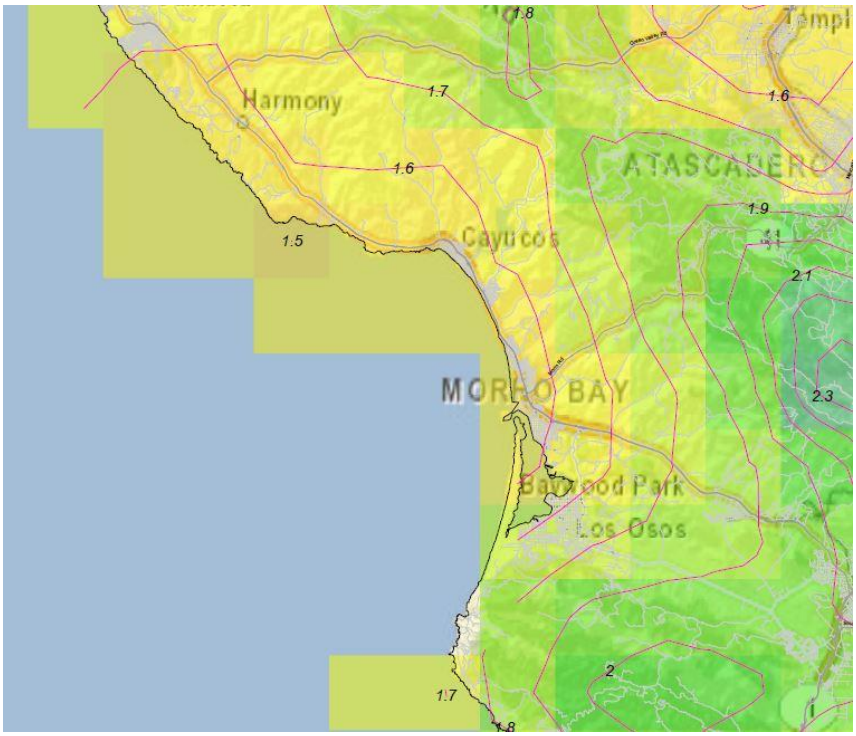
4. Provide log of each profile boring

5. Provide report summarizing data and discussing the potential for use of infiltration based SCMs on the site or area(s) tested.

APPENDIX I
85th and 95th Percentile 24-hr Rainfall Depth Maps



85th Percentile



95th Percentile

APPENDIX J
Operations, Maintenance and Reporting

POST CONSTRUCTION STORMWATER MANAGEMENT SYSTEM OPERATIONS AND MAINTENANCE FORMS INSTRUCTIONS

INSTRUCTIONS FOR RECORDING CONSTRUCTIVE NOTIFICATION

After determining a development is required to complete a Stormwater Control Plan (SWCP) with post construction Best Management Practices (BMPs) the applicant (land owner/developer) shall record a Constructive Notification for Private Stormwater Management System Operations and Maintenance with the San Luis Obispo County Clerk-Recorder's office. The constructive notification shall be recorded prior to occupancy.

The standard constructive notification and attachments can be obtained from the following locations:

- http://www.morro-bay.ca.us/stormwater
- Public Works Department 595 Harbor Ave., Morro Bay Ca 93442

The applicant shall fill out the constructive notification (per the instructions below) and submit to the City for review and approval. Following City approval, the applicant shall have the constructive notification notarized then filed with the County Clerk-Recorder. The property owner shall then complete annual self-inspections and submit to the Planning and Building Department.

OVERVIEW OF PROCEDURE

I. Complete Constructive Notification and Exhibits

The applicant shall fill out the three (3) part constructive notification prior to submitting for review and approval. (See DETAILED INSTRUCTIONS below for filling out the Constructive Notification, EXHIBIT A and EXHIBIT B forms.)

II. Submit the Constructive Notification & Exhibits to the Engineering Department

Upon completion of the draft constructive notification, the applicant shall submit a copy to the City for review and approval. The constructive notification shall be submitted to the following address (or submitted via email to the City contact person):

City of Morro Bay, Public Works Department
595 Harbor Avenue
Morro Bay, CA 93465

III. Revise and Resubmit (if applicable)

The applicant shall make any necessary modifications to the constructive notification based on the City's review. The revised constructive notification shall then be resubmitted to the City.

IV. Notarize Constructive Notification

Following City approval (including City Representative signature on EXHIBIT A), the constructive notification shall be notarized.

The applicant shall retain a notary public to notarize the constructive notification. The applicant shall sign the constructive notification and the notary shall complete and sign the constructive notification.

V. Record Constructive Notification

Following notarizing, the constructive notification (and Exhibits) shall be recorded with the County Clerk Recorders office located at:

County of San Luis Obispo Clerk-Recorder's Office
1055 Monterey Street Room D120
San Luis Obispo, CA 93408-3237

The Clerk-Recorder will keep the document for processing and mail the original back to the Public Services Department. The applicant may purchase a copy of the constructive notification.

For additional information on recording documents and associated fees, visit the County Clerk- Recorder's website at http://www.slocounty.ca.gov/clerk.htm.

VI. Inspections

Annually, the current property owner (or representative) shall complete a self-inspection of the Project Stormwater Management System. EXHIBIT B of the recorded constructive notification shall be completed and submitted annually by June 15th to:

City of Morro Bay
Public Works Department
Annual Reporting Requirements
955 Shasta Ave,
Morro Bay, CA 93442

-or-

pnewman@morrobayca.gov
Subject: Annual Reporting Requirements

For questions please contact the Engineering Department at (805) 772-6215 or the Public Services Department at (805) 772-6261.

DETAILED INSTRUCTIONS

CONSTRUCTIVE NOTIFICATION

The following information shall be completed:

- Property Address
- Property APN
- Permit/Project #
- Property Legal Description NOTE: The legal description is available in the property owner's title report.

See section IV above regarding Notarize Constructive Notification

EXHIBIT A - POST CONSTRUCTION STORMWATER MANAGEMENT SYSTEM OPERATIONS AND MAINTENANCE PLAN

PART 1A - GENERAL INFORMATION

1 **Property APN(s):** If the project has a shared Stormwater Management System (i.e. HOA), insert all the Property APNs served by the SYSTEM. Highlight the Property APN which contains the shared Structural Stormwater Control Measures (SCMs), such as a basin.

2 **Project Address(es):** where the Structural Stormwater Control Measures (SCMs) to be maintained are located.

3-6 Self-explanatory

7-8 **Designer** and **Company/Firm:** Insert name of the original designer of the stormwater management system. In the case that the Designer is not longer available or practicing, the Company/Firm to which the designer worked will be considered responsible to supply information regarding the SYSTEM.

9-11 Self-explanatory

12 **Estimated Annual Cost for Maintenance Once Established (Attach Cost Estimate Spreadsheet):** Designer to provide an estimate of annual cost to owner for services to inspect, maintain, and report on SYSTEM per instructions provide in this Exhibit. (Consider line item for inflation.)

13 **Other Pertinent Info:** For example, is the SYSTEM shared? Specifics of how will it be managed.

PART 1B - STRUCTURAL CONTROL MEASURE (SCM) DETAILS

General: Data provided on these sheets should match information provided in the Stormwater Control Plan (SWCP) or other plans approved by the City of Morro Bay for the permitted project.

A Structural Control Measure (SMC) is defined by the RWQCB as: *Any structural facility designed and constructed to mitigate the adverse impacts of stormwater and urban runoff pollution.*

Numbers for SCMs shall be assigned by the City of Morro Bay officials at approval of the project SWCP or other plan.

1 **Purpose(s) of SCM (check all that apply):** See City Morro Bay Stormwater Management Requirements.

2 Self explanatory

3 **Description & Location of SCM (As necessary, refer to PART 2 – Drawings & Photos):** The most effective means to describe the location of SCMs on a project site is in reference to a Drawing. (See Part 2 explanation.) If there is a basin serving multiple properties on its own lot, note which properties (by APN) the SCM serves.

4 **Drainage Design Criteria:** Per approved SWCP or Drainage Plan.

5 **Design Details (as applicable):** Per approved SWCP or Drainage Plan. If it does not apply, write N/A.

6 Self-explanatory

7 **SCM Inspection & Maintenance Requirements:** These requirements will be based on the design life and considerations of the SCMs ability to meet requirements for water quality and/or flow control as set out by RWQCB and the City Morro Bay.

Short Term Requirements include those things that will be required within a reporting year, i.e. clearing of debris, sediment or other obstructions to inlets.

Long-Term Requirements would be those things done on multiple-year schedule (2-year, 5 year, 10 year) to ensure the continual proper functioning of the SCM. For example, for a filter strip or biofiltration swale, include a plan ensuring the vegetation is healthy and method for replacement of plants (planting plan). For basins (infiltration or detention), provide a schedule for vegetation management and sediment removal. The replacement of inlet grates or other devices that could rust or degrade should be considered in this area, along with the design life.

PART 2 - DRAWINGS AND PHOTOS

The official documents related to the approved design of the Project's Stormwater Control Measure(s) (signed by the Engineer of Record) are required as a part of the Plan. If changes are made to the design of the SCM during construction, Record Drawings must be submitted. Include dated photos of the completed SCM with pertinent notes (i.e. direction from which the photo was taken.)

Reduced size Site and Drainage Plans and/or Details Sheets shall be provided. Any relevant details shall be copied at the original scale on 8.5x11 (for example, on an exhibit) for inclusion in the Plan. Ensure any exhibits include all the listed components.

PART 3 - CERTIFICATION AND APPROVAL

Along with the Owner and Designer, the designer or a 3rd party professional engineer, geologist, architect or landscape architect is required to field verify the Stormwater Control Measure(s) per RWQCB Resolution No. R3-2013-0032 Attachment 1, Section D. Field Verification of Post-Construction Stormwater Control Measures. Prior to the submittal of this Plan, it is recommended that the field verifier signatory and City official signatory visit the site together to inspect the SCMs, discuss the proposed plan and any potential issues prior to submittal.

EXHIBIT B - POST CONSTRUCTION STORMWATER MANAGEMENT SYSTEM OPERATIONS AND MAINTENANCE CHECKLIST TEMPLATE

General: The initial purpose of Exhibit B is to produce a template checklist which will be used for inspections and submitted to the City annually by June 15th. The approved template will be included in the recorded document, so consideration of checklist items that meet the short and long term maintenance requirements of the SCM is important. Since each SYSTEM design is different, it is the responsibility of the designer/engineer to advise the owner in completing the checklist, which must be approved by the City prior to recordation.

For this reason, the template can change in content to meet the particular SCM's maintenance needs. Two examples are included for a Biofiltration Area/Swale and Catch Basin(s).

Suggestions for Inspection Timing: Note that the official rainy season in California is October 15th – April 15th, so annual inspections of SYSTEMS would logically occur before October 15th to beat the possibility of a storm coming before any required maintenance is undertaken. Monthly inspections could be scheduled along with planned landscaping maintenance of the overall site, so the removal of vegetation debris or sediment could be done simultaneously. If the Project is a HOA-run development or similar, it will be beneficial to consider the project Operations & Maintenance schedule and add the SYSTEM maintenance therein.

Inspectors: It is required for a licensed Civil Engineer or Qualified SWPPP Practitioner (QSP) to sign off on the checklist annually. However, this does not prevent the signatory from delegating inspection responsibly to trained maintenance staff. All inspectors must be listed, and initialed to designate who did each inspection. However, the responsibility for certifying that the information provided is true & correct rests on the signatory.

Corrective Action/ Required Maintenance: Inspectors shall estimate how long it will take to rectify the situation in discussion with the owner, and re-inspect promptly. Any issues that are not addressed shall be recorded.

RECORDING REQUESTED BY:

WHEN RECORDED, PLEASE RETURN TO
County of San Luis Obispo Planning and Building Department
County Government Center, Room 208
San Luis Obispo, CA 93408
Ph: (805) 781-5600

NOTICE OF ADDITIONAL INFORMATION

Building Permit #_____ (*for office use–staff to provide*)

Property Address:

_____
(Street No. & Street Name, City, State, Zip)

Property APN: _____ **Permit/Project#:** _____

Property Legal Description:

_____

_____

Owner of the aforesaid property does here by give

CONSTRUCTIVE NOTIFICATION

For Private Stormwater Management System Operations and Maintenance

The Applicant (Individual, Married Person, a HOA, A for Profit, or non-Profit Corporation), herein after referred to as **"OWNER"** of the real property referenced above, hereby required by existing City codes and regulations to utilize "on-site stormwater management systems (i.e. structural and/or non-structural) to minimize runoff and pollutants in runoff and to provide permanent storm drainage to control, manage, retain, treat, infiltrate and dispose of" (1) "on-site storm drainage for the Project" and (2) "ancillary street and site drainage from the adjoining street and sites" as stipulated in the approved project plans and contained within the required Stormwater Management System Operations & Maintenance Plan.

The Owner is solely responsible for the **Private Stormwater Management System,** hereinafter referred to as "**SYSTEM**" and attached as Exhibit "A". The Owner agrees to the following conditions in compliance with all local, state, federal laws and regulations:

1. **MAINTENANCE**: OWNER shall maintain, monitor, inspect, clean and repair the SYSTEM as required in Exhibit "A" – Post Construction Stormwater Management System Operations & Maintenance Plan.
2. **DOCUMENT & REPORT:** OWNER shall document all maintenance, monitoring, inspections, cleanings and repairs made to the SYSTEM in the annual report **submitted to the City by June 15th of each year** in the form as approved by the City as detailed in Exhibit "B" – Post Construction Stormwater Management System Operations & Maintenance Checklist.

3. **CITY RIGHTS & AUTHORITY:** the City has the right and authority to inspect the SYSTEM to determine compliance with this constructive notification (i.e. maintenance, monitoring, inspections, cleanings, repairs,

documentation and reporting) which may result in enforcement activities and/or abatement if necessary pursuant to existing and future laws and regulations.

1. **FAILURE TO MAINTAIN, MONITOR, INSPECT, CLEAN, REPAIR AND REPORT SYSTEM:** Failure to maintain, monitor, inspect, clean, repair, or document and report as required herein shall constitute a public nuisance. The City may remedy such public nuisance through any of the applicable procedures as set forth in the City of Morro Bay Municipal Code, and/or may pursue any other legal or equitable remedies to abate such public nuisance.

5. **INDEMNIFICATION:** Owner further agrees to defend, indemnify, protect and hold the City and its agents, officers and employees harmless from and against any and all claims asserted or liability established for damages or injuries to any person or property, including to Owner's tenants, guests, invitees, agents or employees, which arise from or are connected with or caused or claimed by the acts or omissions of Owner, and its agents, employees or contractors, in performing the obligations specified herein, and all expenses of investigating and defending against same; provided, however, that Owner's duty to indemnify and hold harmless all not include any claims or liability arising from the established sole negligence or willful misconduct of the City, its agents, officers or employees.

6. **BINDING ON FUTURE <u>OWNERS</u>:** This covenant shall run with the land and shall be binding upon the undersigned owners, their heirs, executors, administrators, assigns and successors in interest.

OWNER(S) OF RECORD:

_____ _____

(Owner's Signature) (Owner's Signature)

_____ _____

(Owner Print Name & Title) (Owner Print Name & Title)

State of California
County of San Luis Obispo

On_____,20 ,before me,_____
_____ and _____

_____, personally appeared before___ who proved to me on the basis of satisfactory evidence to be the person(s) whose name(s) is/are subscribed to the within instrument and acknowledged to me that he/she/they executed the same in his/her/their authorized capacity(ies), and that by his/her/their signatures(s) on the instrument the person(s) or the entity upon behalf of which the person(s) acted, executed the instrument.

I certify under PENALTY OF PERJURY under the laws of the State of California that the foregoing paragraph is true and correct.

WITNESS my hand and official seal.

_____(Seal), Notary Public

City of Morro Bay

For Private Stormwater Management System Operations and Maintenance

EXHIBIT A – Post Construction Stormwater Management System Operations & Maintenance Plan

The 'Stormwater Management System Operations & Maintenance Plan' is to be filled out by landowner/designer and attached to the Constructive Notification for Private Stormwater Management System Operations and Maintenance upon recording. **NOTE: EXHIBIT B – 'Post Construction Stormwater Management System Operations & Maintenance Checklist' template is to be attached to the Constructive Notification. The approved and recorded template shall be filled out by the owner and submitted to the City by June 15th of each year.**

PART 1A – General Information

General Information		
1	**Property APN(s):**	
2	**Project Address(es):**	
3	**Owner:**	
4	**Address:**	
5	**Phone:**	
6	**Email:**	
Stormwater Management System Information		
7	**Designer:**	☐CE ☐QSP ☐QSD ☐ Other
8	**Company/Firm:**	
9	**Address:**	
10	**Phone:**	
11	**Email:**	
12	**Estimated Annual Cost for Maintenance Once Established*:**	
13	**Other Pertinent Info:**	

***Attach Cost Estimate Spreadsheet**

PART 1B: STRUCTURAL CONTROL MEASURE (SCM) DETAILS

SCM#: _____

1. Purpose(s) of SCM (check all that apply):	☐ Water Treatment	☐ Runoff Retention	☐ Peak Management
2. Type(s) of SCM Installed:	☐ Retention/Infiltration Basin, Trench, or Swale ☐ Subsurface Basin ☐ Detention Basin	☐ Biofiltration Swale ☐ Catch Basin ☐ Filter Strip(s)	☐ Water Quality Unit ☐ Proprietary Devices ☐ Other: _____
3. Description & Location of SCM (As necessary, refer to PART 2 – Drawings & Photos):	☐ Onsite Description:	☐ Offsite	

4. Drainage Design Criteria:	Design Storm Flow (cfs):			
	Design Storm Capacity (ft^3):			
5. Design Details (As applicable):	Length (ft):		Surface Area (ft^2):	
	Width (ft):		Capacity/Volume (ft^3):	
	Depth (ft):		Vegetation Height (in):	
	Slope (ft/ft):		Design Life (yrs):	
6. SCM Product Specifications (attach applicable specification sheets):	Product Name:			
	Manufacturer/Model Number:			
	Number Installed:			
	Product Life:			
7. SCM Inspection & Maintenance Requirements:	Date of installation:			
	Short Term Required Maintenance (describe or attach plan):			
	Long Term Required Maintenance (describe or attach plan):			

Include additional pages for multiple SCMs as necessary.

Page ___ of ___

PART 2 – Drawings & Photos

In addition to the location description, provide a copy of drawings showing each Structural Control Measure. Provide a plan view showing SCM location(s) relative to the parcel property lines. Include any details of the SCM and any additional sheets, reduced site plans or dated post construction photos to clearly define the limits of the SCM(s).

Ensure the drawings include the following:

- ☐ North Arrow
- ☐ Scale or Dimensions
- ☐ Length/Width
- ☐ Volume Depth
- ☐ Slopes
- ☐ Surface Area
- ☐ Cross Section(s)
- ☐ Unique Number for Each Structural Control Measure (SCM) as assigned by the City

PART 3 – Certification and Approval

I certify the information provided in EXHIBIT A – Post Construction Stormwater Management System Operations & Maintenance Plan and EXHIBIT B – Post Construction Stormwater Management System Operations & Maintenance Checklist to be true and correct.

Owner (Print Name)	Signature	Date
Designer (Print Name)	Signature	Date

The Post Construction Stormwater Management System has been reviewed, field verified and approved by (professional engineer, geologist, architect or landscape architect) :

Verifier (Print Name)	License No.	Signature	Date

The 'Post Construction Stormwater Management System Operations & Maintenance Plan' and 'Post Construction Stormwater Management System Operations & Maintenance Checklist' has been reviewed and approved by:

City Representative (Print Name, Title)	Signature	Date

EXAMPLE TEMPLATE

City of Morro Bay

For Private Stormwater Management System Operations and Maintenance

EXHIBIT B – Post Construction Stormwater Management System Operations & Maintenance Checklist

The following TEMPLATE shall be tailored to the Project SCMs and submitted with EXHIBIT A – Post Construction Stormwater Management System Operations & Maintenance Plan for approval. Remove all unnecessary text or instructions such as 'EXAMPLE' prior to submission. Contact the City for electronic forms.

Completed forms shall be <u>submitted annually by June 15th</u> to:

City of Morro Bay
Public Works Department
955 Shasta Ave
Morro Bay, CA 93442

-or-

pnewman@morrobayca.gov
Subject: Annual Reporting Requirements

General Information			
Property APN(s):			
Project Address(es):			
Owner:			
Address:			
Phone:		**Email:**	
Report Year:			
Inspector(s) (First & Last Name, Initials):			
Date of Inspection(s):			
SCM Number(s) Inspected:			

I certify the provided information to be true and correct and that the Structural Stormwater Control Measures (SCMs) on my property have been maintained, monitored, inspected, cleaned and repaired as required in EXHIBIT A – Post Construction Stormwater Management System Operations & Maintenance Plan.

Owner	**Inspecting Party Representative** ☐**Licensed Civil Engineer or** ☐**QSP No.**_____
Printed Name	Printed Name
Signature	Signature
Date	Date

During this reporting period (check all that apply):

☐ Completed inspections as required in EXHIBIT A – Post Construction Stormwater Management System Operations & Maintenance Plan
☐ Completed required short and long term maintenance as required in EXHIBIT A
☐ Completed corrective action(s) per the inspection (if applicable)
☐ Updated the EXHIBIT A - Post Construction Stormwater Management System O&M Plan to reflect revised site conditions. (Attach any updates.)
☐ No spills or system upsets occurred on site.
☐ Cleaned all spills promptly and reported the spill as required. *

*For all site spills, list spill date, content, volume and resolution:

Date	Content	Volume	Resolution	Inspector's Initials

Property APN: _____ SCM # _____ Page ___ of ___

EXAMPLE 1: Biofiltration Areas /Swales

Inspection and Maintenance Checklist

Inspected by (Print Name, Initials): *If multiple, list all.*		Report Year:	
Areas Inspected: *(see SCM location map in Post Construction Stormwater Management System Operations & Maintenance Plan)*		*If corrective action is required AND a re-inspection is warranted, indicate* Re-check date:	

Inspection frequency key: A = Annually on_____ ___(*Specify Date, i.e. October 15th*) of each year, M = Monthly, S = after major storm events

Inspection Items	Inspection Frequency	Date Inspected	Inspectors Initials	Maintenance Needed? (Yes/No)	Comments/Description
Is there standing water longer than 1 week after a storm event?	S				
Evidence of erosion?	S				
Vegetation appropriate and healthy?	A				
Area free of debris?	M				
Inlets free of obstructions?	M				
Is there obviously trapped sediment in need of removal (covers vegetation or greater than 3-inches at any spot)?	A				

Inspector comments: *(Use additional sheets or back of this sheet if more room is necessary, include Inspector's initials.)*

Overall condition of facility: ☐ Acceptable ☐ Unacceptable

Corrective Action Needed	**Due Date**

The next routine inspection is scheduled for approximately: _____

Property APN: _____ SCM # _____ Page ___ of ___

EXAMPLE 2: Catch Basin(s)

Inspection and Maintenance Checklist

Inspected by (Print Name, Initials): *If multiple, list all.*		Report Year:	
Areas Inspected: *(see SCM location map in Post Construction Stormwater Management System Operations & Maintenance Plan)*		*If corrective action is required AND a re-inspection is warranted, indicate* Re-check date:	

Inspection frequency key: A = Annually on _____(*Specify Date, i.e. October 15th*) of each year, M = Monthly, S = after major storm events

Inspection Items	Inspection Frequency	Date Inspected	Inspector's Initials	Maintenance Needed? (Yes/No)	Comments/Description
Inlets free of obstructions?	A				
Basins free of obstructions, debris (vegetation)?	A				
Drainage area & slopes leading to catch basin free of sediment & debris?	A				
Is there obviously trapped sediment in need of removal (greater than 3 inches)?	A				

Inspector comments: *(Use additional sheets or back of this sheet if more room is necessary, include Inspector's initials.)*

Overall condition of facility: ☐ Acceptable ☐ Unacceptable

Corrective Action Needed	**Due Date**

The next routine inspection is scheduled for approximately: _____

APPENDIX K

STORMWATER CONTROL PLAN CHECKLIST

A Stormwater Control Plan (SWCP) prepared by a Professional Engineer is required for all non-exempt projects. A preliminary SWCP is required for Planning Permit approval and a final SWCP shall be required prior to issuance of a Building Permit. The SWCP shall include the following (check all that apply or mark N/A):

Preliminary SWCP for Planning Permit application	COMPLETED
1. Project Information	
a. Project name	
b. Application number	
c. Address and assessor's parcel number	
d. Name of Applicant	
e. Name of Owner	
f. Project Phase number (if project is being constructed in phases)	
g. Project Type (e.g., commercial, industrial, multi-unit residential, mixed-use, public), and description	
2. Project Areas	
a. Total project site area	
b. Total new impervious surface area	
c. Total replaced impervious surface area	
d. Total new pervious area	
e. Calculation of Net Impervious Area	
3. Acknowledgement of the Performance Requirements that apply:	
a. PR No.1 – Site Design and Runoff Reduction	
b. PR No.2 – Water Quality Treatment	
c. PR No. 3 – Runoff Retention	
d. PR No. 4 – Peak Management	
4. Site Assessment Summary	
5. Summary of Site Design and Stormwater Control Measures selected for the project.	
6. Location and general configuration of all SCMs used shown on the plans	

Final SWCP for Building Permit application (all of above plus the following):	COMPLETED
1. List and describe all LID Measures Used	
a. List all site design measure incorporated into the design of the project	
b. Location and detail for all runoff reduction measures used	
c. Location and detail for all post-construction structural SCMs	
2. Summary Table of Runoff Reduction Measures and Structural Stormwater Control Measures, by Drainage Management Area and the entire site	
3. Supporting Calculations used to comply with the applicable on-site performance requirements	
4. Documentation demonstrating infeasibility where on-site compliance cannot be achieved	
5. Documentation demonstrating percentage of the project's Equivalent Impervious Surface Area dedicated to retention-based Stormwater Control Measures	
6. Documentation certifying that the selection, sizing, and design of the Stormwater Control Measures meet the applicable full or partial performance requirements.	
7. O&M Plan for all structural SCMs to ensure long-term performance	
8. Statement of Compliance: Statement that Performance Requirements has been met on-site, or, if not achievable:	
a. Documentation of the volume of runoff for which compliance cannot be achieved on-site and the associated off-site compliance requirements.	
b. Statement of intent to comply with Performance Requirements through Alternative Compliance	

STORMWATER MANAGEMENT GUIDANCE MANUAL FOR LOW IMPACT DEVELOPMENT & POST-CONSTRUCTION REQUIREMENTS

EZ MANUAL

March 6, 2014
(Updated: July 1, 2017)

This Guidance Manual is a supplement to the City of Morro Bay Department of Public Works Standard Drawings and Specifications

City of Morro Bay, California

APPROVED: _____ 23 August 2017

Rob Livick, PE/PLS, Director of Public Works/City Engineer — Date

Introduction

Post-Construction Stormwater Management Performance Requirements

The primary objective of these Post-Construction Stormwater Management Performance Requirements is to minimize the downstream impact of increased stormwater runoff that often occurs as the result of development or redevelopment projects. The Post-Construction Requirements emphasize protecting and, where degraded, restoring key watershed processes to create and sustain healthy watersheds. Maintenance and restoration of watershed processes is necessary to protect water quality and beneficial uses.

The intention of this Guidance Manual is to provide developers a tool to both determine the specific requirements for a given project and to plan and design the project so that those requirements are met as efficiently as possible. The requirements in this Manual are based on Resolution R3-2013-0032 and Attachments adopted July 12, 2013 by the Central Coast Regional Water Quality Control Board. In the event of conflict, the requirements of Resolution R3-2013-0032 shall take precedence over those contained in this Guidance Manual. Attachment 1 of the Resolution, which contains the referenced requirements, can be viewed or downloaded at: http://www.waterboards.ca.gov/centralcoast/water_issues/programs/stormwater/docs/lid/hydromod_lid_docs/2013_0032_attach1_post_construction_requirements.pdf

This EZ Manual can be used for individual single-family residence projects (Net Impervious Area <15,000 SF) in Morro Bay; if your project doesn't fit into this category then the Main Manual shall be used.

Definitions Related to Post-Construction Requirements

Discretionary Approval – A project approval which requires the exercise of judgment or deliberation when the MS4 decides to approve or disapprove a particular activity, as distinguished from situations where the MS4 merely has to determine whether there has been conformity with applicable statutes, ordinances, or regulations.

Dispersion – The practice of routing stormwater runoff from impervious areas, such as rooftops, walkways, and patios, onto the surface of adjacent pervious areas. Stormwater runoff is dispersed via splash block, dispersion trench, or sheet flow and soaks into the ground as it moves slowly across the surface of the pervious area.

Evapotranspiration (ET) – The loss of water to the atmosphere by the combined processes of evaporation (from soil and plant surfaces) and transpiration (from plant tissues).

Gross Impervious Area – Impervious surfaces that are created or replaced by the project. Manufactured permeable surfaces (pervious paving, gapped paving stones, etc.) may be considered as a pervious surface and are considered on a case by case basis. Do not include the surface area of decks with gaps that allow runoff to drain to permeable surfaces below. Gross Impervious Area is used in the initial determination of performance requirements.

Impervious Surface – A hard, non-vegetated surface area that prevents or significantly limits the entry of water into the soil mantle, as would occur under natural conditions prior to development. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or storage areas, concrete or asphalt paving, oiled, macadam or other surfaces which similarly impede the natural infiltration of stormwater. Open, uncovered retention/detention facilities shall not be considered as impervious surfaces for purposes of determining whether the thresholds for application of Performance Requirements are exceeded. However, for modeling purposes, open, uncovered facilities that retain/detain water (e.g., retention ponds, pools) shall be considered impervious surfaces. There are three methods of calculating impervious surface area, depending on the context of the calculation. For more details, see ***Net Impervious Area, Gross Impervious Area, and Equivalent Impervious Area*** definitions.

Landscaped Areas – Areas of soil and vegetation not including any impervious surfaces of ancillary features such as impervious patios, BBQ areas, and pools.

Low Impact Development (LID) – A stormwater and land use management strategy that strives to mimic pre-disturbance hydrologic processes of infiltration, filtration, storage, evaporation, and transpiration by emphasizing conservation, use of on-site natural features, site planning, and distributed stormwater management practices that are integrated into a project design.

Ministerial Approval – A project approval which involves little or no personal judgment by the MS4 as to the wisdom or manner of carrying out the project and only involves the use of fixed standards or objective measurements.

Native Vegetation – Vegetation comprised of plant species indigenous to the Central Coast Region and which reasonably could have been expected to naturally occur on the site.

Net Impervious Area – The sum of new and replaced post-project impervious areas, minus any reduction in total imperviousness from the pre-project to post-project condition: *Net Impervious Area = (New and Replaced Impervious Area) – (Reduced Impervious Area Credit),* where *Reduced Impervious Area Credit* is the total pre-project to post-project reduction in impervious area, if any.

New Development – Land disturbing activities that include the construction or installation of buildings, roads, driveways and other impervious surfaces. Development projects with pre-existing impervious surfaces are not considered New Development.

Permeable or Pervious Surface – A surface that allows varying amounts of stormwater to infiltrate into the ground. Examples include pasture, native vegetation areas, landscape areas, and permeable pavements designed to infiltrate.

Pre-Project – Stormwater runoff conditions that exist onsite immediately before development activities occur. This definition is not intended to be interpreted as that period before any human-induced land activities occurred. This definition pertains to redevelopment as well as initial development.

Project Site – The area defined by the legal boundaries and any area with off-site improvement of a parcel or parcels of land within which the new development or redevelopment takes place and is subject to these Post-Construction Stormwater Management Requirements.

Rainwater Harvest – Capture and storage of rainwater or stormwater runoff for later use, such as irrigation (without runoff), domestic use (e.g. toilets), or storage for fire suppression.

Receiving Waters – Bodies of water, surface water systems or groundwater that receive surface water runoff through a point source, sheet flow or infiltration.

Redevelopment – On a site that has already been developed, construction or installation of a building or other structure subject to the Permittee's planning and building authority including: 1) the creation or addition of impervious surfaces; 2) the expansion of a building footprint or addition or replacement of a structure; or 3) structural development including construction, installation or expansion of a building or other structure. It does not include routine road maintenance, nor does it include emergency construction activities required to immediately protect public health and safety.

Replaced Impervious Surface – The removal of existing impervious surfaces down to bare soil, base course or foundation slab and replacement with new impervious surface. Replacement of impervious surfaces that are part of routine maintenance activities are not considered replaced impervious surfaces.

Routine Road Maintenance – includes pothole and square cut patching; overlaying existing asphalt or concrete pavement with asphalt or concrete without expanding the area of coverage; shoulder grading;

reshaping/regrading drainage systems; crack sealing; resurfacing with in-kind material without expanding the road prism or altering the original line and grade and/or hydraulic capacity of the road.

Self-Retaining Areas – (also called "zero discharge" areas), are designed to retain some amount of rainfall (by ponding and infiltration and/or evapotranspiration) without producing stormwater runoff. Self-Retaining Areas may include graded depressions with landscaping or pervious pavement.

Self-Treating Areas – are a portion of a Regulated Project in which infiltration, evapotranspiration and other natural processes remove pollutants from stormwater. The self-treating areas may include conserved natural open areas and areas of native landscaping. The self-treating area only treats the rain falling on itself and does not receive stormwater runoff from other areas.

Single-Family Residence – The building of one single new house or the addition and/or replacement of impervious surface associated with one single existing house, which is not part of a larger plan of development.

Stormwater Control Measures (SCM) – Stormwater management measures integrated into project designs that emphasize protection of watershed processes through replication of pre-development runoff patterns (rate, volume, duration). Physical control measures include, but are not limited to, bioretention/rain gardens, permeable pavements, roof downspout controls, dispersion, soil quality and depth, minimal excavation foundations, vegetated roofs, and water use. Design control measures include but are not limited to conserving and protecting the function of existing natural areas, maintaining or creating riparian buffers, using onsite natural drainage features, directing runoff from impervious surfaces toward pervious areas, and distributing physical control measures to maximize infiltration, filtration, storage, evaporation, and transpiration of stormwater before it becomes runoff.

Stormwater Control Plan – A plan, developed by the Regulated Project applicant, detailing how the project will achieve the applicable Post-Construction Stormwater Management Requirements (for both onsite and offsite systems). A preliminary Stormwater Control Plan is required for Planning Permits and the final version is required prior to issuance of a Building Permit. See Appendix B for required contents.

Tributary Area – The entire project area except for undisturbed areas of planted areas with native vegetation that do not receive runoff from other areas and impervious surface areas that discharge to infiltration areas that will not produce runoff or create nuisance ponding. The Drainage Management Areas are smaller Tributary Areas that cumulatively make up the Tributary Area of the entire site.

Does My Project Need to Meet Post-Construction Performance Requirements?

Projects subject to these Post-Construction Performance Requirements include all New Development or Redevelopment projects that create and/or replace ≥15,000 square feet of impervious surface (collectively over the entire project site). In general, the larger the impervious surface created or replaced, the more rigorous the requirements become. However, a single family residence (SFR) project has a higher threshold before advance requirements apply. Consequently, these two elements (gross impervious area and project type) need to be determined and quantified as a first step in the process.

1. **Gross Impervious Area**: Gross Impervious Area is the total of newly created and replaced impervious surfaces. Existing impervious surfaces that are within the project site but are not being replaced do not count in this calculation. Impervious surfaces are any hard, non-vegetated surface areas that prevent or significantly limit the entry of water into the soil. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or storage areas, concrete or asphalt paving, oiled, macadam or other surfaces which similarly impede the natural infiltration of stormwater. Manufactured permeable surfaces (pervious paving, gapped paving stones, etc.) may be considered as pervious surfaces and are considered on a case by case basis. Surface areas of decks with gaps that allow runoff to drain to permeable surfaces below are not considered imperious areas. For redevelopment projects, both new and replaced impervious surfaces are included. If the Gross Impervious Area is less than 15,000 square feet, the project is exempt from requirements.
2. **Type of Project**: A list of types of projects that are exempt for all stormwater requirements is detailed below. If not exempt:
    a. Is the project a new development or redevelopment project? Projects are classified as redevelopment if the project replaces or adds to existing impervious surfaces. Projects located on land with no existing impervious surfaces are considered new development.
    b. Does the project involve the construction or reconstruction of one detached single family residence (SFR)? If not, the applicant must use the Main Manual.

The Performance Requirement Determination Form in Appendix A is provided to document the results of the above assessment. It shall be completed and filed with the Planning permit application. If the project is exempt, no further documentation is required. If not exempt, a calculation of the Net Impervious Area is required.

PROJECTS EXEMPT FROM STORMWATER REQUIREMENTS

Project that are exempt from the Post-Construction Performance are as follows (check any box on the list and no further action is required):

- ☐ Road and Parking Lot maintenance:
  - o Road surface repair including slurry sealing, fog sealing, and pothole and square cut patching
  - o Overlaying existing asphalt or concrete pavement with asphalt or concrete without expanding the area of coverage
  - o Shoulder grading
  - o Cleaning, repairing, maintaining, reshaping, or re-grading drainage systems
  - o Crack sealing
  - o Resurfacing with in-kind material without expanding the road or parking lot
  - o Practices to maintain original line and grade, hydraulic capacity, and overall footprint of the road or parking lot
  - o Repair or reconstruction of the road because of slope failures, natural disasters, acts of God or other man-made disaster
- ☐ Sidewalk and bicycle path or lane projects, where no other impervious surfaces are created or replaced, built to direct stormwater runoff to adjacent vegetated areas
- ☐ Trails and pathways, where no other impervious surfaces are replaced or created, and built to direct stormwater runoff to adjacent vegetated areas
- ☐ Underground utility projects that replace the ground surface with in-kind material or materials with similar runoff characteristics
- ☐ Curb and gutter improvement or replacement projects that are not part of any additional creation or replacement of impervious surface area (e.g., sidewalks, roadway)
- ☐ Second-story additions that do not increase the building footprint
- ☐ Roof repair or replacement
- ☐ Raised (not built directly on the ground) decks, stairs, or walkways designed with spaces to allow for water drainage
- ☐ Photovoltaic systems installed on/over existing roof or other impervious surfaces, and panels located over pervious surfaces with well-maintained grass or vegetated groundcover, or panel arrays with a buffer strip at the most down gradient row of panels
- ☐ Temporary structures (in place for less than six months)
- ☐ Electrical and utility vaults, sewer and water lift stations, backflows and other utility devices
- ☐ Above-ground fuel storage tanks and fuel farms with spill containment system

Net Impervious Area Calculation

Net Impervious Area is the Gross Impervious Area minus any reduction in total imperviousness from the pre-project to post-project condition: Net Impervious Area = (Gross Impervious Area) - (Reduced Impervious Area Credit), where Reduced Impervious Area Credit is the total pre-project to post-project reduction in impervious area, if any. The result of this calculation is used to determine if a project is subject to the requirements described in this EZ Manual

Examples of Calculating **Net Impervious Area**

Example 1:

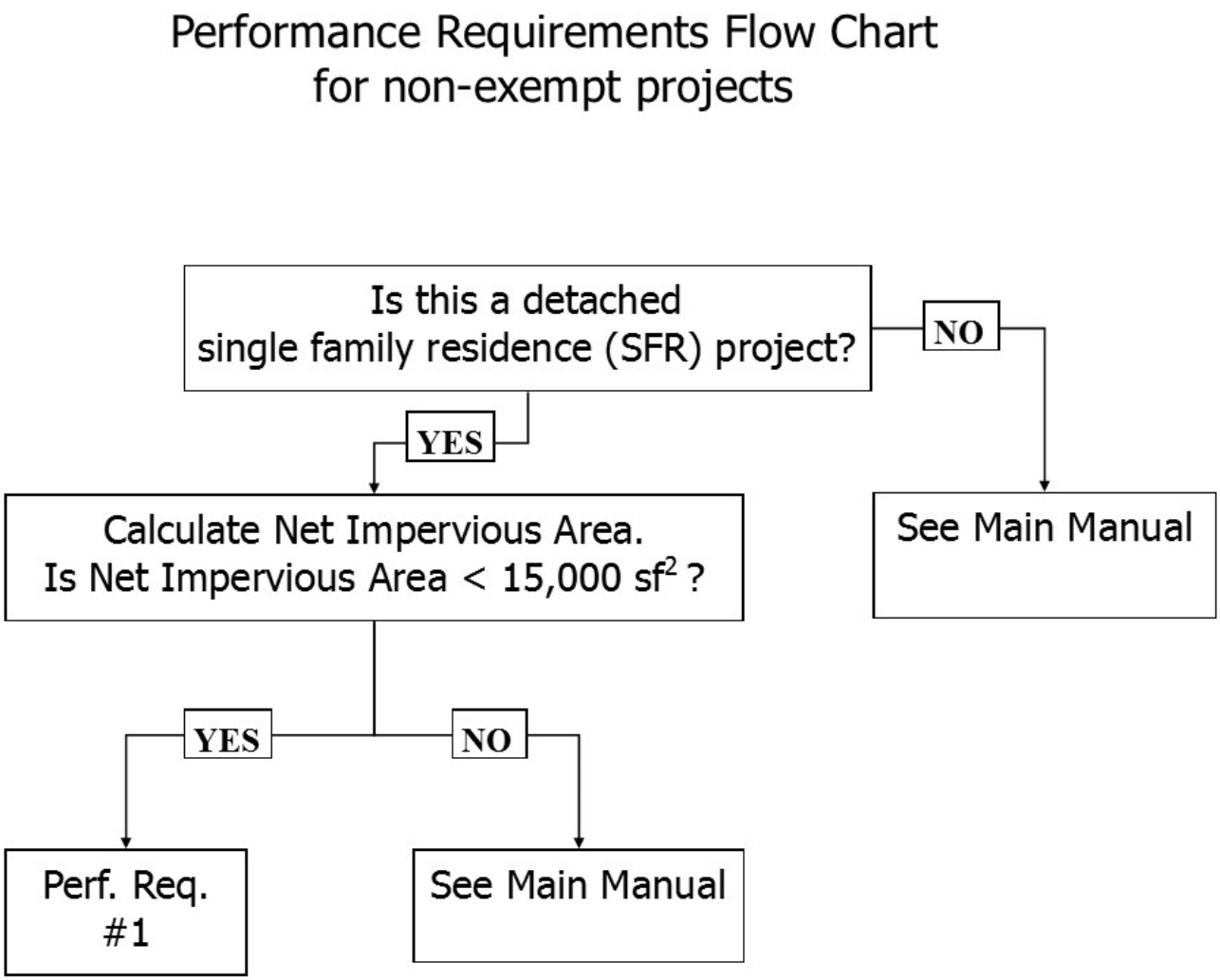
The project is a property that is an existing residence with 20,000 sf of impervious surface, including residence, garage, driveway, tennis court, etc. The new project will redevelop the site and have a total impervious area of 18,000 sf.
The **Reduced Imperious Area Credit** is 20,000 – 18,000 = 2,000 sf.
The **Net Impervious Area** is 18,000 – 2, 000 = 16,000 sf.
The **Net Impervious Area** is 16,000 sf which is greater than 15,000 sf .
The project is subject to requirements in the Main Manual.

Example 2:

The project is a property that is an existing residence with 20,000 sf of impervious surface, including residence, garage, driveway, tennis court, etc. The new project will redevelop the site, replacing the 4,000 SF paved tennis court with natural grass bocce courts and landscaping resulting in a total impervious area of 16,000 sf.
The **Reduced Imperious Area Credit** is 20,000 – 16,000 = 4,000 sf.
The **Net Impervious Area** is 16,000 – 4, 000 = 12,000 sf.
The **Net Impervious Area** is 12,000 sf which is less than 15,000 sf .
The project is subject to requirements in this Manual.

Once the Net Impervious Area Calculation is determined, use the following flow chart to determine the Post-Construction Performance Requirements for the project. Complete the Performance Requirement Determination Form (Appendix A) once the Flow Chart has been used to determine Performance Requirements.

Flow Chart


Performance Requirements Flow Chart
for non-exempt projects
Is this a detached
single family residence (SFR) project?
NO
YES
See Main Manual
Calculate Net Impervious Area.
Is Net Impervious Area < 15,000 sf² ?
YES
NO
Perf. Req.
#1
See Main Manual

Performance Requirement No. 1

Site Design and Runoff Reduction

Projects subject to Performance Requirement No. 1 (PR.1) are:

Projects that create and/or replace ≥ 2,500 square feet of impervious surface (collectively over the entire project site), including detached single-family homes.

The Project Engineer shall submit a stamped and signed copy of the Performance Requirement No.1 Certification, as included on the following page; certifying Low Impact Development design strategies are included in the project design. Each strategy that has been incorporated into the design should be initialed by the project engineer, or marked NA if not applicable.

PERFORMANCE REQUIREMENT NO. 1 CERTIFICATION		
	LOW IMPACT DEVELOPMENT (LID) DESIGN STRATEGY	INCORPORATED
1.	Limit disturbance of creeks and natural drainage features.	
2.	Minimize compaction of highly permeable soils.	
3.	Limit clearing and grading of native vegetation at the site to the minimum area needed to build the project, allow access, and provide fire protection.	
4.	Minimize impervious surfaces by concentrating improvements on the least sensitive areas of the site, while leaving the remaining land in a natural undisturbed state.	
5.	Minimize stormwater runoff by implementing one or more of the following design measures:	
	a) Direct roof runoff into cisterns or rain barrels for reuse.	
	b) Direct roof runoff onto vegetated areas safely away from building foundations and footings.	
	c) Direct runoff from sidewalks, walkways, and/or patios onto vegetated areas safely away from building foundations and footings.	
	d) Direct runoff from driveways and/or uncovered parking lots onto vegetated areas safely away from building foundations and footings.	
	e) Construct bike lanes, driveways, uncovered parking lots, sidewalks, walkways, and patios with permeable surfaces.	

I, _____, acting as the Project Engineer for _____ project, located at _____, hereby state that the Site Design and Runoff Reductions design strategies initialed above have been incorporated into the design of the project.

_____ _____

Signature Date

APPENDIX A

SFR PERFORMANCE REQUIREMENT DETERMINATION FORM

The following form shall be completed for all SFR development and redevelopment projects. Projects that are exempt from performance requirements are required to complete Section 1 & 2 only.

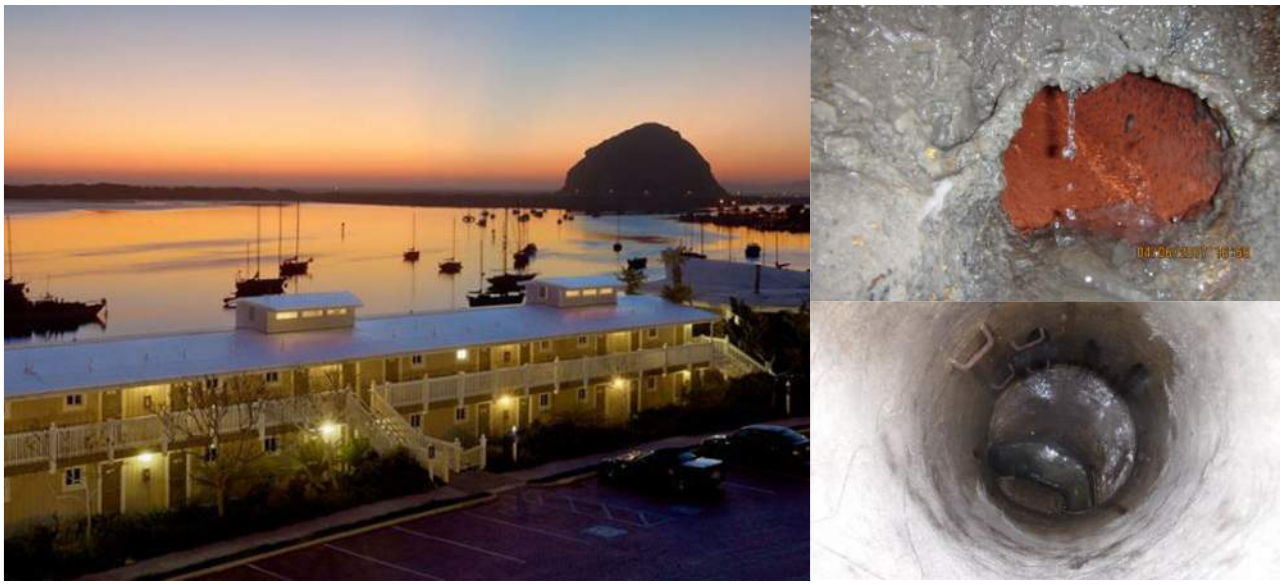
Section 1: General Information	
Project name	
Project Address	
Assessor's Parcel Number(s)	
Name of Applicant	
Applicant email address:	
Applicant phone:	
Project Type (e.g. single-family residential, commercial, etc.)	
Section 2: Area Information (ft^2)	
Total Project Area	
Total Existing impervious surface area	
Proposed Gross Impervious Area (list only the surface areas that are being created or replaced)	
a. Rooftops	
b. Driveways	
c. Patios	
d. Parking Lots	
e. Other	
Total Gross Impervious Area	
If Gross Impervious Area <2,500 ft^2, write "EXEMPT". Otherwise continue to Sec. 3	
Section 3: PR Determination	
Net Impervious Area (from page 7)	
Performance Requirements (from Flow Chart)	

Appendix F

V&A 2017 SEWER FLOW MONITORING AND INFLOW/INFILTRATION STUDY

City of Morro Bay

2017 Sewer Flow Monitoring and Inflow/Infiltration Study



Prepared for:

City Of Morro Bay
595 Harbor St.
Morro Bay, CA 93442

Date:

Draft: September 2017
Final: January 2018

Prepared by:

V&A consulting engineers

V&A Project No. 17-0050

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Abbreviations and Acronyms

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

Terms and Definitions

Term	Definition
Average dry weather flow (ADWF)	Average flow rate or pattern from days without noticeable inflow or infiltration response. ADWF usage patterns for weekdays and weekends differ and must be computed separately. ADWF 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 fullness of the pipe used in capacity analysis.
Design storm	A theoretical storm event of a given duration and intensity that aligns with historical frequency records of rainfall events. For example, a 10-year, 24-hour design storm is a storm event wherein the volume of rain that falls in a 24-hour period would historically occur once every 10 years. Design storm events are used to predict I/I response and are useful for modeling how a collection system will react to a given set of storm event scenarios.
Infiltration and inflow	Infiltration and inflow (I/I) rates are calculated by subtracting the ADWF flow curve from the instantaneous flow measurements taken during and after a storm event. Flow in excess of the baseline consists of inflow, rainfall-responsive infiltration, and rainfall-dependent infiltration. **Total I/I** is the total sum in gallons of additional flow attributable to a storm event.
Infiltration, *groundwater*	Groundwater infiltration (**GWI**) is groundwater that enters the collection system through pipe defects. GWI depends on the depth of the groundwater table above the pipelines as well as the percentage of the system that is submerged. The variation of groundwater levels and subsequent groundwater infiltration rates is seasonal by nature. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.
Infiltration, *rainfall-dependent*	Rainfall-dependent infiltration (**RDI**) is similar to groundwater infiltration but occurs as a result of storm water. The storm water percolates into the soil, submerges more of the pipe system, and enters through pipe defects. RDI is the slowest component of storm-related infiltration and inflow, beginning gradually and often lasting 24 hours or longer. The response time depends on the soil permeability and saturation levels.
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.
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 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.
Synthetic hydrograph	A set of algorithms has been developed to approximate the actual I/I hydrograph. The synthetic hydrograph is developed strictly using rainfall data and response parameters representing response time, recession coefficient and soil saturation.

Executive Summary

Scope and Purpose

V&A Consulting Engineers (V&A) has completed sanitary sewer flow monitoring within the City of Morro Bay, CA (City). Flow monitoring was performed over several periods from April 7, 2017 to July 13, 2017 at 15 open-channel flow monitoring sites. V&A was also provided data from four City-owned flow meters and the City's influent flume in order to provide more information about wet weather flows. The dates of the data provided was from November 7, 2016 through July 18, 2017 for the flume and January 3, 2017 through February 27, 2017 for the City-owned meters. There were four general purposes of this overall project.

1. Establish the baseline sanitary flows at the flow monitoring sites during two distinct periods with different flow regimes
    a. Non-holiday periods: from April 7, 2017 to July 11, 2017, excluding the Memorial Day weekend (May 27 – 29, 2017) and the week of the Fourth of July holiday (July 1 - 9, 2017).
    b. Holiday periods: during the two peak periods, May 27 – 29, 2017 and July 1 - 9, 2017, that see a heavy tourist presence and its influence on the sanitary flows.
2. Establish the peak flow condition during the rainfall events
3. Quantify inflow/infiltration (I/I) at the applicable flow monitoring sites.
4. Assess and calibrate the City's influent flume. The results of this work will be presented in a separate technical memorandum.

Rainfall Monitoring

There were a series of rainfall events that occurred from January 2017 through February 2017, of which five rainfall events were defined. Rainfall Event 1 from January 7, 2017 through January 12, 2017 was one of the largest rainfall events. This event was classified as a 12-year, 6-Hour rainfall event at one of the rain gauges used for analysis. This rainfall event, as well as several others, was of sufficient strength to elicit an I/I response from the installed City-owned flow monitoring sites that was suitable for I/I analysis.

There was not sufficient rainfall during the V&A flow monitoring period to elicit an analyzable I/I response. All I/I analysis within this study was based on data from the four City-owned flow meters and the City's influent flume.

Site Flow Monitoring and Capacity Results

Peak measured flows and the corresponding flow levels (depths) are important to understand the capacity limitations of a collection system. The peak flows and flow levels reported are from the peak measurements as taken across the entirety of the flow monitoring period. Peak flows and levels may not correspond to a rainfall event.

The following capacity analyses terms are defined as follows:

- **d/D Ratio:** The d/D ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). The d/D ratio for each site was computed based on the maximum depth of flow for the study.
- **Peaking Factor:** Peaking factor is defined as the peak measured flow divided by the ADWF. Peaking factors are influenced by many factors including size and topography of tributary area and the amount and characteristics of I/I entering the collection system. Flow attenuation and flow restrictions will also affect the peaking factor.

Table ES-1 summarizes the flow monitoring and capacity results for the flow monitoring sites. V&A flow monitoring sites are named "Site #" and the City's flow monitoring sites are named "MB #" in the report. Figure ES-1 and Figure ES-2 show the peak flow schematic for the V&A and City meters. Results of note have been shaded in RED. Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the site locations; hydraulic conditions in other areas of the collection system will differ.

Table ES-1. Capacity Analysis Summary

Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	*d/D* Ratio	Level Surcharged above Crown (ft)	
Site 1	0.18	0.48	2.6	11.75	5.6	0.48	--	April 7 – July 11, 2017
Site 2	0.03	0.08	3.1	8	2.5	0.32	--	
Site 3	0.29	0.89	3.0	15	7.4	0.49	--	
Site 4	0.22	0.58	2.6	17.5	8.4	0.48	--	
Site 5	0.09	0.28	3.1	8	2.5	0.32	--	
Site 6	0.11	0.35	3.1	13	4.1	0.32	--	
Site 7	0.04	0.16	4.1	9.75	3.5	0.36	--	
Site 8	0.07	0.21	3.1	13	7.3	0.56	--	
Site 9	0.02	0.10	4.8	12	3.8	0.32	--	
Site 10	0.07	0.32	4.7	8	4.6	0.57	--	
Site 11	0.06	0.25	3.9	8	3.6	0.46	--	
Site 12	0.13	0.38	3.0	14.375	7.7	0.54	--	
Site 13	0.09	0.31	3.3	15.5	7.0	0.45	--	
Site 14	0.09	0.32	3.5	10	2.5	0.25	--	
Site 15	0.05	0.21	4.6	10	3.5	0.35	--	
MB 1	0.17	0.93	5.6	12	14.0	1.17	0.17	Jan 3 – Feb 27, 2017
MB 2	0.03	0.17	5.9	6	2.5	0.41	--	
MB 3	0.30	2.54	8.6	15	23.6	1.57	0.70	
MB 4	0.22	1.02	4.6	18	6.1	0.34	--	
Morro Bay Flume	0.92	10.08	11.0	27	N/A	N/A	N/A	Nov 7, 2016 – Jul 17, 2017

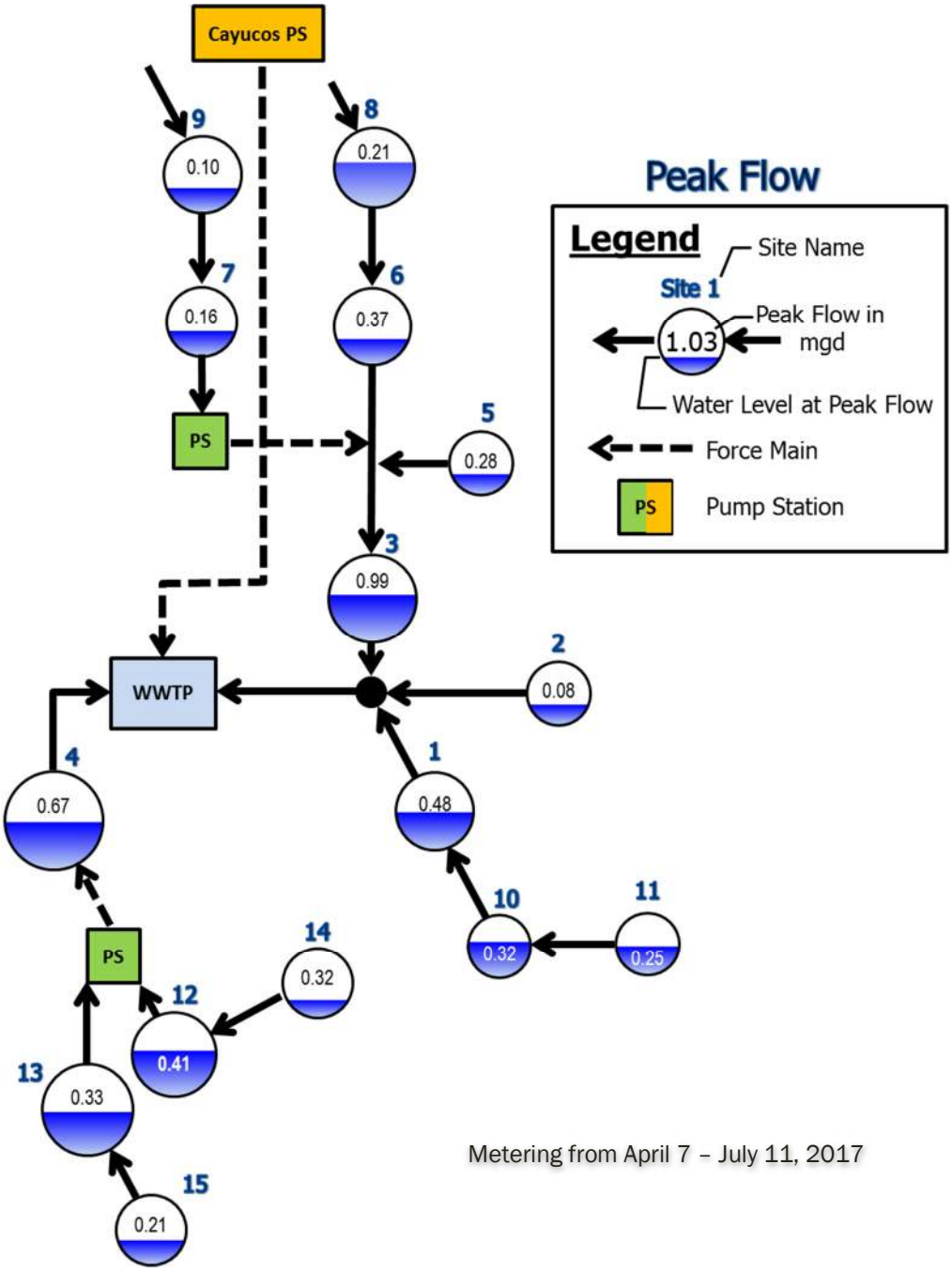

Figure ES-1. Peak Flow Schematic – V&A Sites

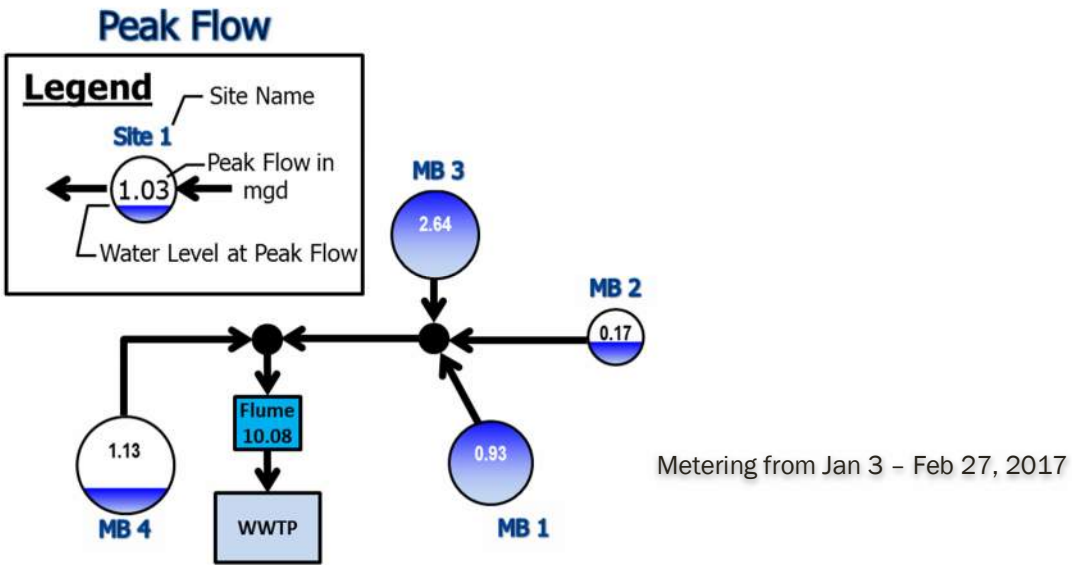

Figure ES-2. Peak Flow Schematic – Morro Bay Sites

The following capacity analysis results are noted:

- **Peaking Factor:** Only three sites, sites MB 1, MB 2 and MB 3, as well as the Morro Bay Influent Flume had peaking factors that exceeded 5. It should be noted that the City-owned sites were installed during the bulk of the rainfall season, while the V&A sites were installed afterwards.
- **d/D Ratio:** Only two of the nineteen flow monitoring sites, MB 1 and MB 3, had d/D ratios that were above the common threshold value of 0.75 d/D. These two sites reached a surcharge condition during the study, also during peak rainfall events.
- **Flume**: The Morro Bay 27-inch Palmer Bowlus Flume had a peak measured flow of 10.08 mgd. The manufacturer American Sigma's drawing states a 10.24 mgd maximum flow and 1.76 maximum head. As a result, the peak measured flow is at 98% capacity.

Infiltration and Inflow Analysis

The I/I results of this study are based on the data of the four City-owned flow meters that were installed during the bulk of the rainfall season. I/I analysis on the flume was not performed as there is no substantive basin that could be isolated. The flume's reaction to the rainfall events is included in Appendix A. Table ES-2 summarizes the I/I results for all basins in the study.

Results of note have been shaded in RED. Please refer to the *I/I Methods* section for more information on inflow and infiltration analysis methods.

Table ES-2. I/I Analysis Summary

Metering Basin	ADWF (mgd)	Peak I/I per ADWF [A]	RDI per ADWF [B]	Total I/I per ADWF (per in-Rain) [C]	Suggestion of GWI?
MB 1	0.17	4.07	75%	2.87	
MB 2	0.03	4.87	79%	2.90	
MB 3	0.30	7.27	87%	4.09	Yes
MB 4	0.22	3.12	10%	0.36	

[A] Event 1, [B] Event 4, [C] Events 1-5 (January 3 – February 27, 2017)

The following inflow/infiltration analysis results are noted:

- **Inflow:** Basin 3 had the highest normalized inflow contribution. Basins 1 and 2 also had elevated levels of inflow.
- **RDI:** Basin 3 had the highest normalized rain-dependent infiltration contribution. Basins 1 and 2 also had elevated levels of RDI.
- **GWI:** Of the Morro Bay owned meters, only MB 3 showed evidence of GWI contribution.

Recommendations

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

1. **Determine I/I Reduction Program:** The City should examine its I/I reduction needs to determine their needs and goals for a future I/I reduction program.
   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 higher inflow/ADWF ratios.
   b. If 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 higher RDI/ADWF ratios.
2. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
   a. Smoke testing
   b. Mini-basin flow monitoring
   c. Night-time reconnaissance work to (1) investigate and determine direct point sources of inflow, and (2) determine the areas and/or pipe reaches responsible for high levels of infiltration contribution.
   d. CCTV inspection
3. **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) has completed sanitary sewer flow monitoring within the City of Morro Bay, CA (City). Flow monitoring was performed over several periods from April 7, 2017 to July 13, 2017 at 15 open-channel flow monitoring sites. V&A was also provided data from 4 City-owned flow meters and the City's influent flume in order to provide more information about wet weather flows. The dates of the data provided were from November 7, 2016 through July 18, 2017 for the flume and January 3, 2017 through February 27, 2017 for the City-owned meters. There were four general purposes of this overall project.

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    b. Holiday period: during the two peak periods, May 27 – 29, 2017 and July 1 - 9, 2017, that see a heavy tourist presence and its influence on the sanitary flows.
2. Establish the peak flow condition during the rainfall events
3. Quantify inflow/infiltration (I/I) at the applicable flow monitoring sites.
4. A fourth goal of this project was to assess and calibrate the City's influent flume. The results of this work will be presented in a separate technical memorandum.

The flow monitoring sites are listed in Table 1-1 and shown in Figure 1-1. V&A flow monitoring sites are named "Site #" and the City's flow monitoring sites are named "MB #" in the report.

Flow monitoring basins are localized areas of a sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. The basin refers to the ground surface area near and enclosed by the pipelines. A basin may refer to the entire collection system upstream from a flow meter or may exclude separately monitored basins upstream. To isolate a particular drainage basin, an addition or subtraction of flows may be required[1]. For this study, there were no basins that required a subtraction in flows.

Figure 1-2 shows the basins used for the I/I analysis and Table 1-2 shows which V&A flow sites are contained within these basins.

[1] There is error inherent in flow monitoring. Adding and subtracting flows increases error on an additive basis. For example, if Site A has an error of ±10% and Site B has an error of ±10%, then the resulting flow when subtracting Site A from Site B would have an error of up to ±20%.

Table 1-1. List of Flow Monitoring Locations

Monitoring Site	Measured Pipe Diameter (in)	Location
Site 1	11.75	Intersection of Main Street and Errol Street, meter on south 11.75" inlet
Site 2	8	Atascadero Road ~85 feet east of Sunset Avenue, meter on west 8" outlet
Site 3	15	Island between Main Street and HWY 1 on-ramp, ~135 feet north of Atascadero Road, meter on north 15" inlet
Site 4	17.5	Embarcadero, ~685 feet north of Coleman Drive, meter on south 17.5" inlet
Site 5	8	371 Avalon Street, meter on east 8" inlet
Site 6	13	Main Street, ~290 feet south of Elena Street, meter on north 13" inlet
Site 7	9.75	Coral Avenue, ~630 feet north of Emerald Circle, meter on north 9.75" inlet
Site 8	13	Main Street, ~400 feet south of Island Street, meter on north 13" inlet
Site 9	12	125 Java Street (near intersection of Sandalwood Avenue), meter on west 12" inlet
Site 10	8	Main Street, just north of intersection with Radcliff Avenue, meter on south 8" inlet
Site 11	8	785 Quintana Road, meter on east 8" inlet
Site 12	14.375	Parking lot of 1154 Embarcadero, meter on south 14.375" inlet
Site 13	15.5	Sidewalk of 910 Embarcadero, meter on south 15.5" inlet
Site 14	10	Intersection of Main Street and Marina Street, meter on east 10" inlet
Site 15	10	301 Embarcadero, ~90 feet north of Morro Bay Launching Facility, meter on south 10" inlet
MB 1	12	Main Street, ~220' south of intersection with Errol Street, meter on south 11.75" inlet
MB 2	6	Atascadero Road ~85 feet east of Sunset Avenue, meter on east 6" inlet
MB 3	15	Island between Main Street and HWY 1 on-ramp, ~135 feet north of Atascadero Road, meter on north 15" inlet
MB 4	18	Embarcadero, ~290' south of entrance to Arroyo Dunes Mobile Home Park, meter on south 18" inlet
Flume	27	Located at Morro Bay Wastewater Treatment Plant

Table 1-2. List of Basins Used in I/I Analysis

I/I Basin	V&A Sites
Basin MB 1	Sites 1, 10, 11
Basin MB 2	Site 2
Basin MB 3	Sites 3, 5, 6, 7, 8, 9
Basin MB 4	Sites 4, 12, 13, 14, 15

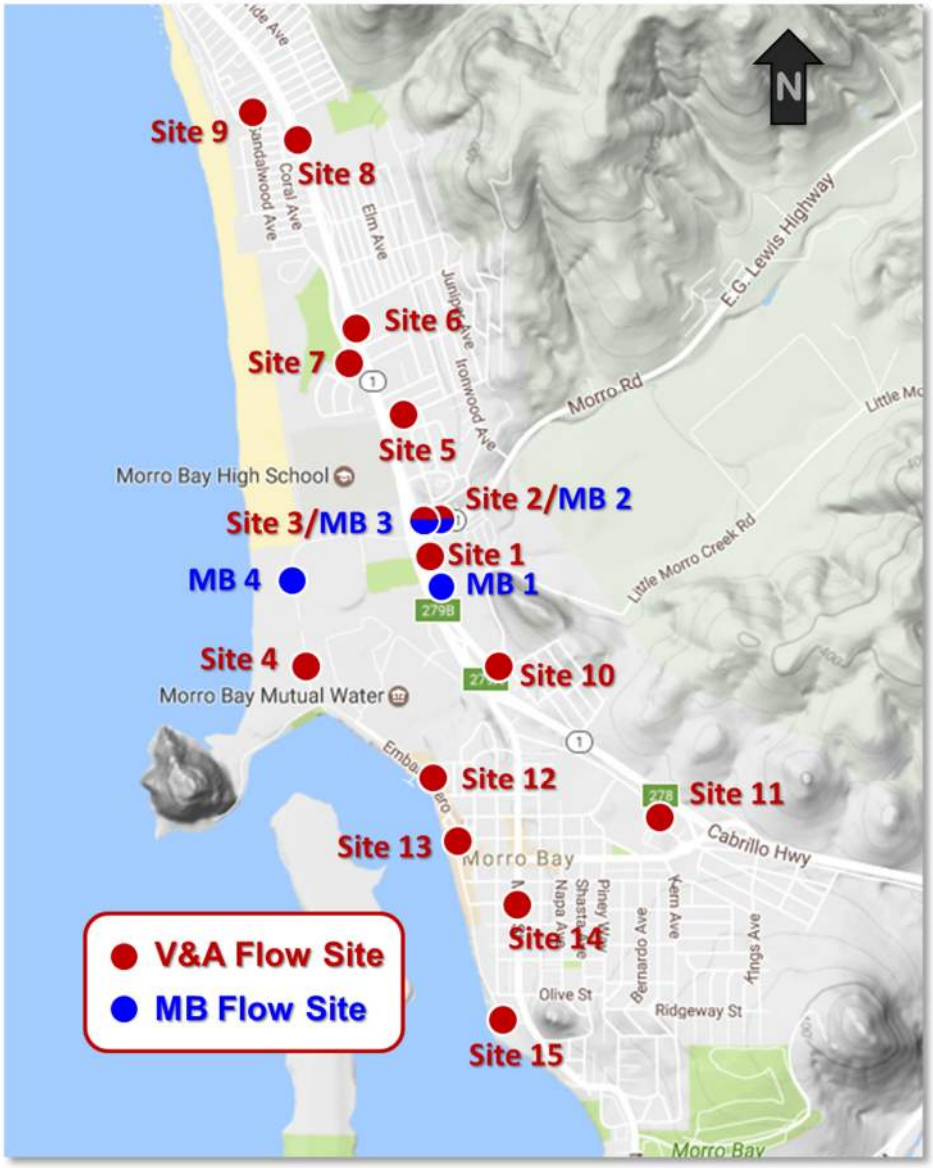


Figure 1-1. Map of Flow Monitoring Sites

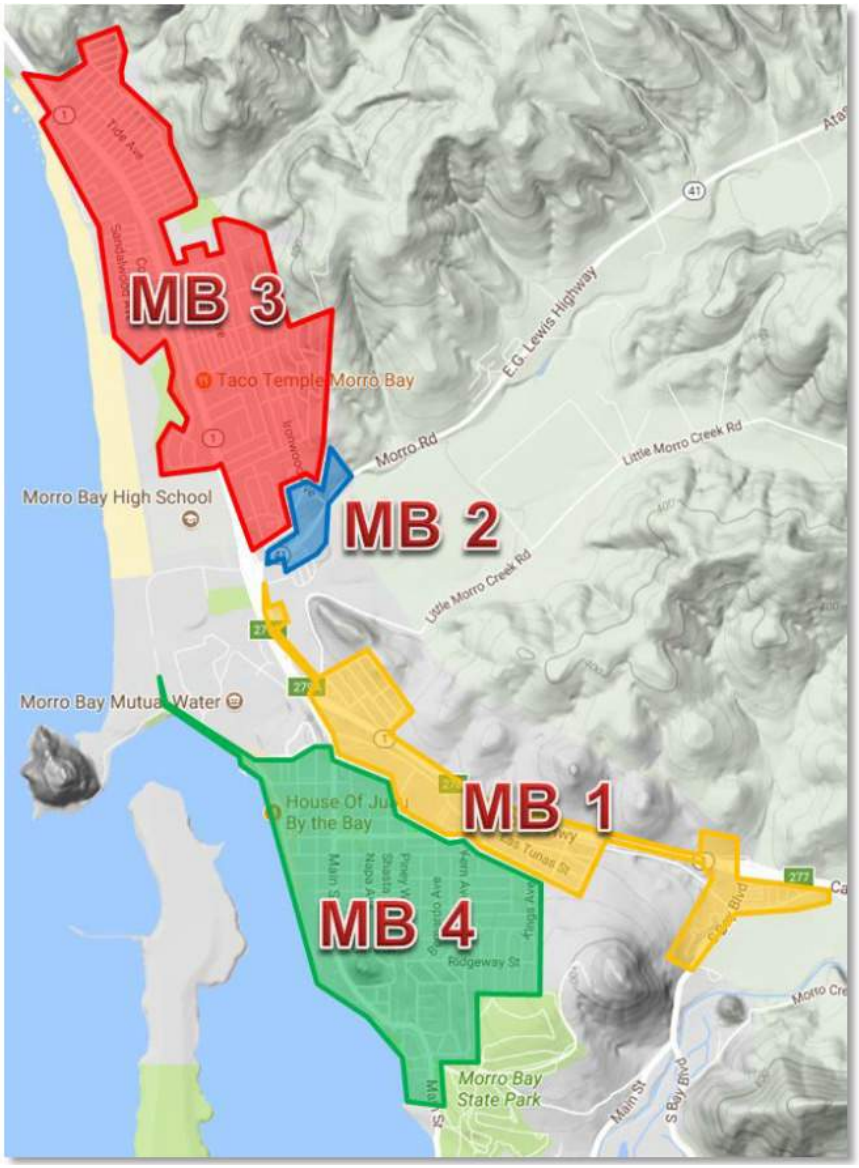


Figure 1-2. Map of Morro Bay Flow Monitoring Basins

2 Methods and Procedures

2.1 Confined Space Entry

A confined space (Photo 2-1) is defined as any space that is large enough and so configured that a person can bodily enter and perform assigned work, has limited or restricted means for entry or exit and is not designed for continuous employee occupancy. In general, the atmosphere must be constantly monitored for sufficient levels of oxygen (19.5% to 23.5%), and the presence of hydrogen sulfide (H_2S) gas, carbon monoxide (CO) gas, and lower explosive limit (LEL) levels. A typical confined space entry crew has members with OSHA-defined responsibilities of Entrant, Attendant and Supervisor. The Entrant is the individual performing the work. He or she is equipped with the necessary personal protective equipment needed to perform the job safely, including a personal four-gas monitor (Photo 2-2). If it is not possible to maintain line-of-sight with the Entrant, then more Entrants are required until line-of-sight can be maintained. The Attendant is responsible for maintaining contact with the Entrants to monitor the atmosphere using another four-gas monitor and maintaining records of all Entrants, if there is more than one. The Supervisor is responsible for developing the safe work plan for the job at hand prior to entering.

Photo 2-1. Confined Space Entry

Photo 2-2. Typical Personal Four-Gas Monitor

2.2 Flow Meter Installation

Isco 2150 flow meters were installed by V&A in the sewer lines listed in Table 1-1. Isco 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur. Manual level and velocity measurements were taken during installation of the flow meters and again when they were removed and were compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. The pipe diameter was also verified in order to accurately calculate the flow cross-section. The continuous depth and velocity readings were recorded by the flow meters on 5-minute intervals. Figure 2-1 shows a typical installation for a flow meter with a submerged sensor.

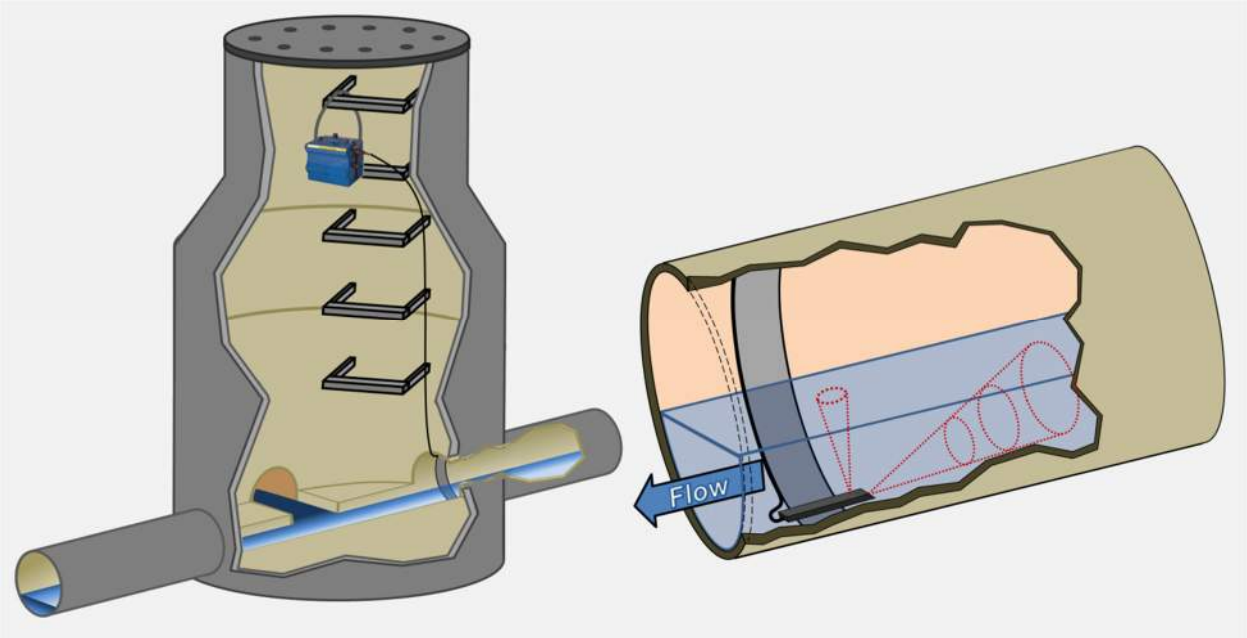


Figure 2-1. Typical Installation for Flow Meter with Submerged Sensor

2.3 Flow Calculation

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

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

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

For circular pipe,

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

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

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

2.4 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 pm, the flows are often decreased (compared to Monday through Thursday). Similarly, flow patterns for Saturday and Sunday were also separated due to their unique evening flow pattern. This type of differentiation can be important when determining I/I response, especially if a rain event occurs on a Friday, Saturday or Sunday evening.

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

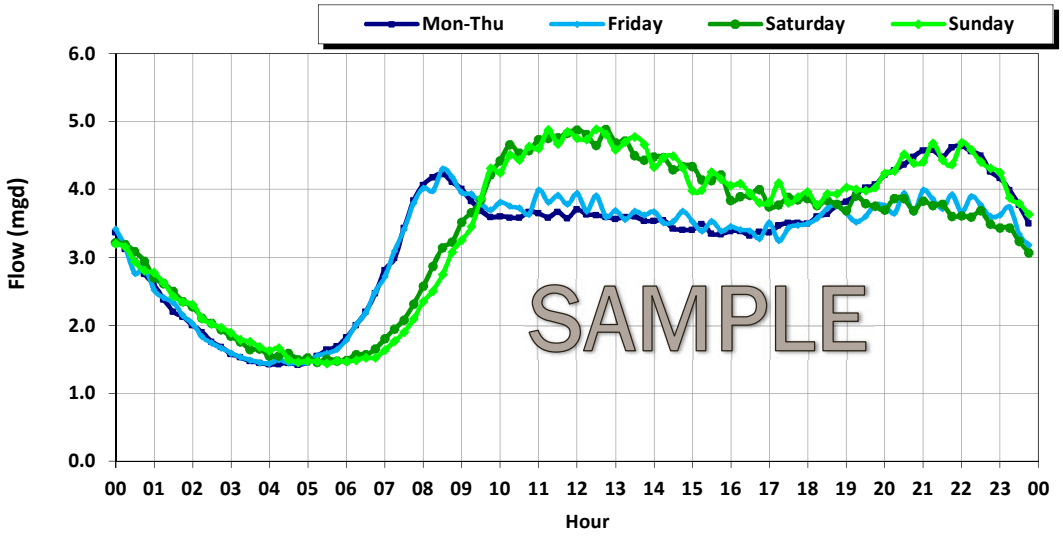


Figure 2-2. Sample ADWF Diurnal Flow Patterns

ADWF curves are taken from "Dry Days", when RDI had the least impact on the baseline flow. For this study, the ADWF curves were also taken from days that did not have major holidays and the resultant tourist traffic. The overall average dry weather flow (ADWF) was calculated per the following equation:

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

2.5 Flow Attenuation

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

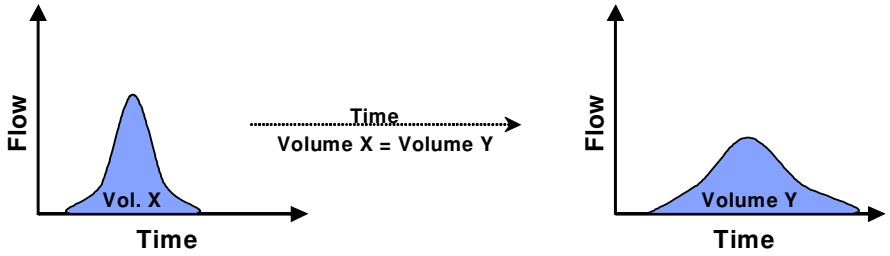


Figure 2-3. Attenuation Illustration

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 (a) not necessarily coincide at the same time, and (b) due to the length and time of travel through the trunk sewers, peak flows will attenuate prior to reaching the treatment facility. The sum of the peak flows of the individual basins within a collection system will usually be greater than the peak flows observed at the treatment facility.

2.6 Inflow / Infiltration Analysis: Definitions and Identification

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

2.6.1 Definition and Typical Sources

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

Figure 2-4 illustrates the possible sources and components of I/I.

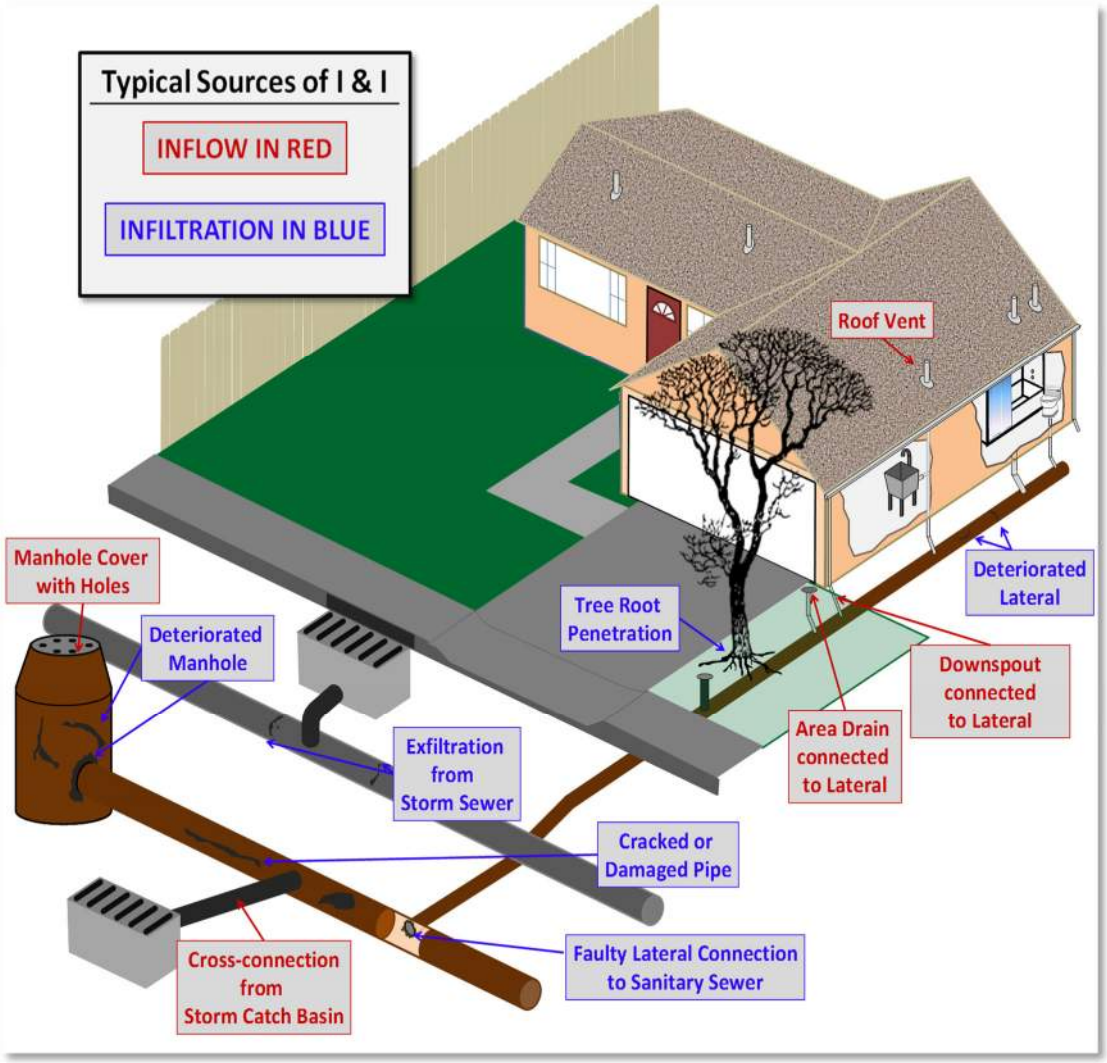


Figure 2-4. Typical Sources of Infiltration and Inflow

2.6.2 Infiltration Components

Infiltration can be further subdivided into components as follows:

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

2.6.3 Impact and Cost of Source Detection and Removal

- **Inflow:**
  - Impact: This component of I/I creates a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Because the response and magnitude of inflow is tied closely to the intensity of the storm event, 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.

2.6.4 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 were 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 were generated for the individual flow monitoring sites and can be found in *Appendix A*.

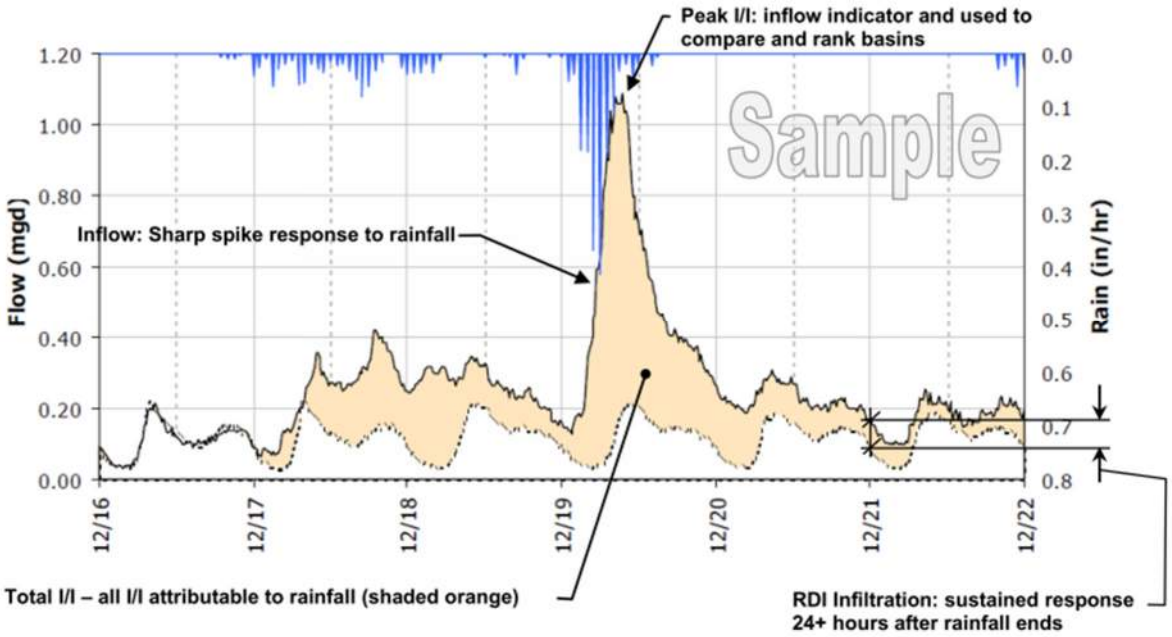


Figure 2-5. Sample Infiltration and Inflow Isolation Graph

2.6.5 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[2].
- **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

[2] I/I flow rate is the real time flow less the estimated average dry weather flow rate. It is an estimate of flows attributable to rainfall. By using peak measured flow rates (inclusive of ADWF), the I/I flow rate would be skewed higher or lower depending on whether the storm event I/I response occurs during low-flow or high-flow hours.

and the time required for flows to return to ADWF levels, different periods may be examined to determine the basins with the greatest or most sustained rainfall-dependent infiltration rates.

- **Total Infiltration:** The total inflow and infiltration is measured in gallons per site and per storm event. Because it is based on total I/I volume, it is an indicator of combined inflow and infiltration and is used to identify the overall volumetric influence of I/I within the monitoring basin.

2.6.6 Normalization Methods

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

- **per-ADWF:** The metric is divided by the established average dry weather flow rate and 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 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 the per-ADWF and the per-ACRE method in this report. In the event of a tie, the per-ADWF method is used to break the tie.

3 Rainfall Results

3.1 Flow Study Rainfall Data

V&A captured rainfall data from 5 publicly available rain gauges maintained by weather enthusiasts[3]. While V&A has no direct control over these gauges, V&A performs QAQC on the data before using it for analysis. V&A used rainfall data from January 1, 2017 through July 2017 to encompass the period when the V&A and City-owned meters were installed. There was a series of rainfall events that occurred from January 2017 through February 2017. Five rainfall events were defined. Table 3-1 and Figure 3-1 show the duration and precipitation of each rainfall event.

Table 3-1. Rainfall Events Used for I/I Analysis

Rainfall Event	Duration	Morro6	Morro28	Losos17	Losos19	Baywo16	Average of 5 RGs
1	1/3/2017 - 1/6/2017	2.44	3.12	2.86	2.60	2.48	2.70
2	1/7/2017 - 1/17/2017	3.42	3.29	2.50	2.33	2.04	2.72
3	1/18/2017 – 1/25/2017	2.75	3.90	3.90	2.18	2.92	3.13
4	2/5/2017 – 2/12/2017	2.23	2.60	2.48	2.44	2.33	2.42
5	2/16/2017 – 2/22/2017	2.54	3.19	3.58	3.77	3.14	3.24
Total over Monitoring Period		***15.53***	***19.22***	***17.99***	***16.22***	***15.76***	***16.95***

[3] National Oceanic and Atmospheric Administration (NOAA) Citizen Weather Observer Program (CWOP) members send data from their private weather stations to the NOAA MADIS server; the data undergoes quality checking and then is distributed.

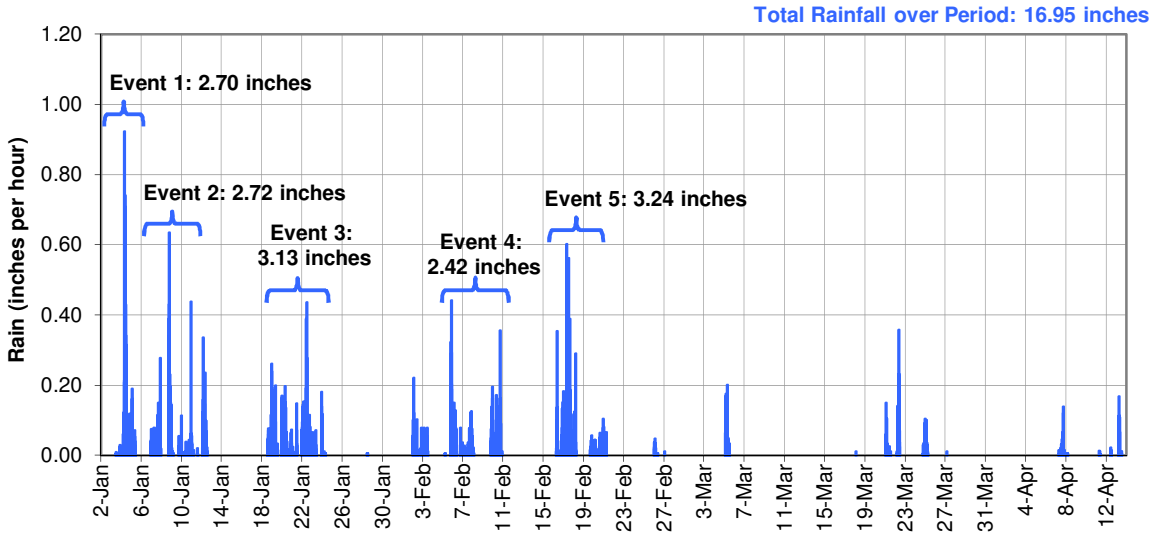


Figure 3-1. Rainfall Activity over Monitoring Period – Average of 5 Rain Gauges

Figure 3-2 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[4] in Morro Bay during this project duration. Rainfall totals for the City were approximately 40% higher than historical normal levels during this time period.

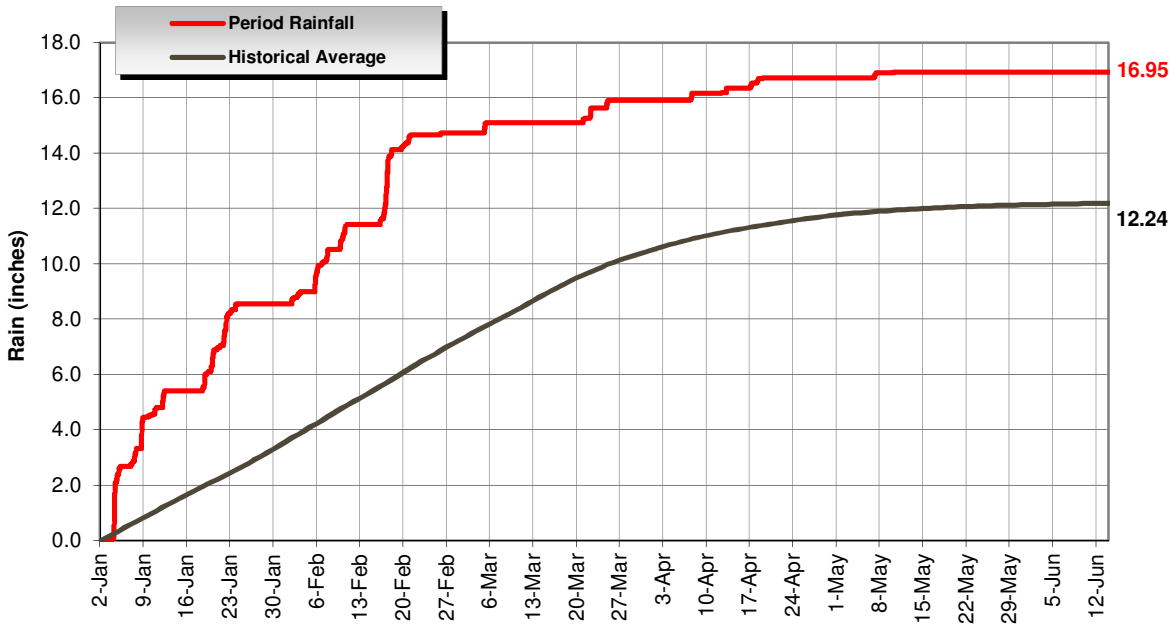


Figure 3-2. Rainfall Accumulation Plot – Average of 5 Rain Gauges

4 Historical data taken from the WRCC (Station 045866 in Morro Bay): http://www.wrcc.dri.edu/summary/climsmnca.html

3.2 Rainfall: Storm Event Classification

It is important to classify the relative size of a major storm event that occurs over the course of a flow monitoring period in order to compare the observed flow response to that occurring during a design storm event (sanitary sewers are often designed to withstand I/I contribution to sanitary flows for specific-sized "design" storm events). Rainfall events are classified by intensity and duration. For example, the National Oceanic and Atmospheric Administration (NOAA) Rainfall Frequency Atlas shown in Figure 3-3[5] classifies a 10-year, 24-hour storm event at the Morro6 rain gauge as 3.65 inches. This means that in any given year, at this specific location, there is a 10% chance that 3.65 inches of rain will fall in any 24-hour period.

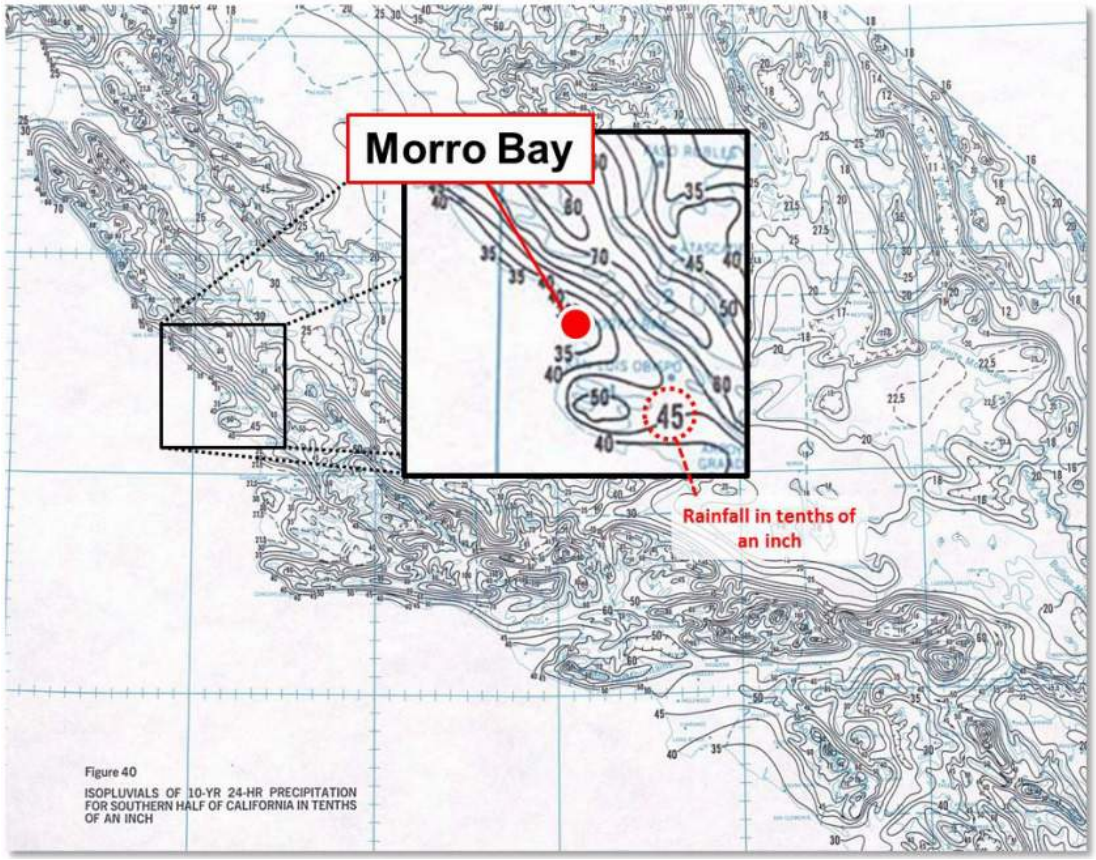


Figure 3-3. NOAA California Map of Isopluvials of 10-year, 24-hour Precipitation

From the NOAA frequency maps, for a specific latitude and longitude, the rainfall densities for period durations ranging from 15 minutes to 60 days are known for rain events ranging from 1-year to 100-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, shown in Figure 3-4 through Figure 3-5 and summarized in Table 3-2.

[5] NOAA Western U.S. Precipitation Frequency Maps Atlas 14, Volume 6, Version 2, 2011: http://www.wrcc.dri.edu/pcpnfreq.html

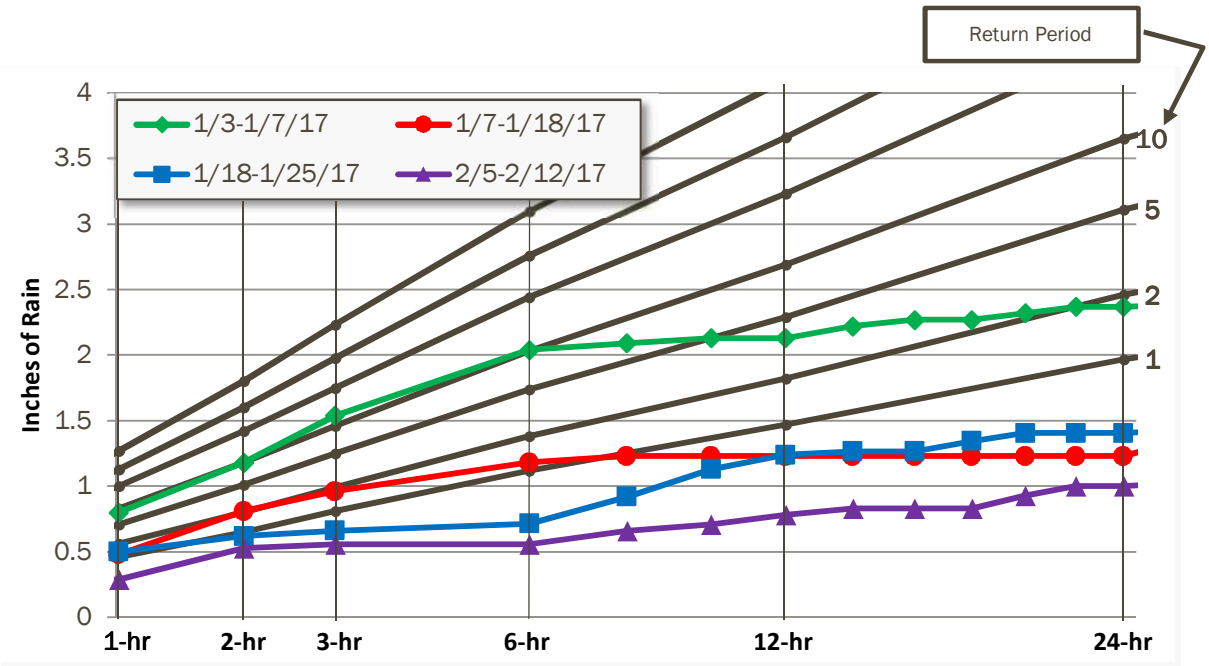


Figure 3-4. Rainfall Event Classification – 24-Hour Period (MORRO6)

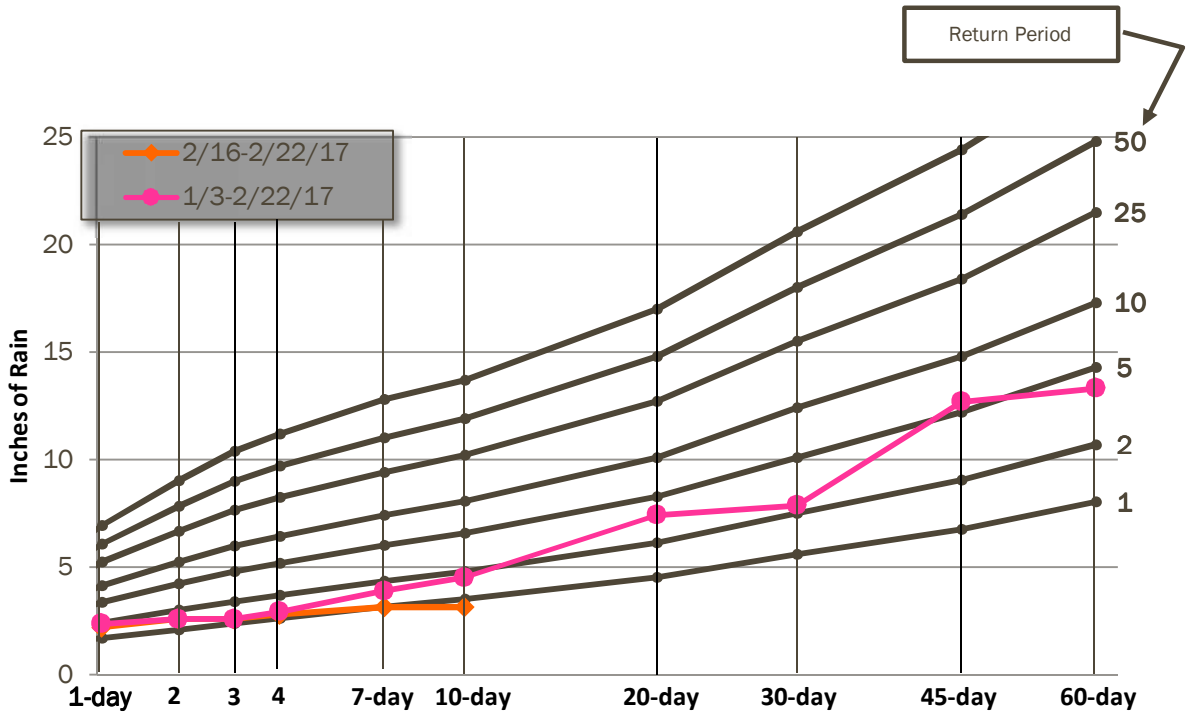


Figure 3-5. Rainfall Event Classification – 60-Day Period (BAYWO16)

Table 3-2. Rainfall Event Classification

Rainfall Event	Morro6	Morro28	Losos17	Losos19	Baywo16
1: Jan 3 - 6	10-Year, 6-Hour	**12-Year, 6-Hour**	10-Year, 3-Hour	10-Year, 3-Hour	5-Year, 3-Hour
2: Jan 7 - 17	1-Year, 6-Hour	**2-Year, 6-Hour**	1-Year, 6-Hour	1-Year, 3-Hour	<1-Year, 24-Hour
3: Jan 18 - 25	<1-Year, 24-Hour	<1-Year, 24-Hour	<1-Year, 24-Hour	<1-Year, 24-Hour	<1-Year, 24-Hour
4: Feb 5 – 12	<1-Year, 24-Hour	<1-Year, 24-Hour	<1-Year, 24-Hour	<1-Year, 24-Hour	<1-Year, 24-Hour
5: Feb 16 - 22	1-Year, 24-Hour	2-Year, 24-Hour	**3-Year, 24-Hour**	**3-Year, 24-Hour**	1-Year, 24-Hour
Season: Jan 3 – Feb 22	5-Year, 60-Day	**10-Year, 60-Day**	**13-Year, 45-Day**	5-Year, 60-Day	5-Year, 45-Day

4 Flow Monitoring Results

4.1 Average Flow Analysis

ADWF curves were established during the two periods: January 13 through January 16 and January 24 through February 1 when RDI had the least impact on the baseline flow. Table 4-1 summarizes the dry weather flow data measured for this study. ADWF curves for each site can be found in *Appendix A.*

Figure 4-1 shows a flow schematic highlighting the average daily flows and levels of the V&A sites. Figure 4-2 shows the same for the City-owned meters.

Table 4-1. Dry Weather Flow

Monitoring Site	Monday-Thursday ADWF (mgd)	Friday ADWF (mgd)	Saturday ADWF (mgd)	Sunday ADWF (mgd)	Overall ADWF (mgd)
Site 1	0.180	0.185	0.185	0.184	0.182
Site 2	0.026	0.029	0.030	0.029	0.027
Site 3	0.281	0.303	0.316	0.321	0.295
Site 4	0.211	0.225	0.256	0.239	0.224
Site 5	0.085	0.093	0.099	0.107	0.091
Site 6	0.110	0.109	0.120	0.118	0.112
Site 7	0.039	0.041	0.041	0.045	0.040
Site 8	0.067	0.068	0.075	0.075	0.069
Site 9	0.019	0.020	0.021	0.023	0.020
Site 10	0.069	0.070	0.068	0.072	0.070
Site 11	0.062	0.063	0.068	0.071	0.064
Site 12	0.121	0.133	0.147	0.136	0.129
Site 13	0.089	0.089	0.105	0.102	0.093
Site 14	0.088	0.086	0.098	0.107	0.092
Site 15	0.042	0.046	0.052	0.052	0.046
MB 1	0.168	0.159	0.160	0.172	0.167
MB 2	0.027	0.031	0.029	0.034	0.029
MB 3	0.287	0.282	0.310	0.333	0.296
MB 4	0.205	0.226	0.258	0.253	0.223
Flume	0.887	0.910	0.982	0.998	0.920

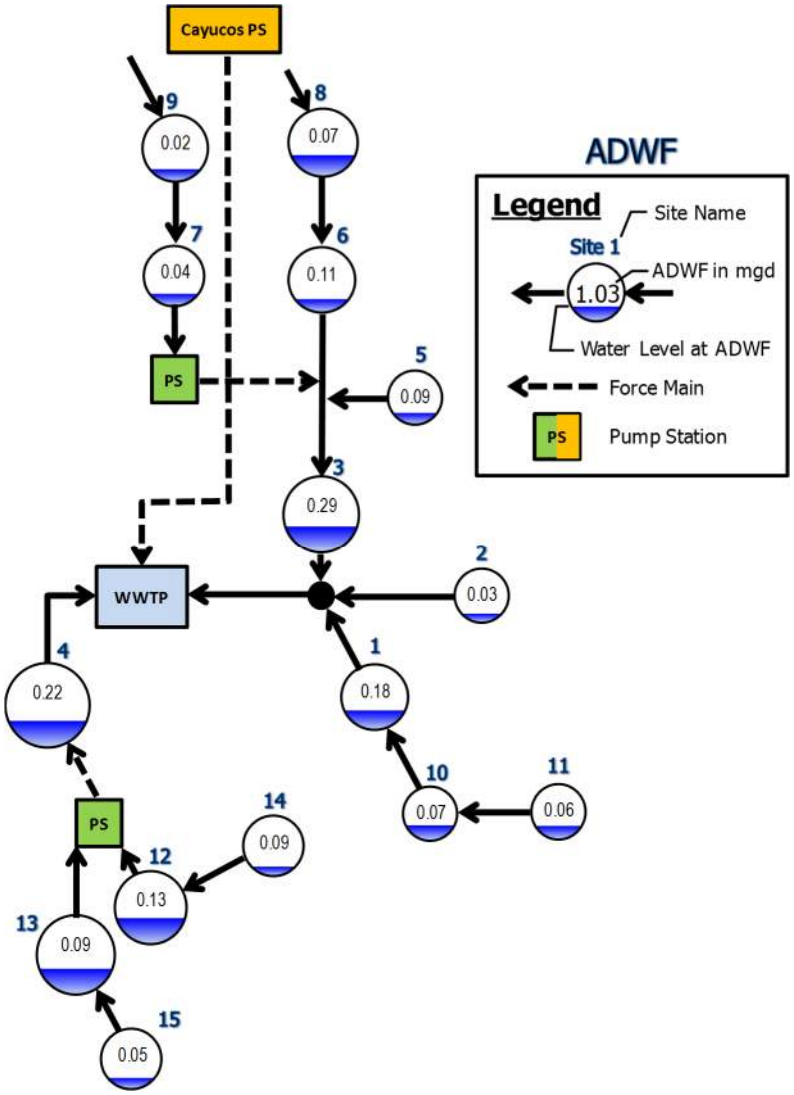


Figure 4-1. Dry Weather Flow Schematic – V&A Sites

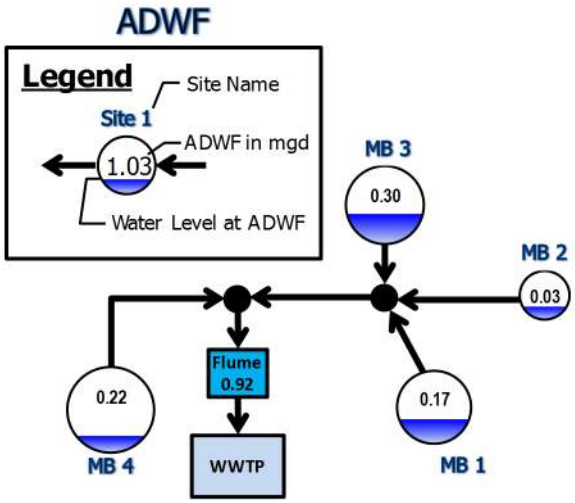


Figure 4-2. Dry Weather Flow Schematic – Morro Bay Sites

4.2 Tourist Flow

As a part of the flow study, V&A was asked to keep flow meters installed during the Memorial Day and July 4th holiday in order to provide the City with data about increases in flow due to the tourist population. Figure 4-3 and Figure 4-4 show the flows for Site 14 during the Memorial Day and July 4th holidays. Table 4-2 summarizes the percentage increase in flows and Figure 4-5 and Figure 4-6 show temperature maps of the V&A basins and the percentage increase in flow.

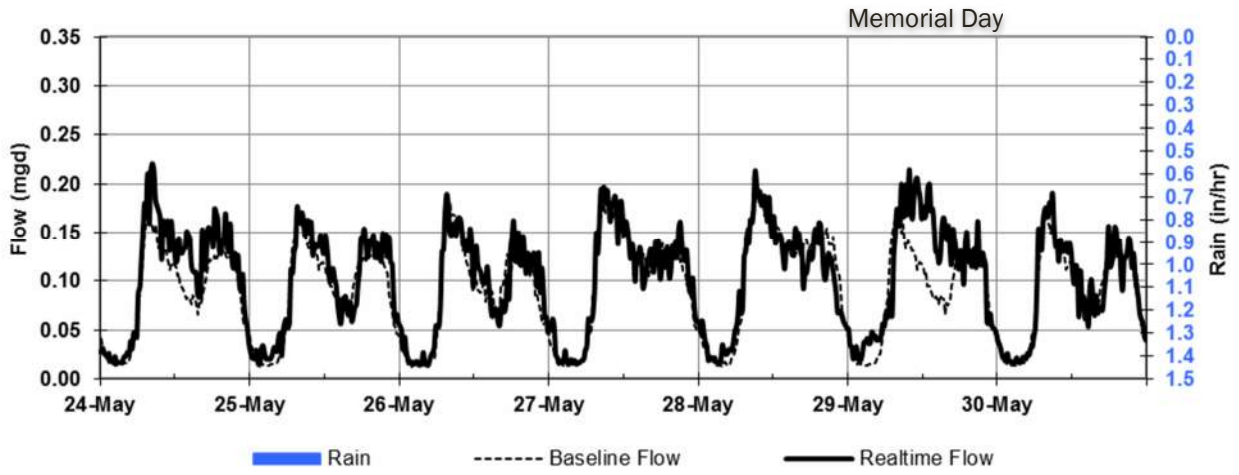


Figure 4-3. Site 14 Flows – Memorial Day Holiday

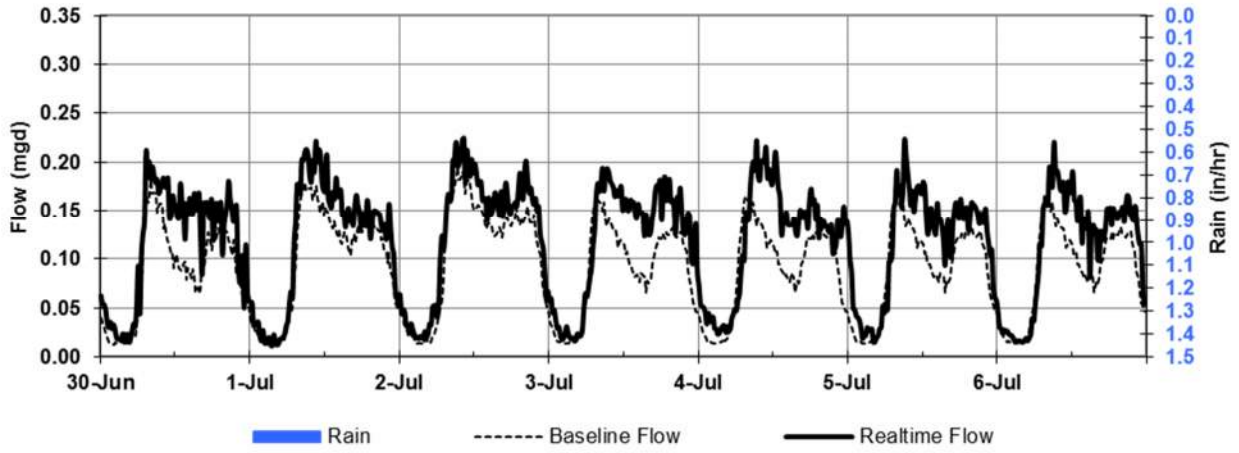


Figure 4-4. Site 14 Flows – July 4th Holiday

Table 4-2. Summary of Holiday Flows

Monitoring Basin*	Memorial Day Holiday Weekend Difference (%)	July 4th Holiday Weekend Difference (%)
Basin 1	n/a	9%
Basin 2	7%	19%
Basin 3	n/a	-18%
Basin 5	23%	12%
Basin 6	n/a	34%
Basin 7	10%	-10%
Basin 8	14%	35%
Basin 9	20%	70%
Basin 11	16%	6%
Basin 12	25%	32%
Basin 13	15%	0%
Basin 14	10%	29%
Basin 15	2%	48%
Flume	7%	28%

* Basin 4 not ranked because basin boundary could not be determined, Basin 10 not ranked due to small relative contributing flows within basin, n/a – data not available

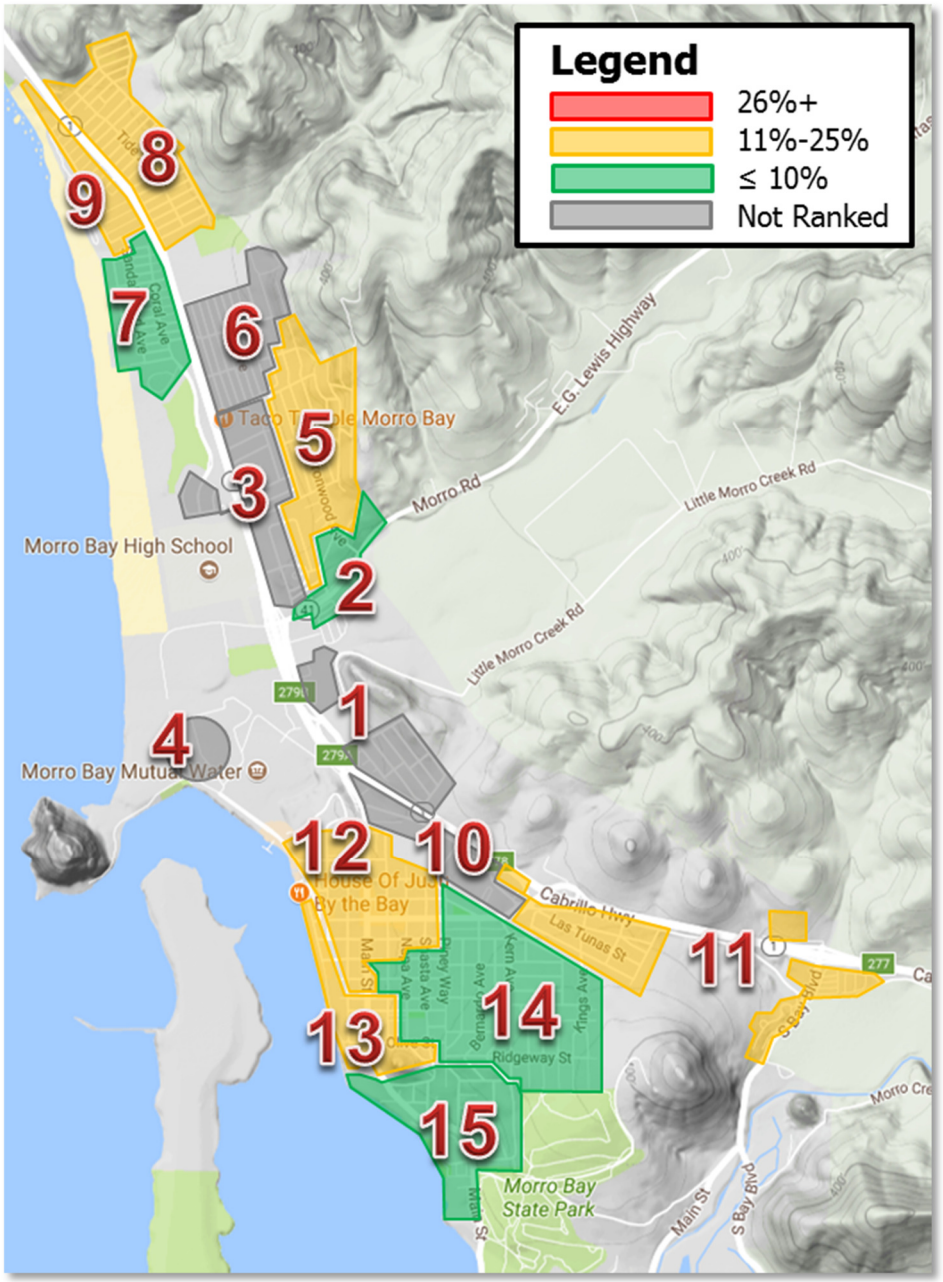


Figure 4-5. Memorial Day Holiday Flows temperature map

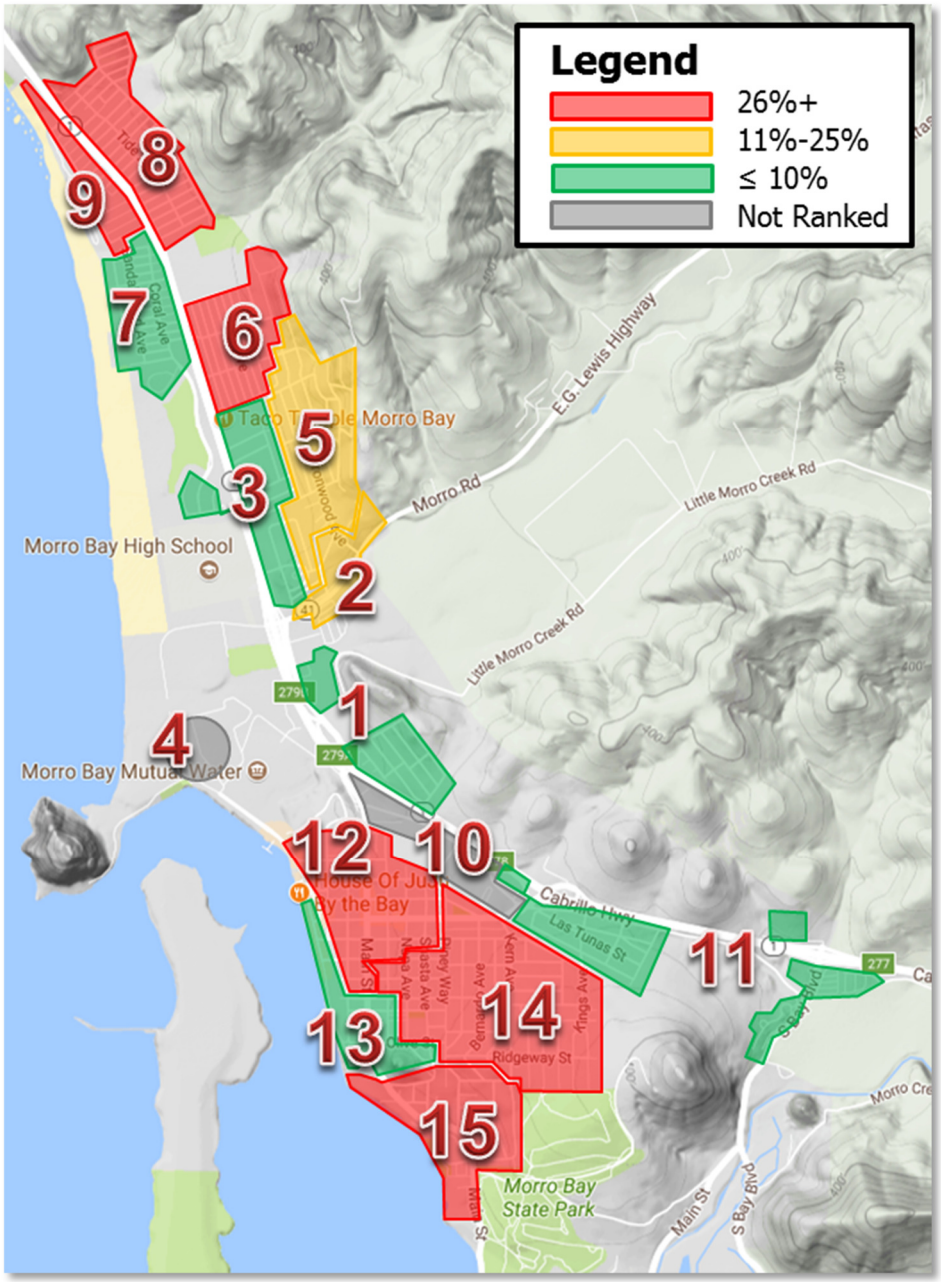


Figure 4-6. July 4th Holiday Flows temperature map

4.3 Capacity Analysis: Peaking Factor and d/D Ratio

Peak measured flows and the corresponding flow levels (depths) are important to understand the capacity limitations of a collection system. The peak flows and flow levels reported are from the peak measurements as taken across the entirety of the flow monitoring period. Peak flows and levels may not correspond to a rainfall event. There were several instances of backflow conditions (detailed below) due to capacity constraints and the inability of the local collection system to handle peak wet weather flows.

The following capacity analysis terms are defined as follows:

- **Peaking Factor:** Peaking factor 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 tributary area and the amount and characteristics of I/I entering the collection system. Flow attenuation and flow restrictions will also affect the peaking factor. Ideally wet weather peaking factors would be approaching 3 for new systems.
- **d/D Ratio:** The d/D ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). Standards for d/D ratio vary from agency to agency, but typically range between $d/D \leq 0.5$ and $d/D \leq 0.75$. The d/D ratio for each site was computed based on the maximum depth of flow for the flow monitoring study.

Table 4-3 summarizes the peak recorded flows, levels, *d/D* ratios, and peaking factors per site during the flow monitoring period. Results of note have been shaded in RED. Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the point site locations. Hydraulic conditions in other areas of the collection system will differ.

Table 4-3. Capacity Analysis Summary

Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	*d/D* Ratio	Level Surcharged above Crown (ft)
Site 1	0.18	0.48	2.6	11.75	5.6	0.48	--
Site 2	0.03	0.08	3.1	8	2.5	0.32	--
Site 3	0.29	0.89	3.0	15	7.4	0.49	--
Site 4	0.22	0.58	2.6	17.5	8.4	0.48	--
Site 5	0.09	0.28	3.1	8	2.5	0.32	--
Site 6	0.11	0.35	3.1	13	4.1	0.32	--
Site 7	0.04	0.16	4.1	9.75	3.5	0.36	--
Site 8	0.07	0.21	3.1	13	7.3	0.56	--
Site 9	0.02	0.10	4.8	12	3.8	0.32	--
Site 10	0.07	0.32	4.7	8	4.6	0.57	--
Site 11	0.06	0.25	3.9	8	3.6	0.46	--
Site 12	0.13	0.38	3.0	14.375	7.7	0.54	--
Site 13	0.09	0.31	3.3	15.5	7.0	0.45	--
Site 14	0.09	0.32	3.5	10	2.5	0.25	--
Site 15	0.05	0.21	4.6	10	3.5	0.35	--

Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	*d/D* Ratio	Level Surcharged above Crown (ft)
MB 1	0.17	0.93	5.6	12	14.0	1.17	0.17
MB 2	0.03	0.17	5.9	6	2.5	0.41	--
MB 3	0.30	2.54	8.6	15	23.6	1.57	0.70
MB 4	0.22	1.02	4.6	18	6.1	0.34	--
Morro Bay Flume	0.92	10.08	11.0	27	N/A	N/A	N/A

The following capacity analysis results are noted:

- **Peaking Factor:** Only three sites, sites MB 1, MB 2 and MB 3, as well as the Morro Bay Influent Flume had peaking factors that exceeded 5. It should be noted that the City-owned sites were installed during the bulk of the rainfall season, while the V&A sites were installed afterwards.
- **d/D Ratio:** Only two of the nineteen flow monitoring sites, MB 1 and MB 3, had d/D ratios that were above the common threshold value of 0.75 d/D. These two sites reached a surcharge condition during the study.
- **Flume**: The Morro Bay 27-inch Palmer Bowlus Flume had a peak measured flow of 10.08 mgd. The manufacturer American Sigma's drawing states a 10.24 mgd maximum flow and 1.76 maximum head. As a result, the peak measured flow is at 98% capacity.

Figure 4-7 and Figure 4-8 show bar graphs summarizing the site-by-site d/D ratios, surcharge levels and peaking factors, respectively. Figure 4-9 shows a flow schematic showing the peak flow levels for the V&A flow meters. Figure 4-10 shows the same for the City-owned meters.

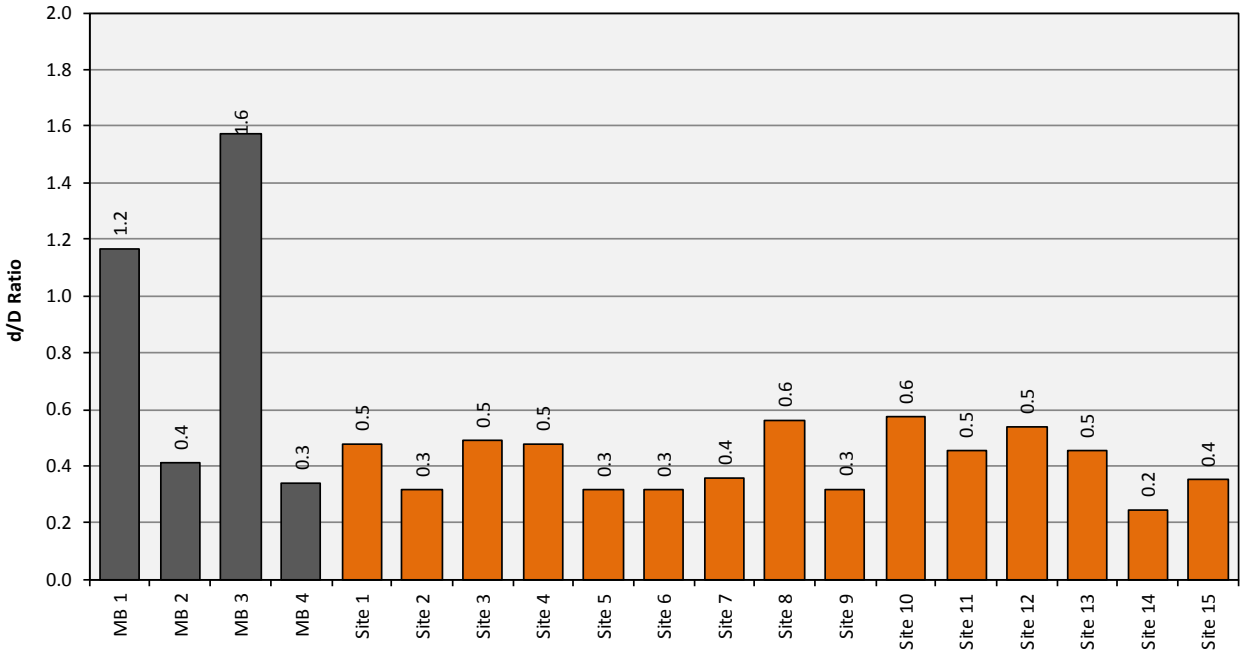


Figure 4-7. Capacity Summary: d/D Ratios

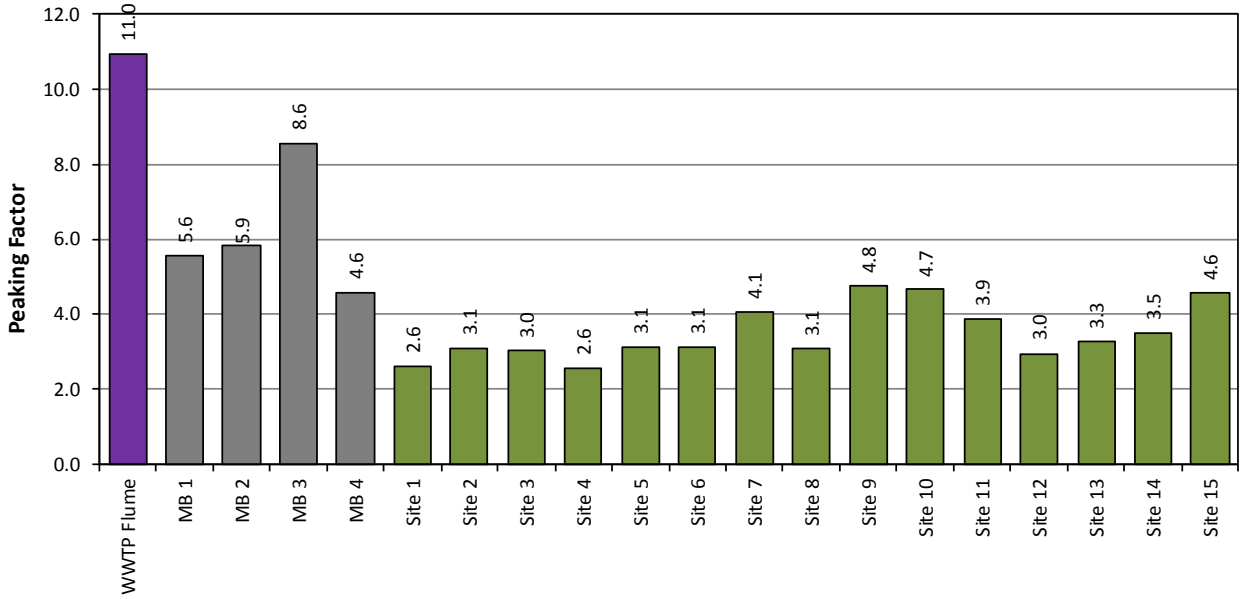


Figure 4-8. Capacity Summary: Peaking Factors

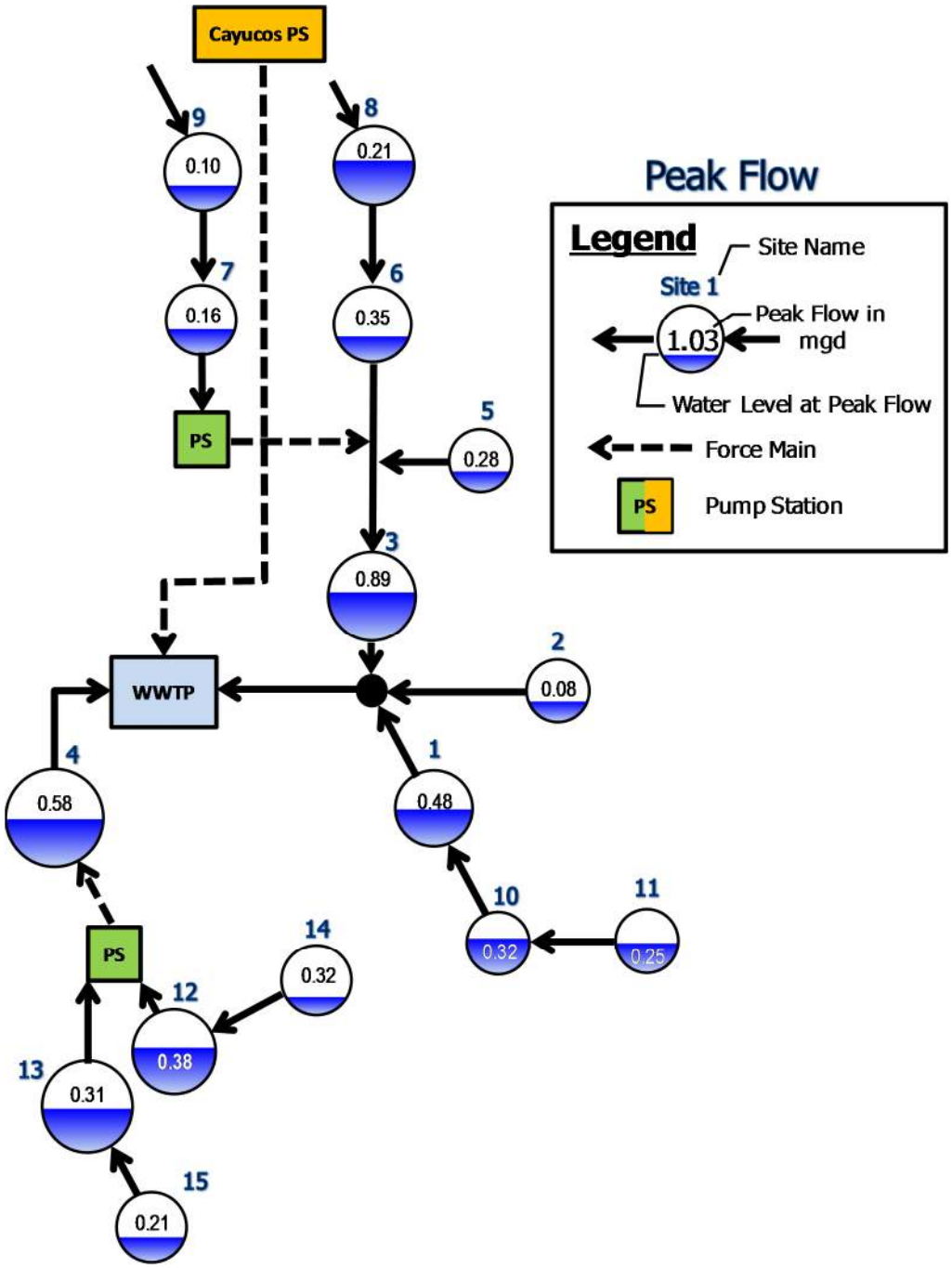


Figure 4-9. Peak Flow Schematic – V&A Sites

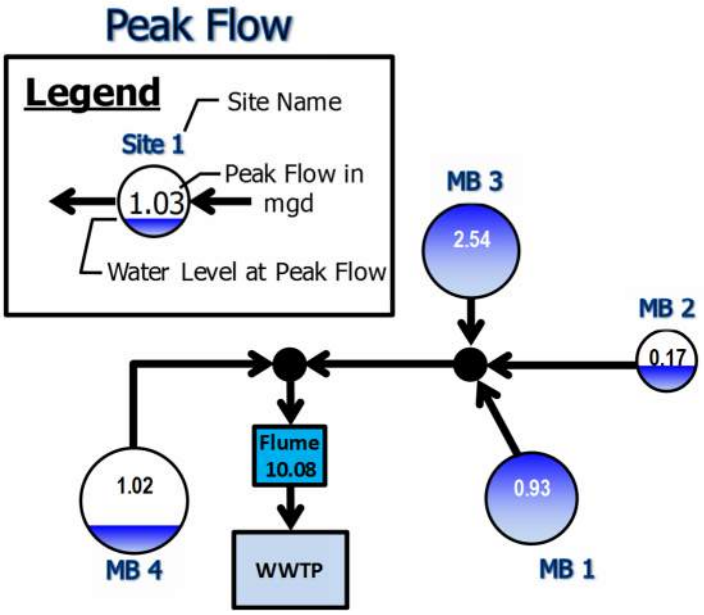


Figure 4-10. Peak Flow Schematic – Morro Bay Sites

5 Inflow and Infiltration Results

5.1 Preface

The following is noted for the I/I analyses of this study:

- The inflow analysis used Event 1 for the analysis (see Figure 5-1 for reference)
- The RDI rate was measured as the average I/I response after Event 4 from February 23, 2017 12:00 am to February 24, 2017 12:00am (see Figure 5-2 for reference).
- I/I analysis on the flume was not performed as there is no substantive basin that could be isolated. The flume's reaction to the rainfall events is included in Appendix A.

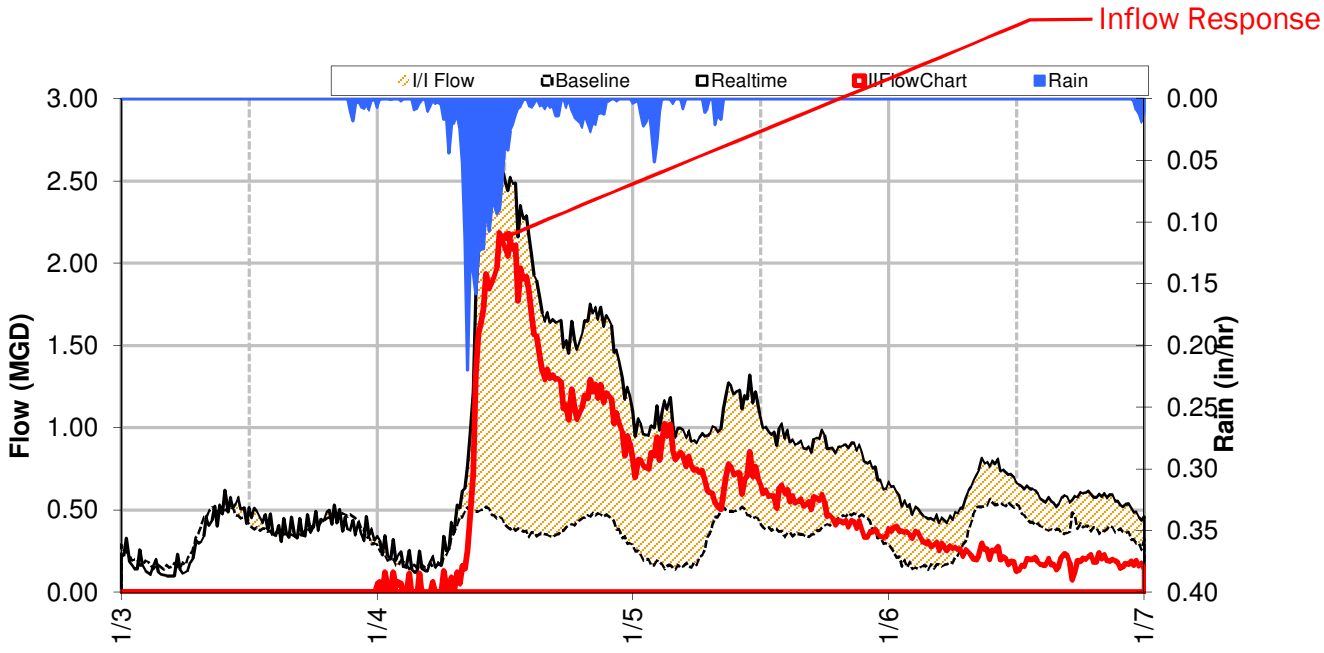


Figure 5-1. I/I Isolation Graph, Site MB 3, Event 1

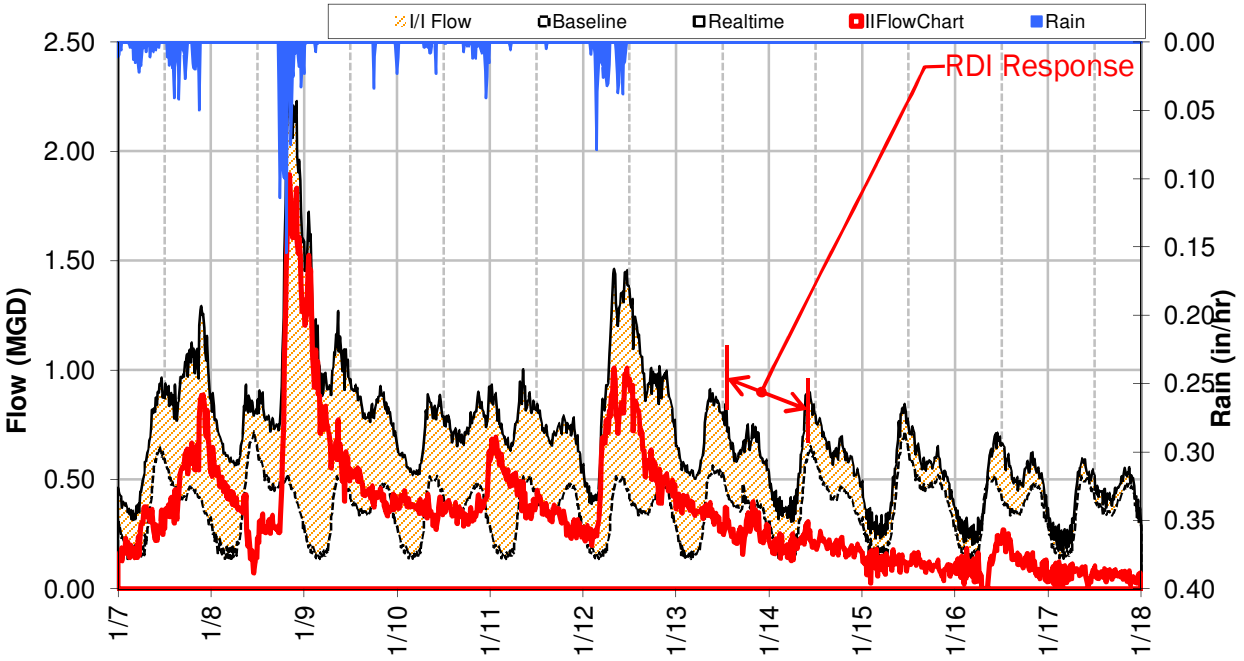


Figure 5-2. I/I Isolation Graph, Site MB 3, Event 2

5.2 January Flume Event

During rain event 1, which was used for inflow analysis, the City influent flume experienced a peak flow event. V&A compared the data from the flume to the data from the four upstream flow meters. As seen in Figure 5-3, the black line is the flume data, the purple line is the sum of sites MB 1, MB 2, MB 3 and MB 4. The data for the flume and the four City meters track each other very well throughout the study and for all of the rainfall events (see January 9 as a typical example) with the exception of the January 4 event. For this event, the flume data far exceeded the upstream measured data. This type of excessive spike was not measured for any of the other strong rainfall events.

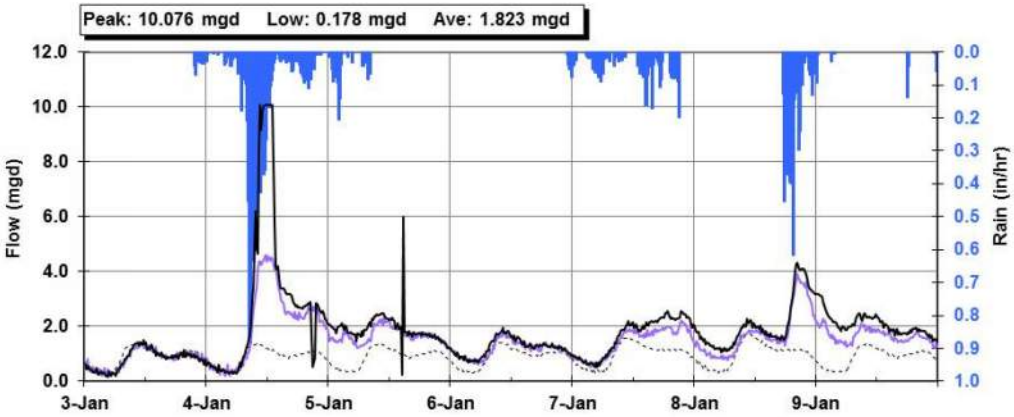


Figure 5-3. Flume and Upstream Meters Comparison – Event 1

Some possible causes of the spike are:

- Possible inflow source that only “activates” during extreme peak intensity rainfall events, located in the area not monitored between Sites 1,2,3,4 and the flume; Errol Street (upstream mobile home park) or Lila H Kaiser Park. These areas are highlighted ‘yellow’ in Figure 5-4. Little Morro Creek is also located in the area.
- Perhaps it was possible there was flow recirculated from the treatment plant back into the system and double-counted at the flume.

Figure 5-4. Area View of 4 City Meters and Flume

5.3 Inflow Results Summary

Inflow is storm water discharged into the sewer system through direct connections such as downspouts, area drains, cross-connections to catch basins, etc. These sources transport rain water directly into the sewer system and the corresponding flow rates are tied closely to the intensity of the storm. This component of I/I often causes a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows.

Table 5-1 and Figure 5-5 summarize the peak measured I/I flows and inflow analysis results. Peak I/I rates for all sites were measured during rain event 1 (January 3 to 6, 2017), as this was the most intensive short-term rainfall event. The top ranked basins with the highest normalized inflow have been shaded in RED.

The following inflow analysis results are noted:

- Basin MB 3 ranked highest for normalized inflow contribution. Basins MB 1 and MB 2 also had elevated levels of inflow.

Table 5-1. Inflow Analysis Summary

Monitoring Basin	ADWF (mgd)	Peak Inflow Rate (mgd)	Peak Inflow per ADWF	Peak Inflow per Acre (gpd/acre)	Inflow Ranking [A]
MB 1	0.17	0.68	4.07	2,750	3
MB 2	0.03	0.14	4.87	2,130	2
MB 3	0.30	2.15	7.27	3,226	1
MB 4	0.22	0.70	3.12	1,018	4

[A] Ranking of 1 represents most inflow after normalization.

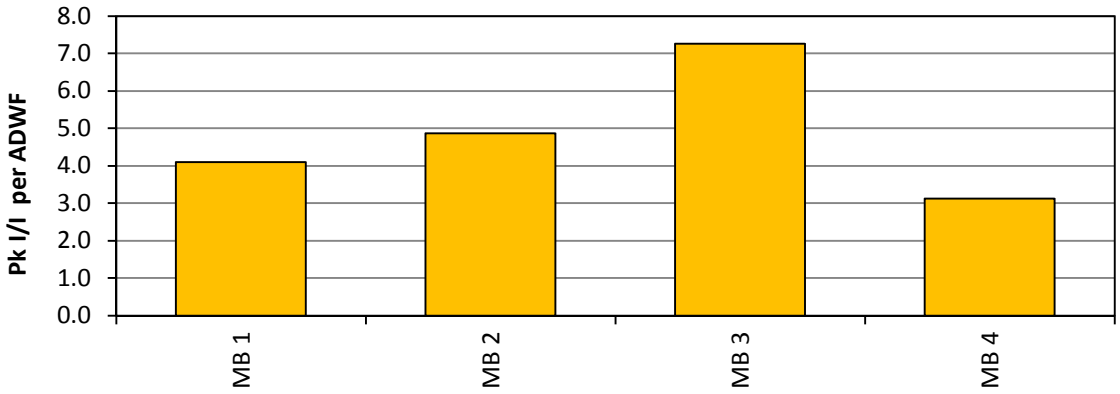


Figure 5-5. Bar Graph: Inflow Analysis Summary – Peak I/I to ADWF

5.4 Infiltration Results Summary

Rain dependent infiltration (RDI) 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 rain water 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 drop off as soils become less saturated and as 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).

Table 5-2 summarizes the calculated RDI flow rates for Rain Event 2. Figure 5-6 shows a bar graph summary. The following infiltration analysis results are noted:

- Basin MB 3 had the highest rates of infiltration. Basins MB 1 and MB 2 also had elevated levels of infiltration.

Table 5-2. Infiltration Analysis Summary

Monitoring Basin	ADWF	RDI Rate (mgd)	RDI per ADWF	RDI per Acre (gpd / acre)	Rank[A]
MB 1	0.17	0.13	75%	509	3
MB 2	0.03	0.02	79%	345	2
MB 3	0.30	0.26	87%	386	1
MB 4	0.22	0.02	10%	33	4

[A] Ranking of 1 represents most inflow after normalization.

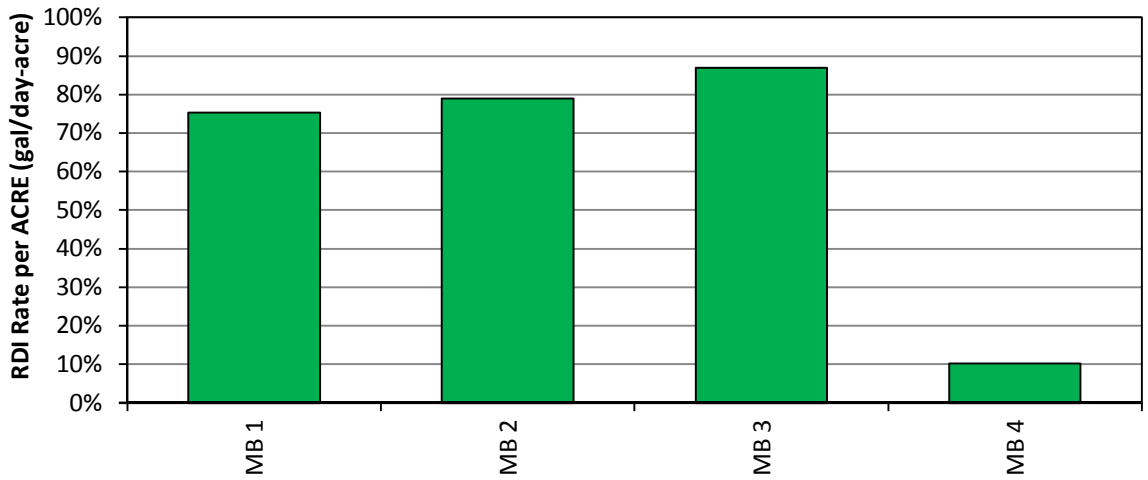


Figure 5-6. Bar Graph: Infiltration Analysis Summary – RDI Rate to ADWF

5.5 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[6]. Figure 5-7 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.

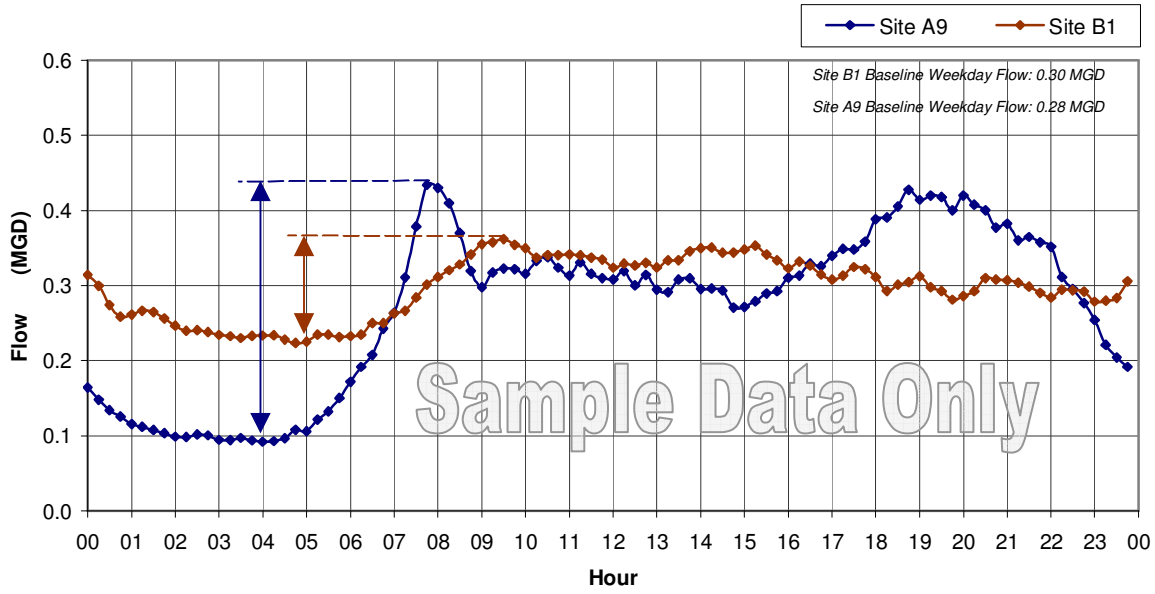


Figure 5-7. Sample Groundwater Infiltration Sample Graph

It can be useful to compare the low-to-ADWF flow ratios for the flow metering 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 5-8 plots the low-to-ADWF flow ratios against the ADWF flows for the sites monitored during this study. The dotted line shows "typical" low-to-ADWF ratios per the WEF[7], [8].

The following GWI results are noted:

- Only basins MB 3 may have some evidence of groundwater infiltration.

[6] 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.

[7] WEF Manual of Practice No. 9, "Design and Construction of Sanitary and Rain Sewers."

[8] 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. Therefore, the WEF typical trend line's slope is closer to 1.0 as the ADWF increases, as shown in the figure.

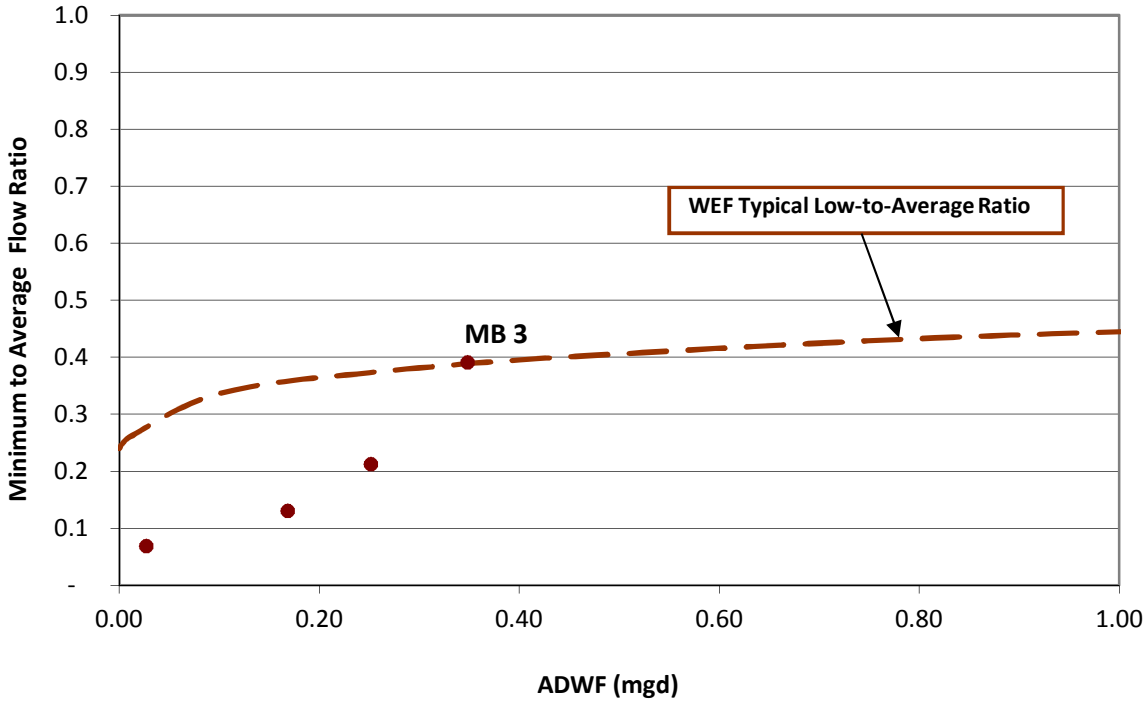


Figure 5-8. Minimum Flow Ratios vs. ADWF[9]

[9] 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.

5.6 Combined I/I Results Summary

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 5-3 summarizes the combined I/I flow results for total I/I measured across the entirety of the flow monitoring period. Results of note have been shaded in RED. Figure 5-9 shows a bar graph summary of the combined I/I analysis.

The following combined I/I analysis results are noted:

- Basin MB 3 had the highest amounts of combined I/I. Basins MB 1 and MB 2 also had elevated levels of combined I/I.

Table 5-3. Combined I/I Analysis Summary

Monitoring Basin	ADWF	Combined I/I (gallons)	Combined I/I per ADWF (per inch-rain)	R Value	Combined I/I per Acre (per inch-rain)	Rank[A]
MB 1	0.17	7,331,000	2.87	7.14%	1,940	3
MB 2	0.03	1,286,000	2.90	4.67%	1,268	2
MB 3	0.30	18,545,000	4.09	6.68%	1,815	1
MB 4	0.22	1,219,000	0.36	0.43%	116	4

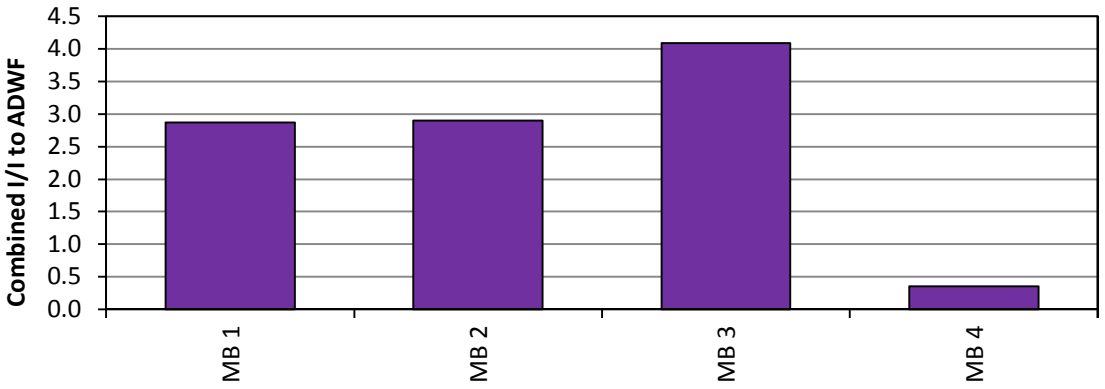


Figure 5-9. Bar Graph: Combined I/I Analysis Summary – Combined I/I Rate to ADWF

6 Recommendations

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

1. **Determine I/I Reduction Program:** The City should examine its I/I reduction needs to determine their needs and goals for a future I/I reduction program.
    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 higher inflow/ADWF ratios.
    b. If 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 higher RDI/ADWF ratios.
2. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
    a. Smoke testing
    b. Mini-basin flow monitoring
    c. Night-time reconnaissance work to (1) investigate and determine direct point sources of inflow, and (2) determine the areas and/or pipe reaches responsible for high levels of infiltration contribution.
    d. CCTV inspection
3. **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 Site Reports: Data, Graphs, Information

City of Morro Bay

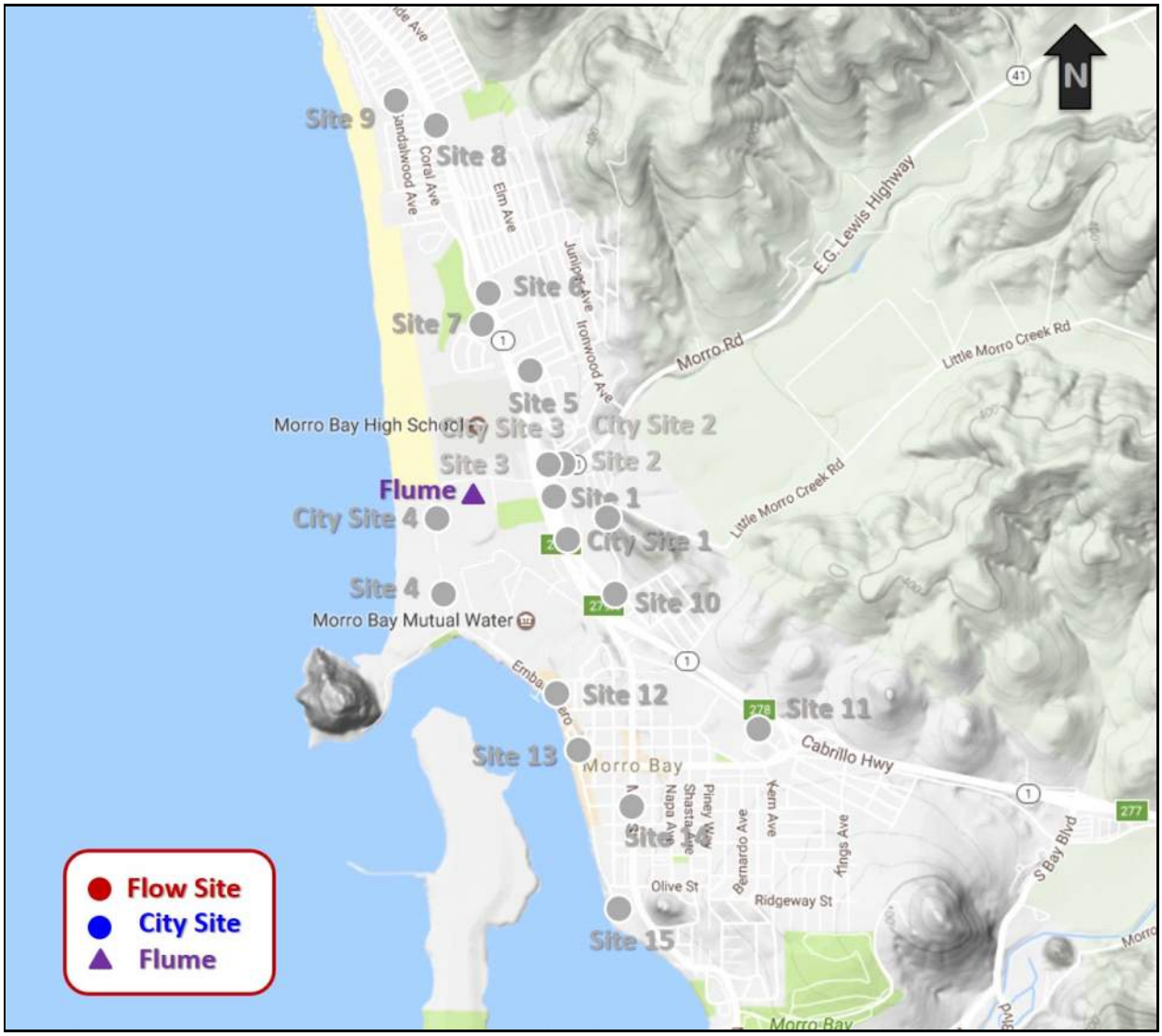
Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 2017 - July 2017

Monitoring Site: WWTP Flume

Location: Wastewater Treatment Plant: 27" Palmer-Bowlus Flume

Data Summary Report



Vicinity Map: WWTP Flume

WWTP FLUME

Site Information

Location: Wastewater Treatment Plant: 27" Palmer-Bowlus Flume

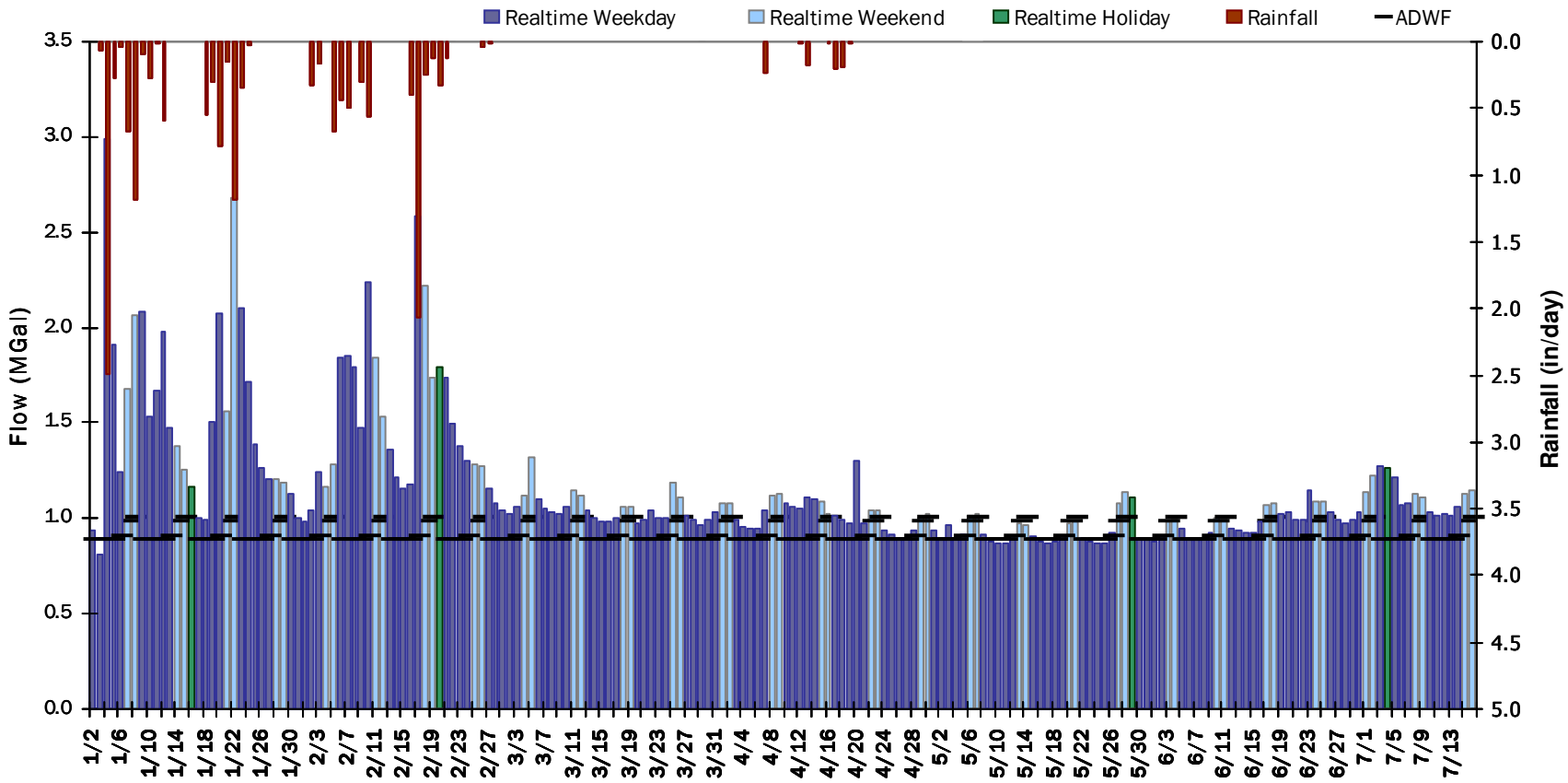
Pipe Diameter: 27 inches

ADWF: 0.920 mgd

Peak Measured Flow: 10.076 mgd

WWTP FLUME

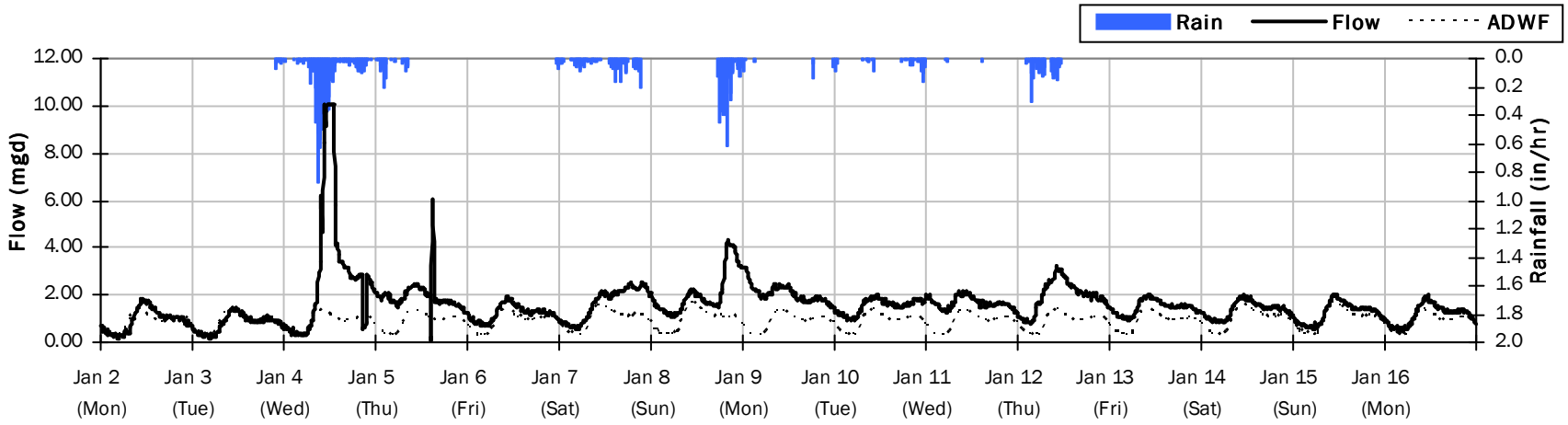
Period Flow Summary: Daily Flow Totals

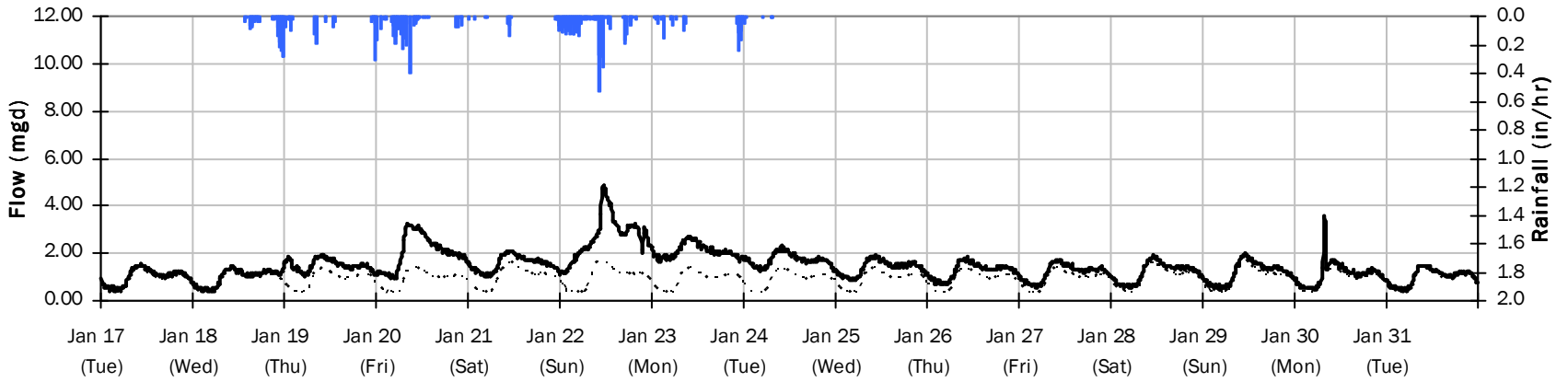


WWTP FLUME

Flow Summary: 1/2/2017 to 1/31/2017

Total Period Rainfall: 9.05 inches **Avg Flow: 1.538 mgd Peak Flow: 10.076 mgd Min Flow: 0.176 mgd**

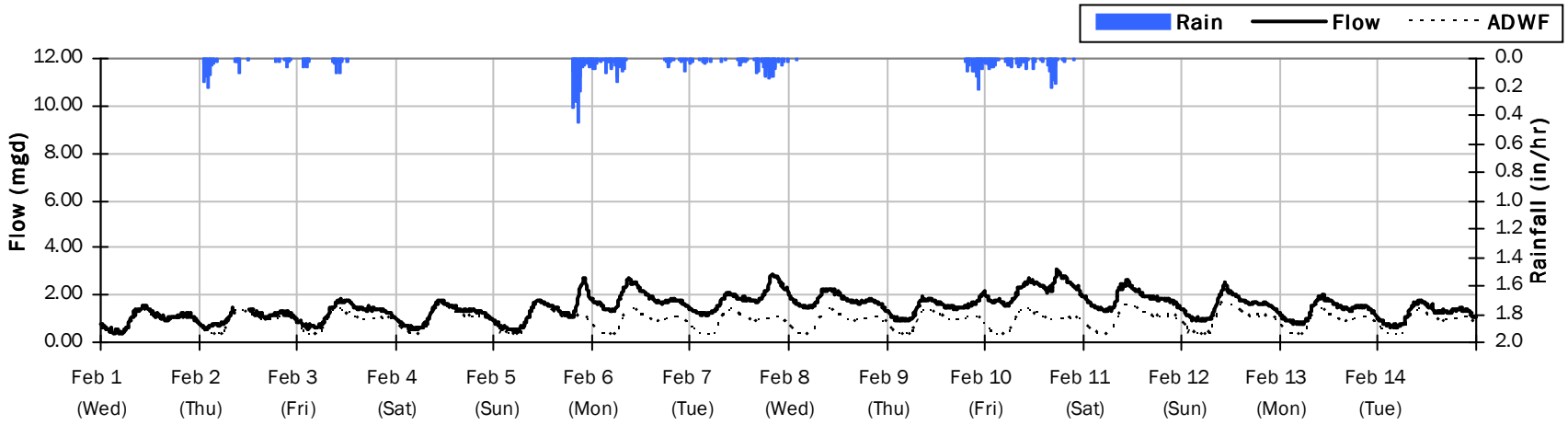


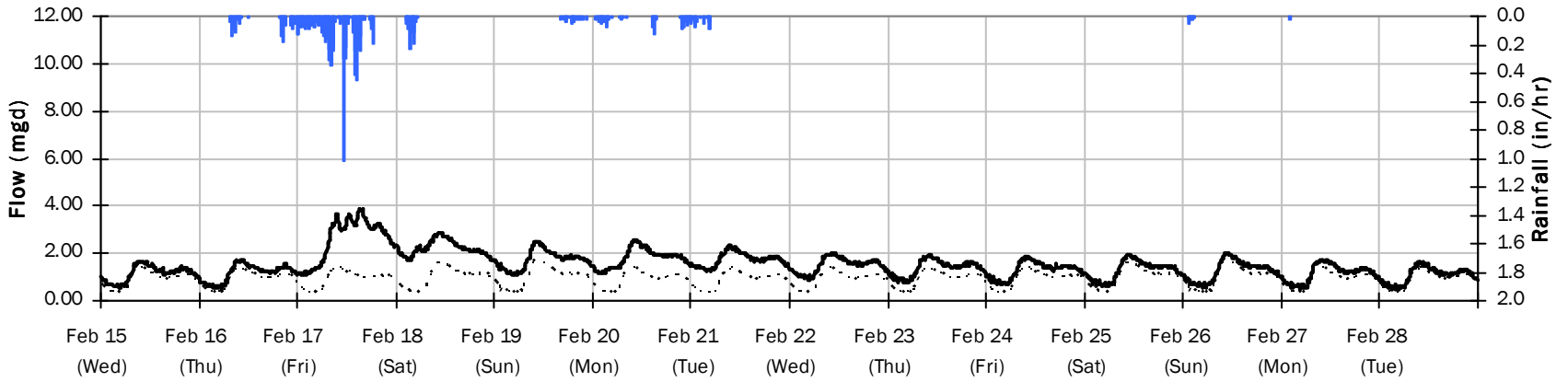


WWTP FLUME

Flow Summary: 2/1/2017 to 2/28/2017

Total Period Rainfall: 6.32 inches **Avg Flow: 1.510 mgd** **Peak Flow: 3.875 mgd** **Min Flow: 0.373 mgd**

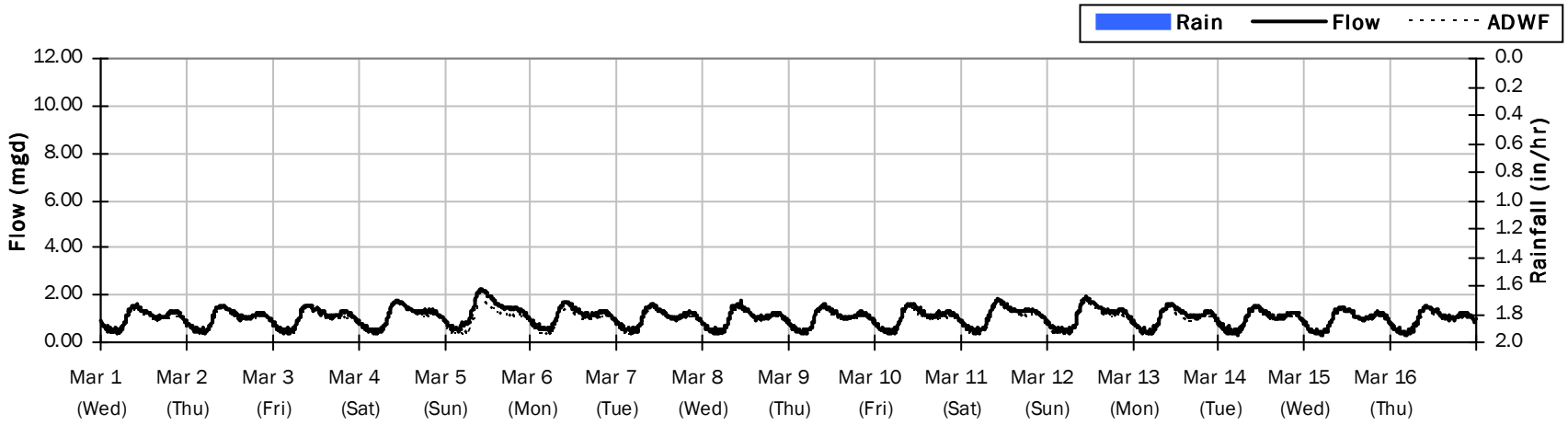


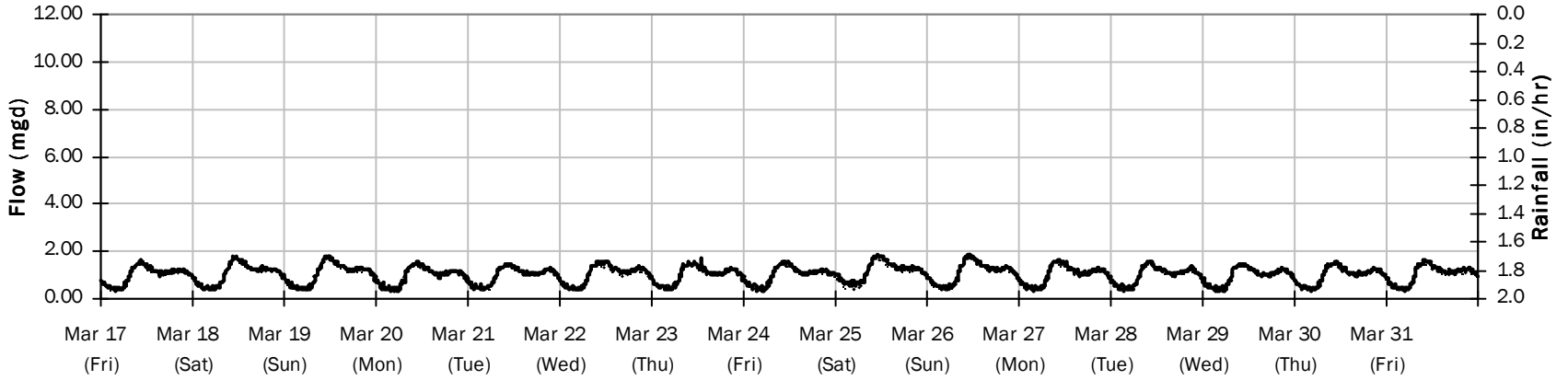


WWTP FLUME

Flow Summary: 3/1/2017 to 3/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 1.050 mgd Peak Flow: 2.262 mgd Min Flow: 0.312 mgd**

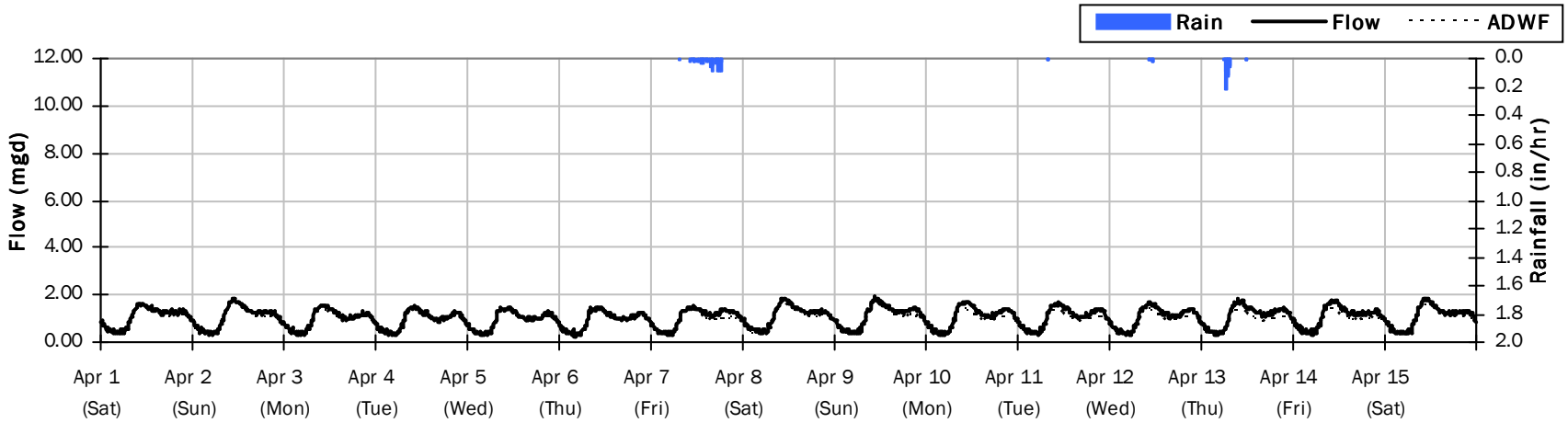


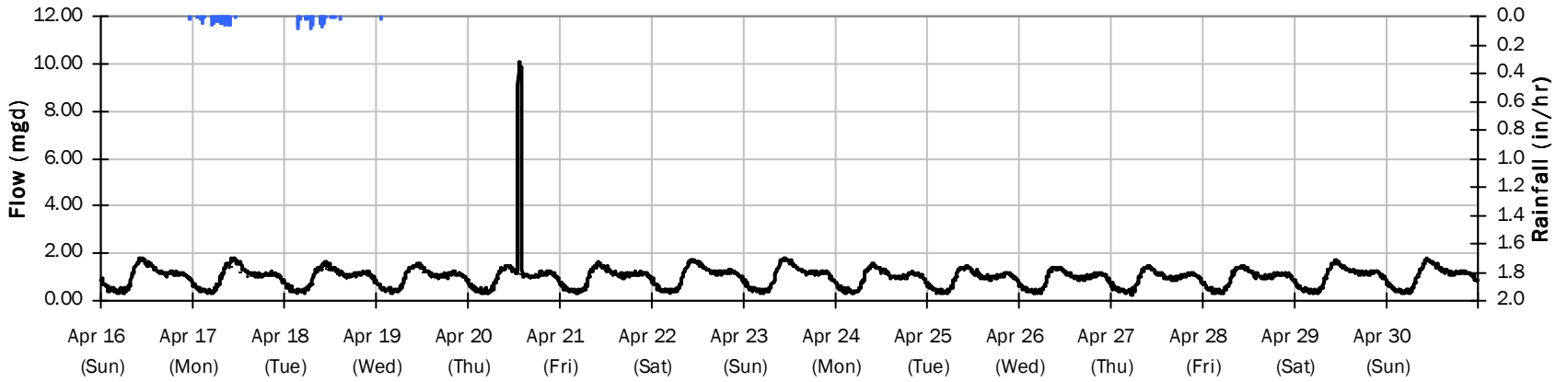


WWTP FLUME

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 1.024 mgd** **Peak Flow: 10.076 mgd** **Min Flow: 0.264 mgd**

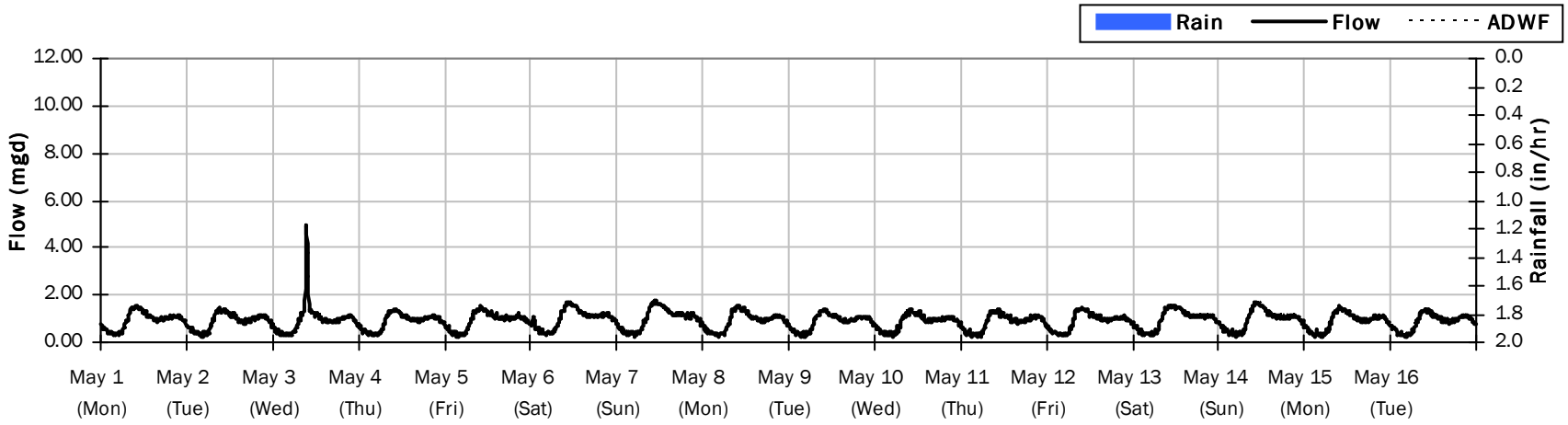


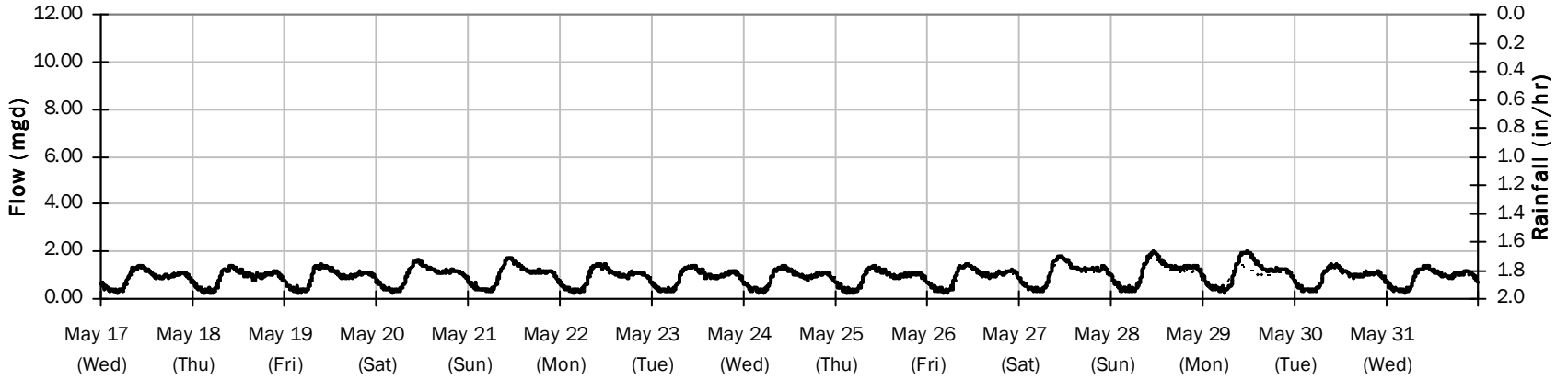


WWTP FLUME

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.933 mgd Peak Flow: 4.888 mgd Min Flow: 0.232 mgd**

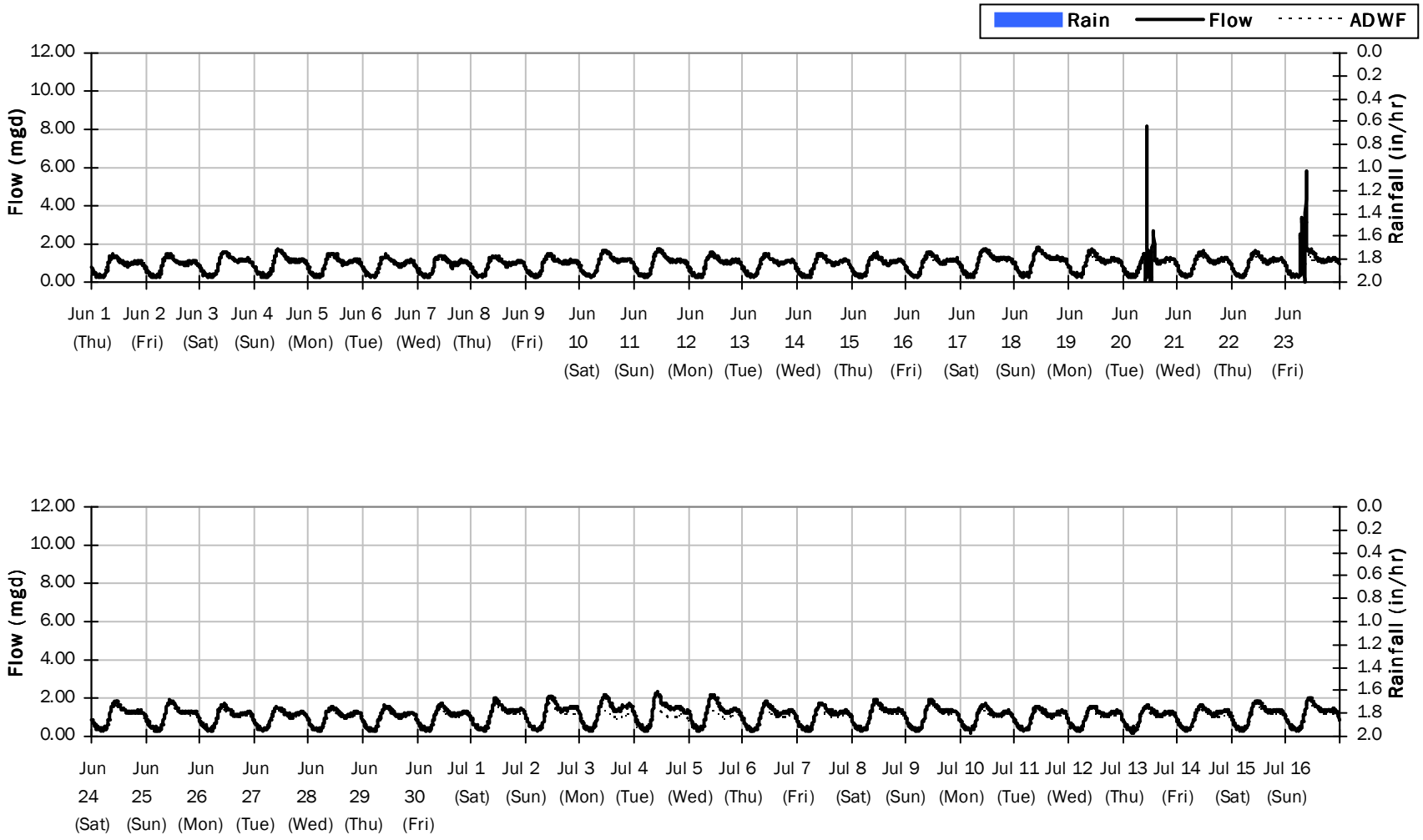




WWTP FLUME

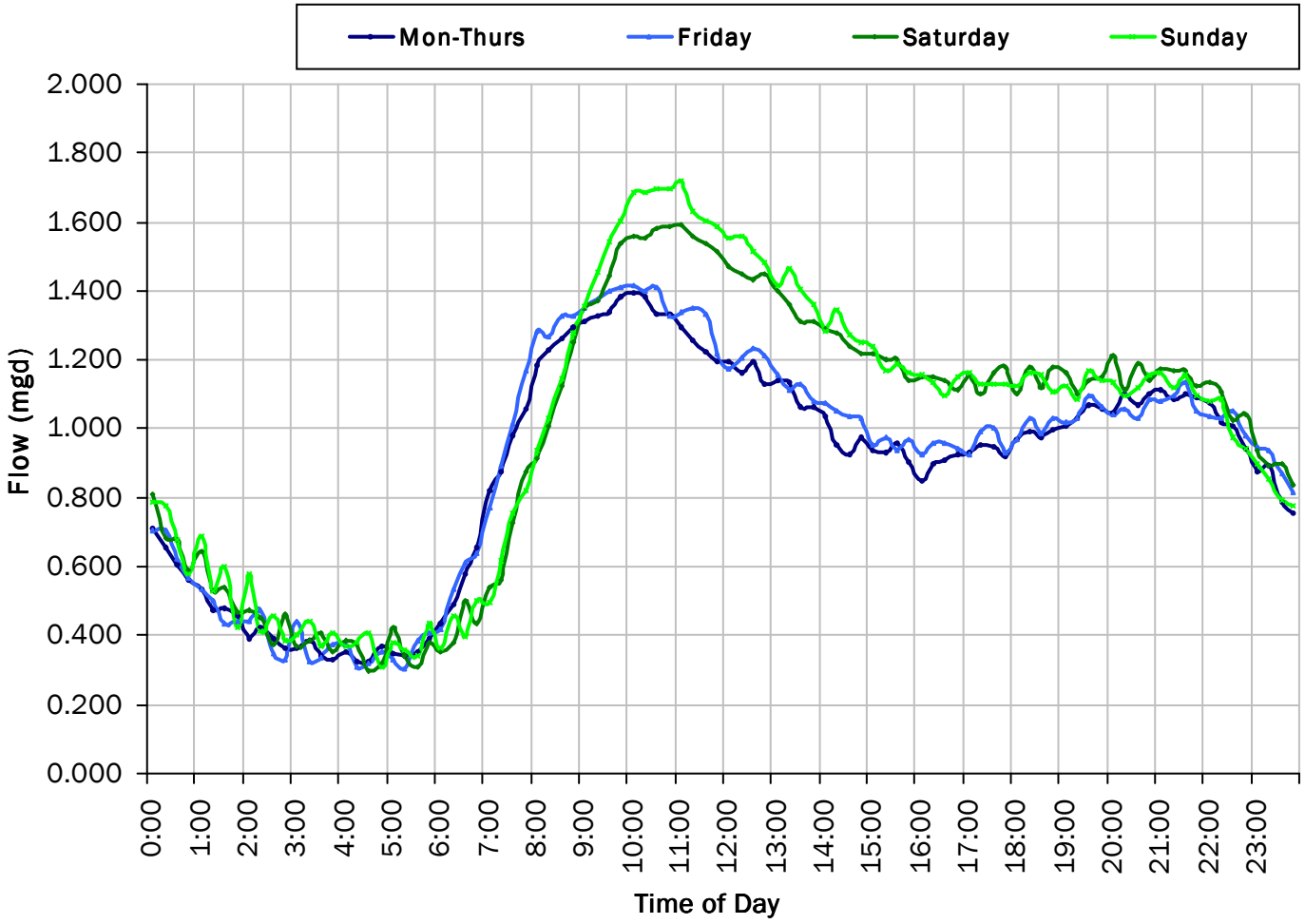
Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 1.033 mgd** **Peak Flow: 8.212 mgd** **Min Flow: 0.035 mgd**



WWTP FLUME

Average Dry Weather Flow Hydrographs

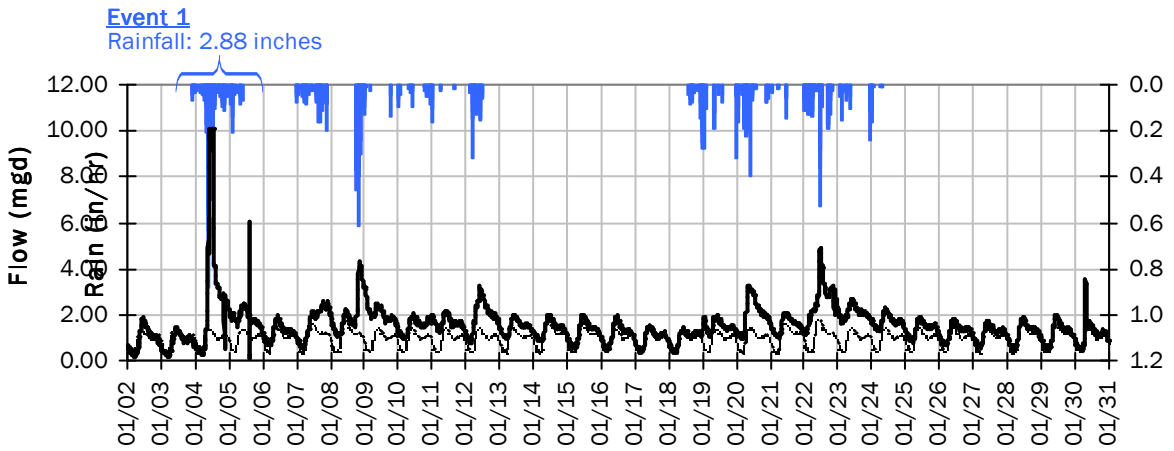


ADWF:

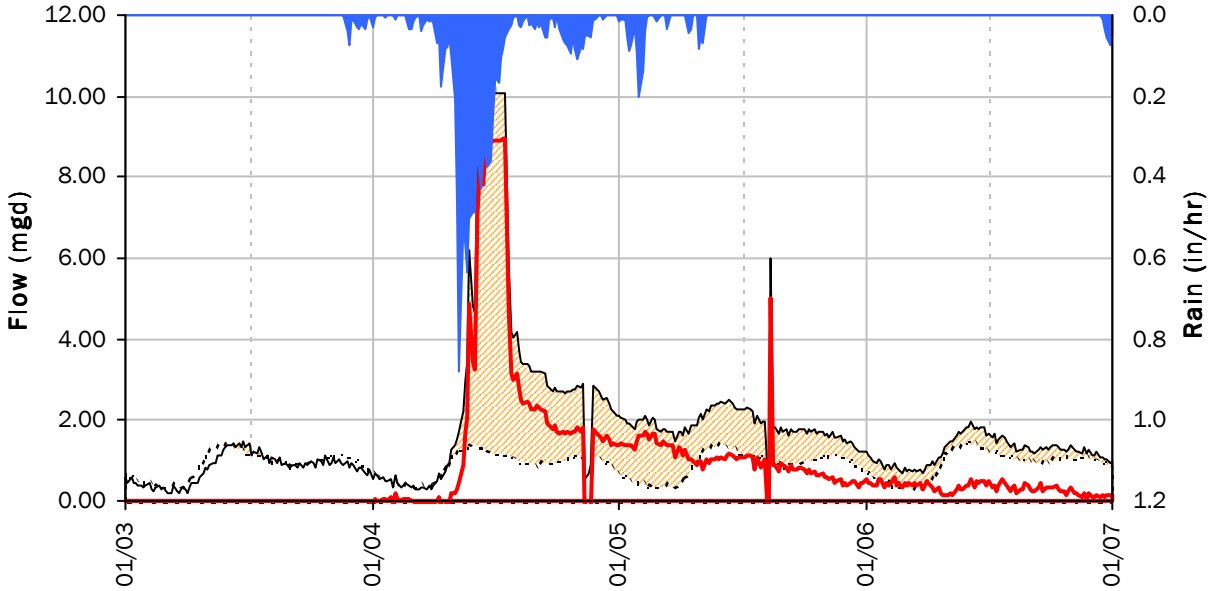
0.920 *mgd*

WWTP FLUME
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

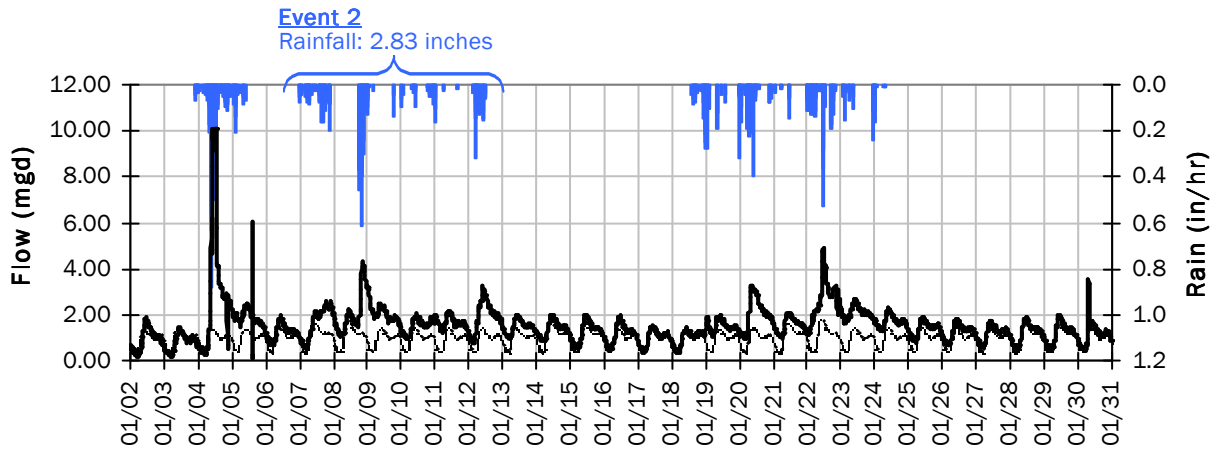


Storm Event I/I Analysis (Rain = 2.88 inches)

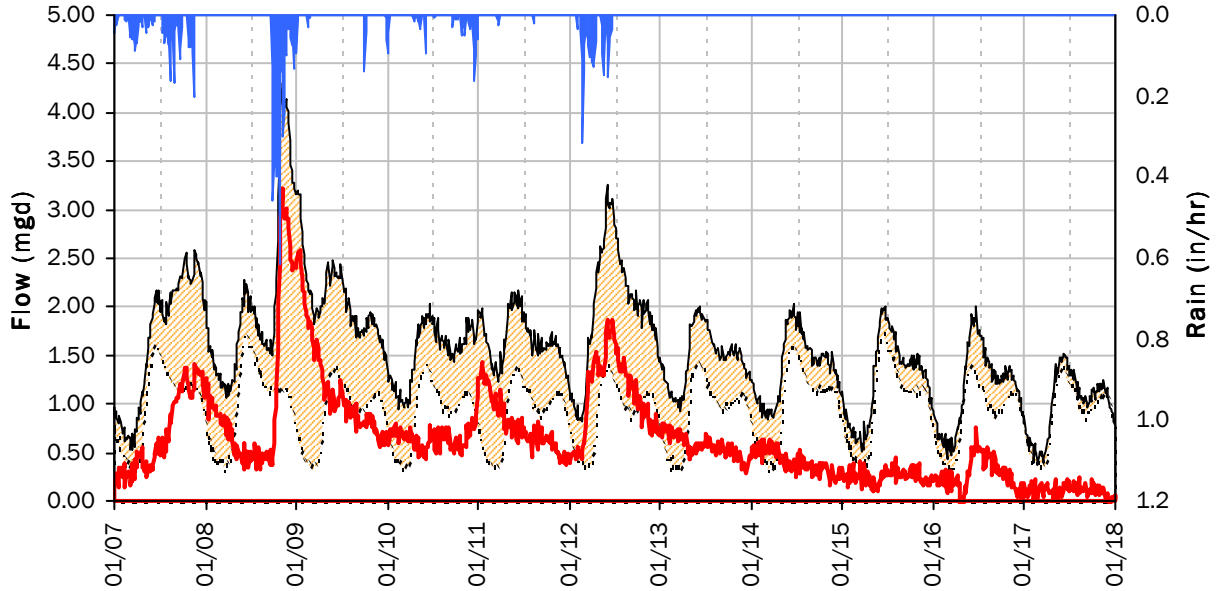
Capacity		Inflow / Infiltration	
Peak Flow:	10.08 *mgd*	**Peak I/I Rate:**	8.94 *mgd*
PF:	10.96	**Total I/I:**	3,447,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

WWTP FLUME
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

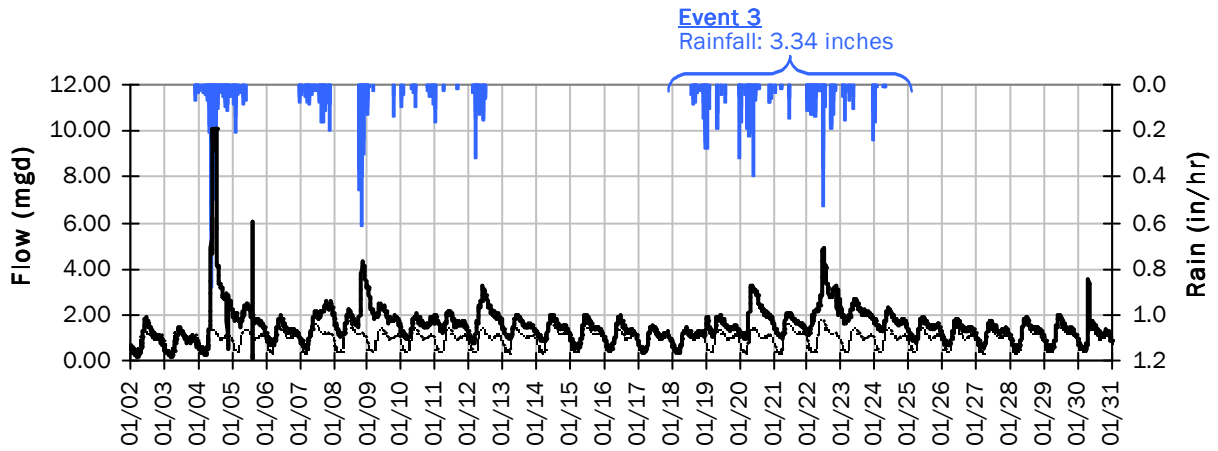


Storm Event I/I Analysis (Rain = 2.83 inches)

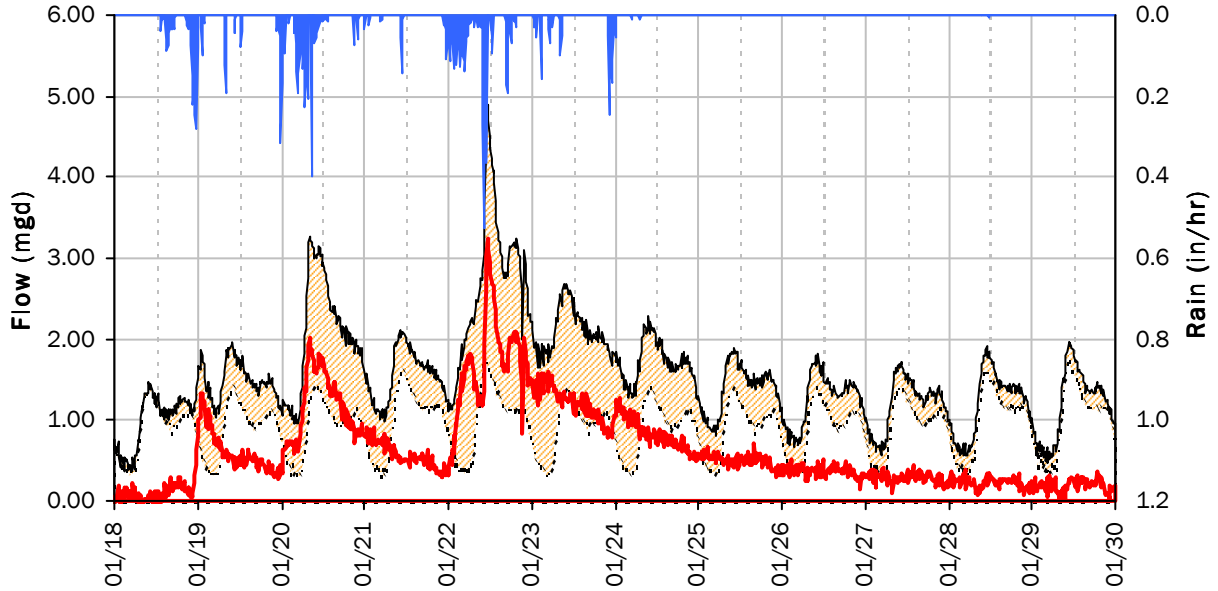
Capacity		Inflow / Infiltration	
Peak Flow:	4.31 *mgd*	**Peak I/I Rate:**	3.21 *mgd*
PF:	4.68	**Total I/I:**	7,080,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

WWTP FLUME
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 3 Detail Graph

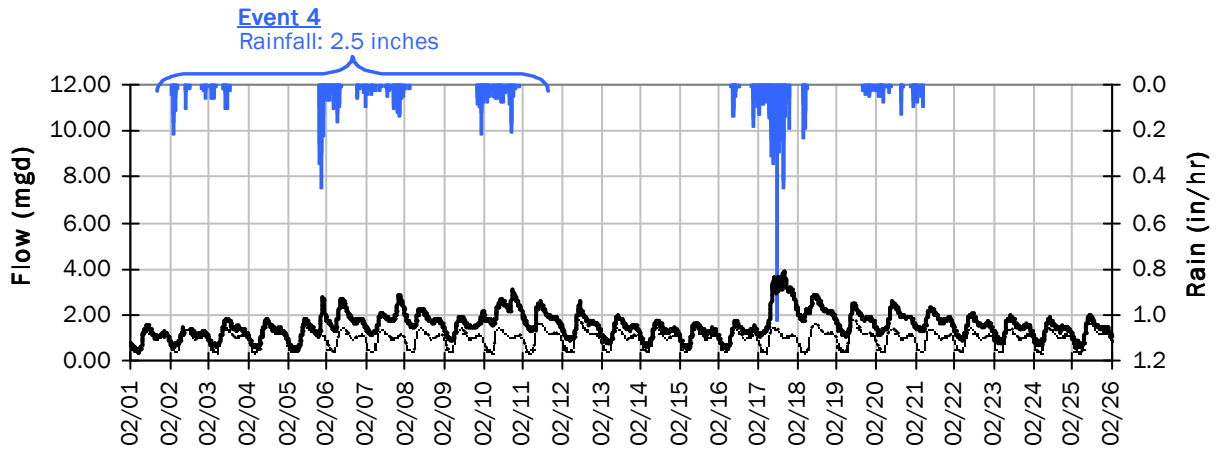


Storm Event I/I Analysis (Rain = 3.35 inches)

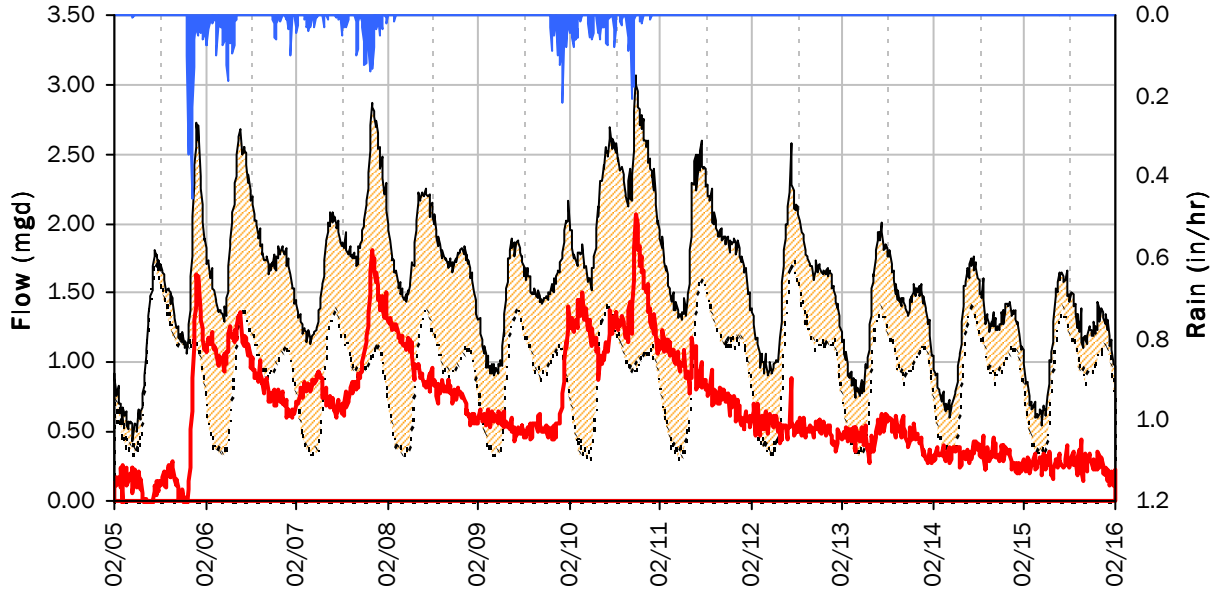
Capacity		Inflow / Infiltration	
Peak Flow:	4.89 *mgd*	**Peak I/I Rate:**	3.26 *mgd*
PF:	5.32	**Total I/I:**	7,778,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

WWTP FLUME
I/I Summary: Event 4

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 4 Detail Graph

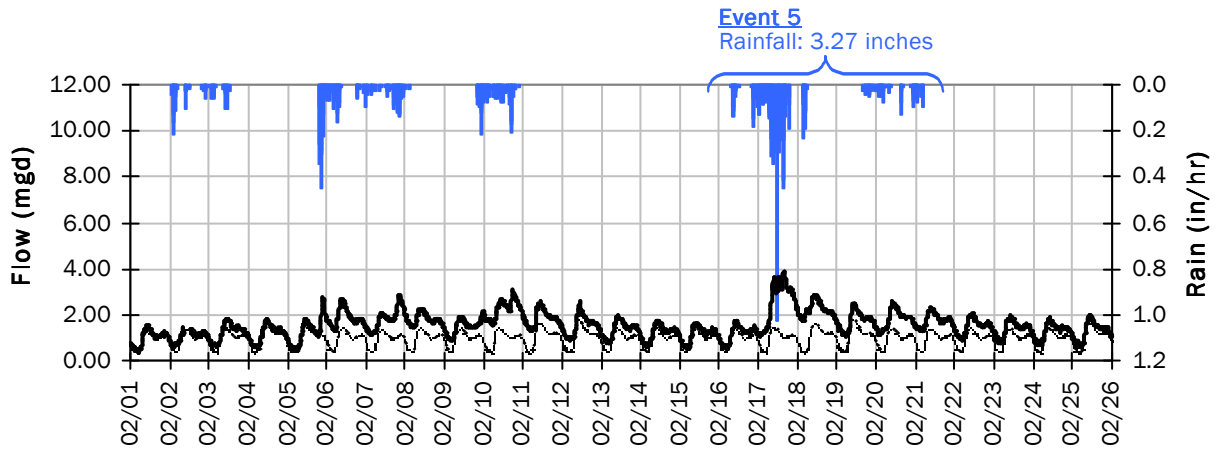


Storm Event I/I Analysis (Rain = 2.50 inches)

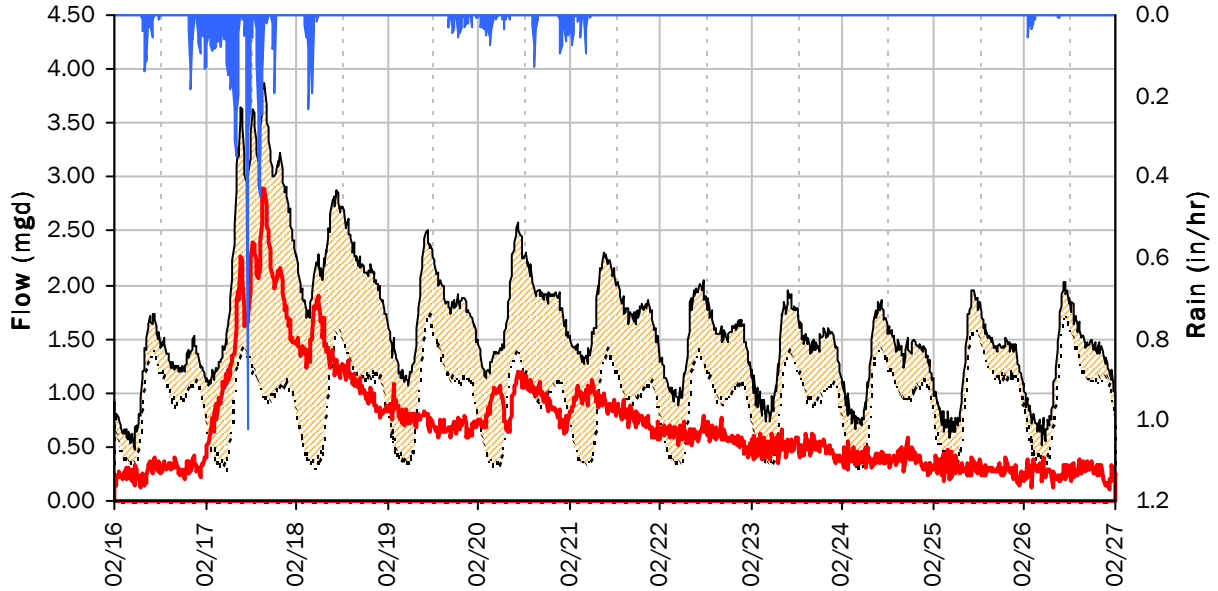
Capacity		Inflow / Infiltration	
Peak Flow:	3.07 *mgd*	**Peak I/I Rate:**	2.06 *mgd*
PF:	3.33	**Total I/I:**	7,472,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

WWTP FLUME
I/I Summary: Event 5

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 5 Detail Graph



Storm Event I/I Analysis (Rain = 3.28 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	3.87 *mgd*	**Peak I/I Rate:**	2.90 *mgd*
PF:	4.21	**Total I/I:**	7,763,000 *gallons*
Peak Level:	*in*		
d/D Ratio:			

City of Morro Bay

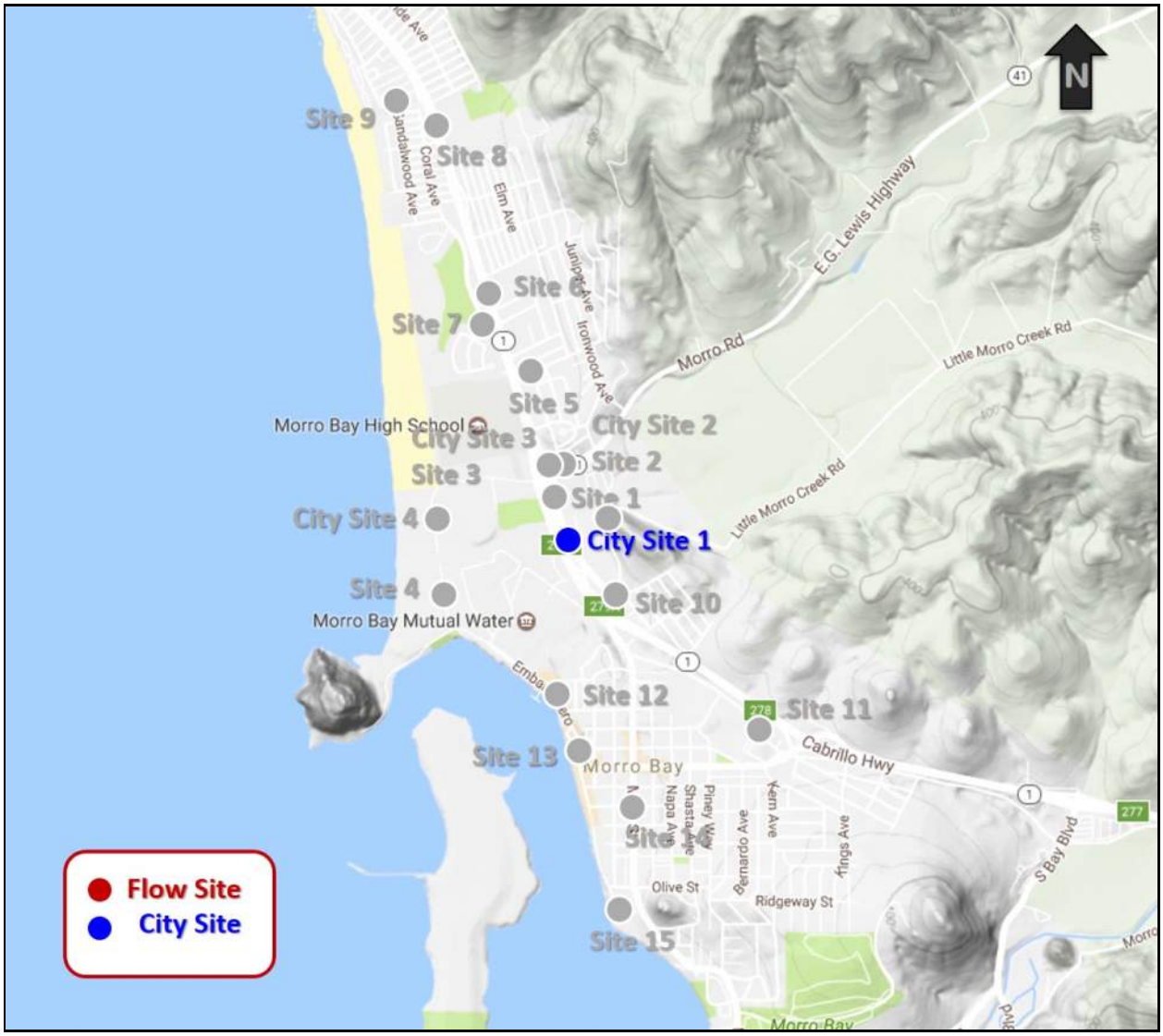
Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 2017 - February 2017

Monitoring Site: MB 1

Location: City Meter at MH 13.36, Main Street ~225 feet south of Errol Street, meter on south 12" inlet

Data Summary Report



Vicinity Map: MB 1

MB 1

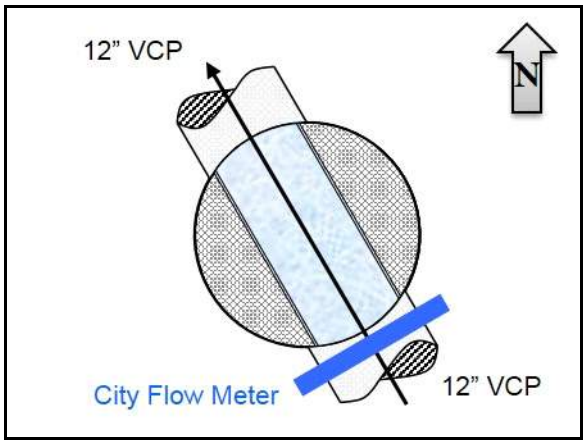
Site Information

Location: City Meter at MH 13.36, Main Street ~225 feet south of Errol Street, meter on south 12" inlet

Pipe Diameter: 12 inches

ADWF: 0.167 mgd

Peak Measured Flow: 0.928 mgd



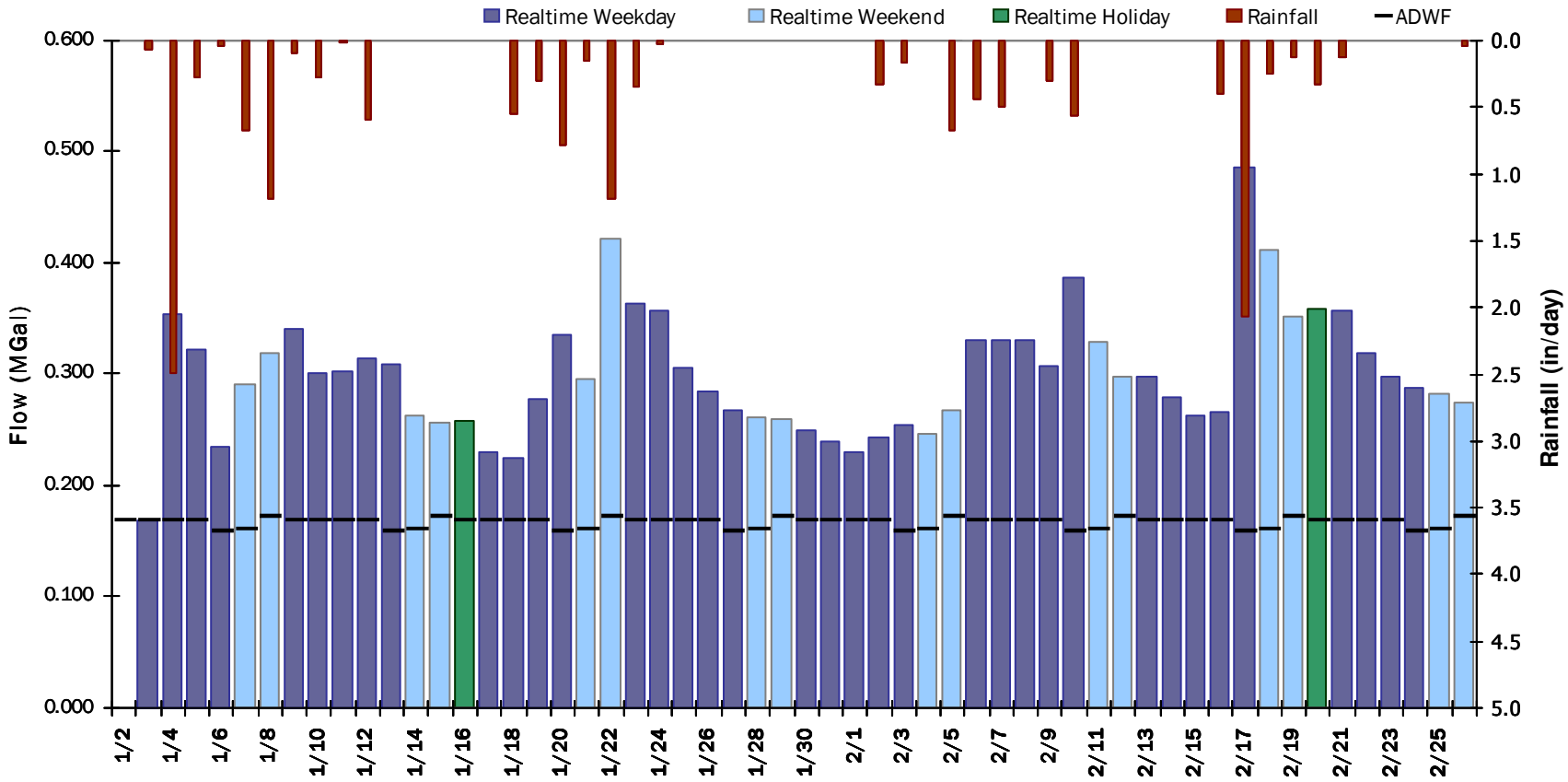
No Photo

MB 1

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.300 MGal **Peak Daily Flow: 0.487 MGal** **Min Daily Flow: 0.169 MGal**

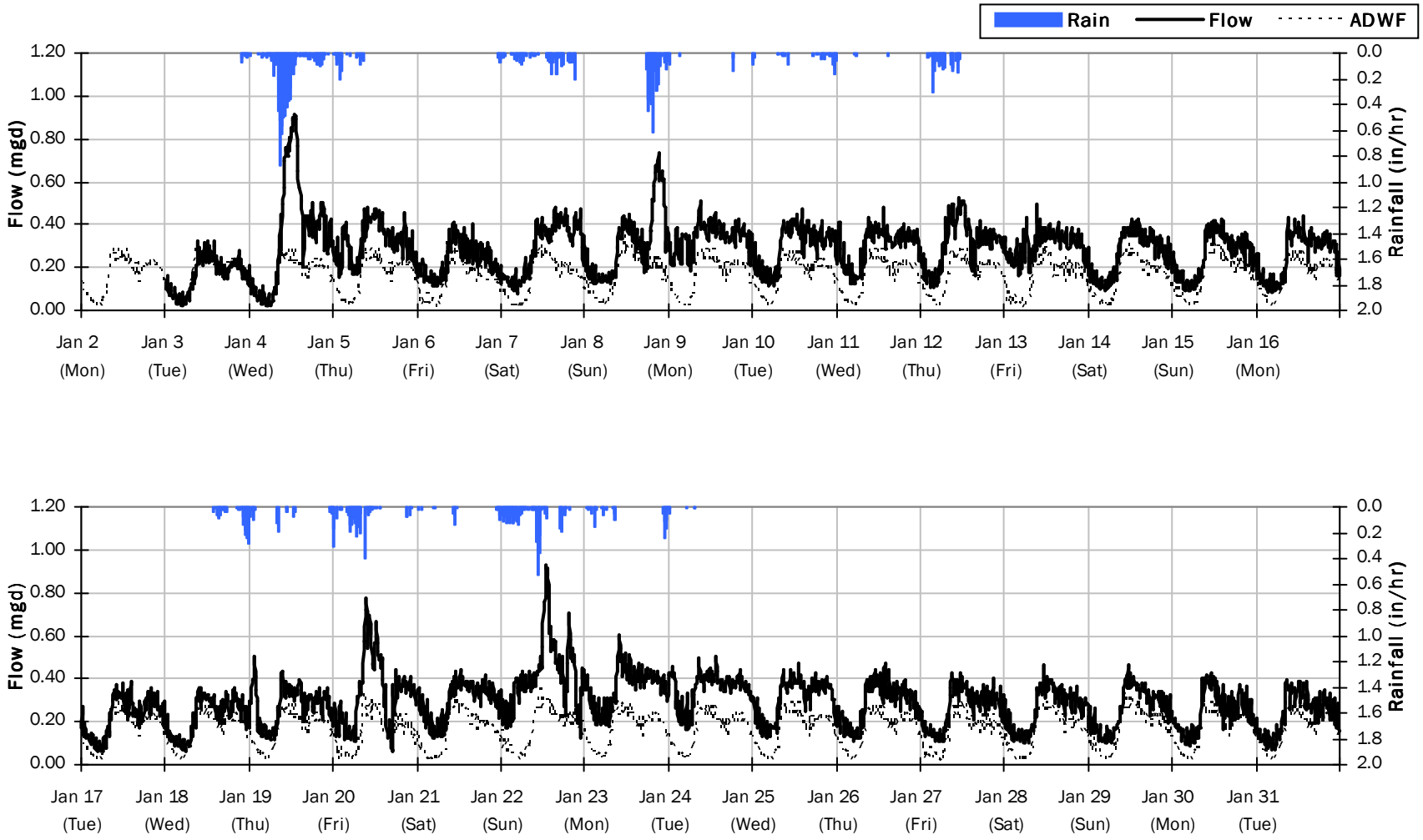
Total Period Rainfall: 15.37 inches



MB 1

Flow Summary: 1/2/2017 to 1/31/2017

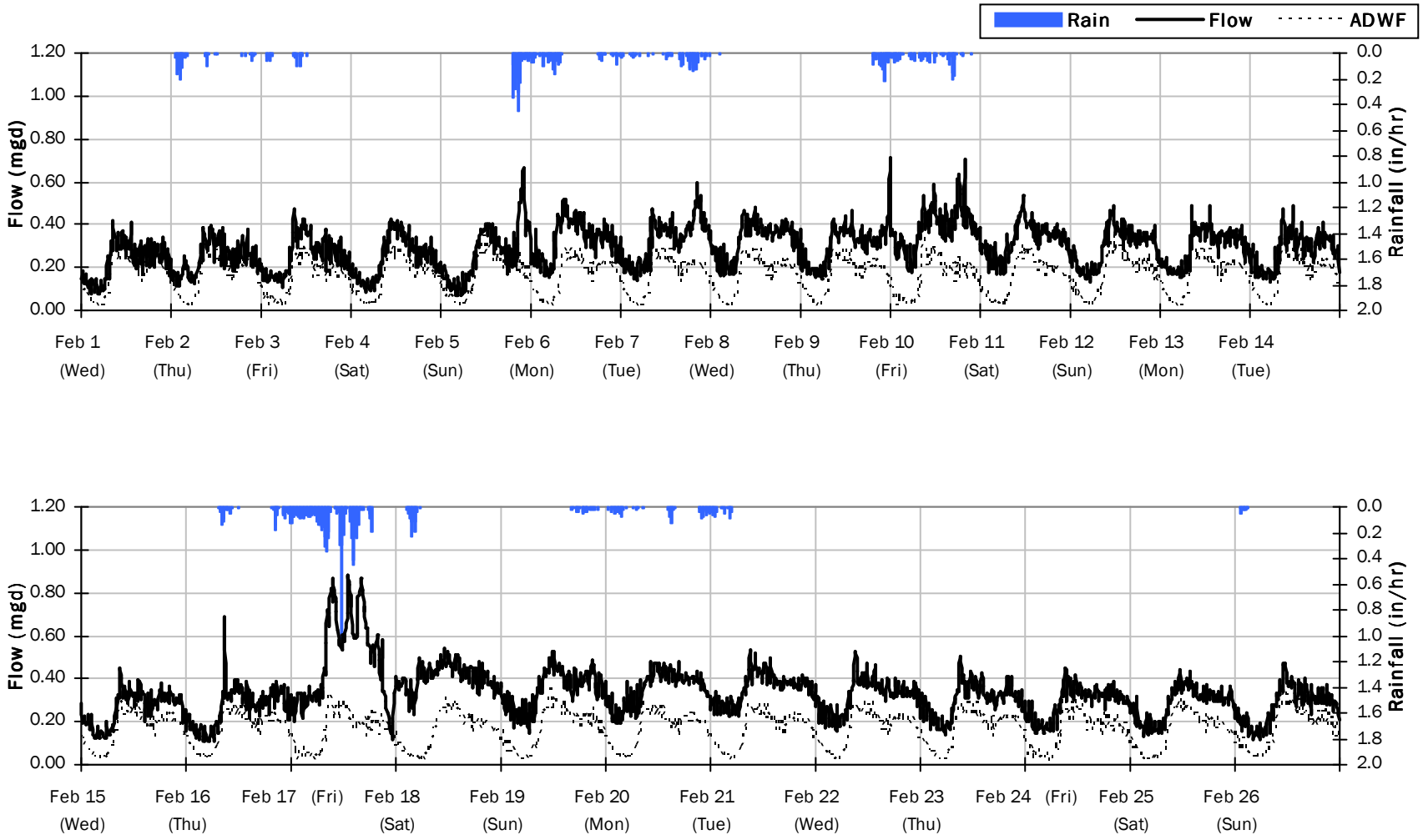
Total Period Rainfall: 9.05 inches **Avg Flow: 0.290 mgd** **Peak Flow: 0.928 mgd** **Min Flow: 0.024 mgd**



MB 1

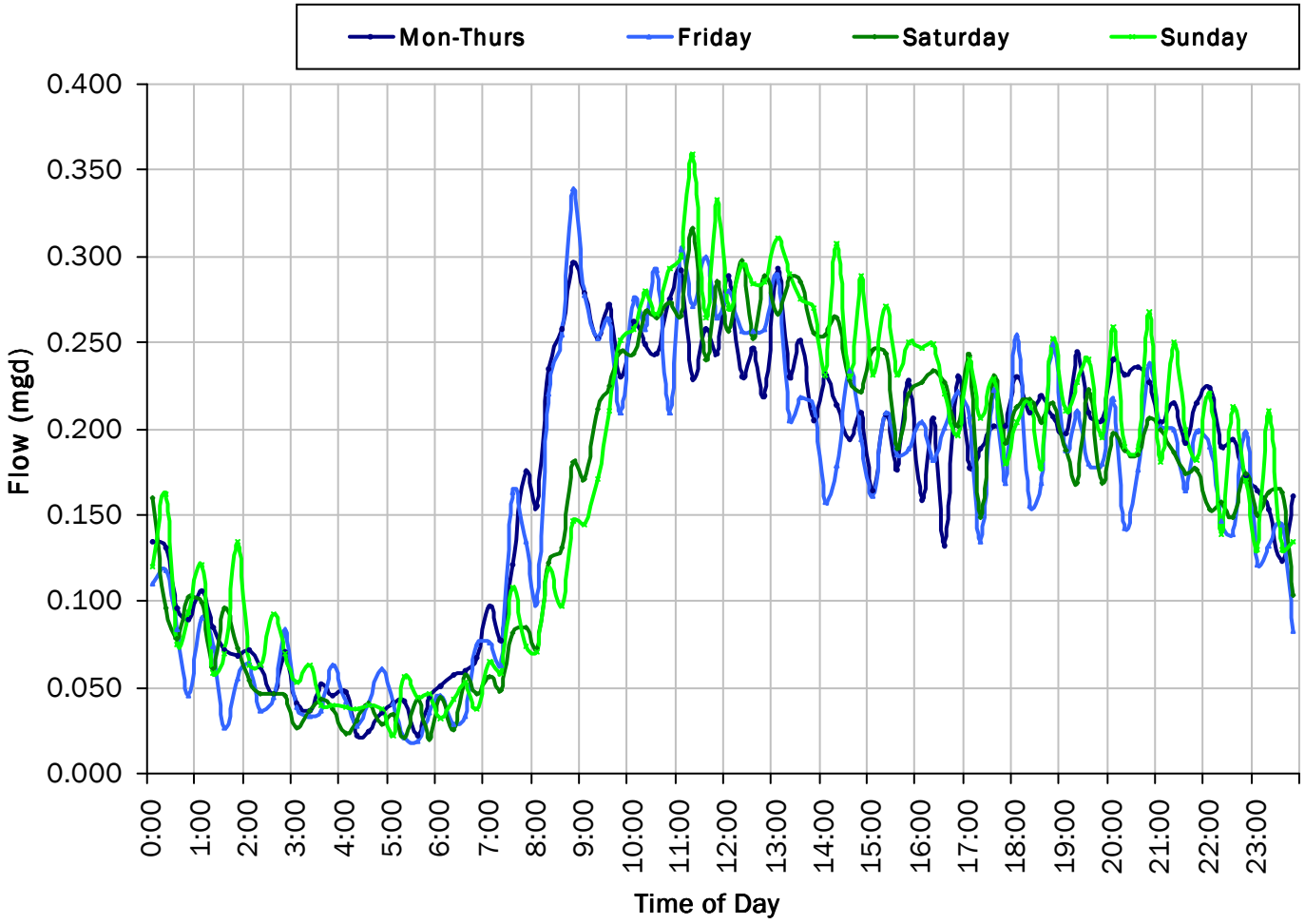
Flow Summary: 2/1/2017 to 2/28/2017

Total Period Rainfall: 6.31 inches **Avg Flow: 0.311 mgd** **Peak Flow: 0.872 mgd** **Min Flow: 0.070 mgd**



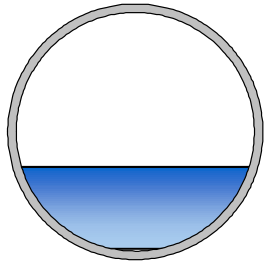
MB 1

Average Dry Weather Flow Hydrographs



ADWF:

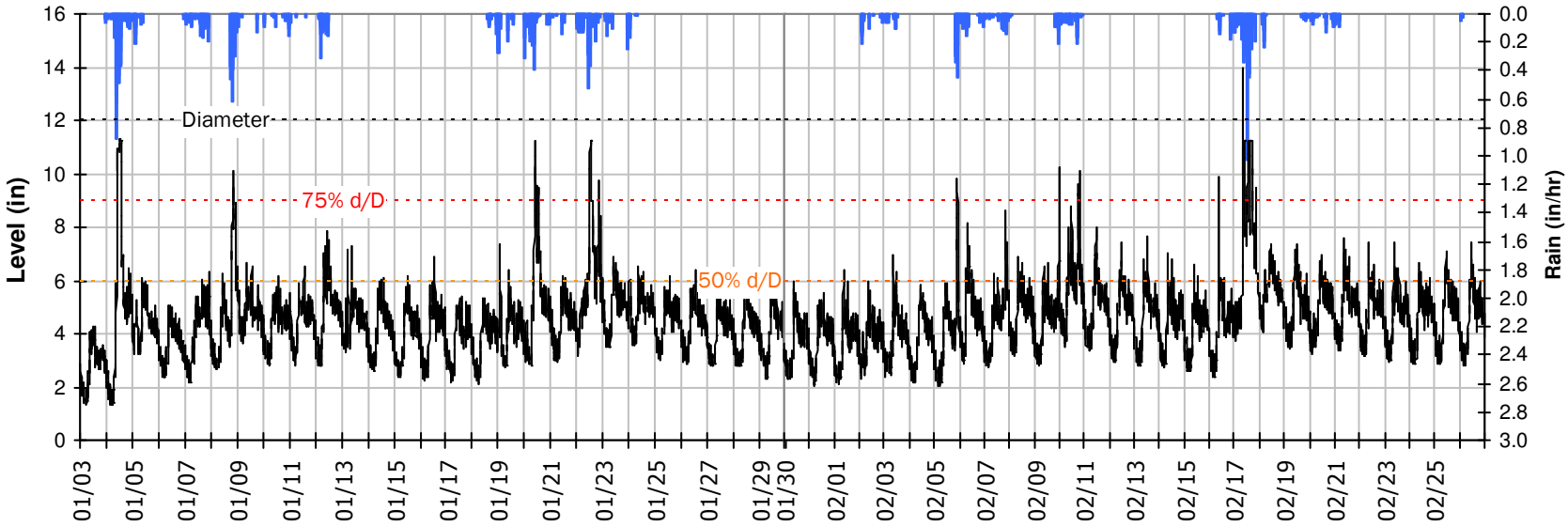
0.167 *mgd*

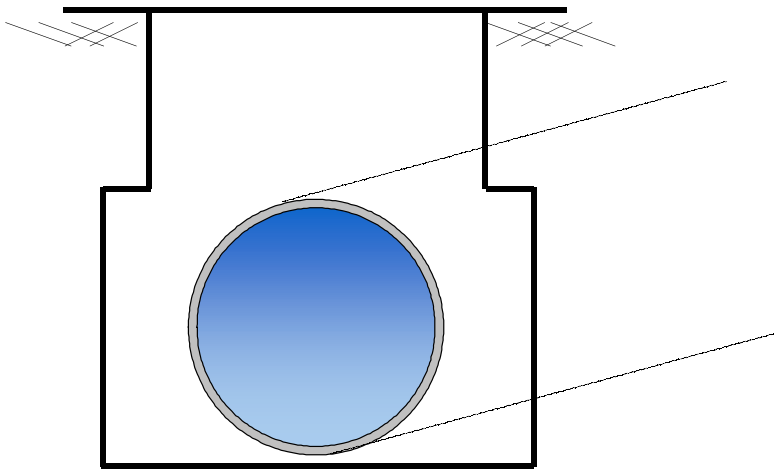


MB 1

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



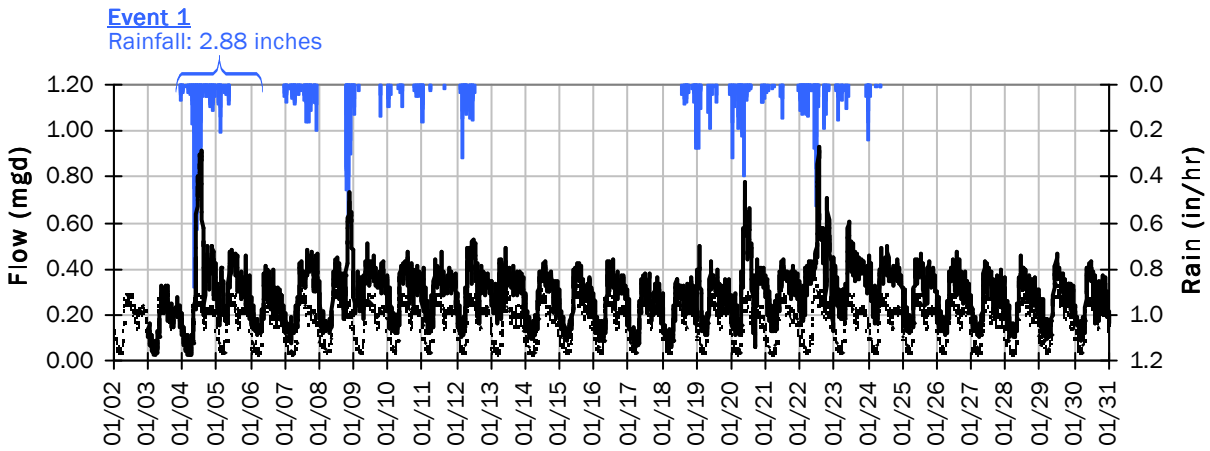


Pipe Diameter:	12	*inches*
Peak Measured Level:	14	*inches*
Peak d/D Ratio:	1.17	

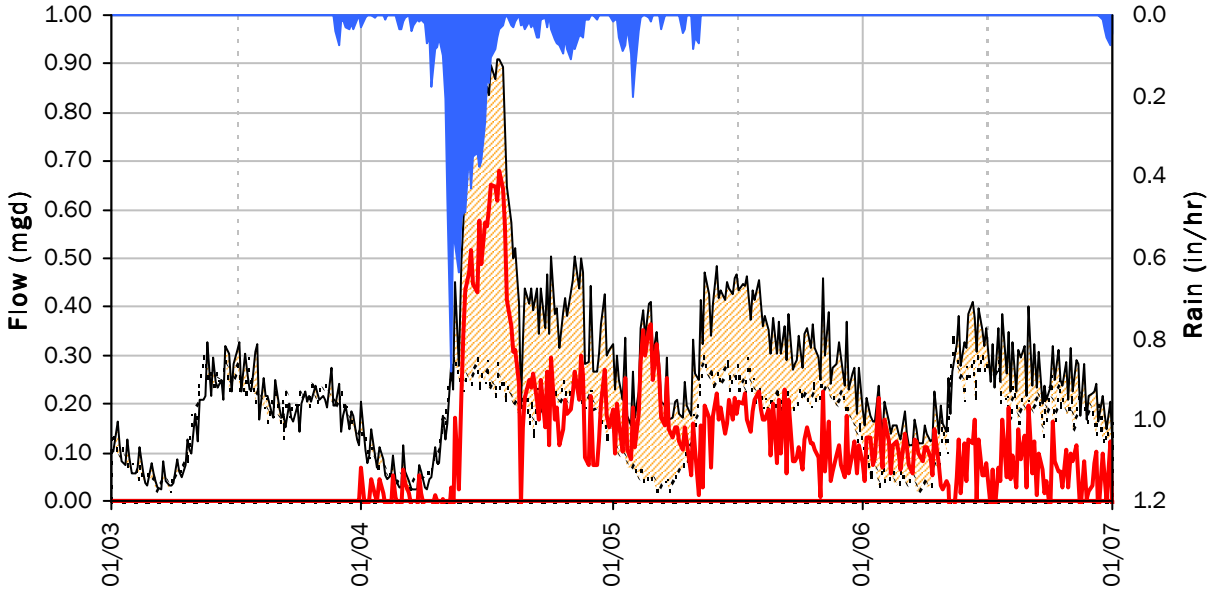
Surcharged 2.0 inches over crown

MB 1
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

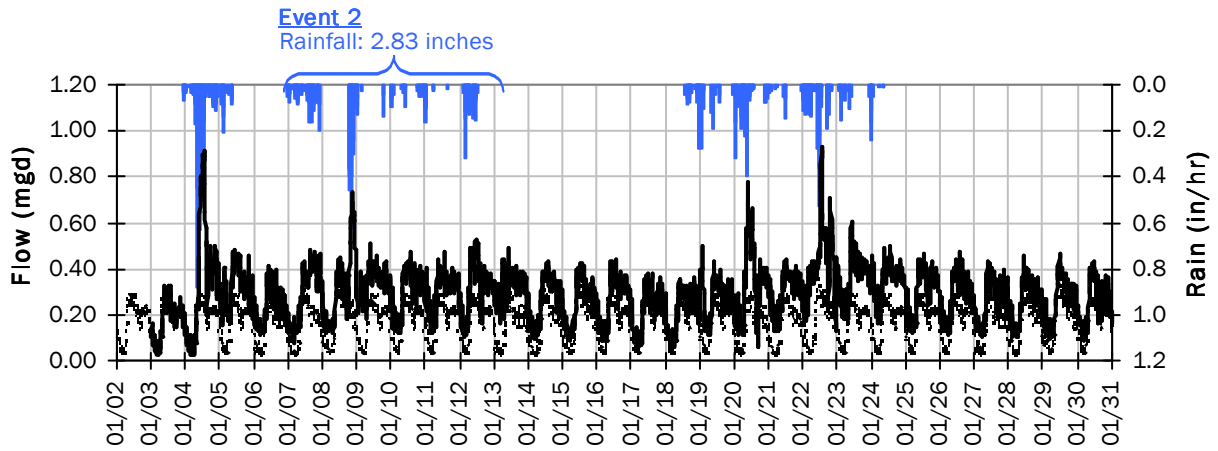


Storm Event I/I Analysis (Rain = 2.88 inches)

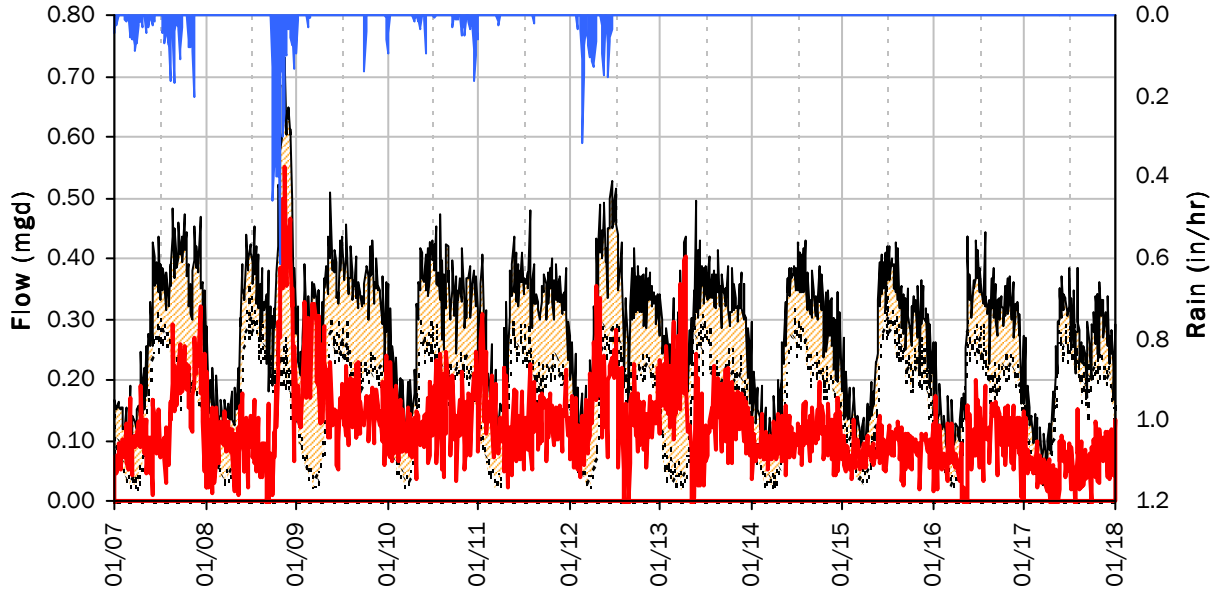
Capacity		Inflow / Infiltration	
Peak Flow:	0.91 *mgd*	**Peak I/I Rate:**	0.68 *mgd*
PF:	5.47	**Total I/I:**	414,000 *gallons*
Peak Level:	11.29 *in*		
d/D Ratio:	0.94		

MB 1
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

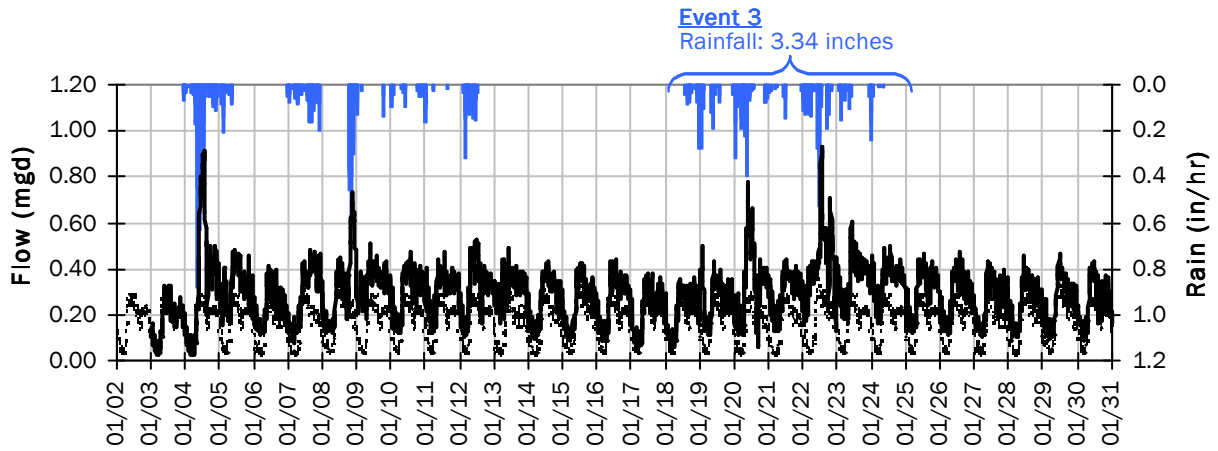


Storm Event I/I Analysis (Rain = 2.83 inches)

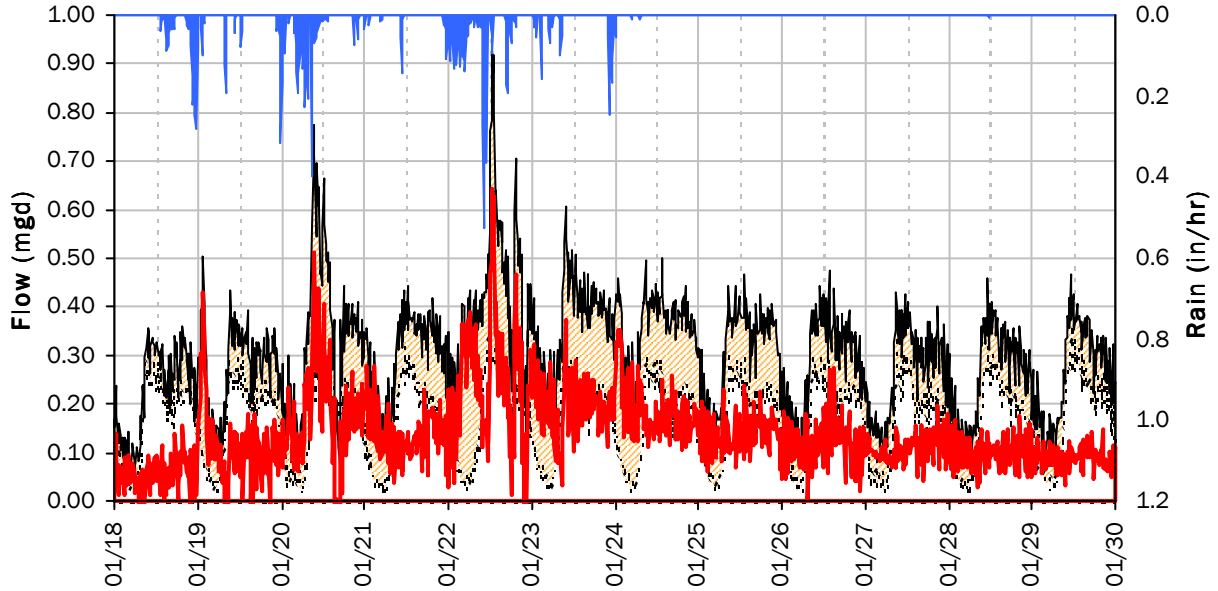
Capacity		Inflow / Infiltration	
Peak Flow:	0.73 *mgd*	**Peak I/I Rate:**	0.55 *mgd*
PF:	4.40	**Total I/I:**	1,345,000 *gallons*
Peak Level:	10.14 *in*		
d/D Ratio:	0.85		

MB 1
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 3 Detail Graph

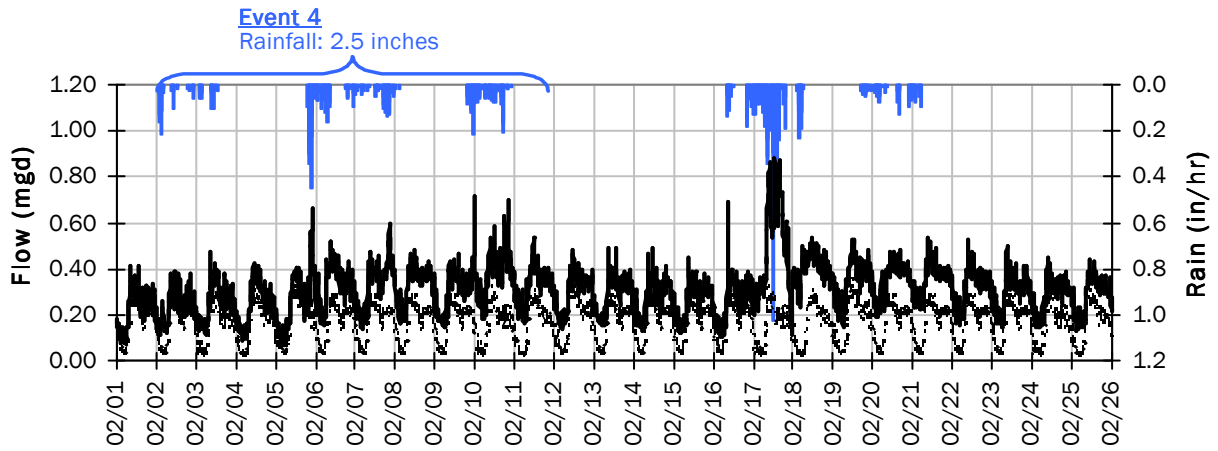


Storm Event I/I Analysis (Rain = 3.35 inches)

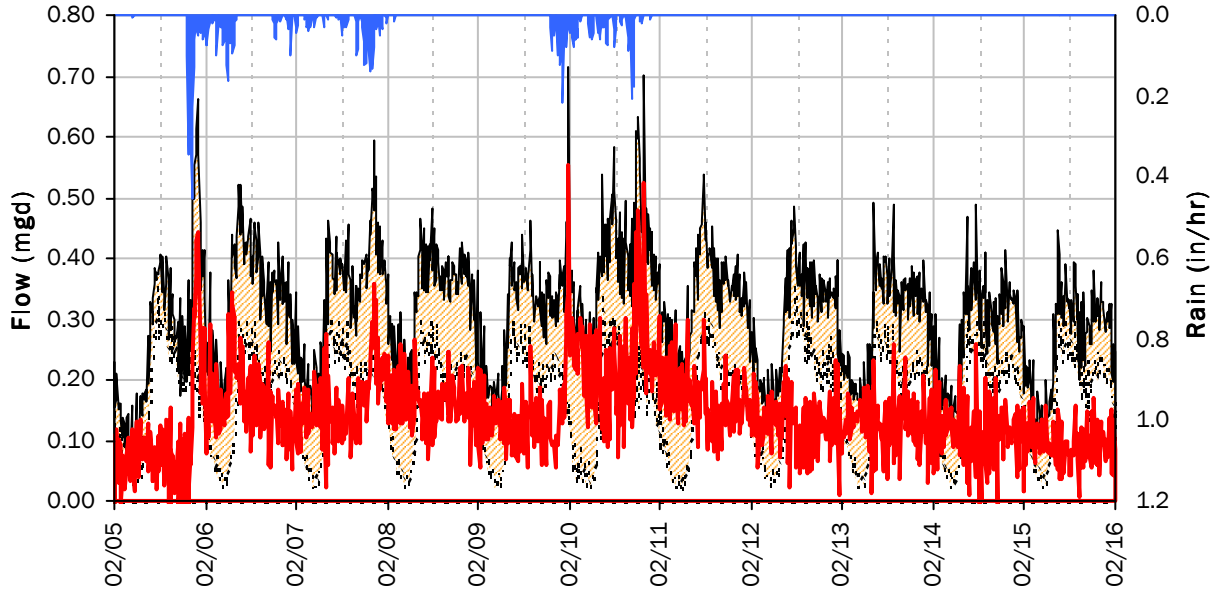
Capacity		Inflow / Infiltration	
Peak Flow:	0.93 *mgd*	**Peak I/I Rate:**	0.64 *mgd*
PF:	5.57	**Total I/I:**	1,662,000 *gallons*
Peak Level:	11.26 *in*		
d/D Ratio:	0.94		

MB 1
I/I Summary: Event 4

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 4 Detail Graph

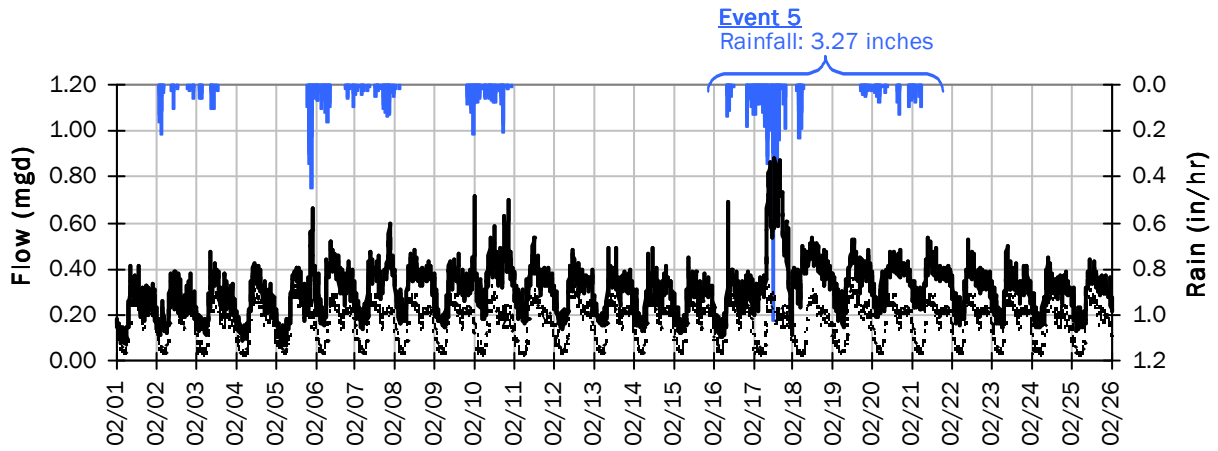


Storm Event I/I Analysis (Rain = 2.50 inches)

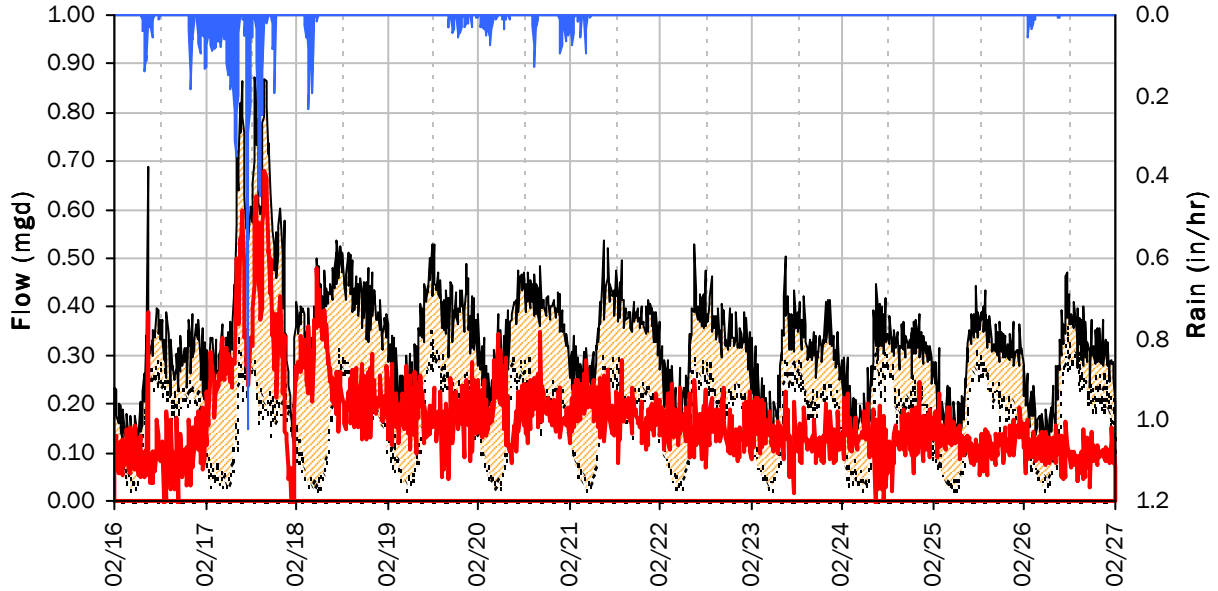
Capacity		Inflow / Infiltration	
Peak Flow:	0.71 *mgd*	**Peak I/I Rate:**	0.55 *mgd*
PF:	4.29	**Total I/I:**	1,577,000 *gallons*
Peak Level:	10.27 *in*		
d/D Ratio:	0.86		

MB 1
I/I Summary: Event 5

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 5 Detail Graph



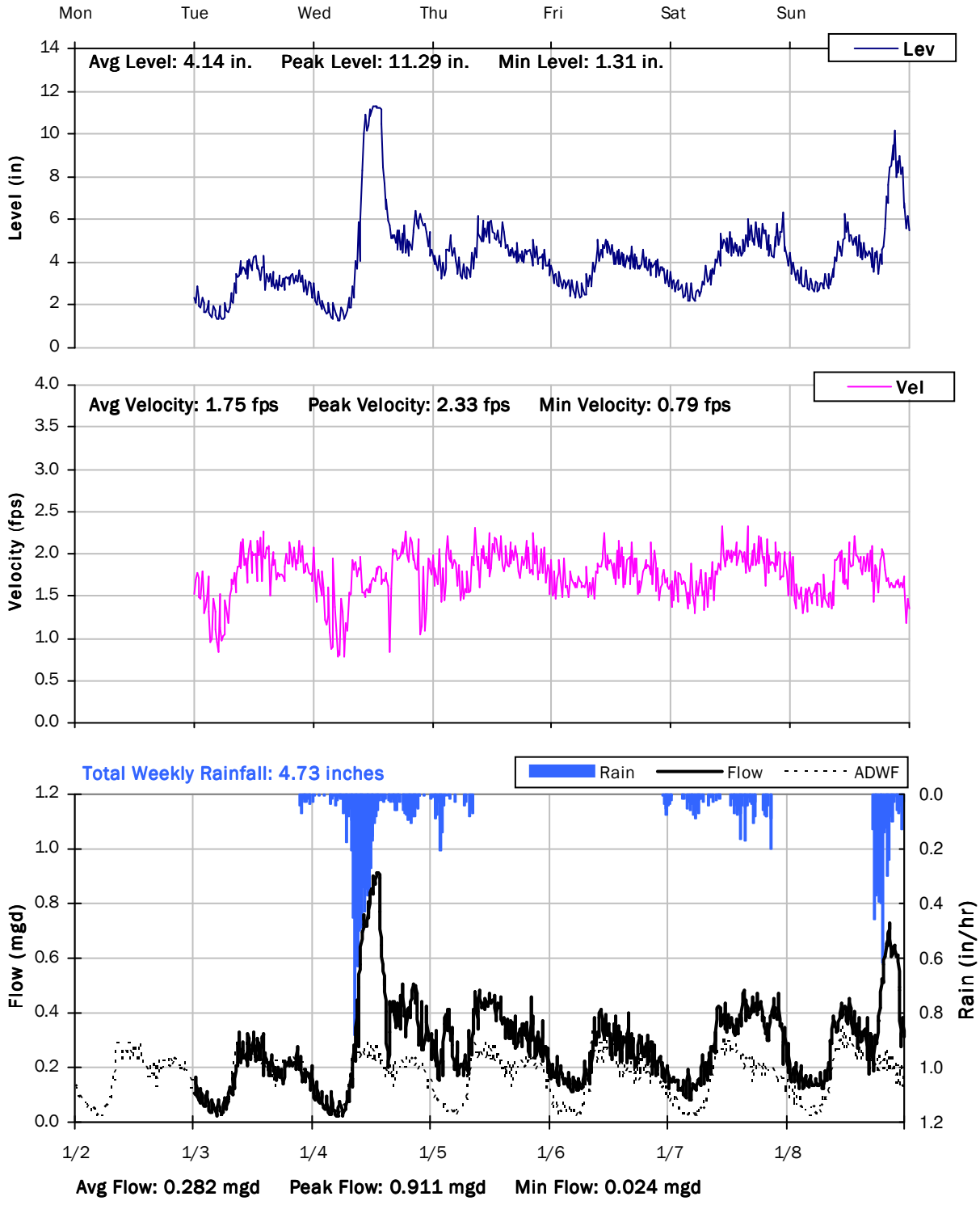
Storm Event I/I Analysis (Rain = 3.28 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.87 *mgd*	**Peak I/I Rate:**	0.68 *mgd*
PF:	5.23	**Total I/I:**	1,864,000 *gallons*
Peak Level:	14.00 *in*		
d/D Ratio:	1.17		

MB 1

Weekly Level, Velocity and Flow Hydrographs

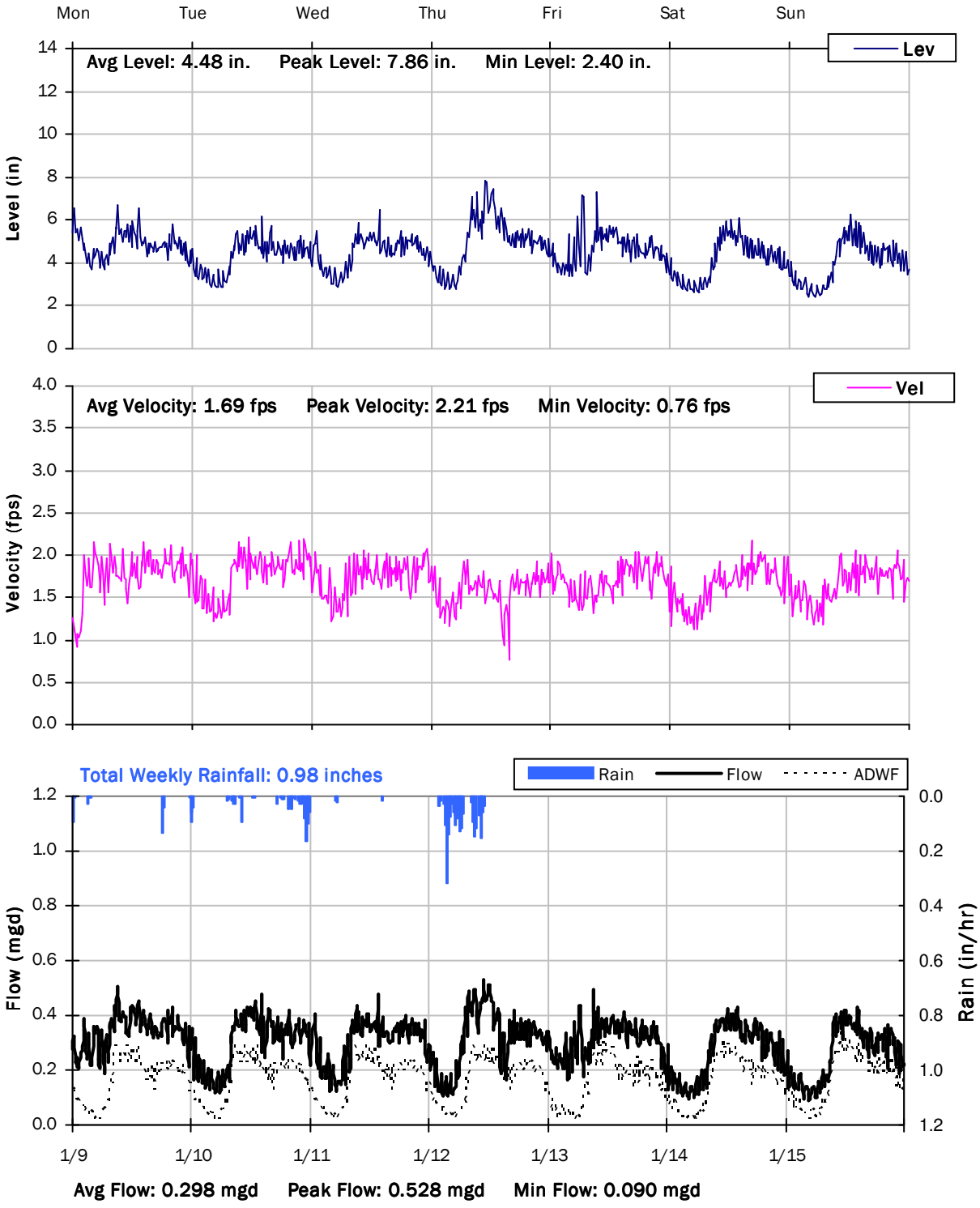
1/2/2017 to 1/9/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

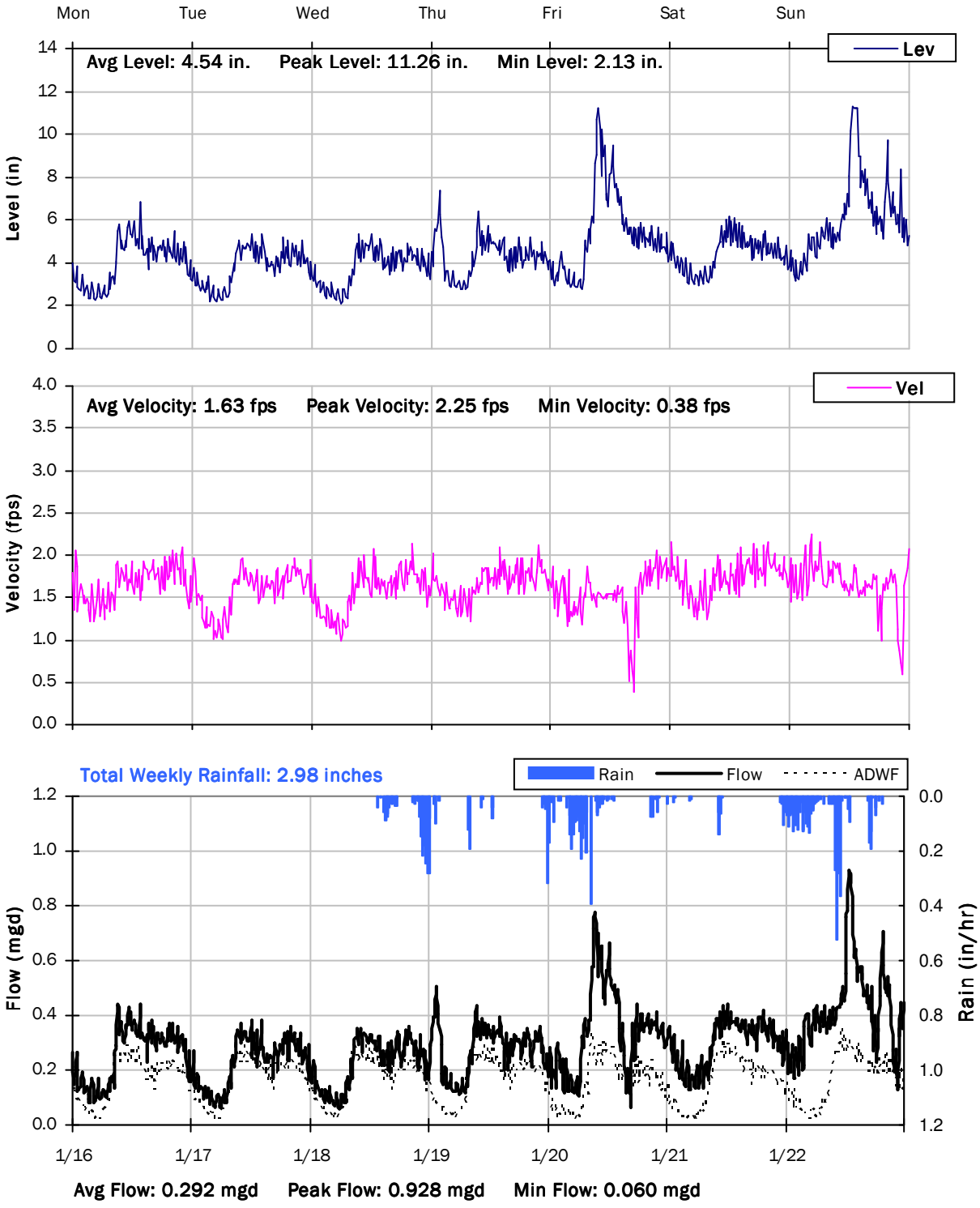
1/9/2017 to 1/16/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

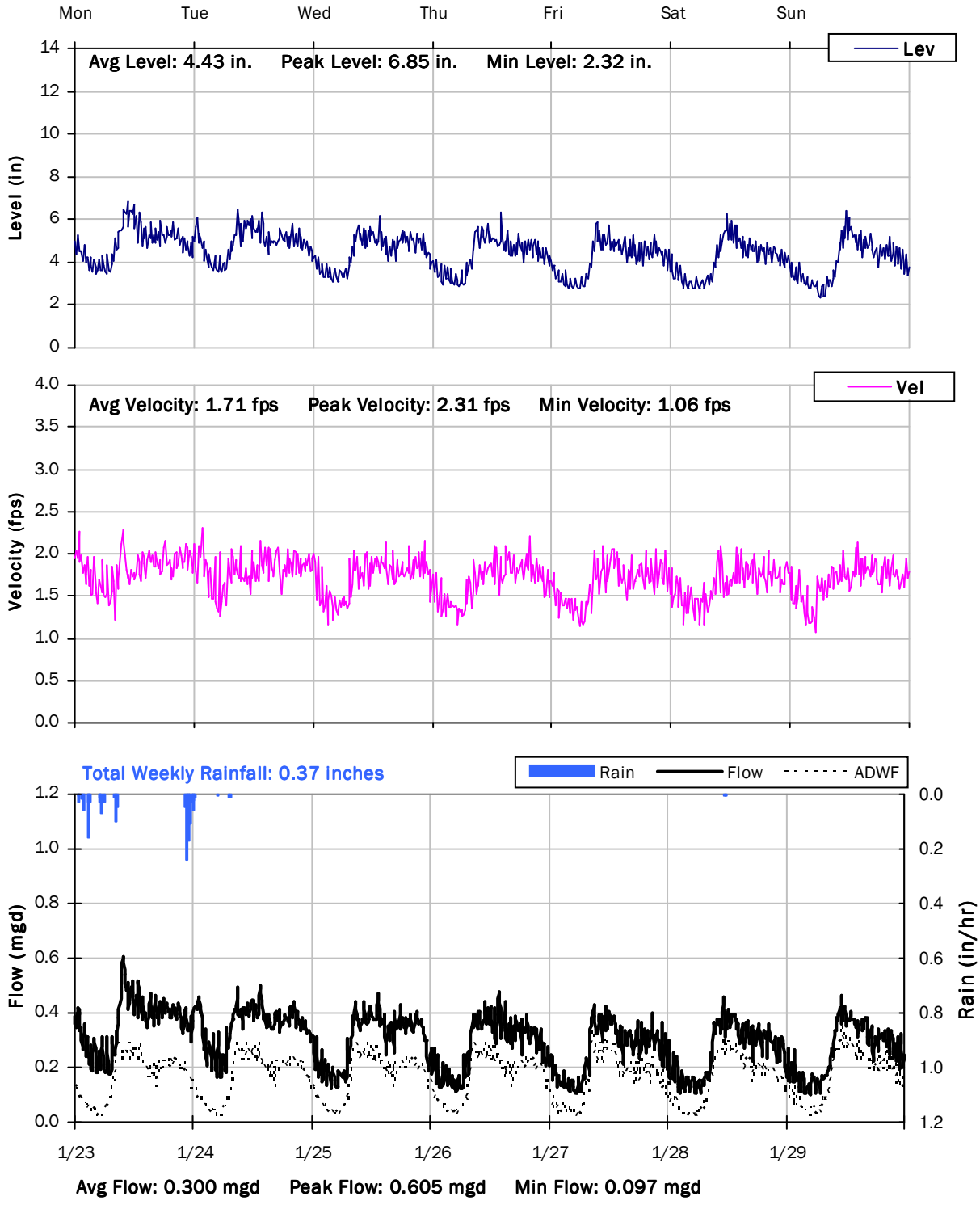
1/16/2017 to 1/23/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

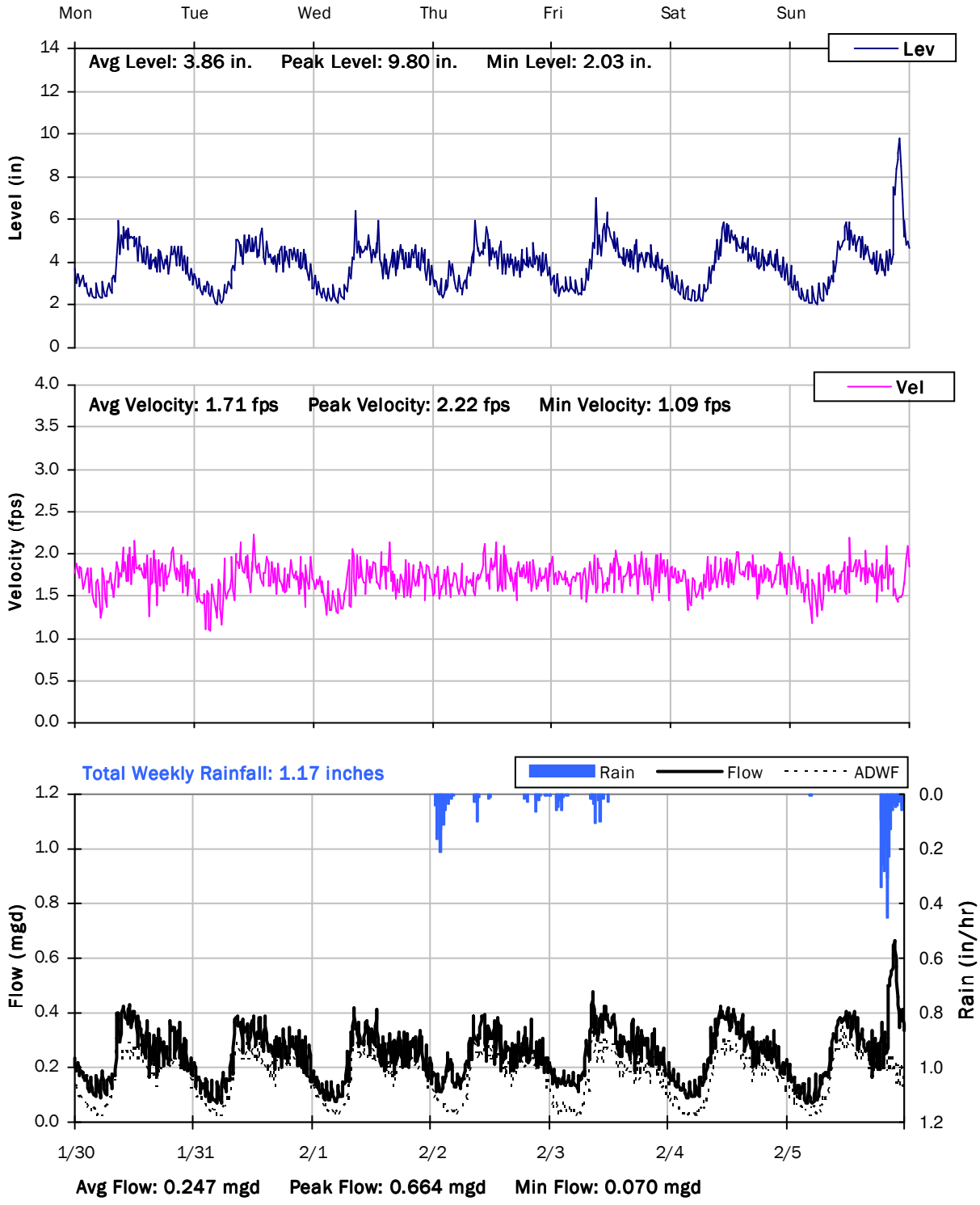
1/23/2017 to 1/30/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

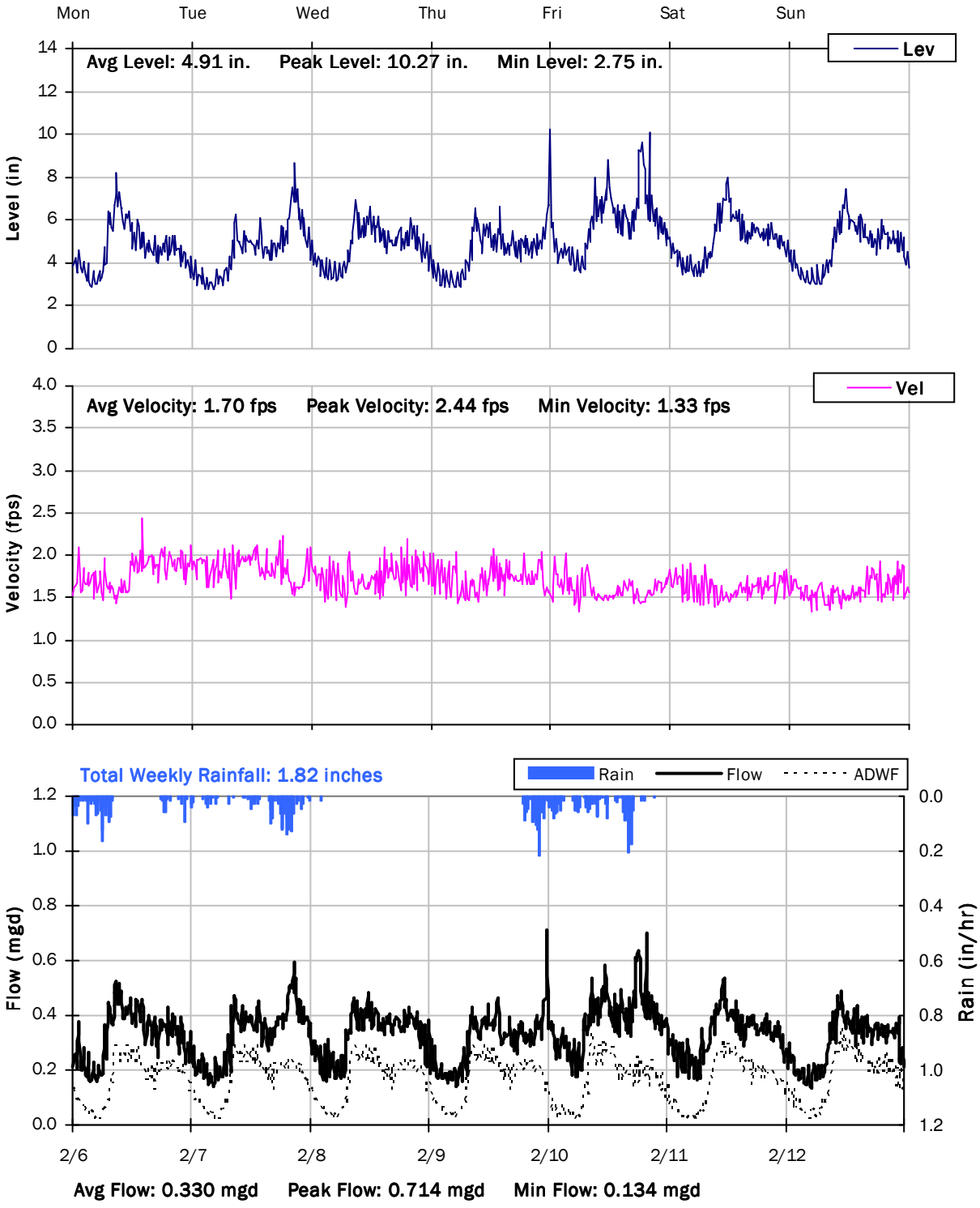
1/30/2017 to 2/6/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

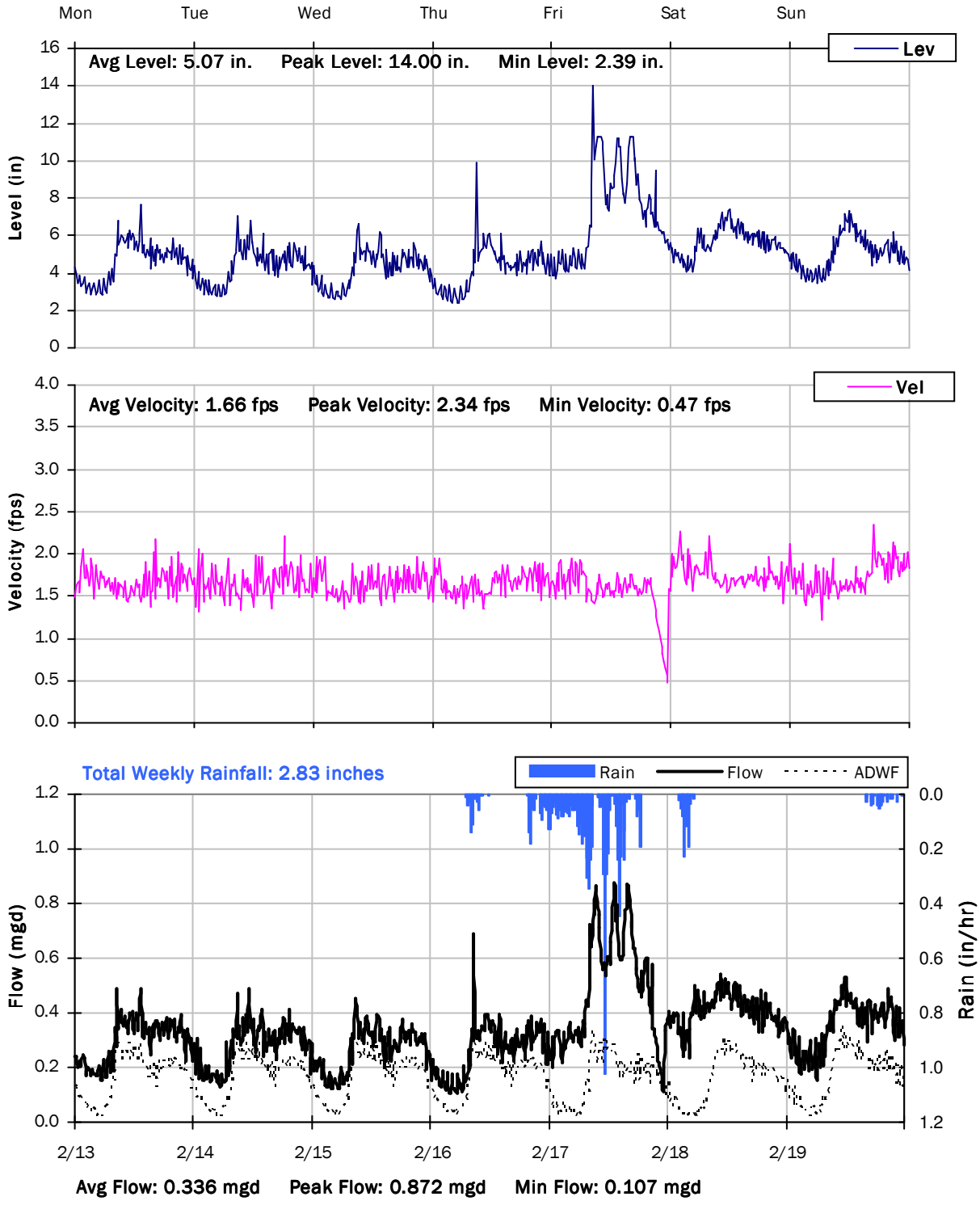
2/6/2017 to 2/13/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

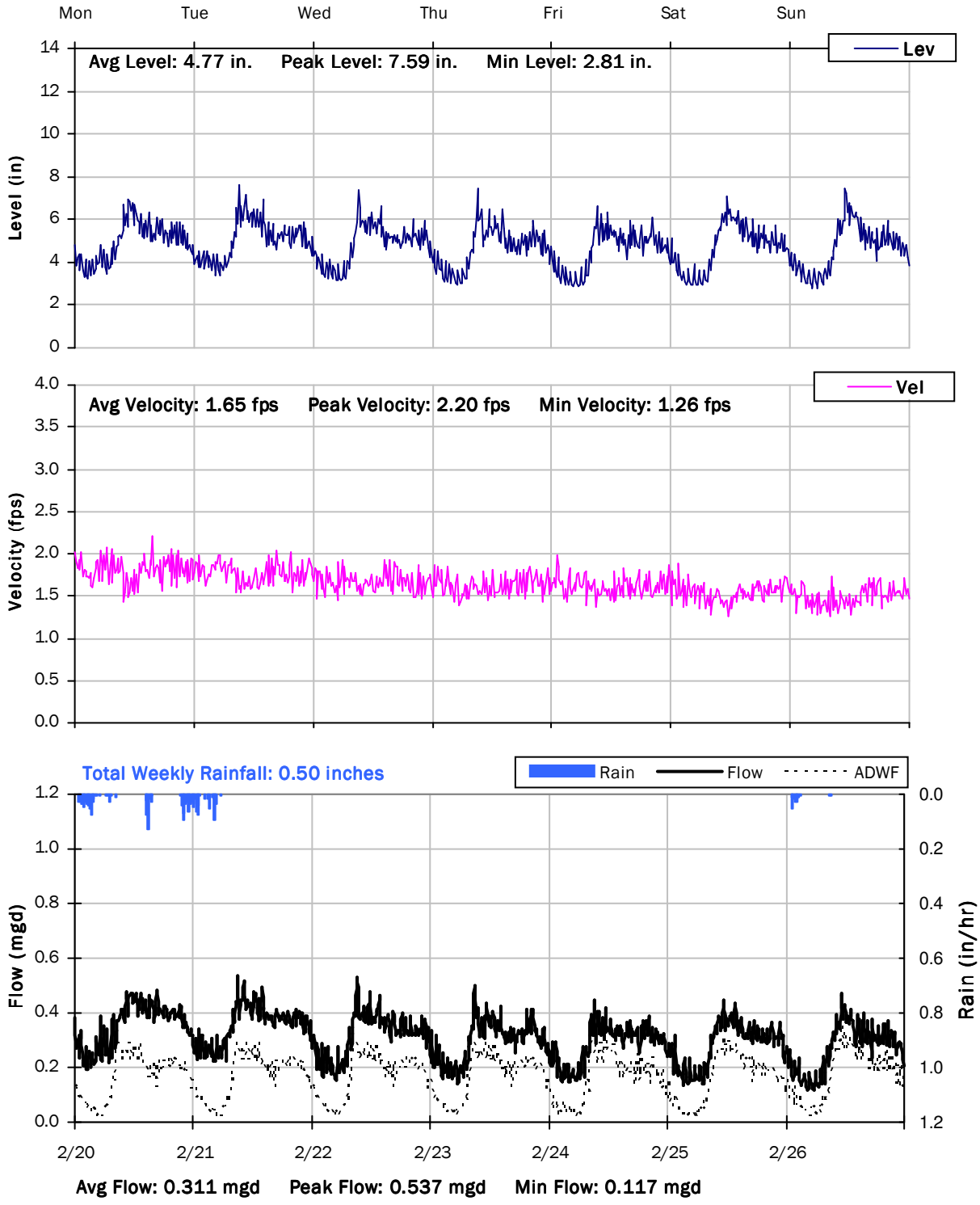
2/13/2017 to 2/20/2017



MB 1

Weekly Level, Velocity and Flow Hydrographs

2/20/2017 to 2/27/2017



City of Morro Bay

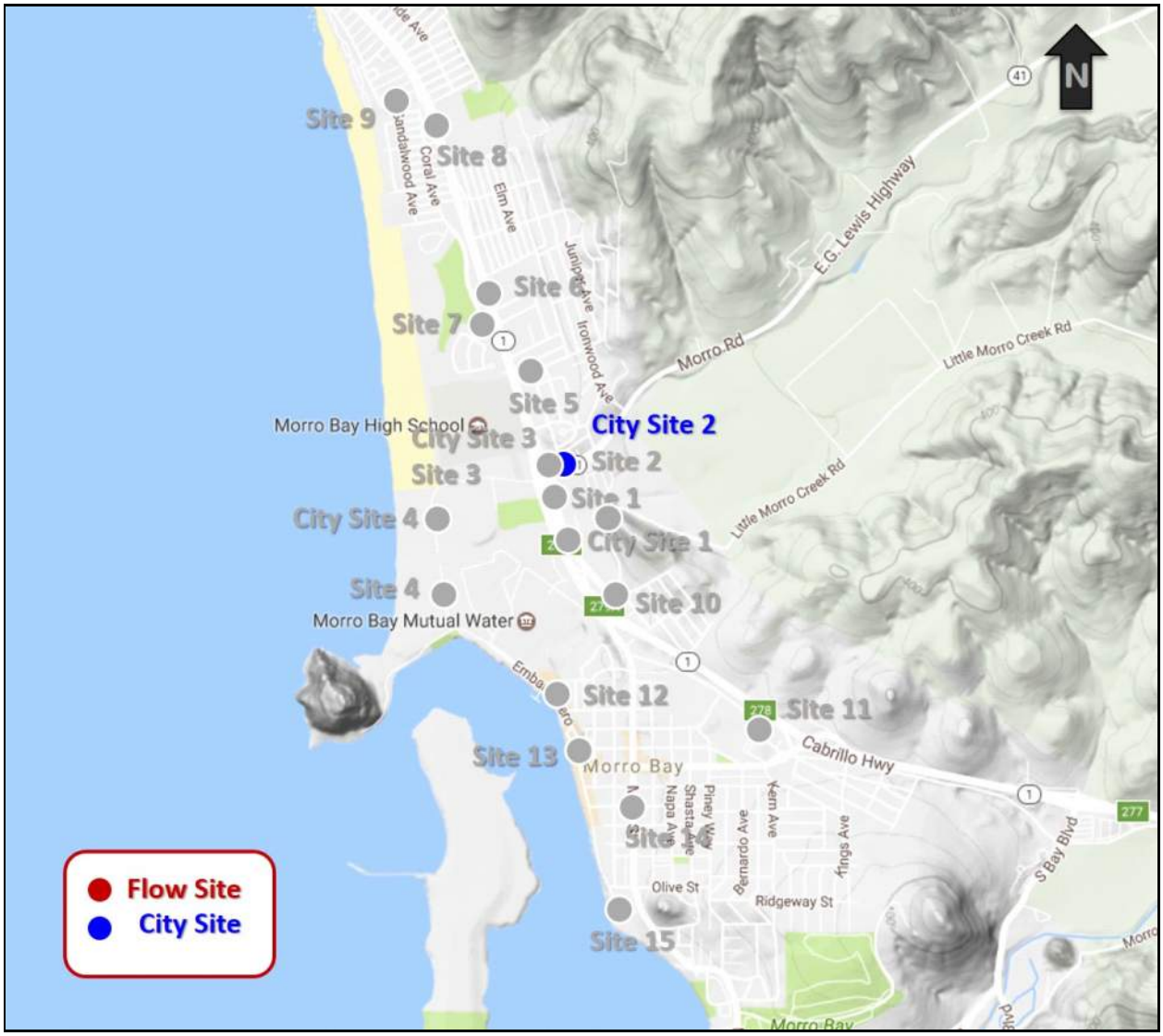
Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 2017 - February 2017

Monitoring Site: MB 2

Location: City Meter at MH 5.19, Atascadero Road ~85 feet east of Sunset Avenue, meter on northeast 6" inlet

Data Summary Report



Vicinity Map: MB 2

MB 2

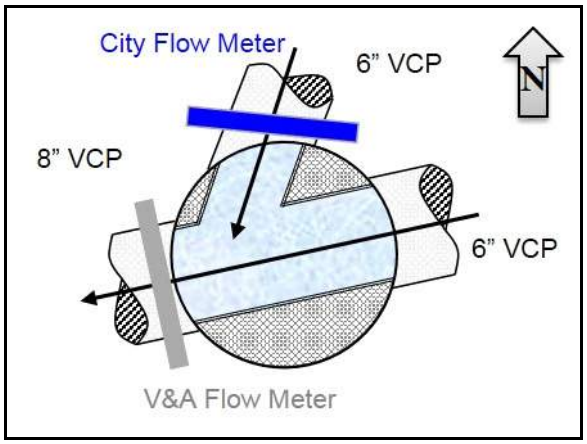
Site Information

Location: City Meter at MH 5.19, Atascadero Road ~85 feet east of Sunset Avenue, meter on northeast 6" inlet

Pipe Diameter: 6 inches

ADWF: 0.029 mgd

Peak Measured Flow: 0.170 mgd

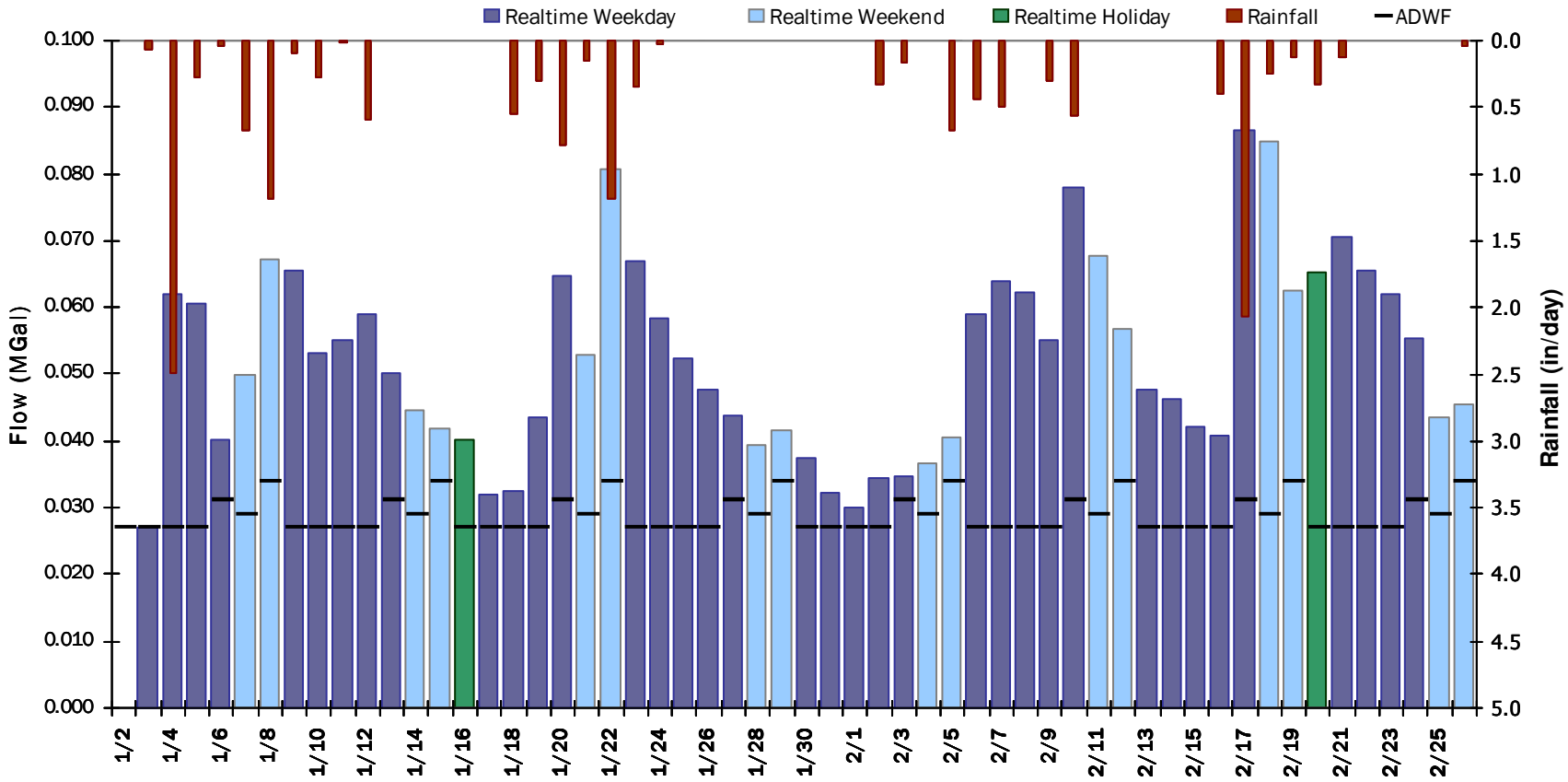


MB 2

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.052 MGal **Peak Daily Flow: 0.086 MGal** **Min Daily Flow: 0.027 MGal**

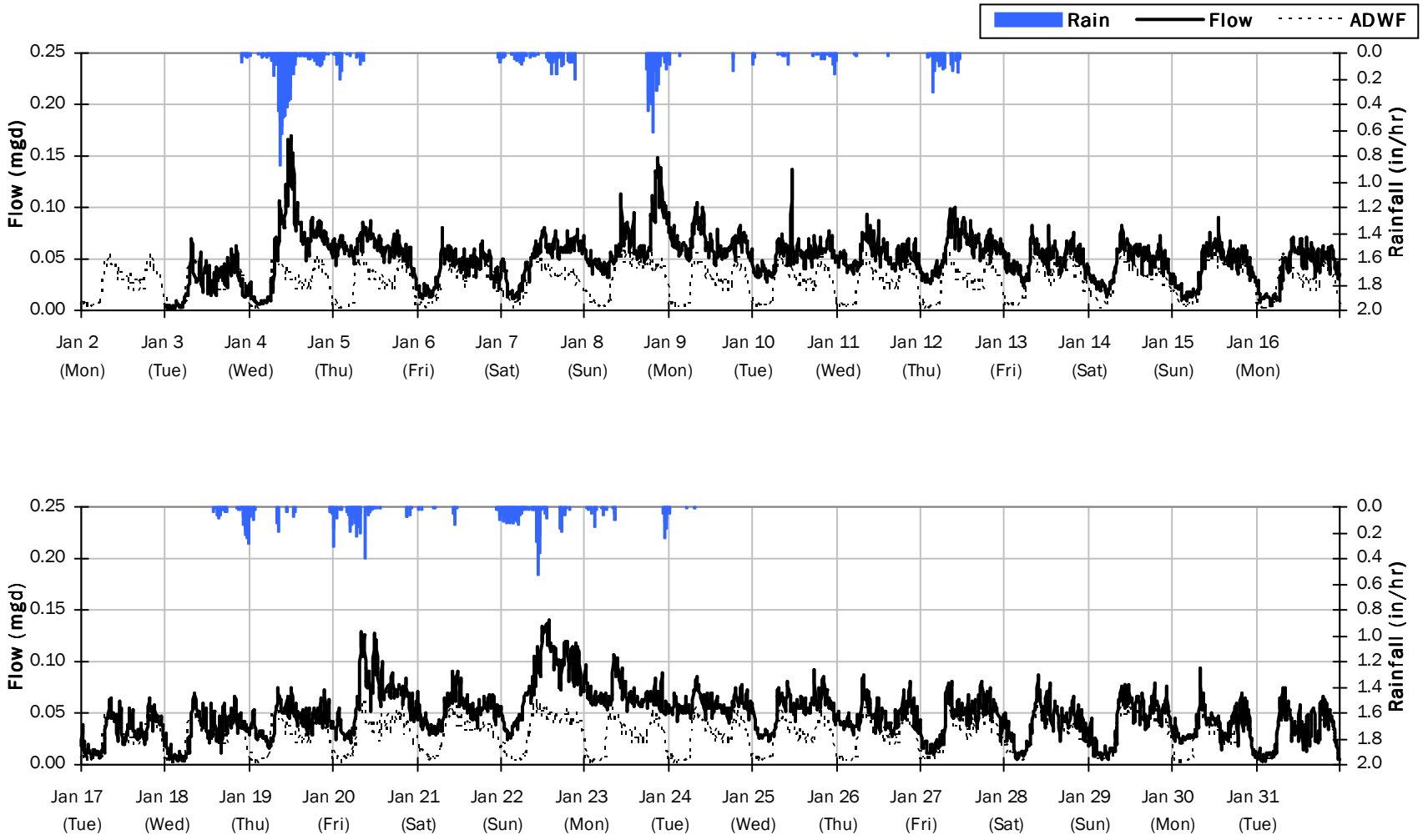
Total Period Rainfall: 15.37 inches



MB 2

Flow Summary: 1/2/2017 to 1/31/2017

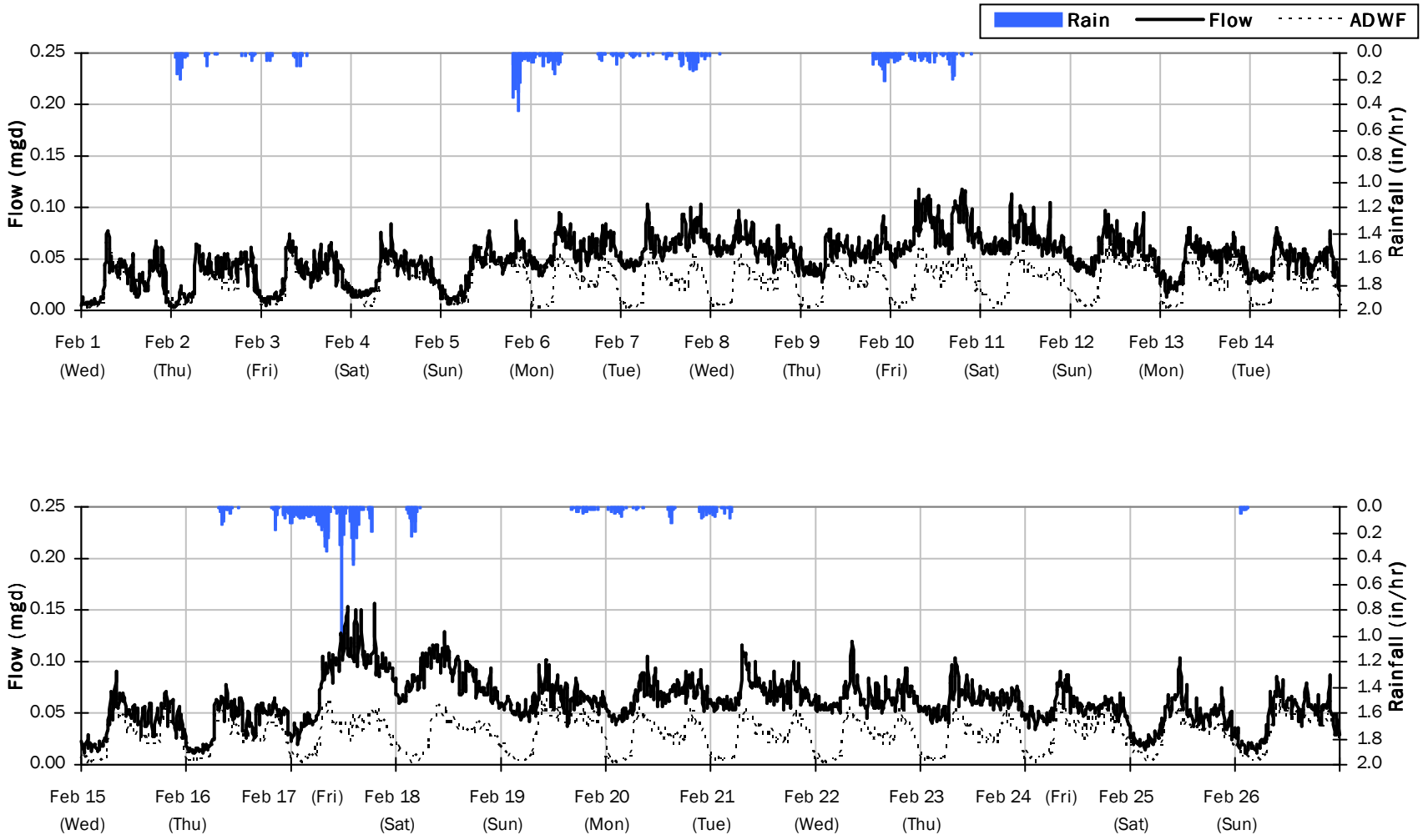
Total Period Rainfall: 9.05 inches **Avg Flow: 0.050 mgd** **Peak Flow: 0.170 mgd** **Min Flow: 0.002 mgd**



MB 2

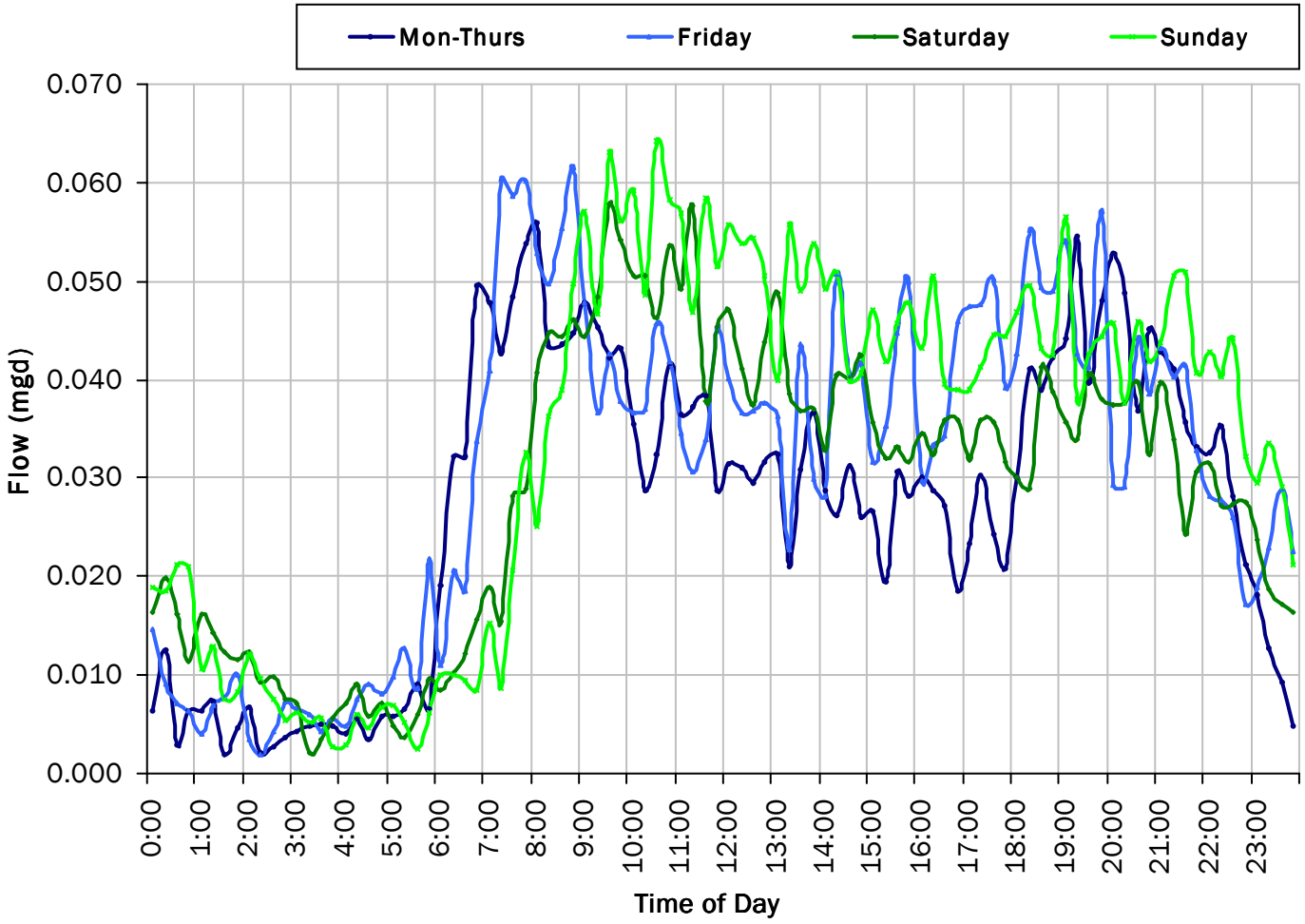
Flow Summary: 2/1/2017 to 2/28/2017

Total Period Rainfall: 6.31 inches **Avg Flow: 0.055 mgd Peak Flow: 0.157 mgd Min Flow: 0.003 mgd**



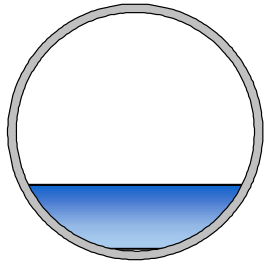
MB 2

Average Dry Weather Flow Hydrographs



ADWF:

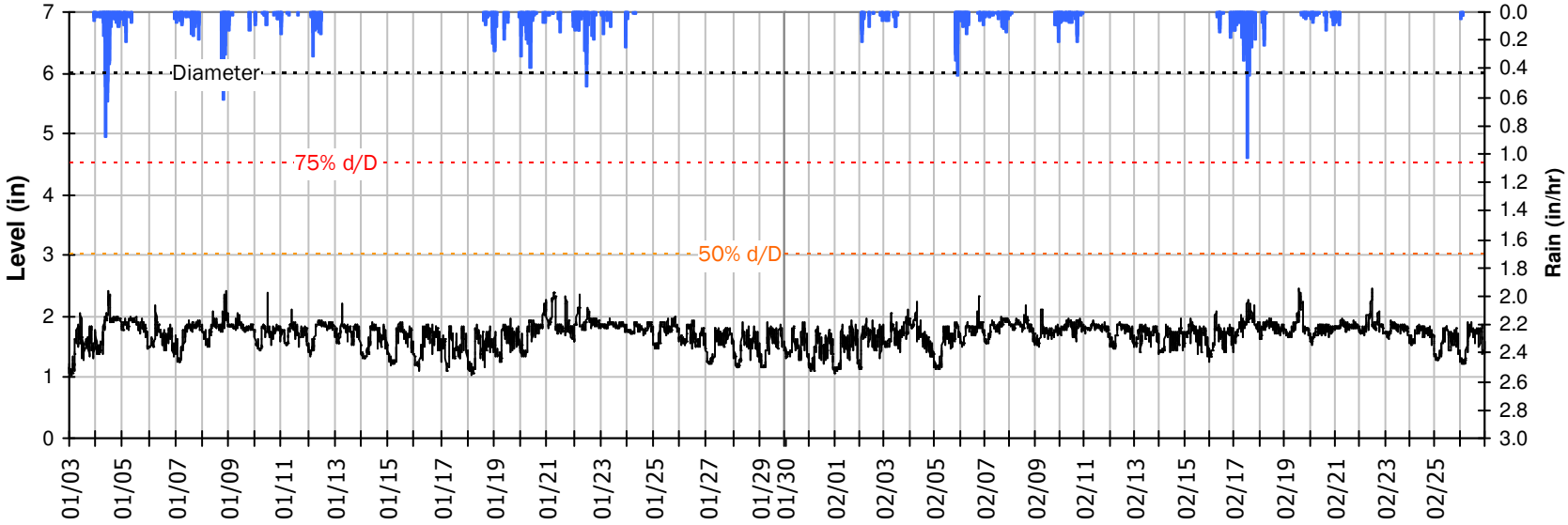
0.029 *mgd*

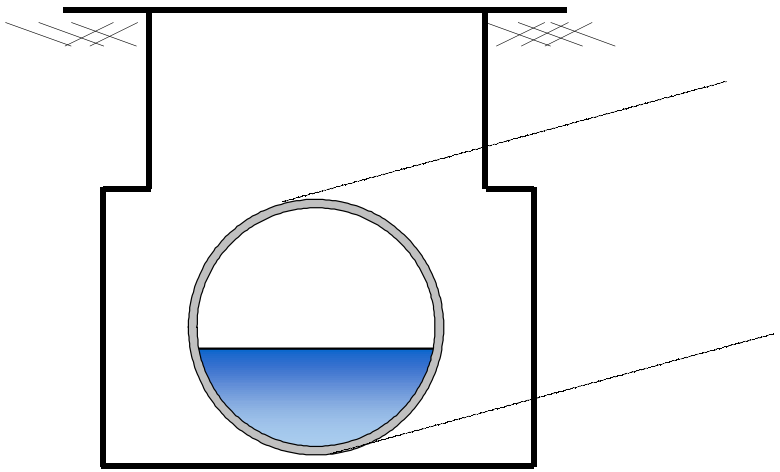


MB 2

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

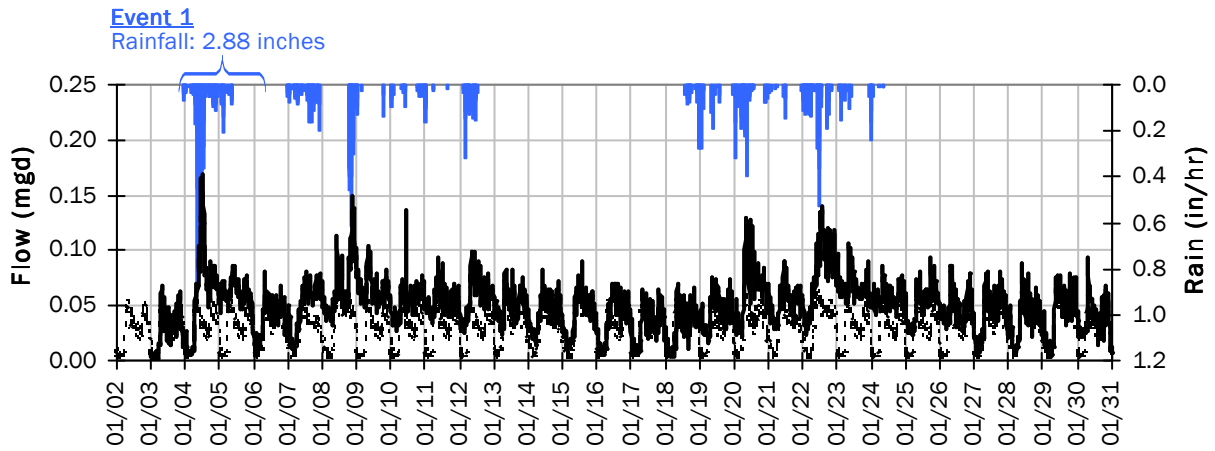




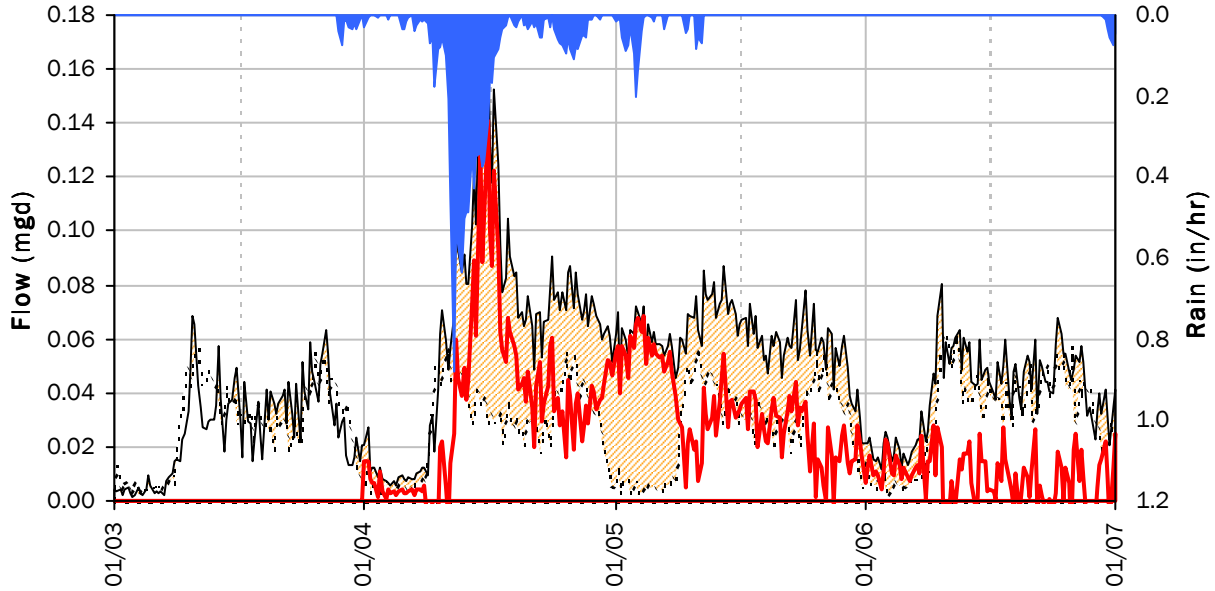
Pipe Diameter:	6	*inches*
Peak Measured Level:	2.47	*inches*
Peak d/D Ratio:	0.41	

MB 2
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

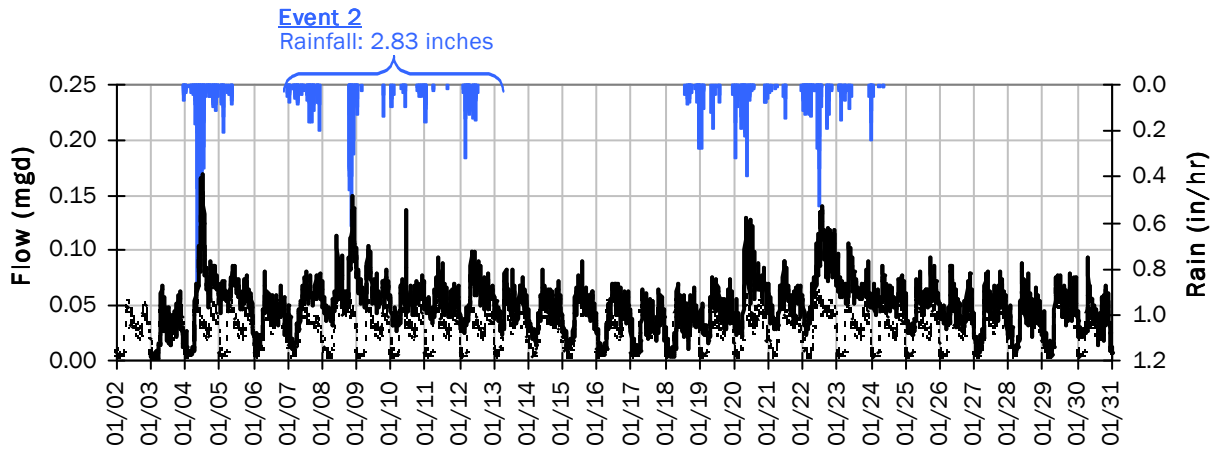


Storm Event I/I Analysis (Rain = 2.88 inches)

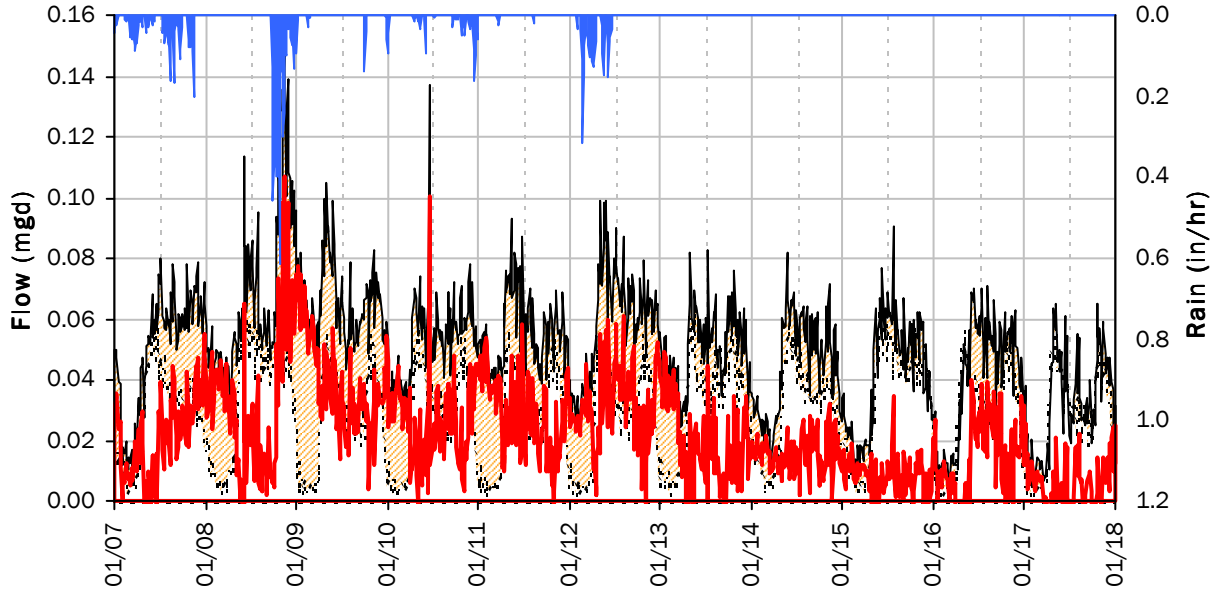
Capacity		Inflow / Infiltration	
Peak Flow:	0.17 *mgd*	**Peak I/I Rate:**	0.14 *mgd*
PF:	5.86	**Total I/I:**	78,000 *gallons*
Peak Level:	2.44 *in*		
d/D Ratio:	0.41		

MB 2
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

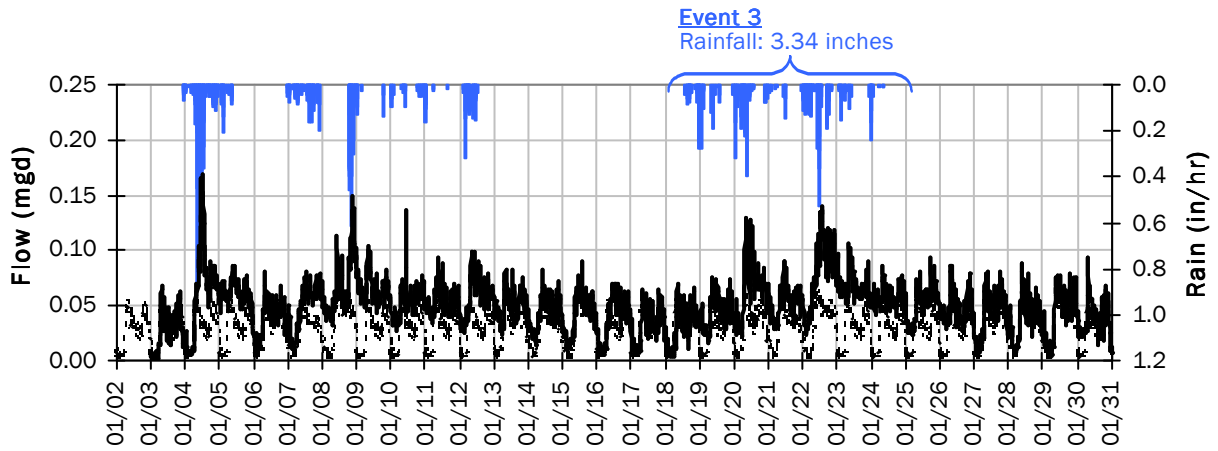


Storm Event I/I Analysis (Rain = 2.83 inches)

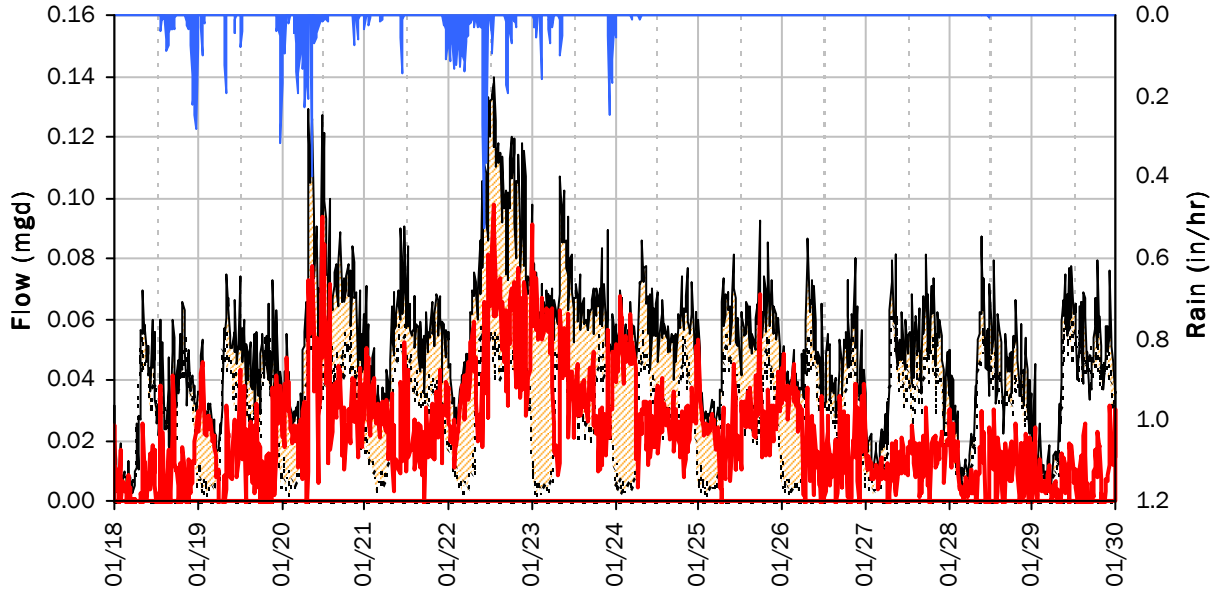
Capacity		Inflow / Infiltration	
Peak Flow:	0.15 *mgd*	**Peak I/I Rate:**	0.11 *mgd*
PF:	5.14	**Total I/I:**	239,000 *gallons*
Peak Level:	2.44 *in*		
d/D Ratio:	0.41		

MB 2
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 3 Detail Graph

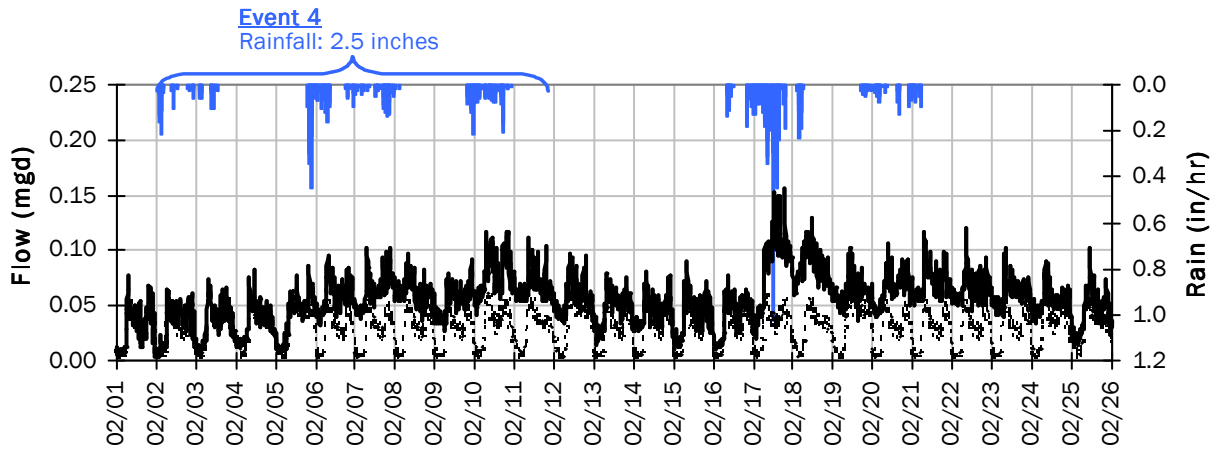


Storm Event I/I Analysis (Rain = 3.35 inches)

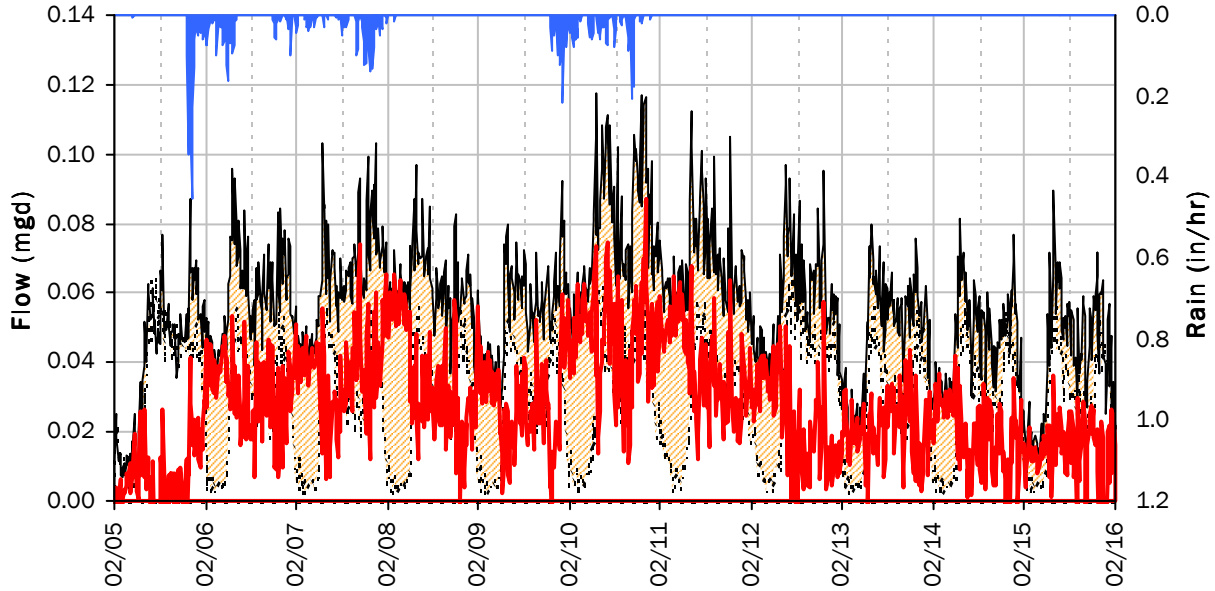
Capacity		Inflow / Infiltration	
Peak Flow:	0.14 *mgd*	**Peak I/I Rate:**	0.10 *mgd*
PF:	4.84	**Total I/I:**	274,000 *gallons*
Peak Level:	2.41 *in*		
d/D Ratio:	0.40		

MB 2
I/I Summary: Event 4

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 4 Detail Graph

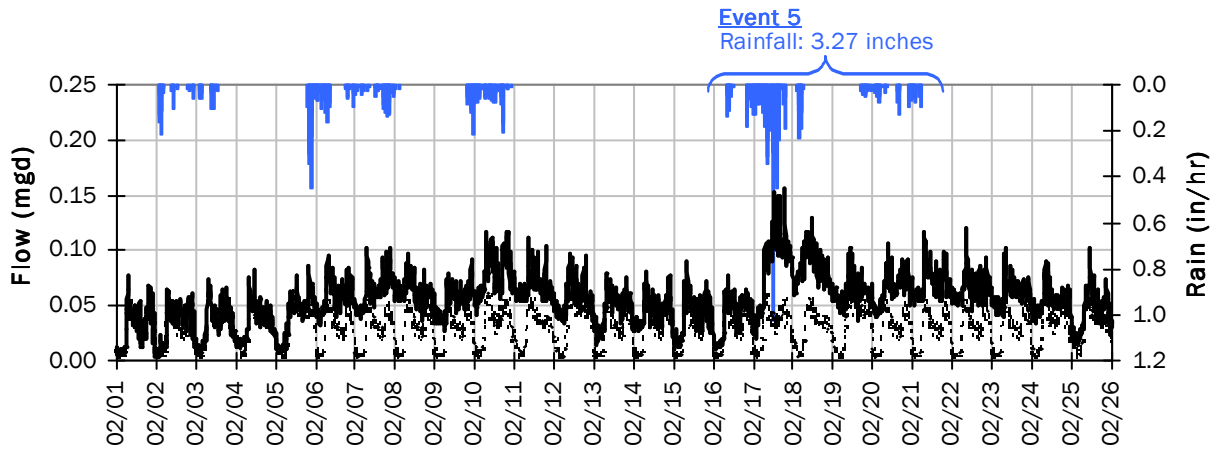


Storm Event I/I Analysis (Rain = 2.50 inches)

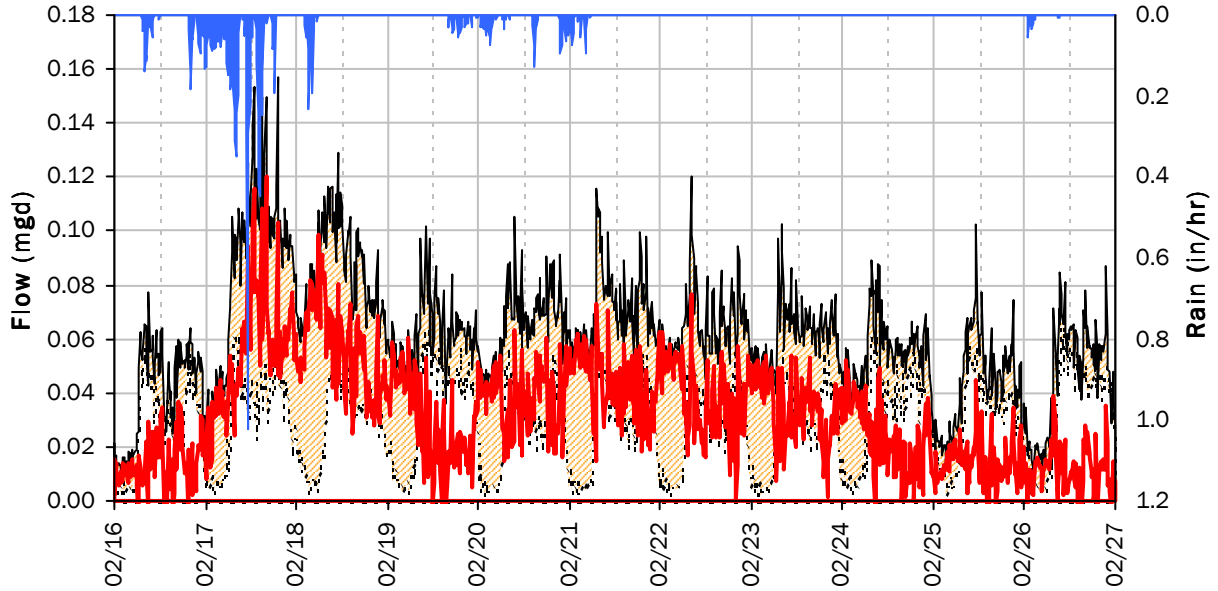
Capacity		Inflow / Infiltration	
Peak Flow:	0.12 *mgd*	**Peak I/I Rate:**	0.09 *mgd*
PF:	4.06	**Total I/I:**	302,000 *gallons*
Peak Level:	2.33 *in*		
d/D Ratio:	0.39		

MB 2
I/I Summary: Event 5

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 5 Detail Graph



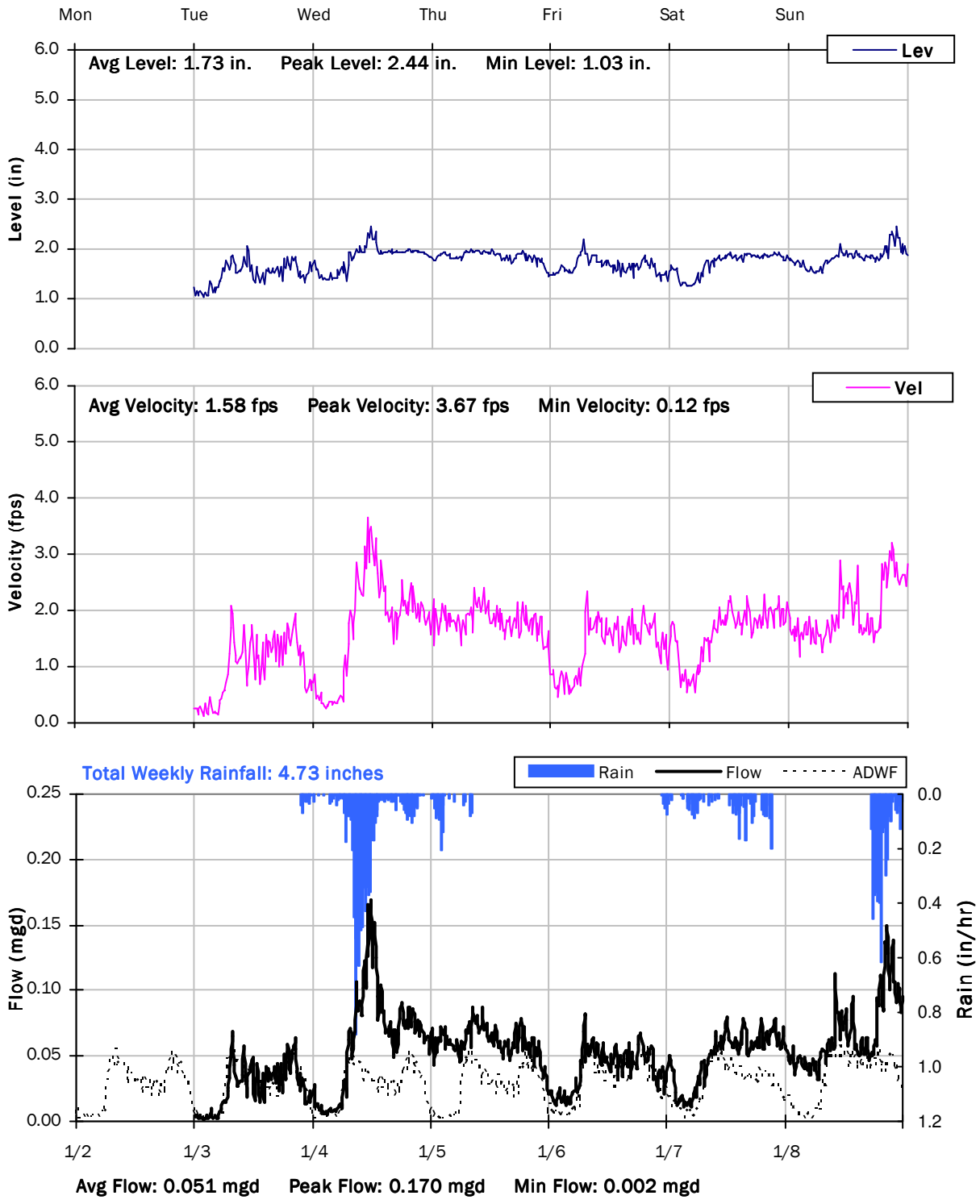
Storm Event I/I Analysis (Rain = 3.28 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.16 *mgd*	**Peak I/I Rate:**	0.12 *mgd*
PF:	5.42	**Total I/I:**	359,000 *gallons*
Peak Level:	2.47 *in*		
d/D Ratio:	0.41		

MB 2

Weekly Level, Velocity and Flow Hydrographs

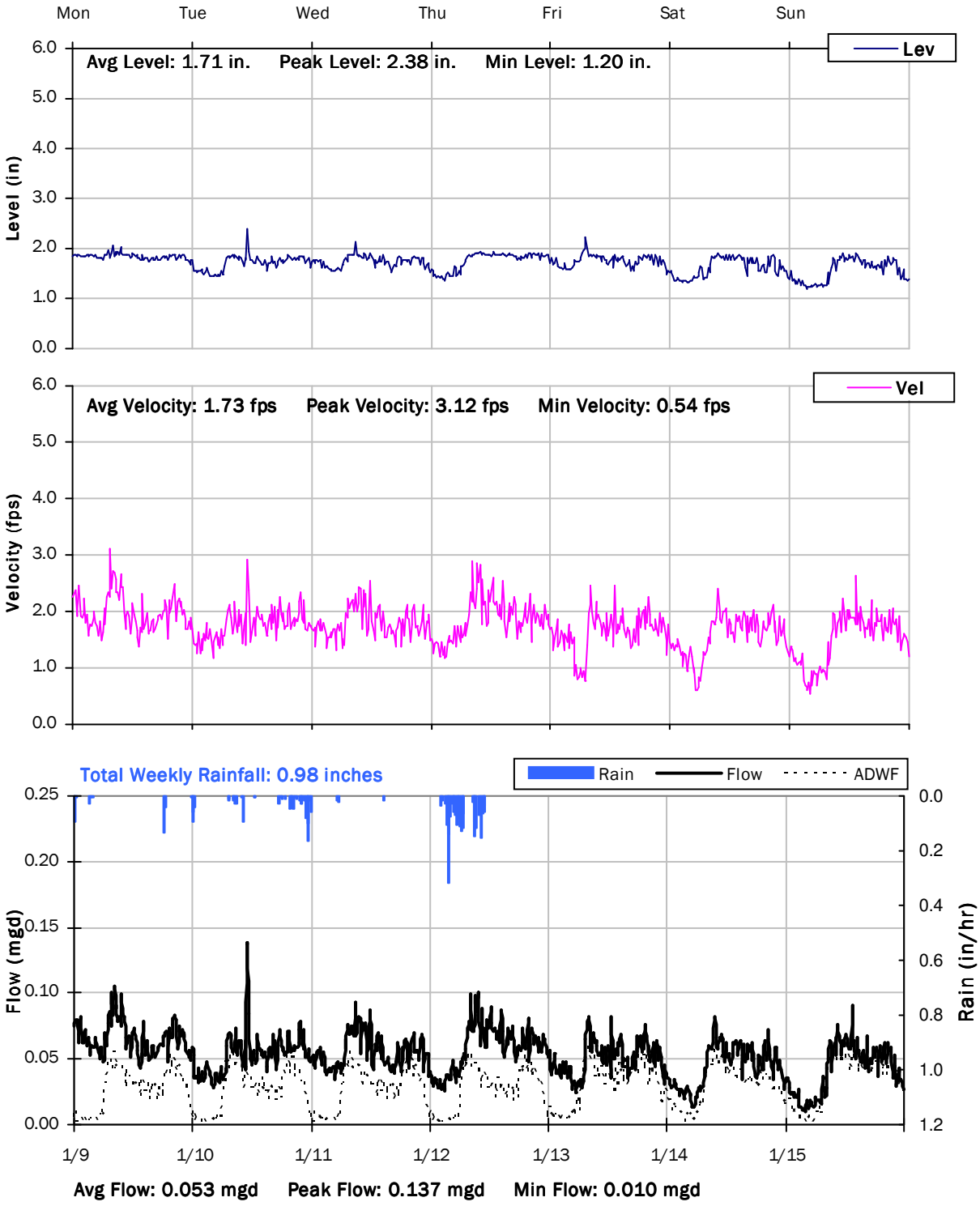
1/2/2017 to 1/9/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

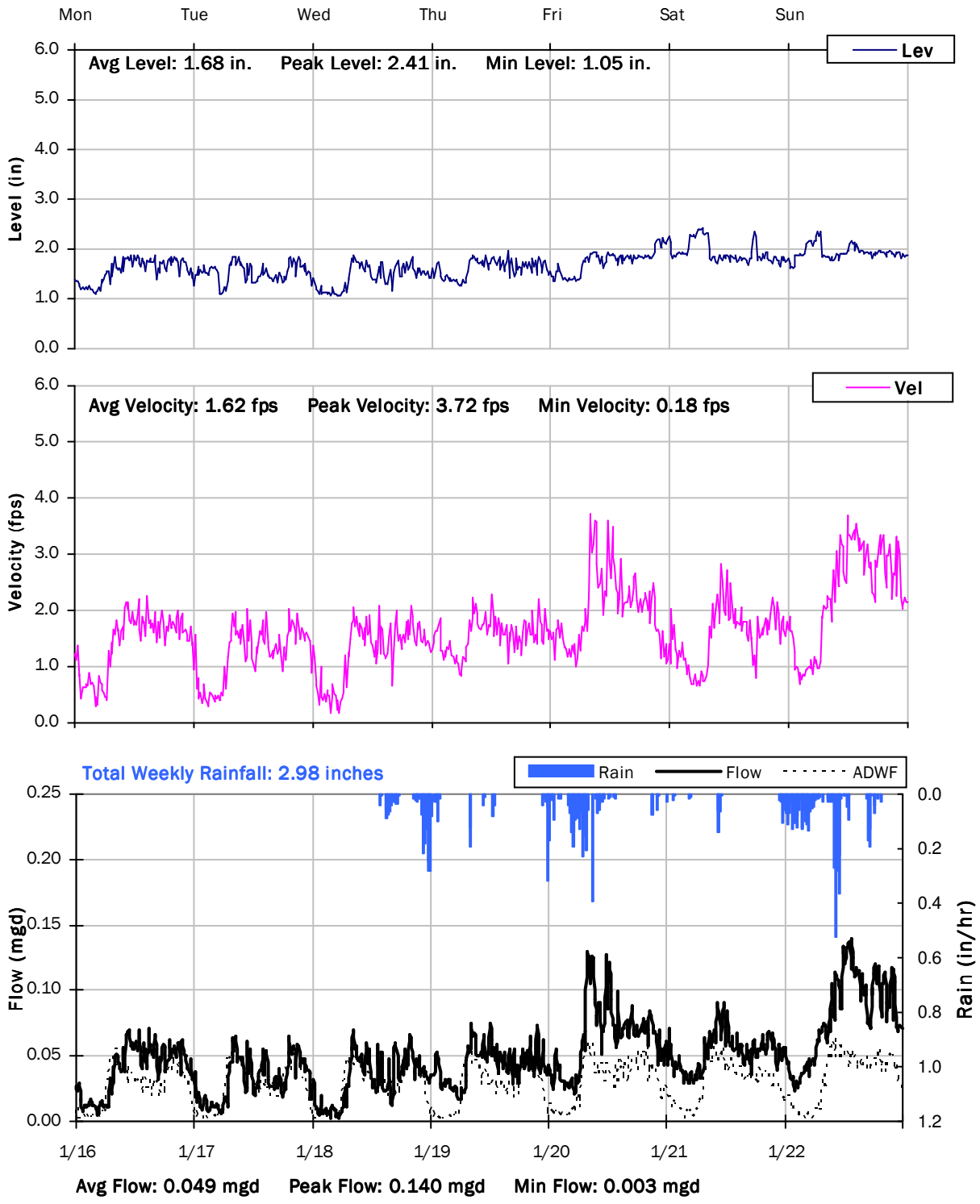
1/9/2017 to 1/16/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

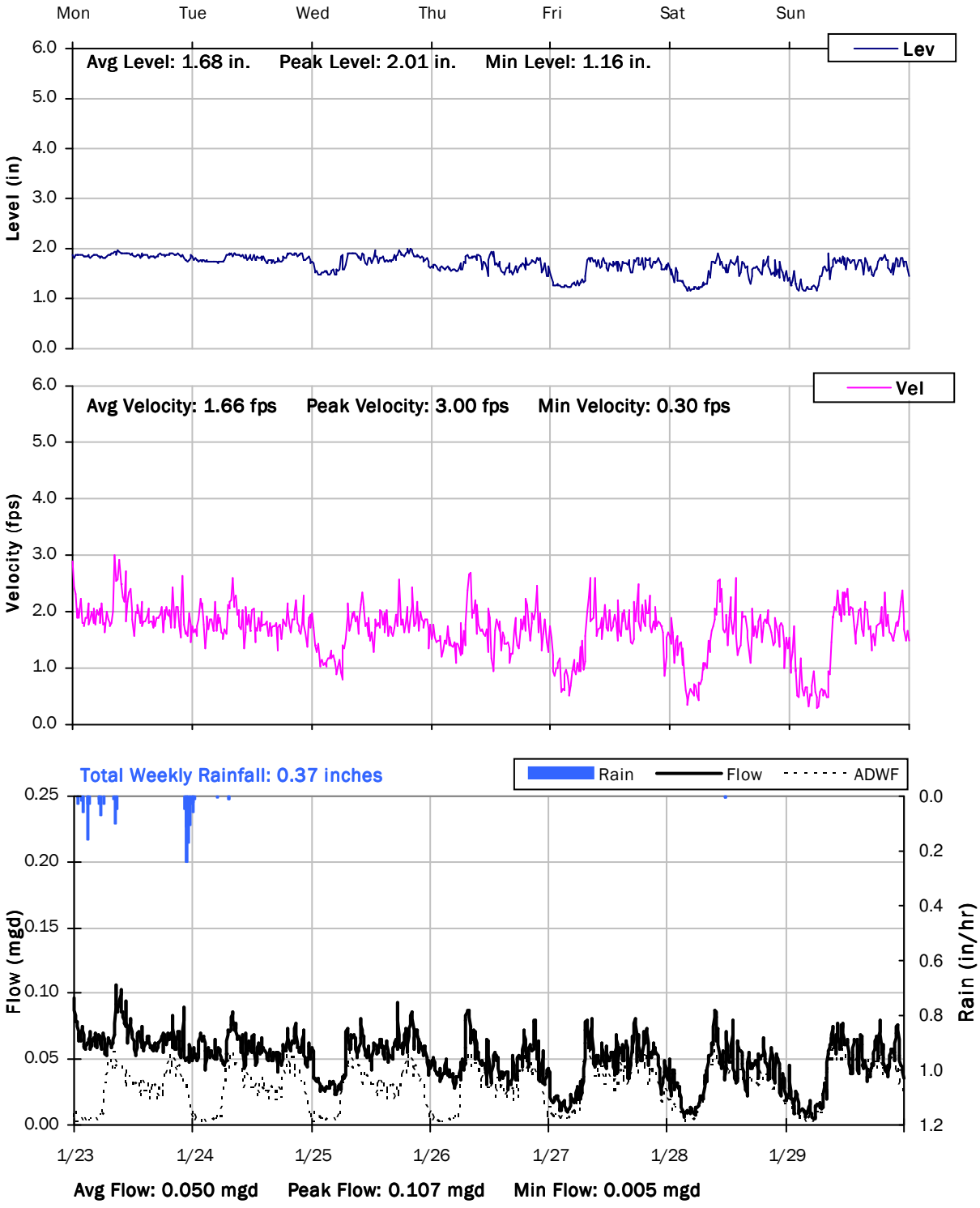
1/16/2017 to 1/23/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

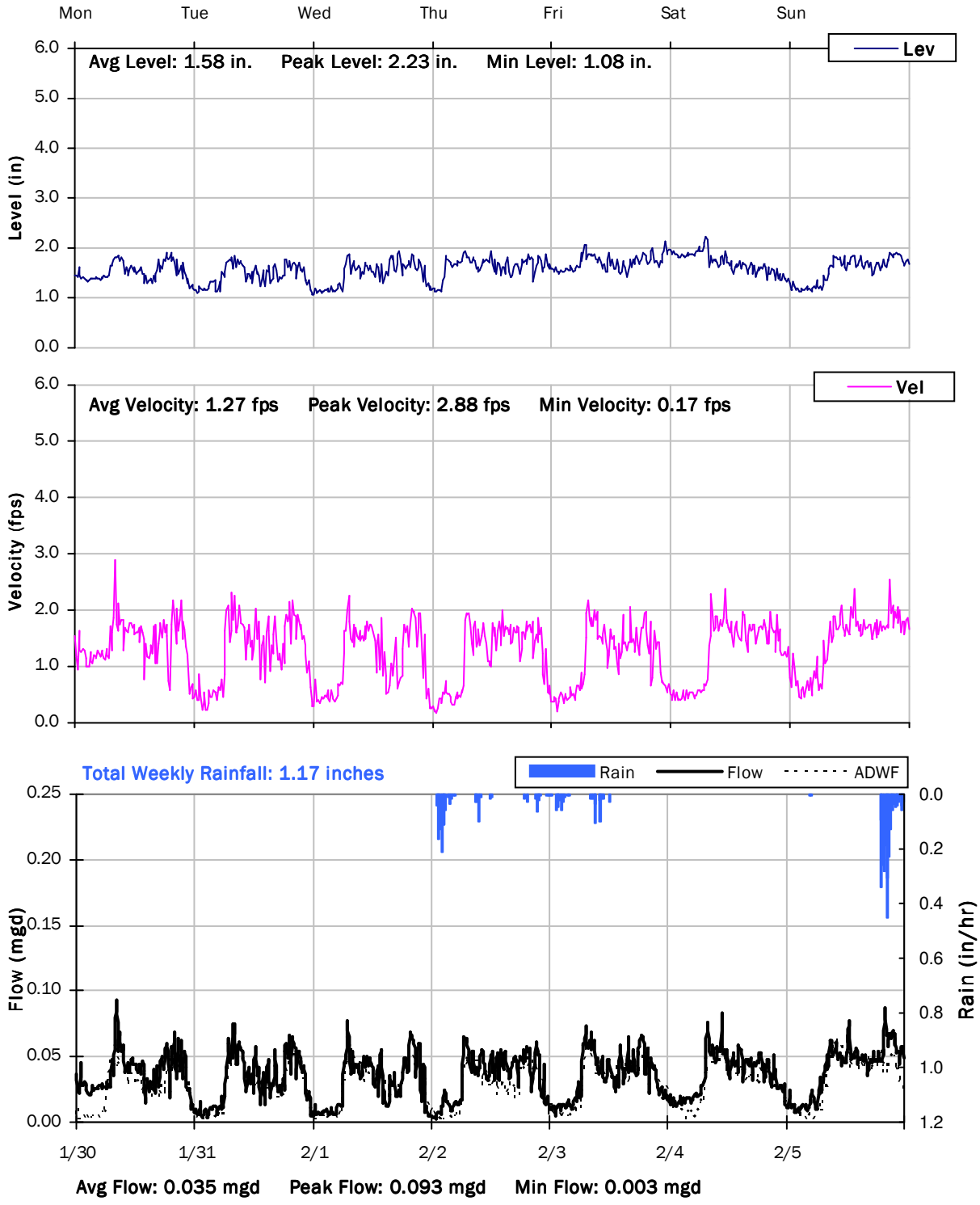
1/23/2017 to 1/30/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

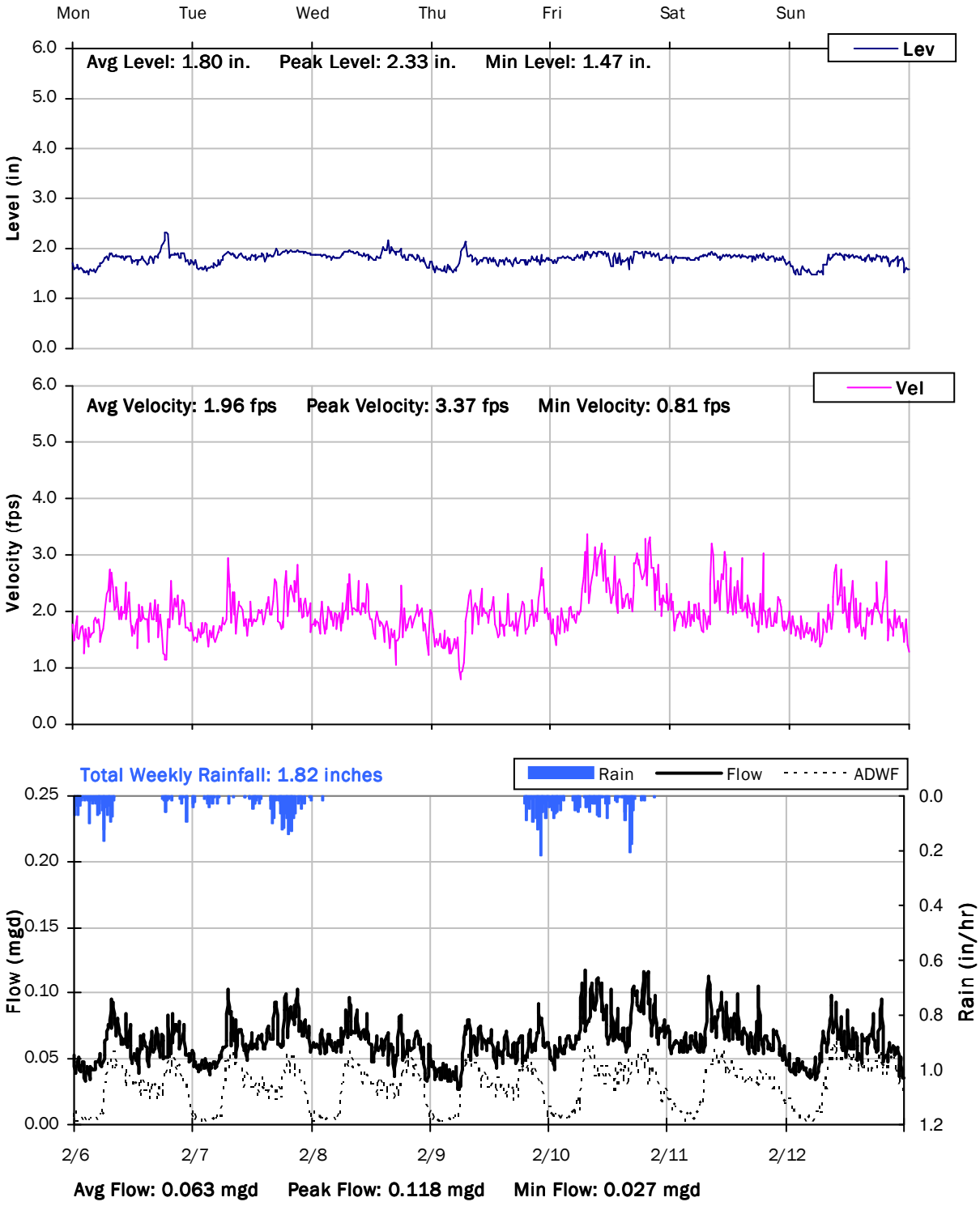
1/30/2017 to 2/6/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

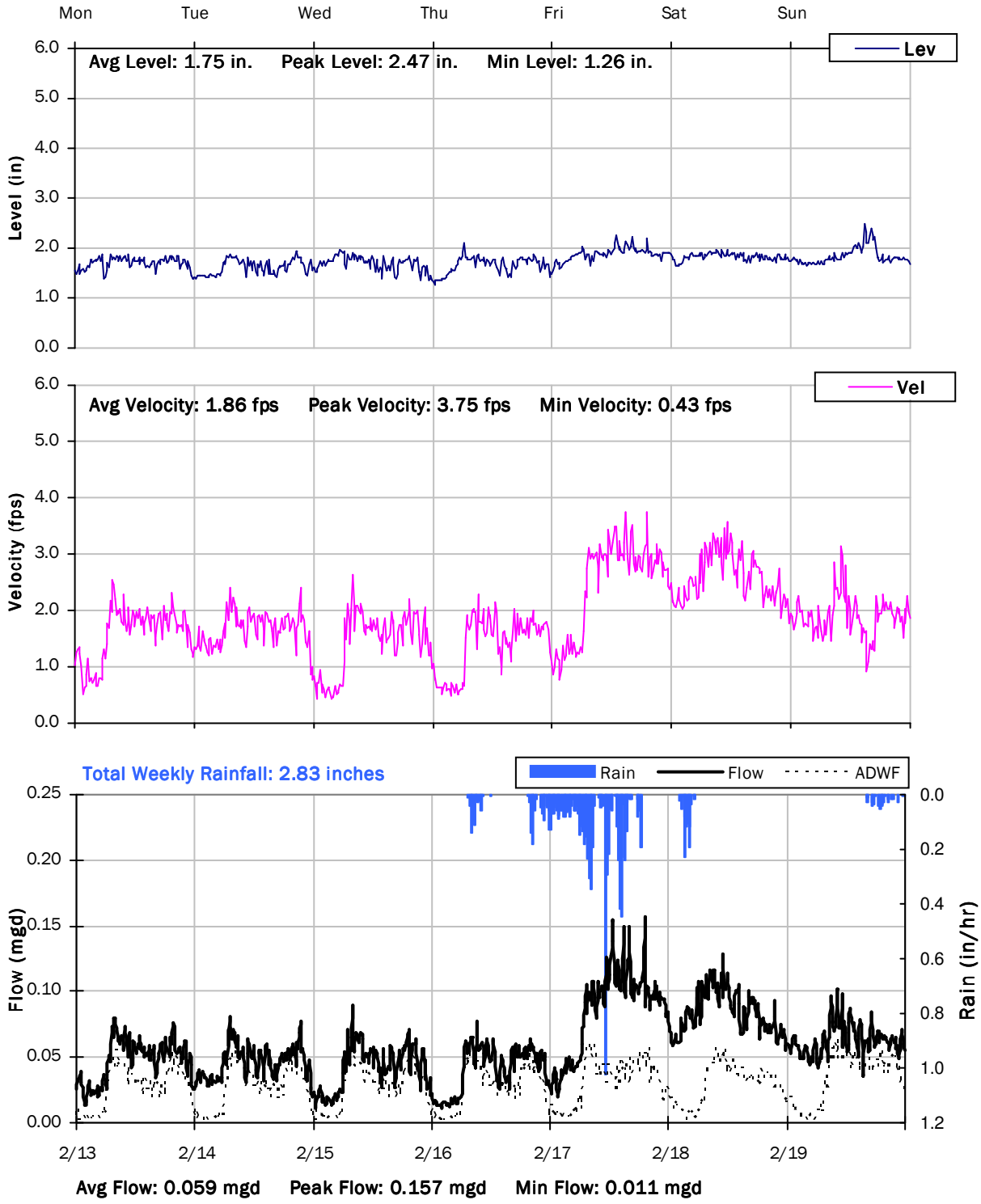
2/6/2017 to 2/13/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

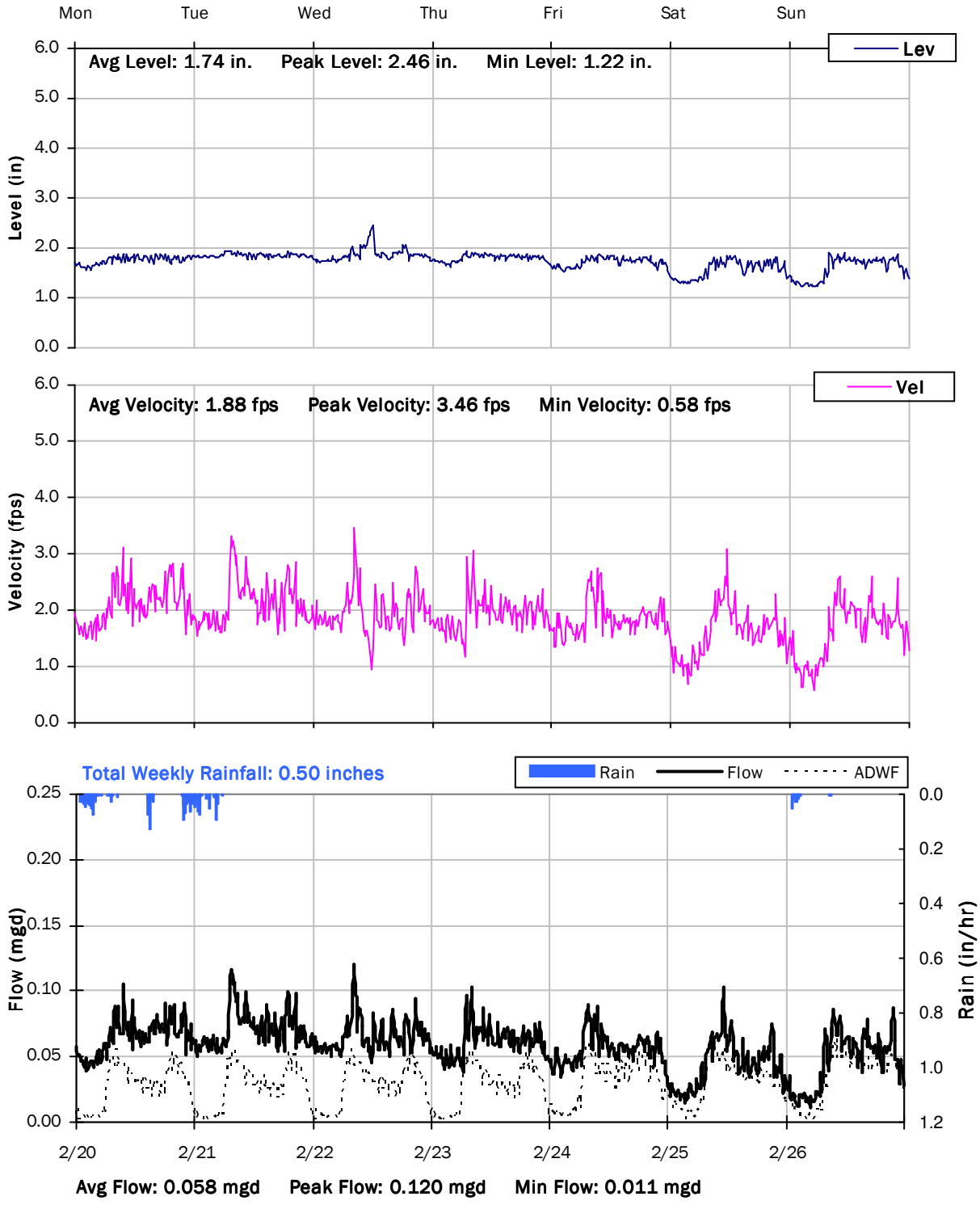
2/13/2017 to 2/20/2017



MB 2

Weekly Level, Velocity and Flow Hydrographs

2/20/2017 to 2/27/2017



City of Morro Bay

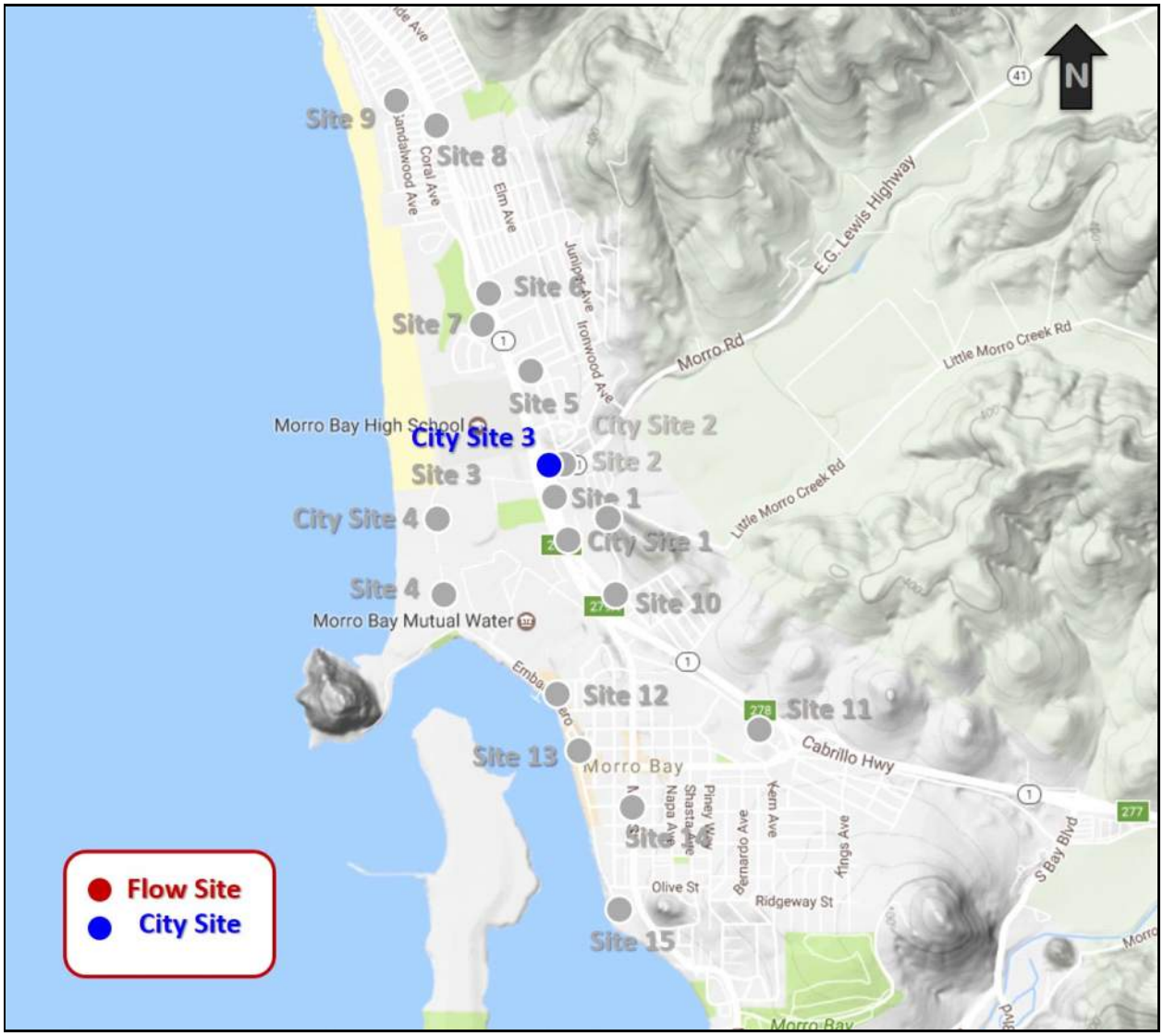
Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 2017 - February 2017

Monitoring Site: MB 3

Location: City Meter at MH 5.25A, Island between Main Street and HWY 1 on-ramp, ~135 feet north of Atascadero Road, meter on north 15" inlet

Data Summary Report



Vicinity Map: MB 3

MB 3

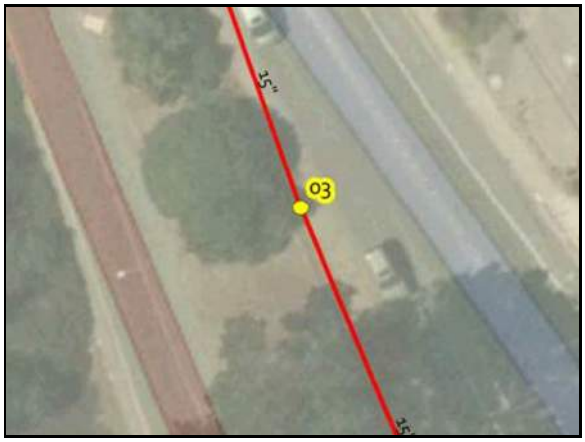
Site Information

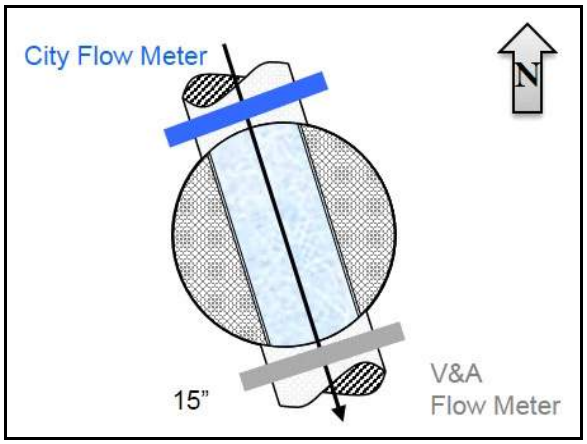
Location: City Meter at MH 5.25A, Island between Main Street and HWY 1 on-ramp, ~135 feet north of Atascadero Road, meter on north 15" inlet

Pipe Diameter: 15 inches

ADWF: 0.296 mgd

Peak Measured Flow: 2.539 mgd



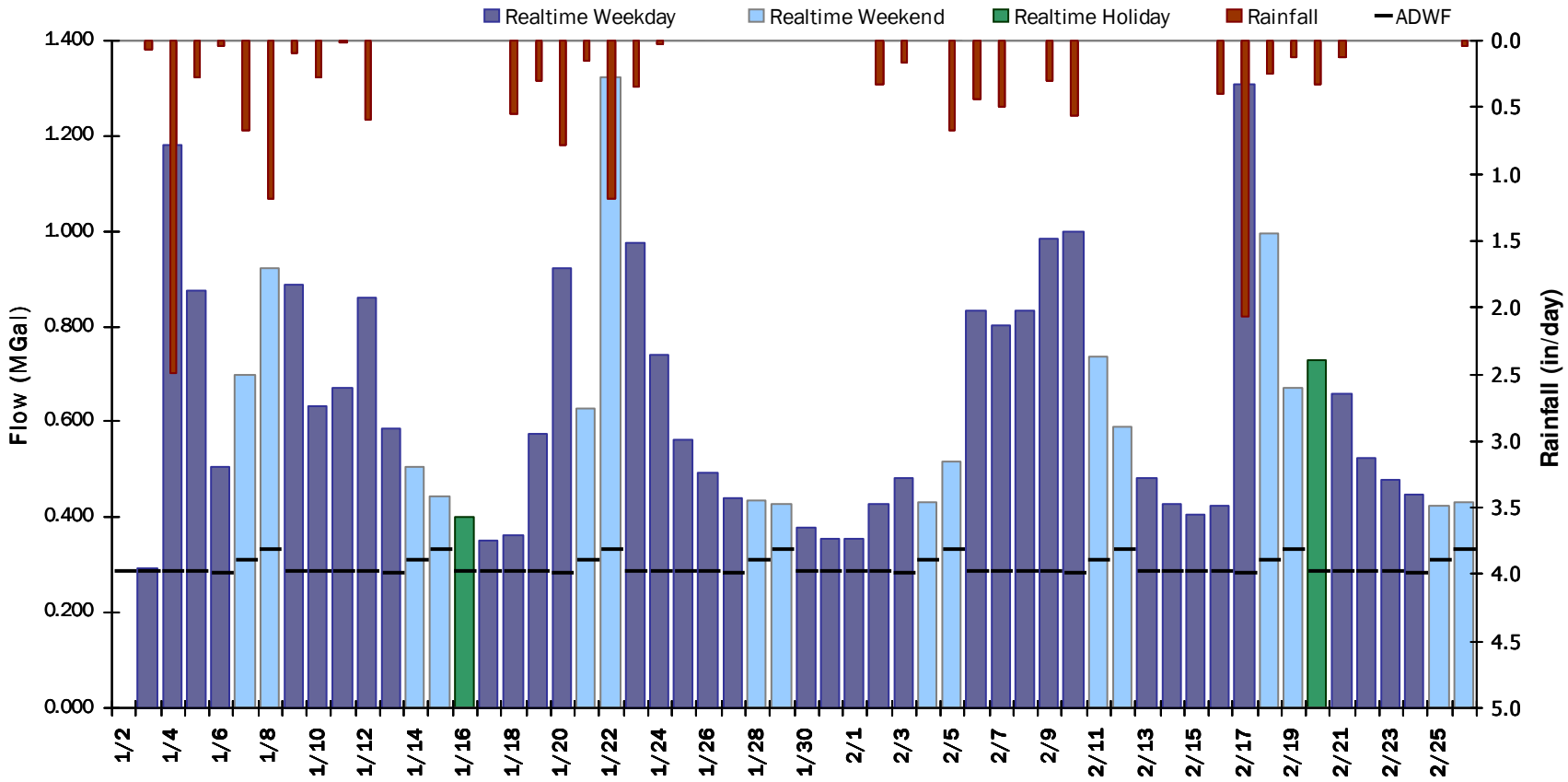


MB 3

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.633 MGal **Peak Daily Flow: 1.322 MGal** **Min Daily Flow: 0.292 MGal**

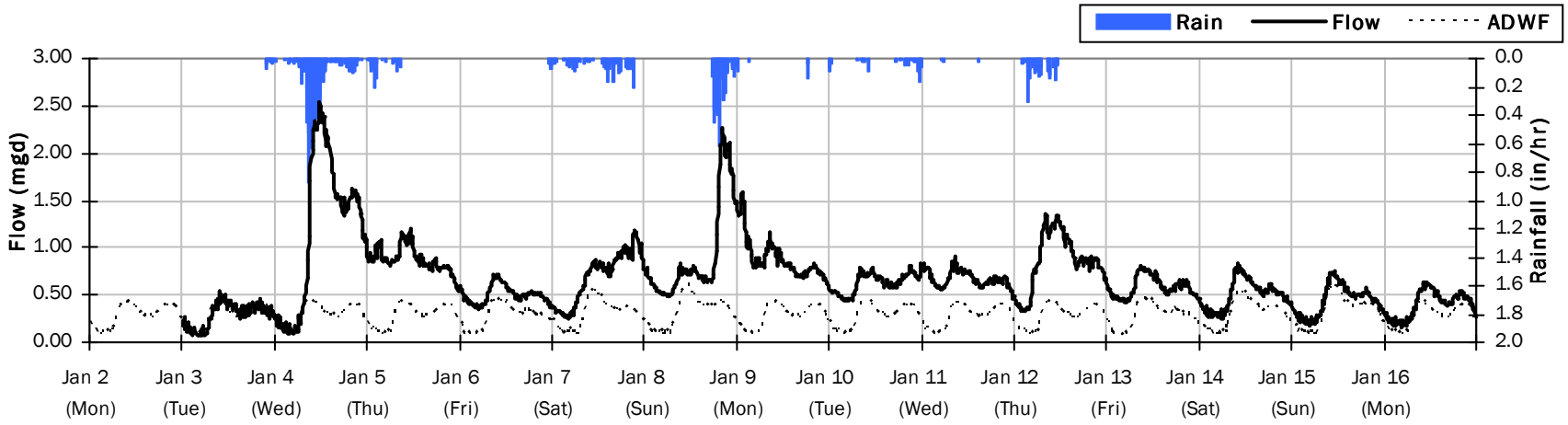
Total Period Rainfall: 15.37 inches

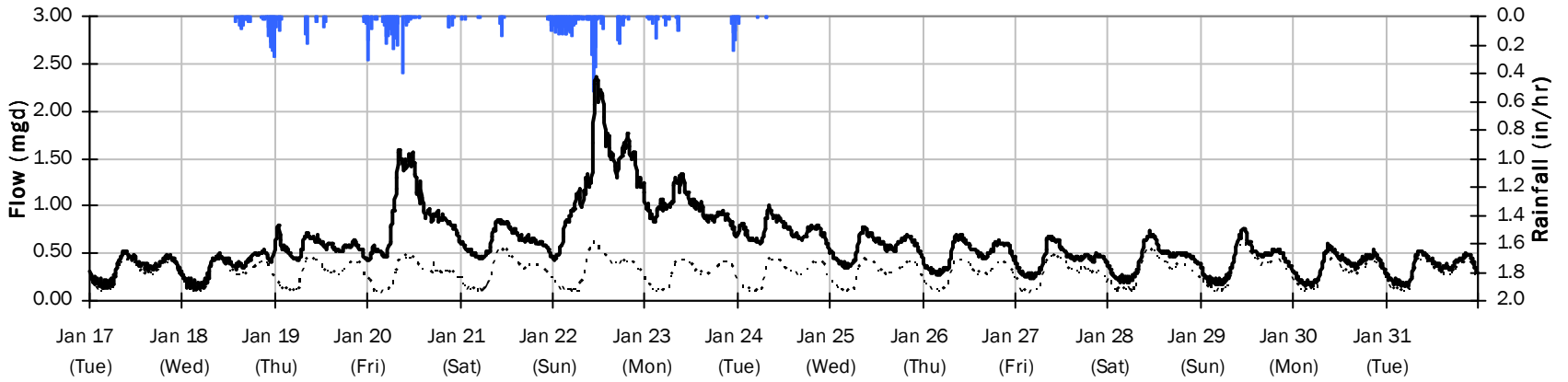


MB 3

Flow Summary: 1/2/2017 to 1/31/2017

Total Period Rainfall: 9.05 inches **Avg Flow: 0.636 mgd** **Peak Flow: 2.539 mgd** **Min Flow: 0.071 mgd**

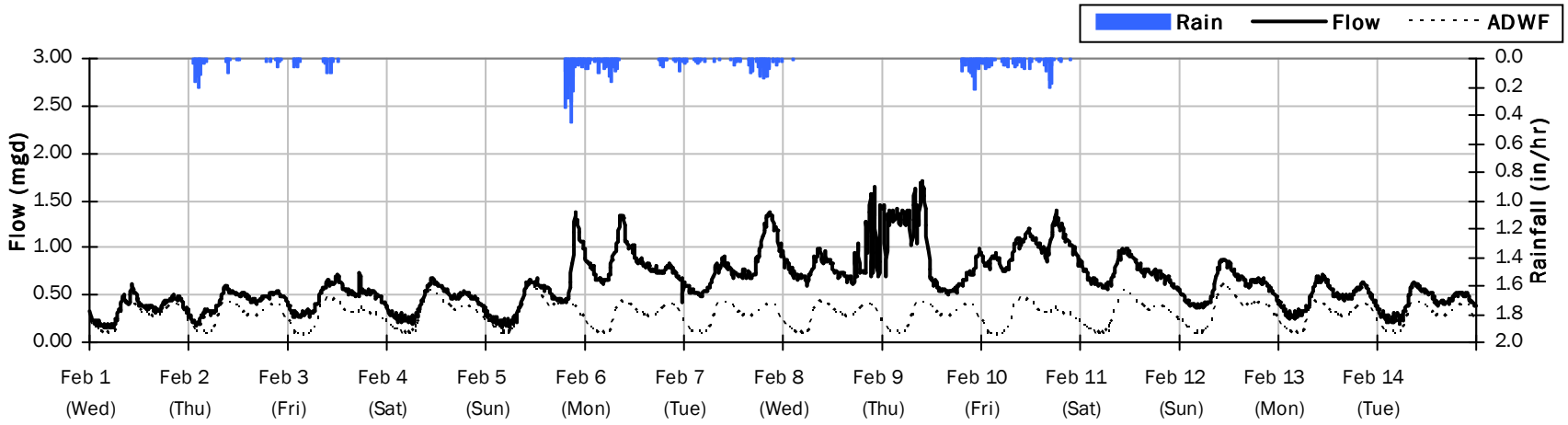


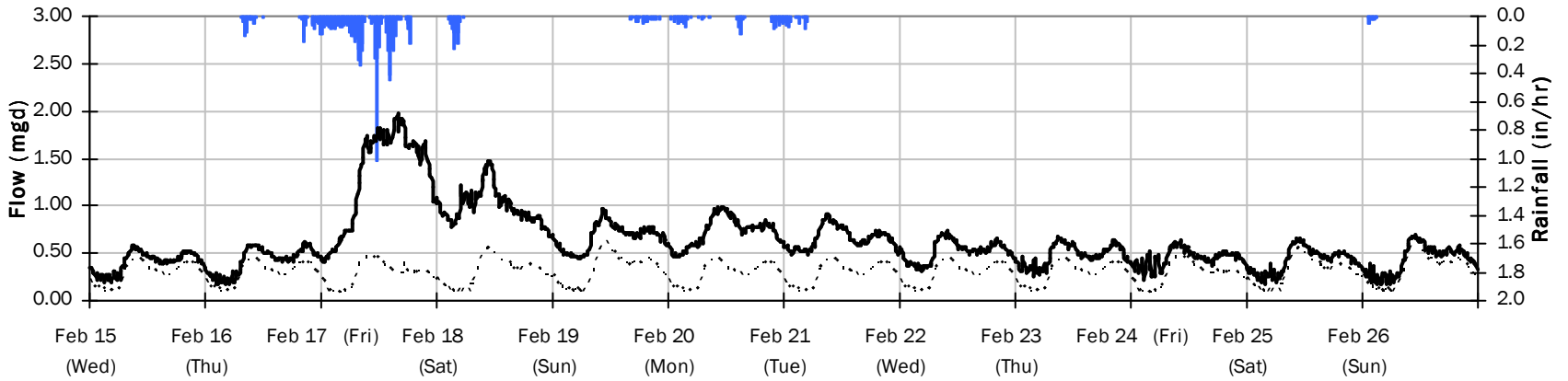


MB 3

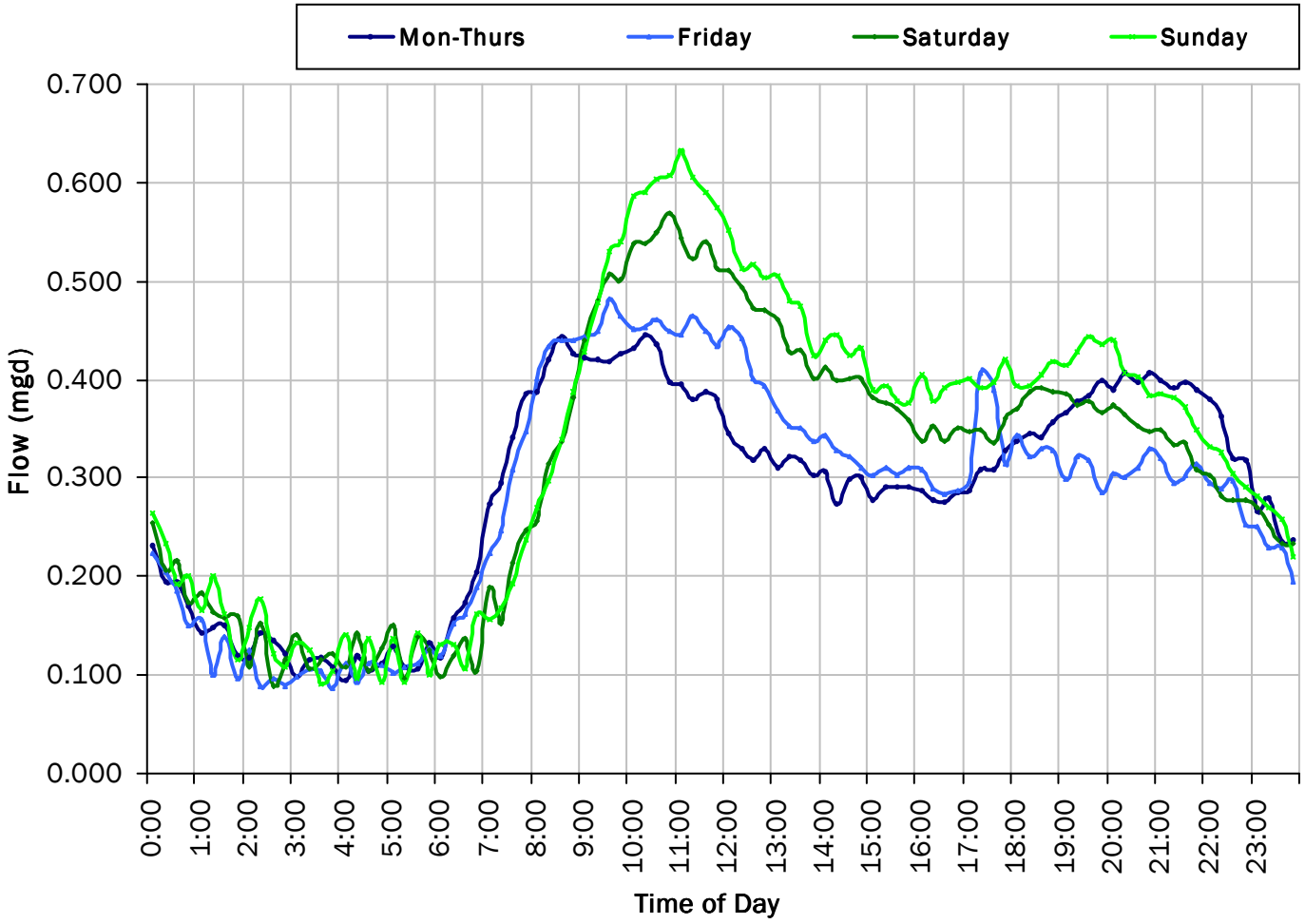
Flow Summary: 2/1/2017 to 2/28/2017

Total Period Rainfall: 6.31 inches **Avg Flow: 0.631 mgd Peak Flow: 1.967 mgd Min Flow: 0.148 mgd**



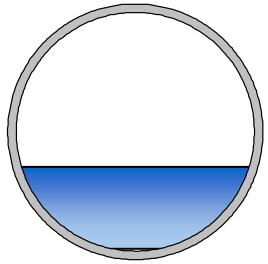


MB 3
Average Dry Weather Flow Hydrographs



ADWF:

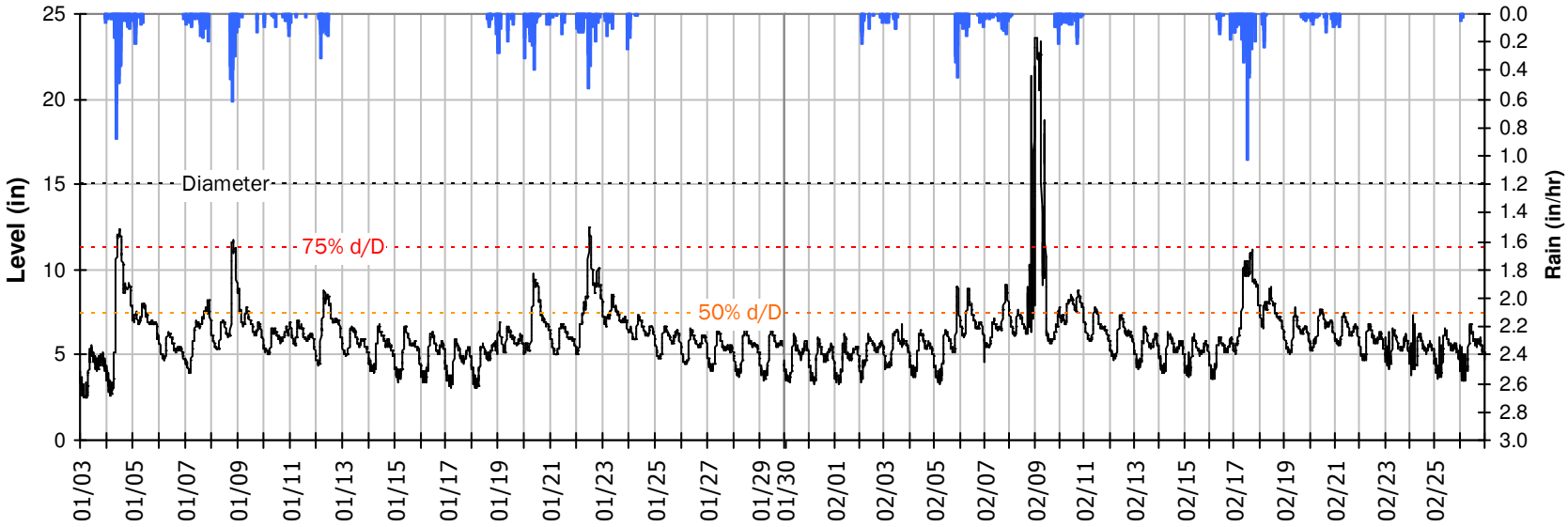
0.296 *mgd*

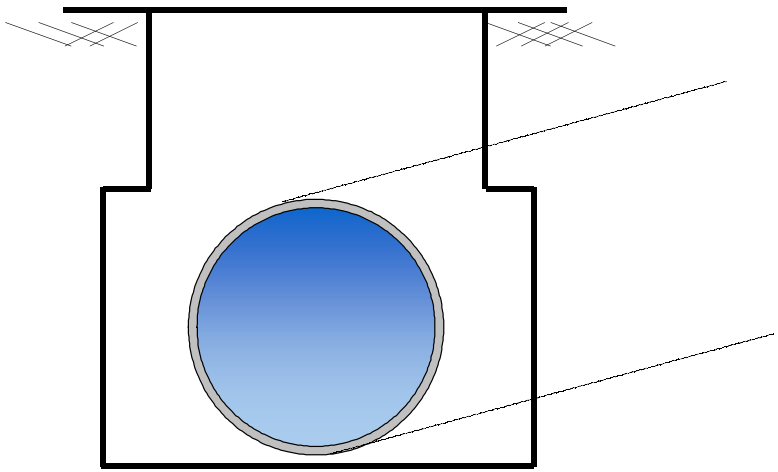


MB 3

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



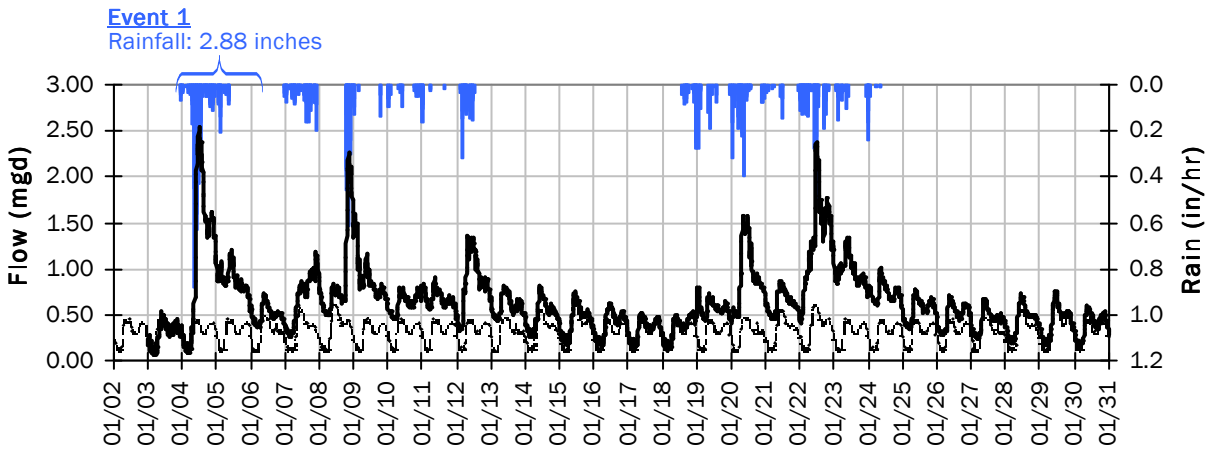


Pipe Diameter:	15	*inches*
Peak Measured Level:	23.6	*inches*
Peak d/D Ratio:	1.57	

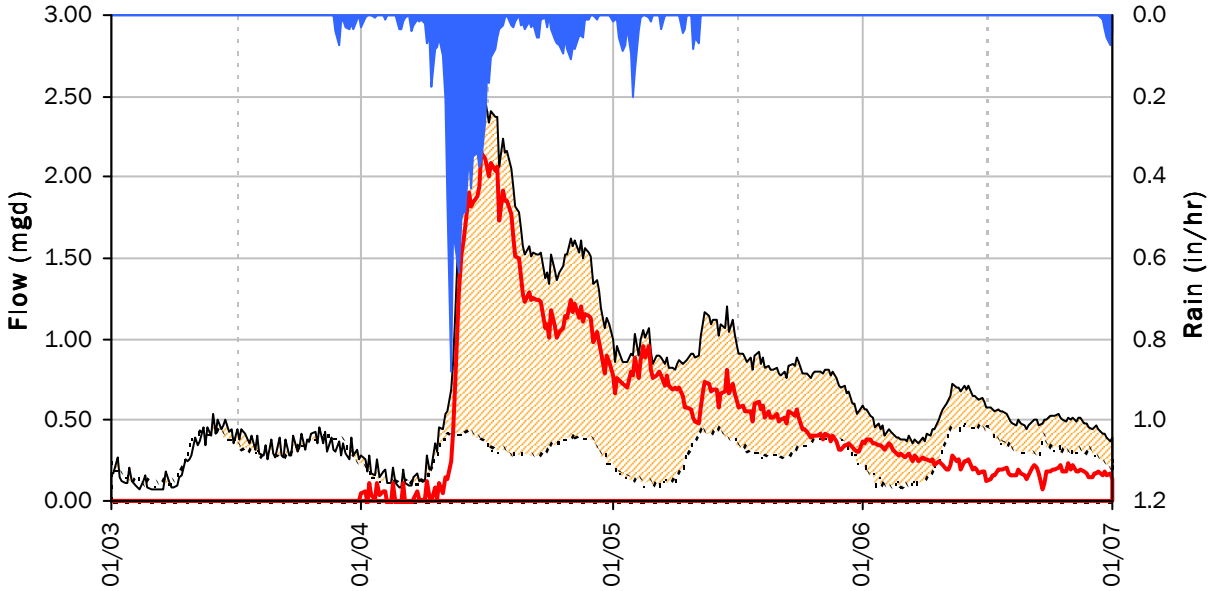
Surcharged 8.6 inches over crown

MB 3
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

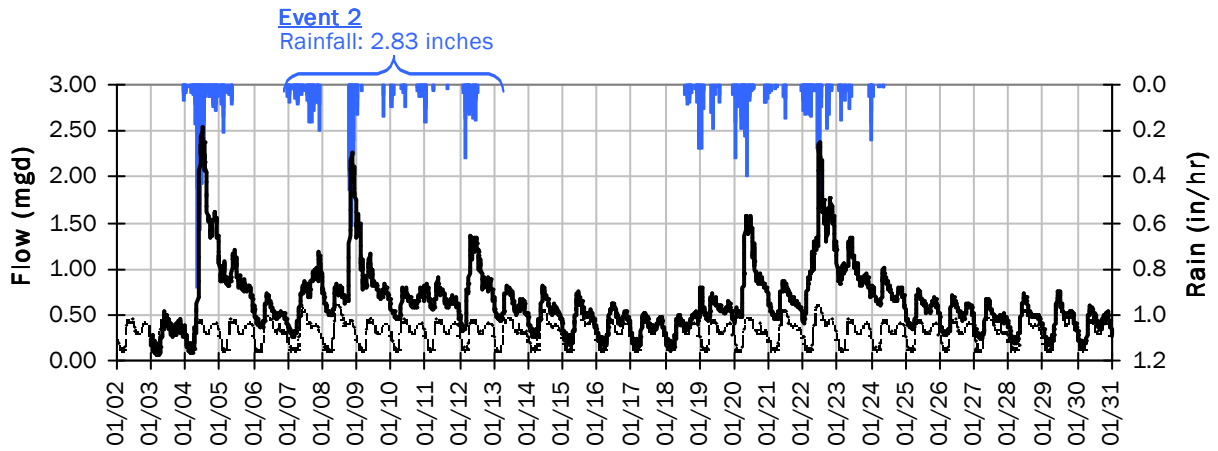


Storm Event I/I Analysis (Rain = 2.88 inches)

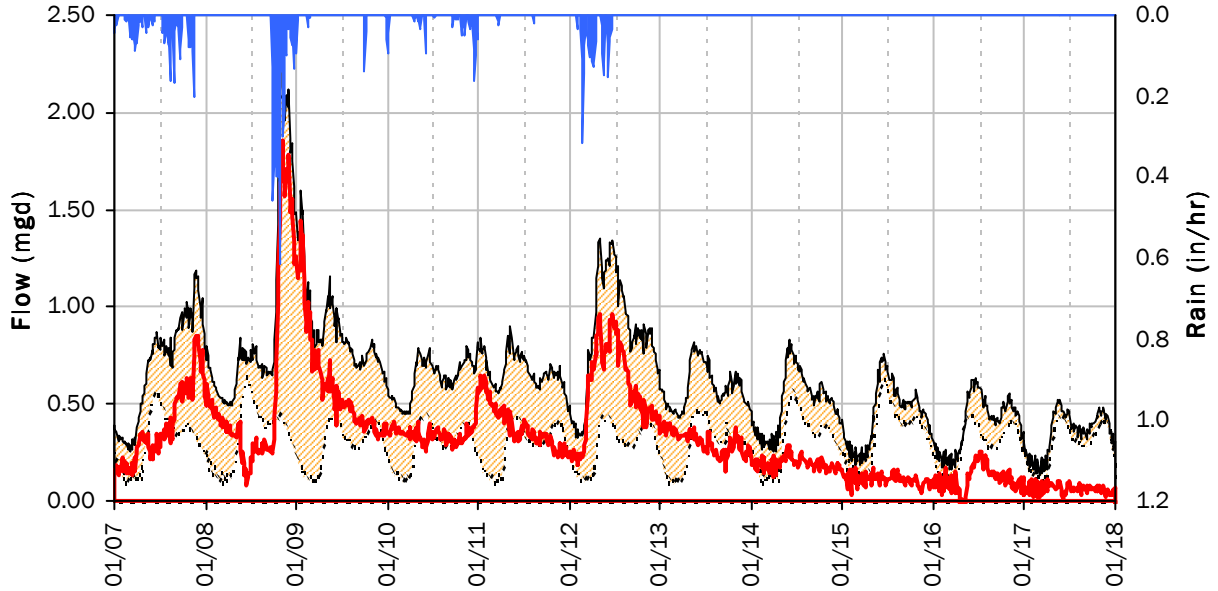
Capacity		Inflow / Infiltration	
Peak Flow:	2.54 *mgd*	**Peak I/I Rate:**	2.15 *mgd*
PF:	8.57	**Total I/I:**	1,709,000 *gallons*
Peak Level:	12.43 *in*		
d/D Ratio:	0.83		

MB 3
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

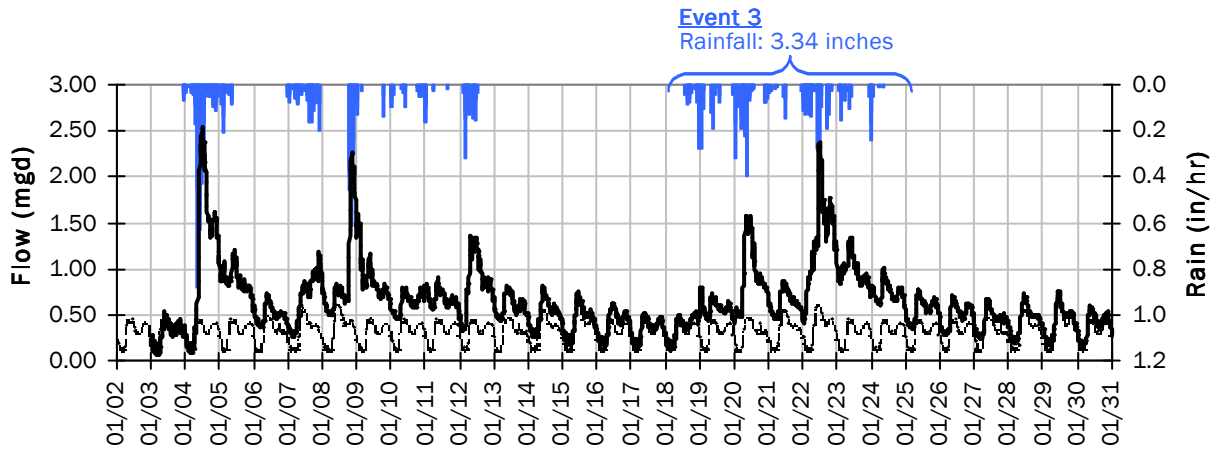


Storm Event I/I Analysis (Rain = 2.83 inches)

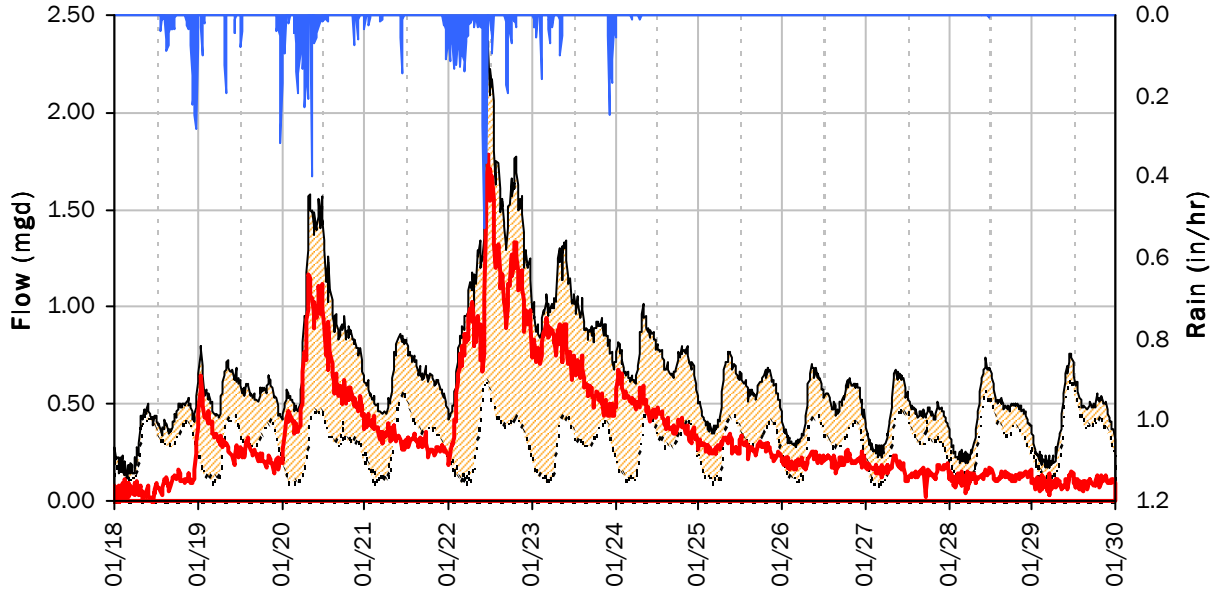
Capacity		Inflow / Infiltration	
Peak Flow:	2.26 *mgd*	**Peak I/I Rate:**	1.86 *mgd*
PF:	7.65	**Total I/I:**	3,668,000 *gallons*
Peak Level:	11.74 *in*		
d/D Ratio:	0.78		

MB 3
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 3 Detail Graph

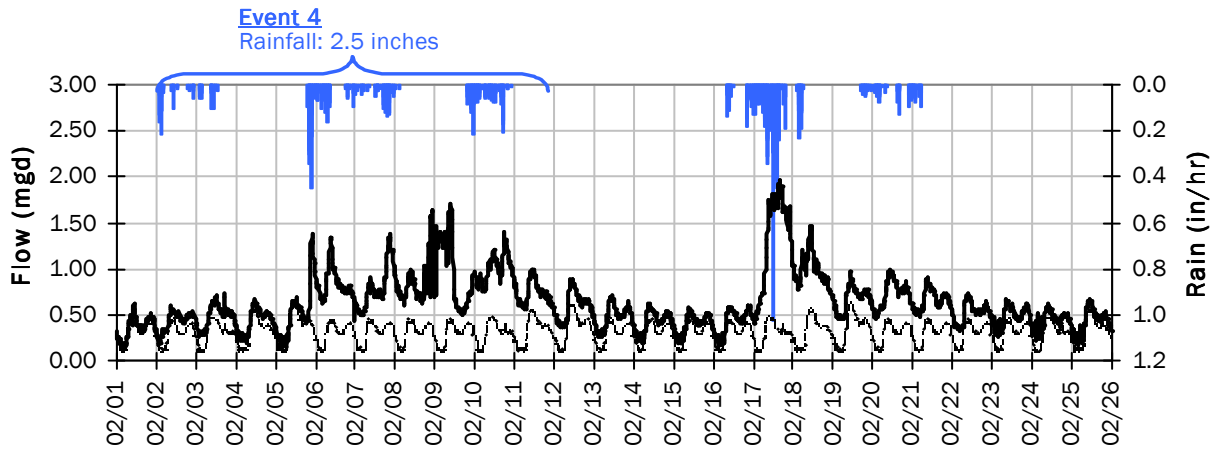


Storm Event I/I Analysis (Rain = 3.35 inches)

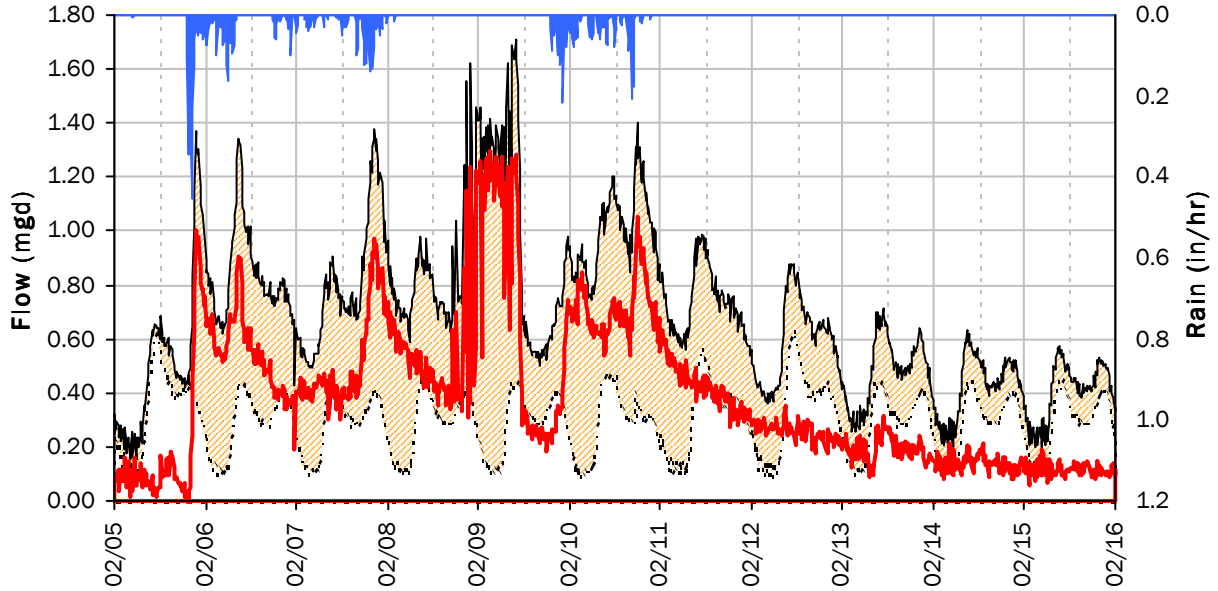
Capacity		Inflow / Infiltration	
Peak Flow:	2.37 *mgd*	**Peak I/I Rate:**	1.78 *mgd*
PF:	8.00	**Total I/I:**	4,318,000 *gallons*
Peak Level:	12.55 *in*		
d/D Ratio:	0.84		

MB 3
I/I Summary: Event 4

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 4 Detail Graph

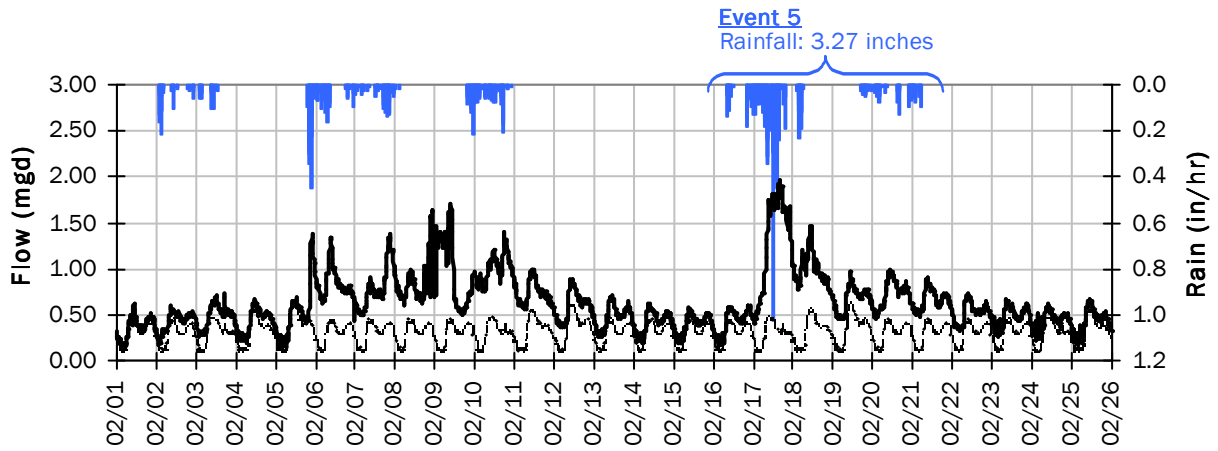


Storm Event I/I Analysis (Rain = 2.50 inches)

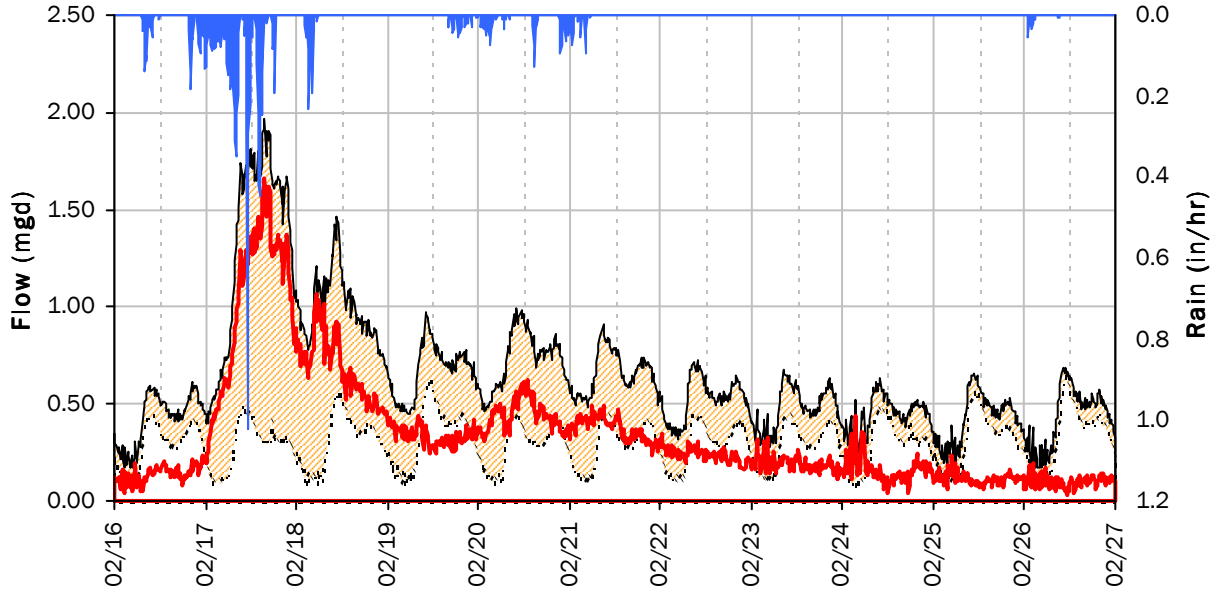
Capacity		Inflow / Infiltration	
Peak Flow:	1.71 *mgd*	**Peak I/I Rate:**	1.30 *mgd*
PF:	5.77	**Total I/I:**	4,344,000 *gallons*
Peak Level:	23.62 *in*		
d/D Ratio:	1.57		

MB 3
I/I Summary: Event 5

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 5 Detail Graph



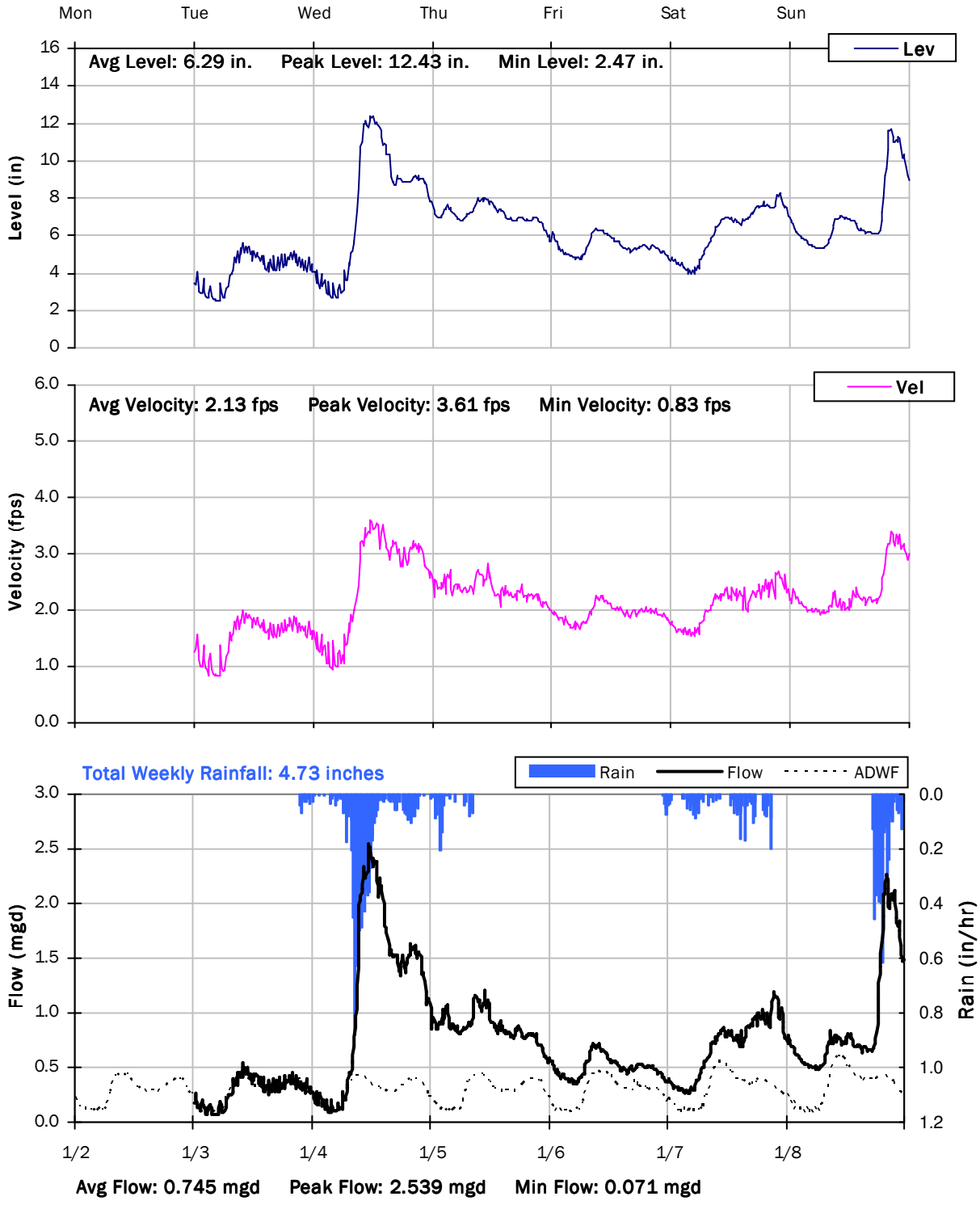
Storm Event I/I Analysis (Rain = 3.28 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	1.97 *mgd*	**Peak I/I Rate:**	1.66 *mgd*
PF:	6.64	**Total I/I:**	3,813,000 *gallons*
Peak Level:	11.14 *in*		
d/D Ratio:	0.74		

MB 3

Weekly Level, Velocity and Flow Hydrographs

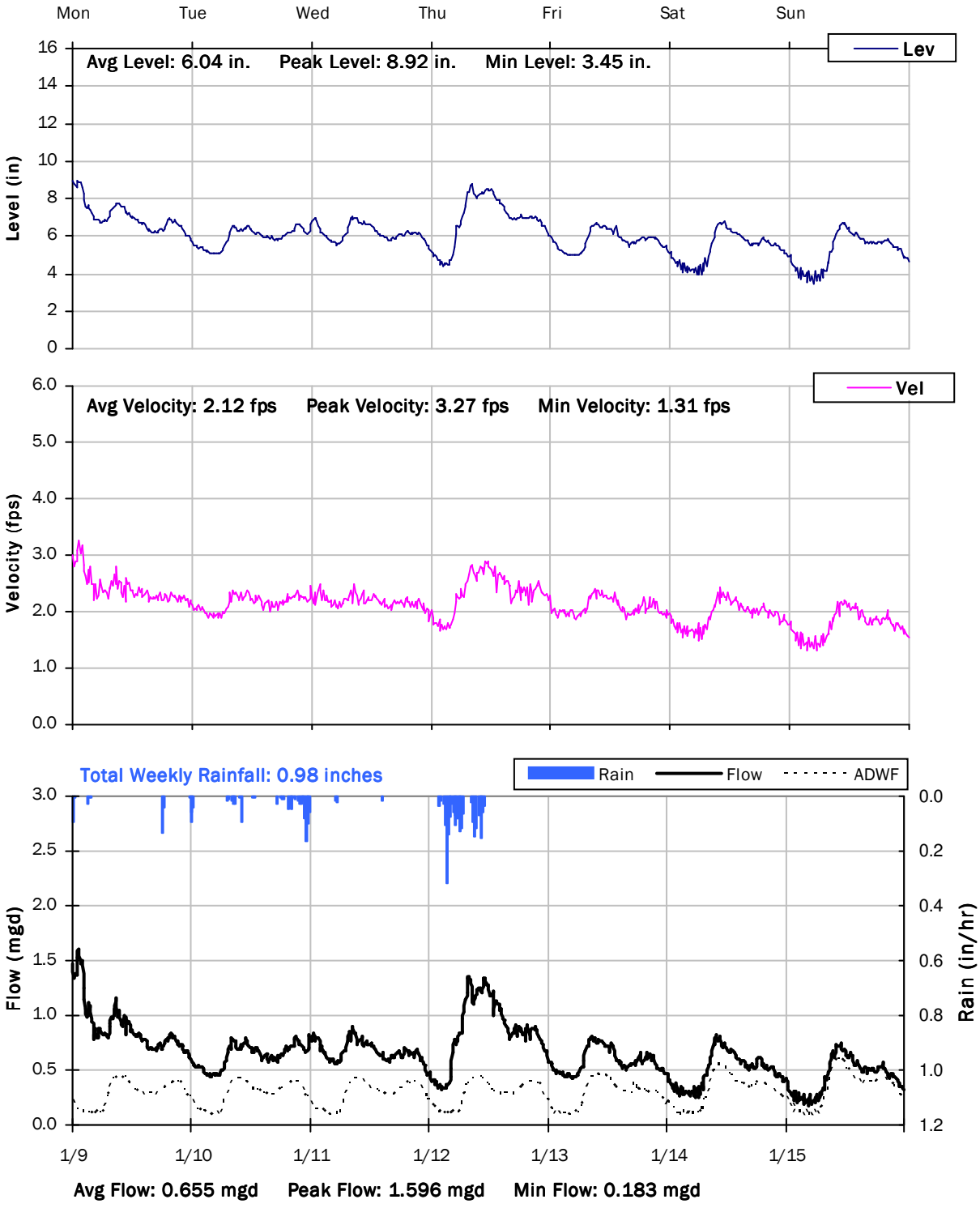
1/2/2017 to 1/9/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

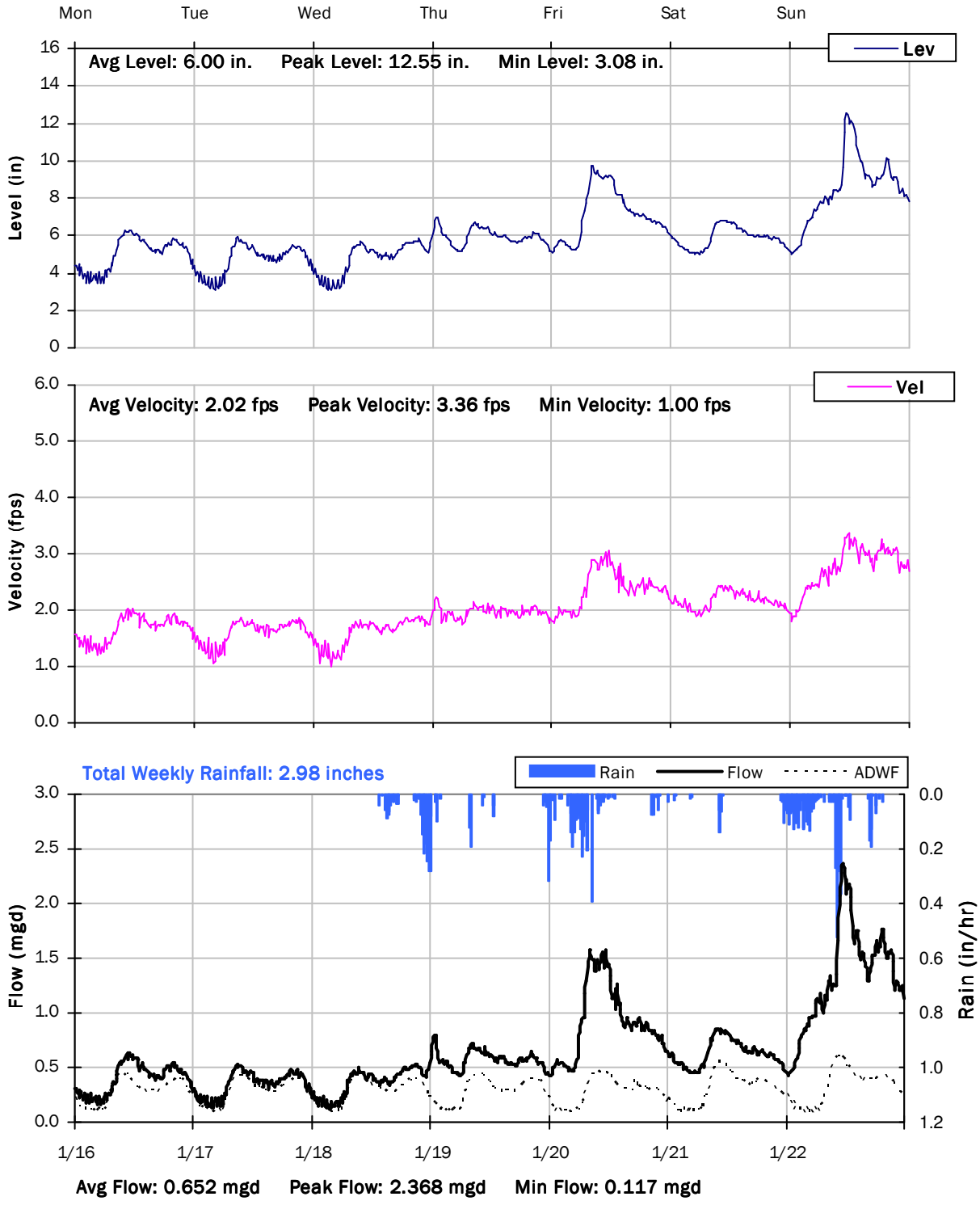
1/9/2017 to 1/16/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

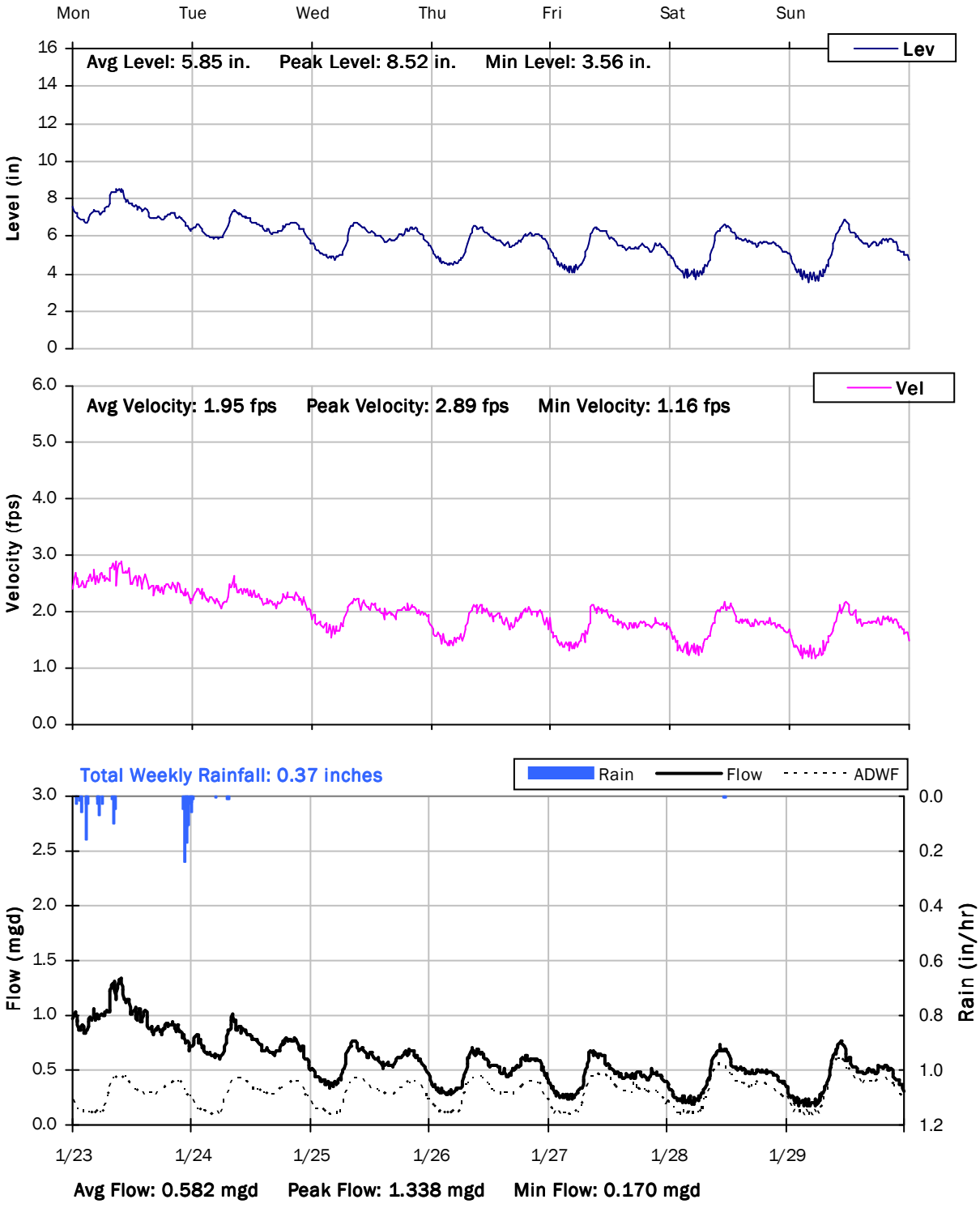
1/16/2017 to 1/23/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

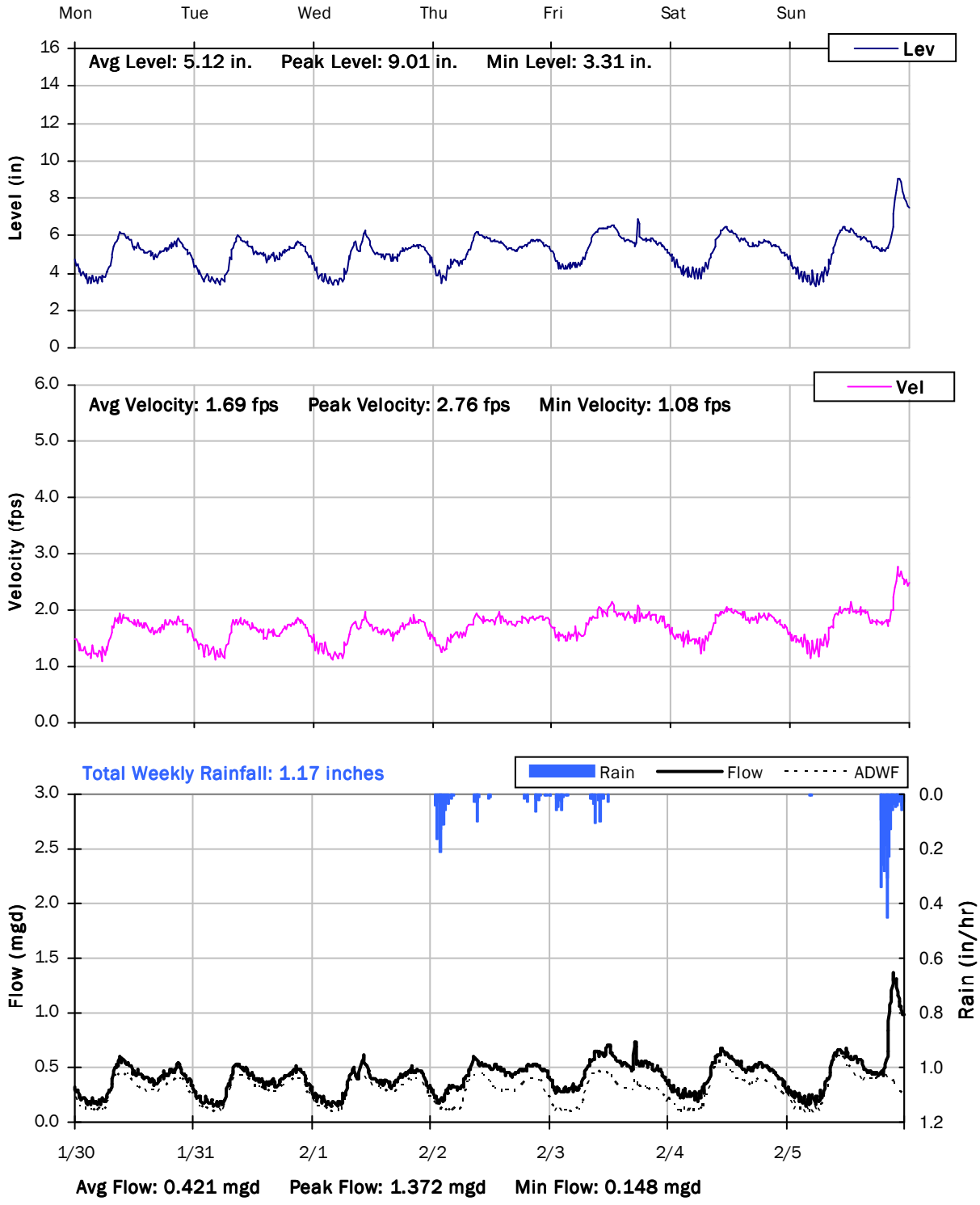
1/23/2017 to 1/30/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

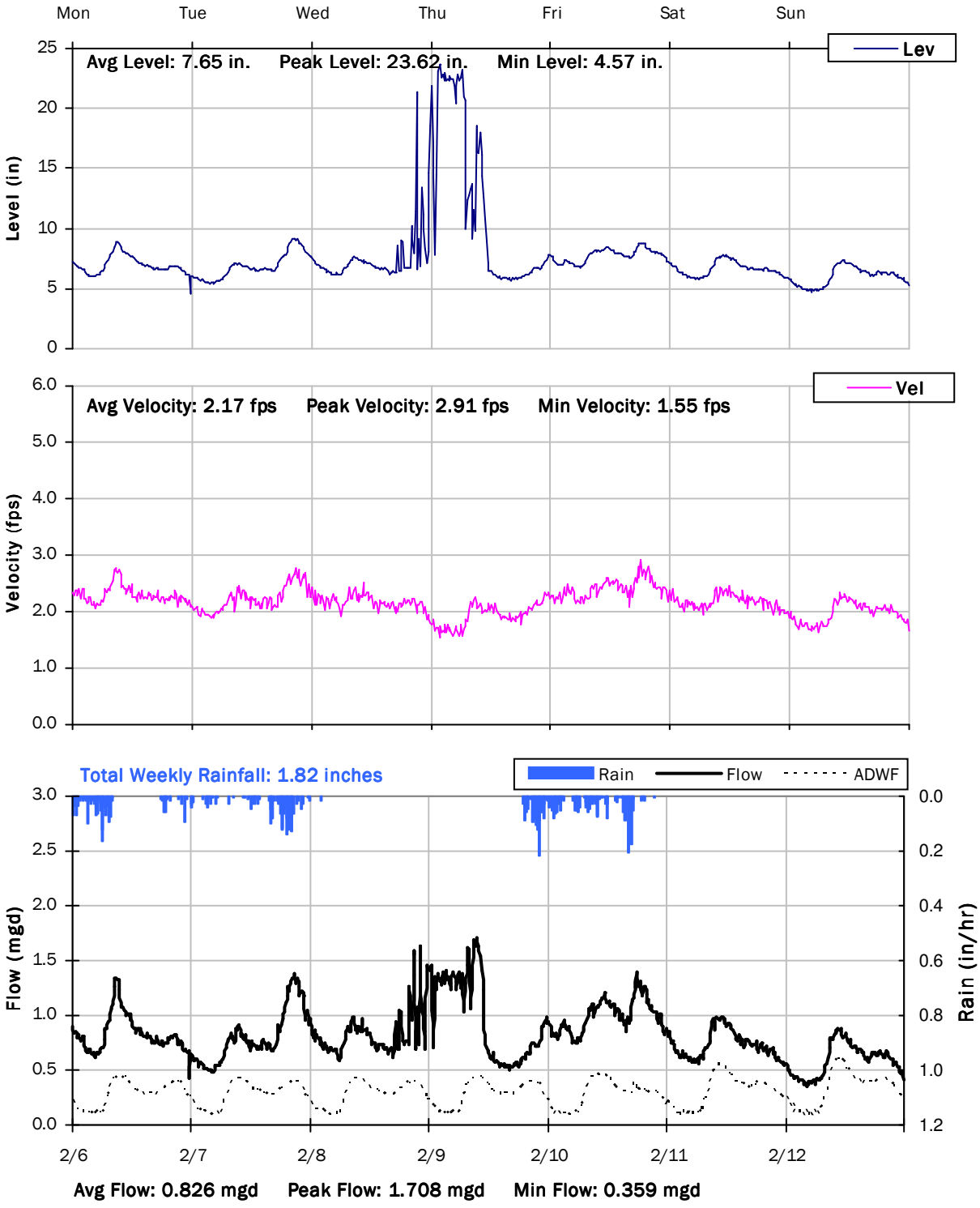
1/30/2017 to 2/6/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

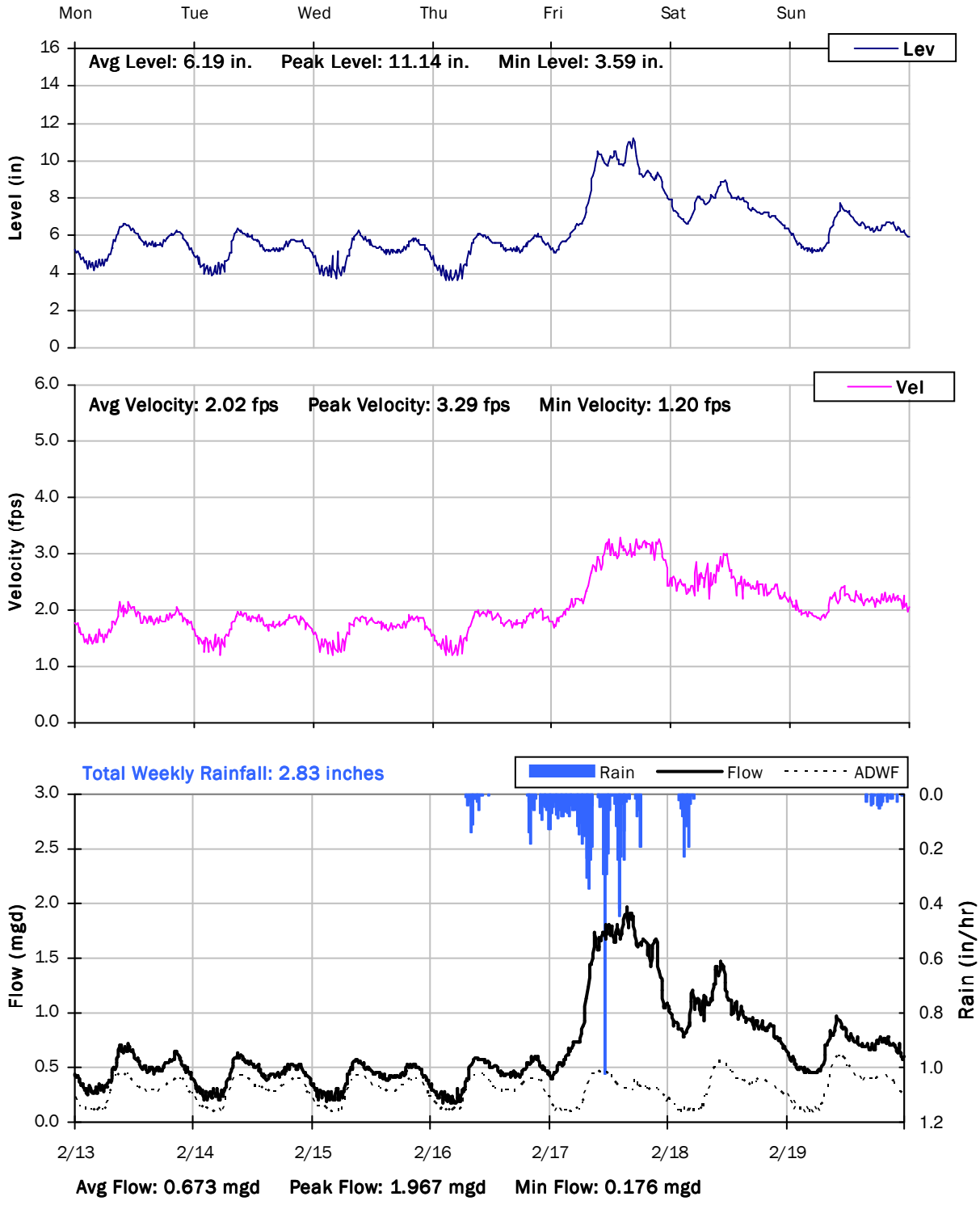
2/6/2017 to 2/13/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

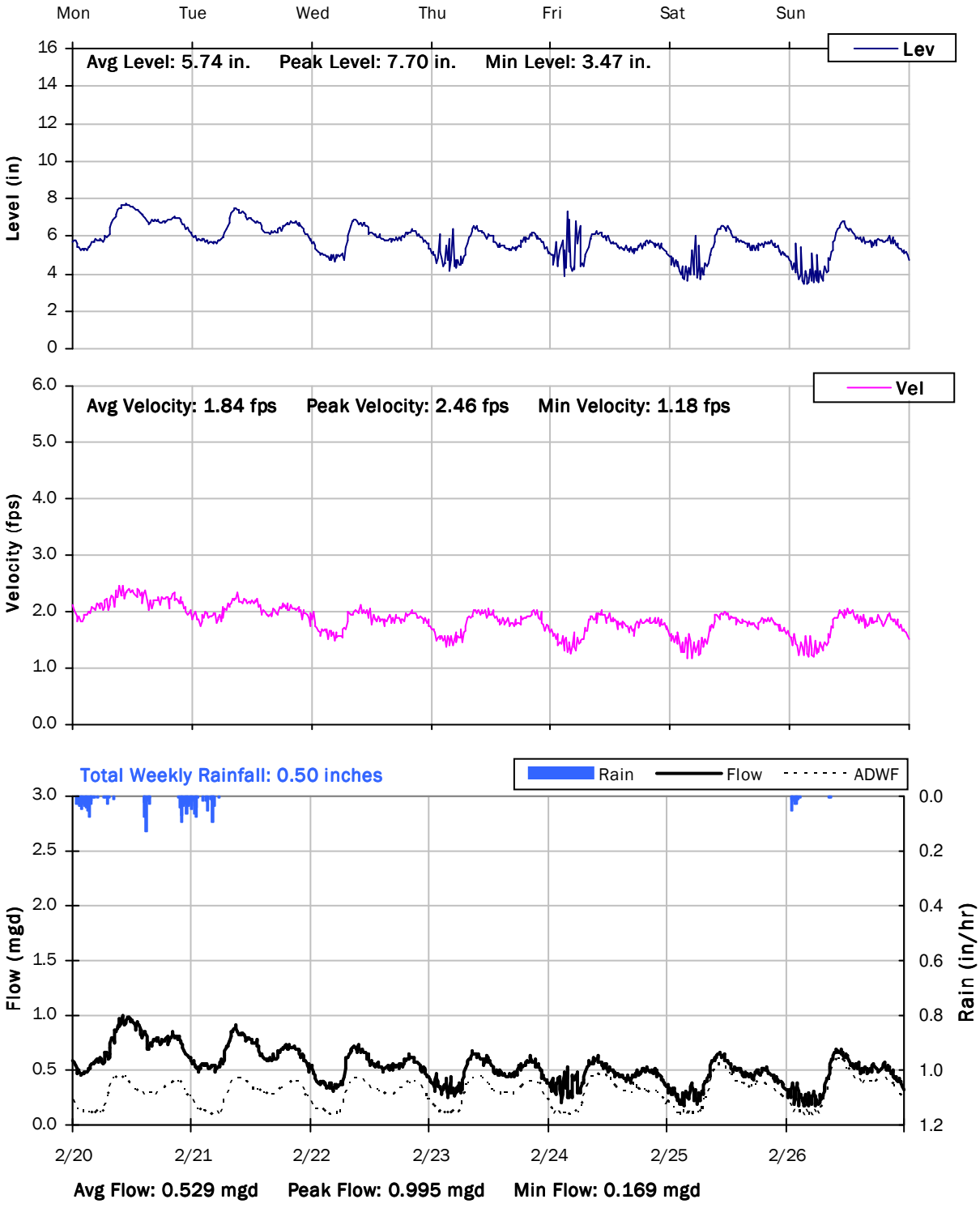
2/13/2017 to 2/20/2017



MB 3

Weekly Level, Velocity and Flow Hydrographs

2/20/2017 to 2/27/2017



City of Morro Bay

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 2017 - February 2017

Monitoring Site: MB 4

Location: City Meter at MH 14.5, Embarcadero ~300 feet south of entrance to Morro Dunes mobile home park, meter on south 18" inlet

Data Summary Report

Vicinity Map: MB 4

MB 4

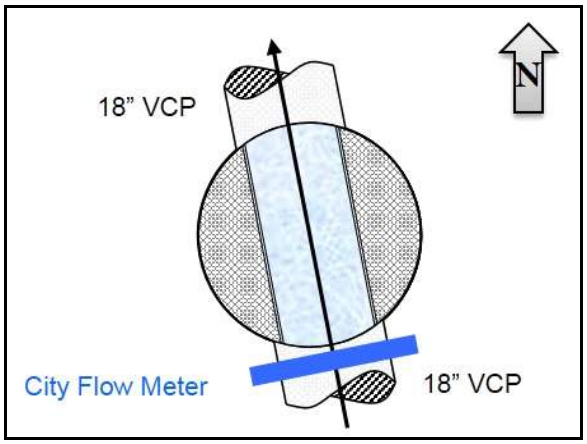
Site Information

Location: City Meter at MH 14.5, Embarcadero ~300 feet south of entrance to Morro Dunes mobile home park, meter on south 18" inlet

Pipe Diameter: 18 inches

ADWF: 0.223 mgd

Peak Measured Flow: 1.023 mgd



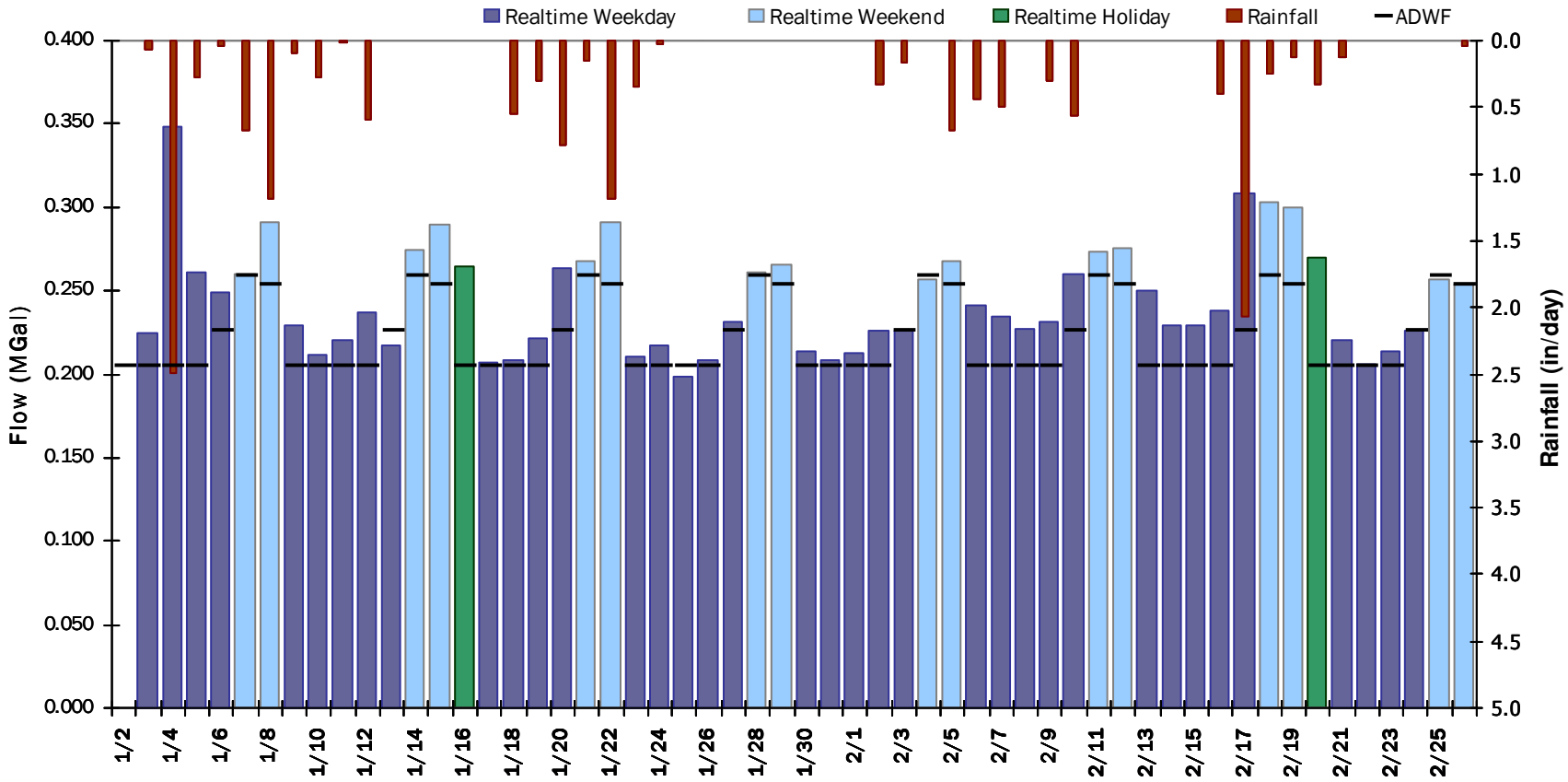
No Photo

MB 4

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.245 MGal **Peak Daily Flow: 0.348 MGal** **Min Daily Flow: 0.199 MGal**

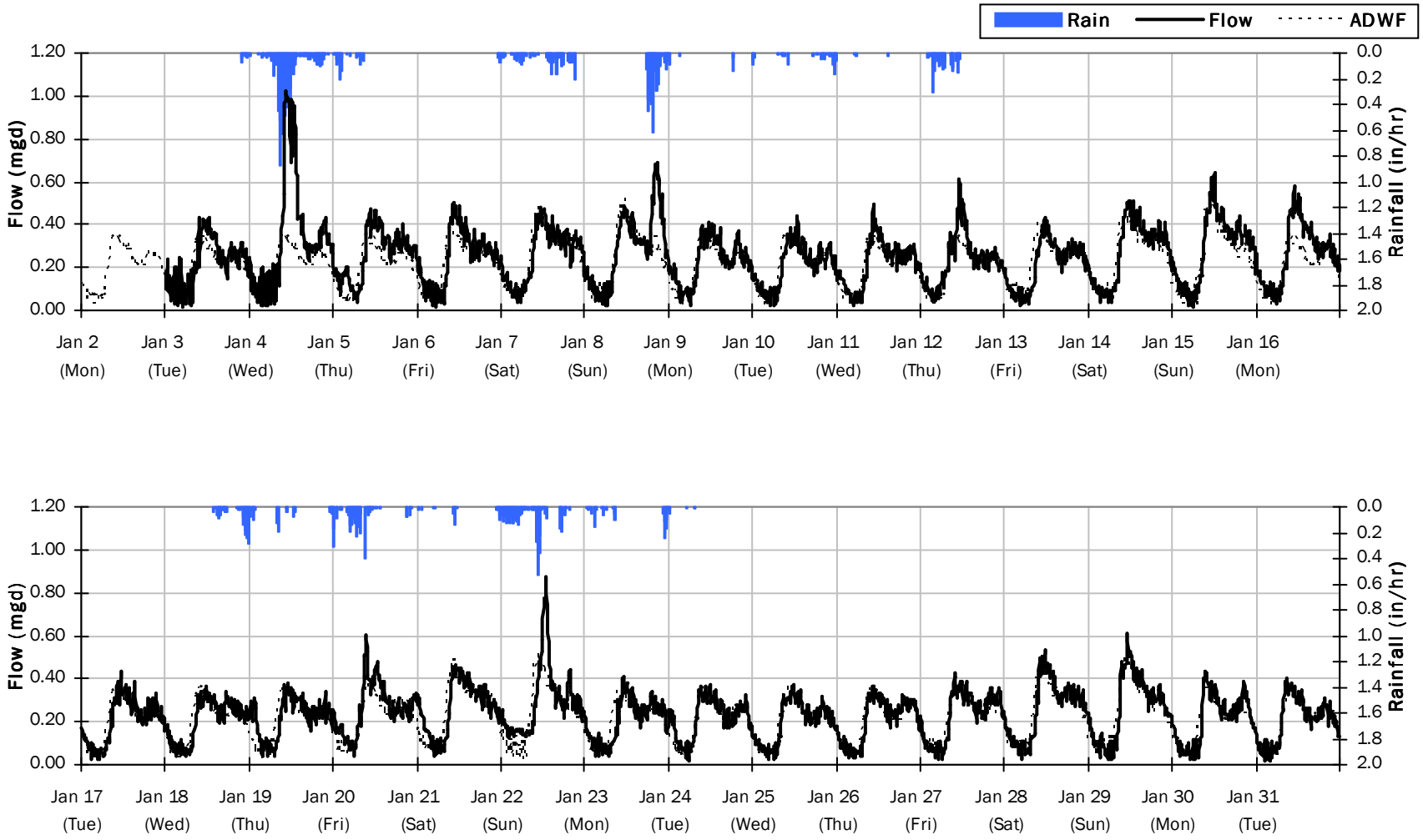
Total Period Rainfall: 15.37 inches



MB 4

Flow Summary: 1/2/2017 to 1/31/2017

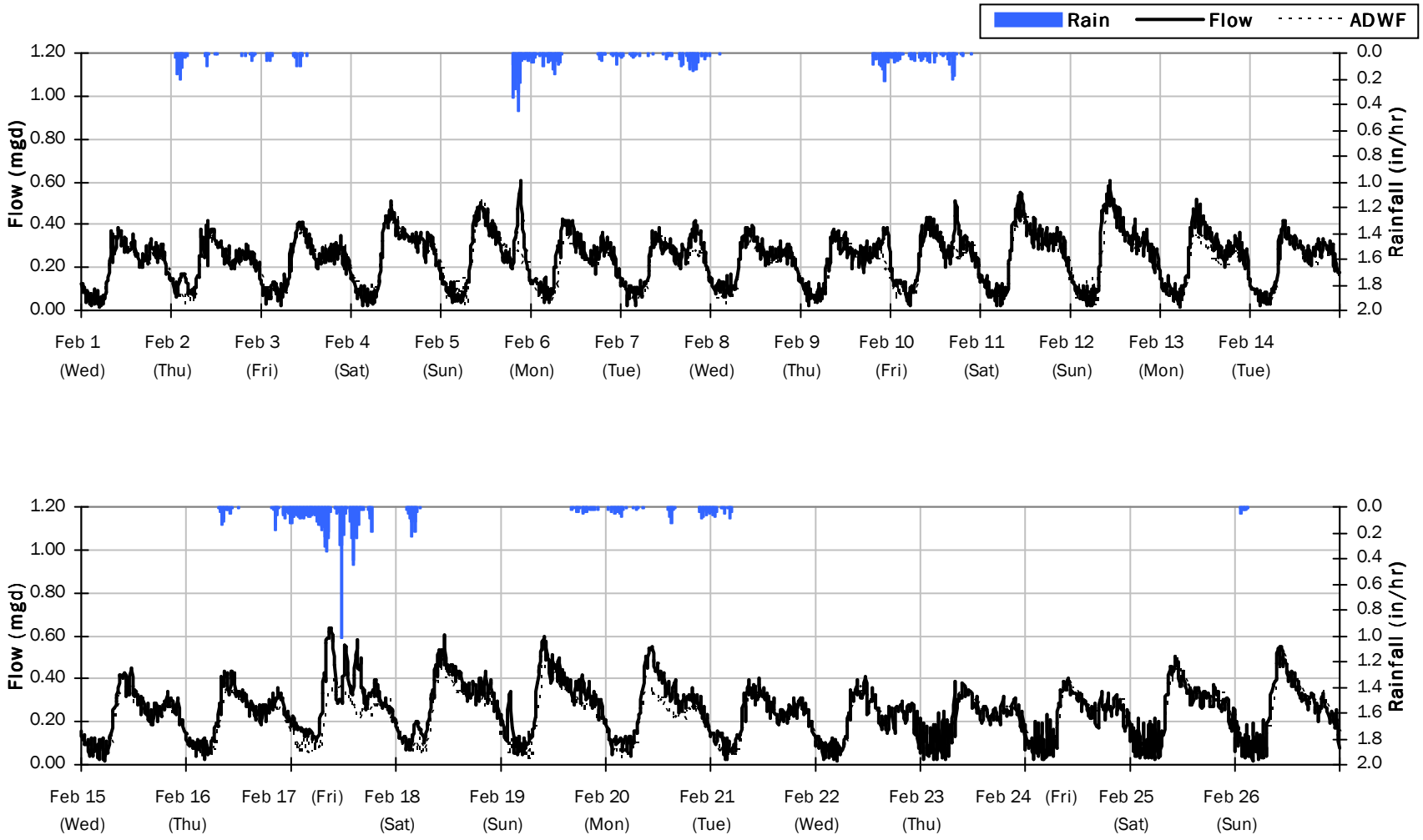
Total Period Rainfall: 9.05 inches **Avg Flow: 0.243 mgd** **Peak Flow: 1.023 mgd** **Min Flow: 0.012 mgd**



MB 4

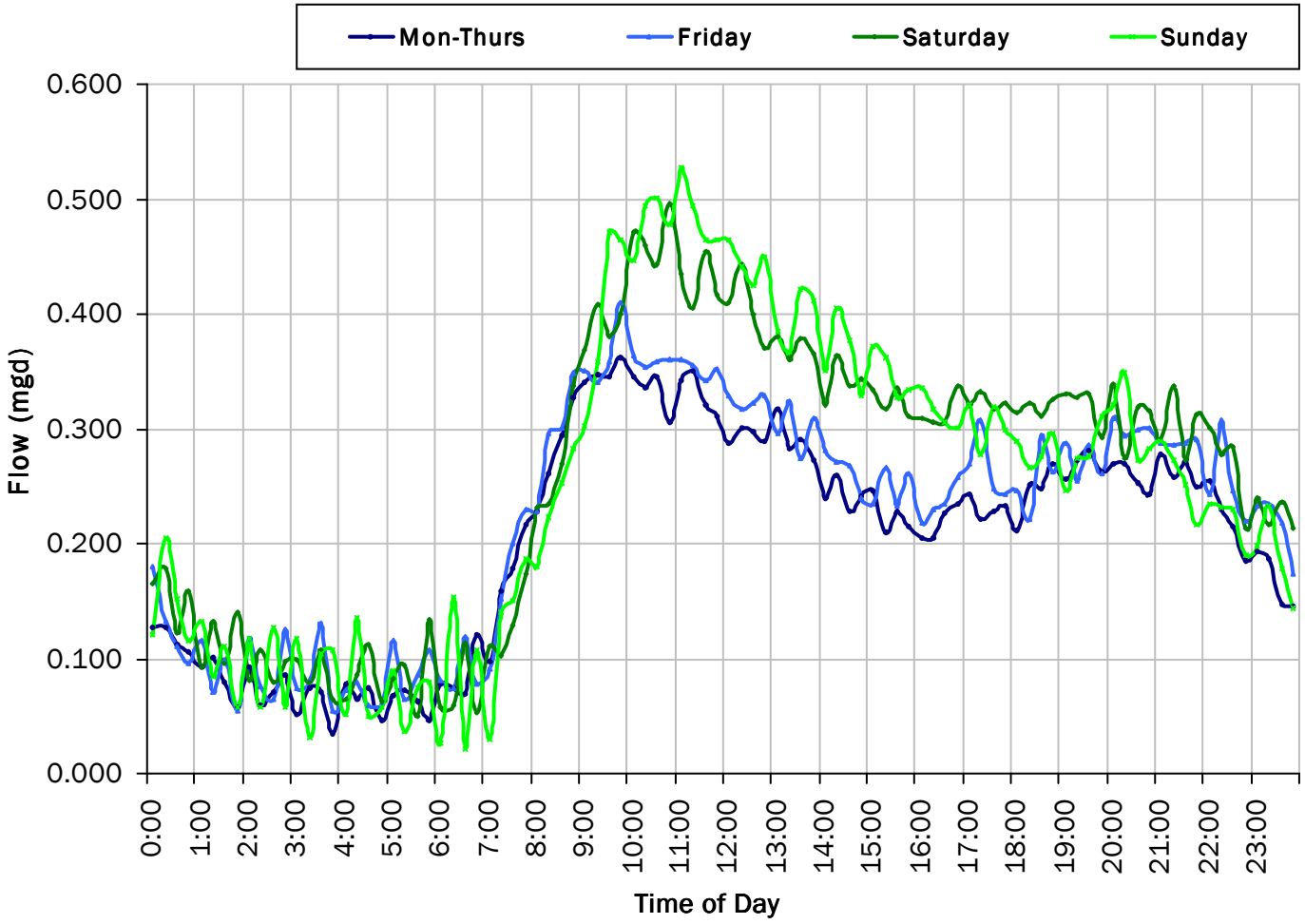
Flow Summary: 2/1/2017 to 2/28/2017

Total Period Rainfall: 6.31 inches **Avg Flow: 0.248 mgd** **Peak Flow: 0.634 mgd** **Min Flow: 0.013 mgd**



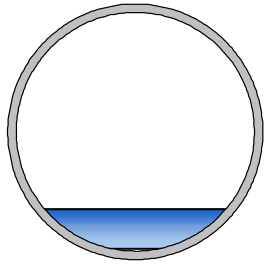
MB 4

Average Dry Weather Flow Hydrographs



ADWF:

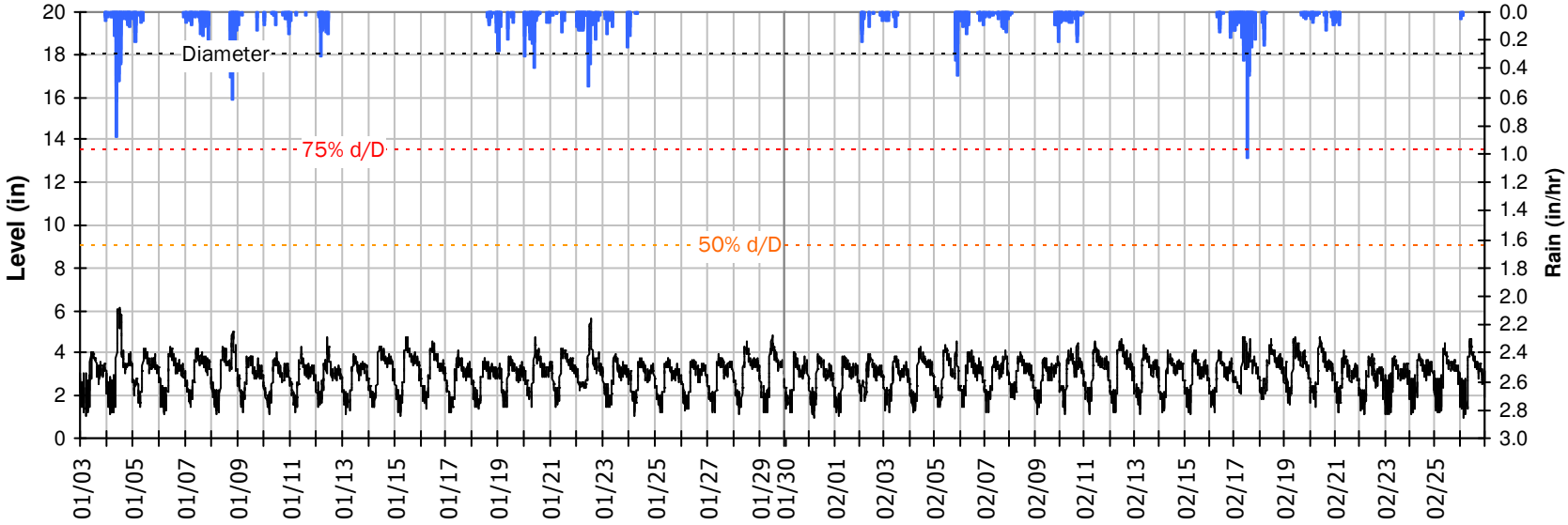
0.223 *mgd*

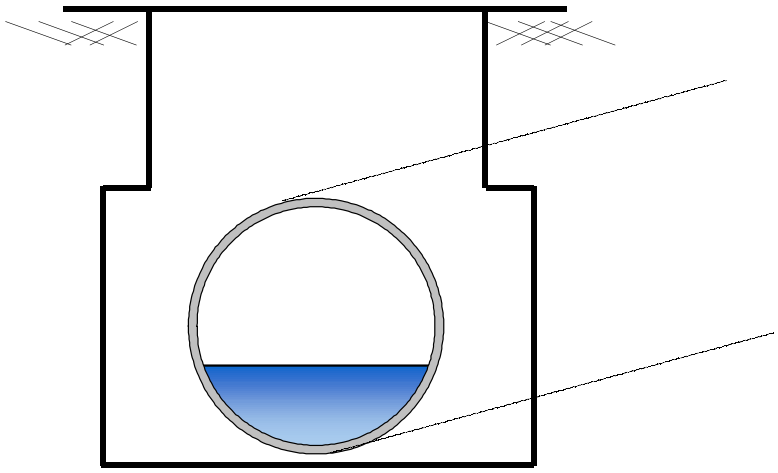


MB 4

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

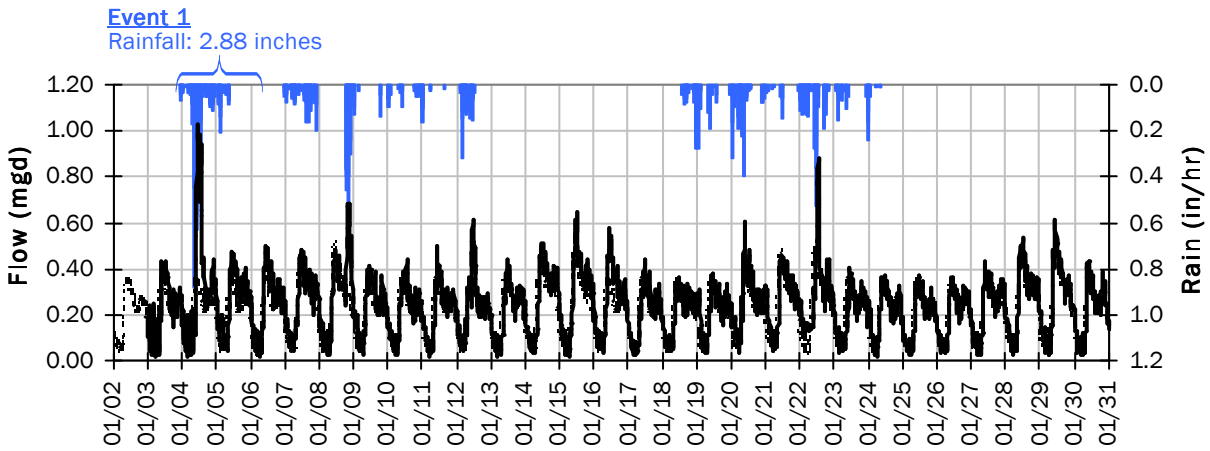




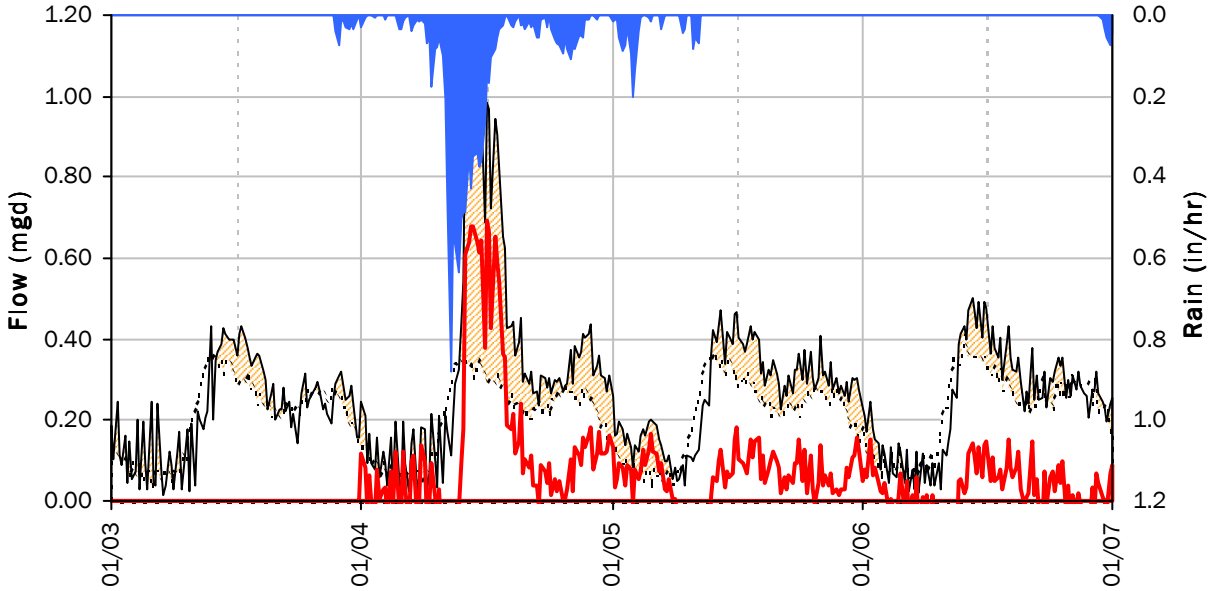
Pipe Diameter:	18	*inches*
Peak Measured Level:	6.1	*inches*
Peak d/D Ratio:	0.34	

MB 4
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

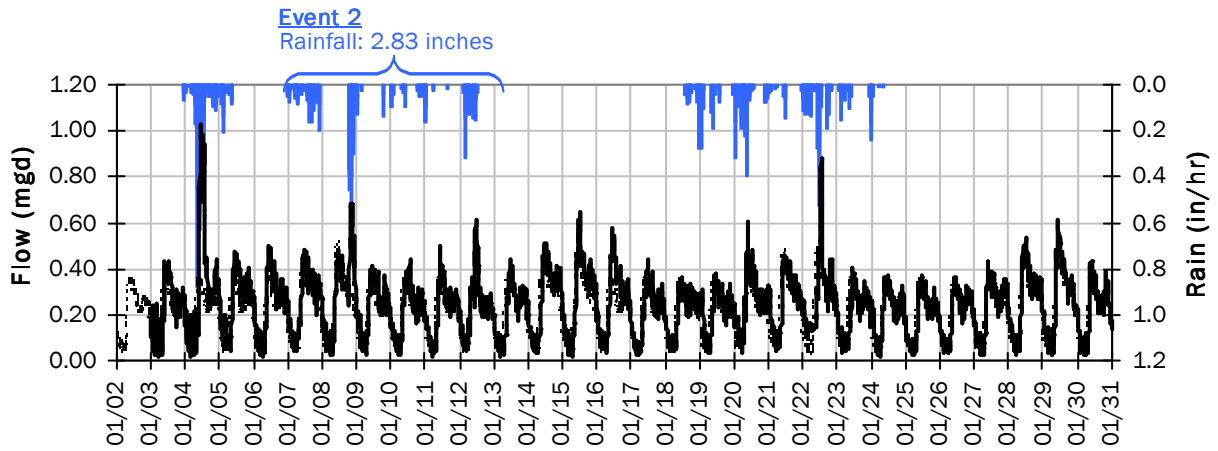


Storm Event I/I Analysis (Rain = 2.88 inches)

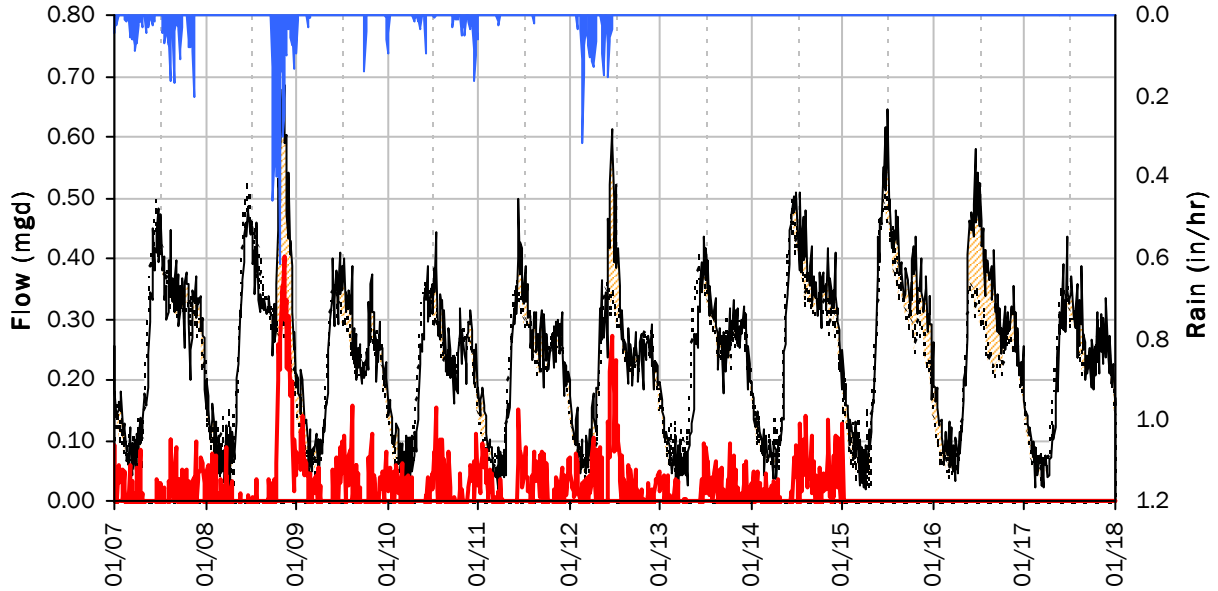
Capacity		Inflow / Infiltration	
Peak Flow:	1.02 *mgd*	**Peak I/I Rate:**	0.70 *mgd*
PF:	4.60	**Total I/I:**	223,000 *gallons*
Peak Level:	6.10 *in*		
d/D Ratio:	0.34		

MB 4
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

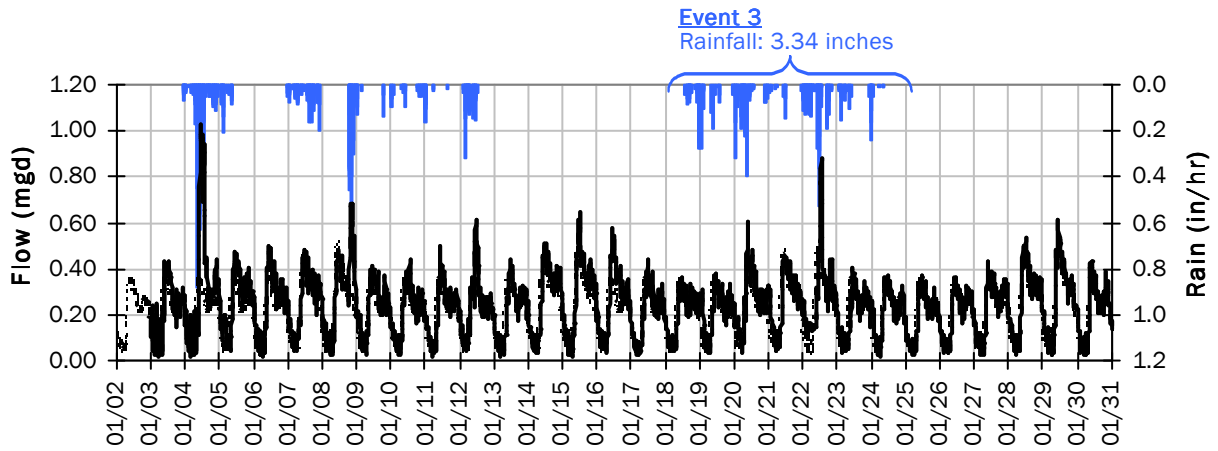


Storm Event I/I Analysis (Rain = 2.83 inches)

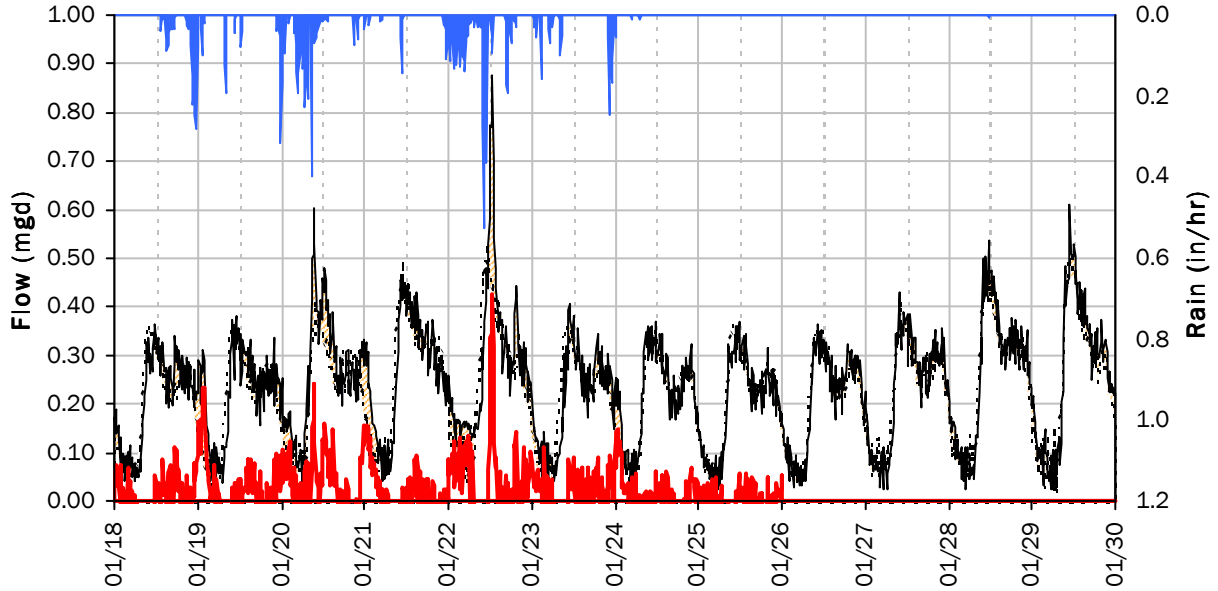
Capacity		Inflow / Infiltration	
Peak Flow:	0.69 *mgd*	**Peak I/I Rate:**	0.40 *mgd*
PF:	3.08	**Total I/I:**	124,000 *gallons*
Peak Level:	5.04 *in*		
d/D Ratio:	0.28		

MB 4
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 3 Detail Graph

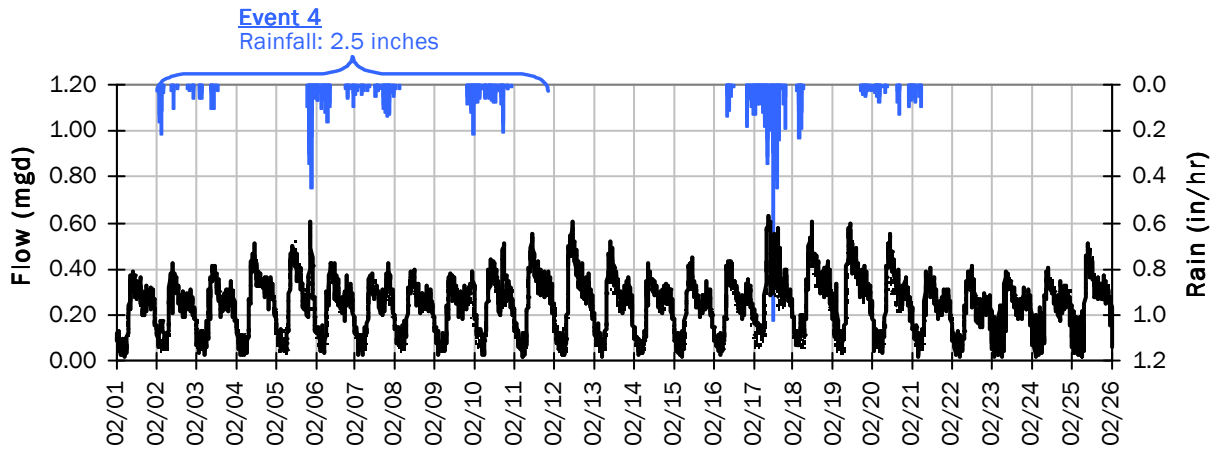


Storm Event I/I Analysis (Rain = 3.35 inches)

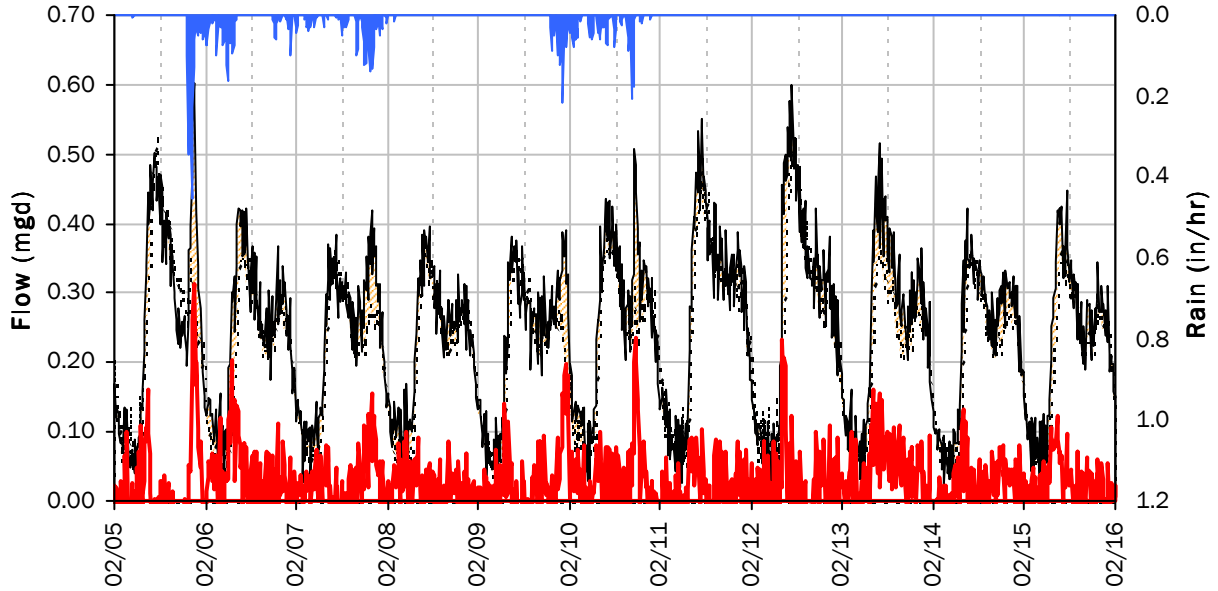
Capacity		Inflow / Infiltration	
Peak Flow:	0.88 *mgd*	**Peak I/I Rate:**	0.43 *mgd*
PF:	3.94	**Total I/I:**	116,000 *gallons*
Peak Level:	5.65 *in*		
d/D Ratio:	0.31		

MB 4
I/I Summary: Event 4

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 4 Detail Graph

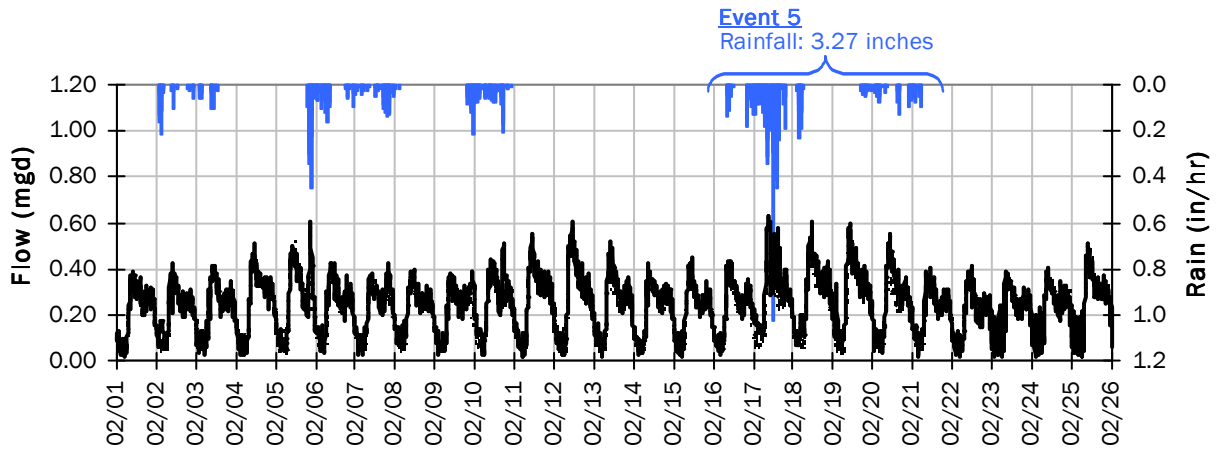


Storm Event I/I Analysis (Rain = 2.50 inches)

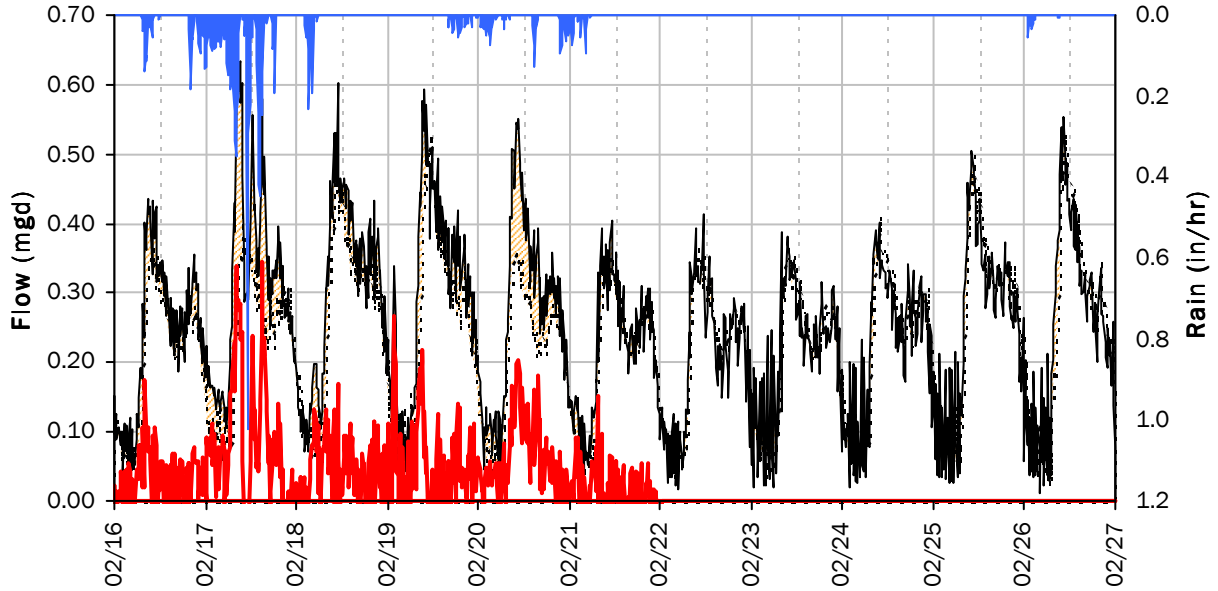
Capacity		Inflow / Infiltration	
Peak Flow:	0.60 *mgd*	**Peak I/I Rate:**	0.31 *mgd*
PF:	2.71	**Total I/I:**	293,000 *gallons*
Peak Level:	4.66 *in*		
d/D Ratio:	0.26		

MB 4
I/I Summary: Event 5

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 5 Detail Graph



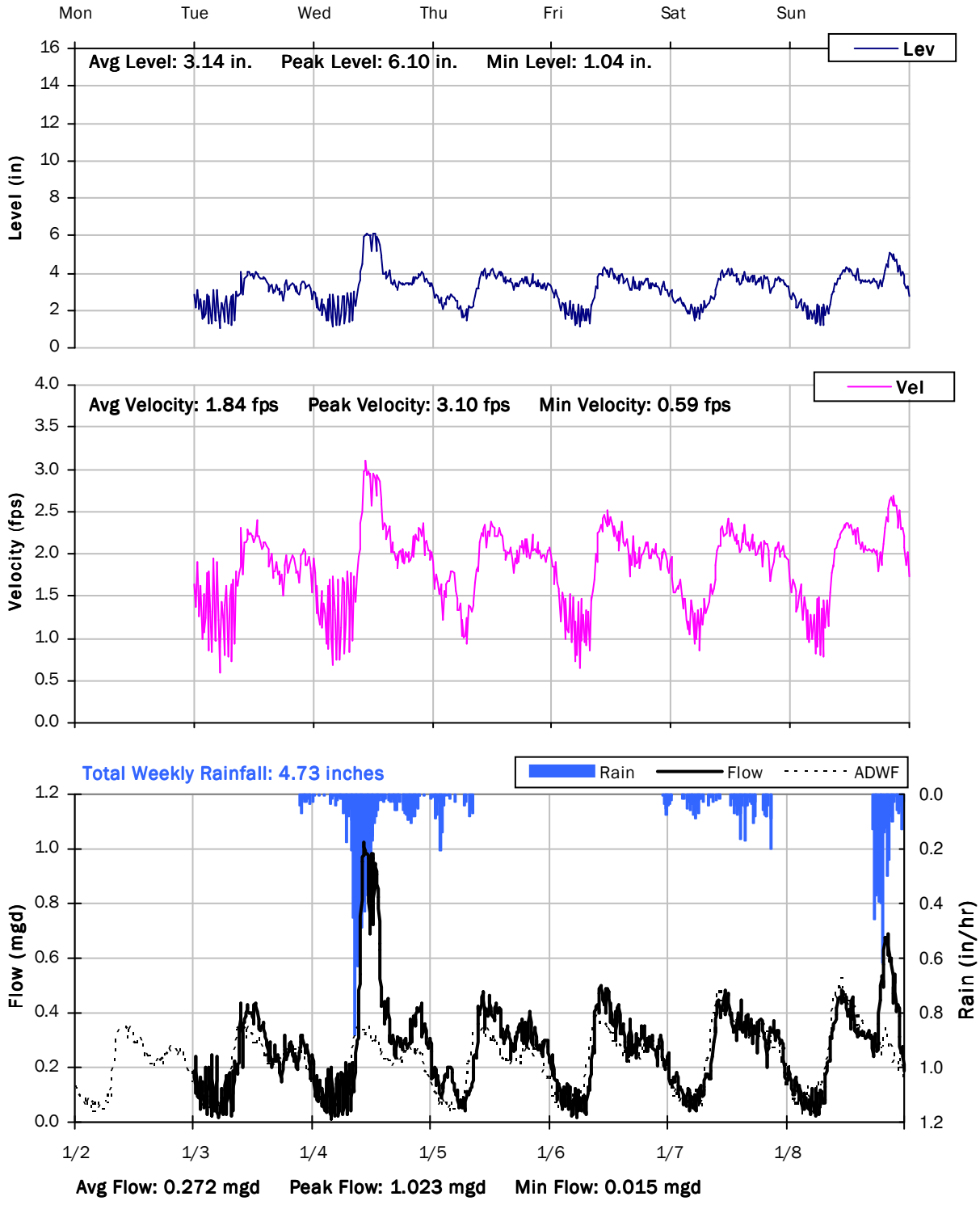
Storm Event I/I Analysis (Rain = 3.28 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.63 *mgd*	**Peak I/I Rate:**	0.34 *mgd*
PF:	2.85	**Total I/I:**	287,000 *gallons*
Peak Level:	4.77 *in*		
d/D Ratio:	0.27		

MB 4

Weekly Level, Velocity and Flow Hydrographs

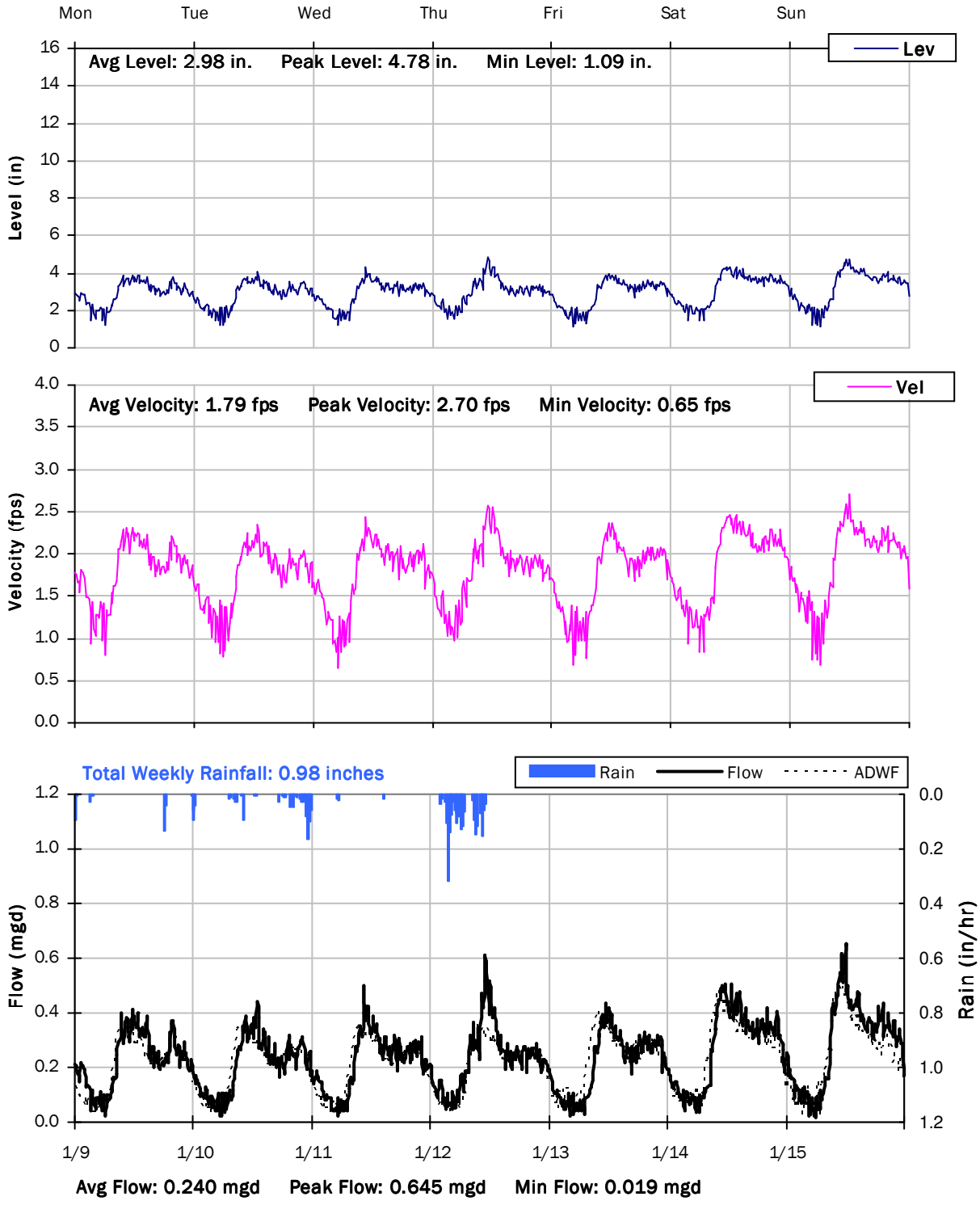
1/2/2017 to 1/9/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

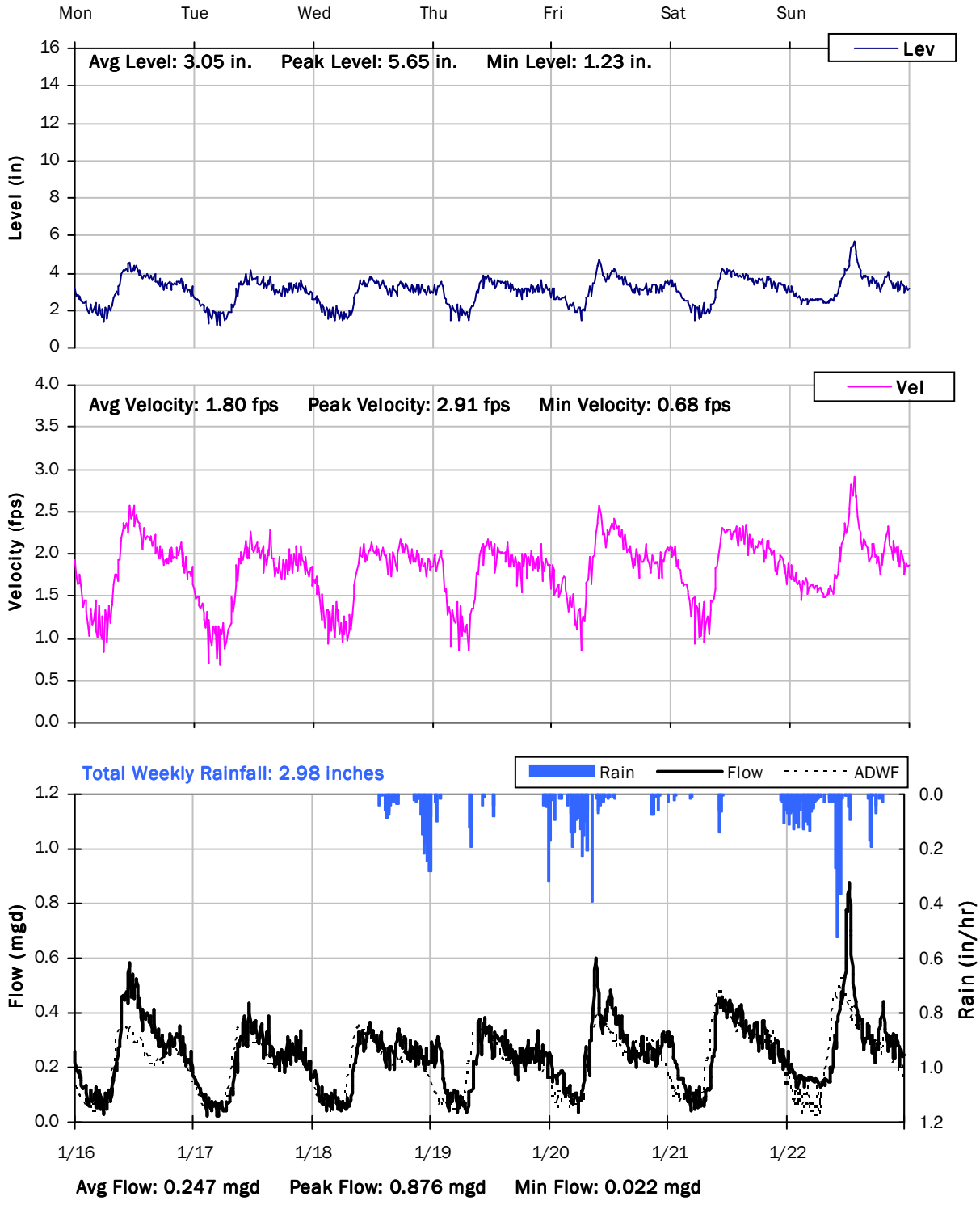
1/9/2017 to 1/16/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

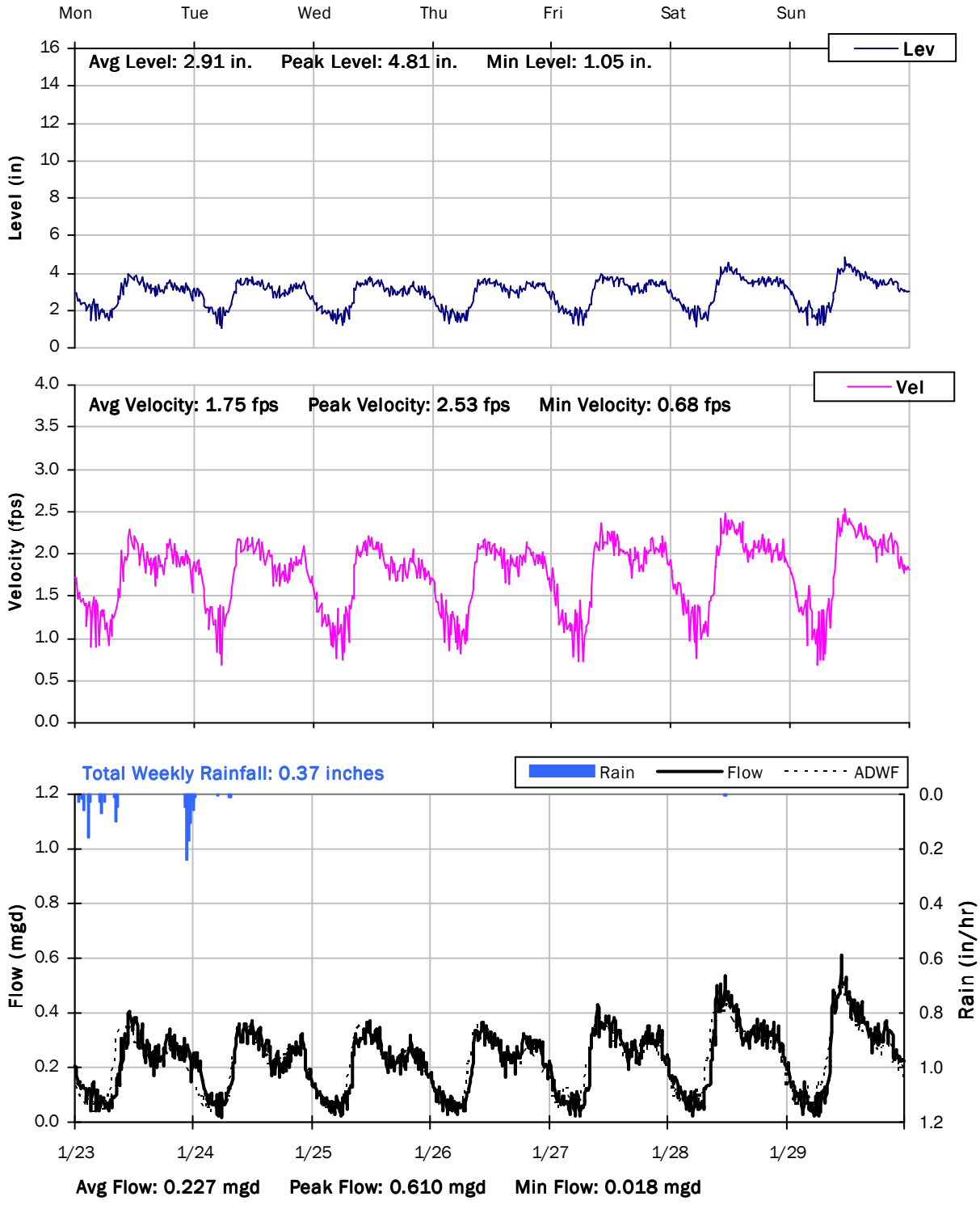
1/16/2017 to 1/23/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

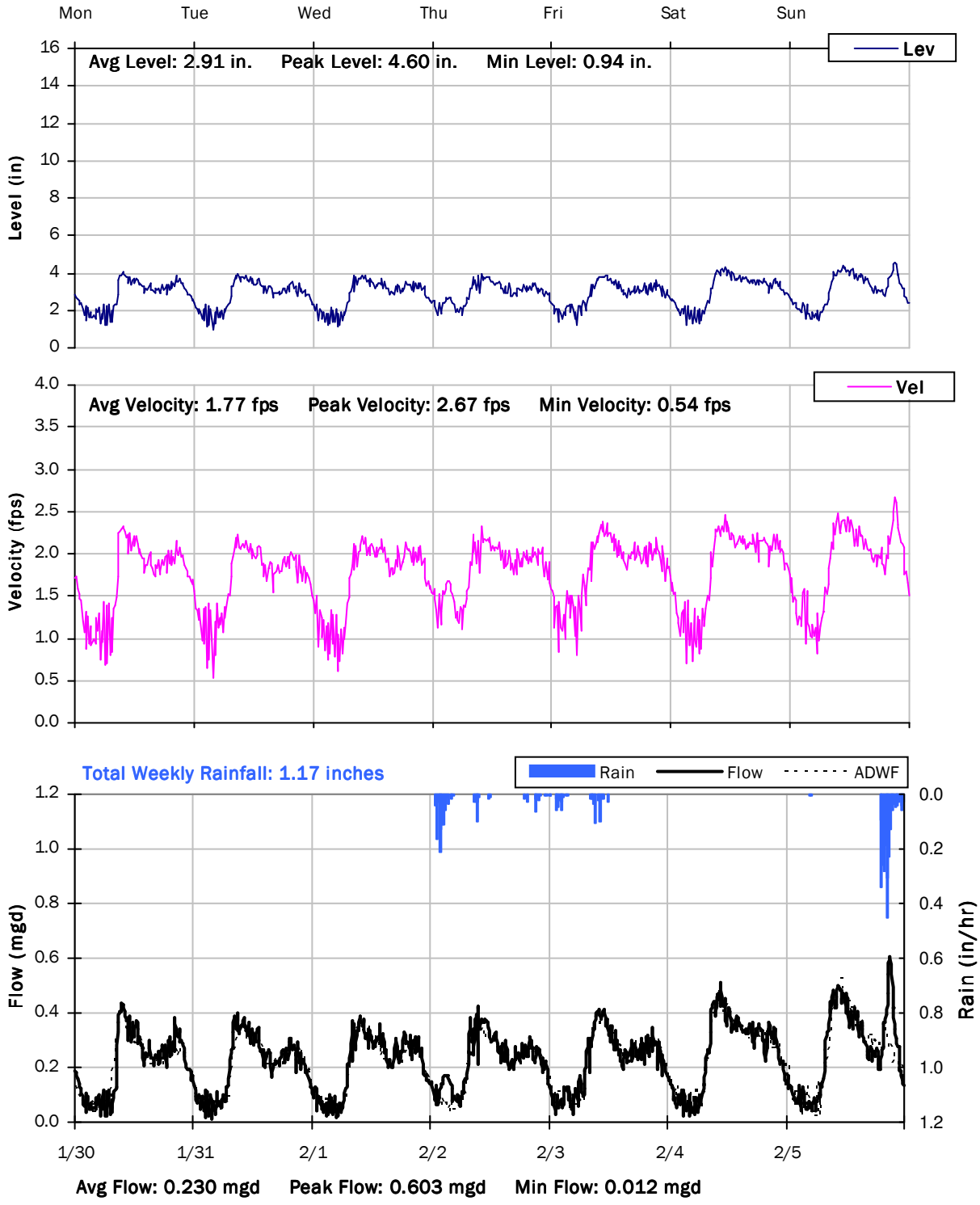
1/23/2017 to 1/30/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

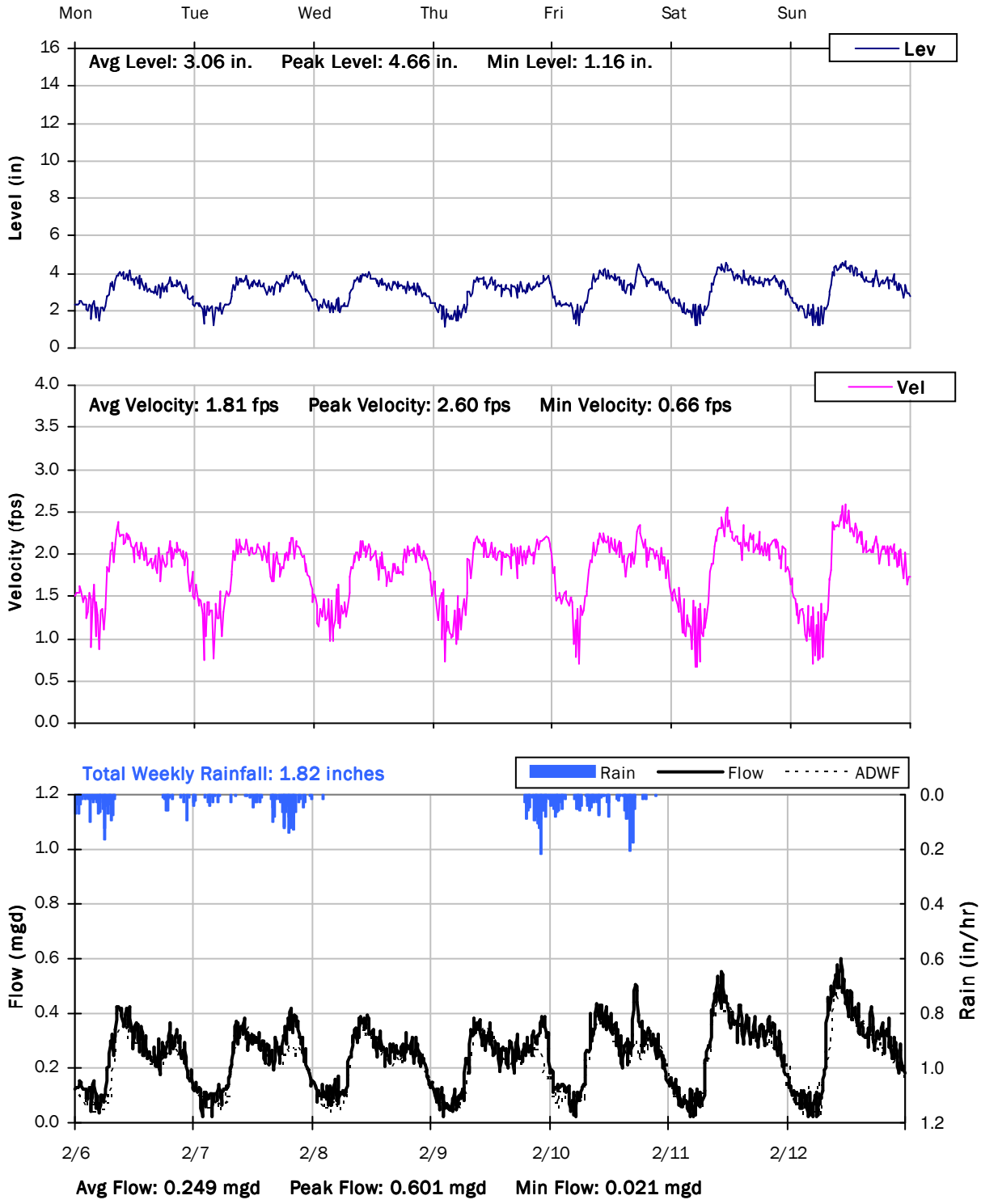
1/30/2017 to 2/6/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

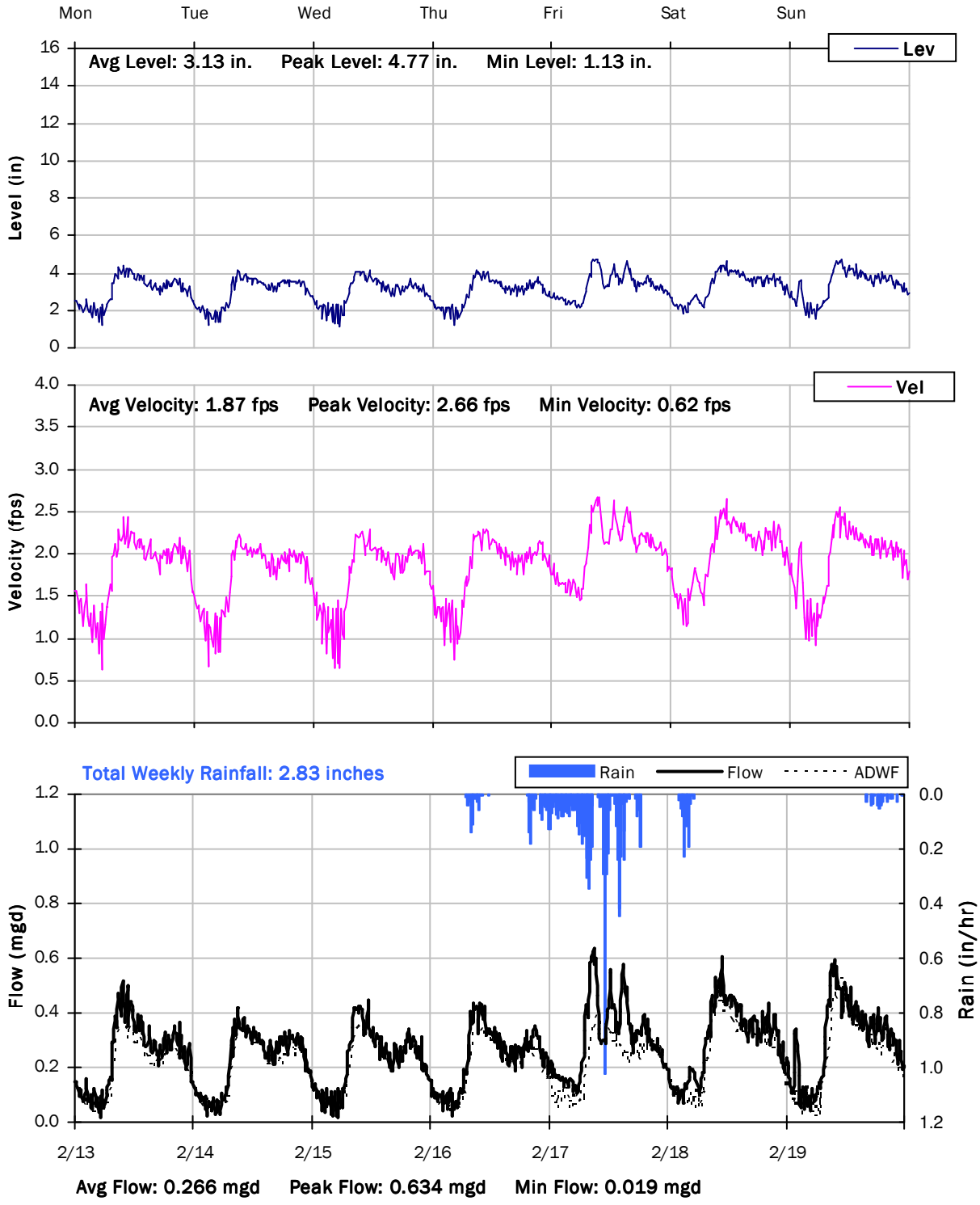
2/6/2017 to 2/13/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

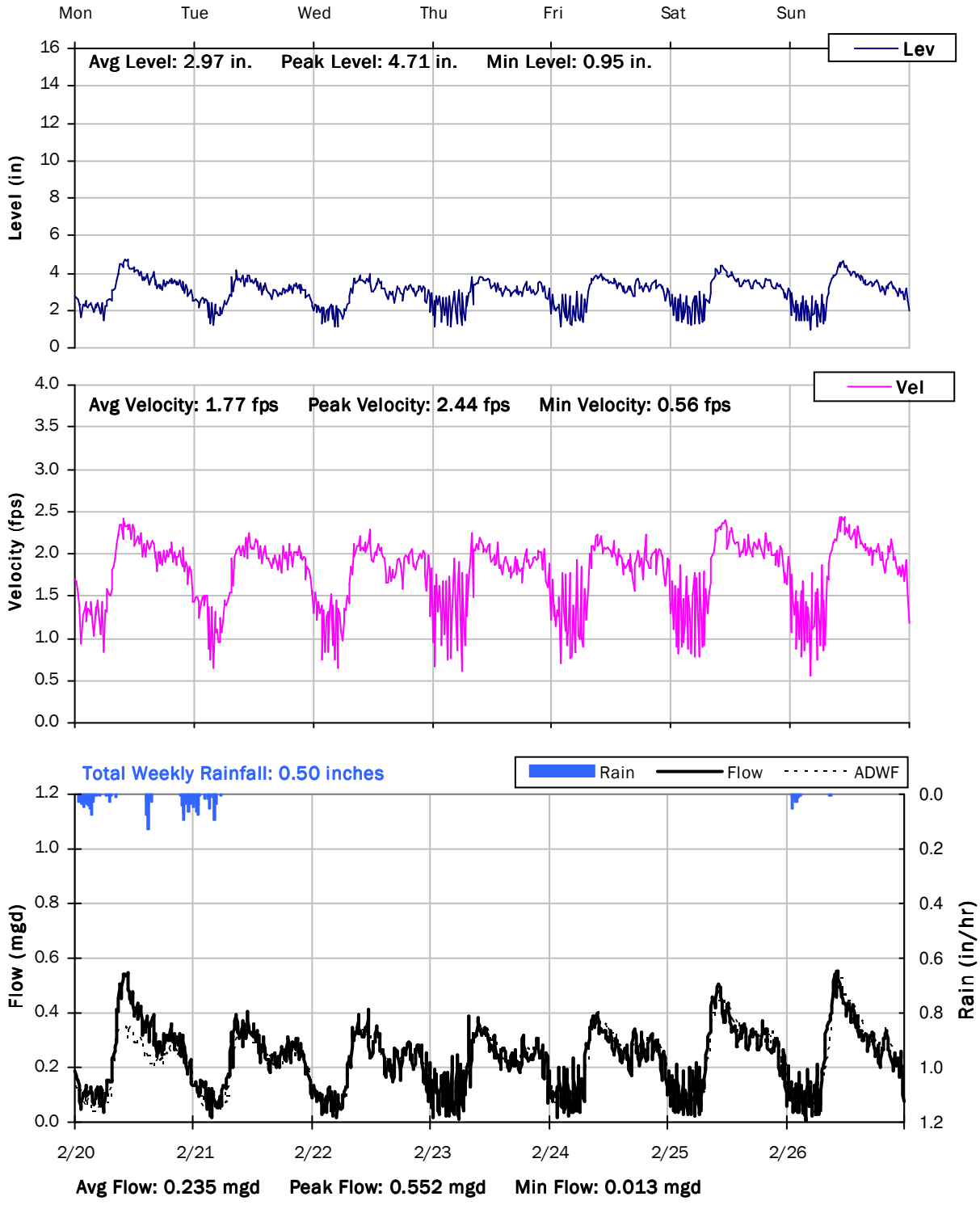
2/13/2017 to 2/20/2017



MB 4

Weekly Level, Velocity and Flow Hydrographs

2/20/2017 to 2/27/2017



City of Morro Bay

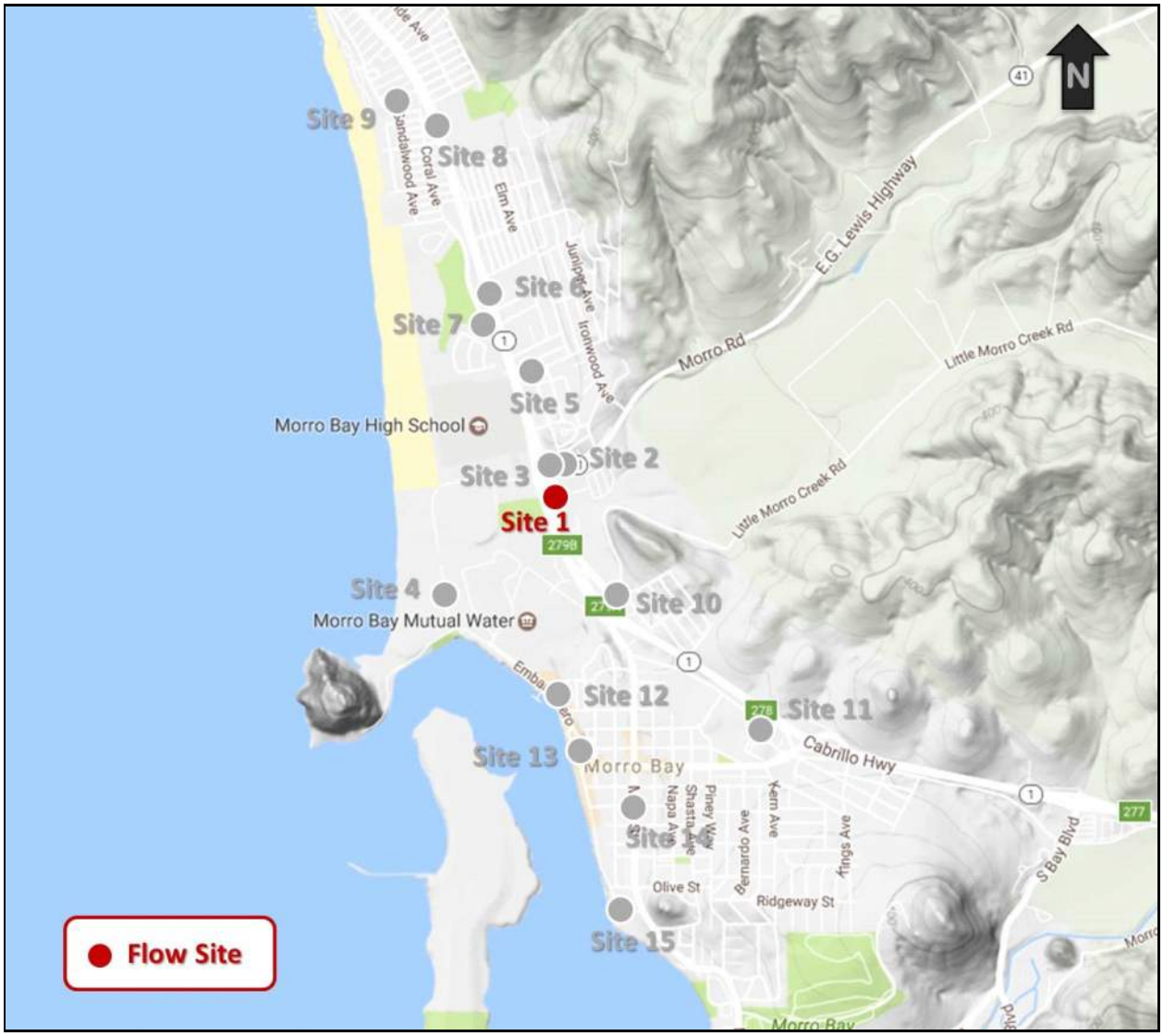
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 01

Location: Intersection of Main Street and Errol Street, meter on south 11.75" inlet

Data Summary Report



Vicinity Map: Site 01

SITE 01

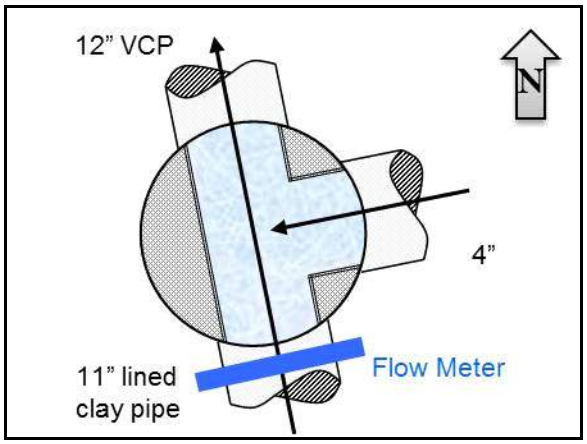
Site Information

Location: Intersection of Main Street and Errol Street, meter on south 11.75" inlet

Pipe Diameter: 11.75 inches

ADWF: 0.182 mgd

Peak Measured Flow: 0.479 mgd



SITE 01

Additional Site Photos

SITE 01

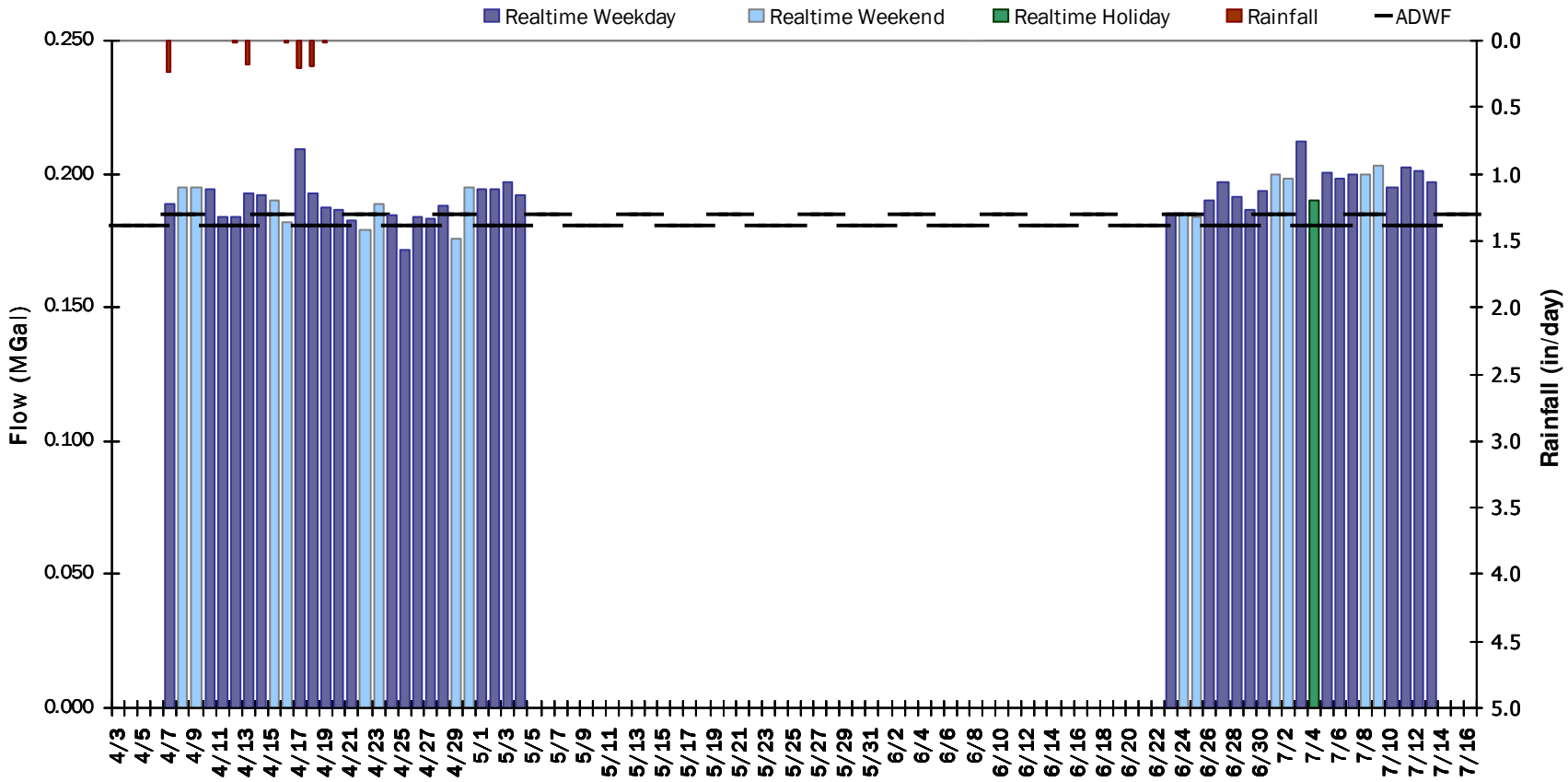
Additional Site Photos

SITE 01

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.192 MGal Peak Daily Flow: 0.212 MGal Min Daily Flow: 0.172 MGal

Total Period Rainfall: 0.85 inches

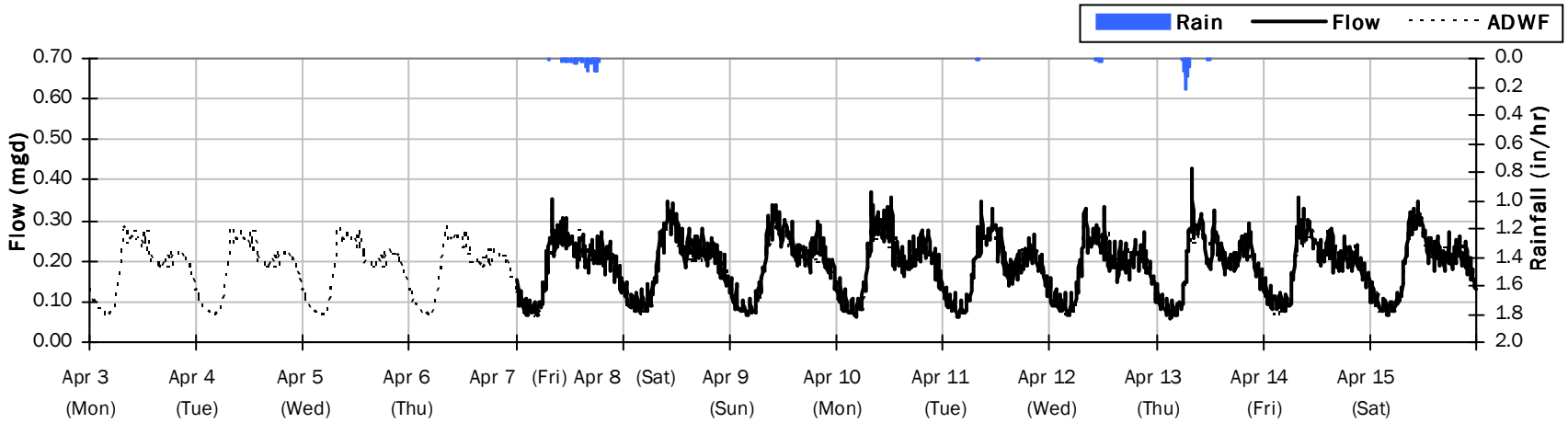


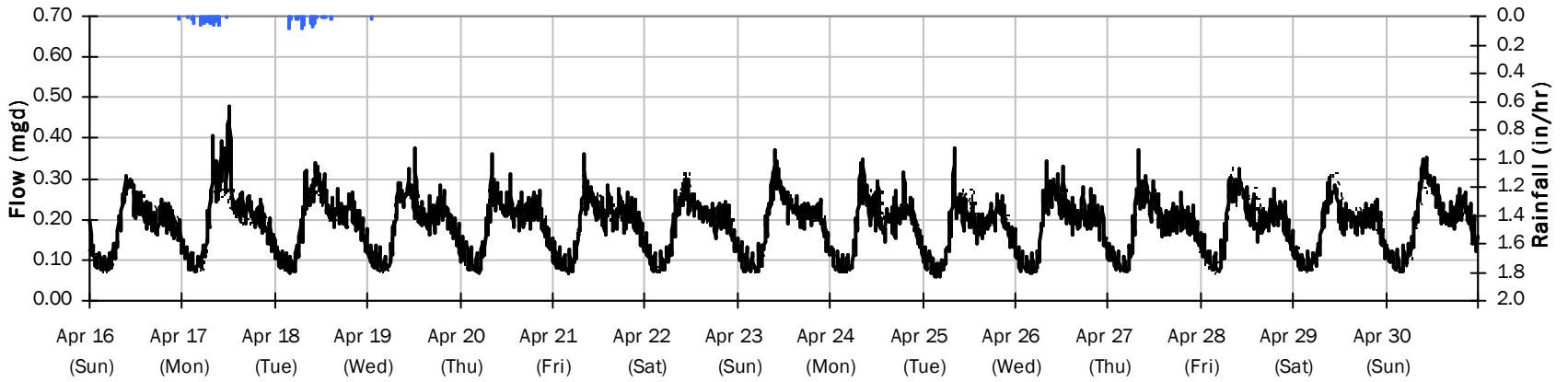
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 01

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.188 mgd** **Peak Flow: 0.479 mgd** **Min Flow: 0.059 mgd**

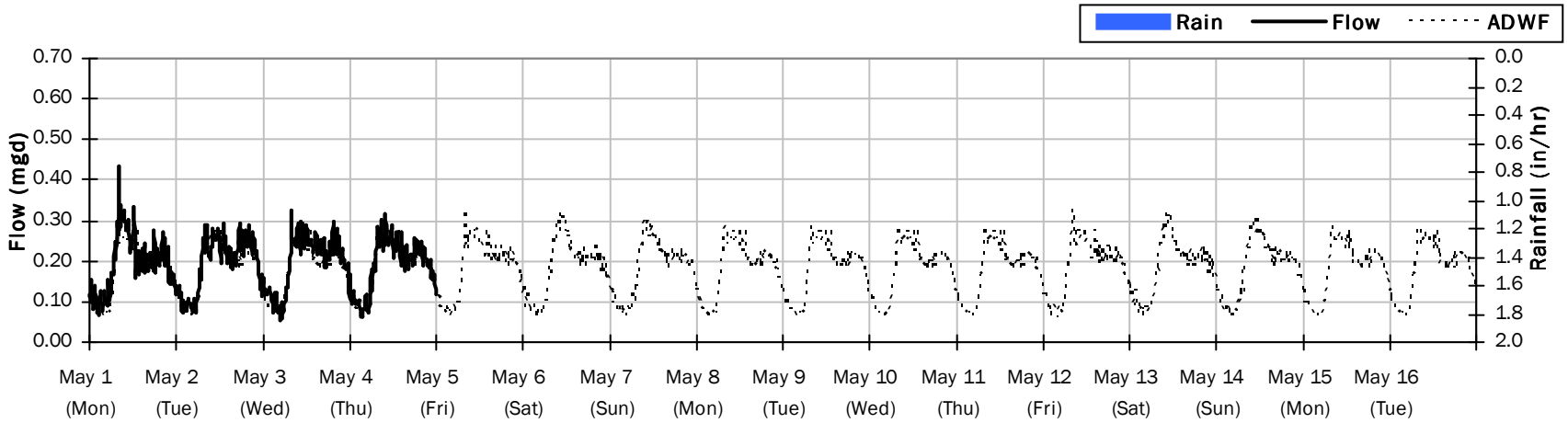


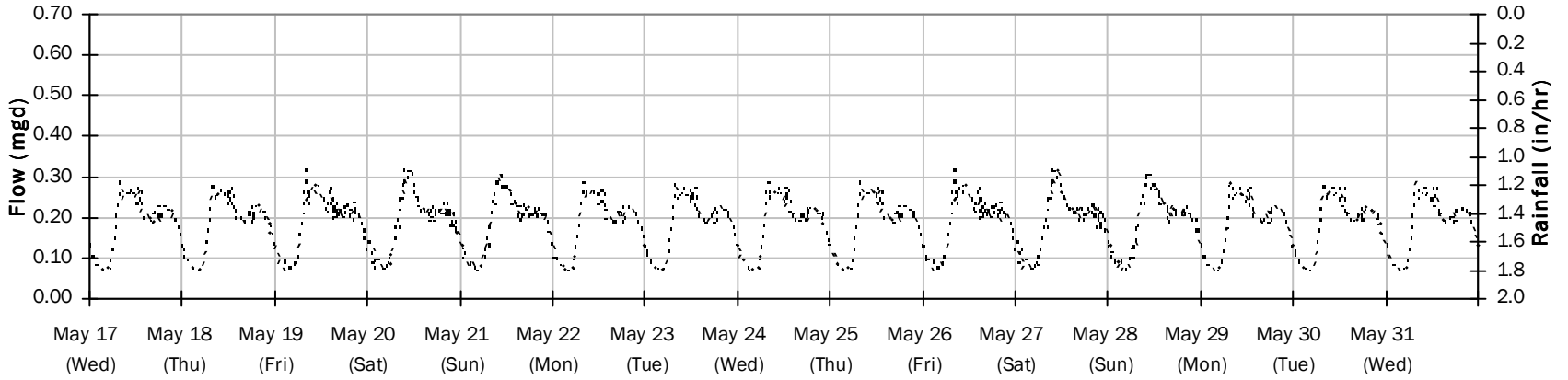


SITE 01

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.194 mgd** **Peak Flow: 0.433 mgd** **Min Flow: 0.056 mgd**



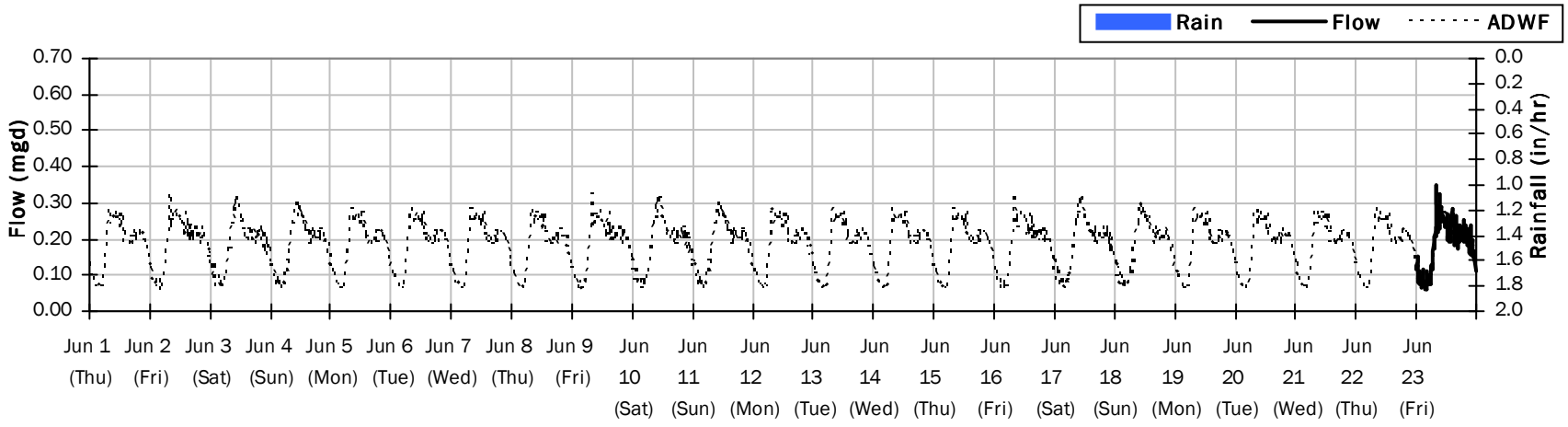


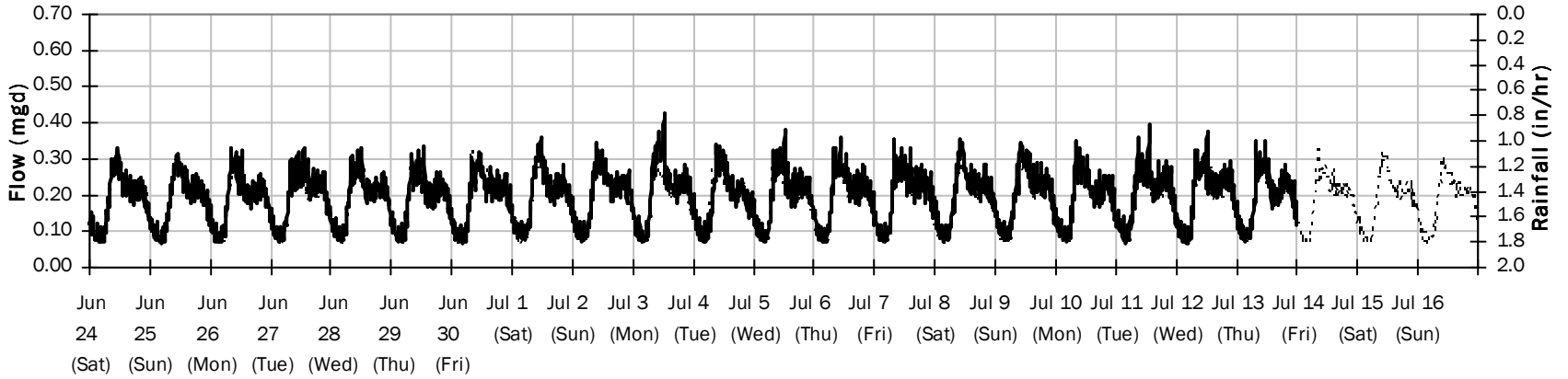
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 01

Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.196 mgd Peak Flow: 0.425 mgd Min Flow: 0.062 mgd**





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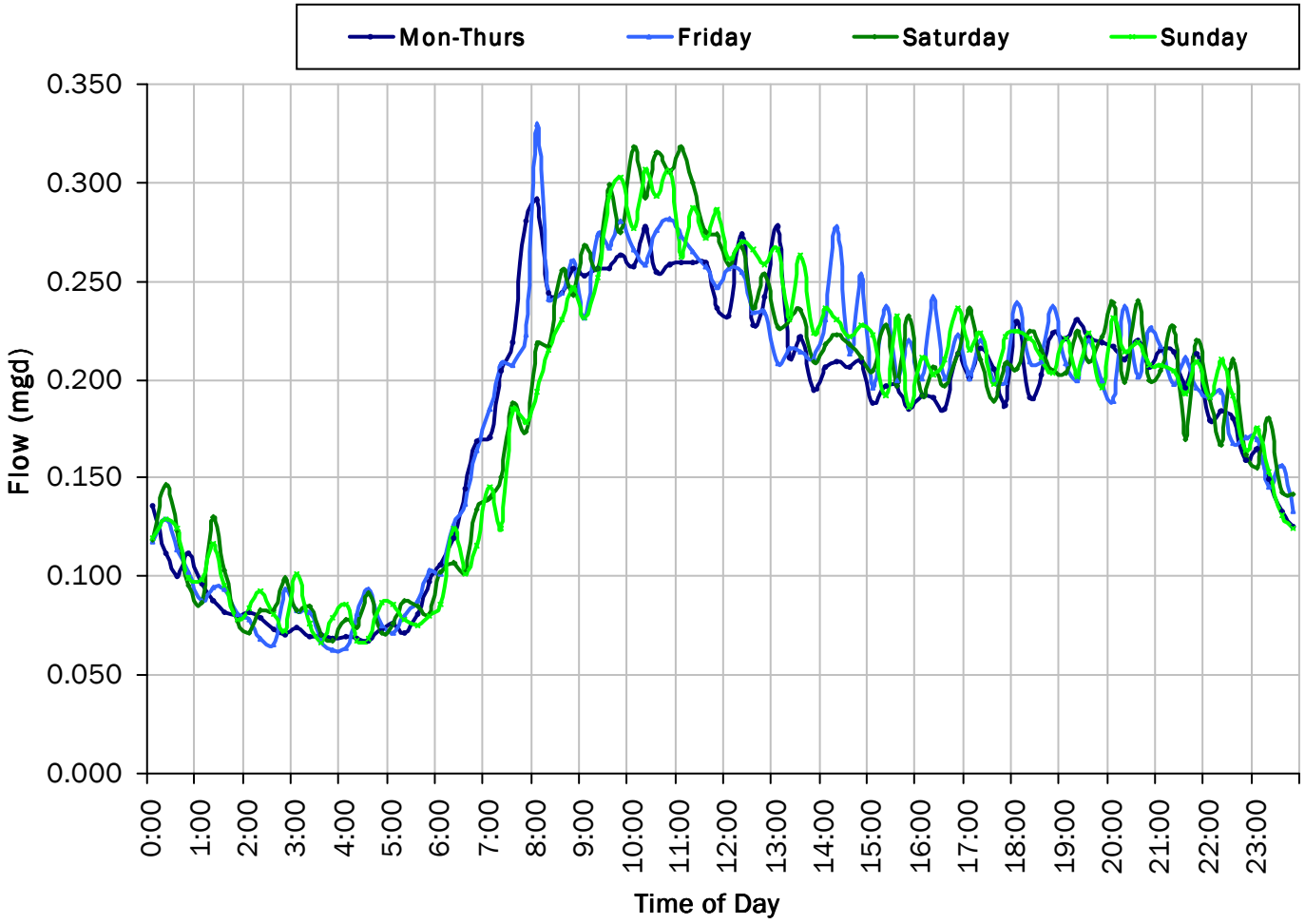
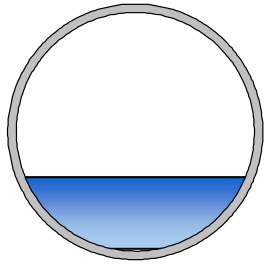
SITE 01

Average Dry Weather Flow Hydrographs



ADWF:

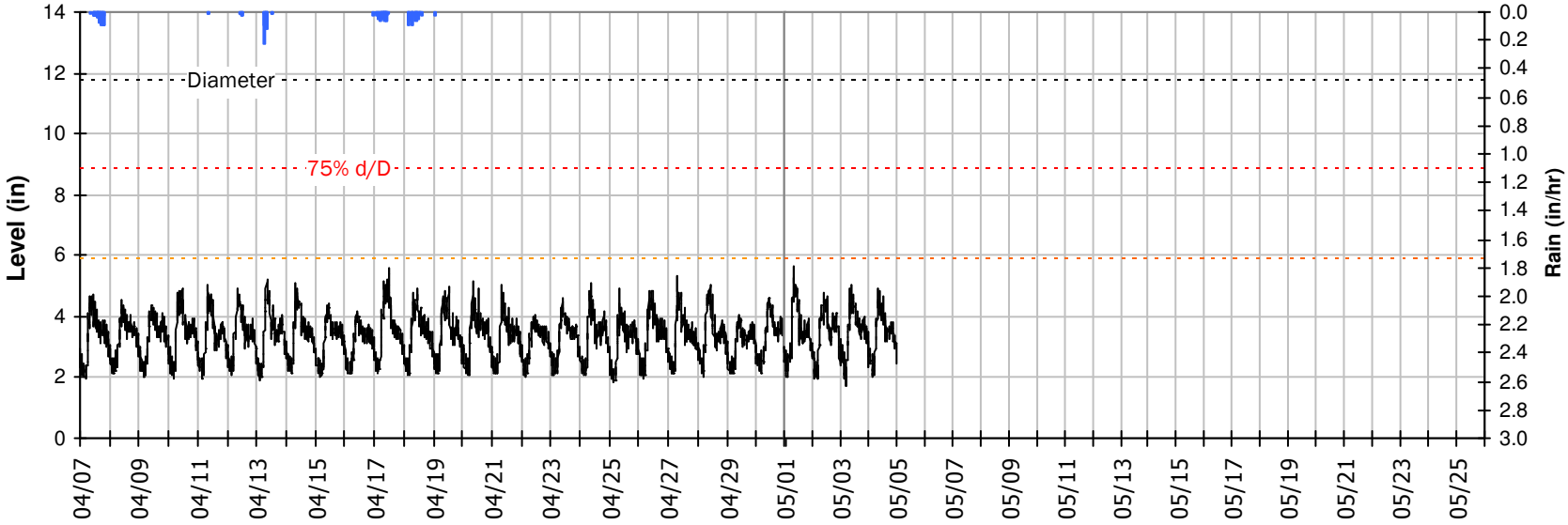
0.182 *mgd*

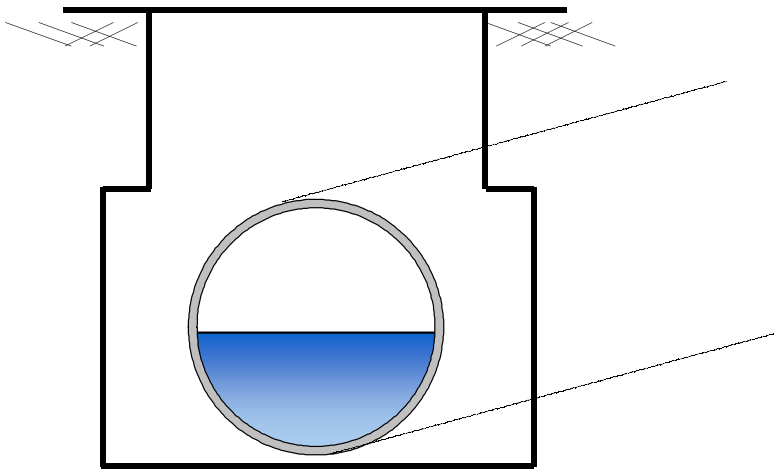


SITE 01

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





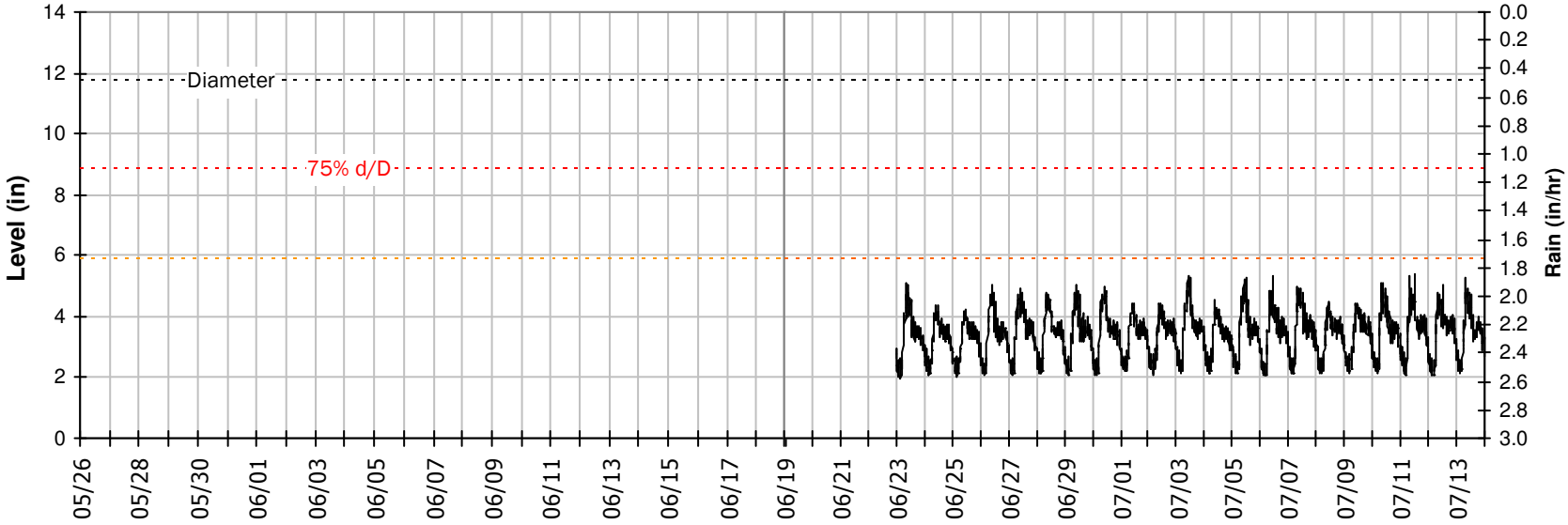
Pipe Diameter:	11.8	*inches*
Peak Measured Level:	5.63	*inches*
Peak d/D Ratio:	0.48	

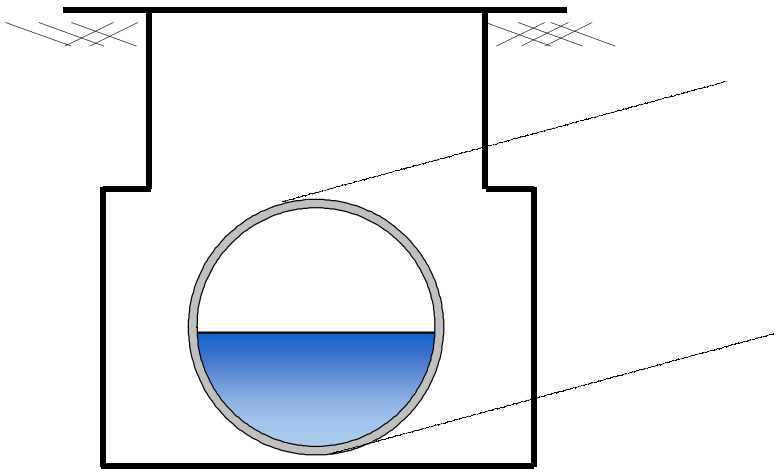
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 01

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





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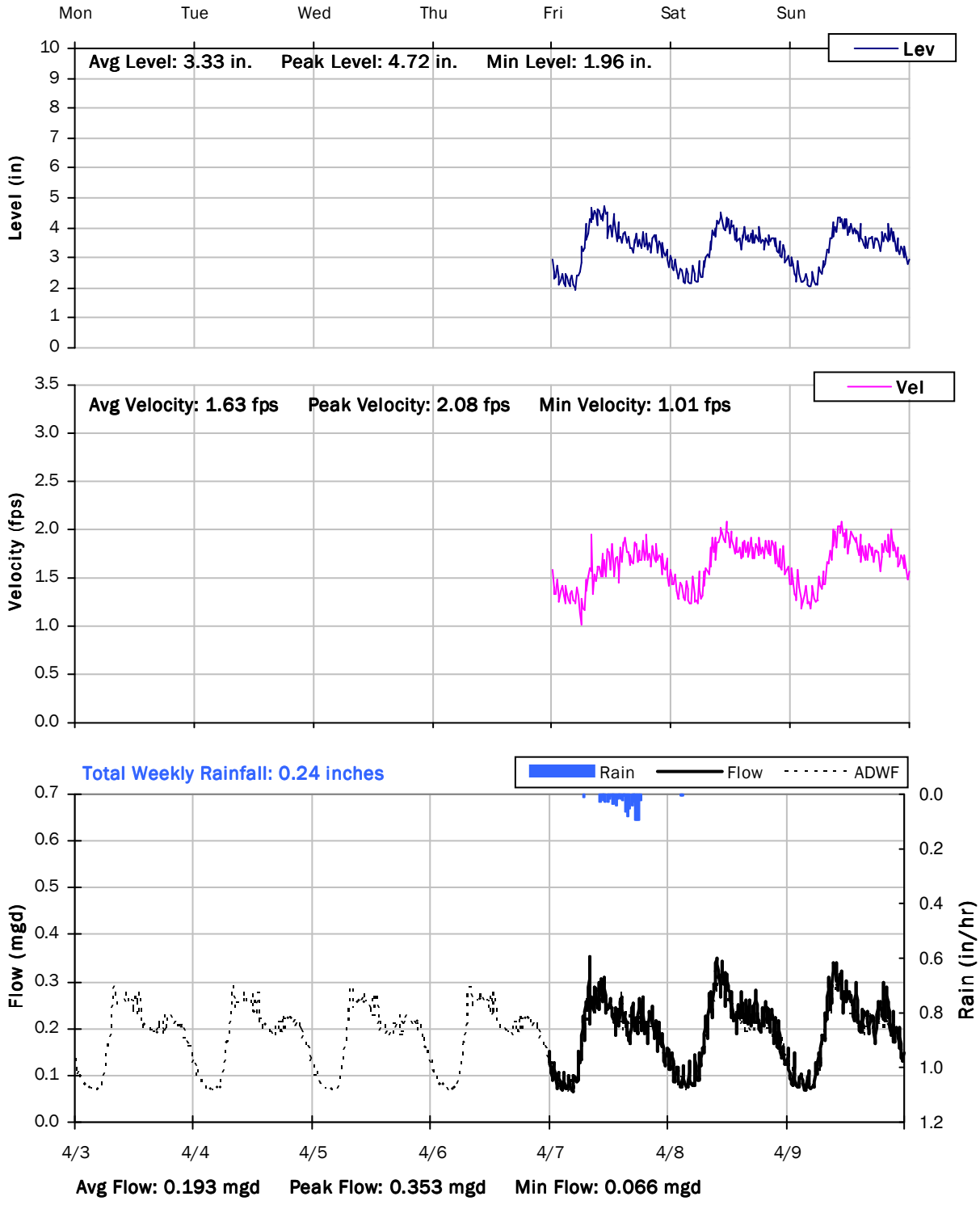
SITE 01

Weekly Level, Velocity and Flow Hydrographs

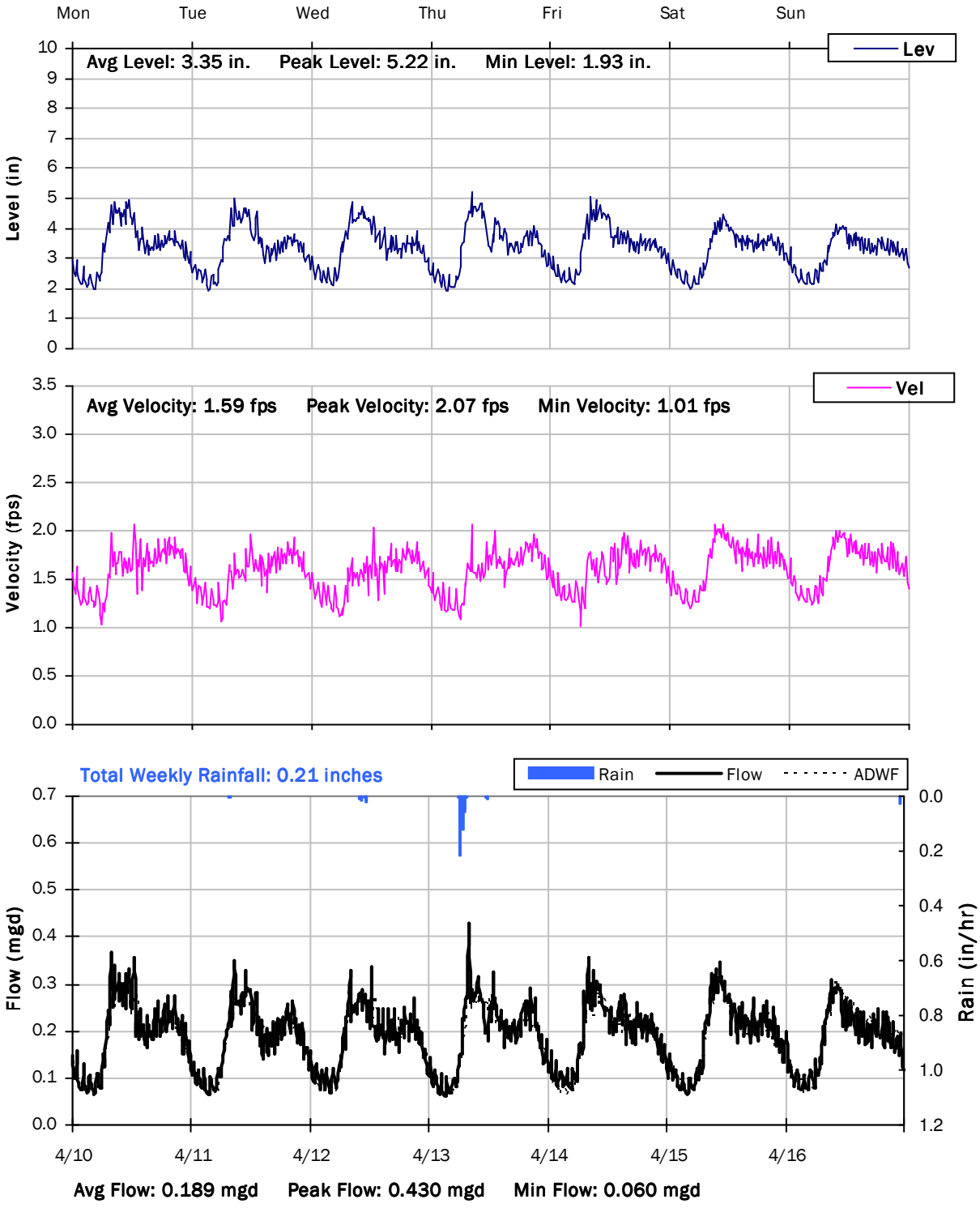
4/3/2017 to 4/10/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

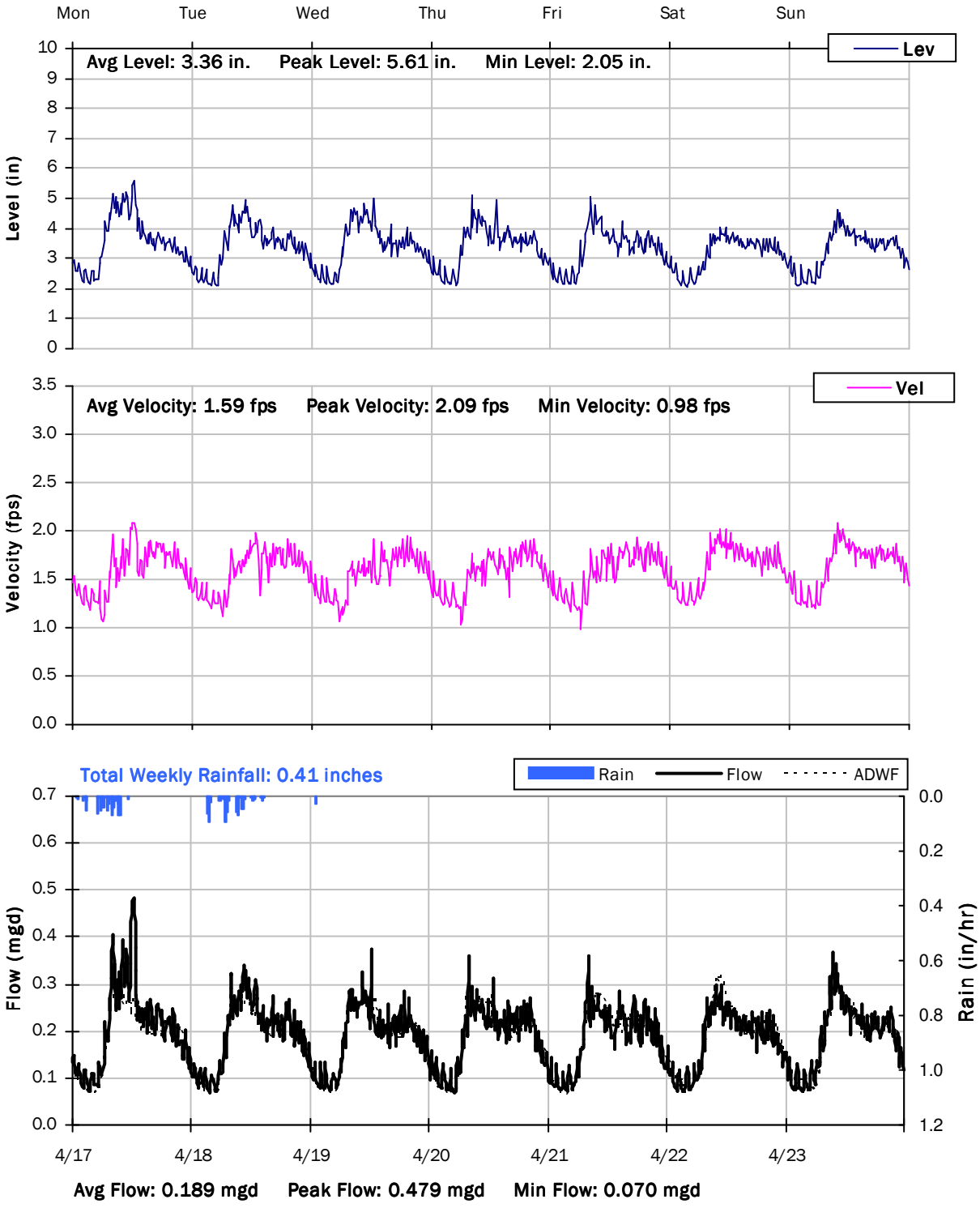
4/10/2017 to 4/17/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

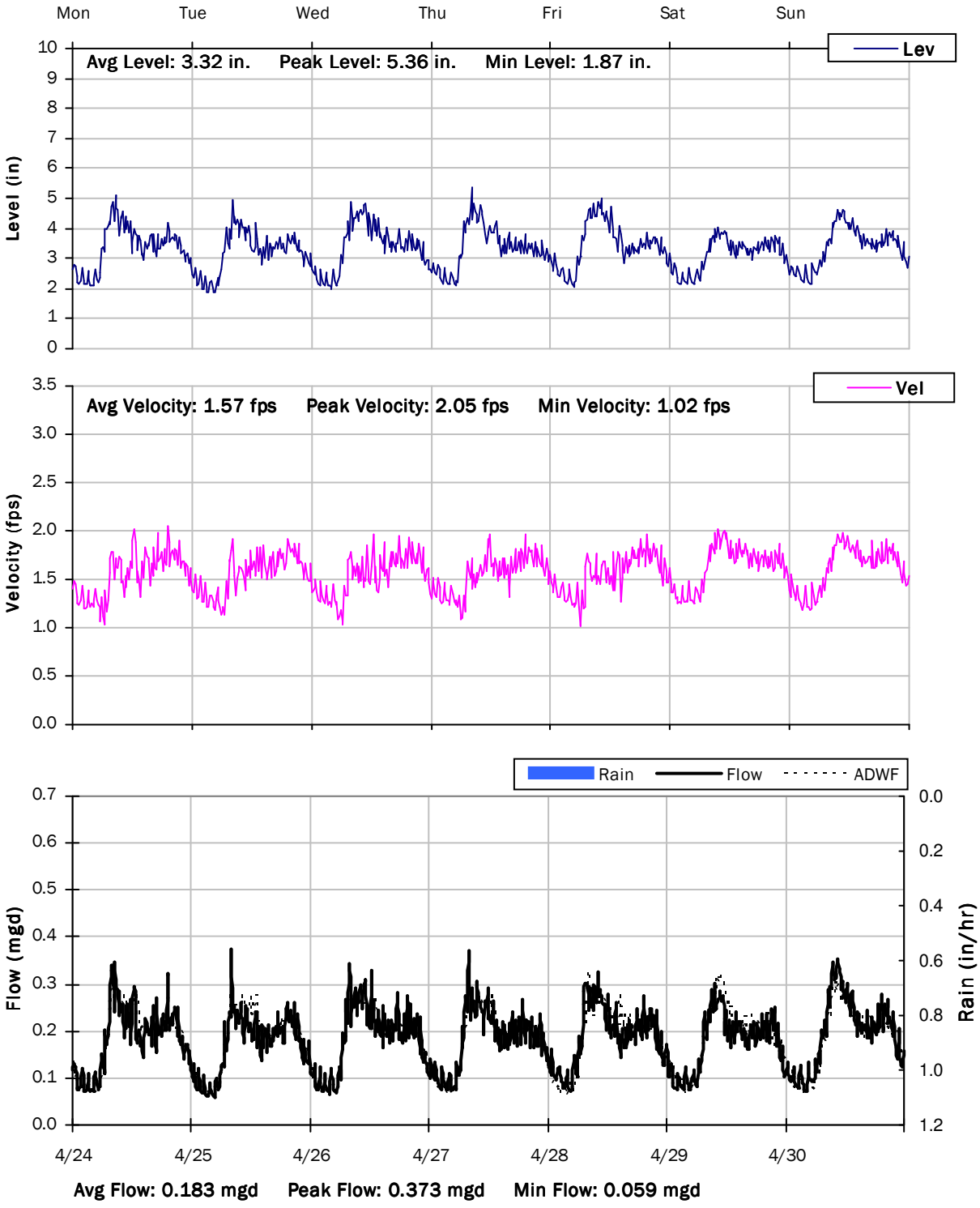
4/17/2017 to 4/24/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

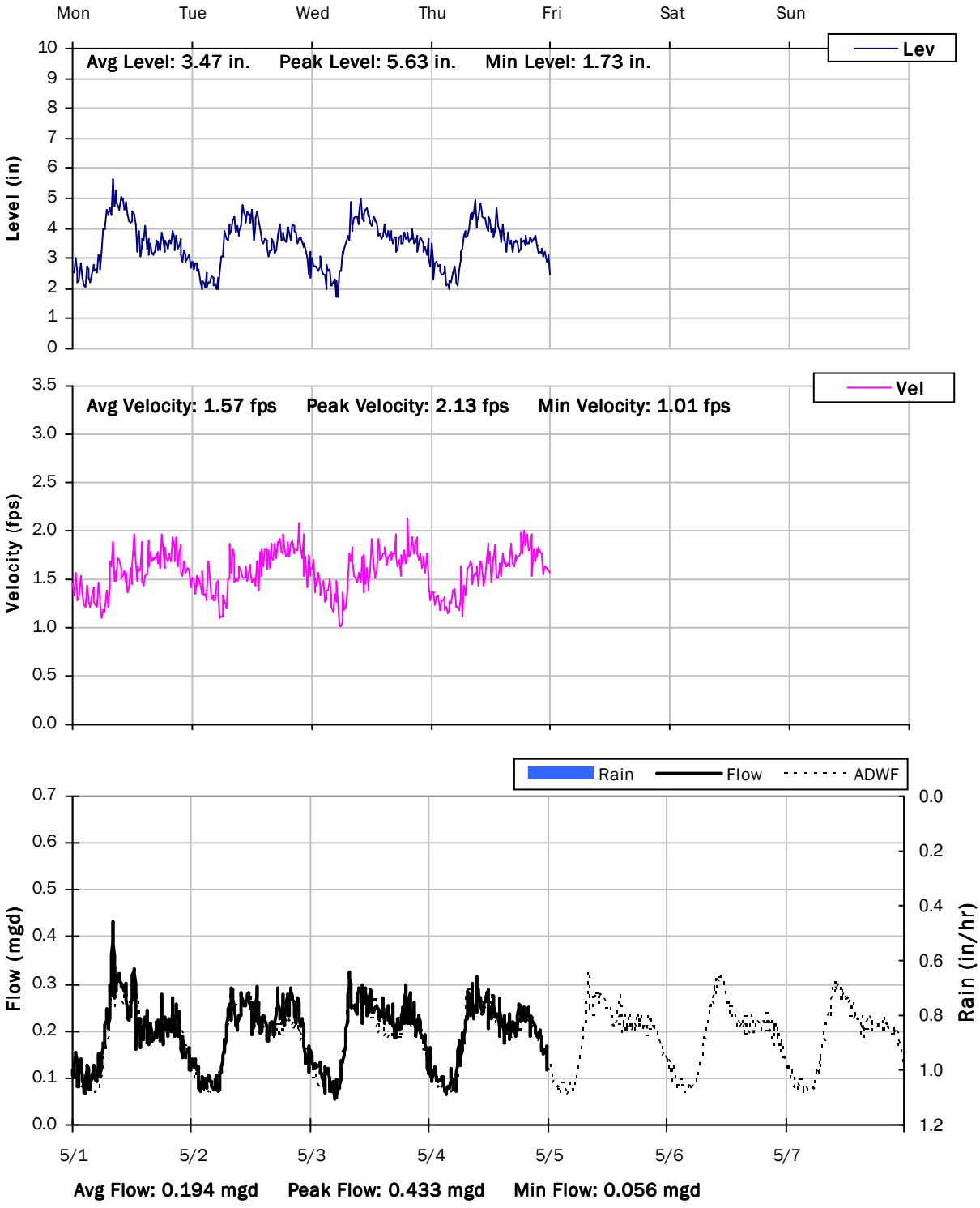
4/24/2017 to 5/1/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

5/1/2017 to 5/8/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

5/8/2017 to 5/15/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 01

Weekly Level, Velocity and Flow Hydrographs

5/15/2017 to 5/22/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 01

Weekly Level, Velocity and Flow Hydrographs

5/22/2017 to 5/29/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 01

Weekly Level, Velocity and Flow Hydrographs

5/29/2017 to 6/5/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 01

Weekly Level, Velocity and Flow Hydrographs

6/5/2017 to 6/12/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 01

Weekly Level, Velocity and Flow Hydrographs

6/12/2017 to 6/19/2017

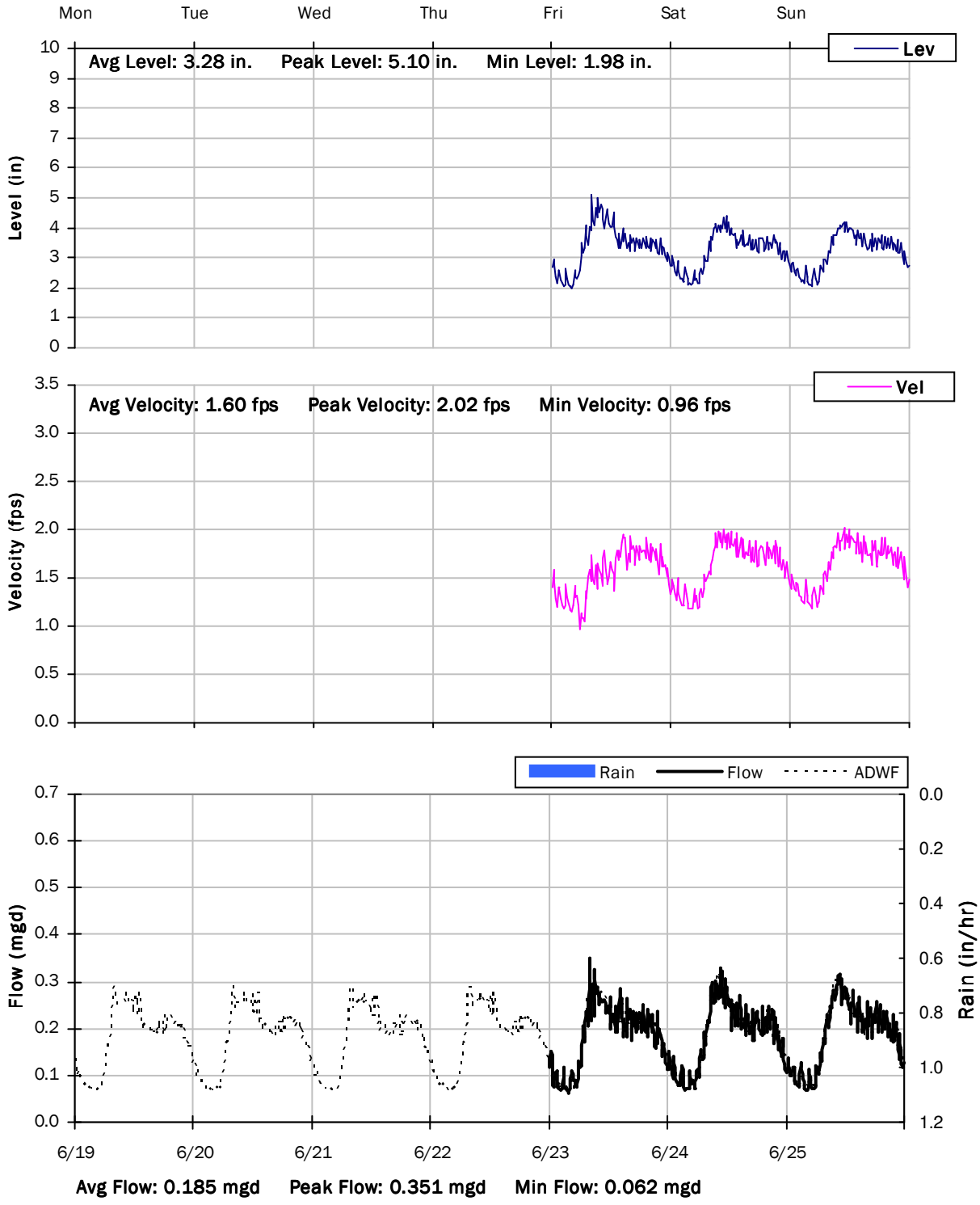
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 01

Weekly Level, Velocity and Flow Hydrographs

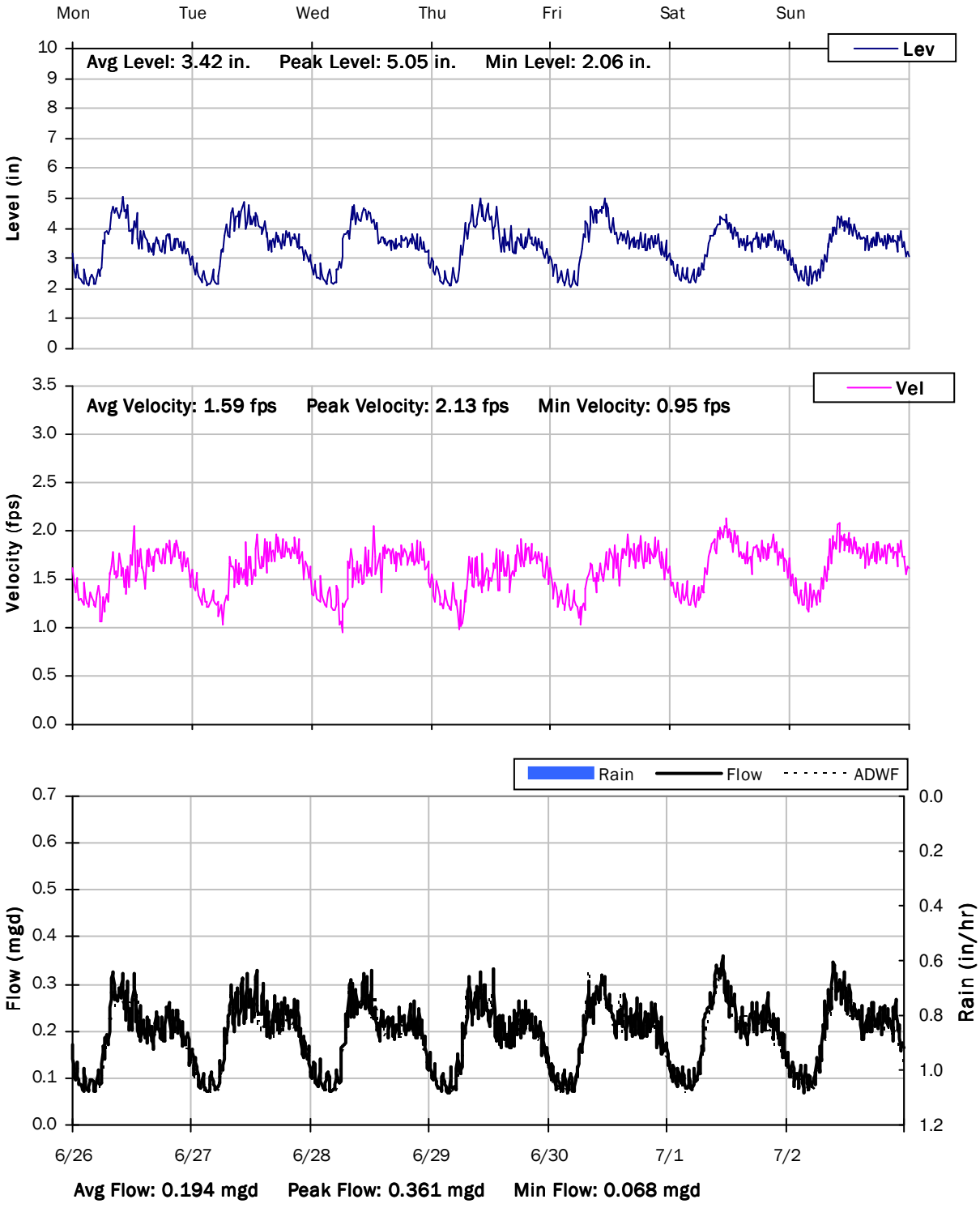
6/19/2017 to 6/26/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

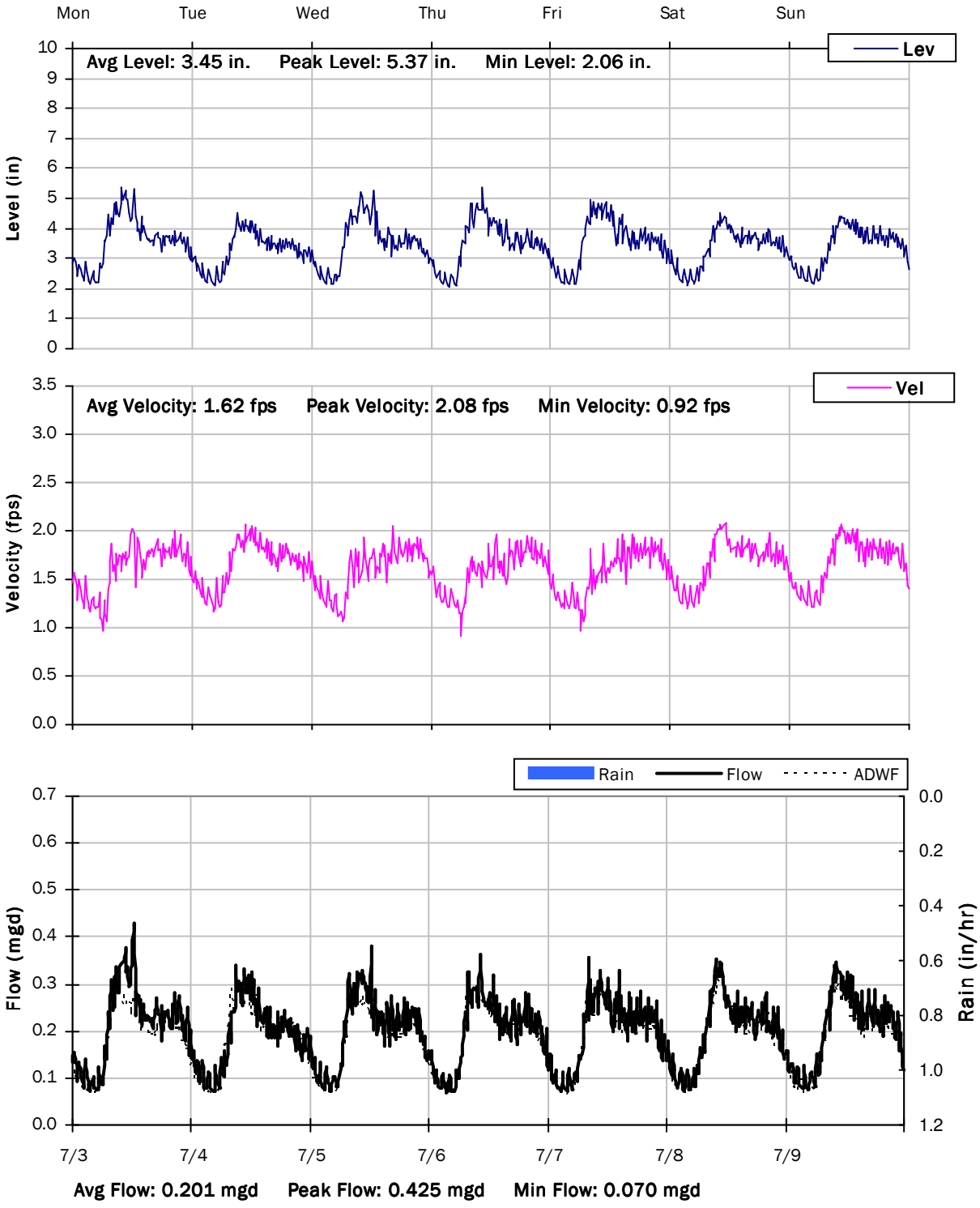
6/26/2017 to 7/3/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

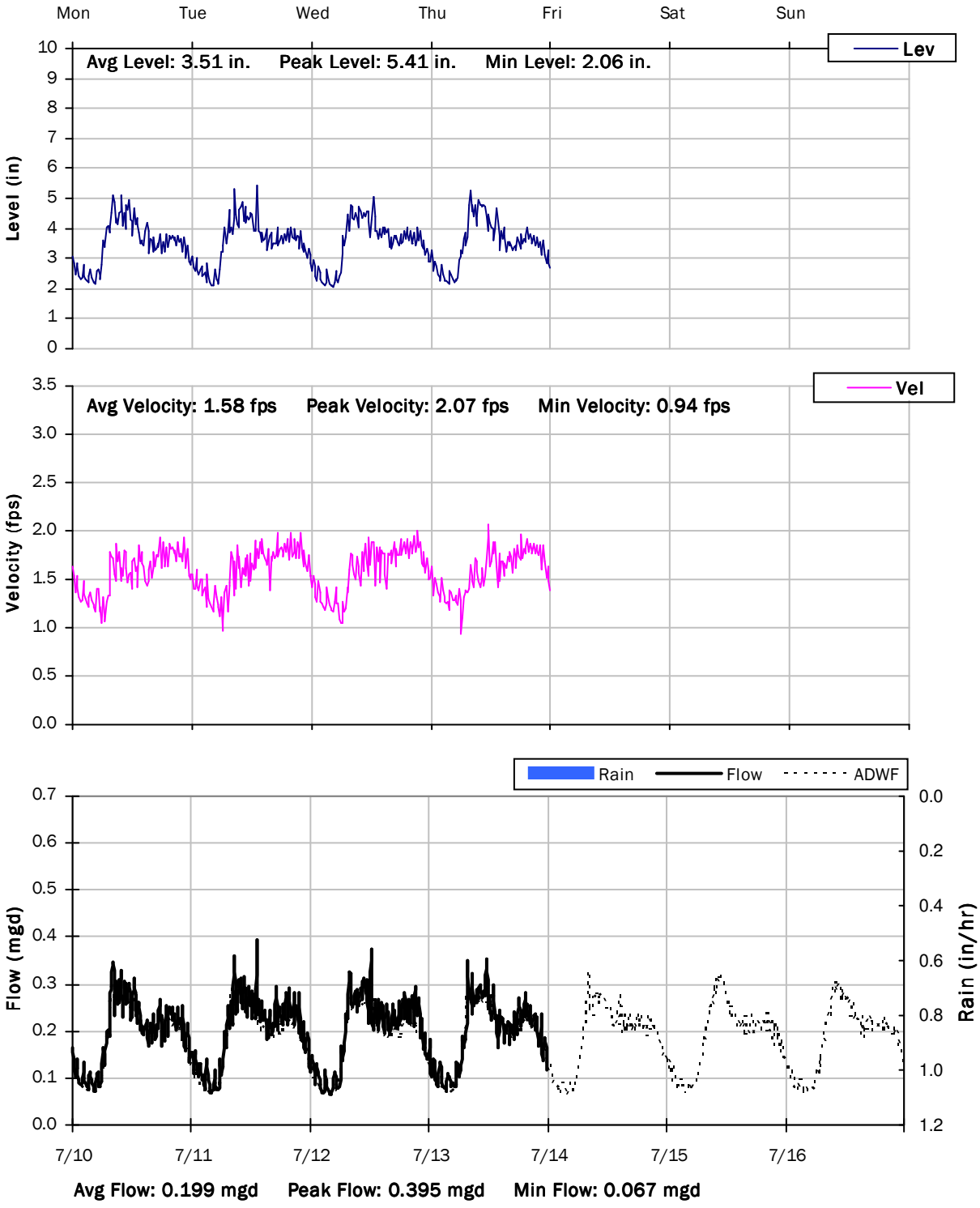
7/3/2017 to 7/10/2017



SITE 01

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

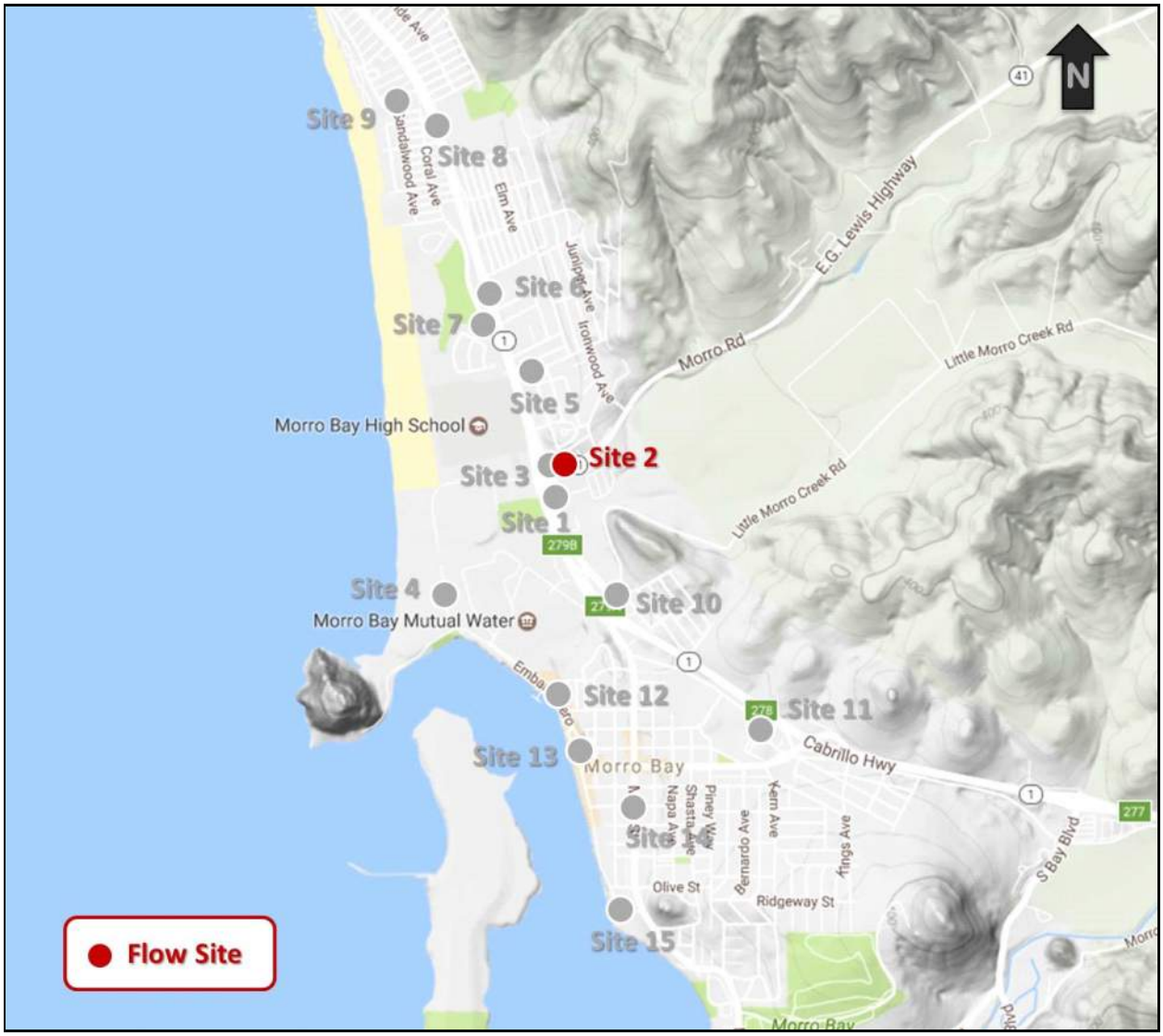
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 02

Location: Atascadero Road ~85 feet east of Sunset Avenue, meter on west 8" outlet

Data Summary Report



Vicinity Map: Site 02

SITE 02

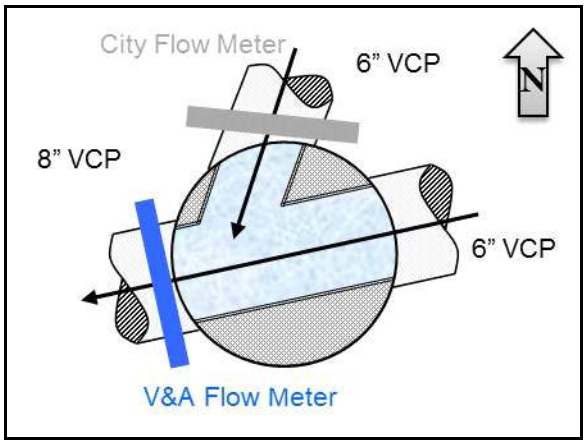
Site Information

Location: Atascadero Road ~85 feet east of Sunset Avenue, meter on west 8" outlet

Pipe Diameter: 8 inches

ADWF: 0.027 mgd

Peak Measured Flow: 0.085 mgd



SITE 02

Additional Site Photos

SITE 02

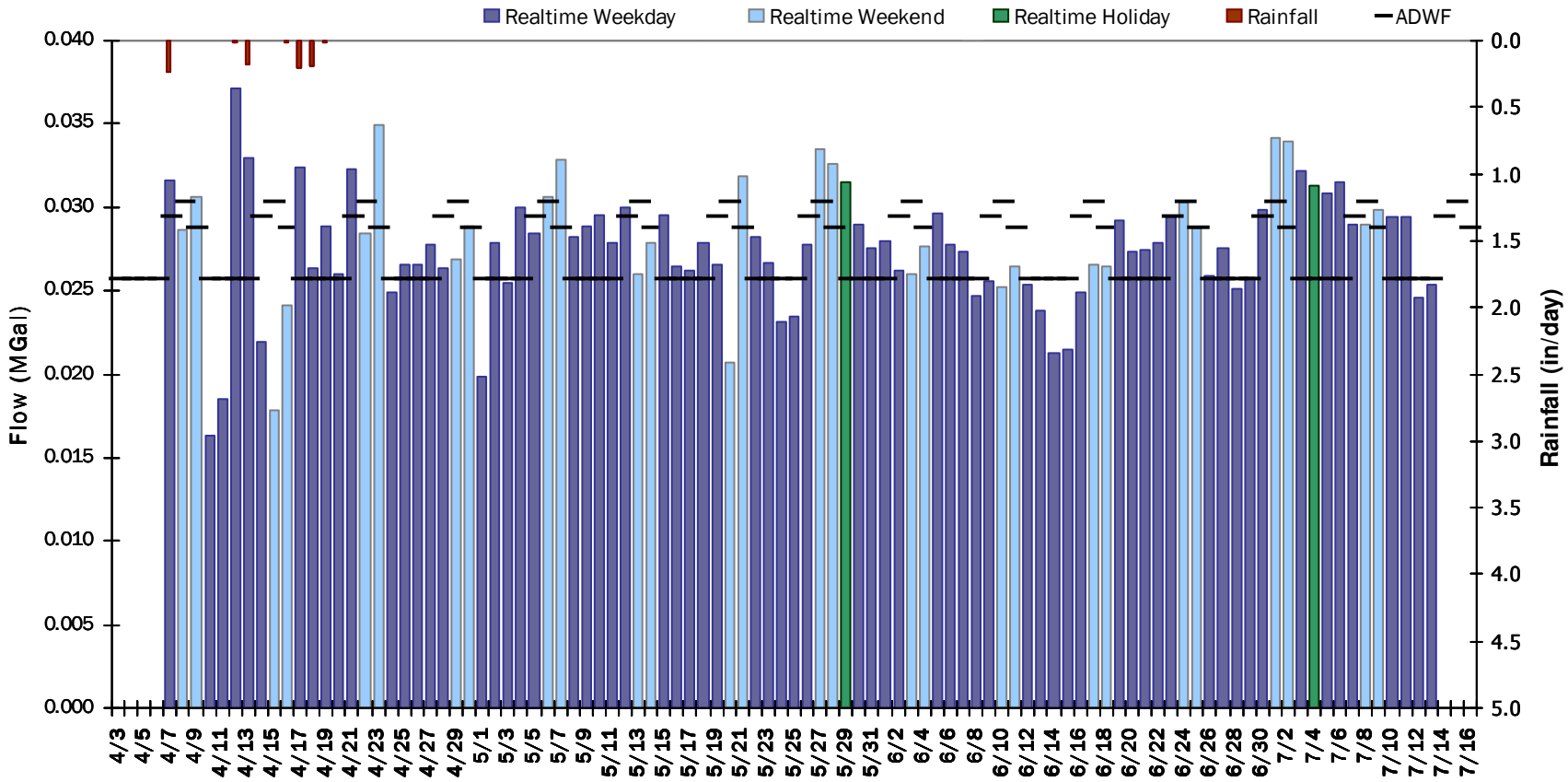
Additional Site Photos

SITE 02

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.028 MGal Peak Daily Flow: 0.037 MGal Min Daily Flow: 0.016 MGal

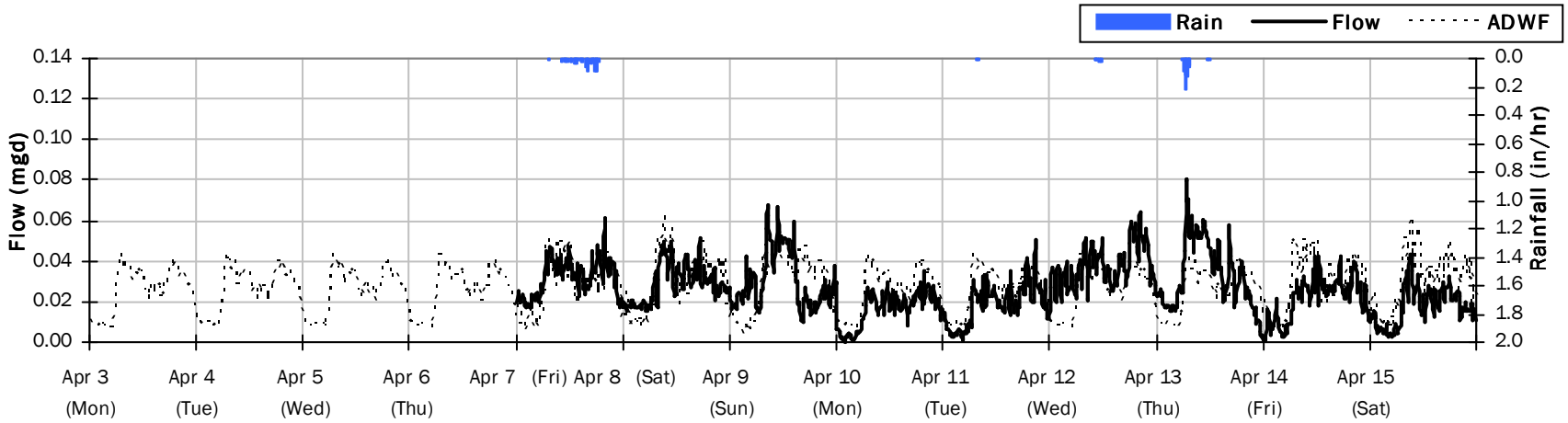
Total Period Rainfall: 0.85 inches

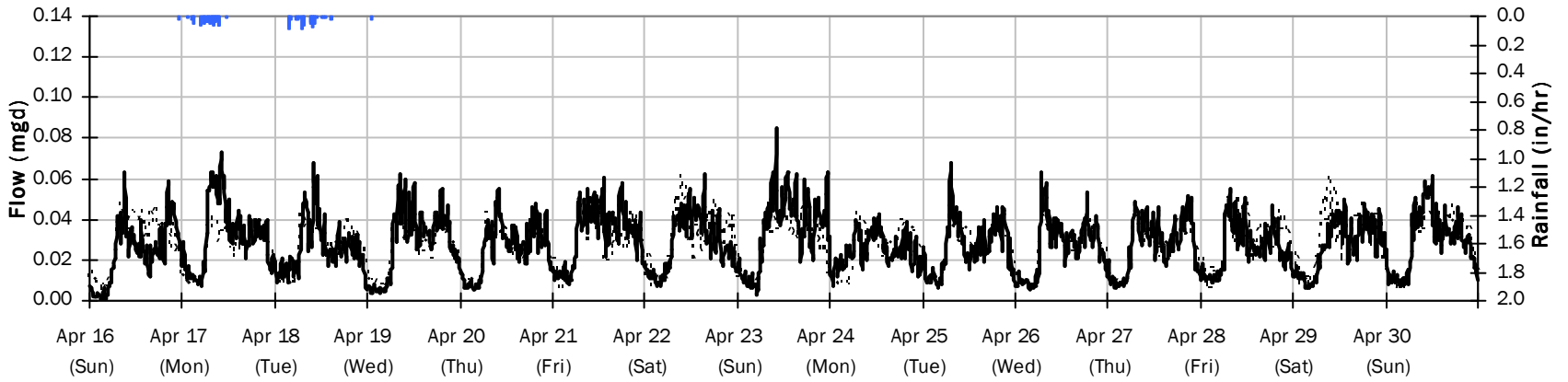


SITE 02

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.027 mgd** **Peak Flow: 0.085 mgd** **Min Flow: 0.000 mgd**

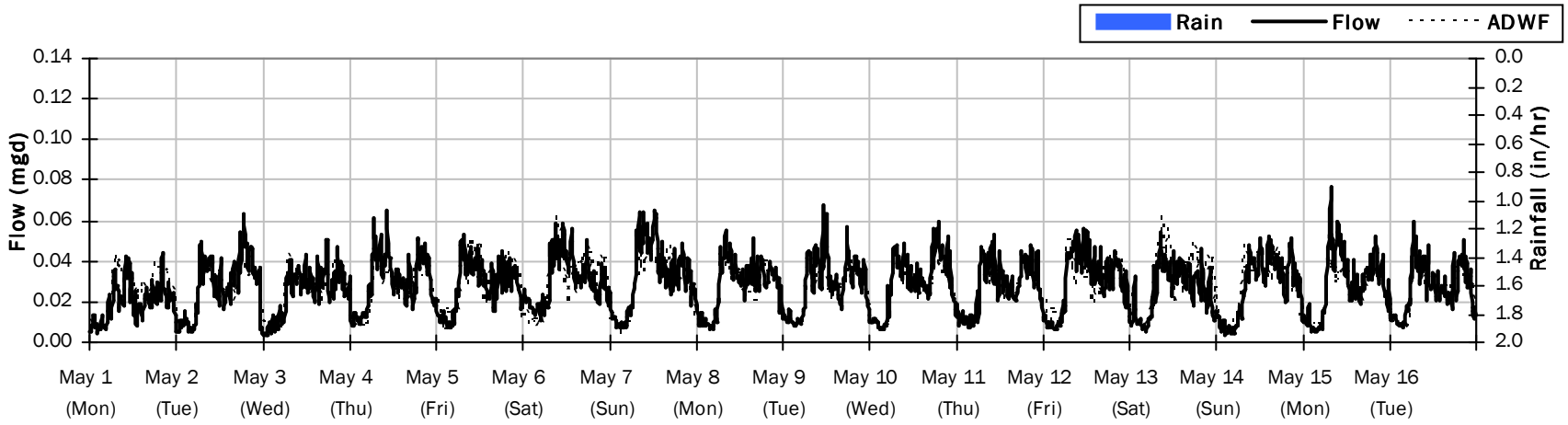


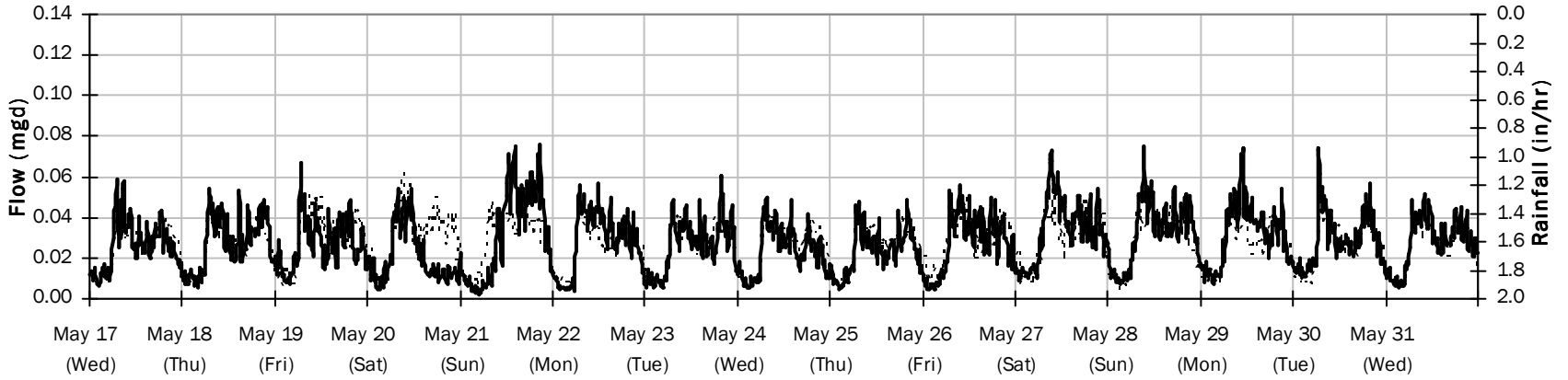


SITE 02

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.028 mgd** **Peak Flow: 0.077 mgd** **Min Flow: 0.002 mgd**

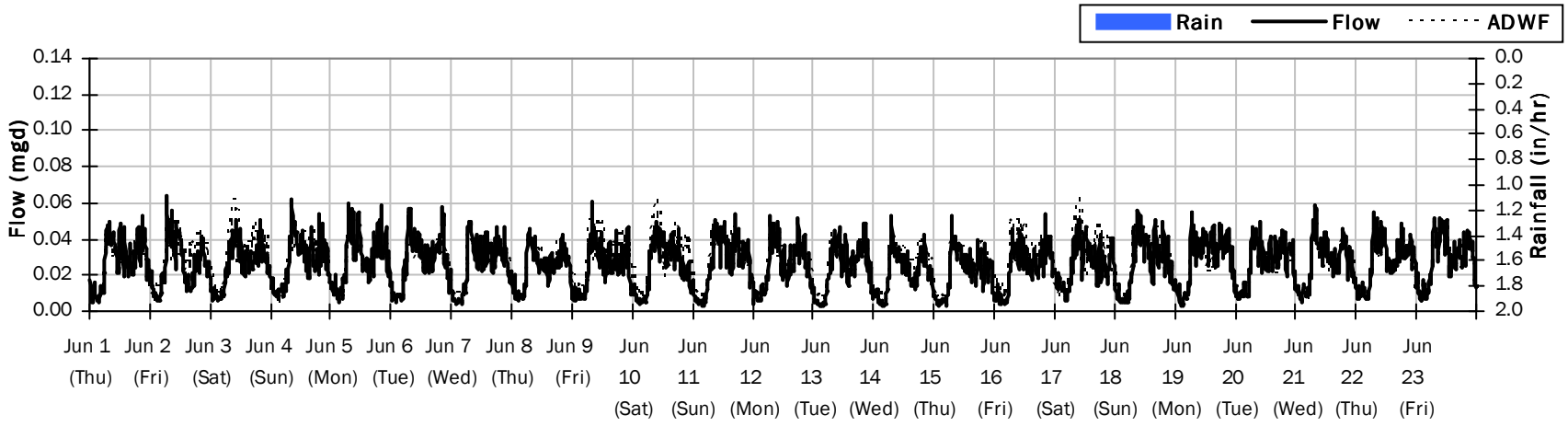


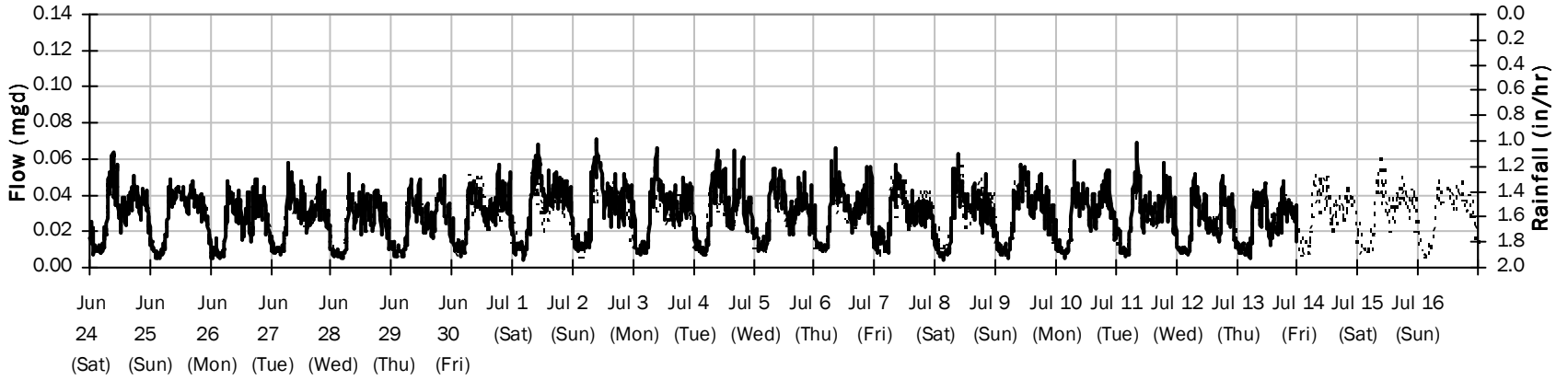


SITE 02

Flow Summary: 6/1/2017 to 7/16/2017

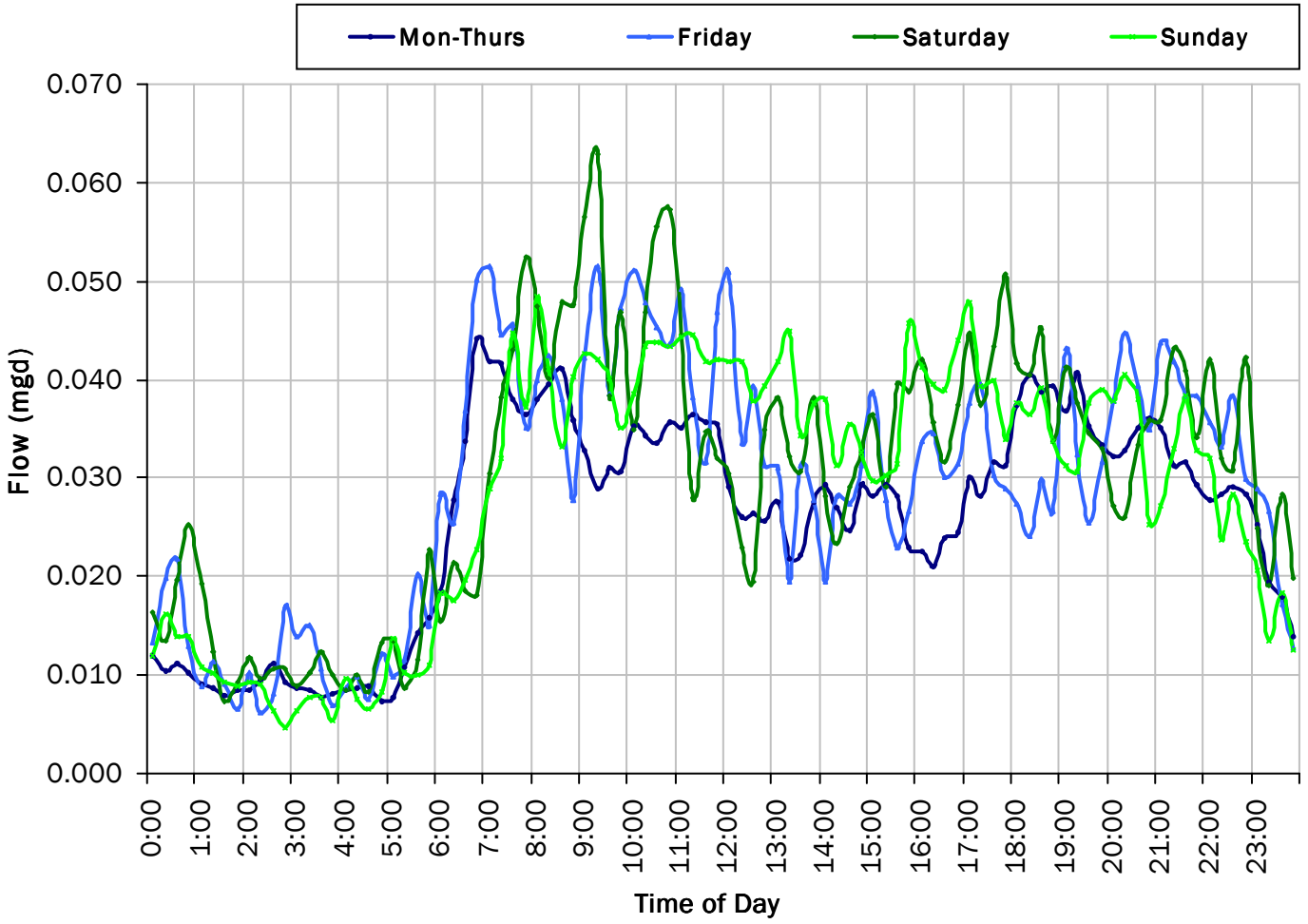
Total Period Rainfall: 0.00 inches **Avg Flow: 0.028 mgd** **Peak Flow: 0.072 mgd** **Min Flow: 0.003 mgd**





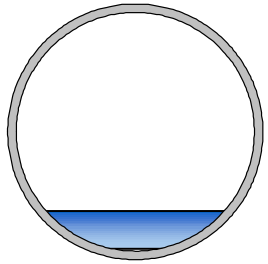
SITE 02

Average Dry Weather Flow Hydrographs



ADWF:

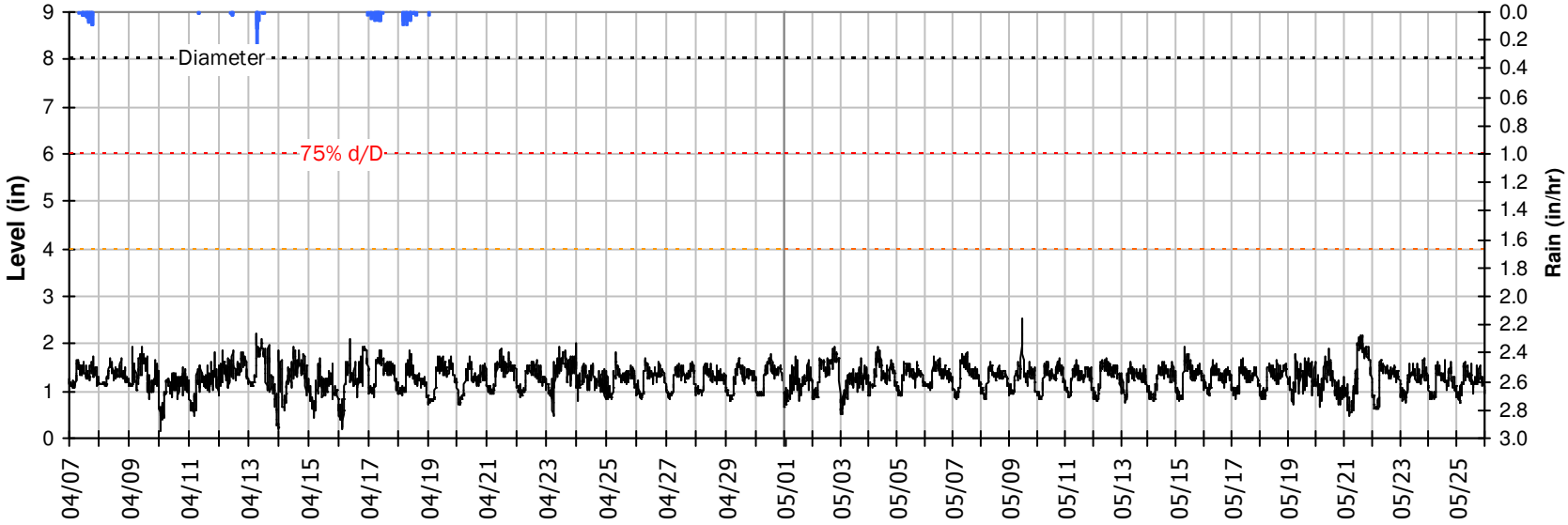
0.027 *mgd*

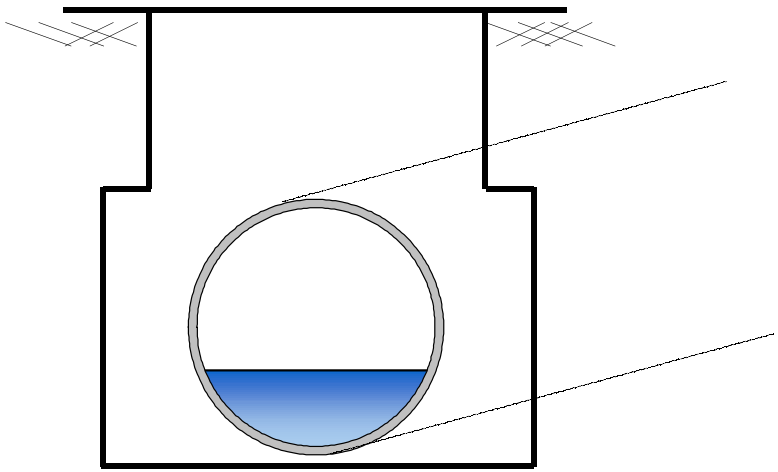


SITE 02

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



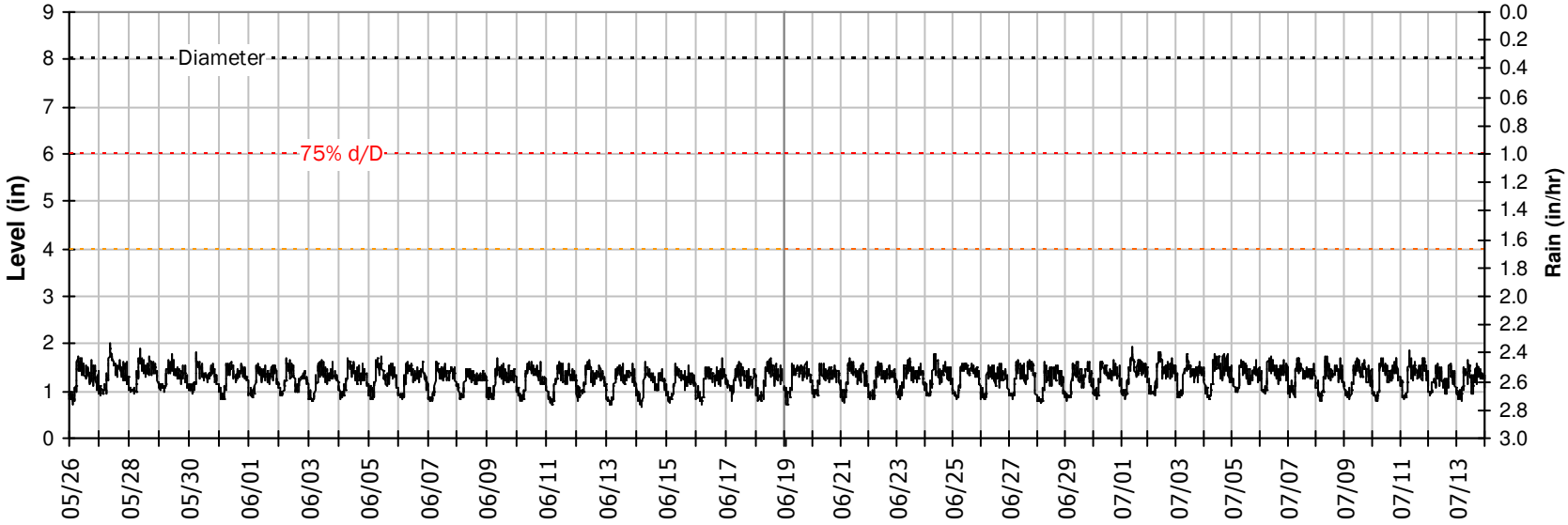


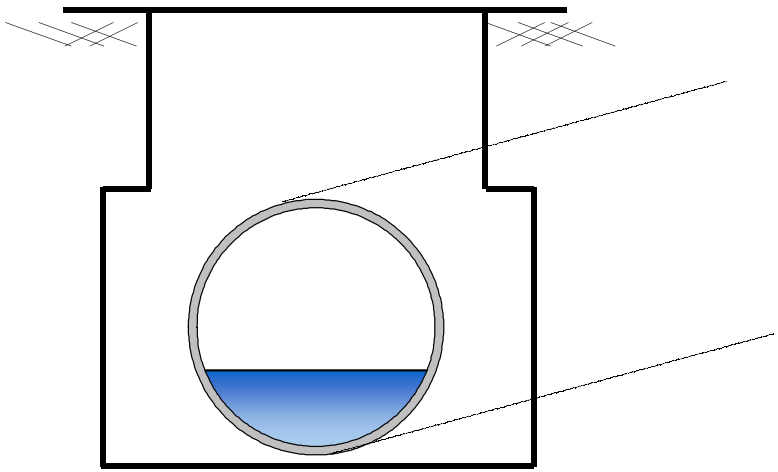
Pipe Diameter:	8	*inches*
Peak Measured Level:	2.53	*inches*
Peak d/D Ratio:	0.32	

SITE 02

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter:	8	*inches*
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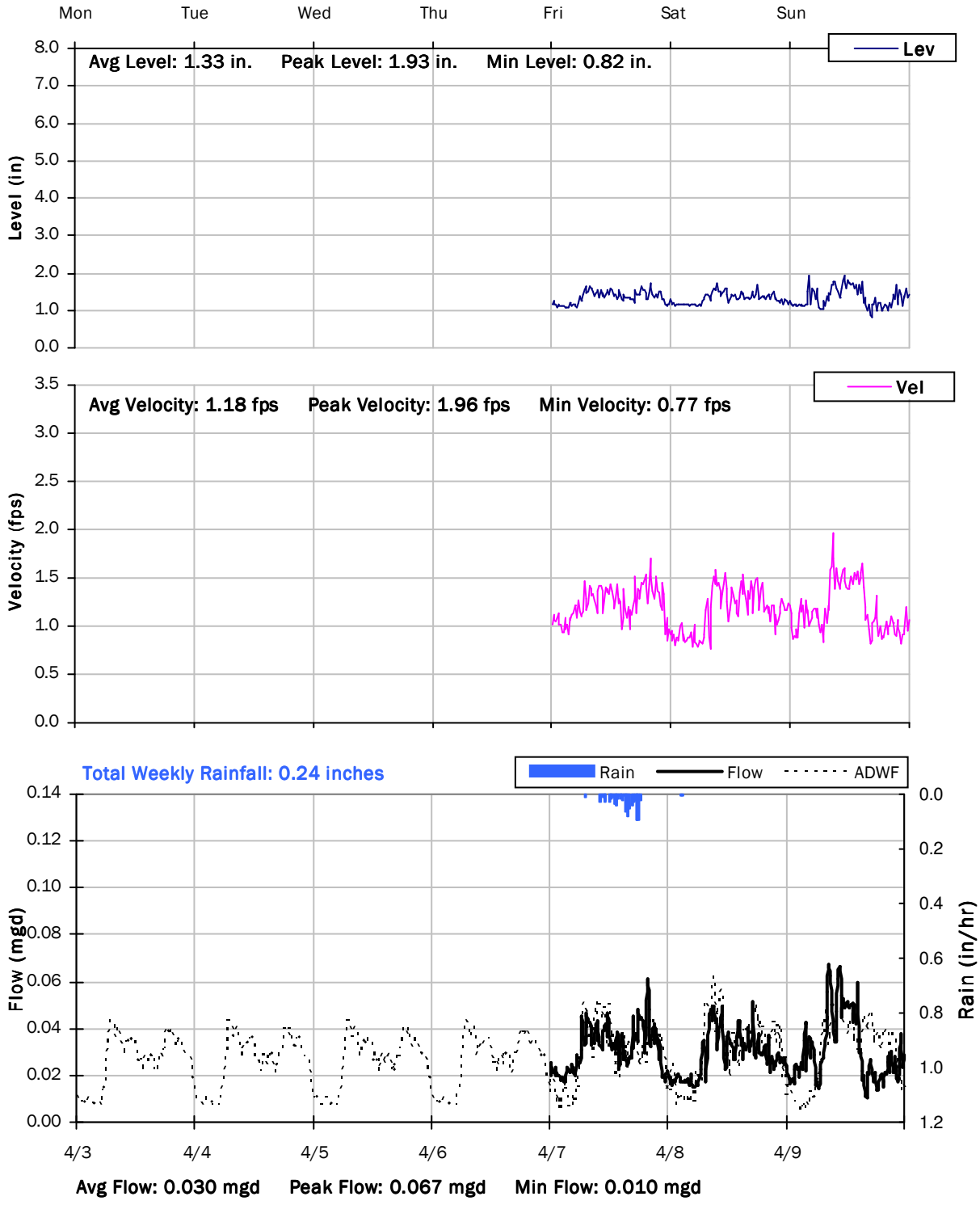
SITE 02

Weekly Level, Velocity and Flow Hydrographs

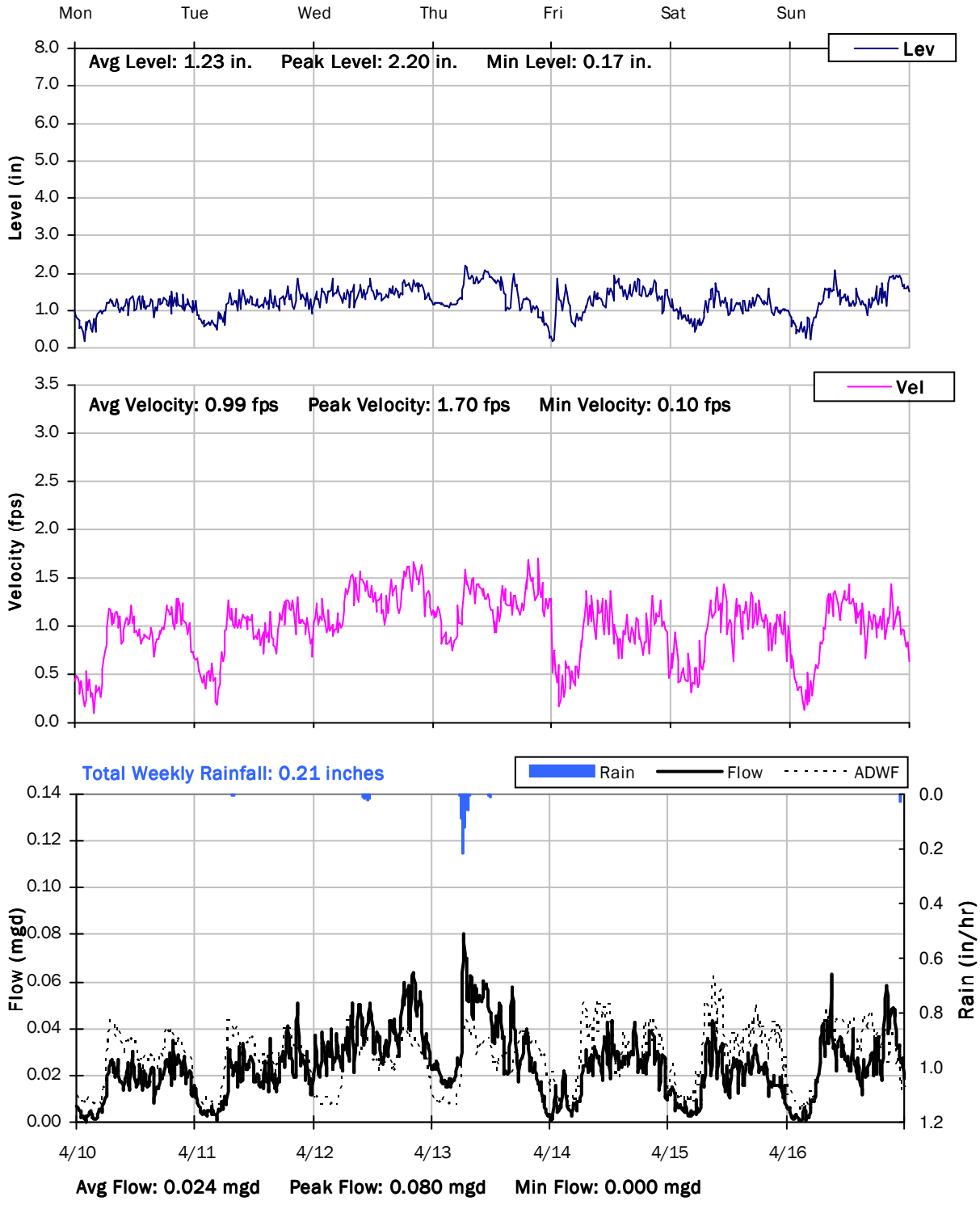
4/3/2017 to 4/10/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

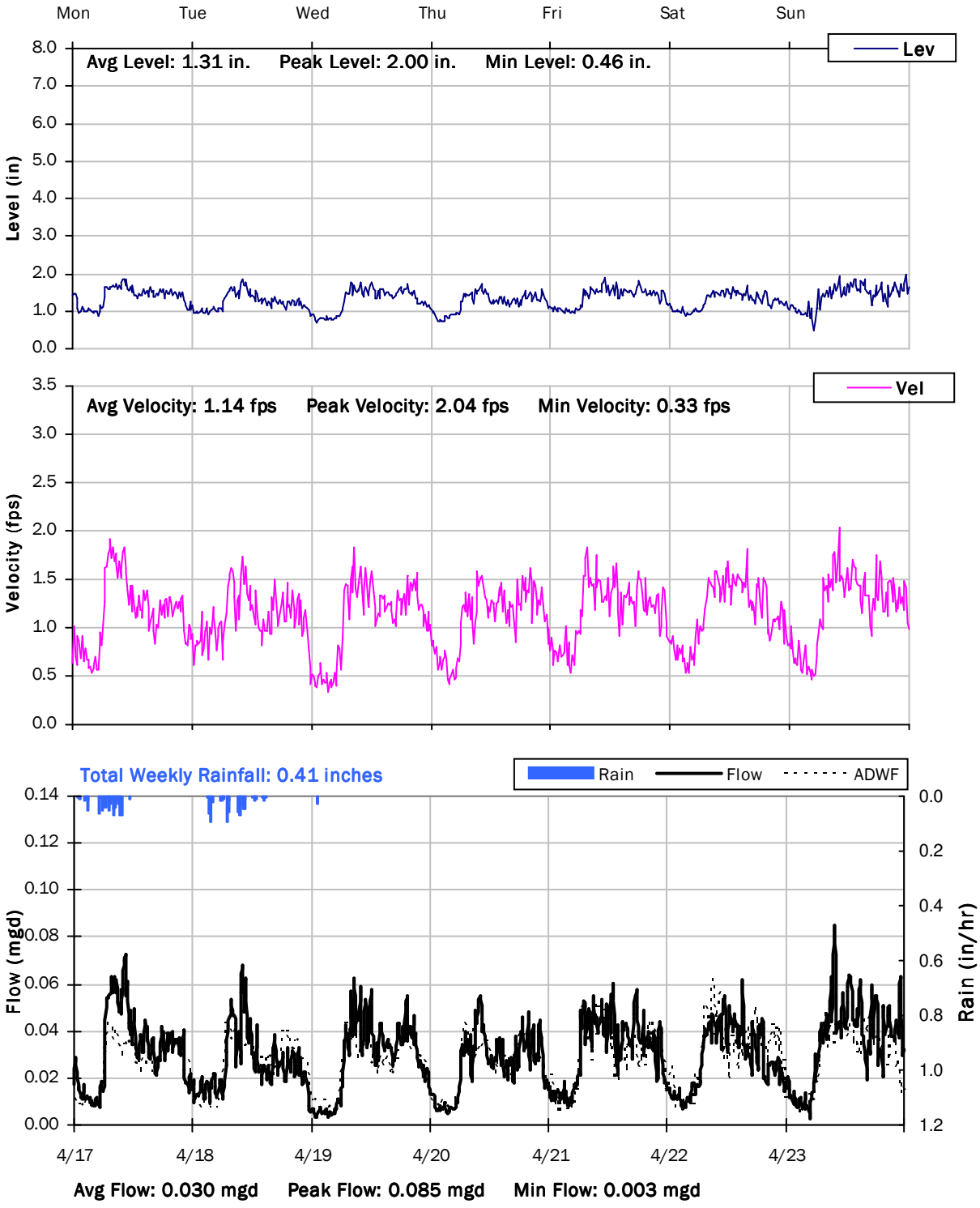
4/10/2017 to 4/17/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

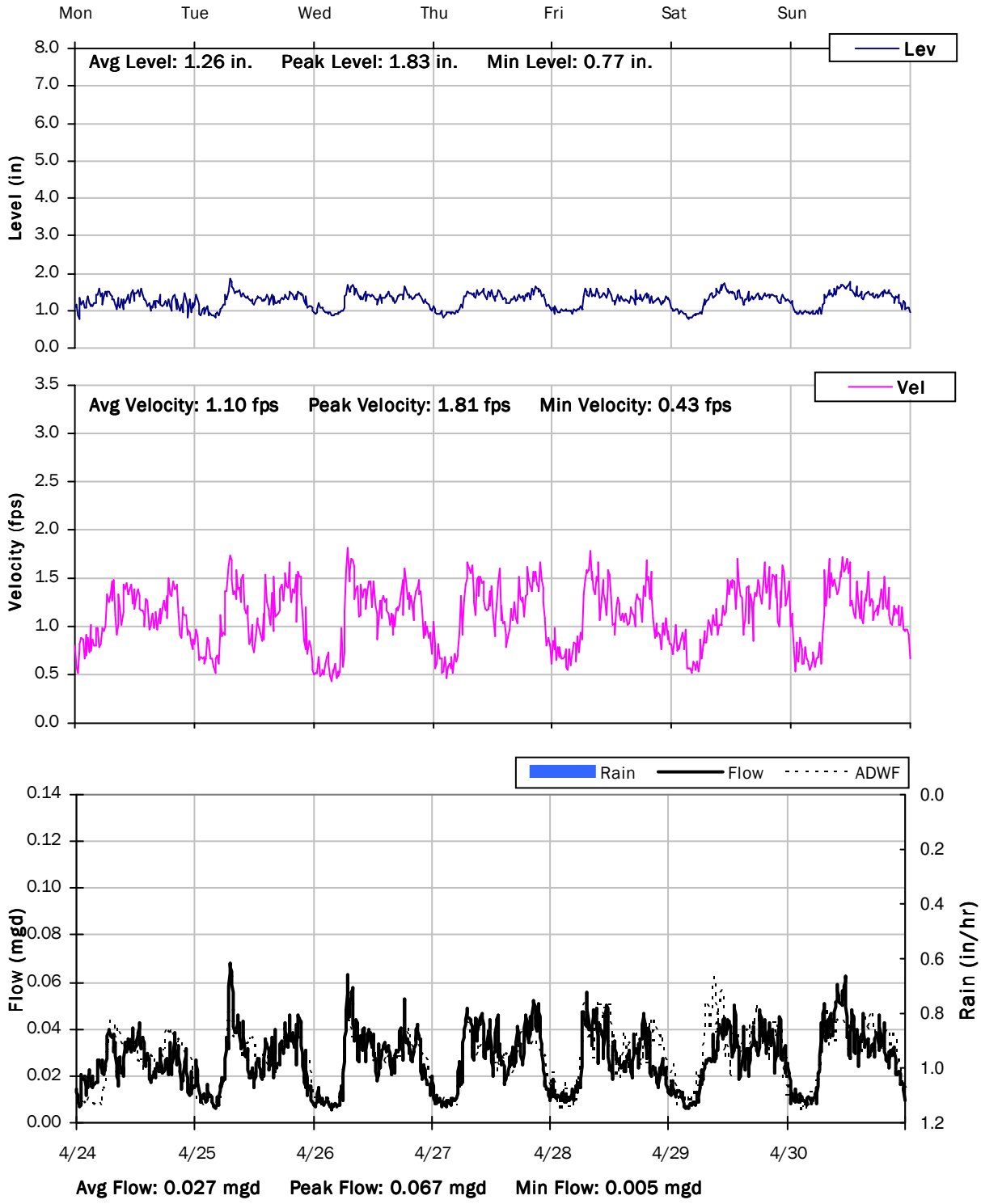
4/17/2017 to 4/24/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

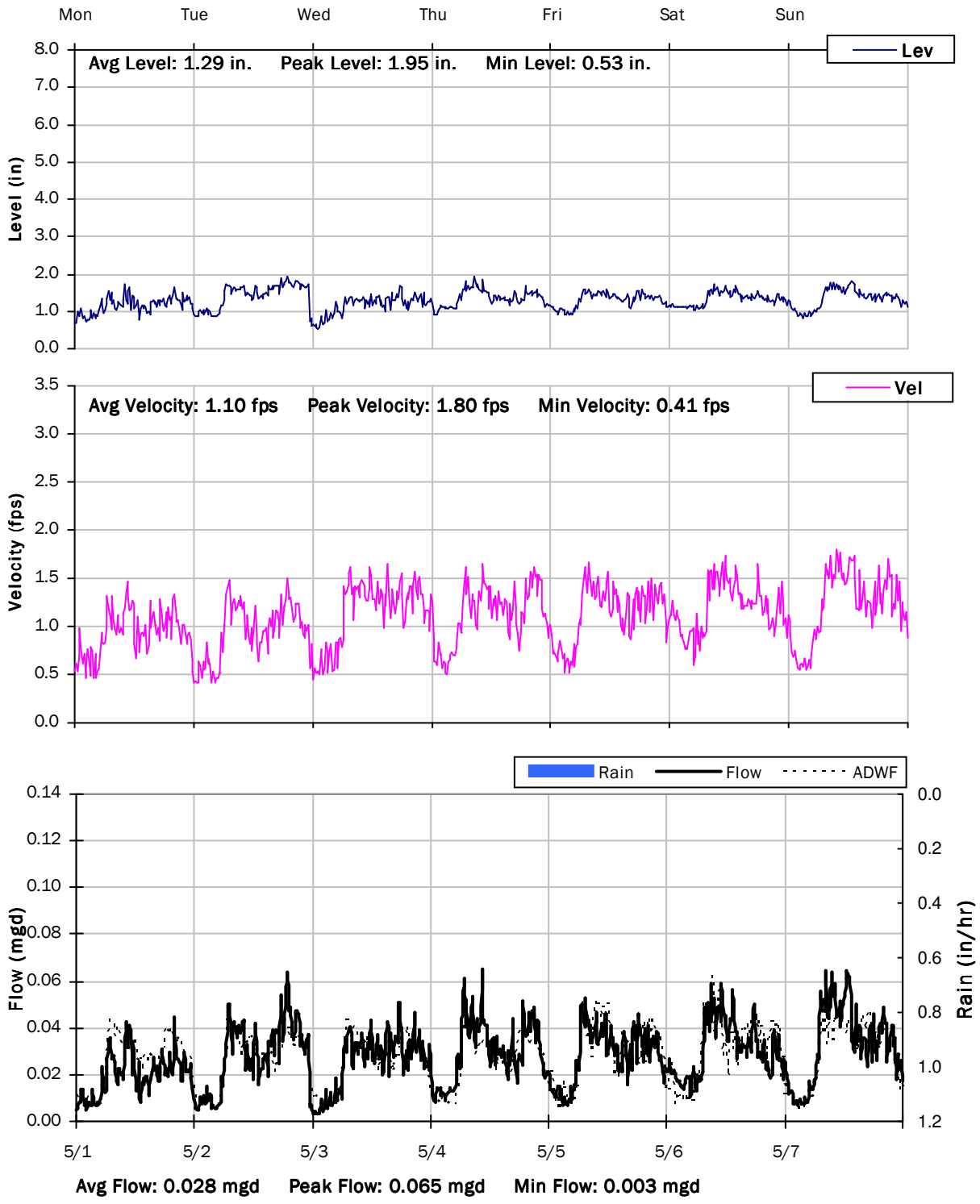
4/24/2017 to 5/1/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

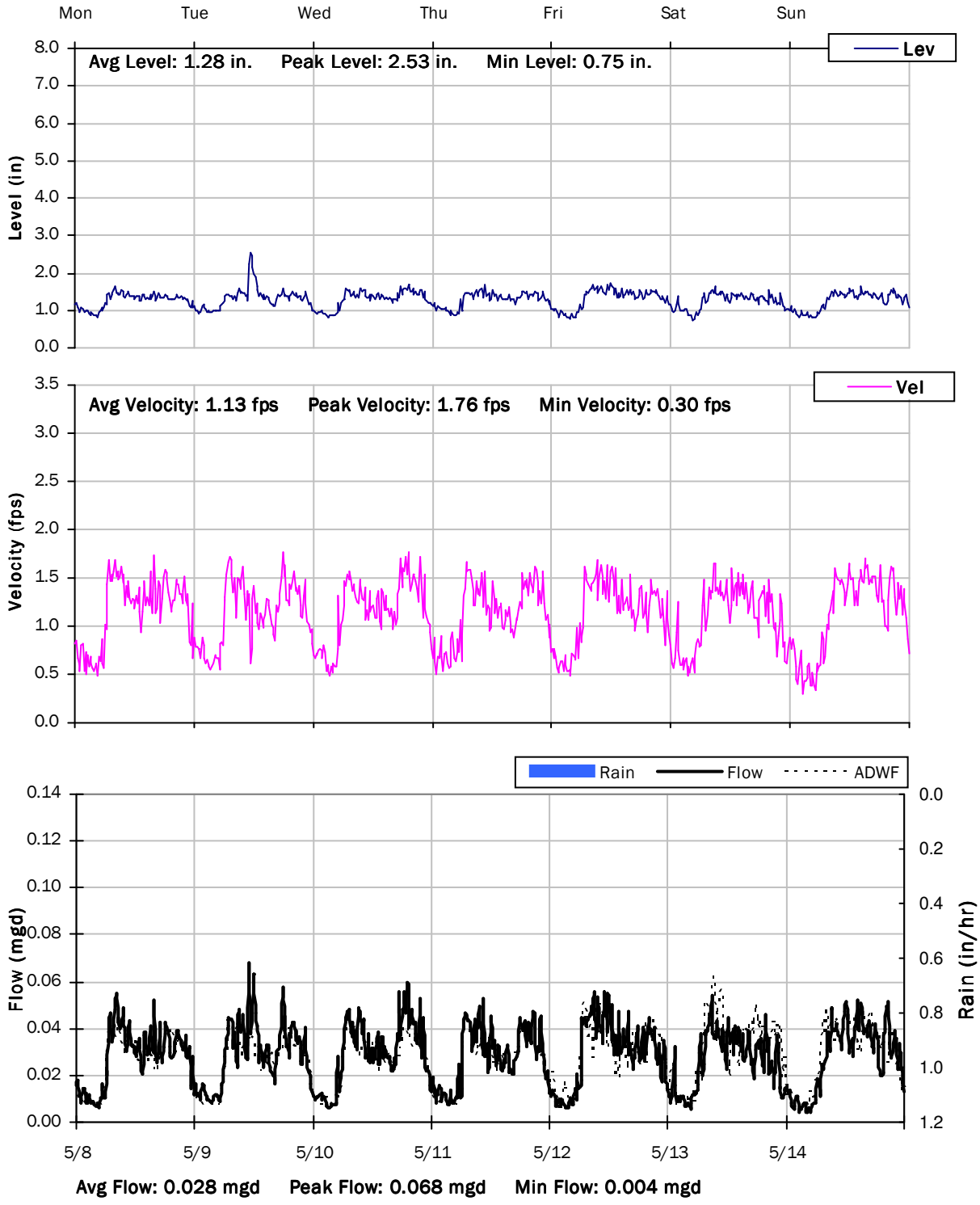
5/1/2017 to 5/8/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

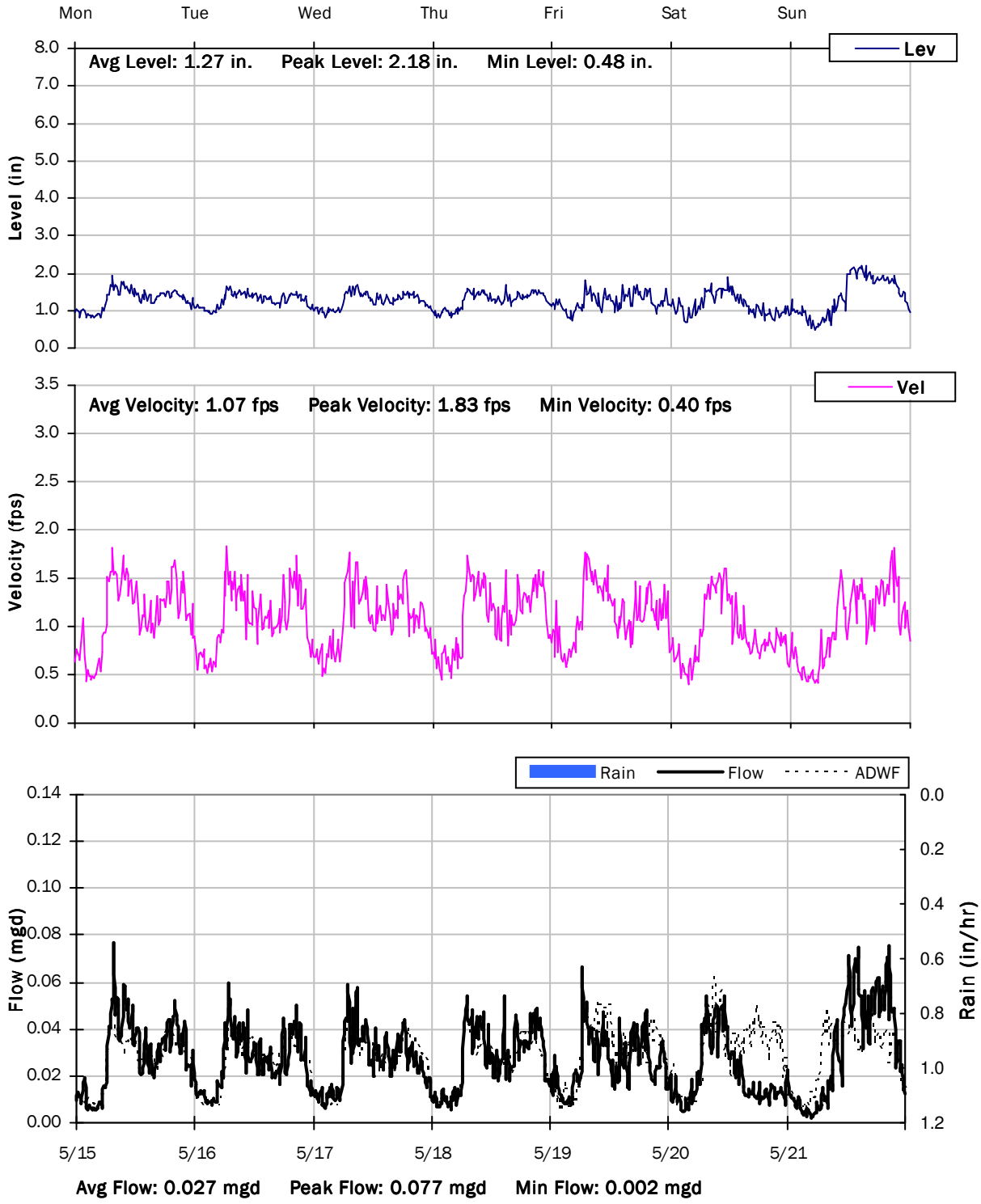
5/8/2017 to 5/15/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

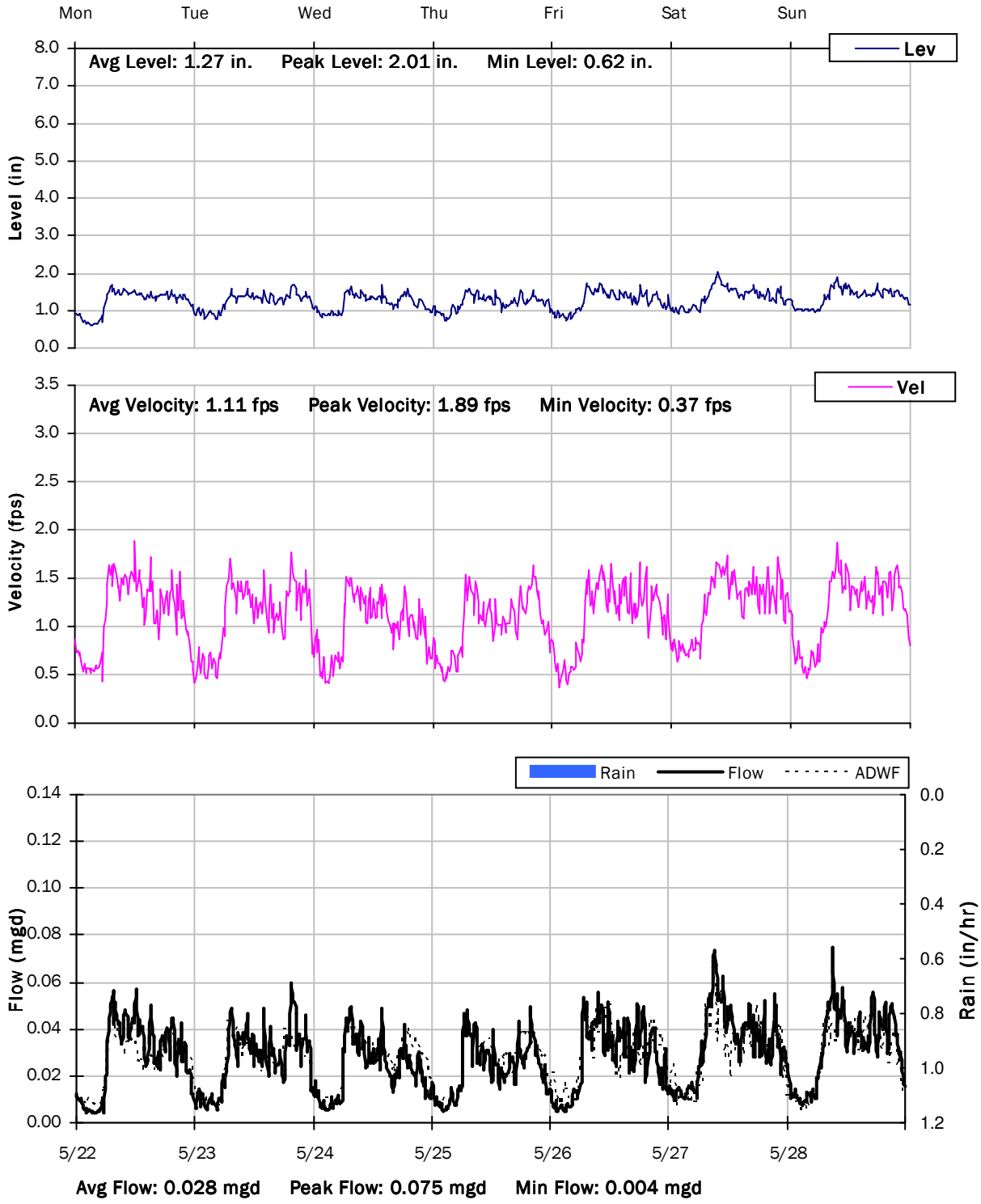
5/15/2017 to 5/22/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

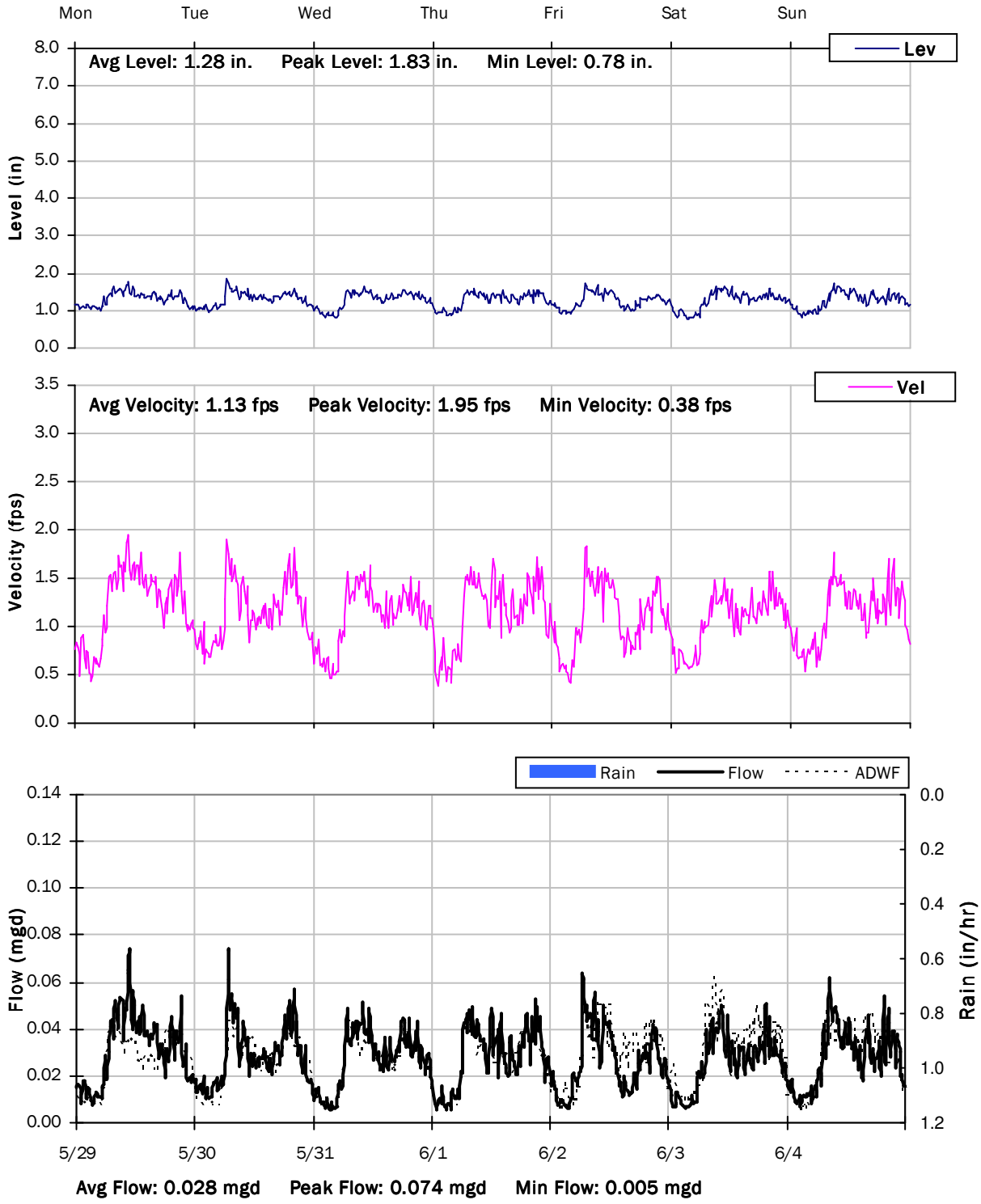
5/22/2017 to 5/29/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

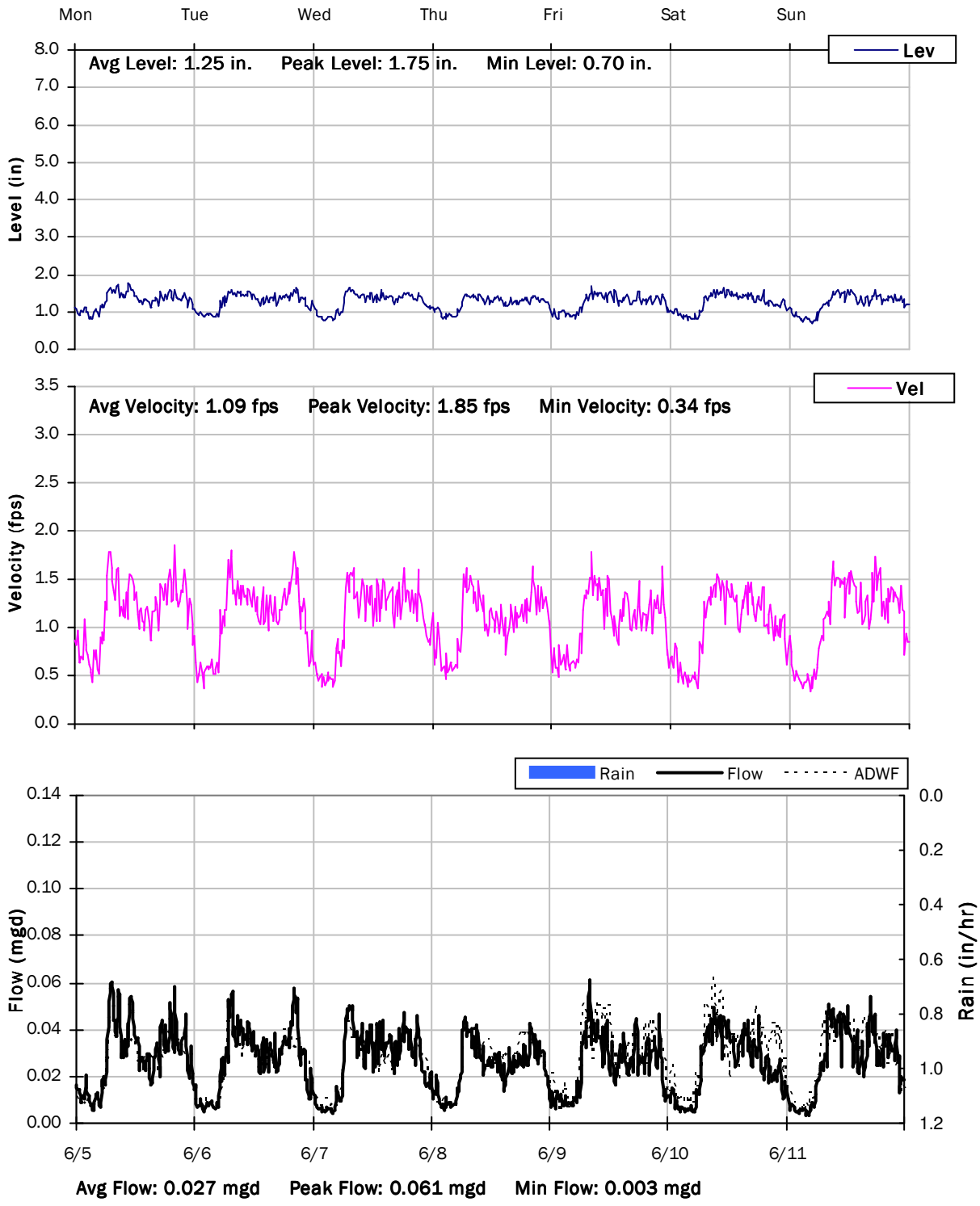
5/29/2017 to 6/5/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

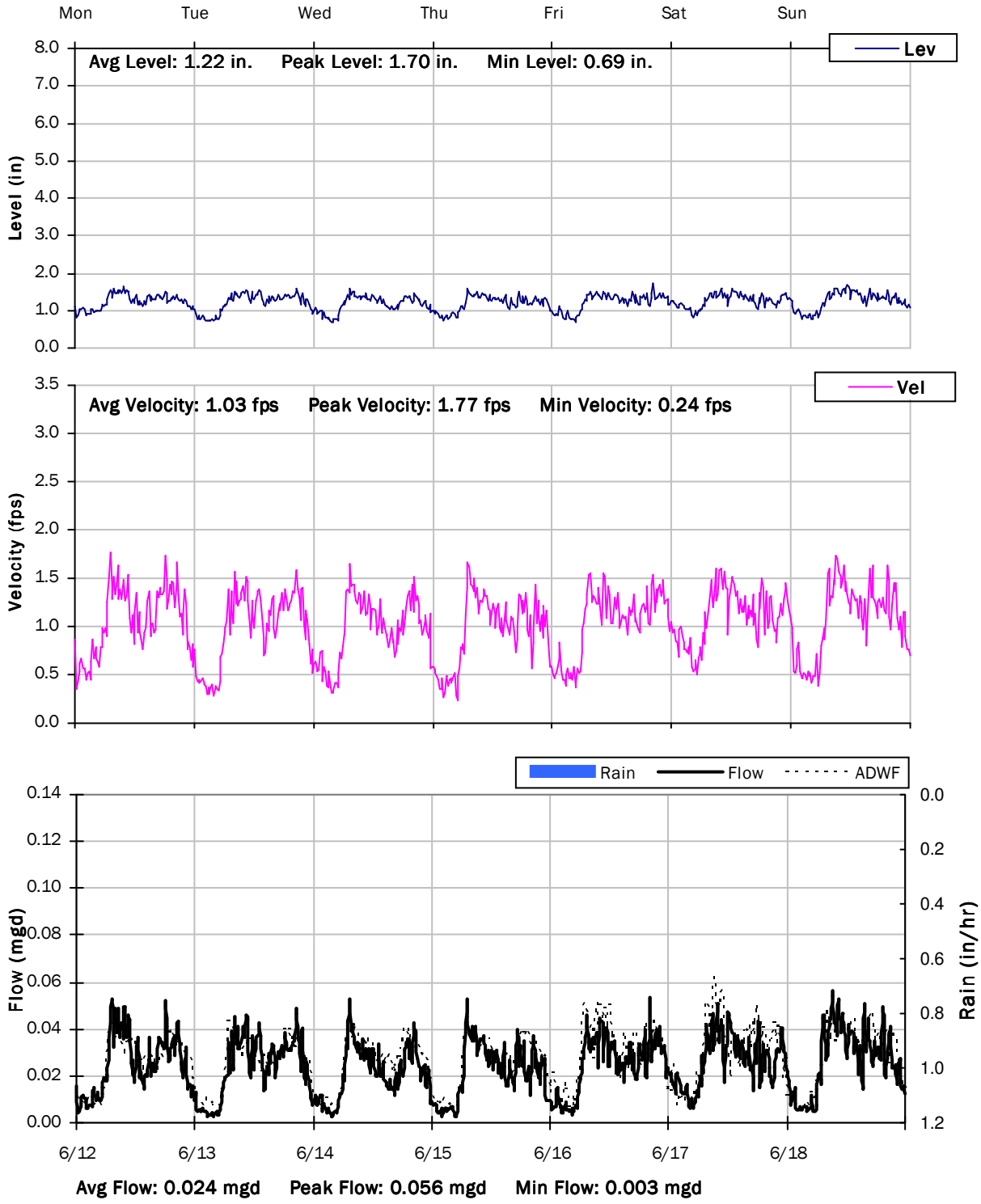
6/5/2017 to 6/12/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

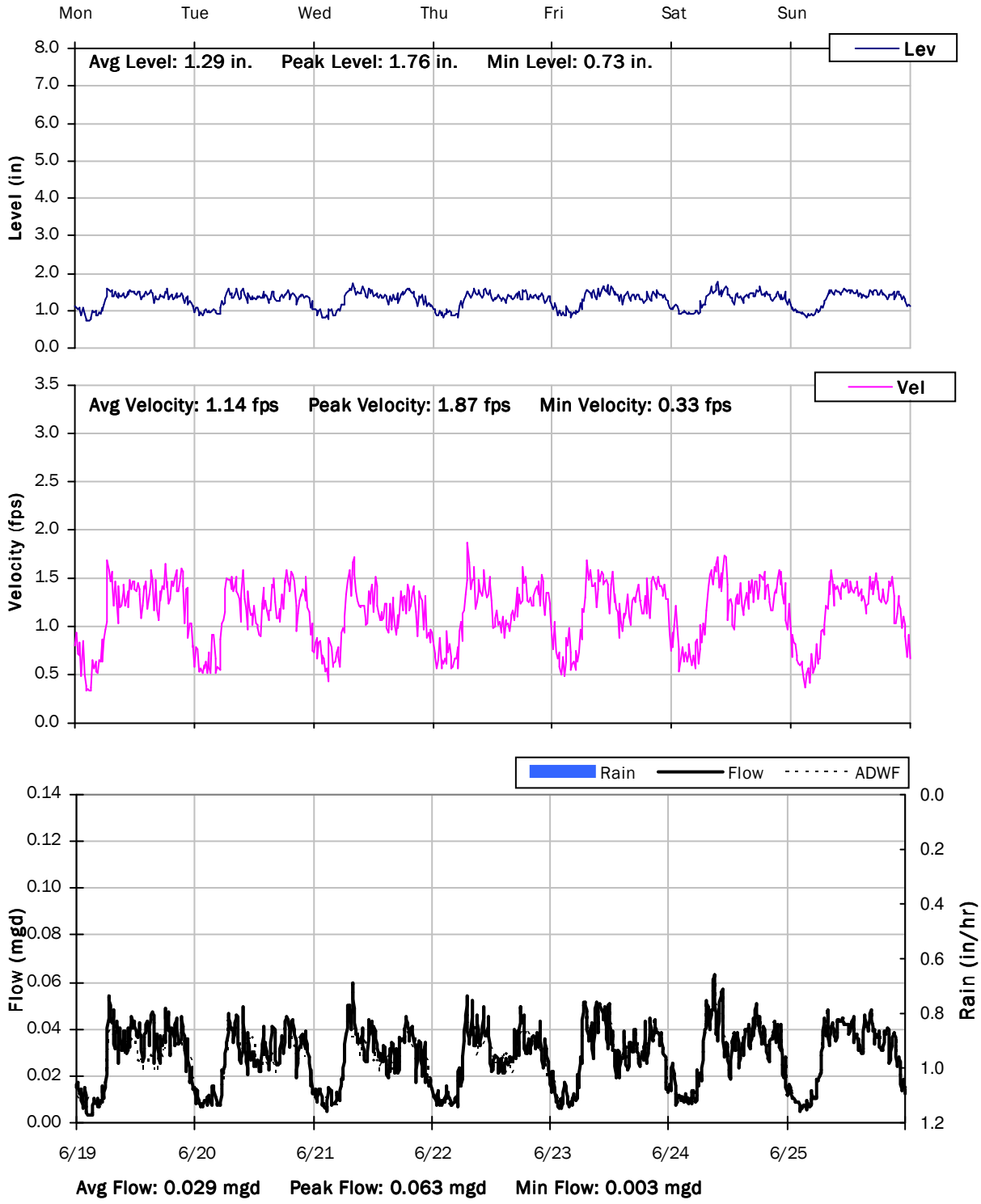
6/12/2017 to 6/19/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

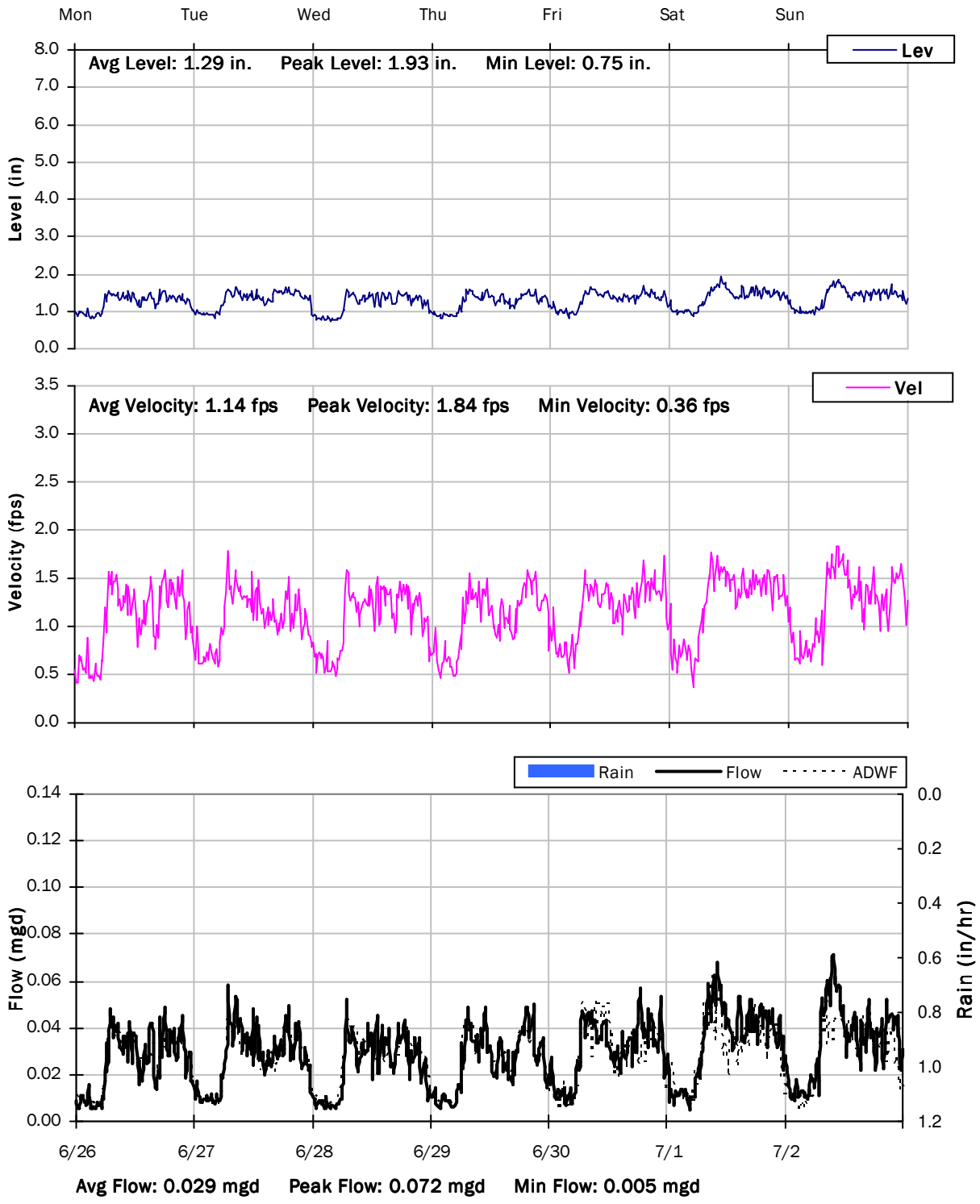
6/19/2017 to 6/26/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

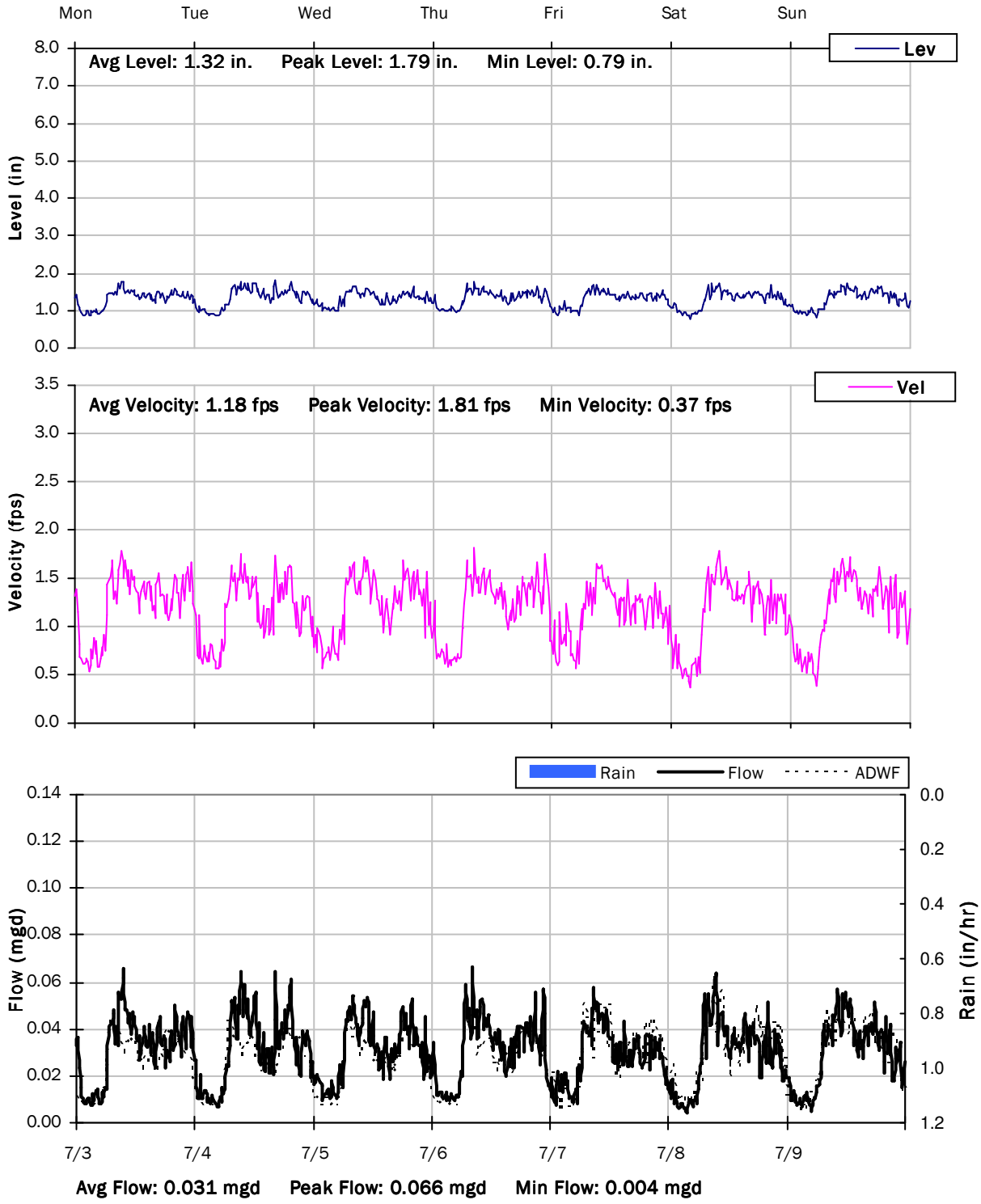
6/26/2017 to 7/3/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

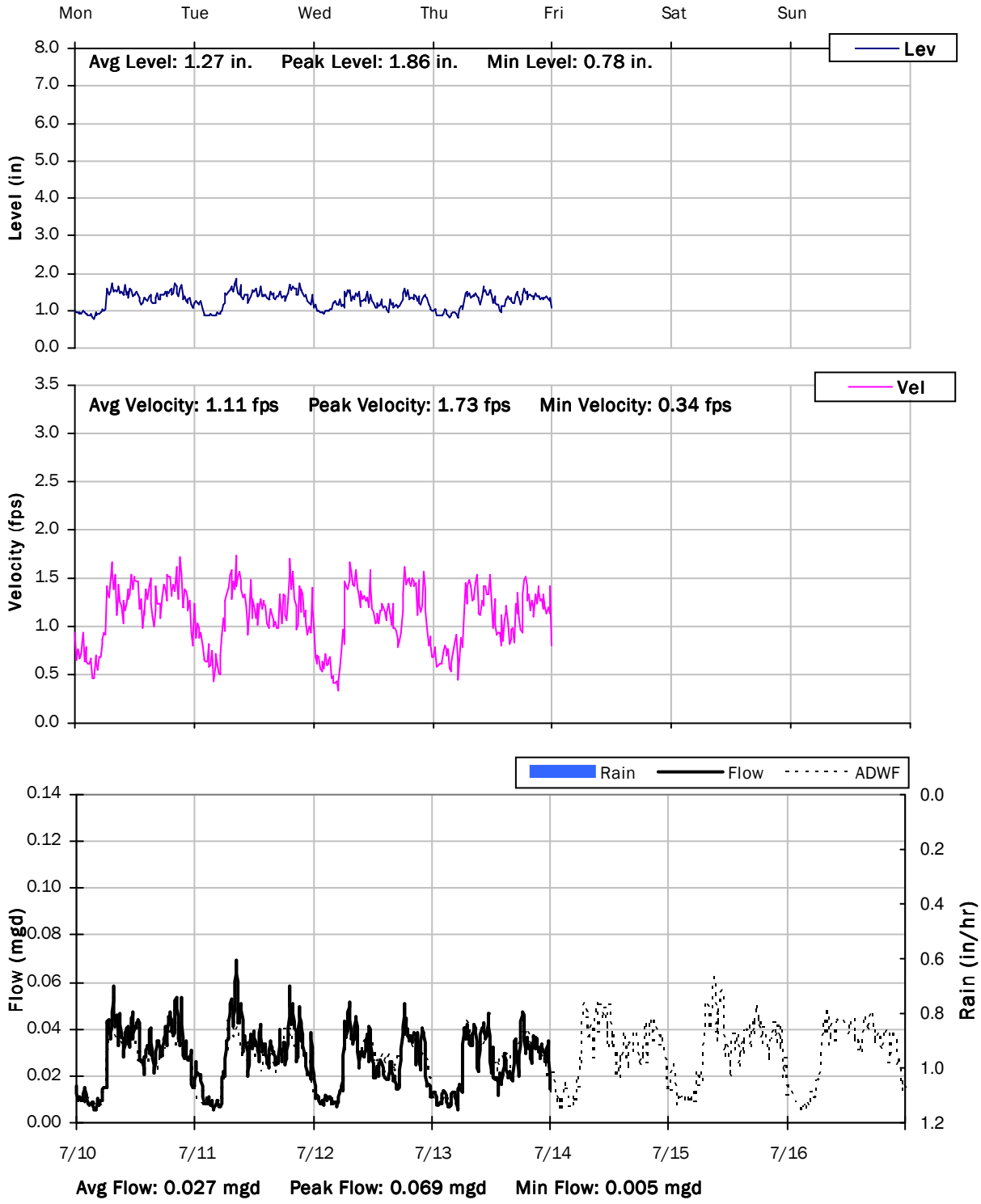
7/3/2017 to 7/10/2017



SITE 02

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

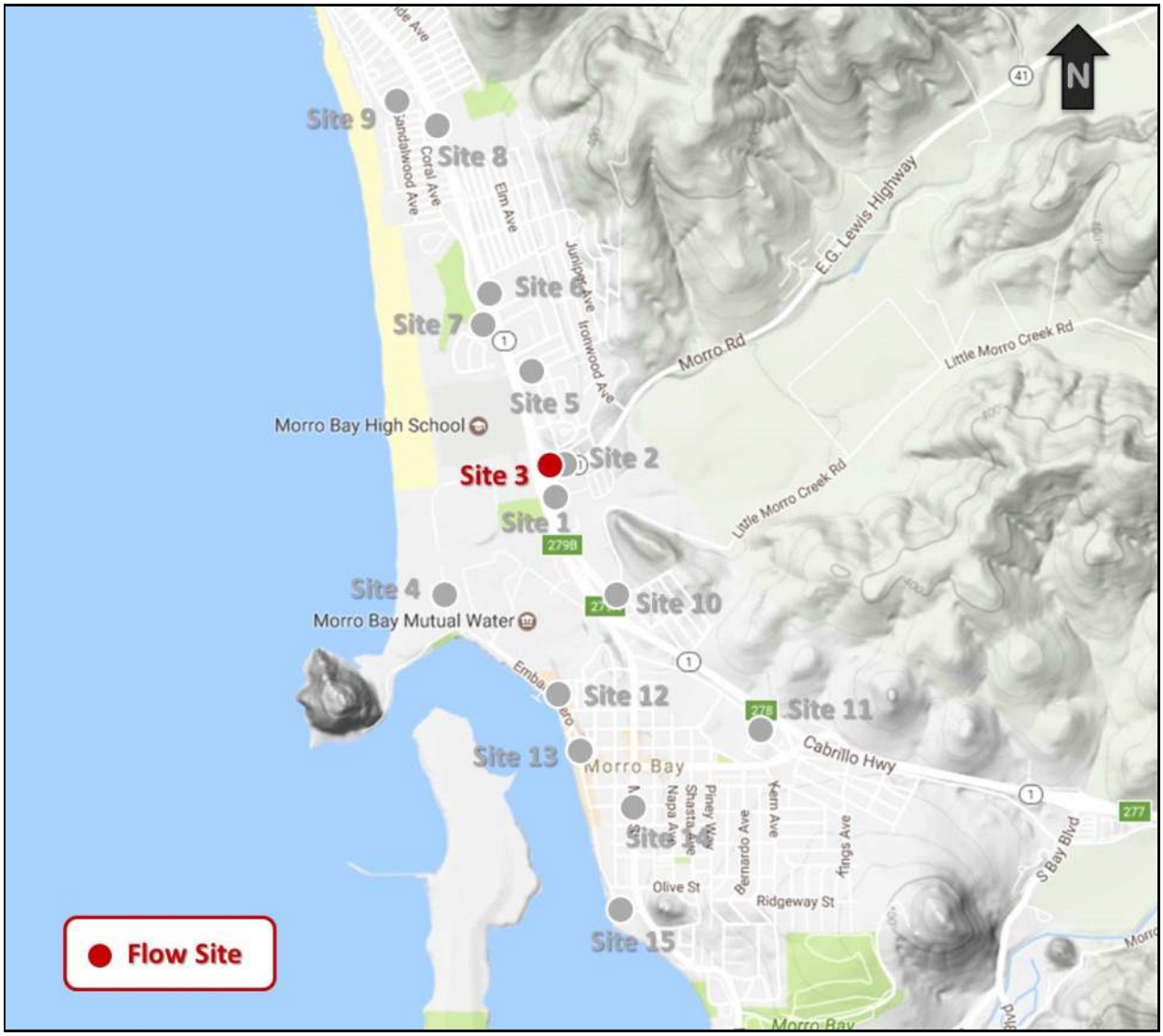
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 03

Location: Island between Main Street and HWY 1 on-ramp, ~135 feet north of Atascadero Road, meter on north 15" inlet

Data Summary Report



Vicinity Map: Site 03

SITE 03

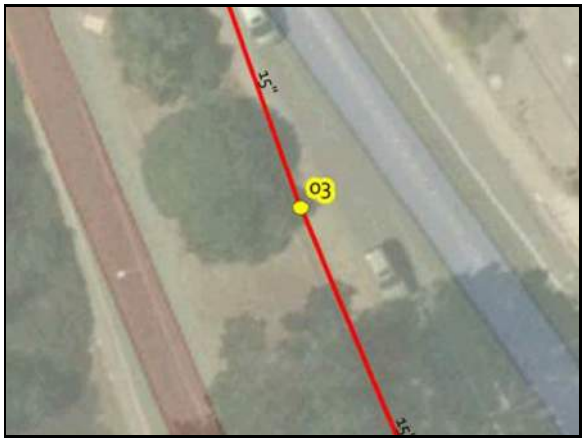
Site Information

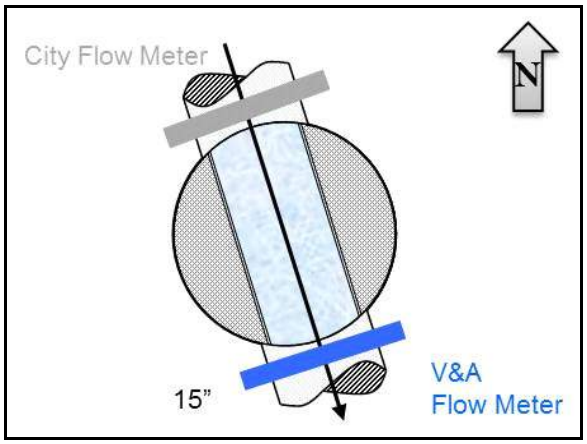
Location: Island between Main Street and HWY 1 on-ramp, ~135 feet north of Atascadero Road, meter on north 15" inlet

Pipe Diameter: 15 inches

ADWF: 0.295 mgd

Peak Measured Flow: 0.889 mgd





SITE 03

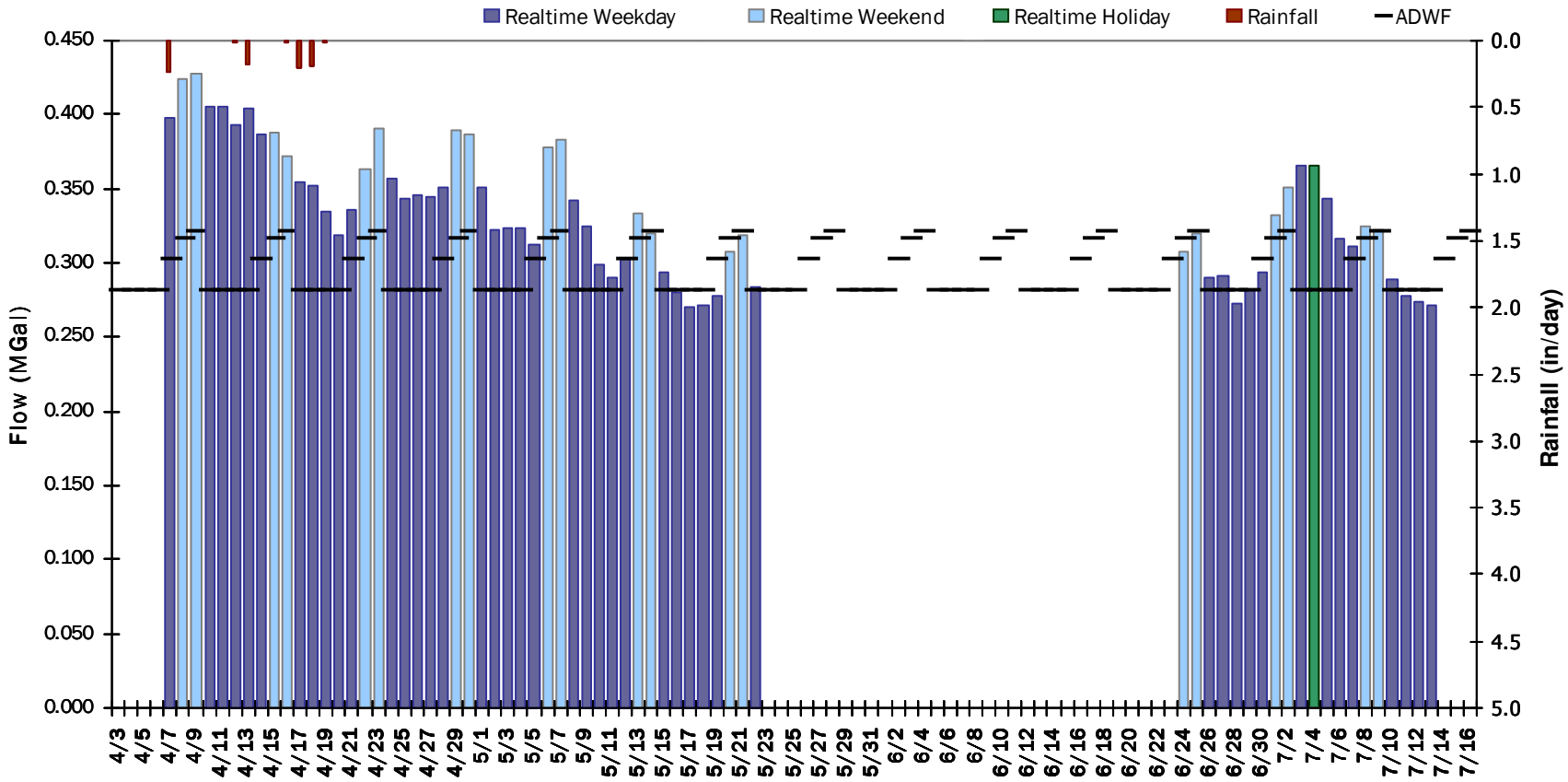
Additional Site Photos

SITE 03

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.335 MGal Peak Daily Flow: 0.428 MGal Min Daily Flow: 0.270 MGal

Total Period Rainfall: 0.85 inches

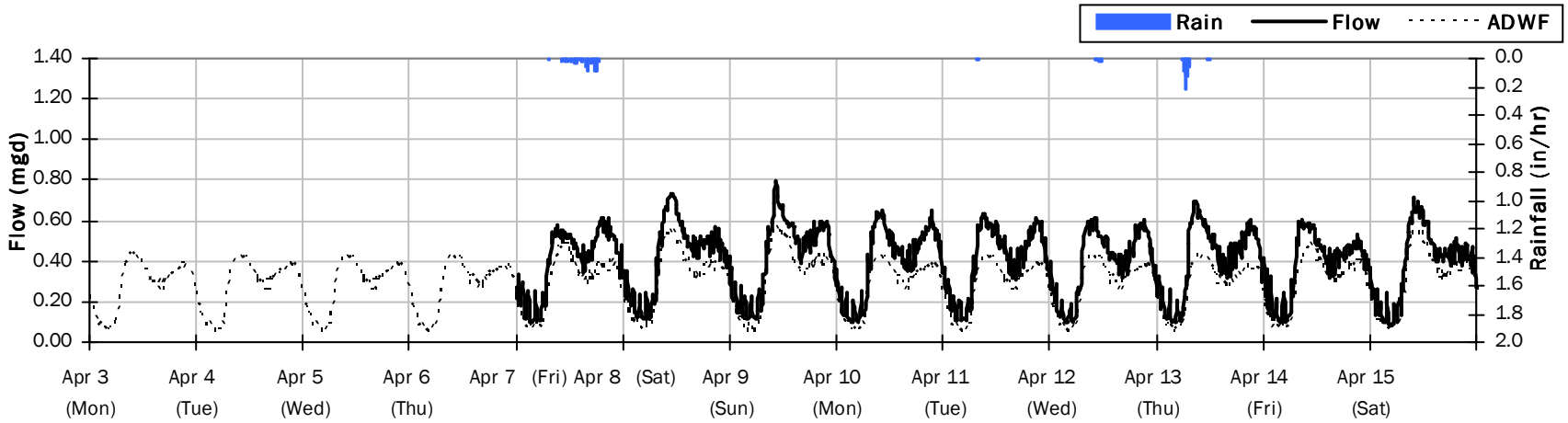


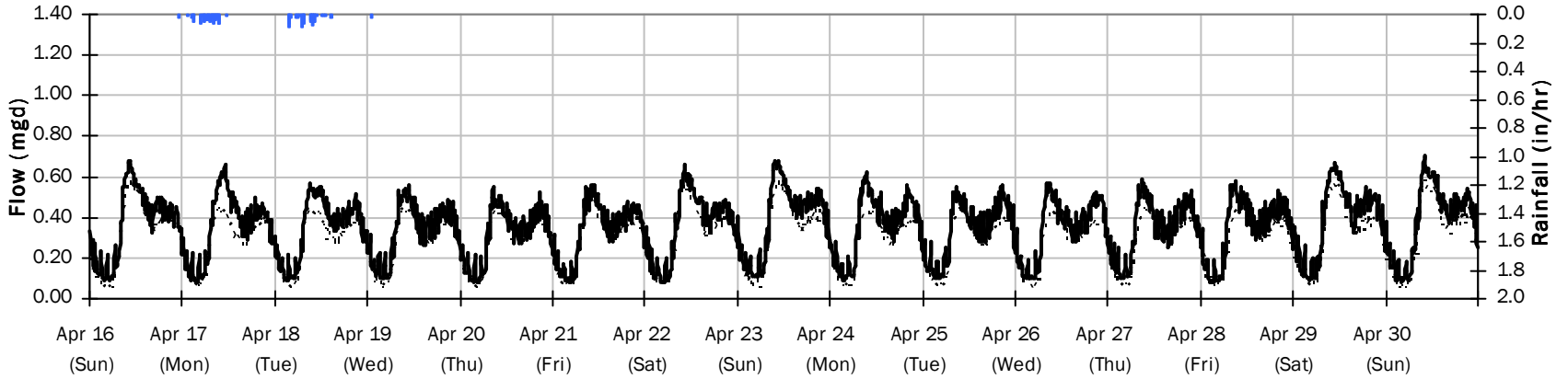
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 03

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.374 mgd** **Peak Flow: 0.794 mgd** **Min Flow: 0.078 mgd**

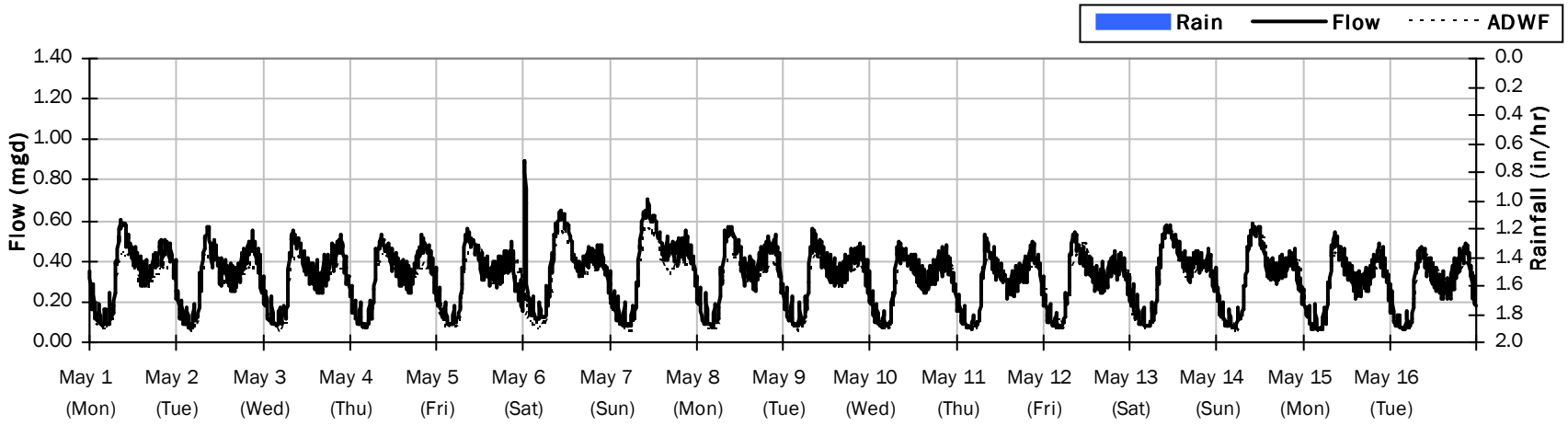


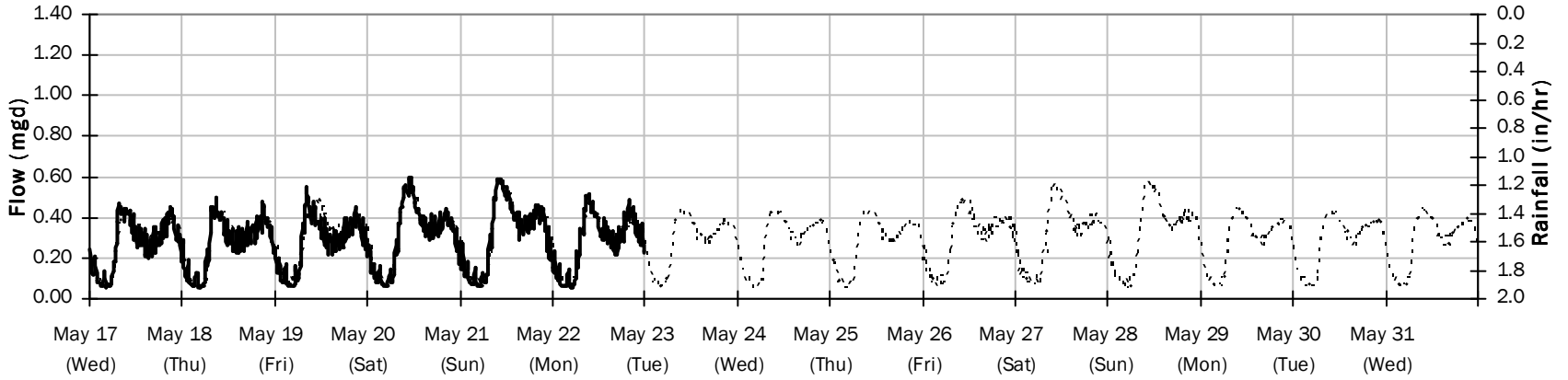


SITE 03

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.314 mgd** **Peak Flow: 0.889 mgd** **Min Flow: 0.055 mgd**



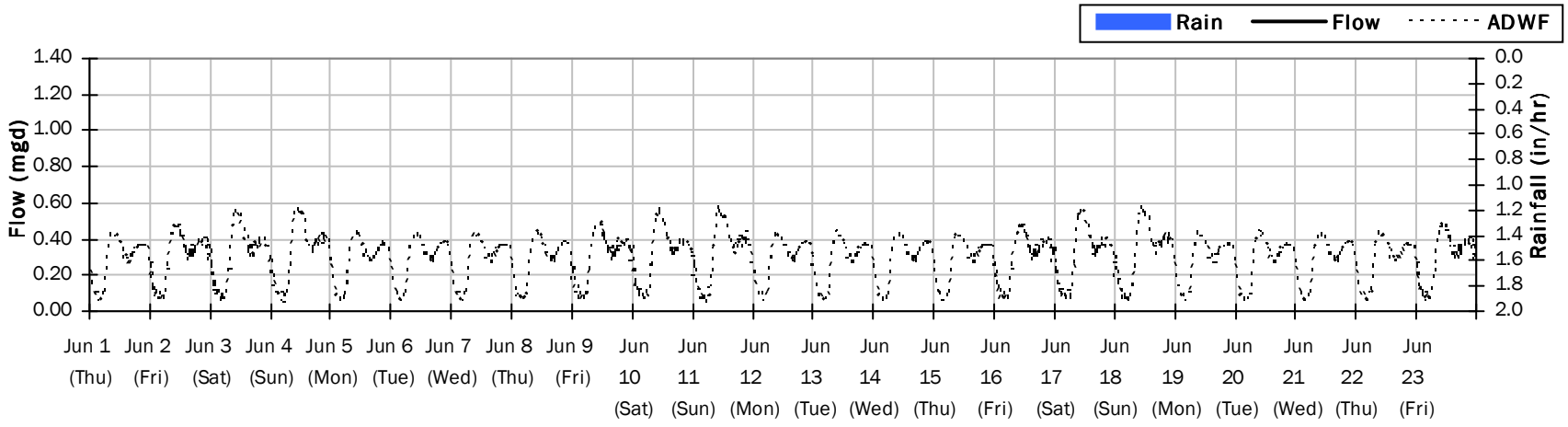


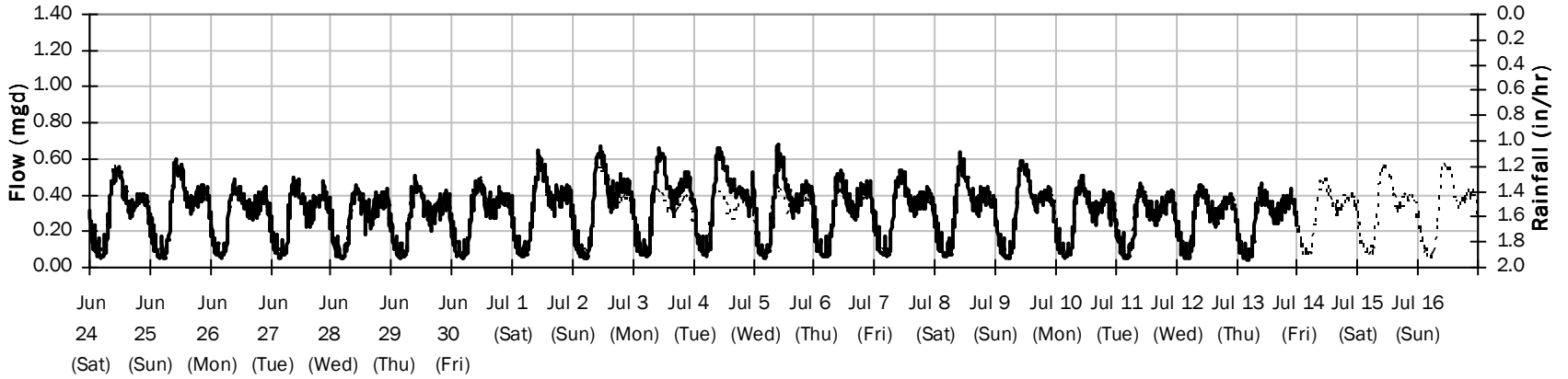
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 03

Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.310 mgd** **Peak Flow: 0.684 mgd** **Min Flow: 0.041 mgd**





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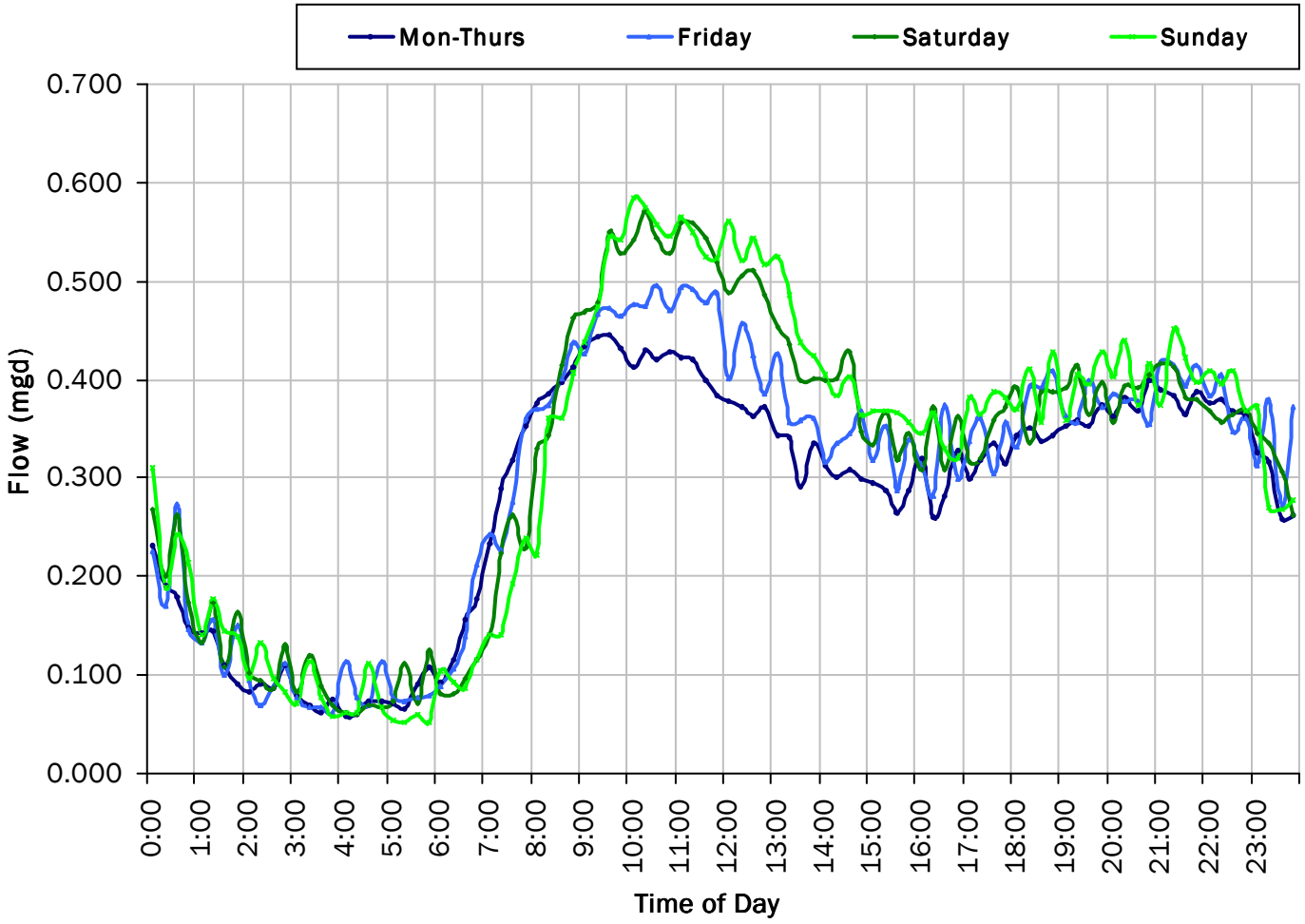
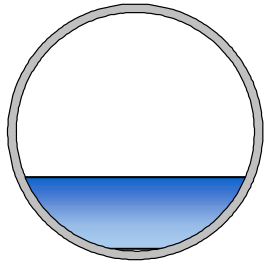
SITE 03

Average Dry Weather Flow Hydrographs



ADWF:

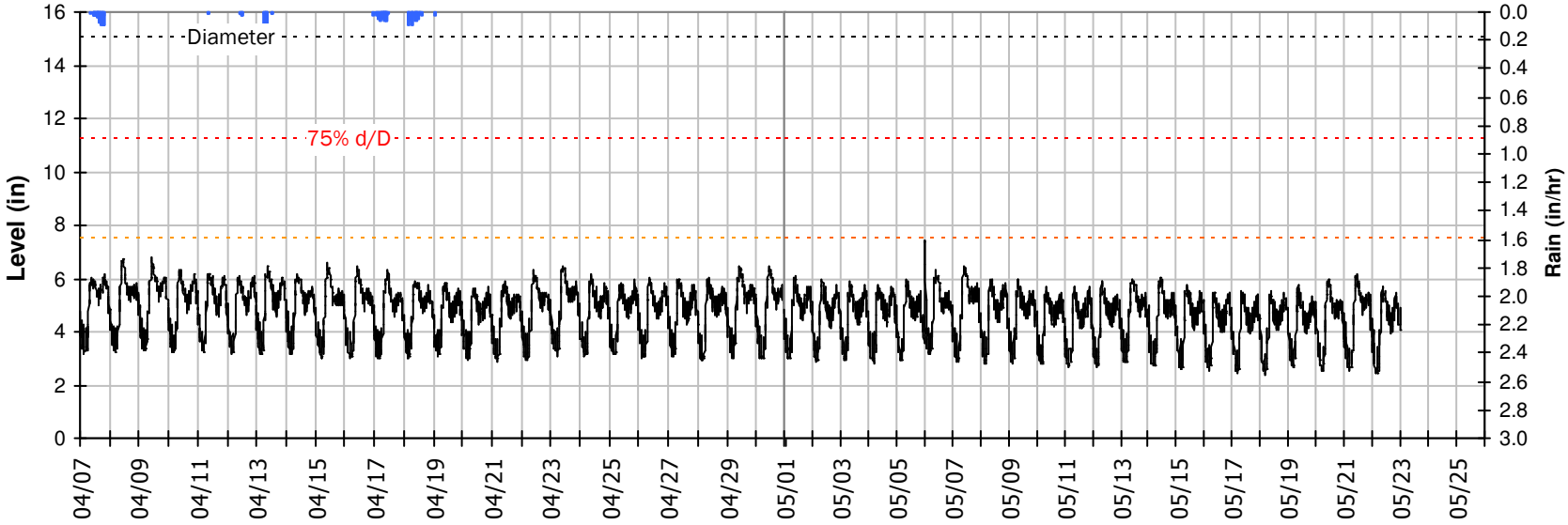
0.295 *mgd*

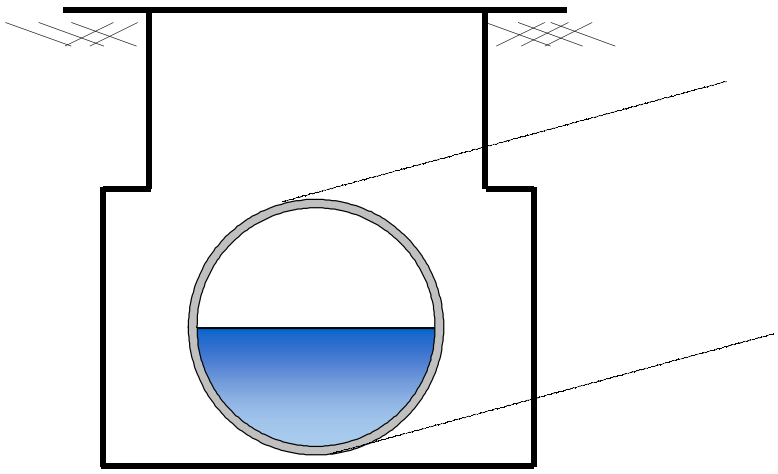


SITE 03

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





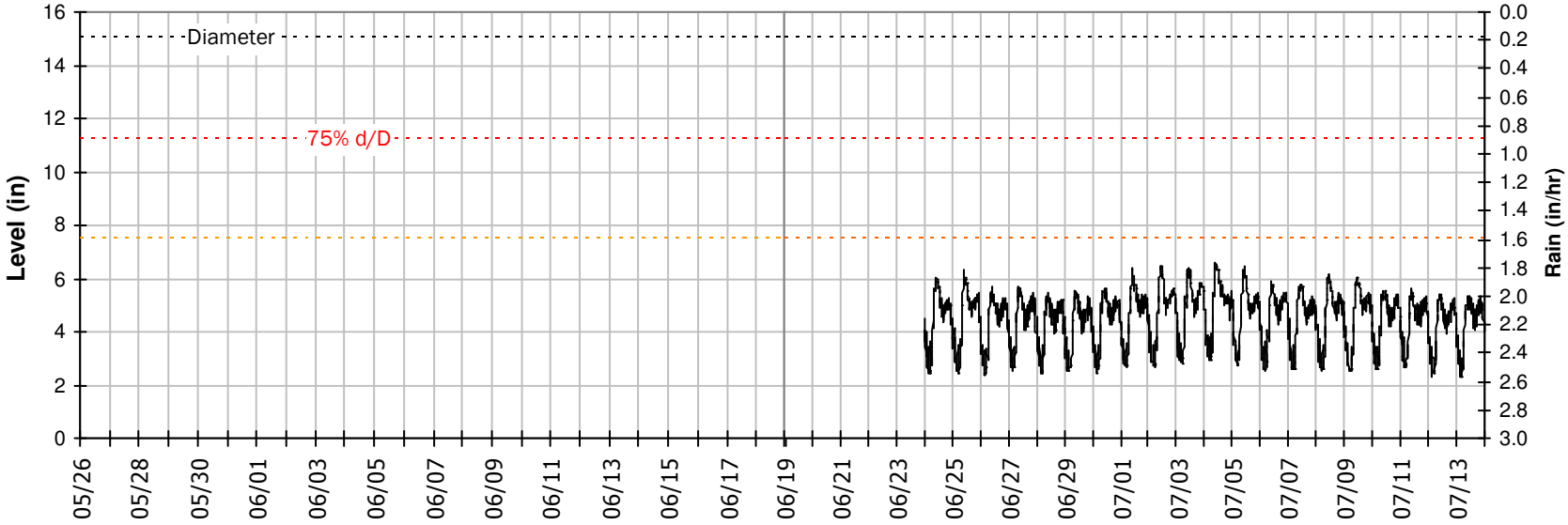
Pipe Diameter:	15	*inches*
Peak Measured Level:	7.36	*inches*
Peak d/D Ratio:	0.49	

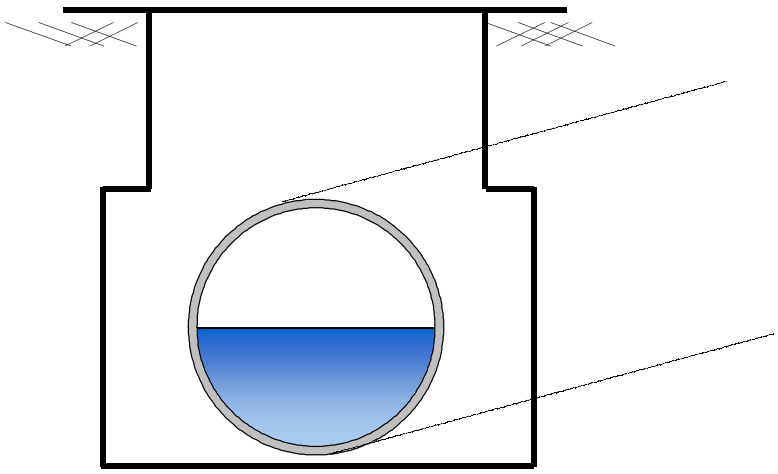
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 03

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





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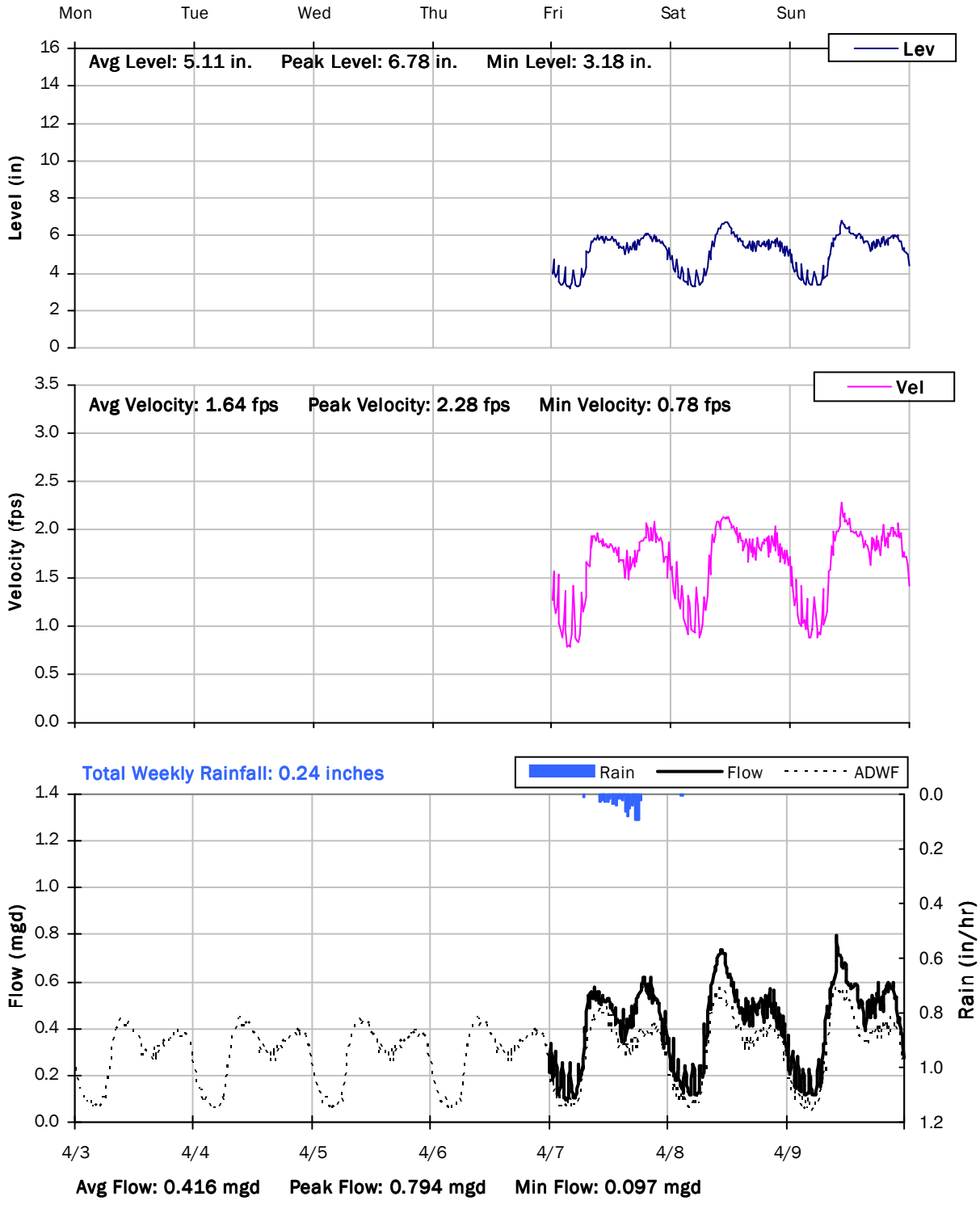
SITE 03

Weekly Level, Velocity and Flow Hydrographs

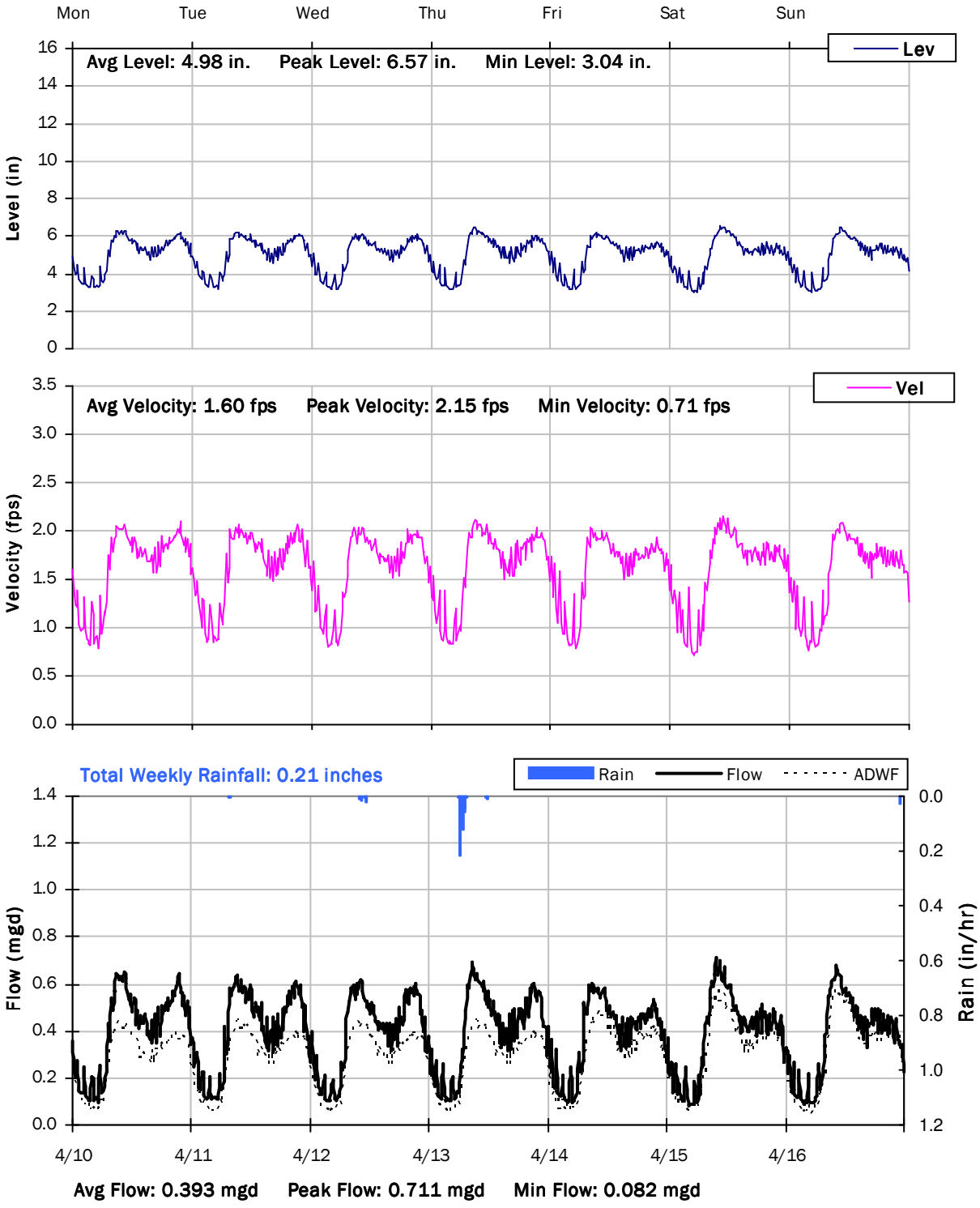
4/3/2017 to 4/10/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

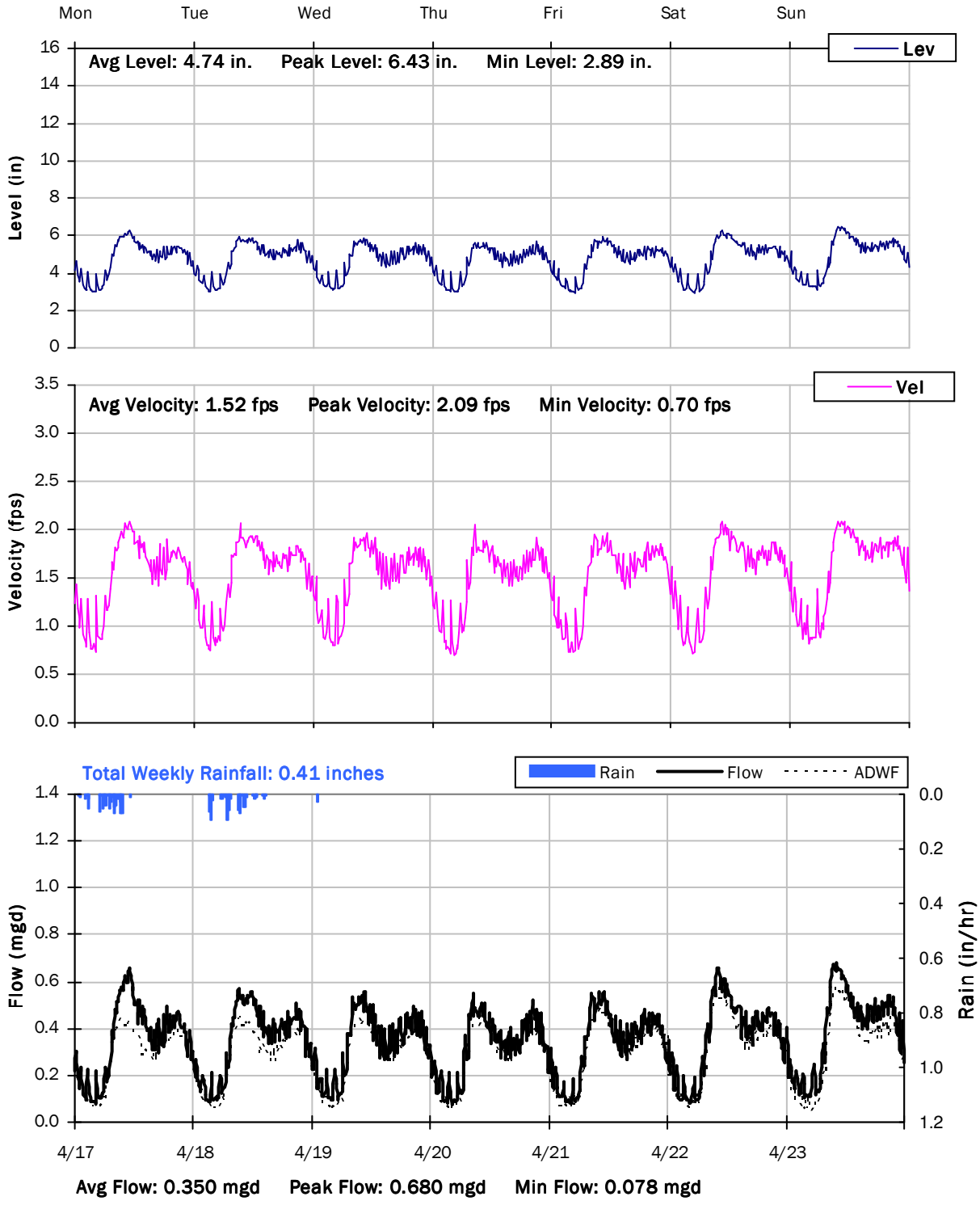
4/10/2017 to 4/17/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

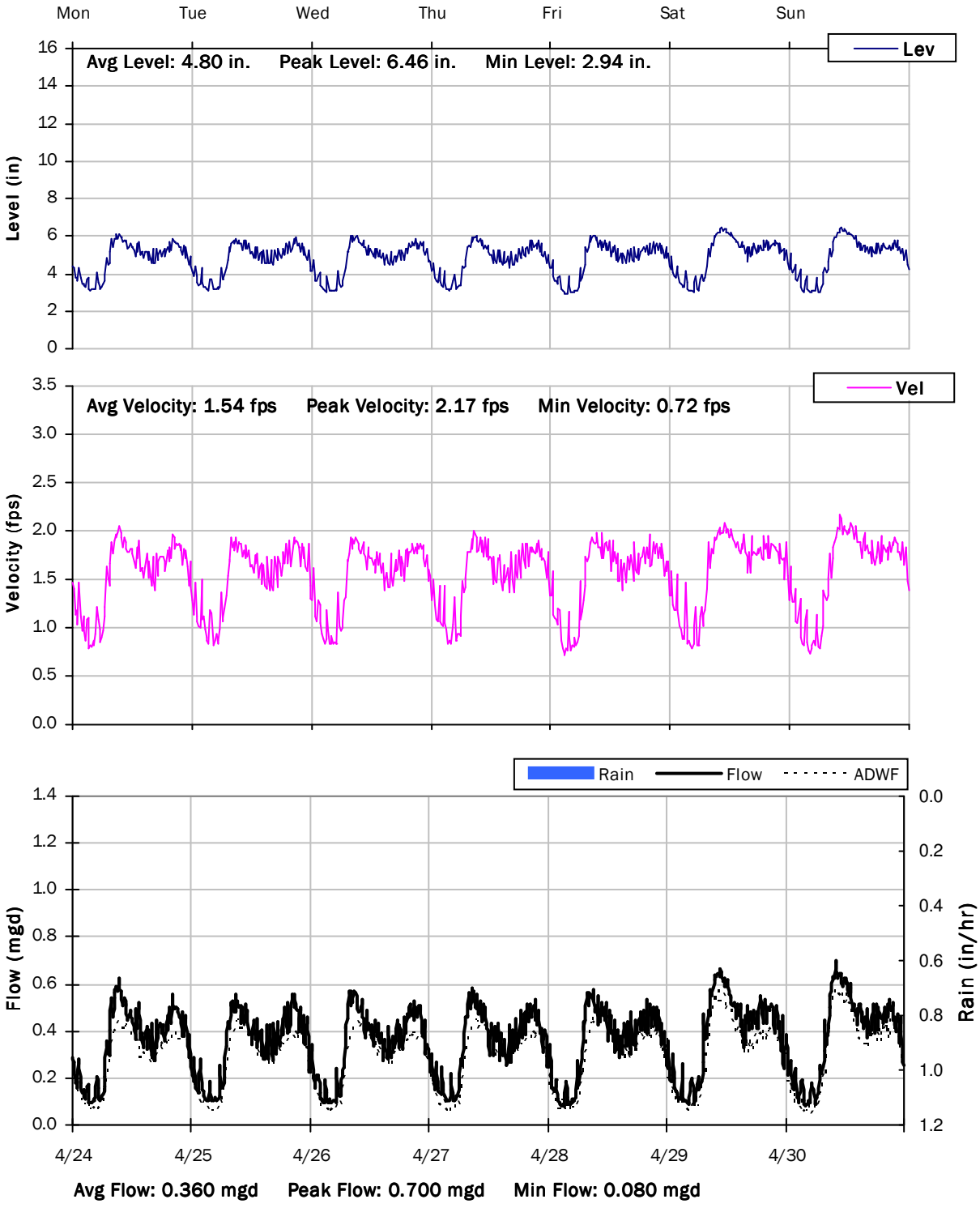
4/17/2017 to 4/24/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

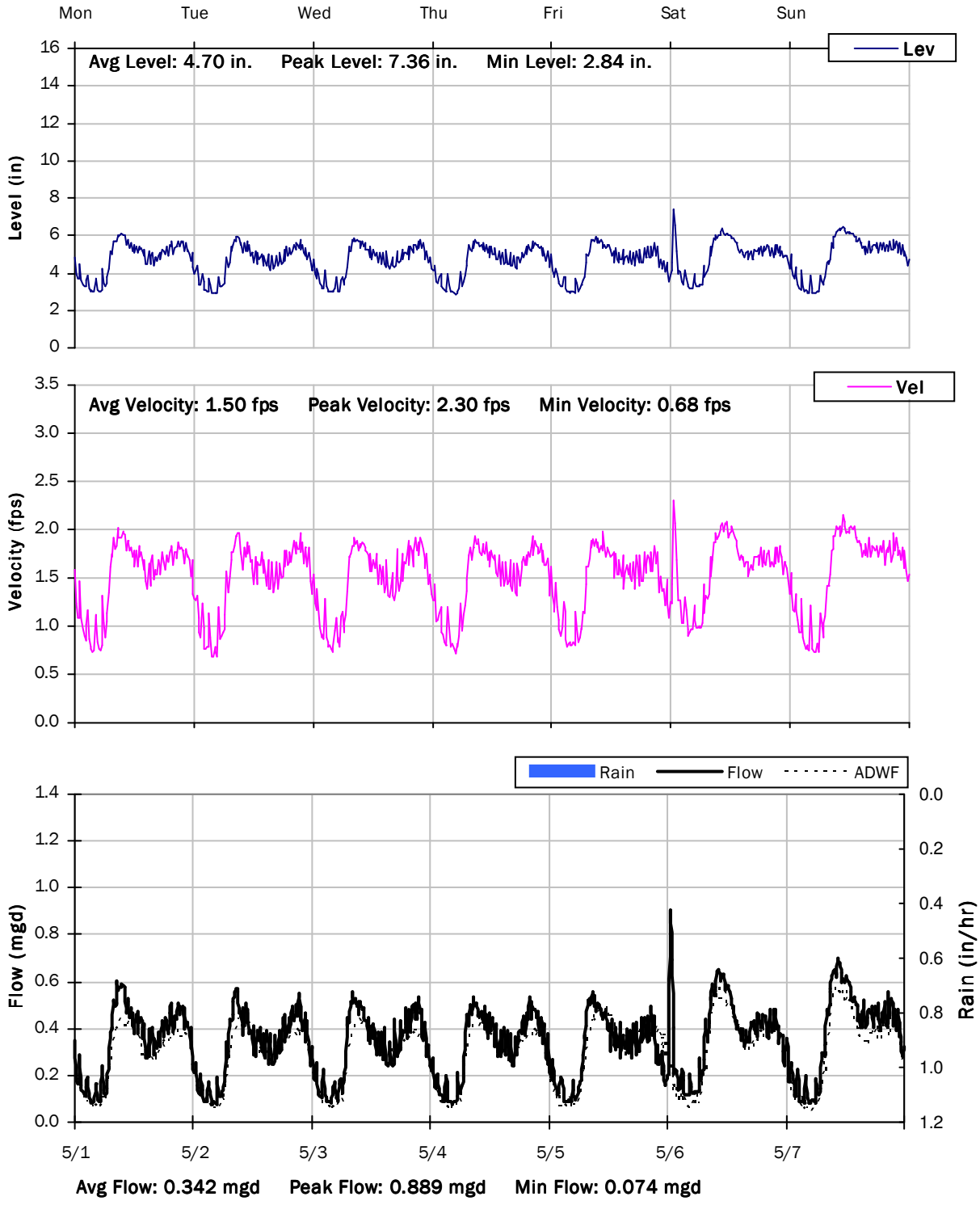
4/24/2017 to 5/1/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

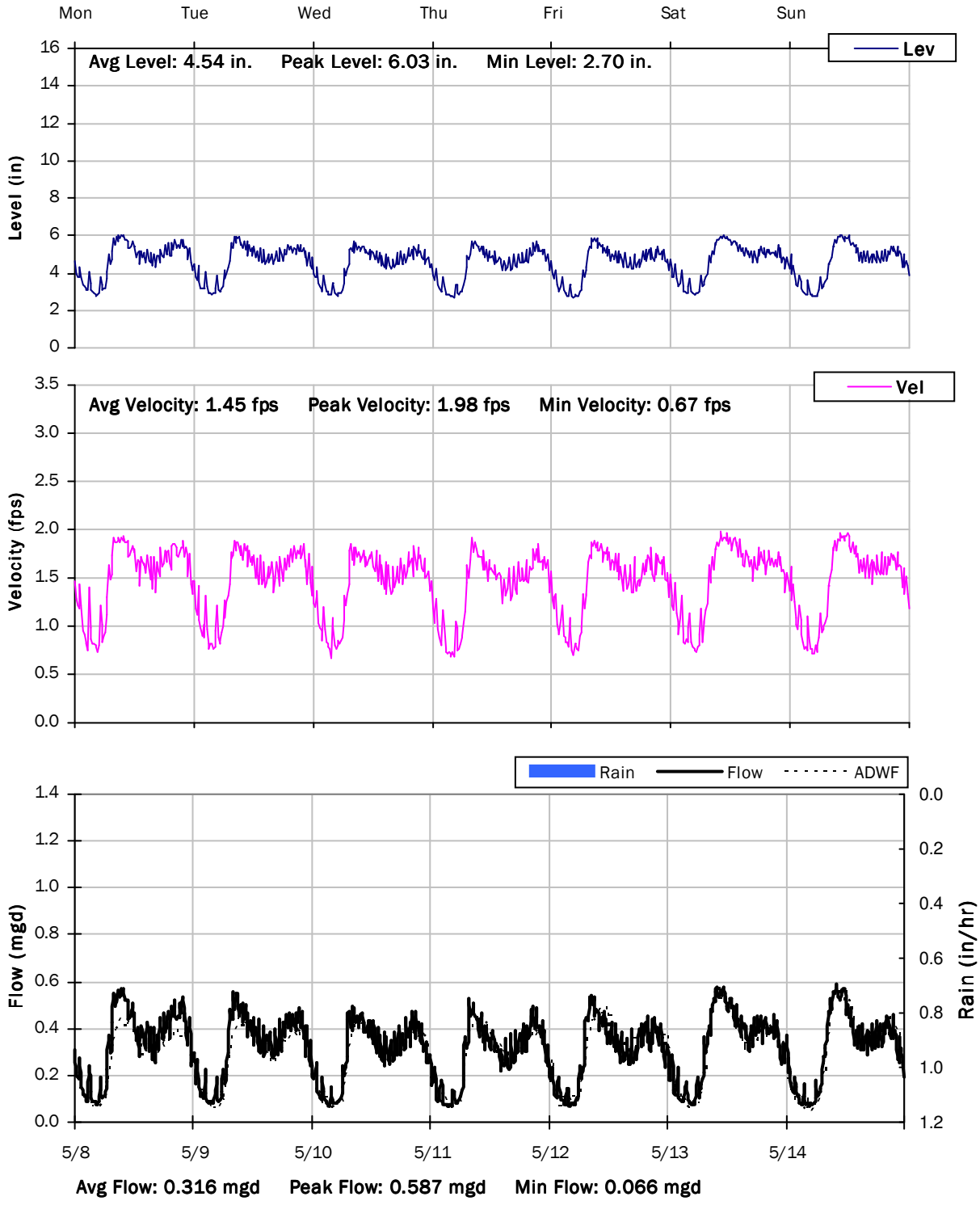
5/1/2017 to 5/8/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

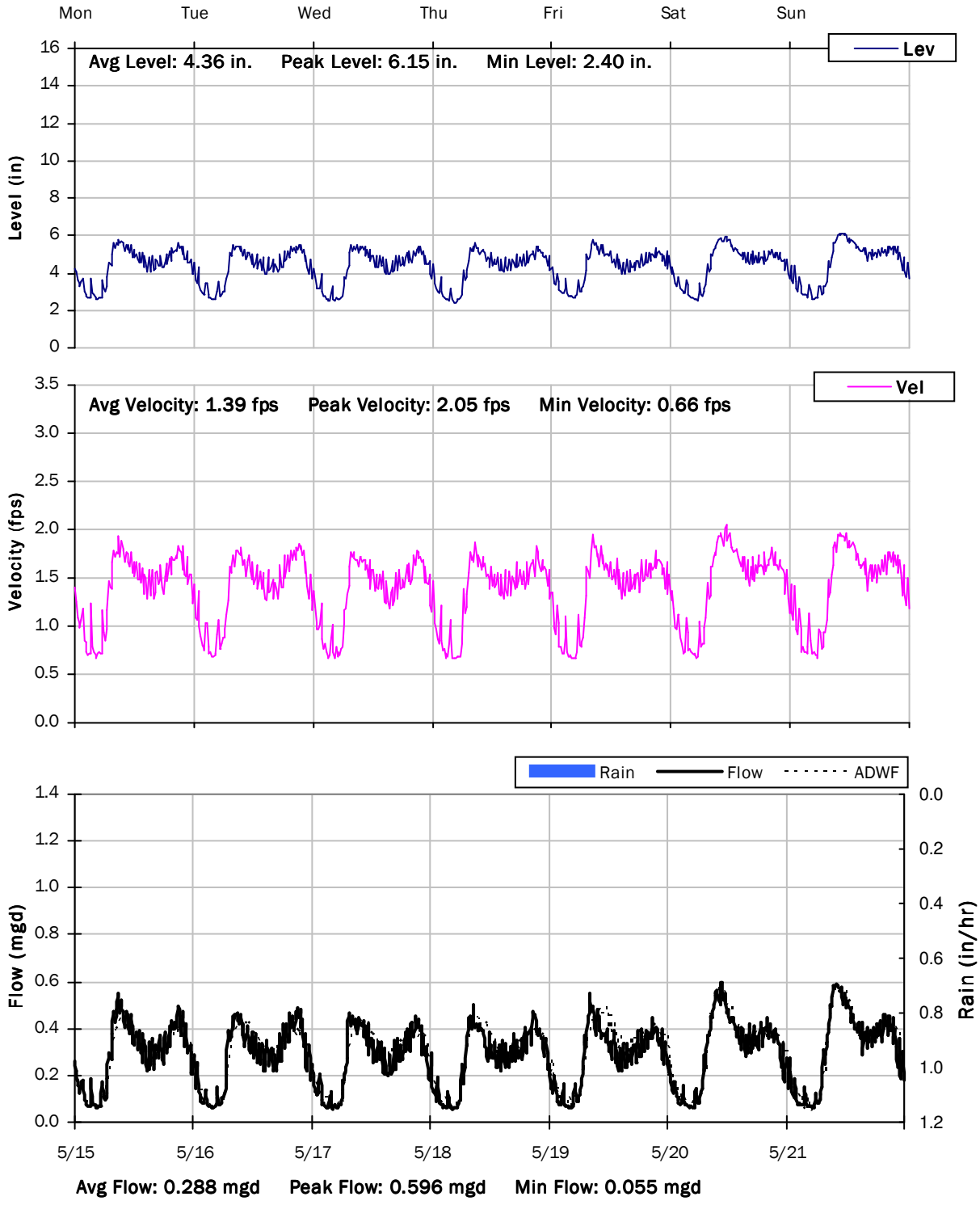
5/8/2017 to 5/15/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

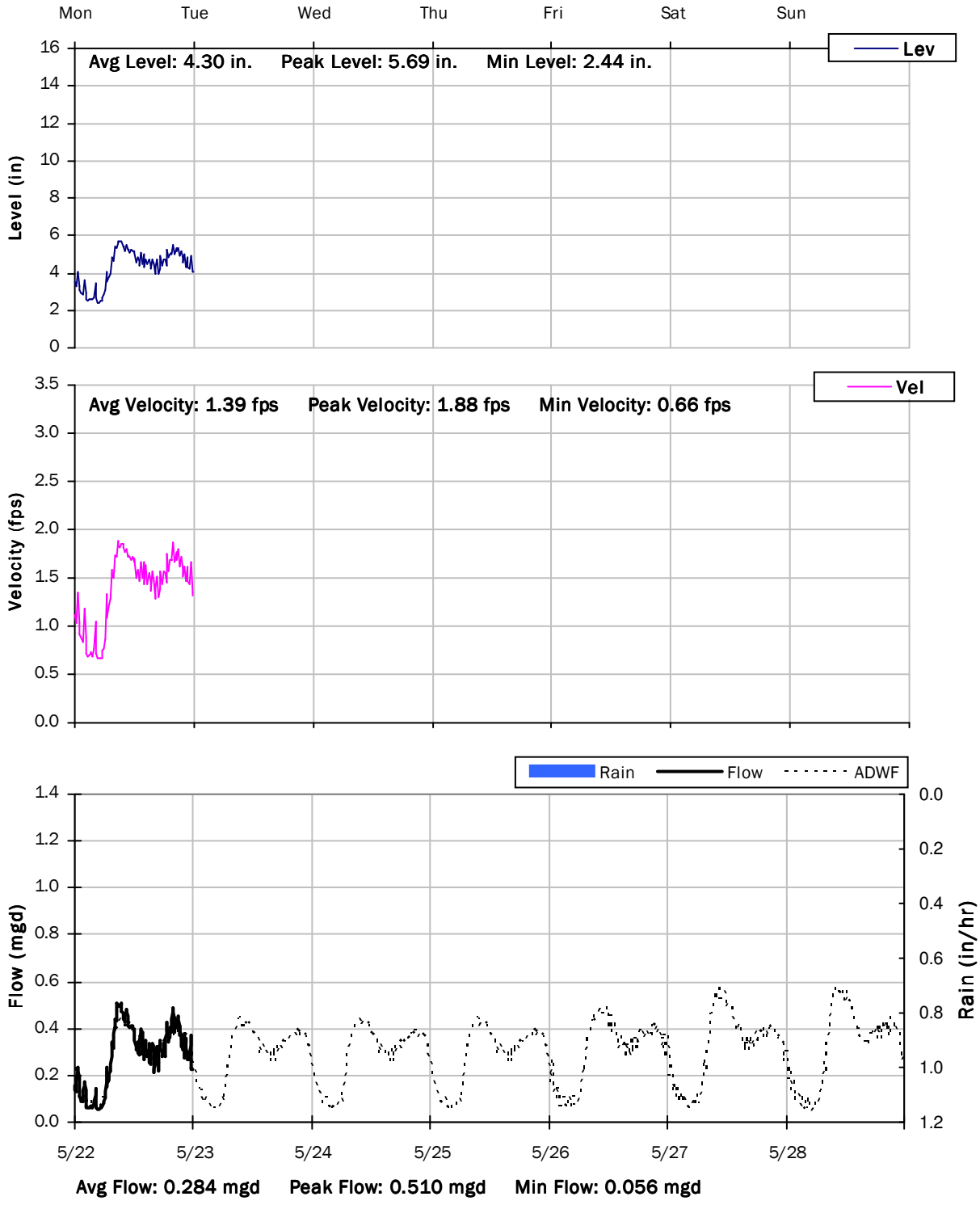
5/15/2017 to 5/22/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

5/22/2017 to 5/29/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

5/29/2017 to 6/5/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 03

Weekly Level, Velocity and Flow Hydrographs

6/5/2017 to 6/12/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 03

Weekly Level, Velocity and Flow Hydrographs

6/12/2017 to 6/19/2017

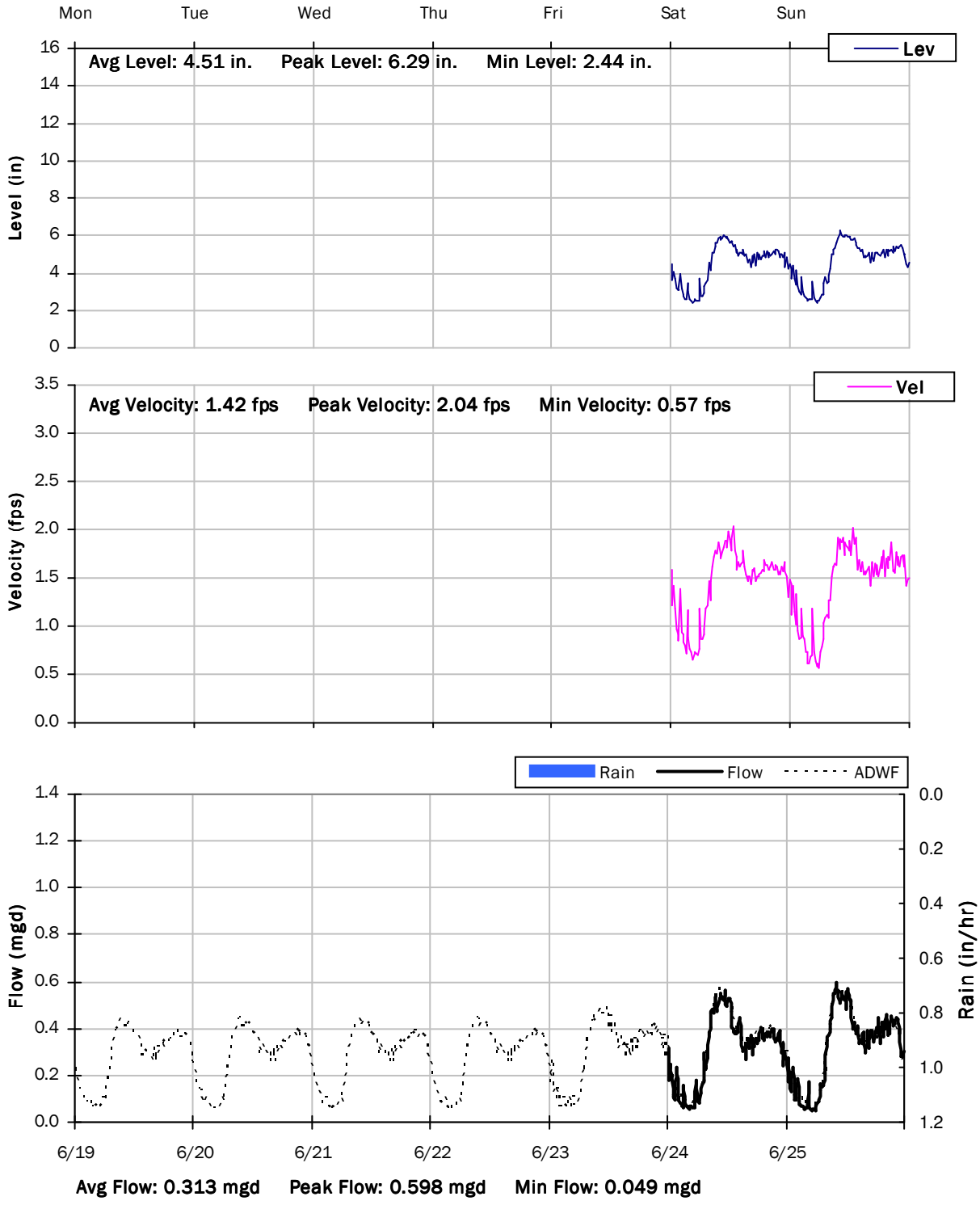
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 03

Weekly Level, Velocity and Flow Hydrographs

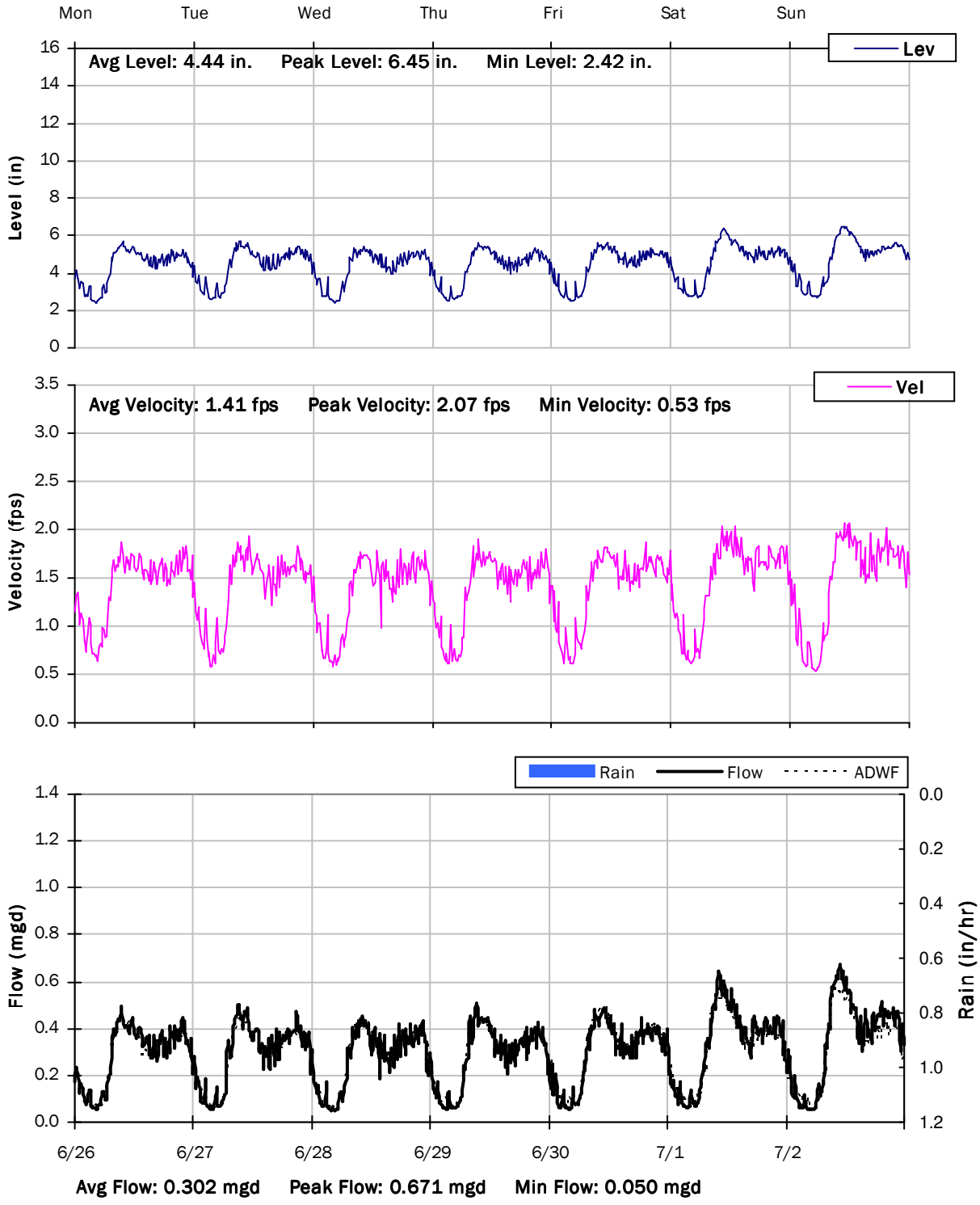
6/19/2017 to 6/26/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

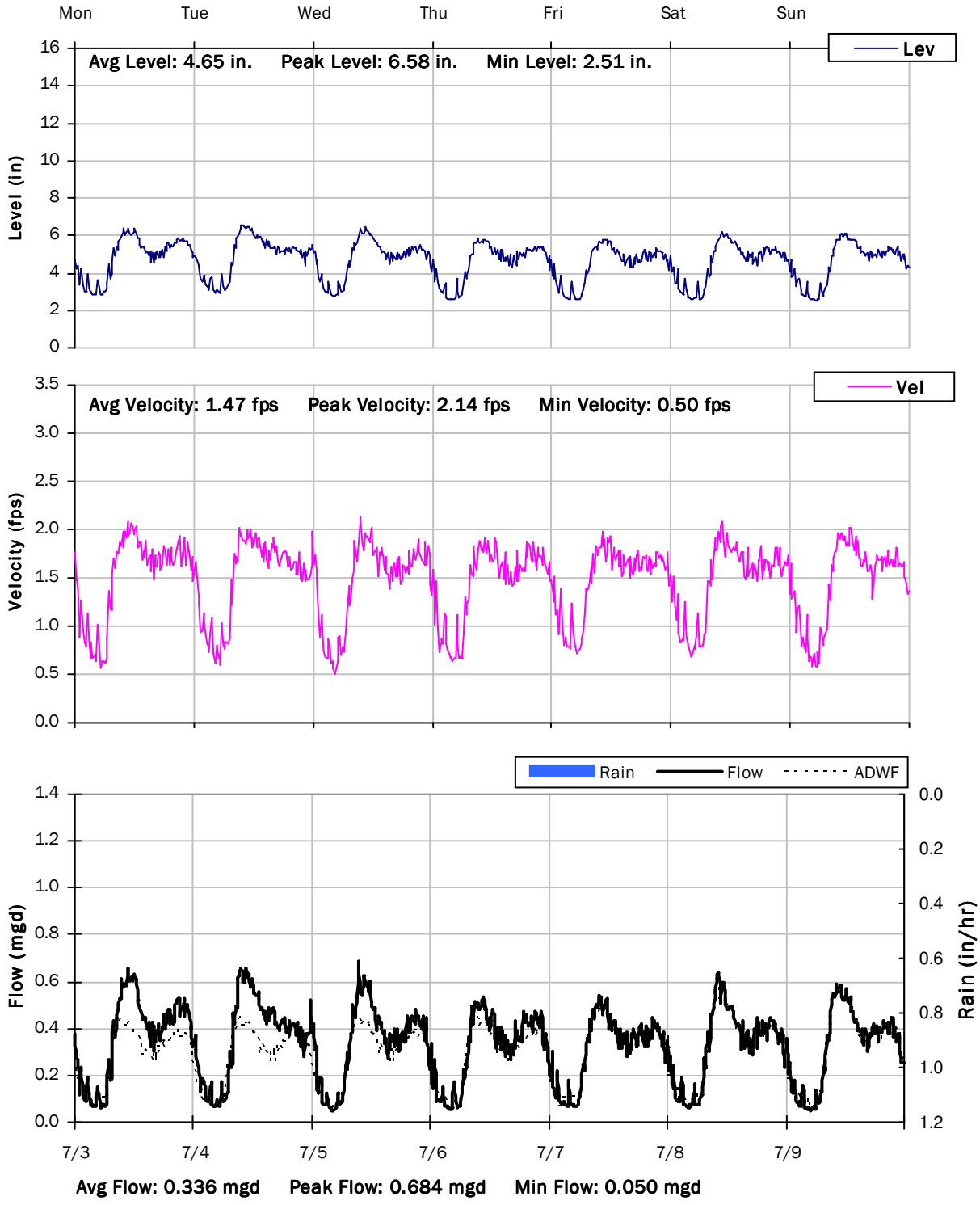
6/26/2017 to 7/3/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

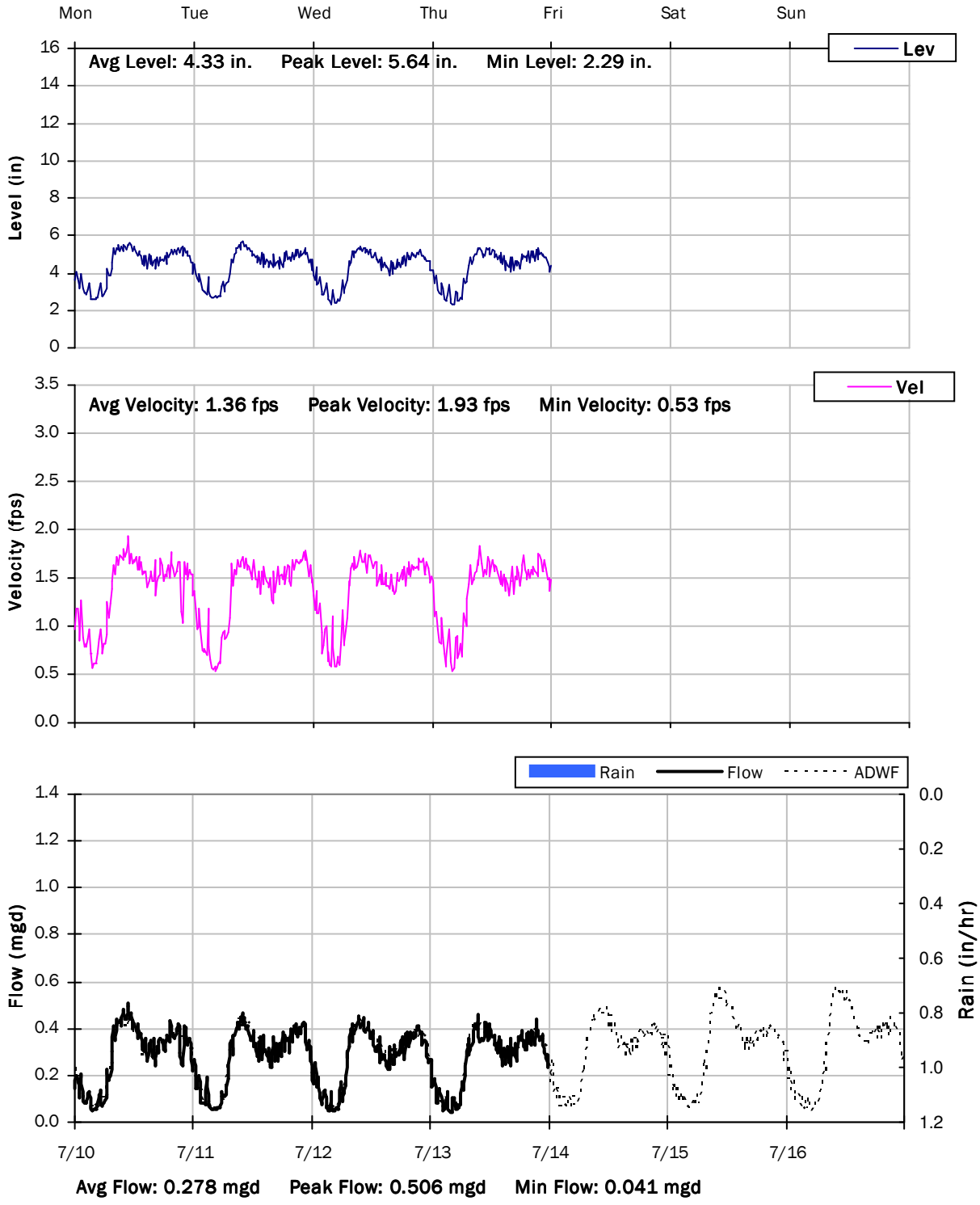
7/3/2017 to 7/10/2017



SITE 03

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

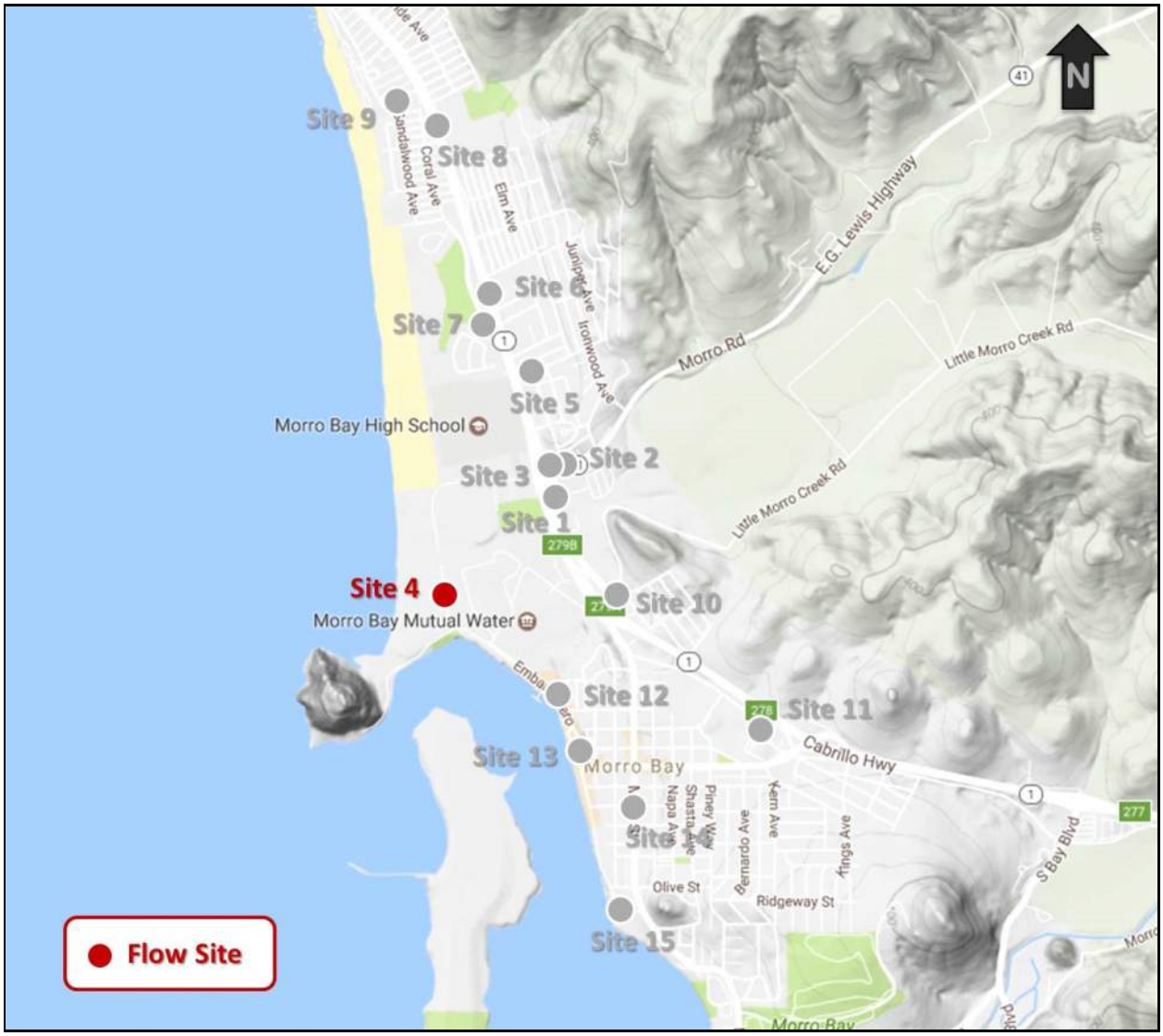
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 04

Location: Embarcadero, ~685 feet north of Coleman Drive, meter on south 17.5" inlet

Data Summary Report



Vicinity Map: Site 04

SITE 04

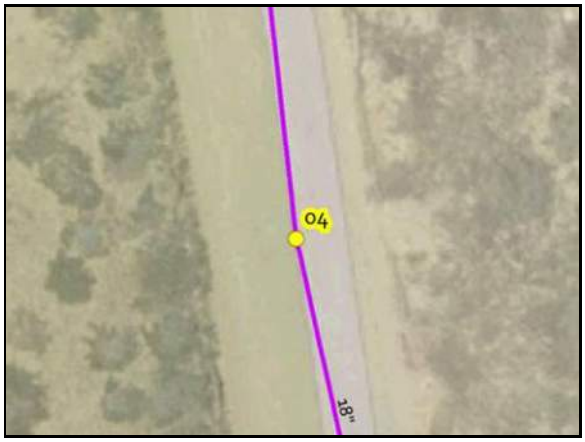
Site Information

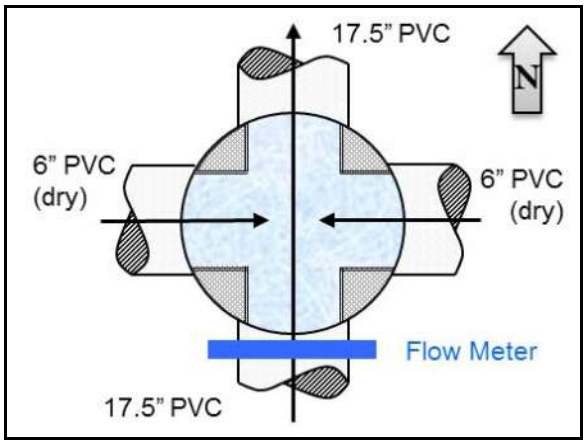
Location: Embarcadero, ~685 feet north of Coleman Drive, meter on south 17.5" inlet

Pipe Diameter: 17.5 inches

ADWF: 0.224 mgd

Peak Measured Flow: 0.578 mgd





SITE 04

Additional Site Photos

SITE 04

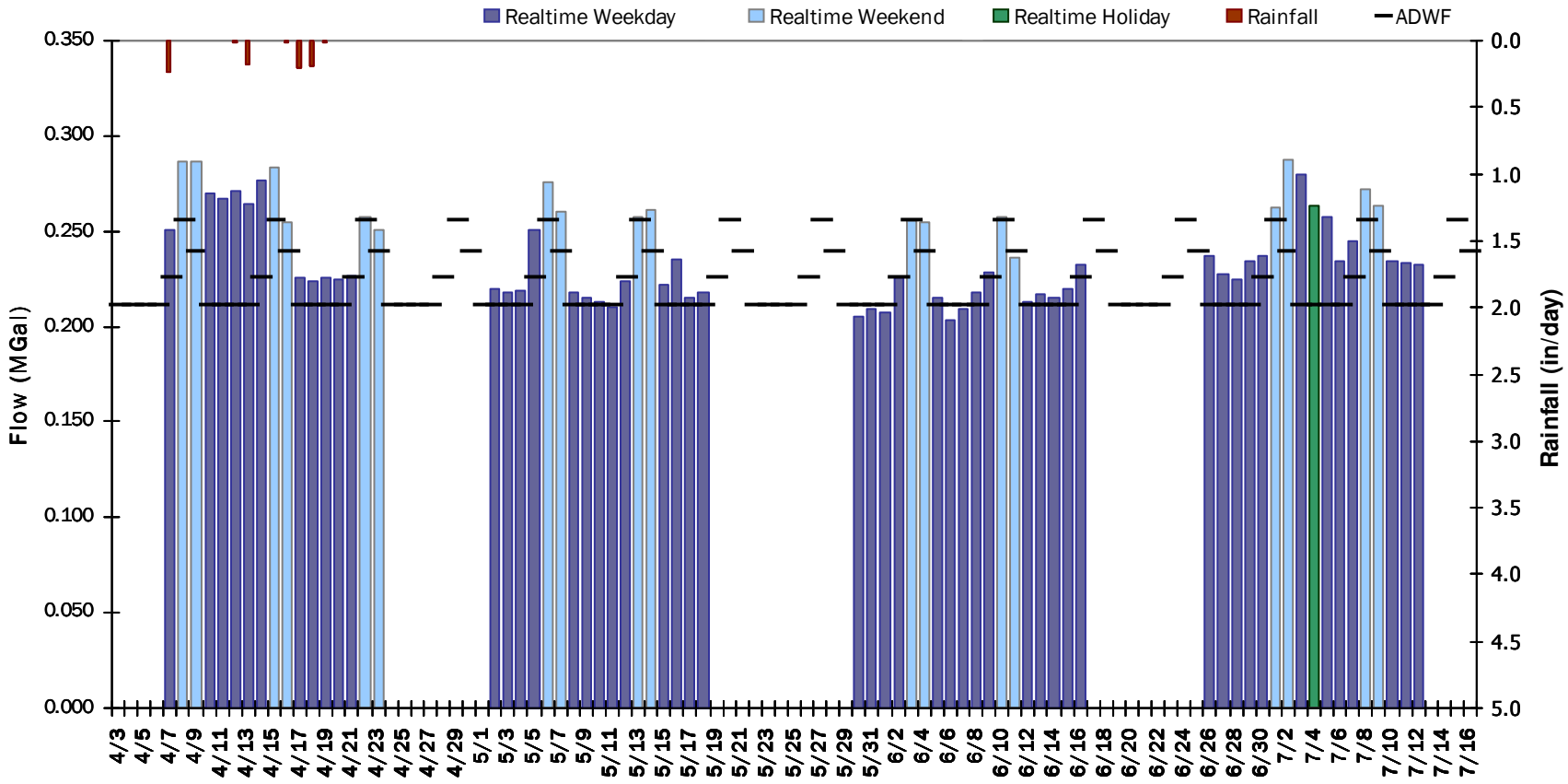
Additional Site Photos

SITE 04

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.240 MGal Peak Daily Flow: 0.288 MGal Min Daily Flow: 0.204 MGal

Total Period Rainfall: 0.85 inches



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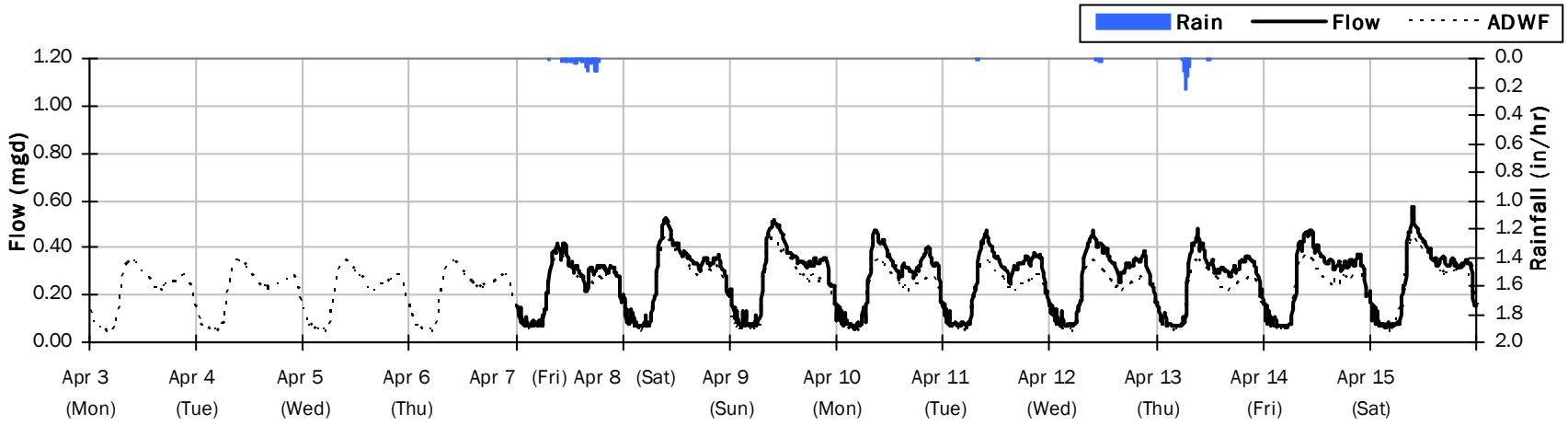
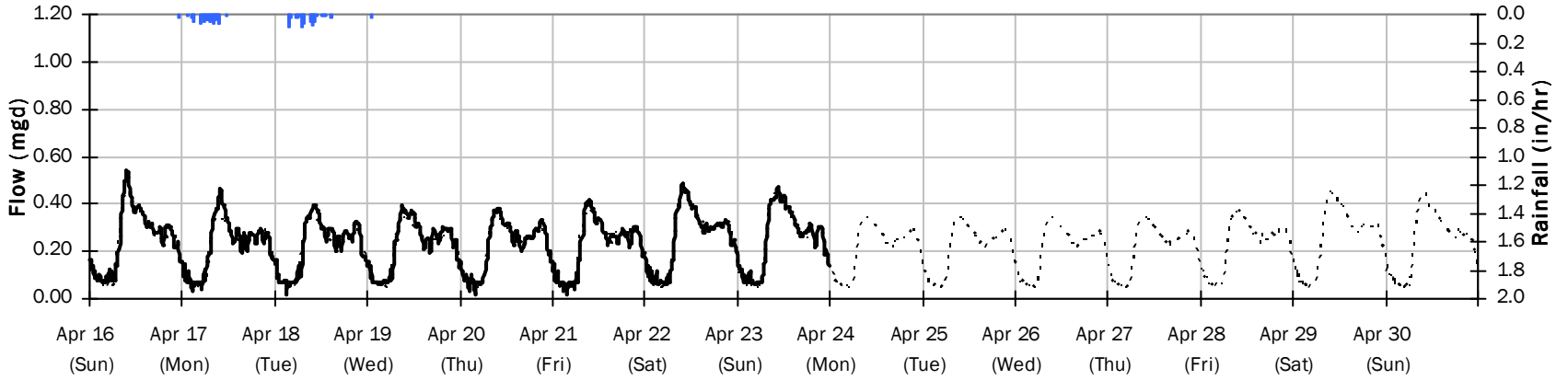
SITE 04

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.256 mgd** **Peak Flow: 0.574 mgd** **Min Flow: 0.014 mgd**



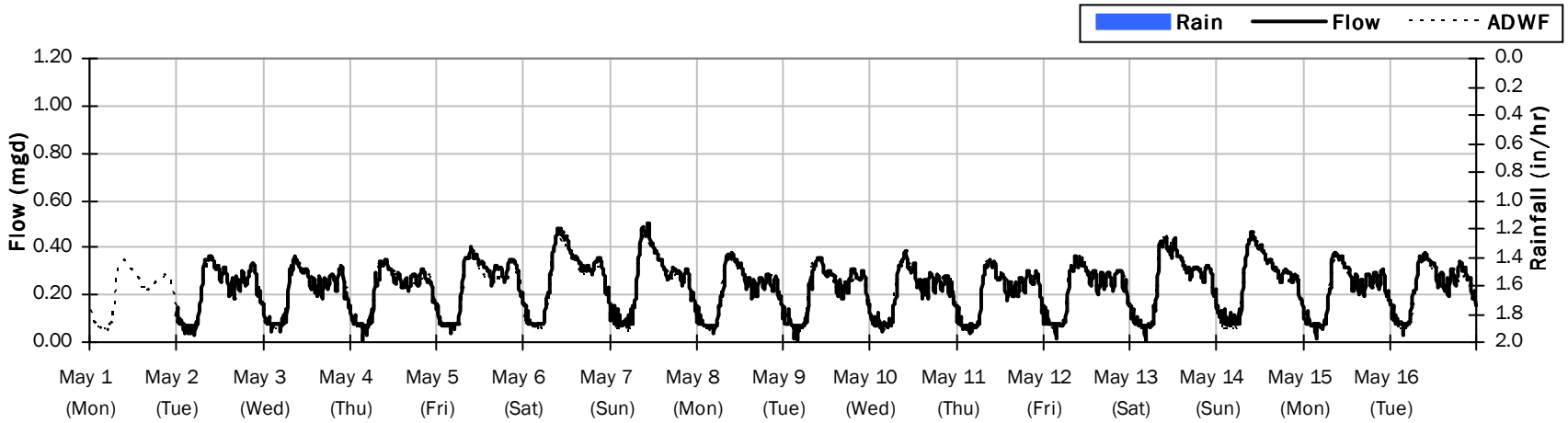


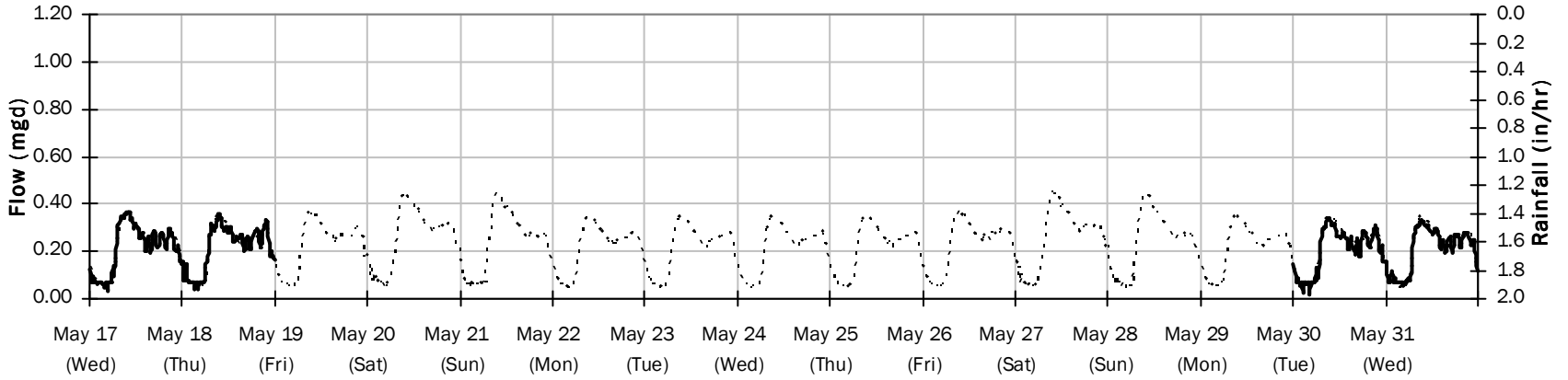
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 04

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.229 mgd Peak Flow: 0.505 mgd Min Flow: 0.004 mgd**



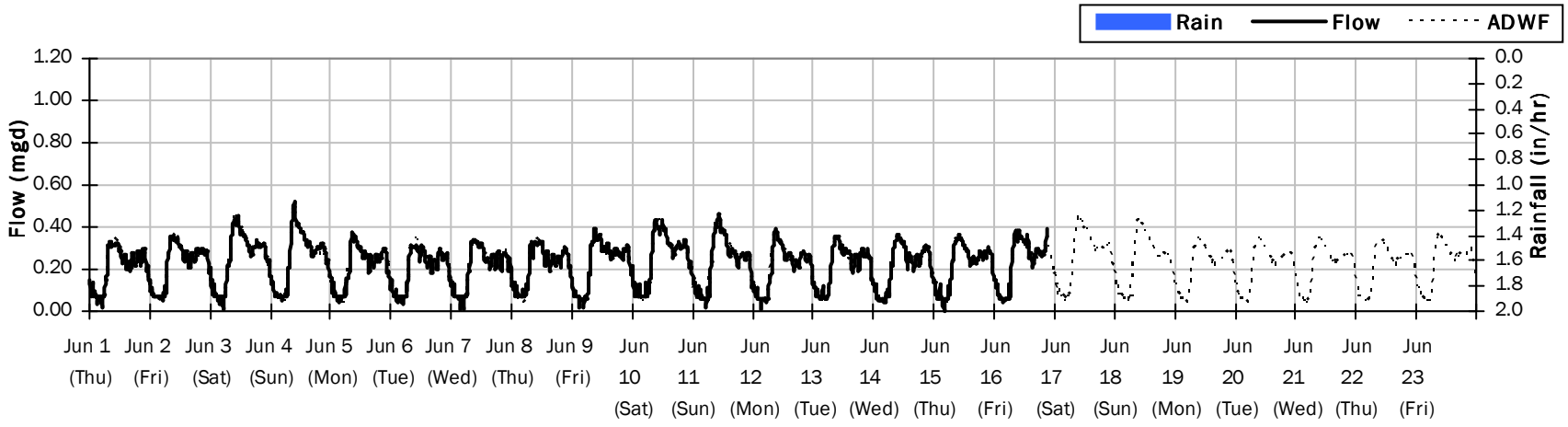


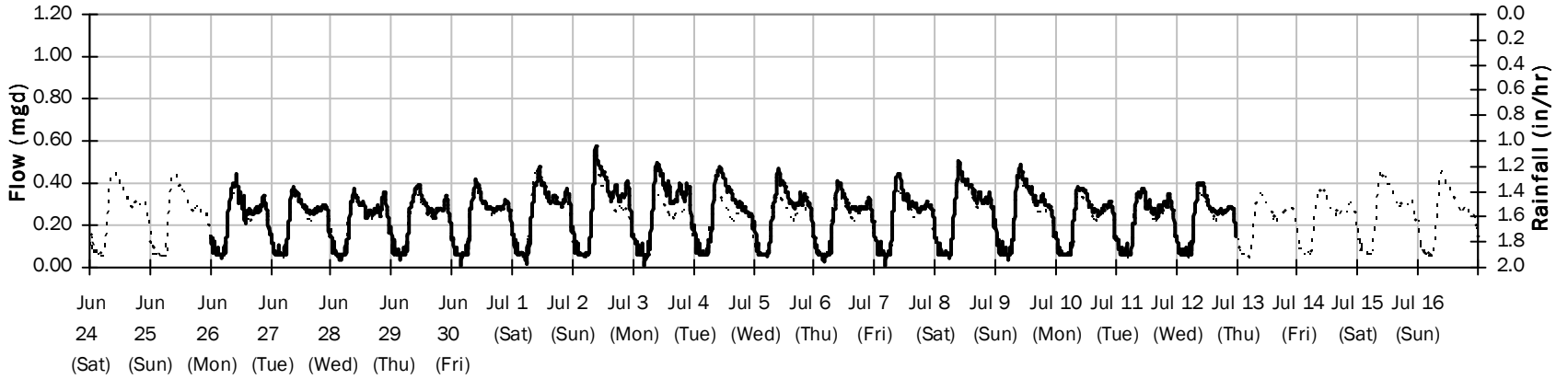
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 04

Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.237 mgd Peak Flow: 0.578 mgd Min Flow: 0.004 mgd**





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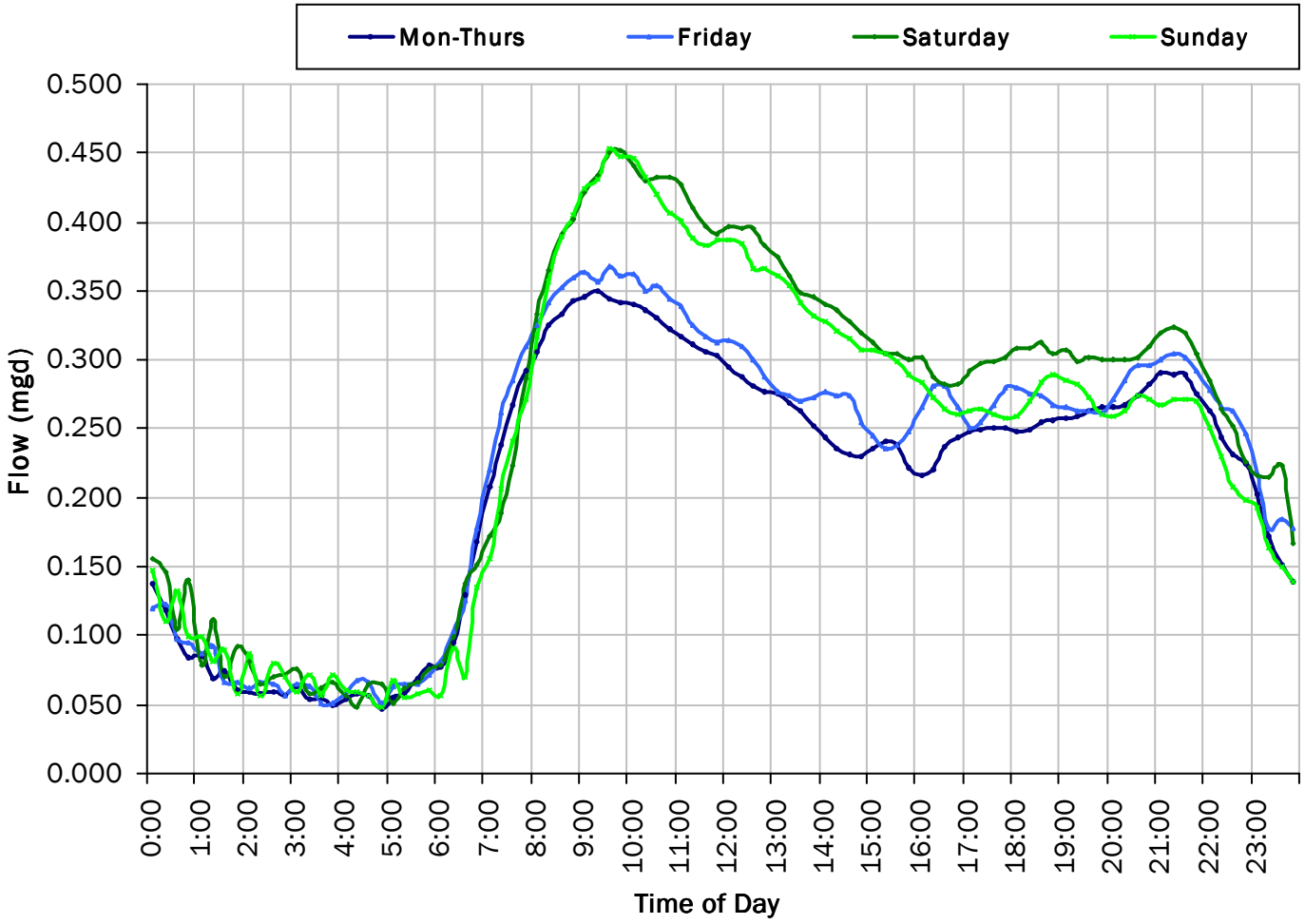
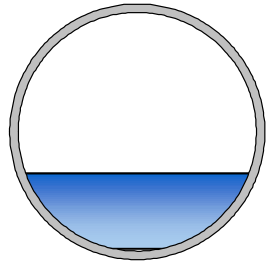
SITE 04

Average Dry Weather Flow Hydrographs



ADWF:

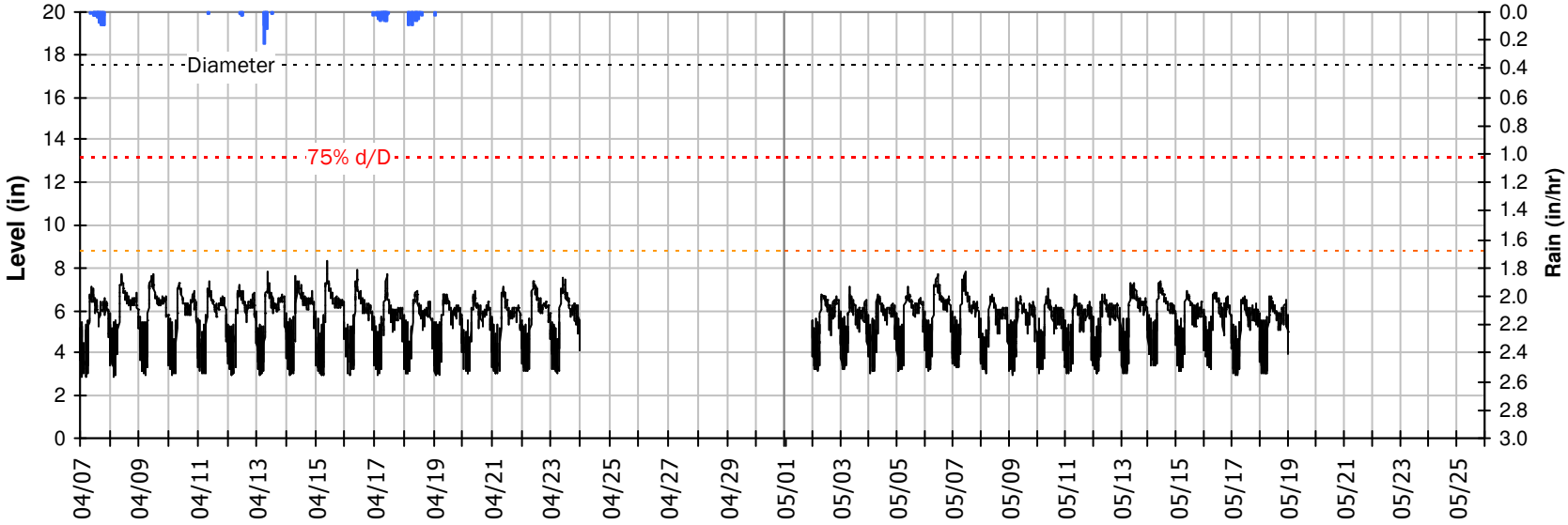
0.224 *mgd*

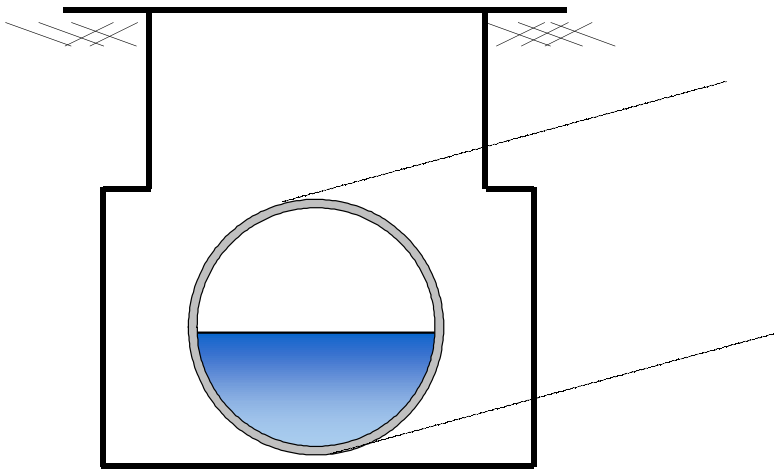


SITE 04

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





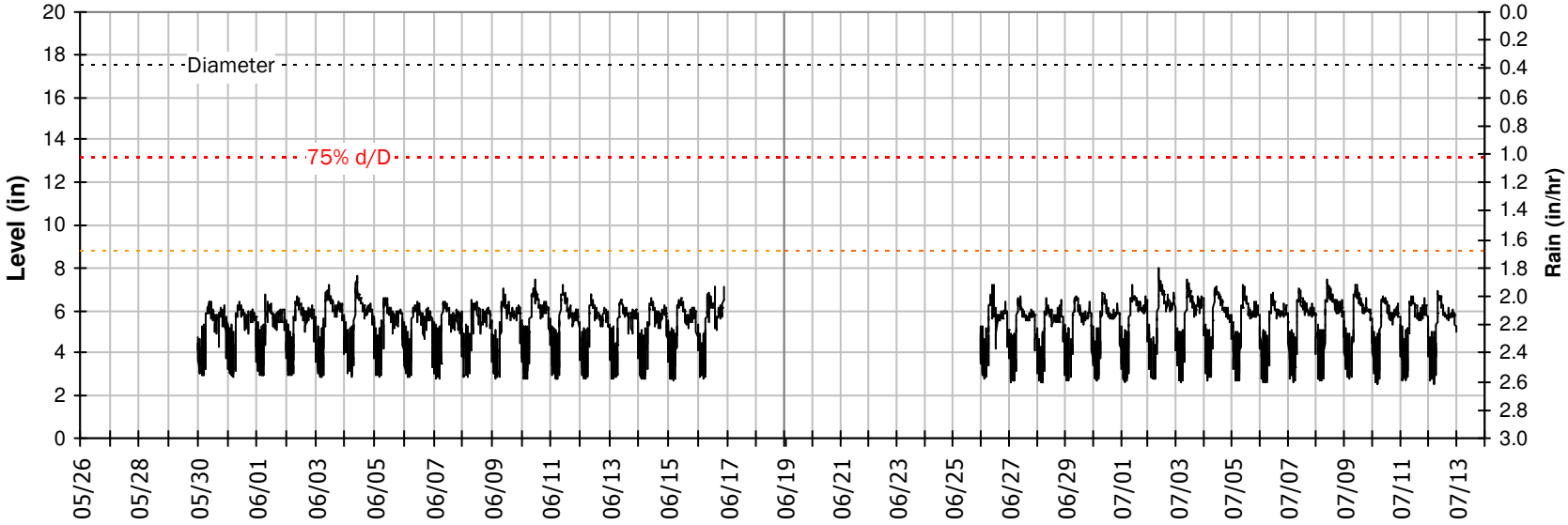
Pipe Diameter:	17.5	*inches*
Peak Measured Level:	8.36	*inches*
Peak d/D Ratio:	0.48	

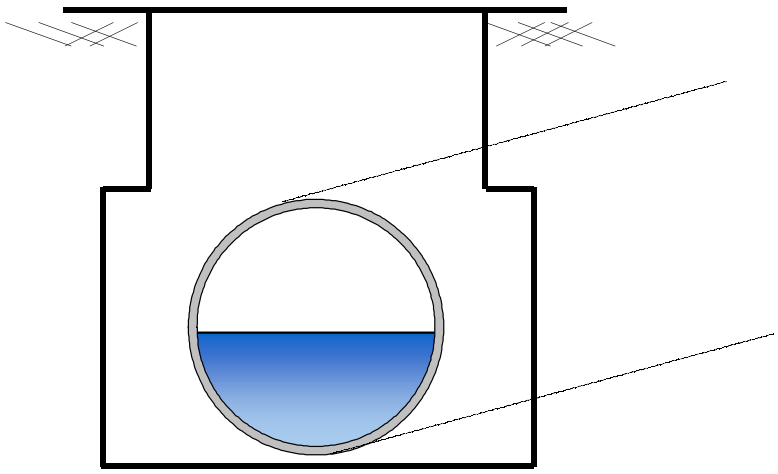
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 04

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





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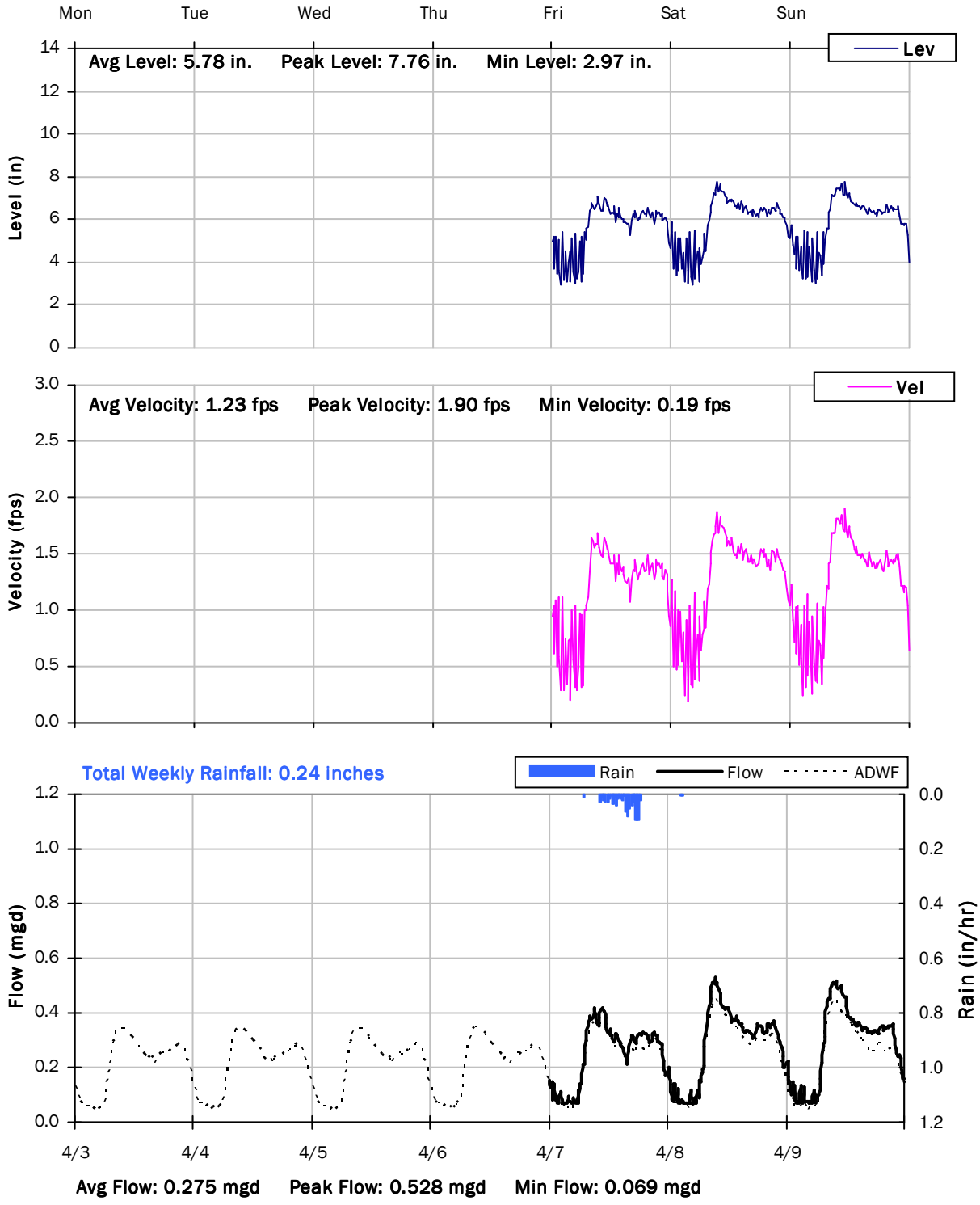
SITE 04

Weekly Level, Velocity and Flow Hydrographs

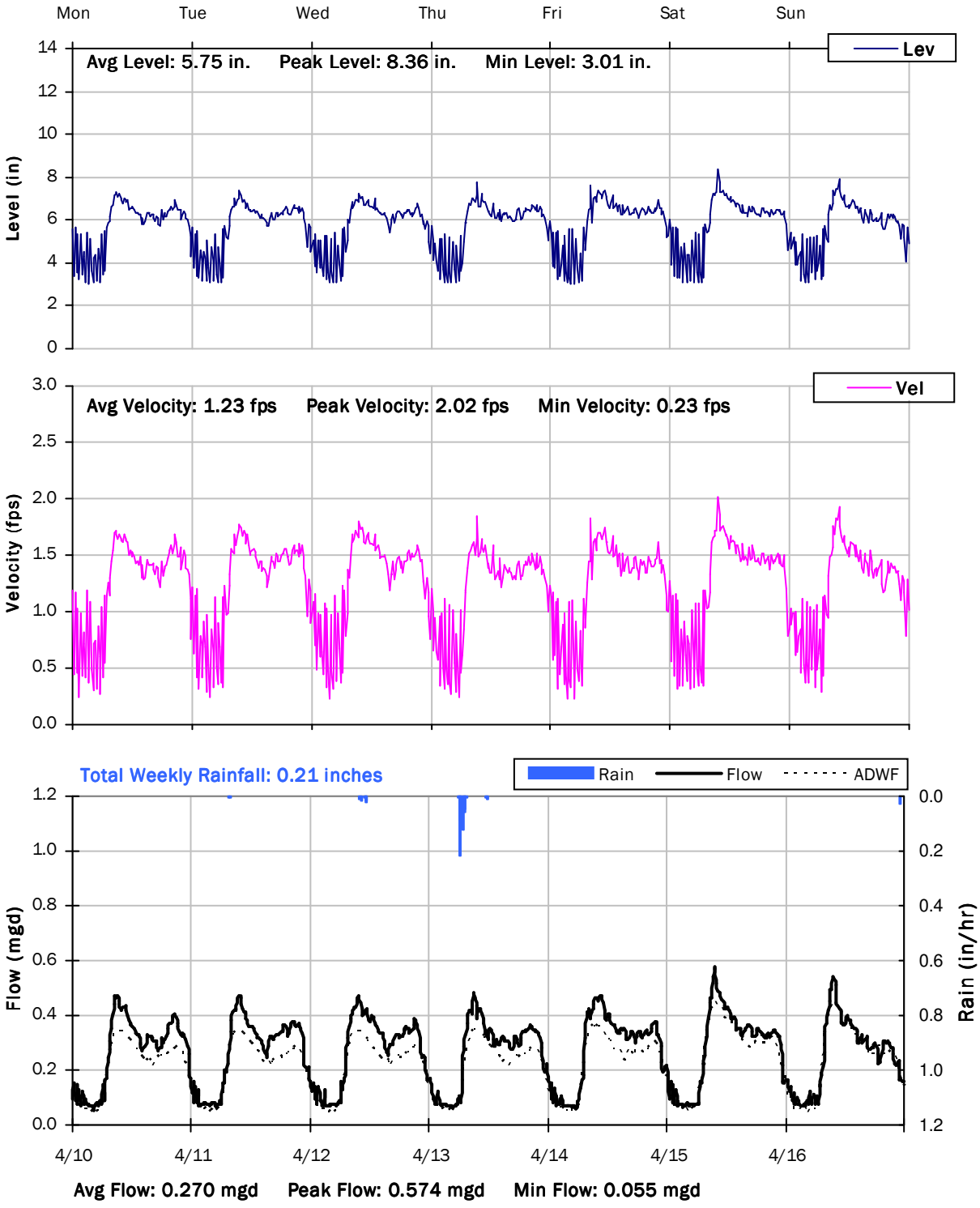
4/3/2017 to 4/10/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

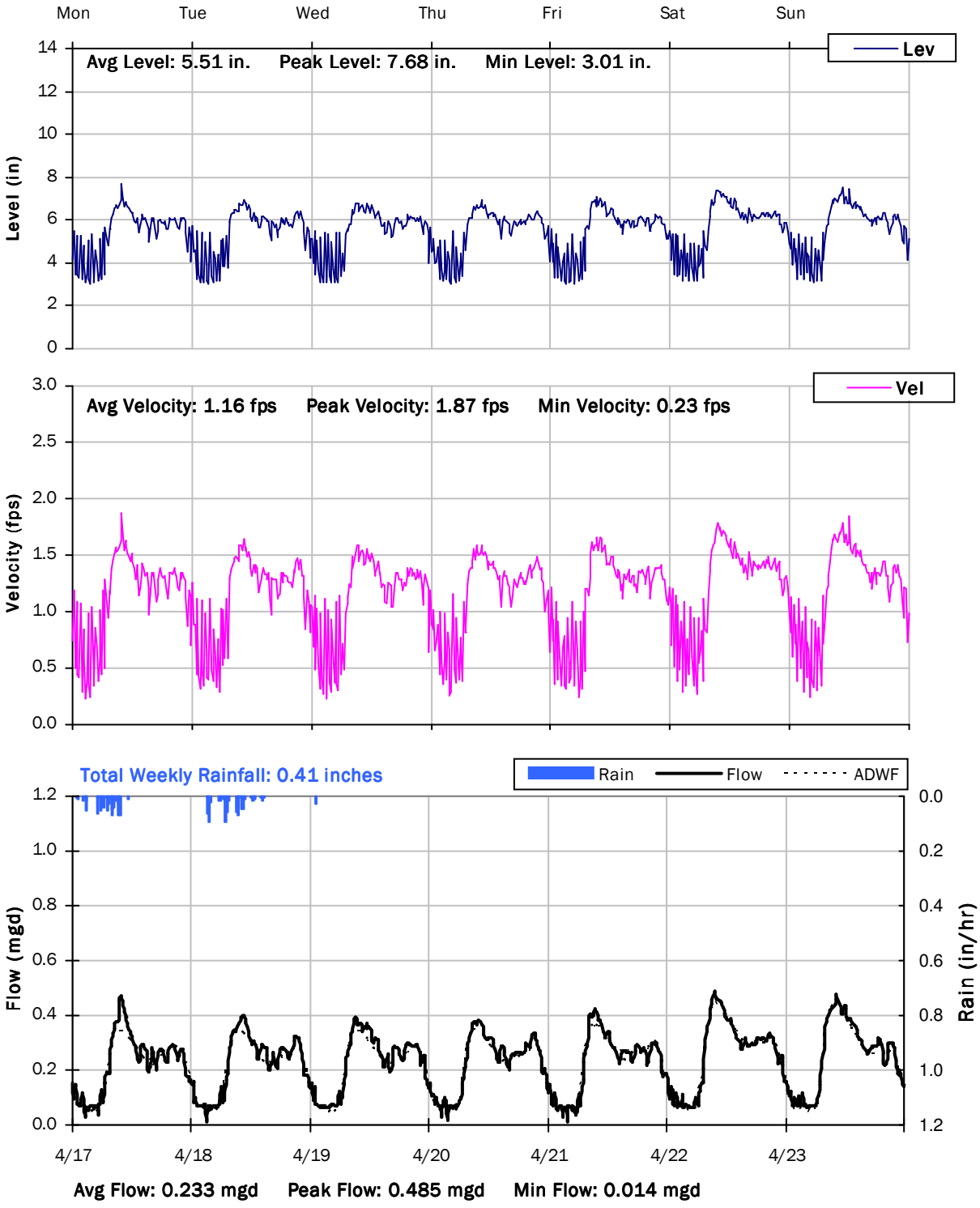
4/10/2017 to 4/17/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

4/17/2017 to 4/24/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

4/24/2017 to 5/1/2017

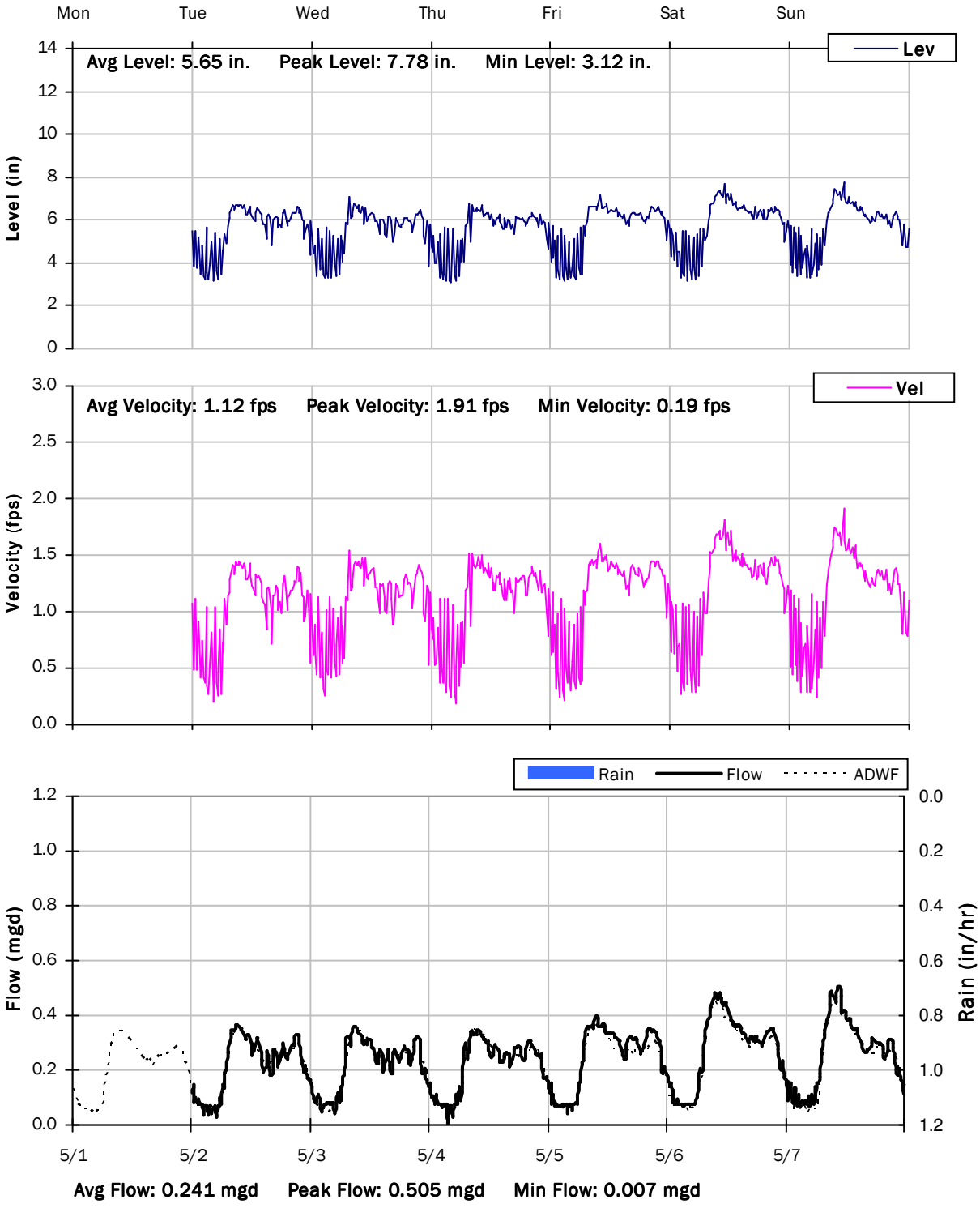
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 04

Weekly Level, Velocity and Flow Hydrographs

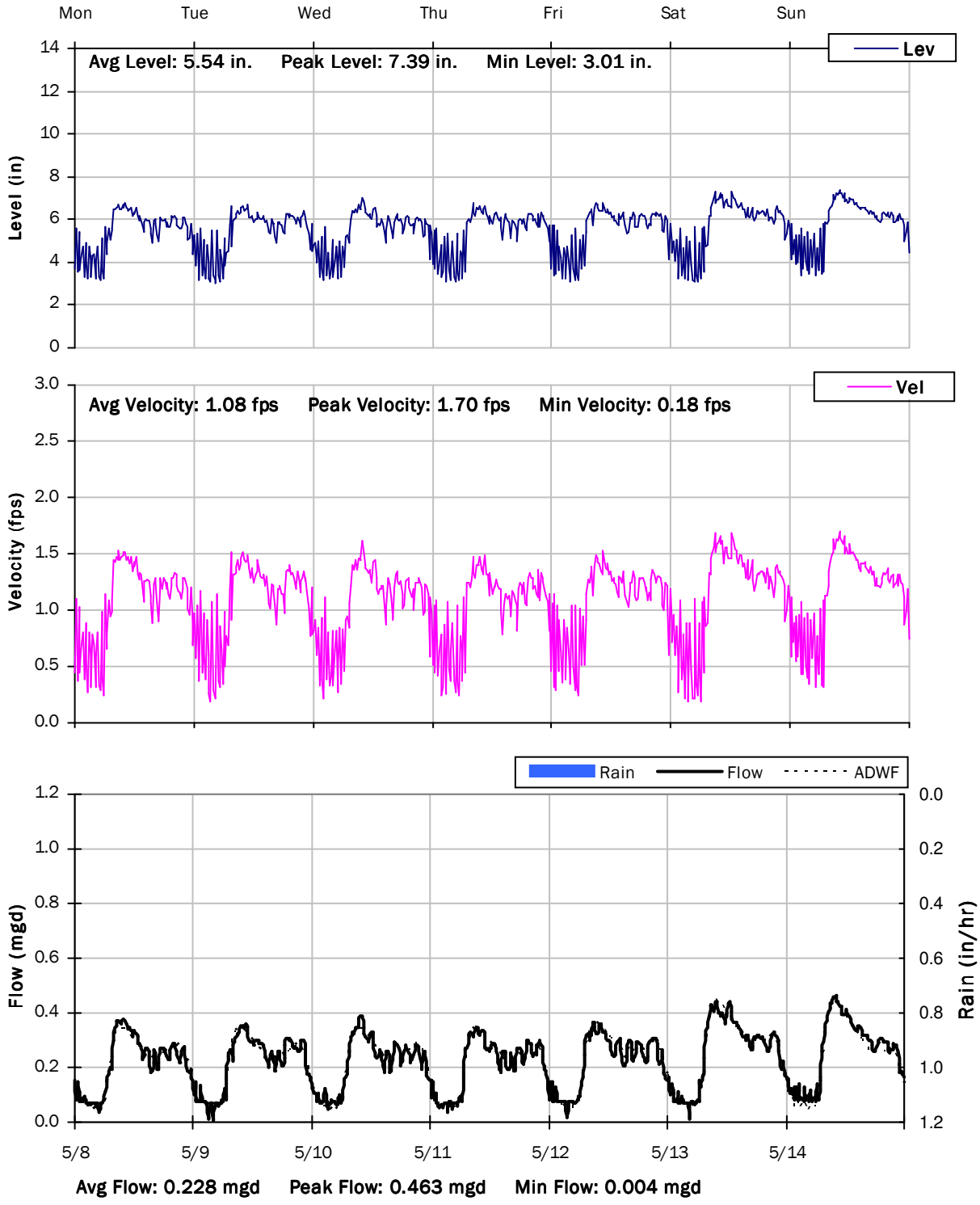
5/1/2017 to 5/8/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

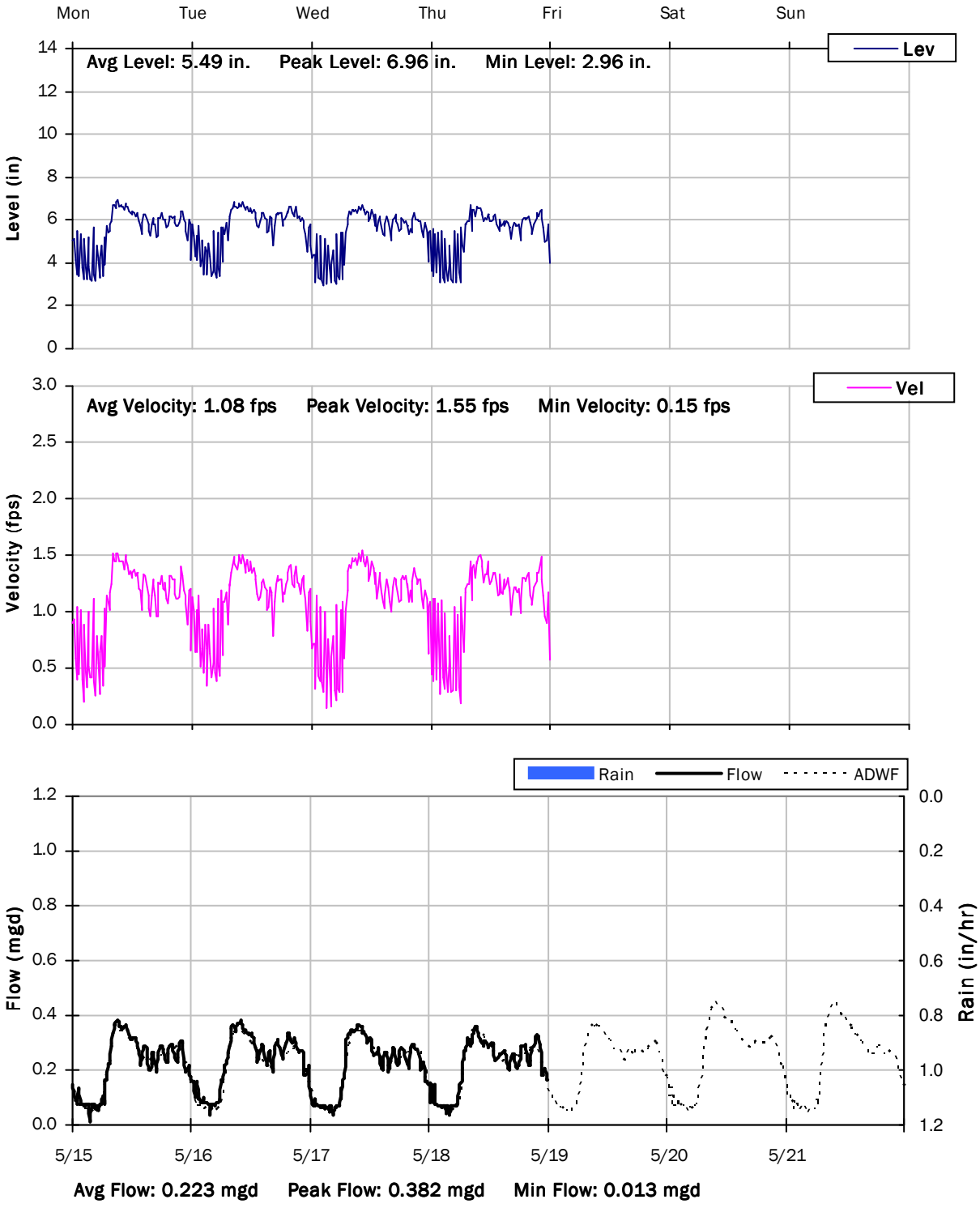
5/8/2017 to 5/15/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

5/15/2017 to 5/22/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

5/22/2017 to 5/29/2017

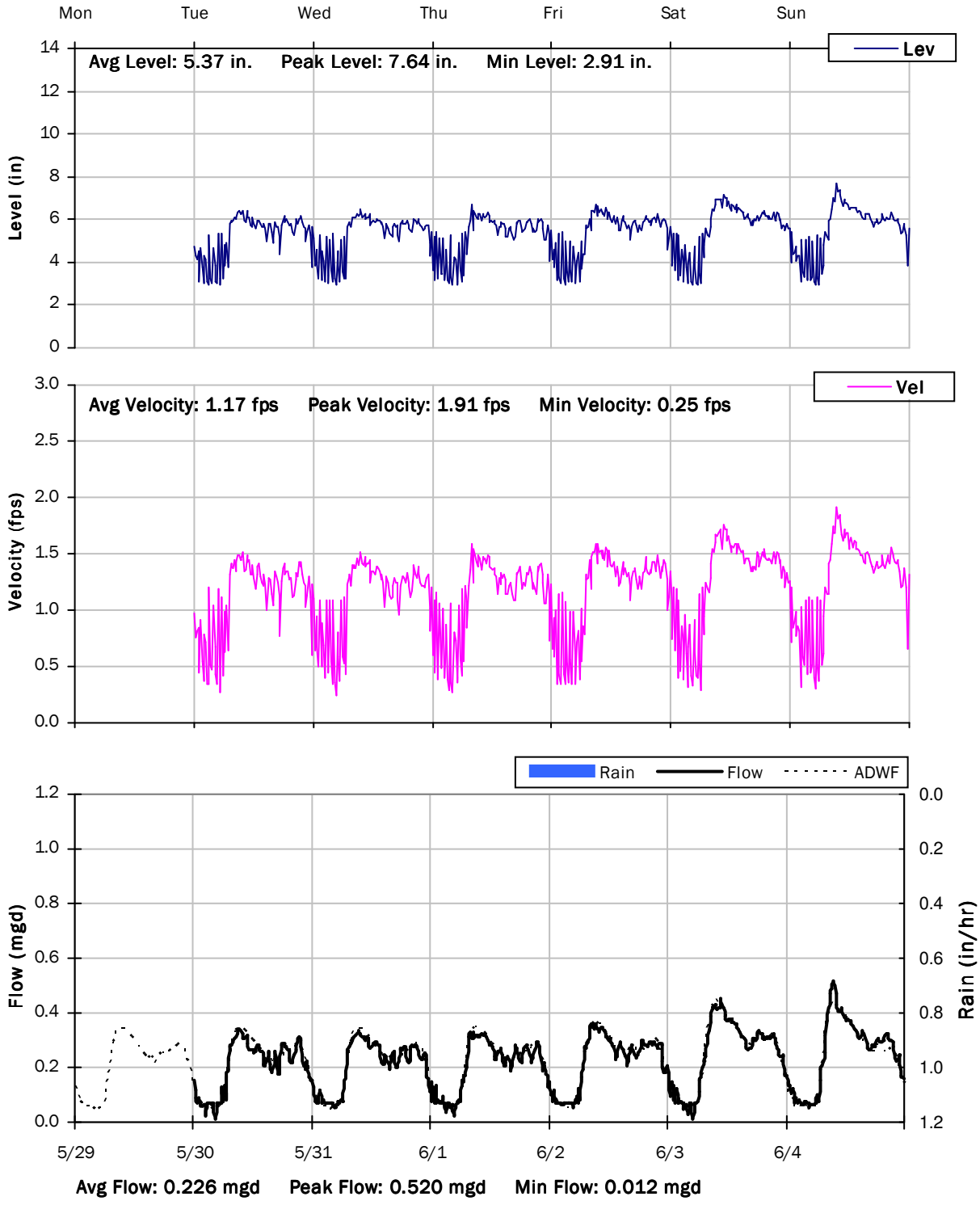
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 04

Weekly Level, Velocity and Flow Hydrographs

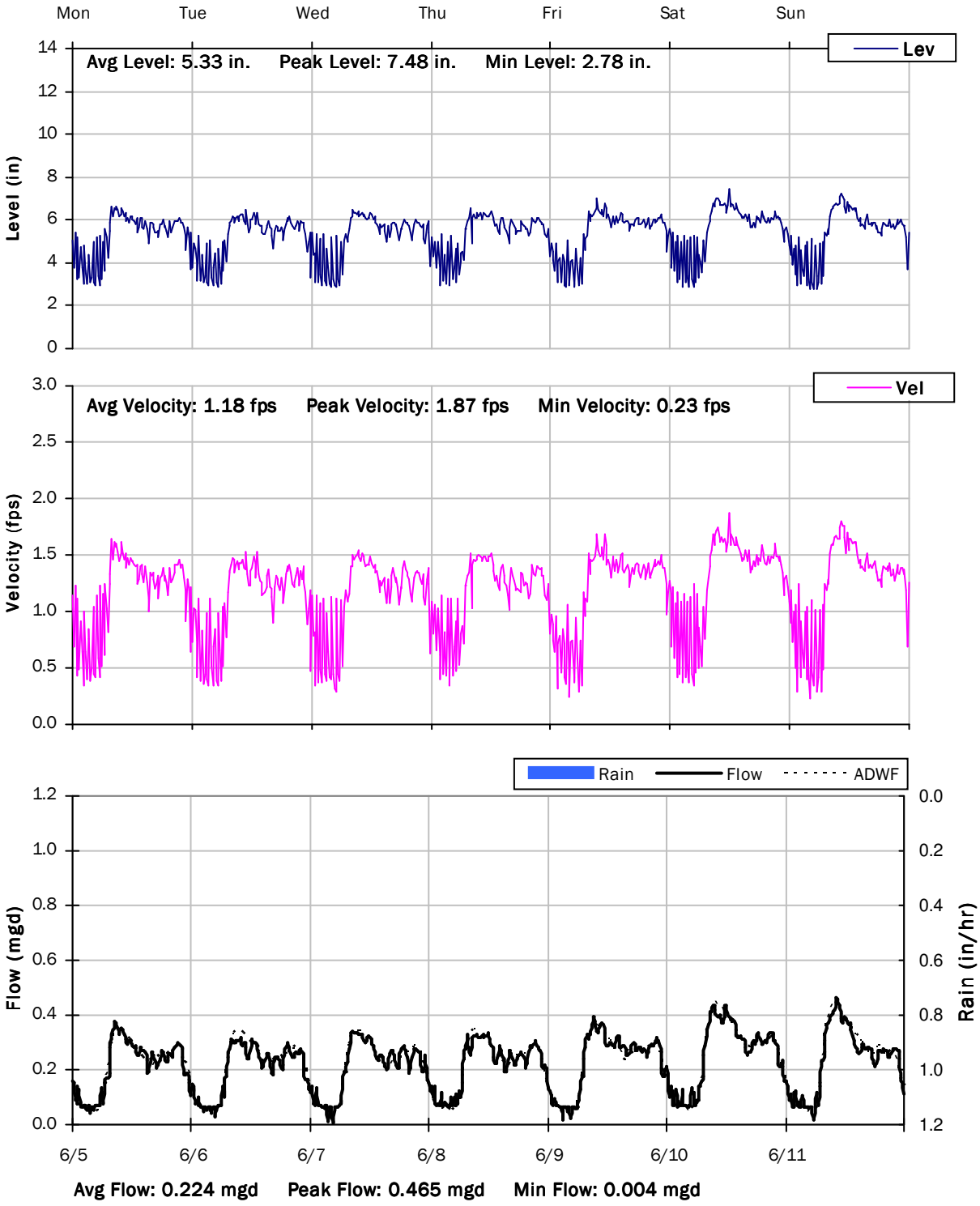
5/29/2017 to 6/5/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

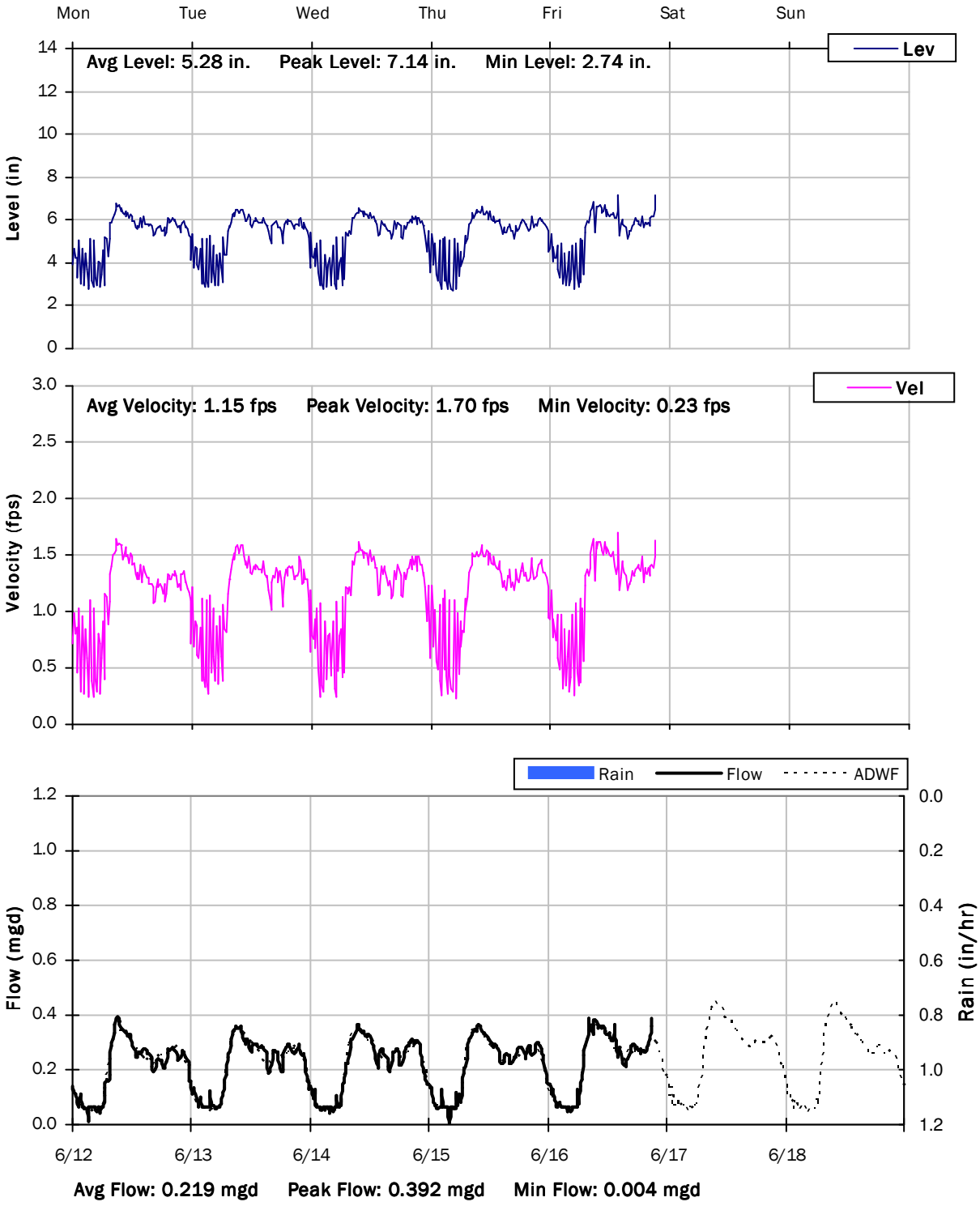
6/5/2017 to 6/12/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

6/12/2017 to 6/19/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

6/19/2017 to 6/26/2017

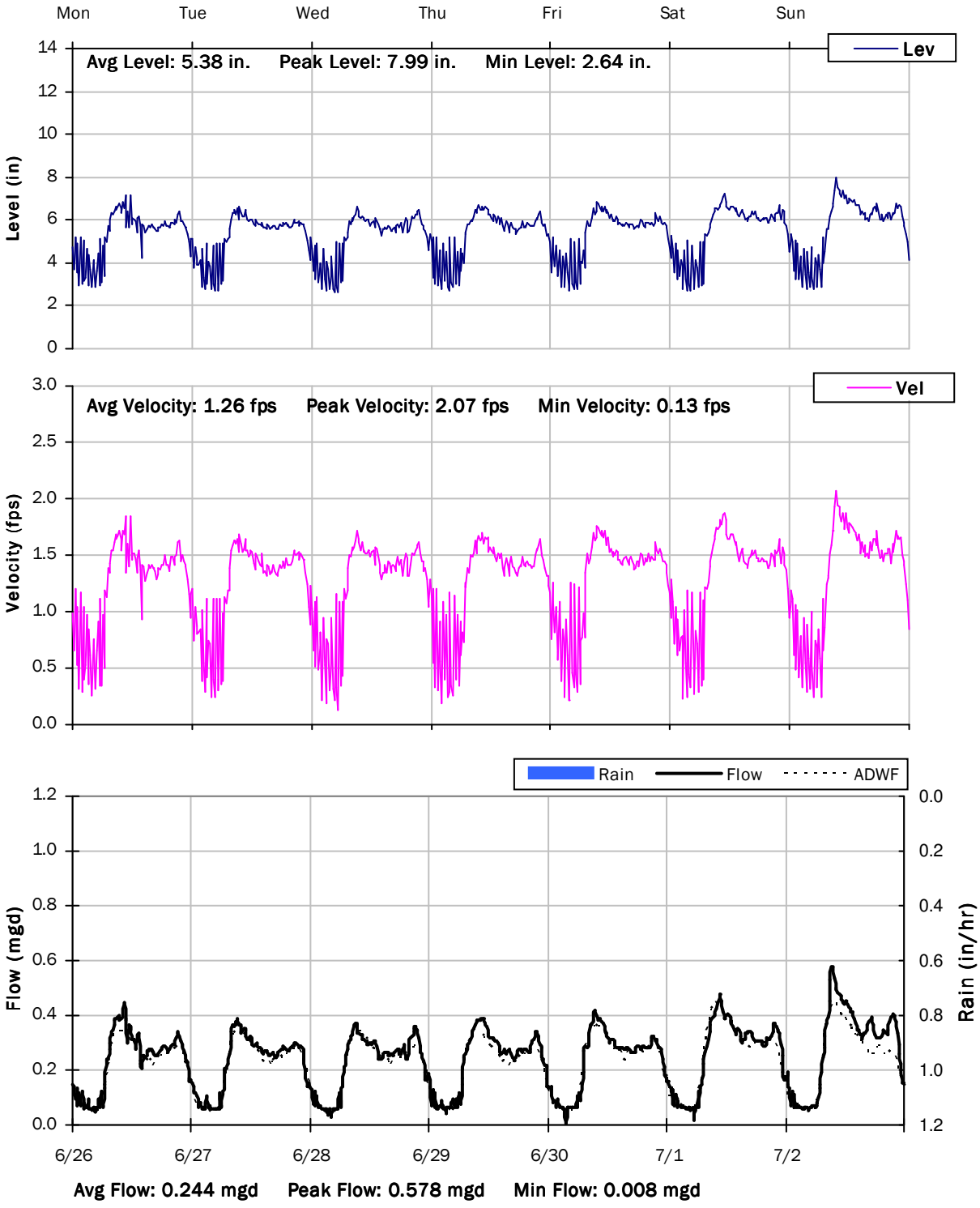
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 04

Weekly Level, Velocity and Flow Hydrographs

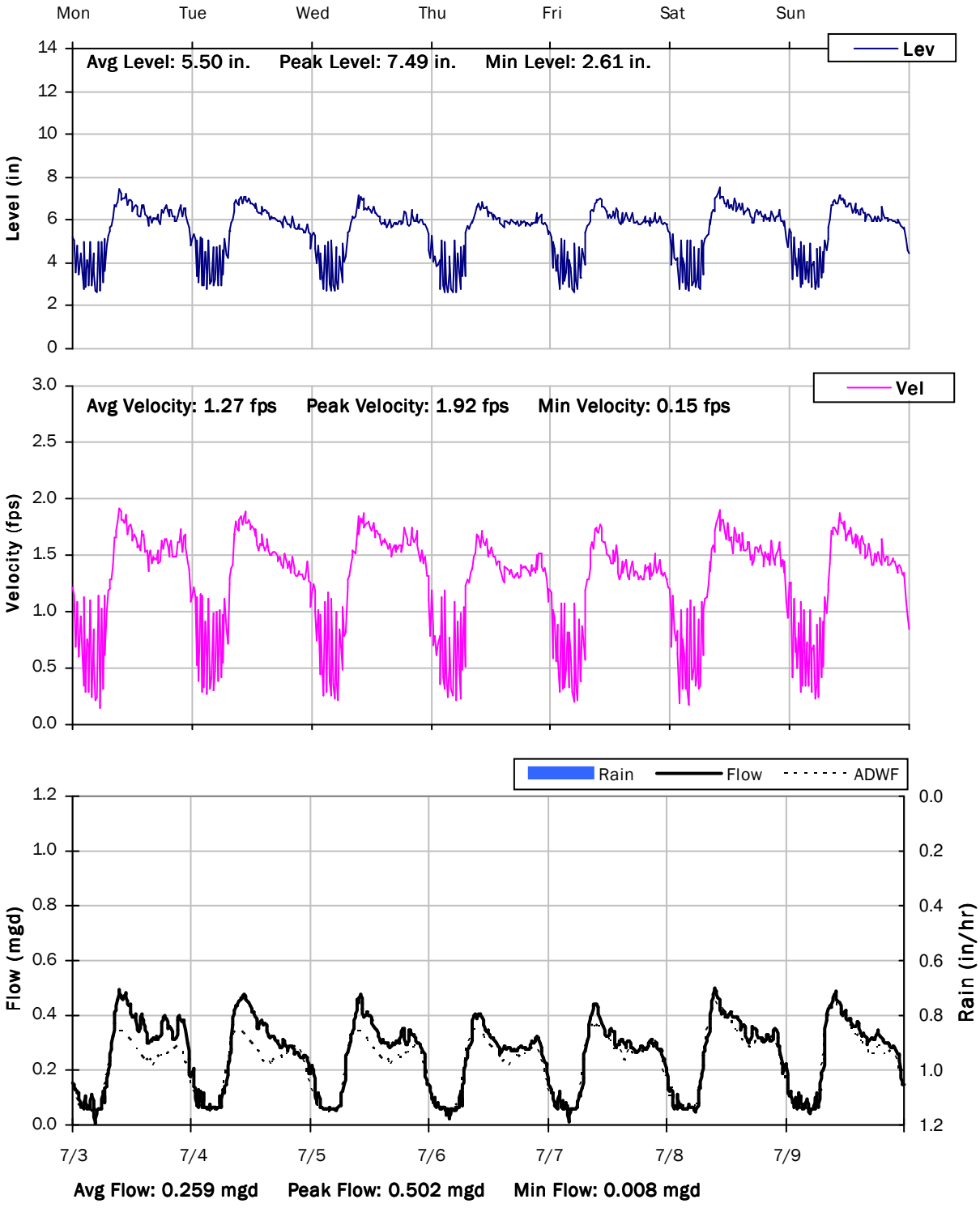
6/26/2017 to 7/3/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

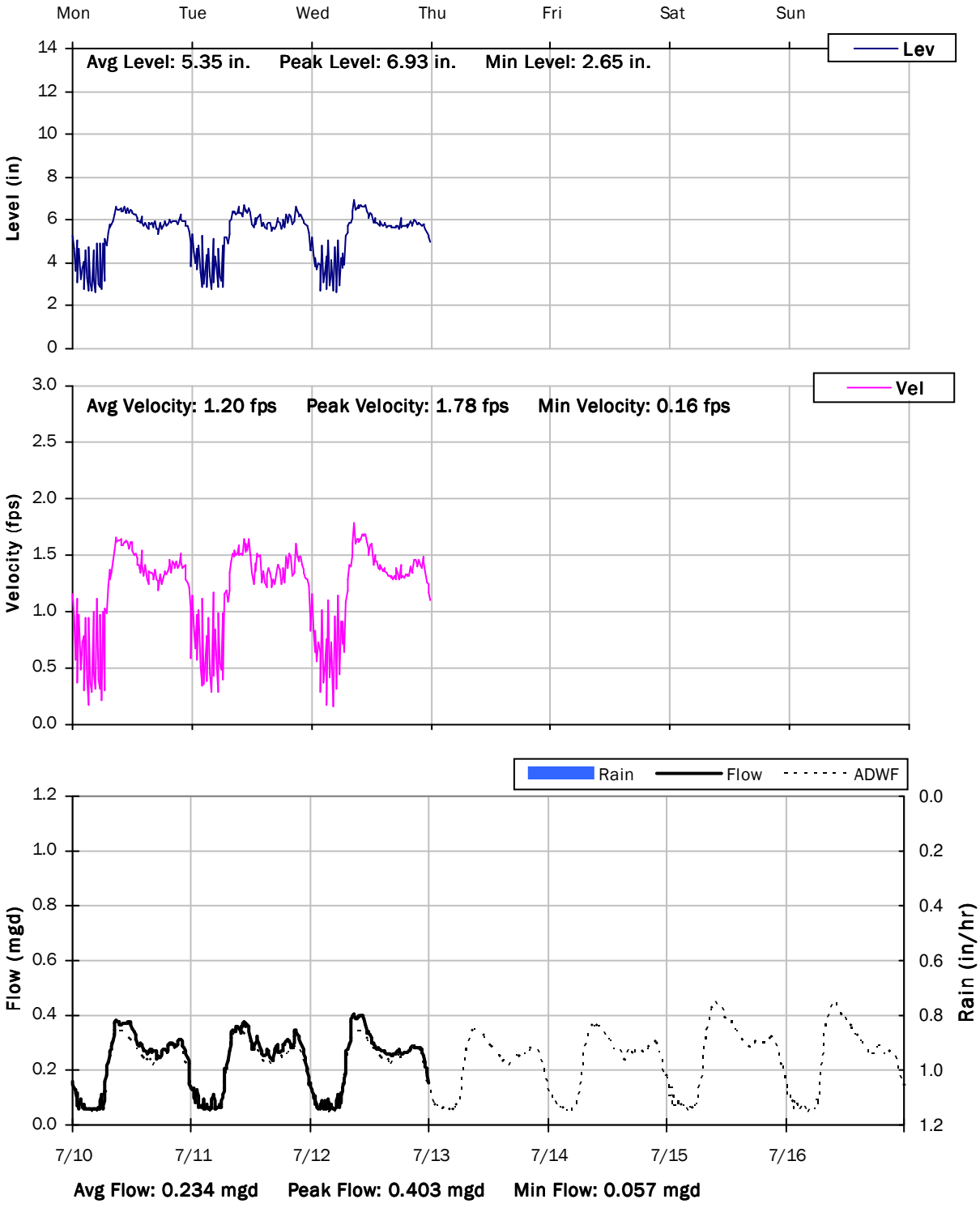
7/3/2017 to 7/10/2017



SITE 04

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

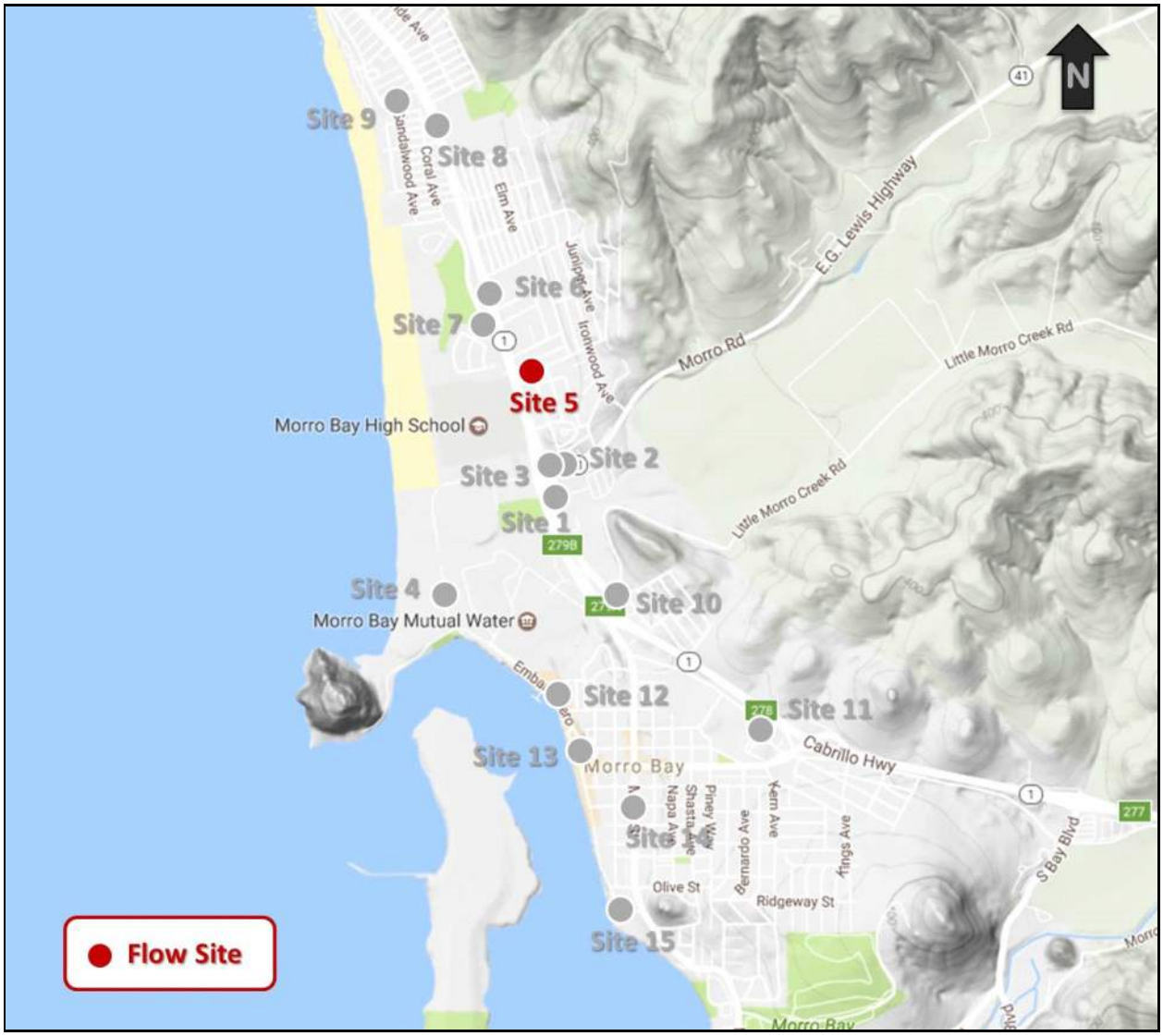
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 05

Location: 371 Avalon Street, meter on east 8" inlet

Data Summary Report



Vicinity Map: Site 05

SITE 05

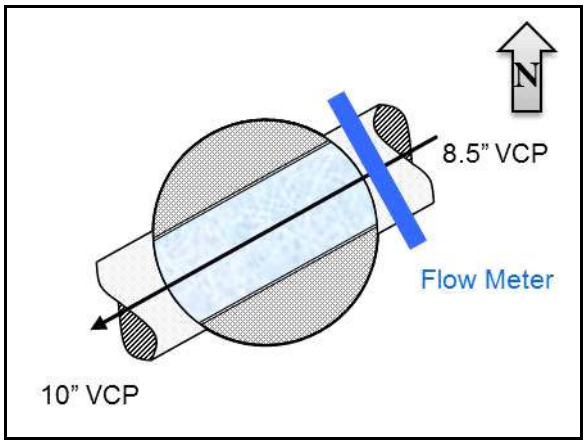
Site Information

Location: 371 Avalon Street, meter on east 8" inlet

Pipe Diameter: 8 inches

ADWF: 0.091 mgd

Peak Measured Flow: 0.285 mgd



SITE 05

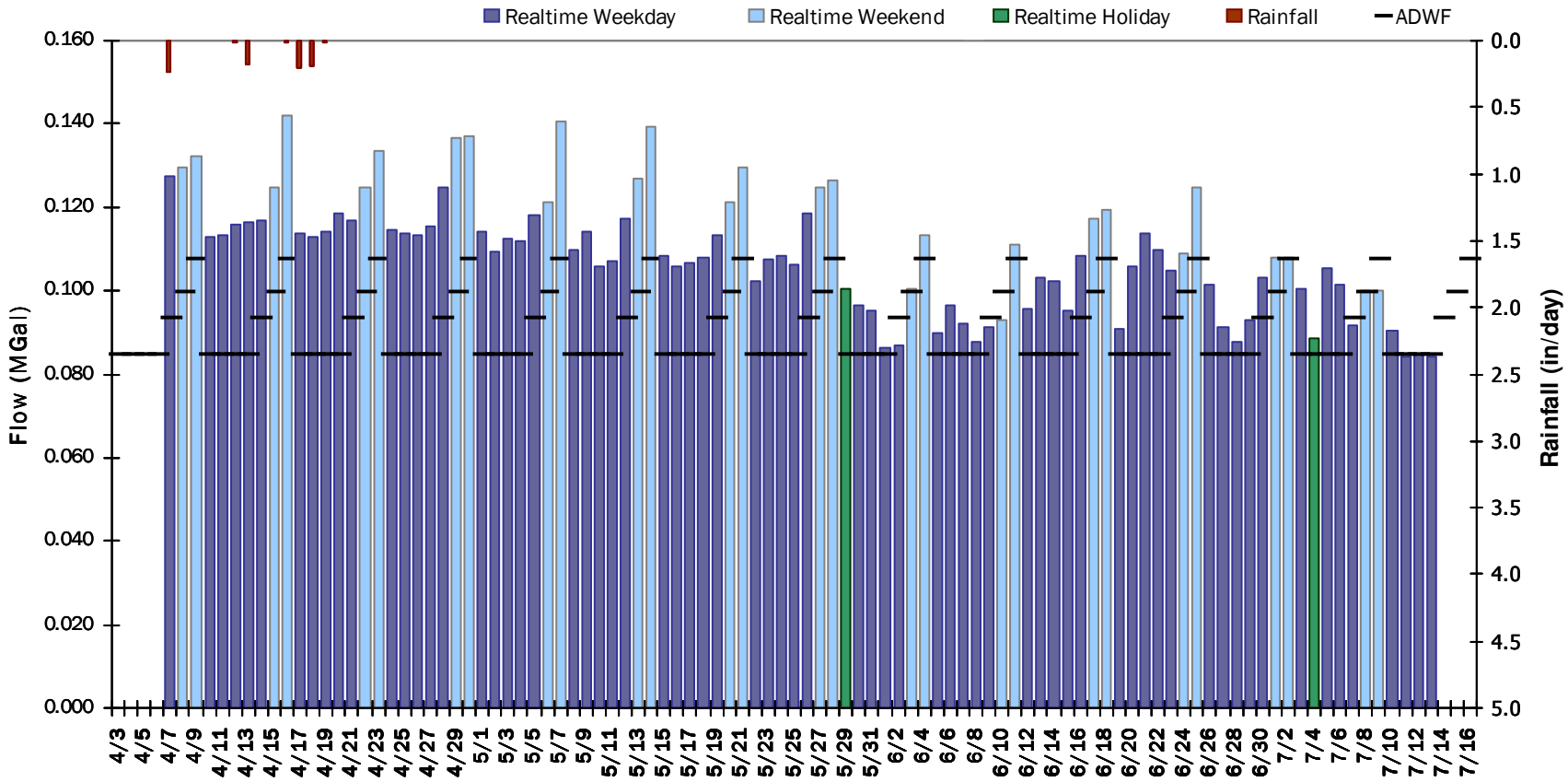
Additional Site Photos

SITE 05

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.109 MGal **Peak Daily Flow: 0.142 MGal** **Min Daily Flow: 0.084 MGal**

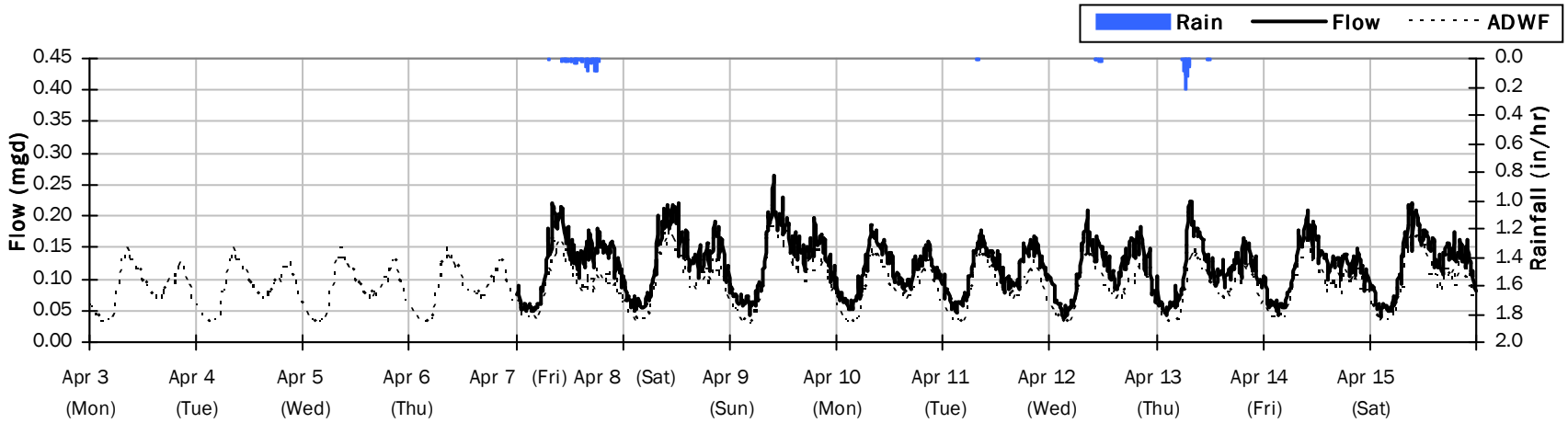
Total Period Rainfall: 0.85 inches

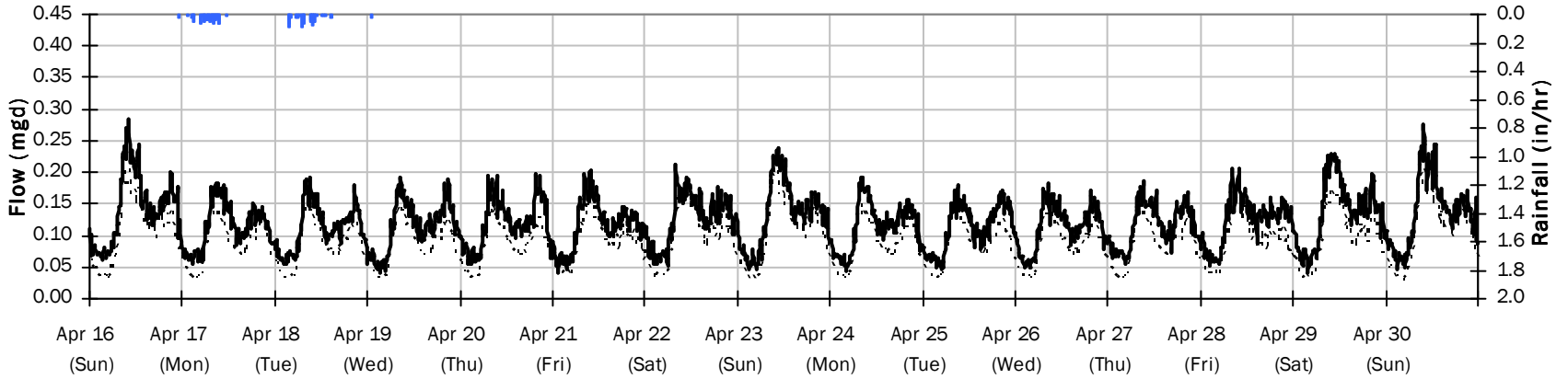


SITE 05

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.122 mgd** **Peak Flow: 0.285 mgd** **Min Flow: 0.038 mgd**

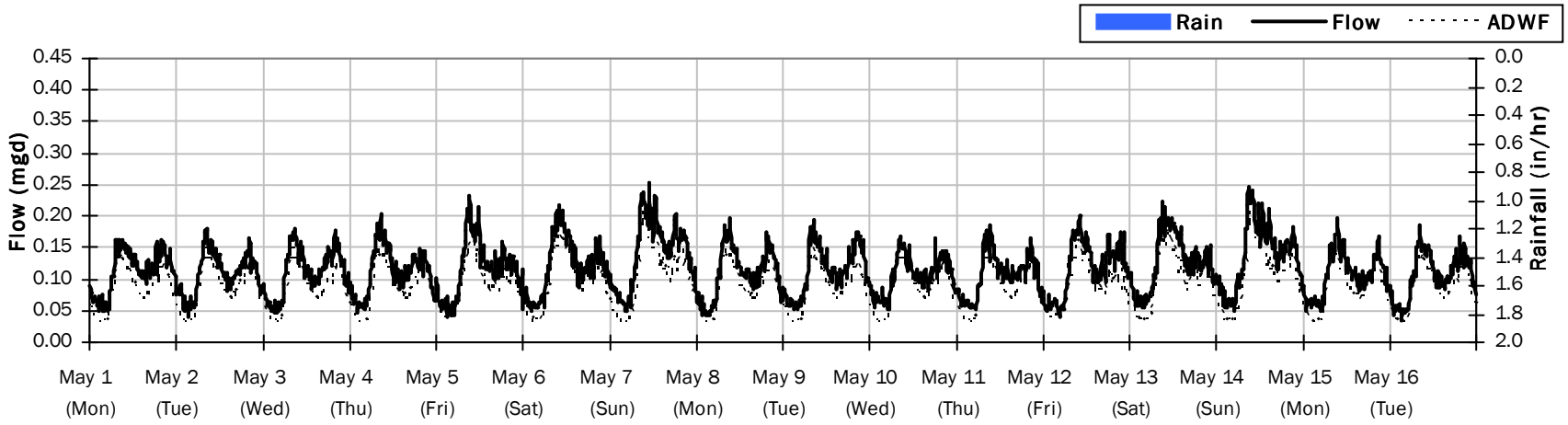


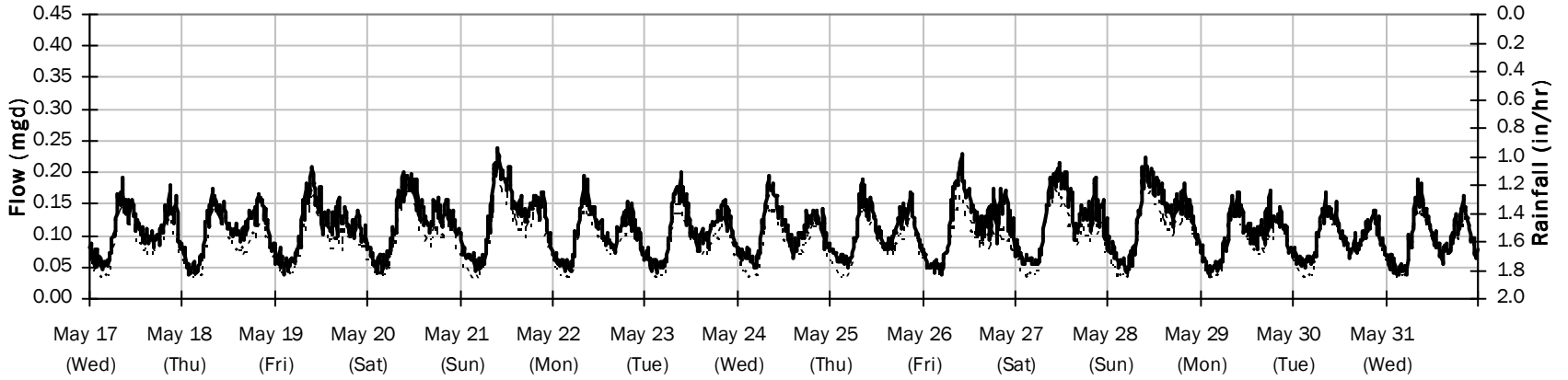


SITE 05

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.114 mgd** **Peak Flow: 0.252 mgd** **Min Flow: 0.035 mgd**

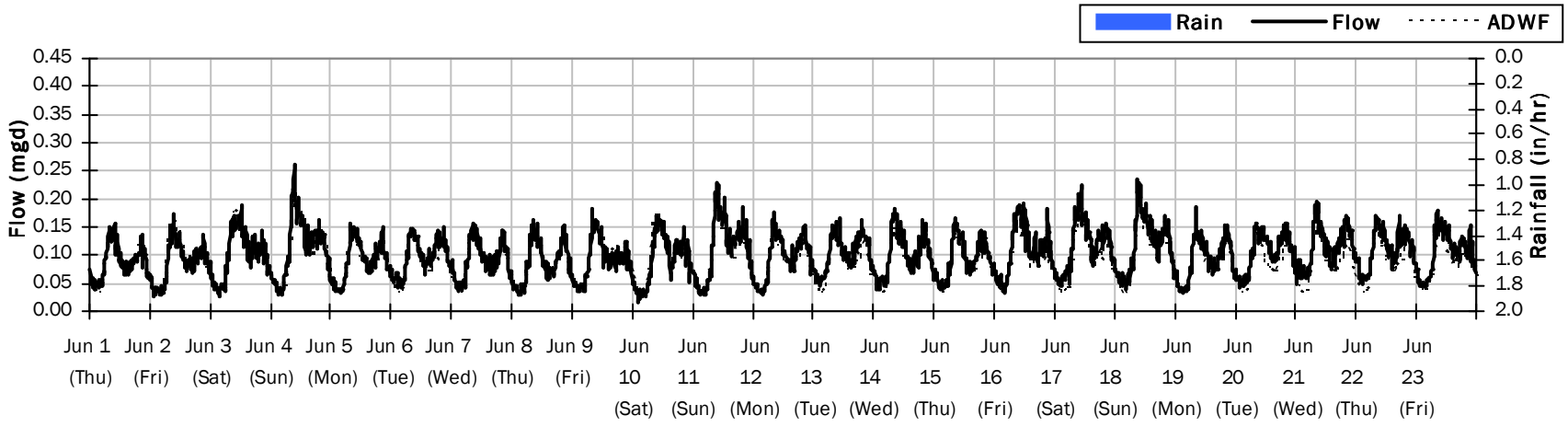


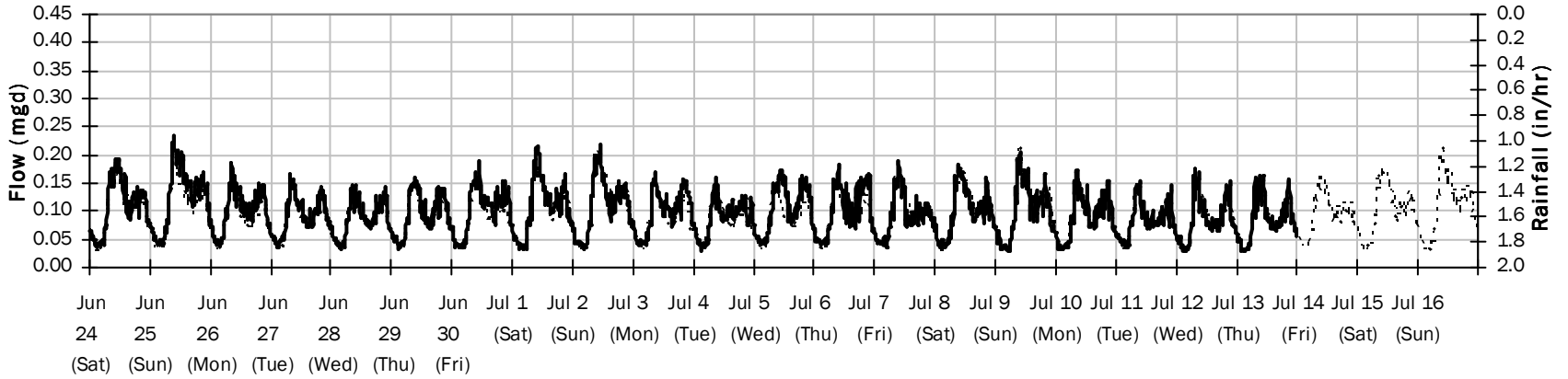


SITE 05

Flow Summary: 6/1/2017 to 7/16/2017

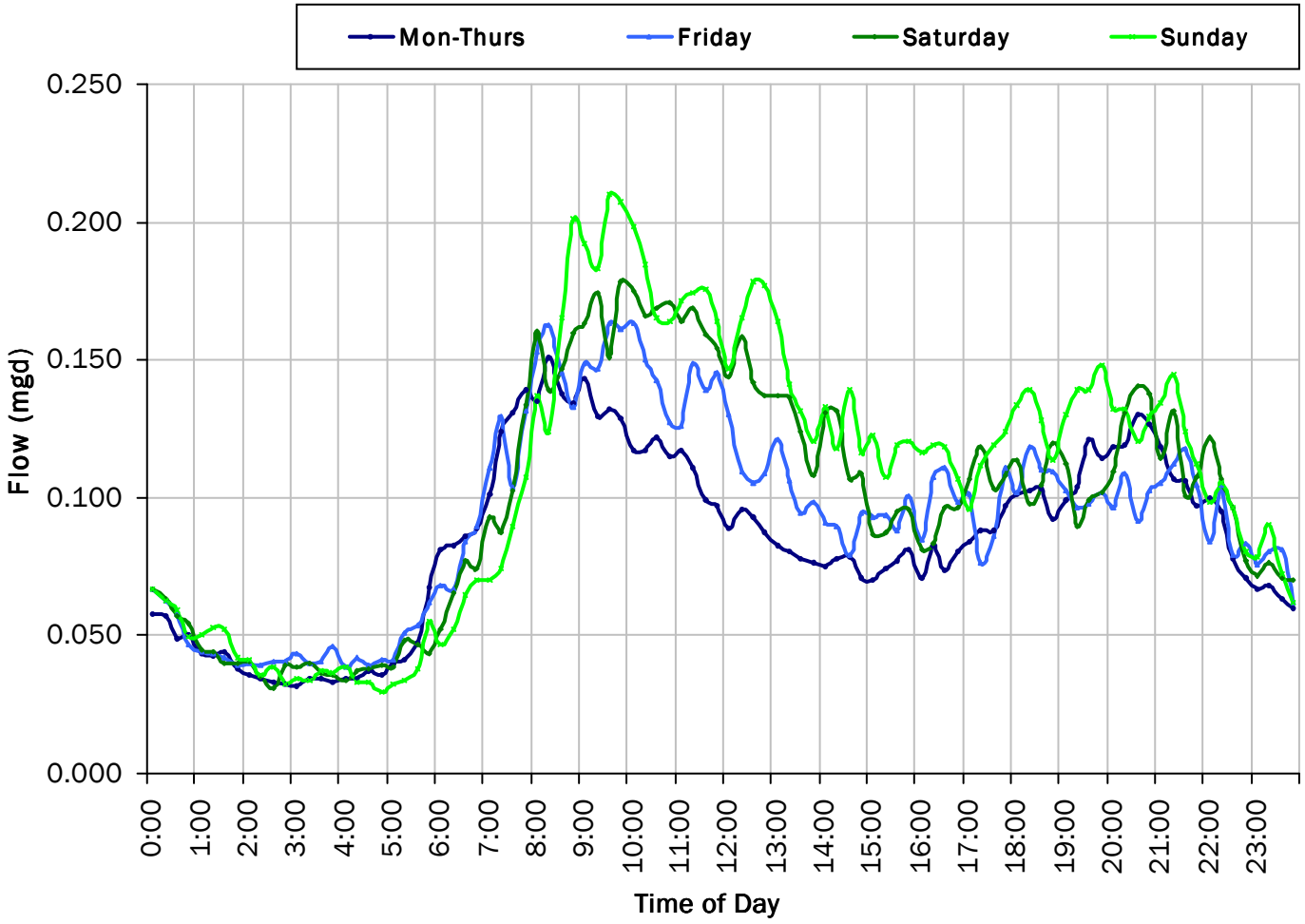
Total Period Rainfall: 0.00 inches **Avg Flow: 0.099 mgd** **Peak Flow: 0.262 mgd** **Min Flow: 0.016 mgd**





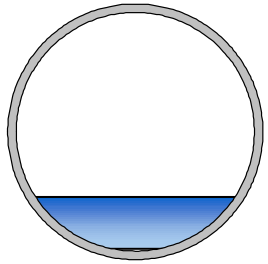
SITE 05

Average Dry Weather Flow Hydrographs



ADWF:

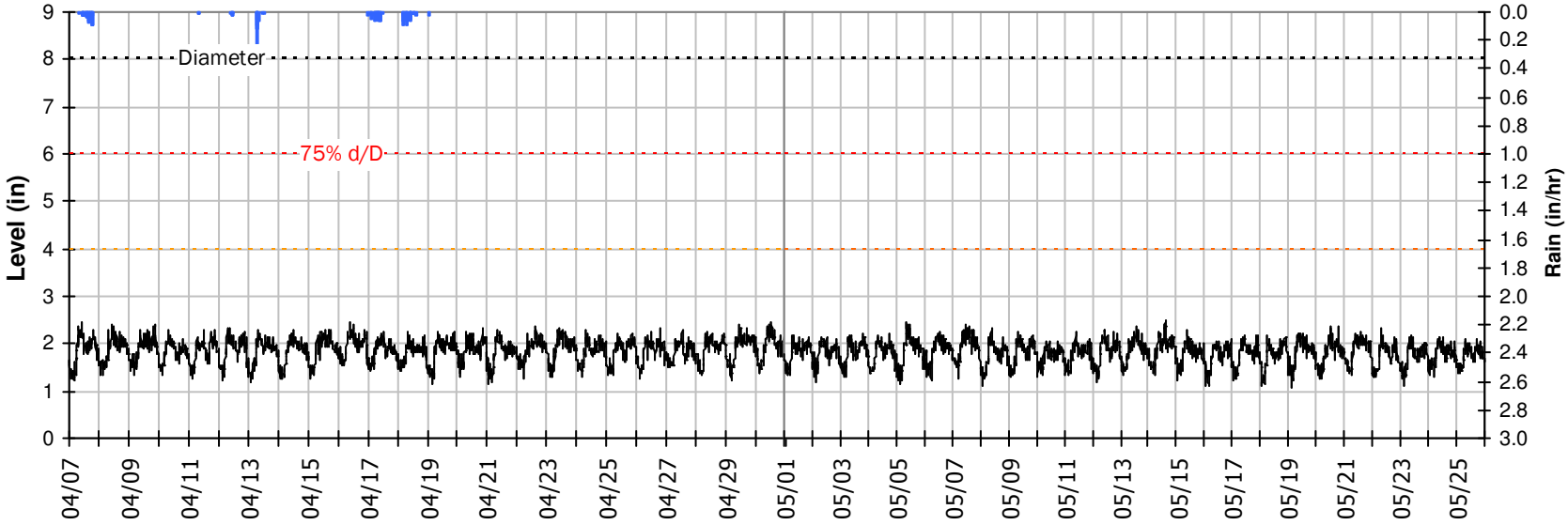
0.091 *mgd*

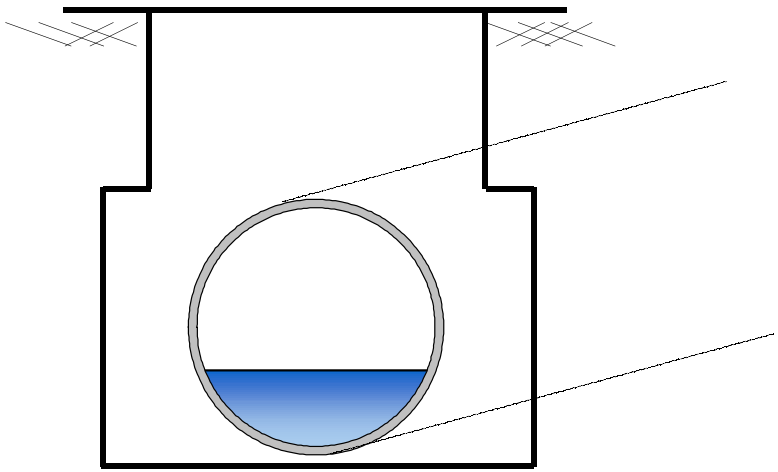


SITE 05

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



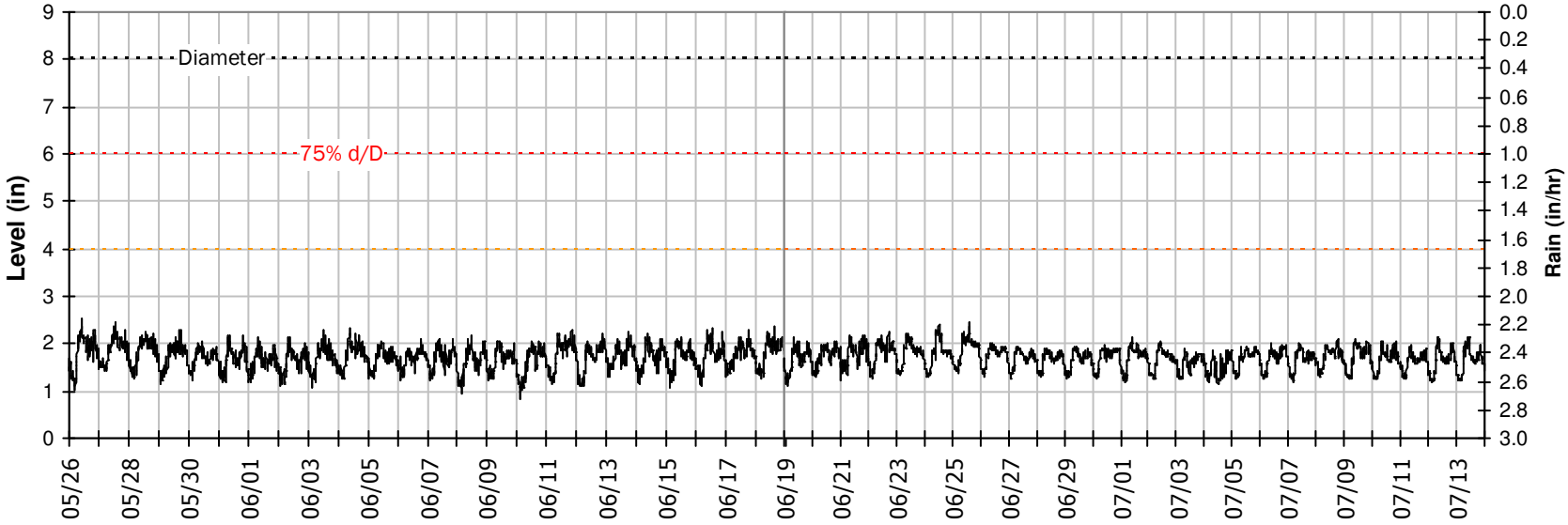


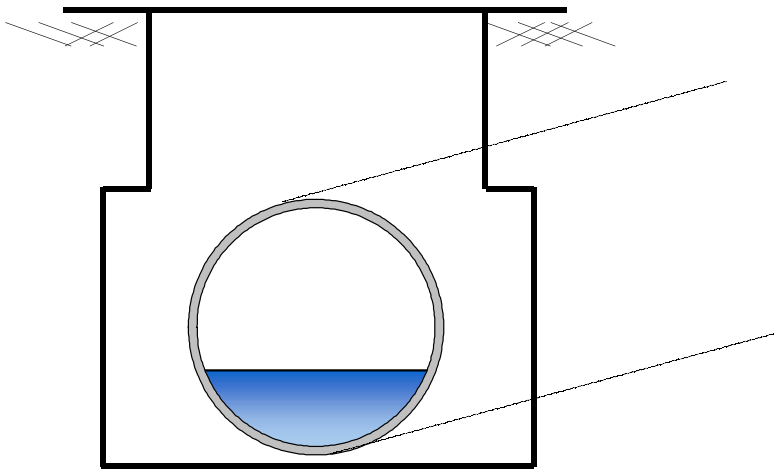
Pipe Diameter:	8	*inches*
Peak Measured Level:	2.53	*inches*
Peak d/D Ratio:	0.32	

SITE 05

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



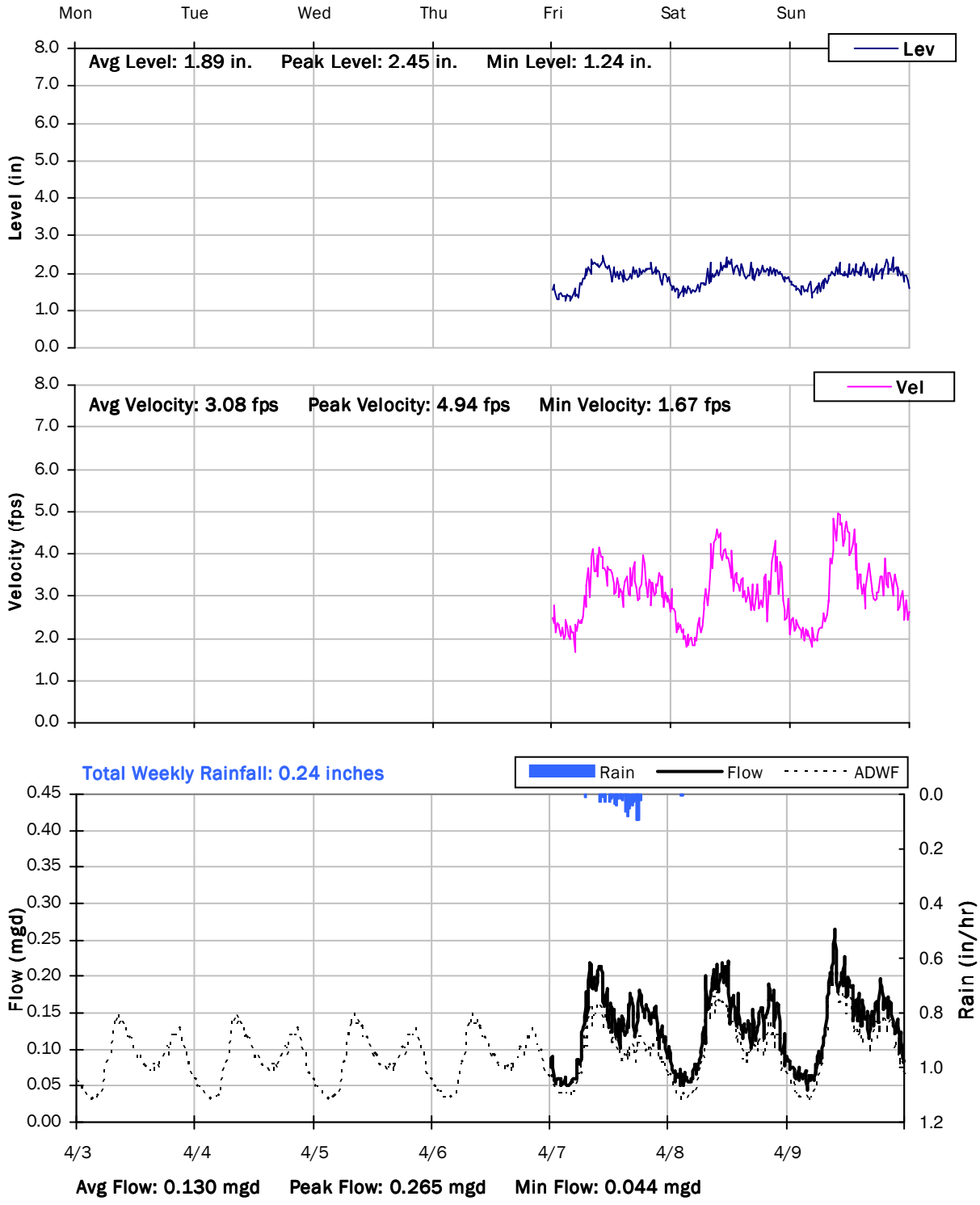


Pipe Diameter:	8	*inches*
Peak Measured Level:	2.53	*inches*
Peak d/D Ratio:	0.32	

SITE 05

Weekly Level, Velocity and Flow Hydrographs

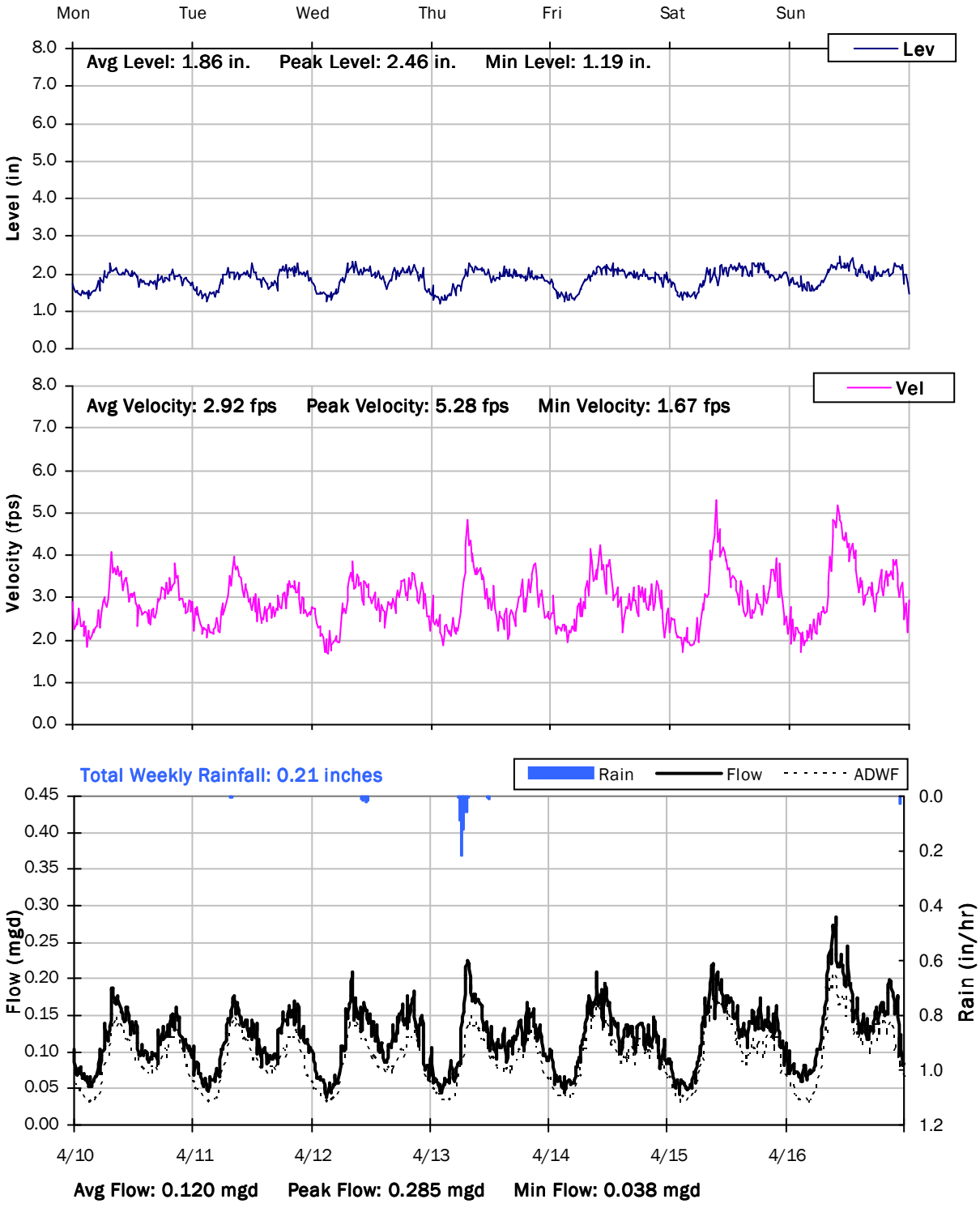
4/3/2017 to 4/10/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

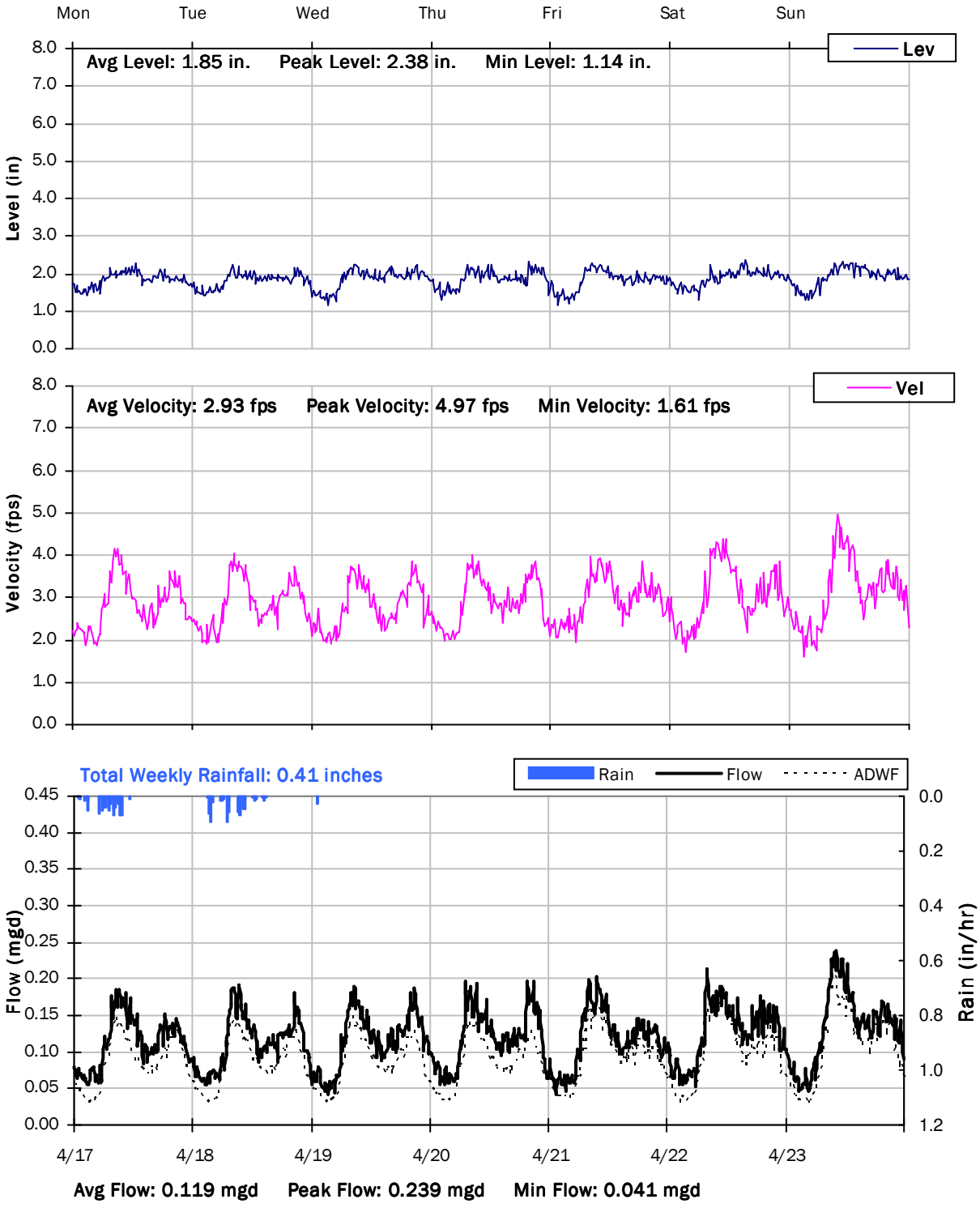
4/10/2017 to 4/17/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

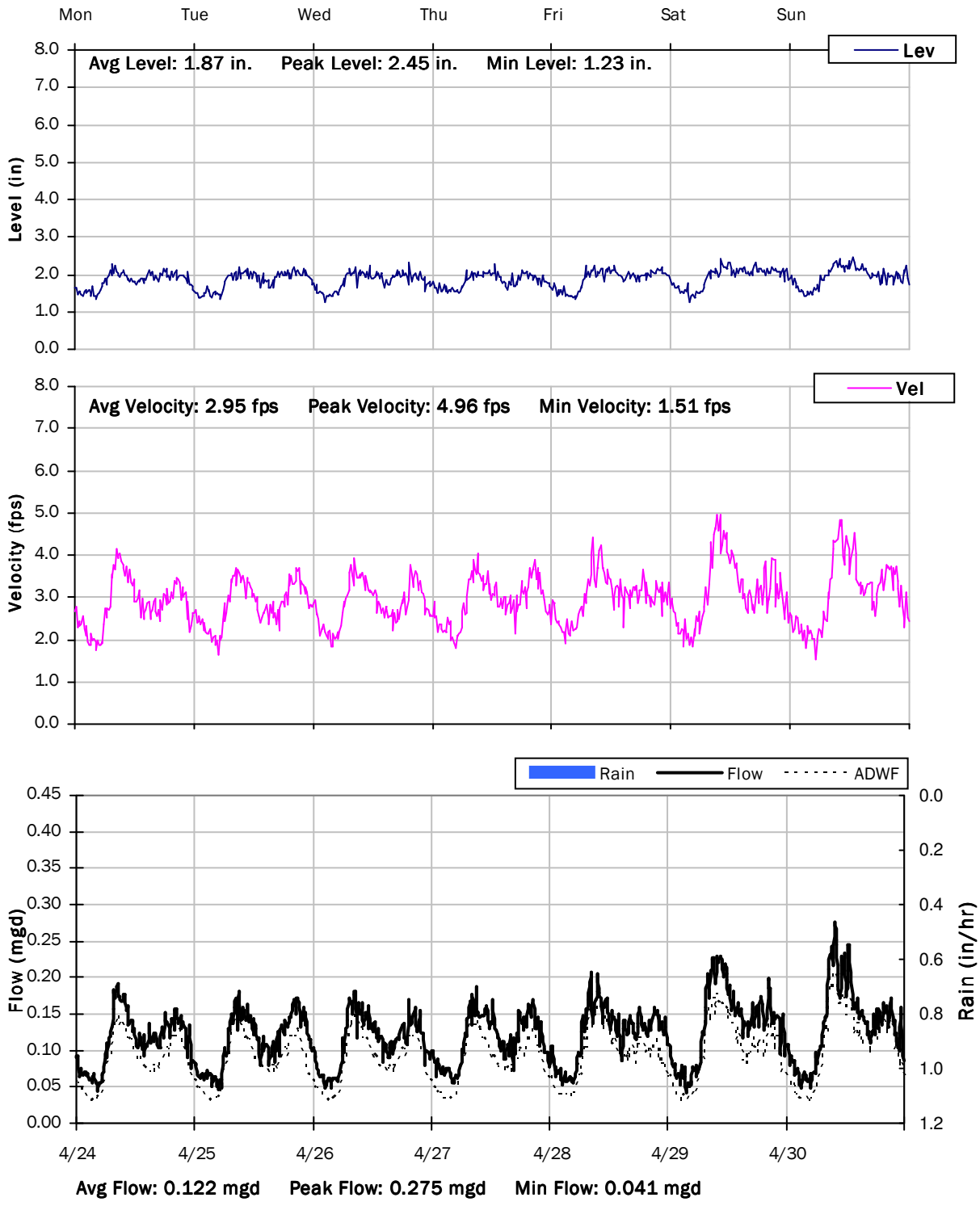
4/17/2017 to 4/24/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

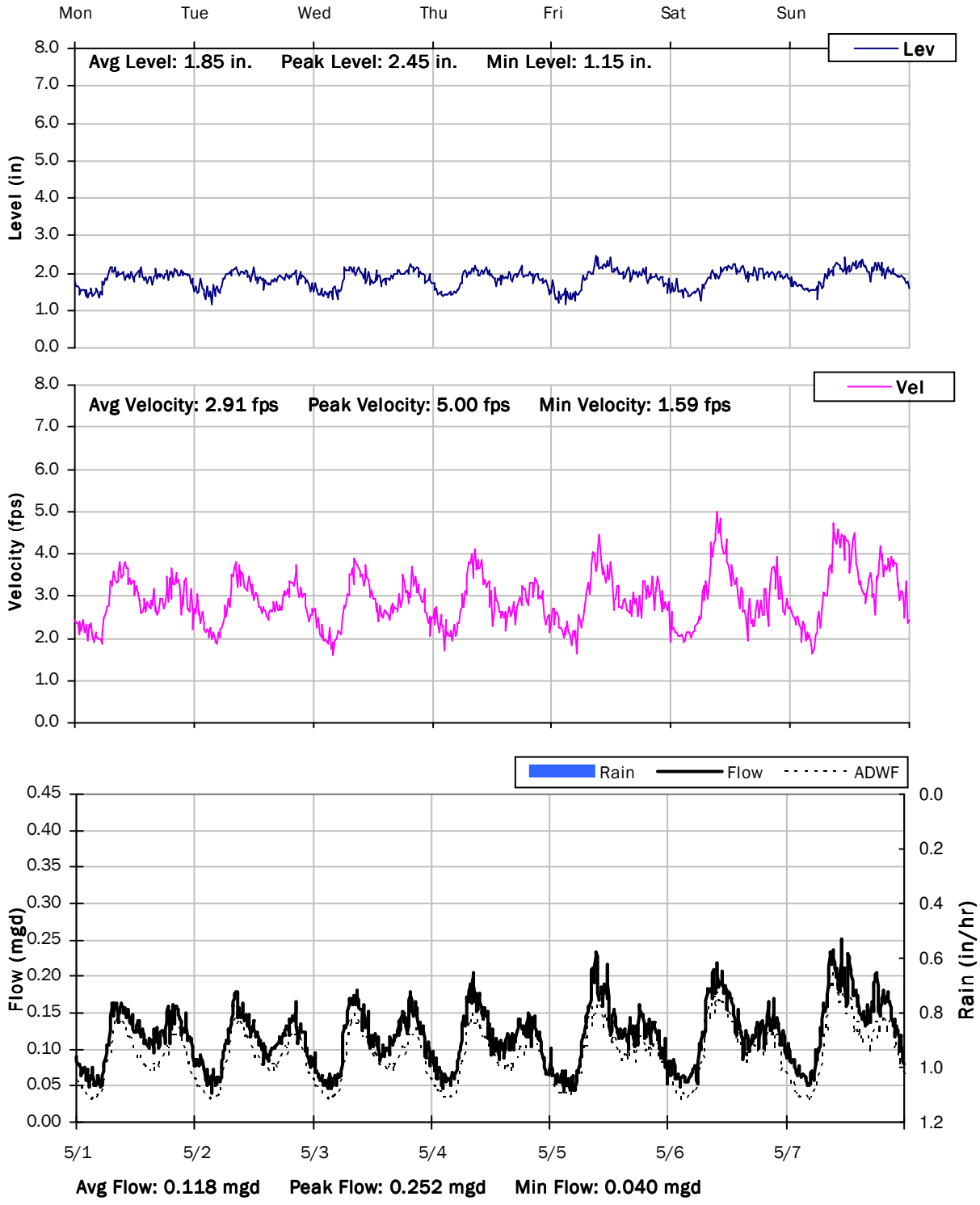
4/24/2017 to 5/1/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

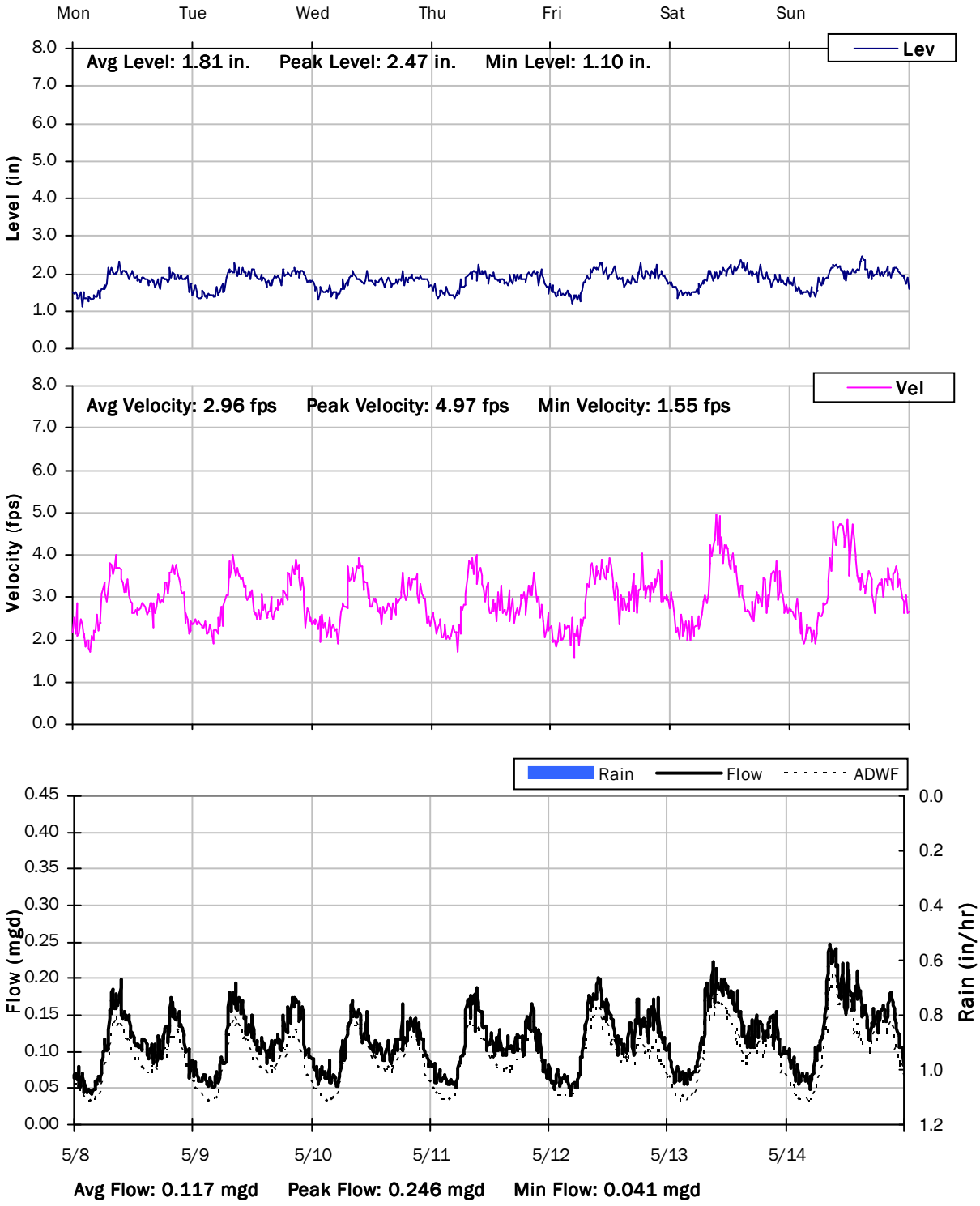
5/1/2017 to 5/8/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

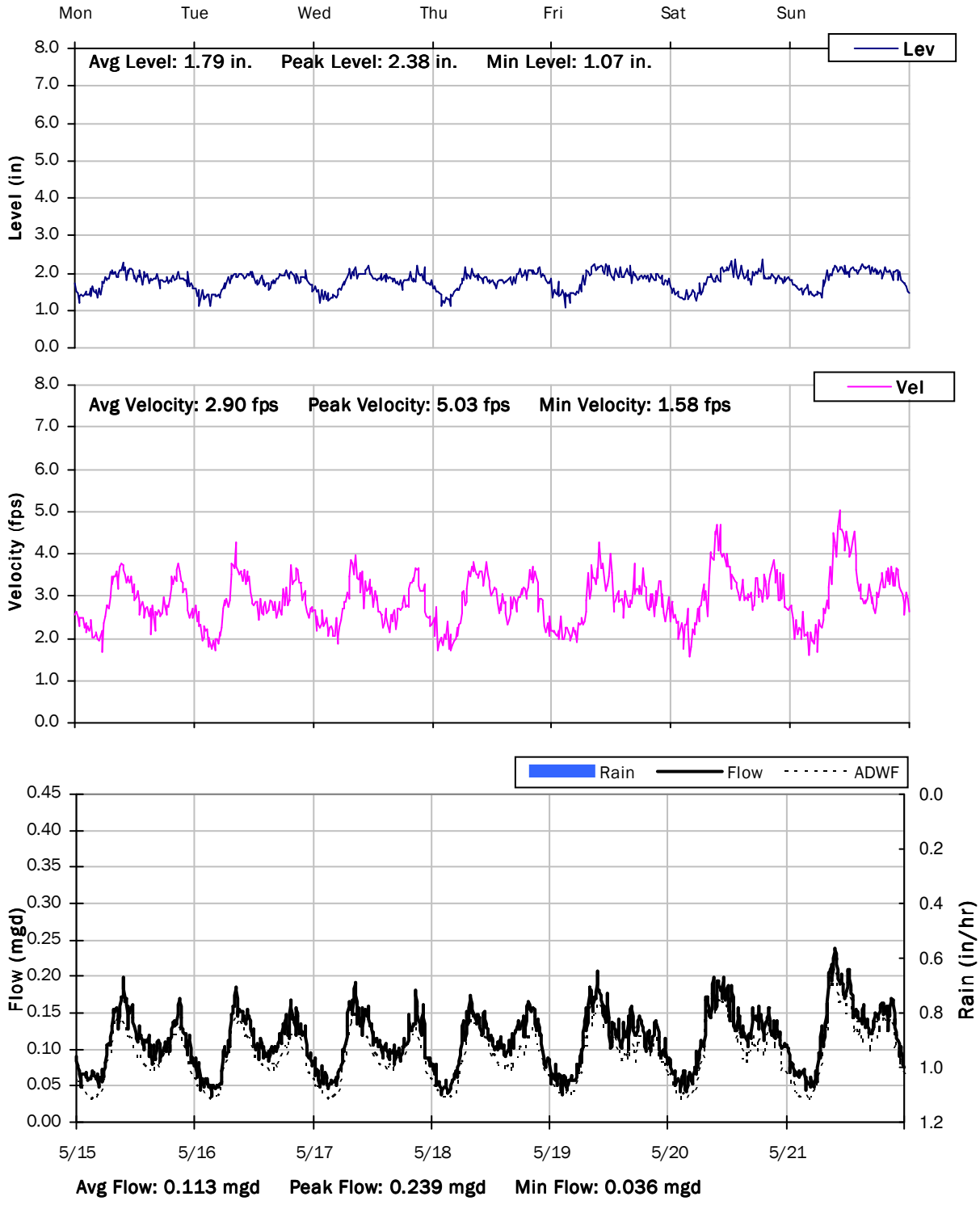
5/8/2017 to 5/15/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

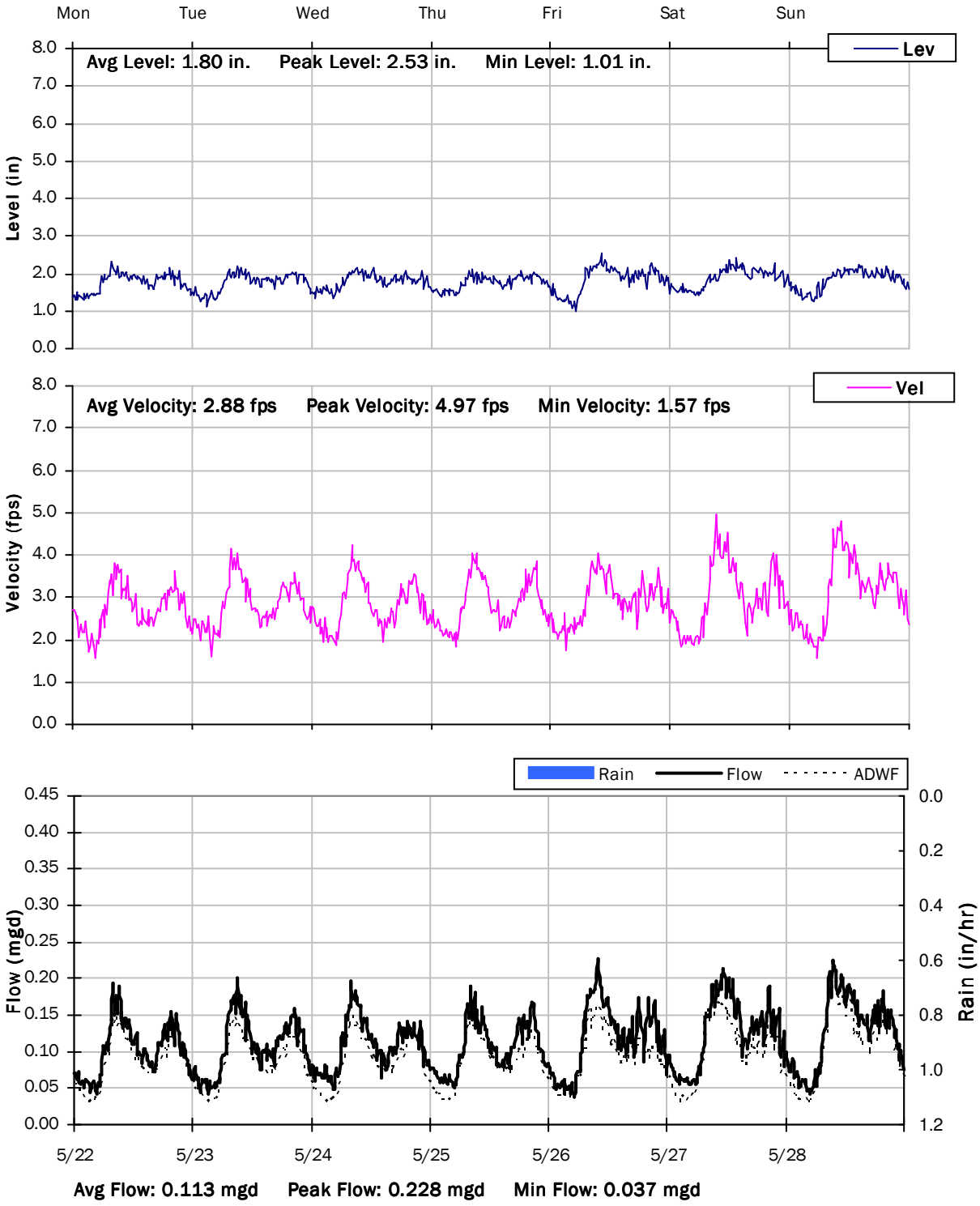
5/15/2017 to 5/22/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

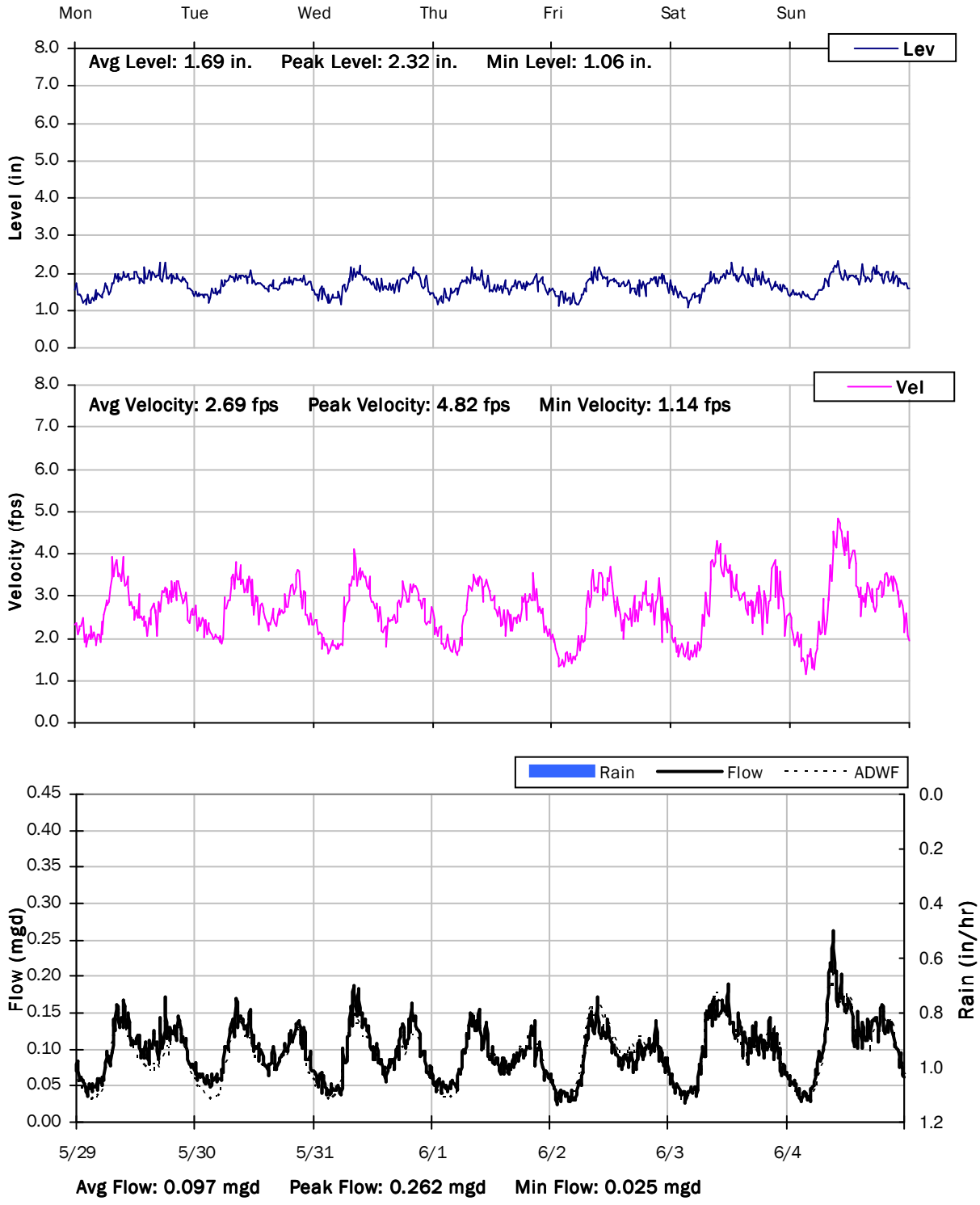
5/22/2017 to 5/29/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

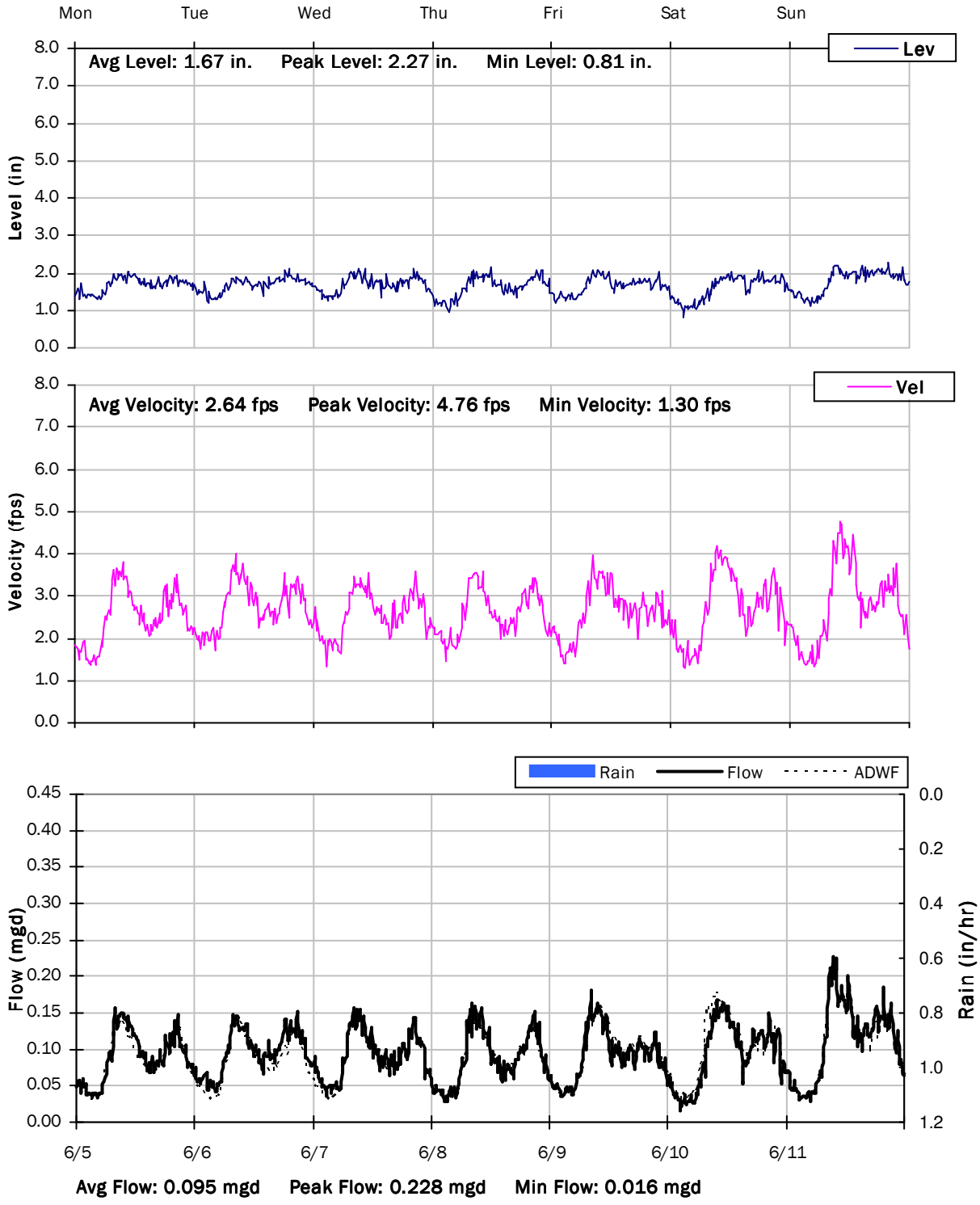
5/29/2017 to 6/5/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

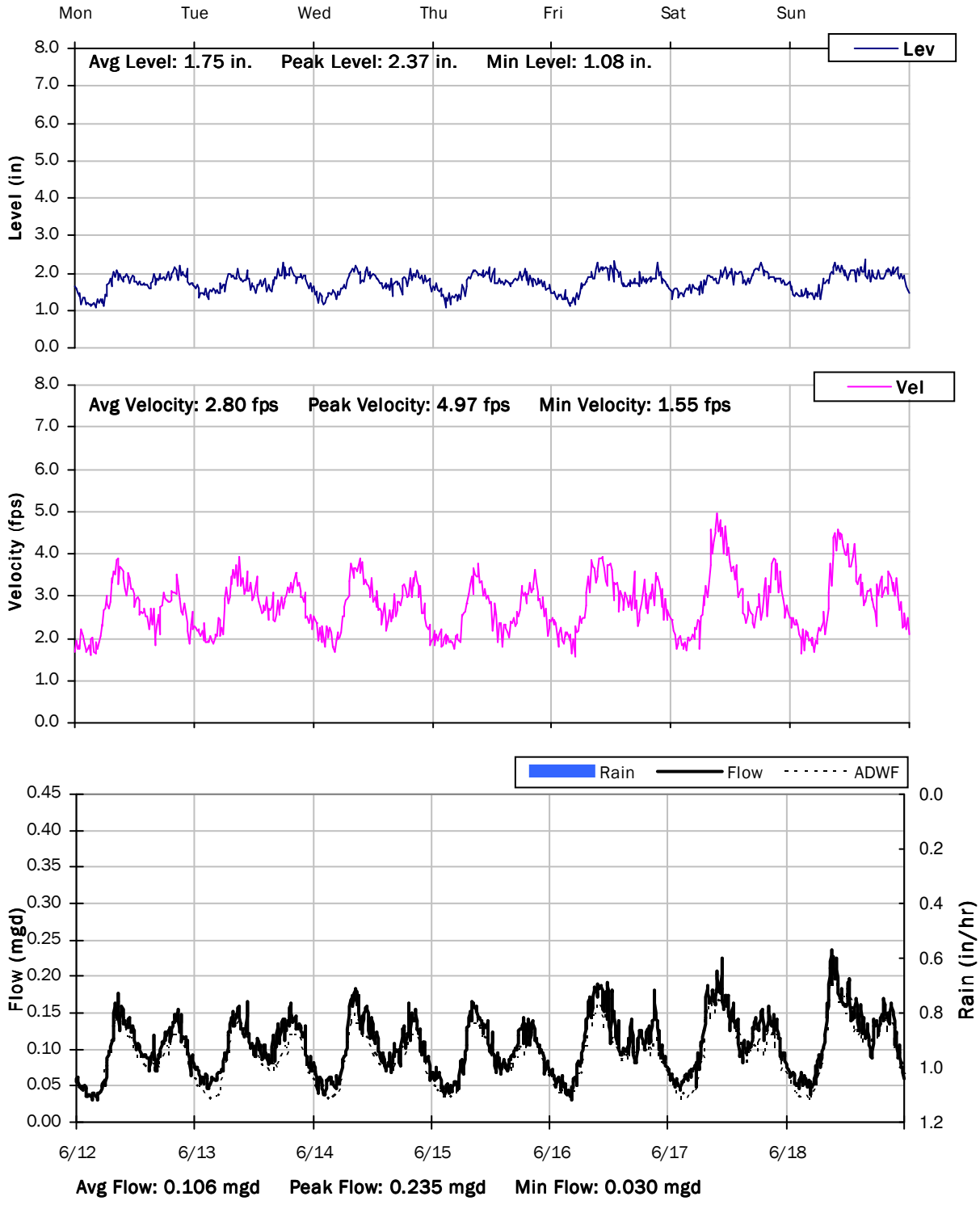
6/5/2017 to 6/12/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

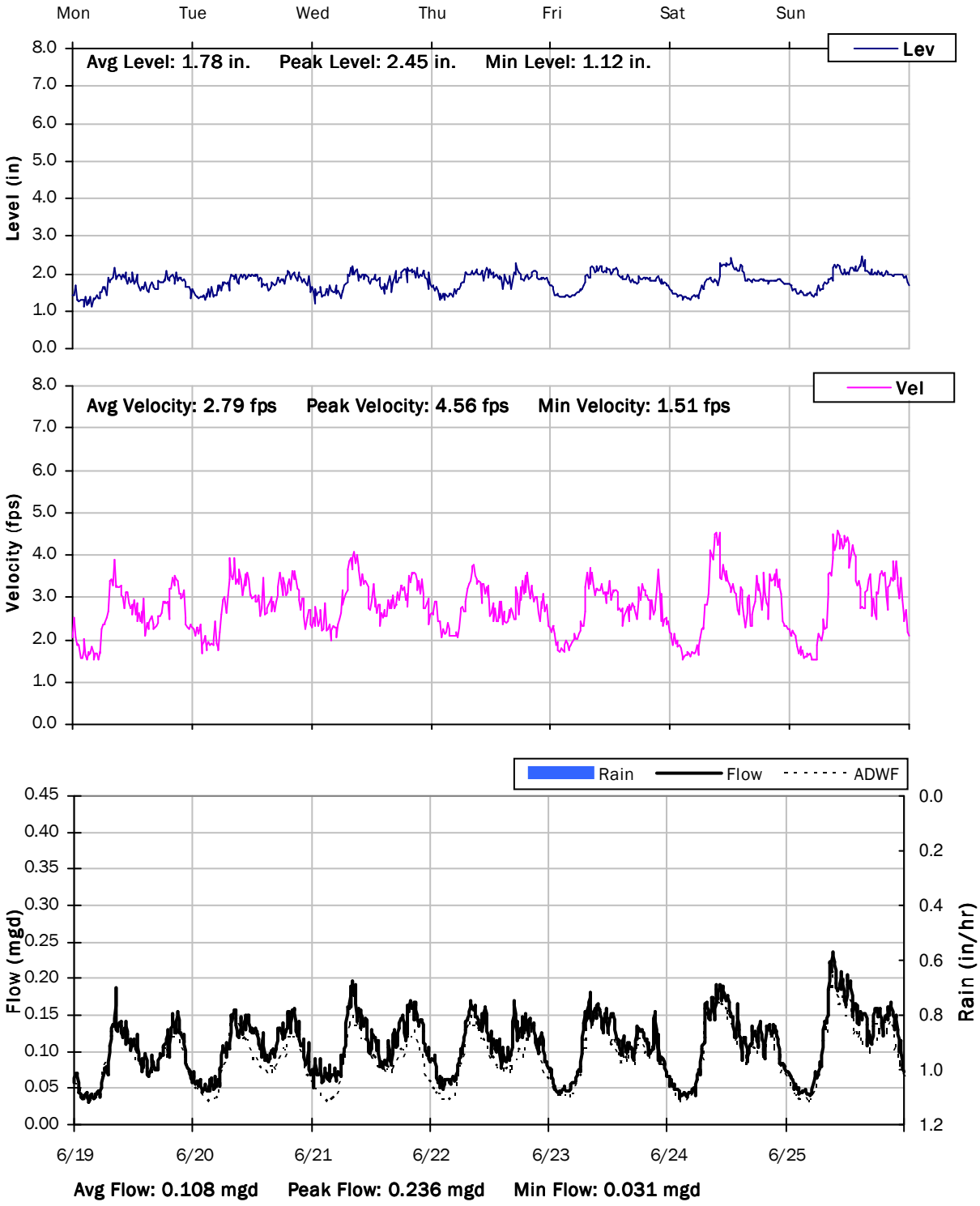
6/12/2017 to 6/19/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

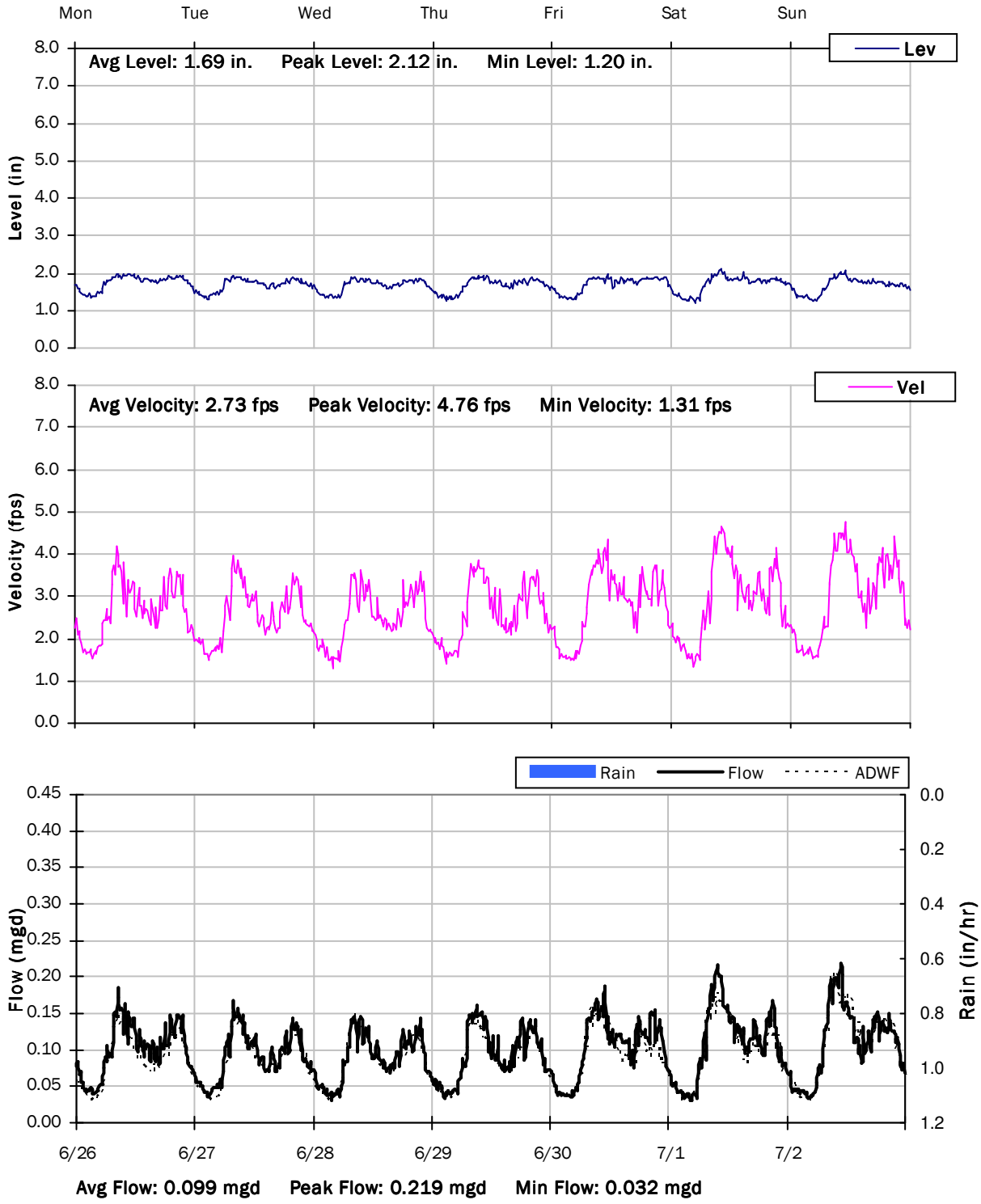
6/19/2017 to 6/26/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

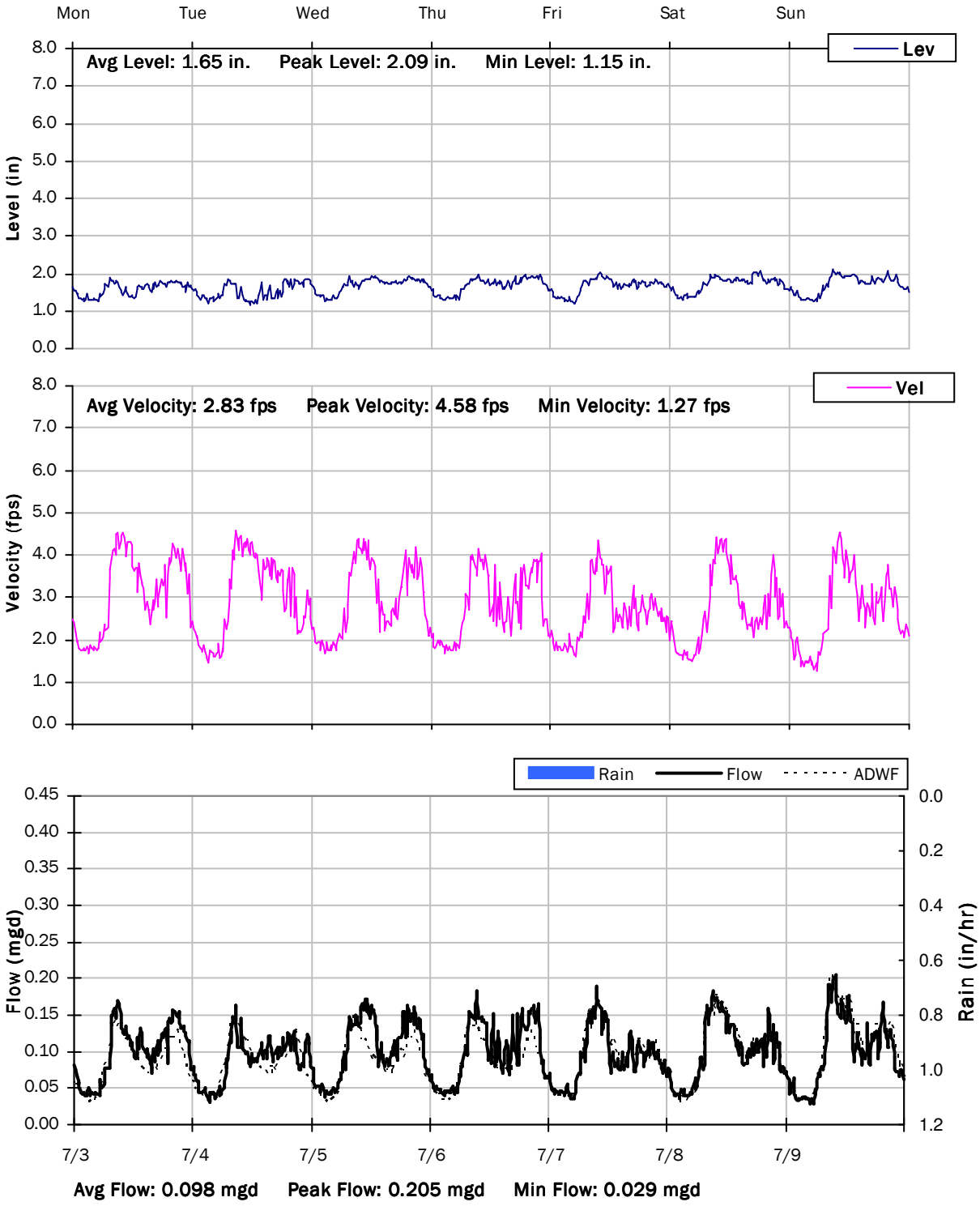
6/26/2017 to 7/3/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

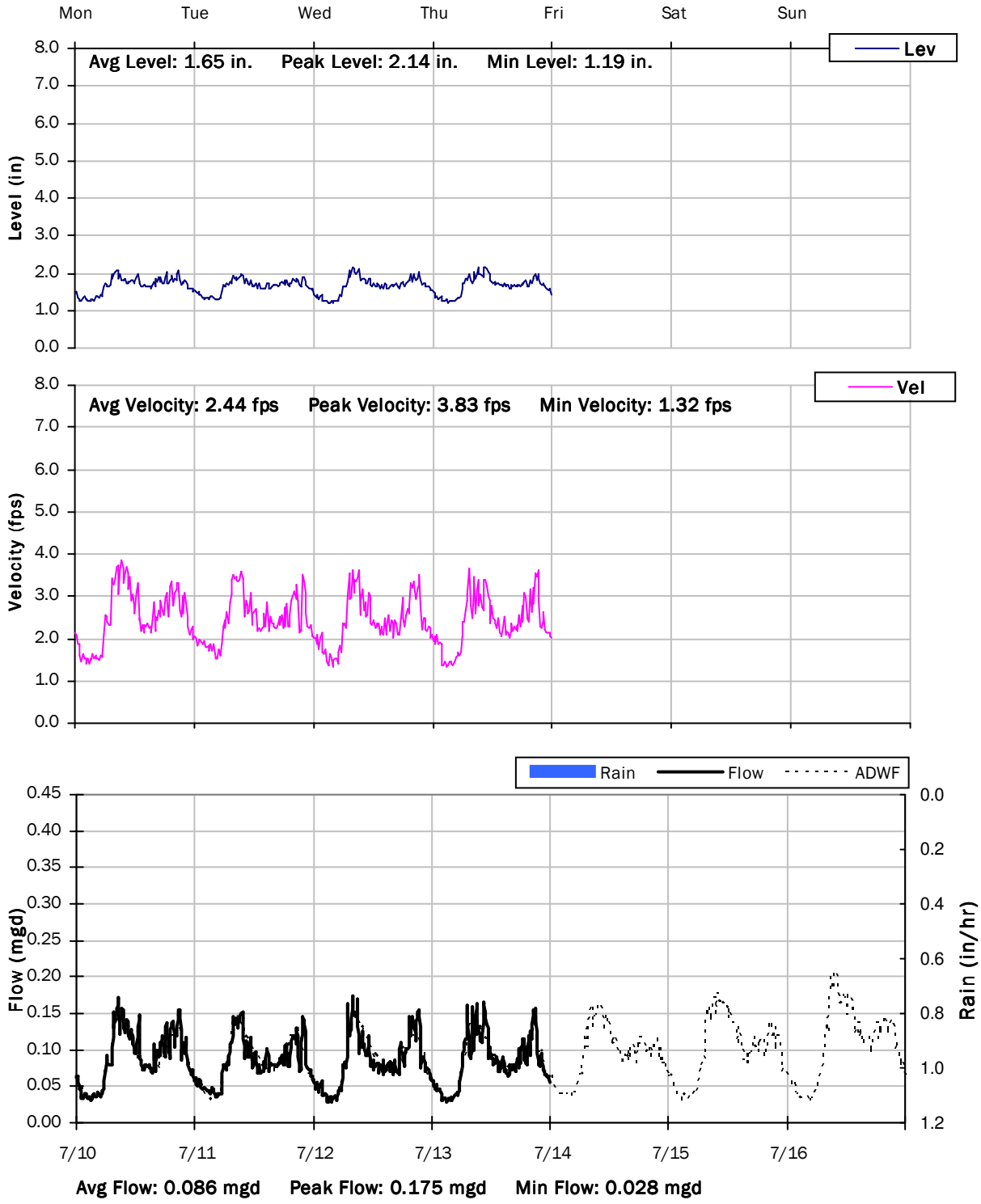
7/3/2017 to 7/10/2017



SITE 05

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

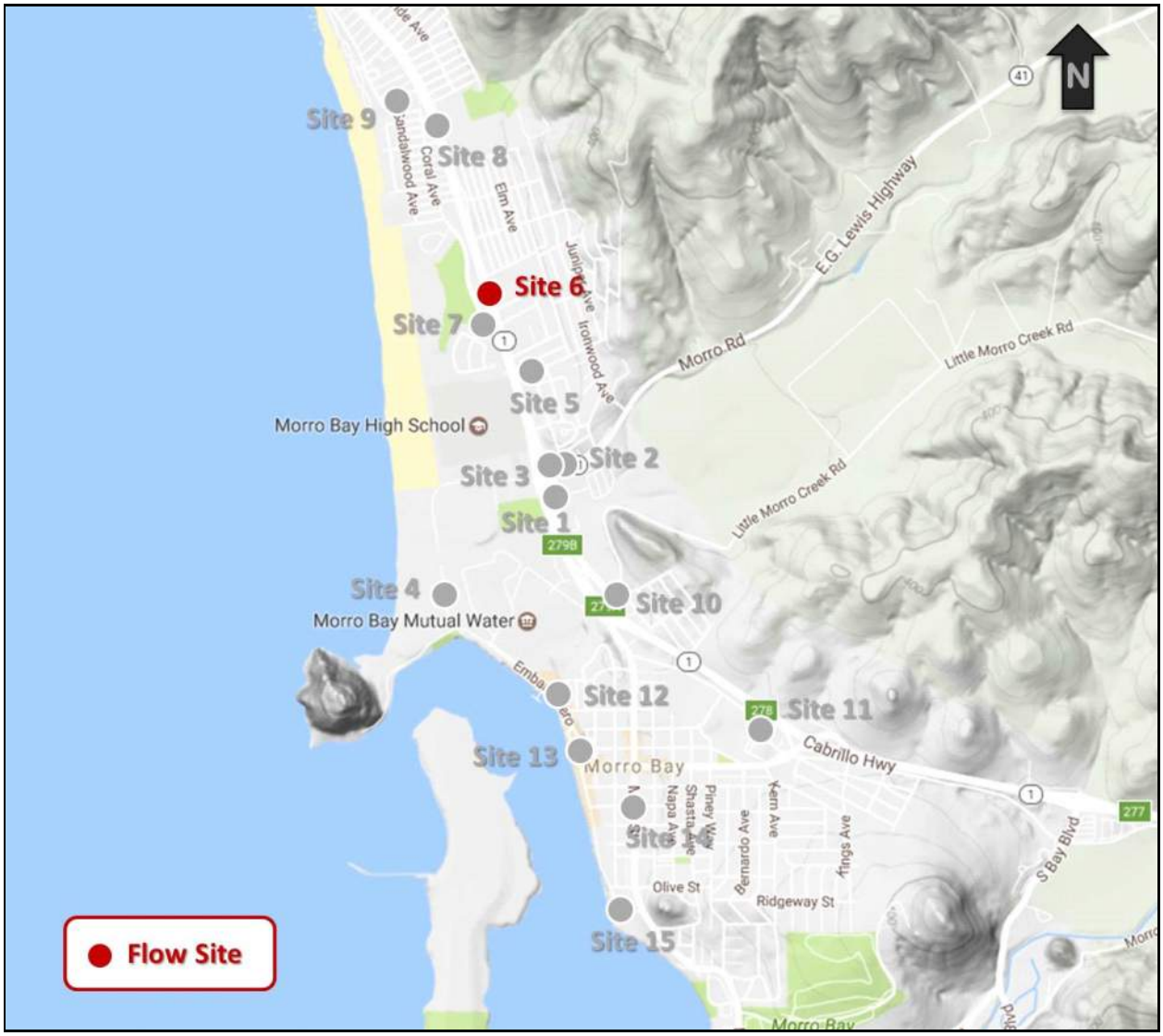
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 06

Location: Main Street, ~290 feet south of Elena Street, meter on north 13" inlet

Data Summary Report



Vicinity Map: Site 06

SITE 06

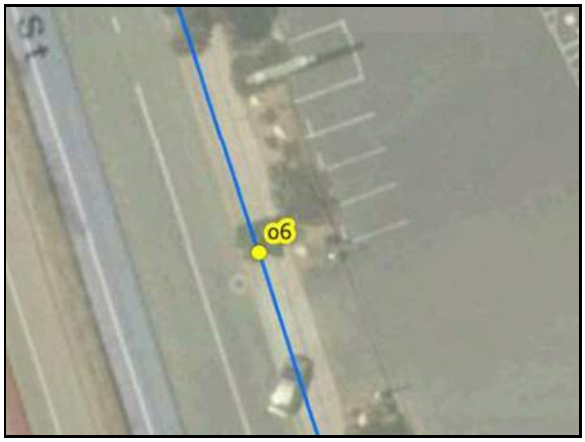
Site Information

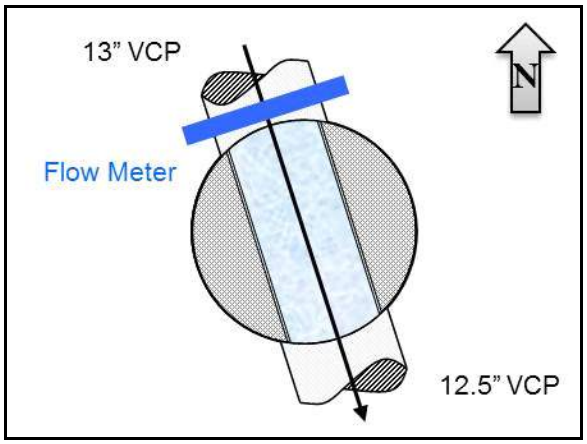
Location: Main Street, ~290 feet south of Elena Street, meter on north 13" inlet

Pipe Diameter: 13 inches

ADWF: 0.112 mgd

Peak Measured Flow: 0.351 mgd





SITE 06

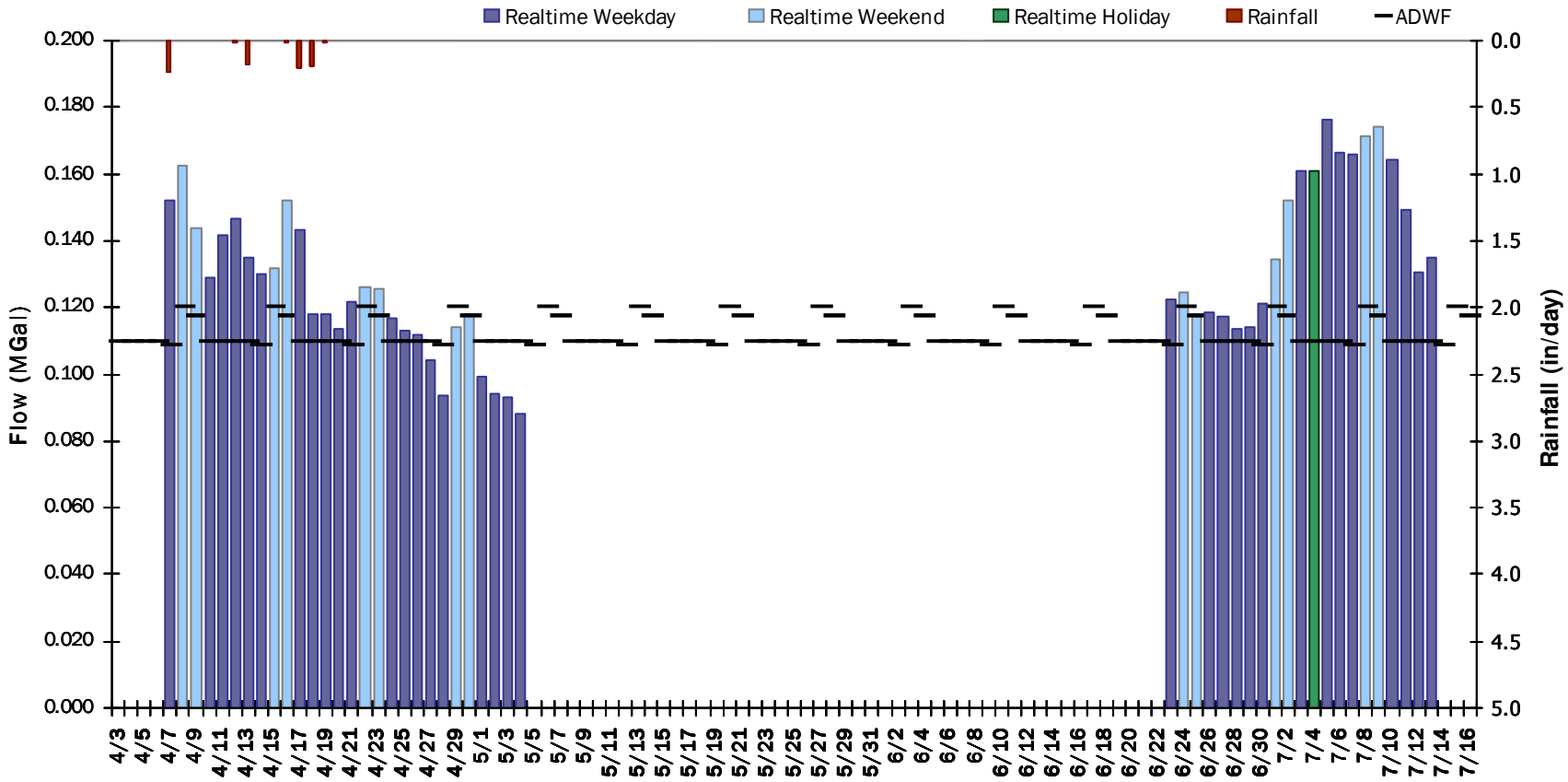
Additional Site Photos

SITE 06

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.131 MGal Peak Daily Flow: 0.176 MGal Min Daily Flow: 0.088 MGal

Total Period Rainfall: 0.85 inches

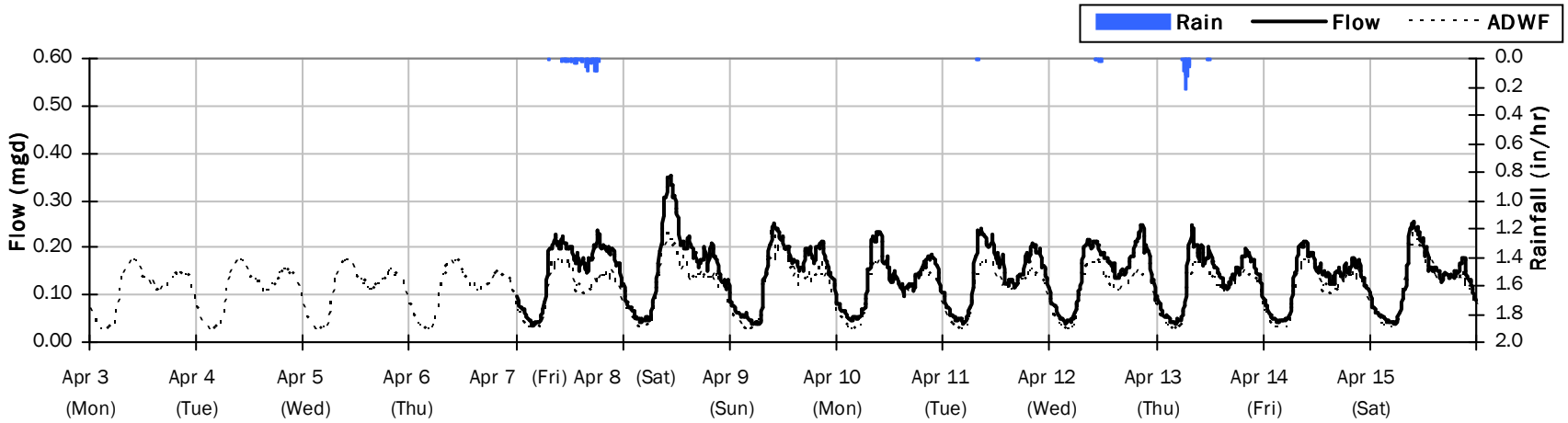


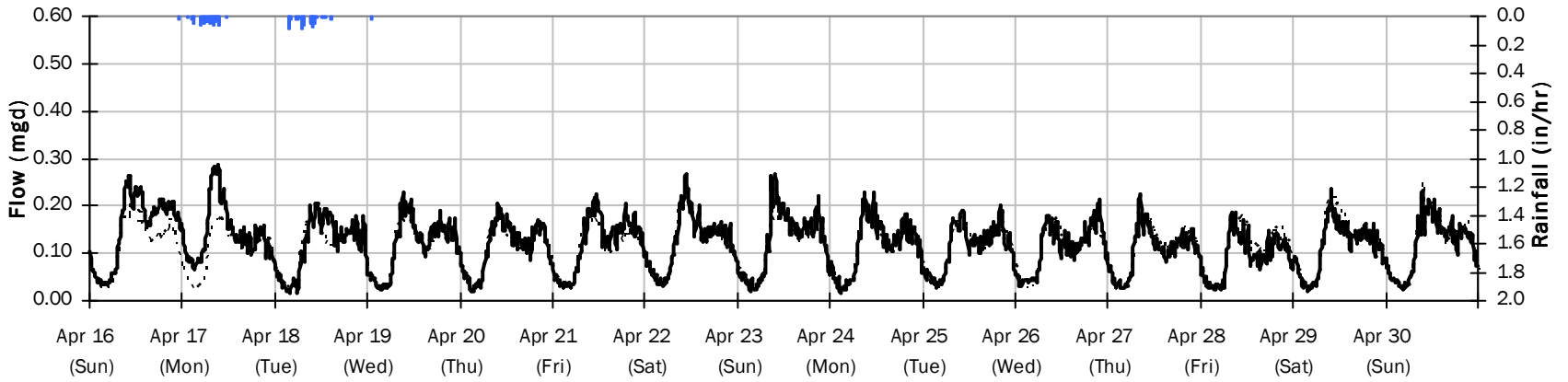
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 06

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.128 mgd** **Peak Flow: 0.351 mgd** **Min Flow: 0.014 mgd**

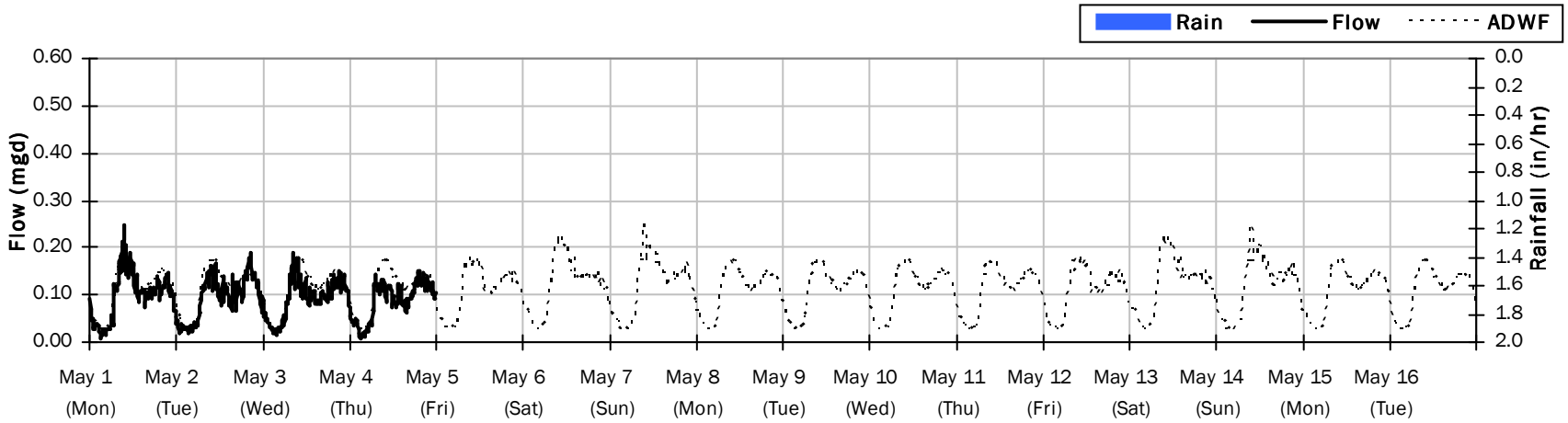


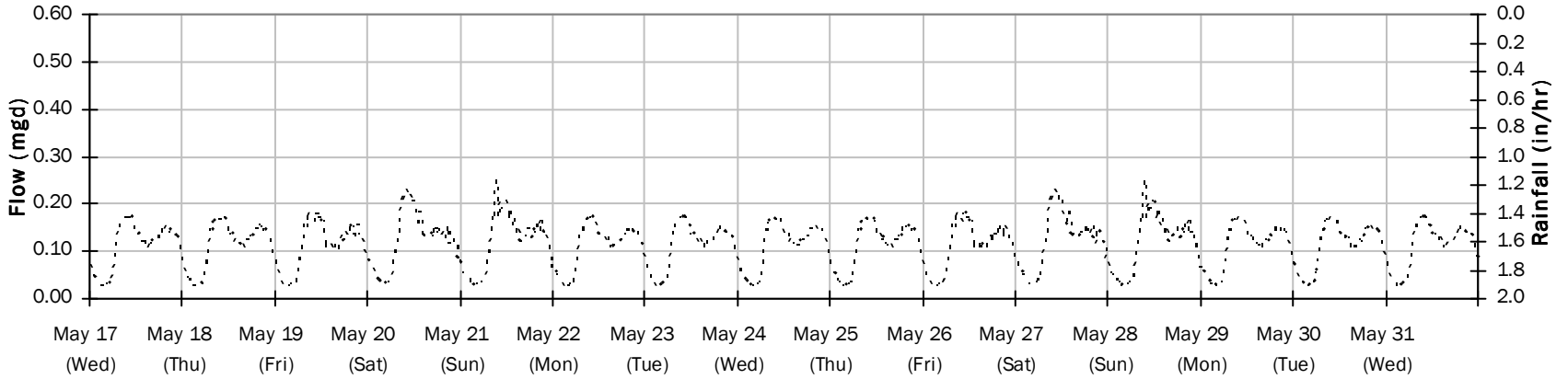


SITE 06

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.094 mgd** **Peak Flow: 0.248 mgd** **Min Flow: 0.008 mgd**



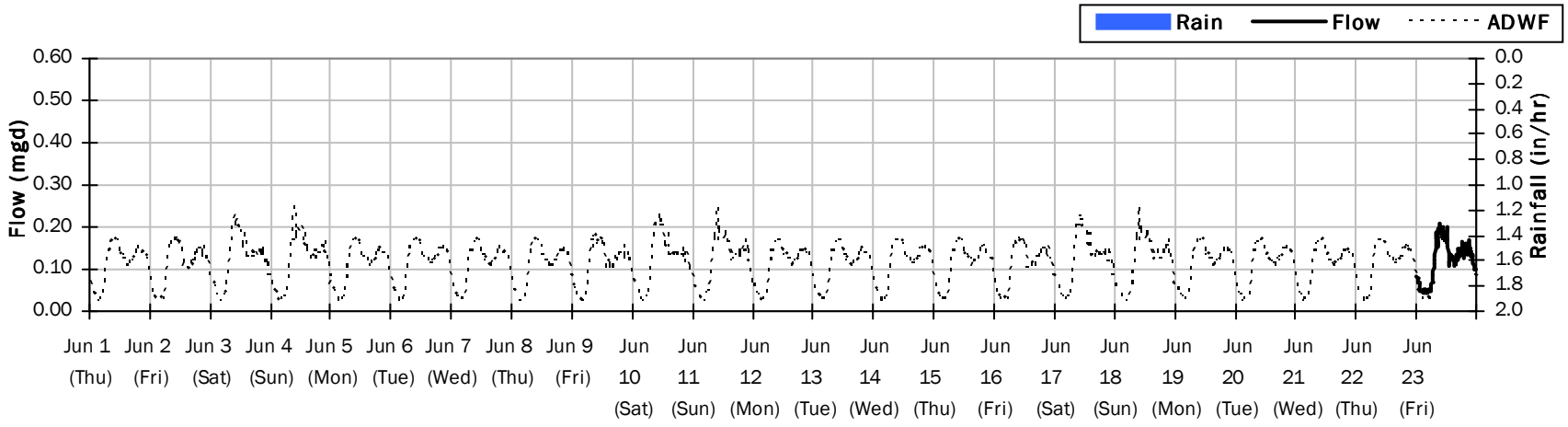


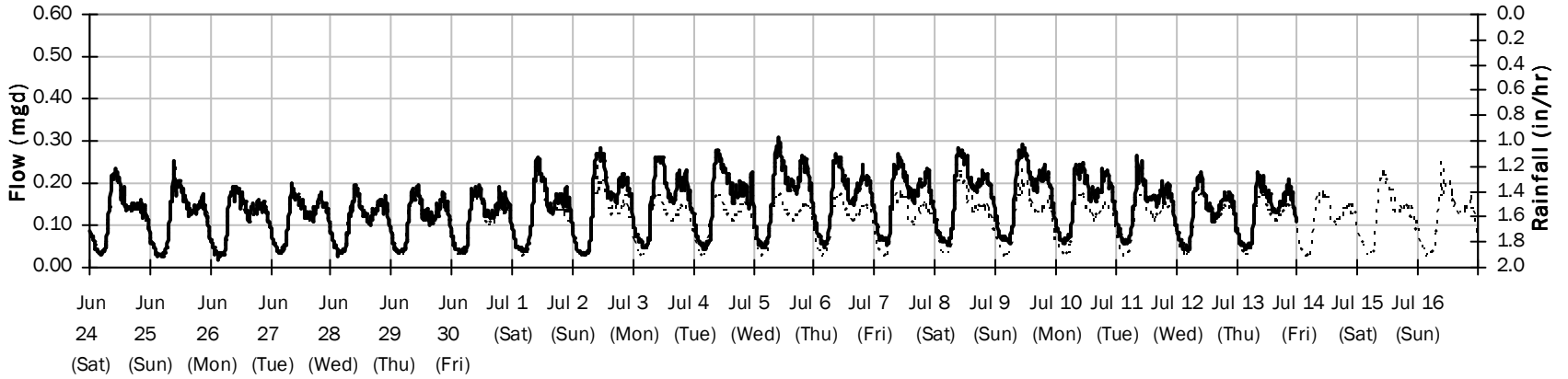
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 06

Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.142 mgd Peak Flow: 0.311 mgd Min Flow: 0.019 mgd**





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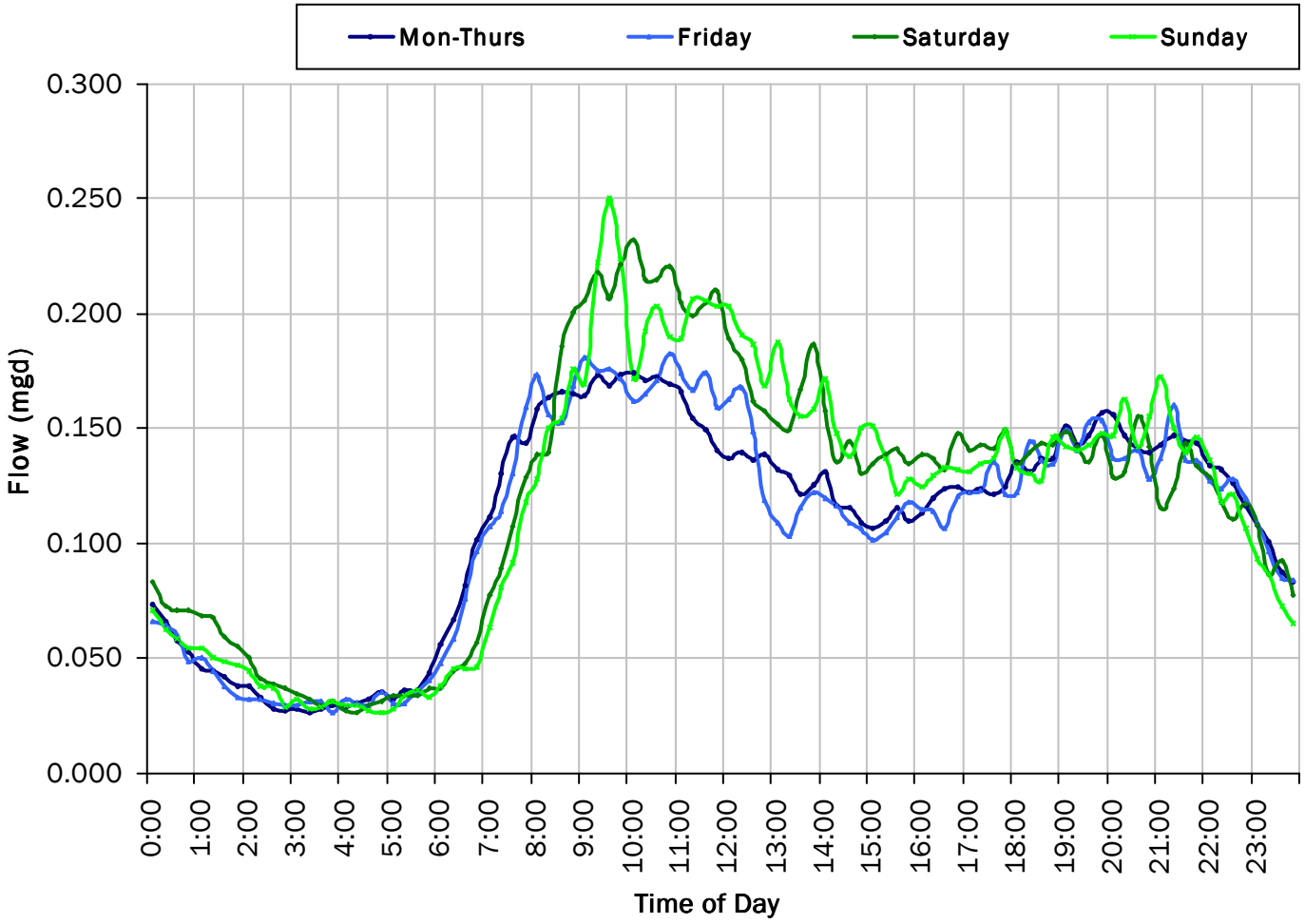
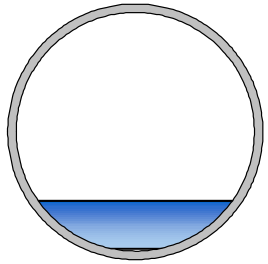
SITE 06

Average Dry Weather Flow Hydrographs



ADWF:

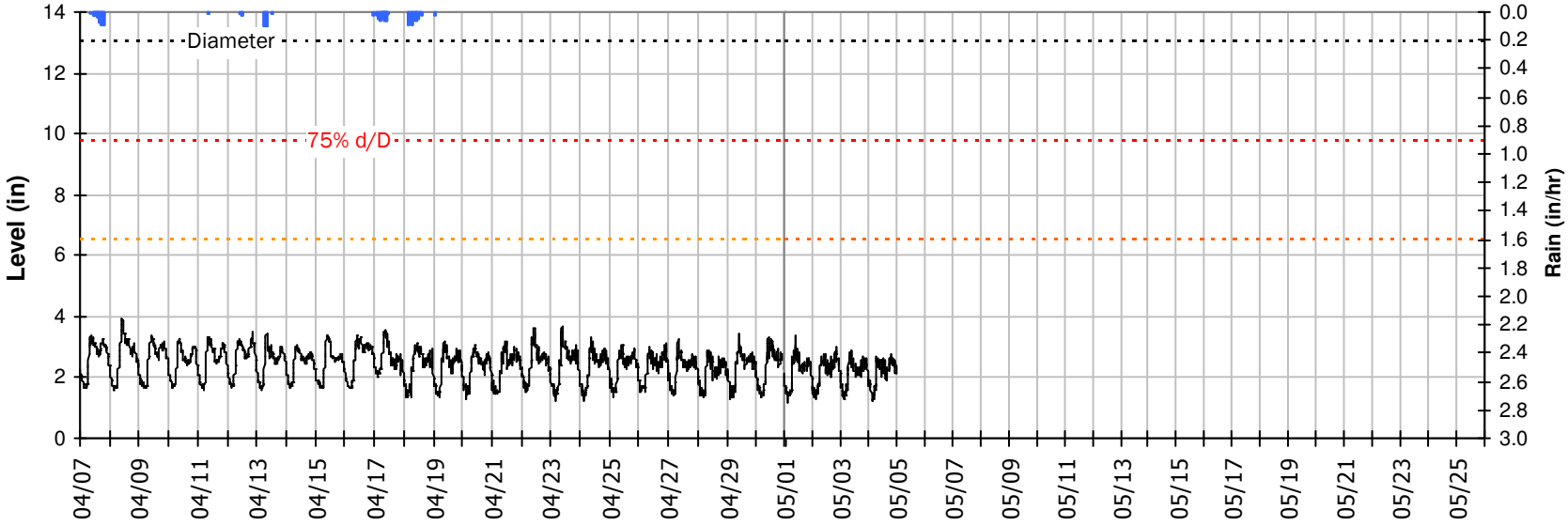
0.112 *mgd*

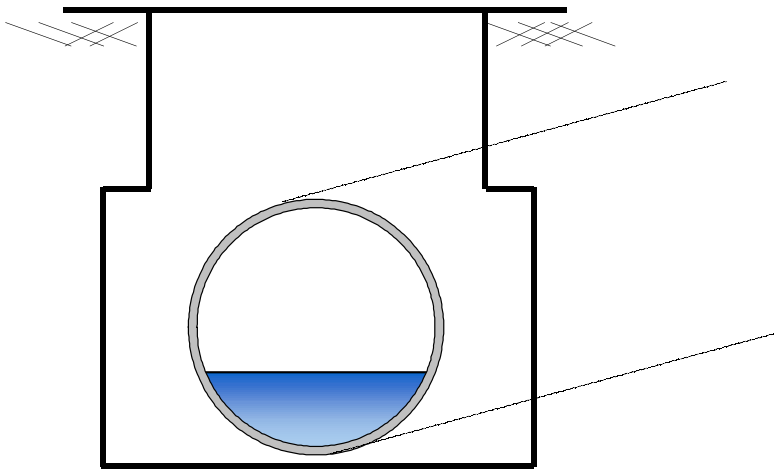


SITE 06

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





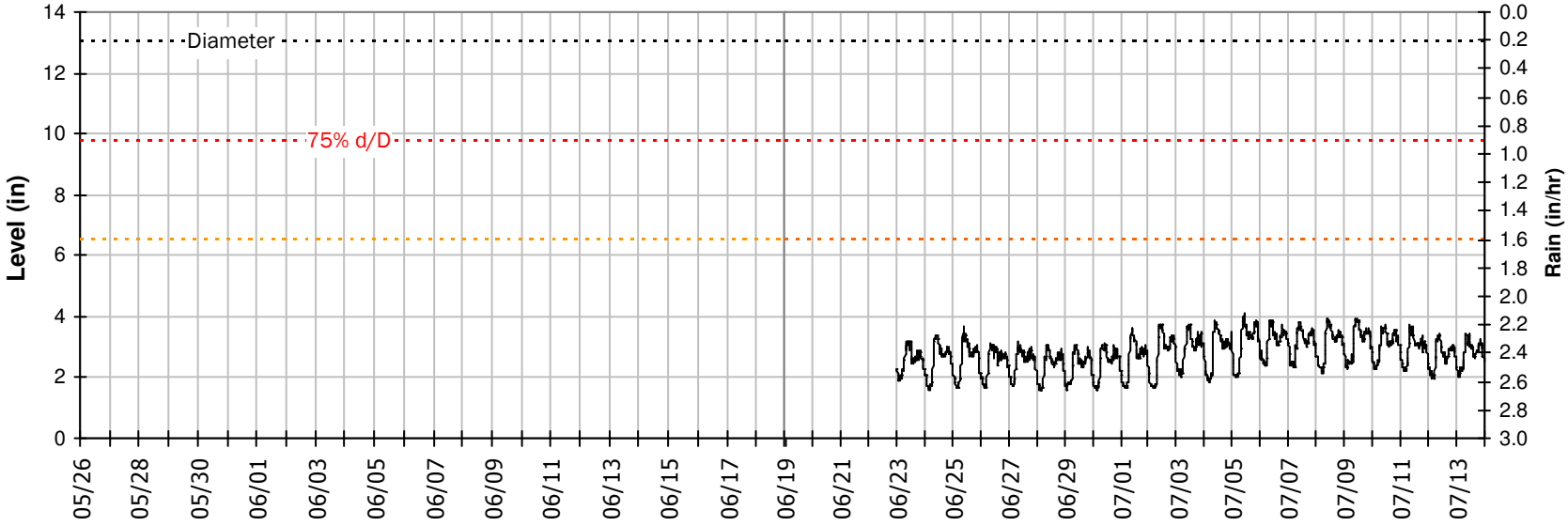
Pipe Diameter:	13	*inches*
Peak Measured Level:	4.1	*inches*
Peak d/D Ratio:	0.32	

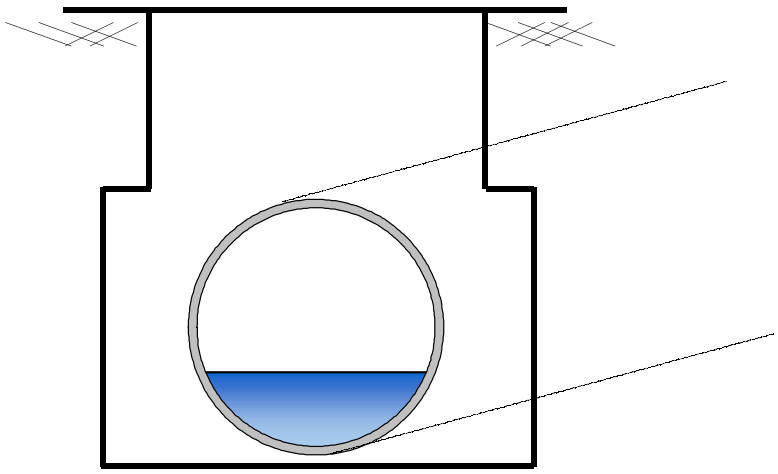
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 06

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter:	13	*inches*
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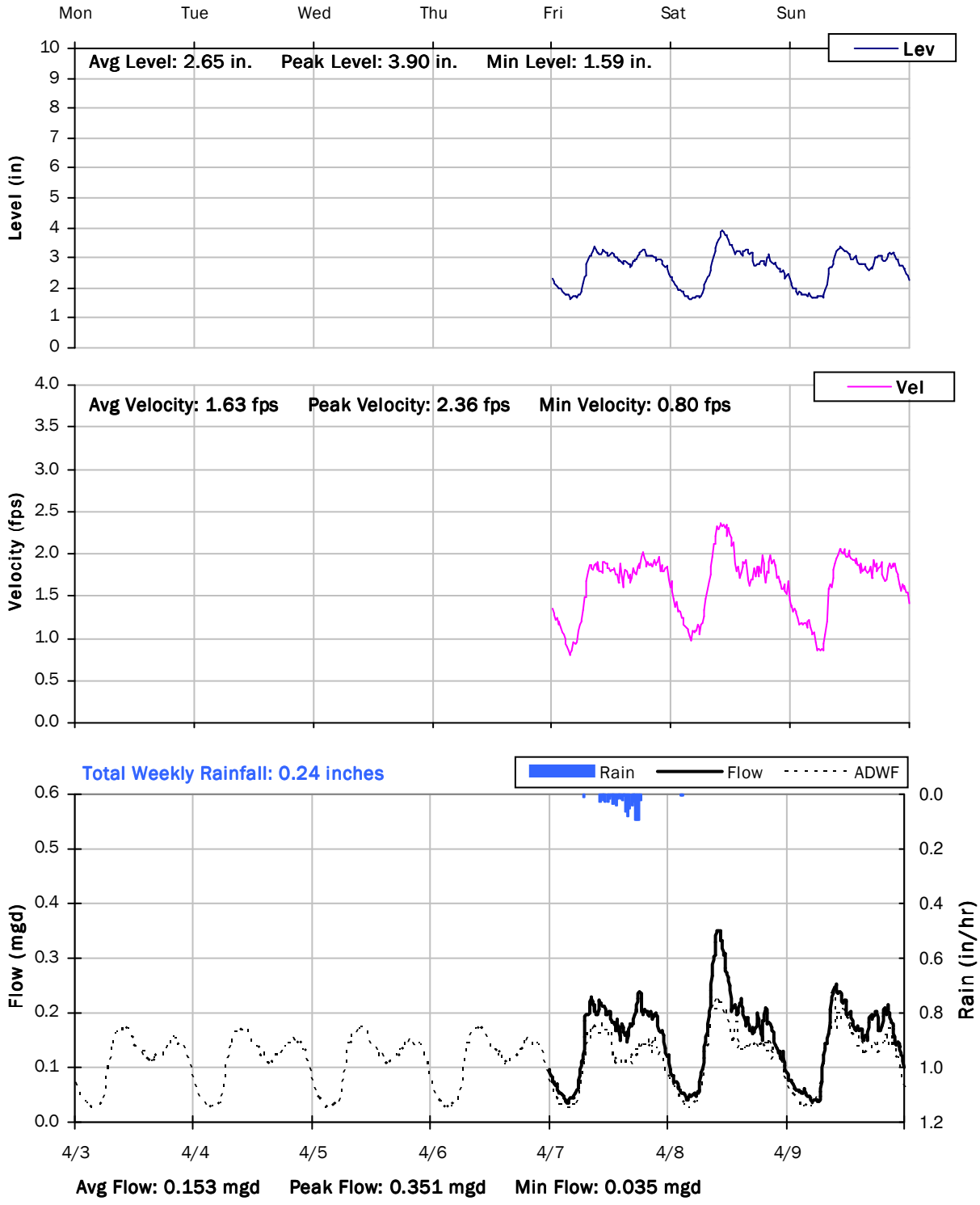
SITE 06

Weekly Level, Velocity and Flow Hydrographs

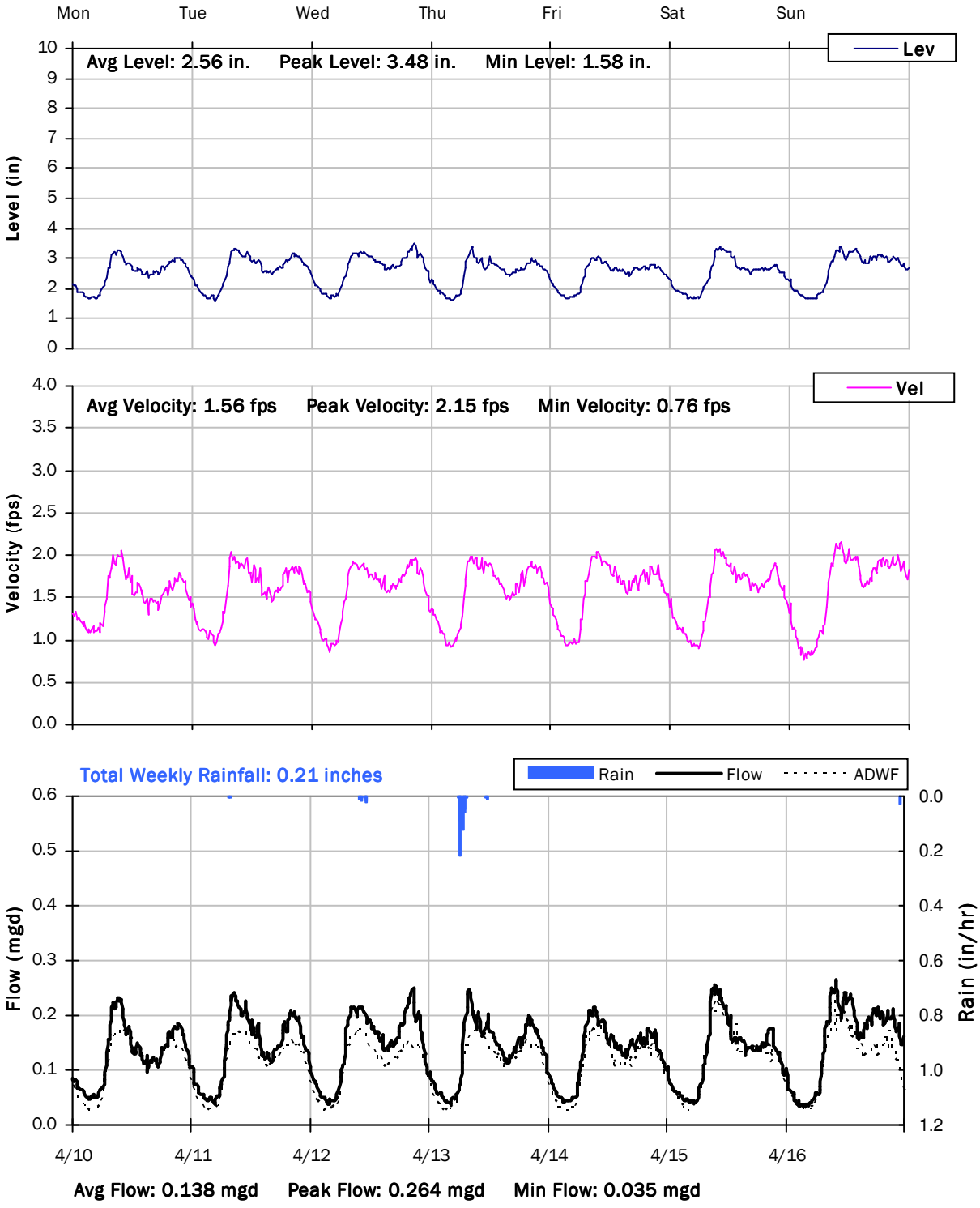
4/3/2017 to 4/10/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

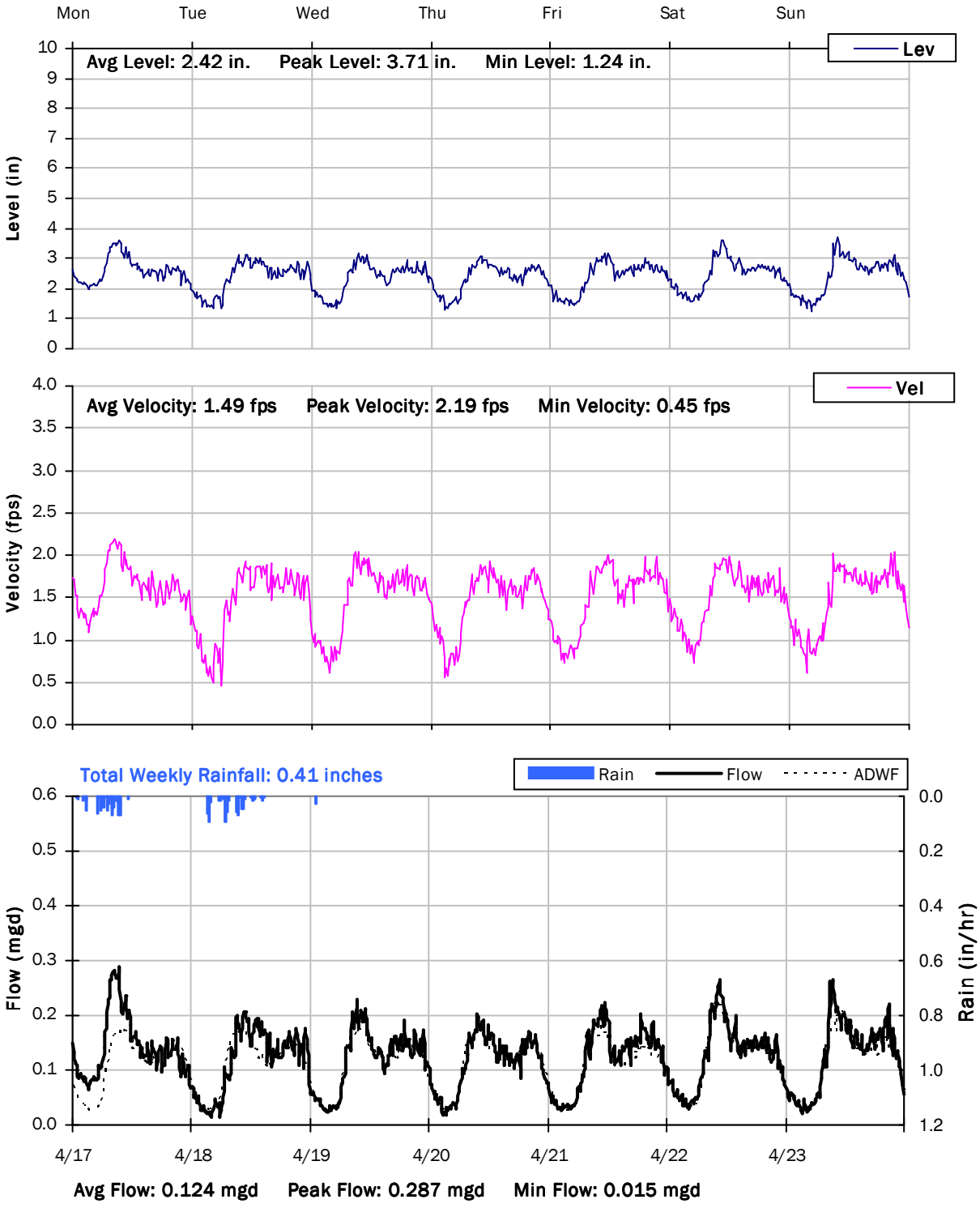
4/10/2017 to 4/17/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

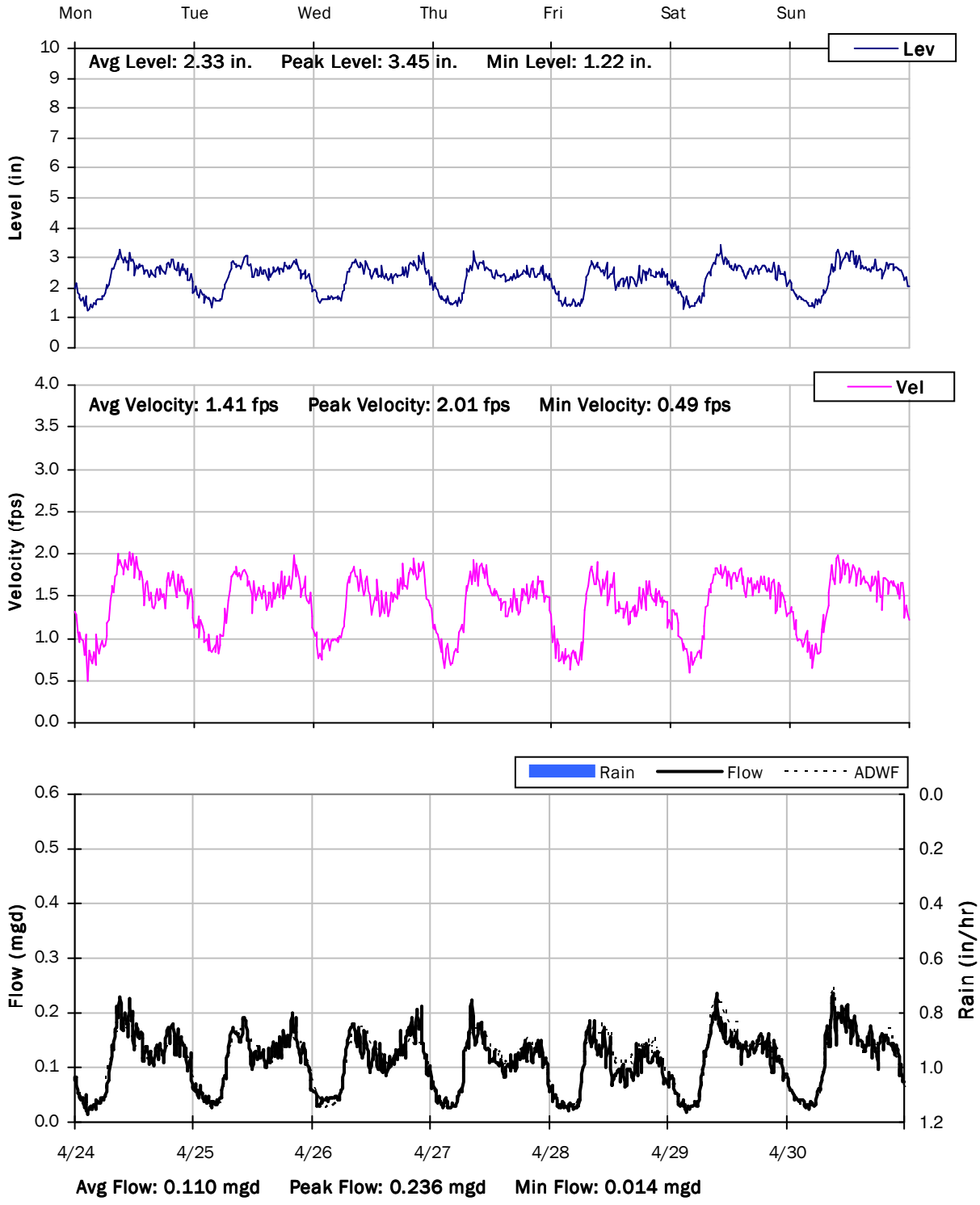
4/17/2017 to 4/24/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

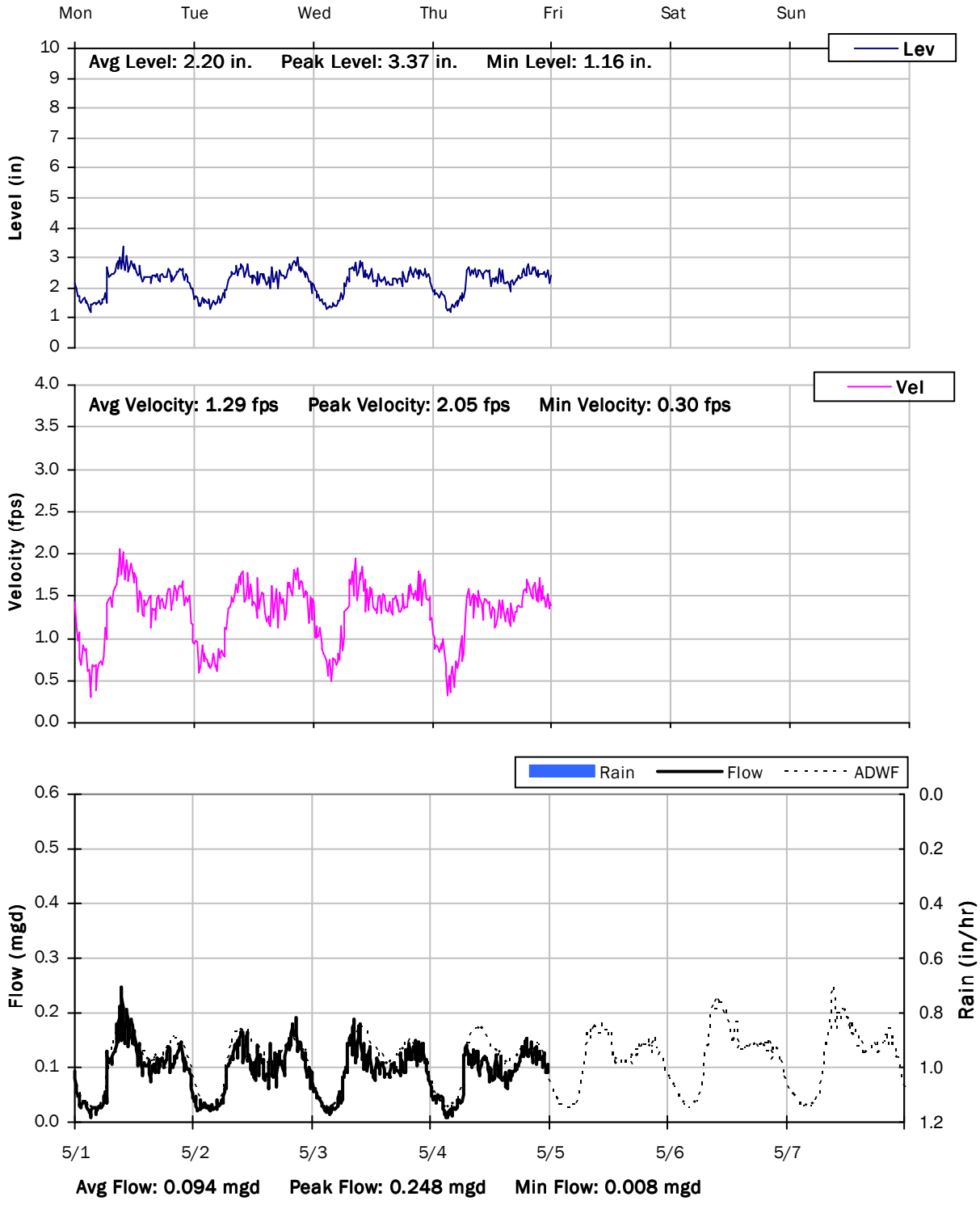
4/24/2017 to 5/1/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

5/1/2017 to 5/8/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

5/8/2017 to 5/15/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 06

Weekly Level, Velocity and Flow Hydrographs

5/15/2017 to 5/22/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 06

Weekly Level, Velocity and Flow Hydrographs

5/22/2017 to 5/29/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 06

Weekly Level, Velocity and Flow Hydrographs

5/29/2017 to 6/5/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 06

Weekly Level, Velocity and Flow Hydrographs

6/5/2017 to 6/12/2017

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 06

Weekly Level, Velocity and Flow Hydrographs

6/12/2017 to 6/19/2017

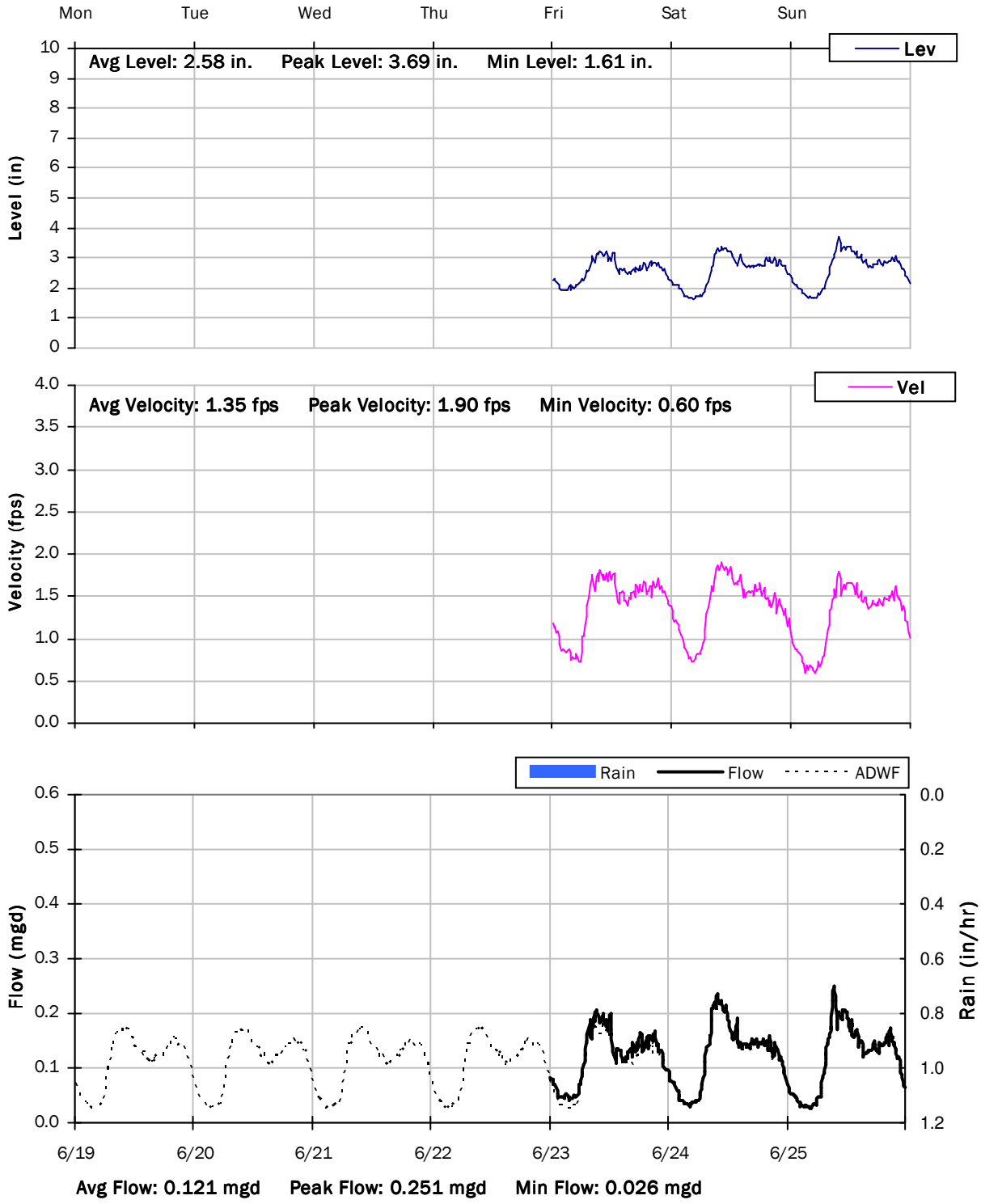
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 06

Weekly Level, Velocity and Flow Hydrographs

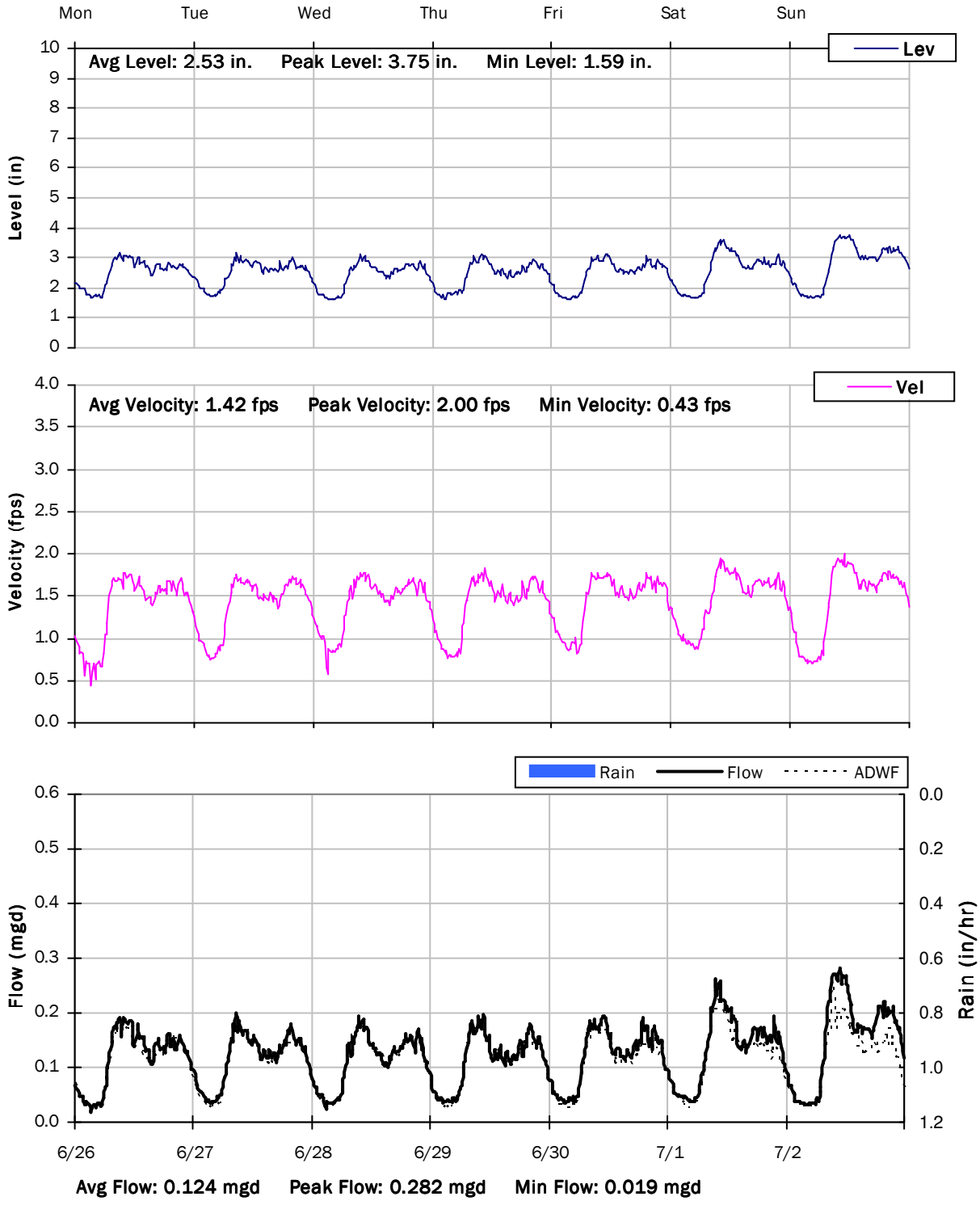
6/19/2017 to 6/26/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

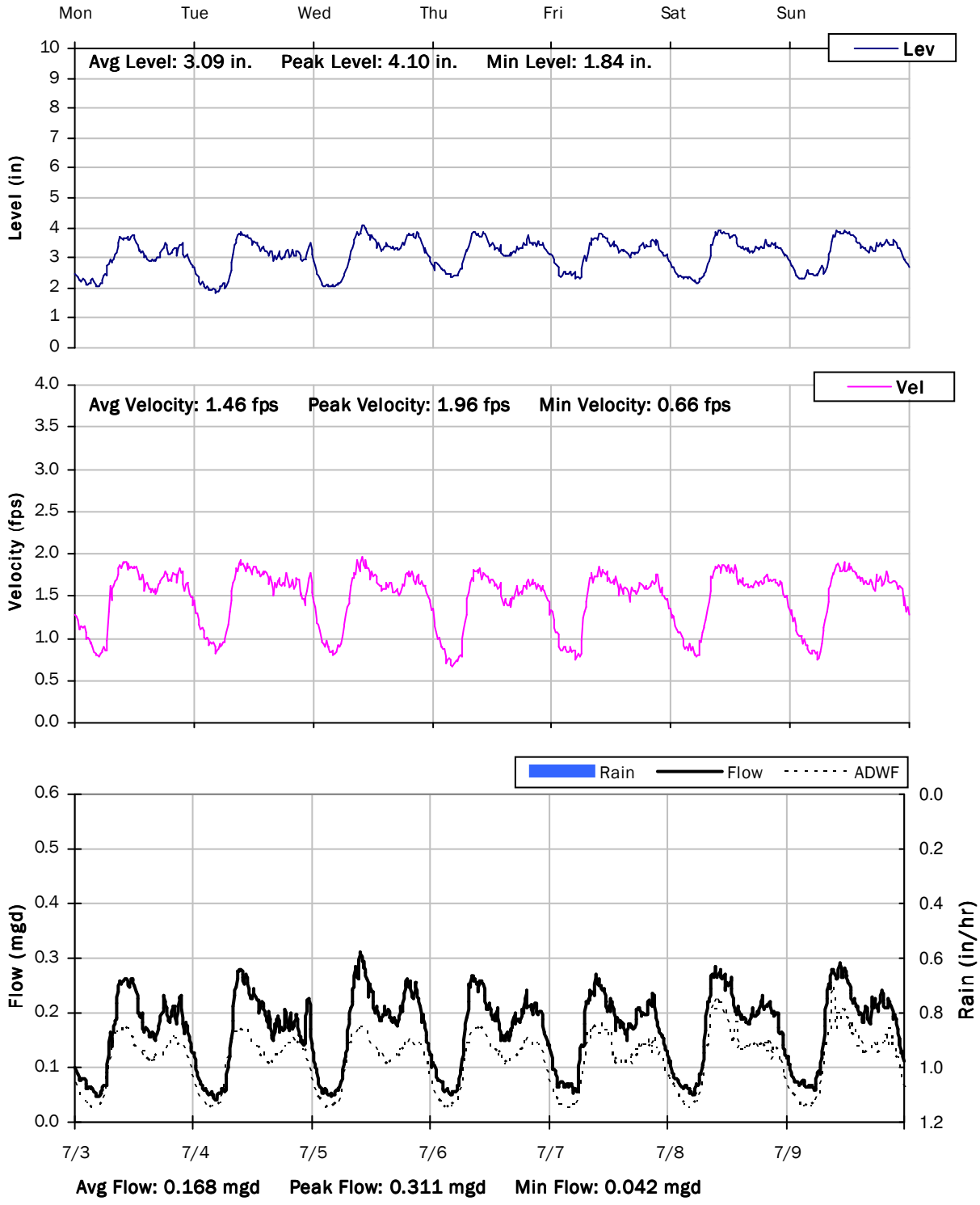
6/26/2017 to 7/3/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

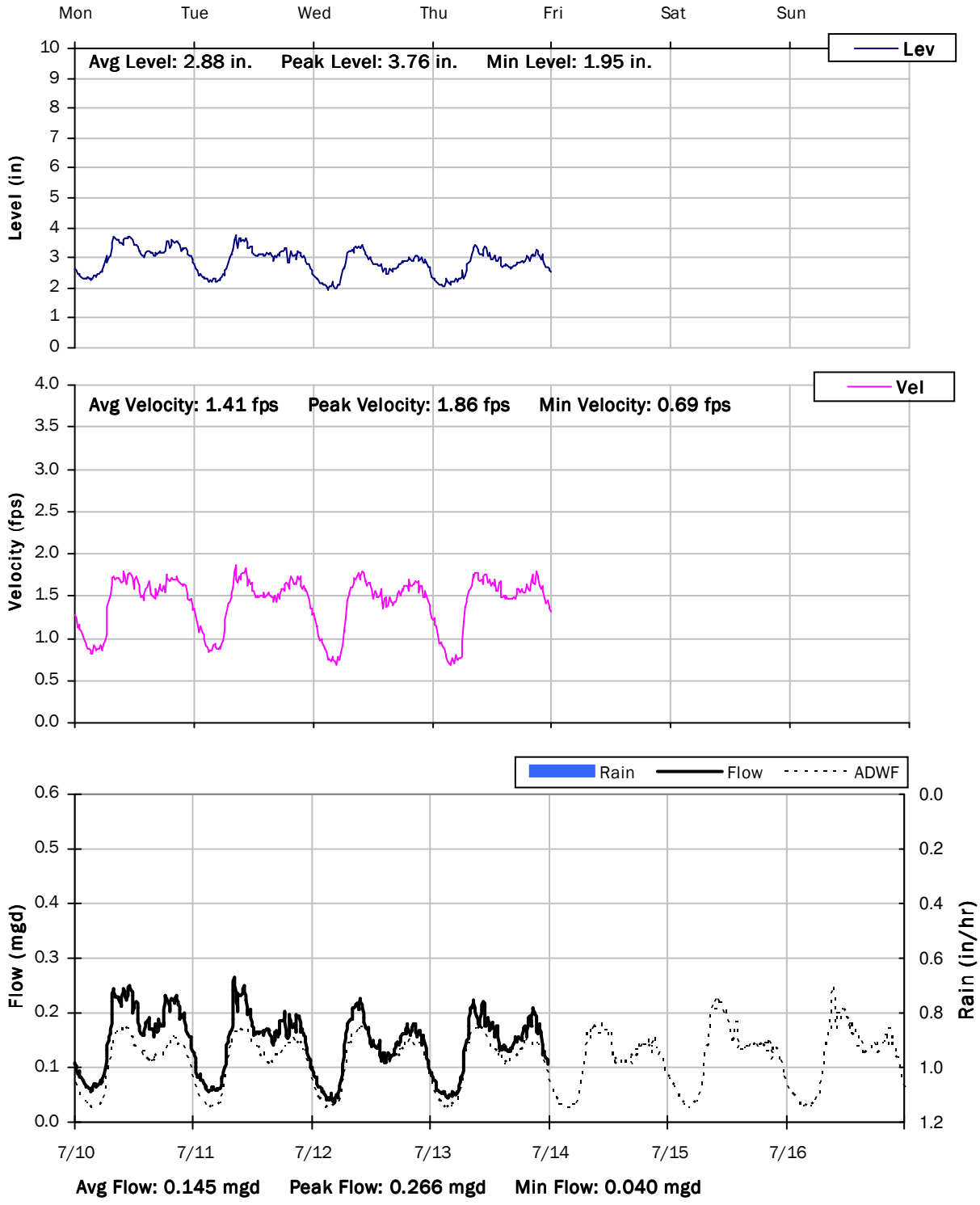
7/3/2017 to 7/10/2017



SITE 06

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

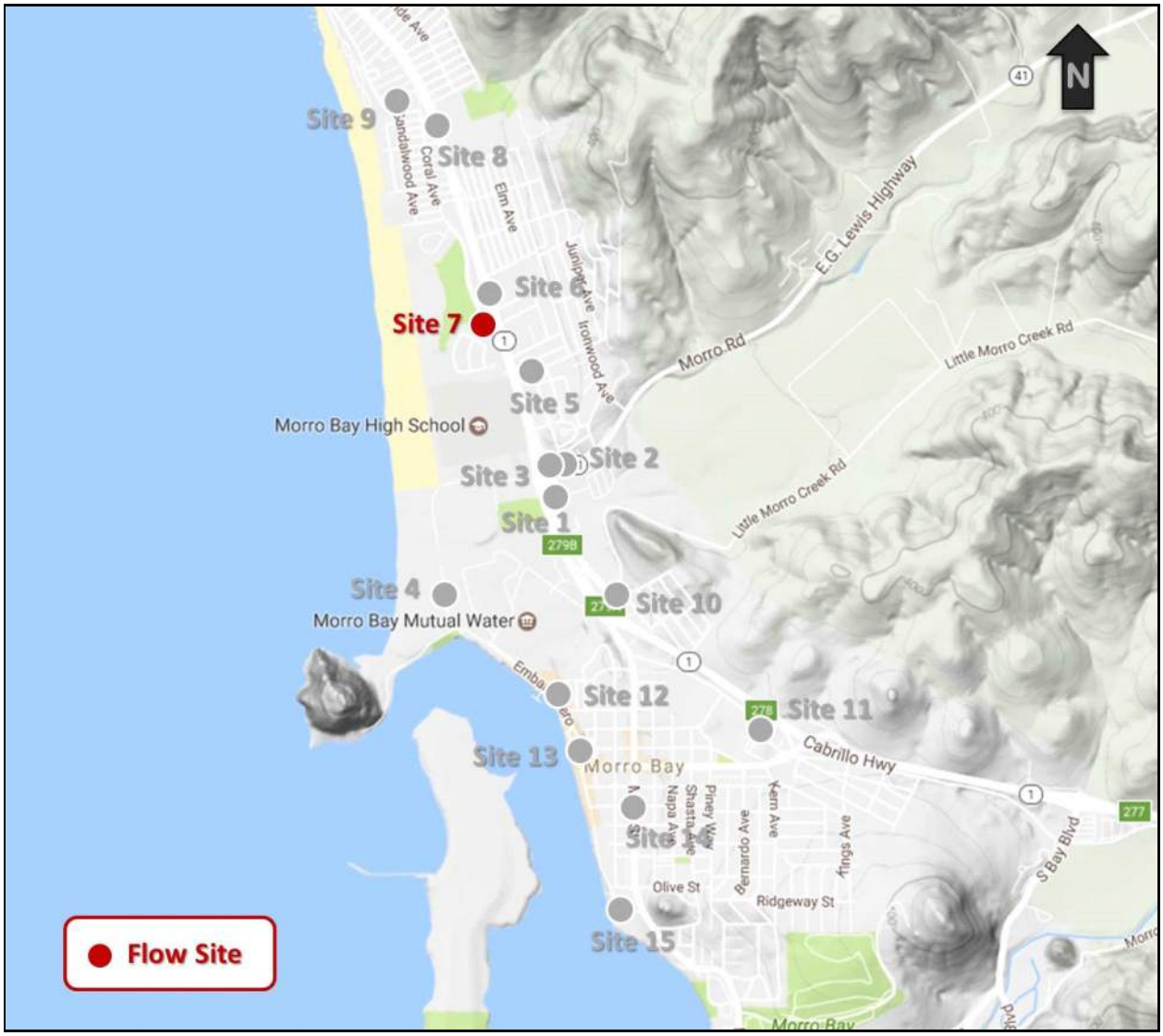
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 07

Location: Coral Avenue, ~630 feet north of Emerald Circle, meter on north 9.75" inlet

Data Summary Report



Vicinity Map: Site 07

SITE 07

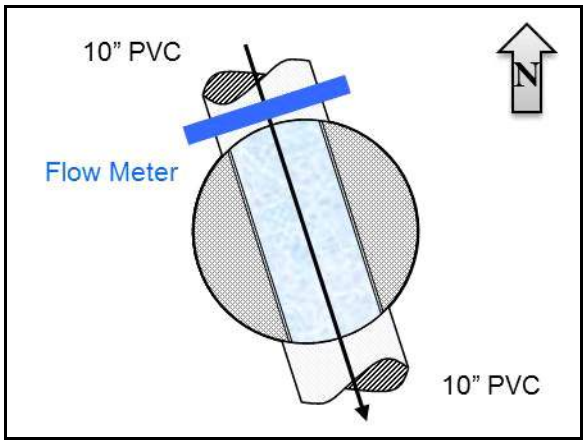
Site Information

Location: Coral Avenue, ~630 feet north of Emerald Circle, meter on north 9.75" inlet

Pipe Diameter: 9.75 inches

ADWF: 0.040 mgd

Peak Measured Flow: 0.164 mgd



SITE 07

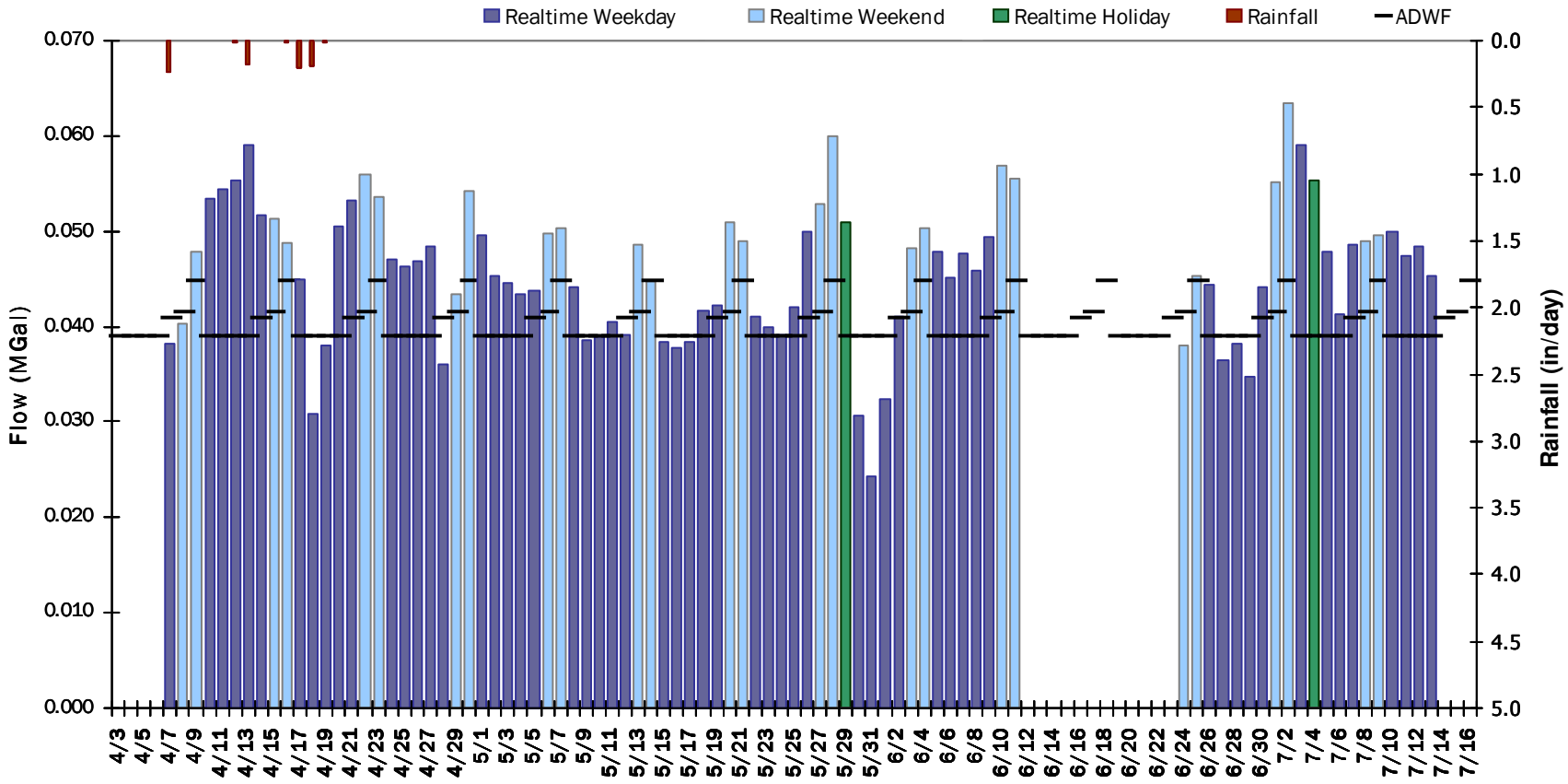
Additional Site Photos

SITE 07

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.046 MGal Peak Daily Flow: 0.063 MGal Min Daily Flow: 0.024 MGal

Total Period Rainfall: 0.85 inches

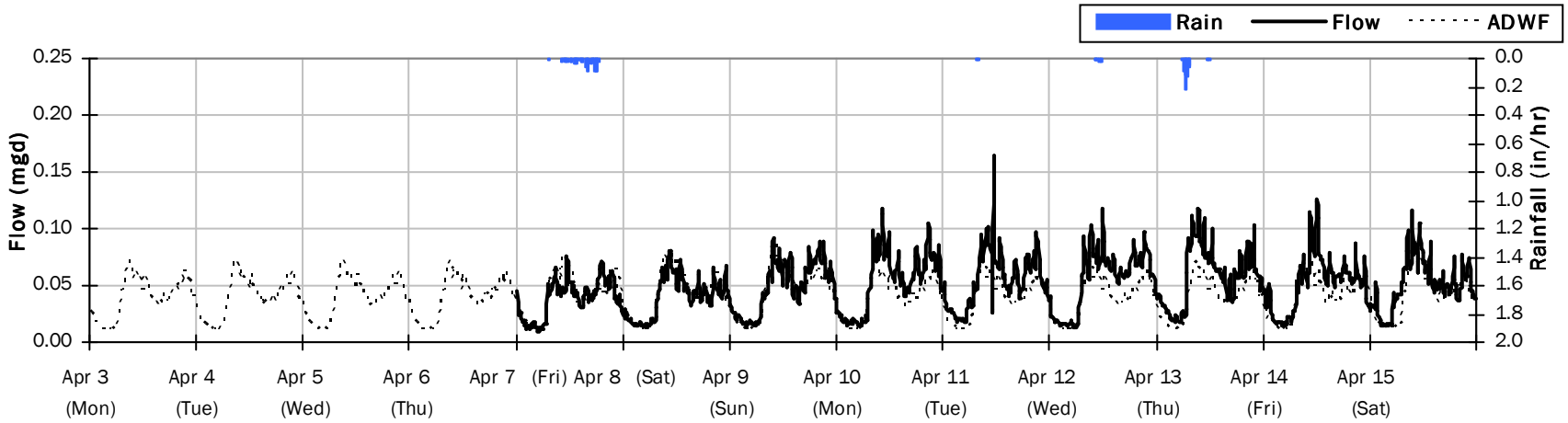


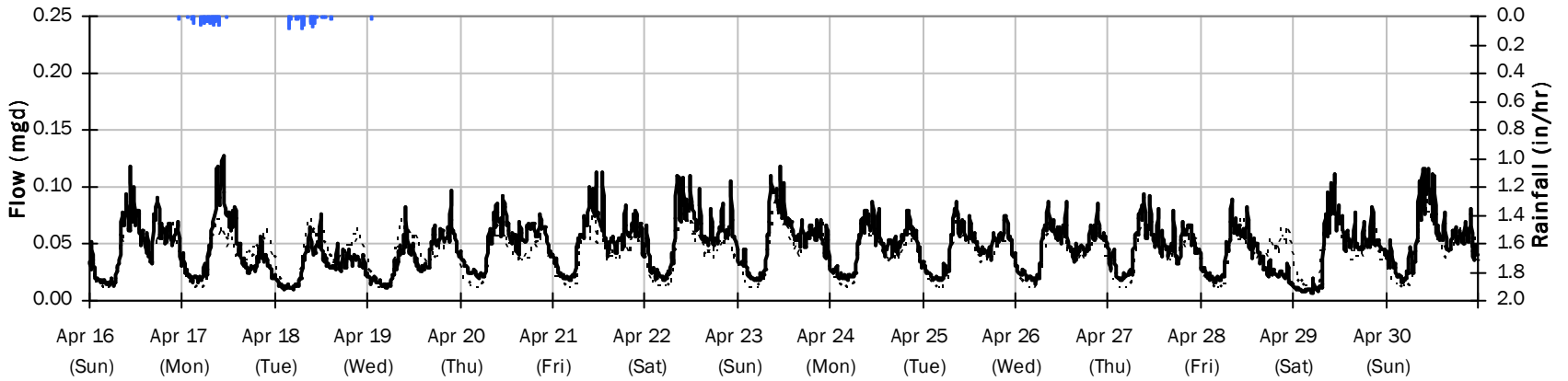
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 07

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.048 mgd** **Peak Flow: 0.164 mgd** **Min Flow: 0.006 mgd**

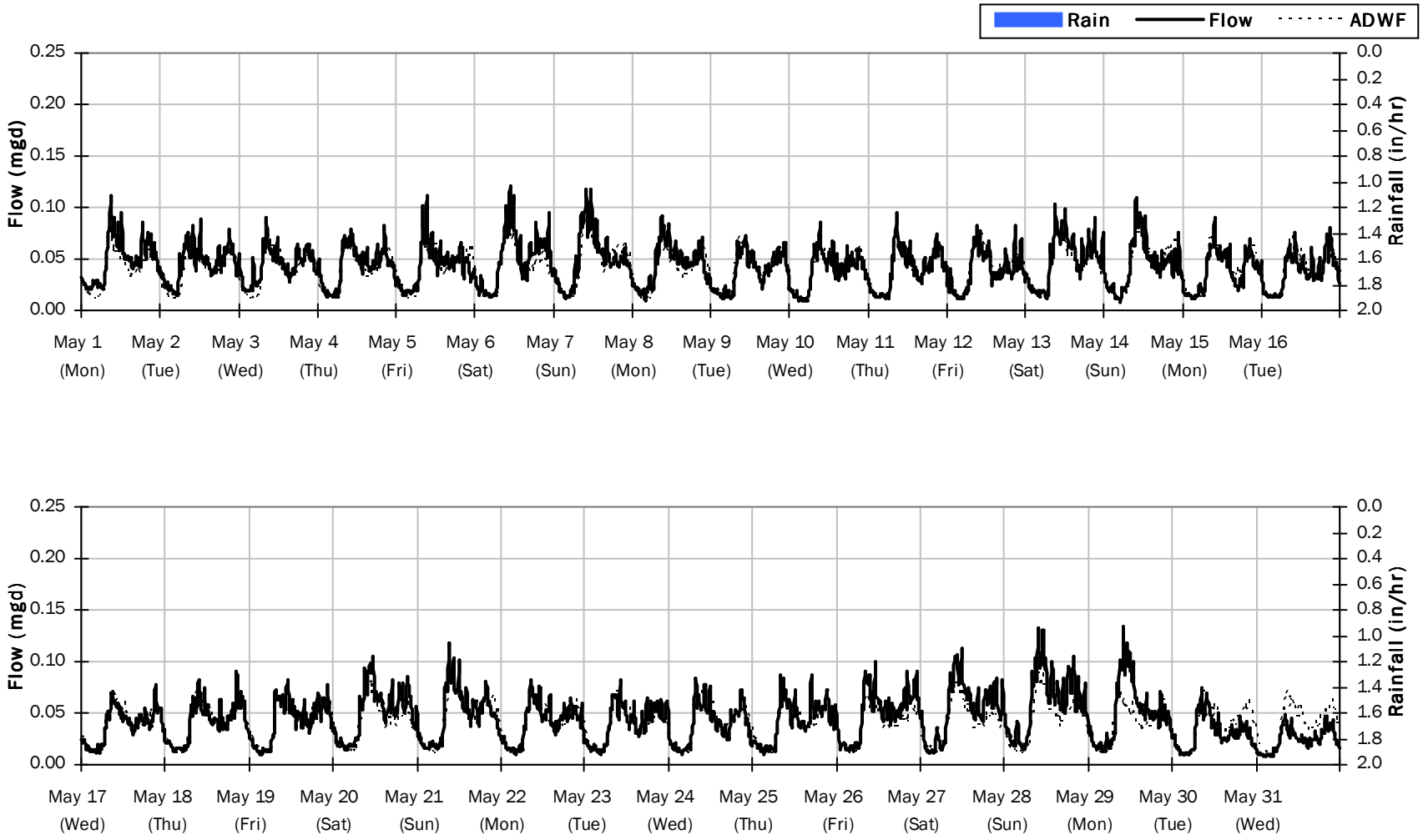




SITE 07

Flow Summary: 5/1/2017 to 5/31/2017

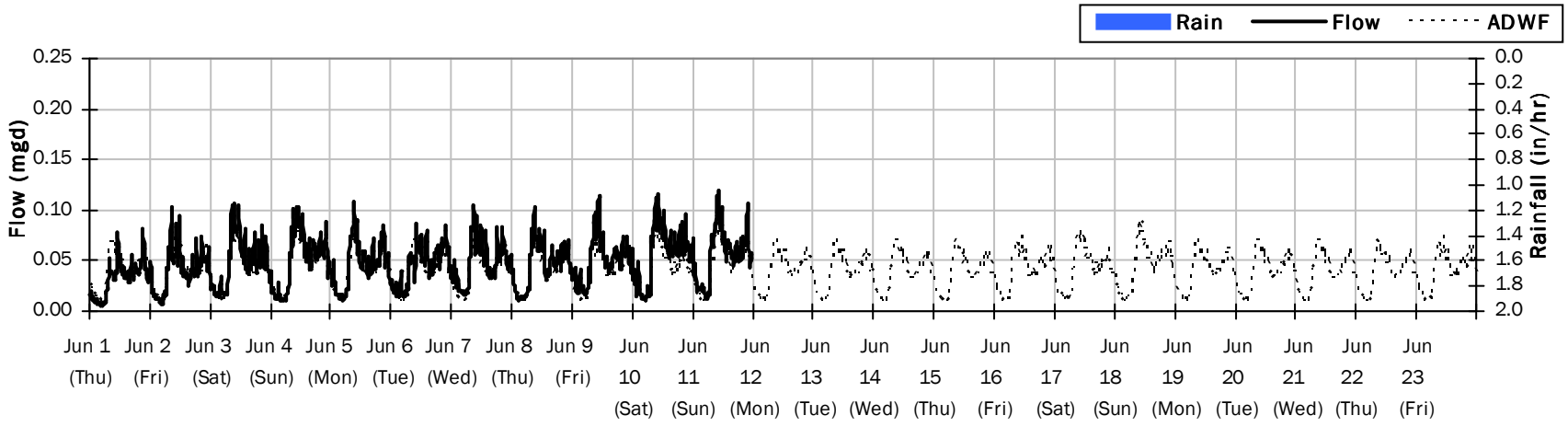
Total Period Rainfall: 0.00 inches **Avg Flow: 0.044 mgd** **Peak Flow: 0.134 mgd** **Min Flow: 0.007 mgd**

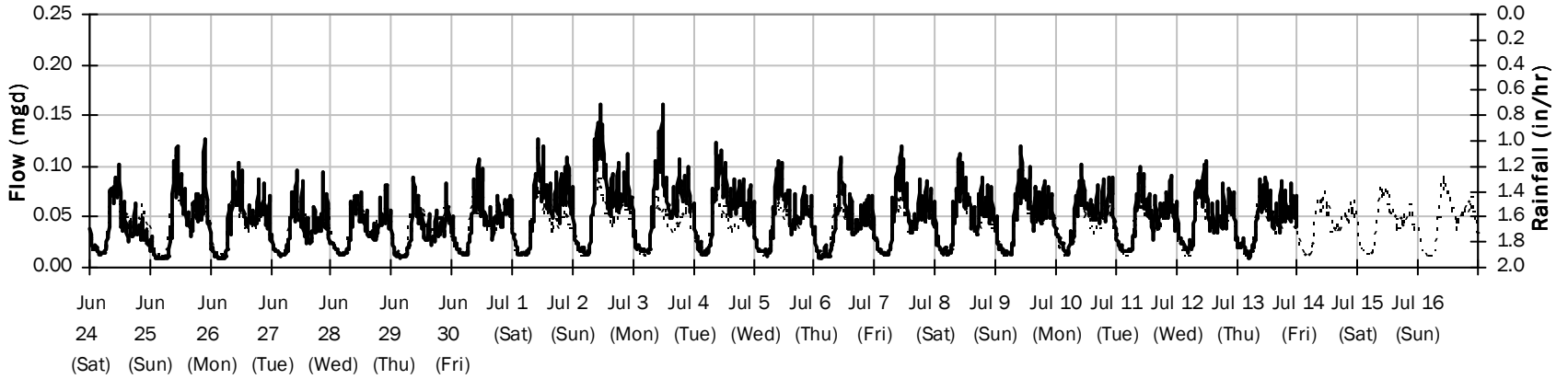


SITE 07

Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.047 mgd** **Peak Flow: 0.161 mgd** **Min Flow: 0.006 mgd**





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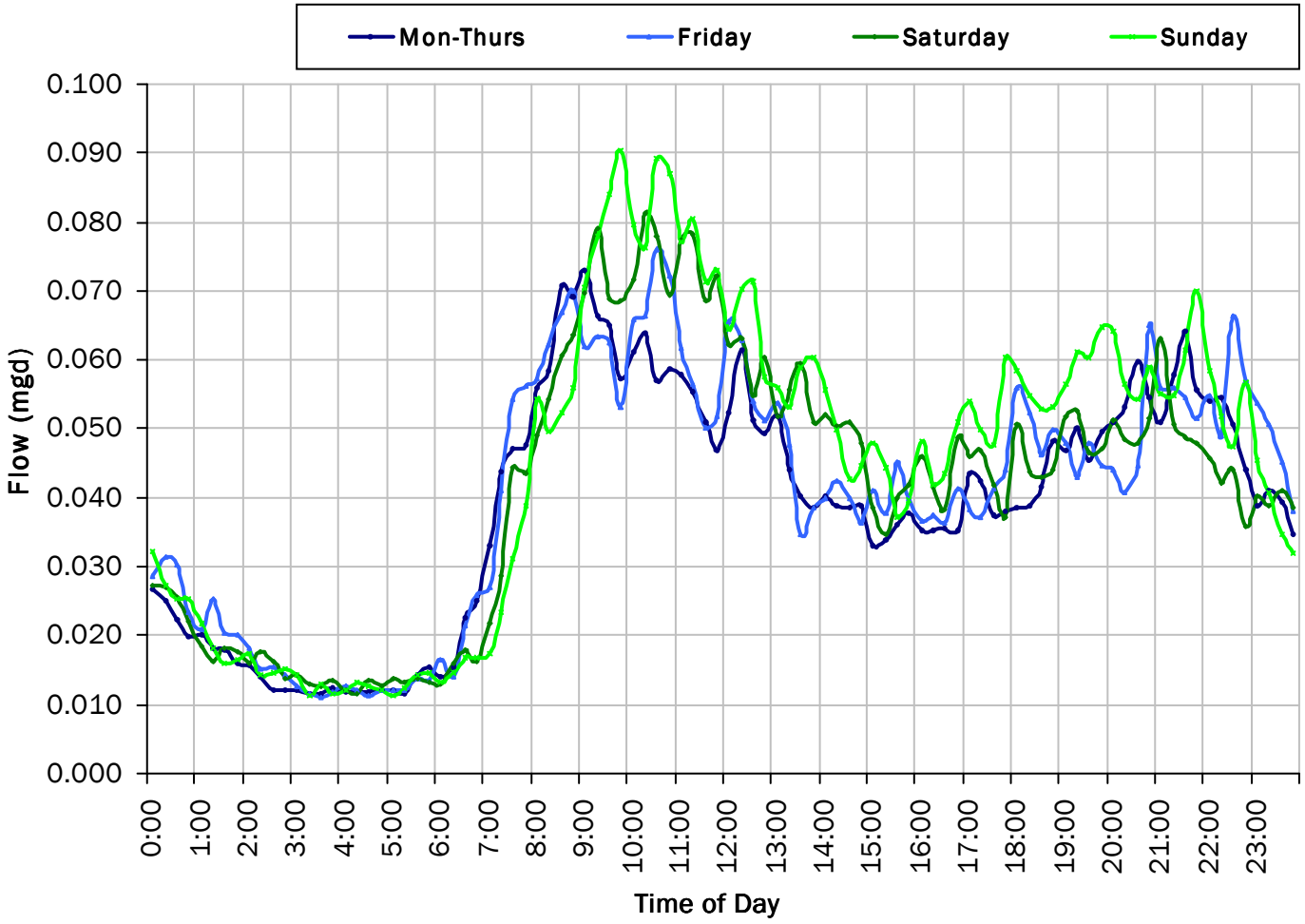
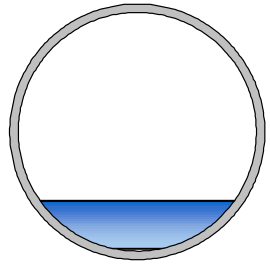
SITE 07

Average Dry Weather Flow Hydrographs



ADWF:

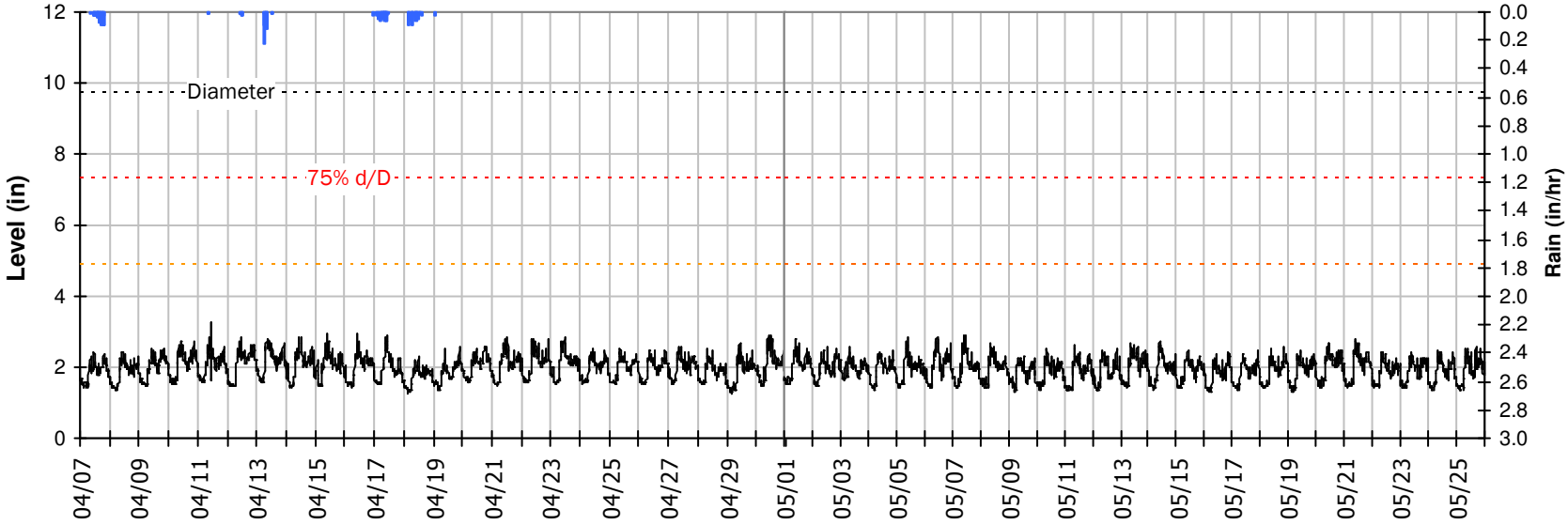
0.040 *mgd*

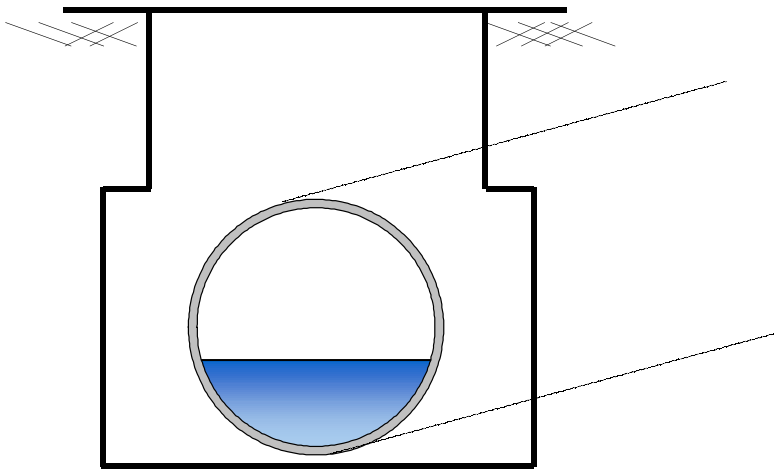


SITE 07

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



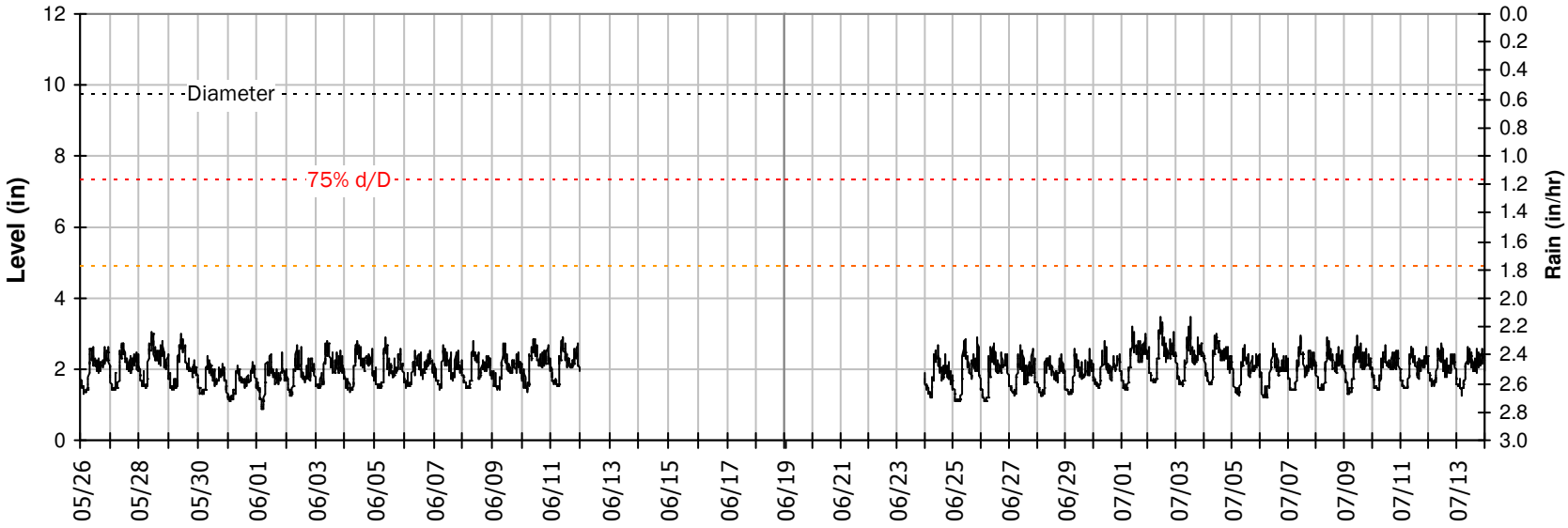


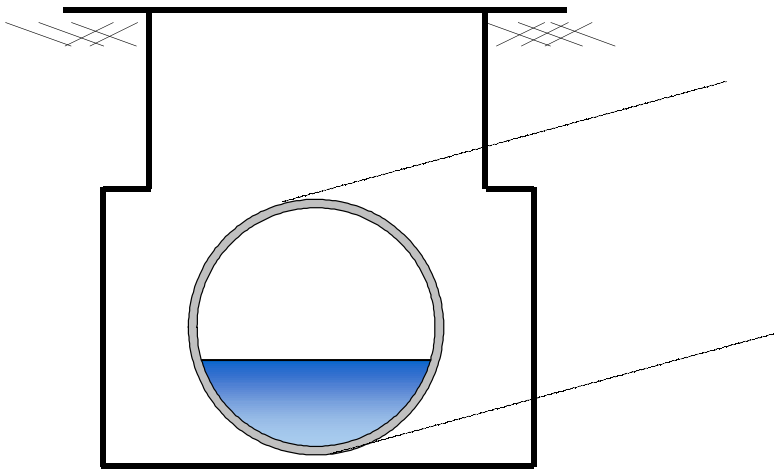
Pipe Diameter:	9.75	*inches*
Peak Measured Level:	3.5	*inches*
Peak d/D Ratio:	0.36	

SITE 07

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





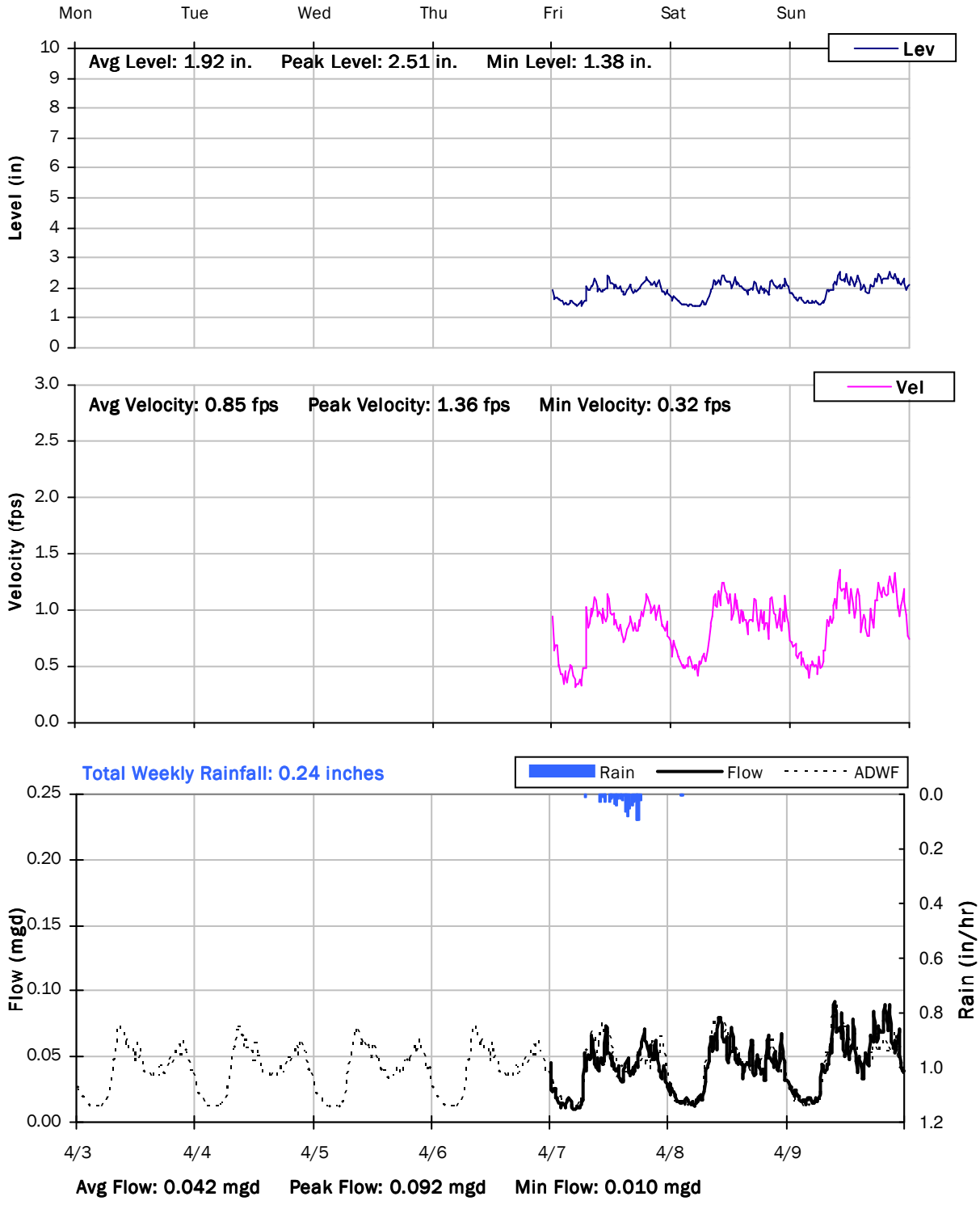
Pipe Diameter:	9.75	*inches*
Peak Measured Level:	3.5	*inches*
Peak d/D Ratio:	0.36	

As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

SITE 07

Weekly Level, Velocity and Flow Hydrographs

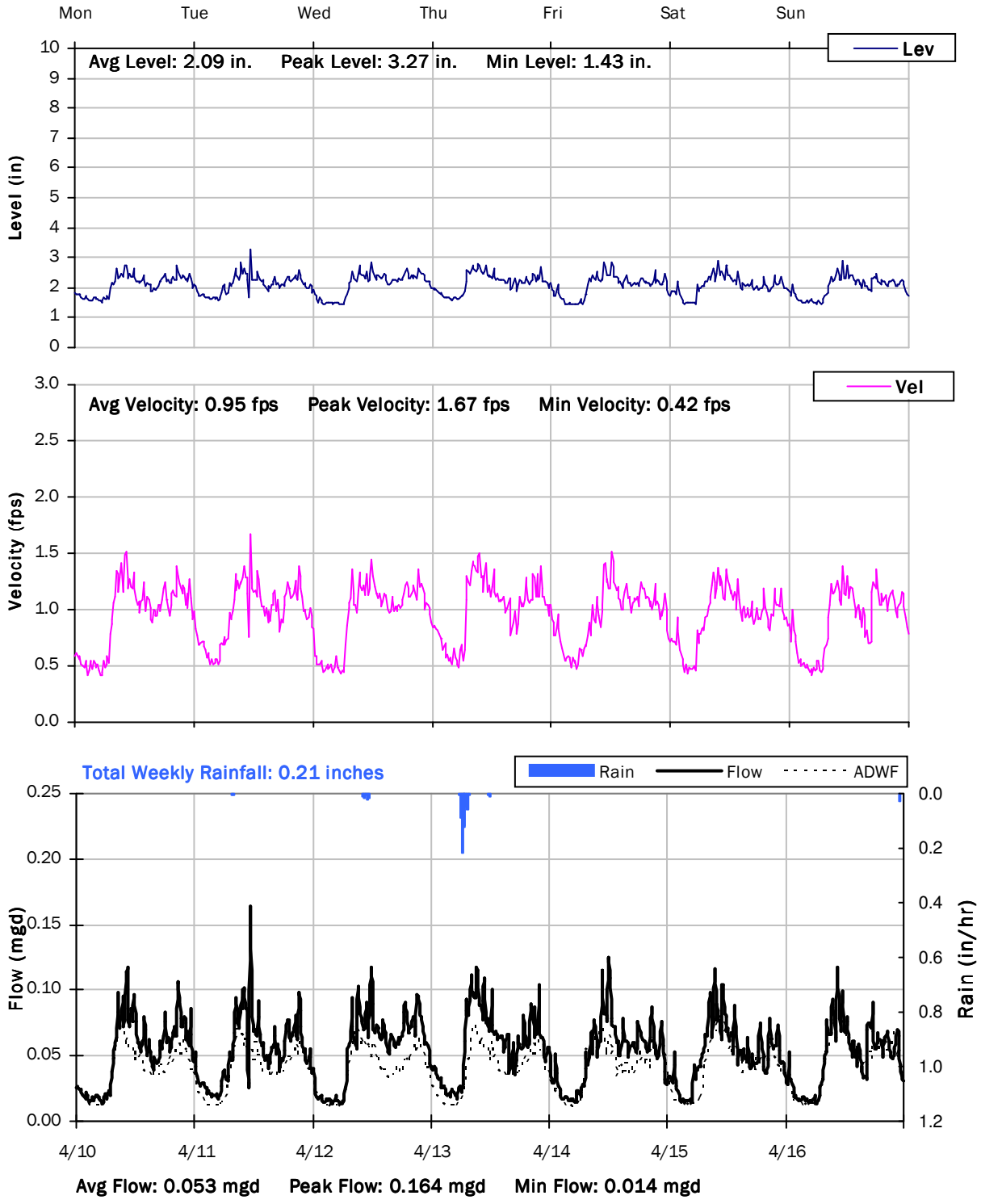
4/3/2017 to 4/10/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

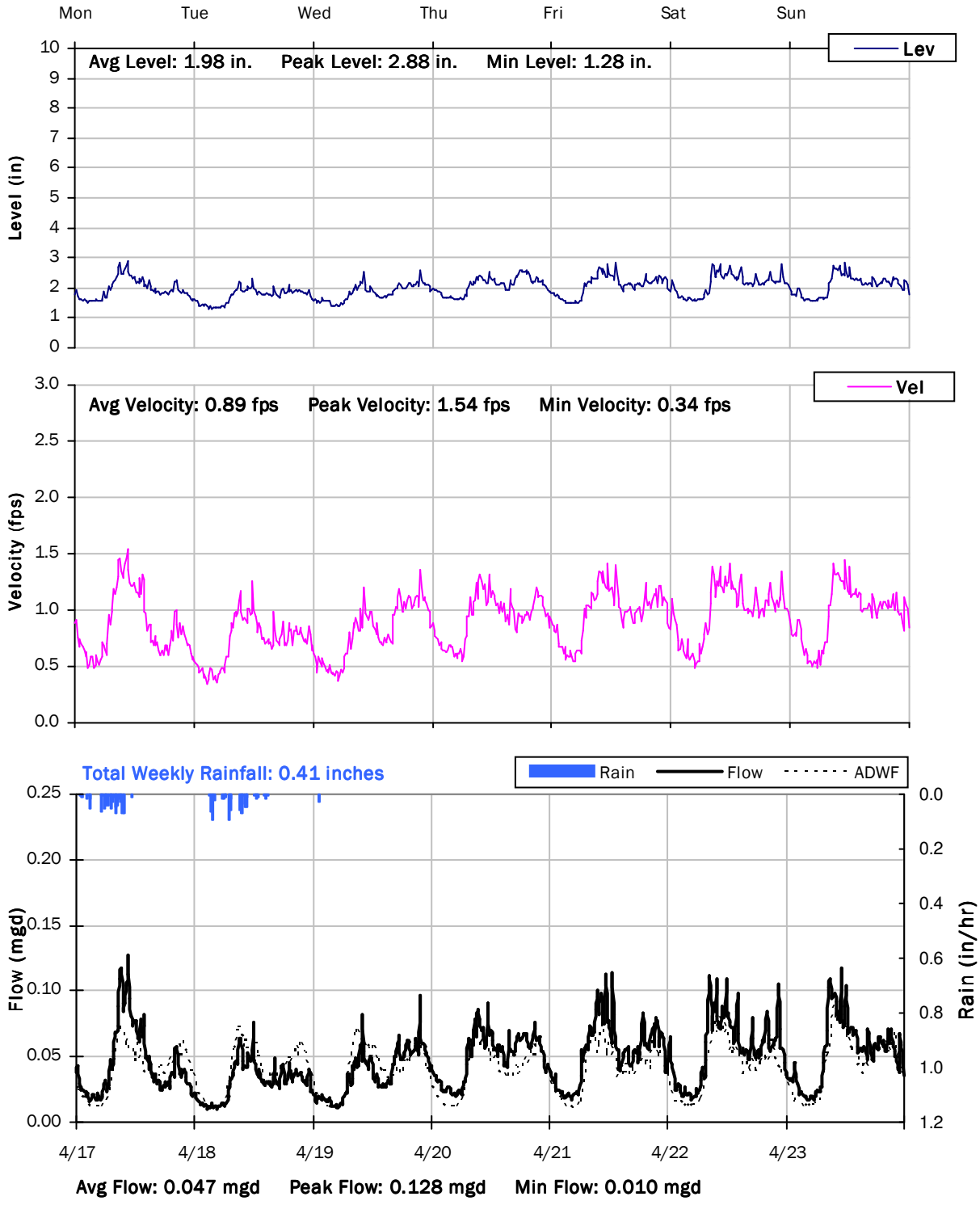
4/10/2017 to 4/17/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

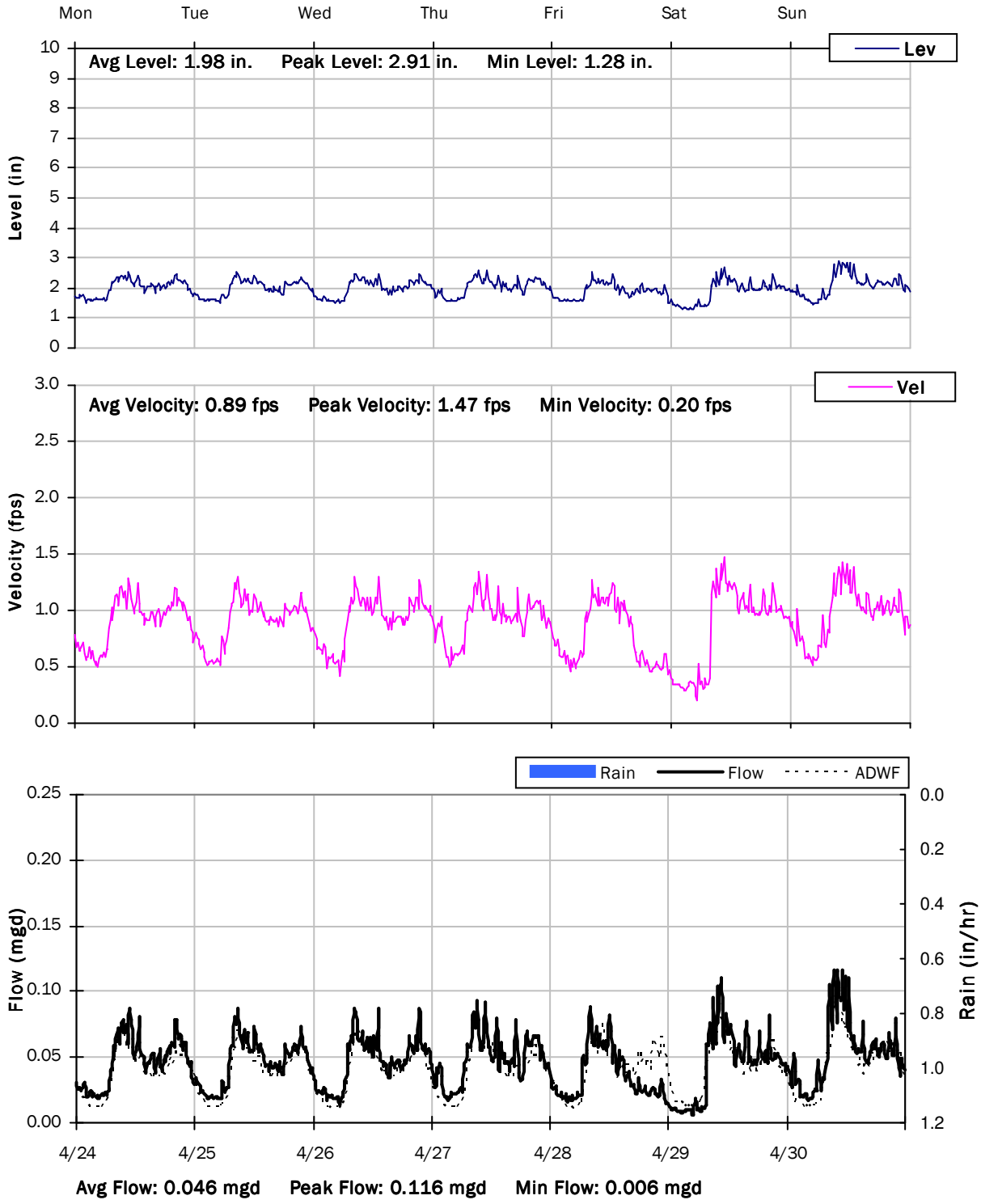
4/17/2017 to 4/24/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

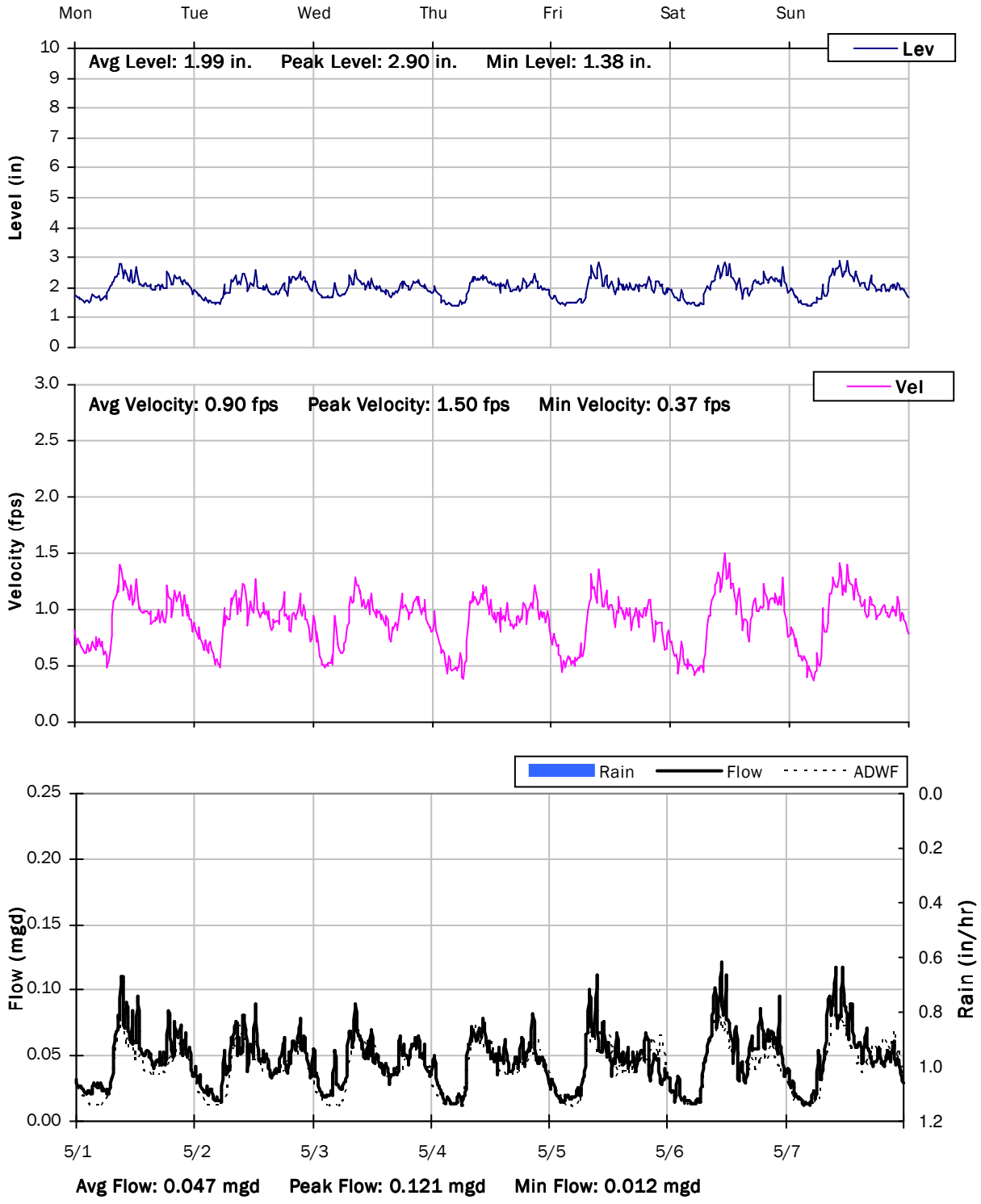
4/24/2017 to 5/1/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

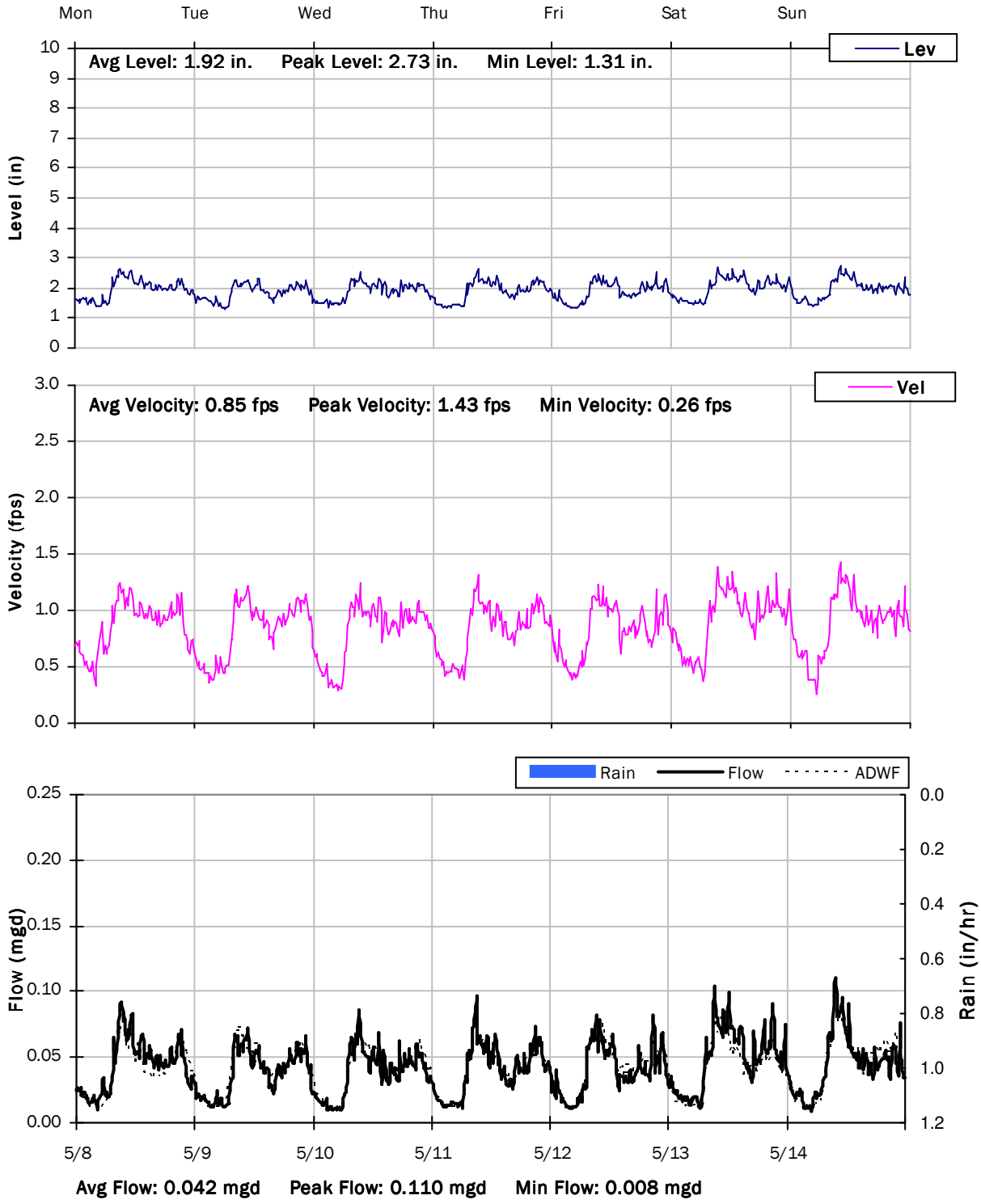
5/1/2017 to 5/8/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

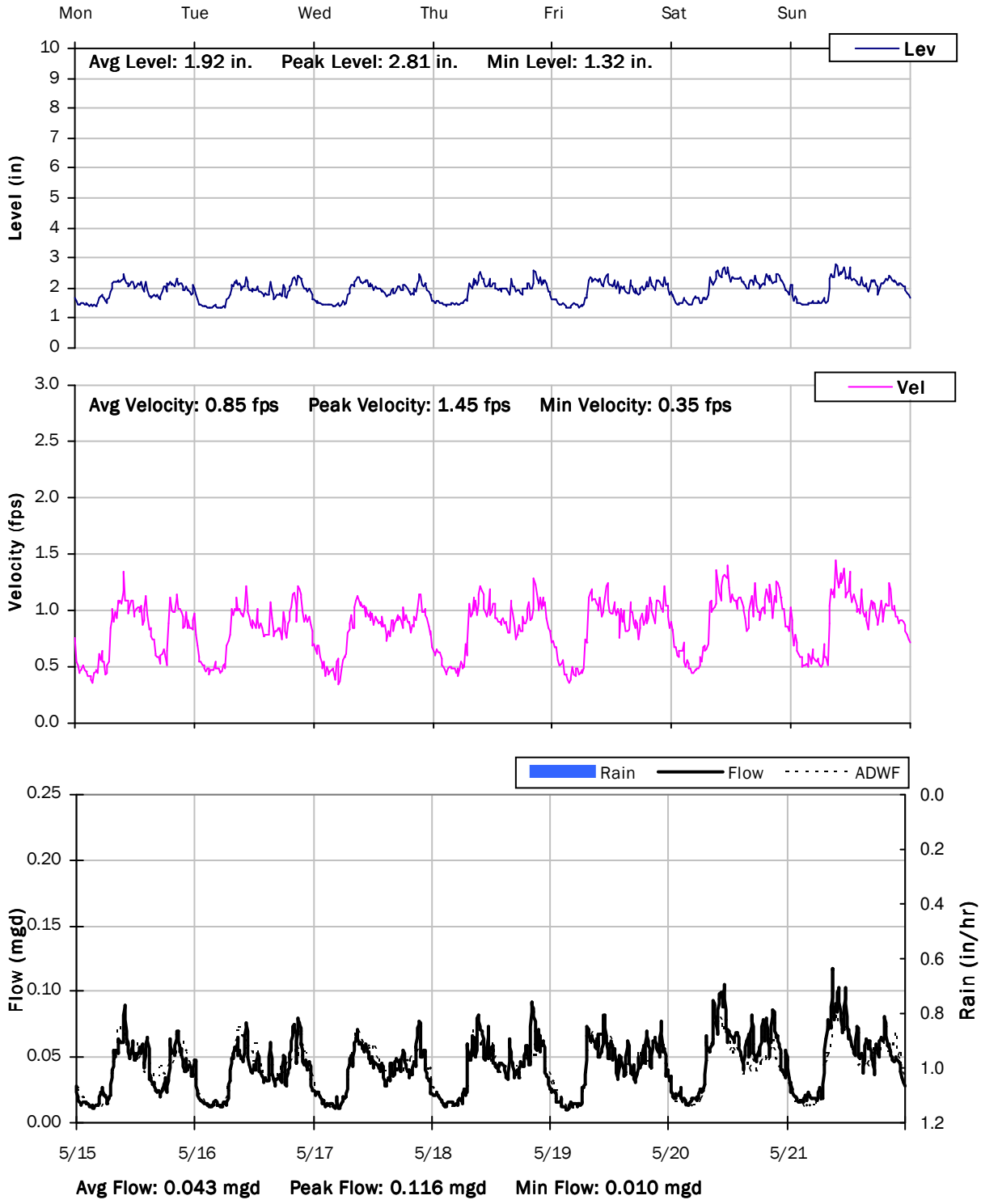
5/8/2017 to 5/15/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

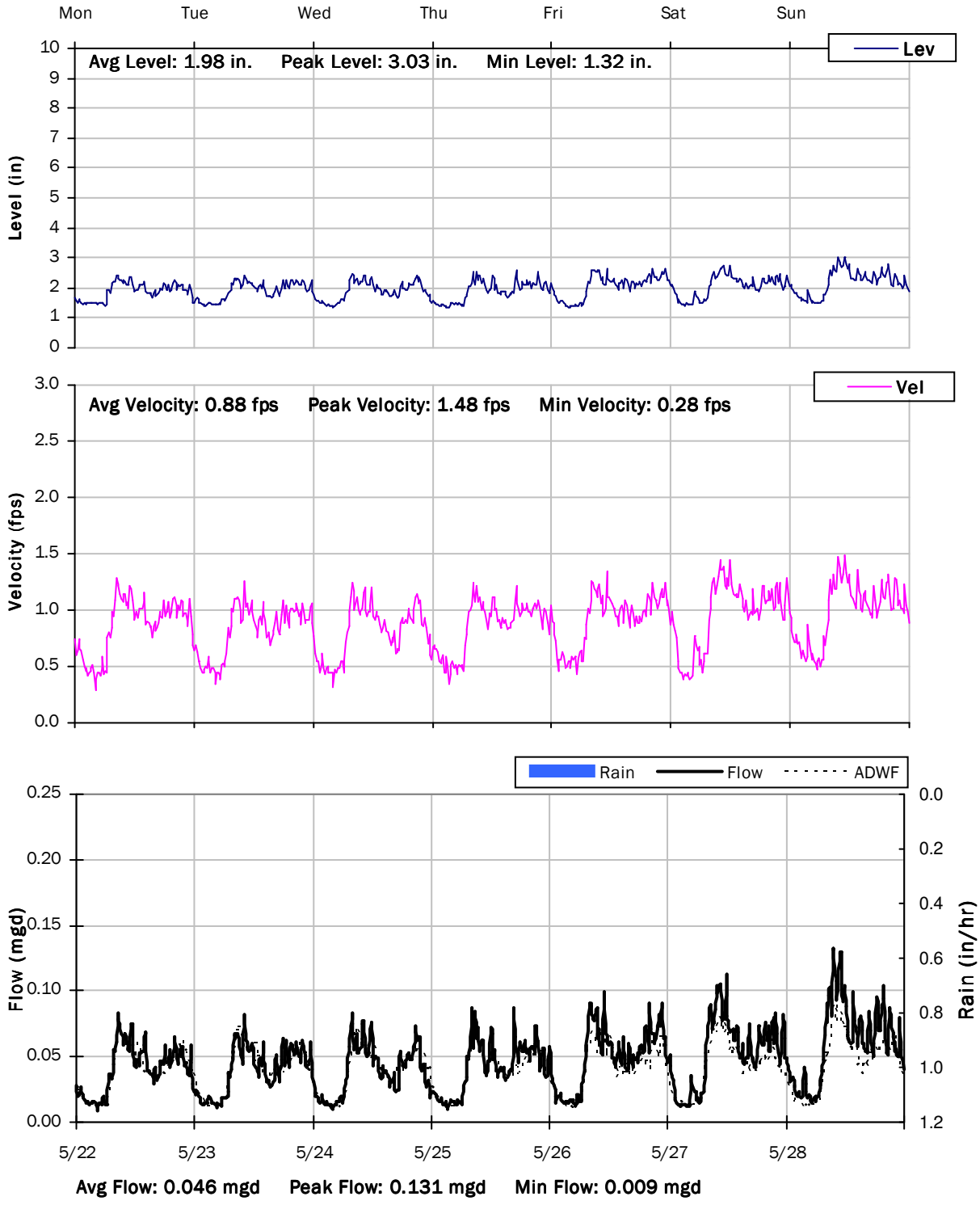
5/15/2017 to 5/22/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

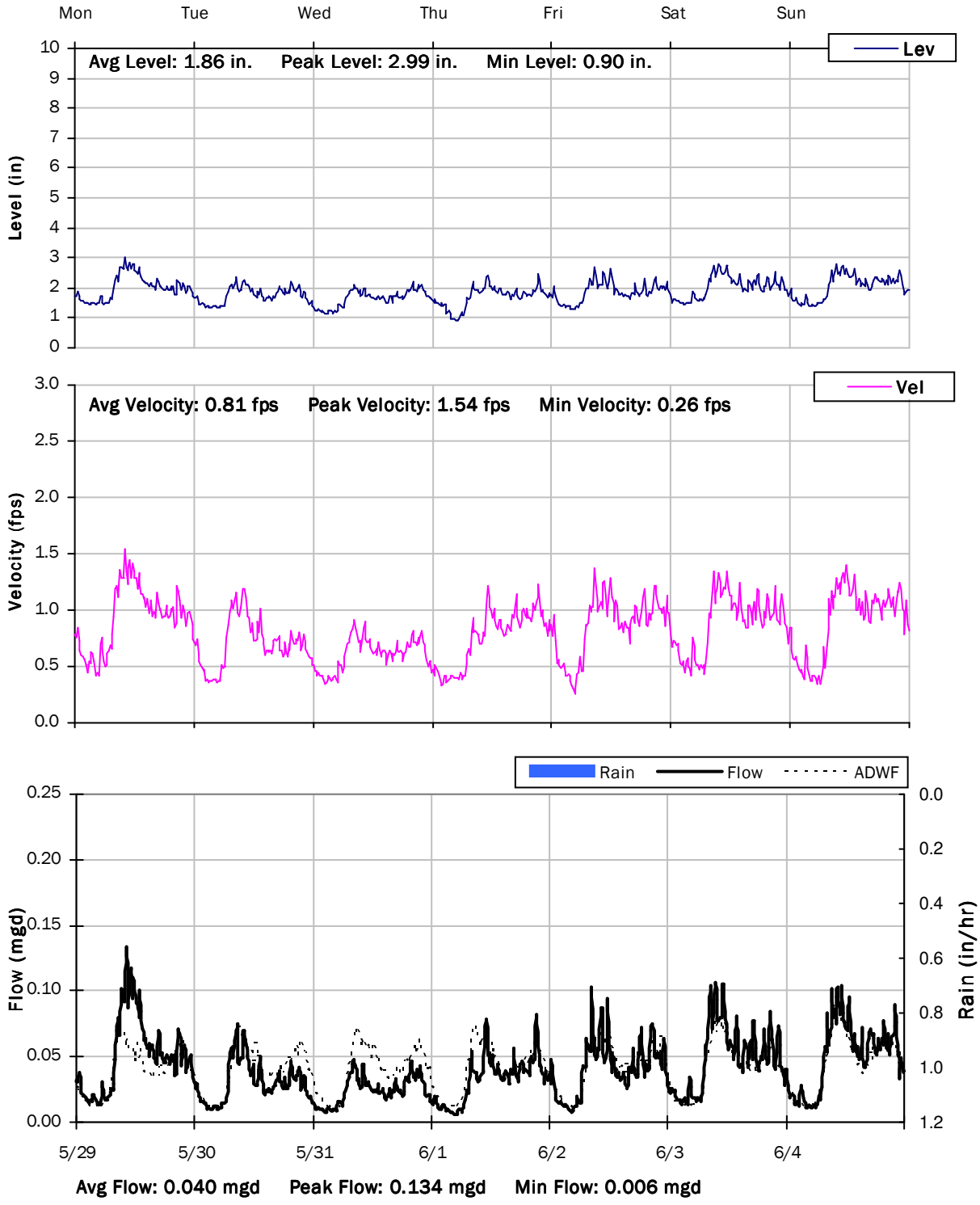
5/22/2017 to 5/29/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

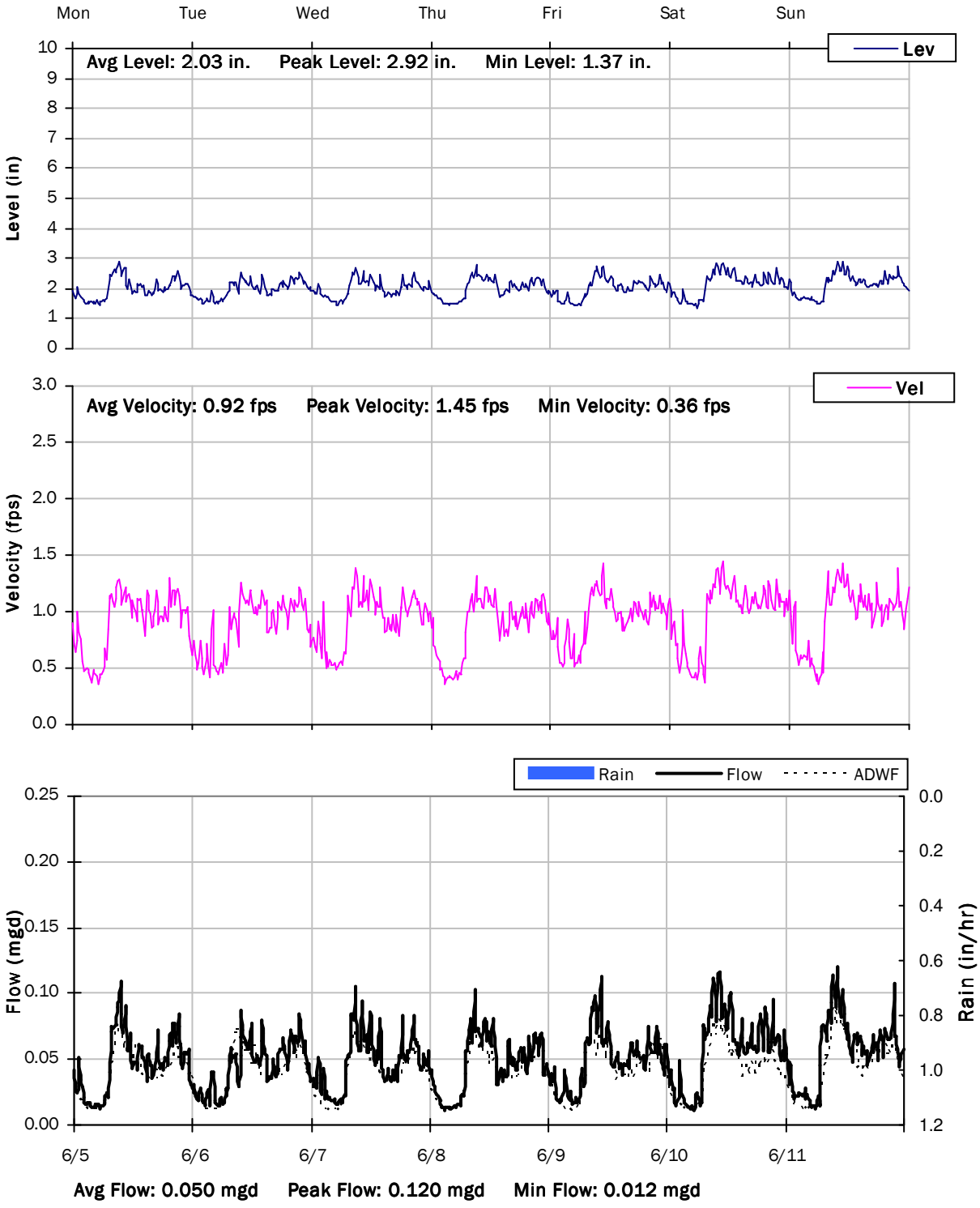
5/29/2017 to 6/5/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

6/5/2017 to 6/12/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

6/12/2017 to 6/19/2017

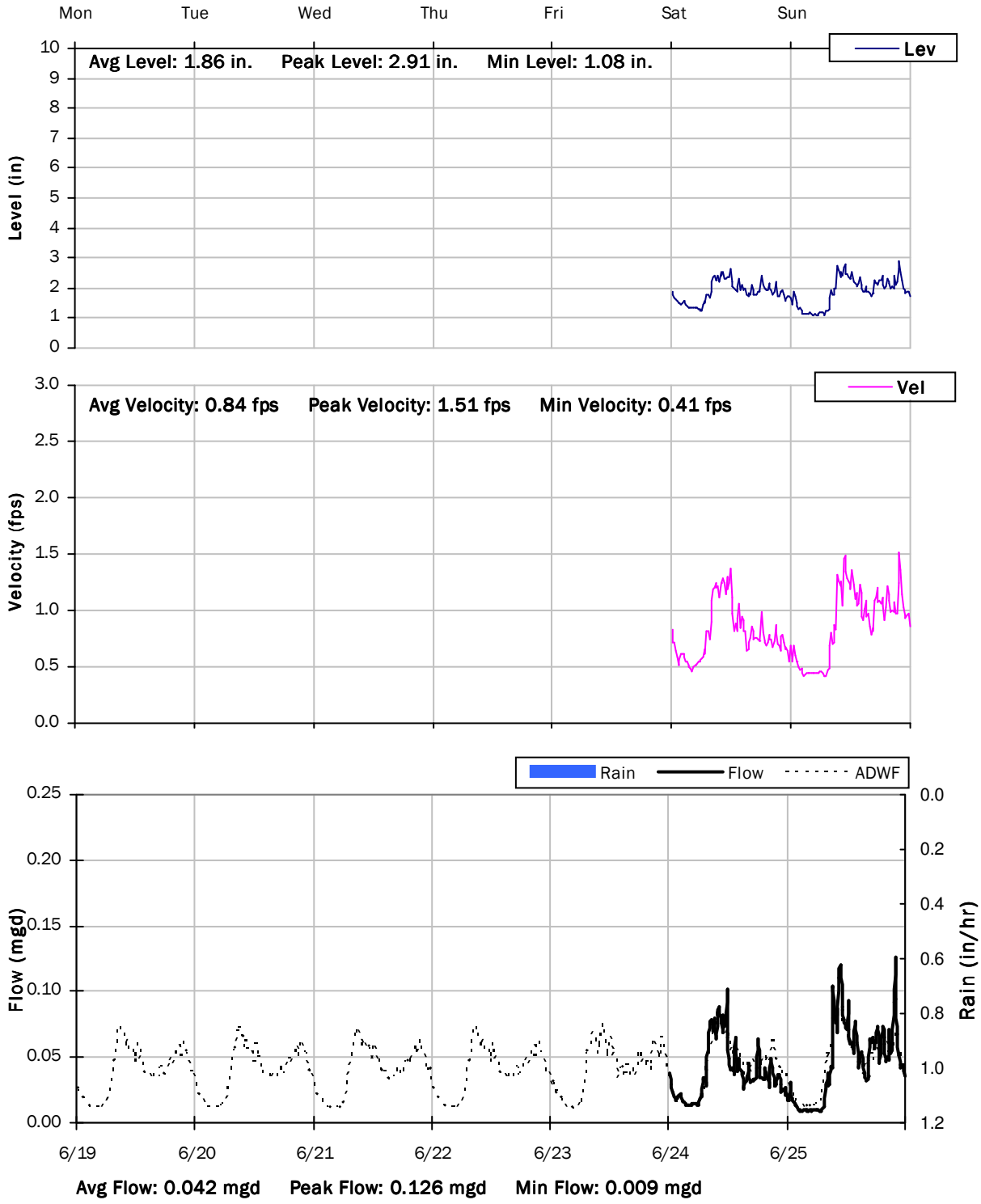
As a part of the scope of work, flow meters were intermittently and permissibly suspended in place, without maintenance, throughout this study. Data gaps indicate unreportable data due to the suspended and unmaintained state of the flow meter.

No reportable data this week.

SITE 07

Weekly Level, Velocity and Flow Hydrographs

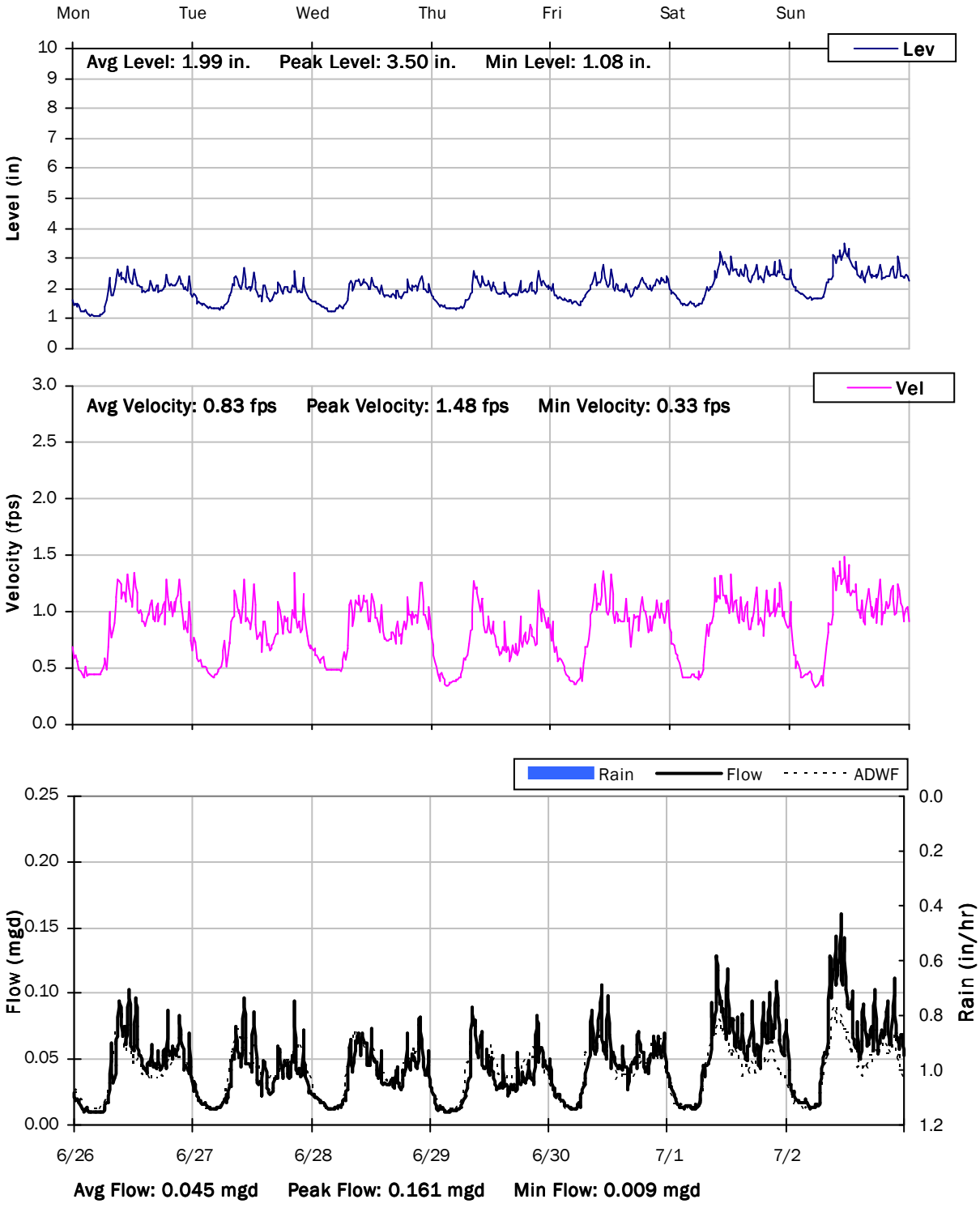
6/19/2017 to 6/26/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

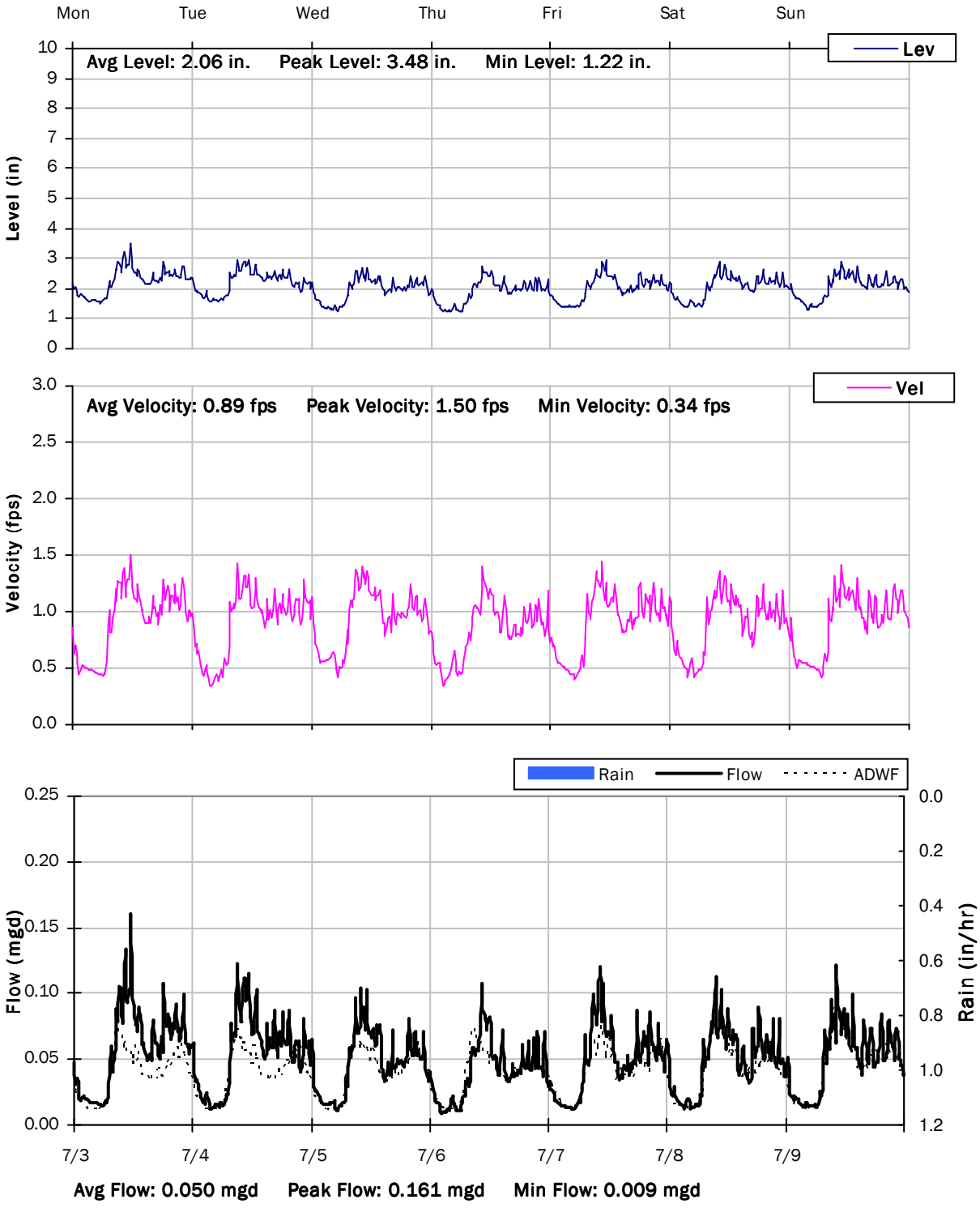
6/26/2017 to 7/3/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

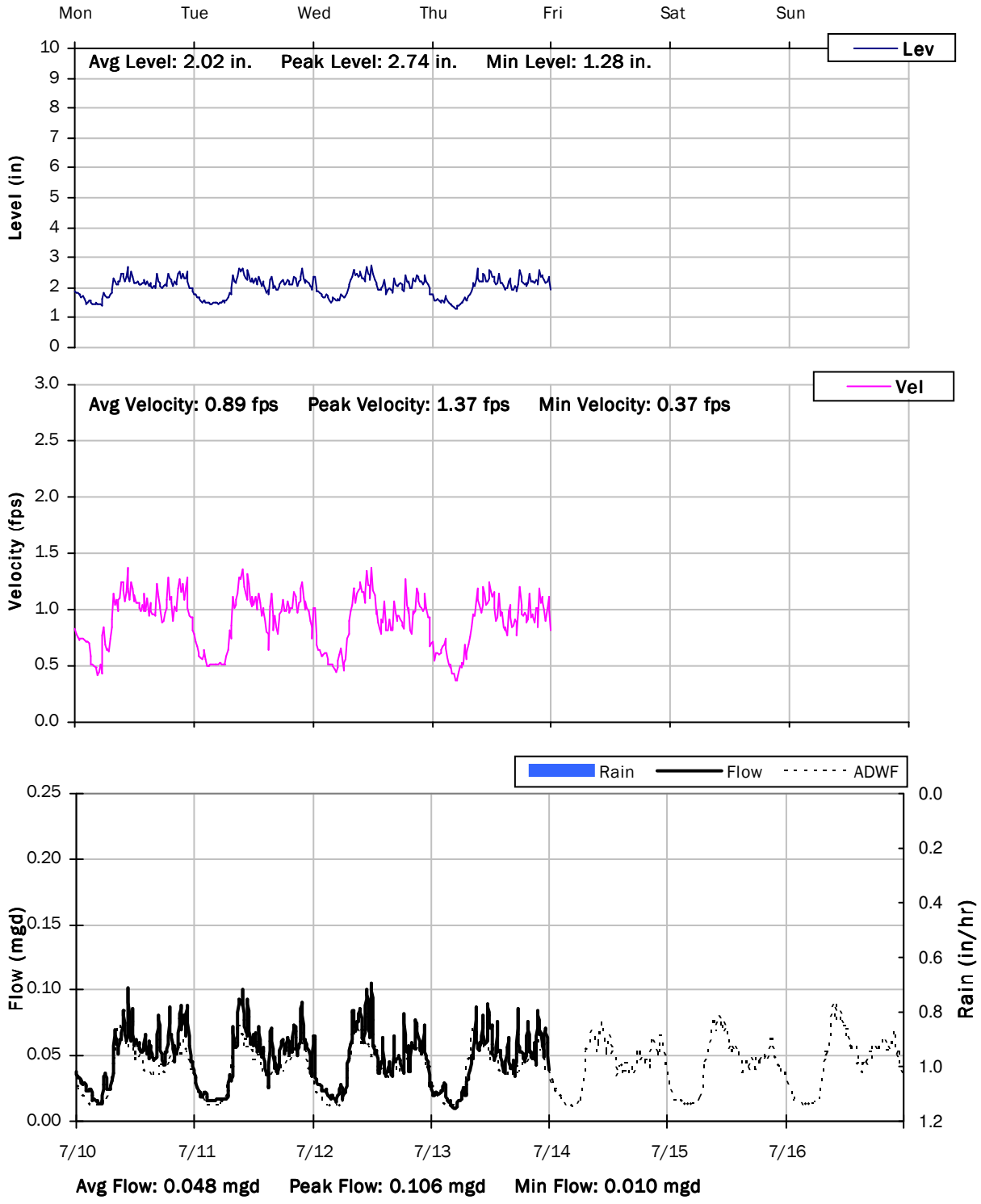
7/3/2017 to 7/10/2017



SITE 07

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

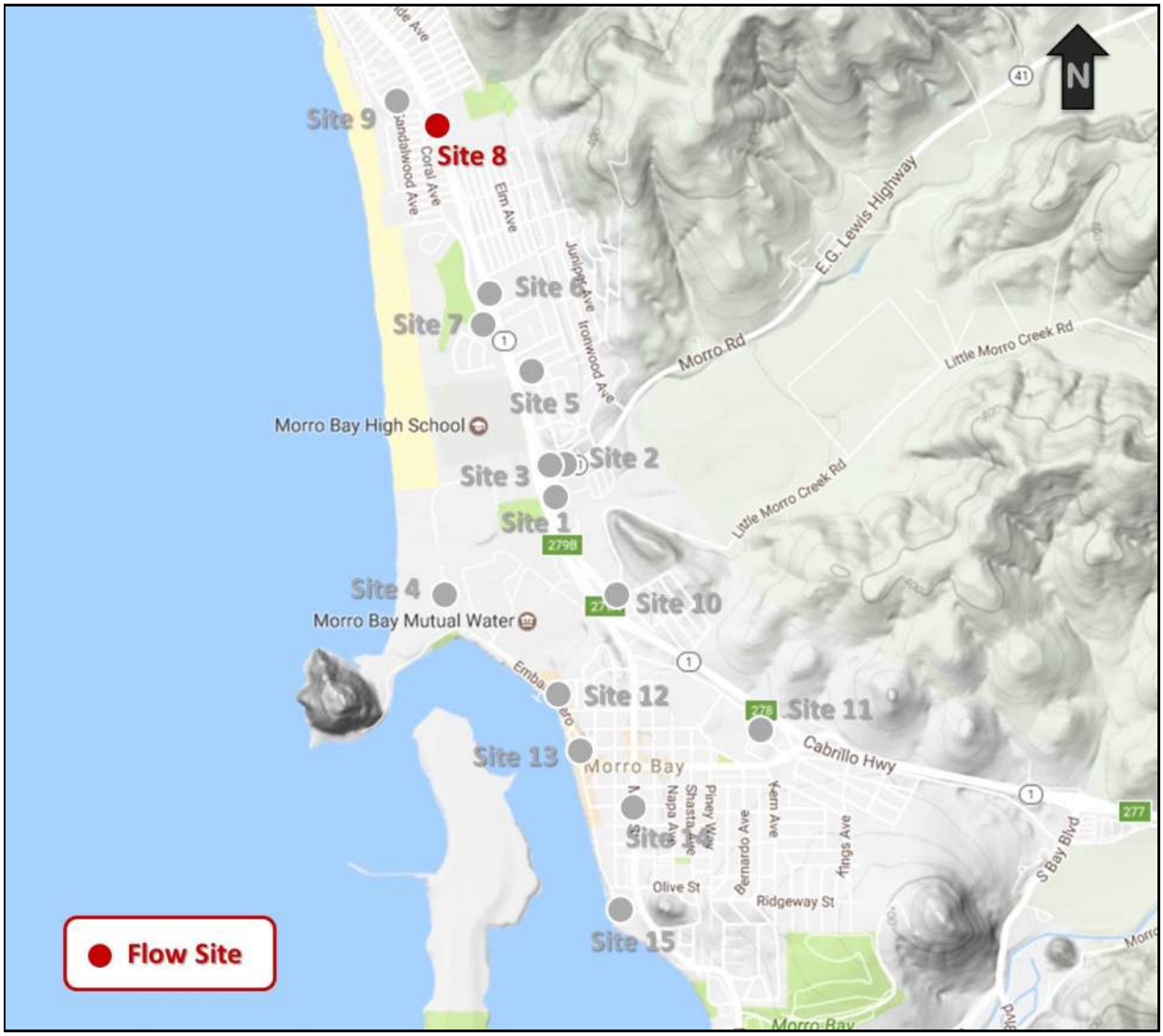
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 08

Location: Main Street, ~400 feet south of Island Street, meter on north 13" inlet

Data Summary Report



Vicinity Map: Site 08

SITE 08

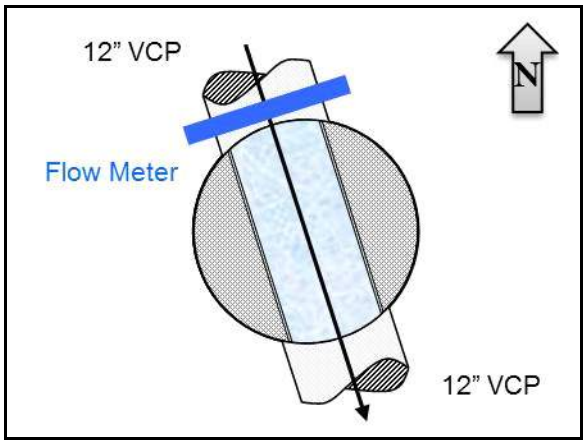
Site Information

Location: Main Street, ~400 feet south of Island Street, meter on north 13" inlet

Pipe Diameter: 13 inches

ADWF: 0.069 mgd

Peak Measured Flow: 0.212 mgd



SITE 08

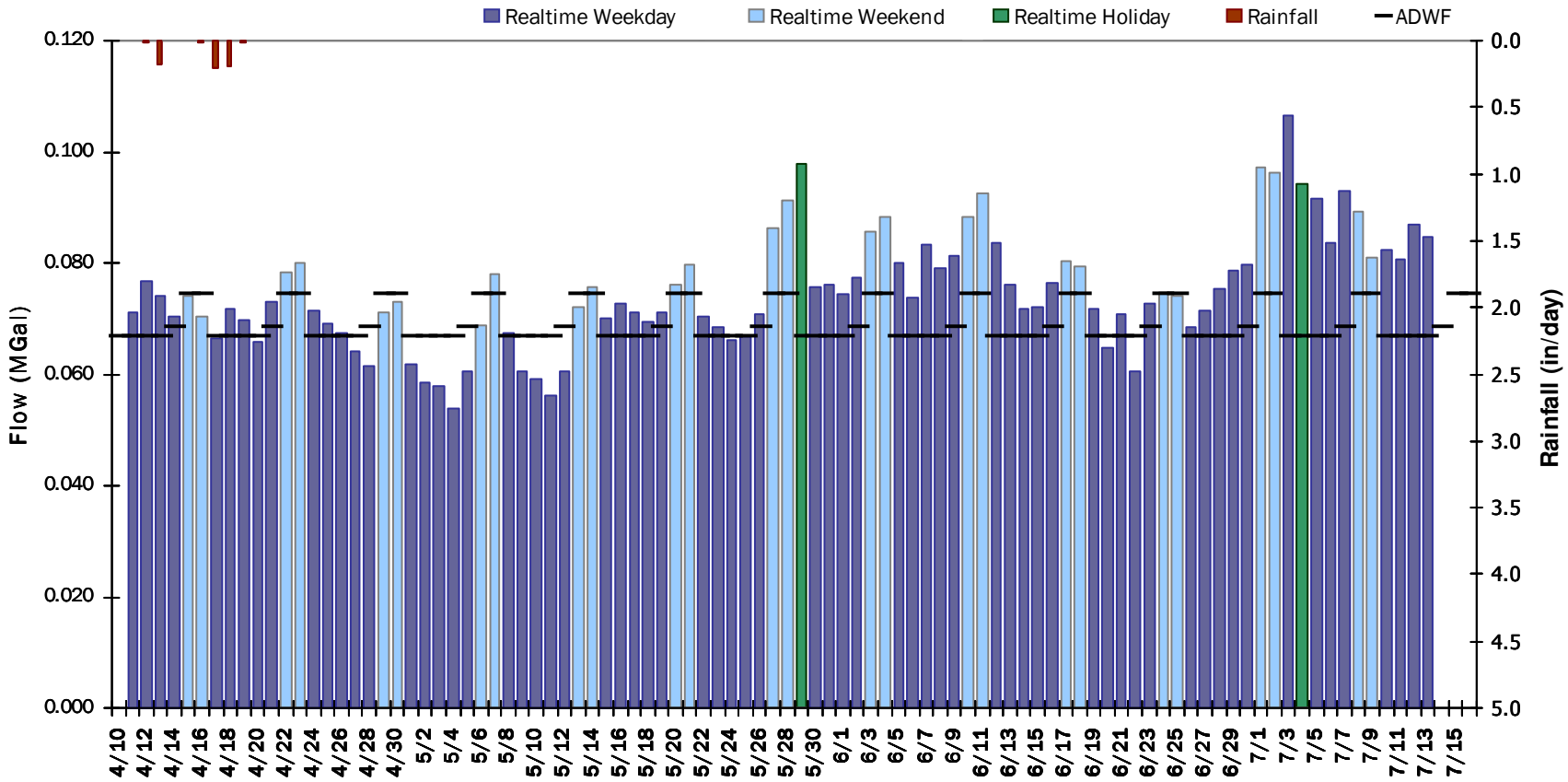
Additional Site Photos

SITE 08

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.075 MGal Peak Daily Flow: 0.106 MGal Min Daily Flow: 0.054 MGal

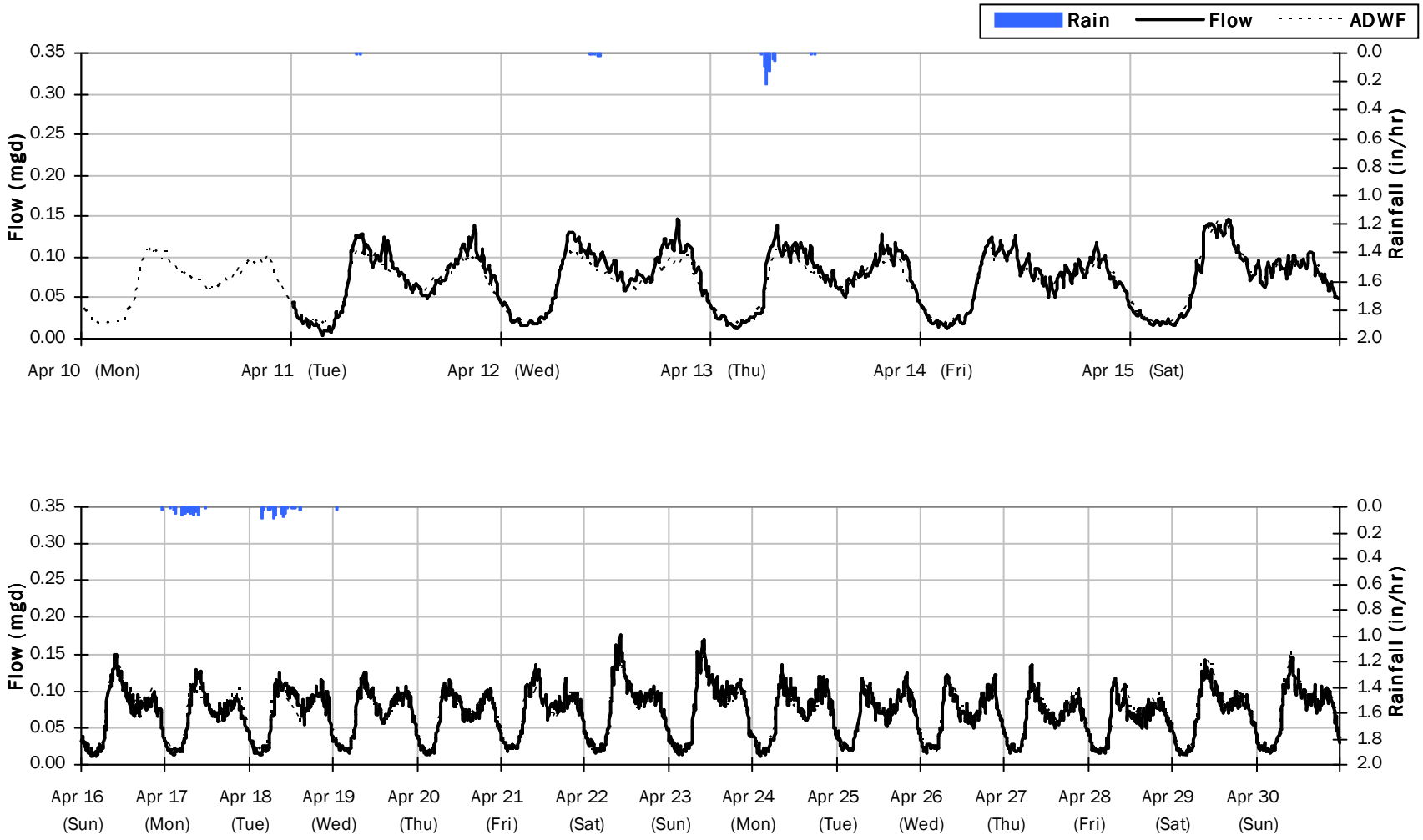
Total Period Rainfall: 0.61 inches



SITE 08

Flow Summary: 4/1/2017 to 4/30/2017

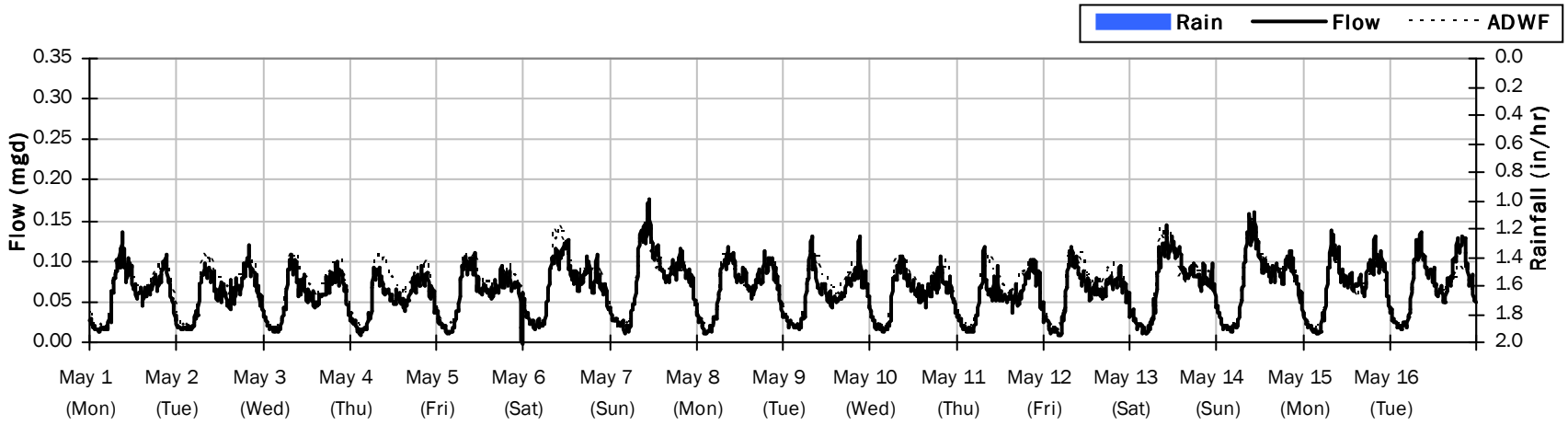
Total Period Rainfall: 0.61 inches **Avg Flow: 0.071 mgd Peak Flow: 0.176 mgd Min Flow: 0.004 mgd**

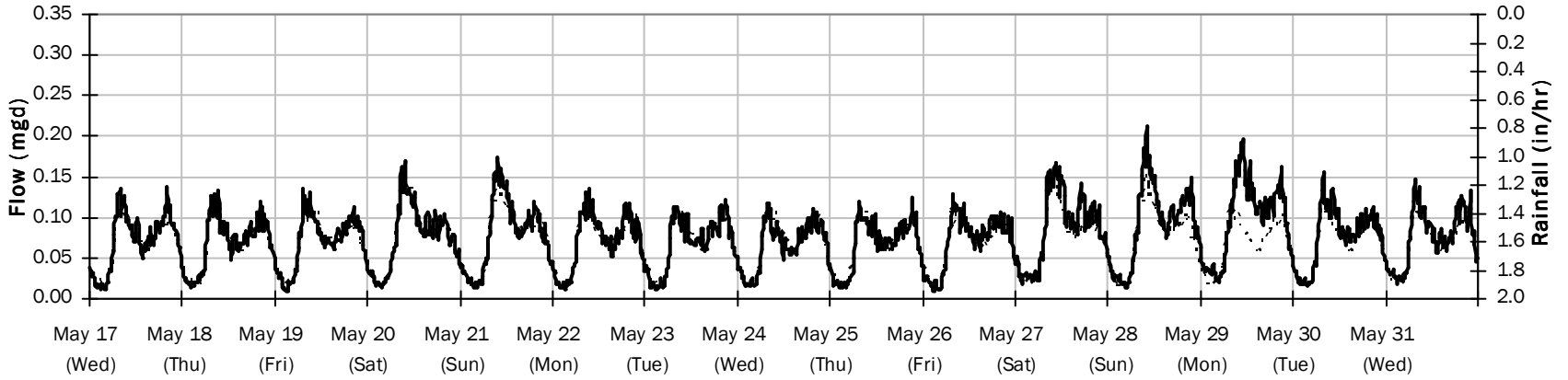


SITE 08

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.070 mgd Peak Flow: 0.212 mgd Min Flow: 0.000 mgd**

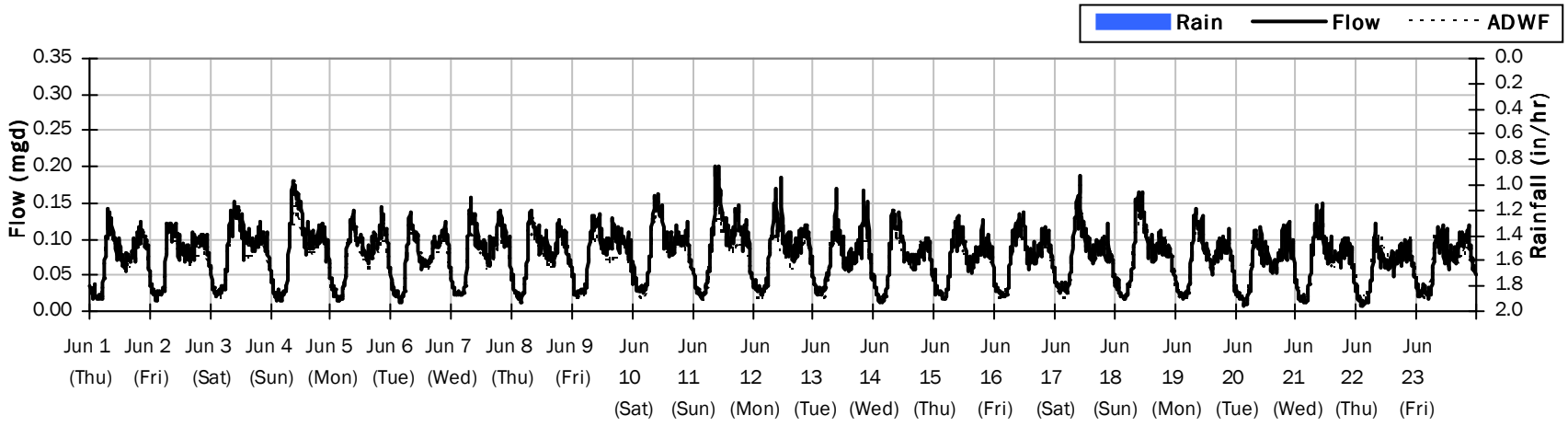


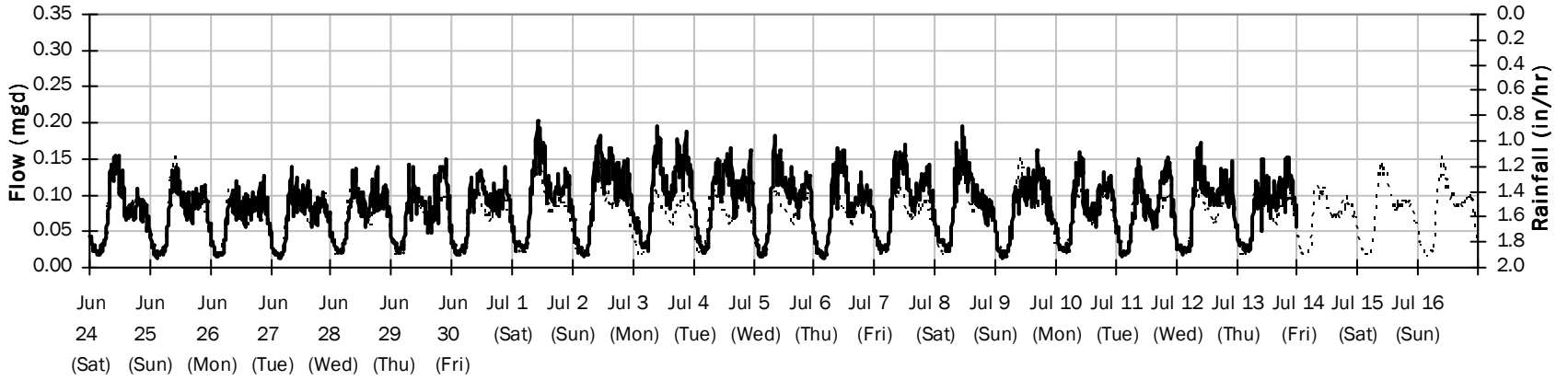


SITE 08

Flow Summary: 6/1/2017 to 7/16/2017

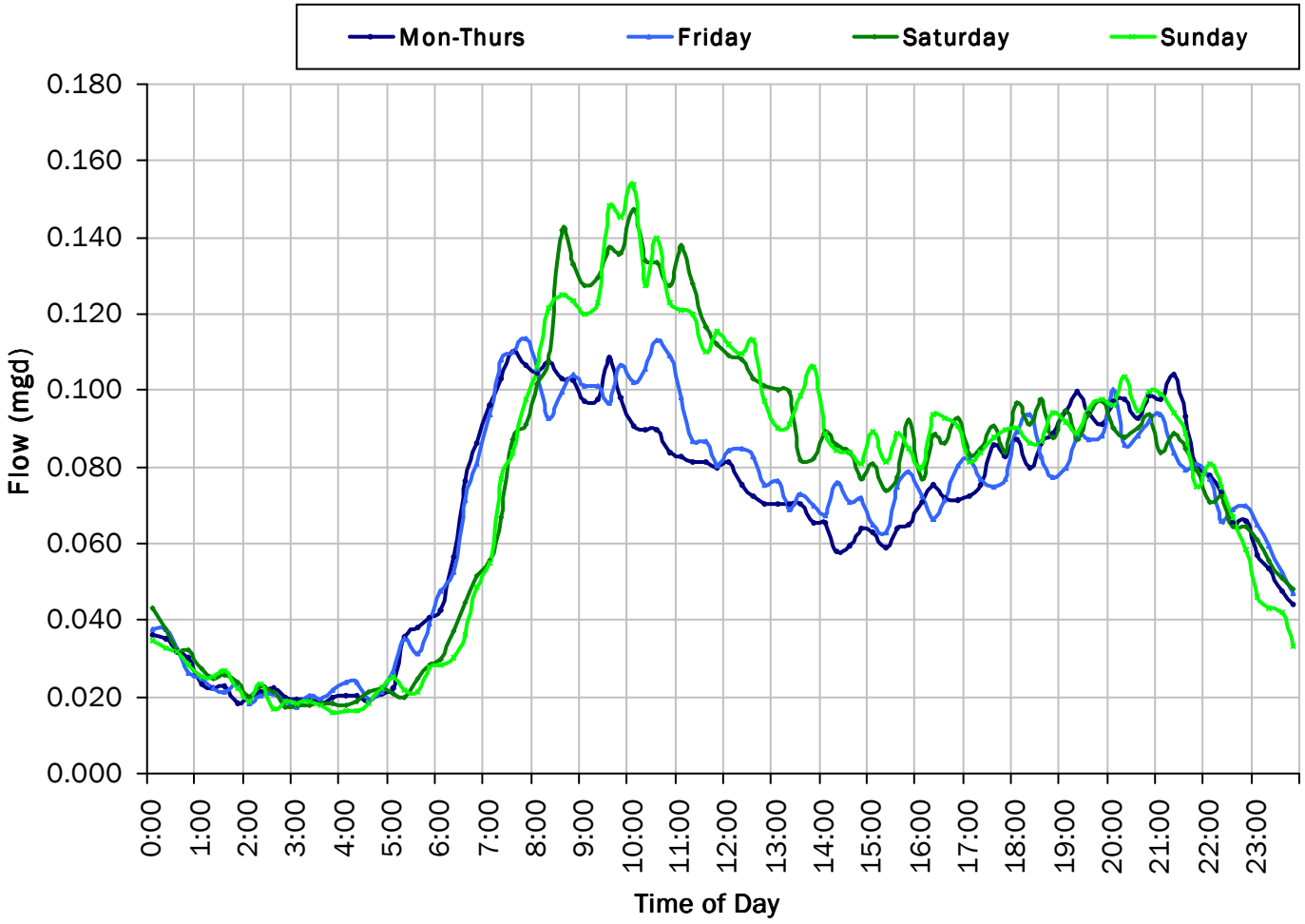
Total Period Rainfall: 0.00 inches **Avg Flow: 0.081 mgd** **Peak Flow: 0.203 mgd** **Min Flow: 0.008 mgd**





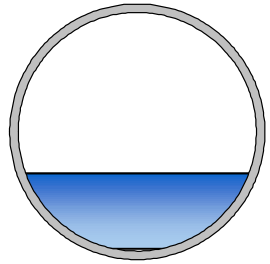
SITE 08

Average Dry Weather Flow Hydrographs



ADWF:

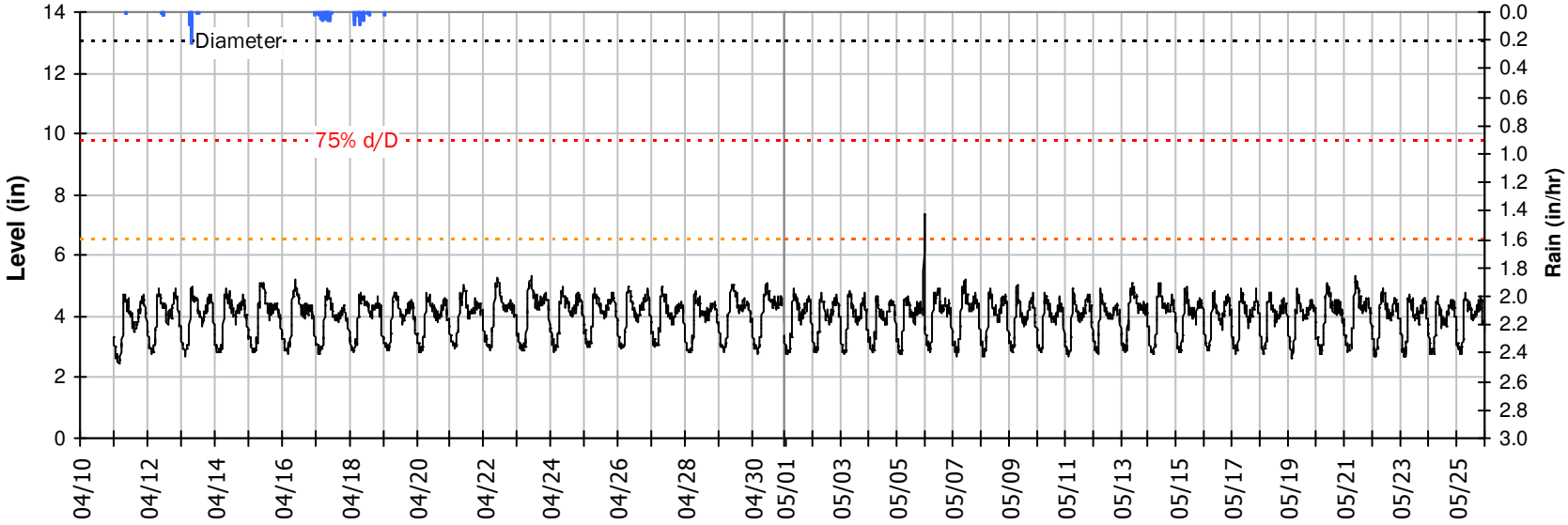
0.069 *mgd*

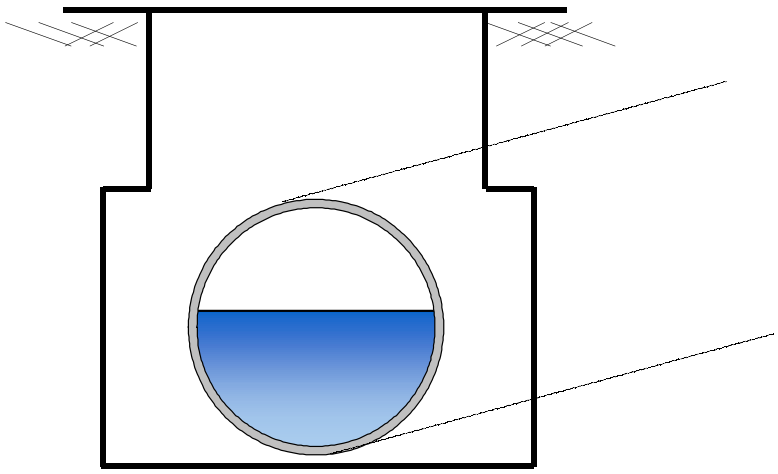


SITE 08

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



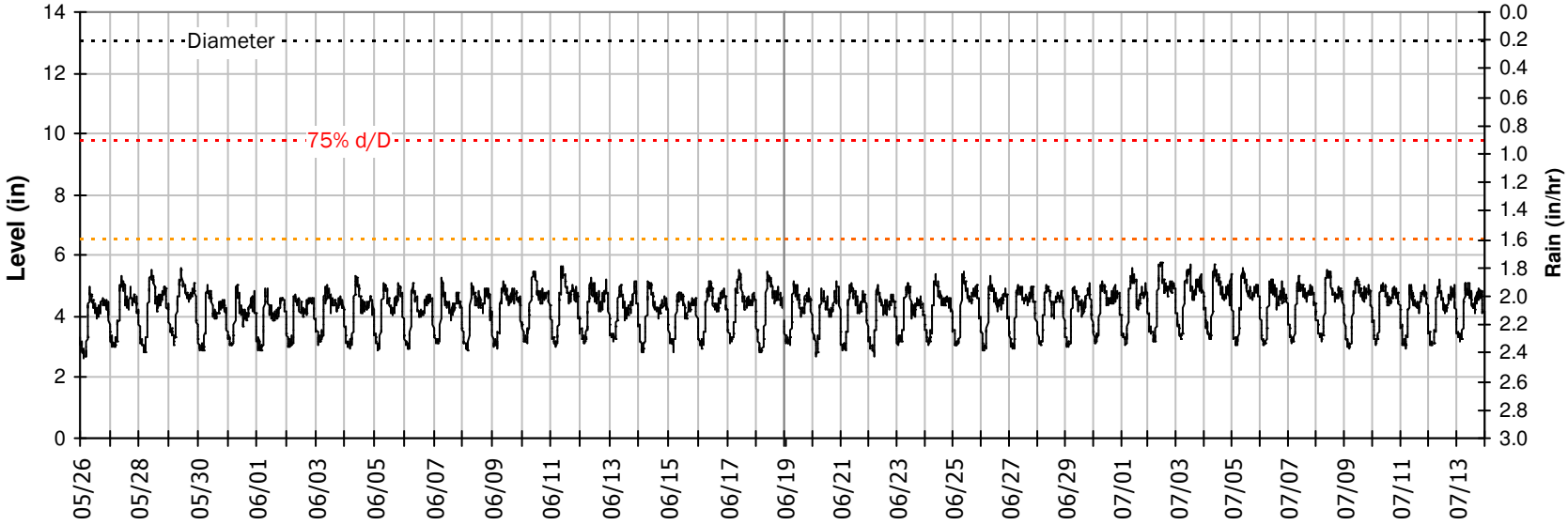


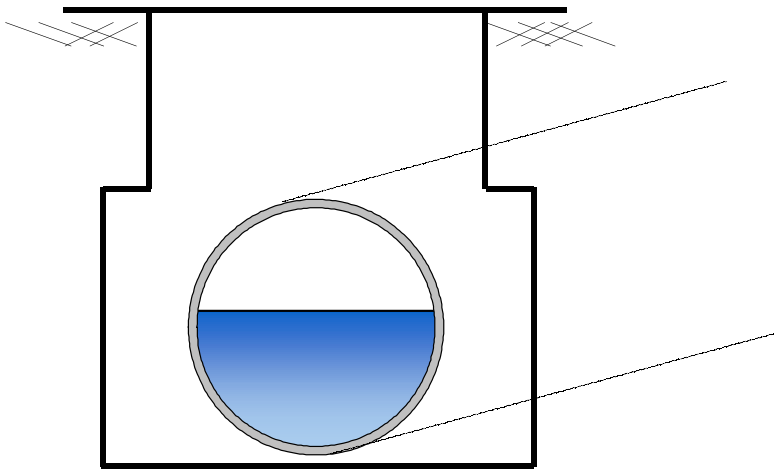
Pipe Diameter:	13	*inches*
Peak Measured Level:	7.32	*inches*
Peak d/D Ratio:	0.56	

SITE 08

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



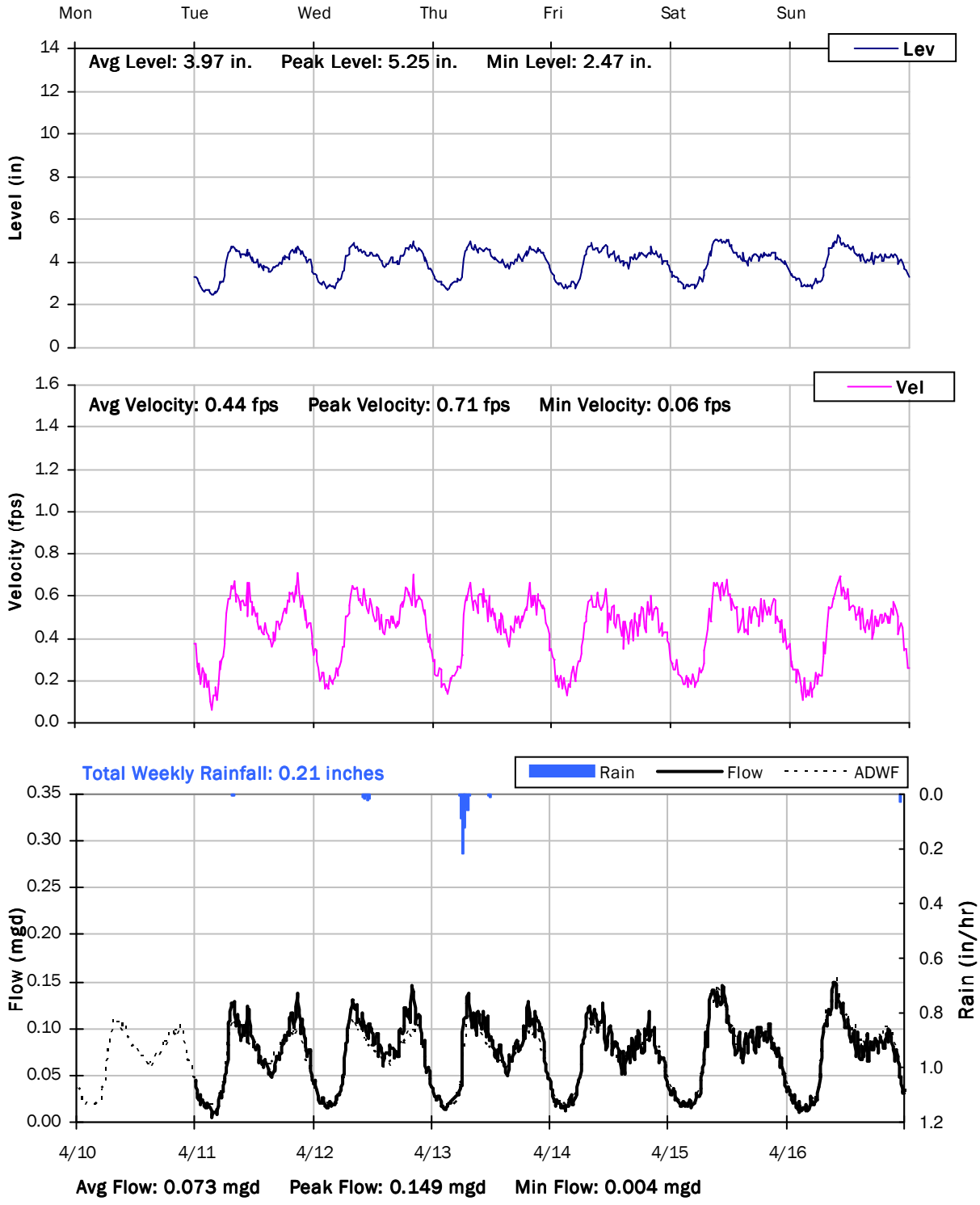


Pipe Diameter:	13	*inches*
Peak Measured Level:	7.32	*inches*
Peak d/D Ratio:	0.56	

SITE 08

Weekly Level, Velocity and Flow Hydrographs

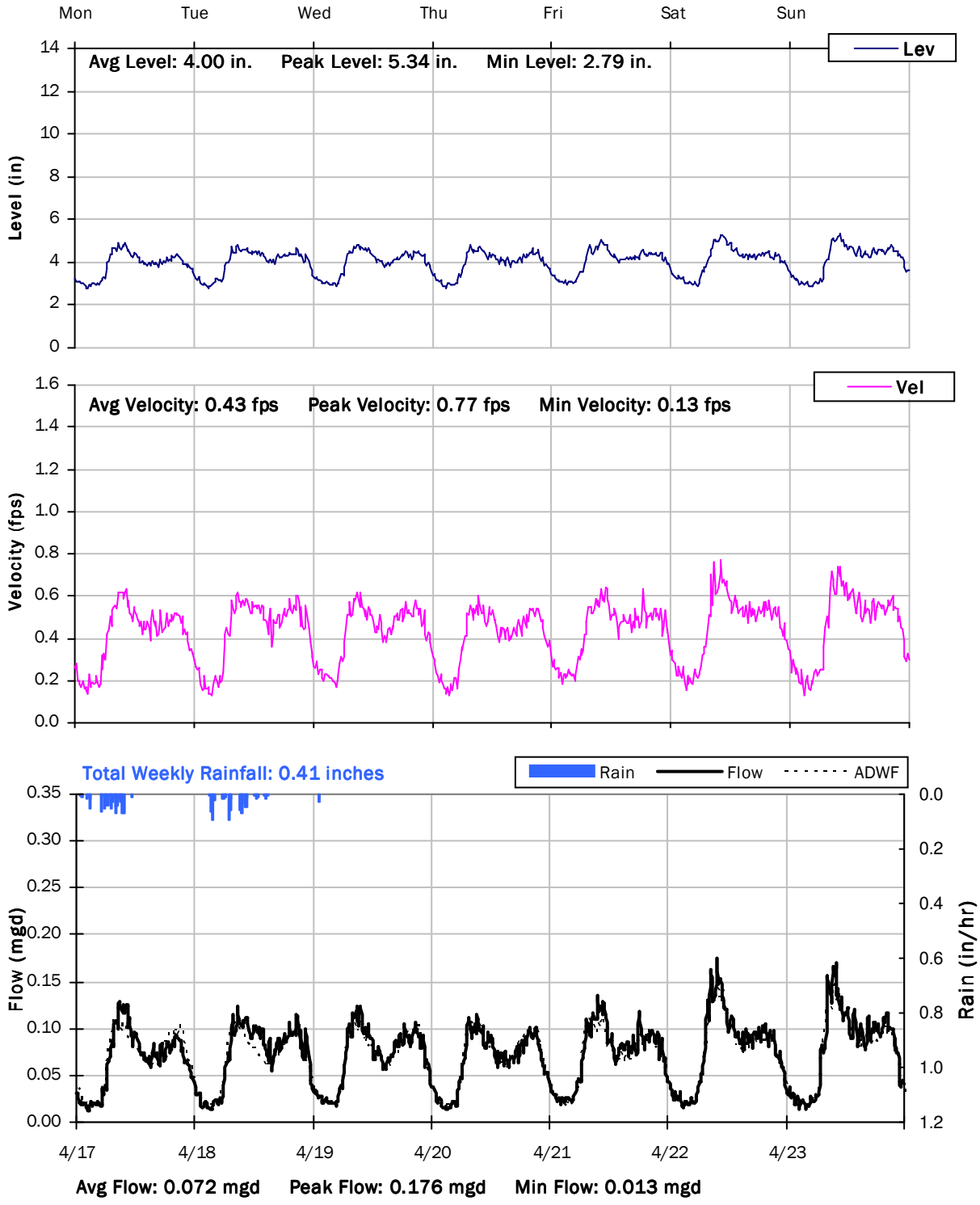
4/10/2017 to 4/17/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

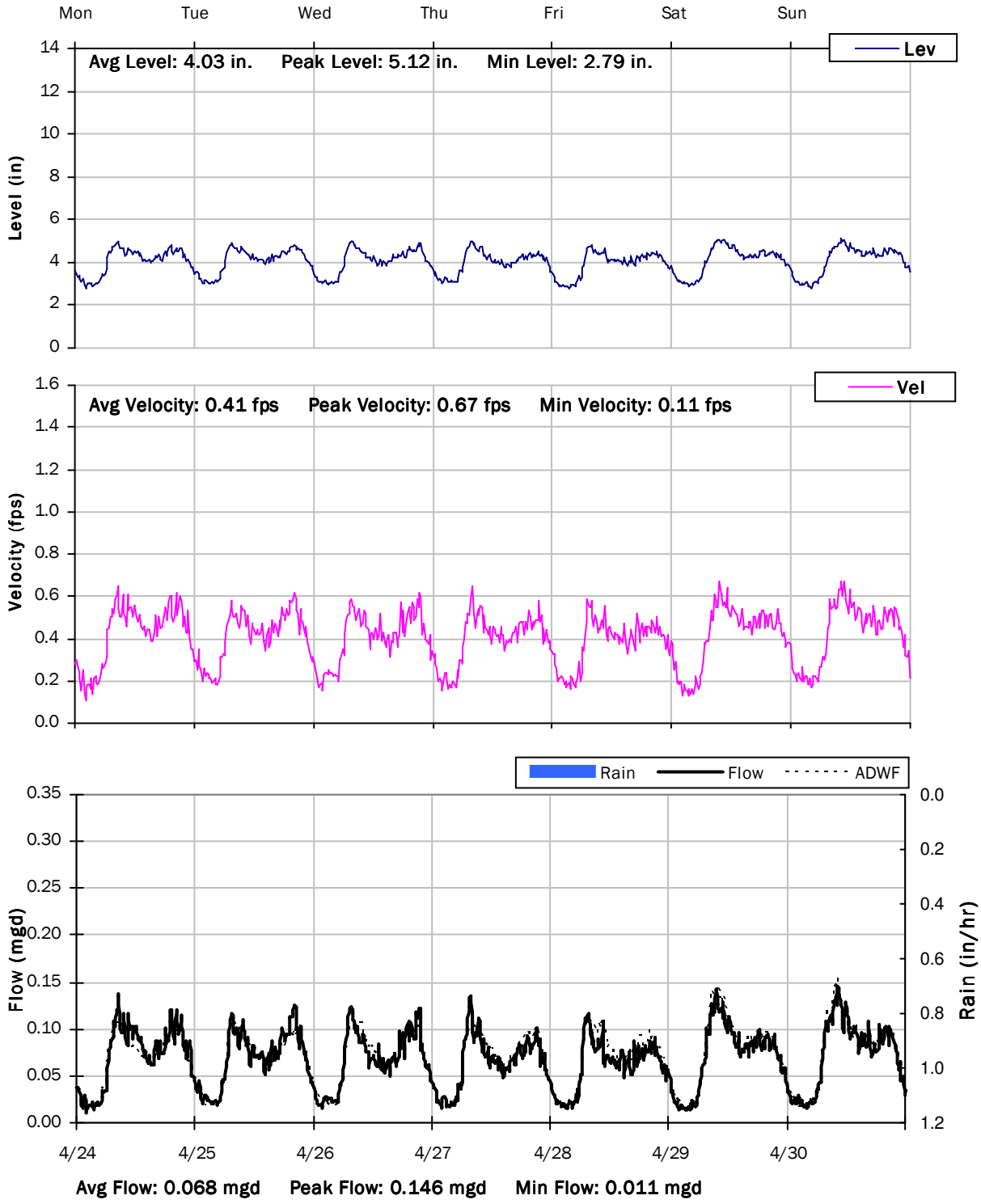
4/17/2017 to 4/24/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

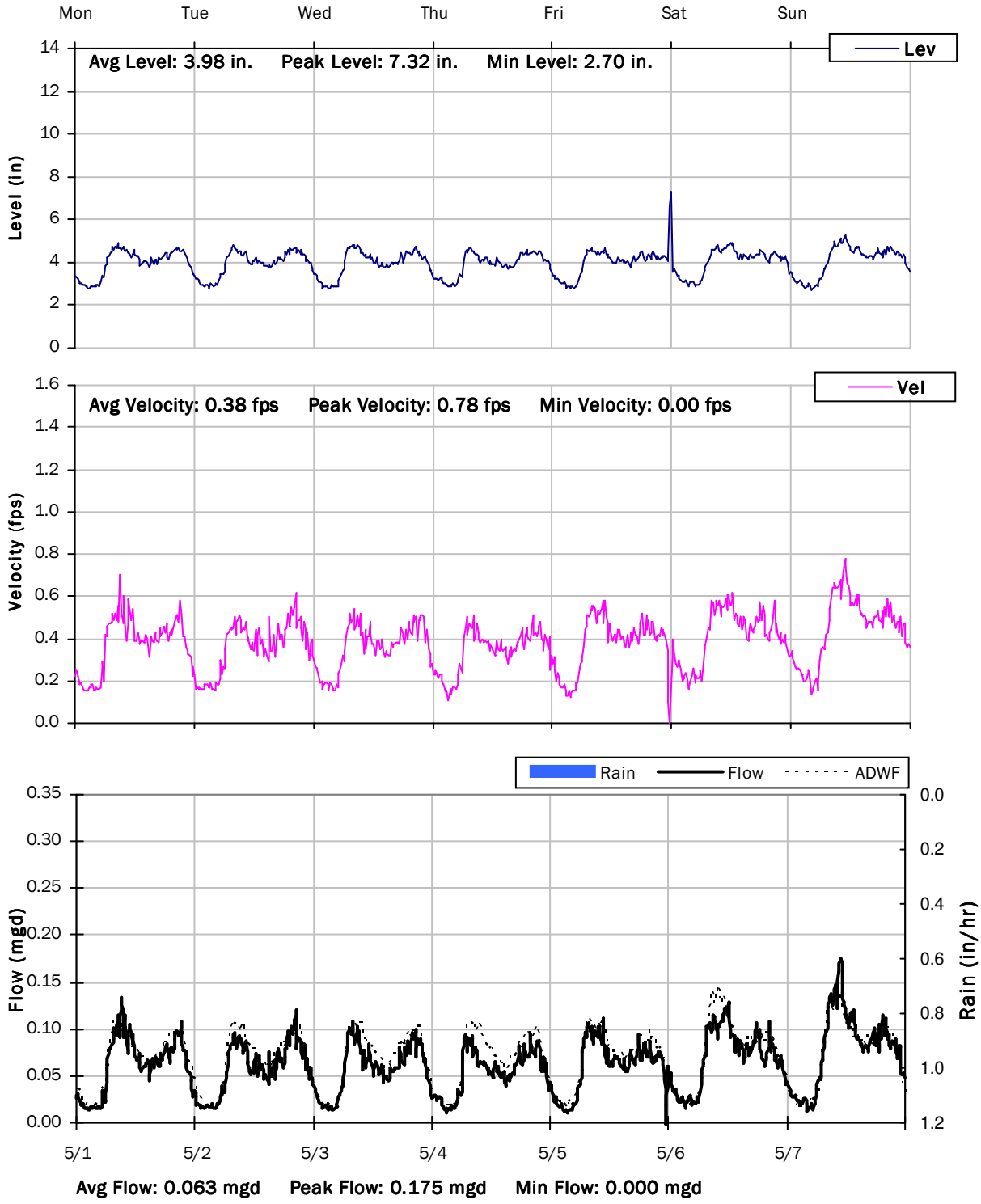
4/24/2017 to 5/1/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

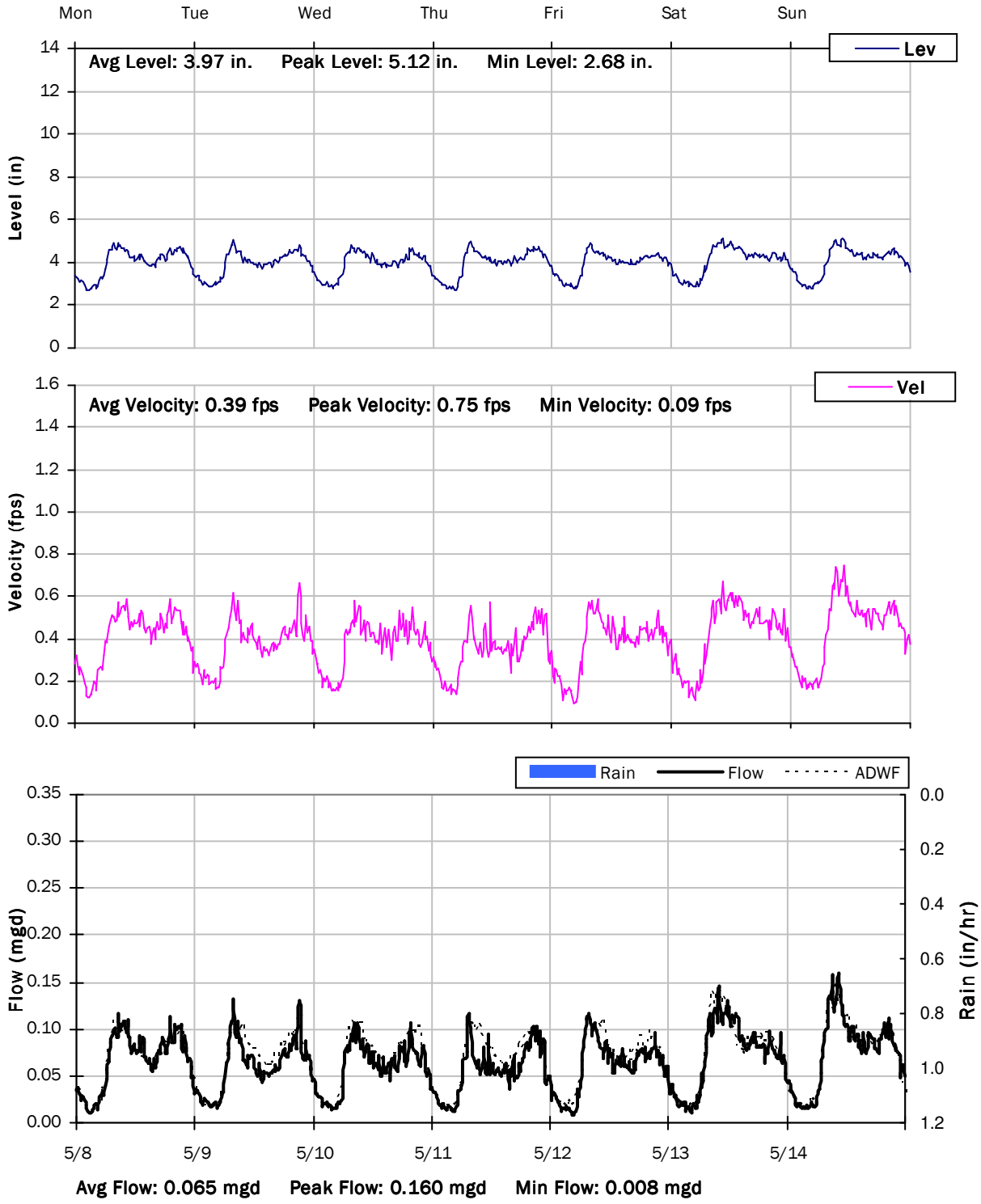
5/1/2017 to 5/8/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

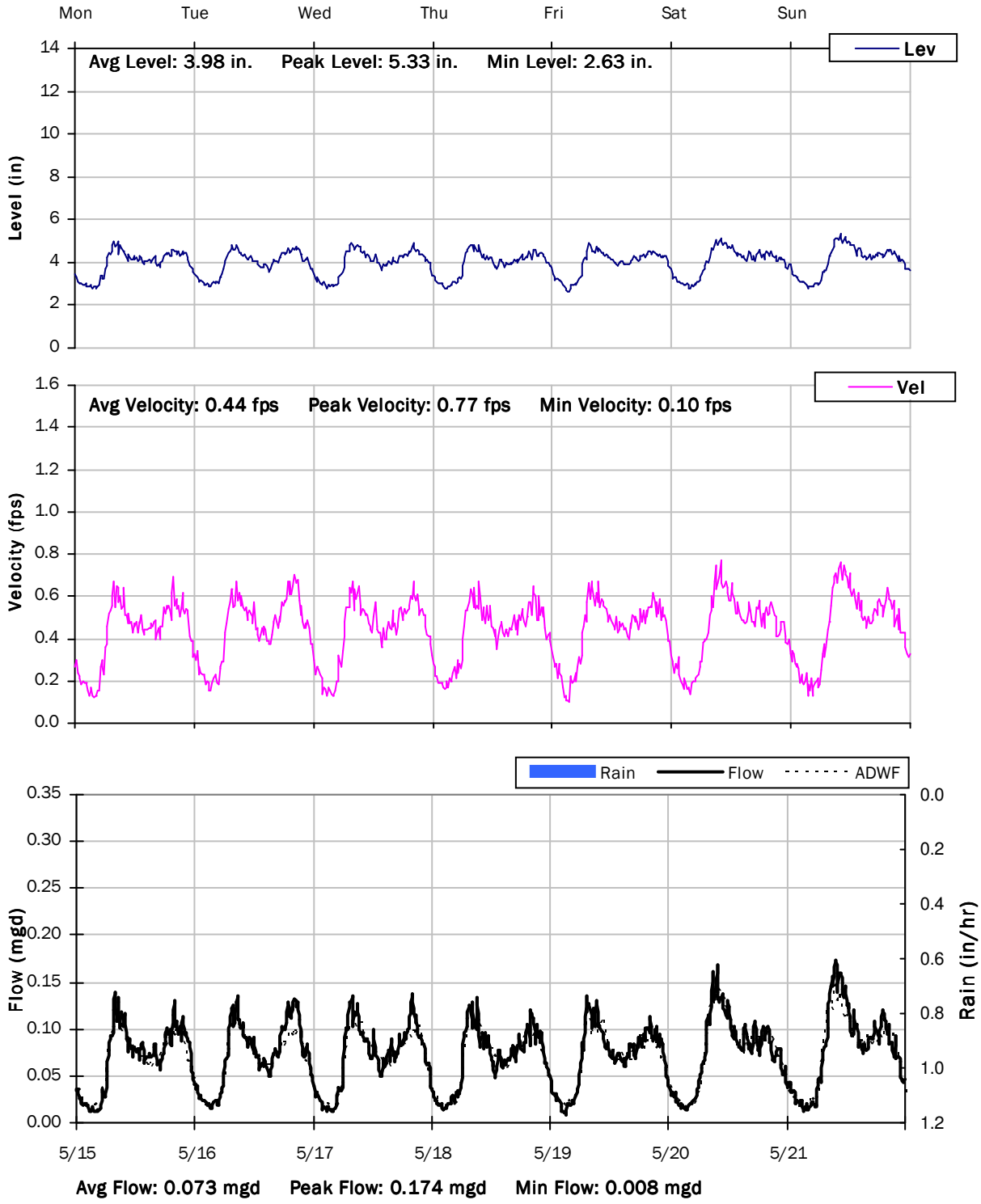
5/8/2017 to 5/15/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

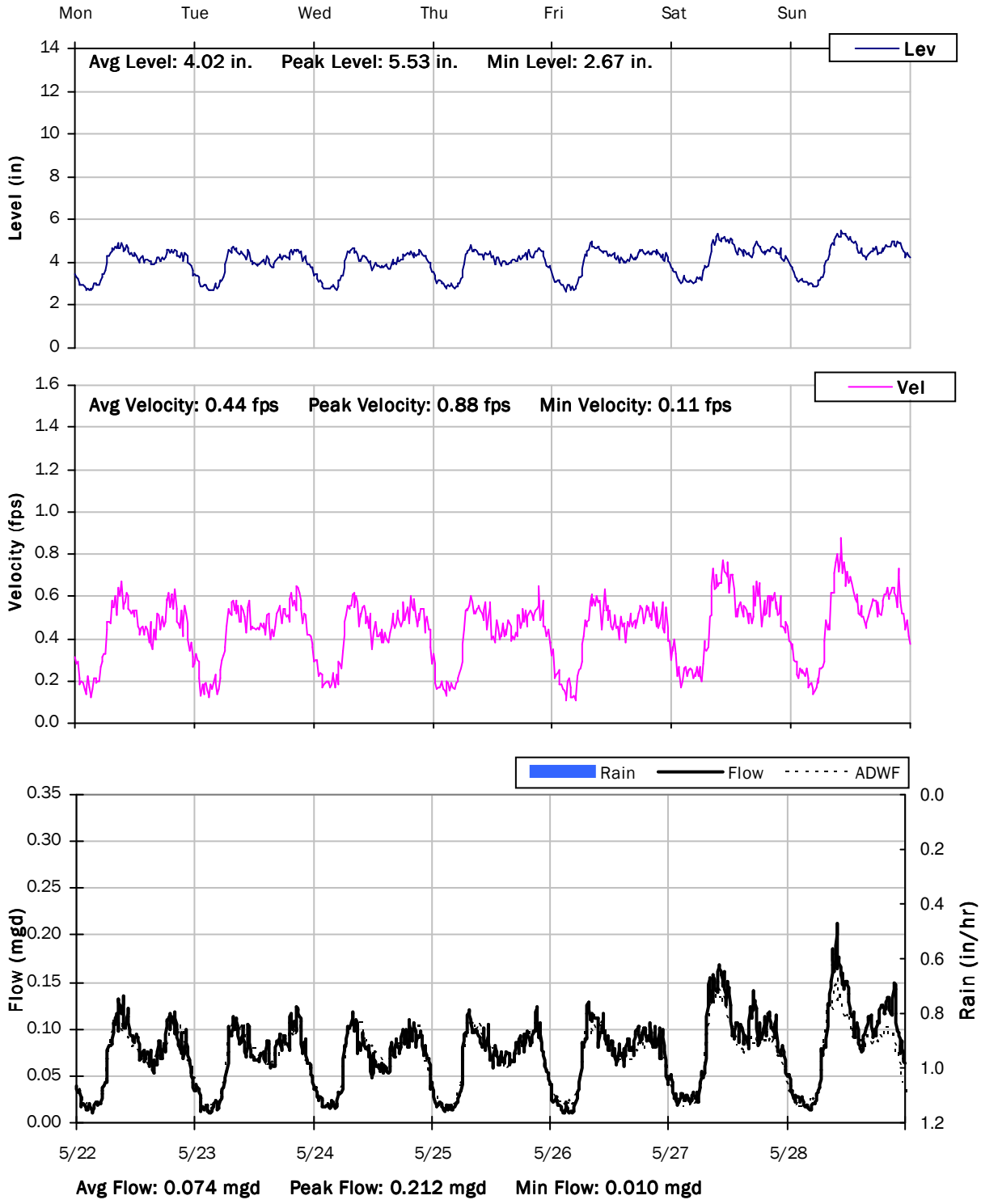
5/15/2017 to 5/22/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

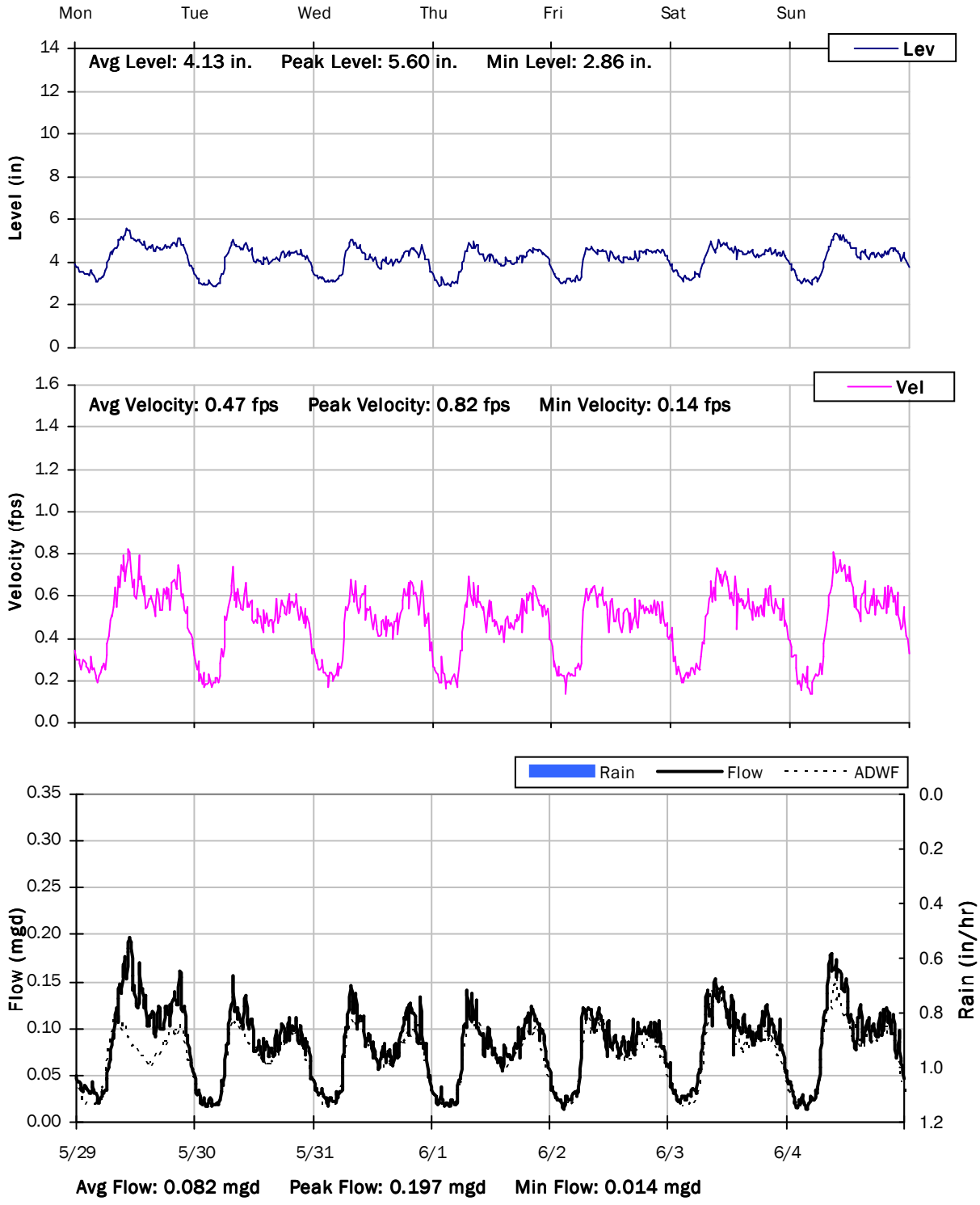
5/22/2017 to 5/29/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

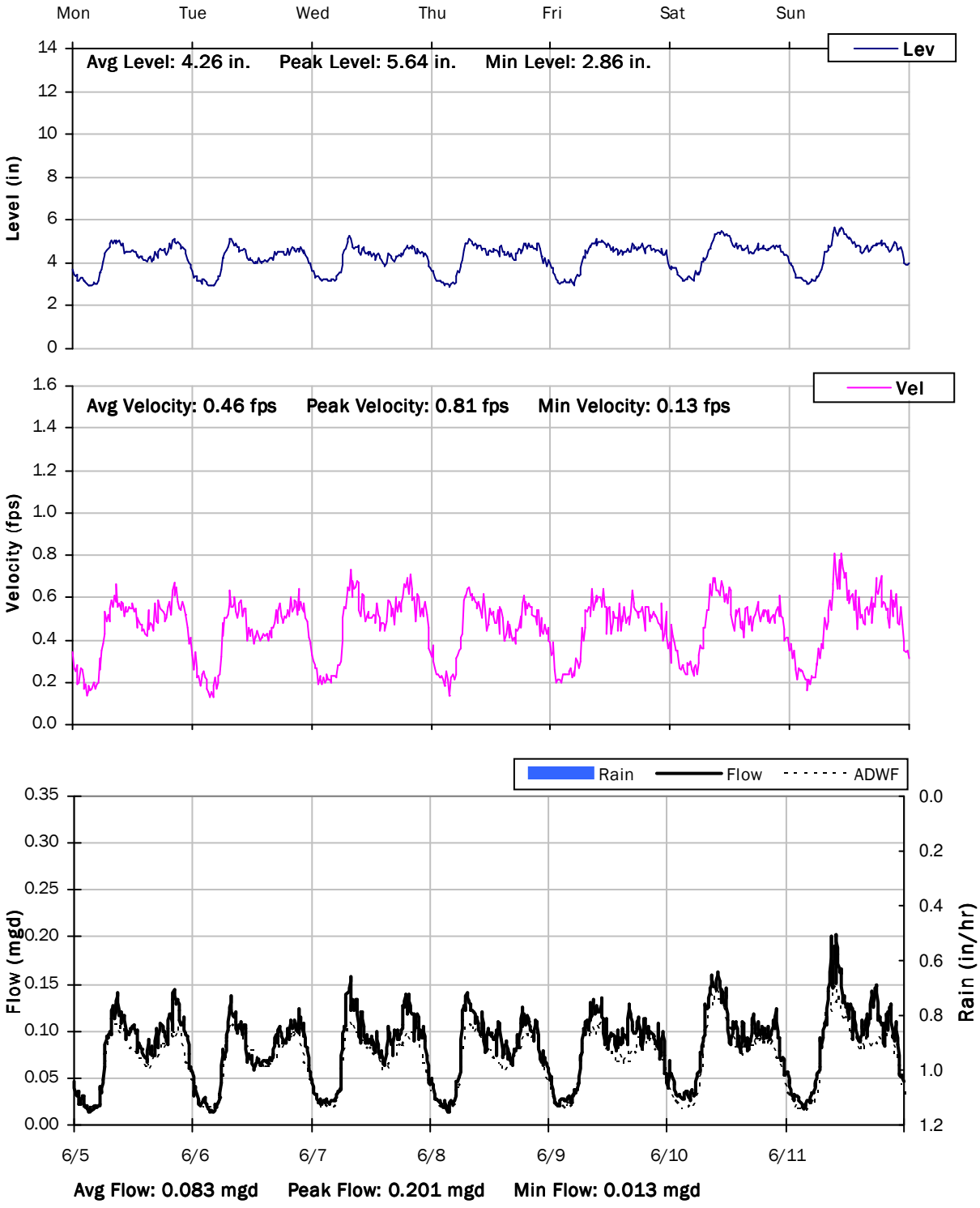
5/29/2017 to 6/5/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

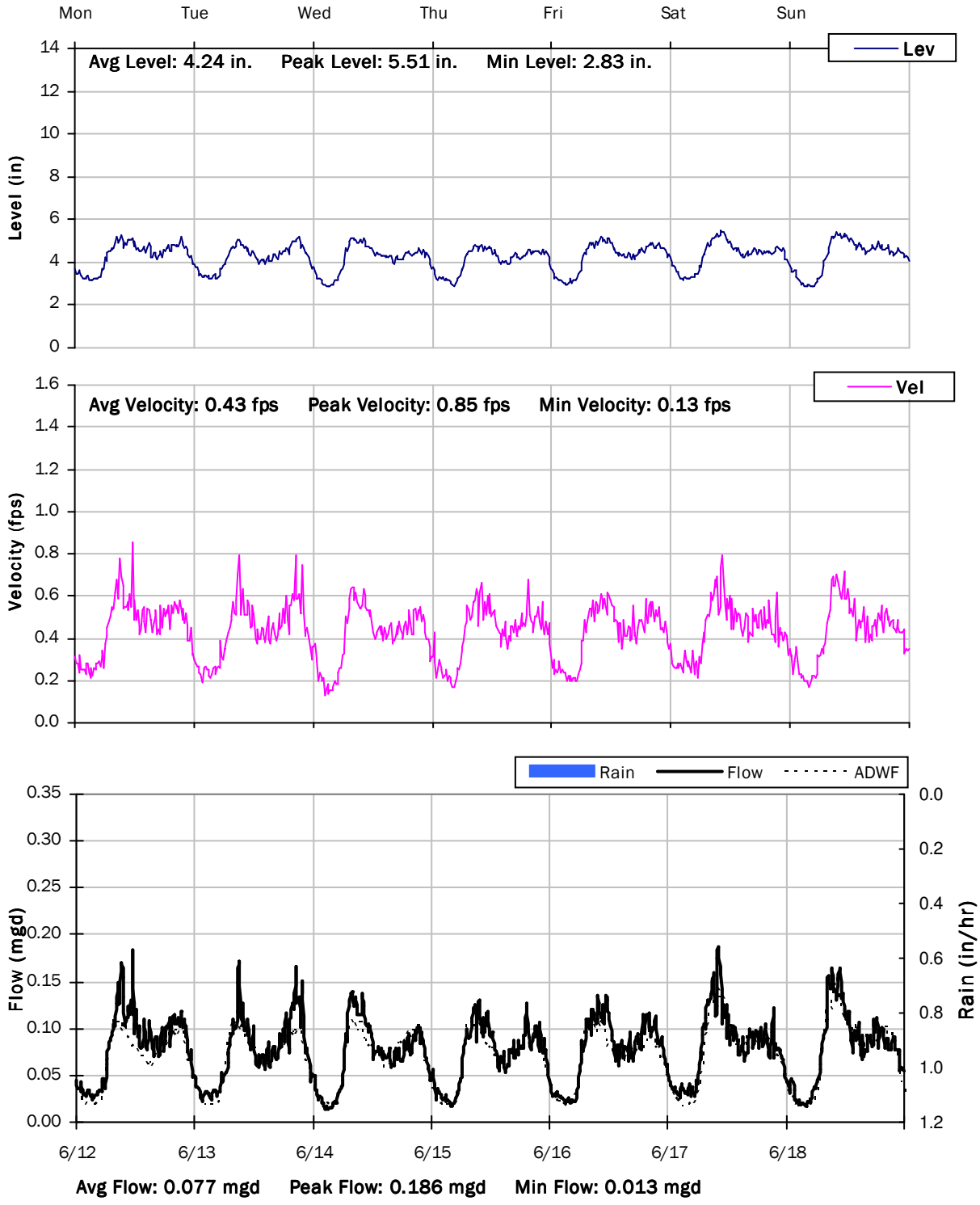
6/5/2017 to 6/12/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

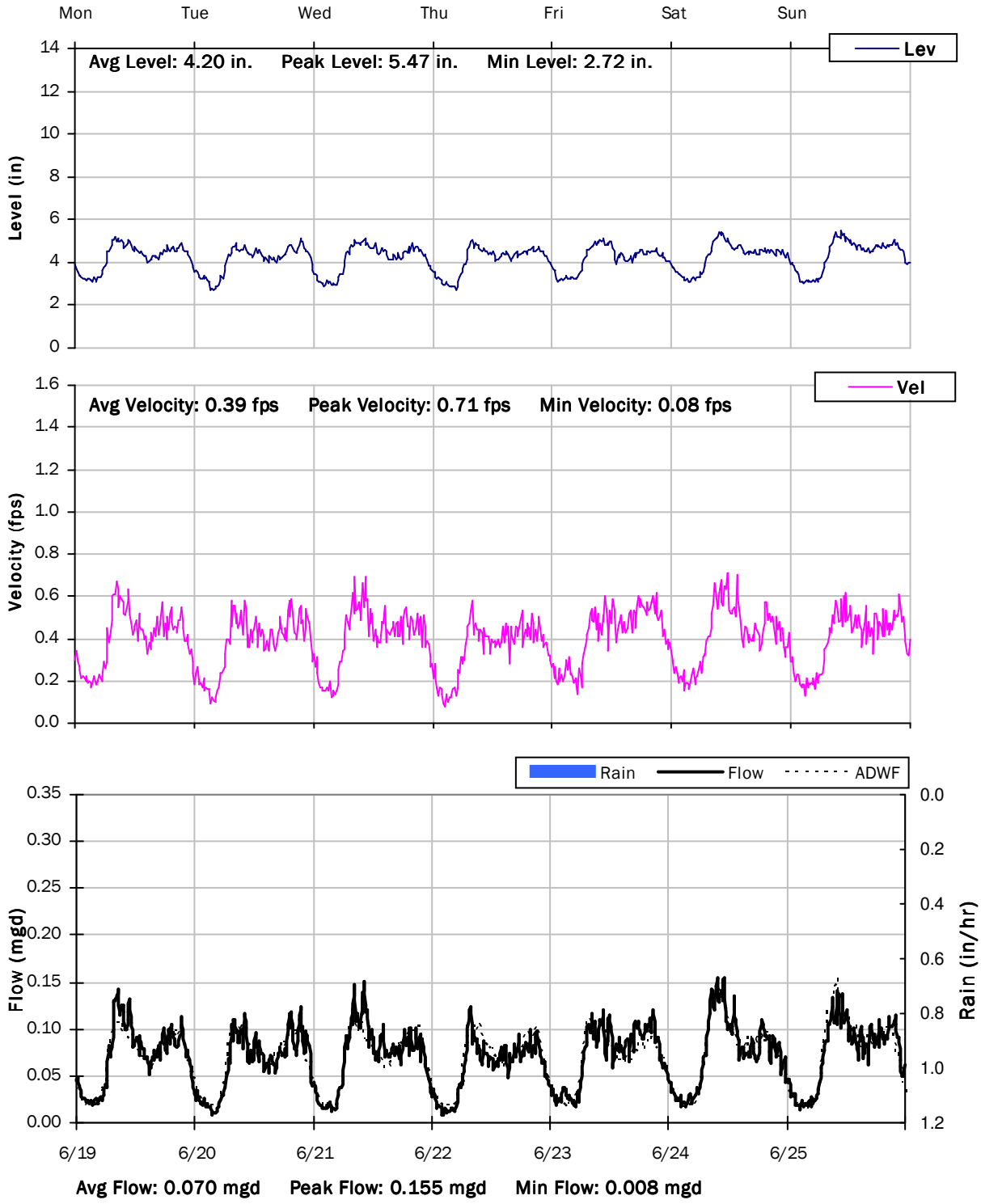
6/12/2017 to 6/19/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

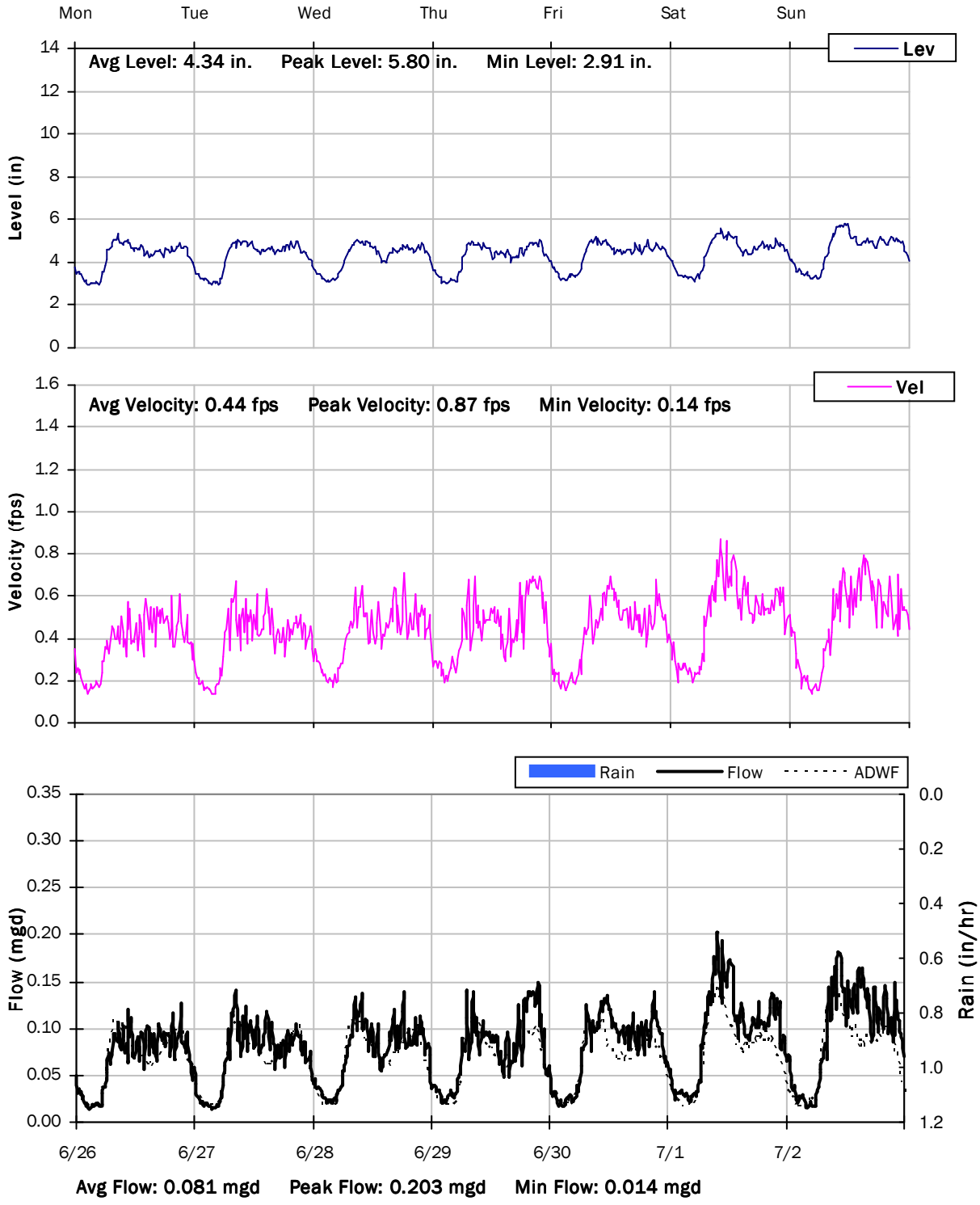
6/19/2017 to 6/26/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

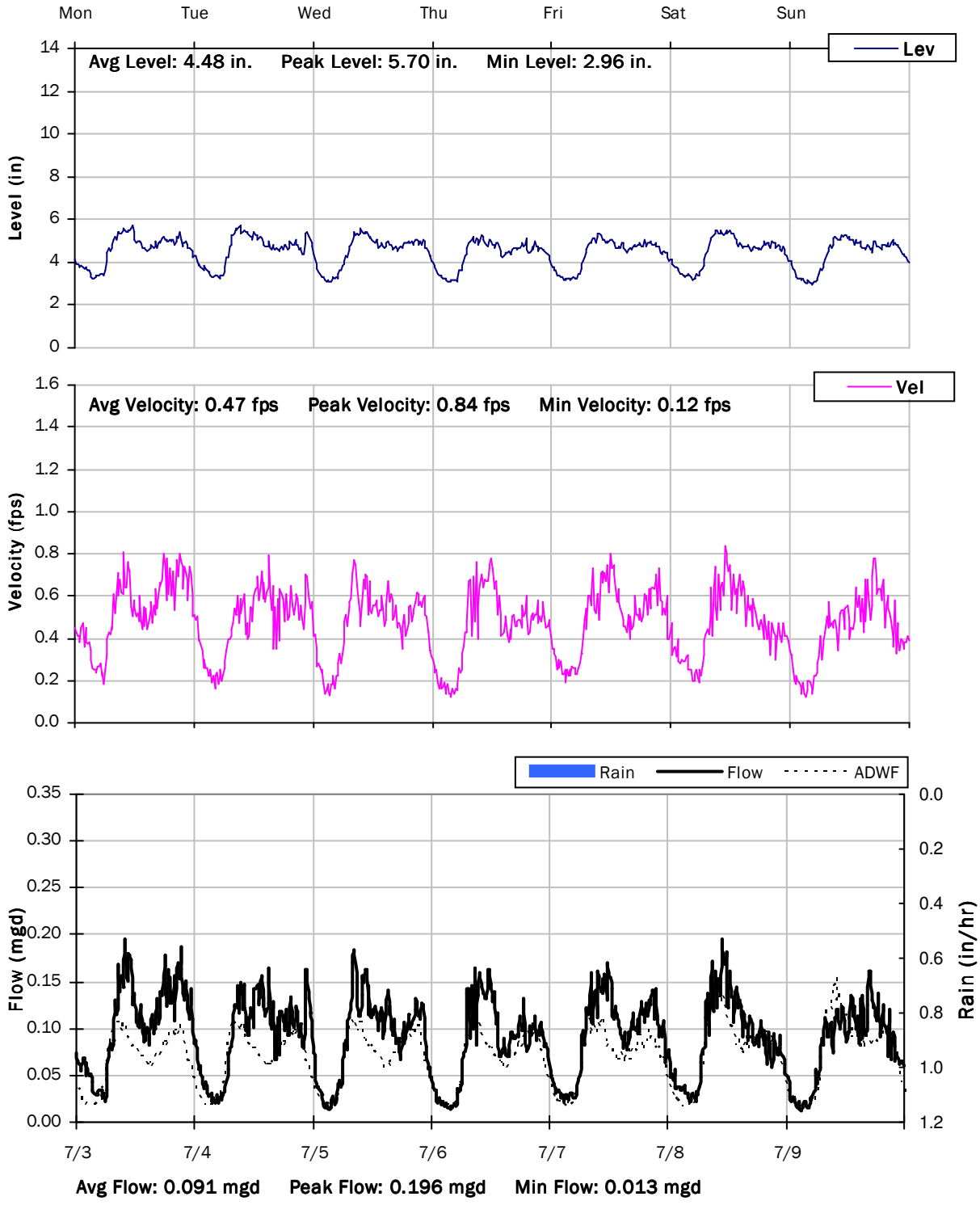
6/26/2017 to 7/3/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

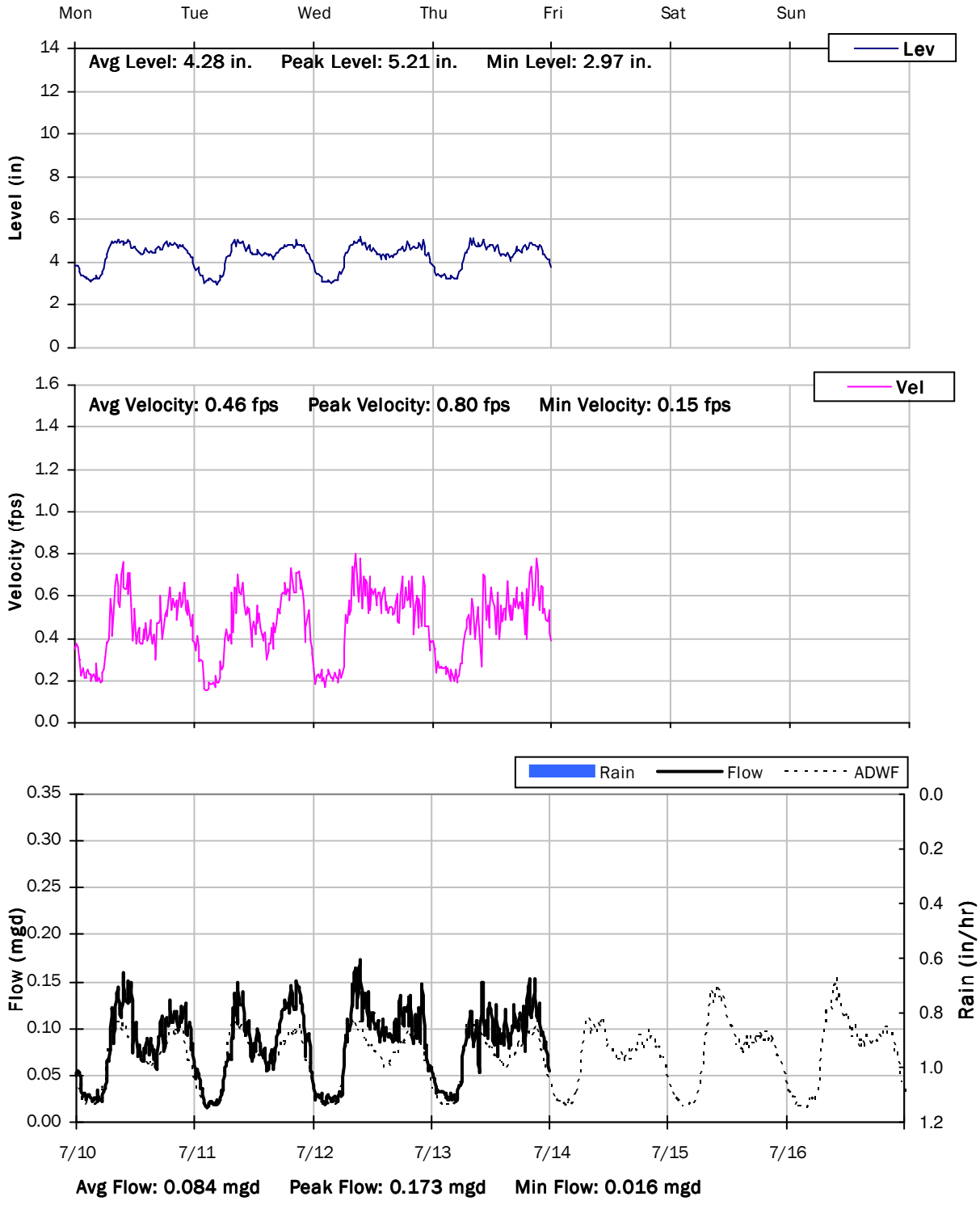
7/3/2017 to 7/10/2017



SITE 08

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

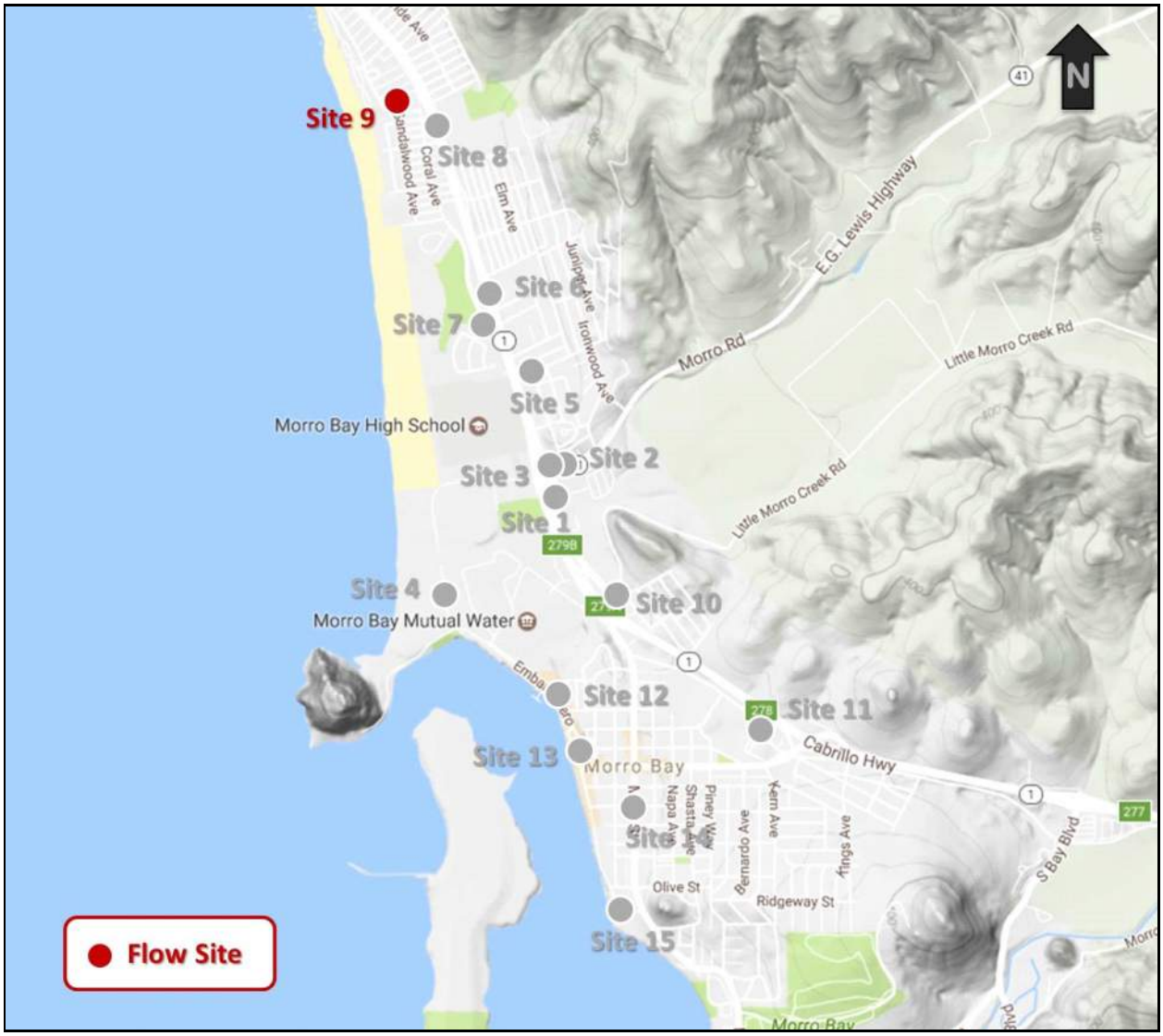
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 09

Location: 125 Java Street (near intersection of Sandalwood Avenue), meter on west 12" inlet

Data Summary Report



Vicinity Map: Site 09

SITE 09

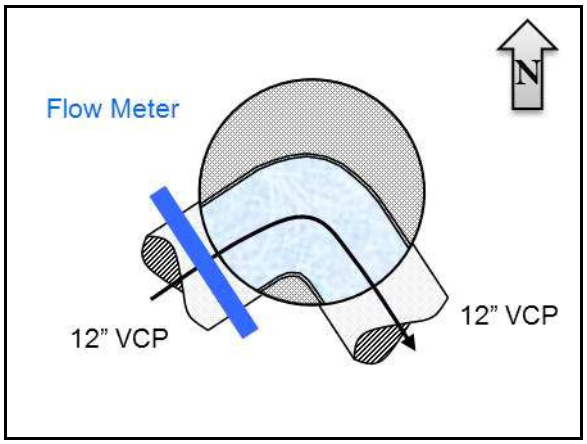
Site Information

Location: 125 Java Street (near intersection of Sandalwood Avenue), meter on west 12" inlet

Pipe Diameter: 12 inches

ADWF: 0.020 mgd

Peak Measured Flow: 0.096 mgd



SITE 09

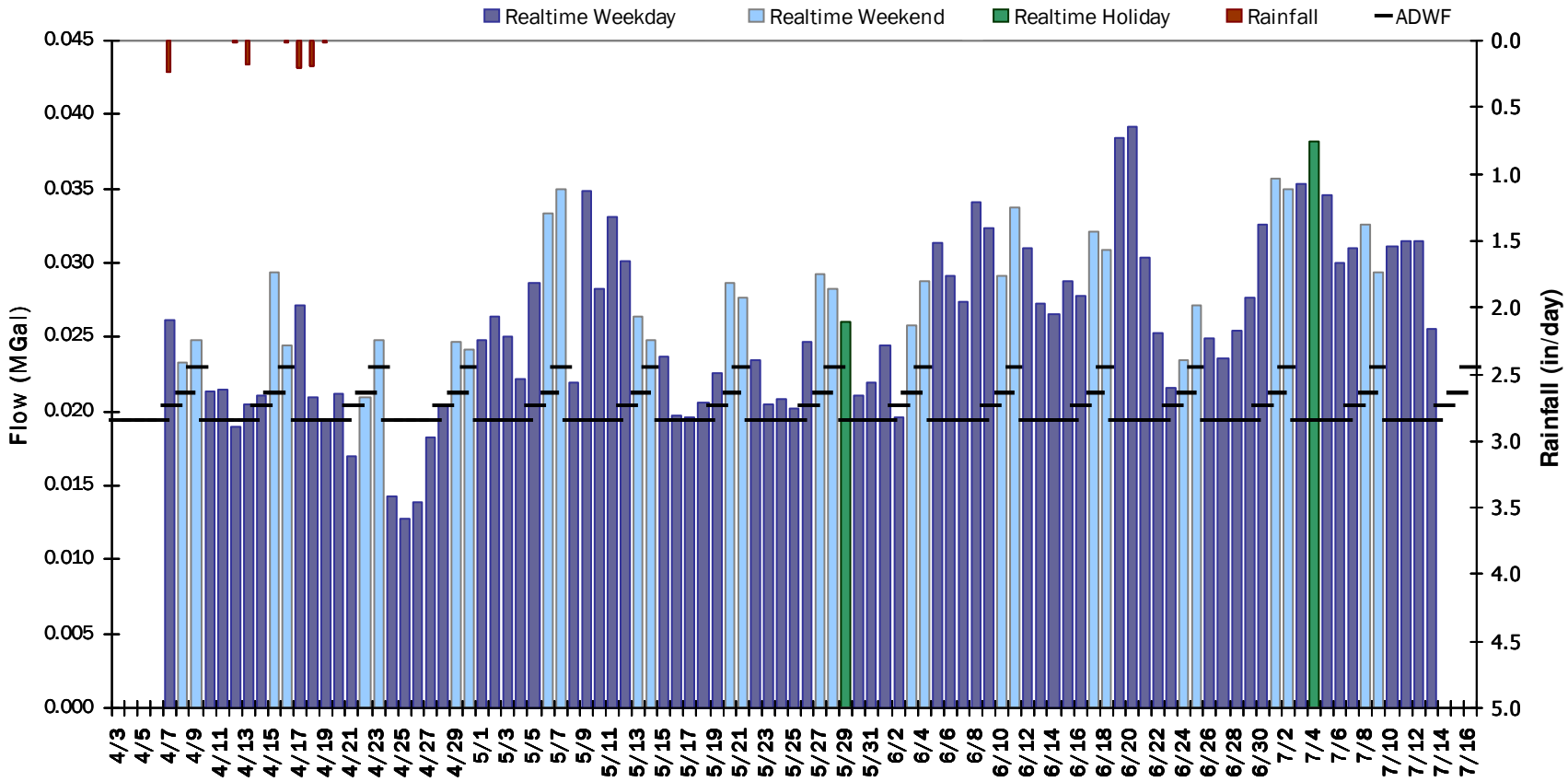
Additional Site Photos

SITE 09

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.026 MGal Peak Daily Flow: 0.039 MGal Min Daily Flow: 0.013 MGal

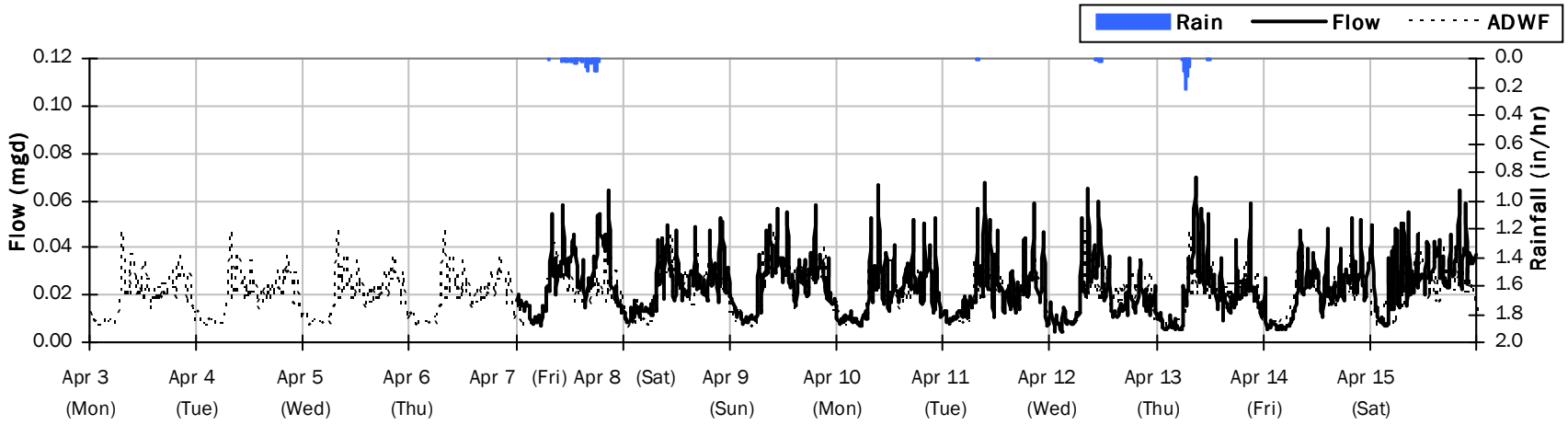
Total Period Rainfall: 0.85 inches

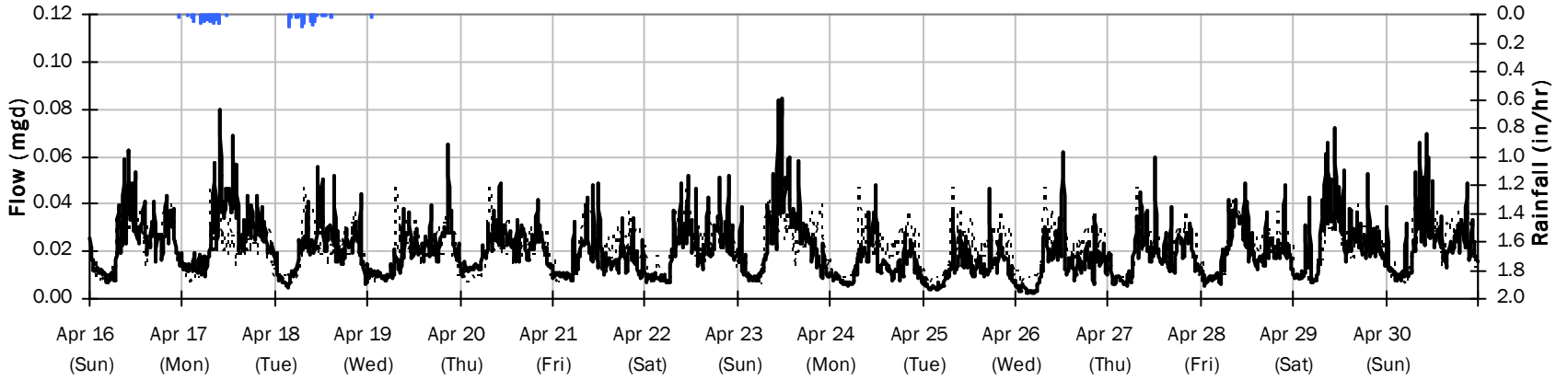


SITE 09

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.021 mgd** **Peak Flow: 0.084 mgd** **Min Flow: 0.002 mgd**

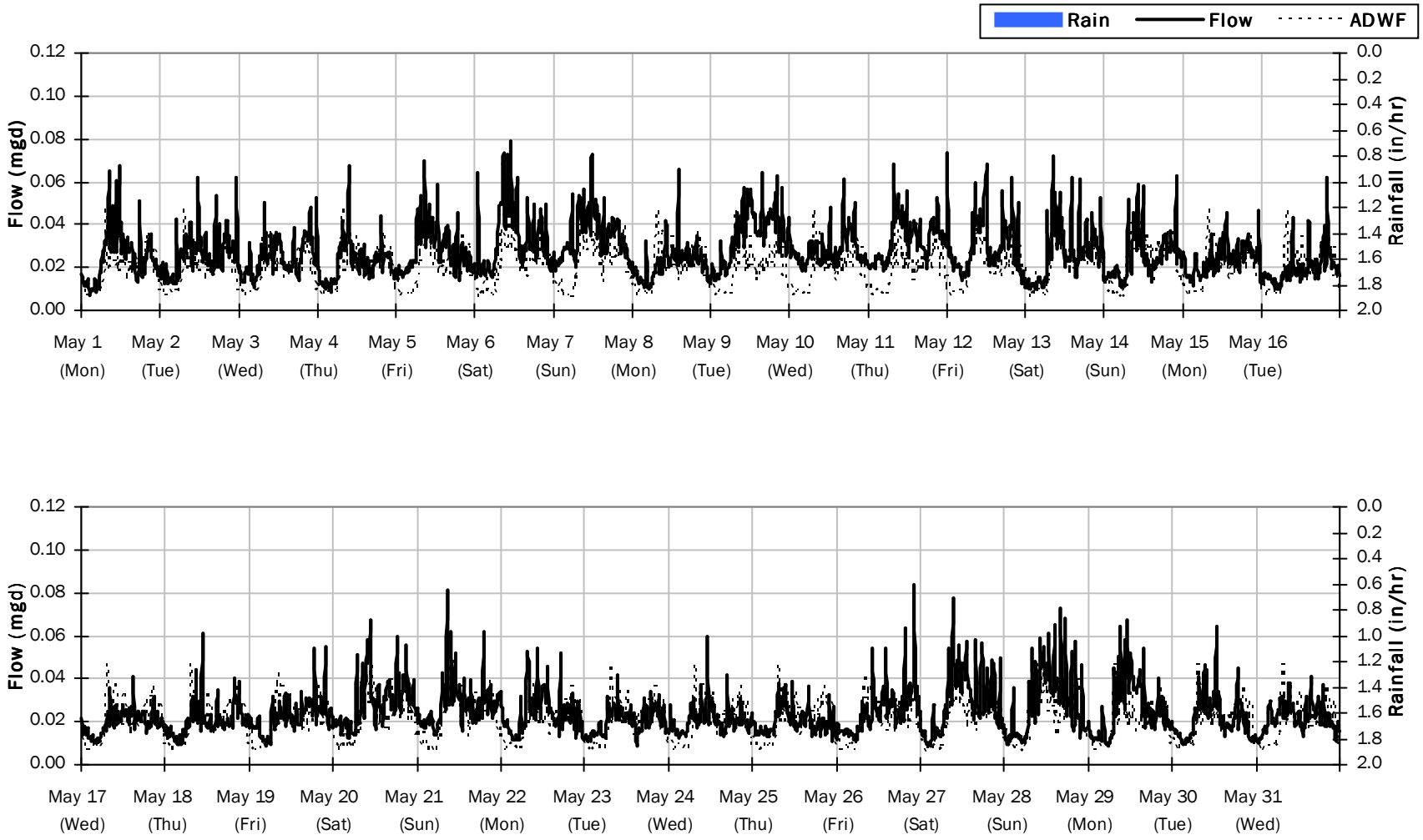




SITE 09

Flow Summary: 5/1/2017 to 5/31/2017

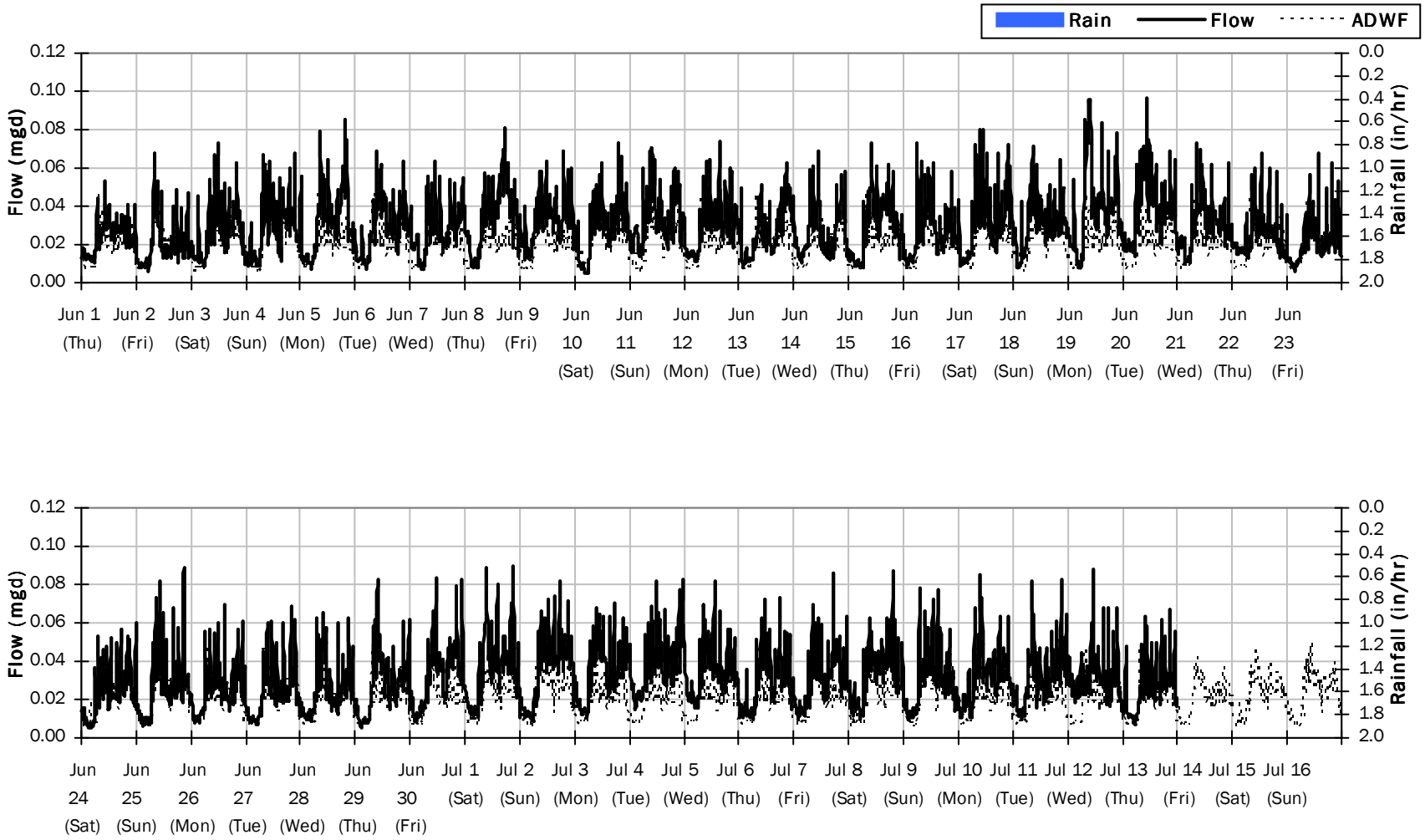
Total Period Rainfall: 0.00 inches **Avg Flow: 0.026 mgd** **Peak Flow: 0.084 mgd** **Min Flow: 0.007 mgd**



SITE 09

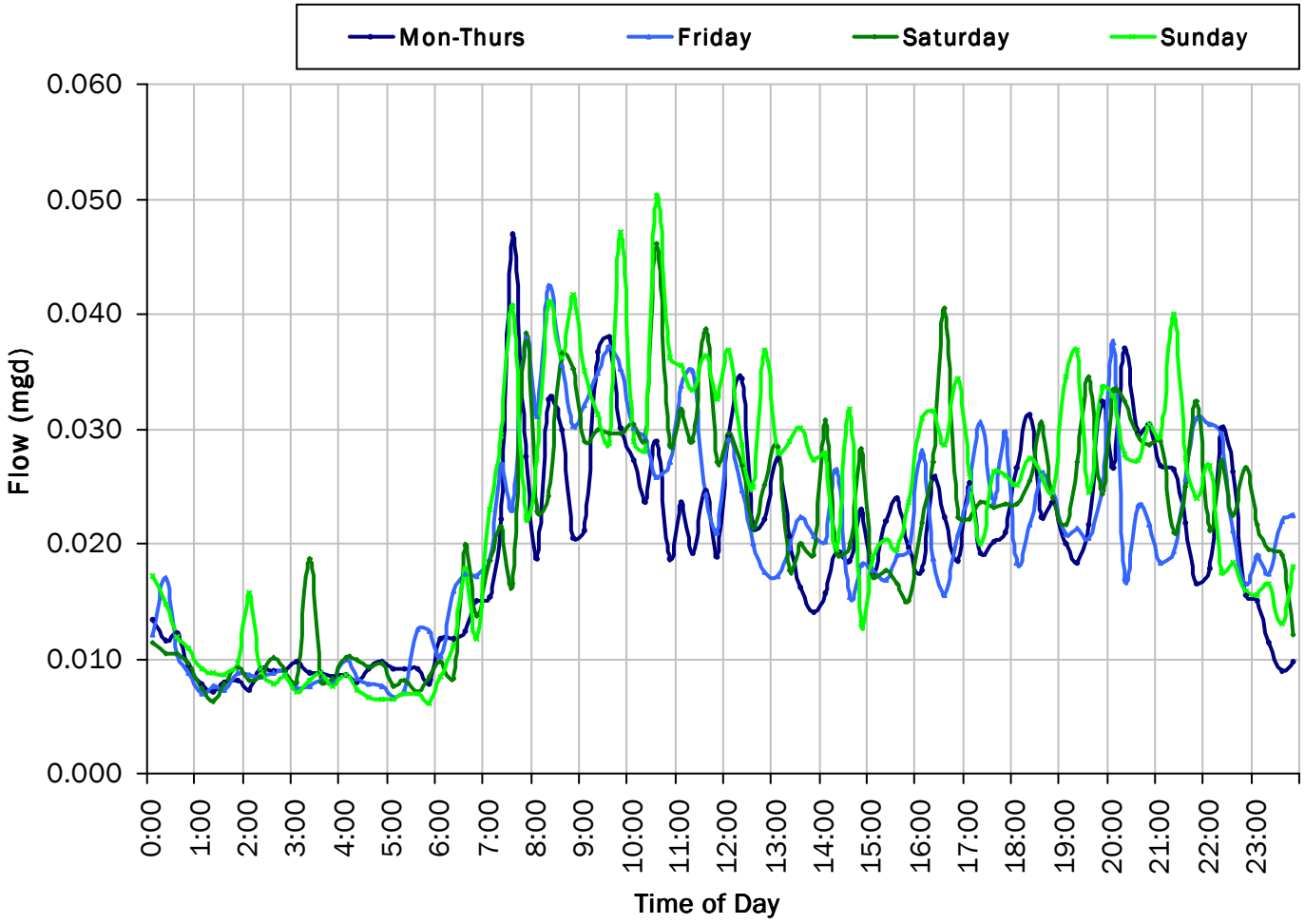
Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.030 mgd Peak Flow: 0.096 mgd Min Flow: 0.005 mgd**



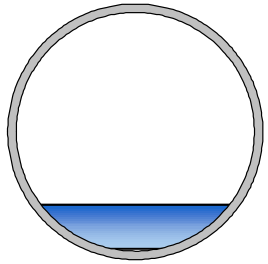
SITE 09

Average Dry Weather Flow Hydrographs



ADWF:

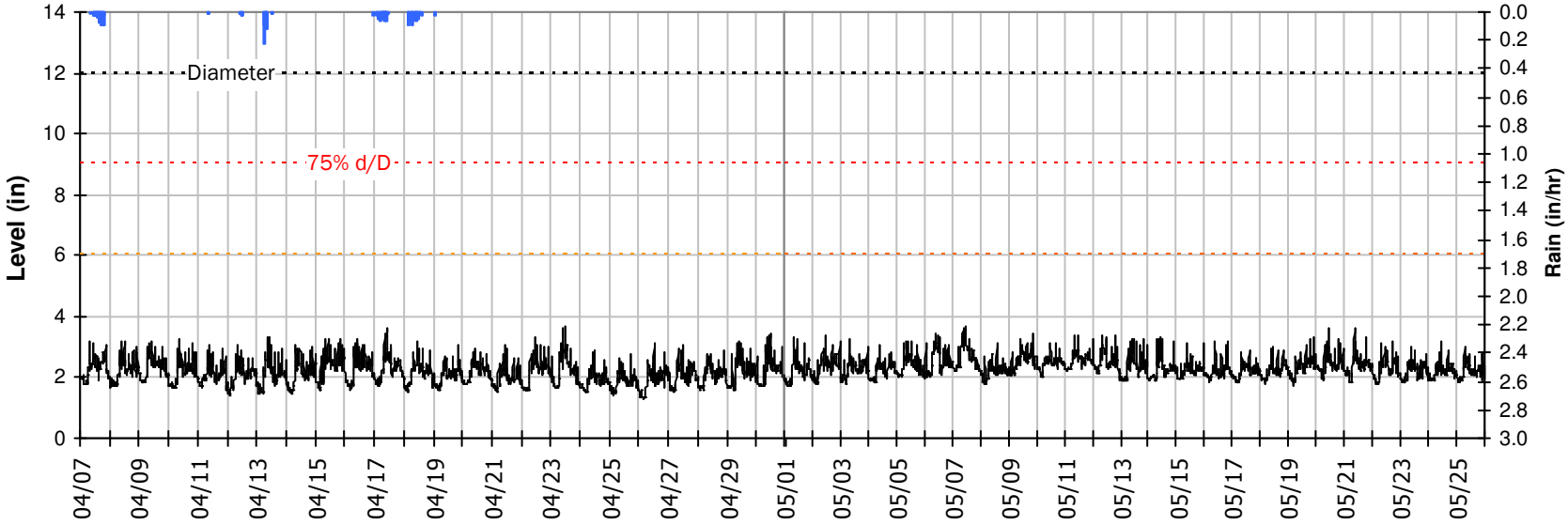
0.020 *mgd*

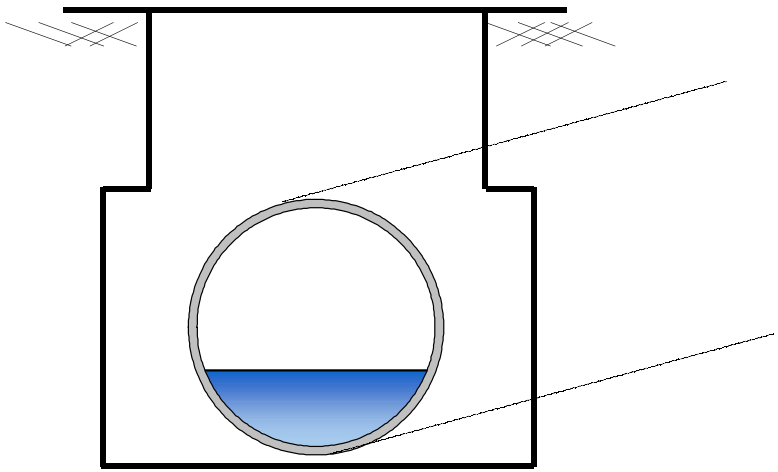


SITE 09

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



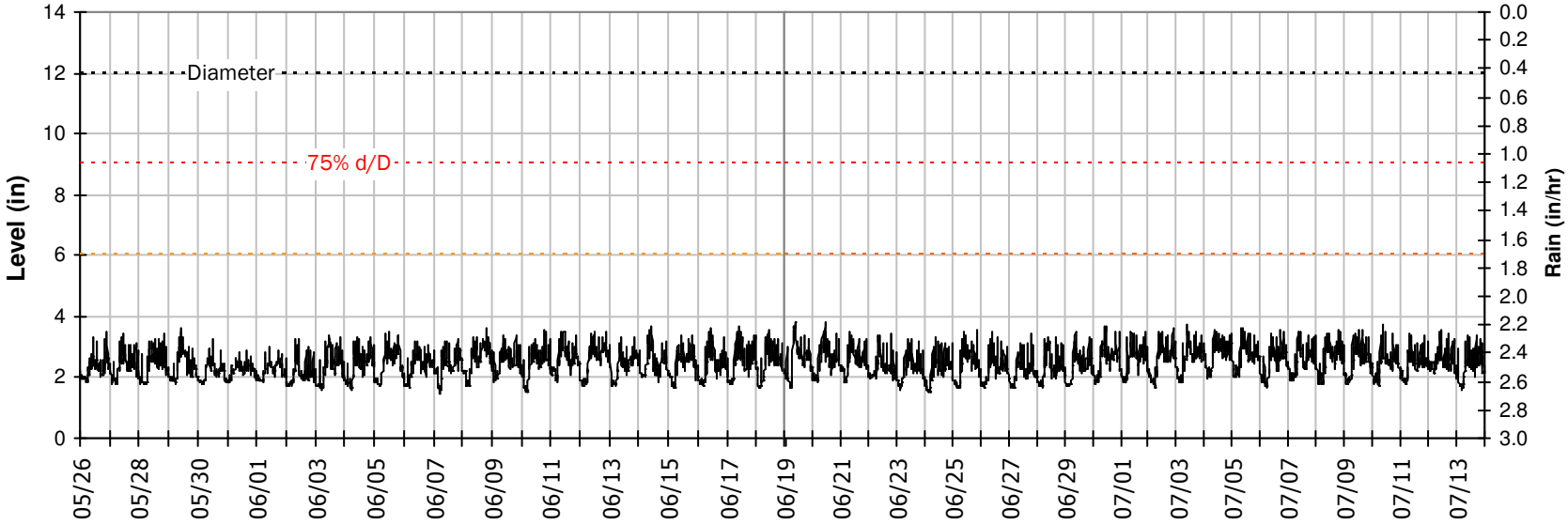


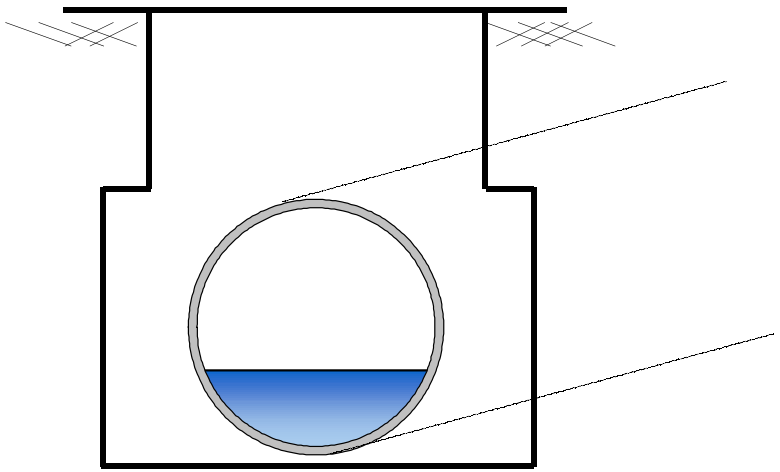
Pipe Diameter:	12	*inches*
Peak Measured Level:	3.82	*inches*
Peak d/D Ratio:	0.32	

SITE 09

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



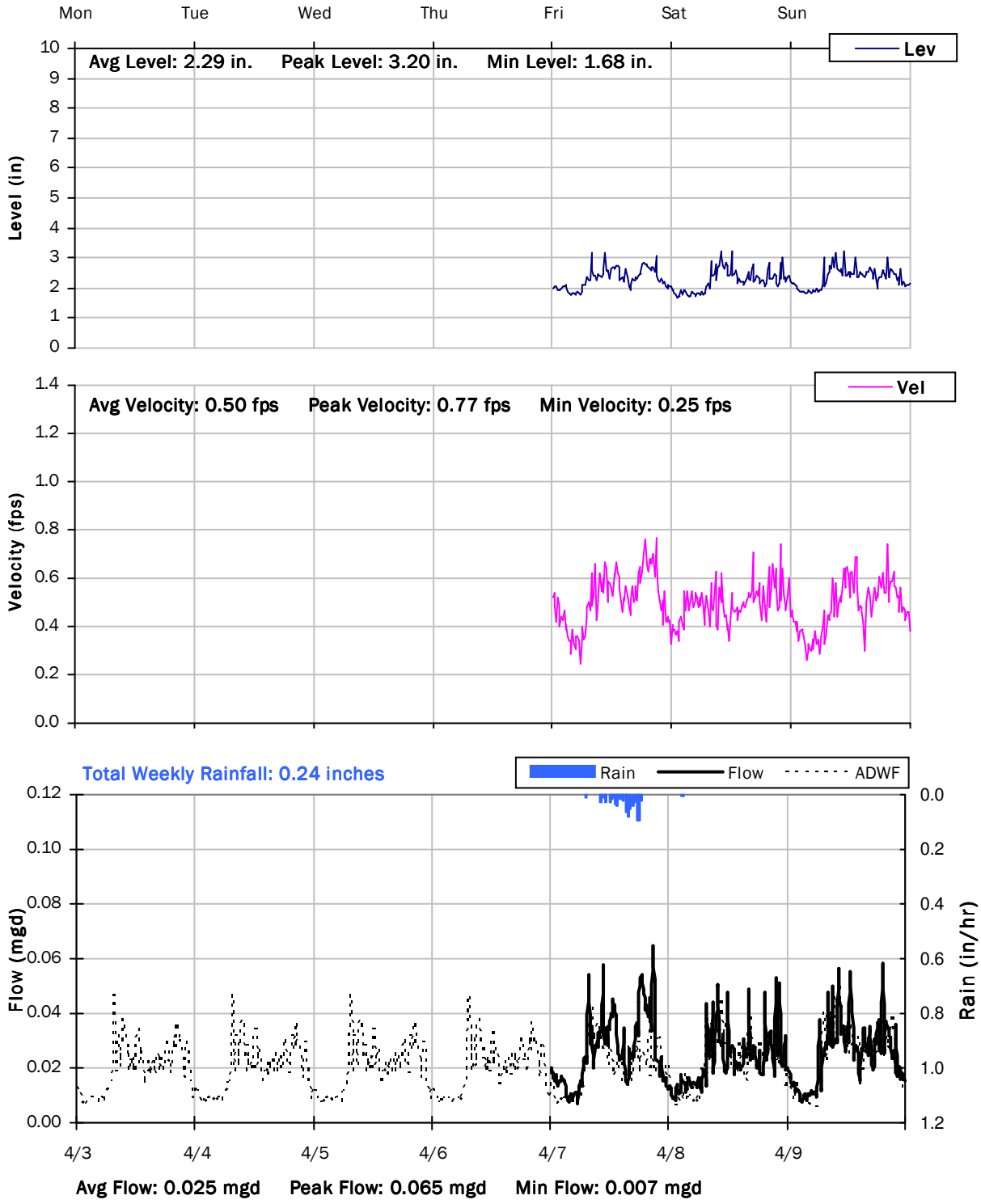


Pipe Diameter:	12	*inches*
Peak Measured Level:	3.82	*inches*
Peak d/D Ratio:	0.32	

SITE 09

Weekly Level, Velocity and Flow Hydrographs

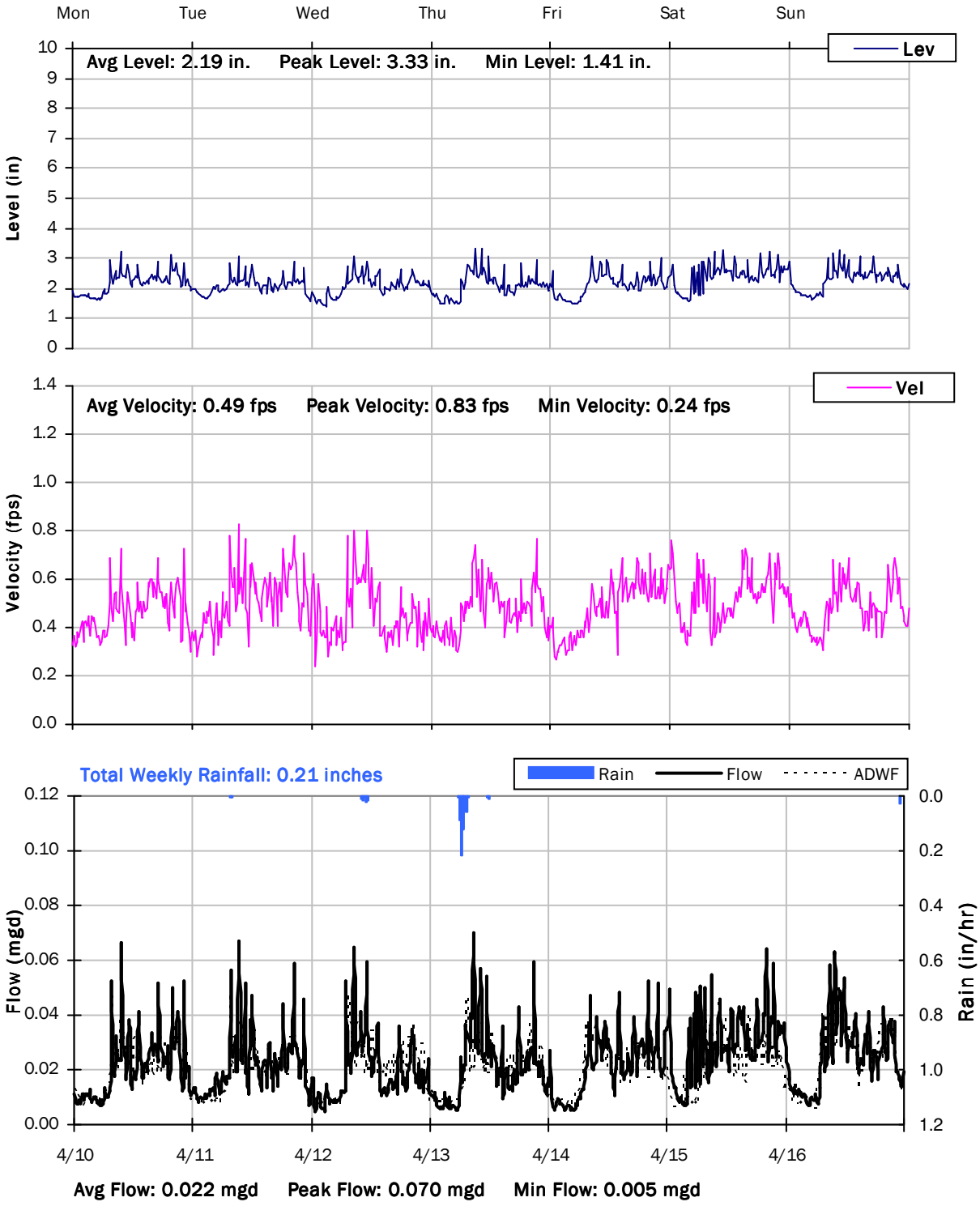
4/3/2017 to 4/10/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

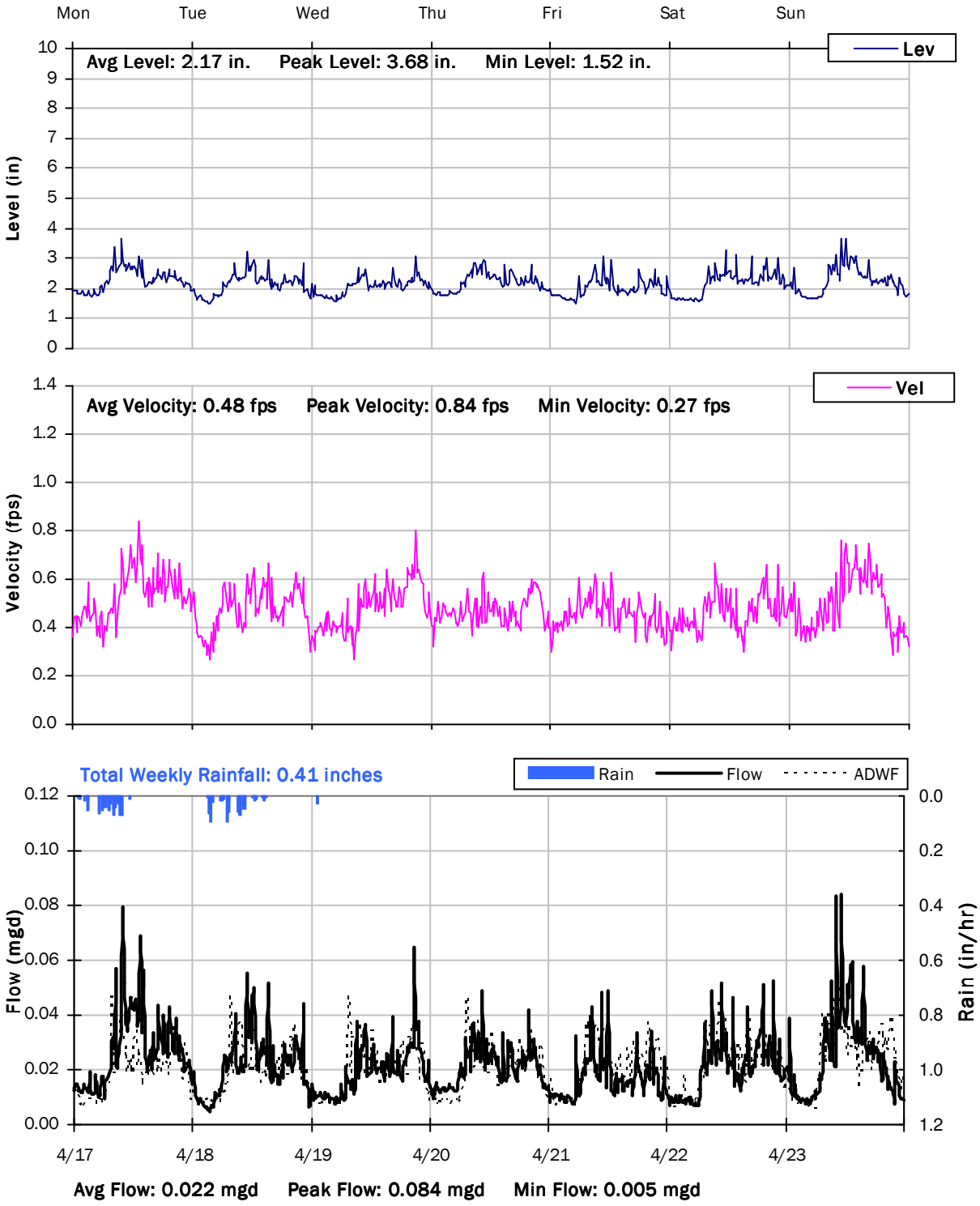
4/10/2017 to 4/17/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

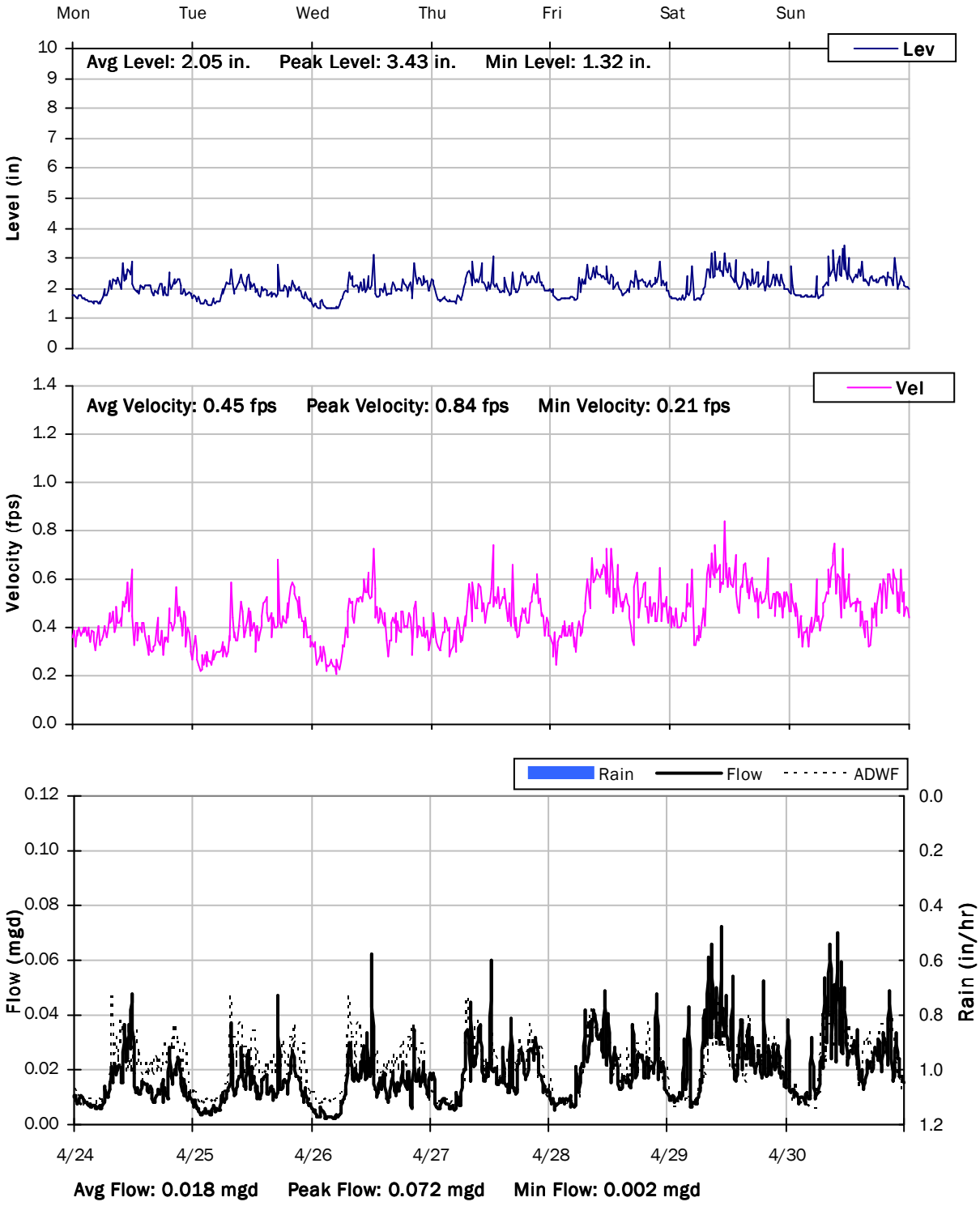
4/17/2017 to 4/24/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

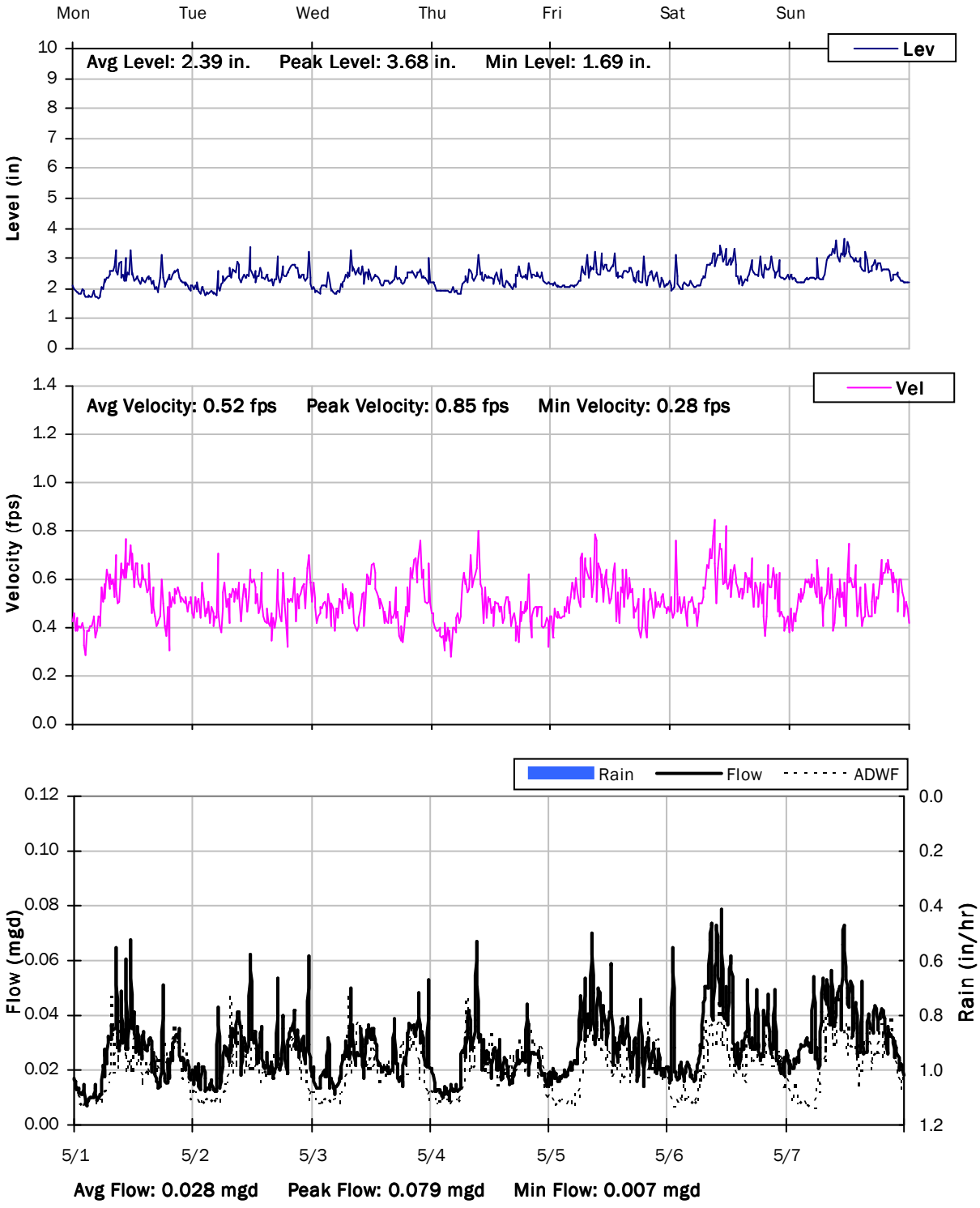
4/24/2017 to 5/1/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

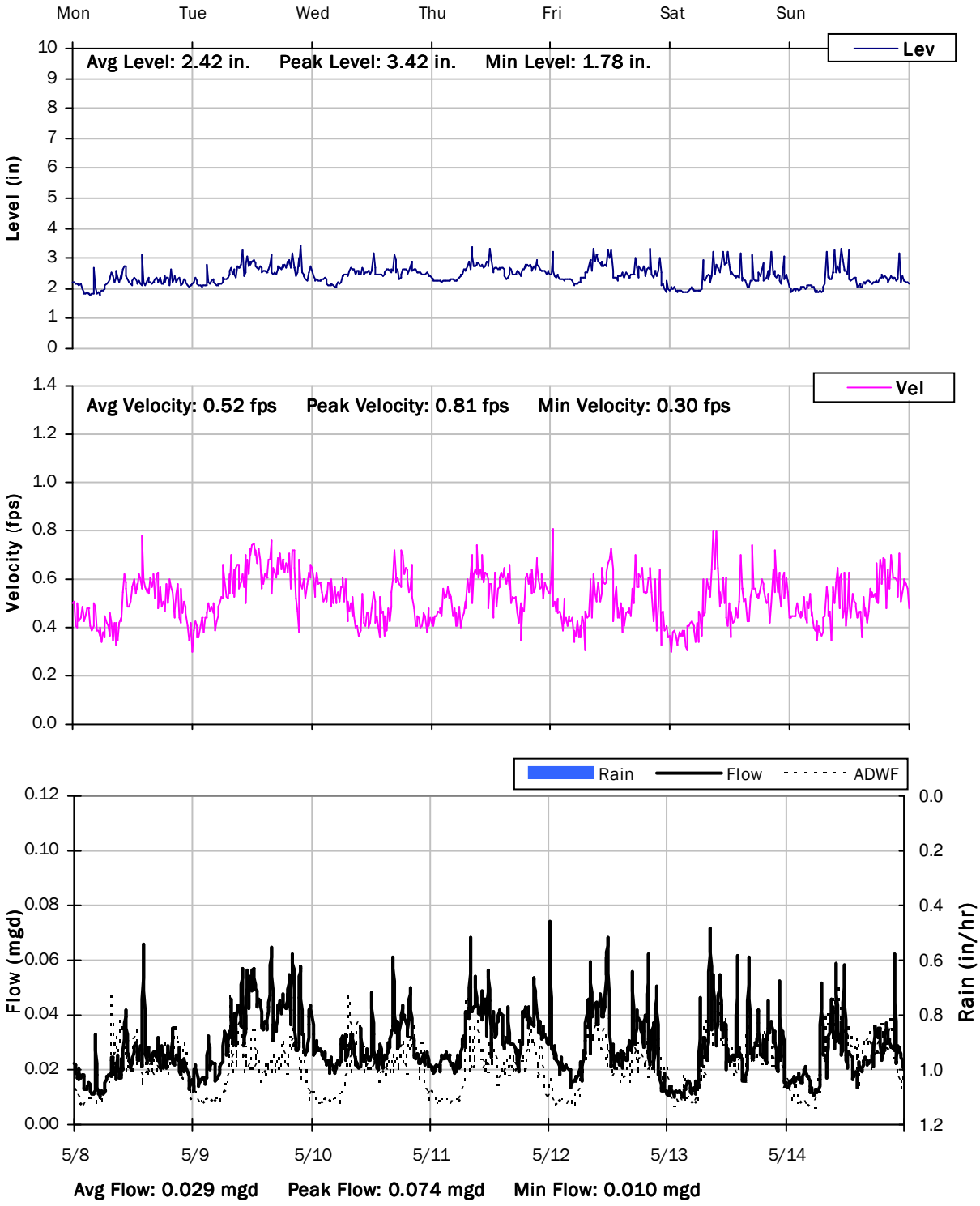
5/1/2017 to 5/8/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

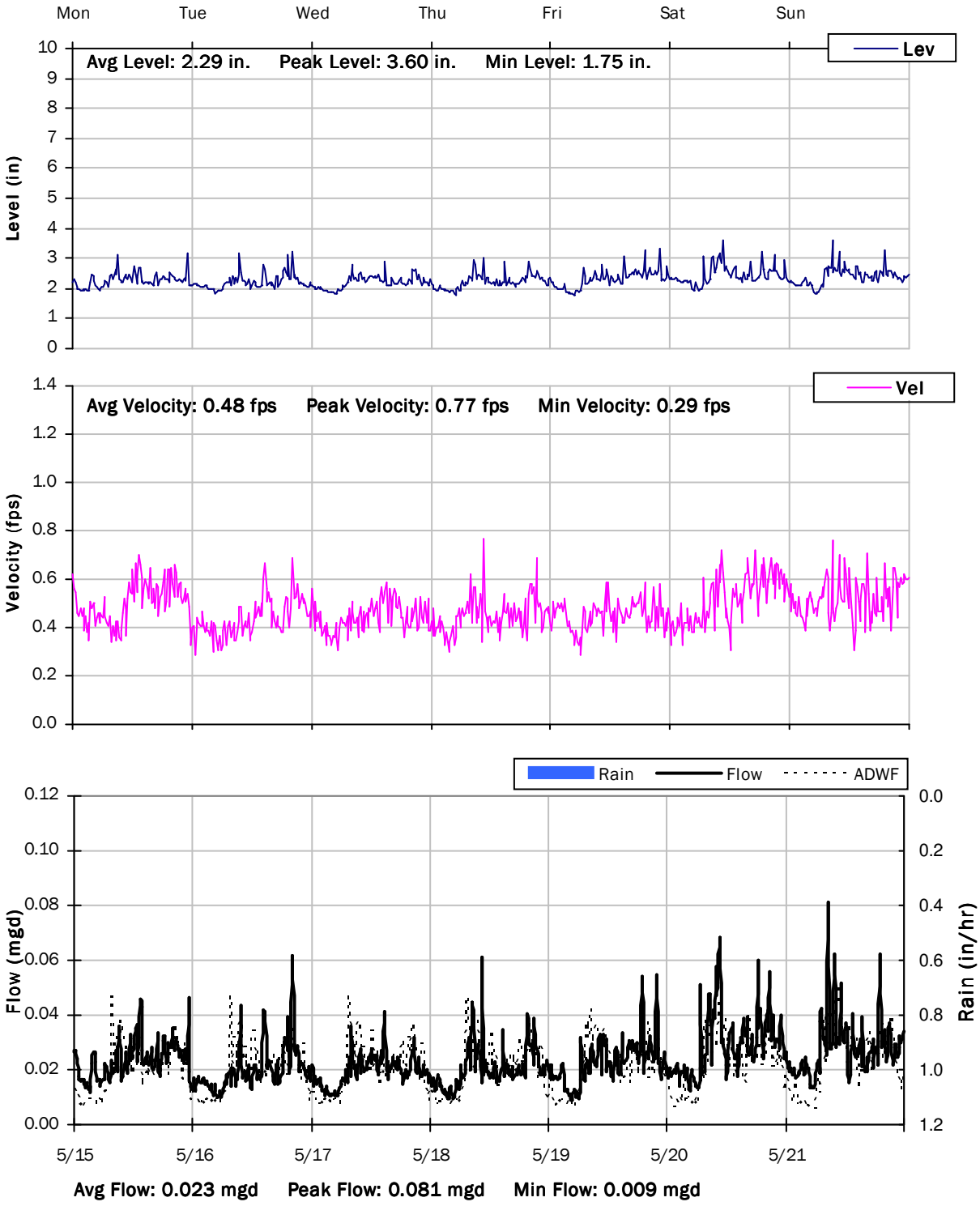
5/8/2017 to 5/15/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

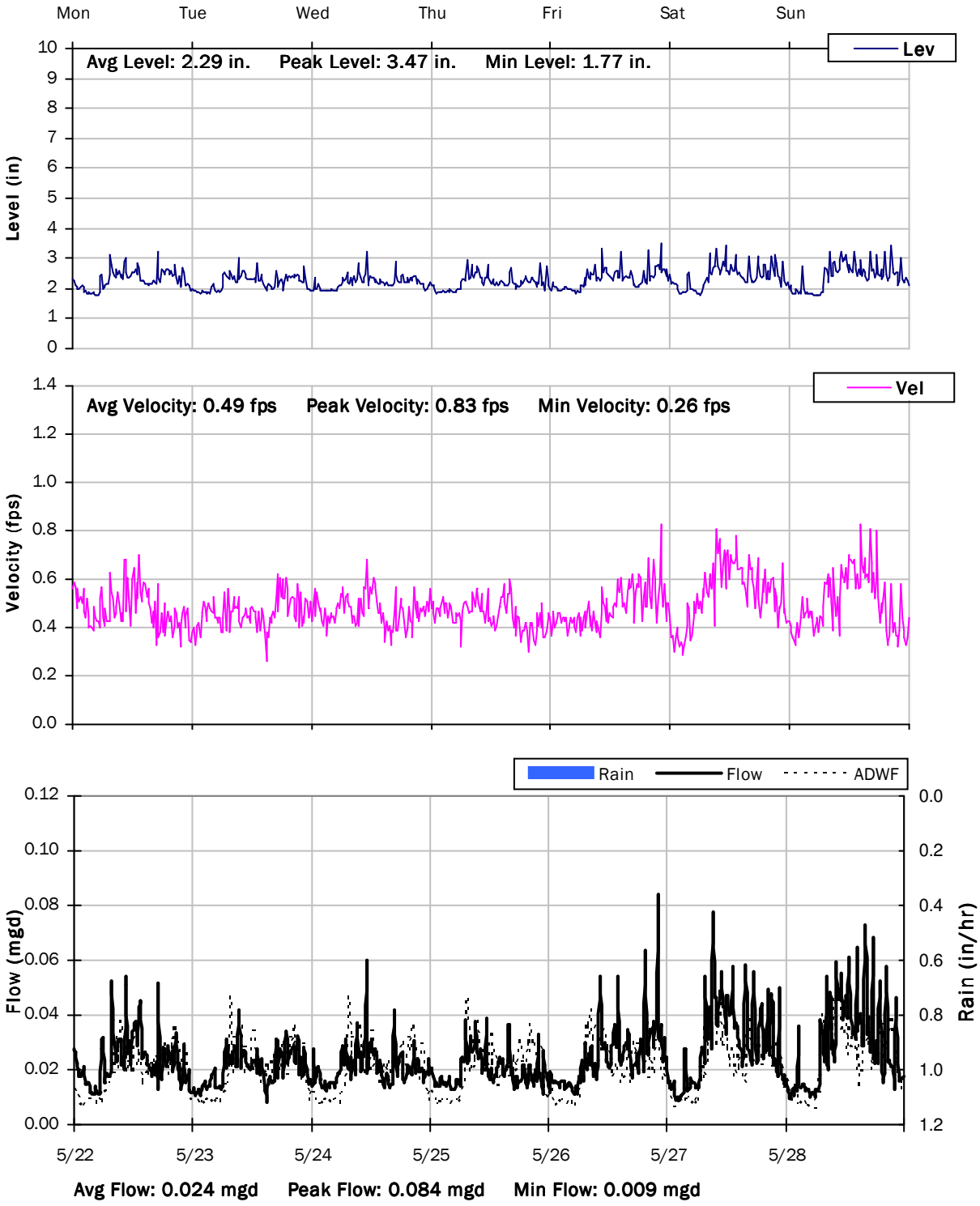
5/15/2017 to 5/22/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

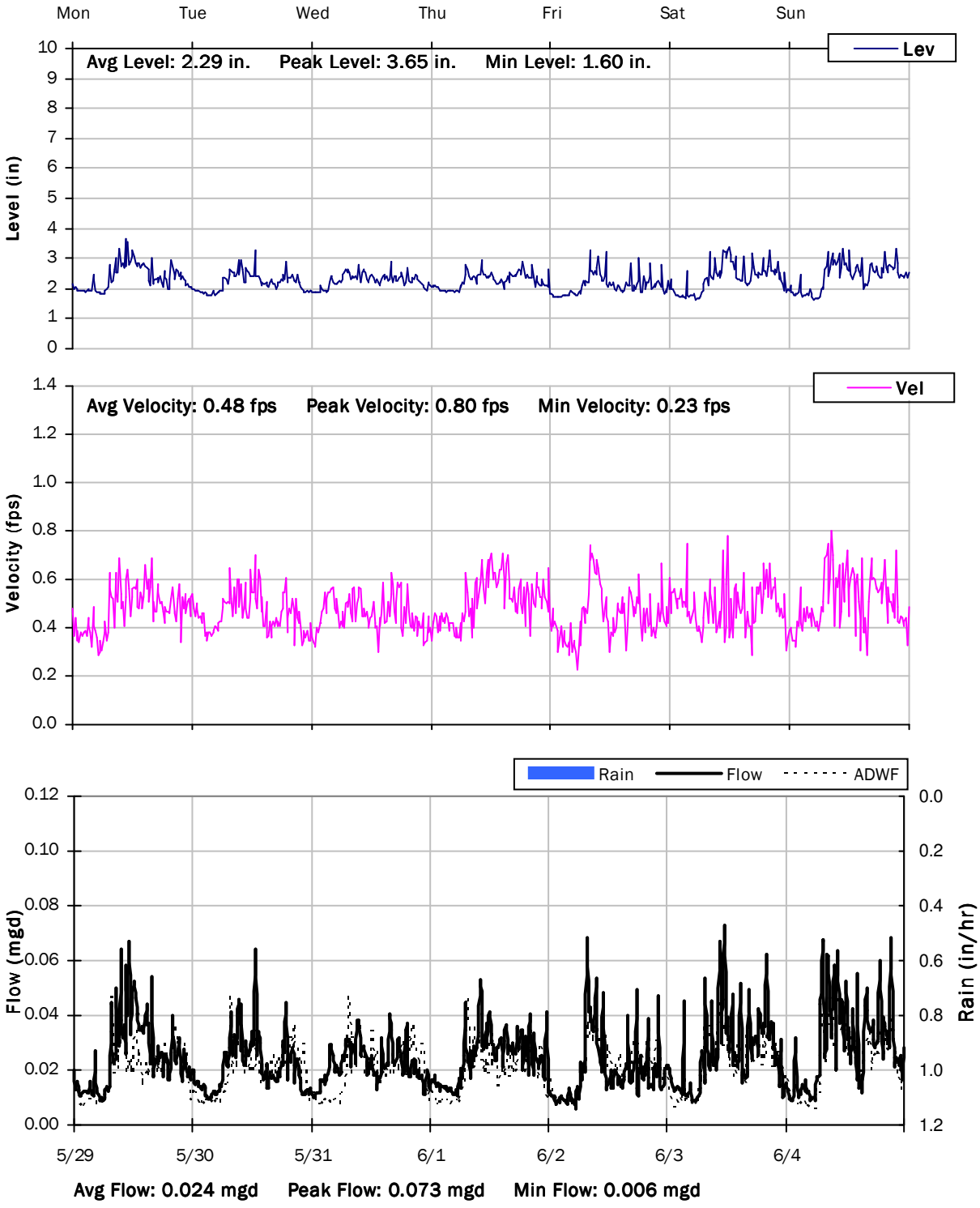
5/22/2017 to 5/29/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

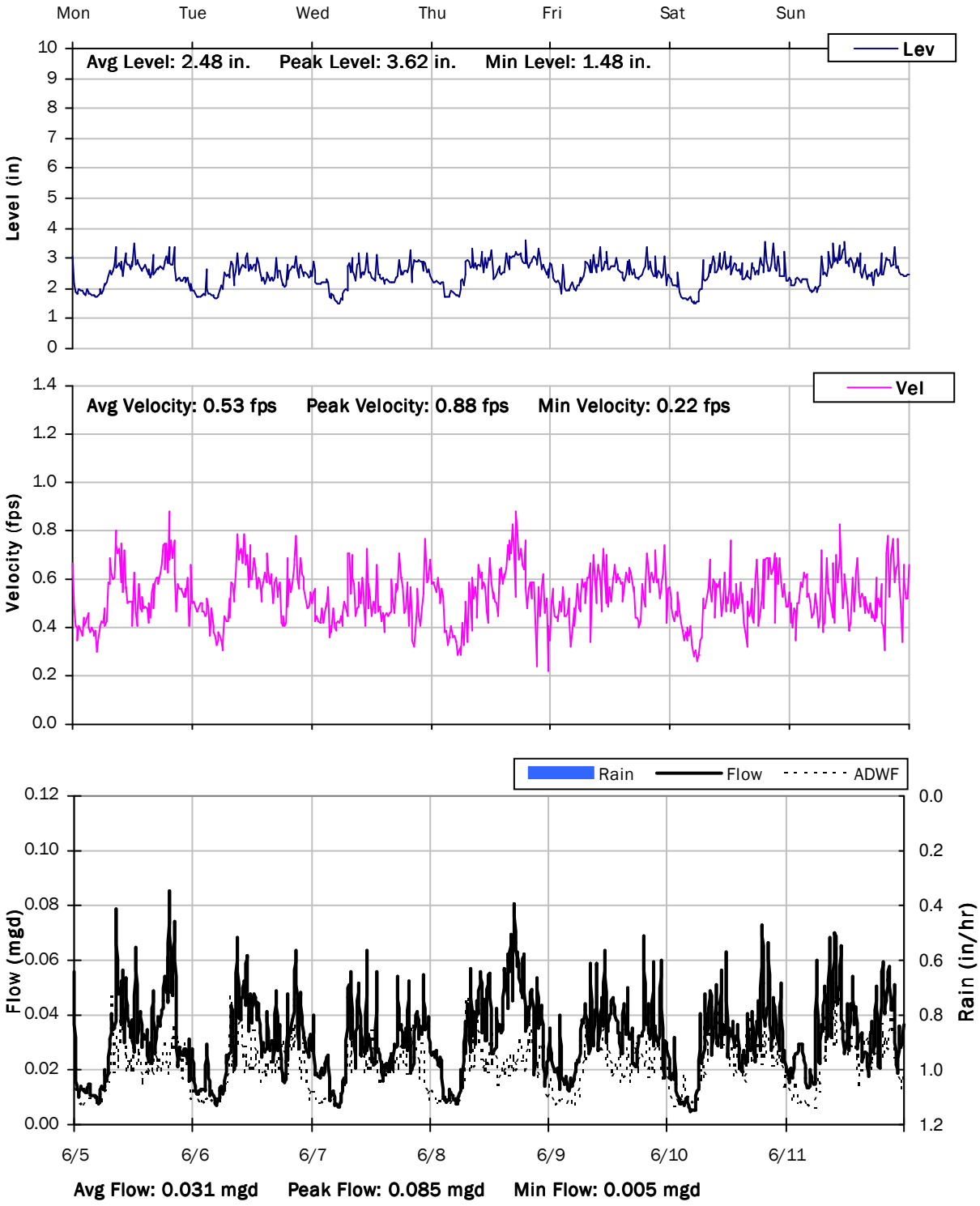
5/29/2017 to 6/5/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

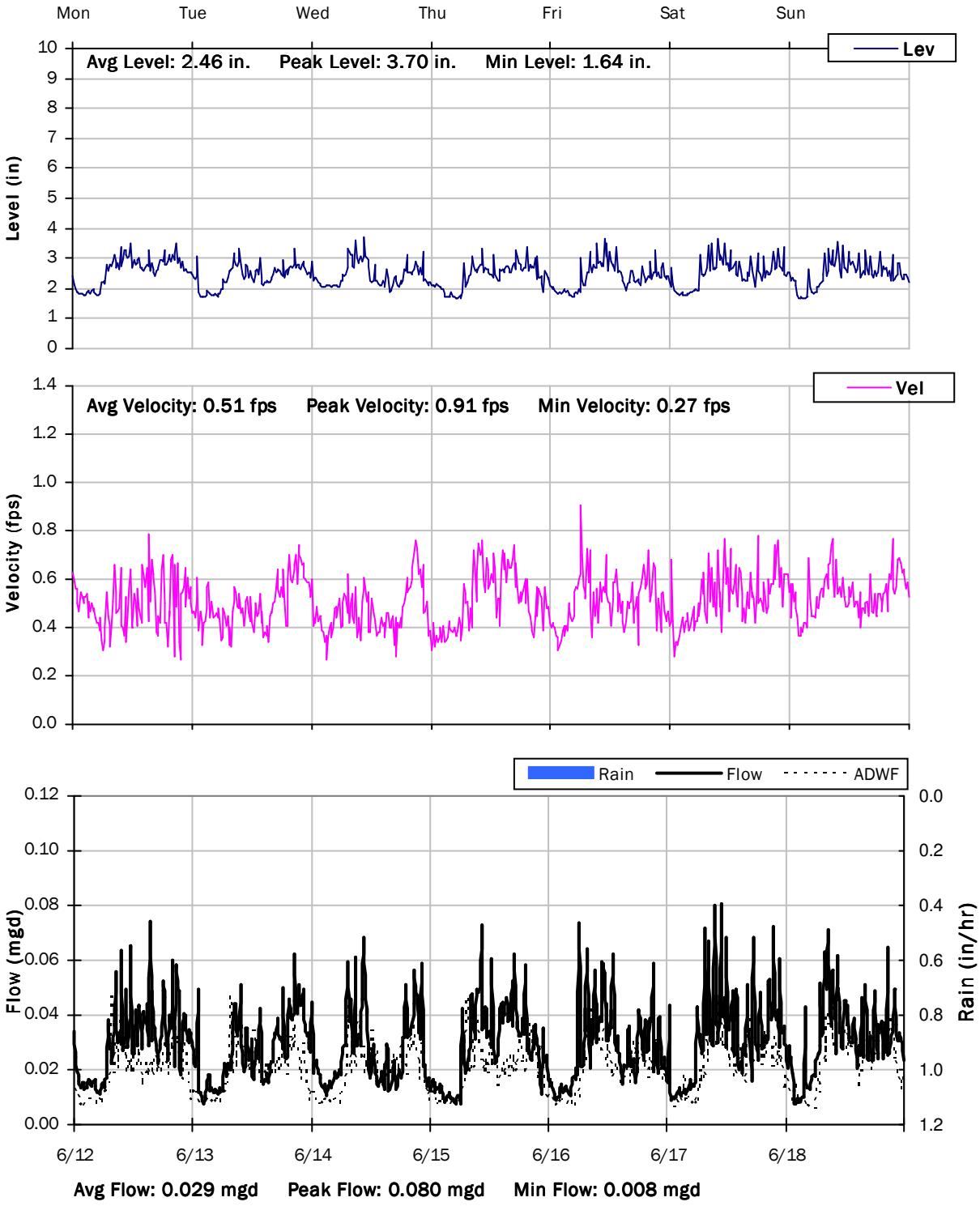
6/5/2017 to 6/12/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

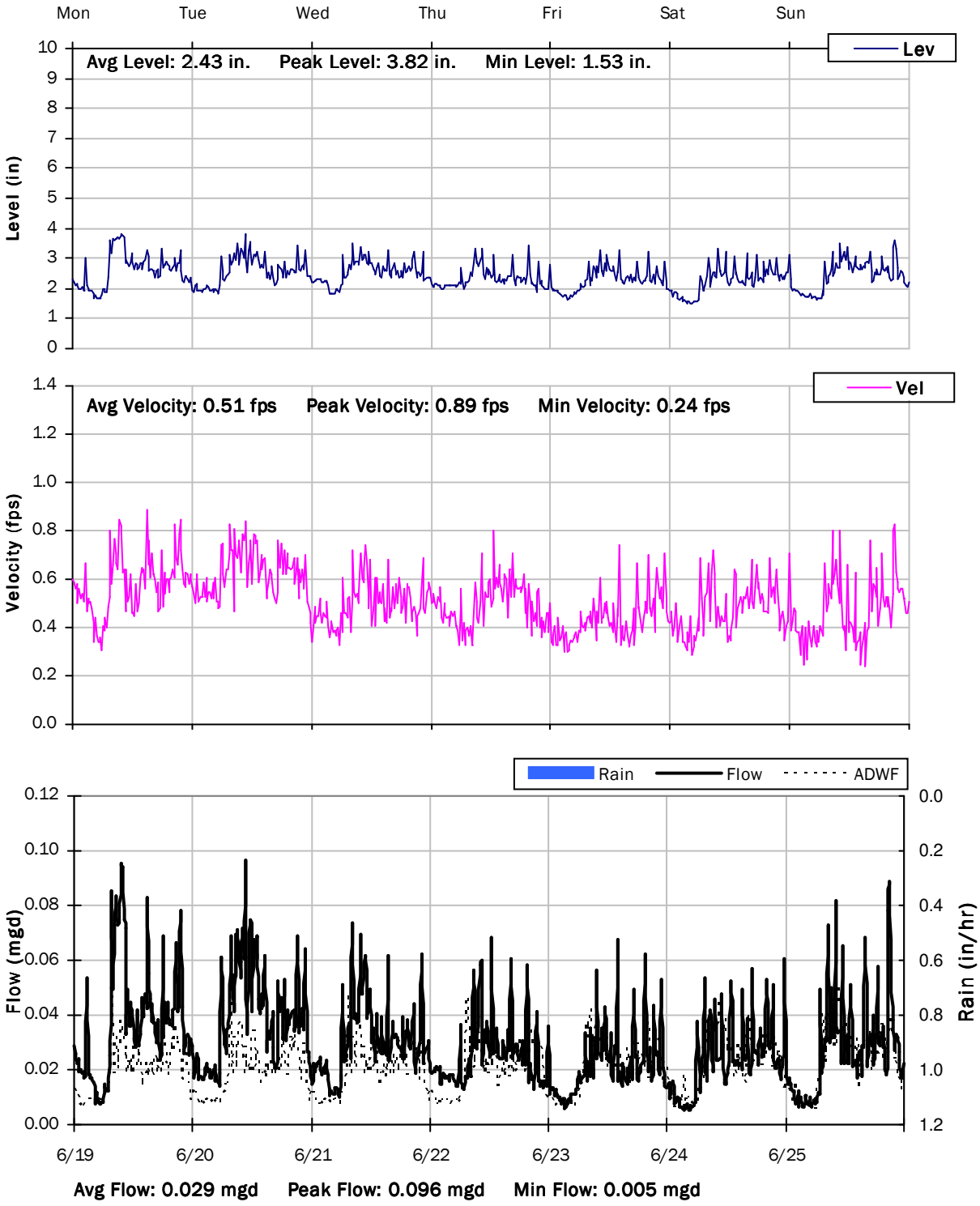
6/12/2017 to 6/19/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

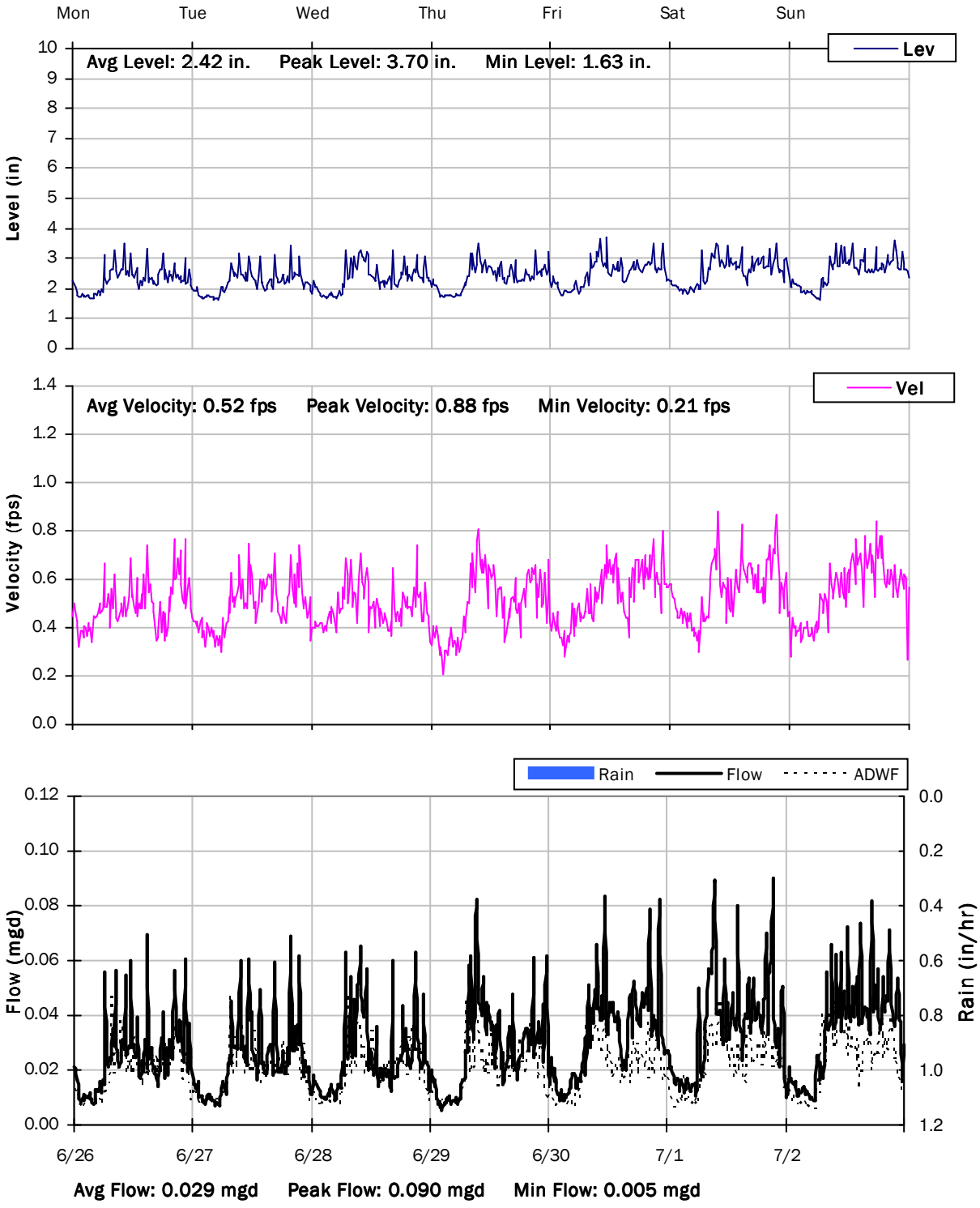
6/19/2017 to 6/26/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

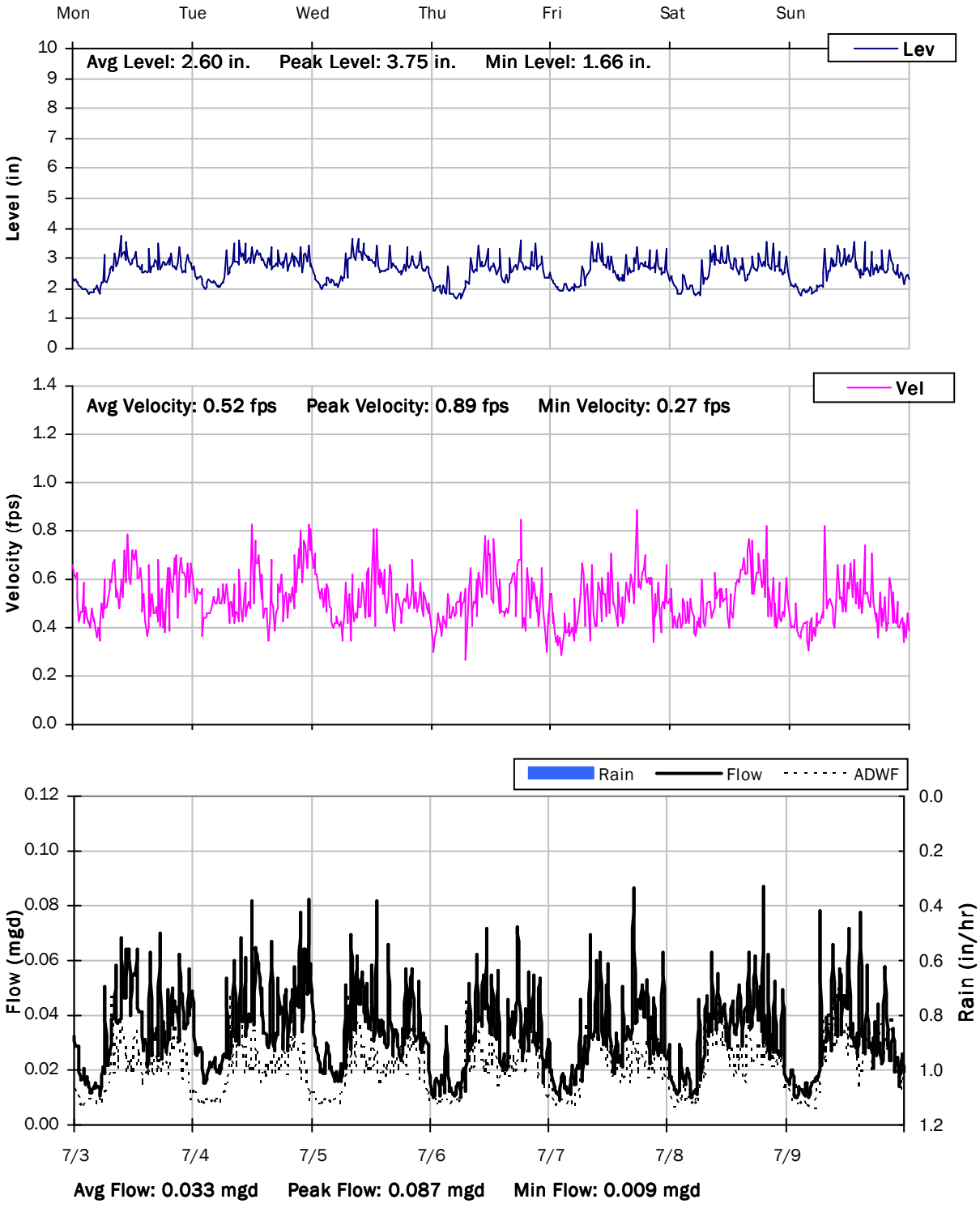
6/26/2017 to 7/3/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

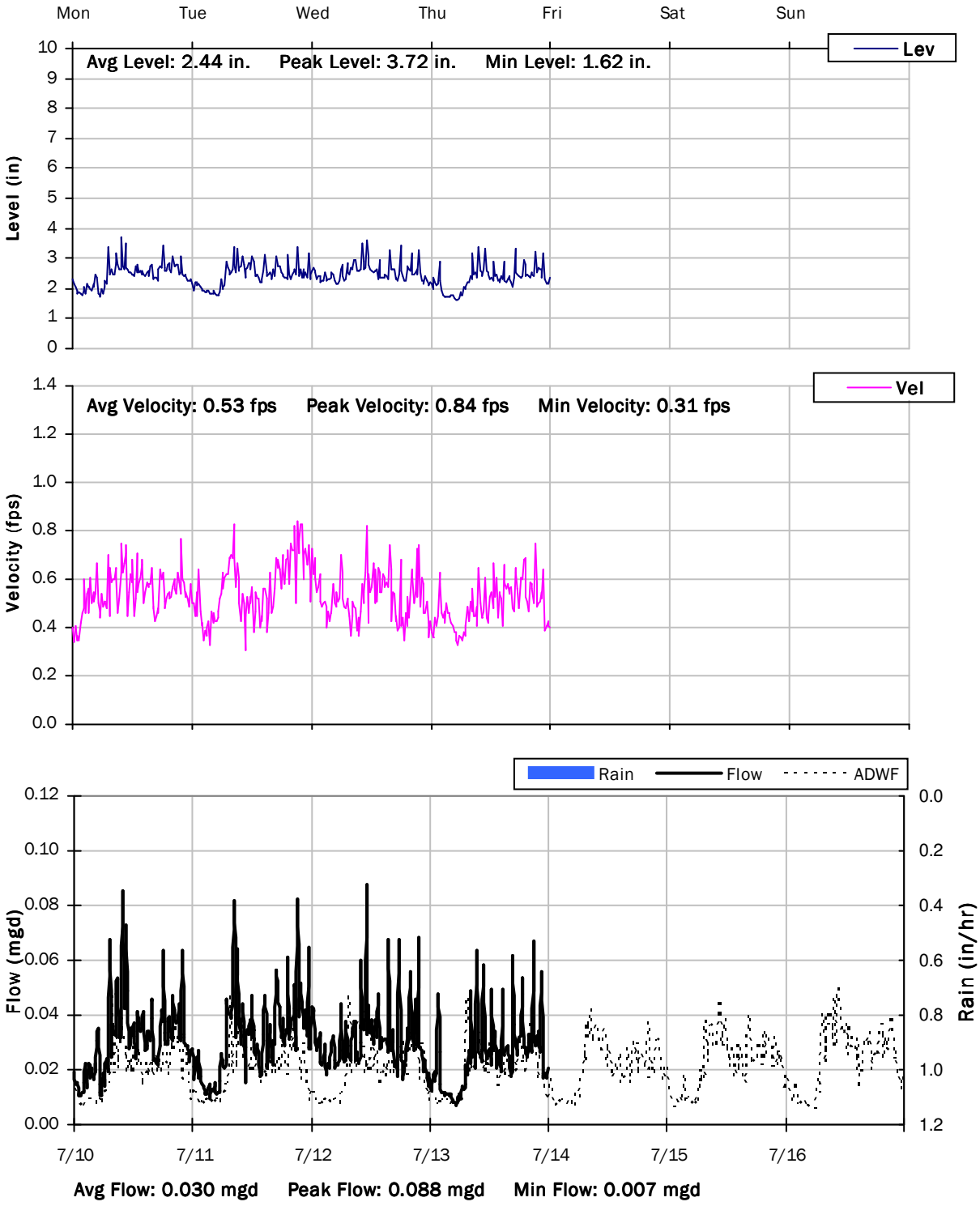
7/3/2017 to 7/10/2017



SITE 09

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

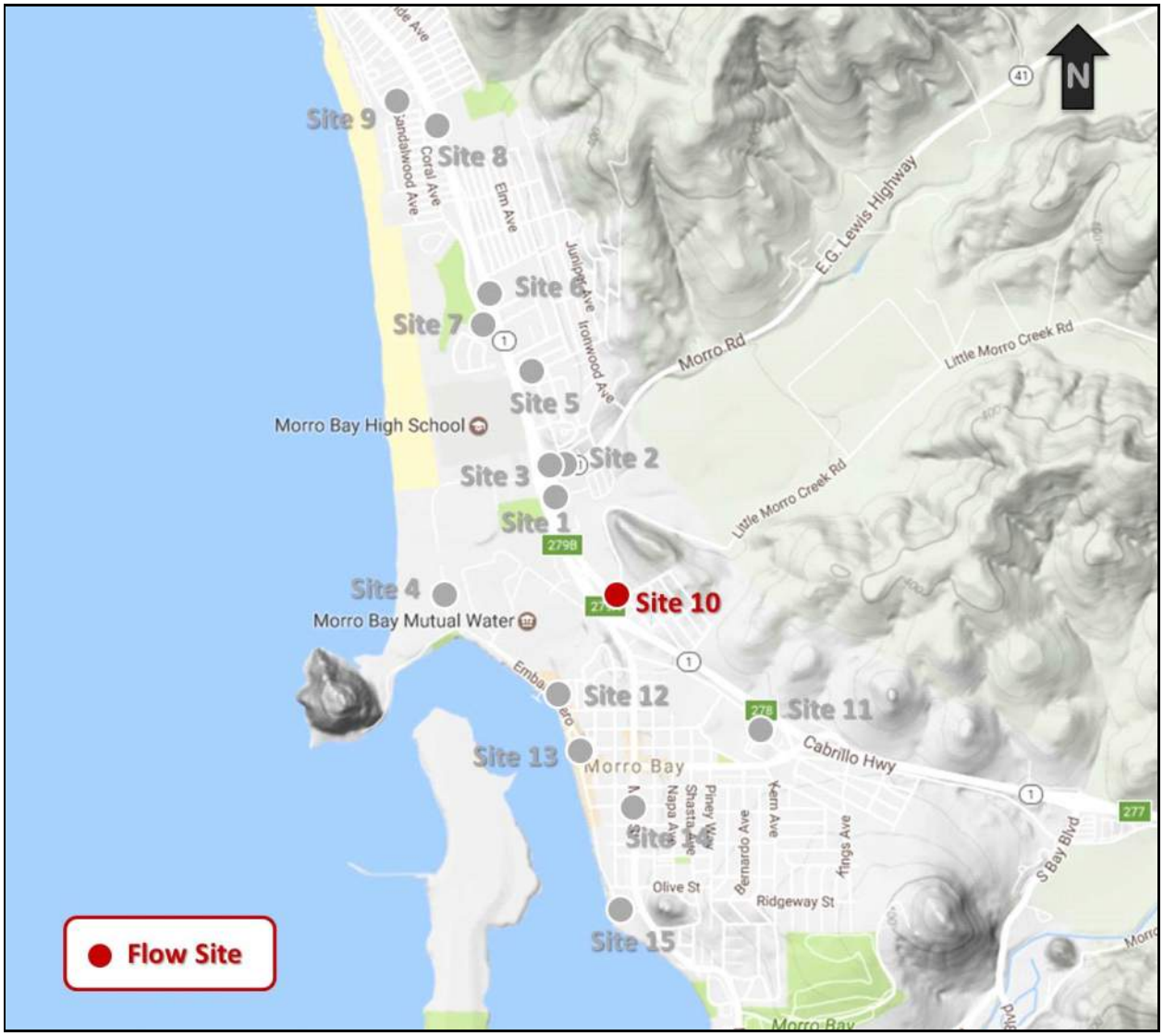
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 10

Location: Main Street, just north of intersection with Radcliff Avenue, meter on south 8" inlet

Data Summary Report



Vicinity Map: Site 10

SITE 10

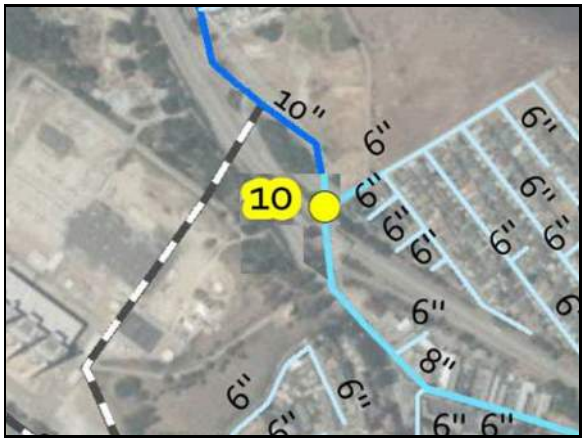
Site Information

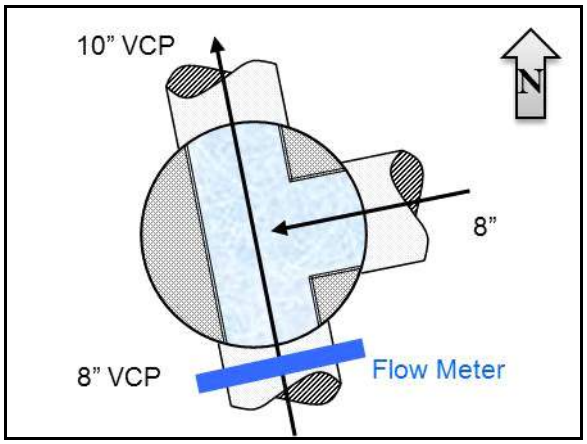
Location: Main Street, just north of intersection with Radcliff Avenue, meter on south 8" inlet

Pipe Diameter: 8 inches

ADWF: 0.070 mgd

Peak Measured Flow: 0.325 mgd





SITE 10

Additional Site Photos

SITE 10

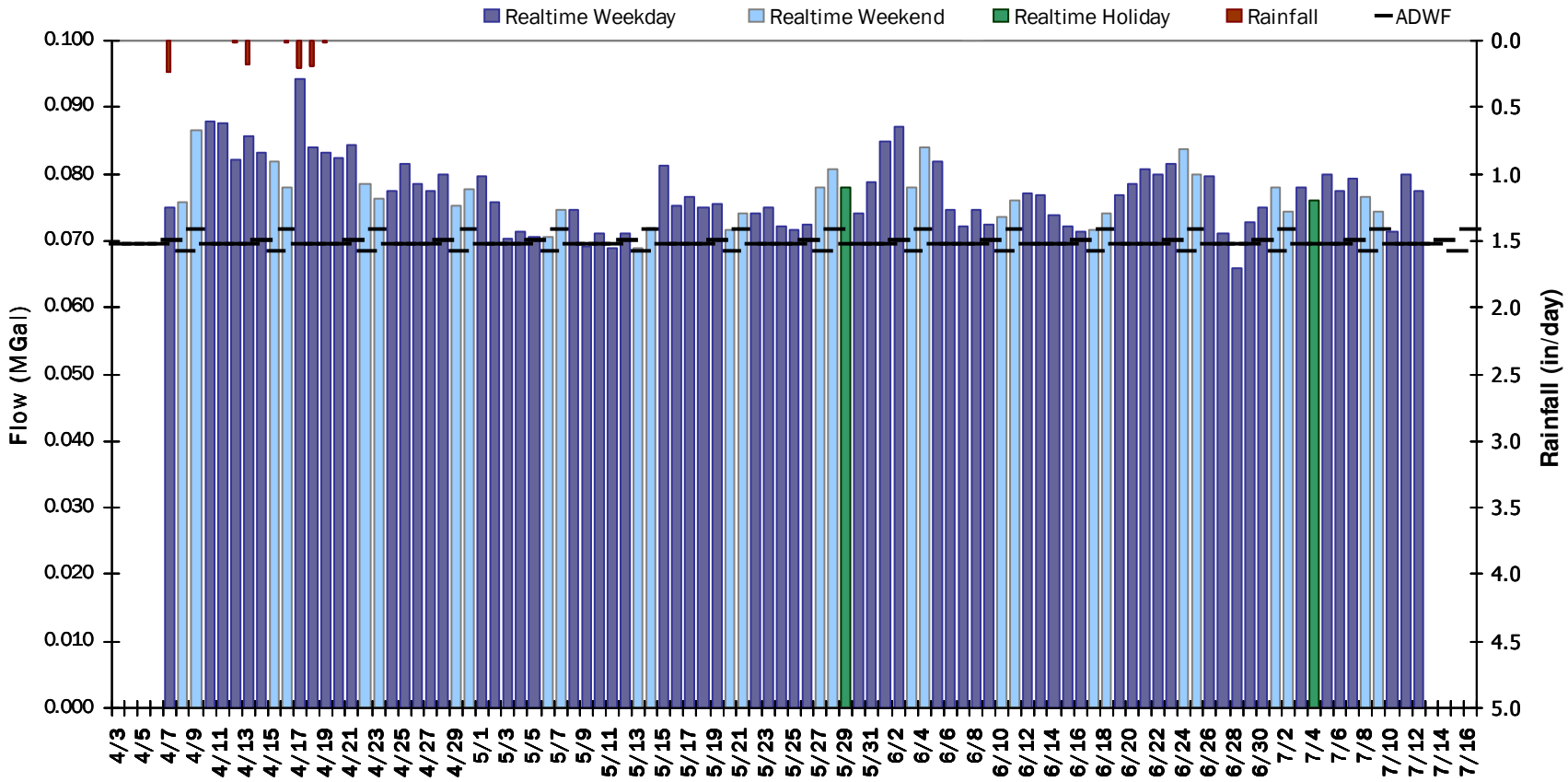
Additional Site Photos

SITE 10

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.077 MGal Peak Daily Flow: 0.094 MGal Min Daily Flow: 0.066 MGal

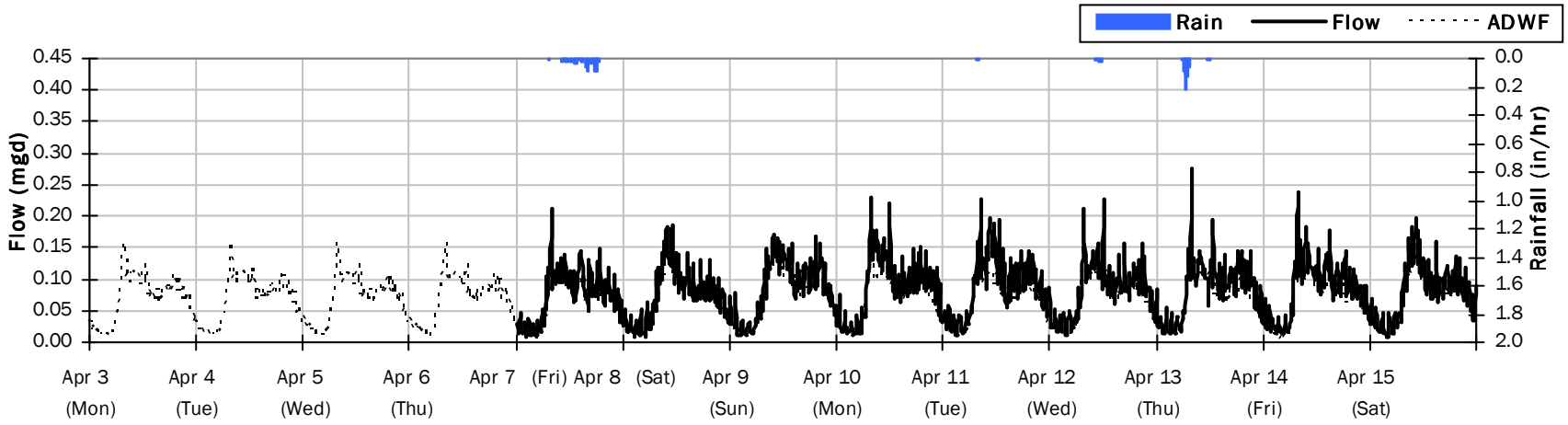
Total Period Rainfall: 0.85 inches

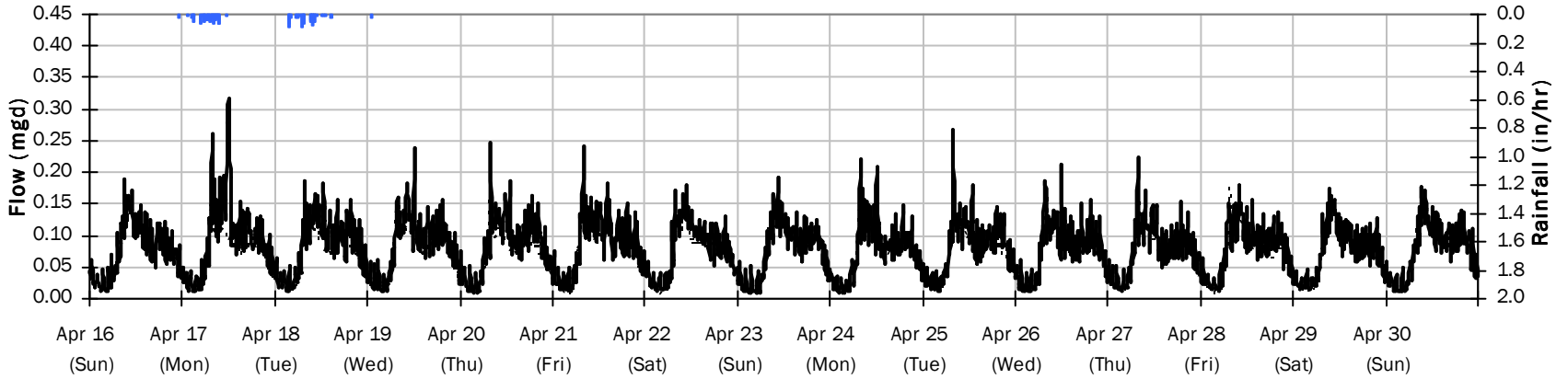


SITE 10

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.081 mgd** **Peak Flow: 0.316 mgd** **Min Flow: 0.009 mgd**

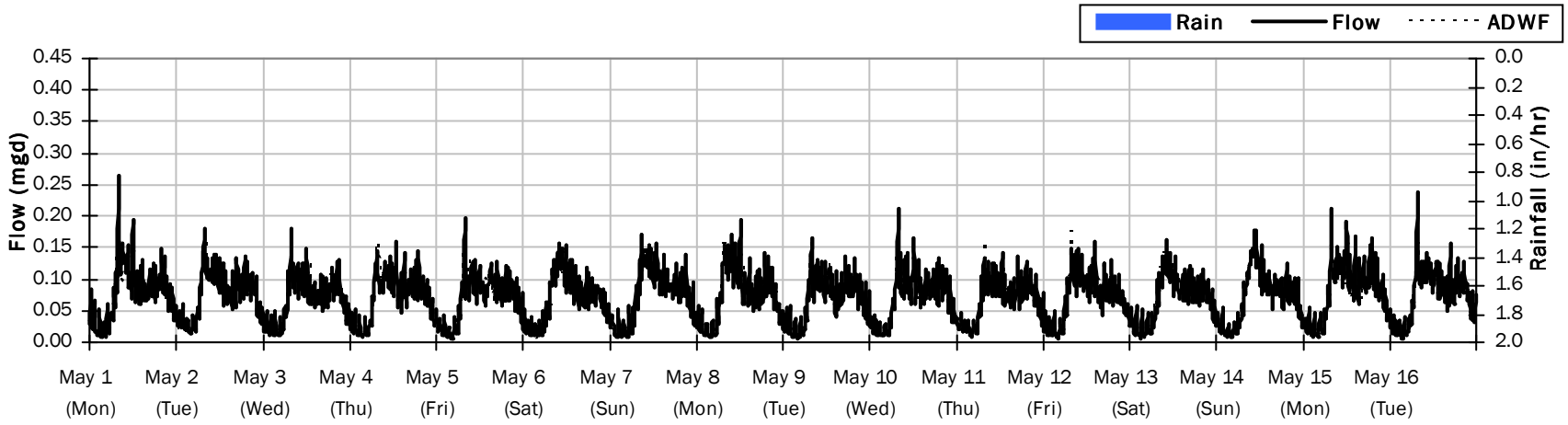


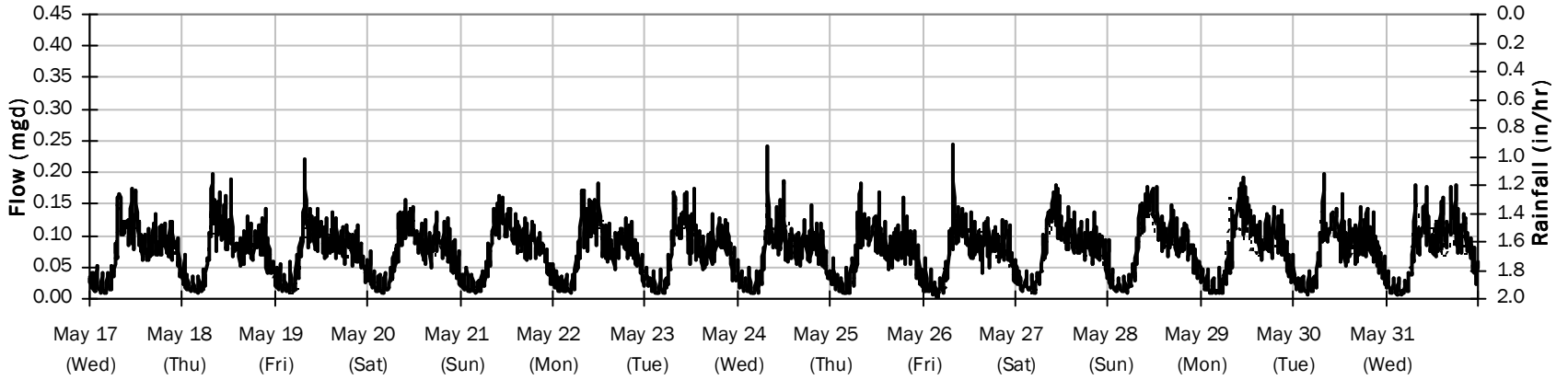


SITE 10

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.074 mgd** **Peak Flow: 0.264 mgd** **Min Flow: 0.004 mgd**

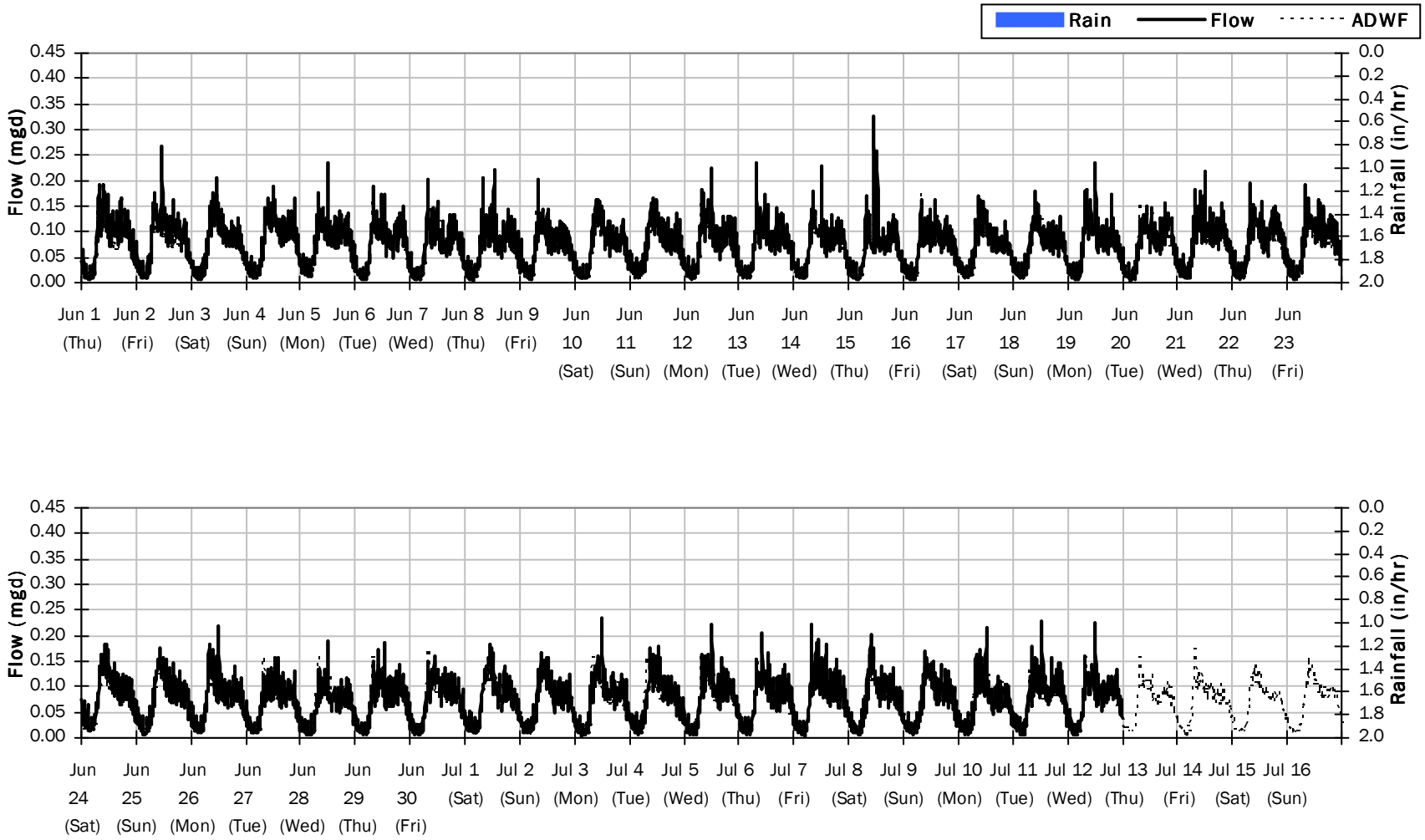




SITE 10

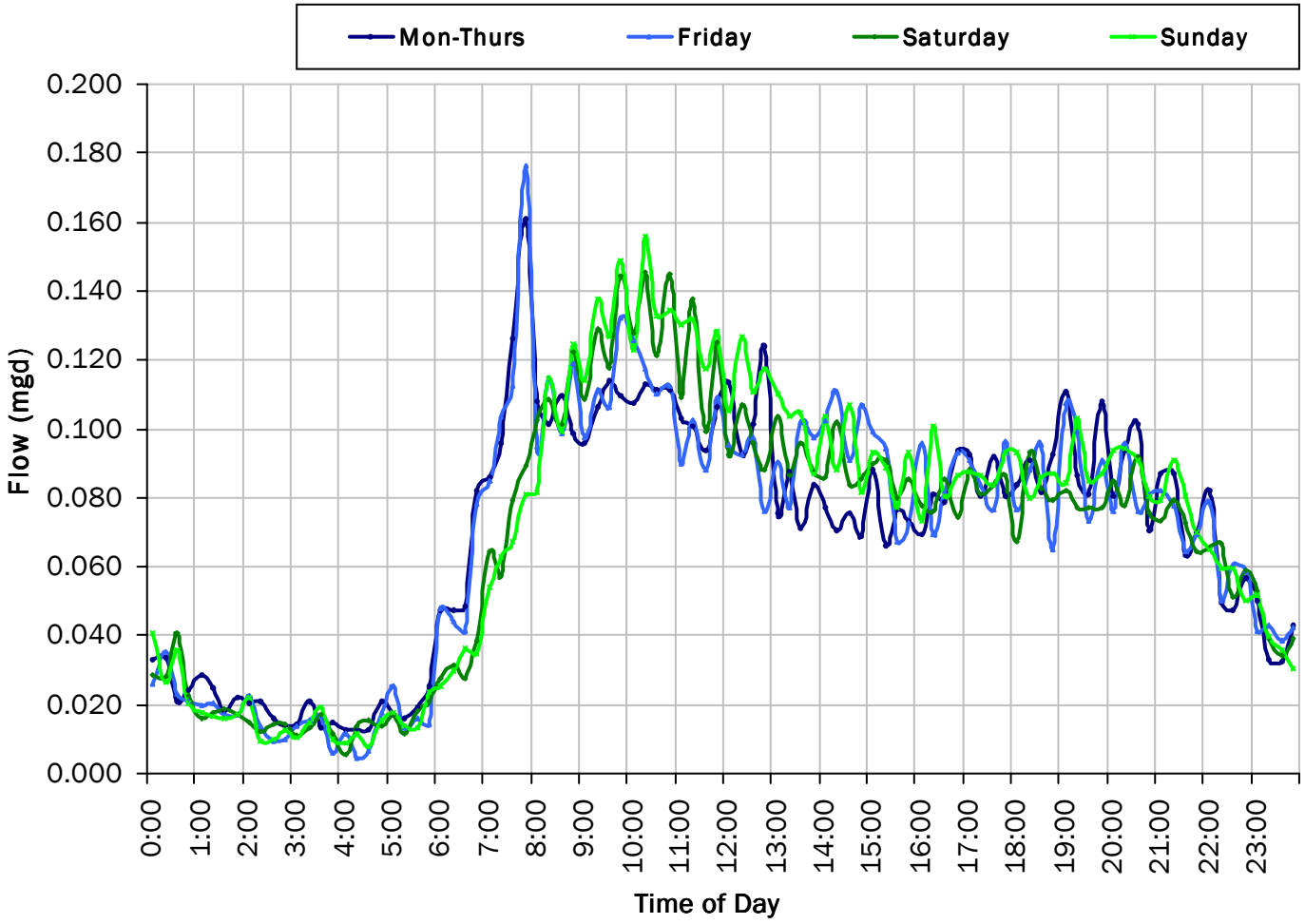
Flow Summary: 6/1/2017 to 7/16/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.077 mgd Peak Flow: 0.325 mgd Min Flow: 0.004 mgd**



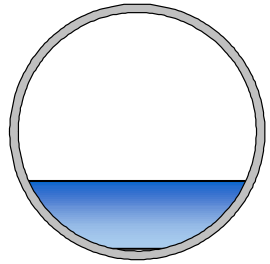
SITE 10

Average Dry Weather Flow Hydrographs



ADWF:

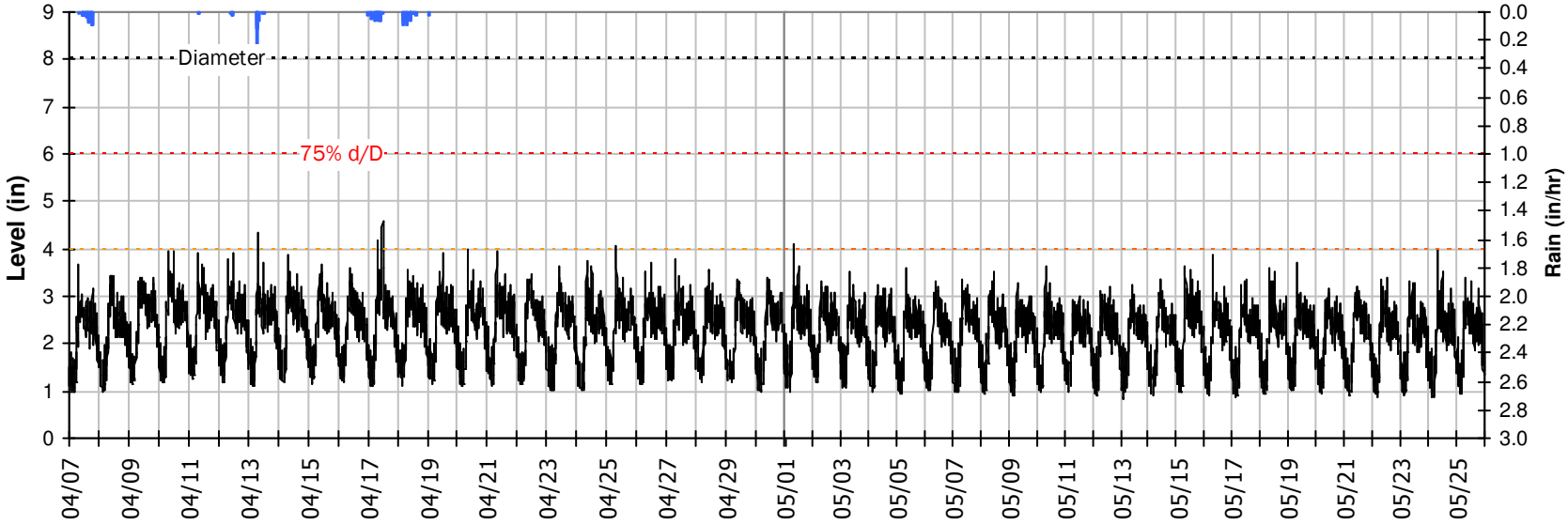
0.070 *mgd*

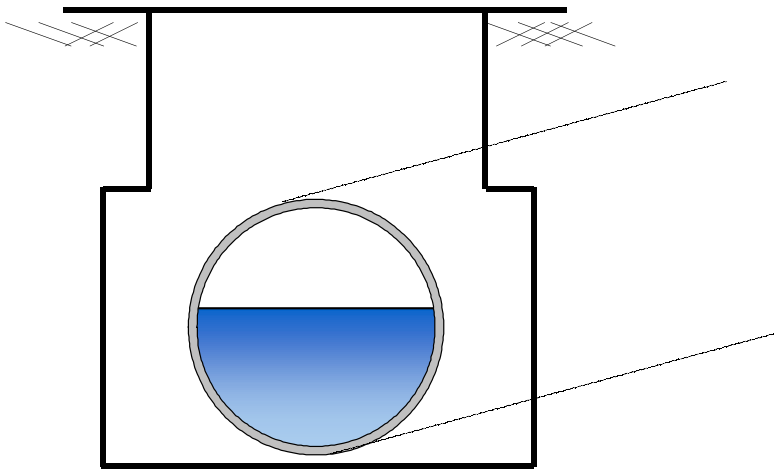


SITE 10

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



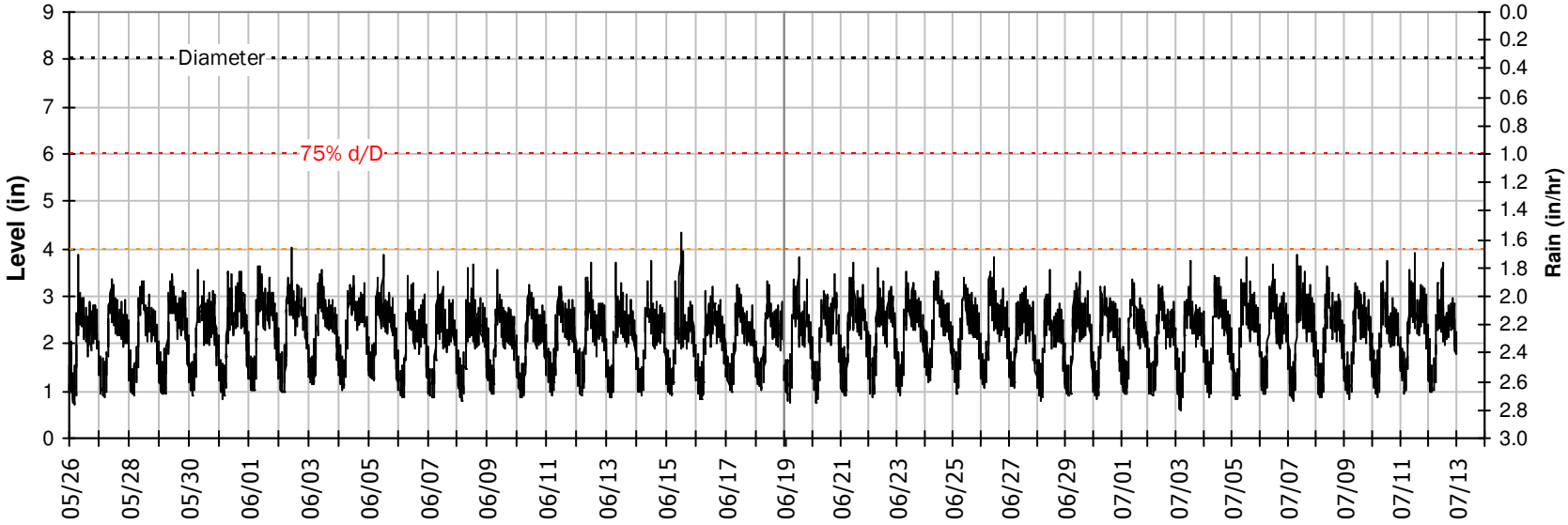


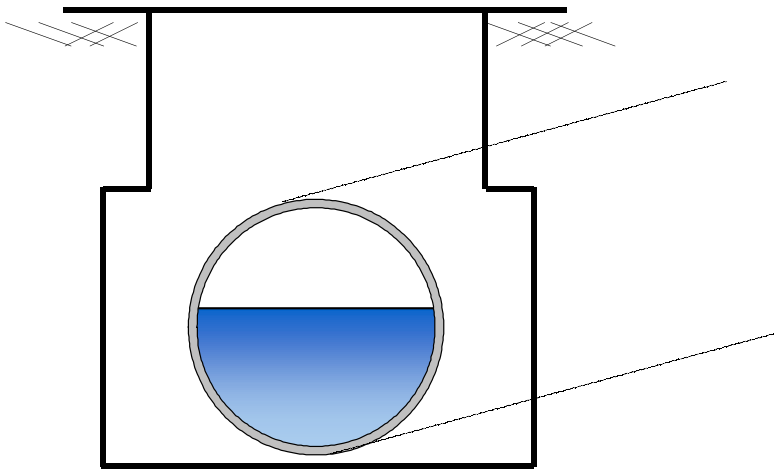
Pipe Diameter:	8	*inches*
Peak Measured Level:	4.57	*inches*
Peak d/D Ratio:	0.57	

SITE 10

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



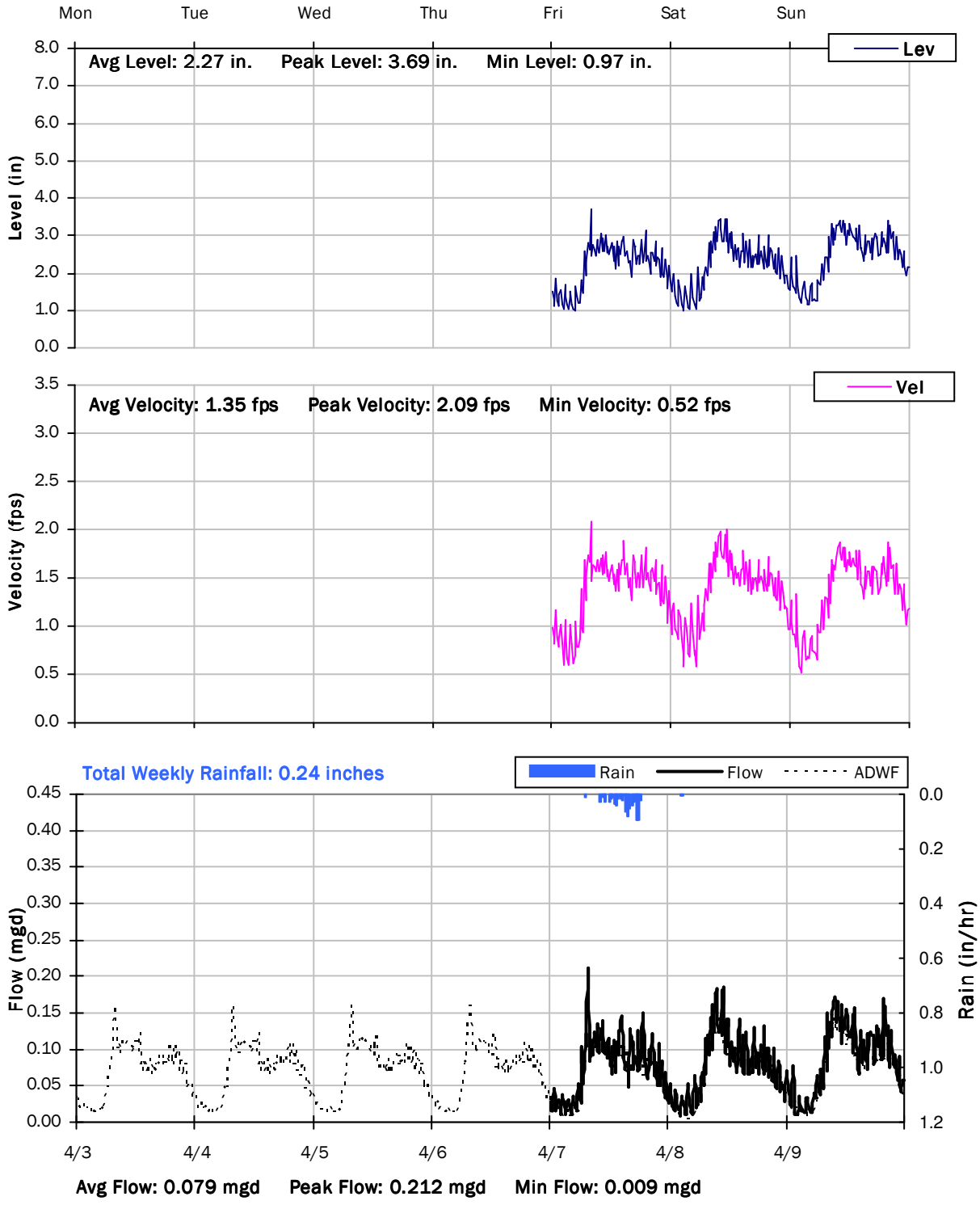


Pipe Diameter:	8	*inches*
Peak Measured Level:	4.57	*inches*
Peak d/D Ratio:	0.57	

SITE 10

Weekly Level, Velocity and Flow Hydrographs

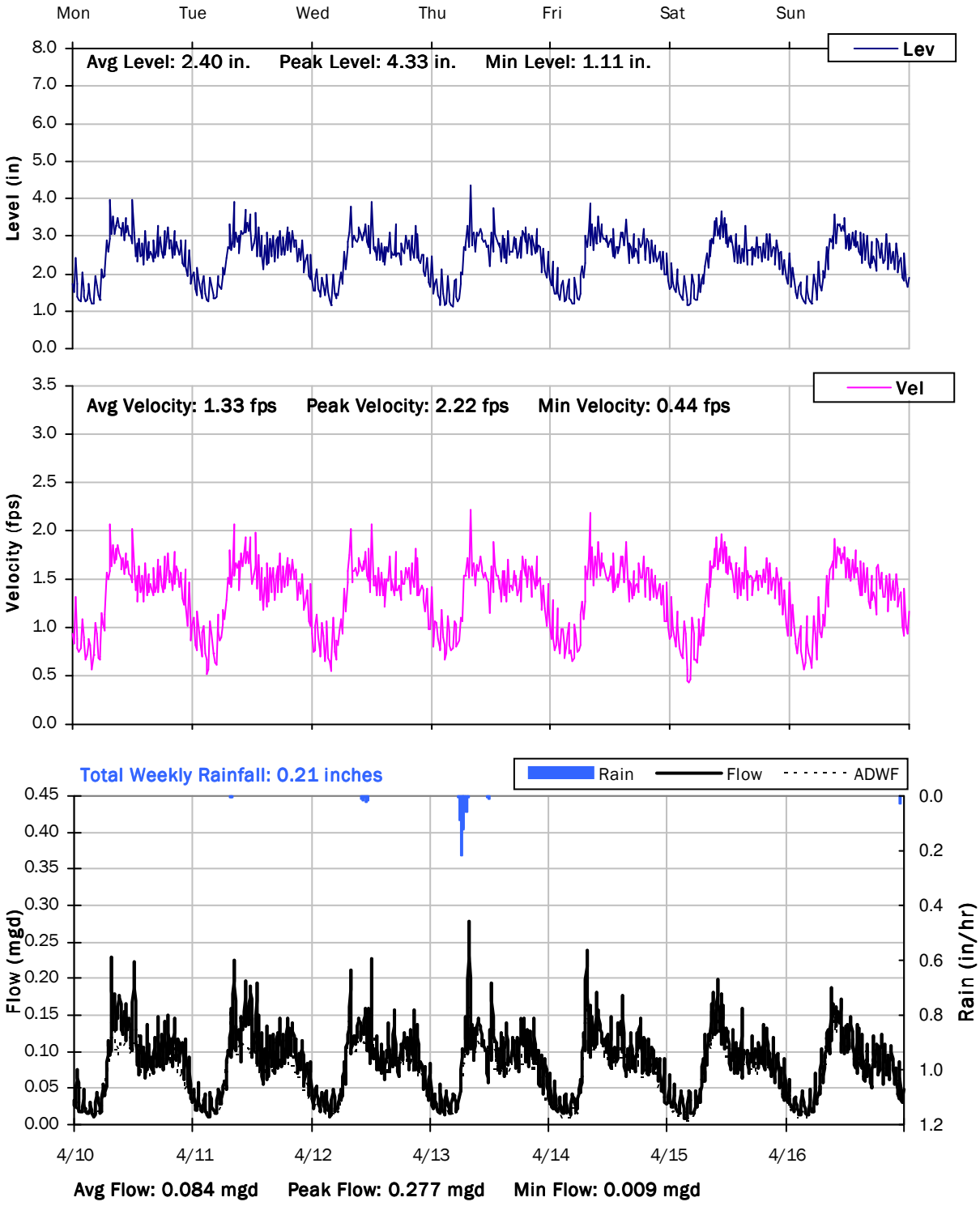
4/3/2017 to 4/10/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

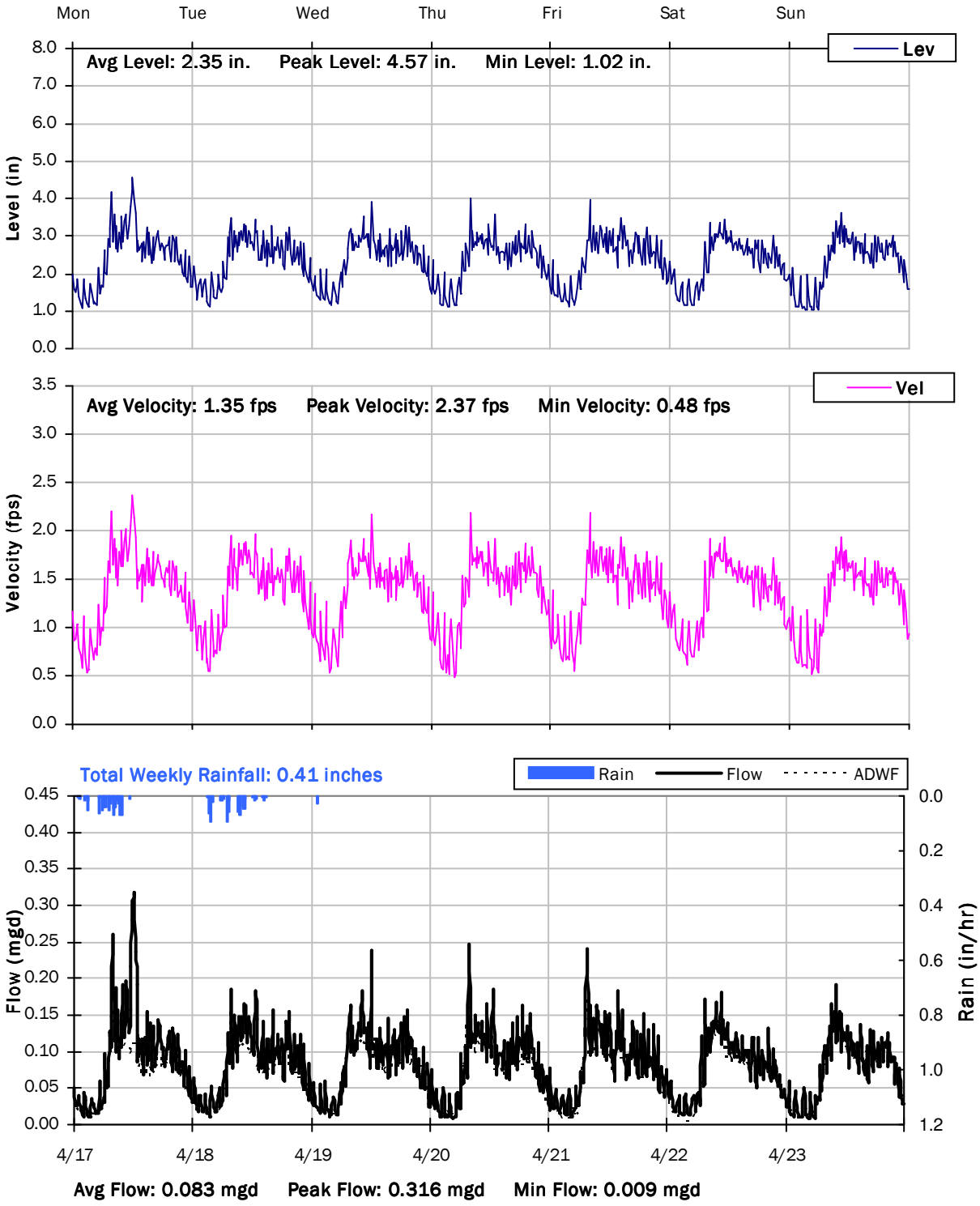
4/10/2017 to 4/17/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

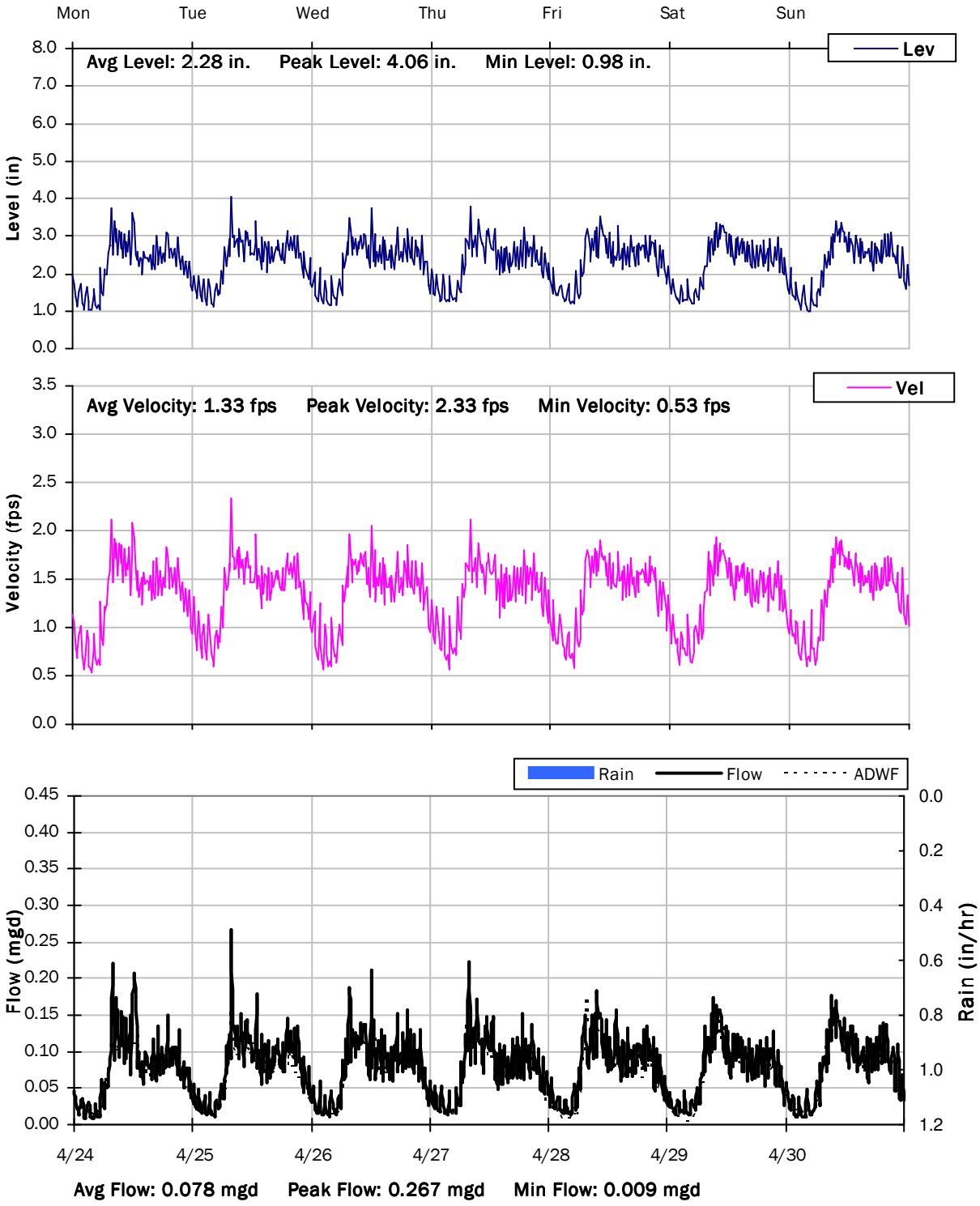
4/17/2017 to 4/24/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

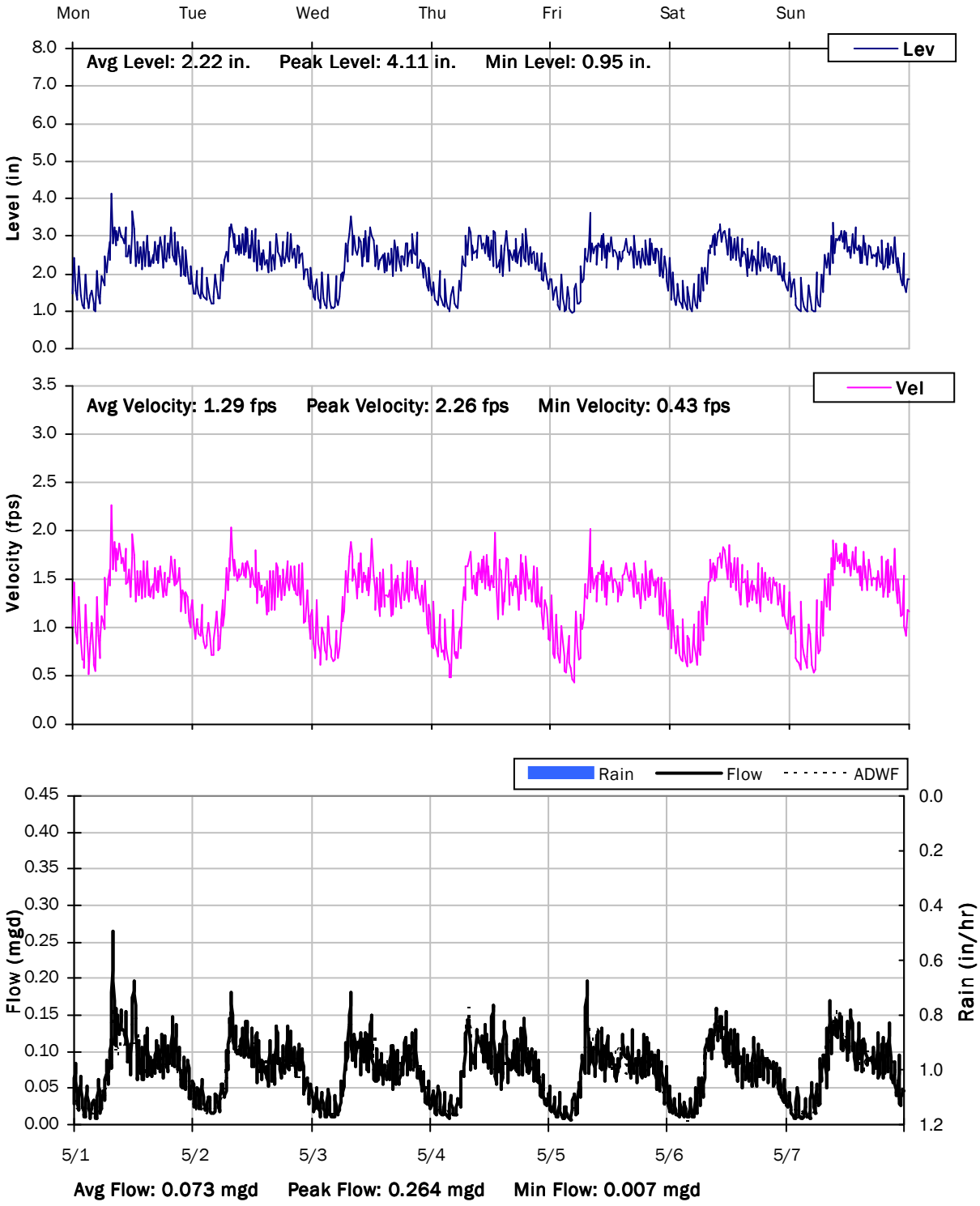
4/24/2017 to 5/1/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

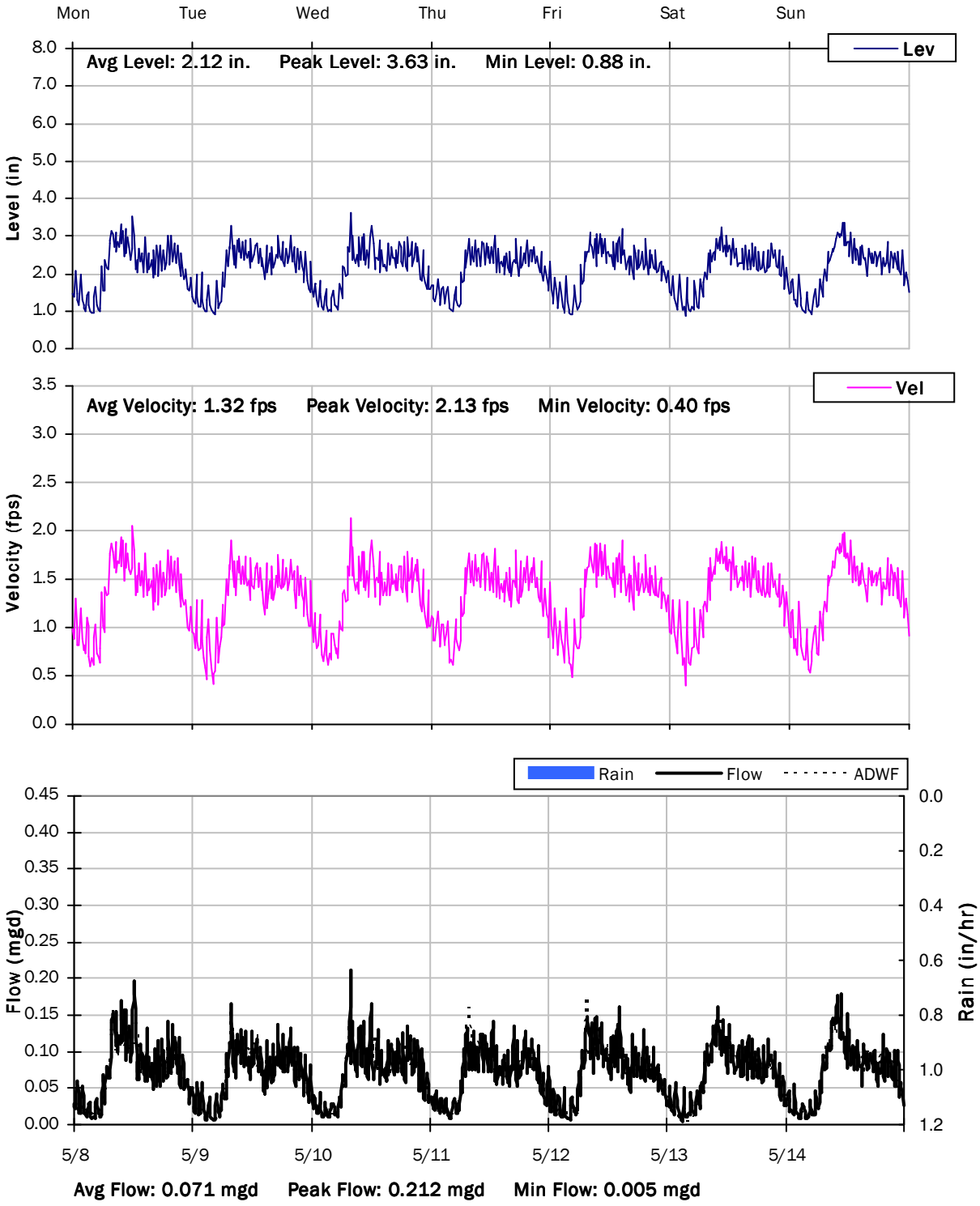
5/1/2017 to 5/8/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

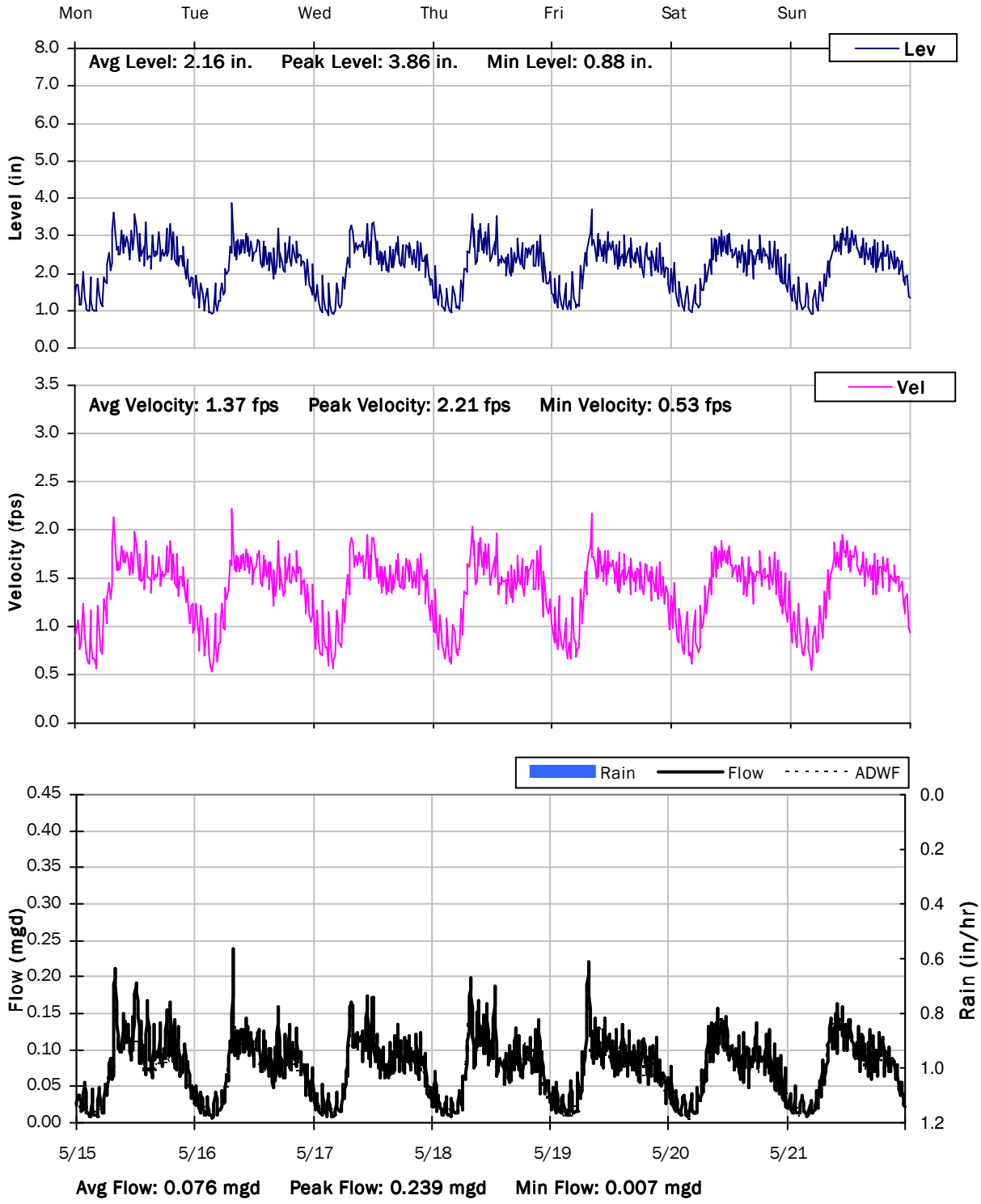
5/8/2017 to 5/15/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

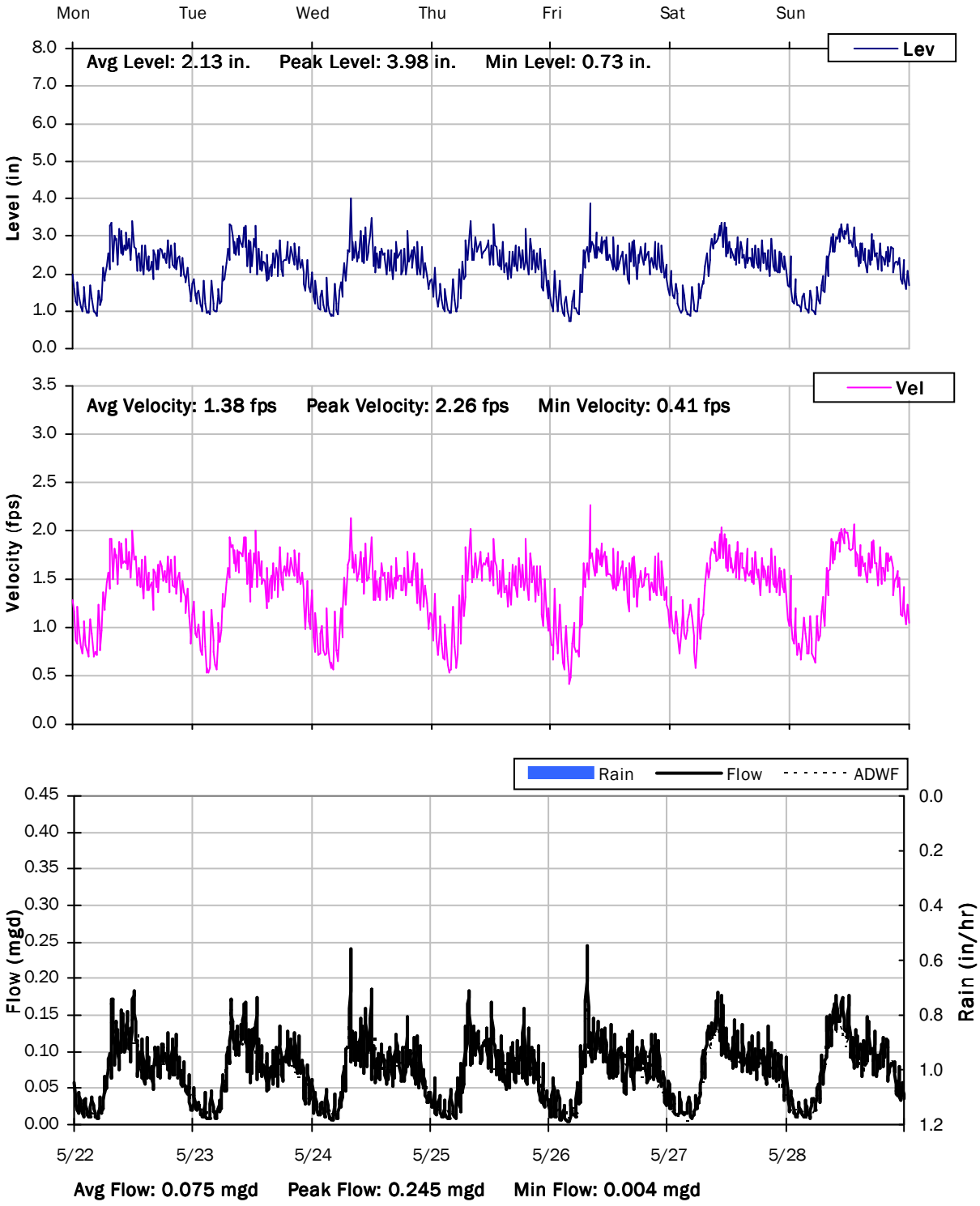
5/15/2017 to 5/22/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

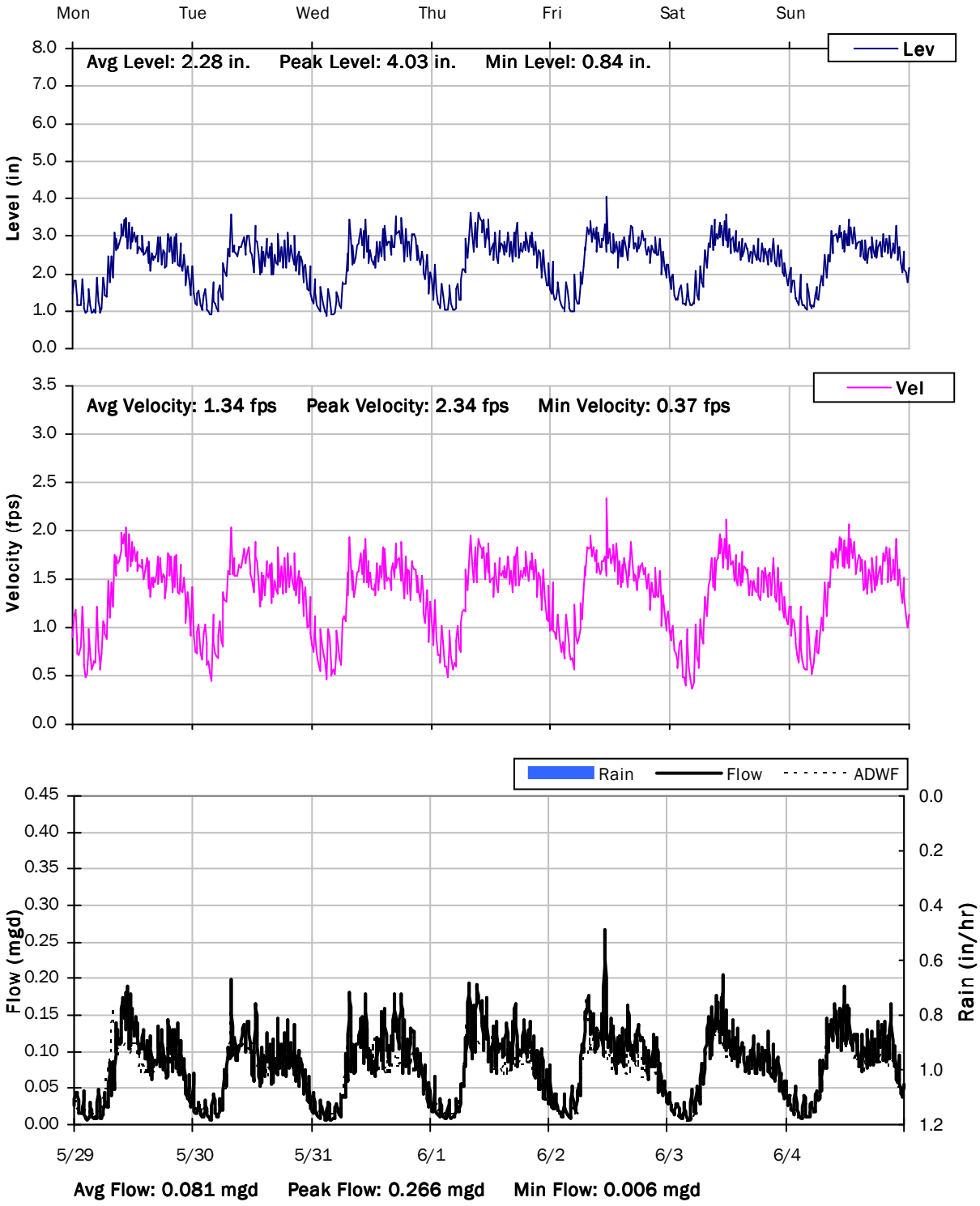
5/22/2017 to 5/29/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

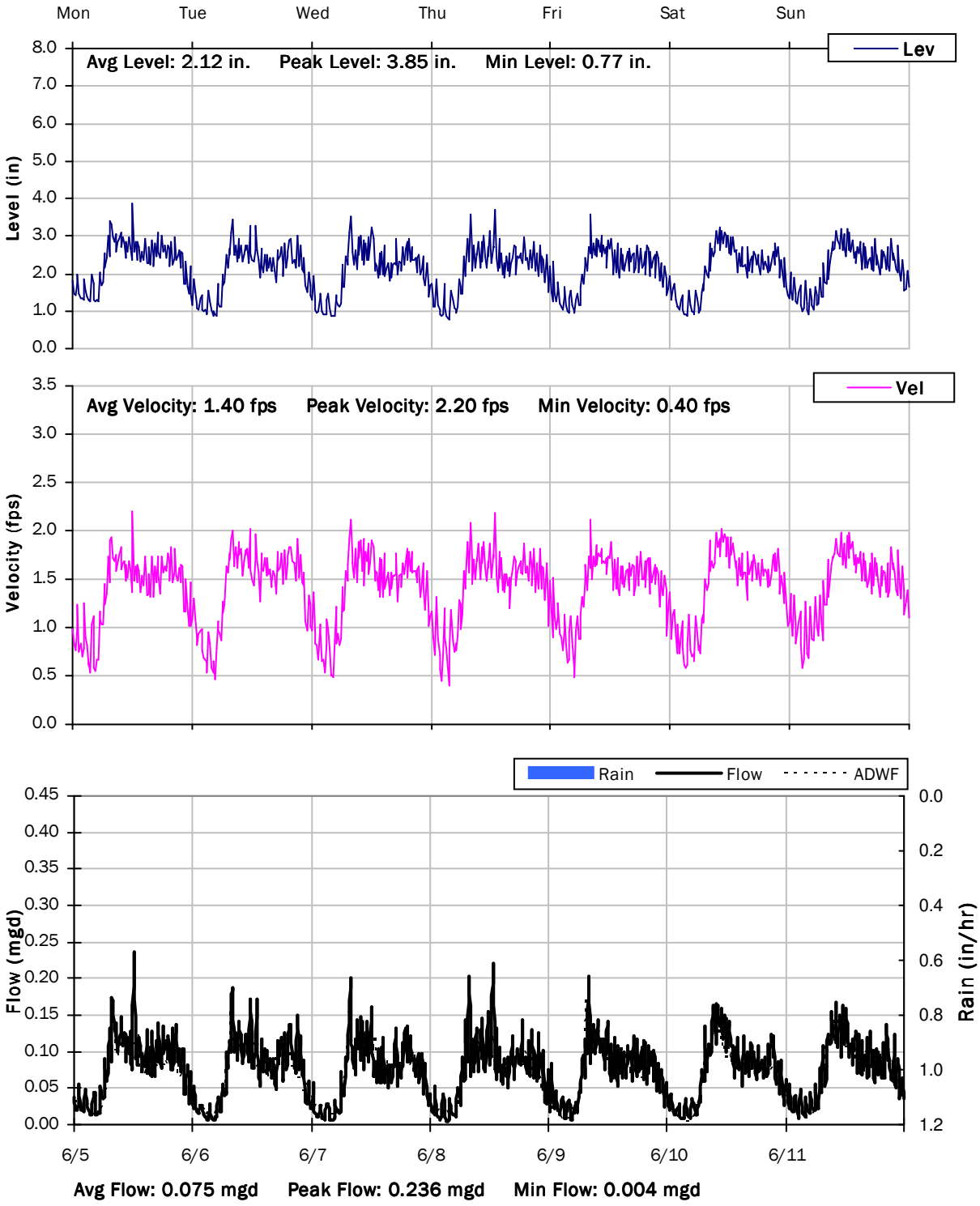
5/29/2017 to 6/5/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

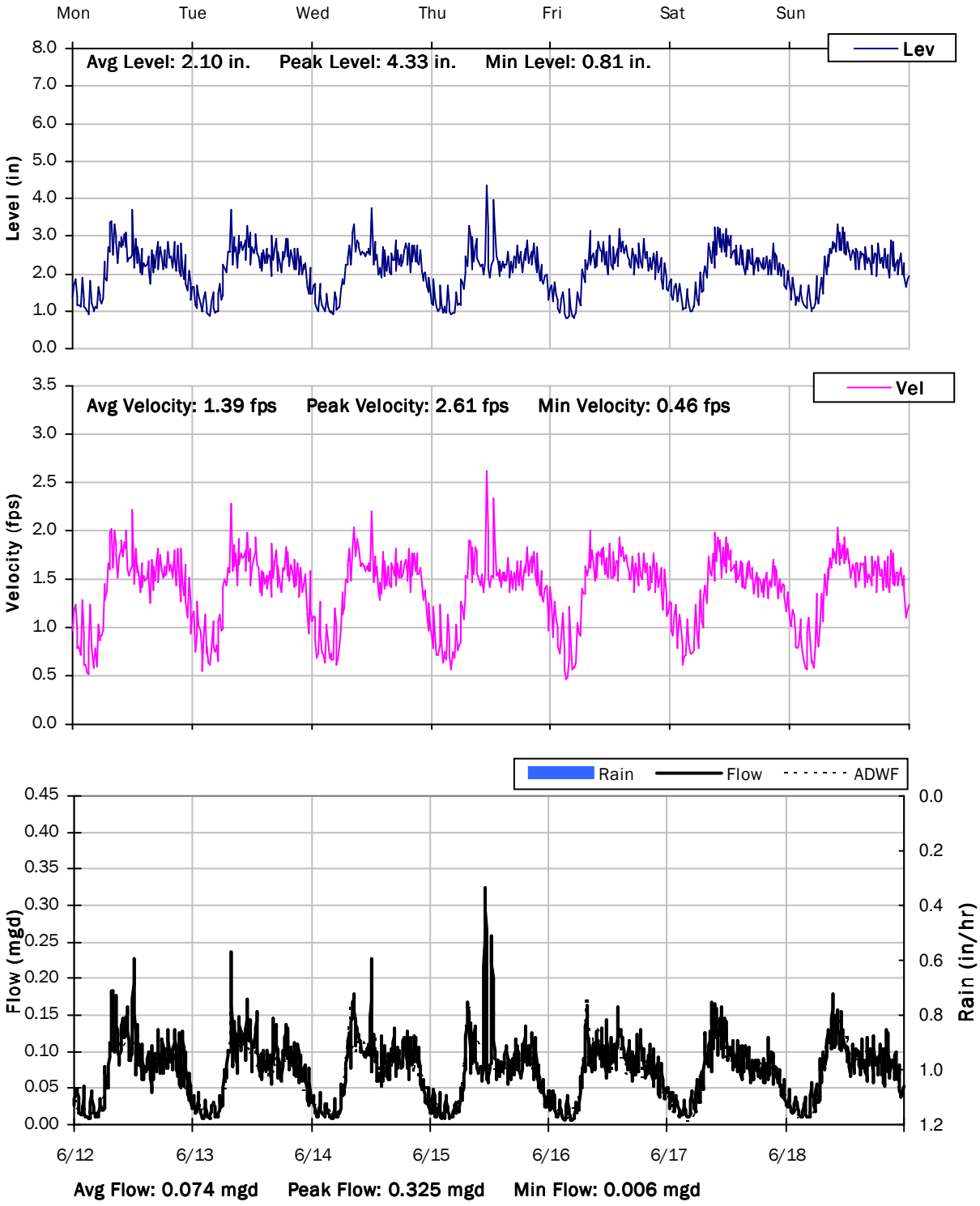
6/5/2017 to 6/12/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

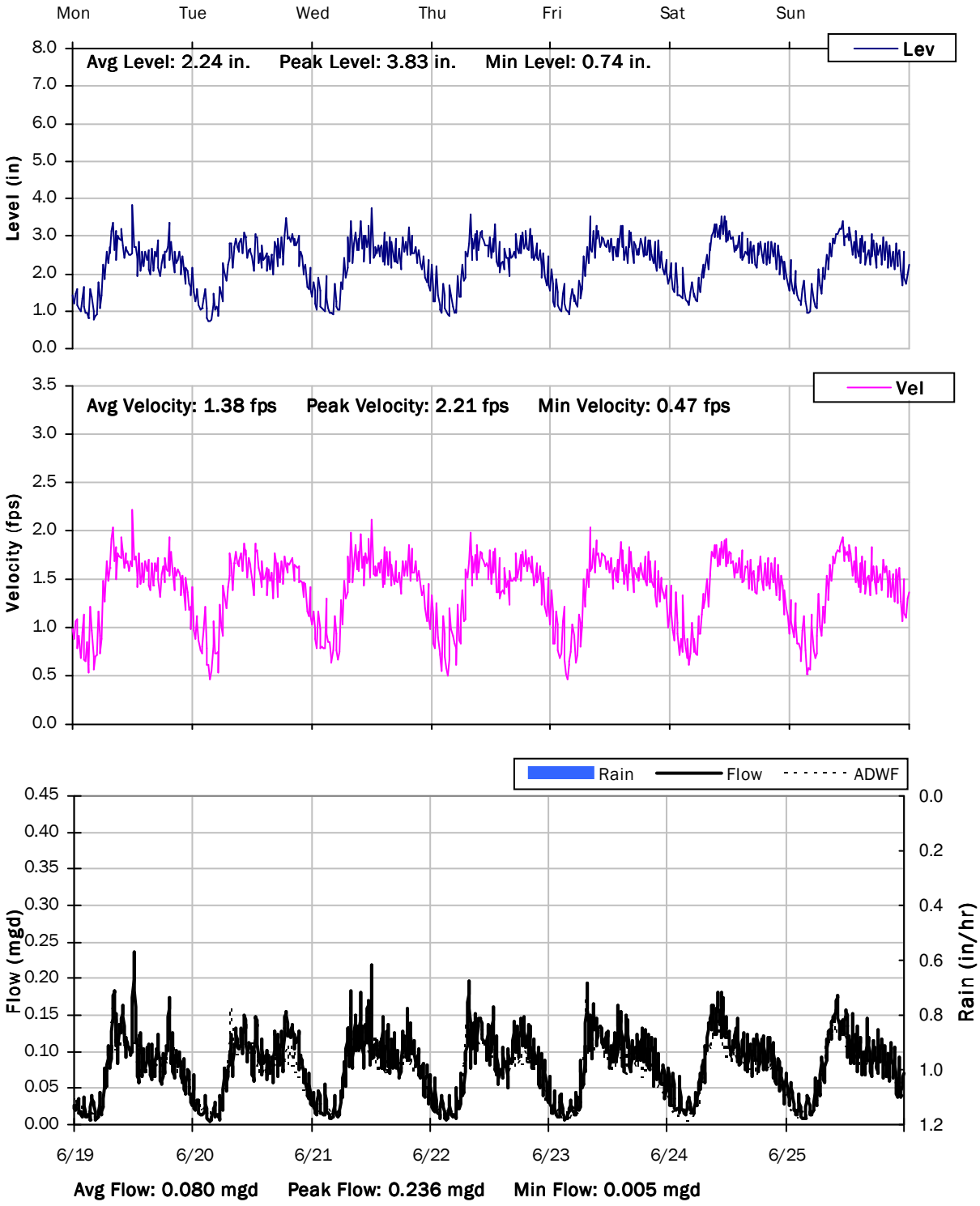
6/12/2017 to 6/19/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

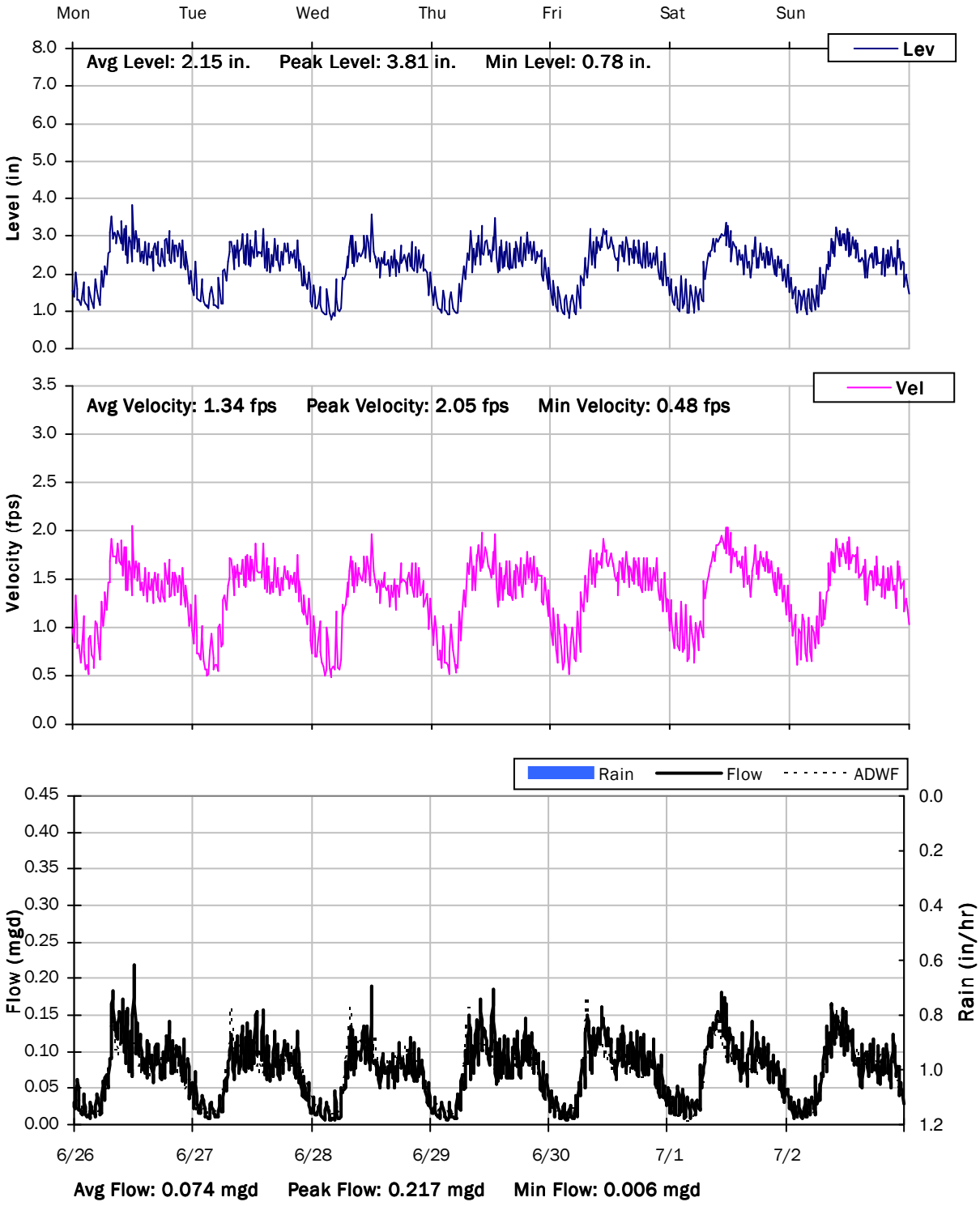
6/19/2017 to 6/26/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

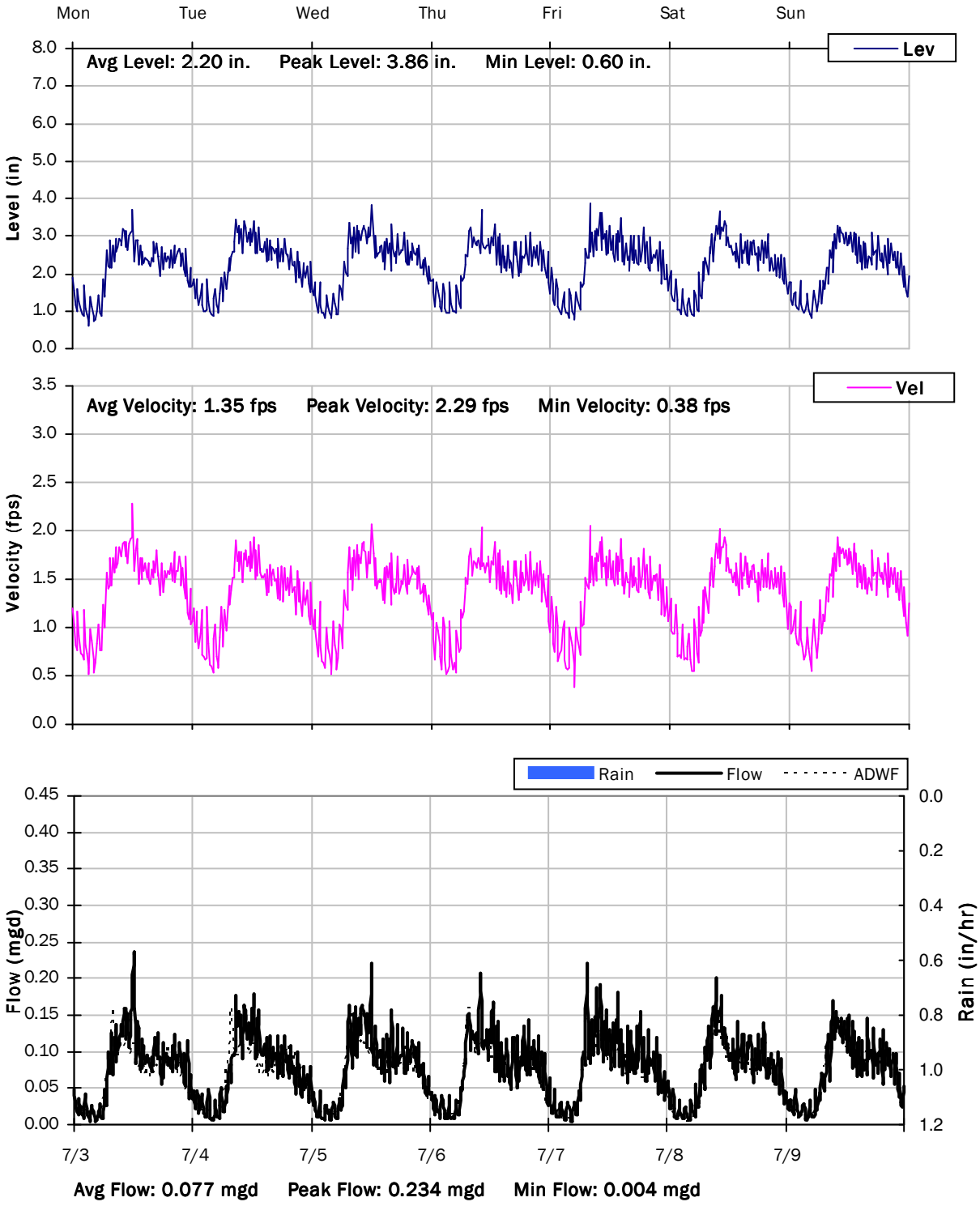
6/26/2017 to 7/3/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

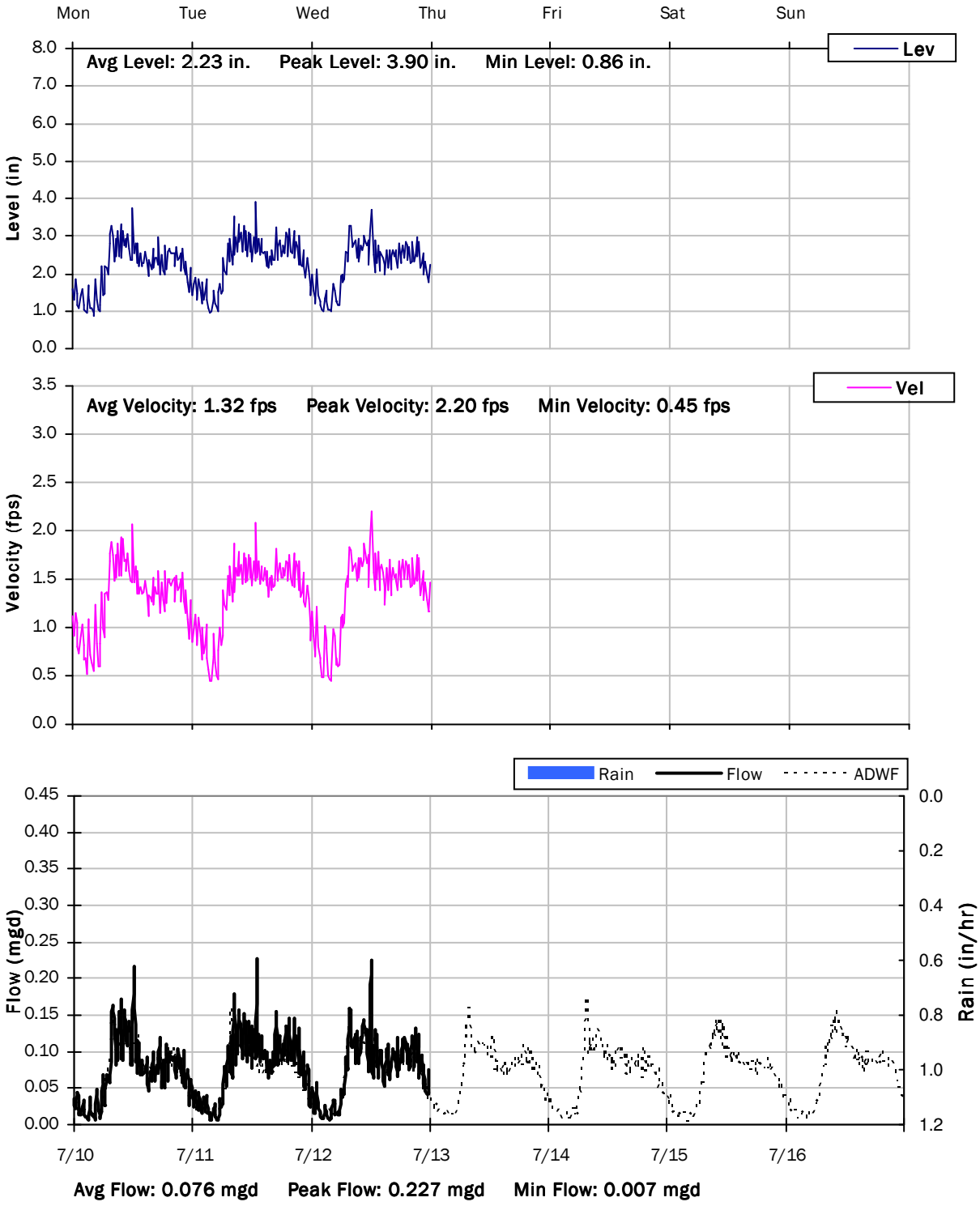
7/3/2017 to 7/10/2017



SITE 10

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

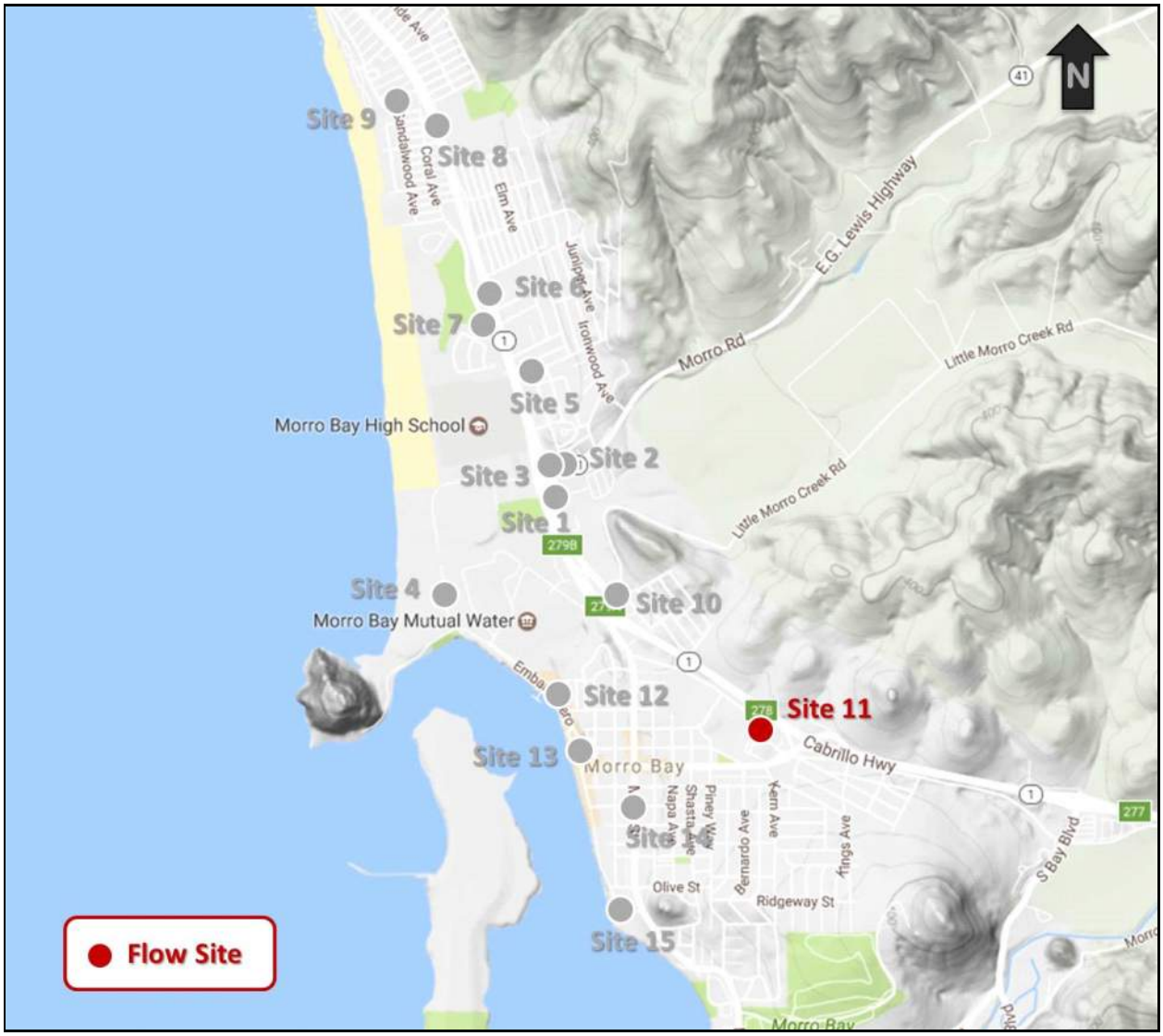
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 11

Location: 785 Quintana Road, meter on east 8" inlet

Data Summary Report



Vicinity Map: Site 11

SITE 11

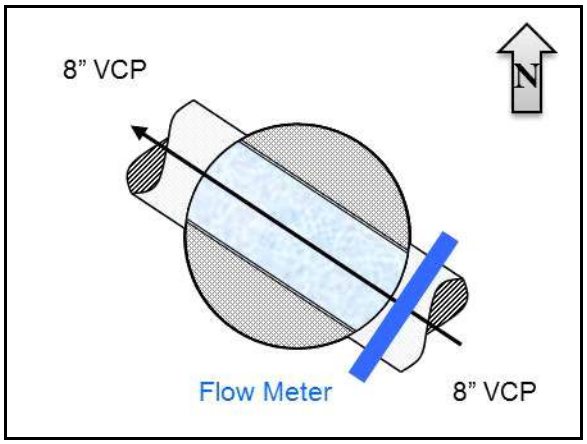
Site Information

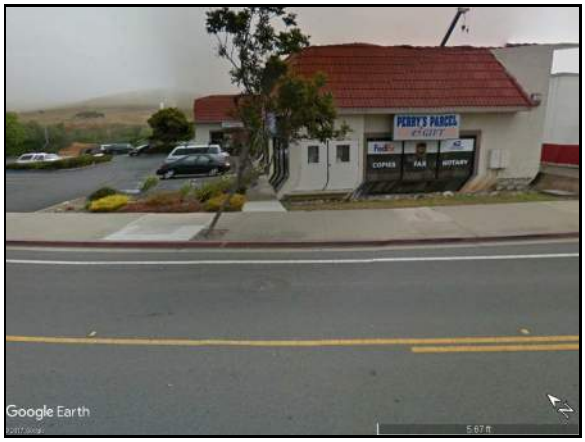
Location: 785 Quintana Road, meter on east 8" inlet

Pipe Diameter: 8 inches

ADWF: 0.064 mgd

Peak Measured Flow: 0.248 mgd





SITE 11

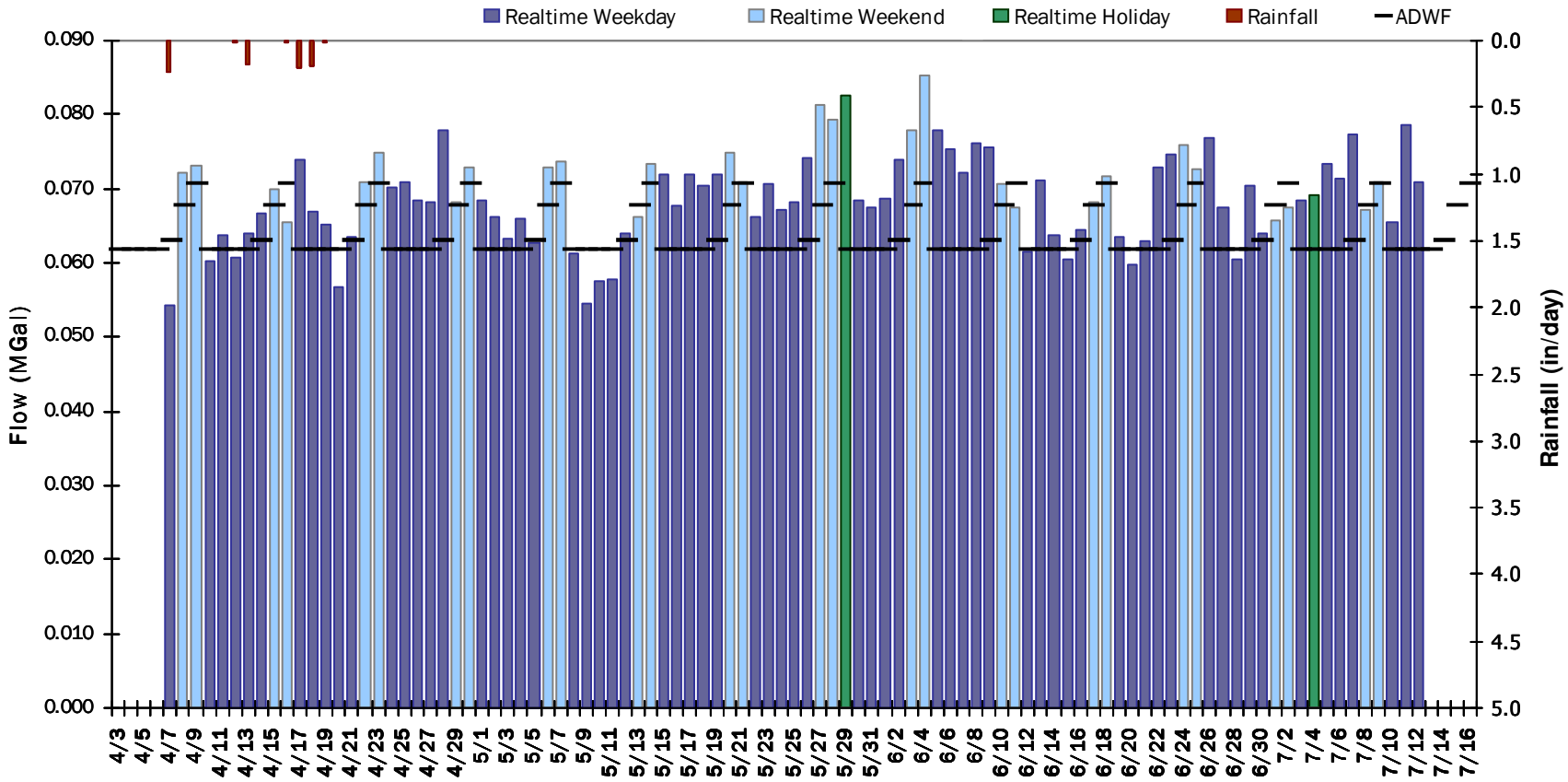
Additional Site Photos

SITE 11

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.069 MGal **Peak Daily Flow: 0.085 MGal** **Min Daily Flow: 0.054 MGal**

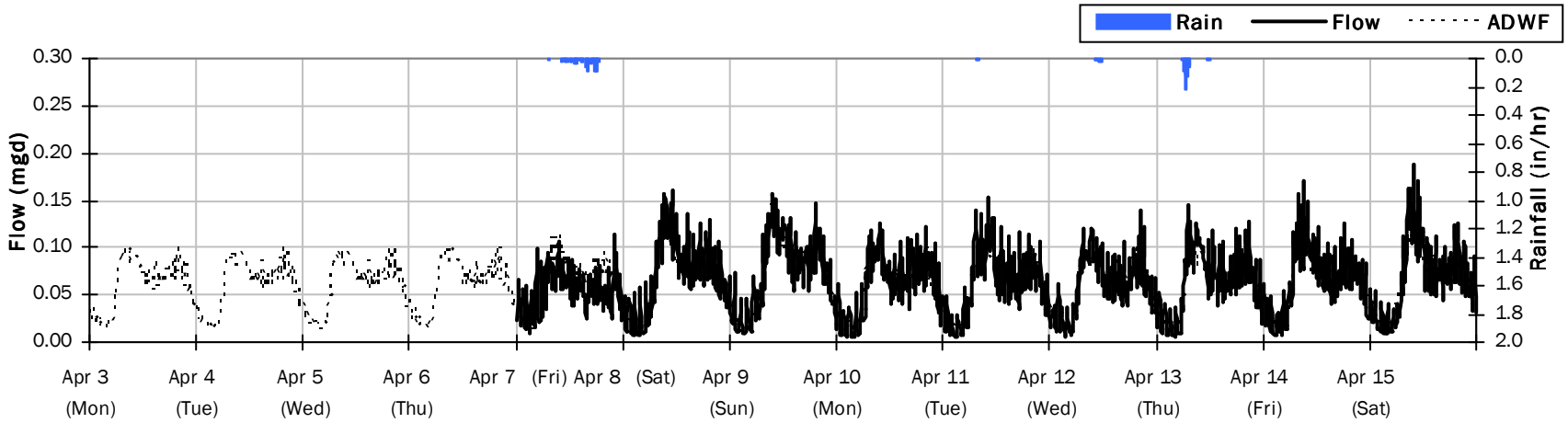
Total Period Rainfall: 0.85 inches

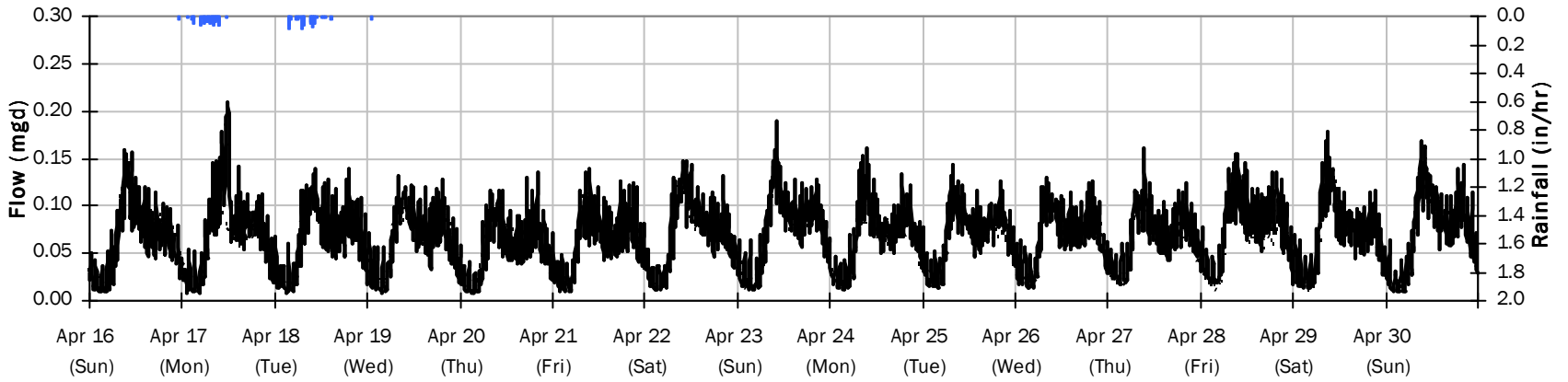


SITE 11

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.067 mgd** **Peak Flow: 0.208 mgd** **Min Flow: 0.006 mgd**

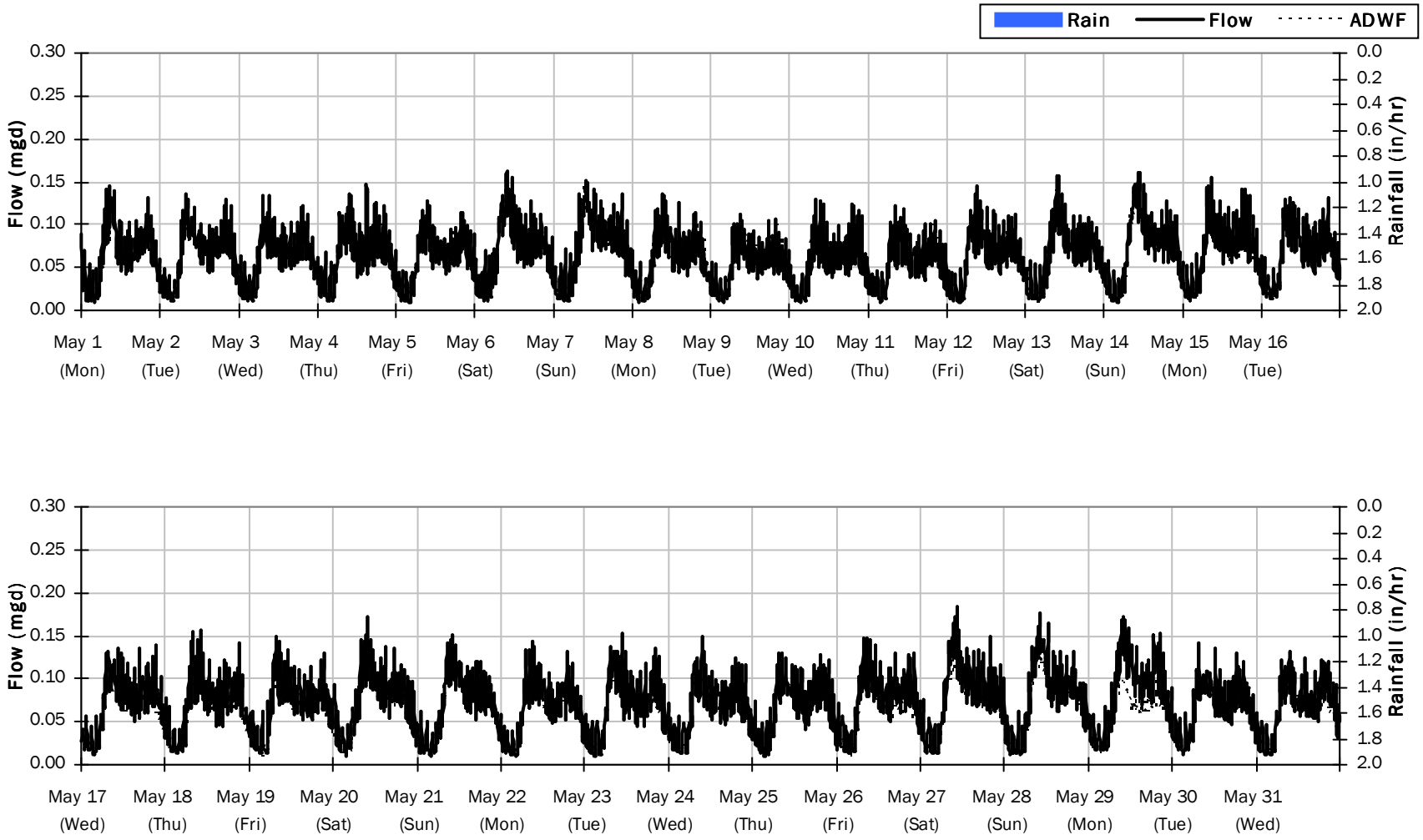




SITE 11

Flow Summary: 5/1/2017 to 5/31/2017

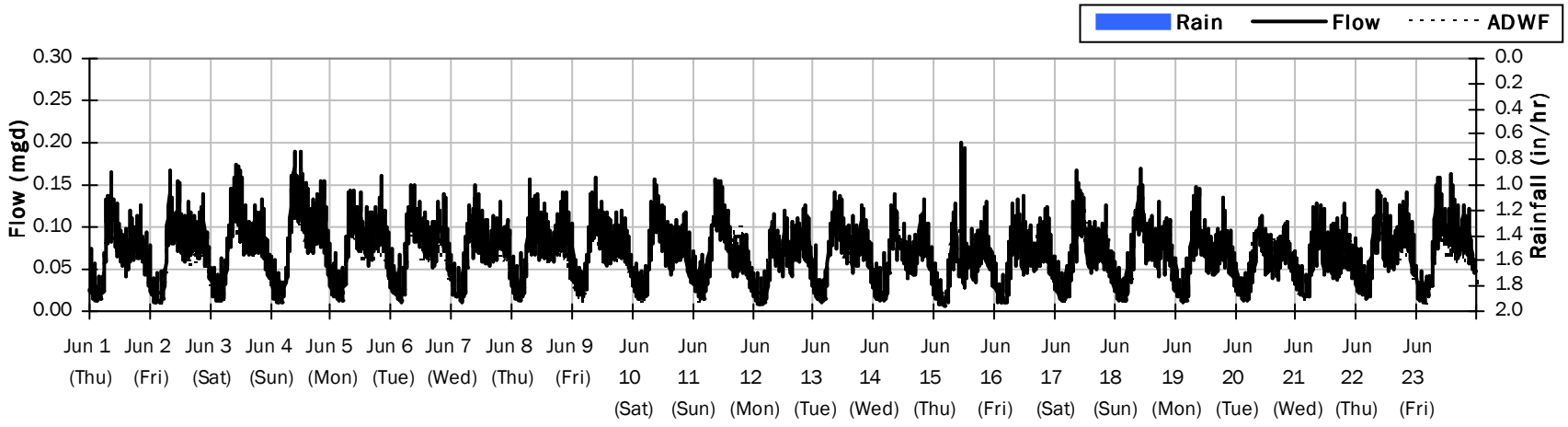
Total Period Rainfall: 0.00 inches **Avg Flow: 0.069 mgd** **Peak Flow: 0.184 mgd** **Min Flow: 0.009 mgd**

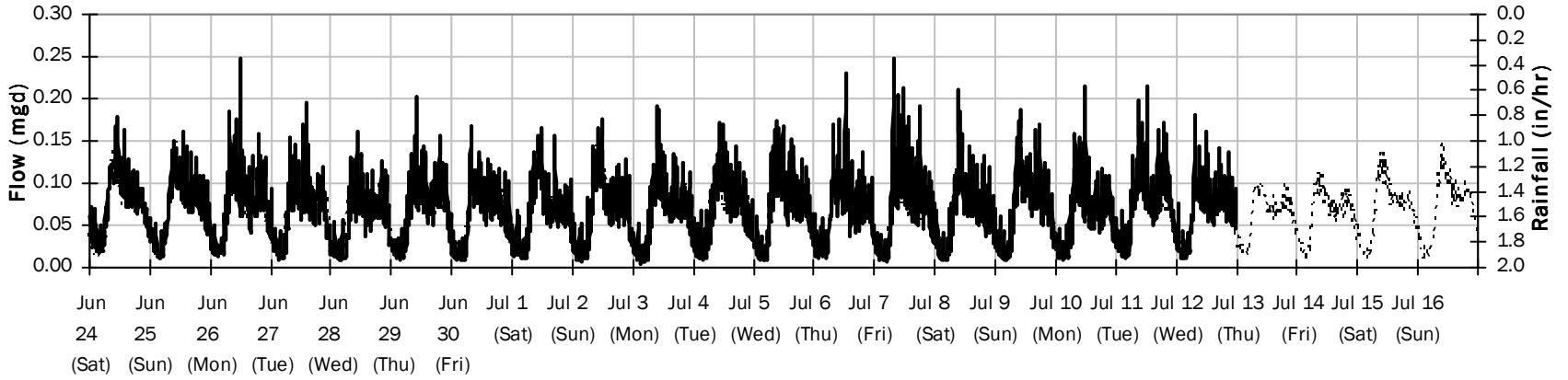


SITE 11

Flow Summary: 6/1/2017 to 7/16/2017

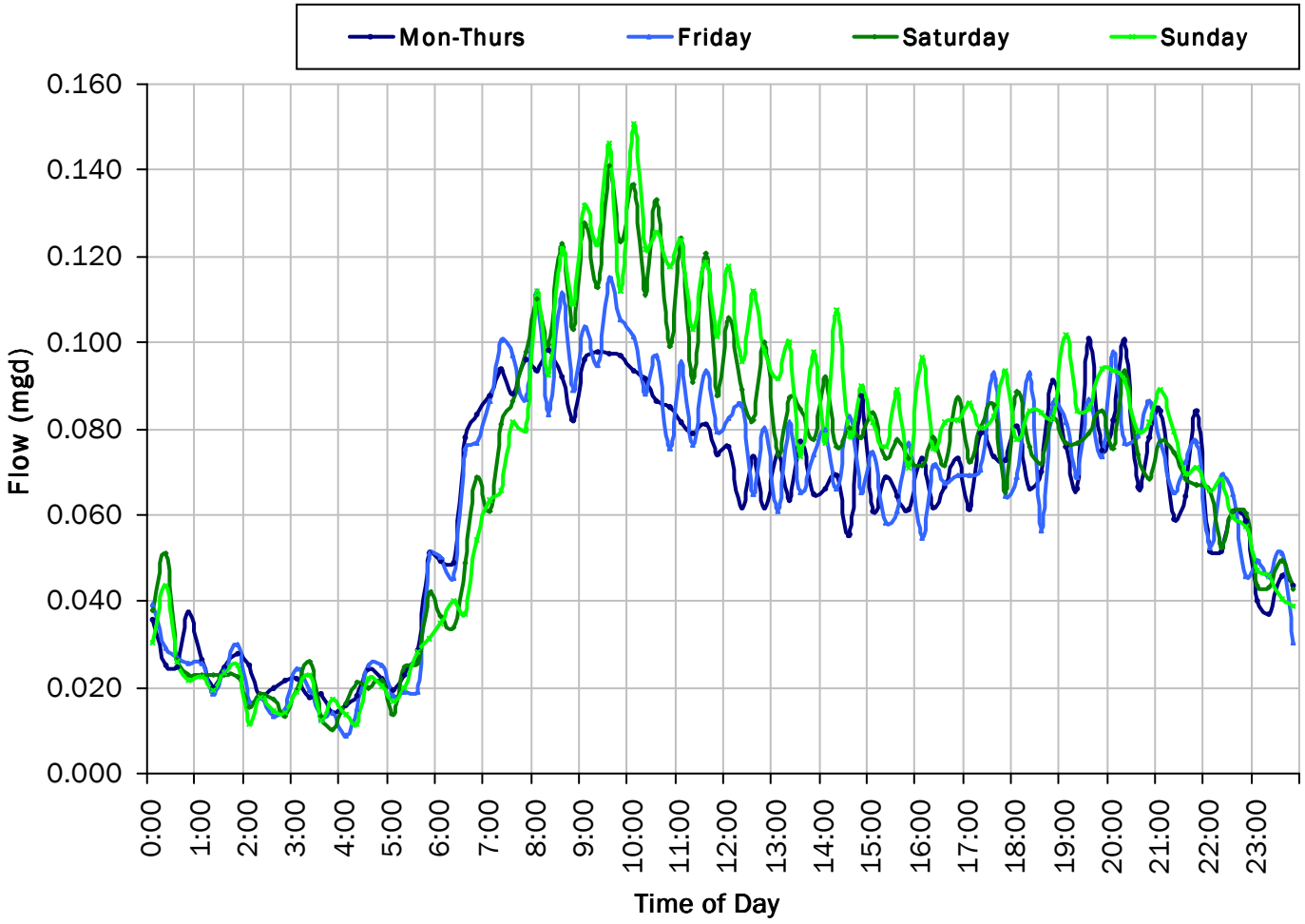
Total Period Rainfall: 0.00 inches **Avg Flow: 0.070 mgd** **Peak Flow: 0.248 mgd** **Min Flow: 0.005 mgd**





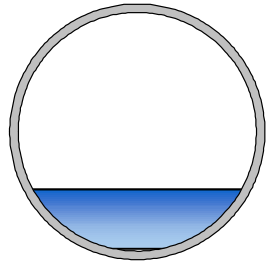
SITE 11

Average Dry Weather Flow Hydrographs



ADWF:

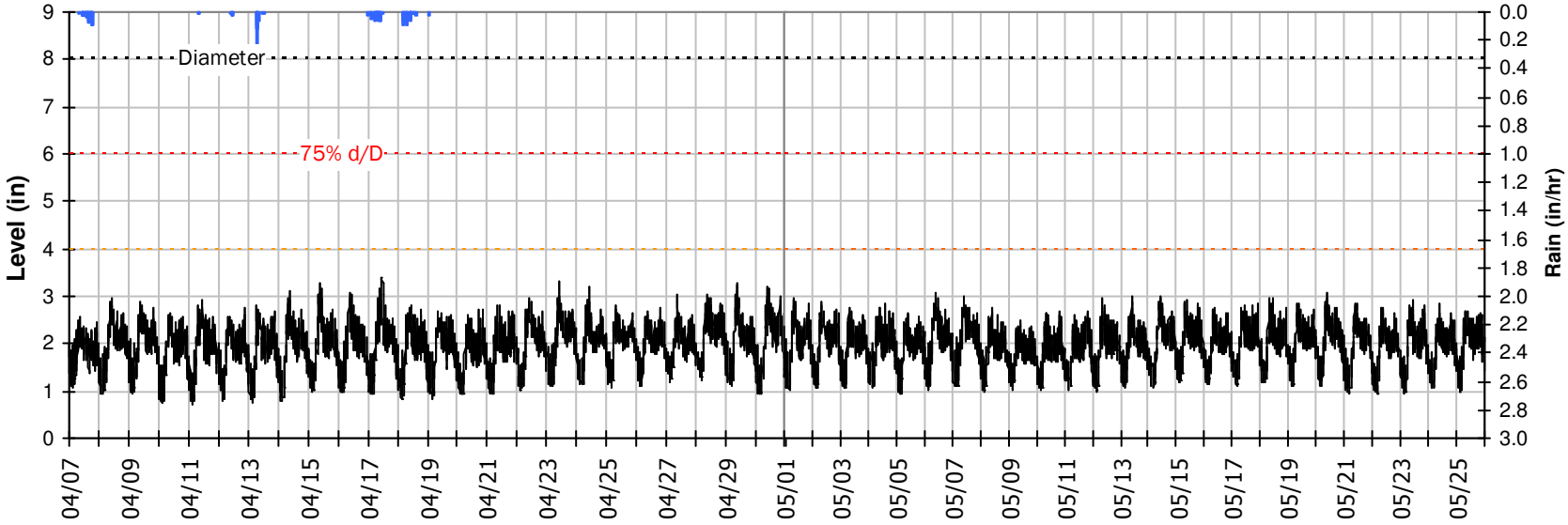
0.064 *mgd*

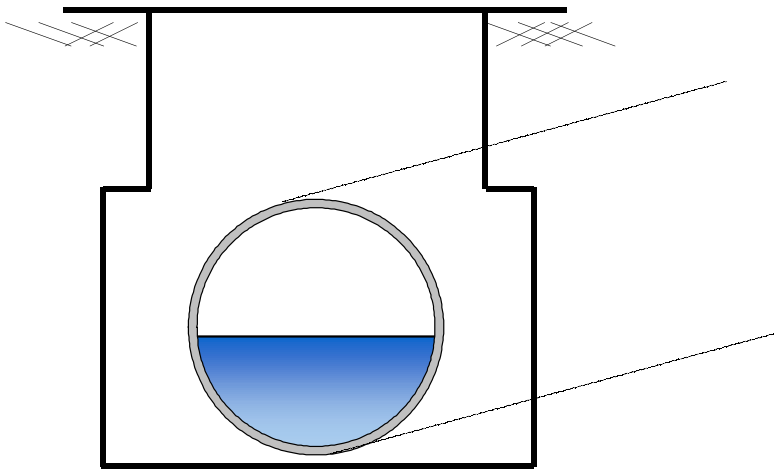


SITE 11

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



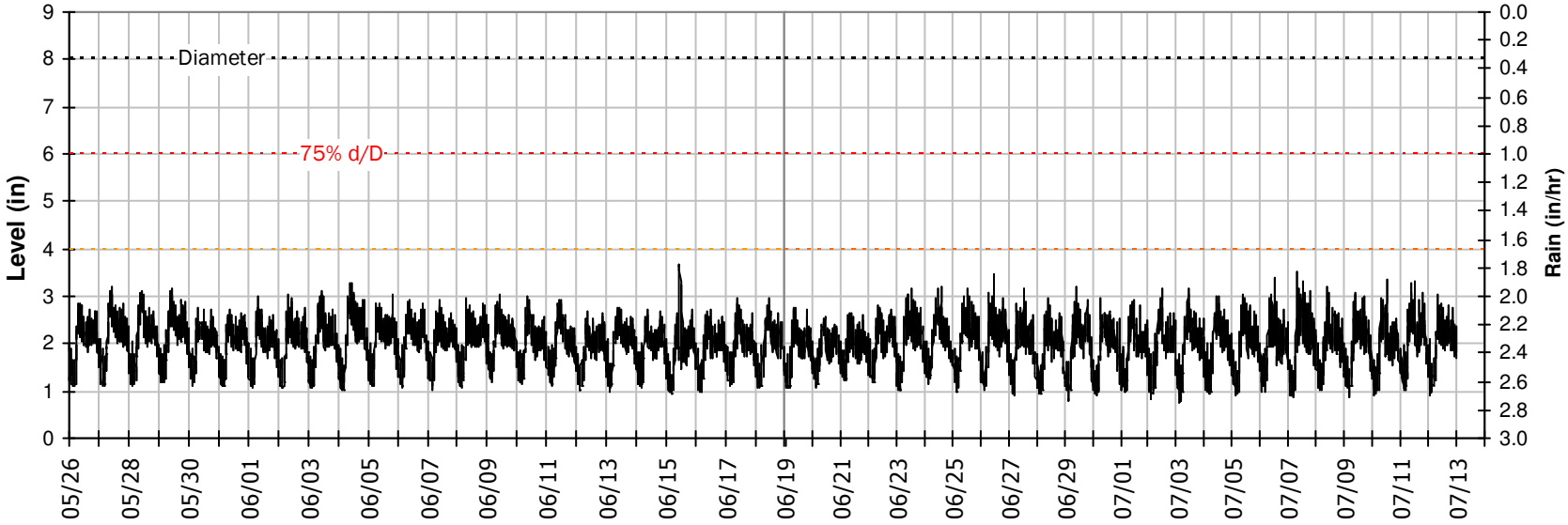


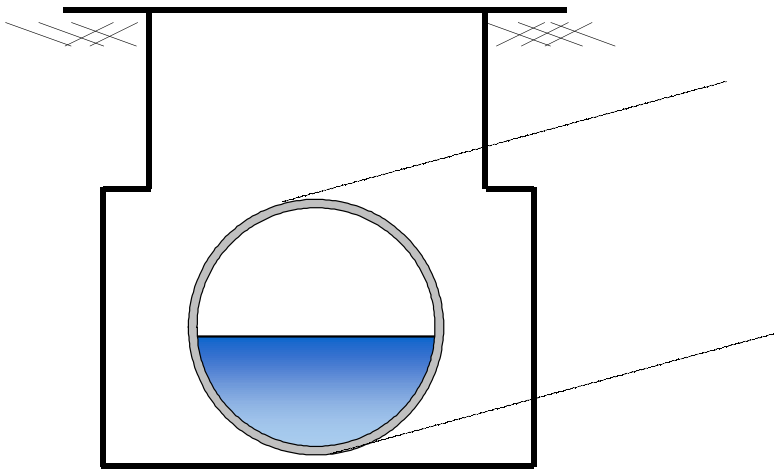
Pipe Diameter:	8	*inches*
Peak Measured Level:	3.64	*inches*
Peak d/D Ratio:	0.46	

SITE 11

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



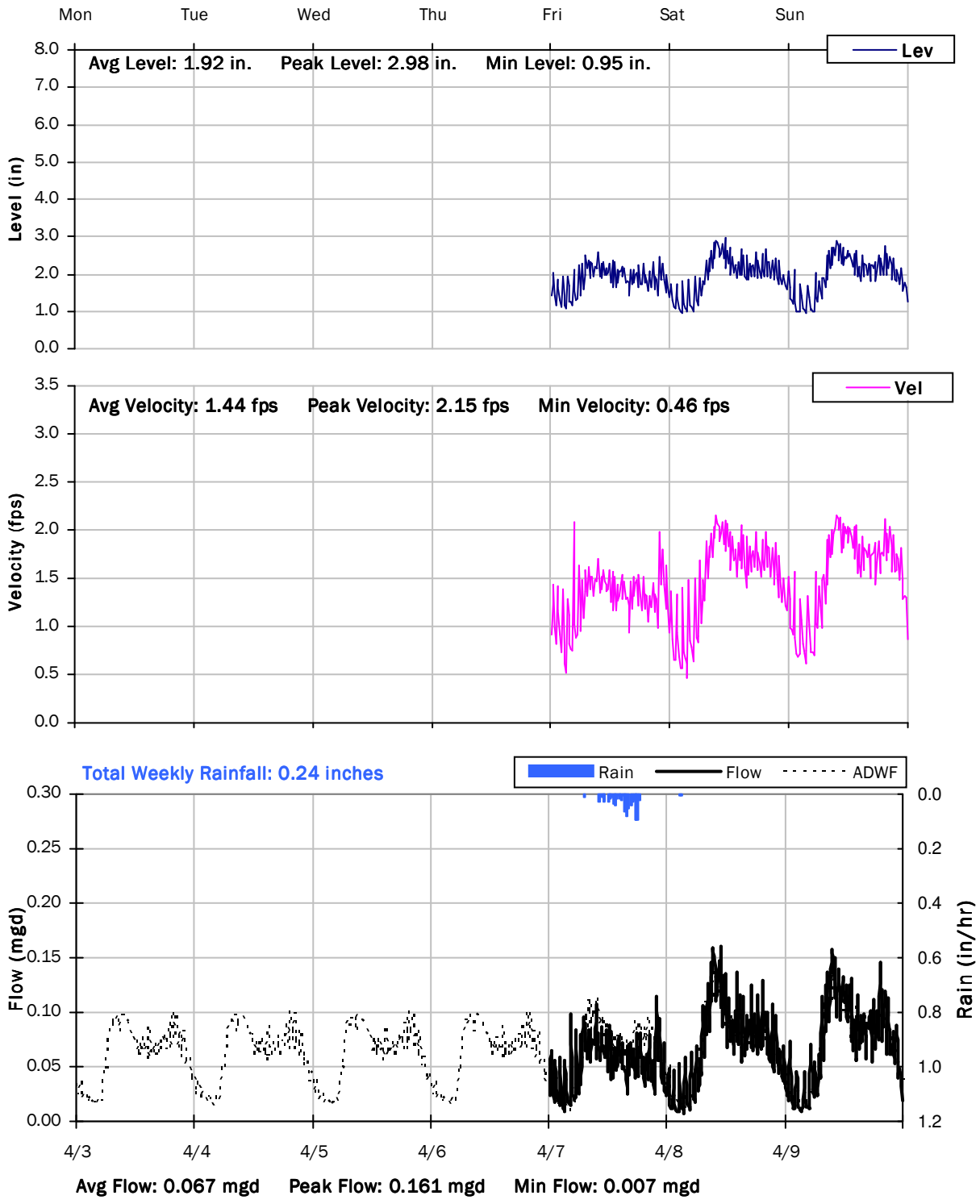


Pipe Diameter:	8	*inches*
Peak Measured Level:	3.64	*inches*
Peak d/D Ratio:	0.46	

SITE 11

Weekly Level, Velocity and Flow Hydrographs

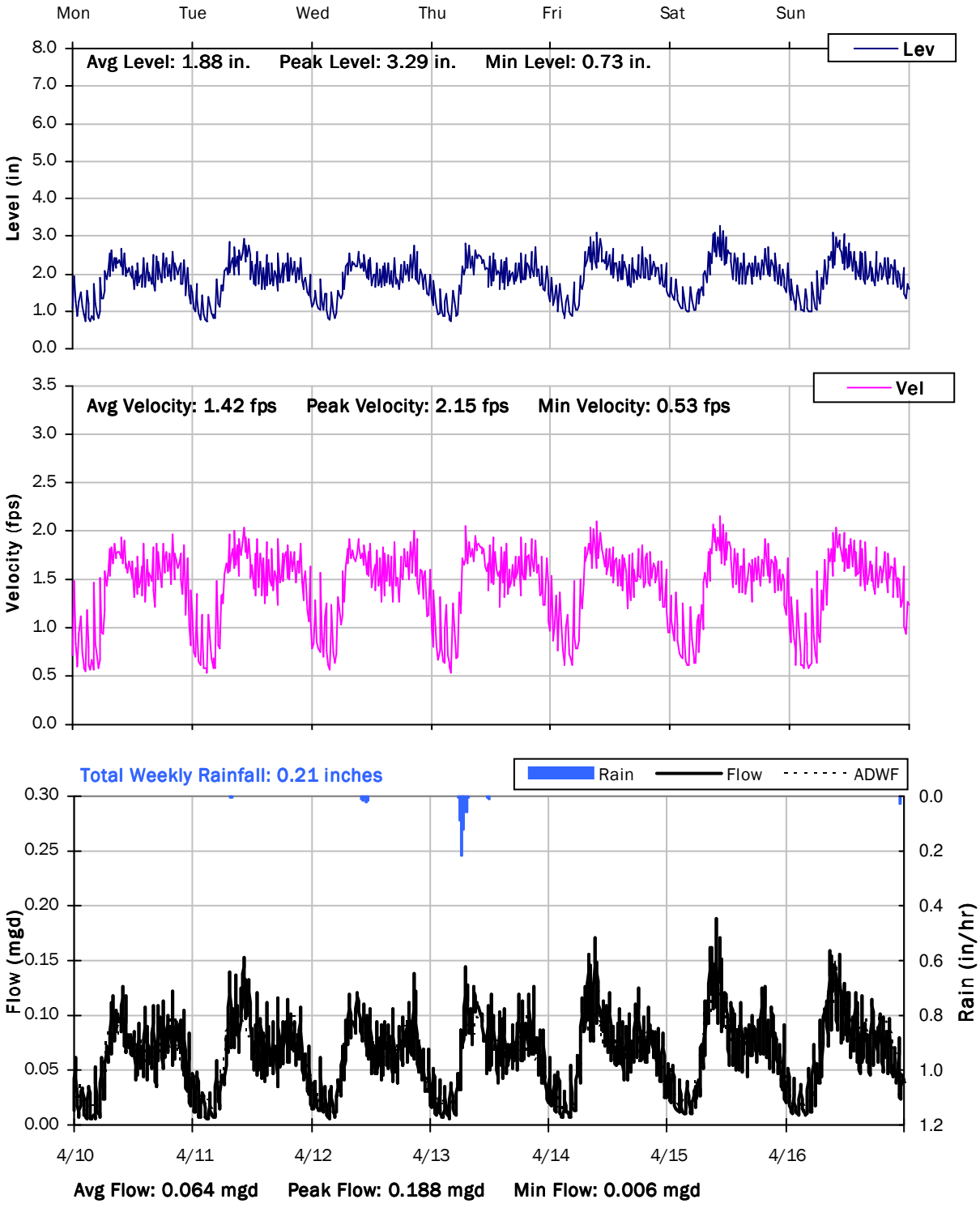
4/3/2017 to 4/10/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

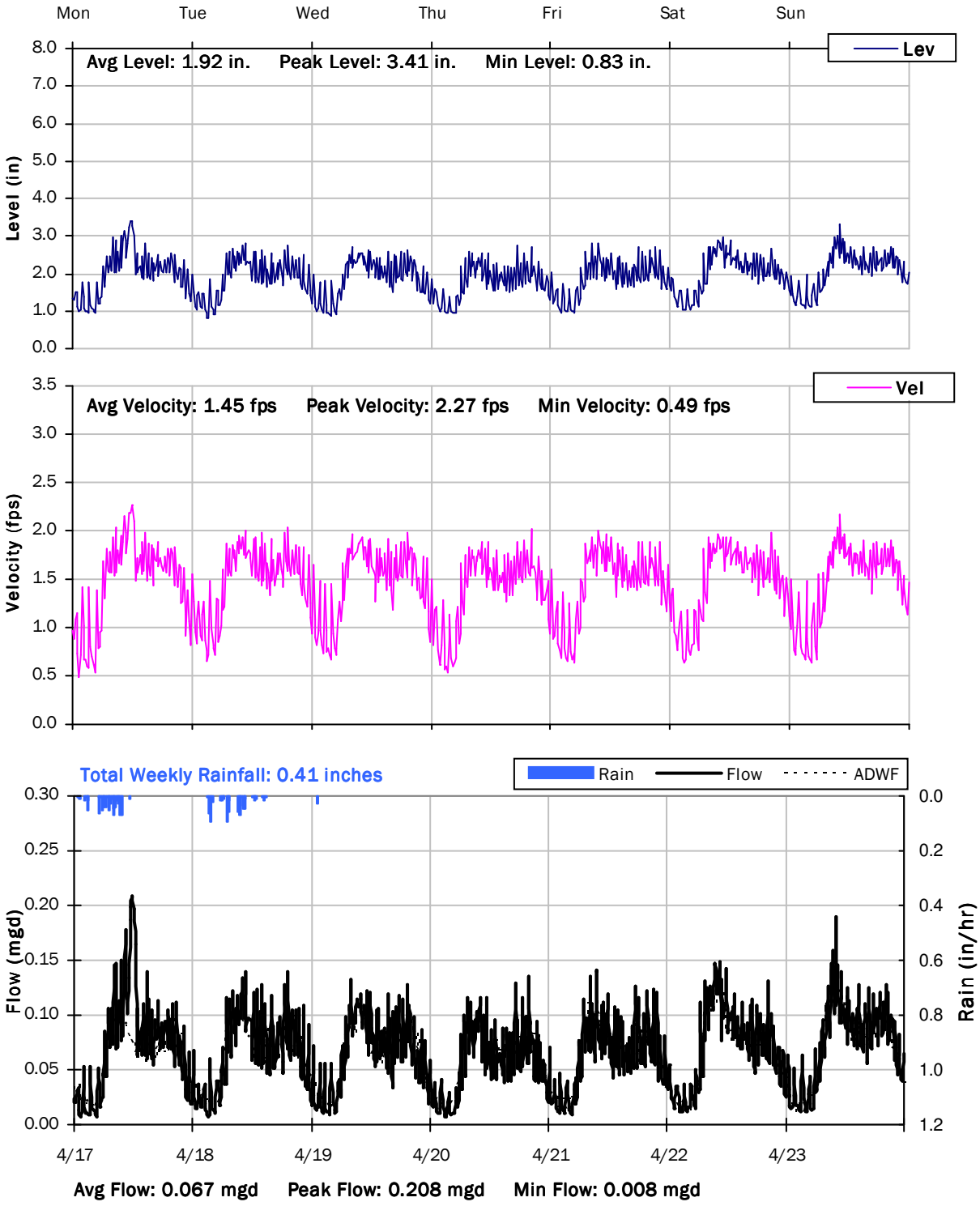
4/10/2017 to 4/17/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

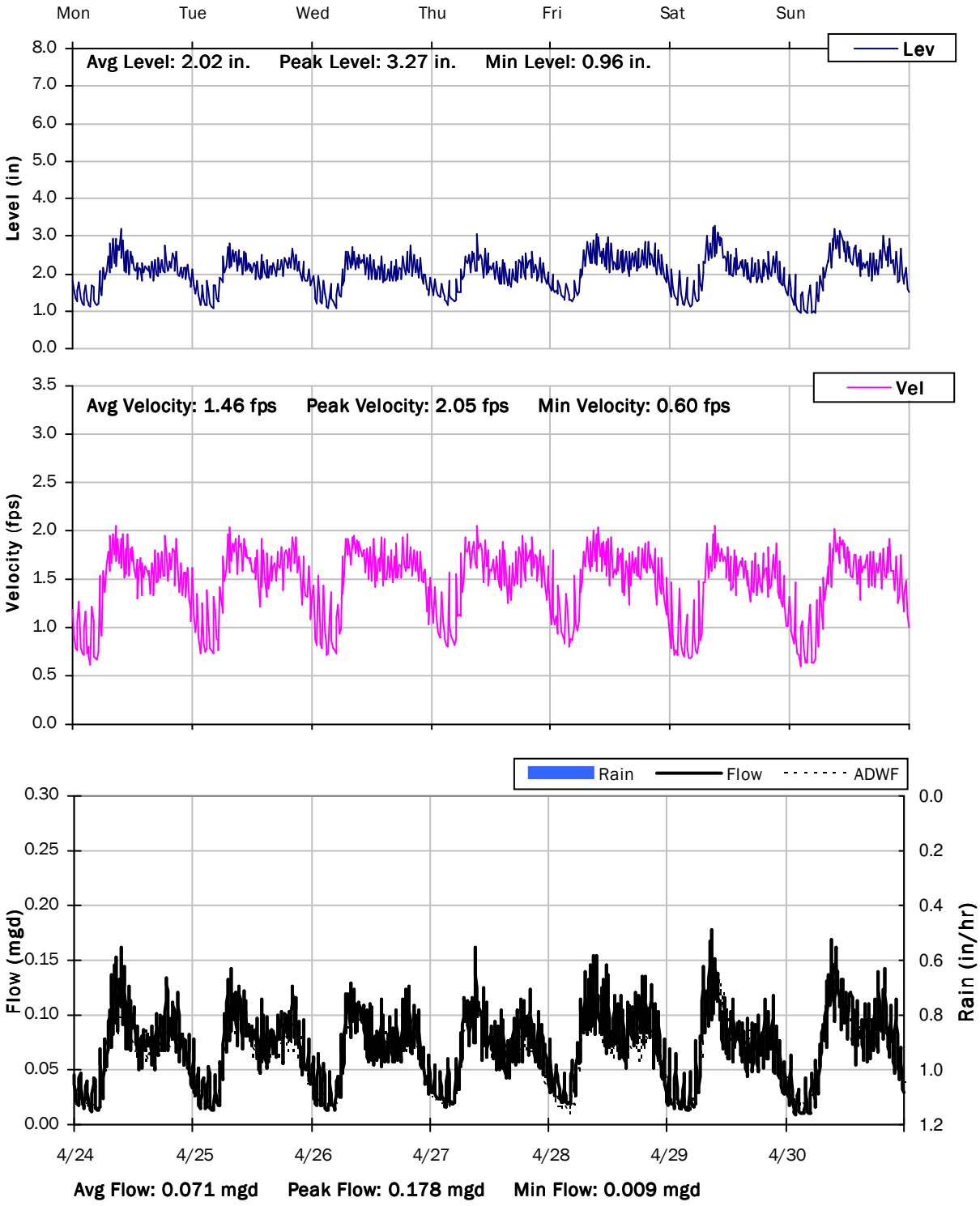
4/17/2017 to 4/24/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

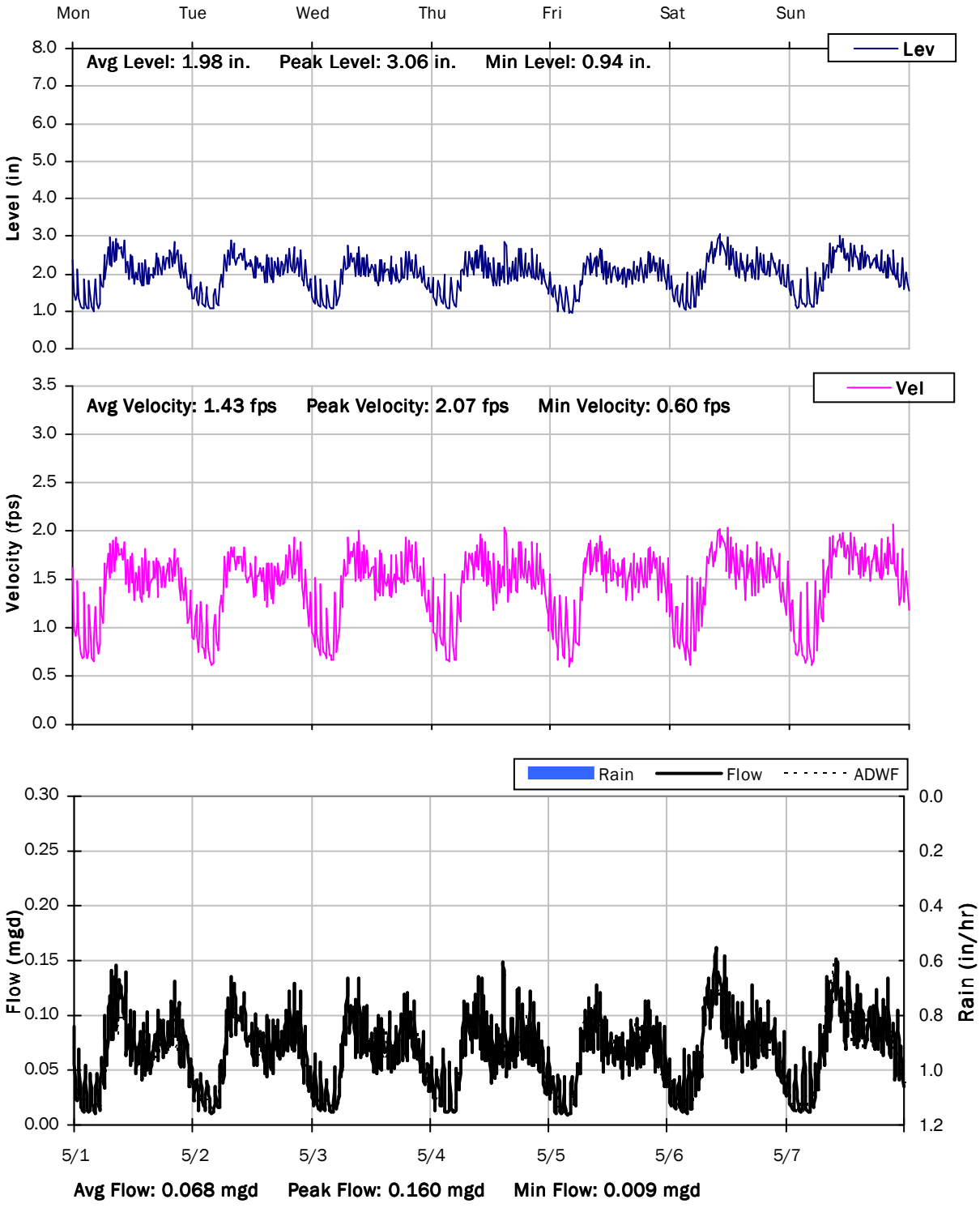
4/24/2017 to 5/1/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

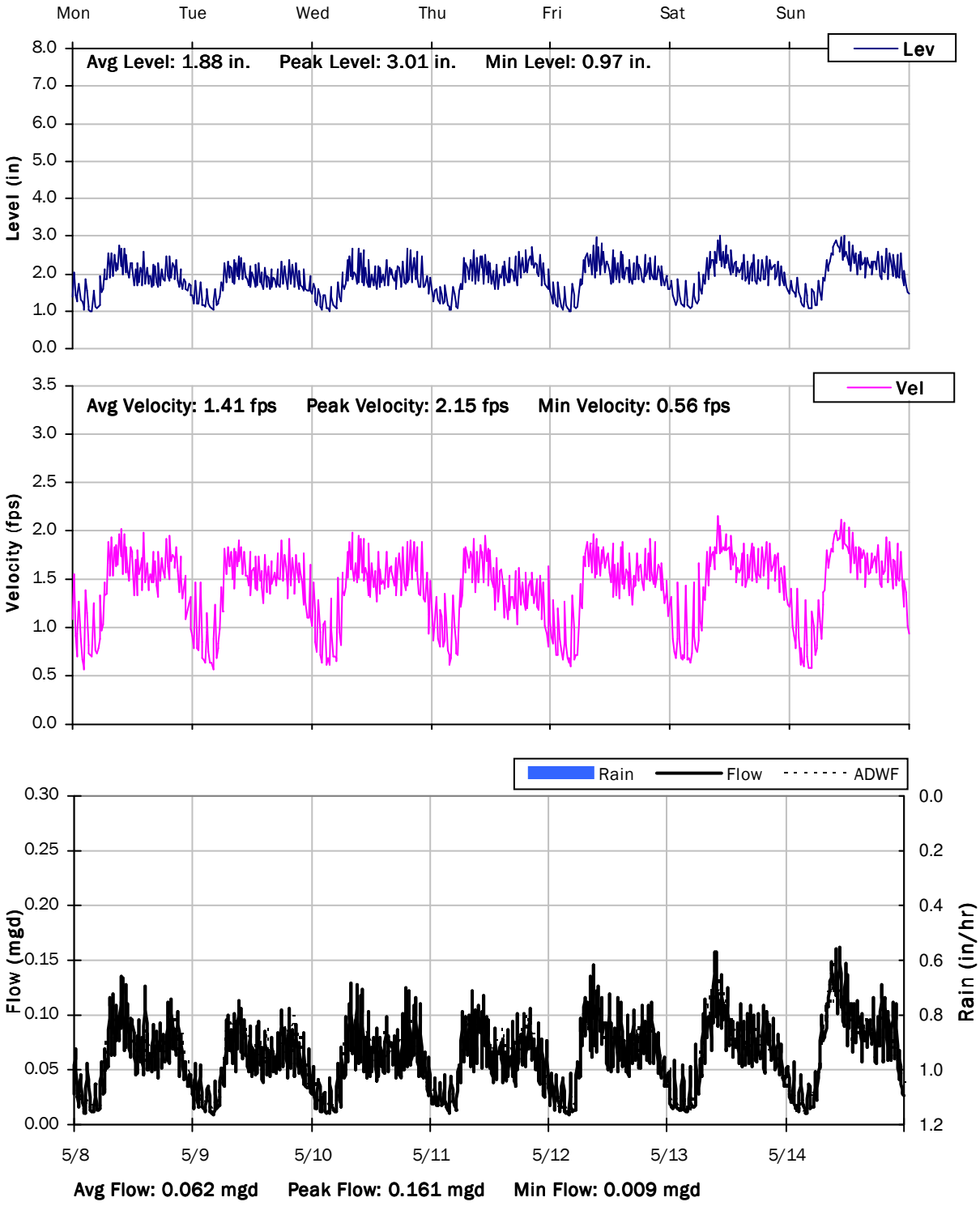
5/1/2017 to 5/8/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

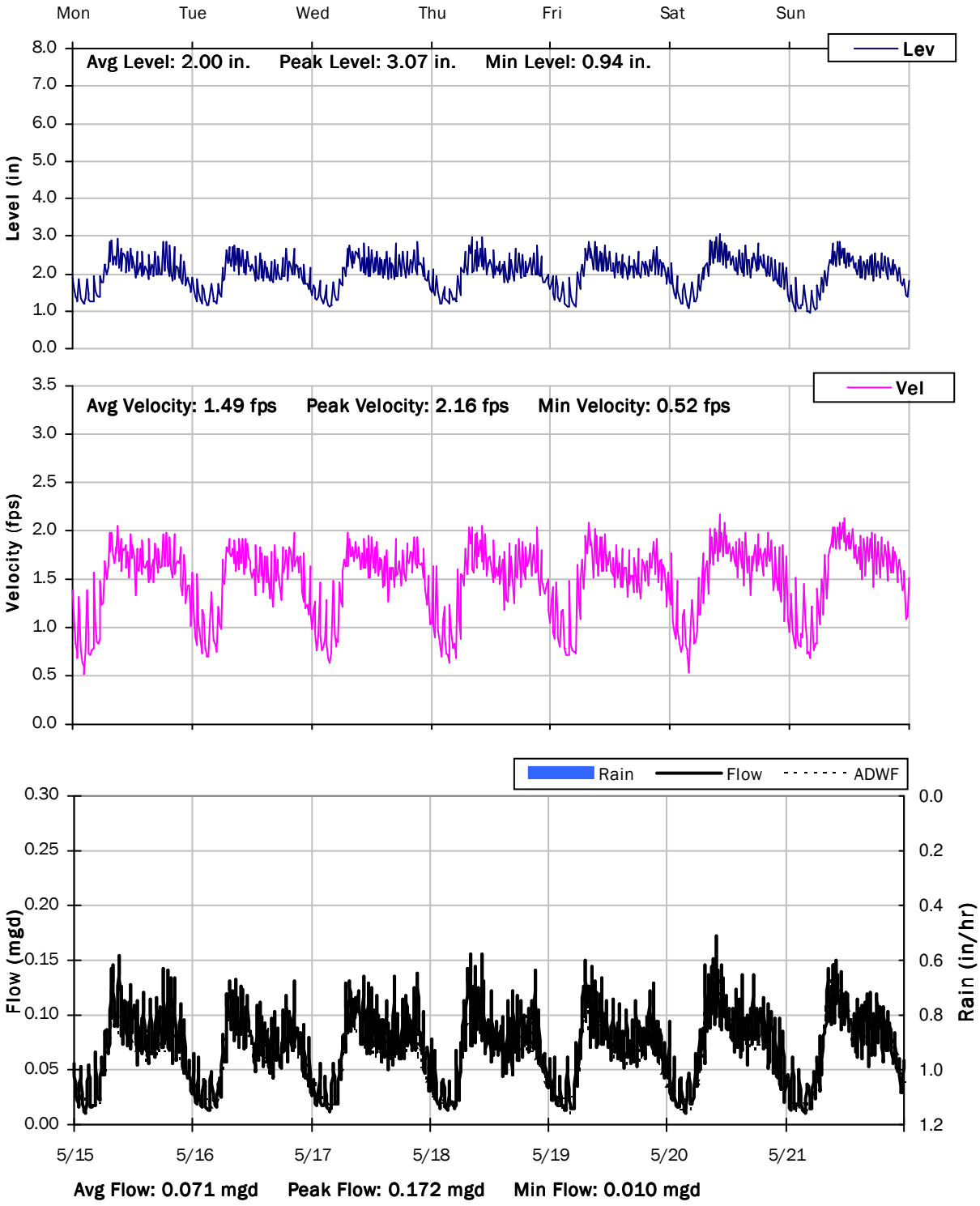
5/8/2017 to 5/15/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

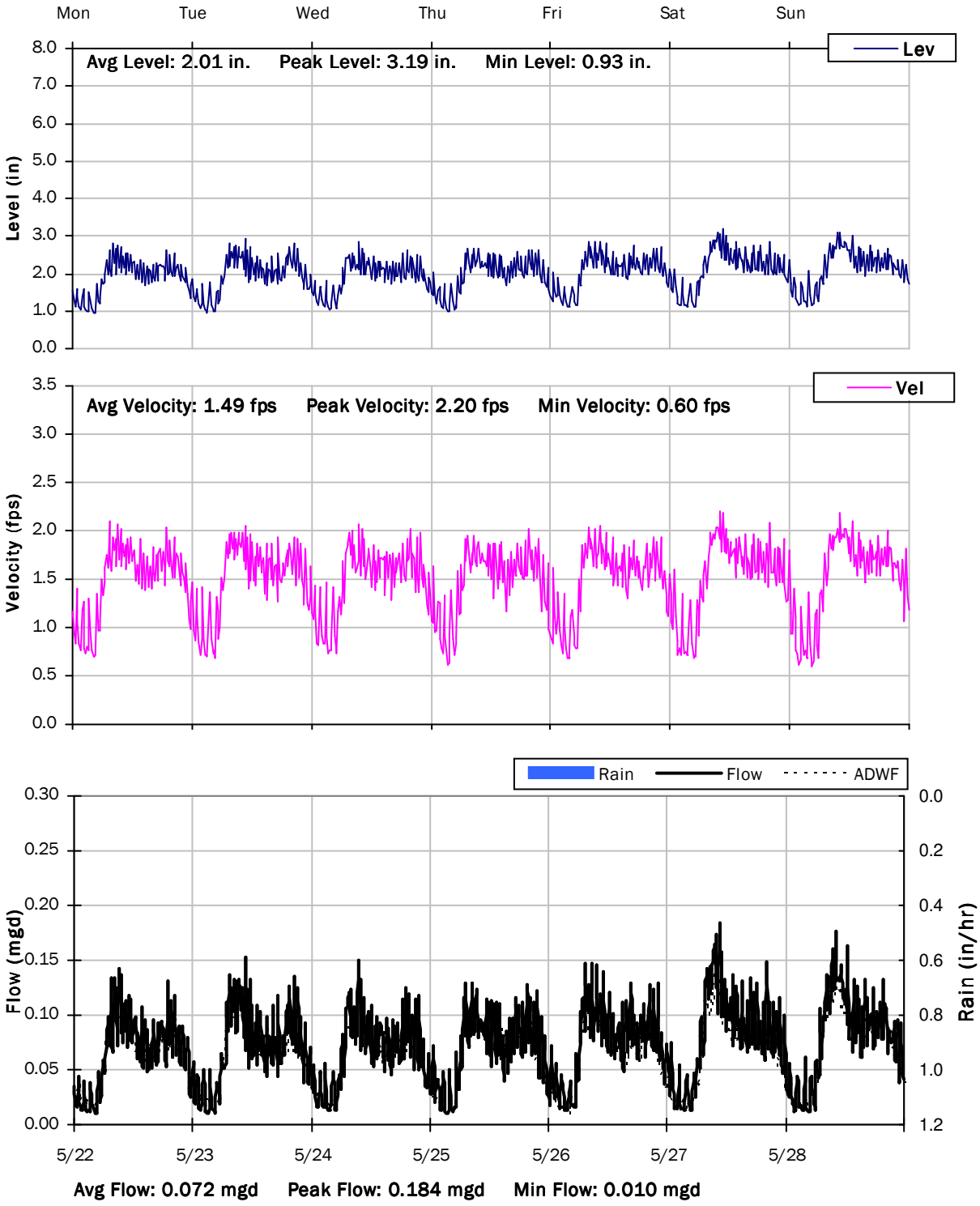
5/15/2017 to 5/22/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

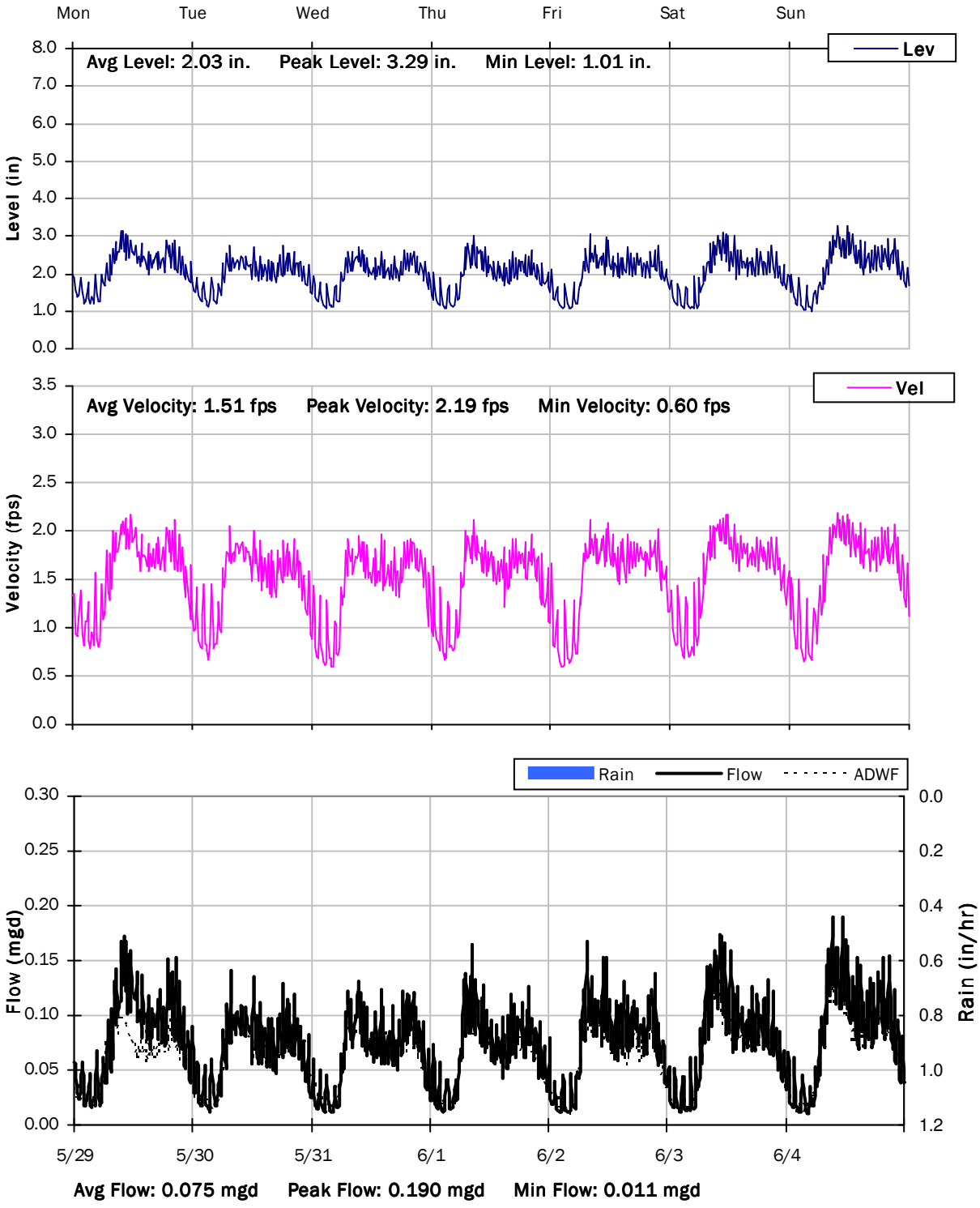
5/22/2017 to 5/29/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

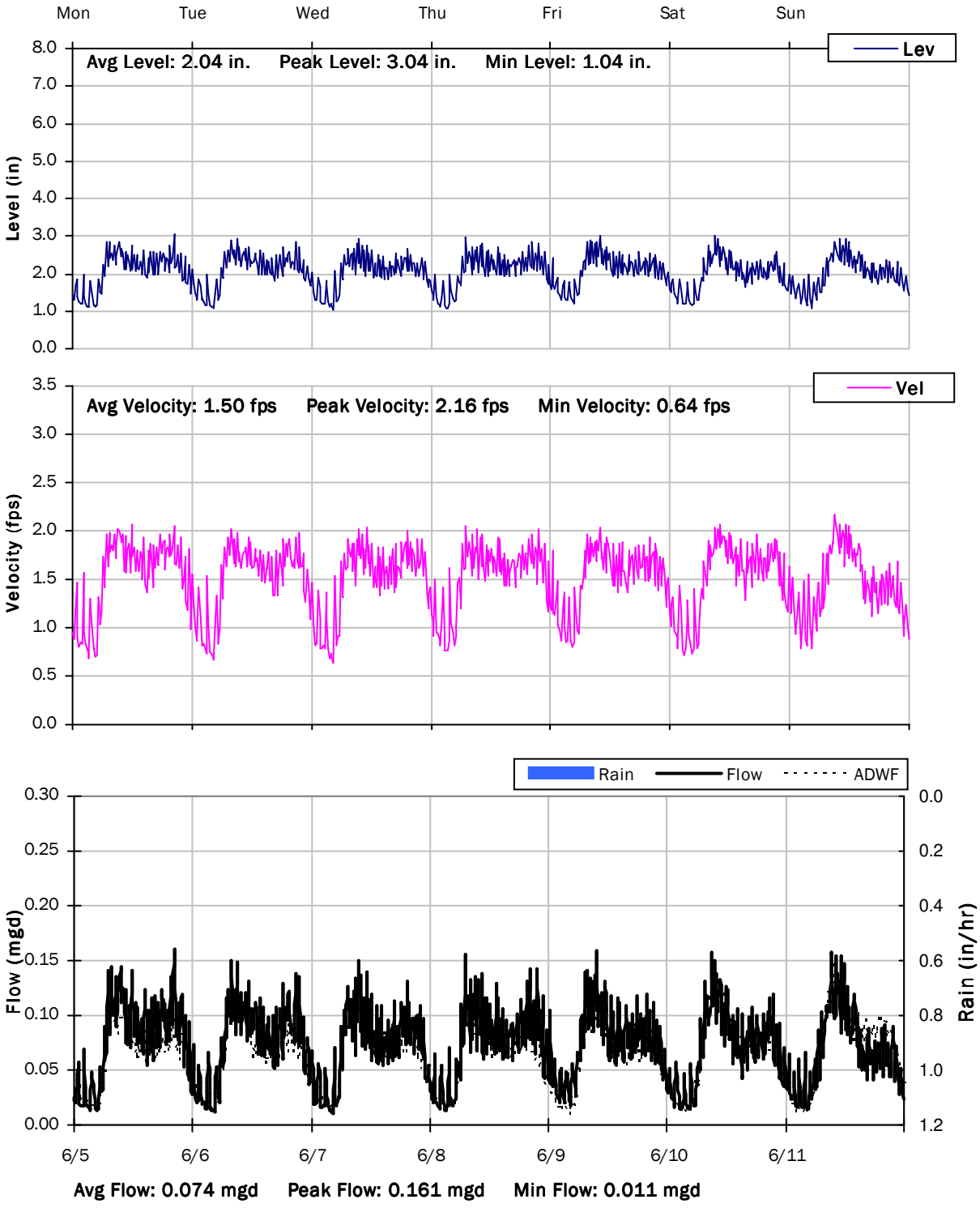
5/29/2017 to 6/5/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

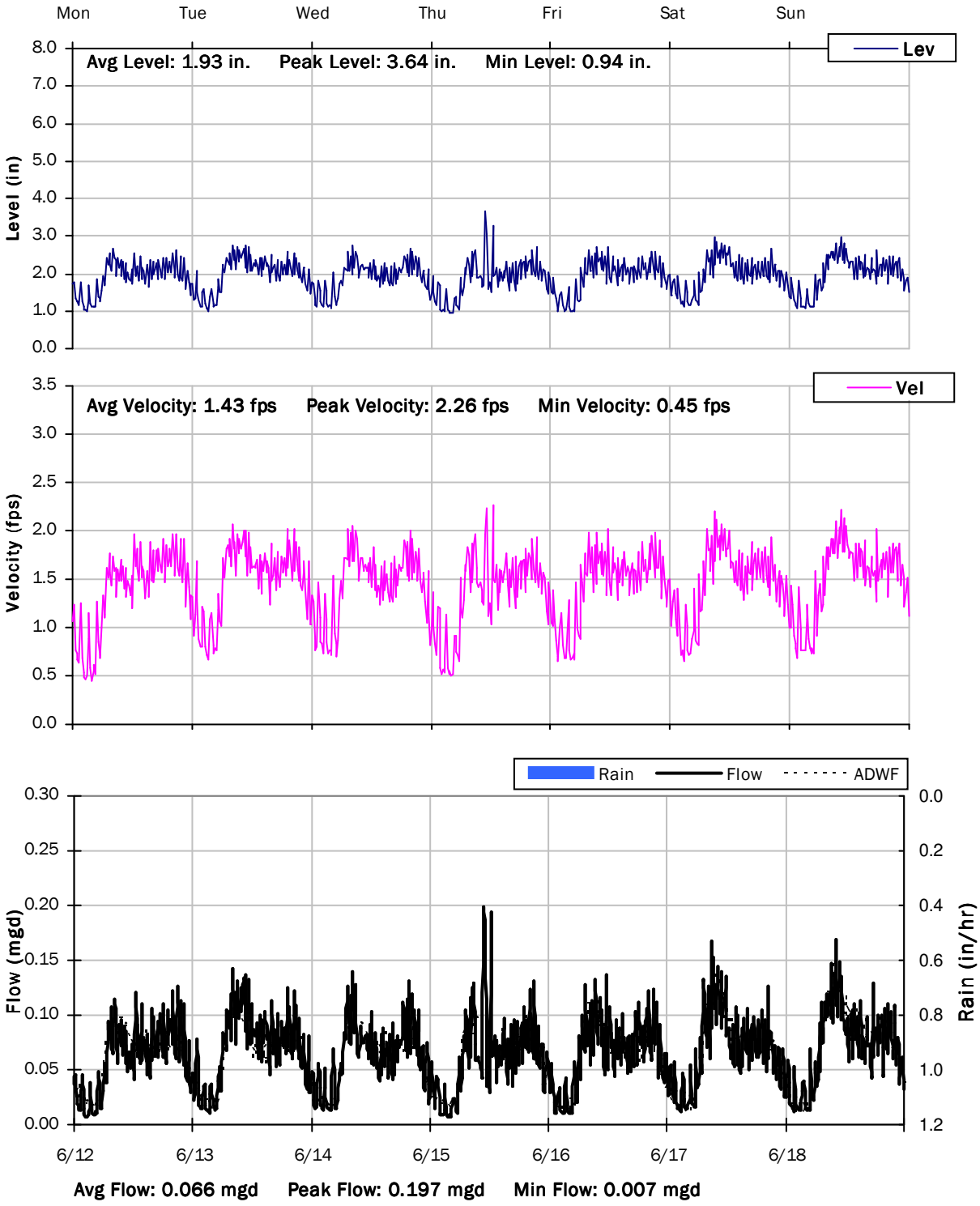
6/5/2017 to 6/12/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

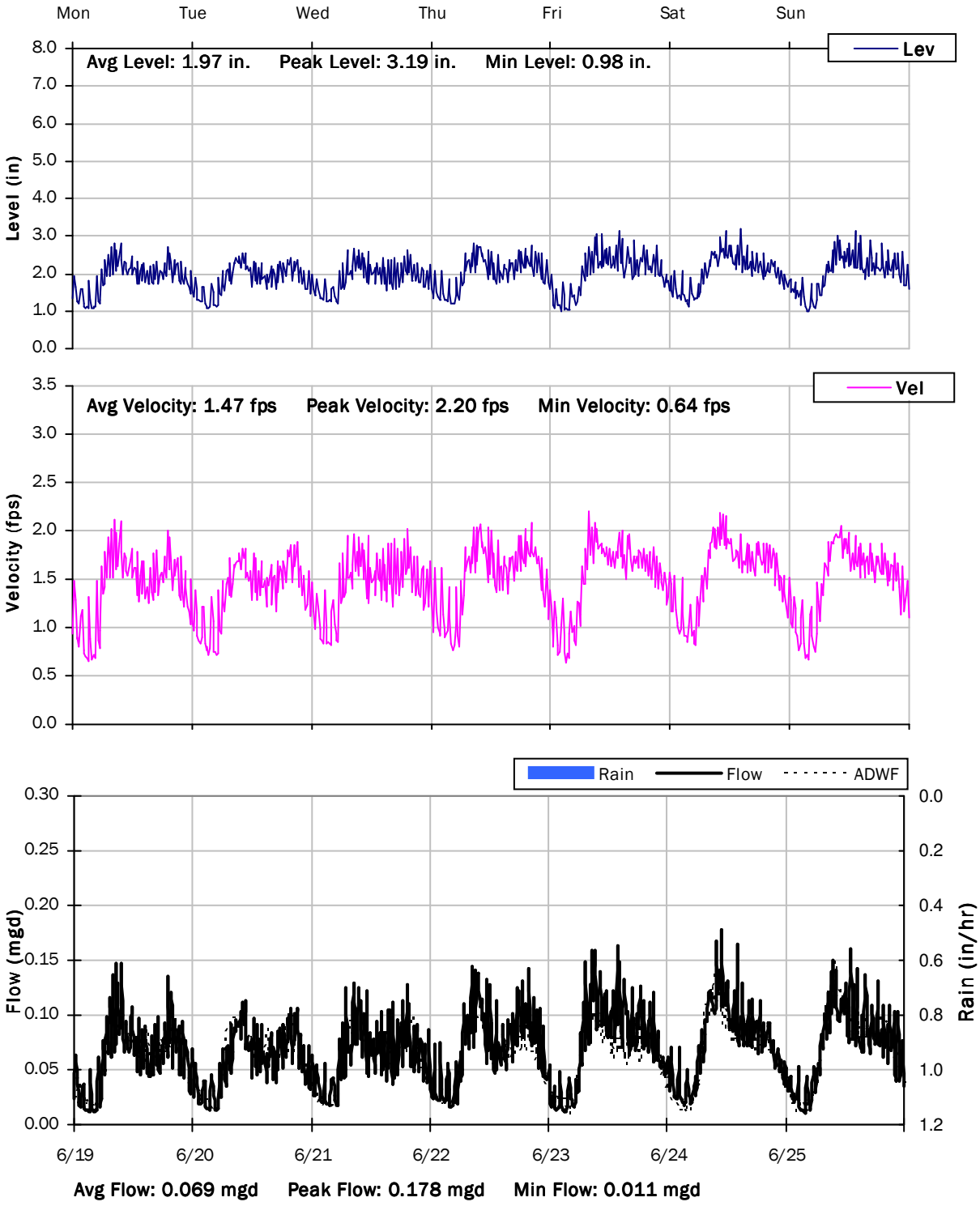
6/12/2017 to 6/19/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

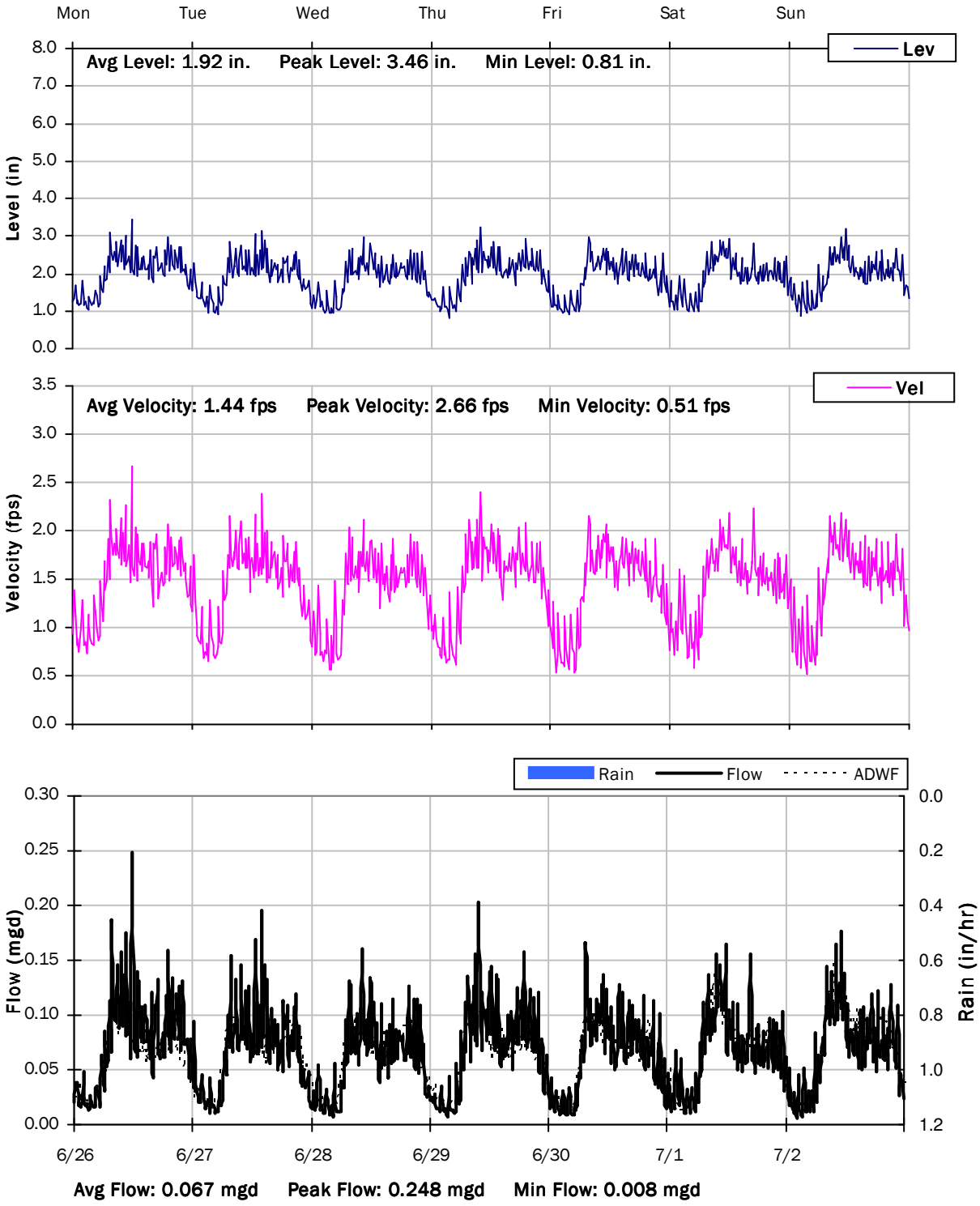
6/19/2017 to 6/26/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

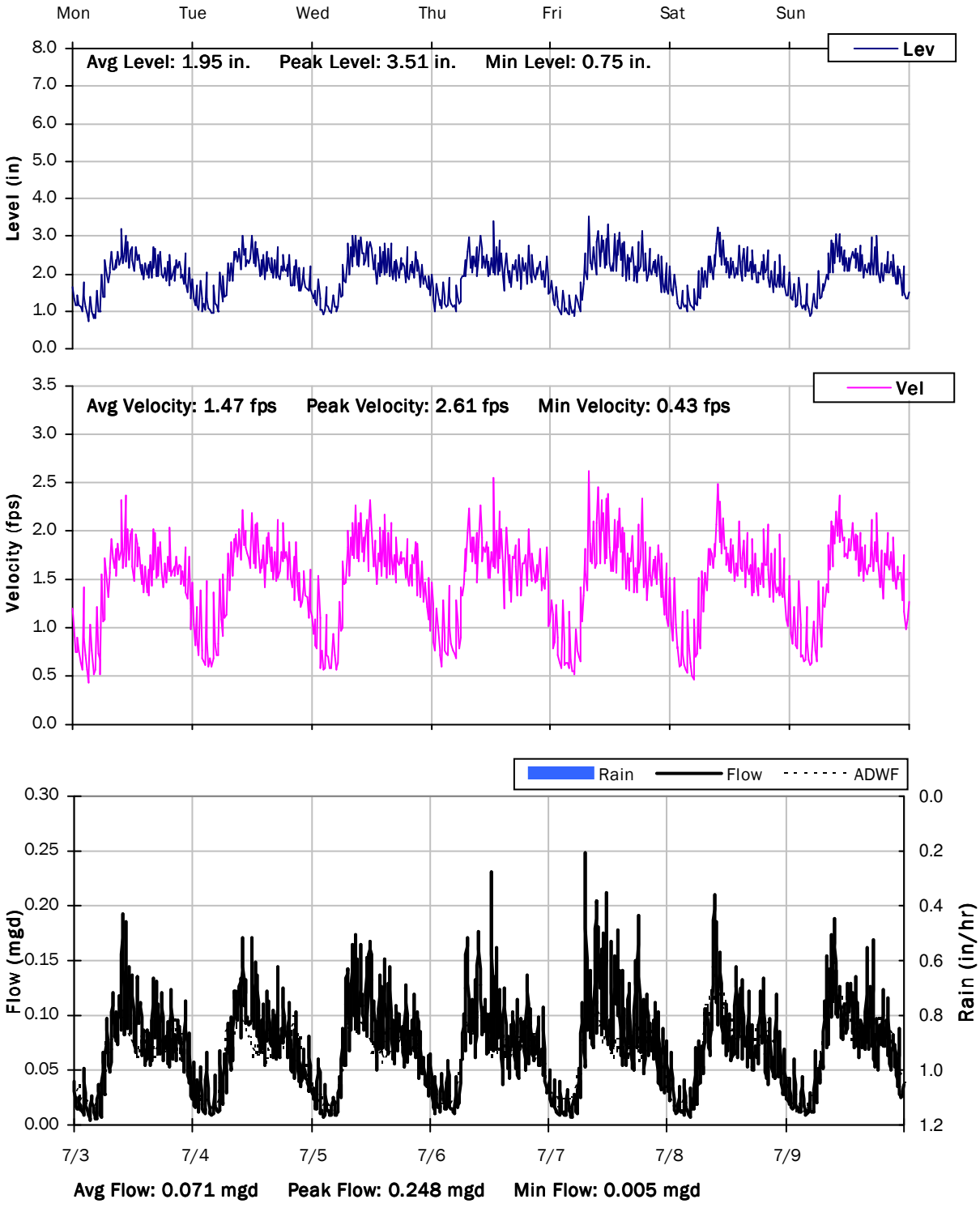
6/26/2017 to 7/3/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

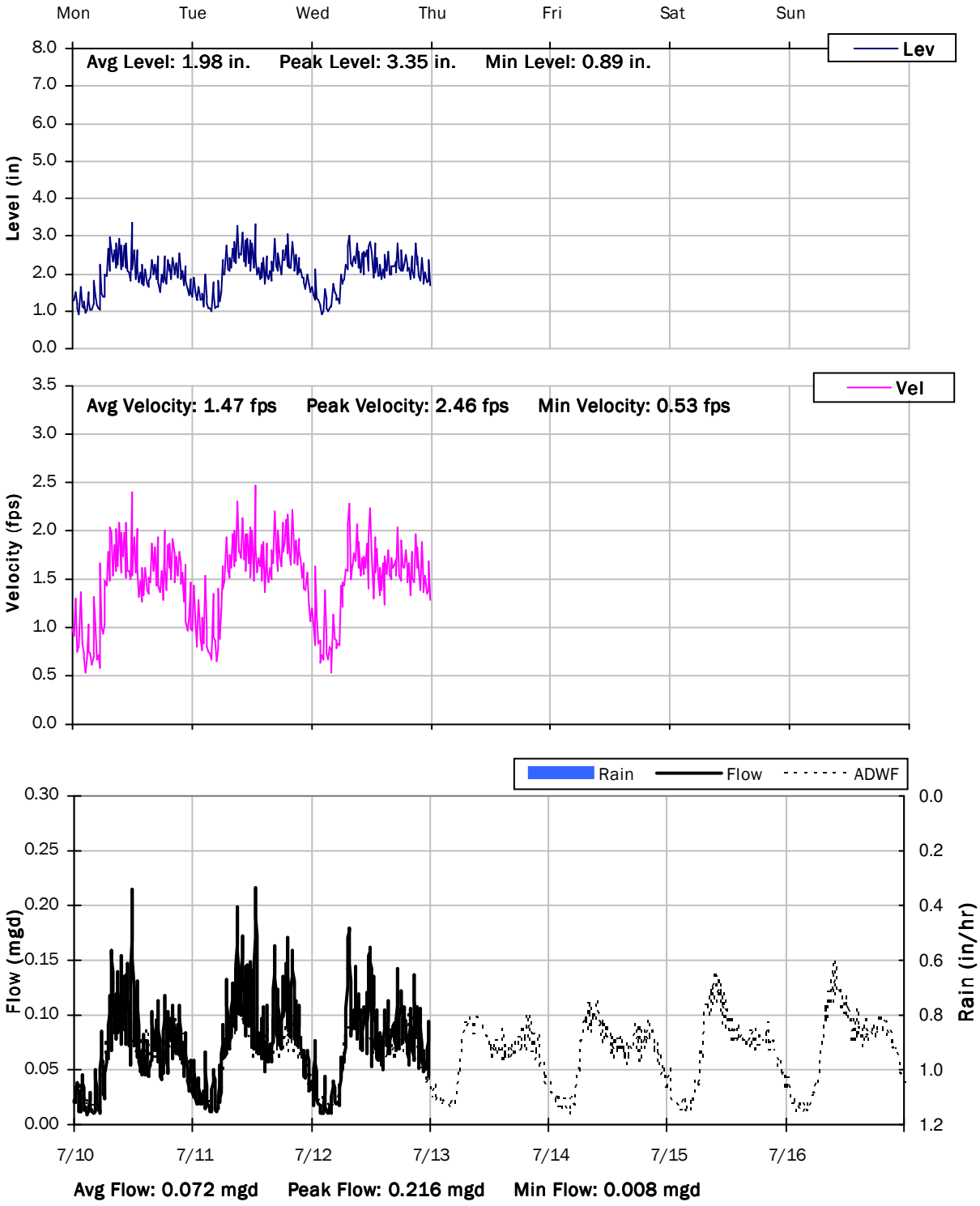
7/3/2017 to 7/10/2017



SITE 11

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

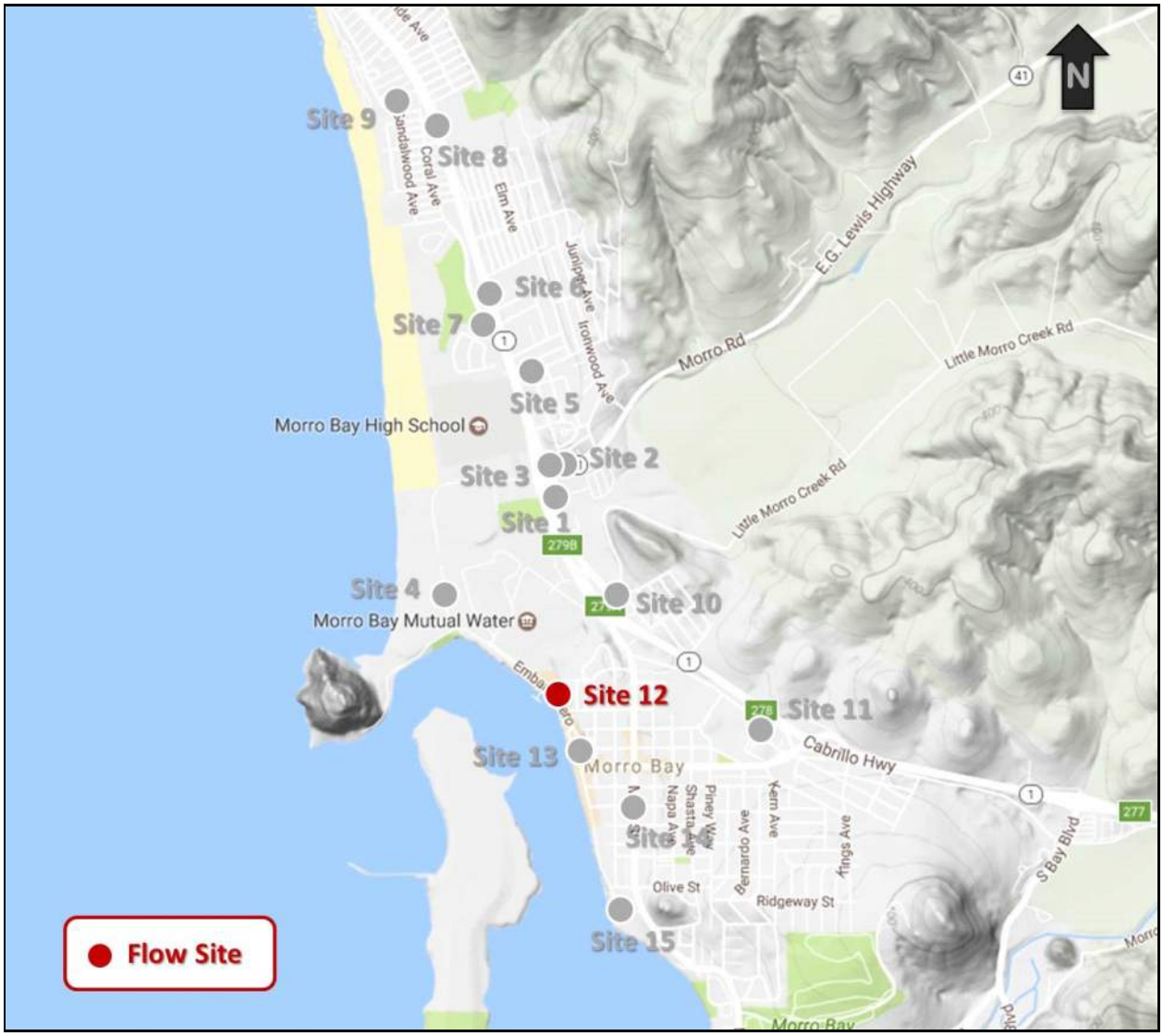
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 12

Location: Parking lot of 1154 Embarcadero, meter on south 14.375" inlet

Data Summary Report



Vicinity Map: Site 12

SITE 12

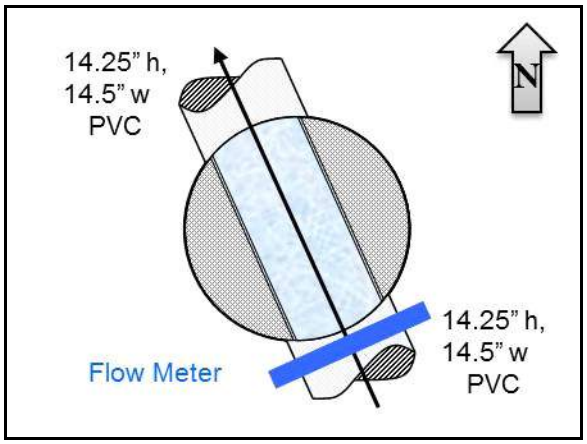
Site Information

Location: Parking lot of 1154 Embarcadero, meter on south 14.375" inlet

Pipe Diameter: 14.375 inches

ADWF: 0.129 mgd

Peak Measured Flow: 0.382 mgd



SITE 12

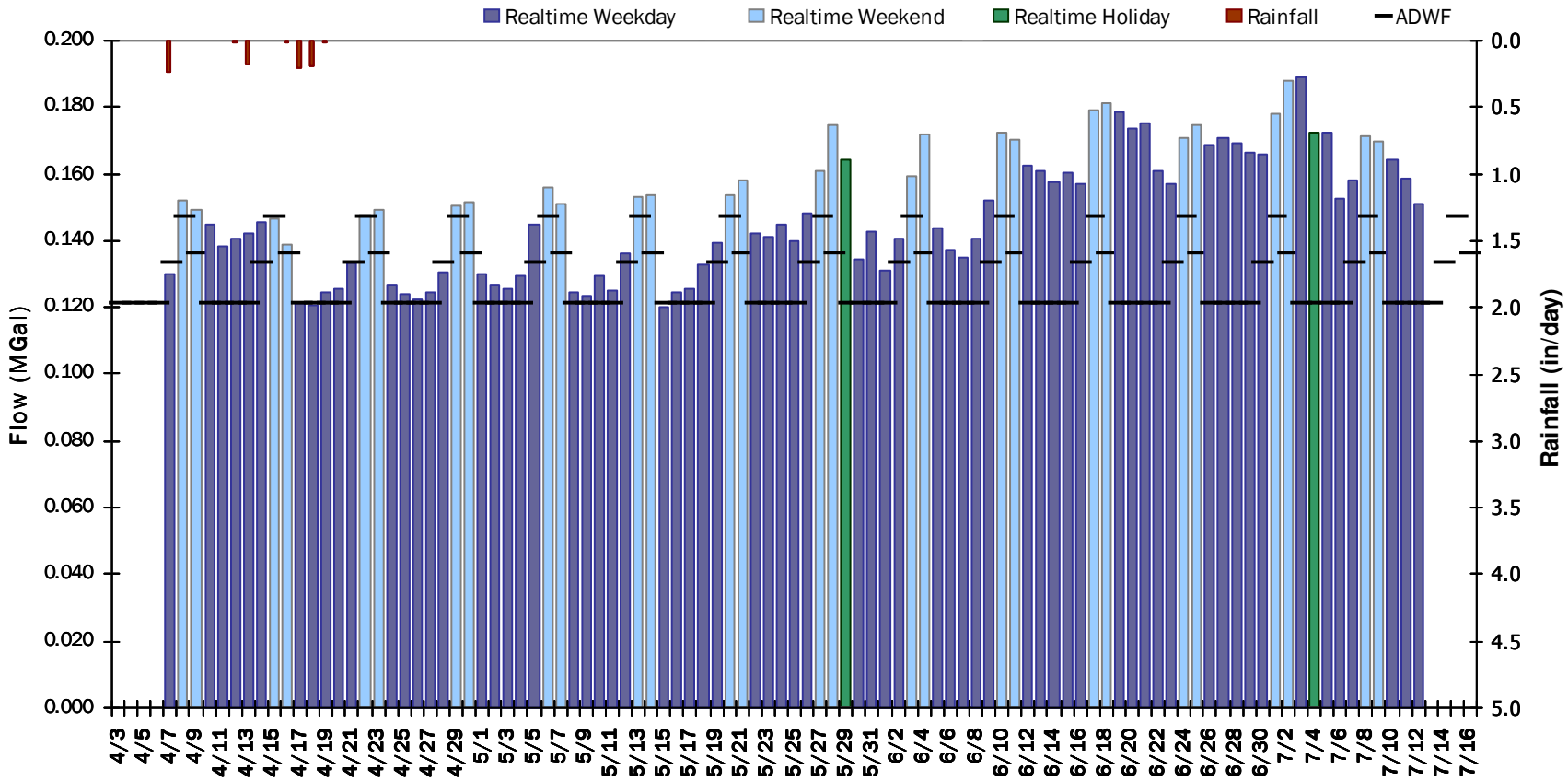
Additional Site Photos

SITE 12

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.150 MGal Peak Daily Flow: 0.189 MGal Min Daily Flow: 0.120 MGal

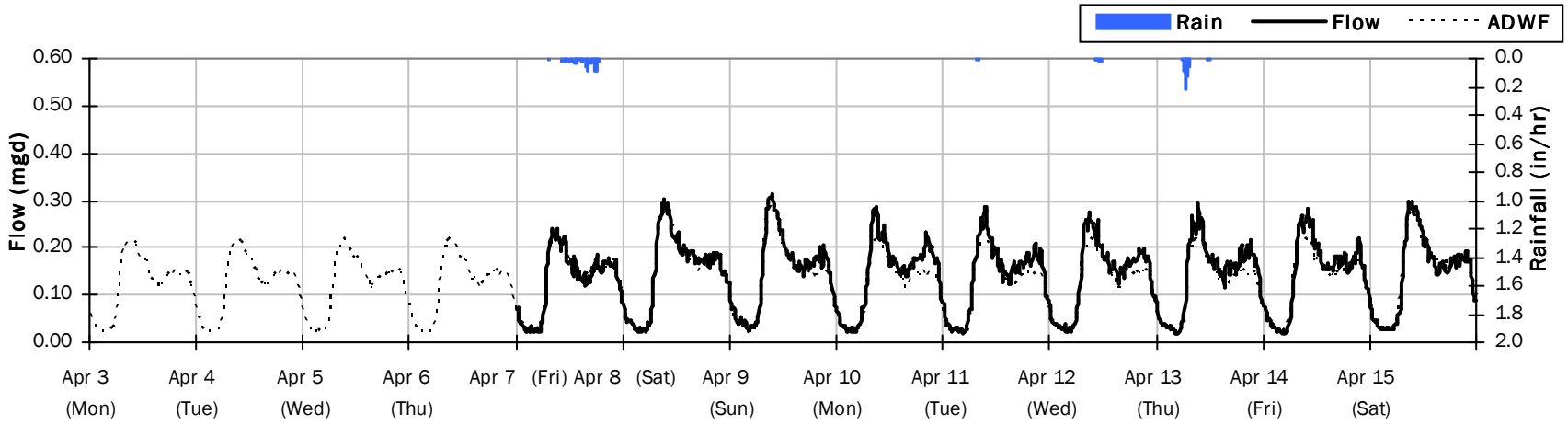
Total Period Rainfall: 0.85 inches

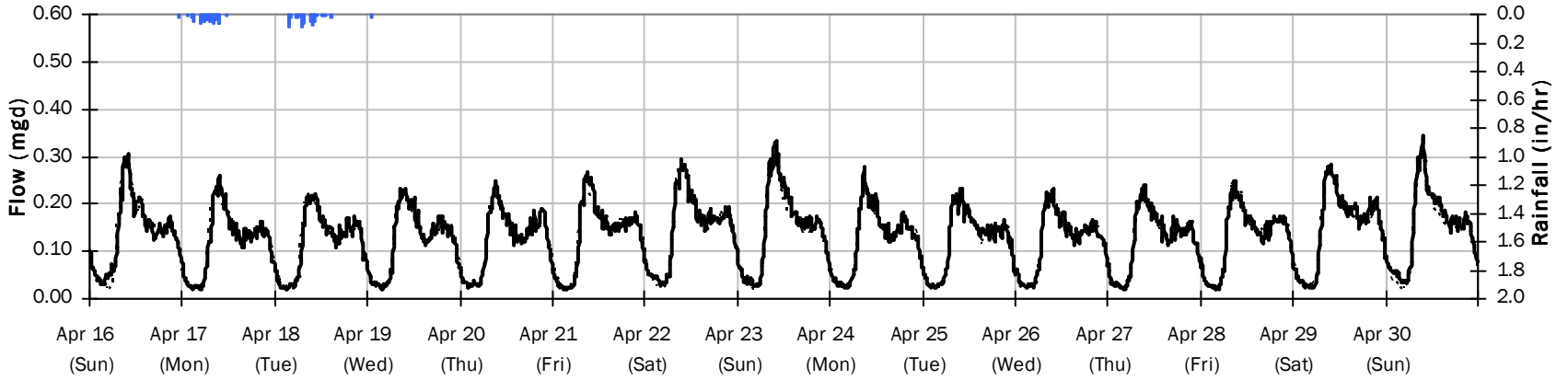


SITE 12

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.137 mgd** **Peak Flow: 0.346 mgd** **Min Flow: 0.018 mgd**

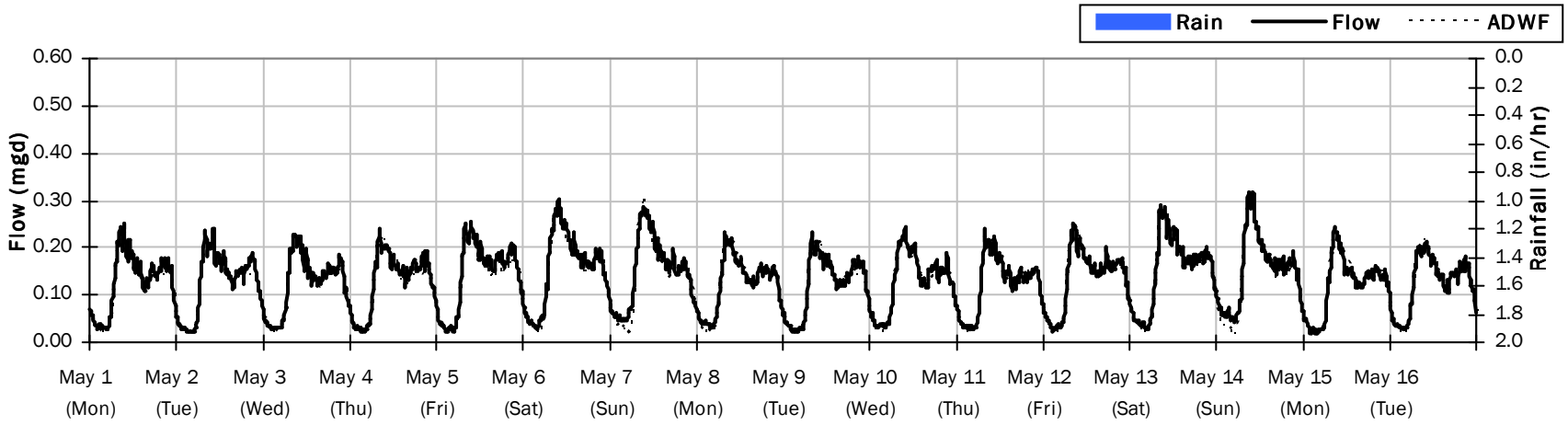


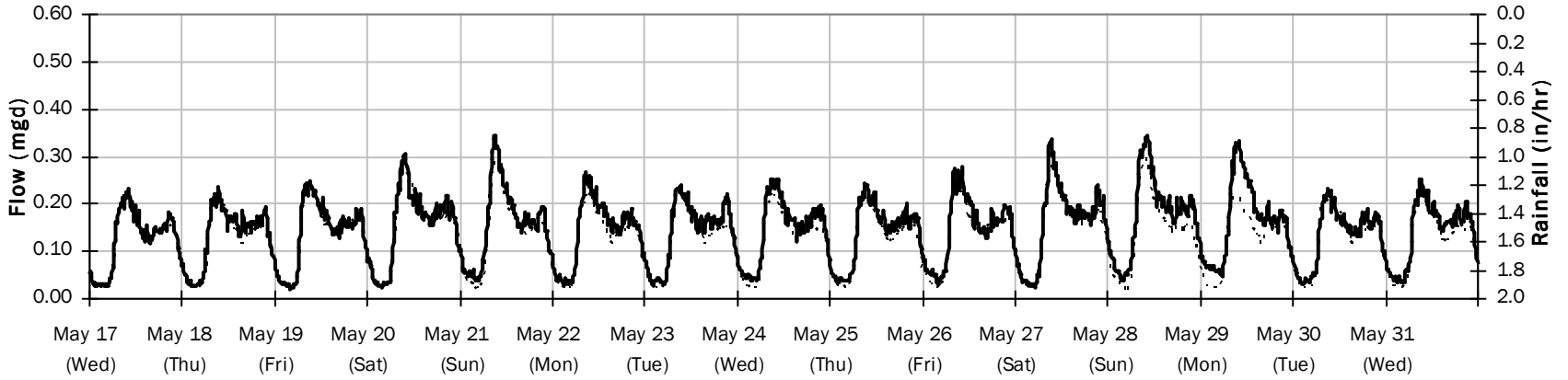


SITE 12

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.141 mgd** **Peak Flow: 0.344 mgd** **Min Flow: 0.018 mgd**

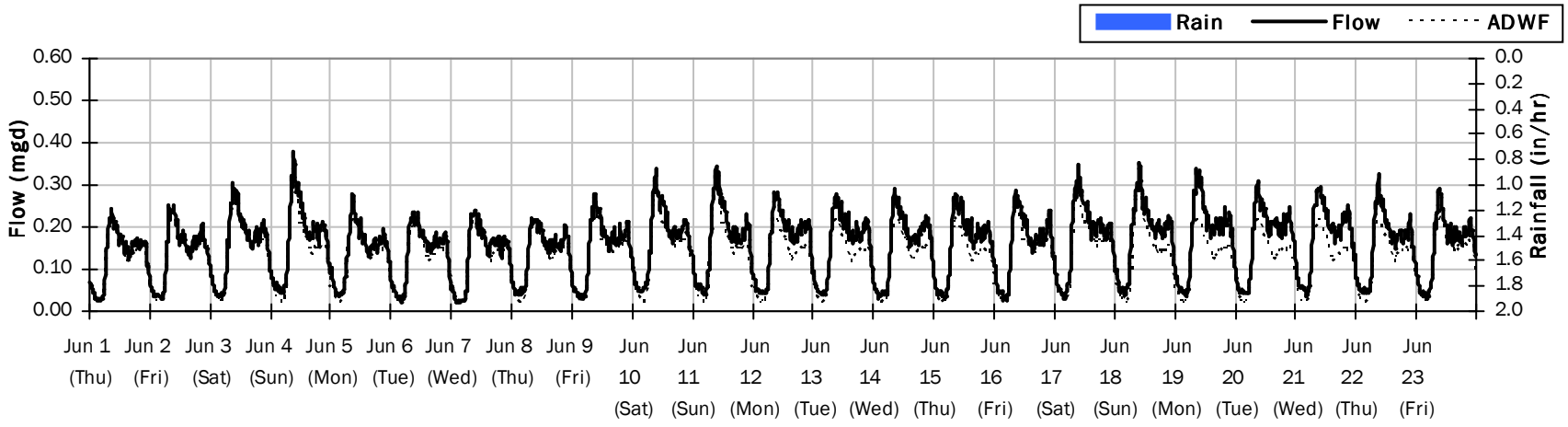


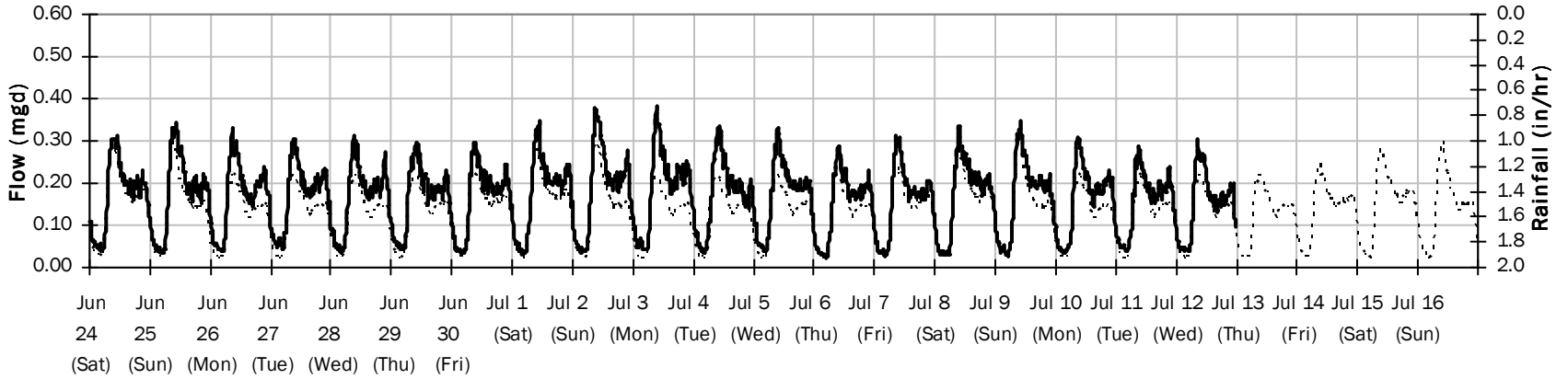


SITE 12

Flow Summary: 6/1/2017 to 7/16/2017

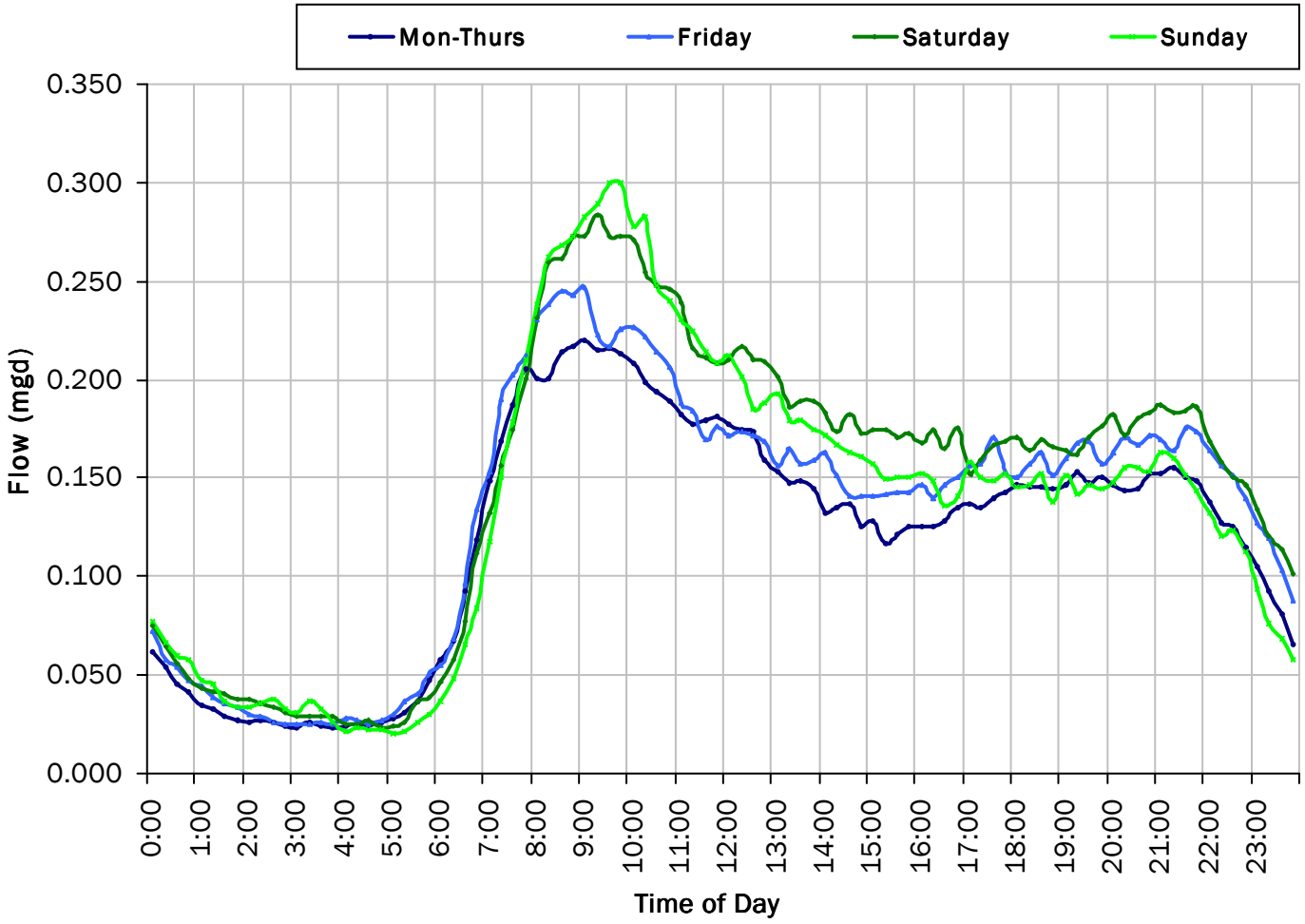
Total Period Rainfall: 0.00 inches **Avg Flow: 0.164 mgd** **Peak Flow: 0.382 mgd** **Min Flow: 0.020 mgd**





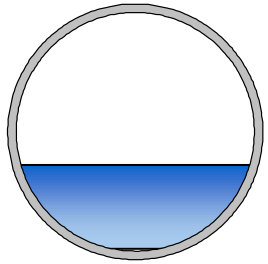
SITE 12

Average Dry Weather Flow Hydrographs



ADWF:

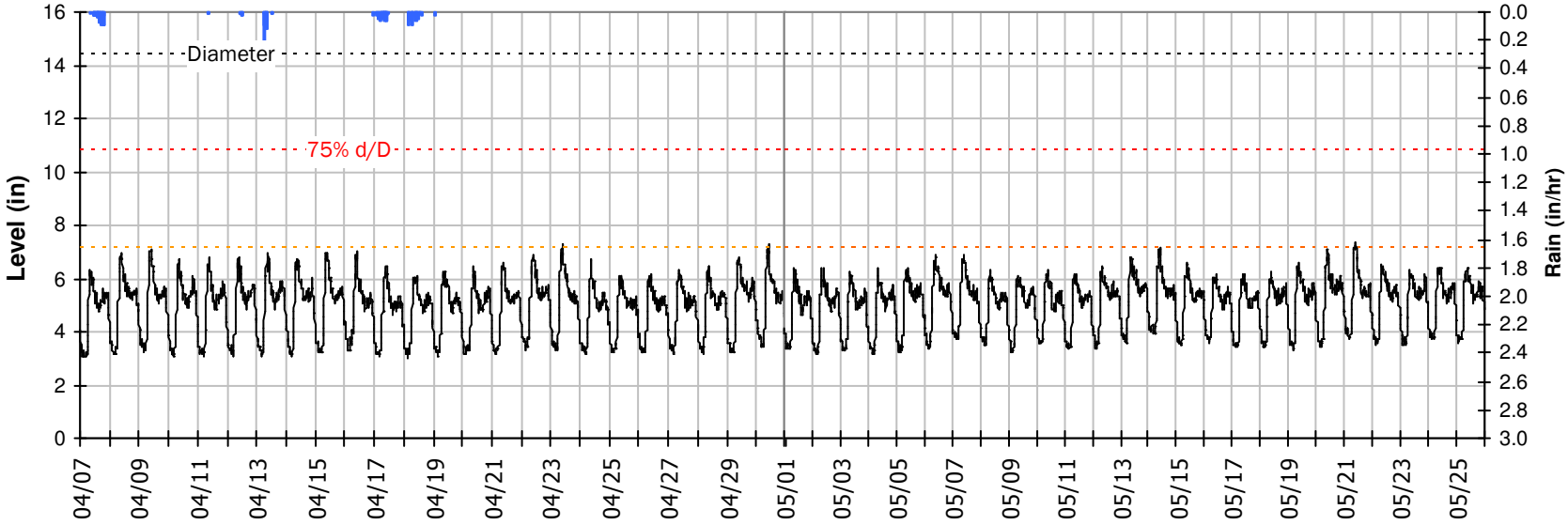
0.129 *mgd*

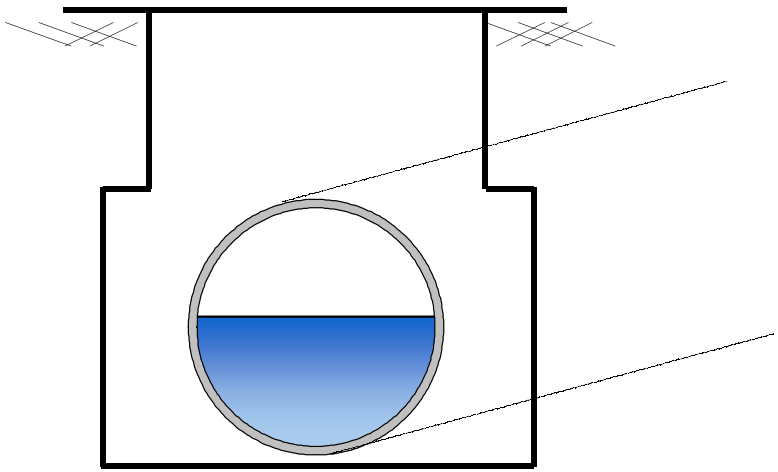


SITE 12

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



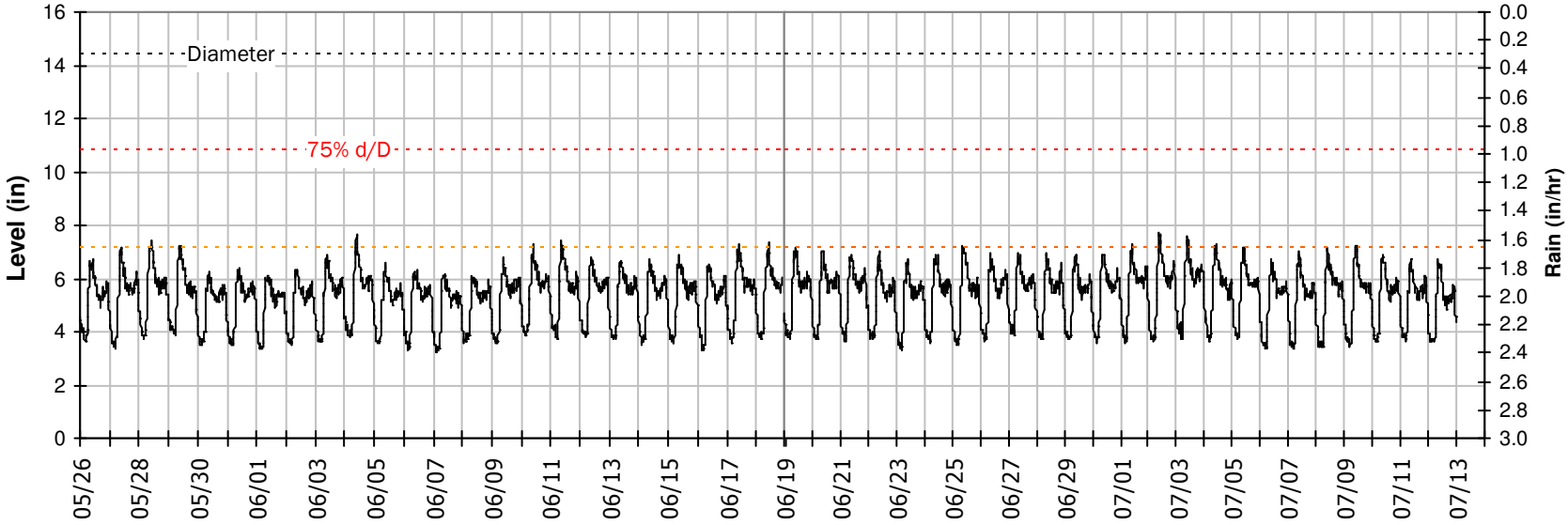


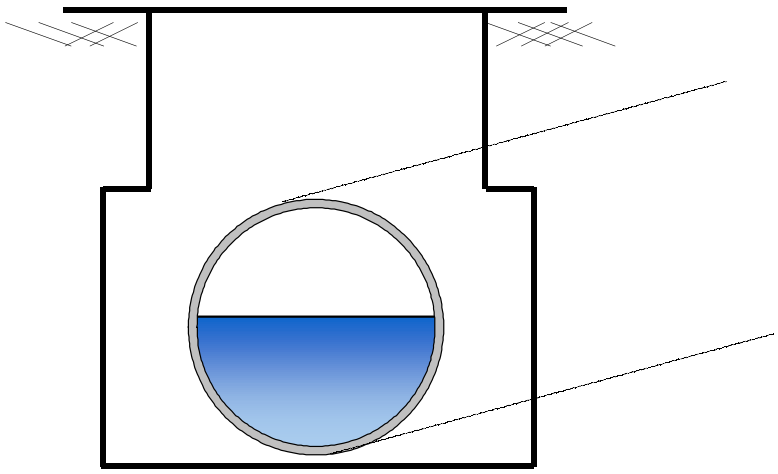
Pipe Diameter:	14.4	*inches*
Peak Measured Level:	7.73	*inches*
Peak d/D Ratio:	0.54	

SITE 12

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



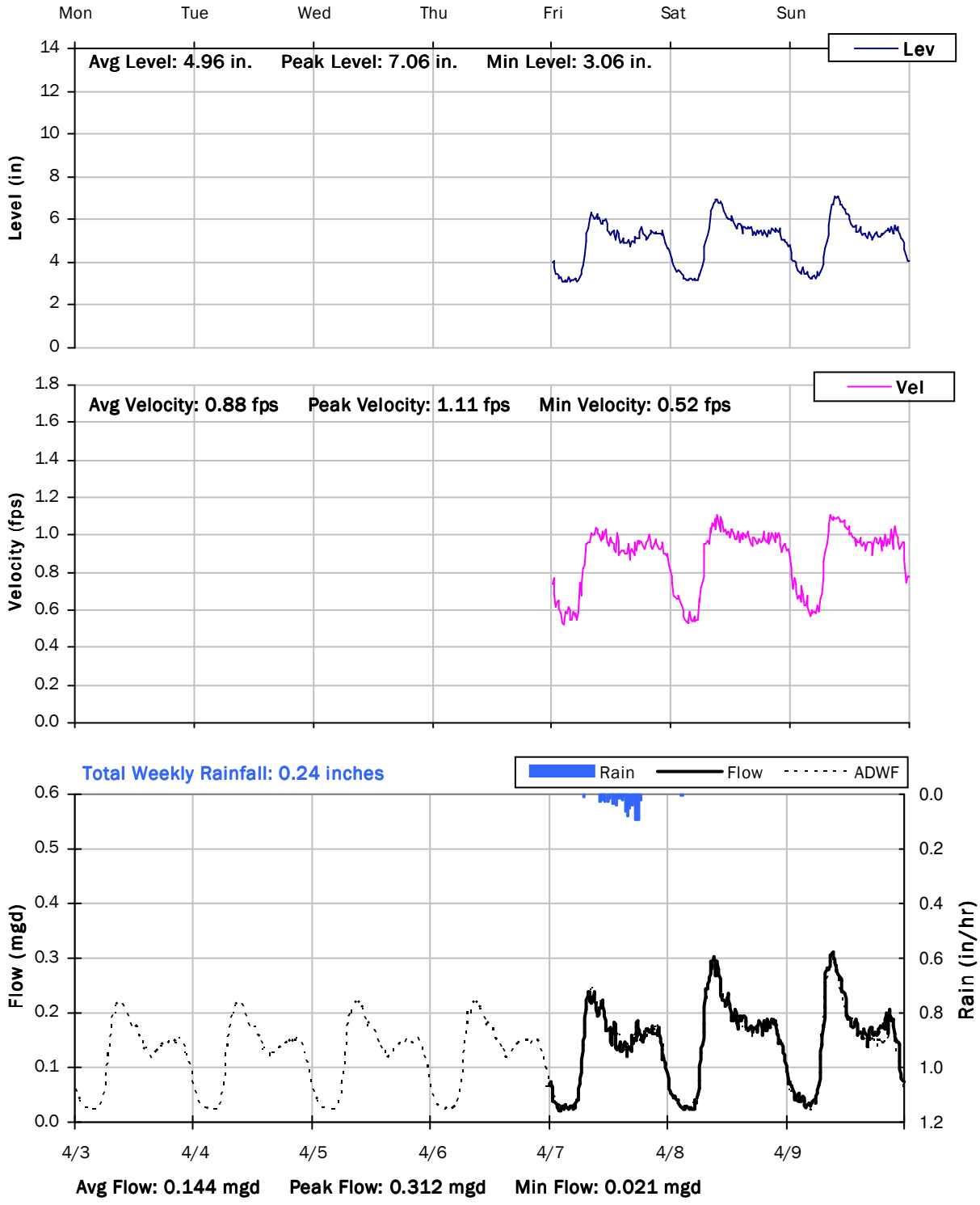


Pipe Diameter:	14.4	*inches*
Peak Measured Level:	7.73	*inches*
Peak d/D Ratio:	0.54	

SITE 12

Weekly Level, Velocity and Flow Hydrographs

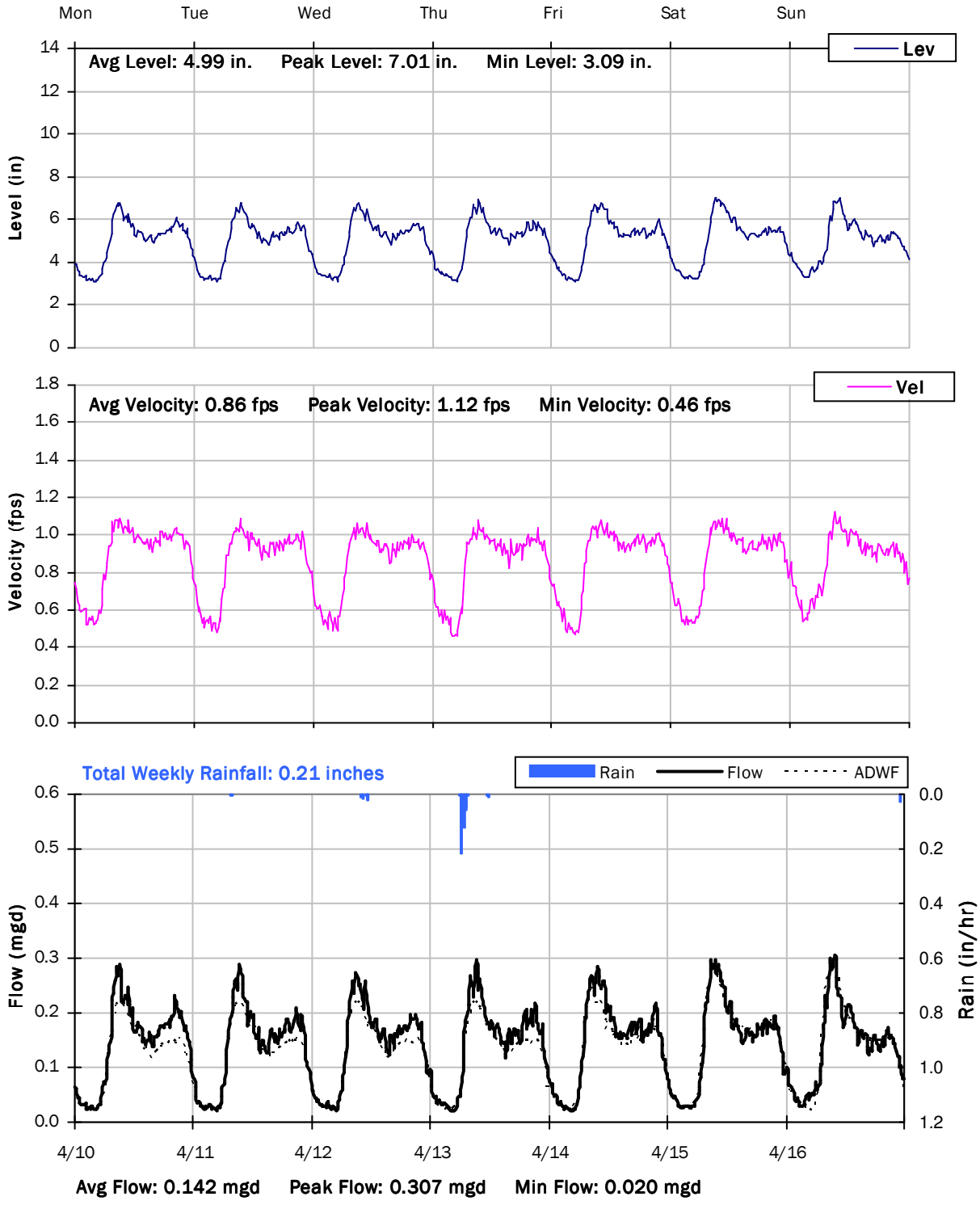
4/3/2017 to 4/10/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

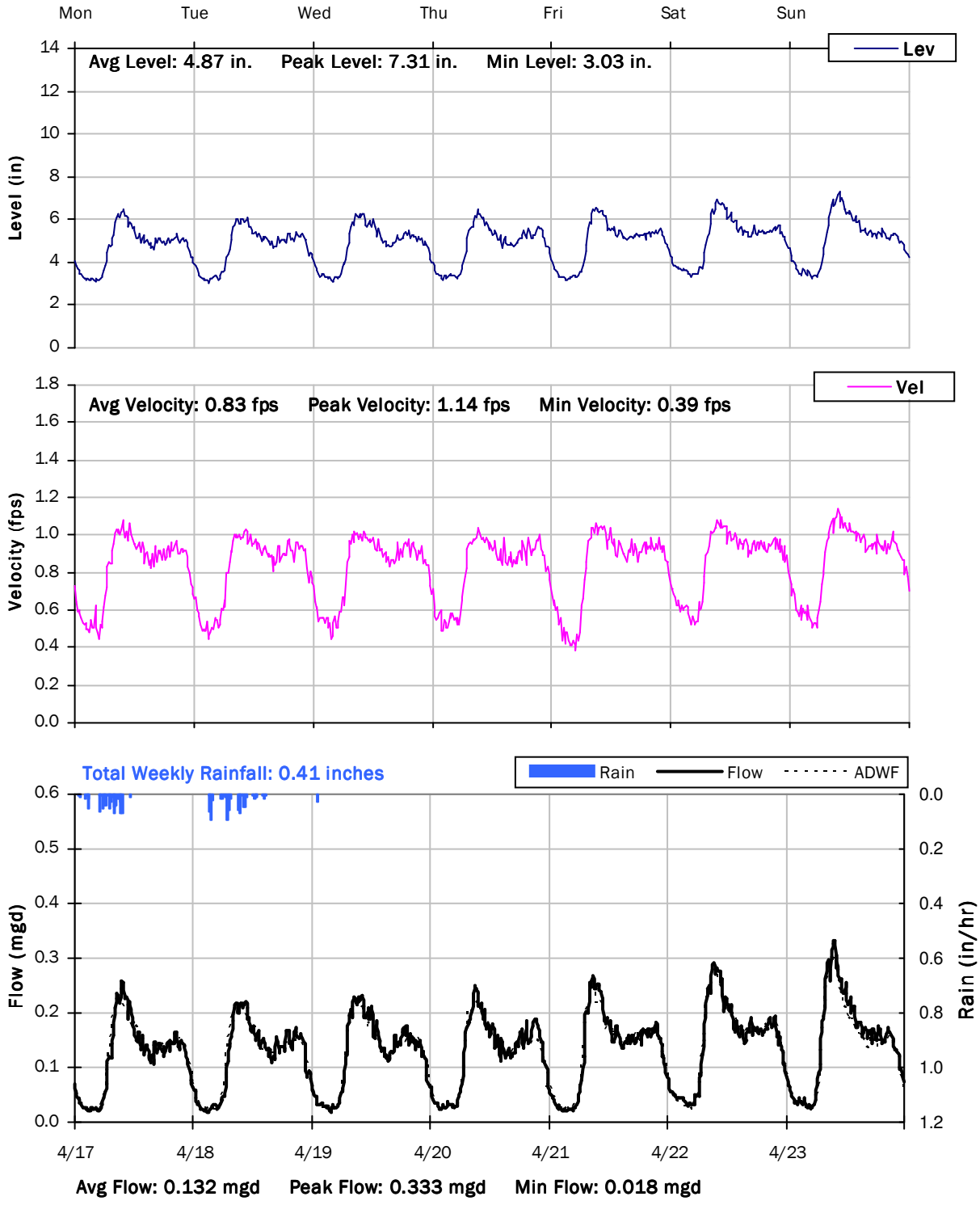
4/10/2017 to 4/17/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

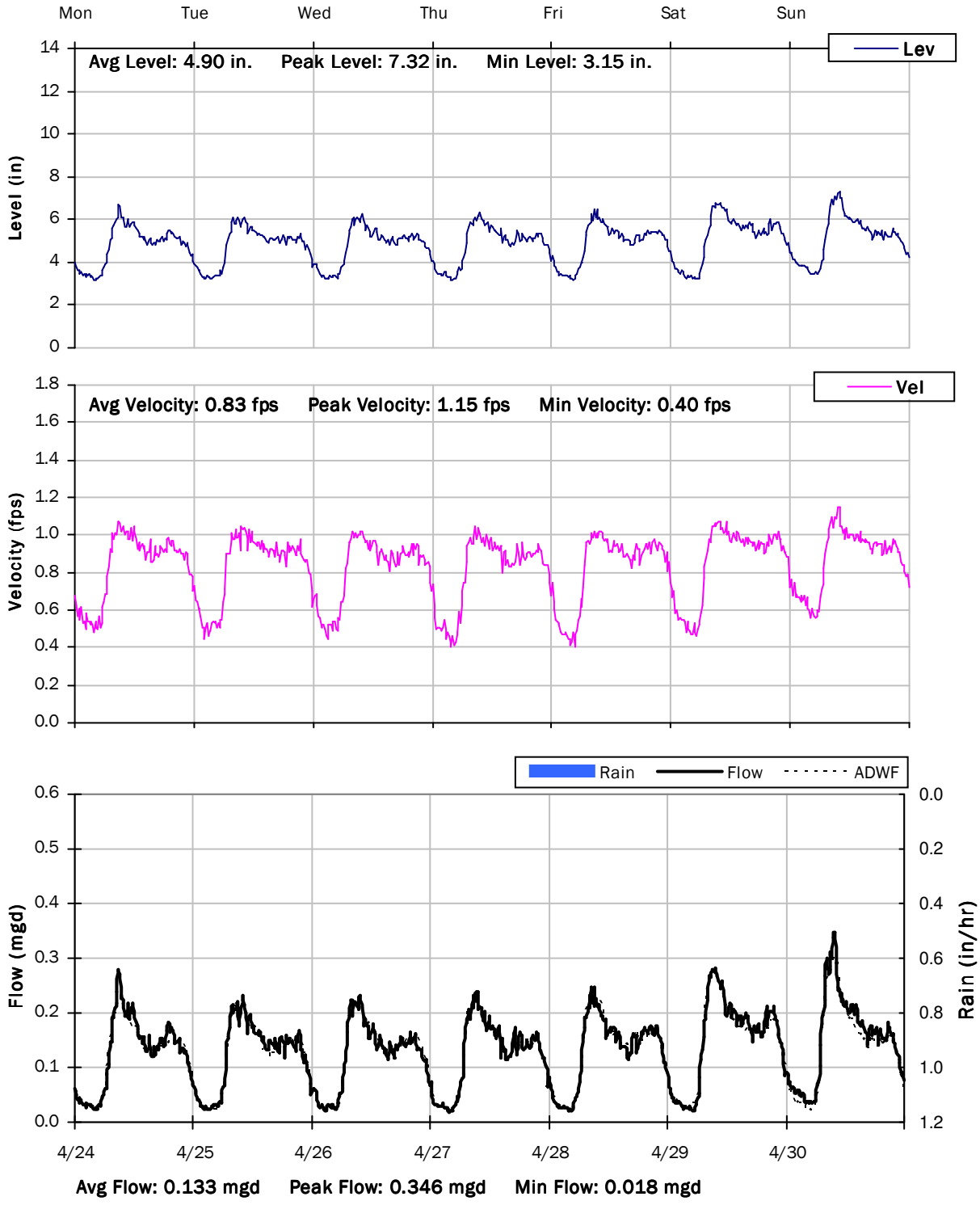
4/17/2017 to 4/24/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

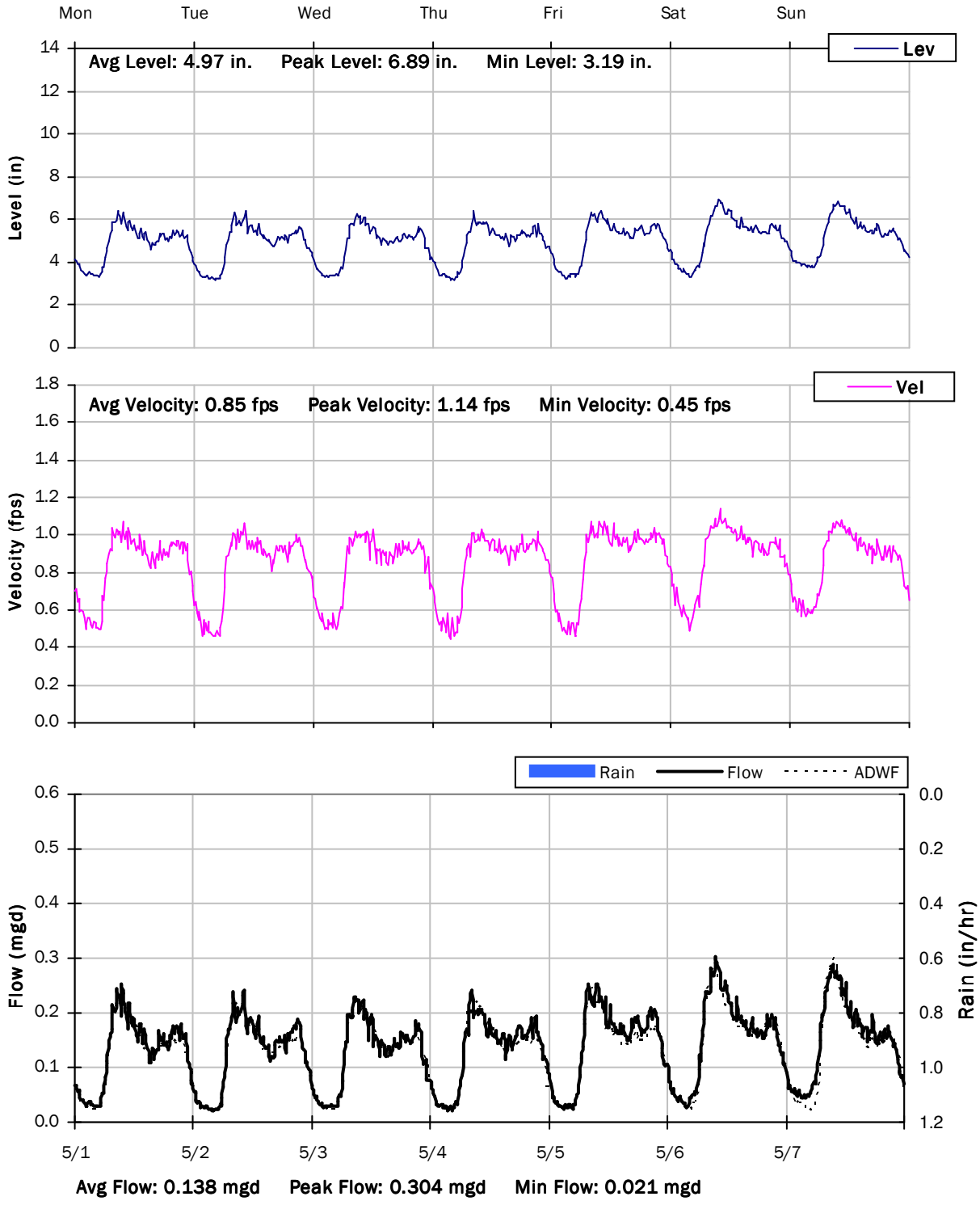
4/24/2017 to 5/1/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

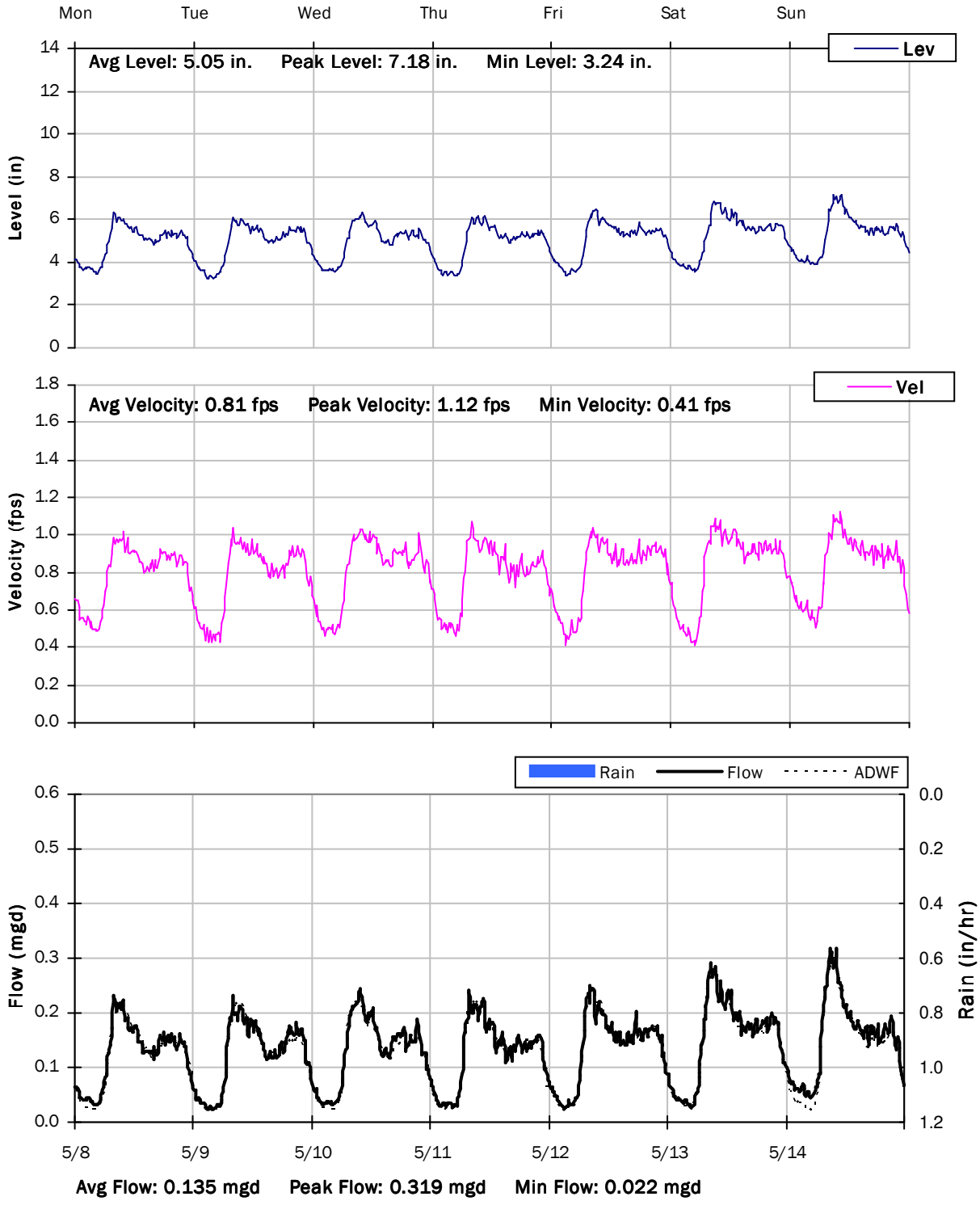
5/1/2017 to 5/8/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

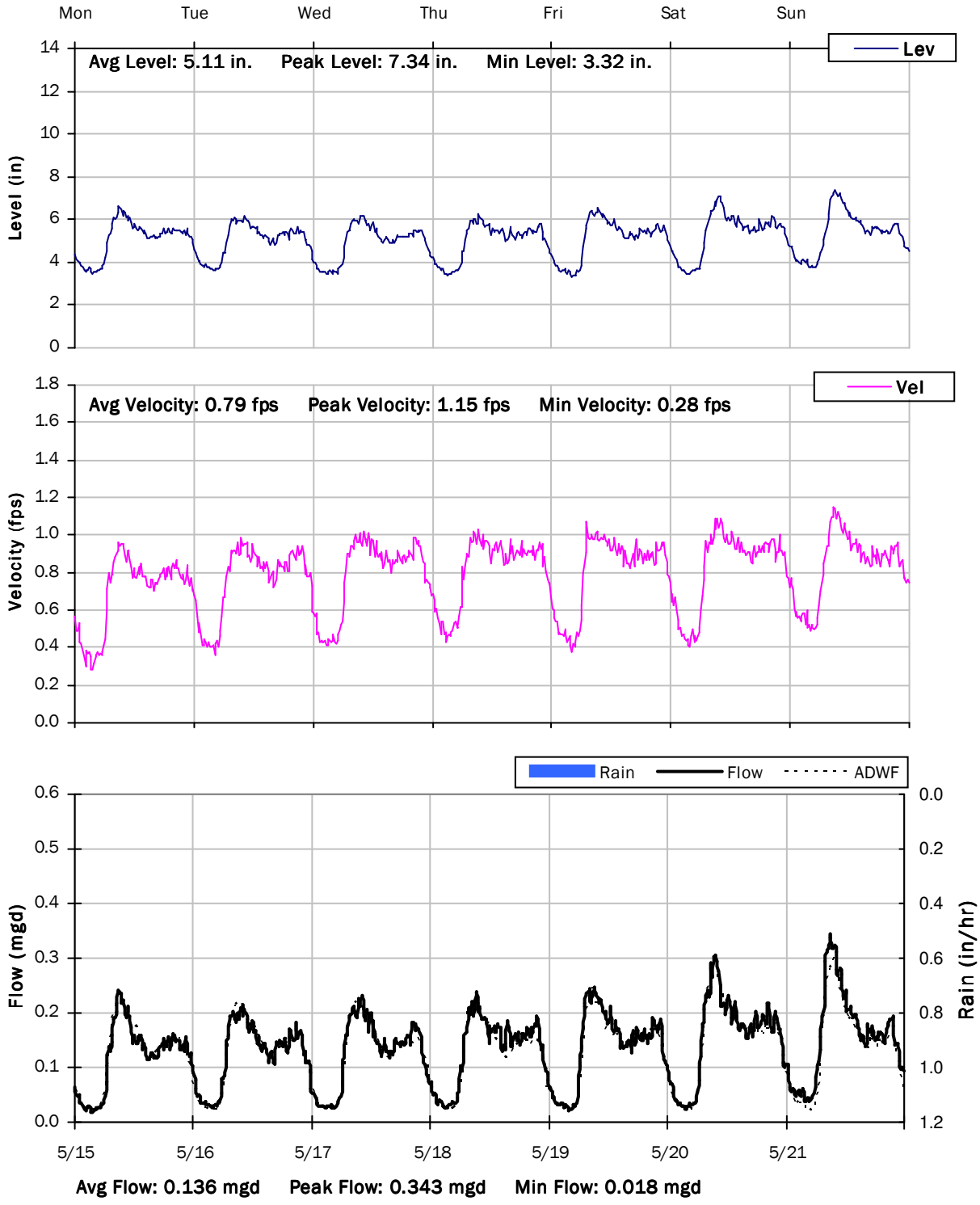
5/8/2017 to 5/15/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

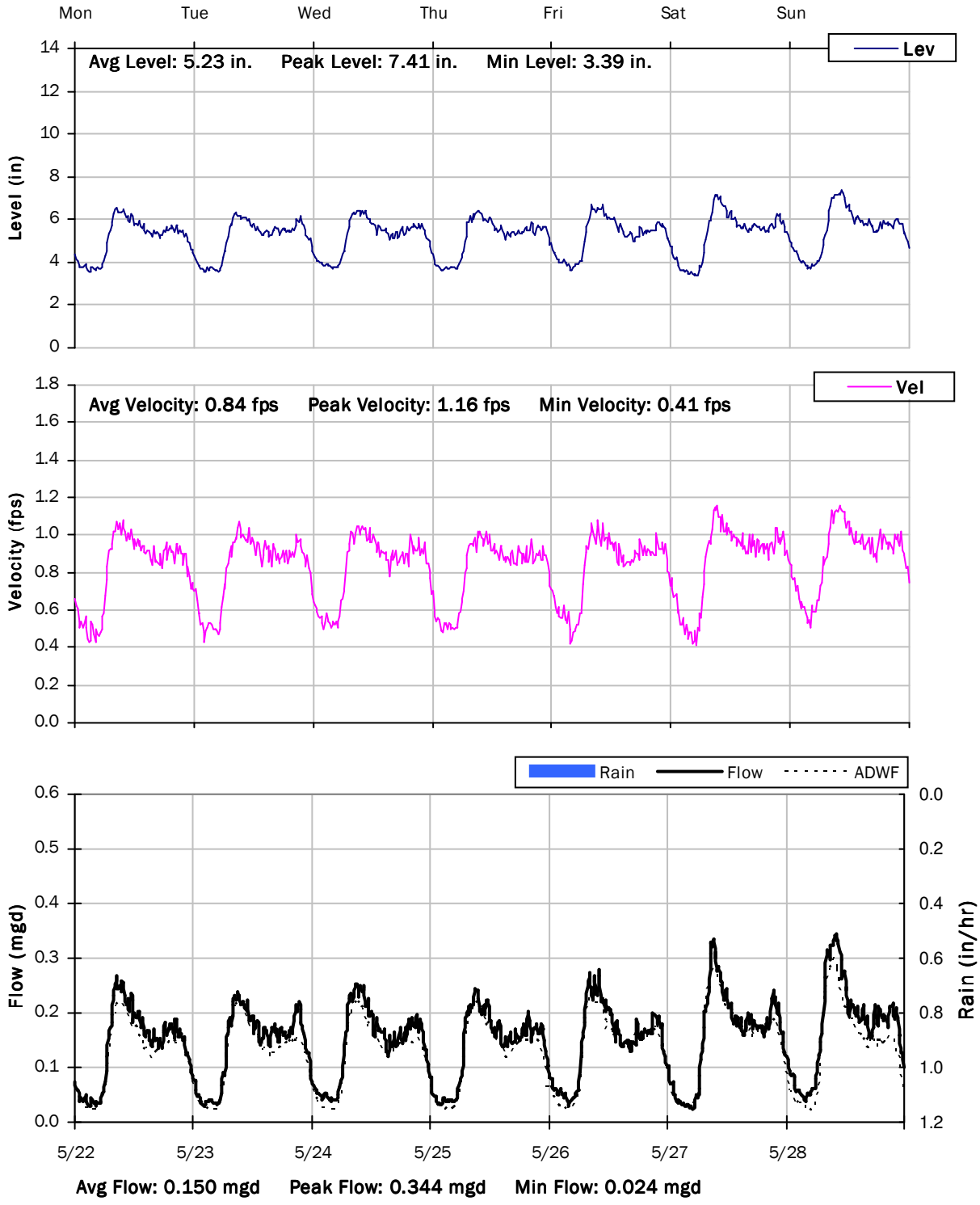
5/15/2017 to 5/22/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

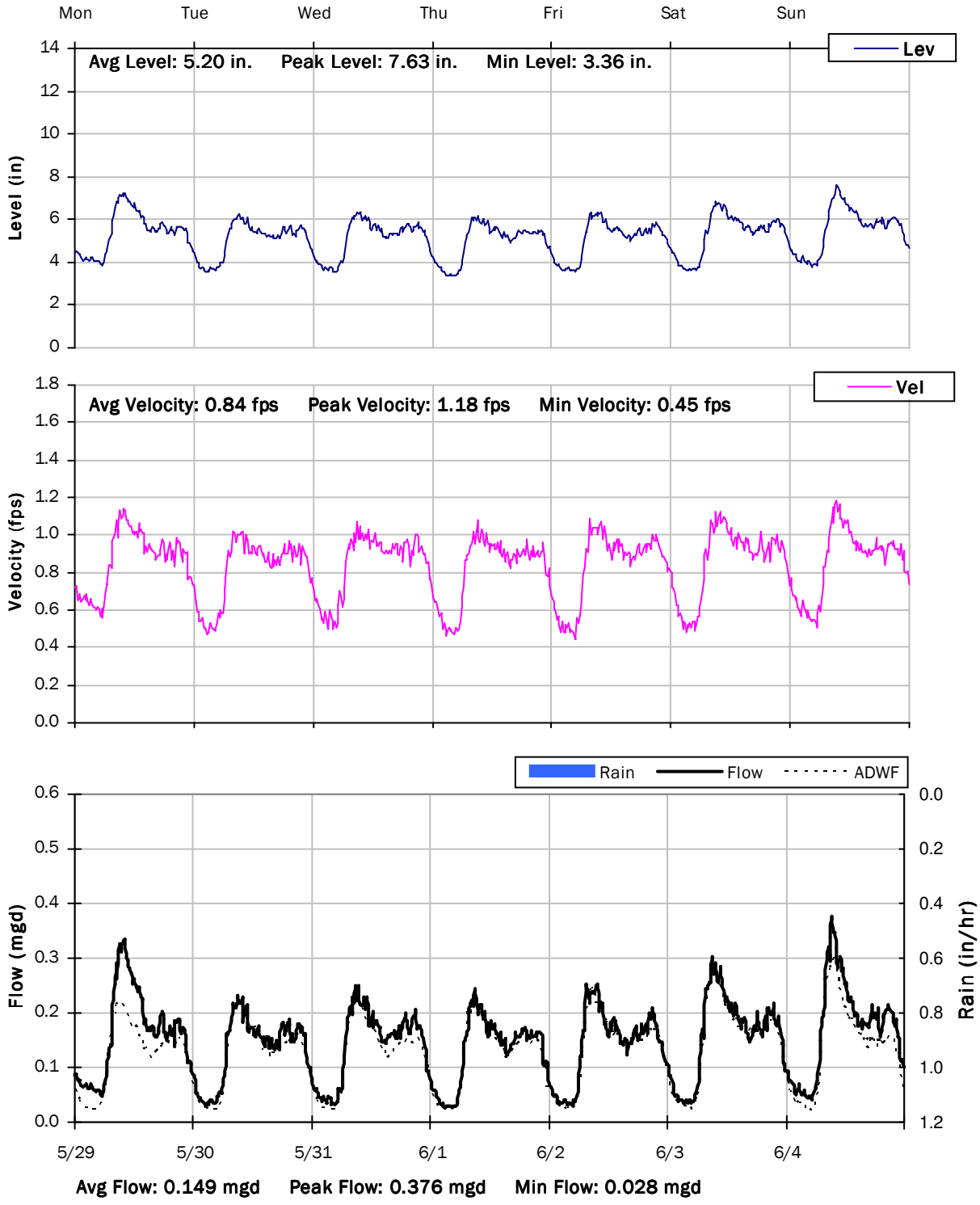
5/22/2017 to 5/29/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

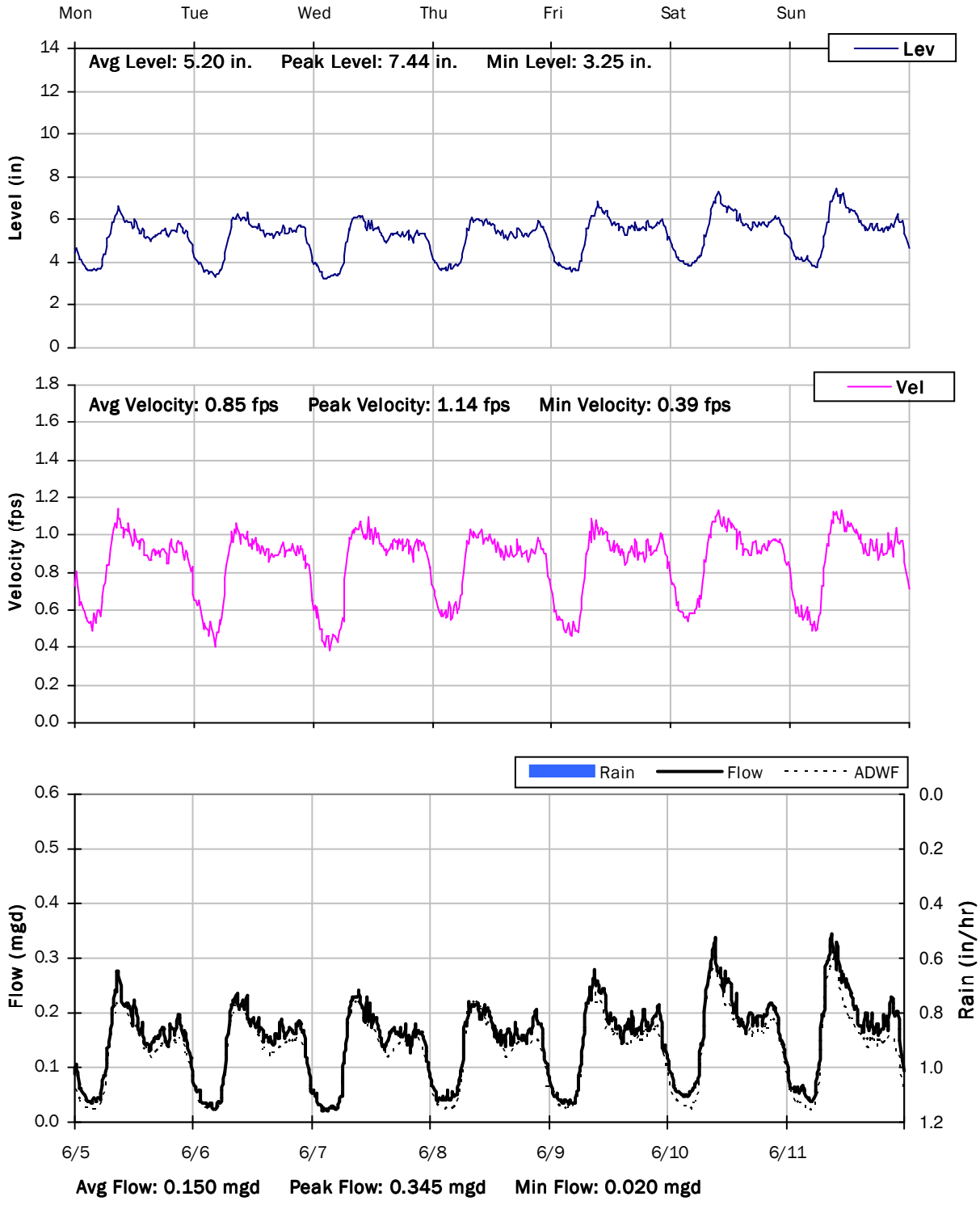
5/29/2017 to 6/5/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

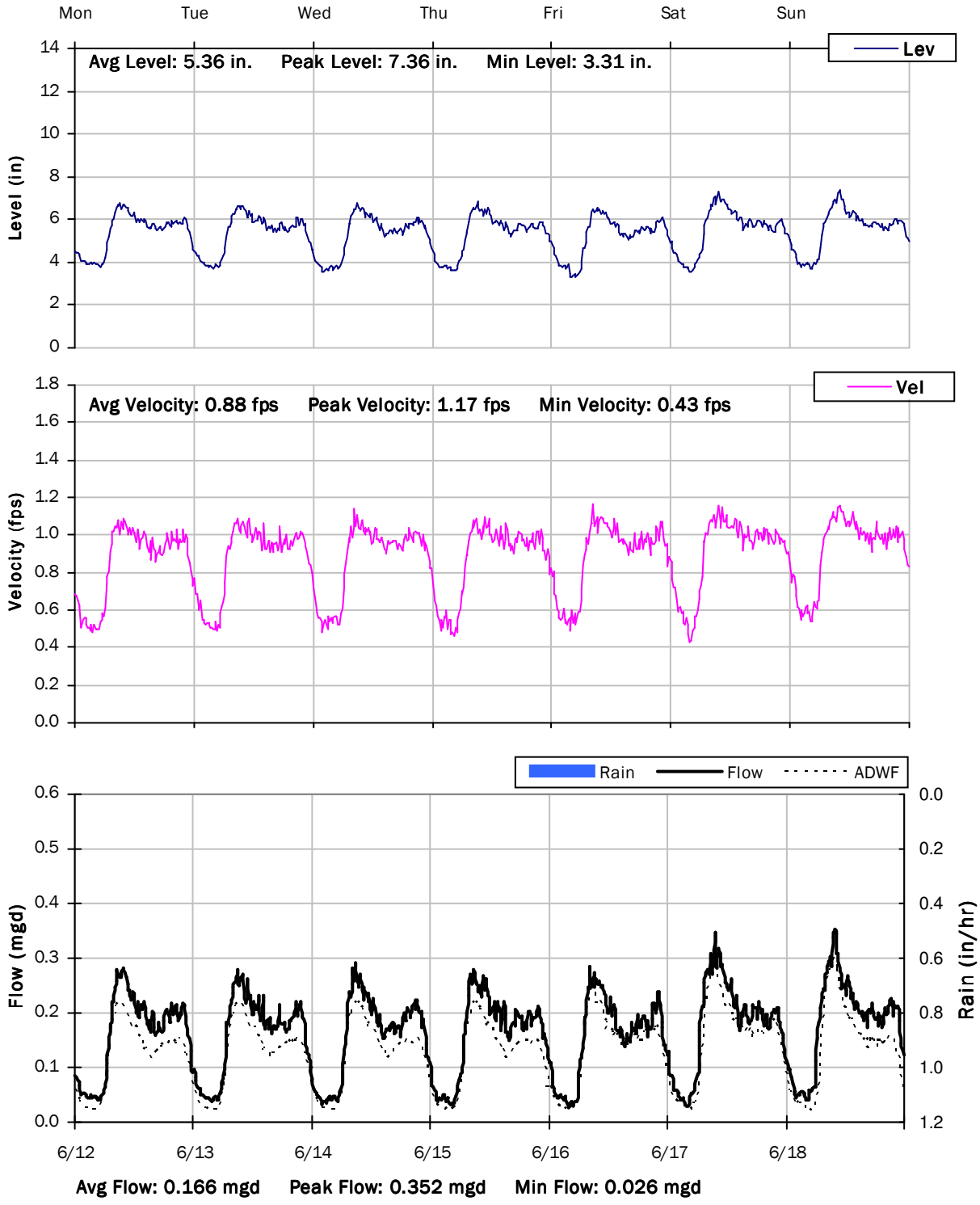
6/5/2017 to 6/12/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

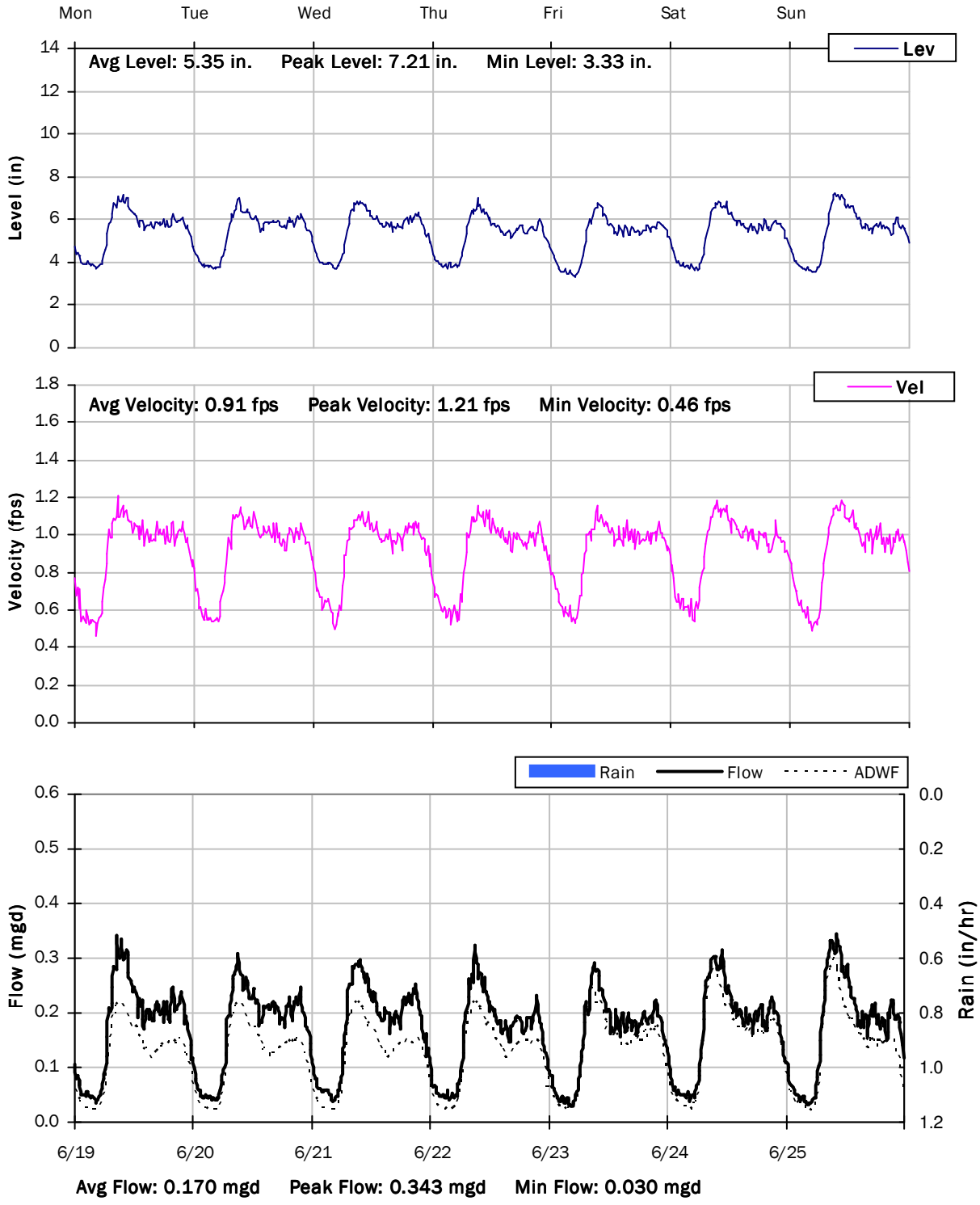
6/12/2017 to 6/19/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

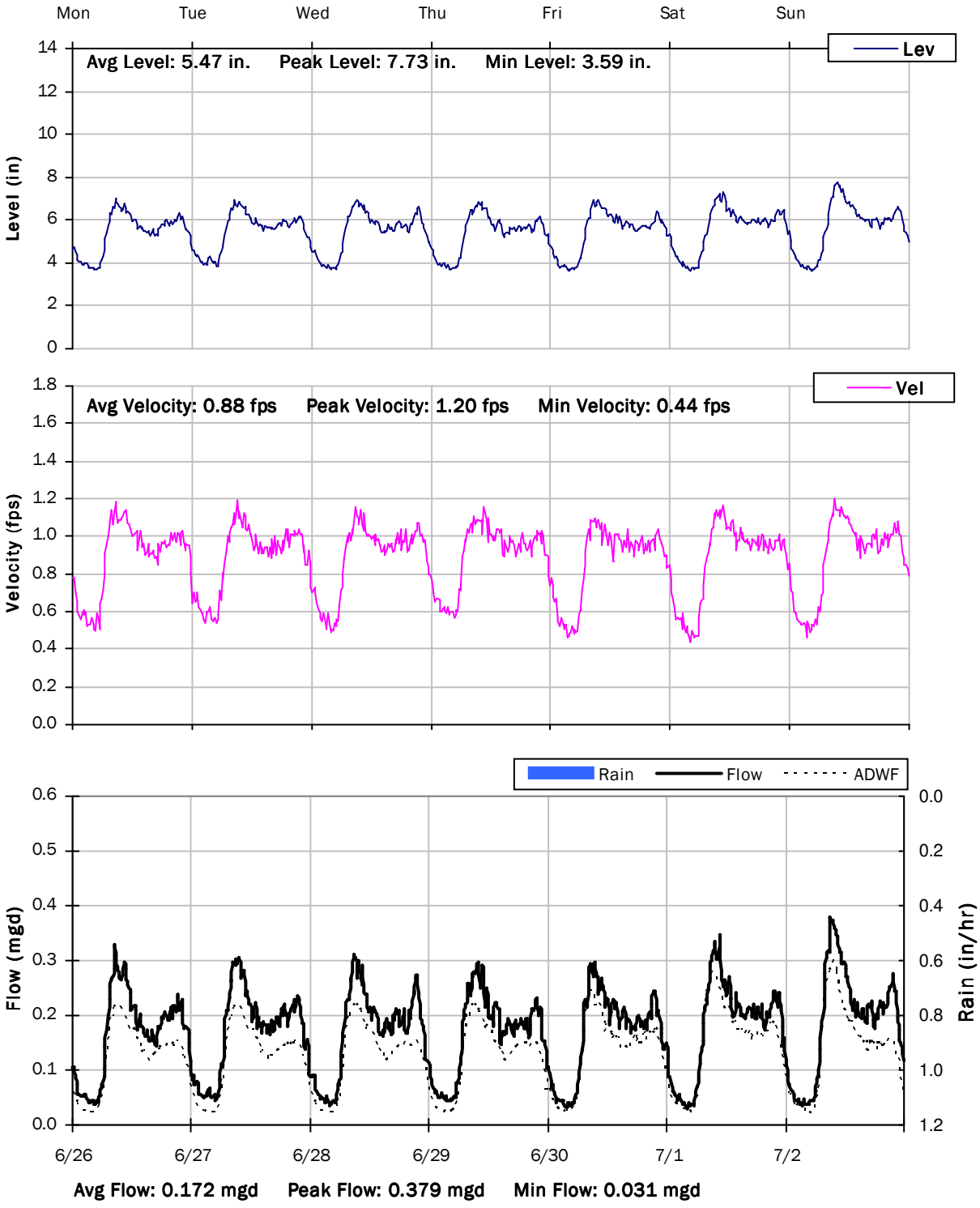
6/19/2017 to 6/26/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

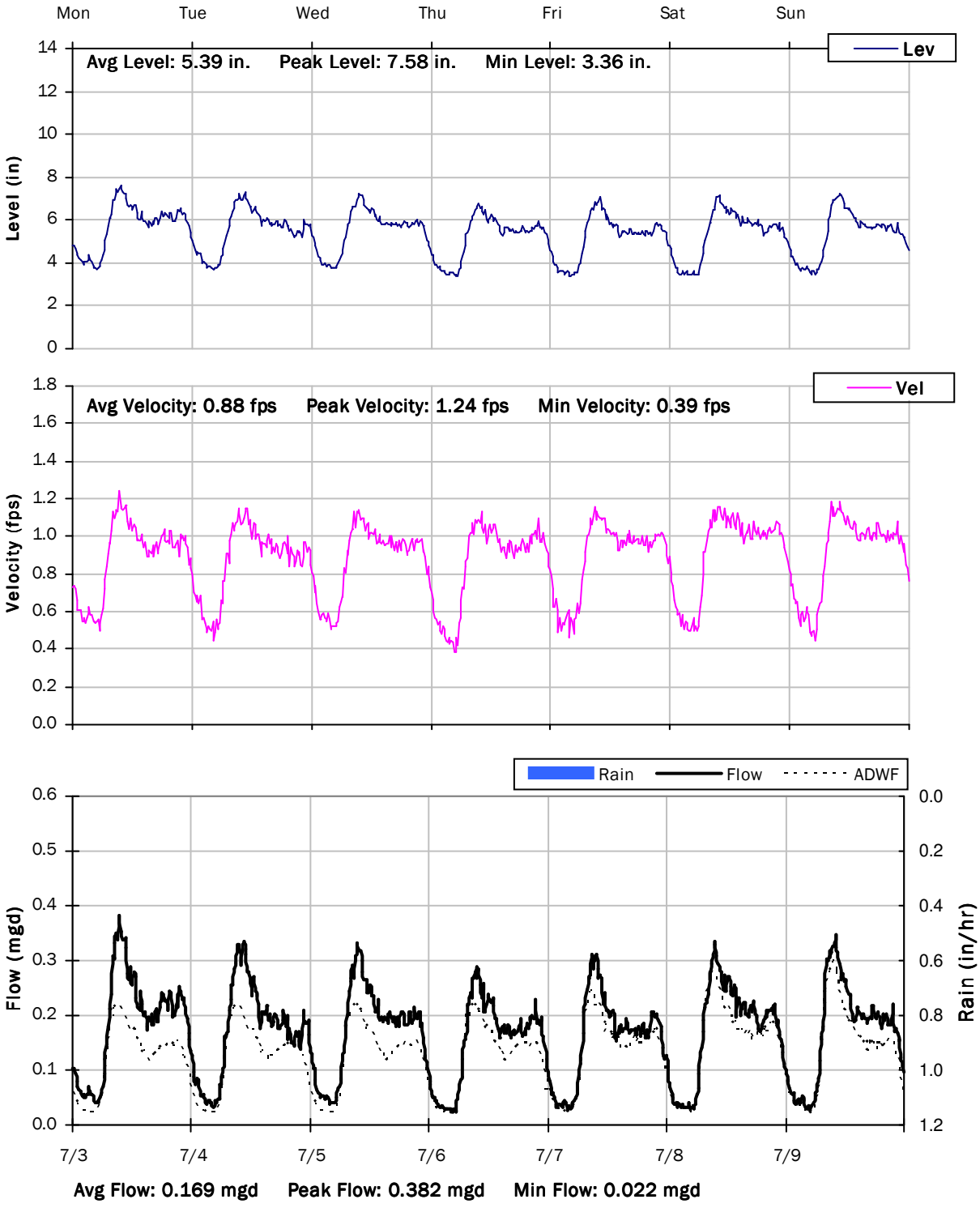
6/26/2017 to 7/3/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

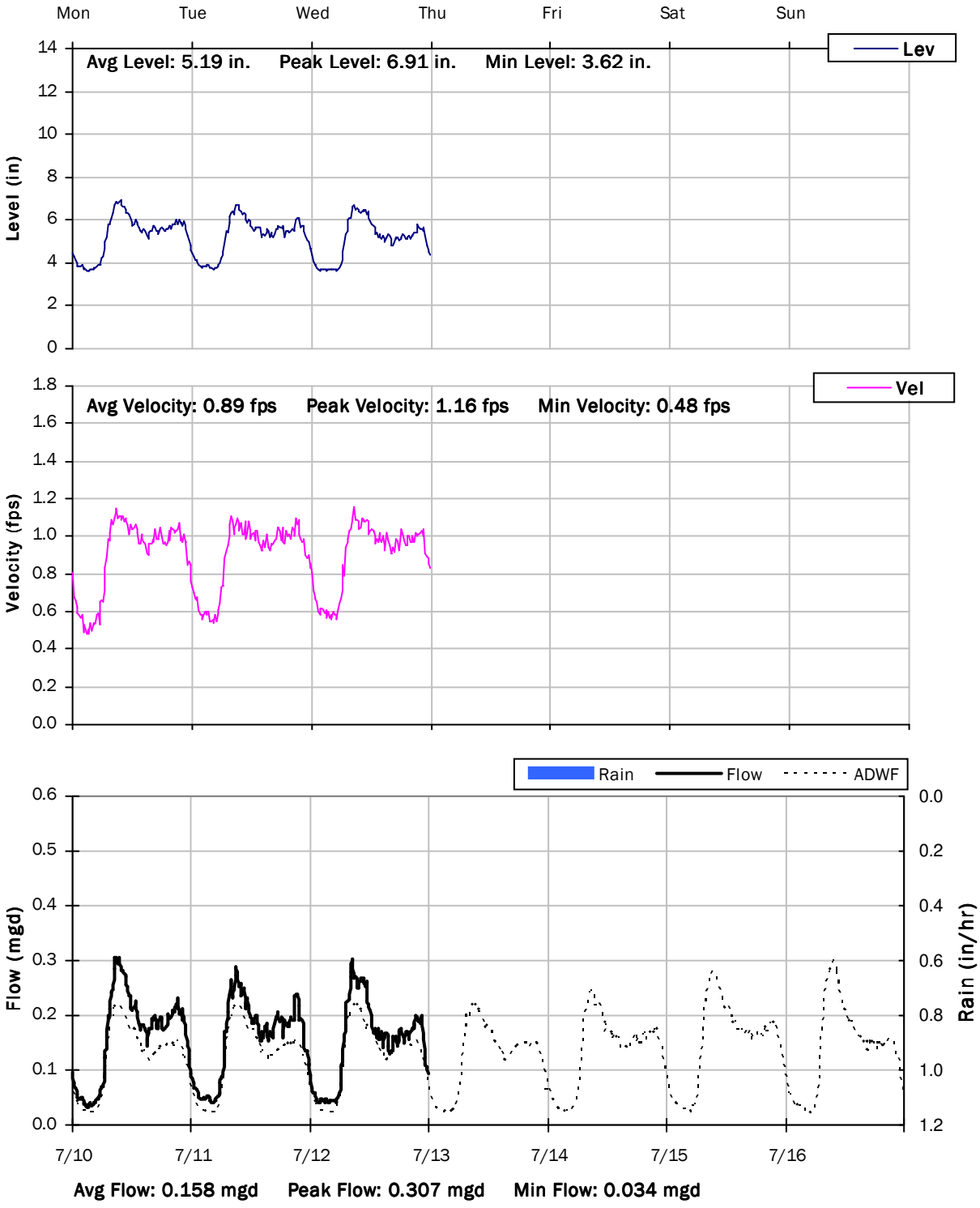
7/3/2017 to 7/10/2017



SITE 12

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

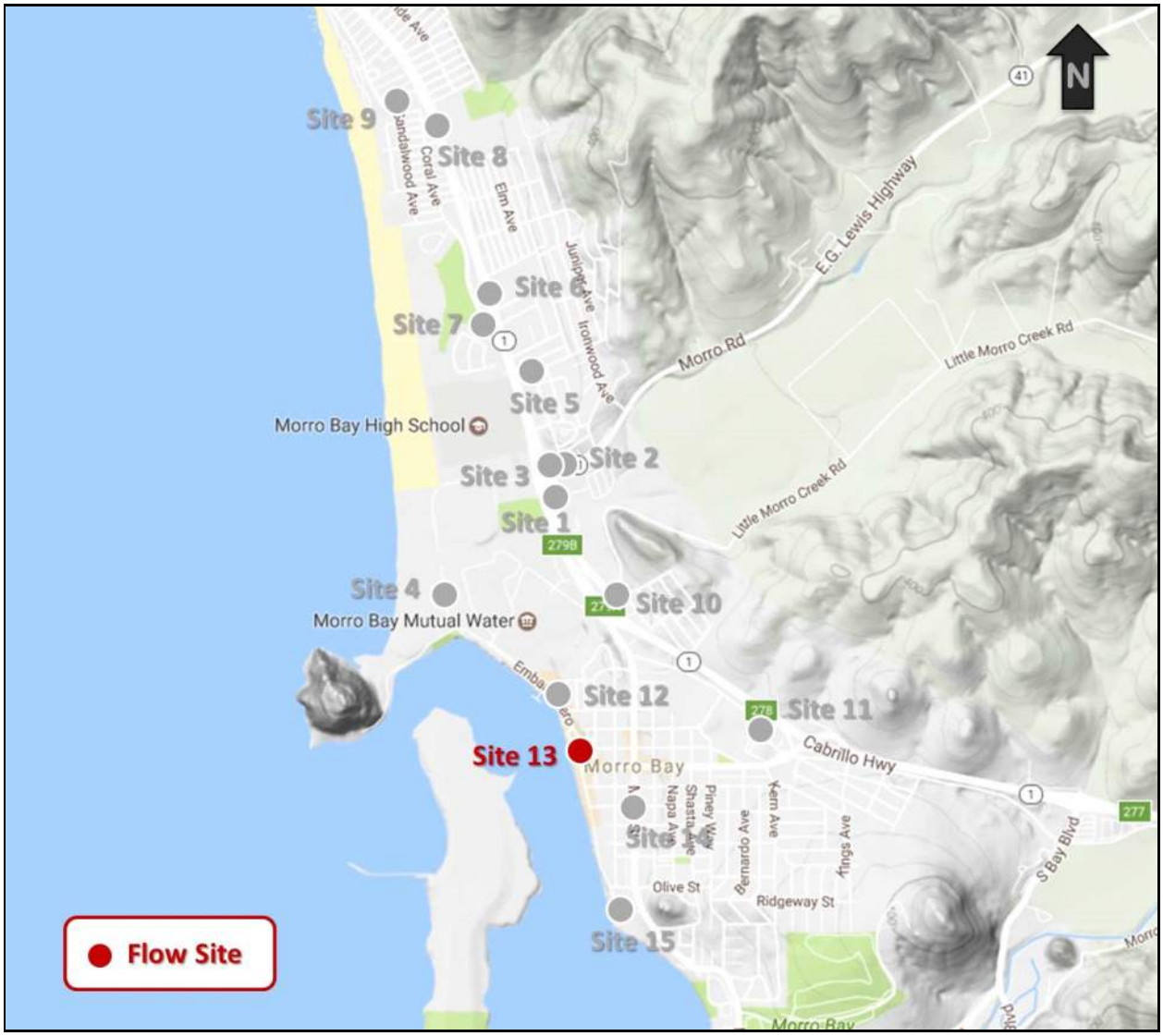
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 13

Location: Sidewalk of 910 Embarcadero, meter on south 15.5" inlet

Data Summary Report



Vicinity Map: Site 13

SITE 13

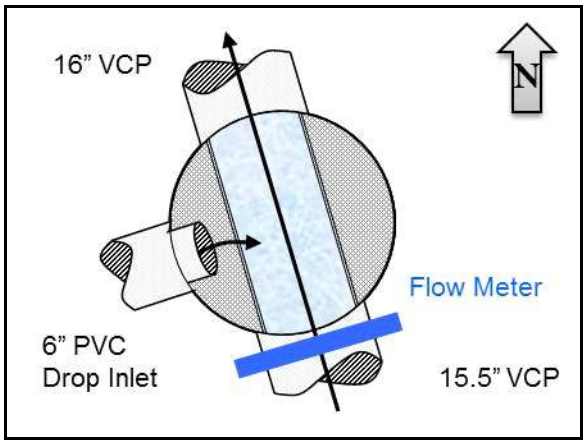
Site Information

Location: Sidewalk of 910 Embarcadero, meter on south 15.5" inlet

Pipe Diameter: 15.5 inches

ADWF: 0.093 mgd

Peak Measured Flow: 0.306 mgd



SITE 13

Additional Site Photos

SITE 13

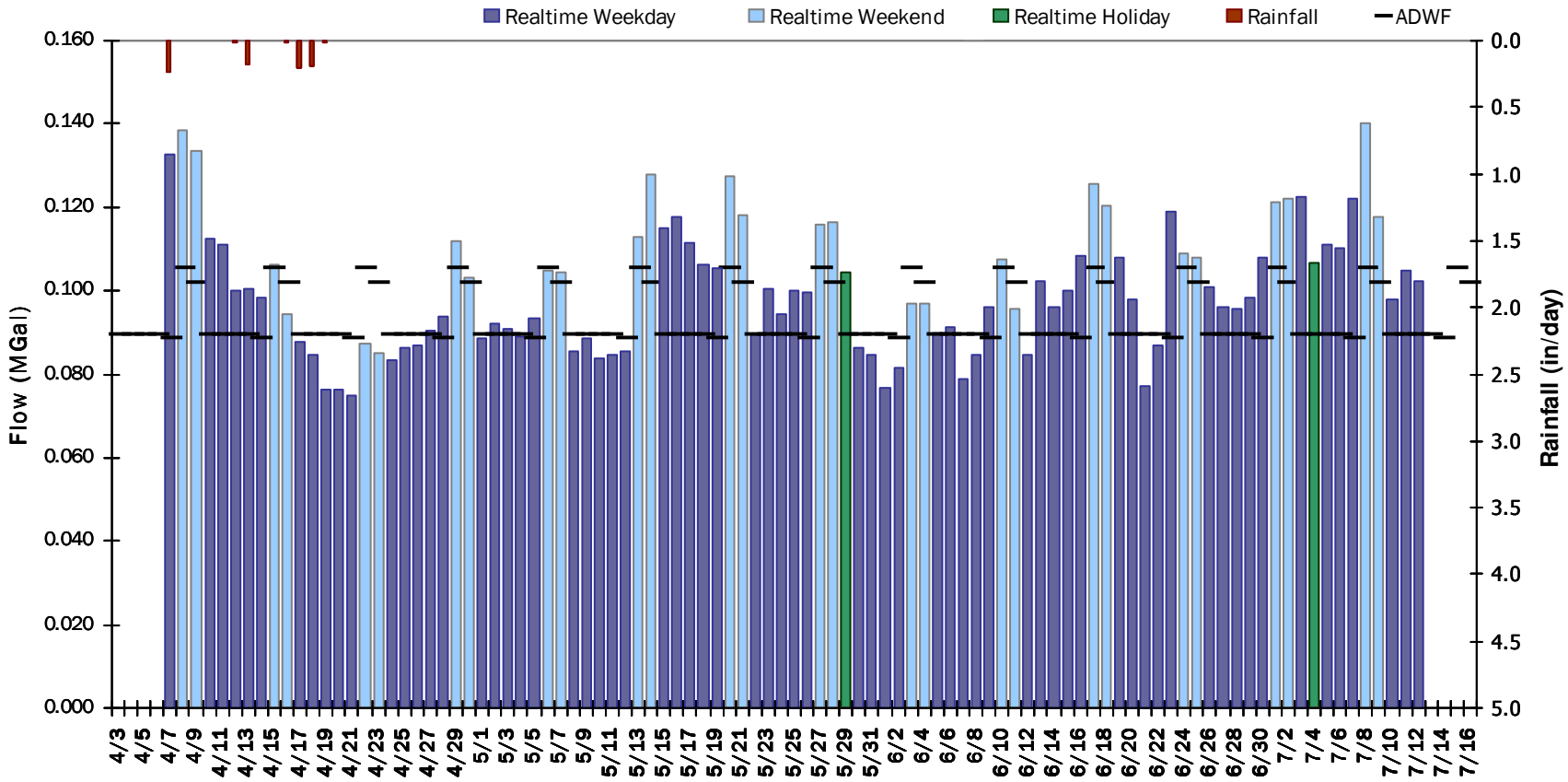
Additional Site Photos

SITE 13

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.101 MGal Peak Daily Flow: 0.140 MGal Min Daily Flow: 0.075 MGal

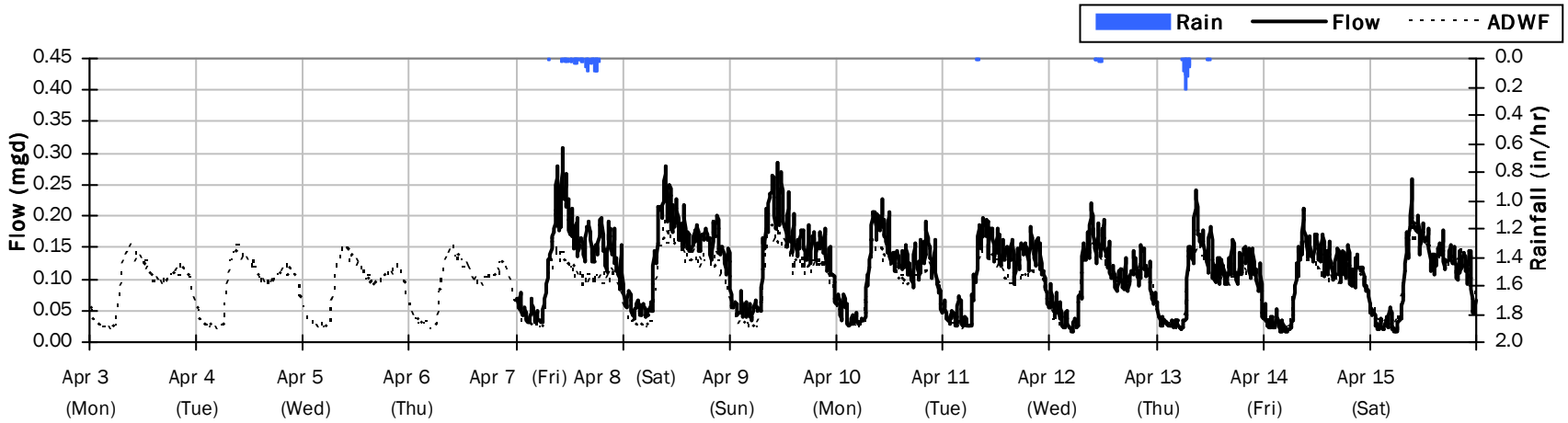
Total Period Rainfall: 0.85 inches

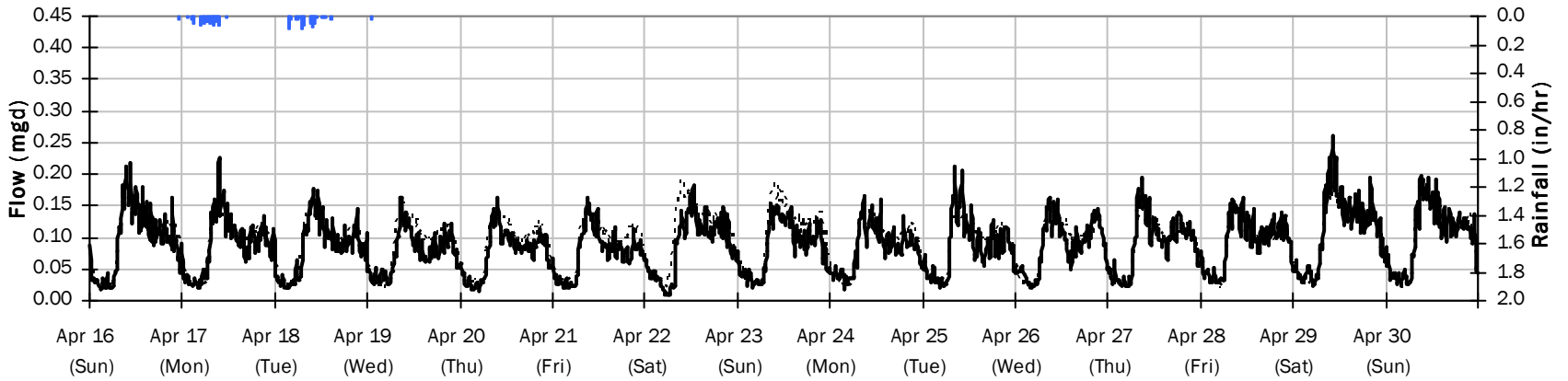


SITE 13

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.098 mgd** **Peak Flow: 0.306 mgd** **Min Flow: 0.008 mgd**

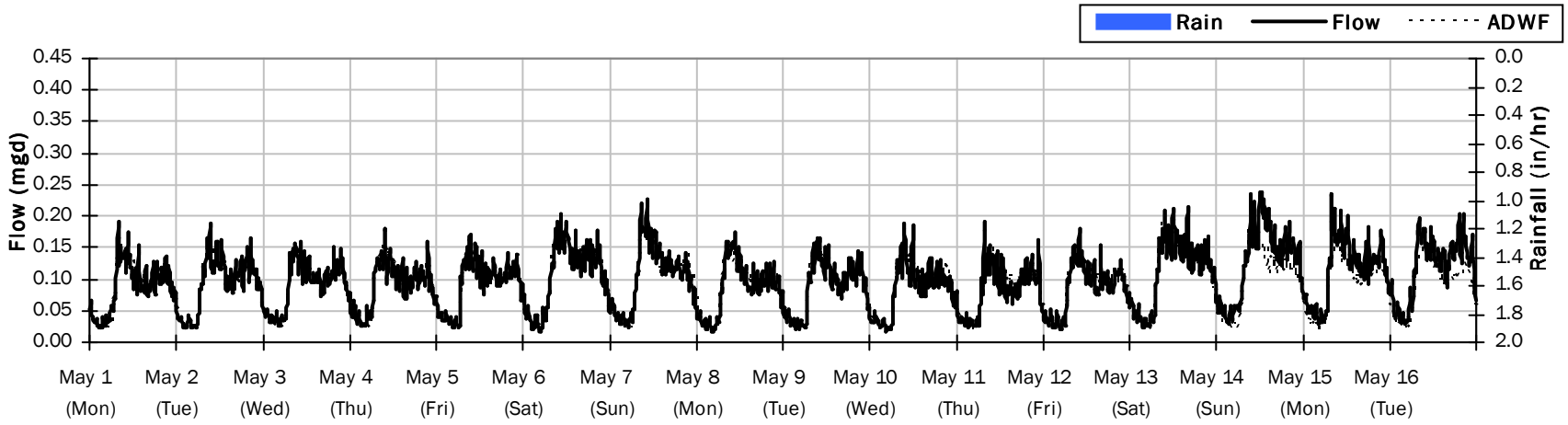


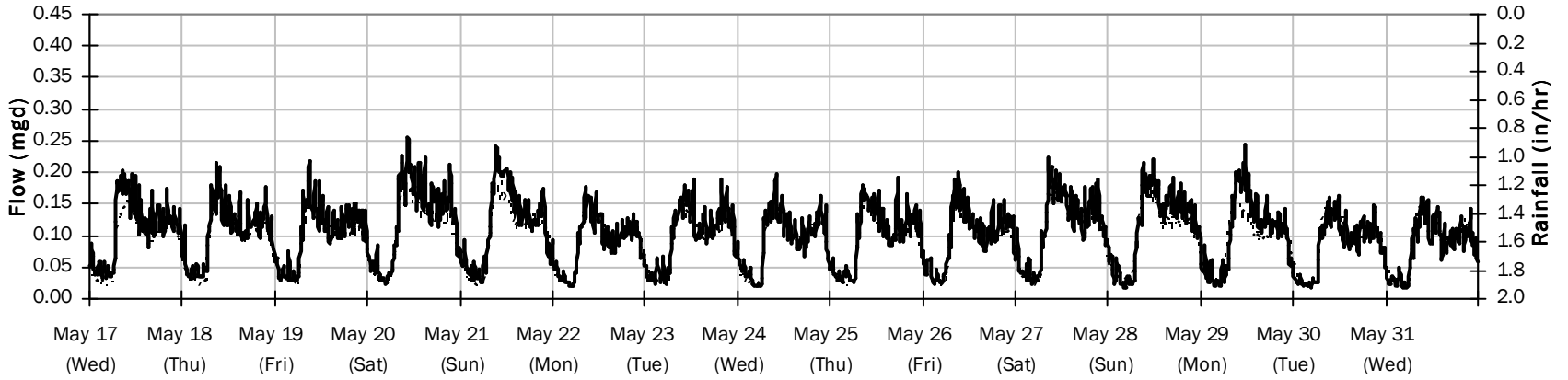


SITE 13

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.101 mgd** **Peak Flow: 0.255 mgd** **Min Flow: 0.016 mgd**

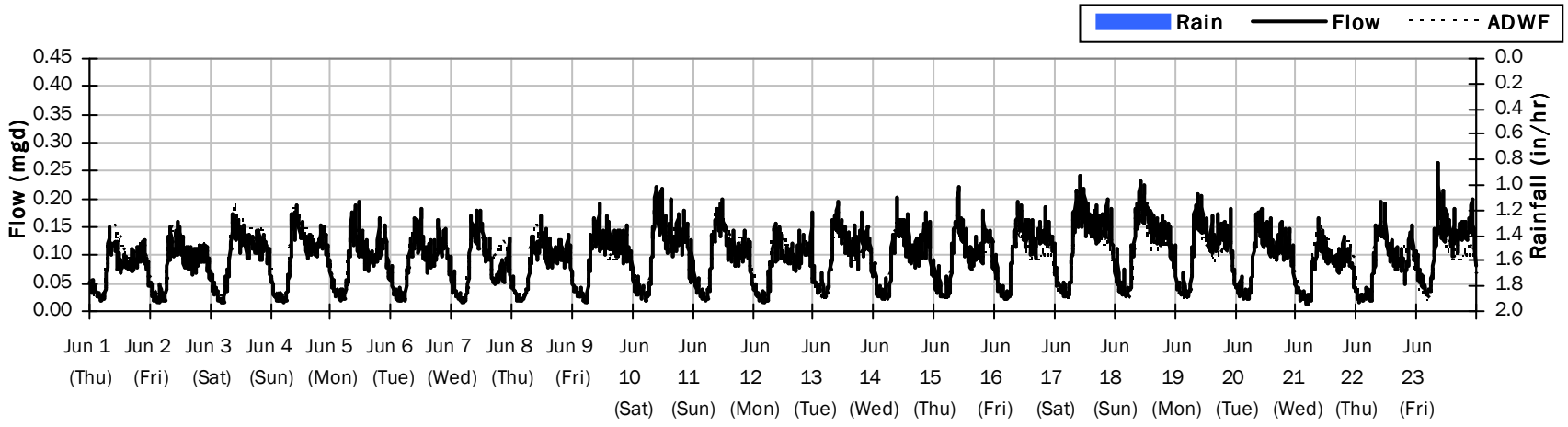


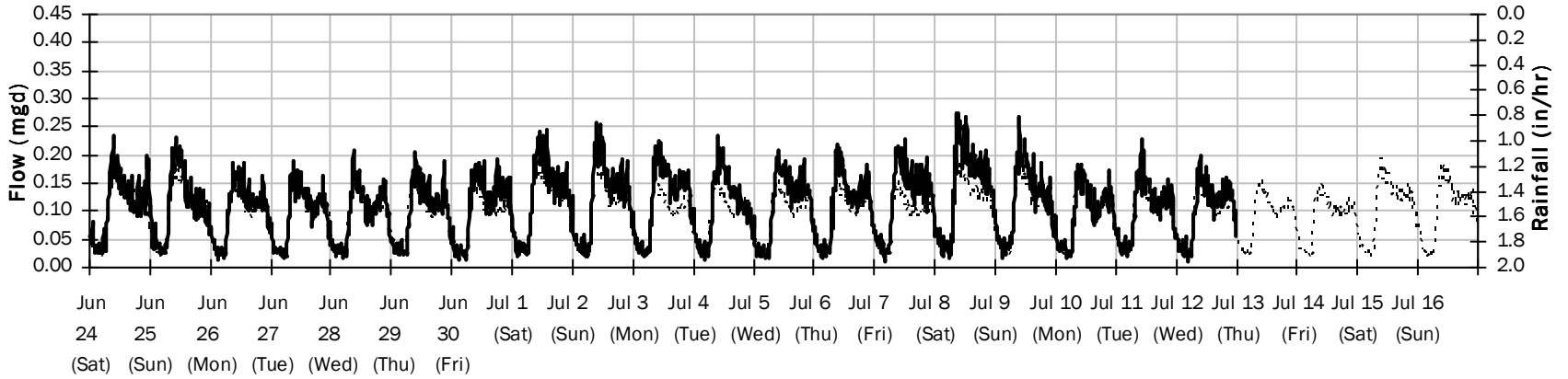


SITE 13

Flow Summary: 6/1/2017 to 7/16/2017

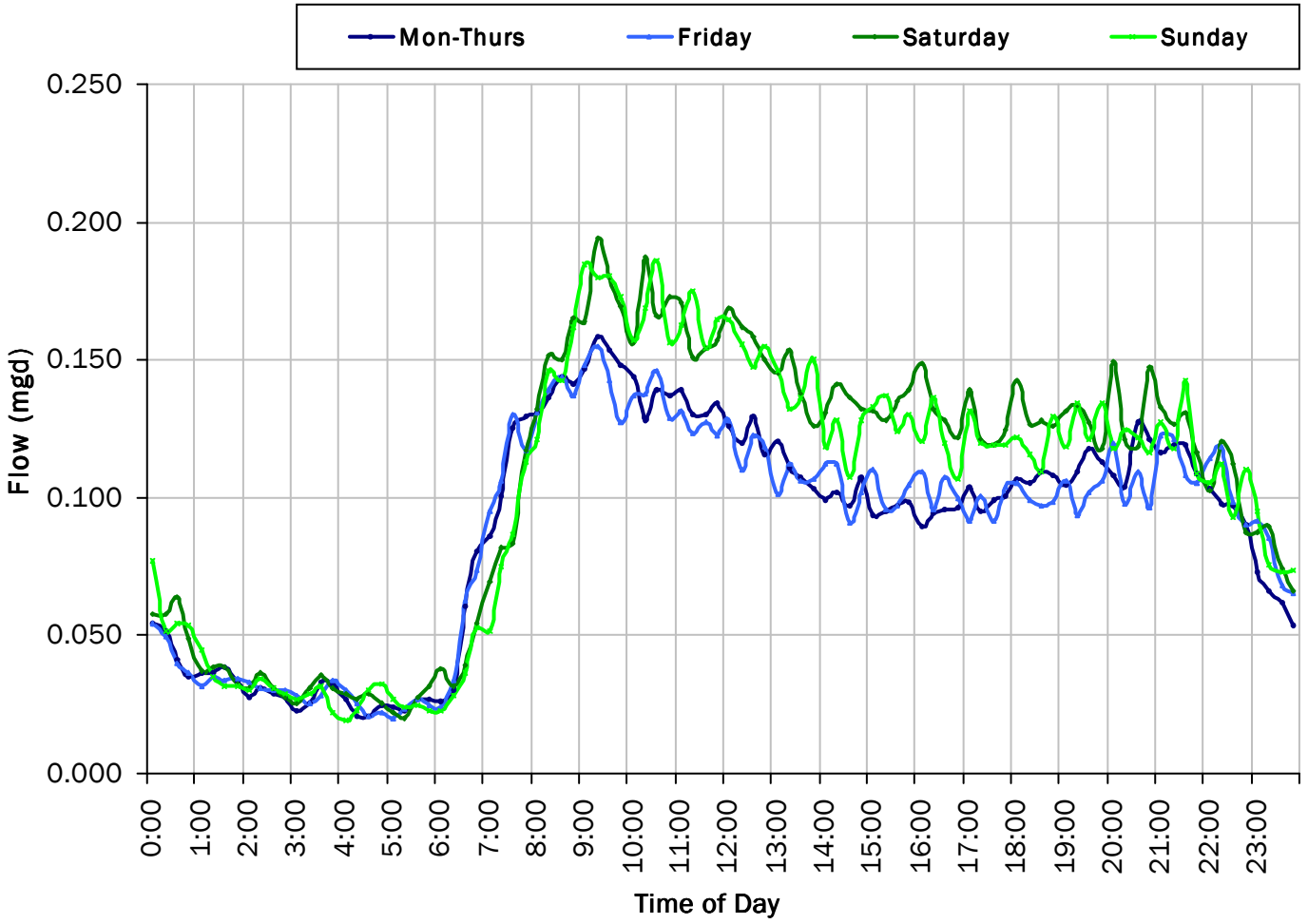
Total Period Rainfall: 0.00 inches **Avg Flow: 0.103 mgd** **Peak Flow: 0.274 mgd** **Min Flow: 0.009 mgd**





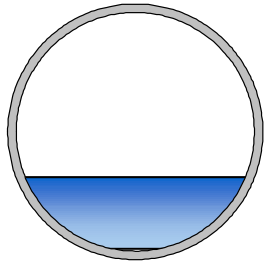
SITE 13

Average Dry Weather Flow Hydrographs



ADWF:

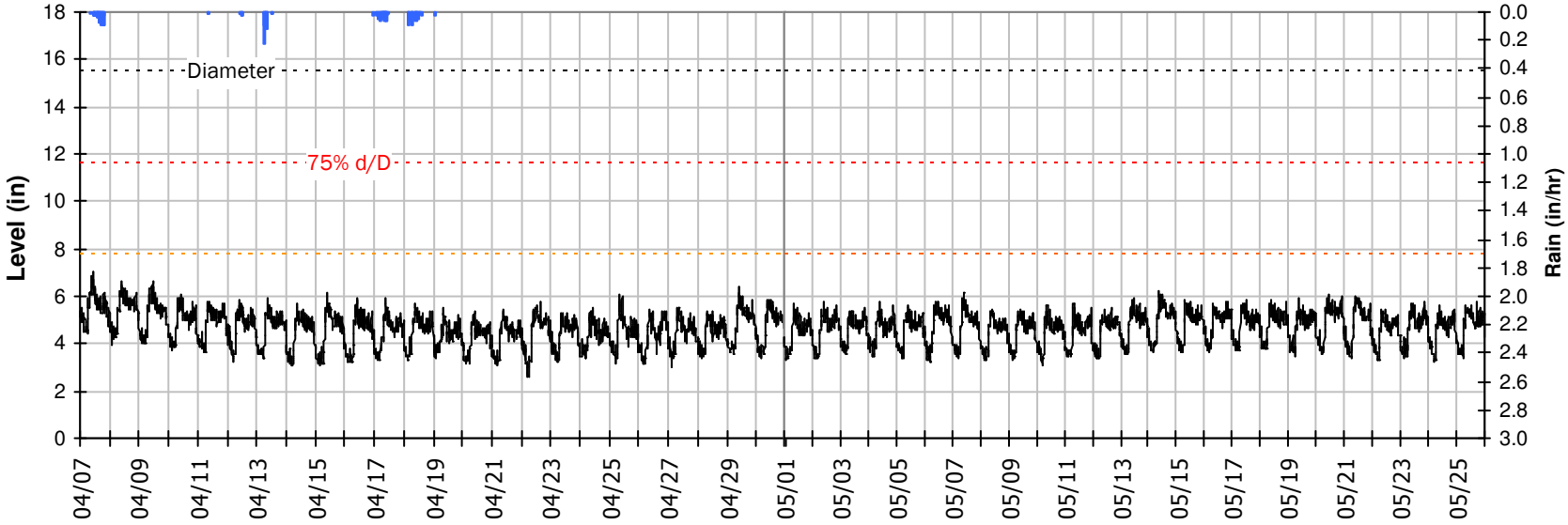
0.093 *mgd*

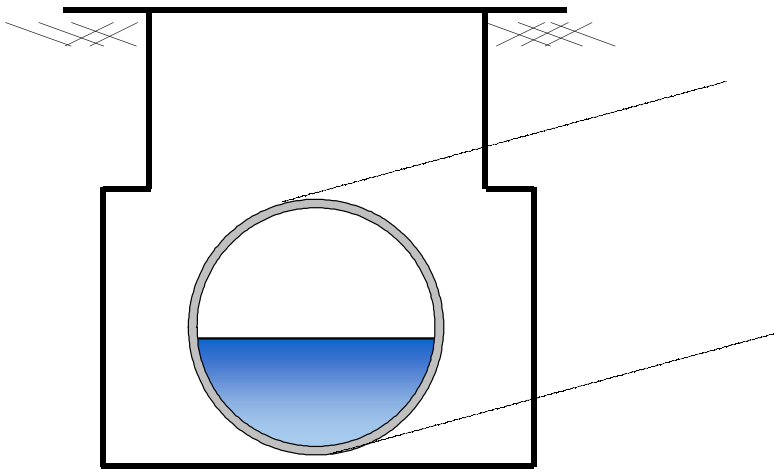


SITE 13

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



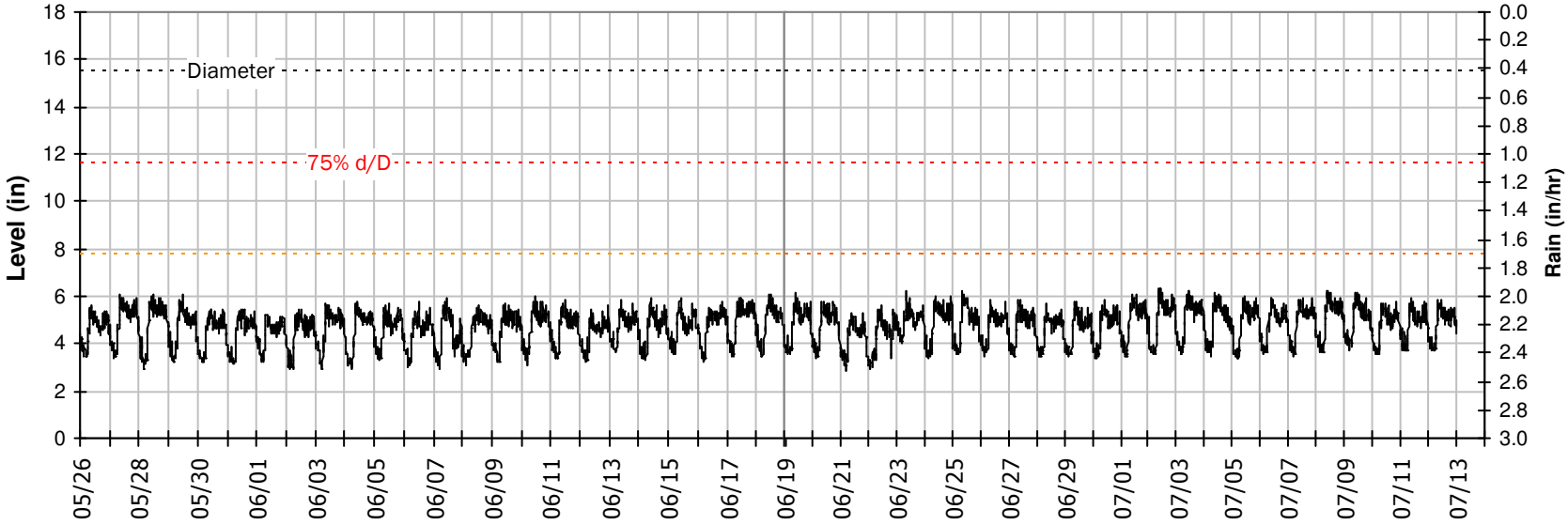


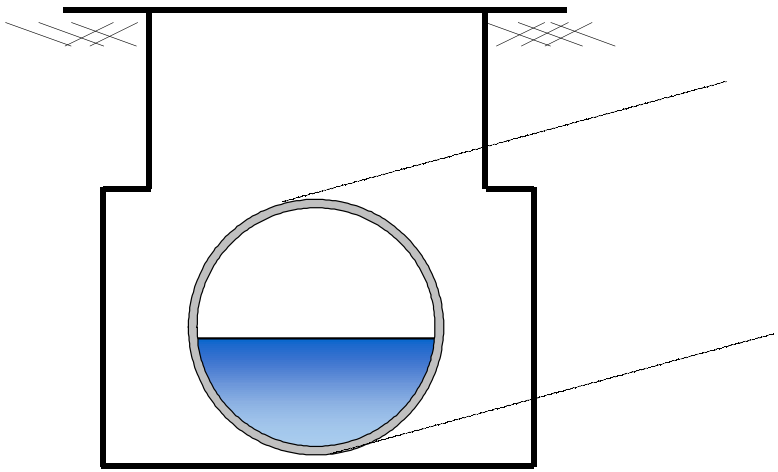
Pipe Diameter:	15.5	*inches*
Peak Measured Level:	7	*inches*
Peak d/D Ratio:	0.45	

SITE 13

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



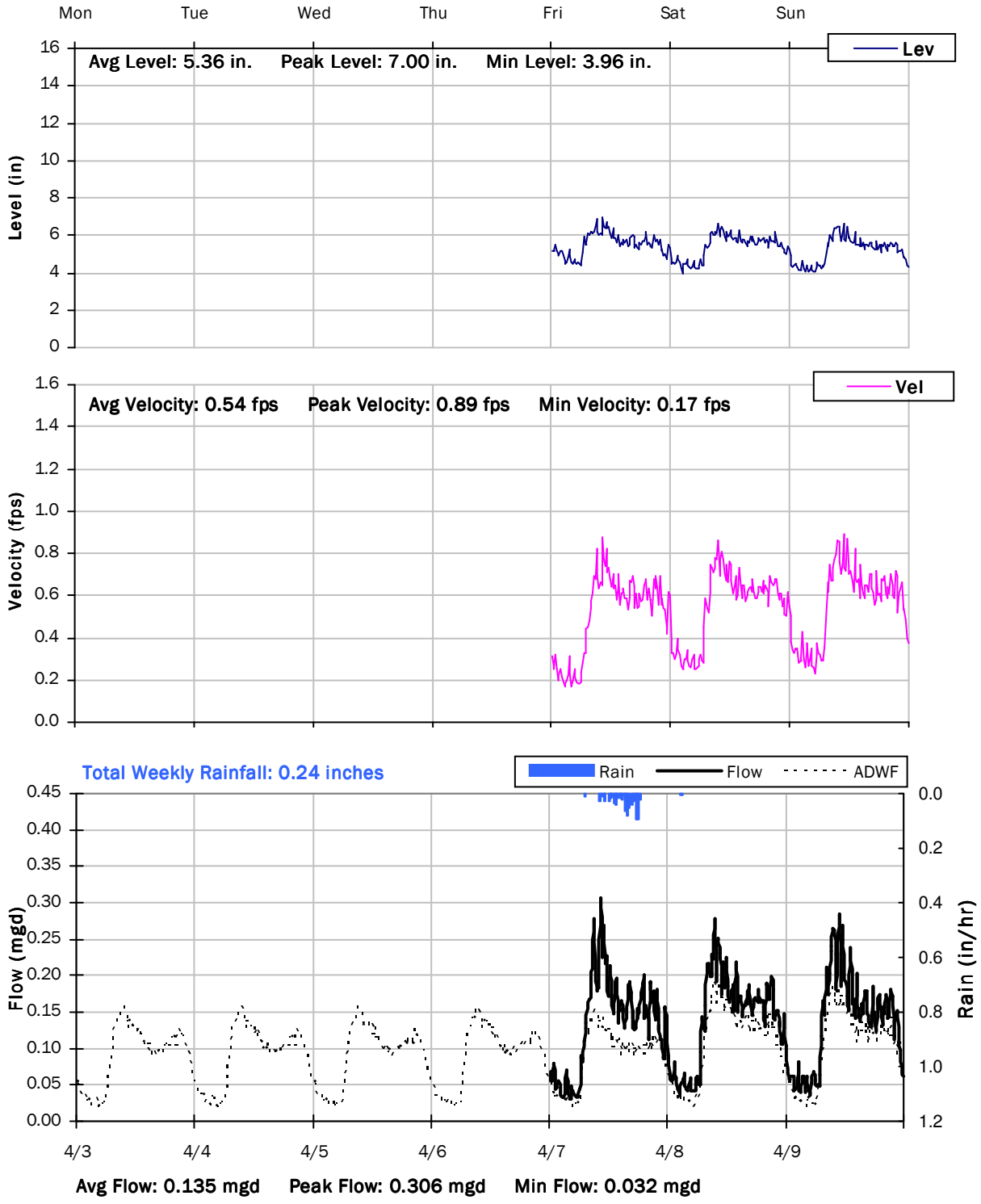


Pipe Diameter:	15.5	*inches*
Peak Measured Level:	7	*inches*
Peak d/D Ratio:	0.45	

SITE 13

Weekly Level, Velocity and Flow Hydrographs

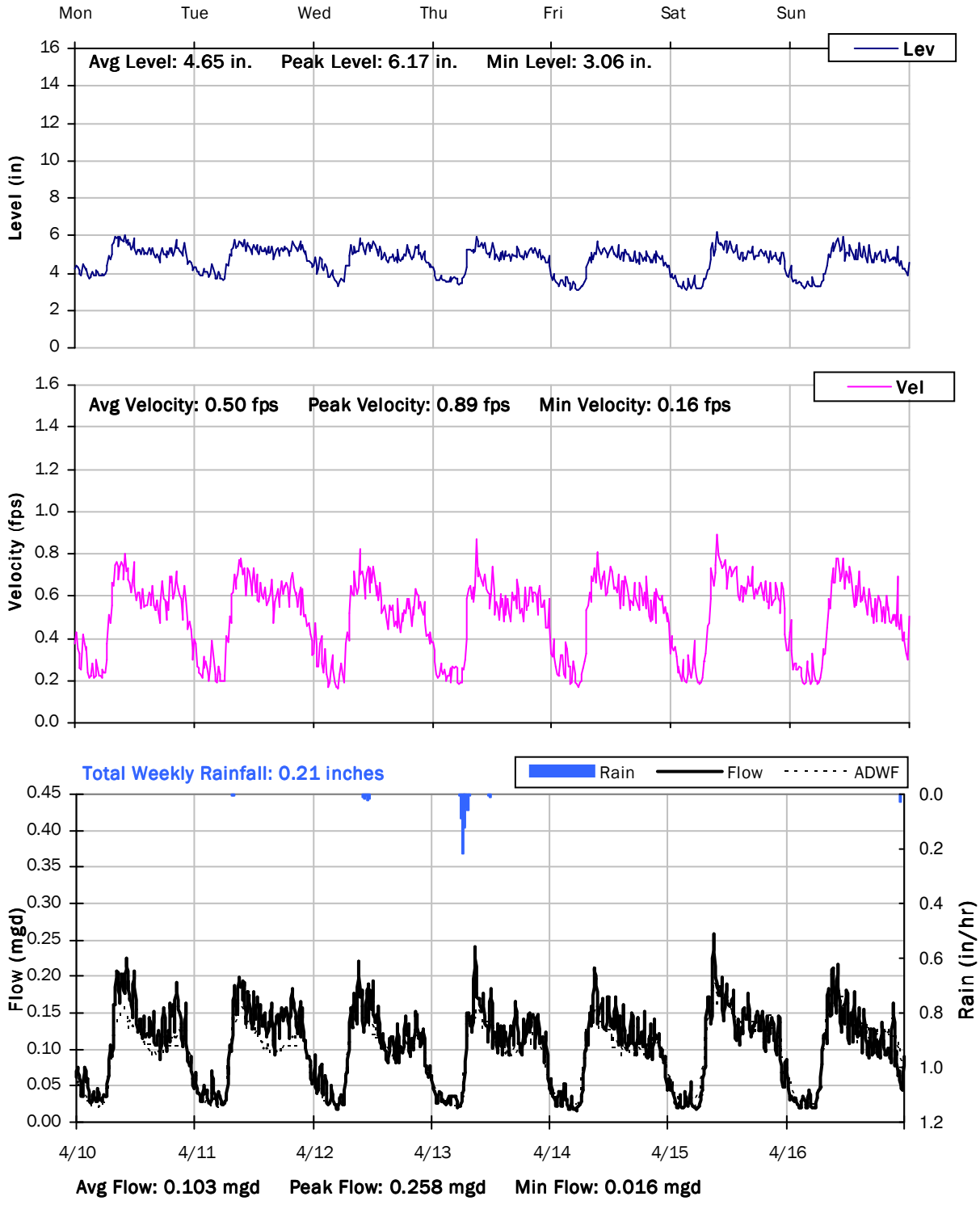
4/3/2017 to 4/10/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

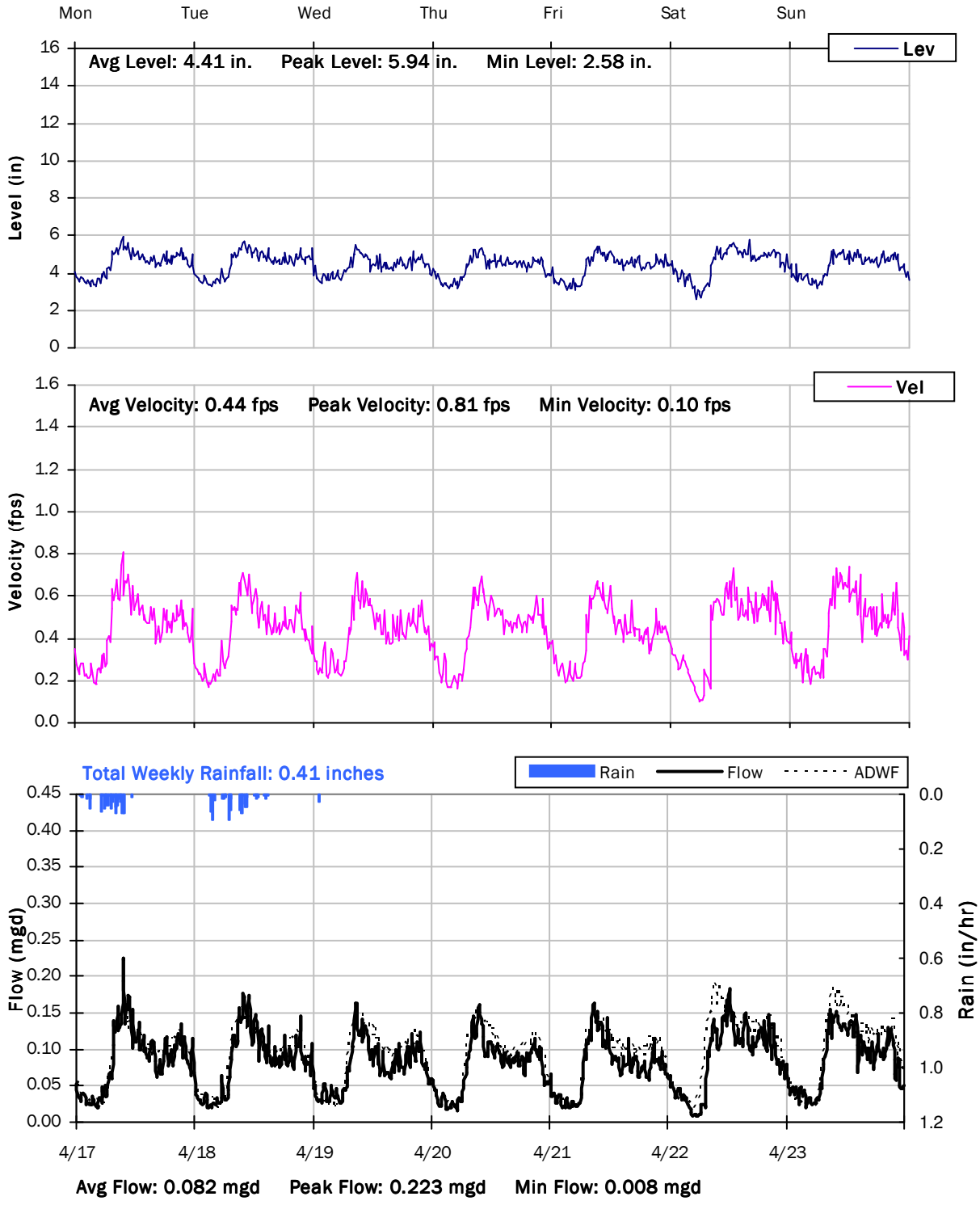
4/10/2017 to 4/17/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

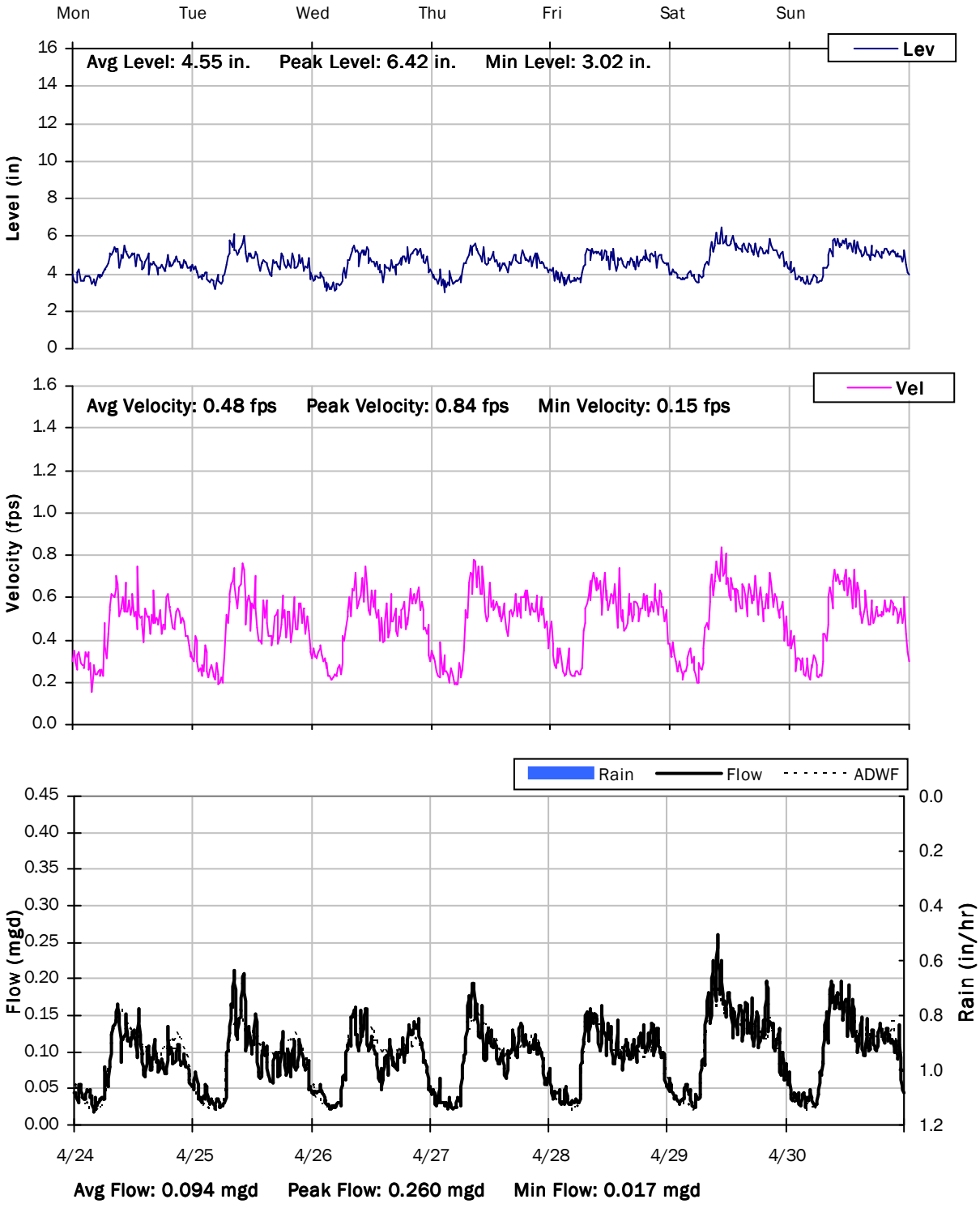
4/17/2017 to 4/24/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

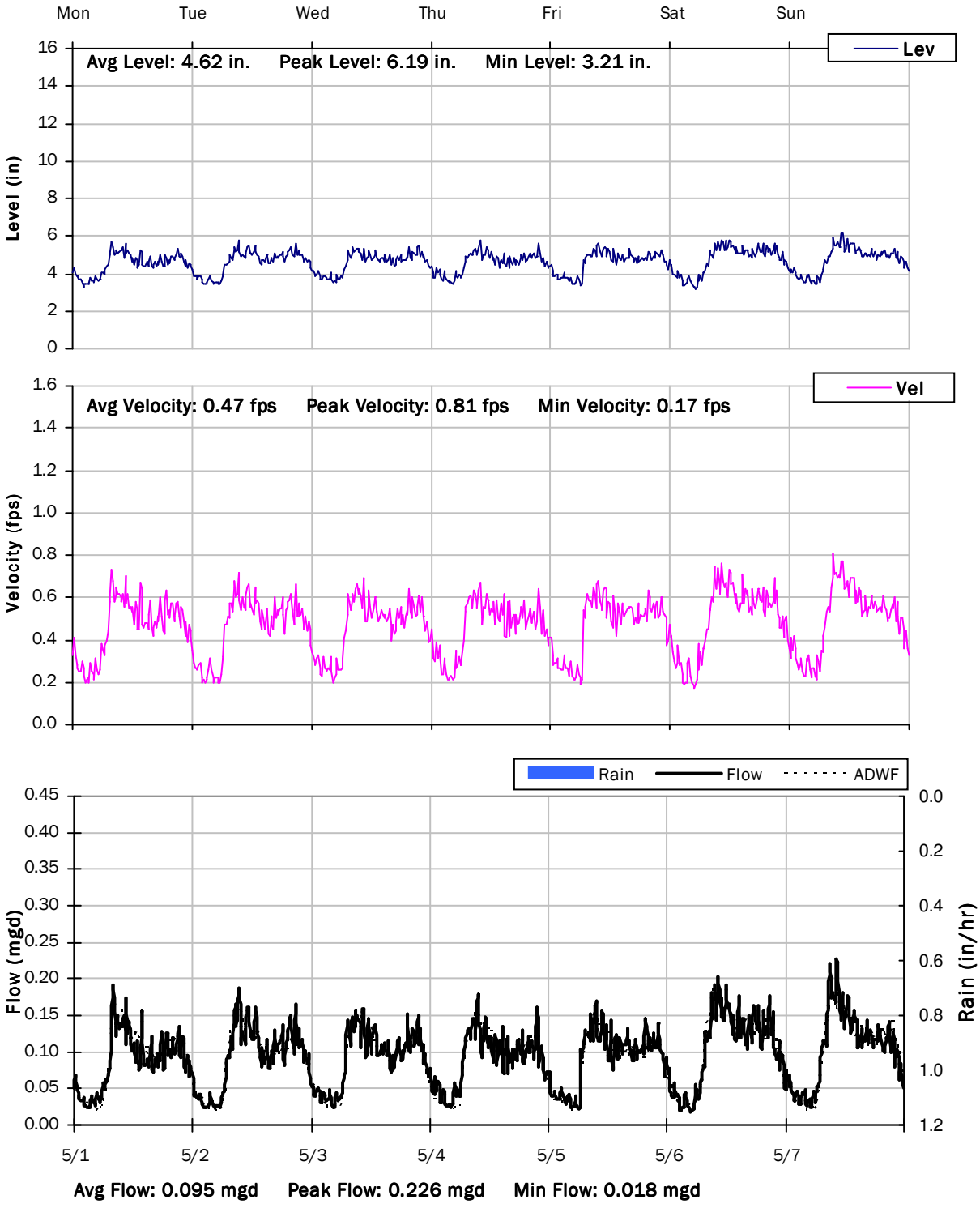
4/24/2017 to 5/1/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

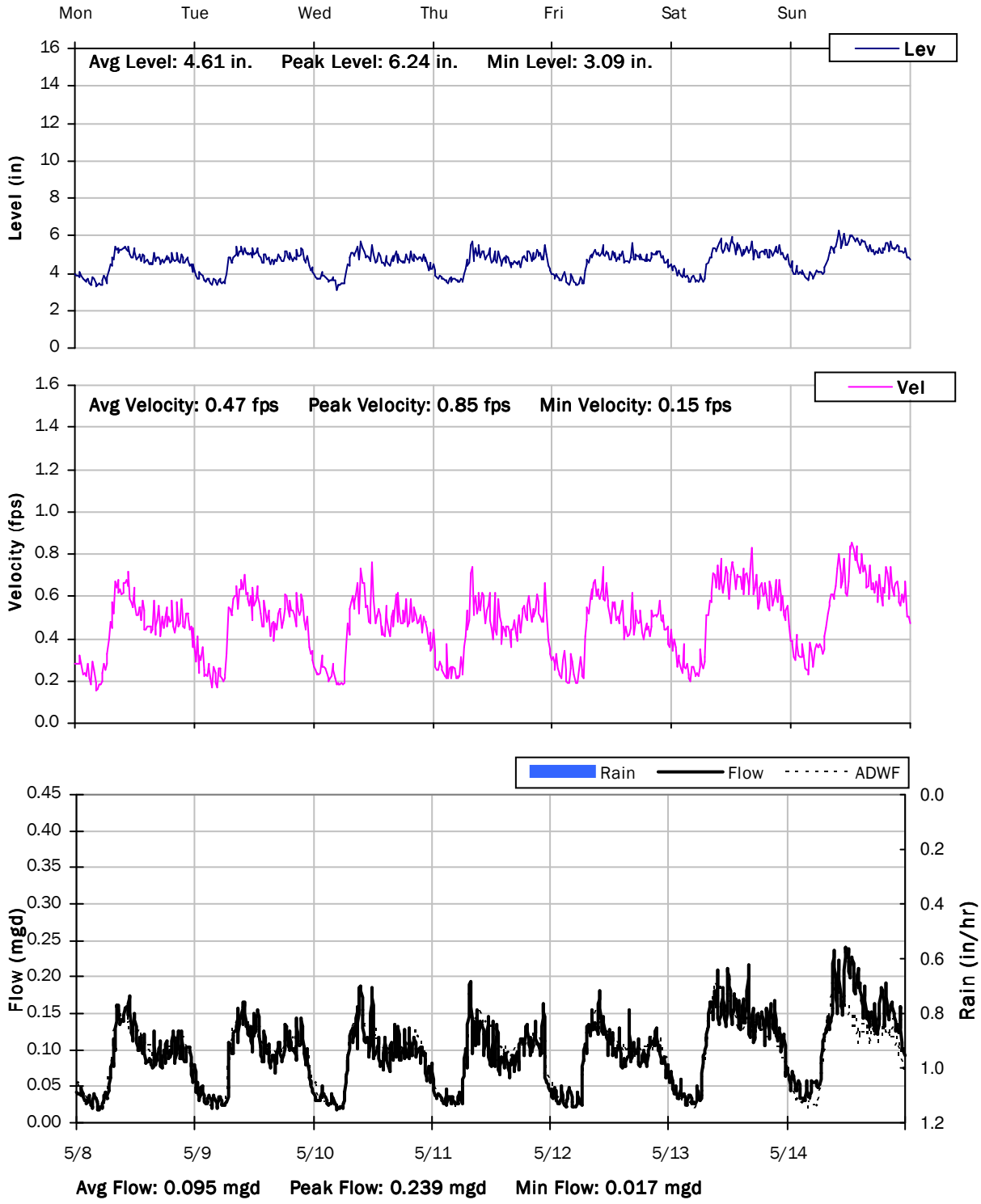
5/1/2017 to 5/8/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

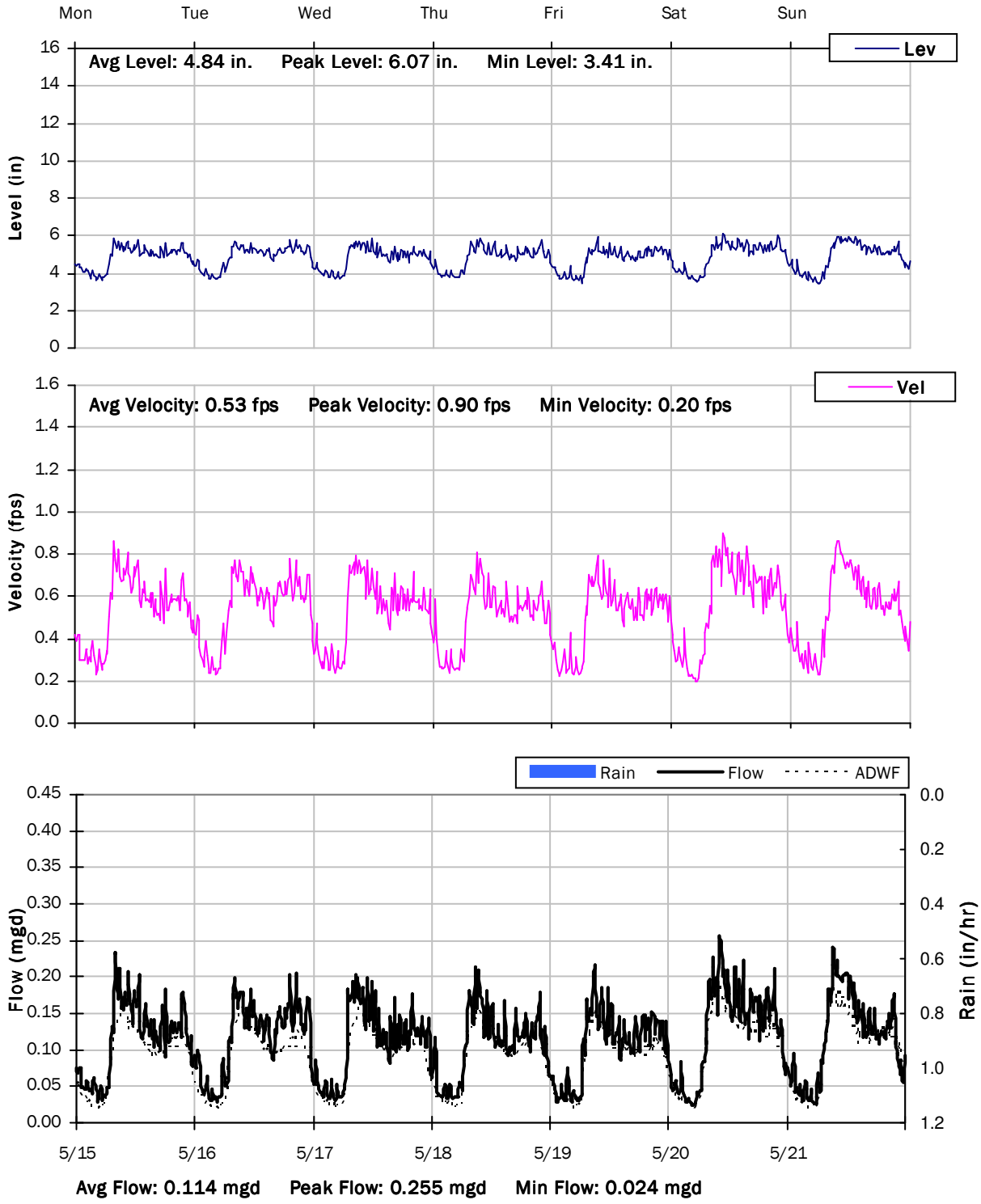
5/8/2017 to 5/15/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

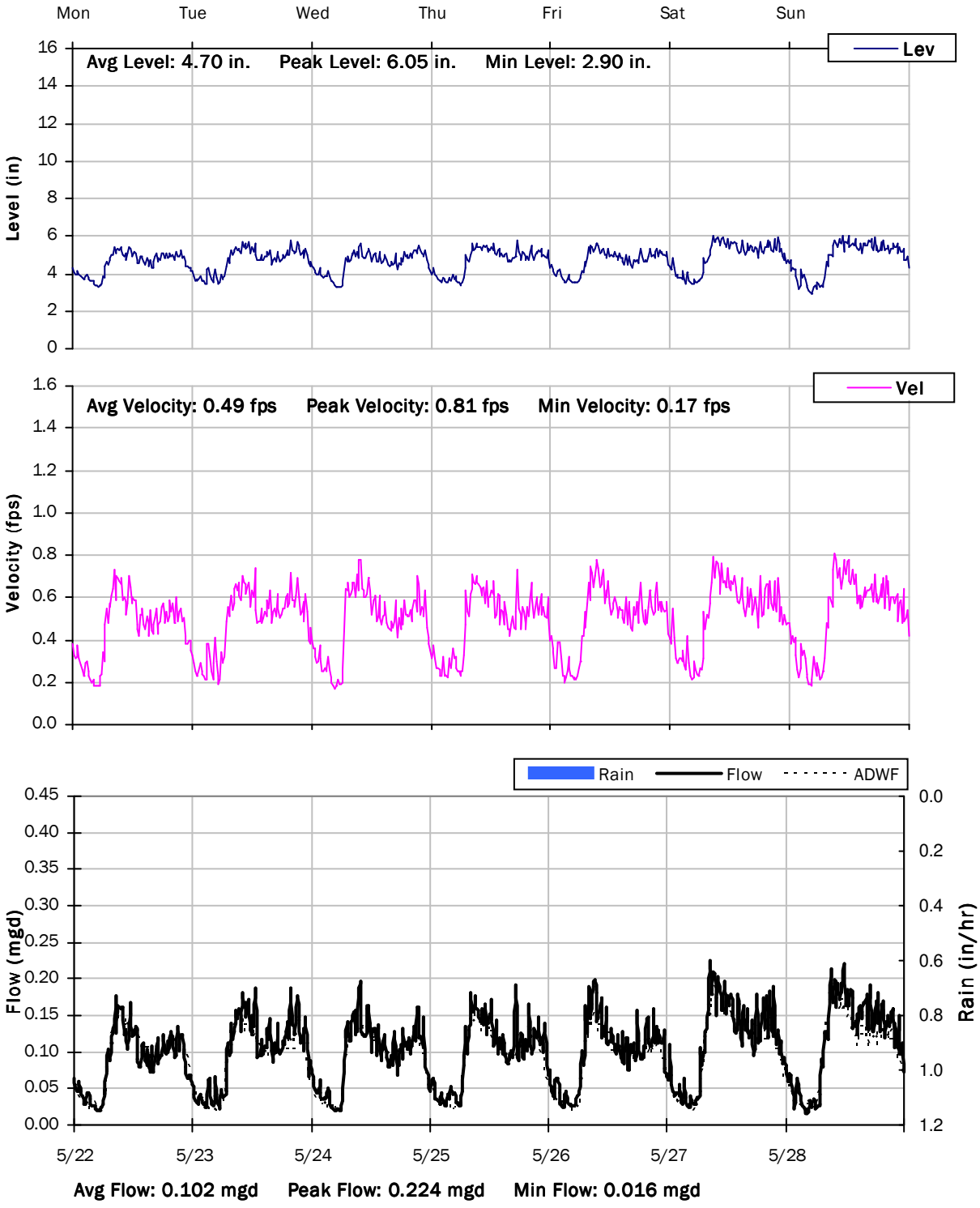
5/15/2017 to 5/22/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

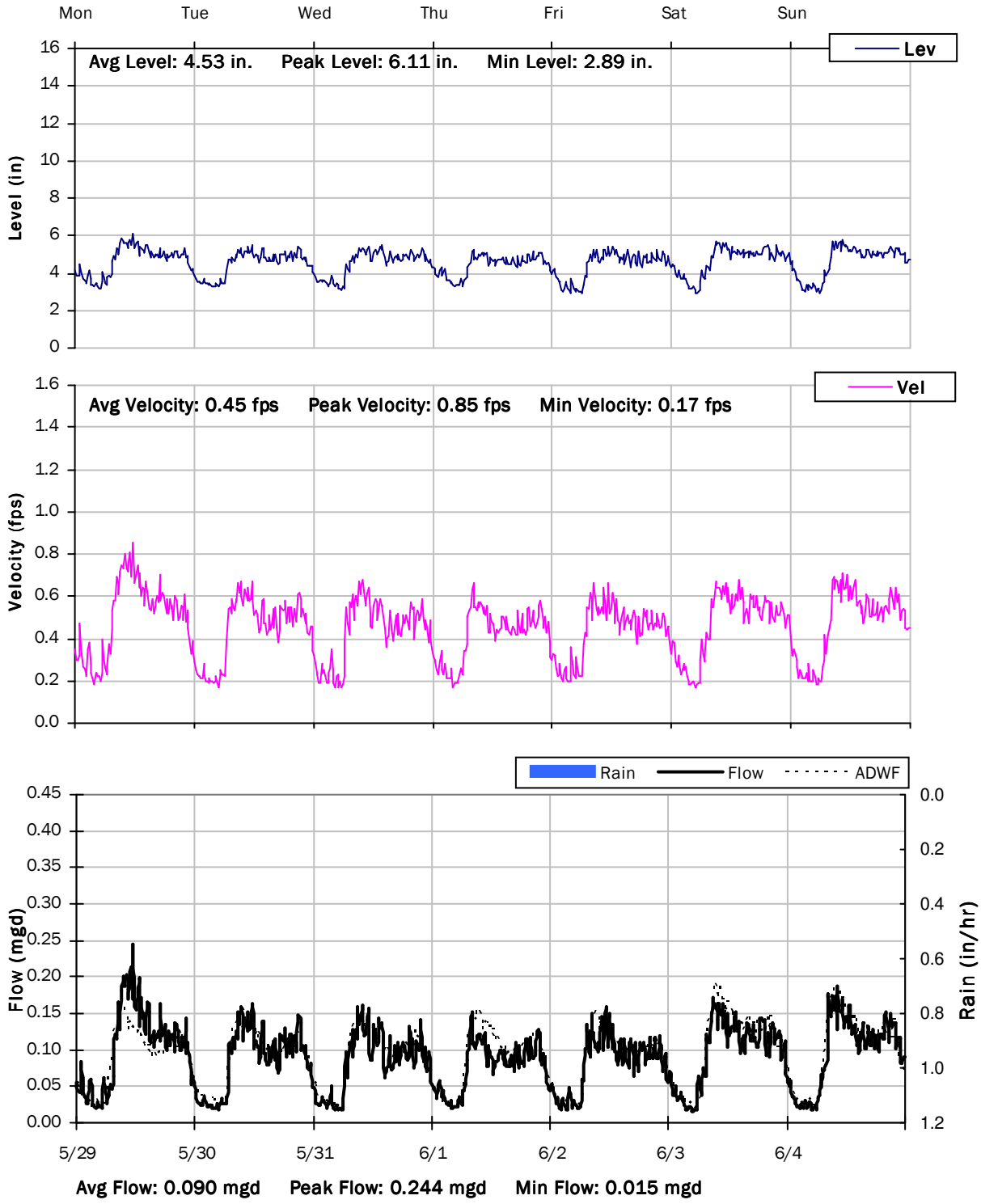
5/22/2017 to 5/29/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

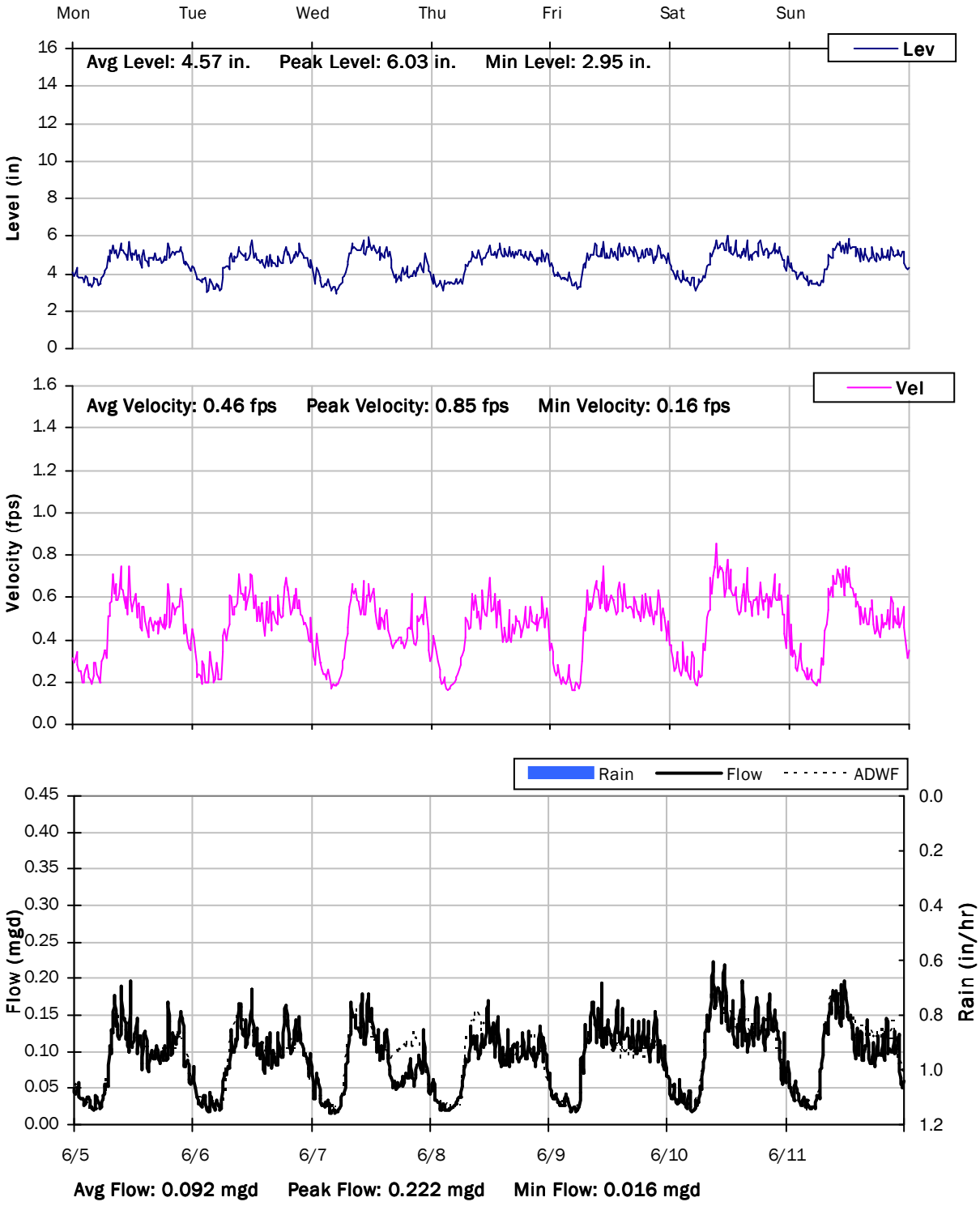
5/29/2017 to 6/5/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

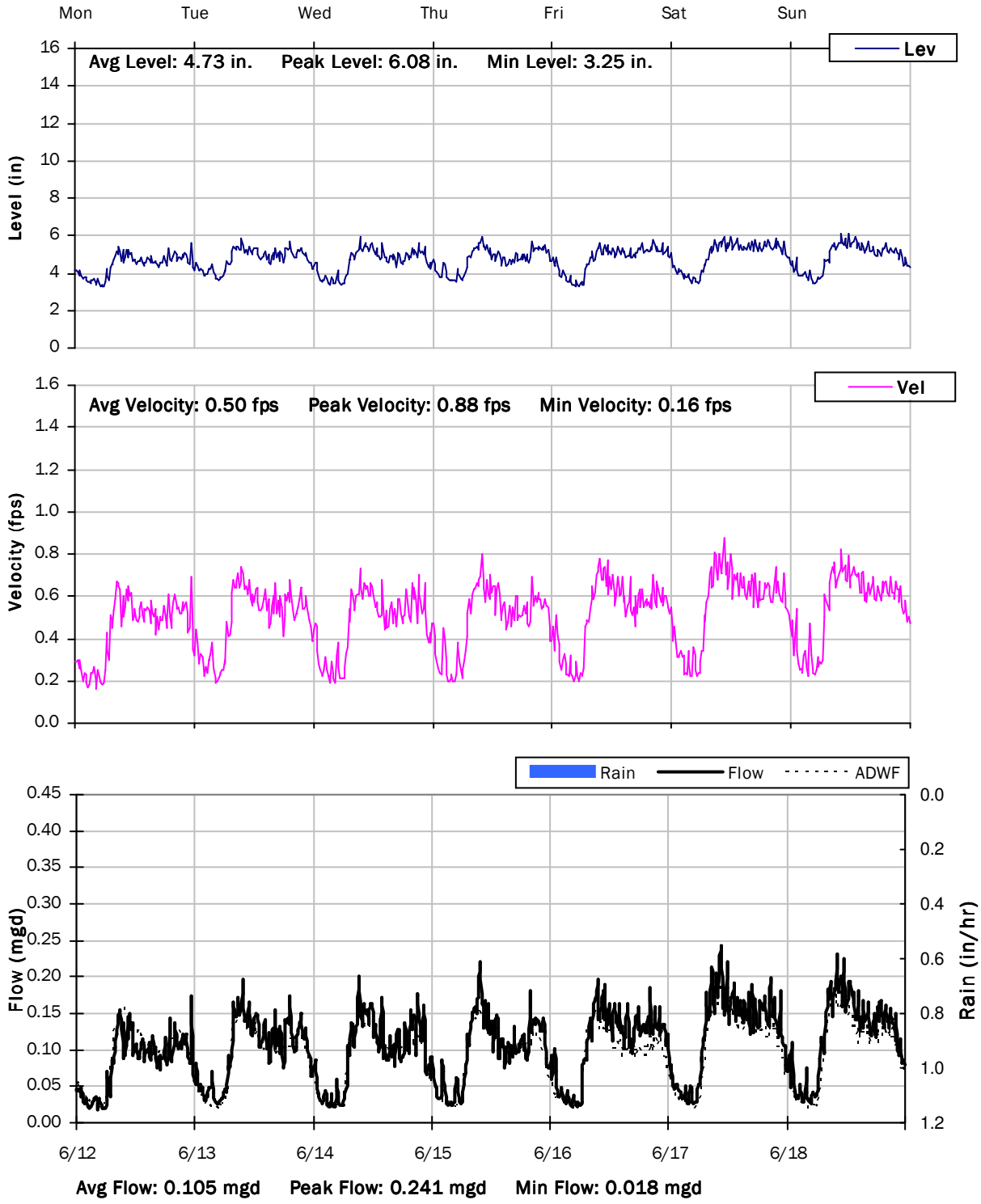
6/5/2017 to 6/12/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

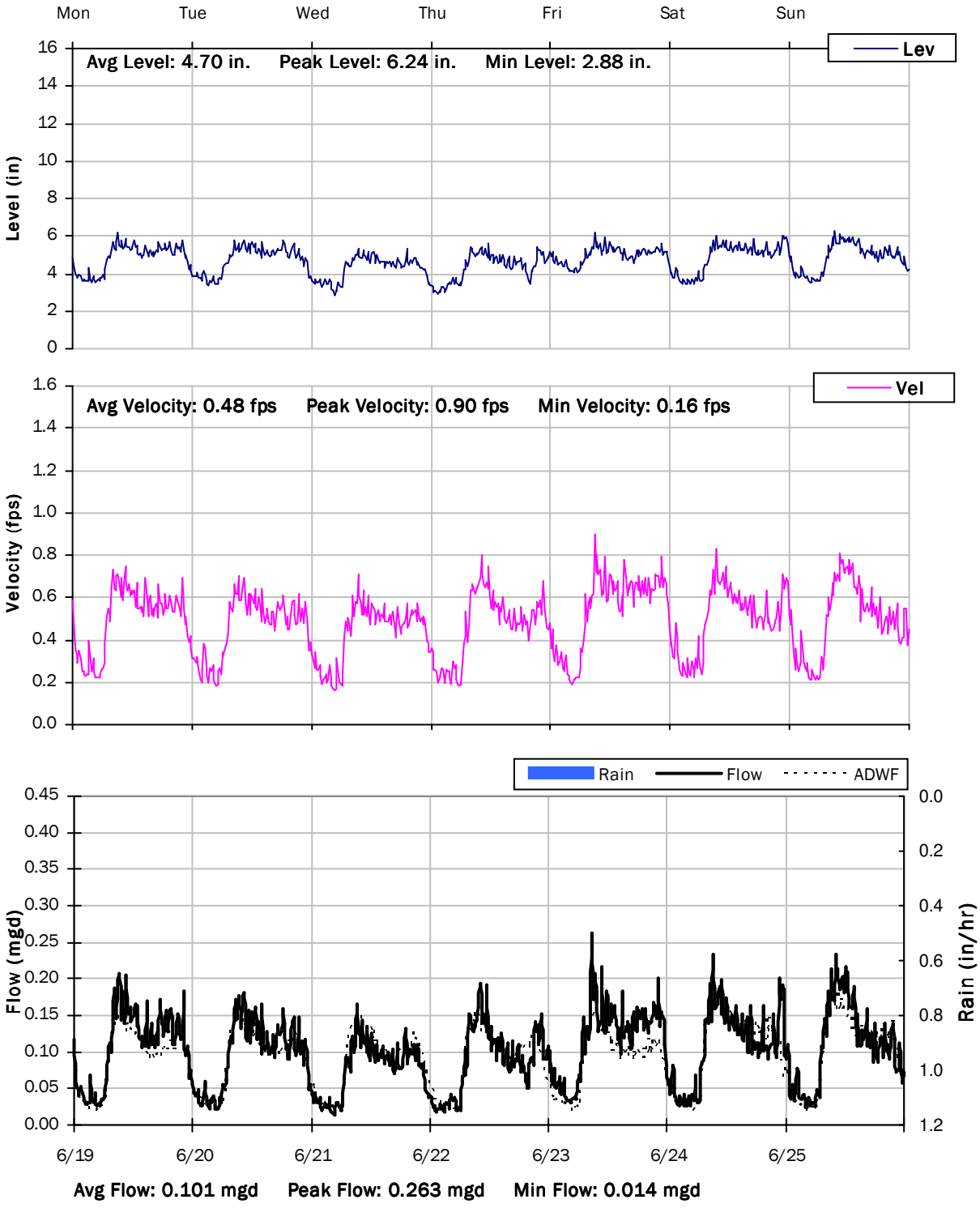
6/12/2017 to 6/19/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

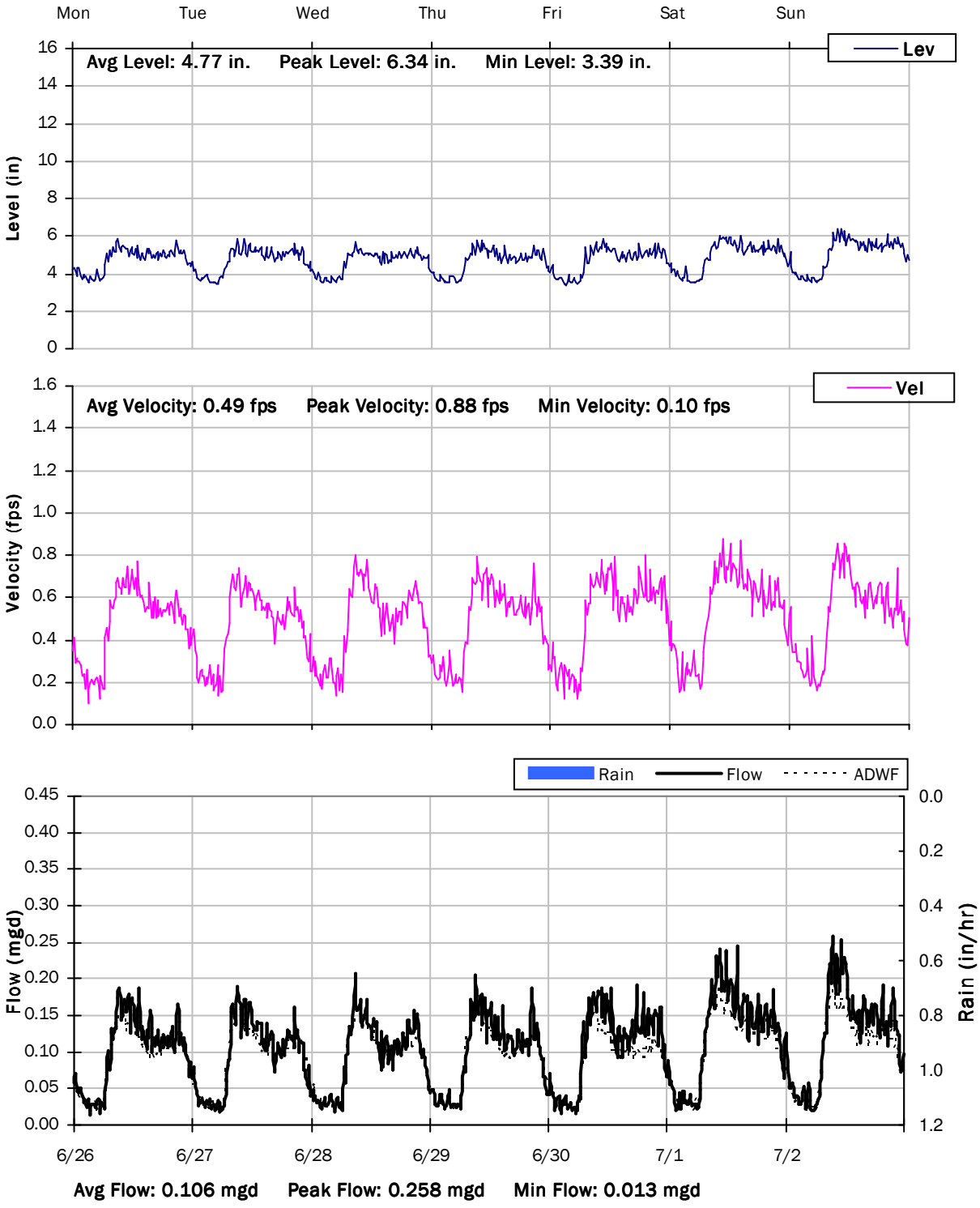
6/19/2017 to 6/26/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

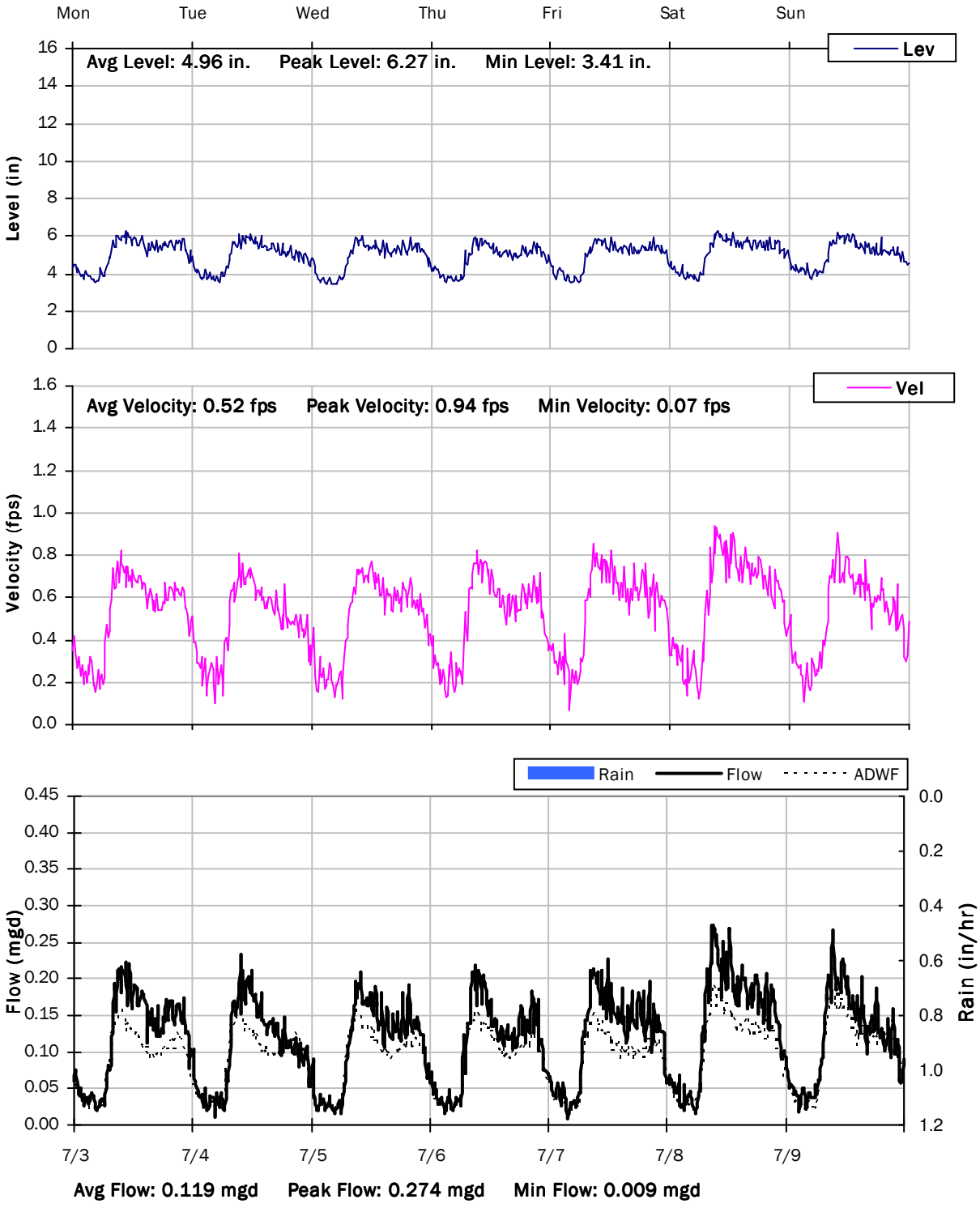
6/26/2017 to 7/3/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

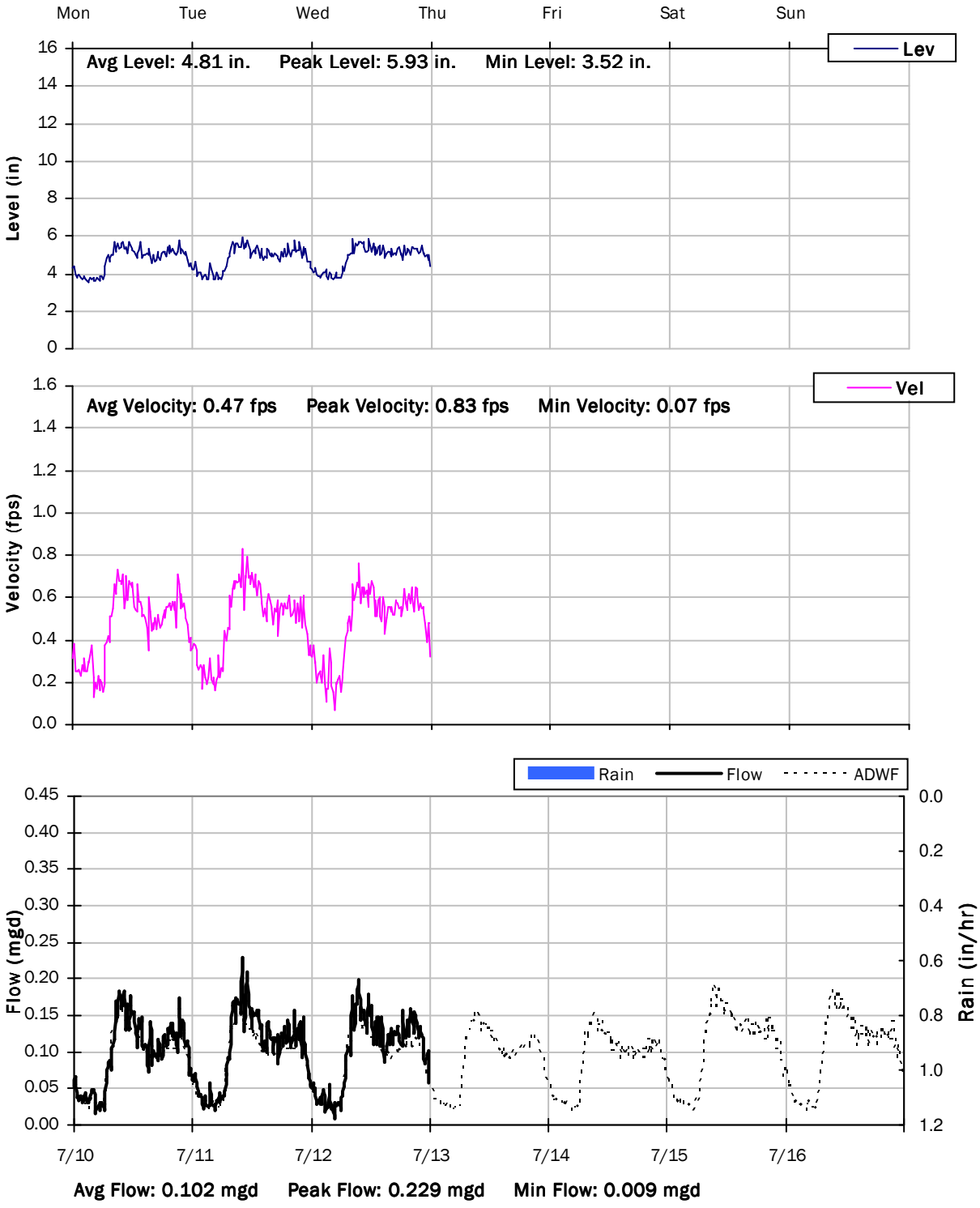
7/3/2017 to 7/10/2017



SITE 13

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

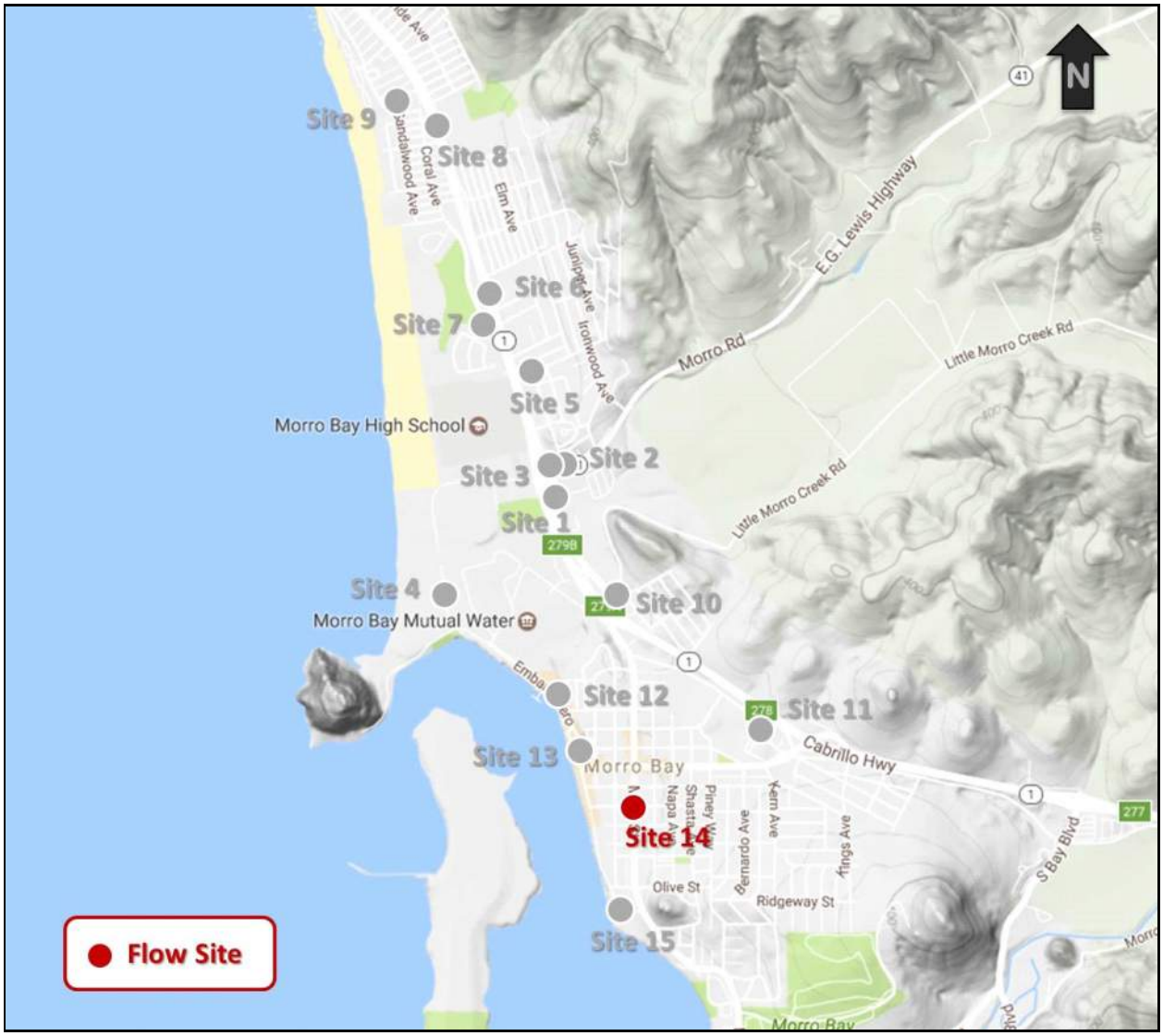
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 14

Location: Intersection of Main Street and Marina Street, meter on east 10" inlet

Data Summary Report



Vicinity Map: Site 14

SITE 14

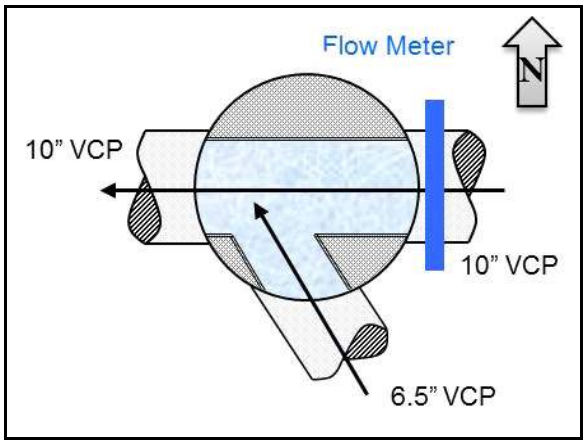
Site Information

Location: Intersection of Main Street and Marina Street, meter on east 10" inlet

Pipe Diameter: 10 inches

ADWF: 0.092 mgd

Peak Measured Flow: 0.323 mgd



SITE 14

Additional Site Photos

SITE 14

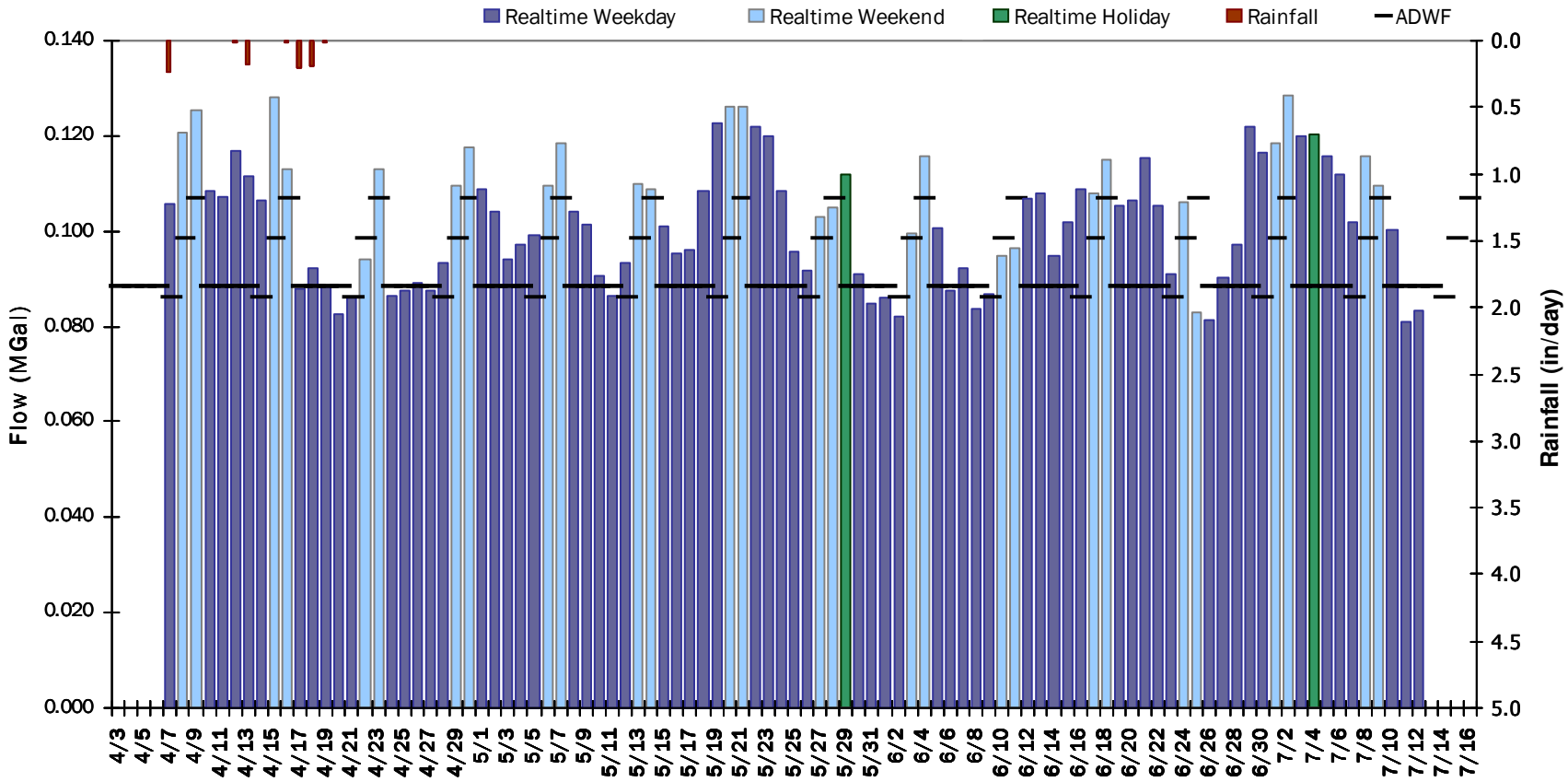
Additional Site Photos

SITE 14

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.103 MGal Peak Daily Flow: 0.128 MGal Min Daily Flow: 0.081 MGal

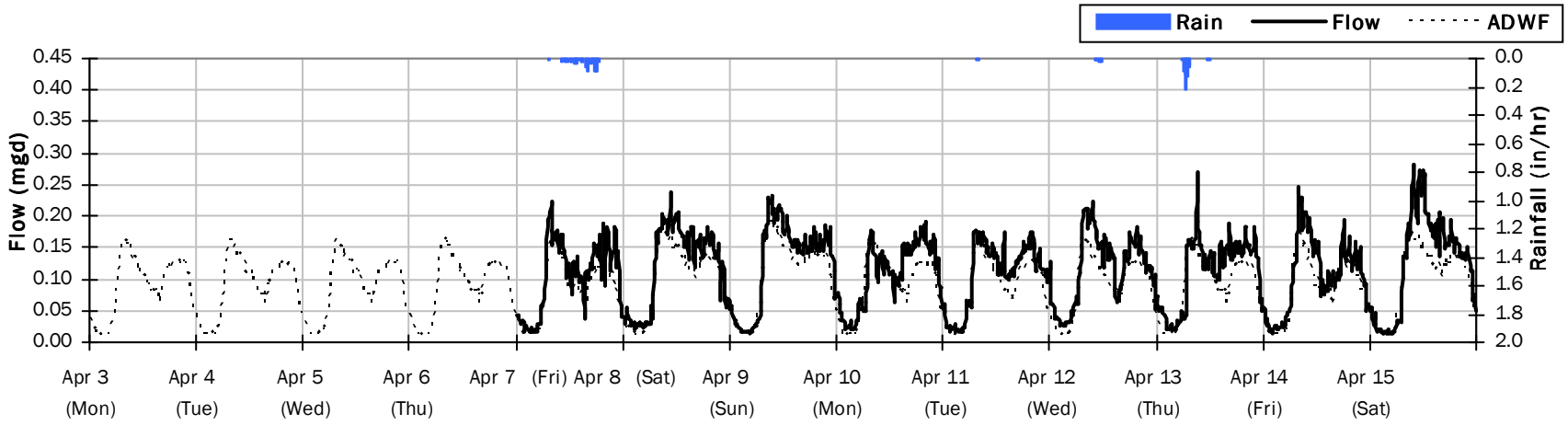
Total Period Rainfall: 0.85 inches

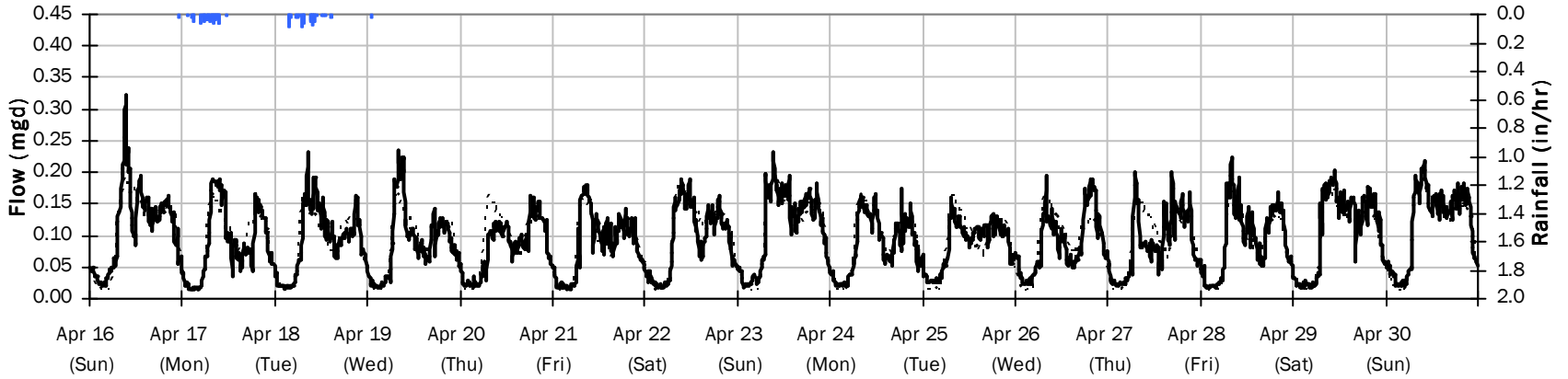


SITE 14

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.102 mgd** **Peak Flow: 0.323 mgd** **Min Flow: 0.013 mgd**

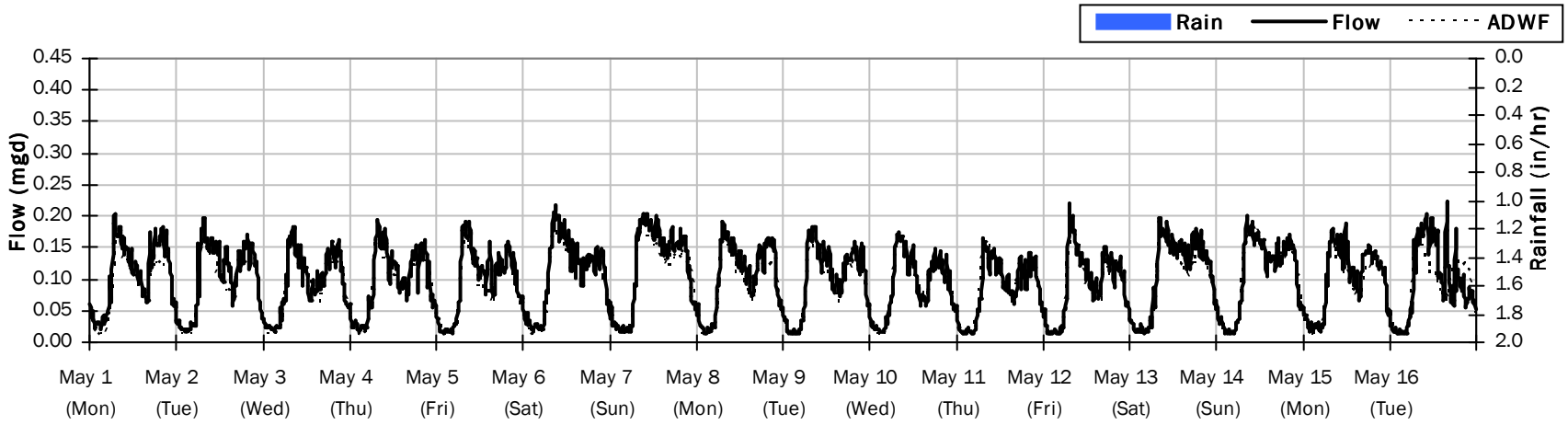


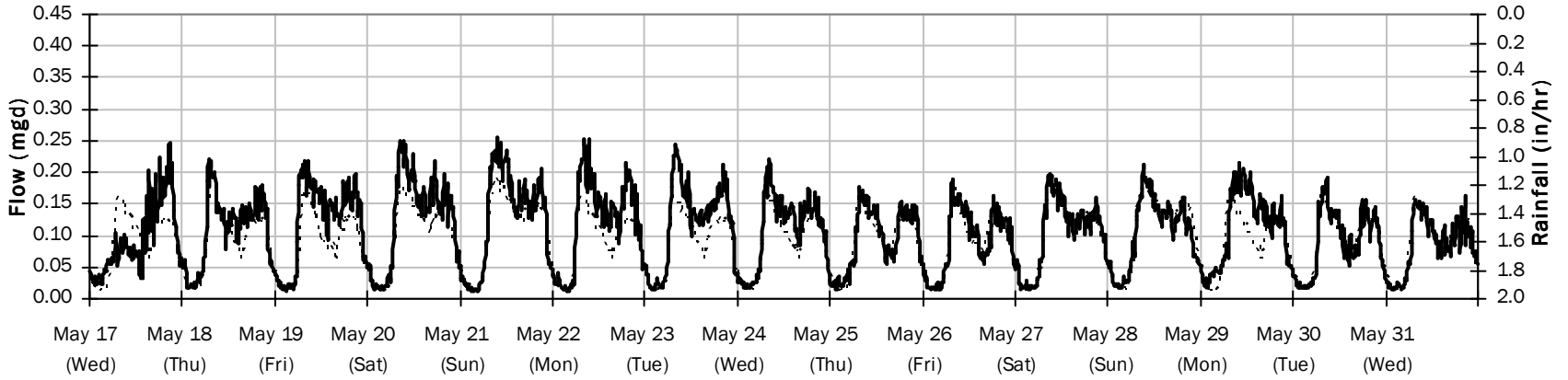


SITE 14

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.104 mgd** **Peak Flow: 0.254 mgd** **Min Flow: 0.012 mgd**

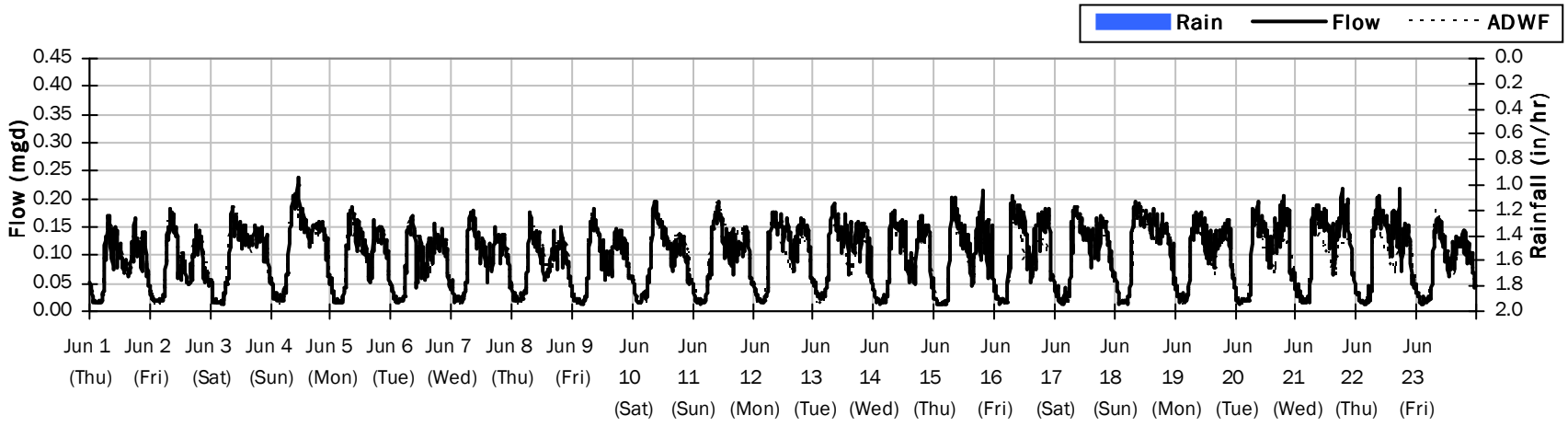


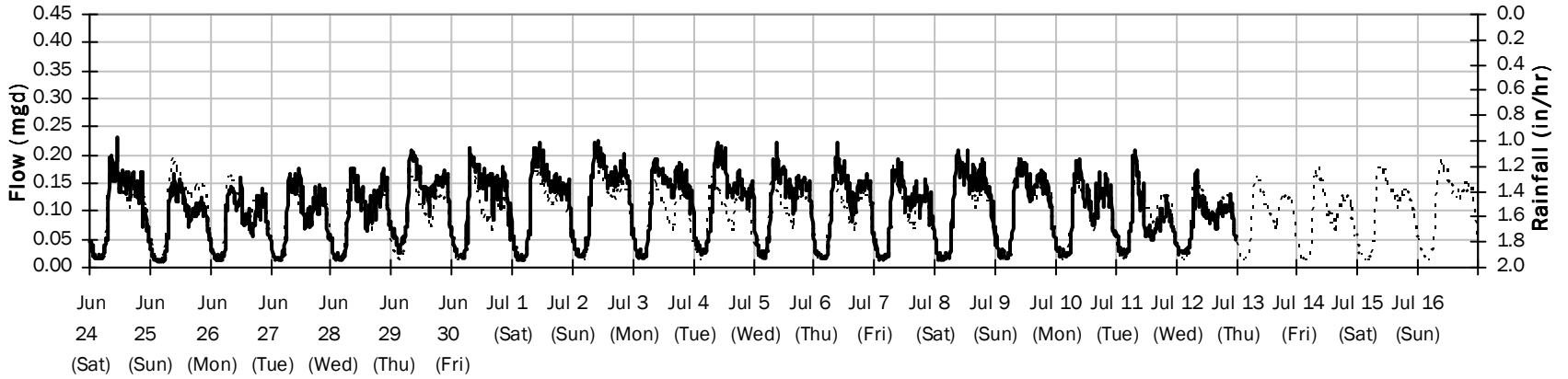


SITE 14

Flow Summary: 6/1/2017 to 7/16/2017

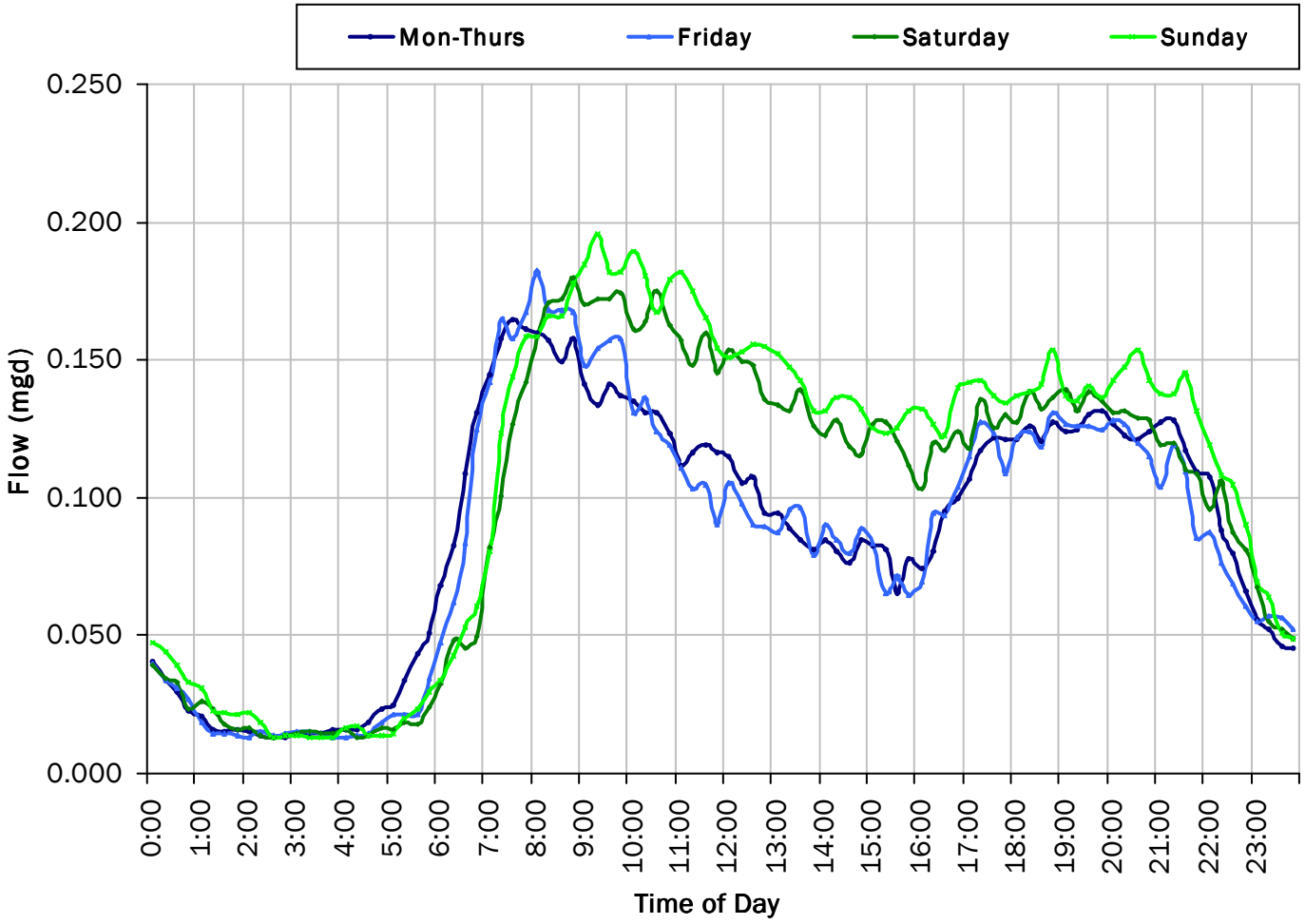
Total Period Rainfall: 0.00 inches **Avg Flow: 0.102 mgd** **Peak Flow: 0.237 mgd** **Min Flow: 0.011 mgd**





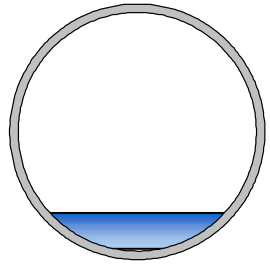
SITE 14

Average Dry Weather Flow Hydrographs



ADWF:

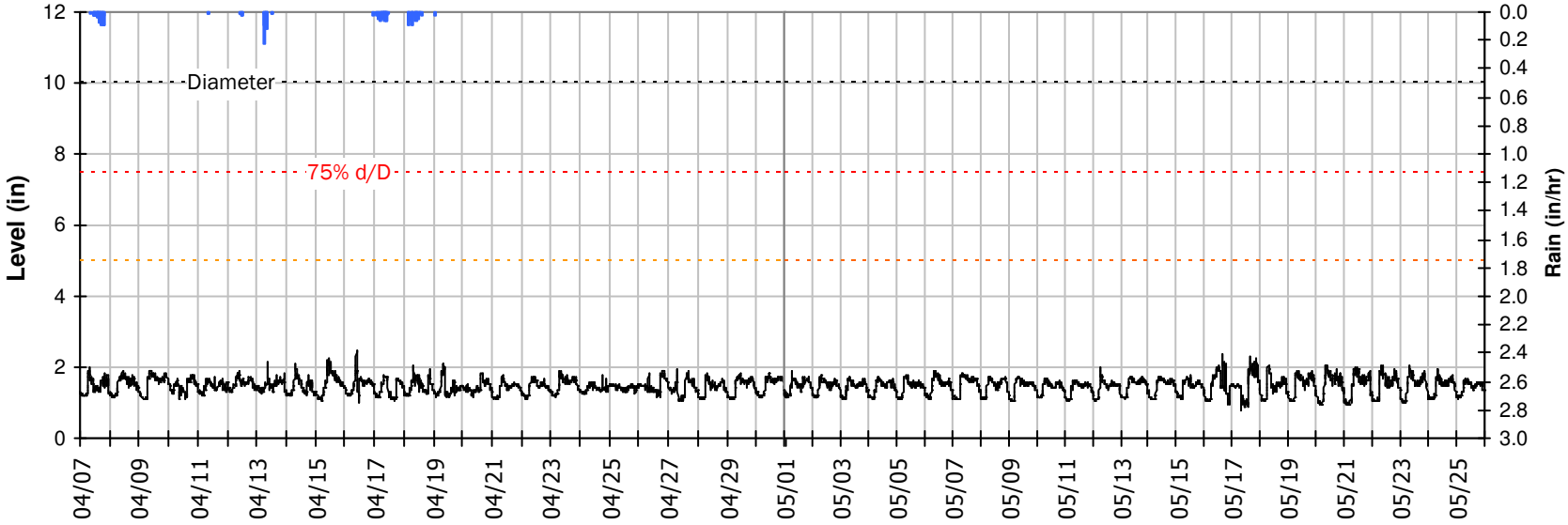
0.092 *mgd*

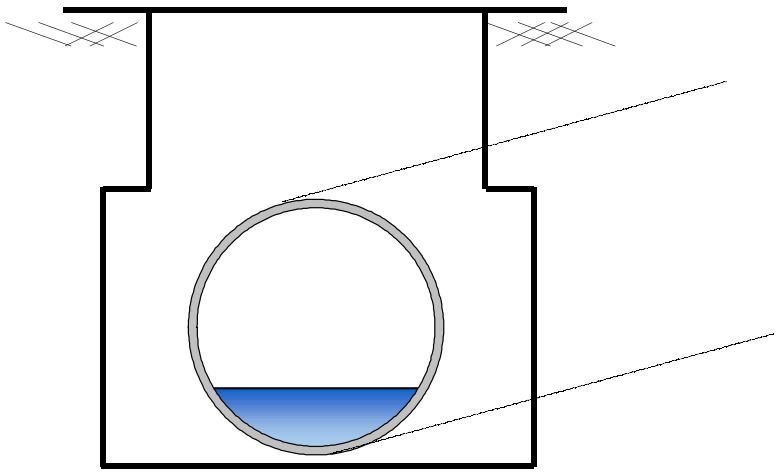


SITE 14

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



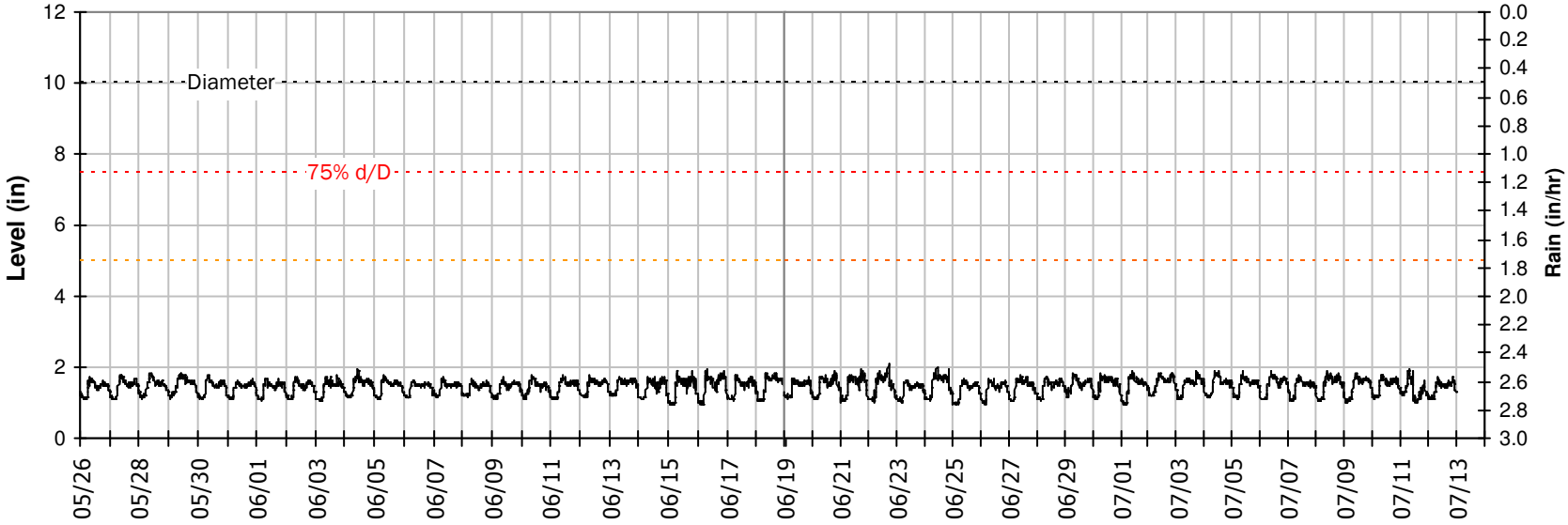


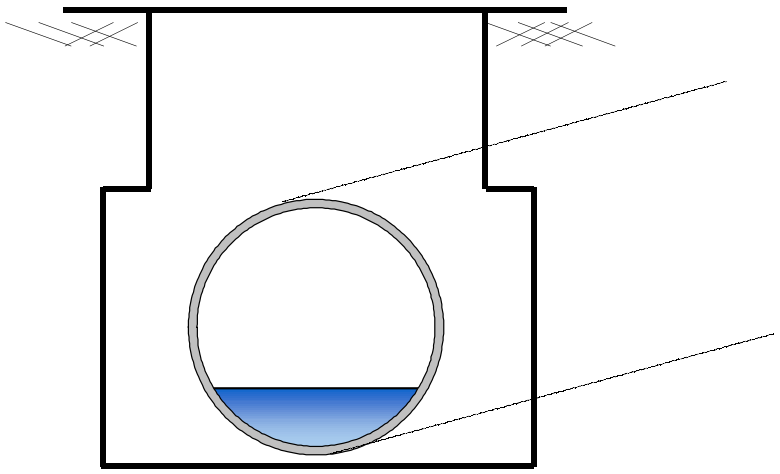
Pipe Diameter:	10	*inches*
Peak Measured Level:	2.46	*inches*
Peak d/D Ratio:	0.25	

SITE 14

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



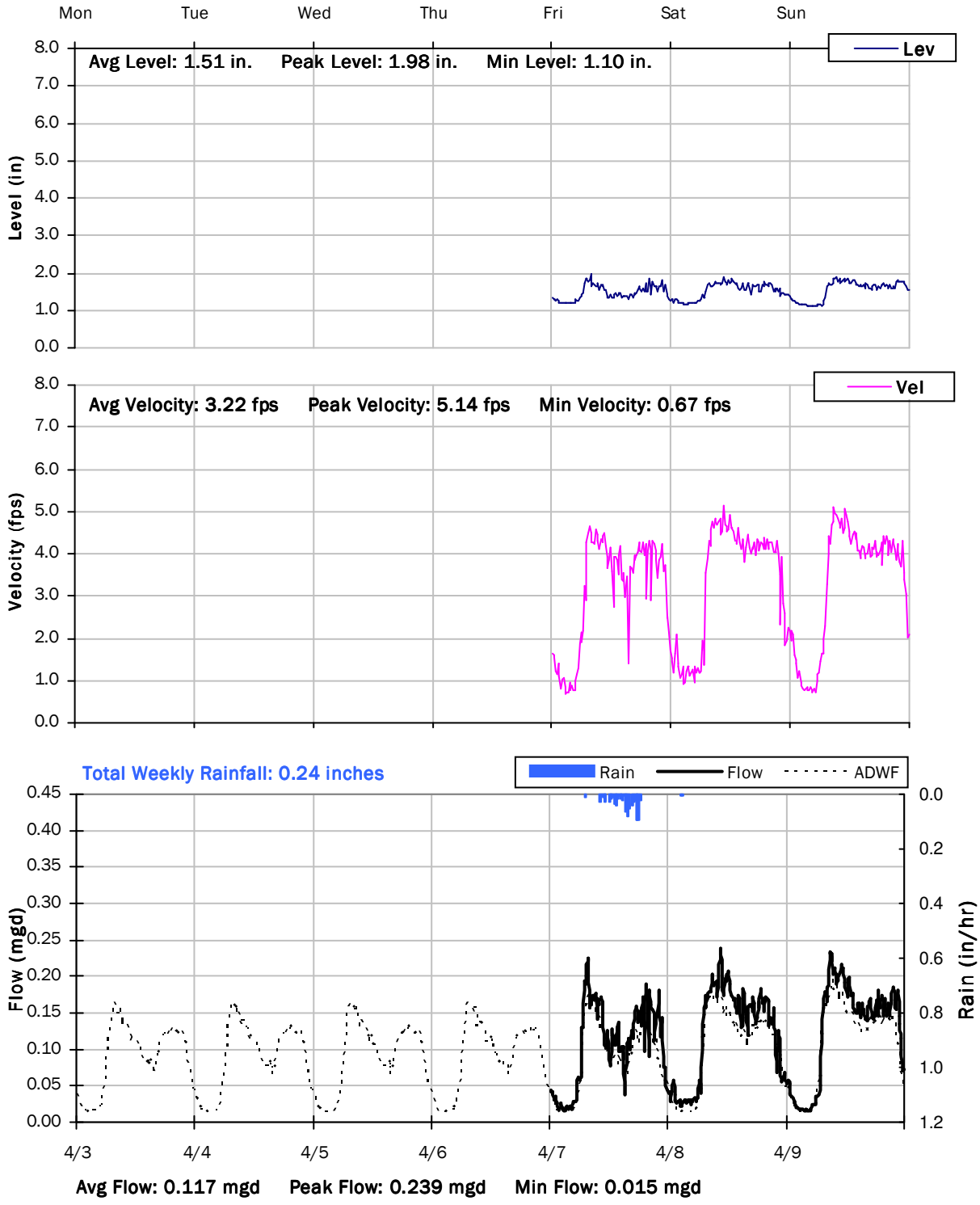


Pipe Diameter:	10	*inches*
Peak Measured Level:	2.46	*inches*
Peak d/D Ratio:	0.25	

SITE 14

Weekly Level, Velocity and Flow Hydrographs

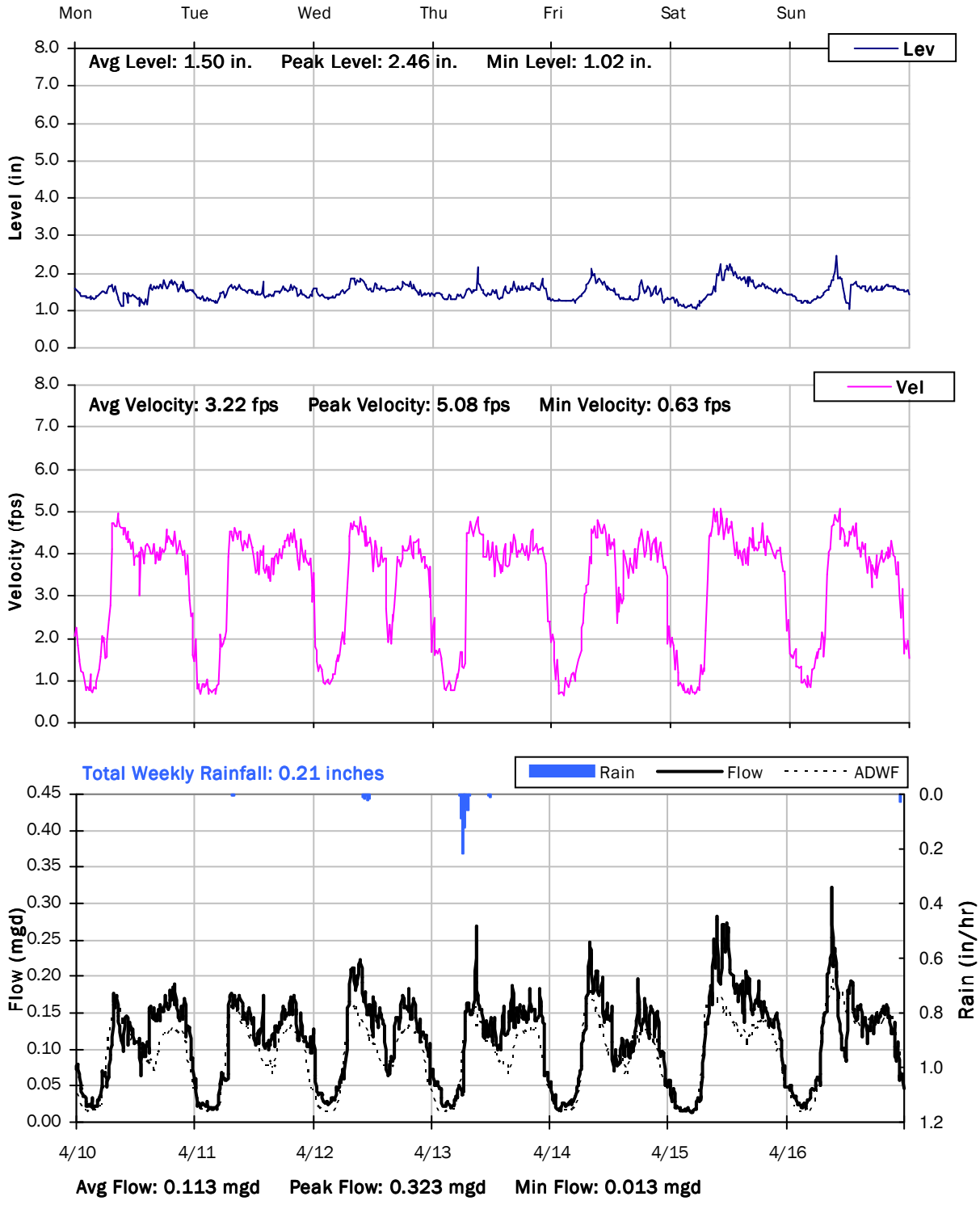
4/3/2017 to 4/10/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

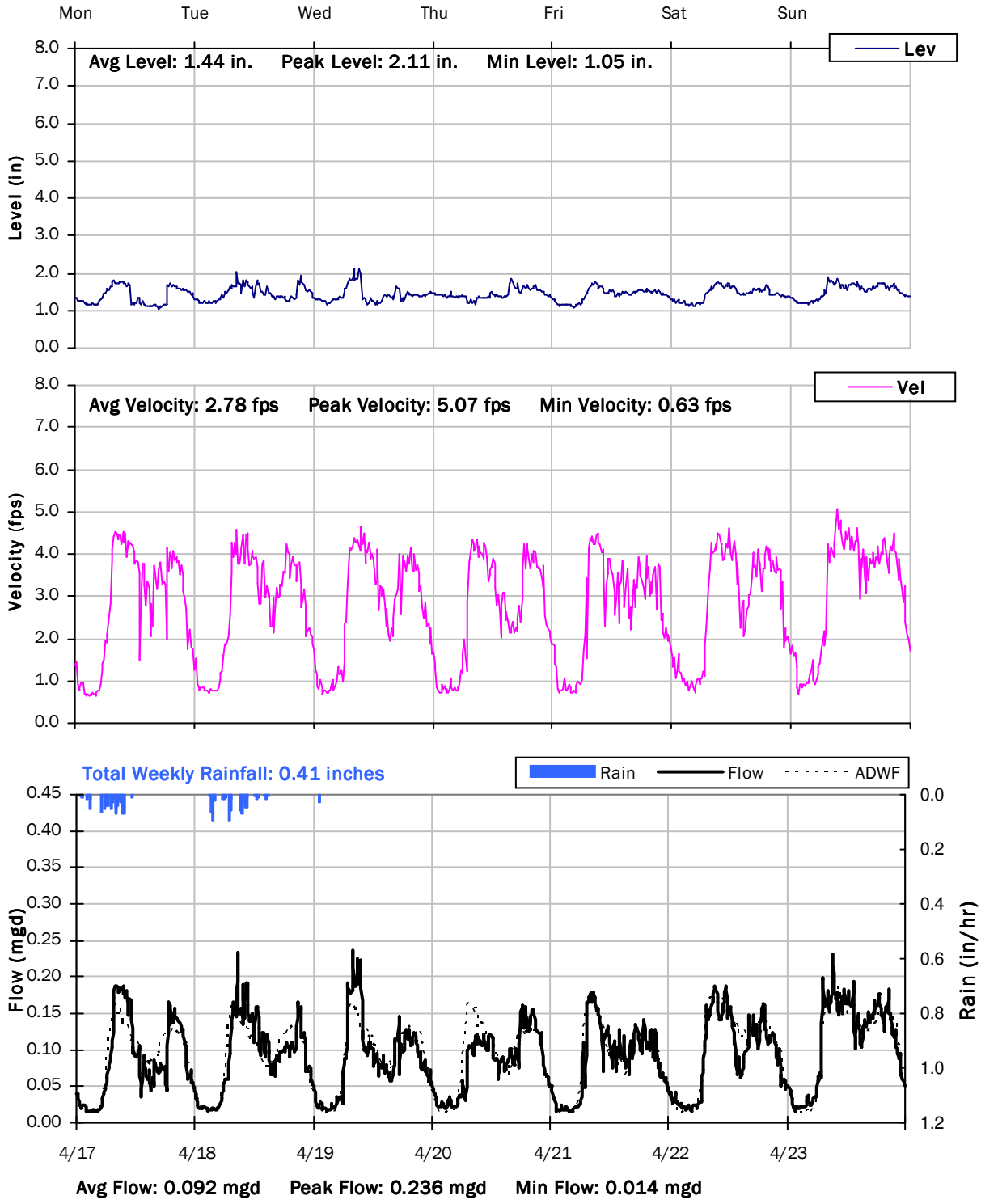
4/10/2017 to 4/17/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

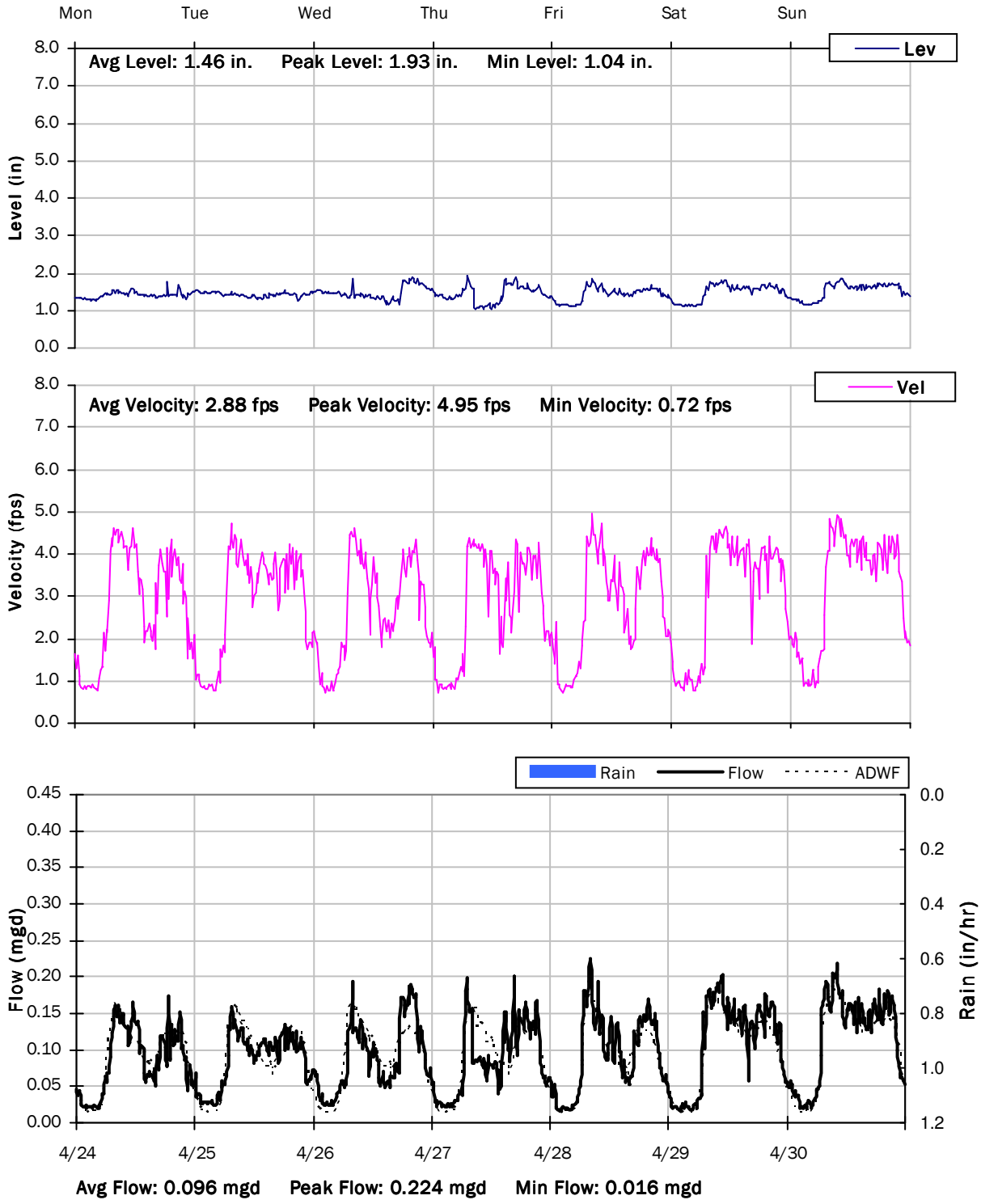
4/17/2017 to 4/24/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

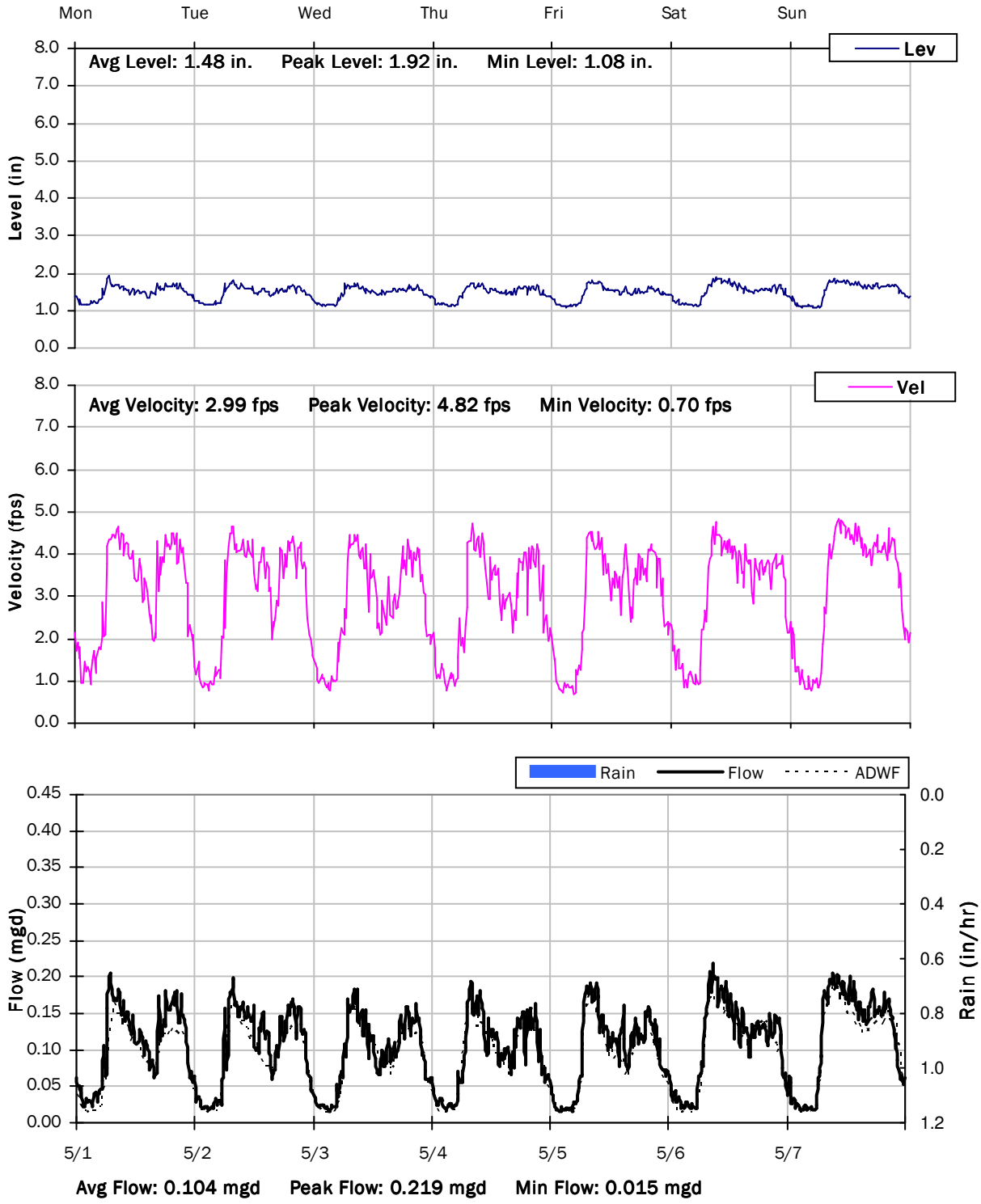
4/24/2017 to 5/1/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

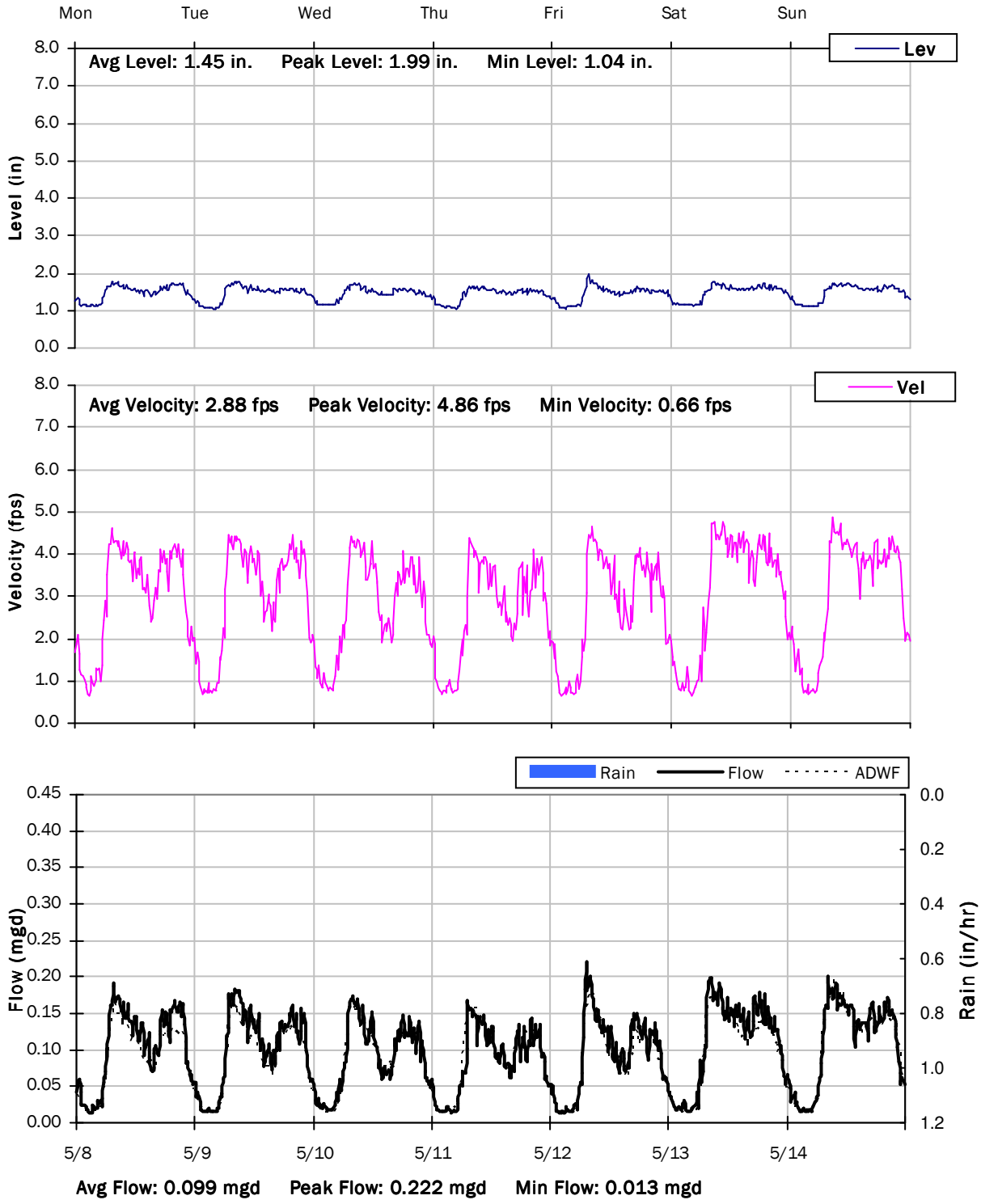
5/1/2017 to 5/8/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

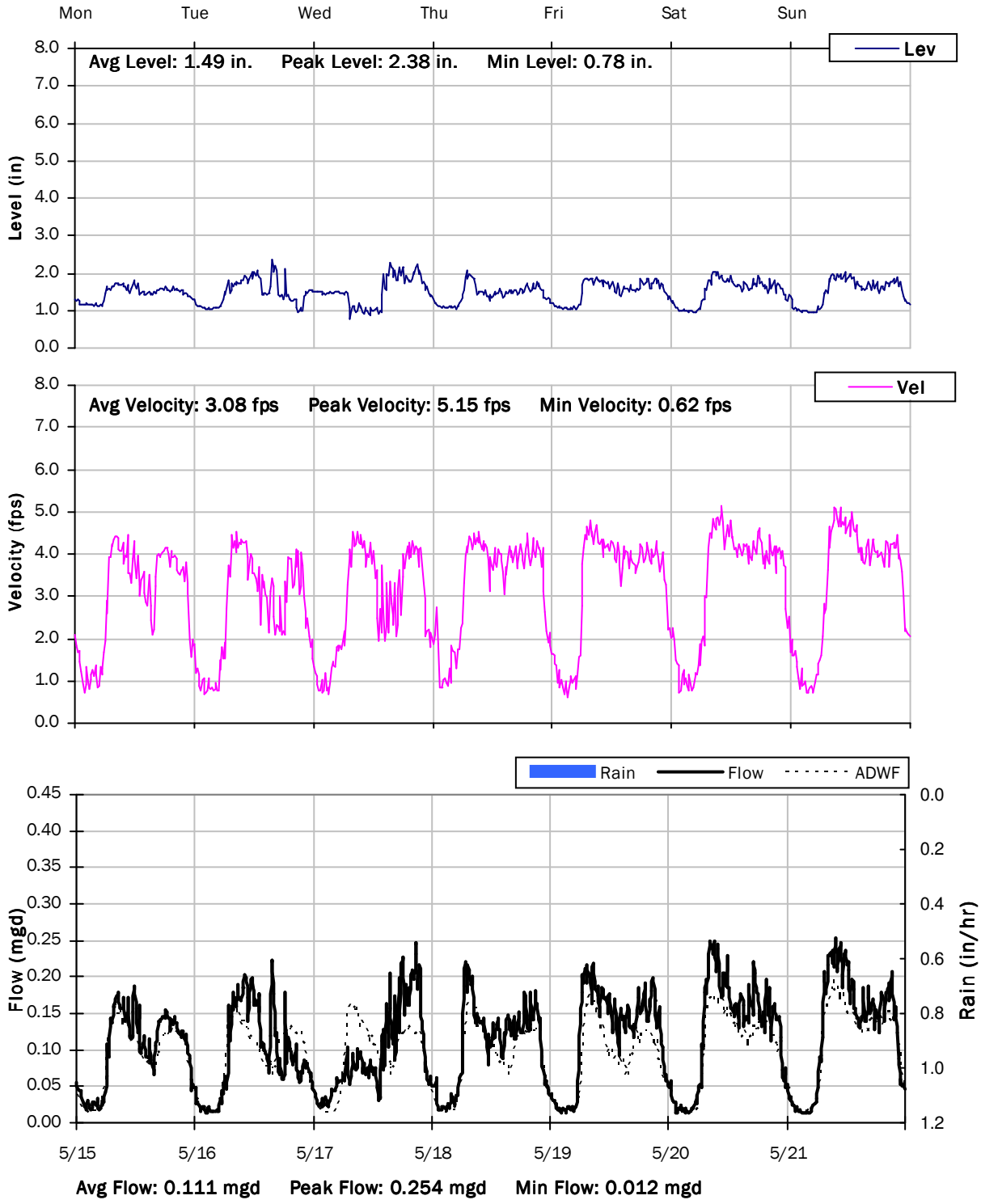
5/8/2017 to 5/15/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

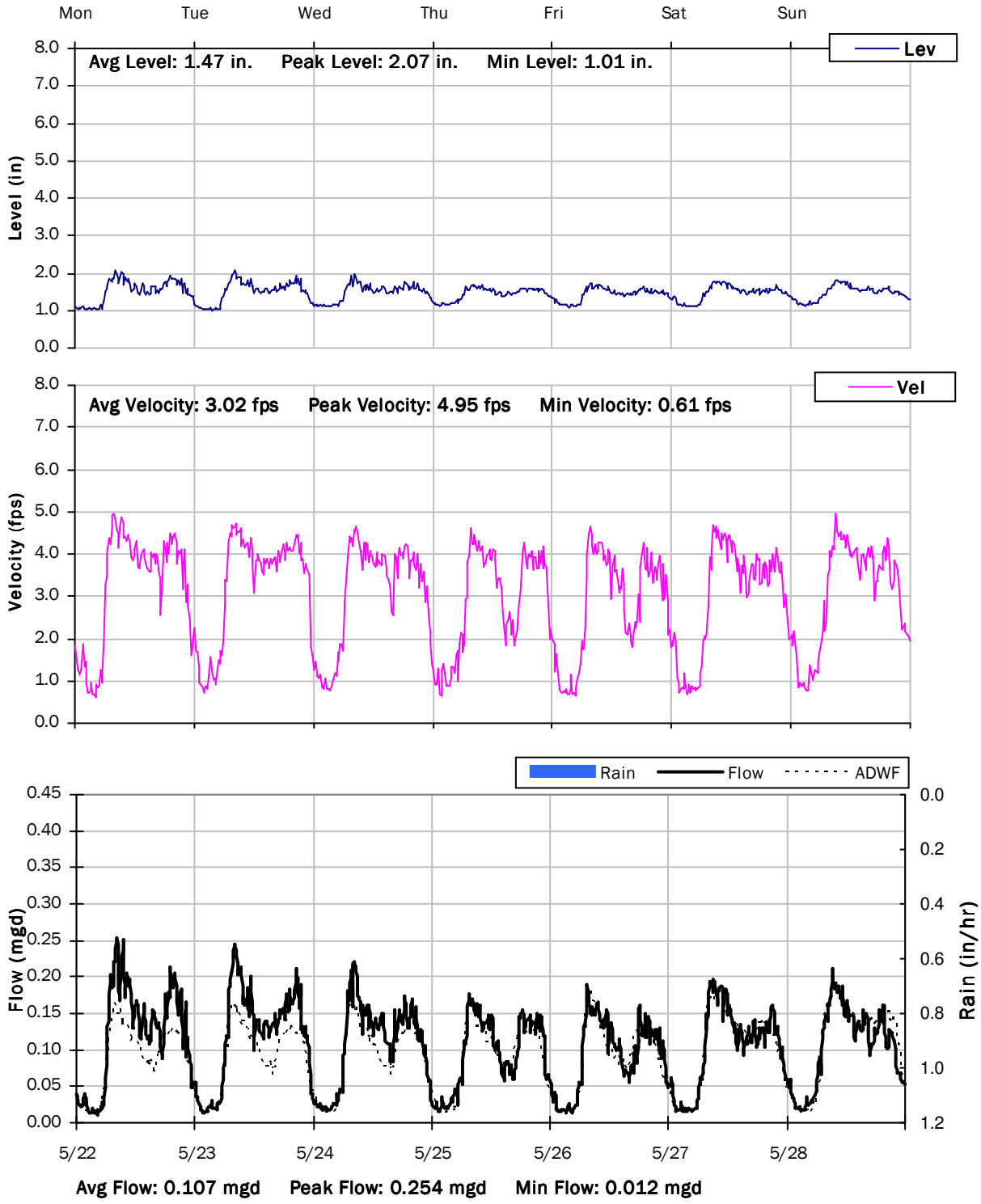
5/15/2017 to 5/22/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

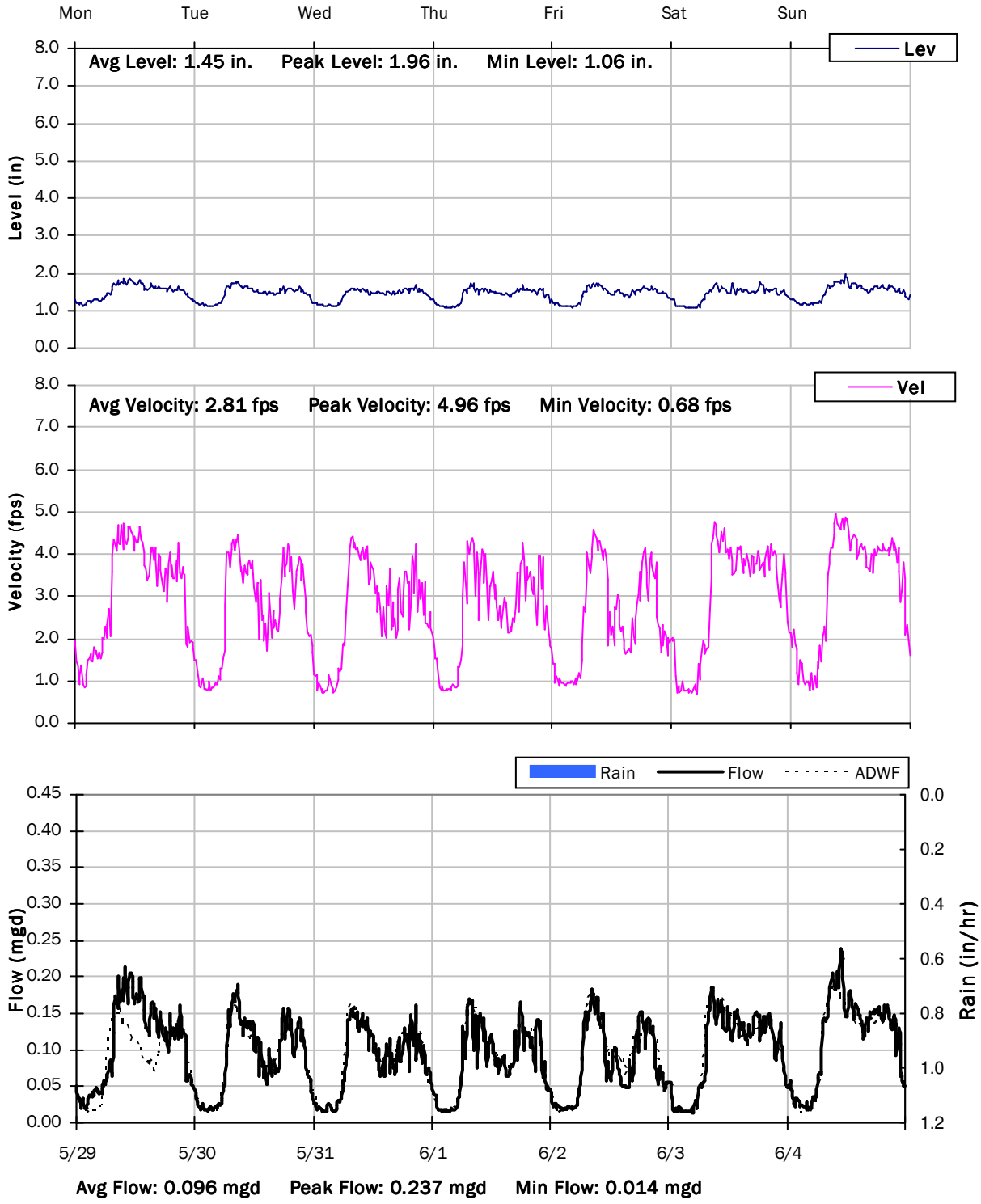
5/22/2017 to 5/29/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

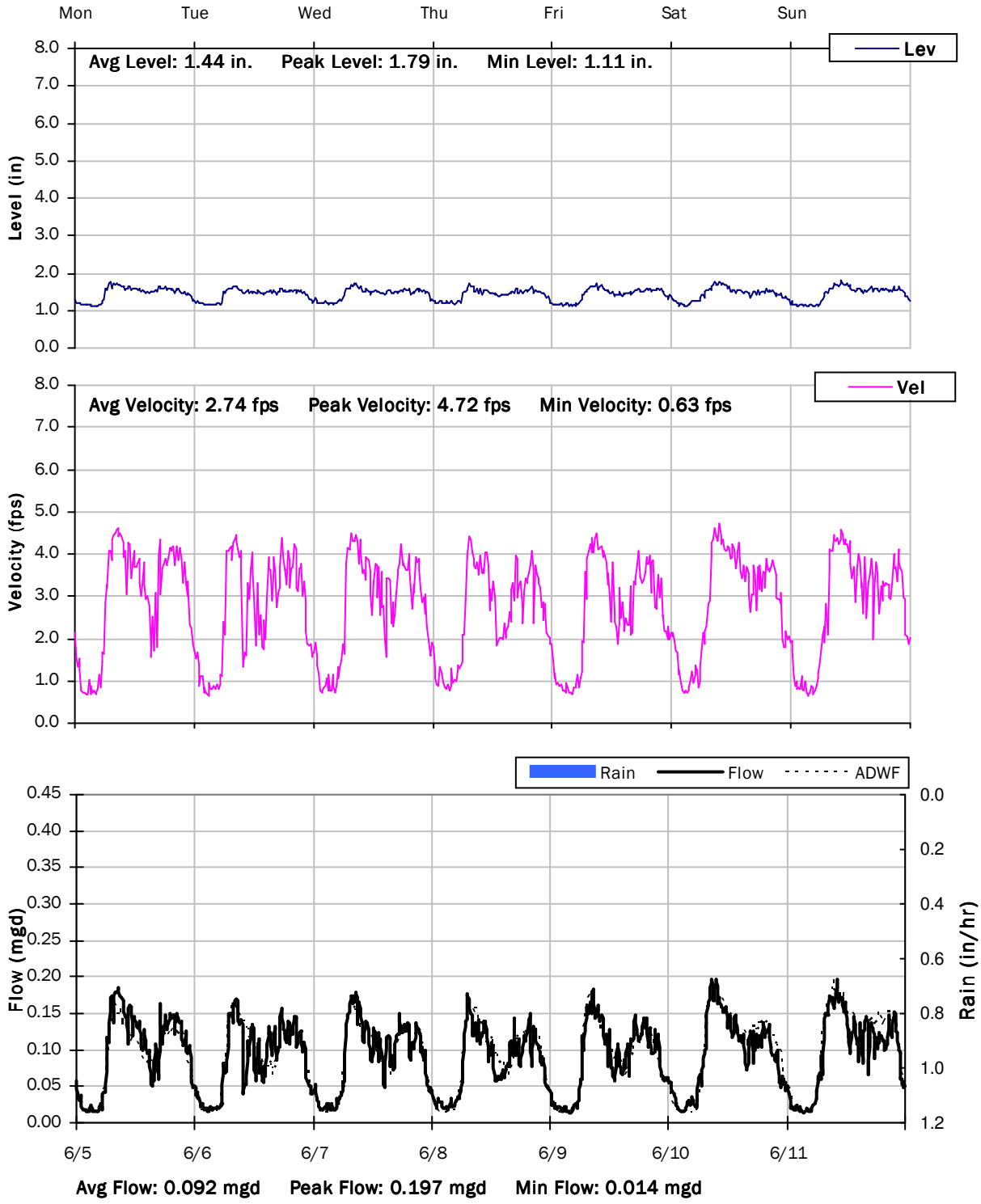
5/29/2017 to 6/5/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

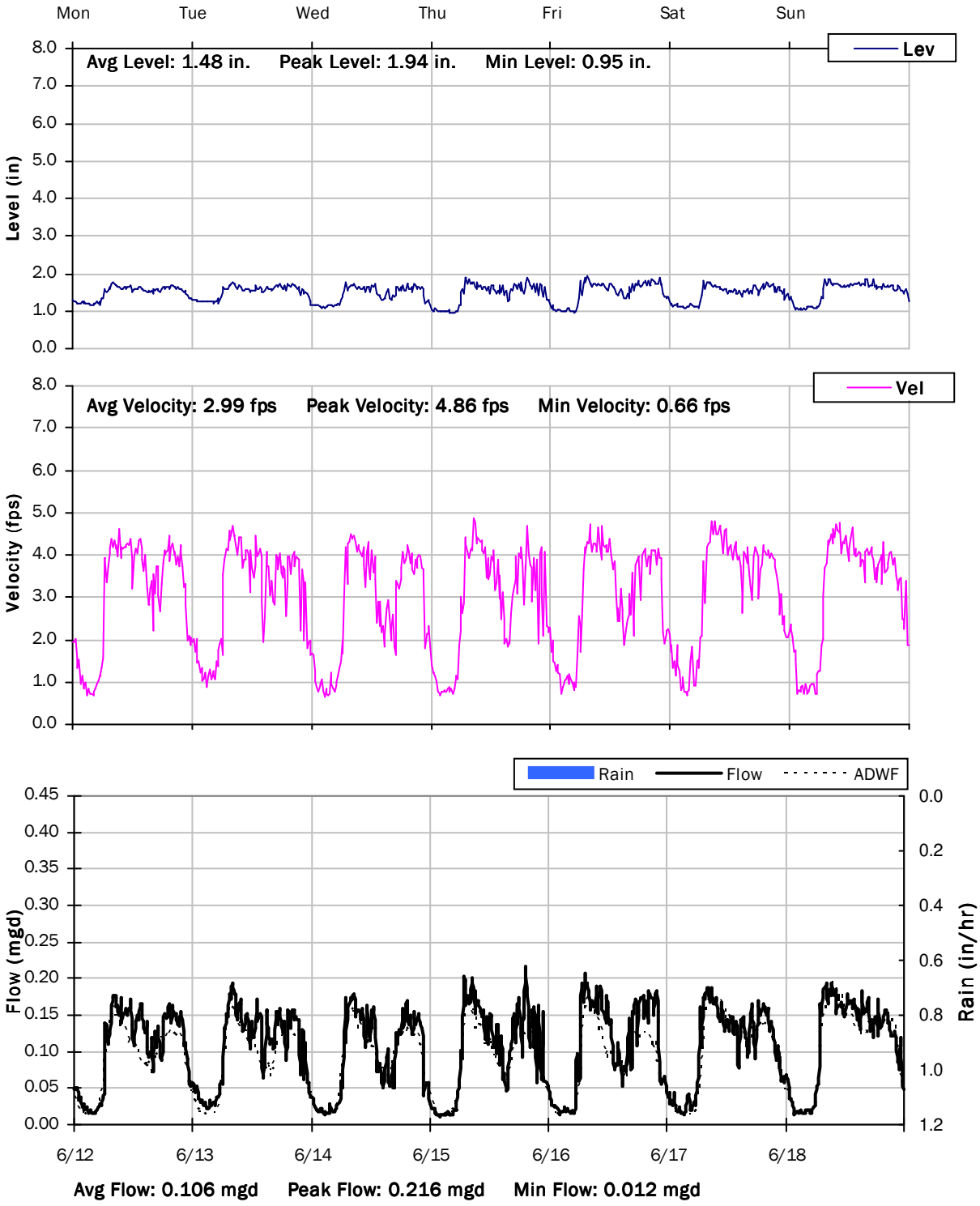
6/5/2017 to 6/12/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

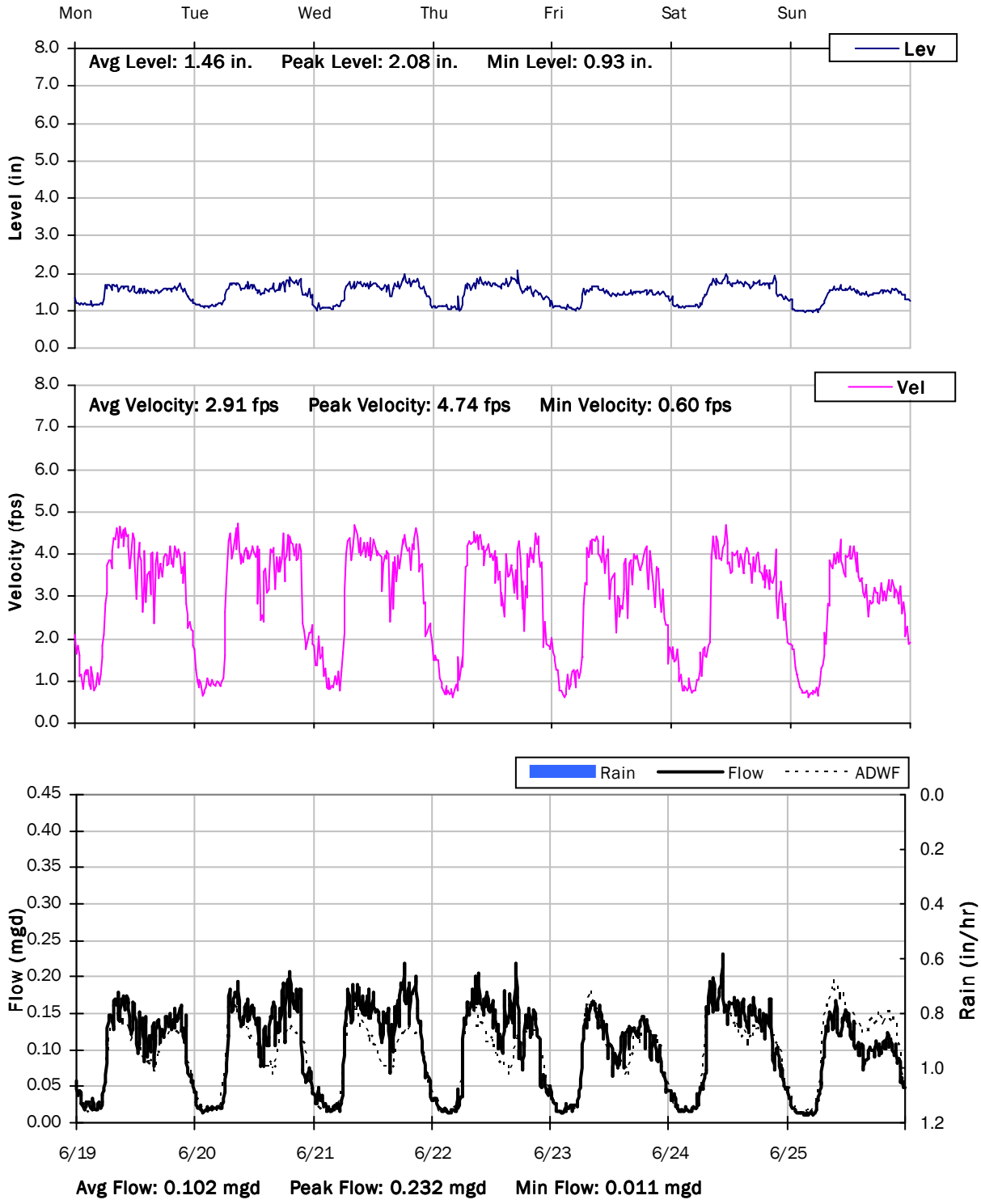
6/12/2017 to 6/19/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

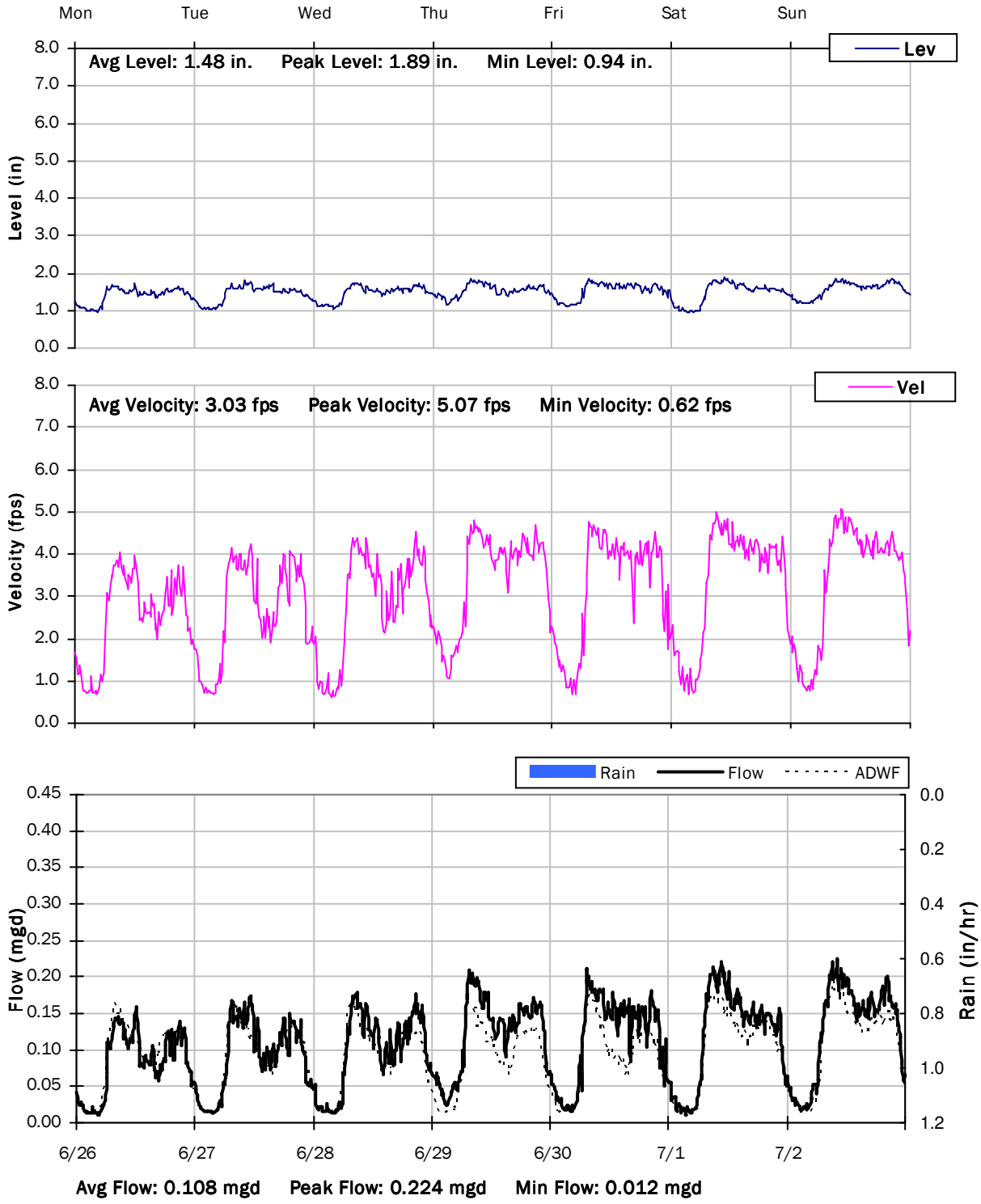
6/19/2017 to 6/26/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

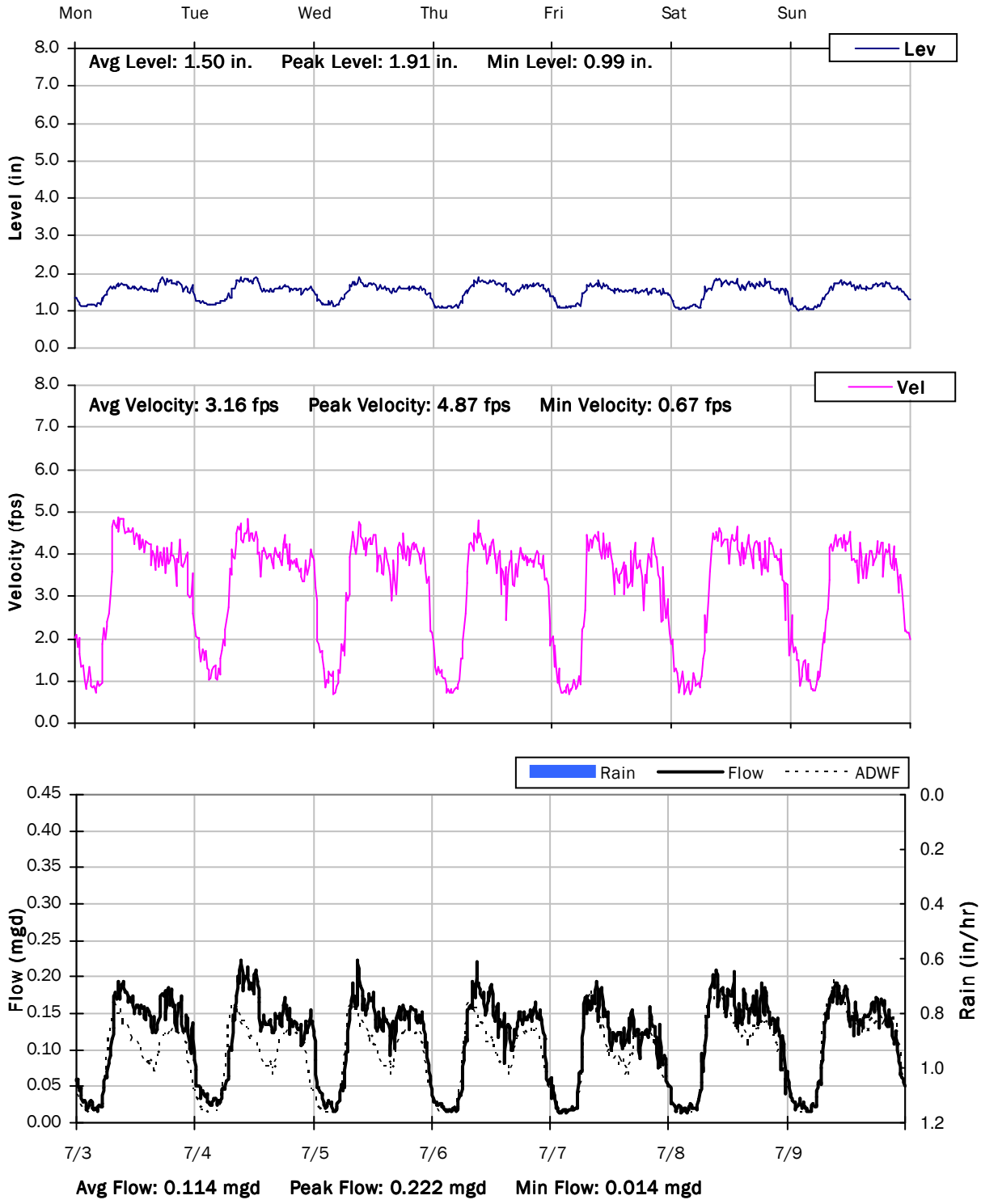
6/26/2017 to 7/3/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

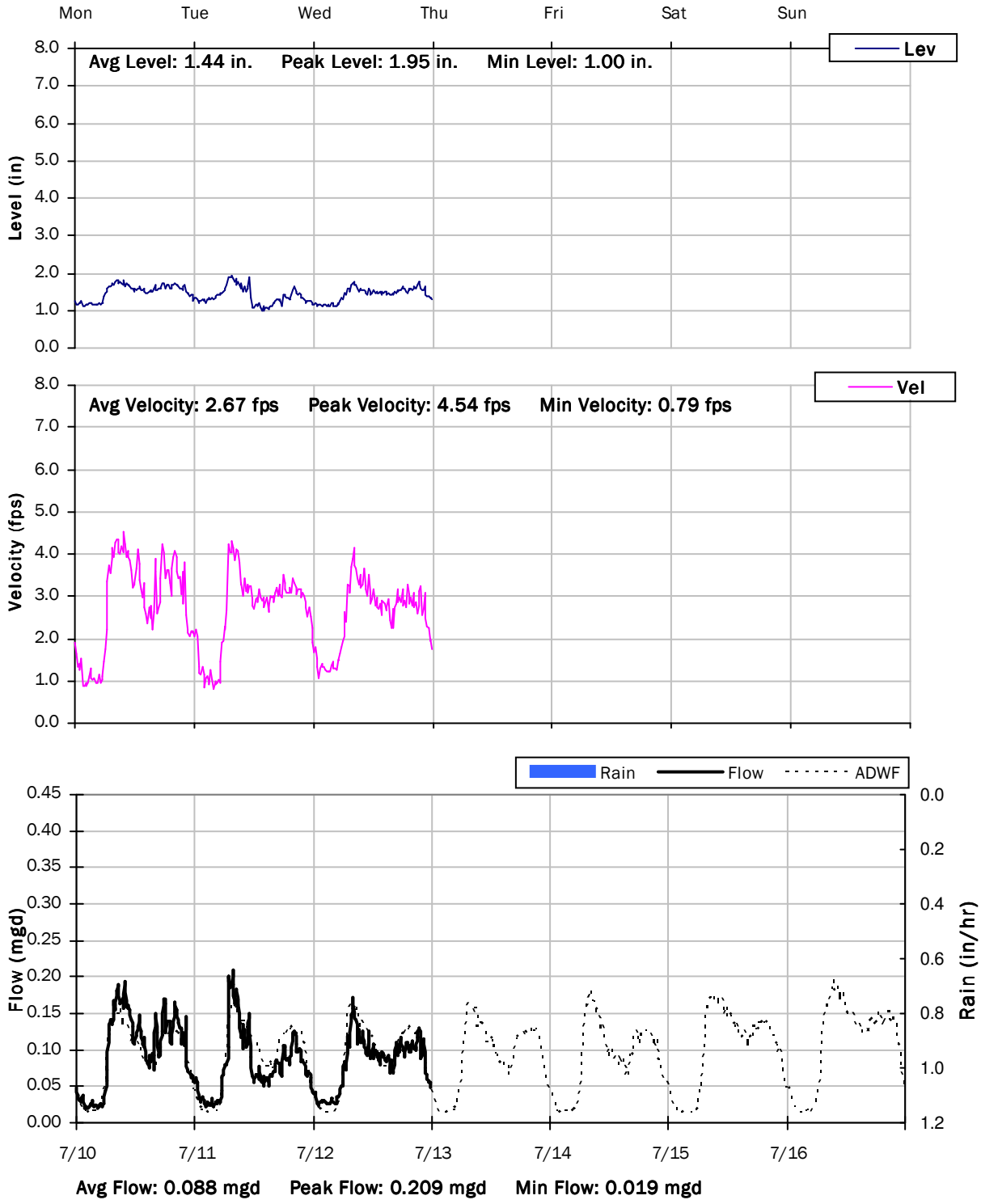
7/3/2017 to 7/10/2017



SITE 14

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



City of Morro Bay

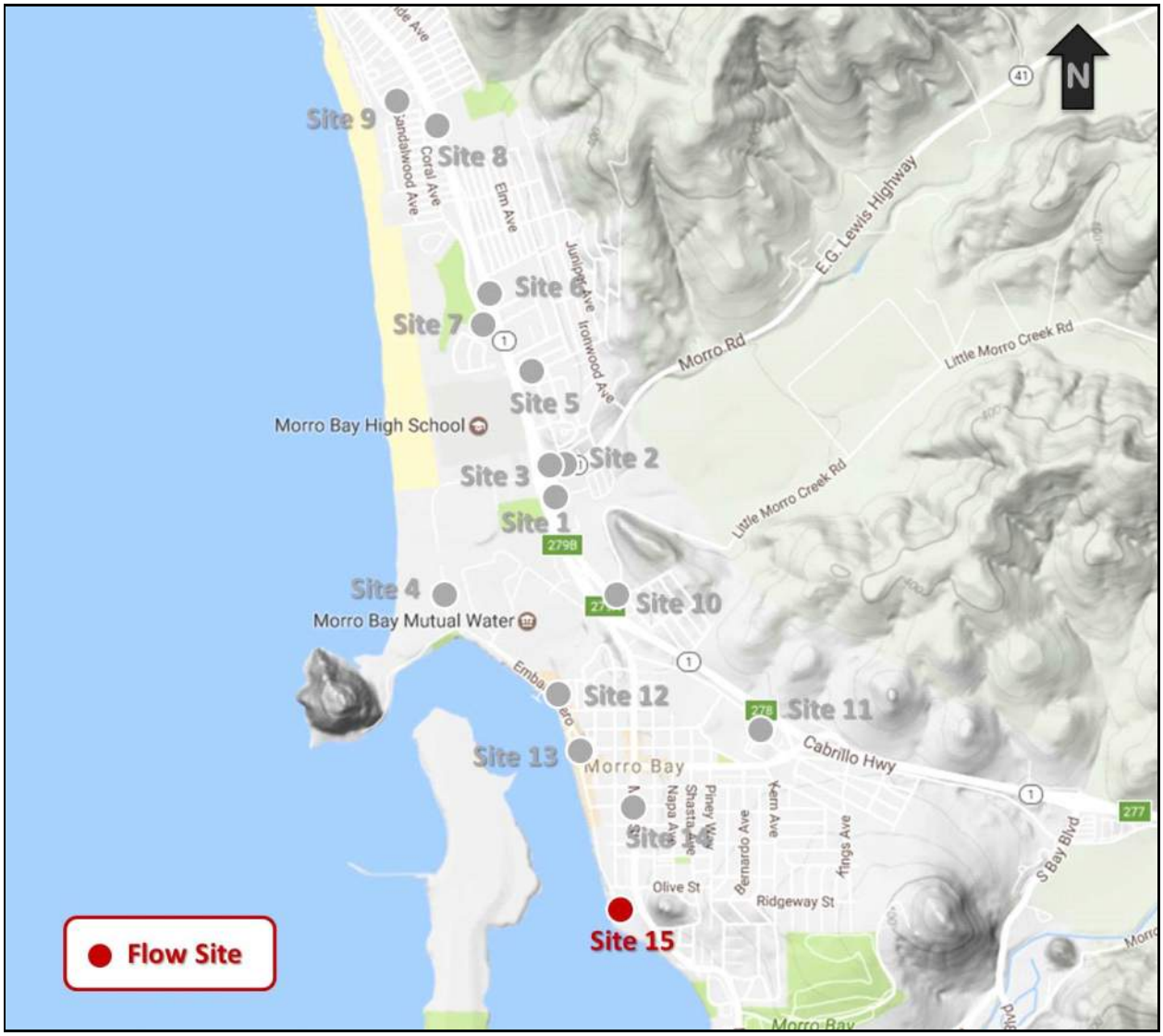
Sanitary Sewer Flow Monitoring

Temporary Monitoring: April 2017 - July 2017

Monitoring Site: Site 15

Location: 301 Embarcadero, ~90 feet north of Morro Bay Launching Facility, meter on south 10" inlet

Data Summary Report



Vicinity Map: Site 15

SITE 15

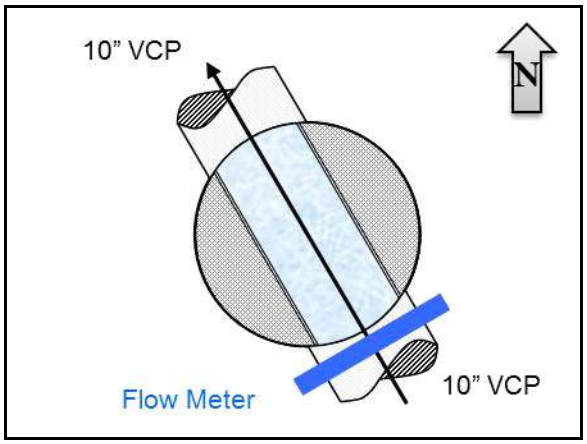
Site Information

Location: 301 Embarcadero, ~90 feet north of Morro Bay Launching Facility, meter on south 10" inlet

Pipe Diameter: 10 inches

ADWF: 0.046 mgd

Peak Measured Flow: 0.208 mgd



SITE 15

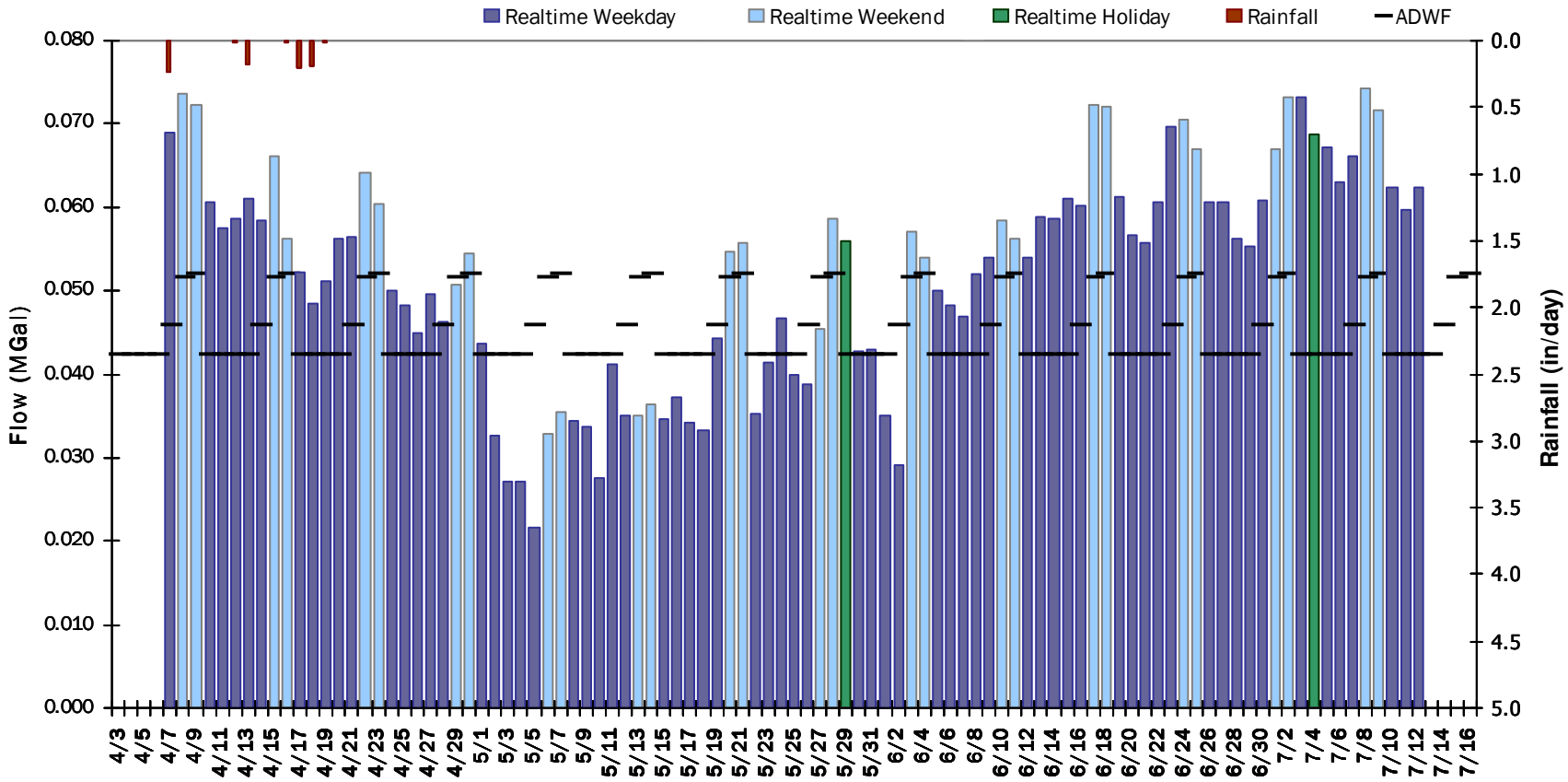
Additional Site Photos

SITE 15

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.053 MGal **Peak Daily Flow: 0.074 MGal** **Min Daily Flow: 0.022 MGal**

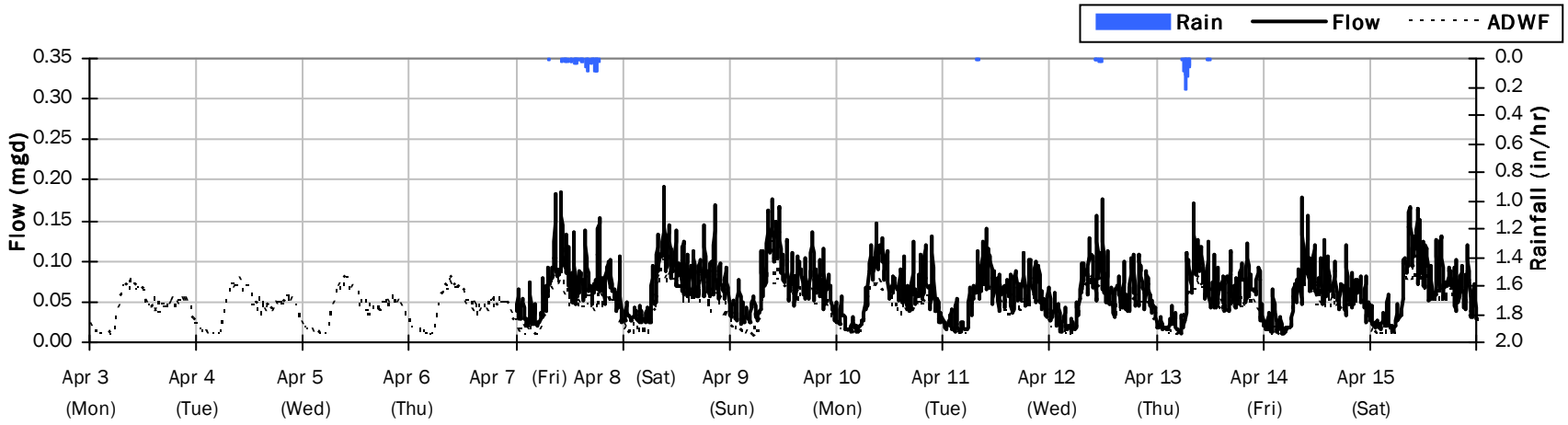
Total Period Rainfall: 0.85 inches

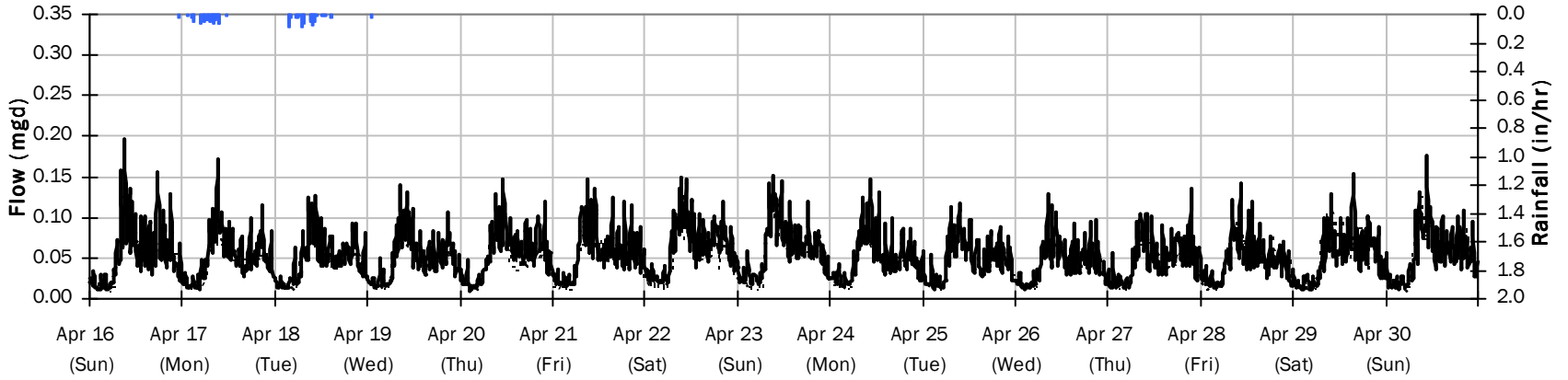


SITE 15

Flow Summary: 4/1/2017 to 4/30/2017

Total Period Rainfall: 0.85 inches **Avg Flow: 0.057 mgd** **Peak Flow: 0.196 mgd** **Min Flow: 0.009 mgd**

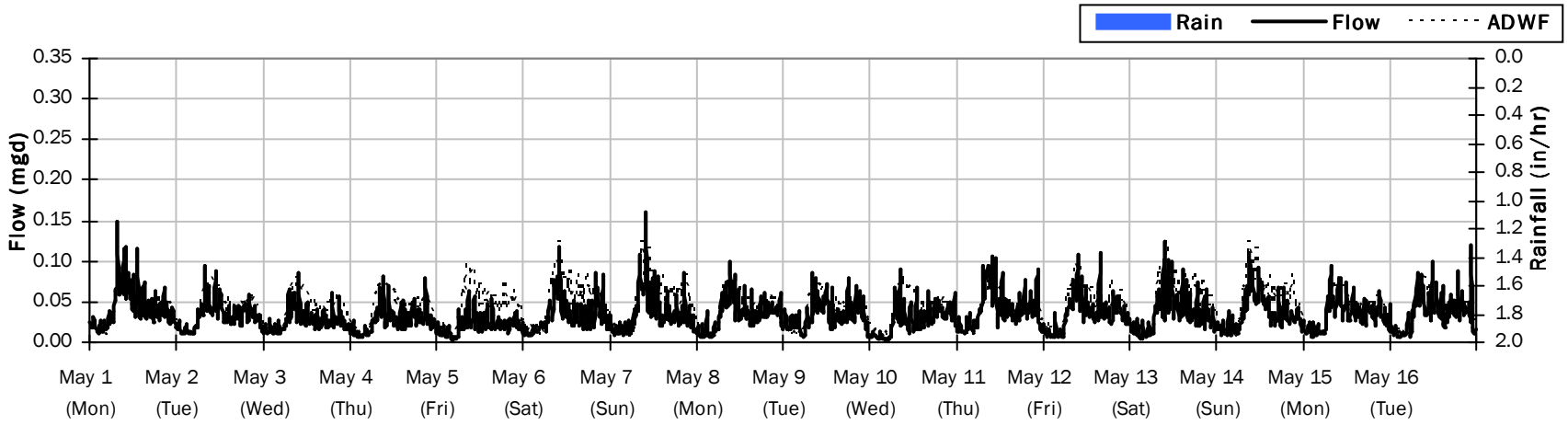


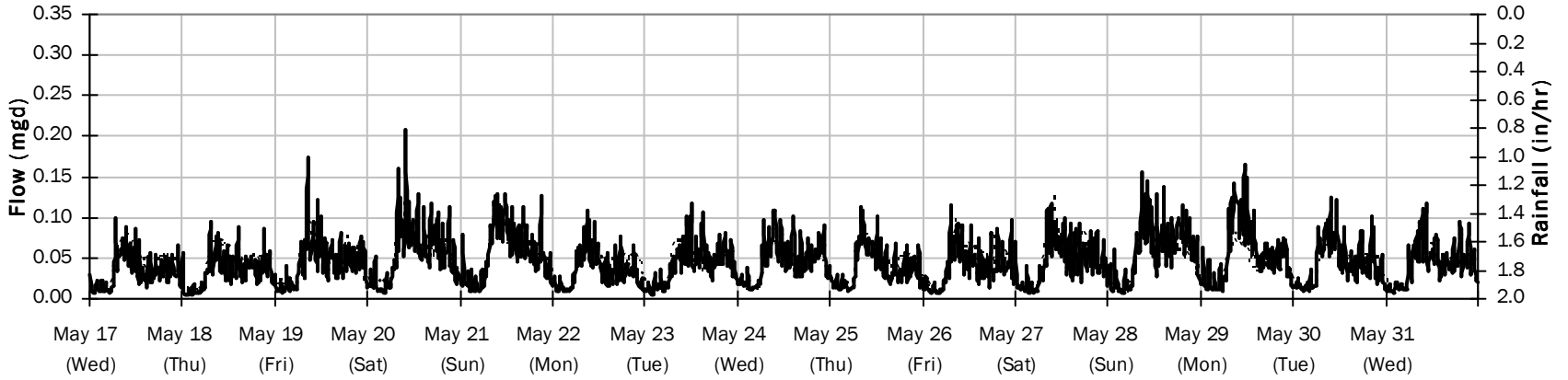


SITE 15

Flow Summary: 5/1/2017 to 5/31/2017

Total Period Rainfall: 0.00 inches **Avg Flow: 0.039 mgd** **Peak Flow: 0.208 mgd** **Min Flow: 0.003 mgd**

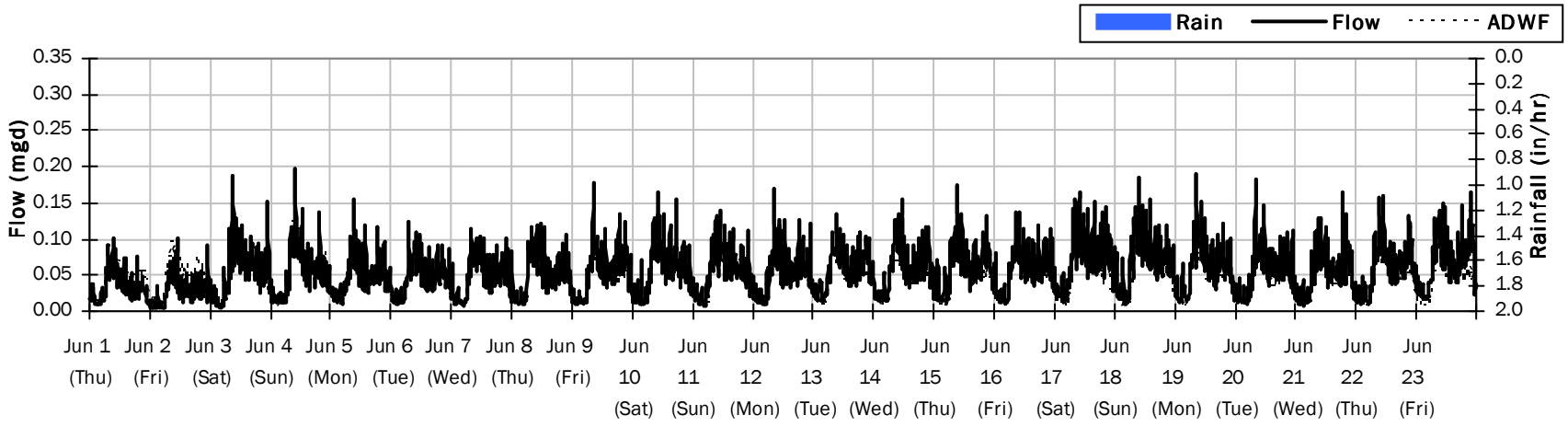


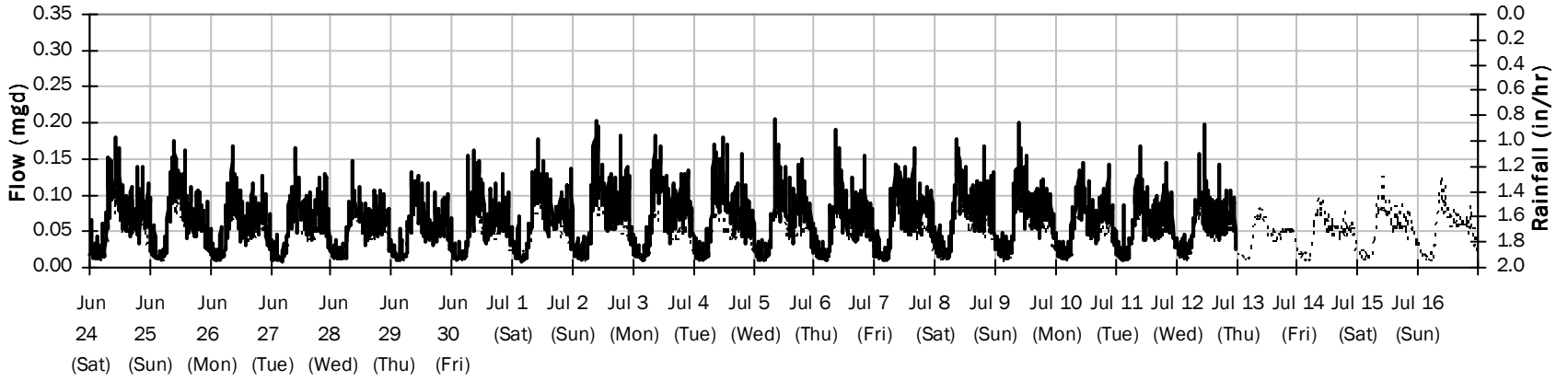


SITE 15

Flow Summary: 6/1/2017 to 7/16/2017

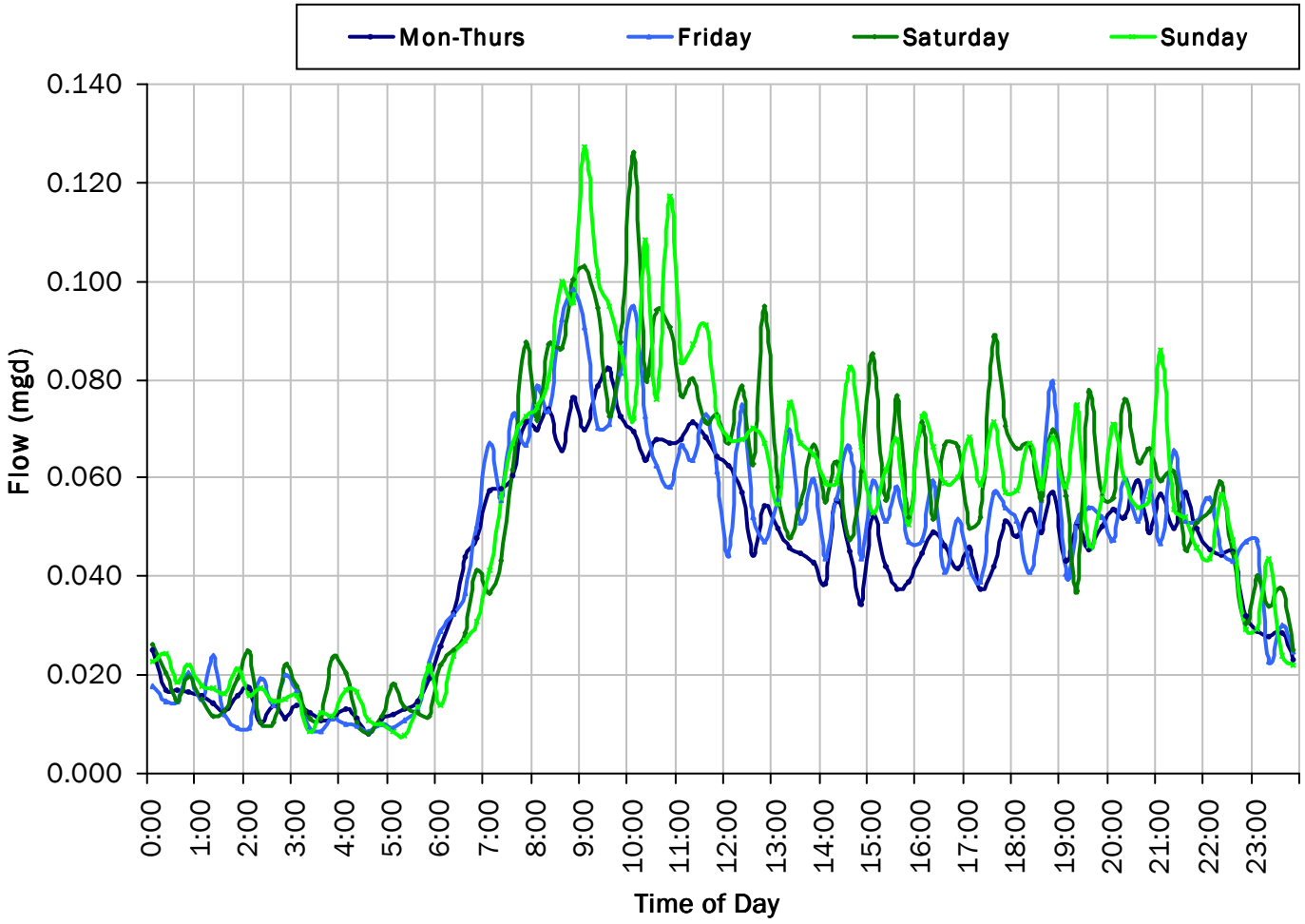
Total Period Rainfall: 0.00 inches **Avg Flow: 0.060 mgd** **Peak Flow: 0.206 mgd** **Min Flow: 0.004 mgd**





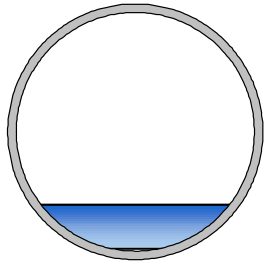
SITE 15

Average Dry Weather Flow Hydrographs



ADWF:

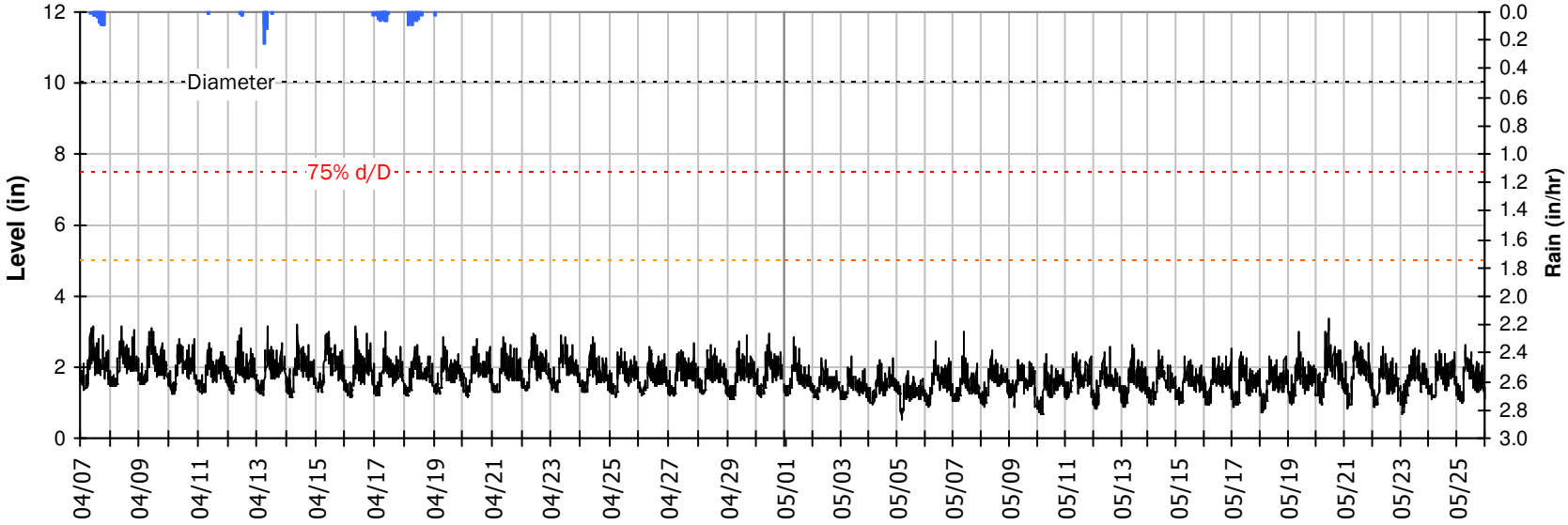
0.046 *mgd*

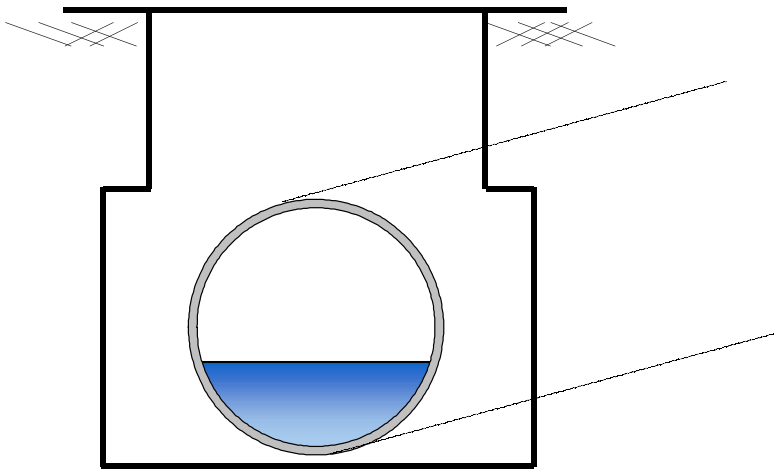


SITE 15

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



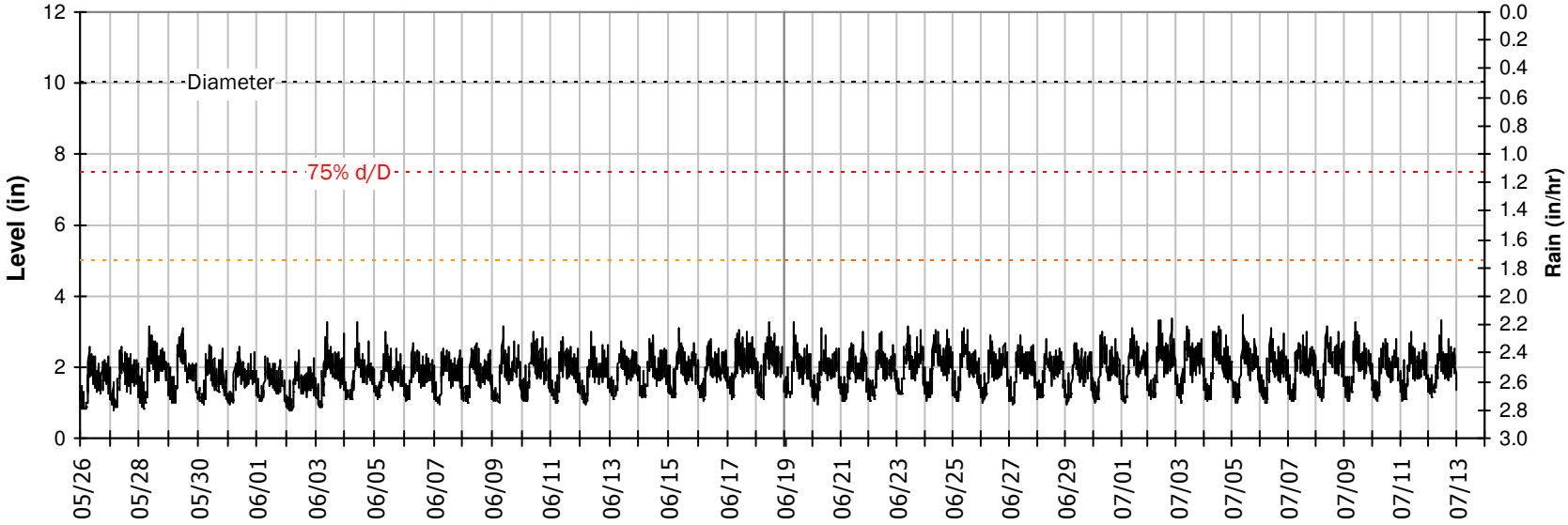


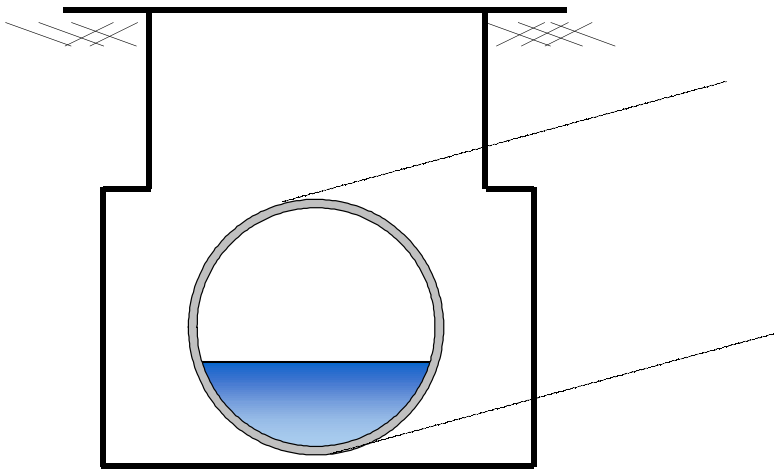
Pipe Diameter:	10	*inches*
Peak Measured Level:	3.5	*inches*
Peak d/D Ratio:	0.35	

SITE 15

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period



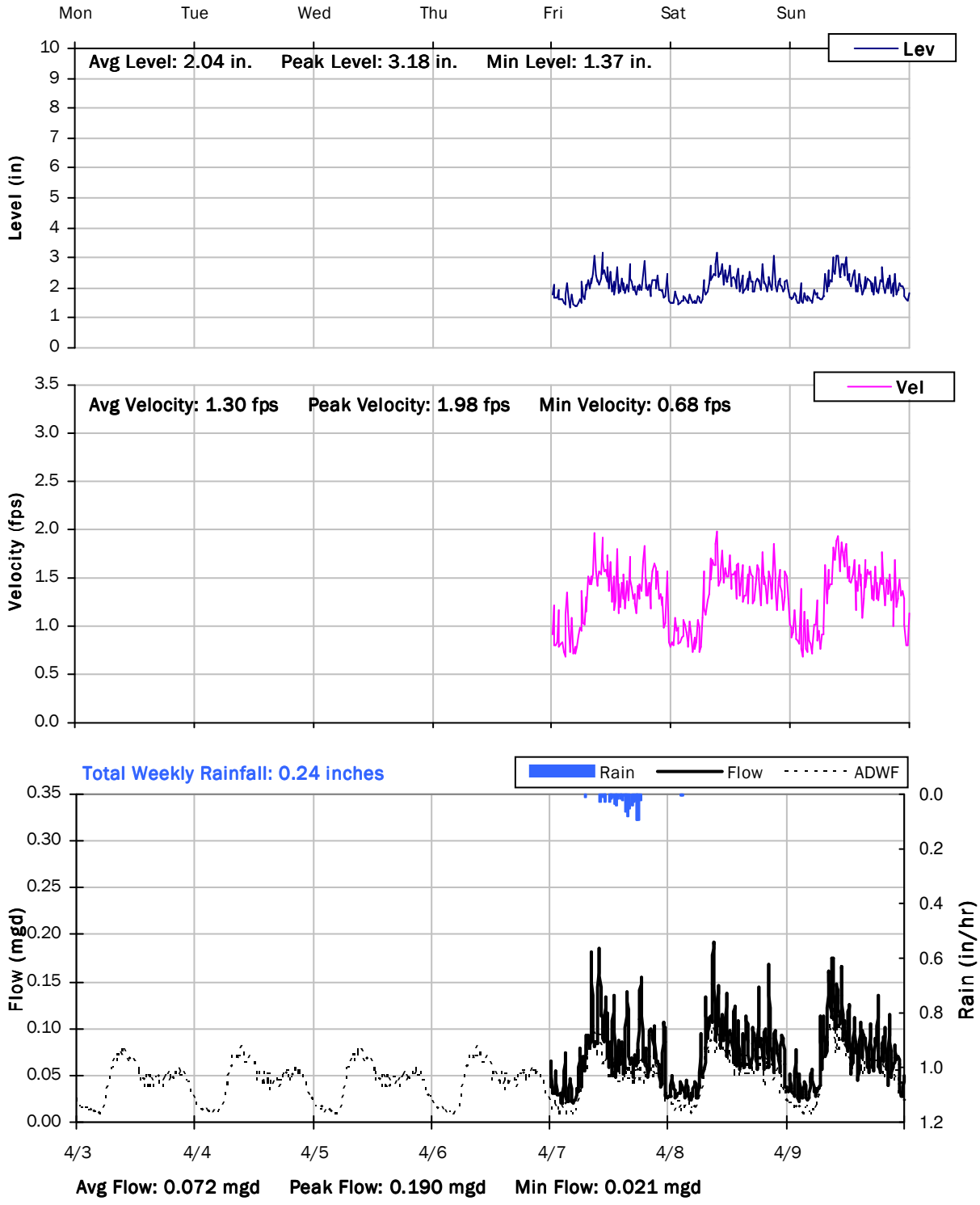


Pipe Diameter:	10	*inches*
Peak Measured Level:	3.5	*inches*
Peak d/D Ratio:	0.35	

SITE 15

Weekly Level, Velocity and Flow Hydrographs

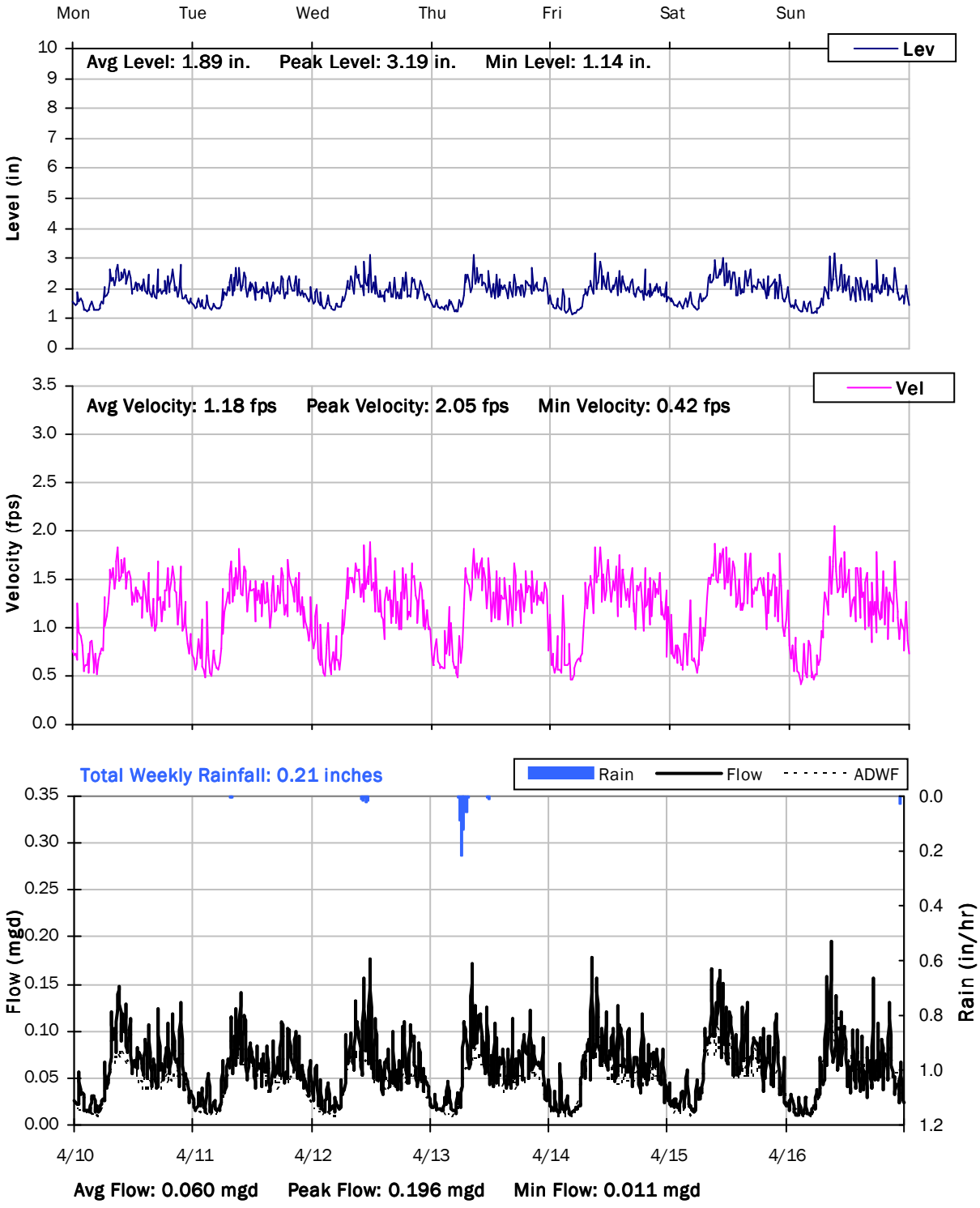
4/3/2017 to 4/10/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

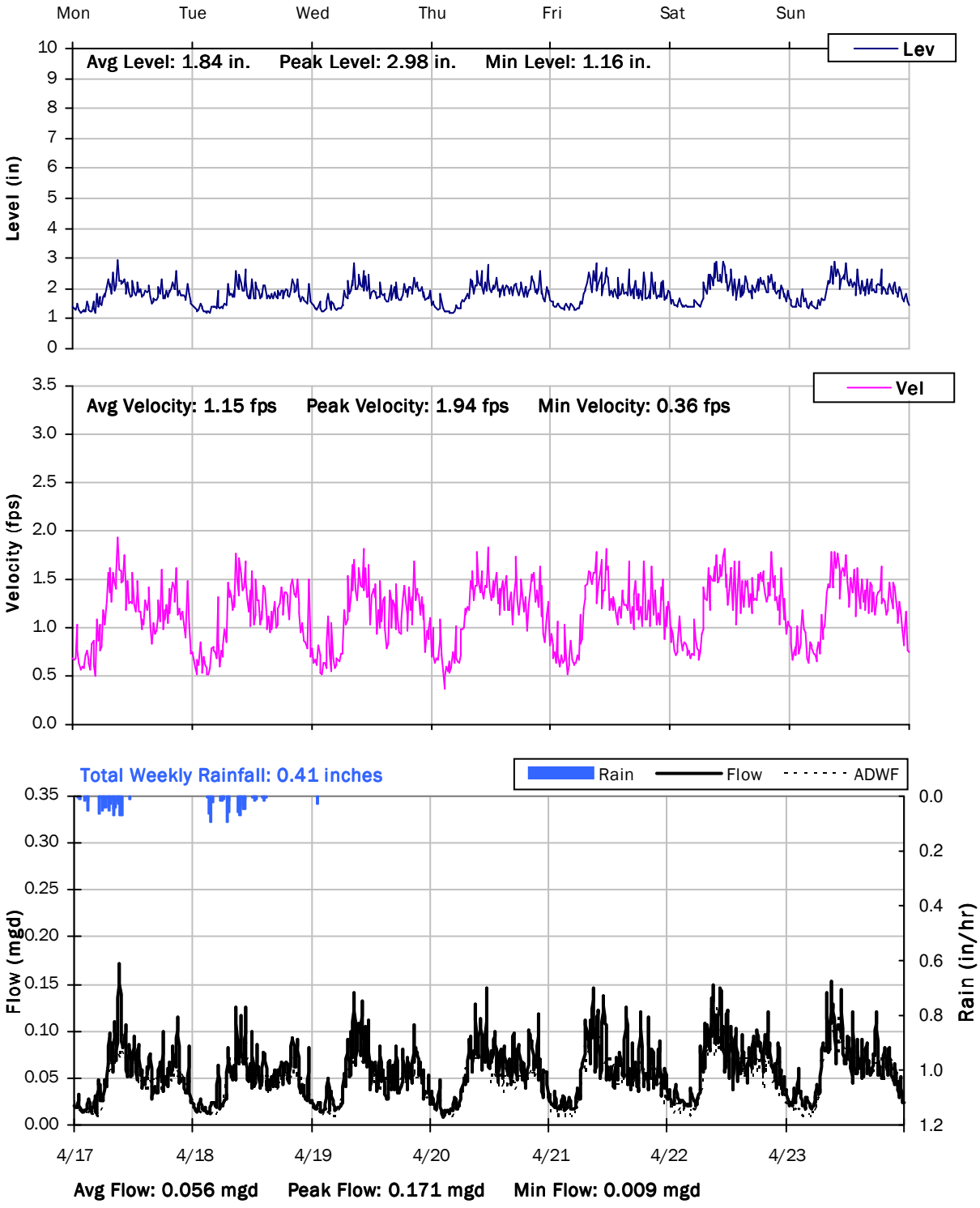
4/10/2017 to 4/17/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

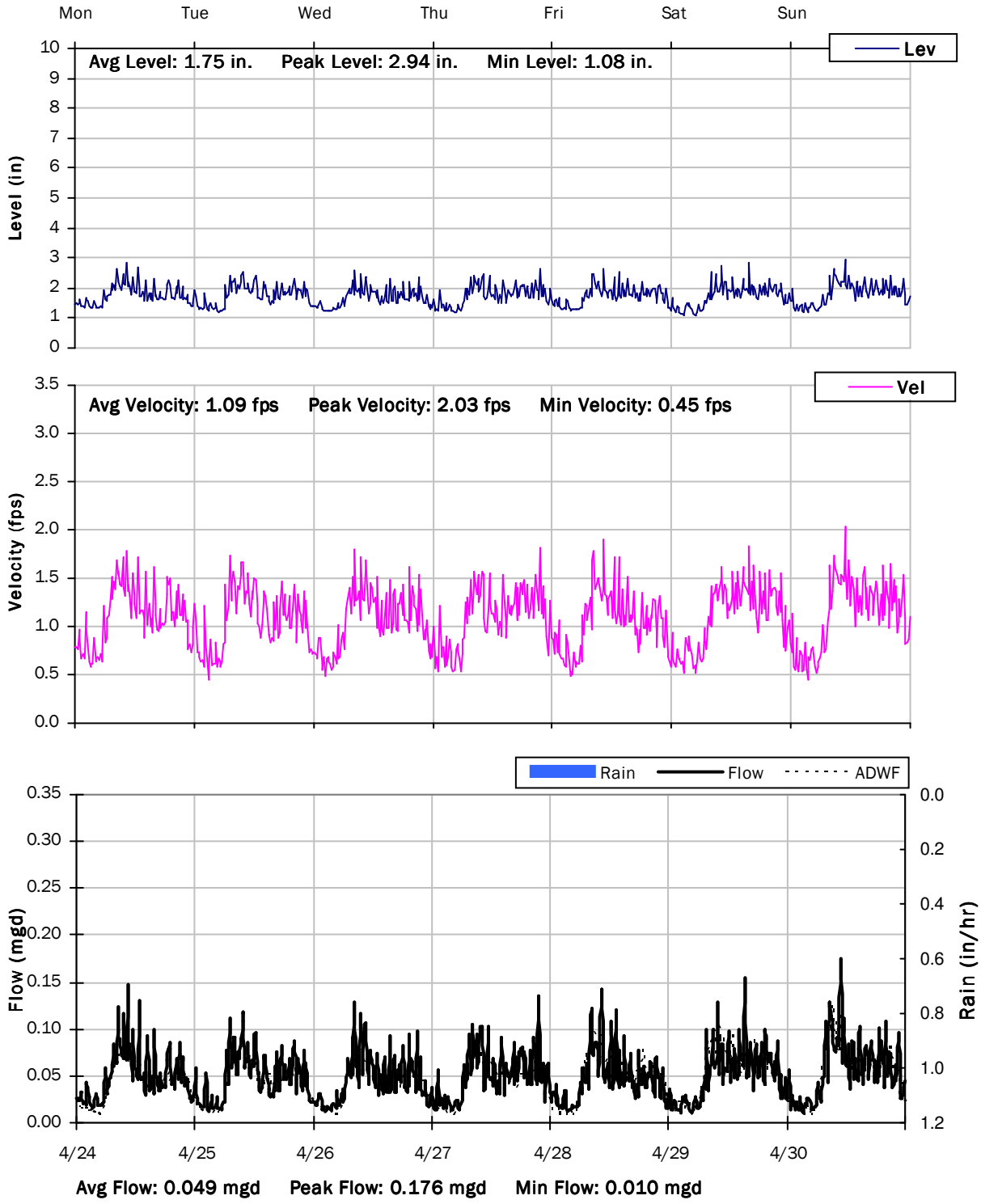
4/17/2017 to 4/24/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

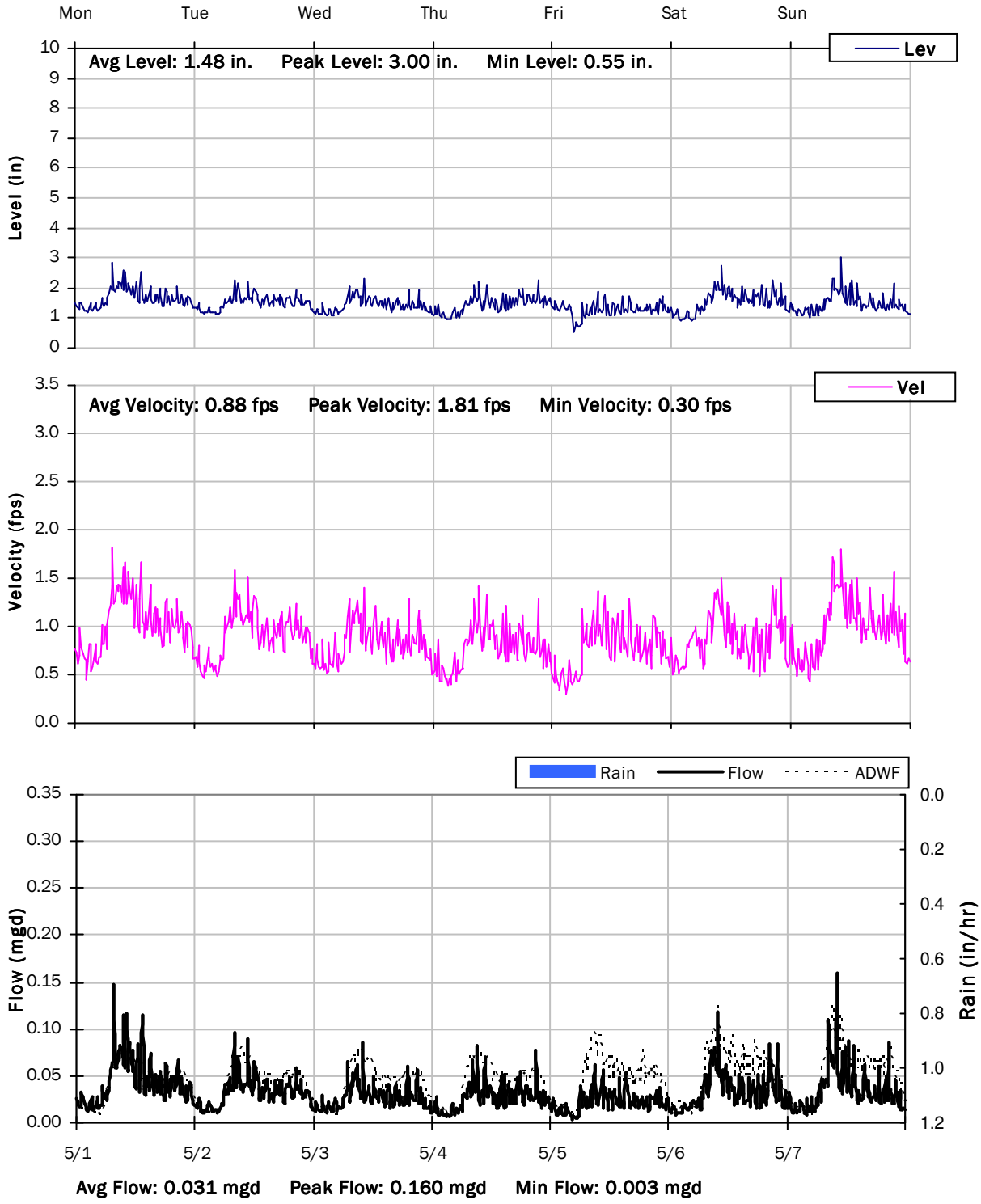
4/24/2017 to 5/1/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

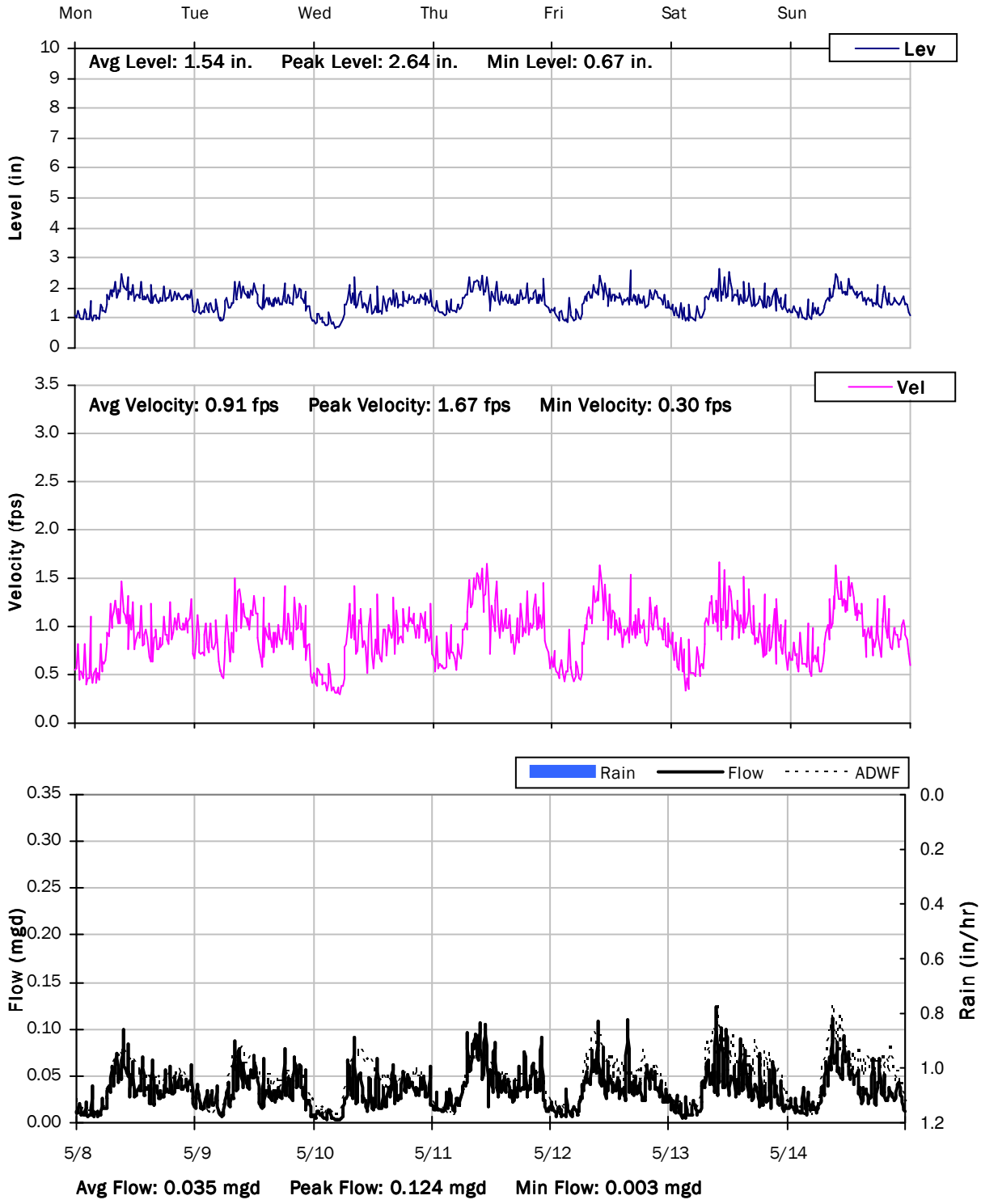
5/1/2017 to 5/8/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

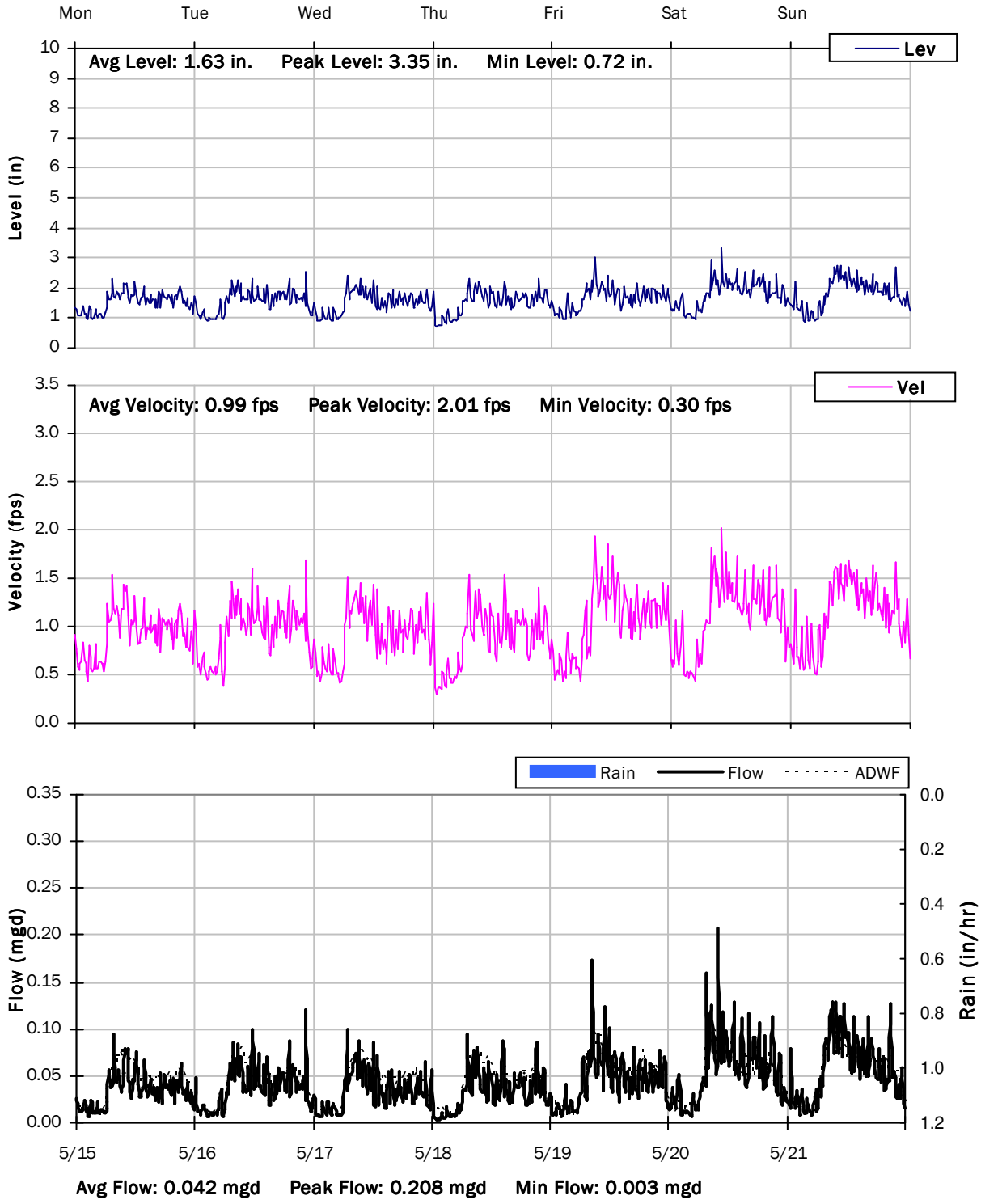
5/8/2017 to 5/15/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

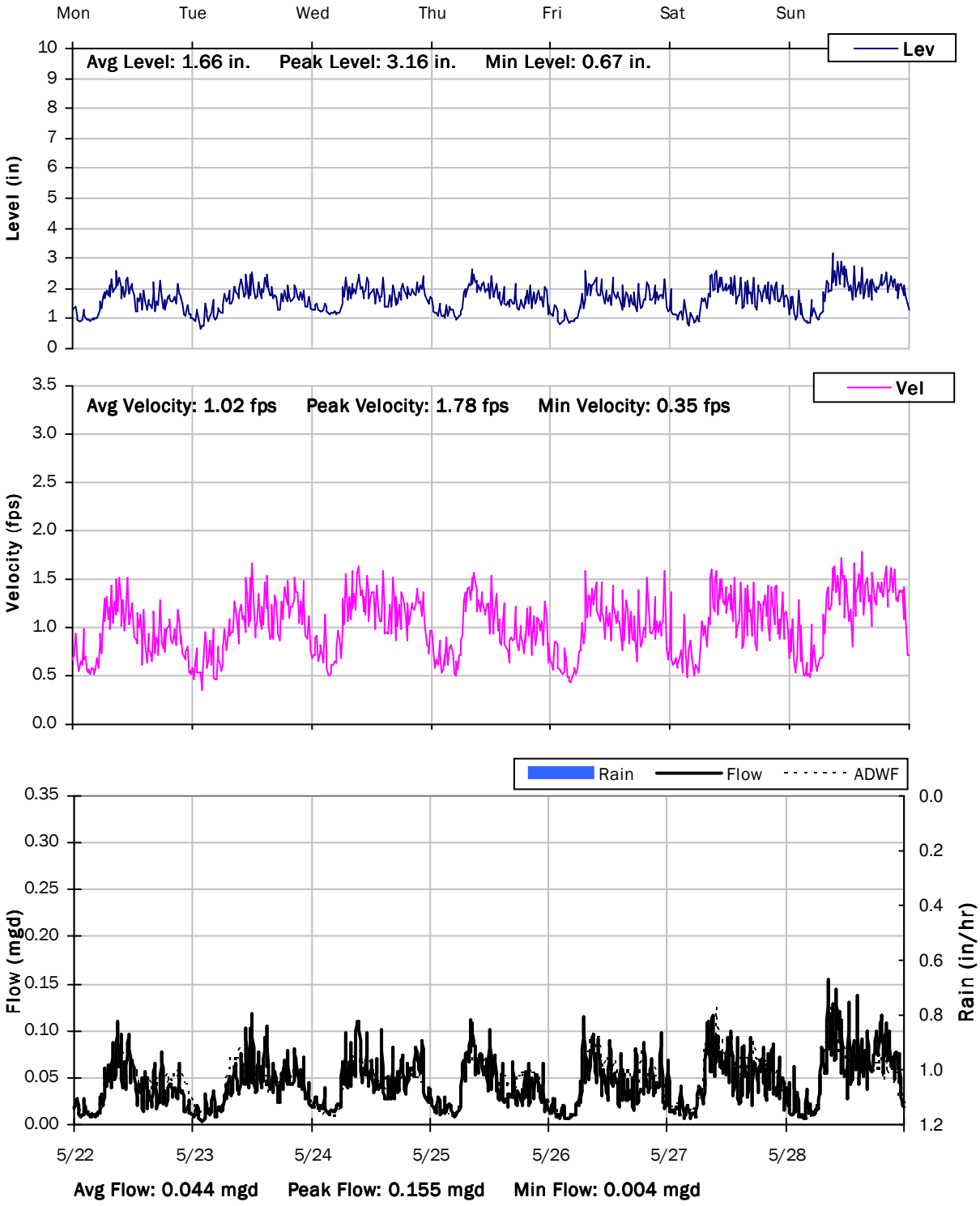
5/15/2017 to 5/22/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

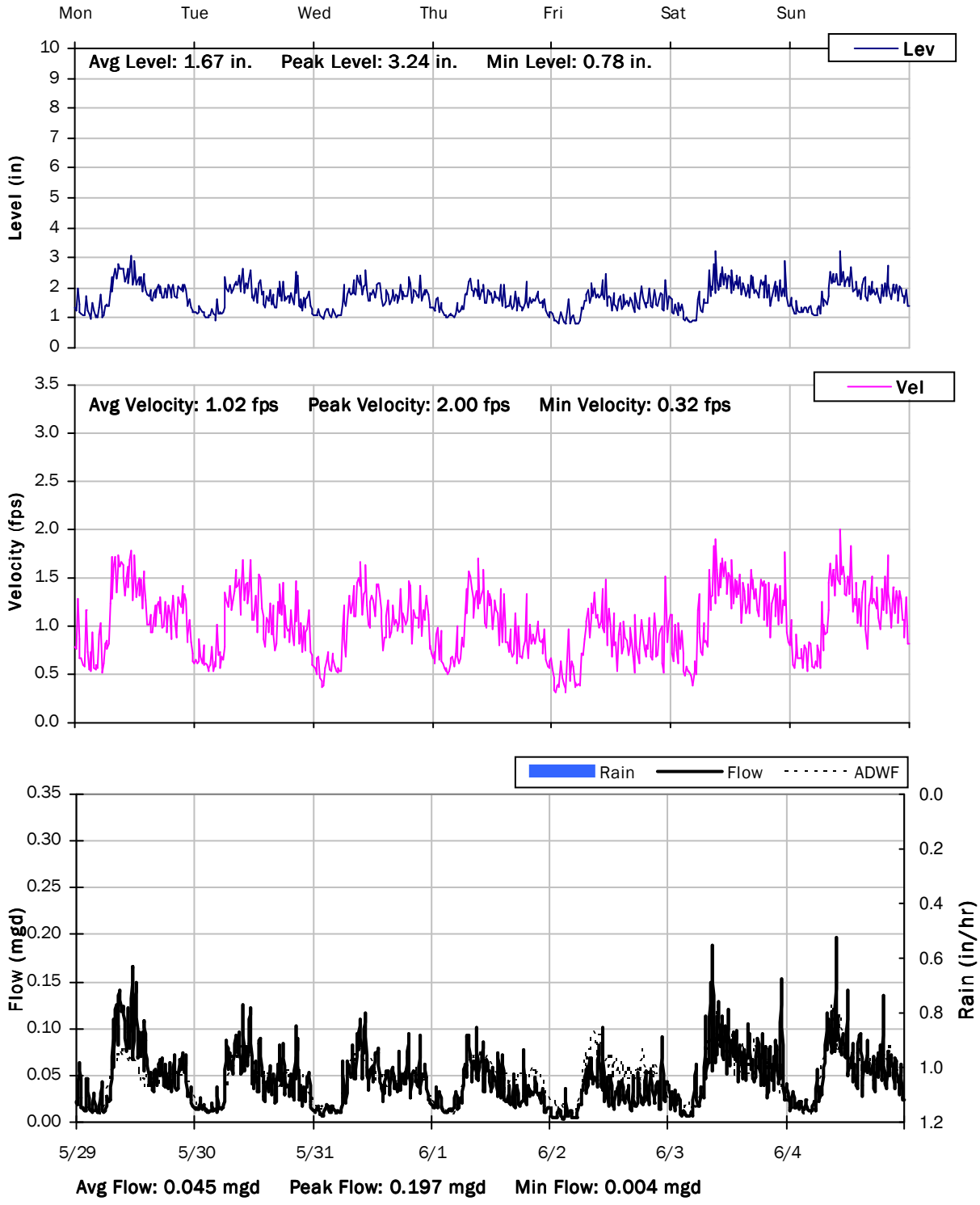
5/22/2017 to 5/29/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

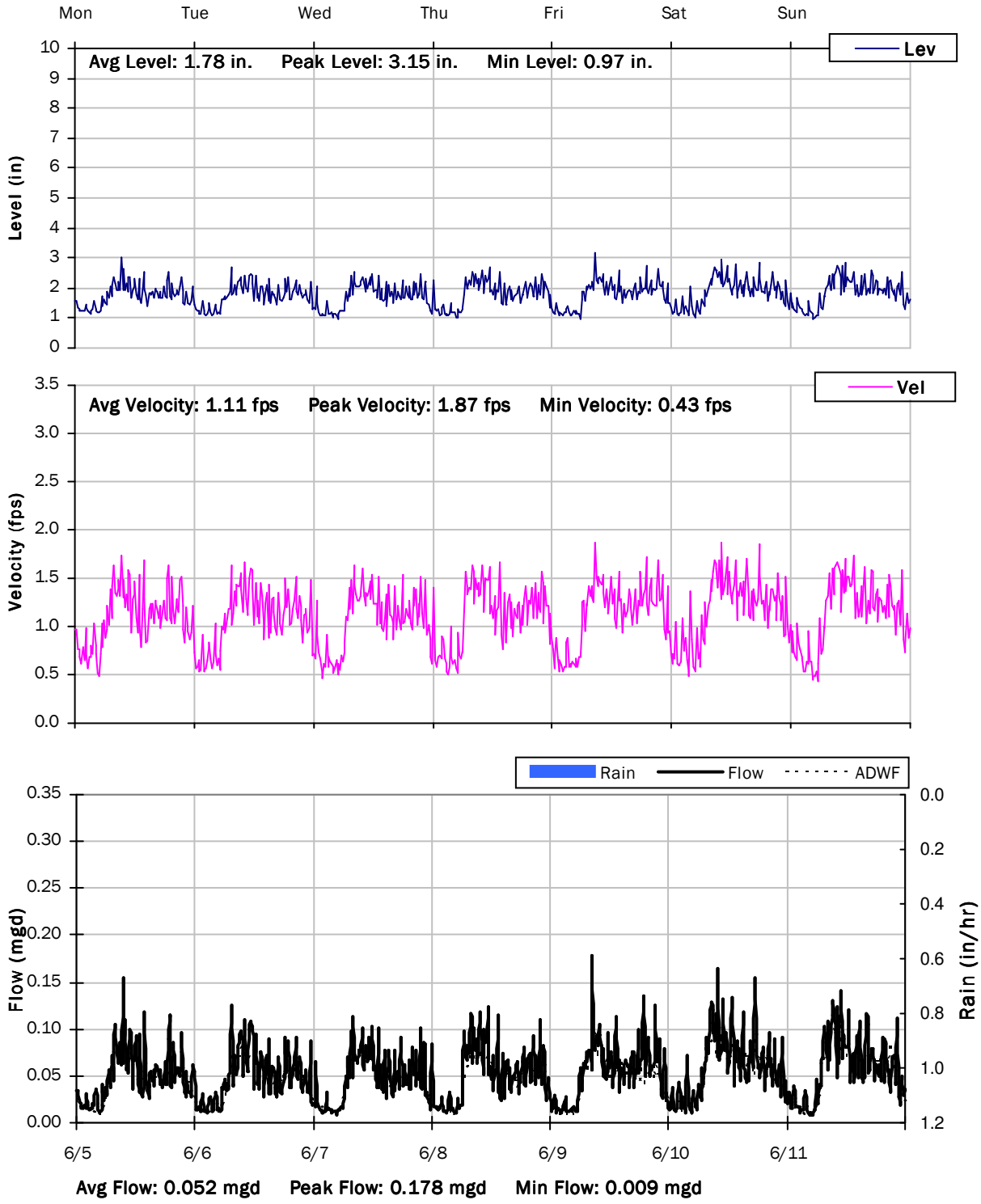
5/29/2017 to 6/5/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

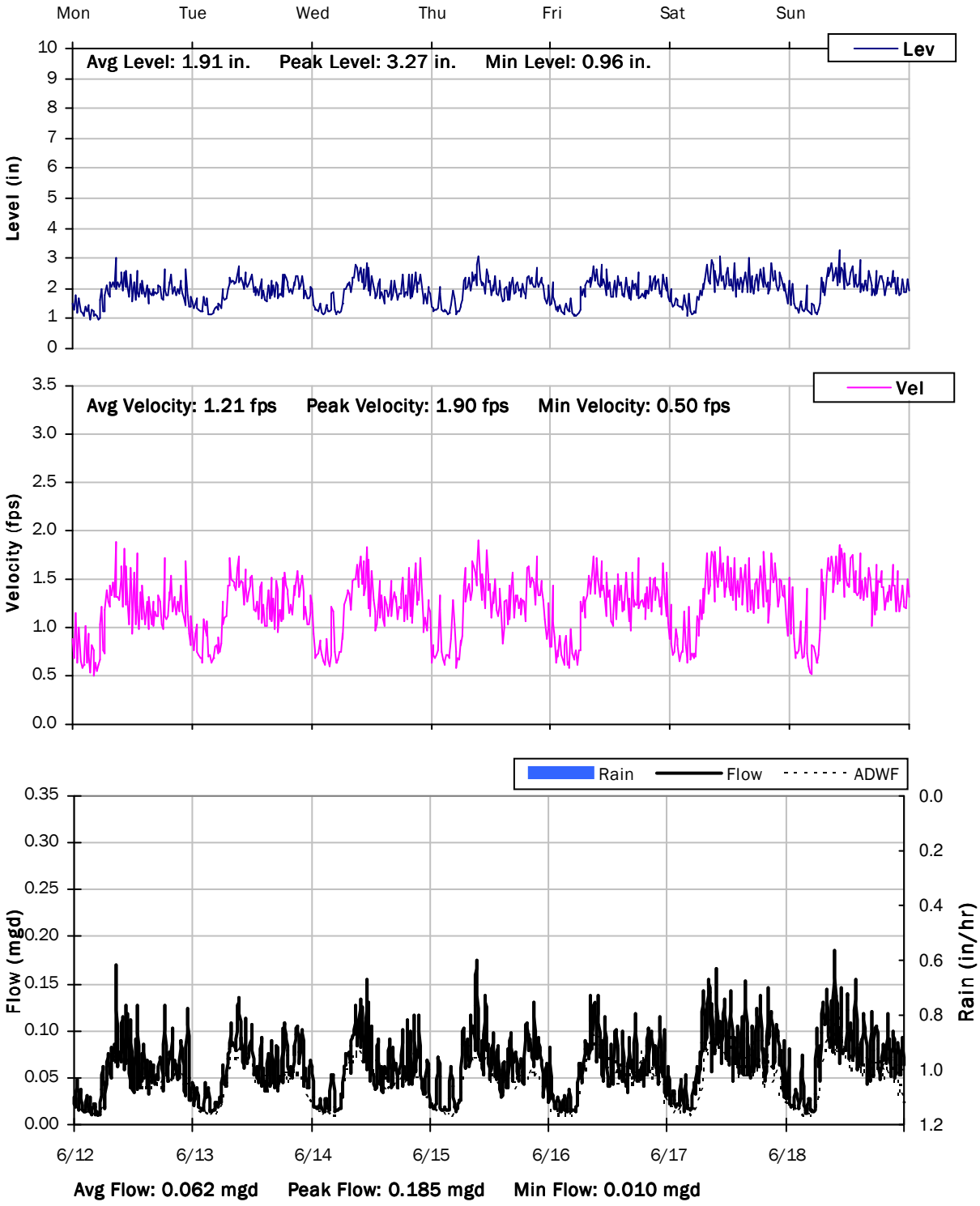
6/5/2017 to 6/12/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

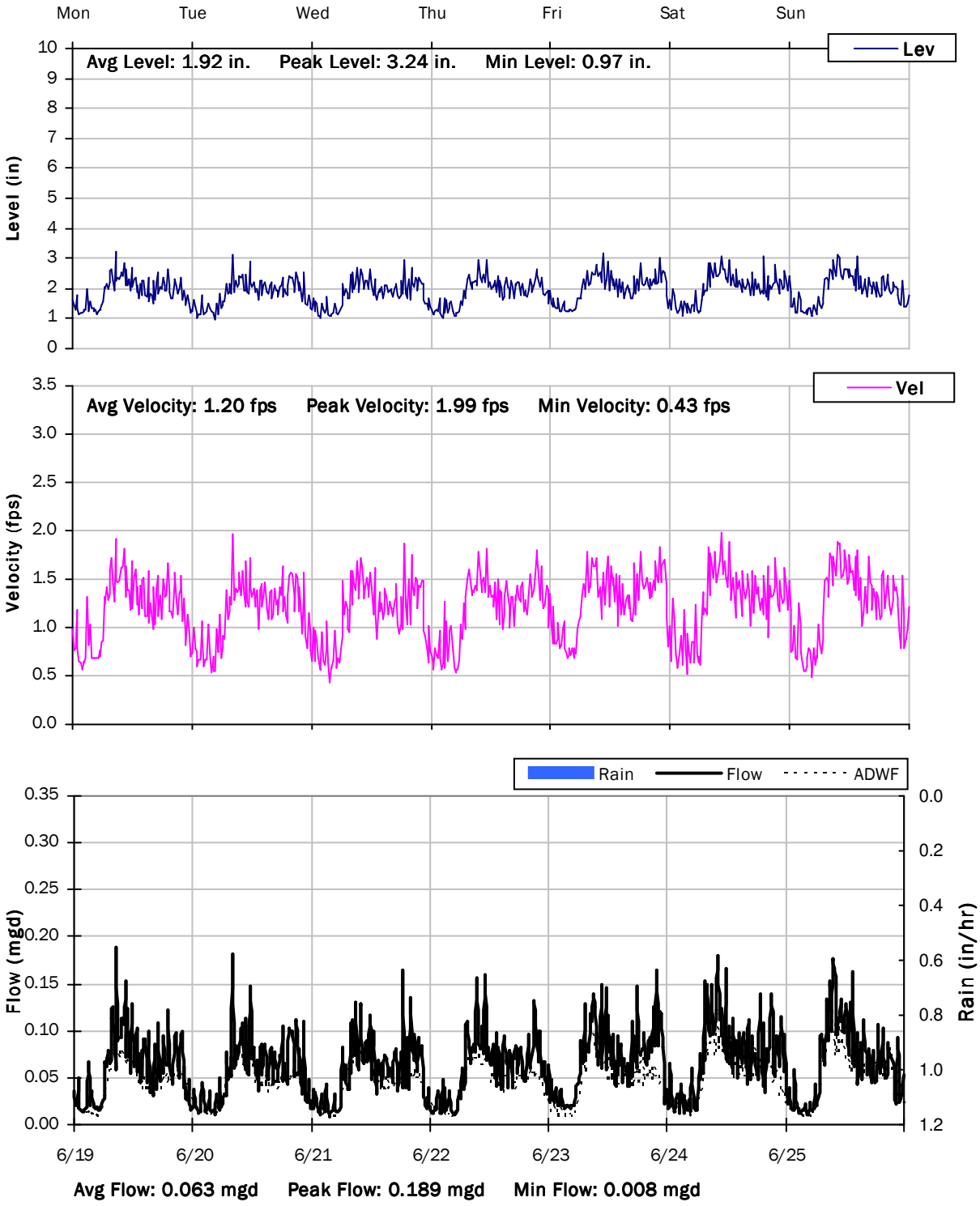
6/12/2017 to 6/19/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

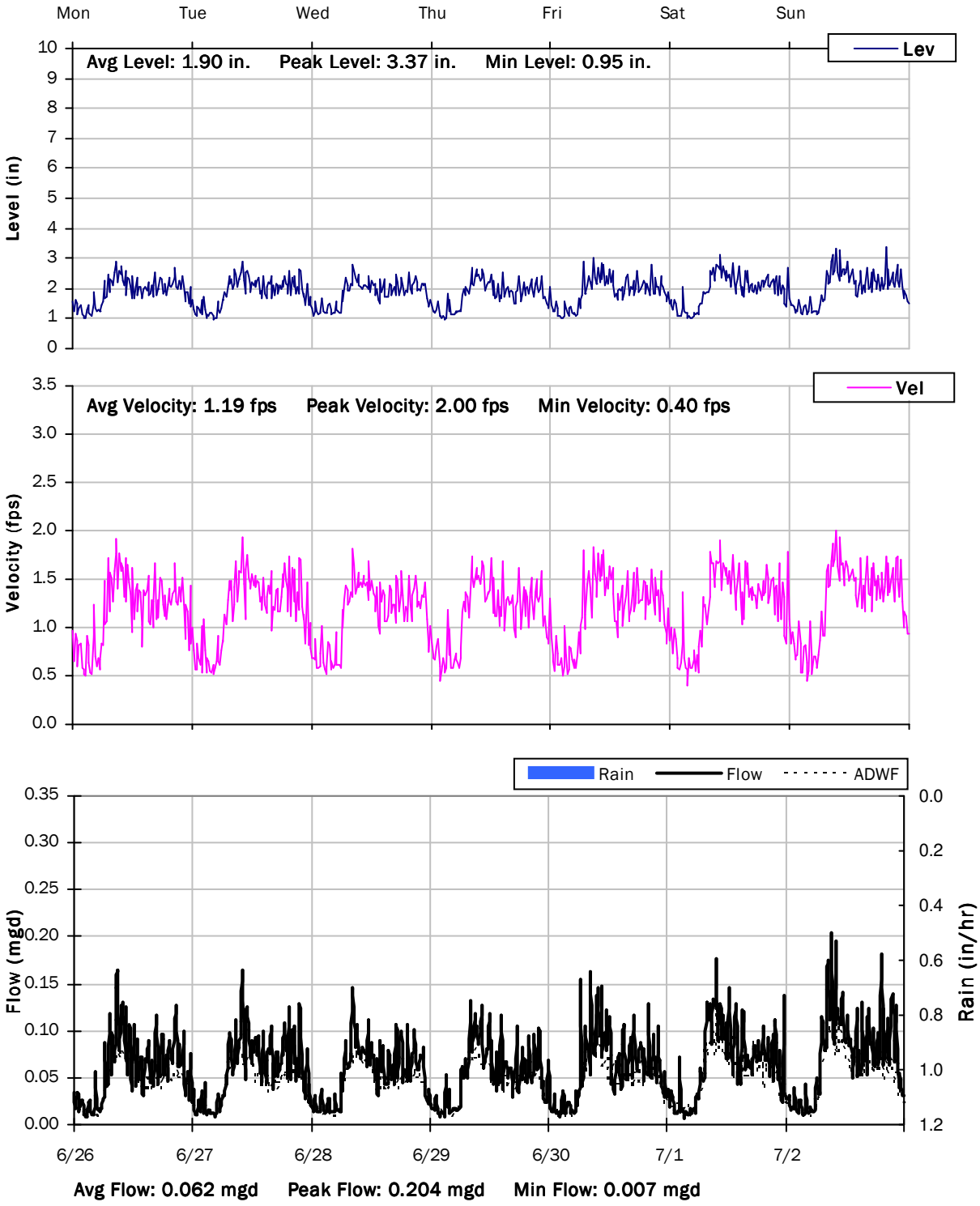
6/19/2017 to 6/26/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

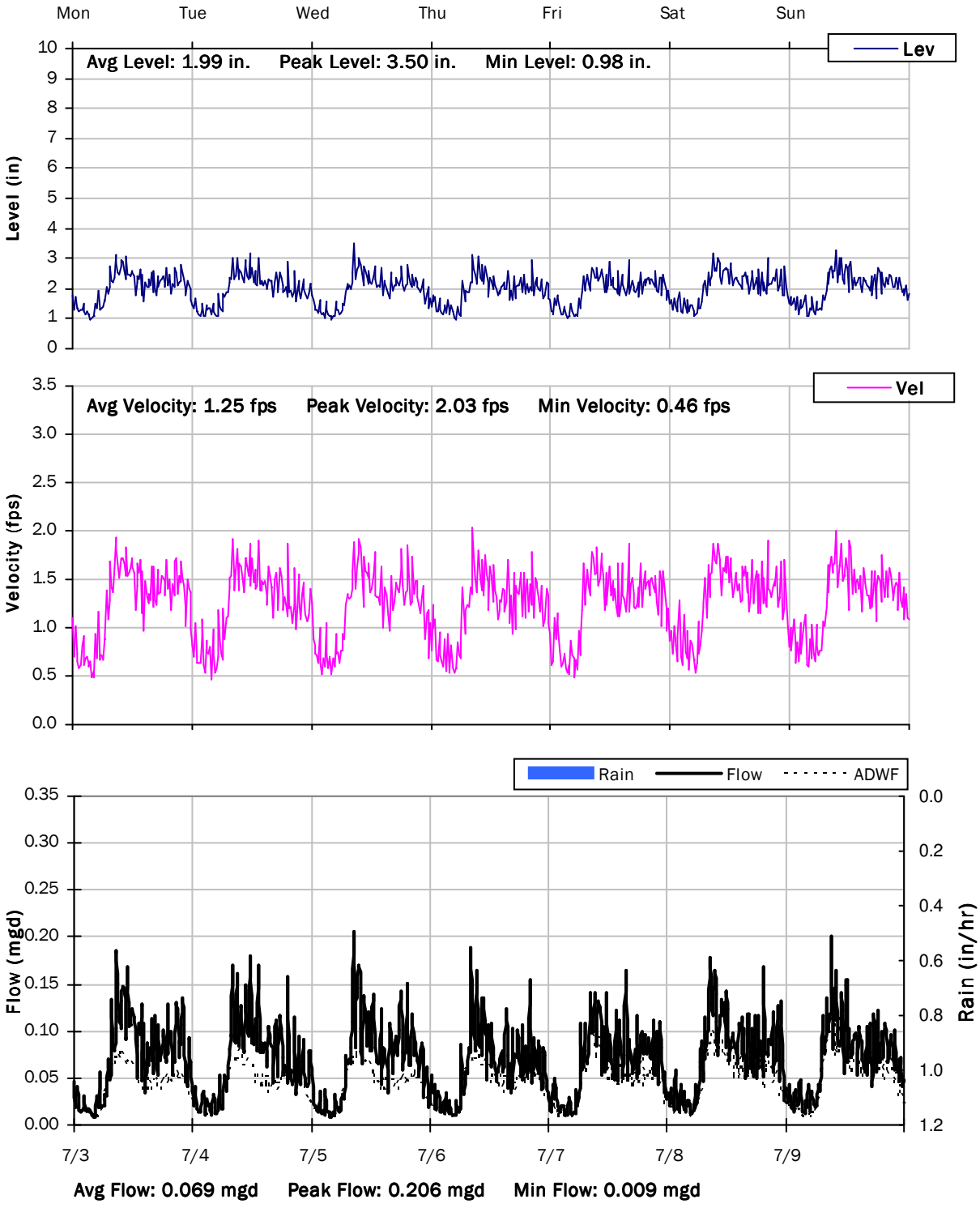
6/26/2017 to 7/3/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

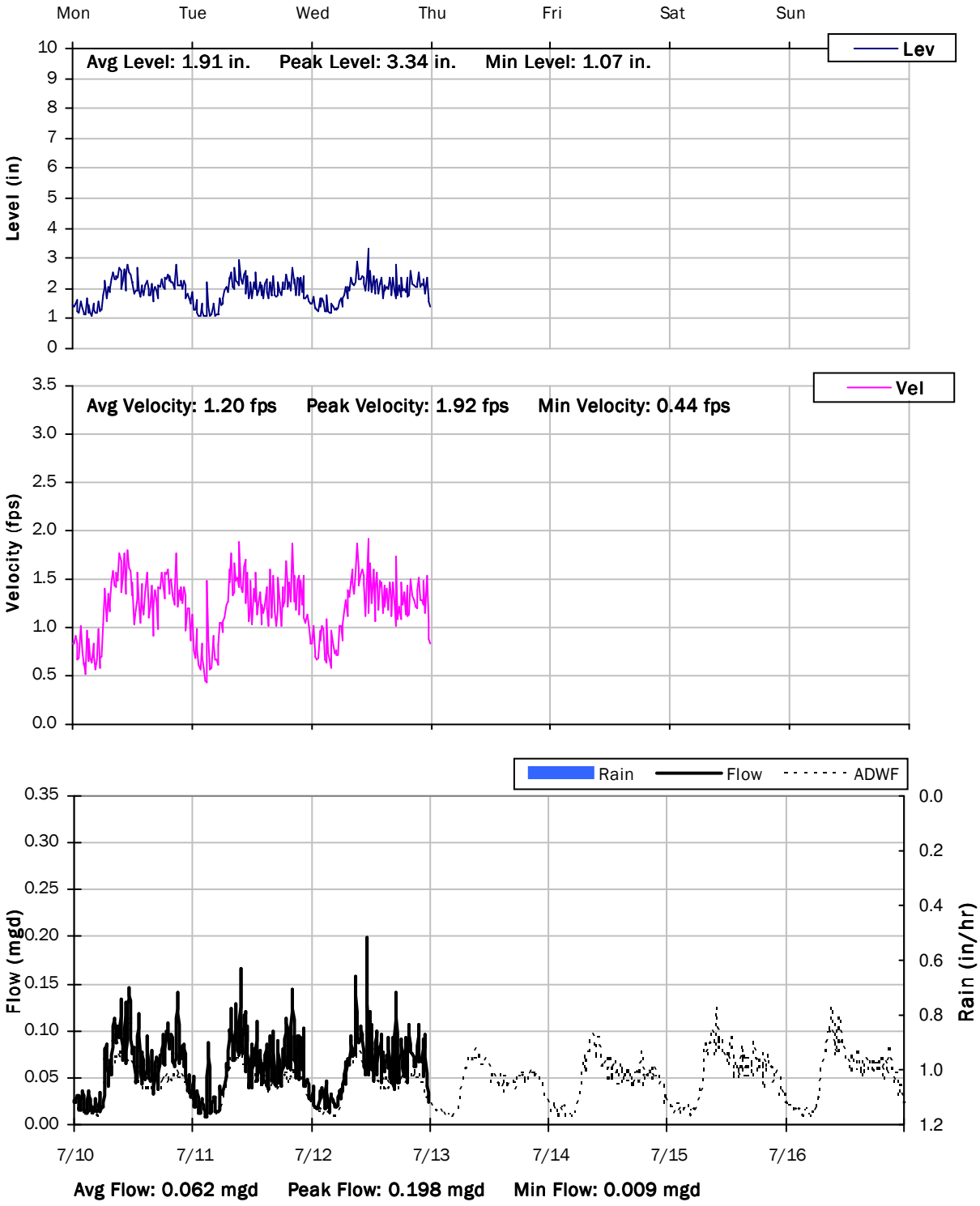
7/3/2017 to 7/10/2017



SITE 15

Weekly Level, Velocity and Flow Hydrographs

7/10/2017 to 7/17/2017



V&A Project No. 17-0050

V&A
consulting engineers
1000 Broadway, Suite 230
Oakland, CA 94607
510.903.6600
510.903.6601, Fax

Appendix G

MORRO BAY EXISTING DESALINATION FACILITY EVALUATION

Engineers...Working Wonders With Water®

710 West Pinedale Avenue, Fresno, California 93711
P. 559.436.6616 F. 559.436.1191

July 26, 2018

Rob Livick
Public Works Director/City Engineer
City of Morro Bay
955 Shasta Avenue
Morro Bay, CA 93442

Subject: Morro Bay Existing Desalination Facility Evaluation

Dear Rob Livick:

DESALINATION BACKGROUND

Since the early 1990s, the City of Morro Bay ("the City") has operated a desalination facility that provides part of the City's water supply. The facility is located within the City's corporation yard near its wastewater treatment facility at 170 Atascadero Road. It obtains its source water from several nearby brackish water wells and five seawater wells located along Embarcadero Road and the Harbor Walk pathway in the City. It discharges effluent to the channel that the now-retired Morro Bay Power Plant formerly used to discharge its power plant cooling water flows.

The facility is able to produce up to the following flows:

- Brackish water: 581 acre-feet per year from the brackish water wells (i.e., an instantaneous treated water flow of 800 gpm from two brackish RO trains), and
- Seawater: up to 645 acre-feet from its seawater wells (i.e., an instantaneous treated water flow of 400 gpm from four RO trains).

Since the late 1990s, the facility has relied solely on source water provided by the brackish water wells, due in part to its higher recovery rate (about 75% compared to a 50% rate for seawater). In 2007, the City determined that the facility would require modifications to more effectively treat the relatively high nitrate levels in the brackish water supply and would also benefit from additional modifications that would allow it to more effectively treat both brackish water and seawater. In 2009, it expanded its brackish water treatment train, but together with the proposed modifications to the seawater treatment system, the proposed modifications could simultaneously treat both brackish water and seawater to produce up to 1,226 acre-feet per year, if necessary.

The City expects to treat seawater primarily during water emergencies, such as that it declared in April 2014, or when there are limitations on its other main sources – i.e., local groundwater and the State Water Project ("SWP") – due to low aquifer levels, limited allocations, reduced reliability of SWP supplies, or other reasons. In 2010, for example, the facility served as the City's primary water source for several months during a partial SWP shutdown. The City's current Urban Water Management Plan ("UWMP"), completed in July 2016, specifically identifies the brackish water treatment facility as part of the City's future water supply portfolio and states that it is also expected to provide up to 645 acre-feet of seawater desalination per year to the City through 2040.

Permitting:

The desalination facility itself is within the City's certified Local Coastal Program ("LCP") jurisdiction. Project components within the State Coastal Commission's retained jurisdiction include the seawater wells, the facility's outfall into the former power plant discharge channel, and portions of the pipelines between the wells and the facility and between the facility and the discharge that run beneath Embarcadero Road and Morro Rock City Beach.

The City originally constructed the facility in response to drought conditions of the early 1990s, and its initial coastal development permits ("CDPs") allowed it to operate for only a limited time. The City has since used the facility intermittently as a backup or emergency water supply, though it has not had a CDP allowing operations since 1999. However, in November 2016, the City requested a coastal development permit for the City to continue intermittent operation of the seawater wells and facility discharge. The request was granted with the following special condition:

- **Special Condition 1:** Prior to issuance of a CDP, the City must provide documentation from the Regional Board that it has approved the proposed discharge during operations using seawater. If the Regional Board requires any physical changes to the discharge structure or its location, the City will be required to obtain an amendment to this CDP unless the Executive Director determines no amendment is legally required.

When the facility operates using brackish water, its effluent has salinity levels of about 21,000 to 26,000 parts per million ("ppm"), which is somewhat less than levels in ambient seawater of about 31,000 to 34,000 ppm. The Central Coast Regional Water Quality Control Board has determined that the discharge is subject to the Board's "General Low-Threat" discharge permit, which is available to discharges that contain minimal amounts of pollutants and pose little threat to water quality. The facility's proposed ongoing use of the seawater wells would increase its discharge salinity to up to about 45,000 to 47,000 ppm and will require the Regional Board to reassess the facility's permit to ensure the facility continues to meet water quality standards and is protective of marine life.

As part of the City's November 2016 CDP approval, the City proposed upgrades the facility by installing new treatment systems, tanks, and other similar infrastructure that would allow simultaneous use of both brackish water and seawater treatment systems, but did not propose any new construction to the project components within the Commission's retained jurisdiction. The proposed upgrades will allow the facility to more effectively treat the two different types of source waters it relies on – brackish groundwater and seawater – which have different water quality characteristics and different treatment needs.

Concurrently, the City has also been working with Commission staff to relocate its wastewater treatment facility and the desalination facility, due to their location in an area subject to coastal hazards, including flooding and tsunami run-up. The work includes site alternative analyses, a planned update to the City's Local Coastal Program ("LCP"), and an expected subsequent submittal of a CDP application for the new proposed facilities. Depending on the eventual location of these facilities, the City may propose keeping the existing seawater wells or may propose alternative locations that are more suitable to serve the new facility location. If new locations for the seawater wells or outfall are proposed, a new CDP will be required.

Current Status & Assessment

The project team inspected the desalination facilities in July 2017. Based on this, and other proposed modifications, the following recommendations are made:

Common Facilities

1. Complete the outfall modeling necessary to demonstrate that the seawater system can be operated without harming the marine environment.
2. The treatment plant is located within the flood plain and tsunami run-up area. If the City wishes to pursue low-interest SRF loan money to finance the rehabilitation of this facility, it will need to be relocated.

Seawater Treatment System

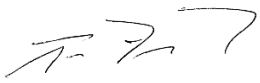
1. Due to the age and condition of the seawater treatment equipment - which includes cartridge filters, iron filters, positive displacement feed pumps and seawater RO trains - the City should plan on replacing the existing equipment entirely. The existing RO treatment equipment is outdated, spare parts are no longer available and may be unsafe to operate. The equipment should be replaced with modern technology that includes automated operation and safety features and may also reduce the energy required by approximately 60 percent.
2. The proposed modifications to the existing facilities within the retained jurisdiction of the coastal commission should include:
    a. Repair and replacement of existing seawater supply well electrical and control infrastructure, which, based on its condition, is unlikely to function and may pose life-safety danger if energized and operated.
    b. Well head piping rehabilitation to facilitate automated operation and water quality monitoring.
    c. Flood proofing, which may include converting existing vaults into pressure rated, water proof vaults.

Brackish Water System

1. The brackish water trains appear to be in good working order and can be reused when the facility is relocated to a new location outside of the flood plain.

Sincerely,

CAROLLO ENGINEERS, INC.



Tom Seacord, P.E.

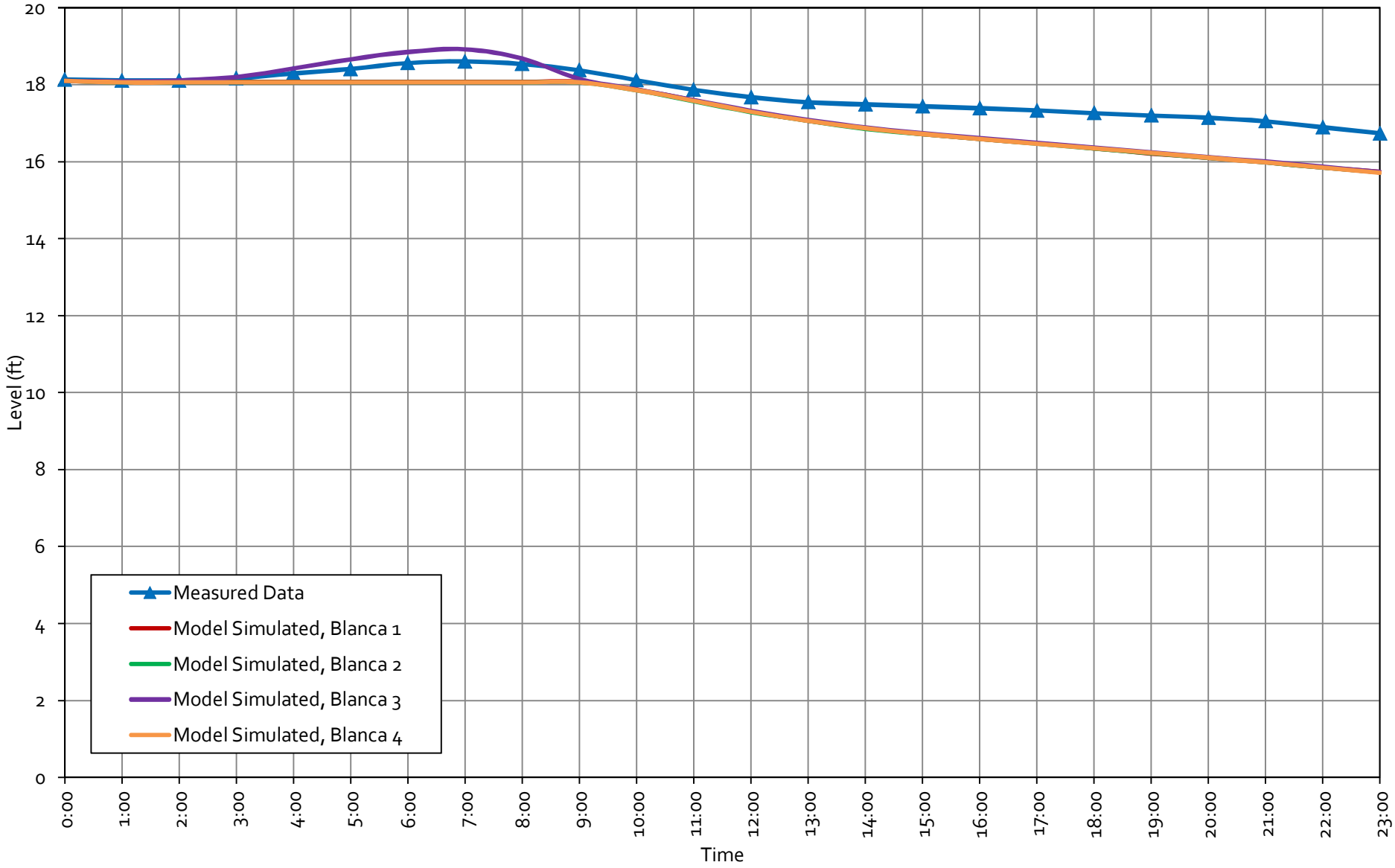
TS:sm

Appendix H

EPS CALIBRATION SHEETS

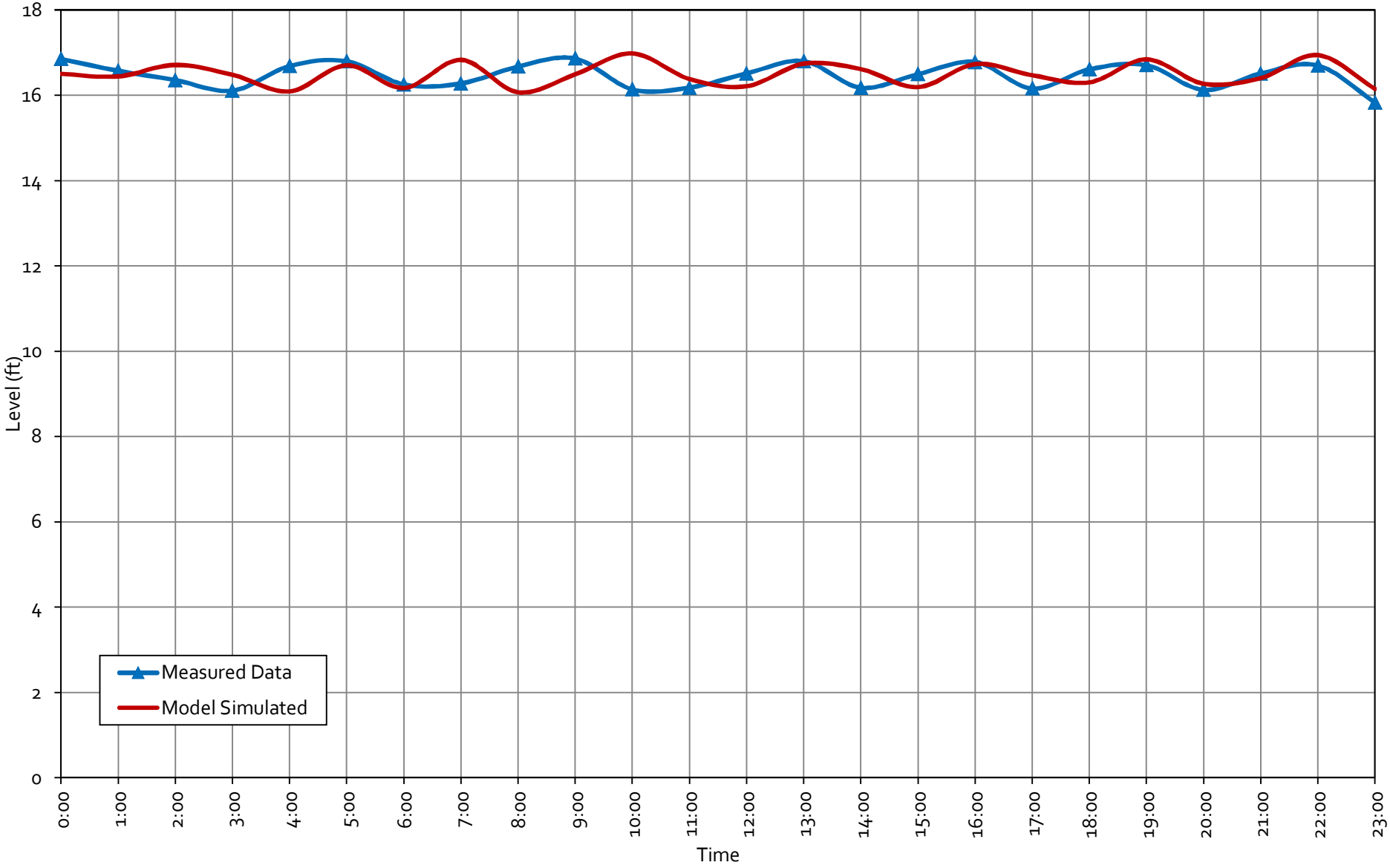
One Water Morro Bay

EPS CALIBRATION - BLANCA TANK LEVEL



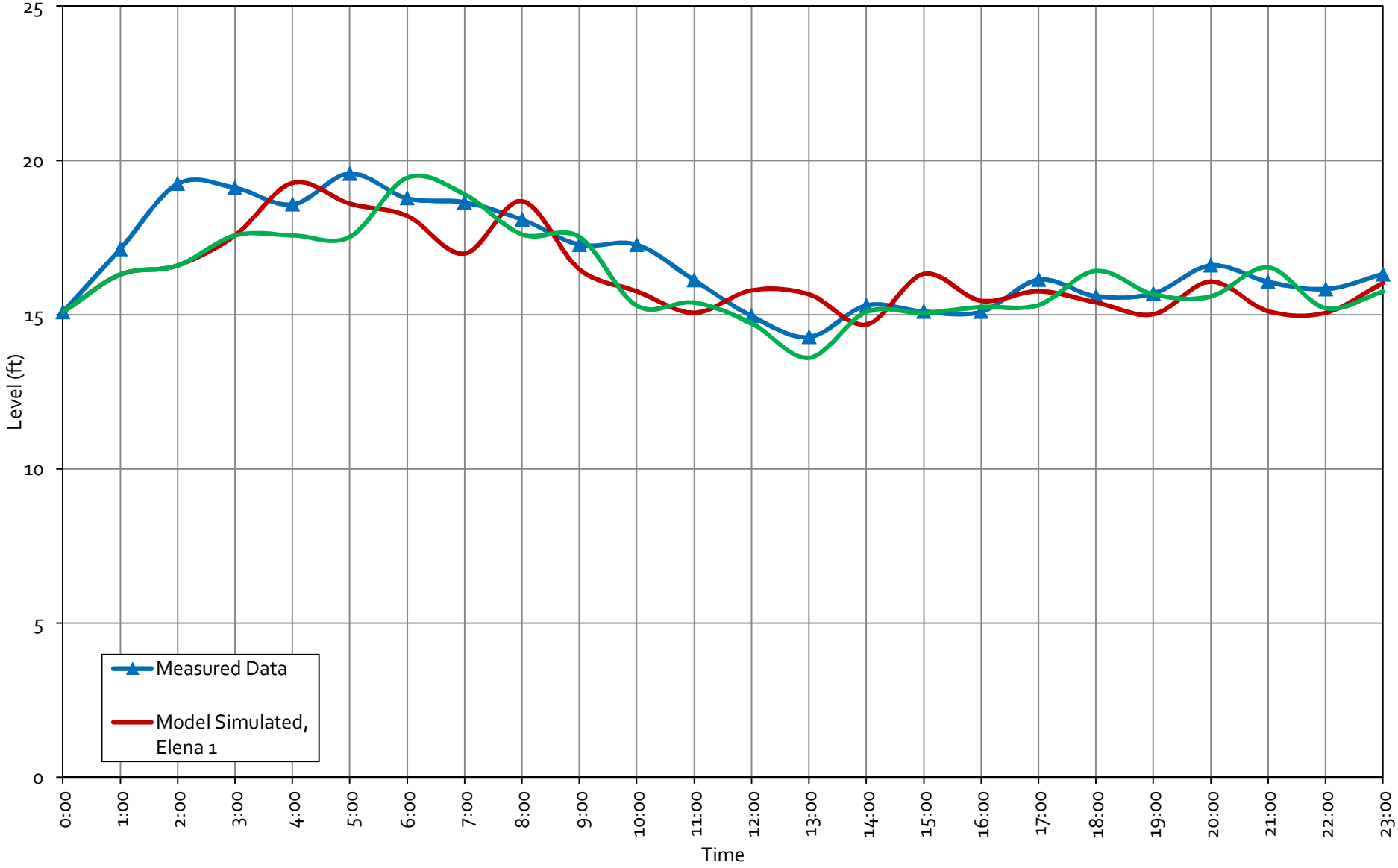
One Water Morro Bay

EPS CALIBRATION - NUTMEG TANK LEVEL



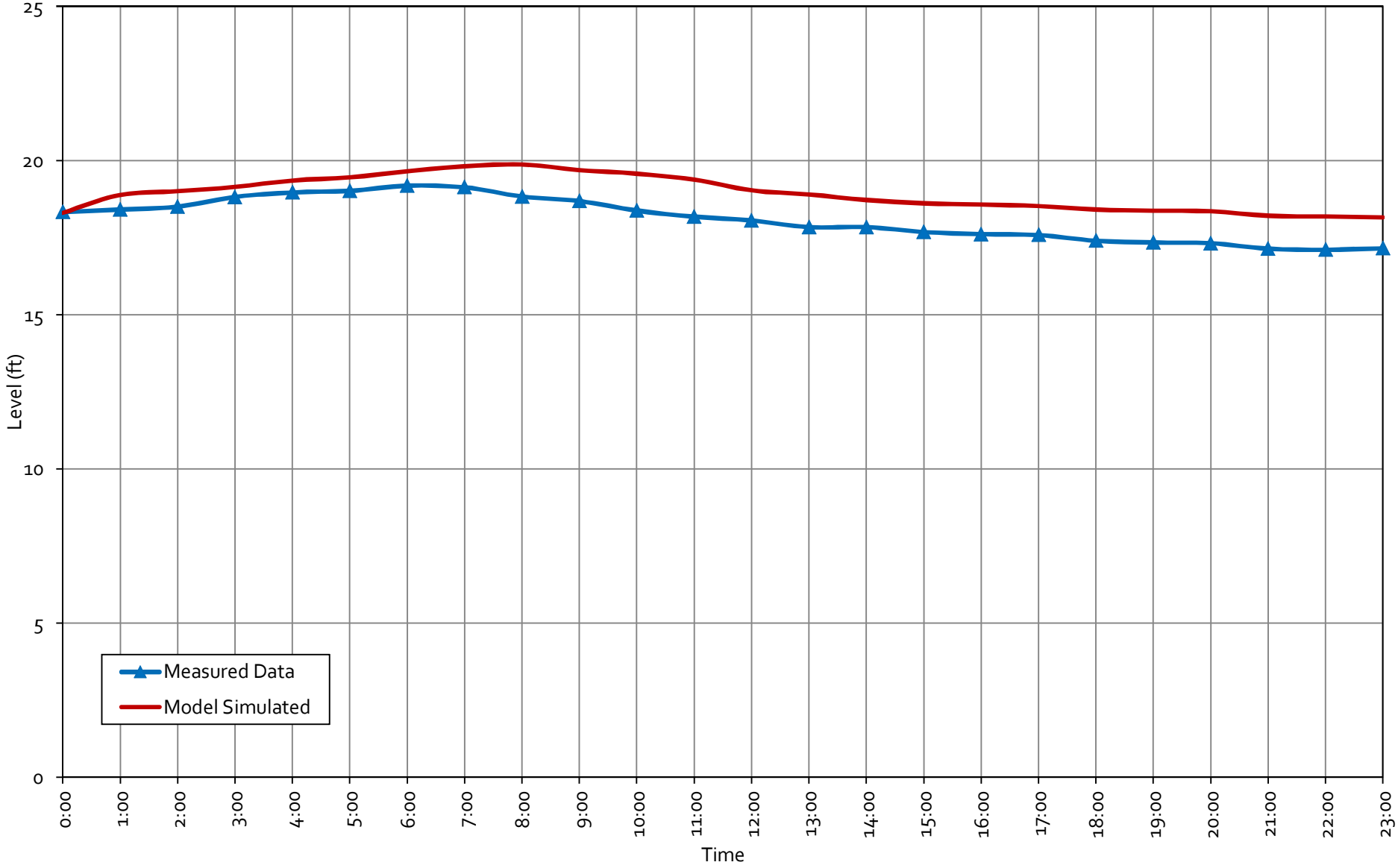
One Water Morro Bay

EPS CALIBRATION - ELENA TANK LEVEL



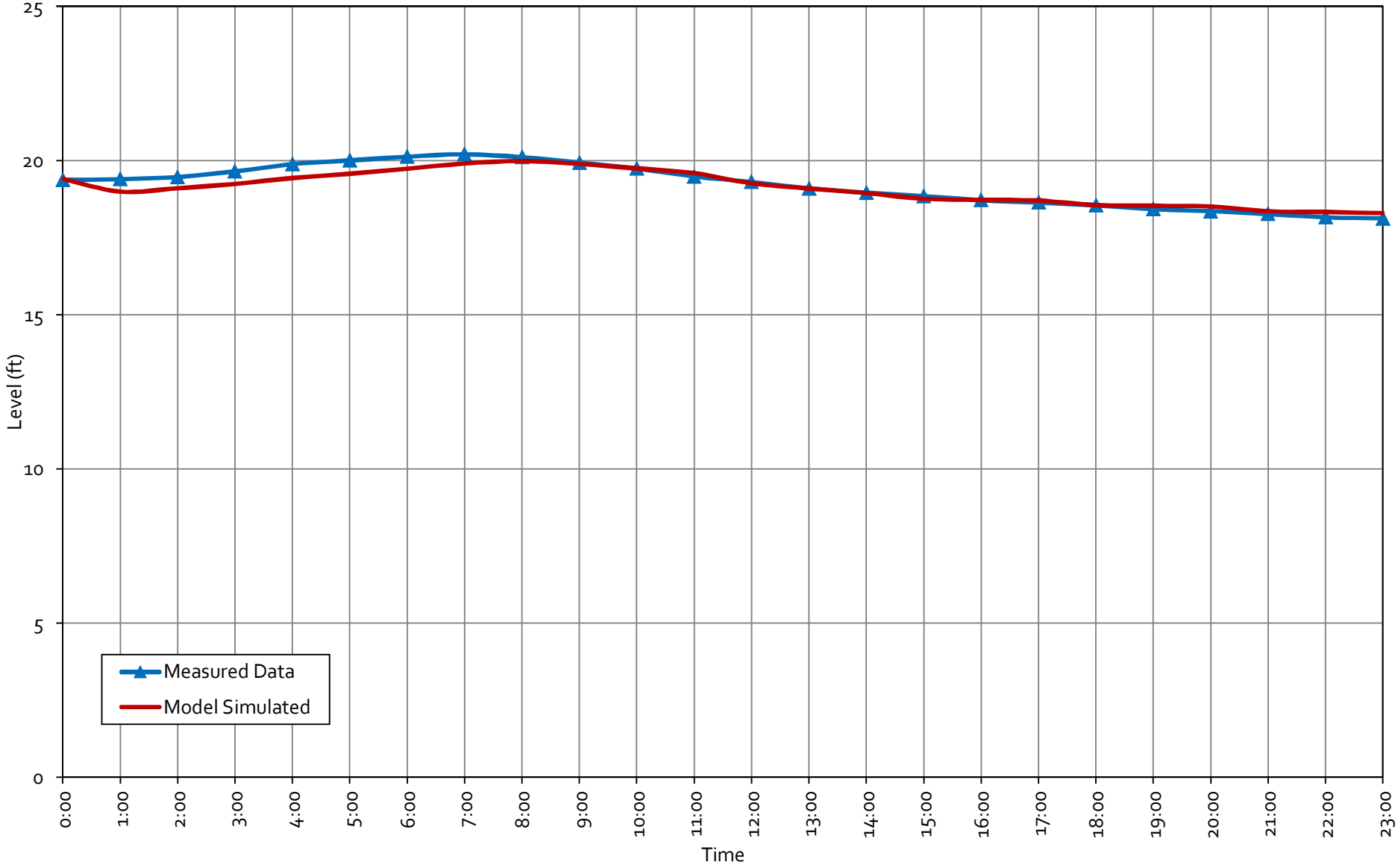
One Water Morro Bay

EPS CALIBRATION - KINGS WEST TANK LEVEL



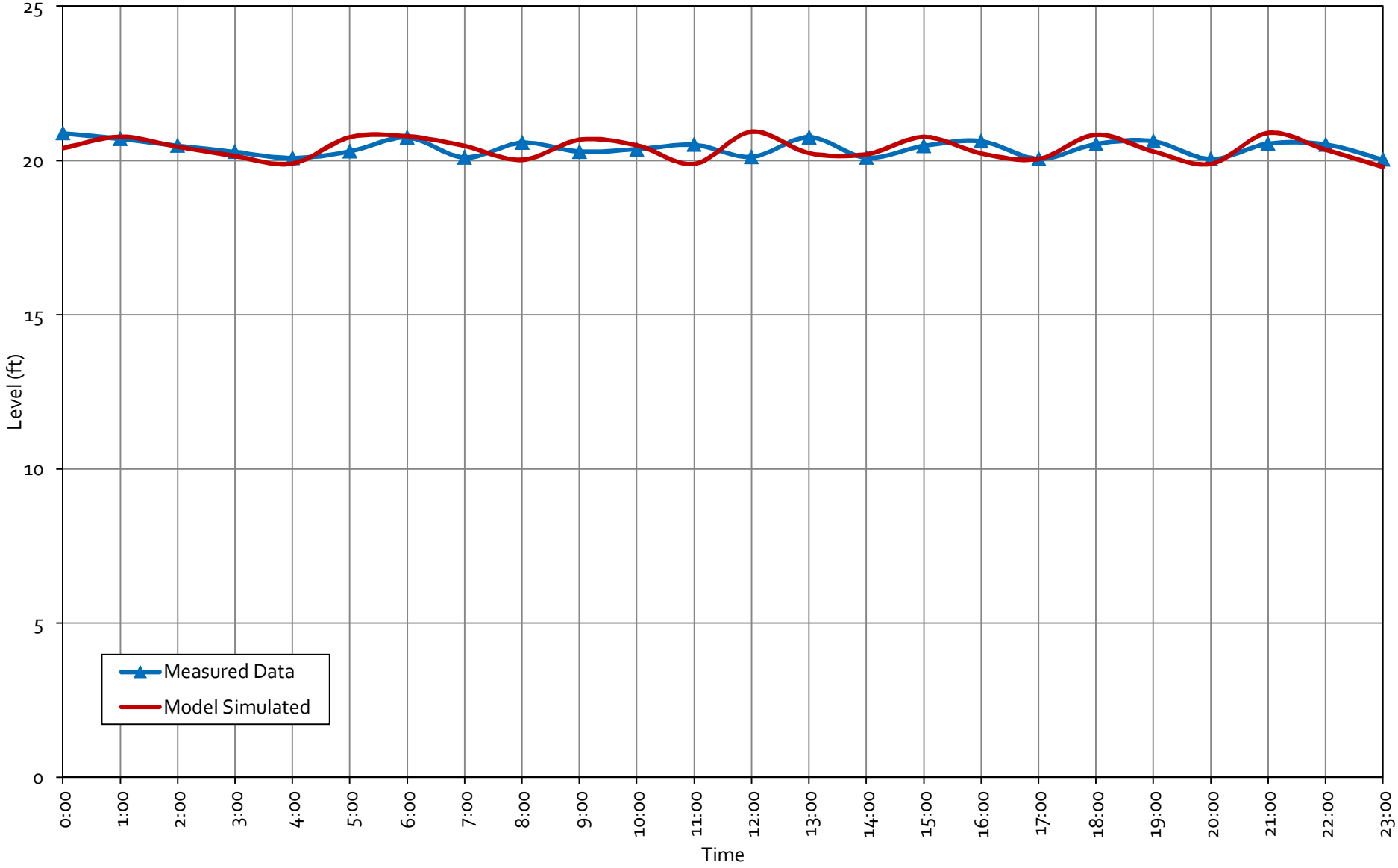
One Water Morro Bay

EPS CALIBRATION - KINGS EAST TANK LEVEL



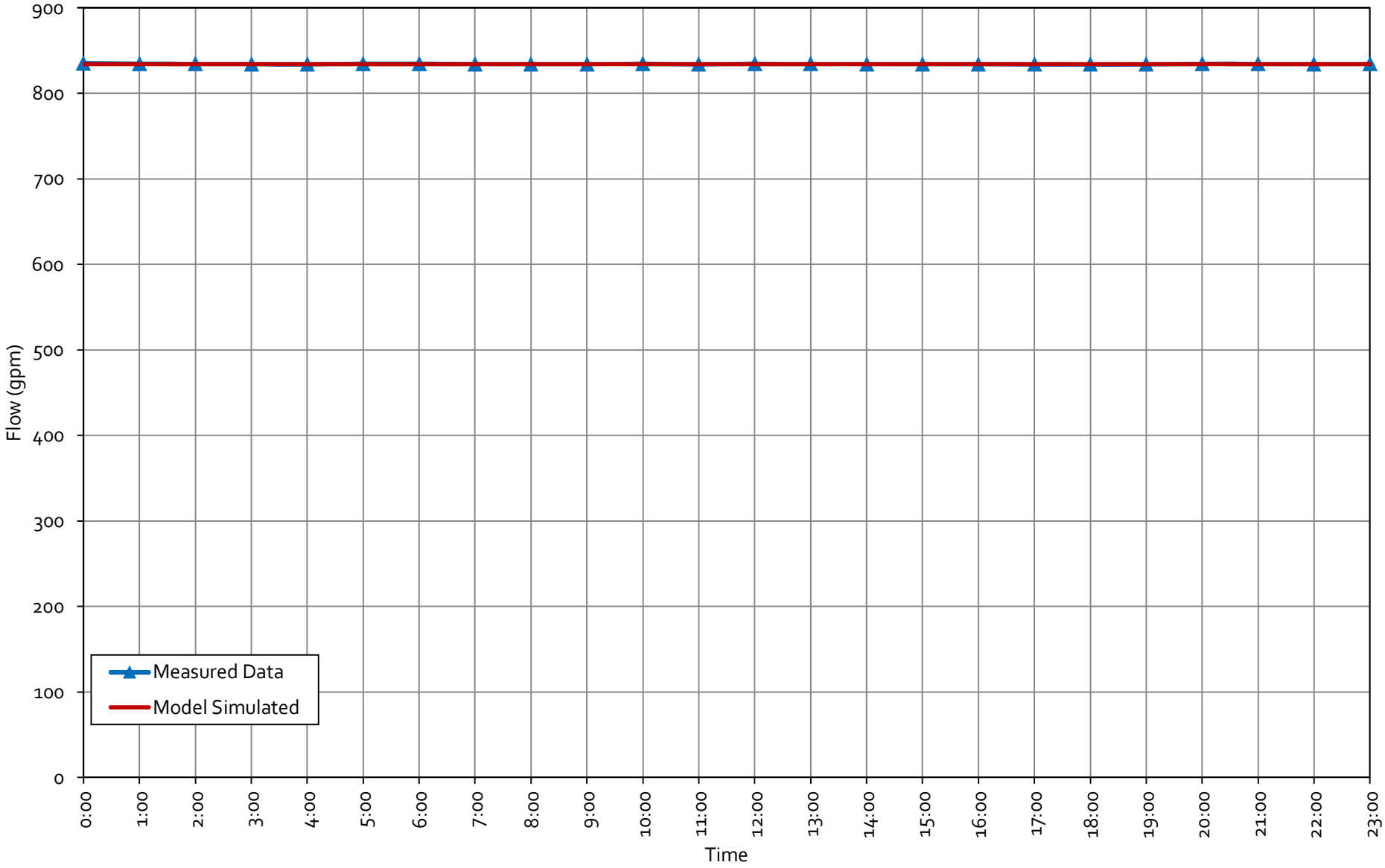
One Water Morro Bay

EPS CALIBRATION - RIDGEWAY TANK LEVEL



One Water Morro Bay

EPS CALIBRATION - STATE WATER FLOW RATE



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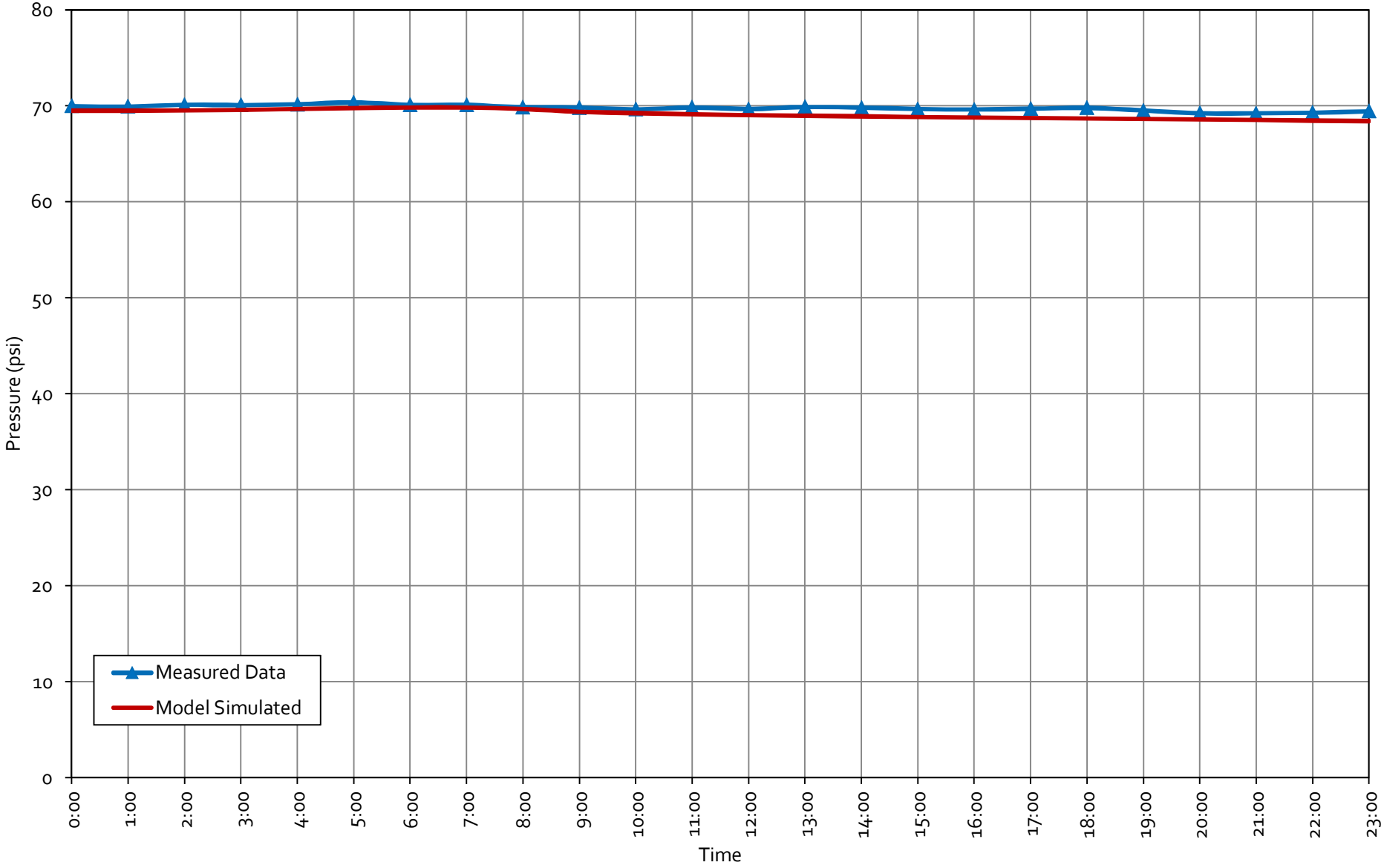
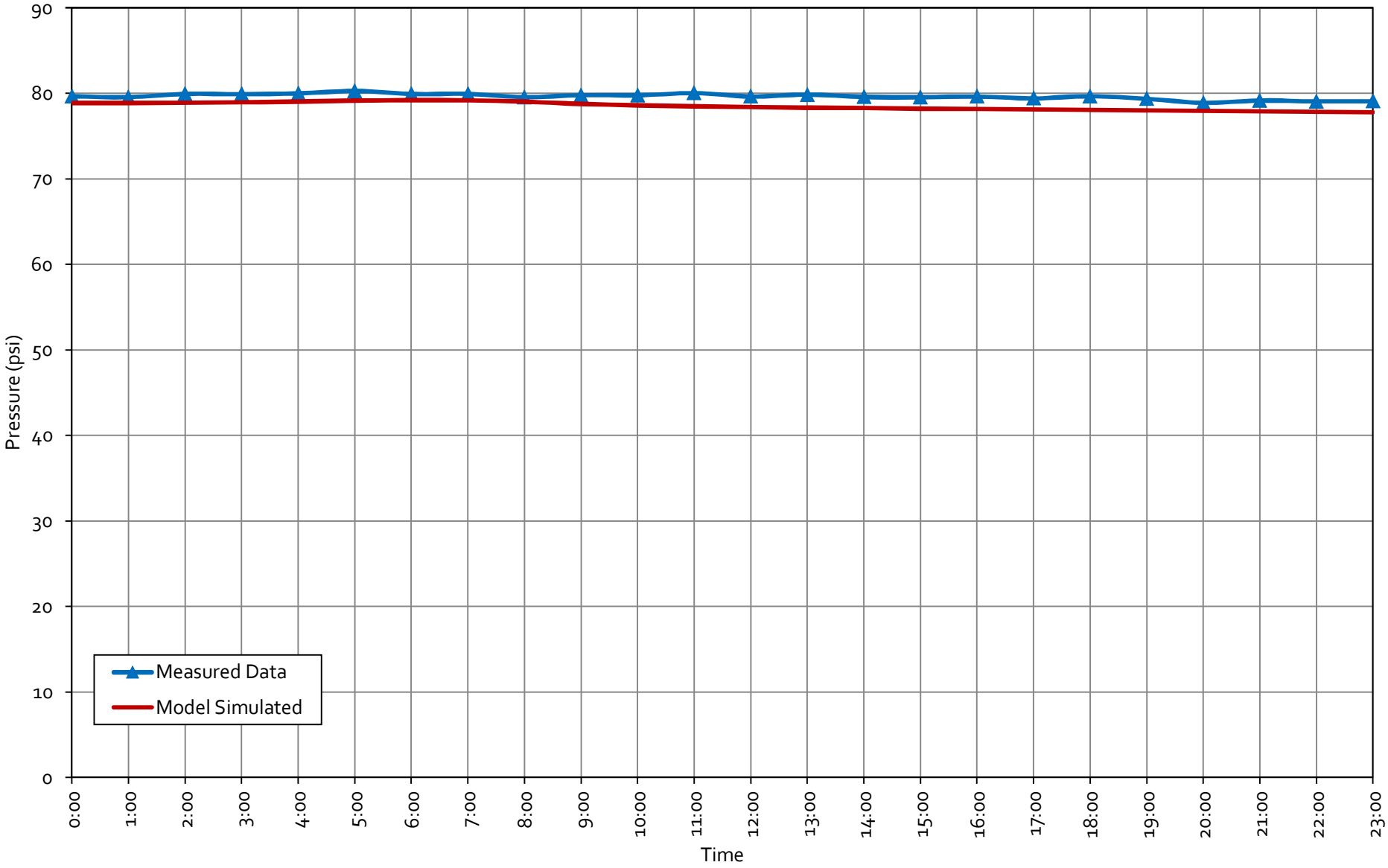
EPS CALIBRATION - PRESSURE LOGGER 30



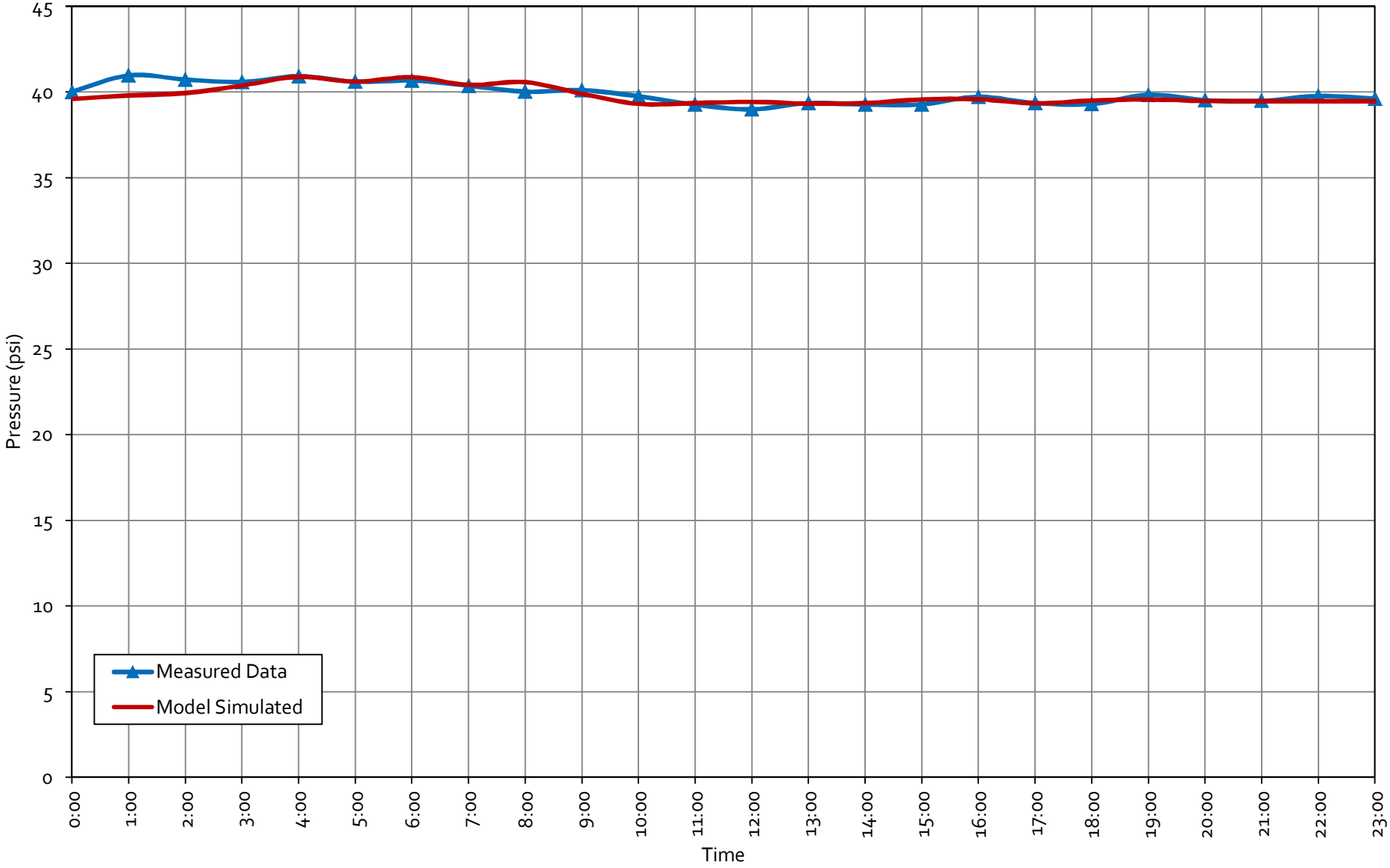
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 32



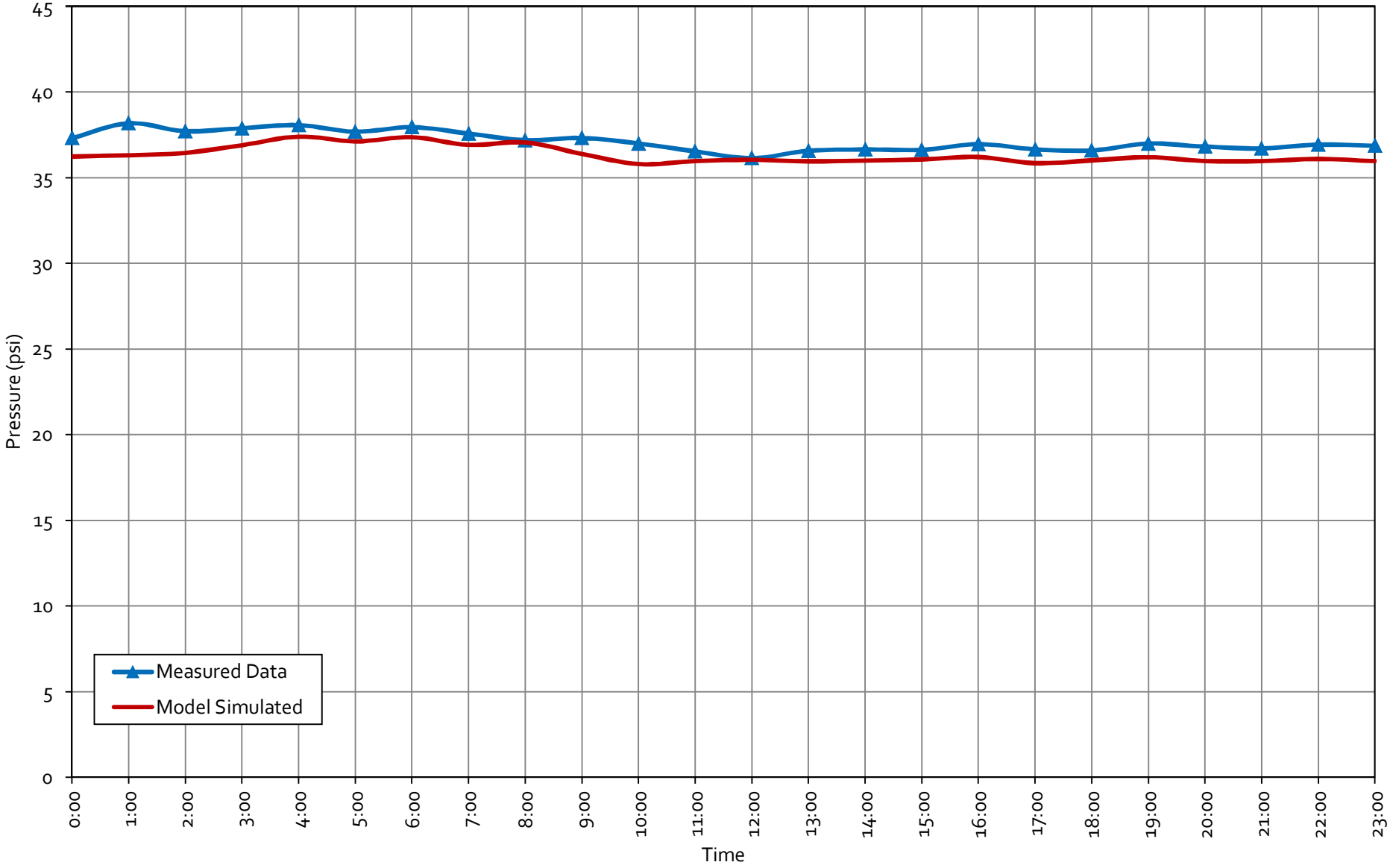
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 33



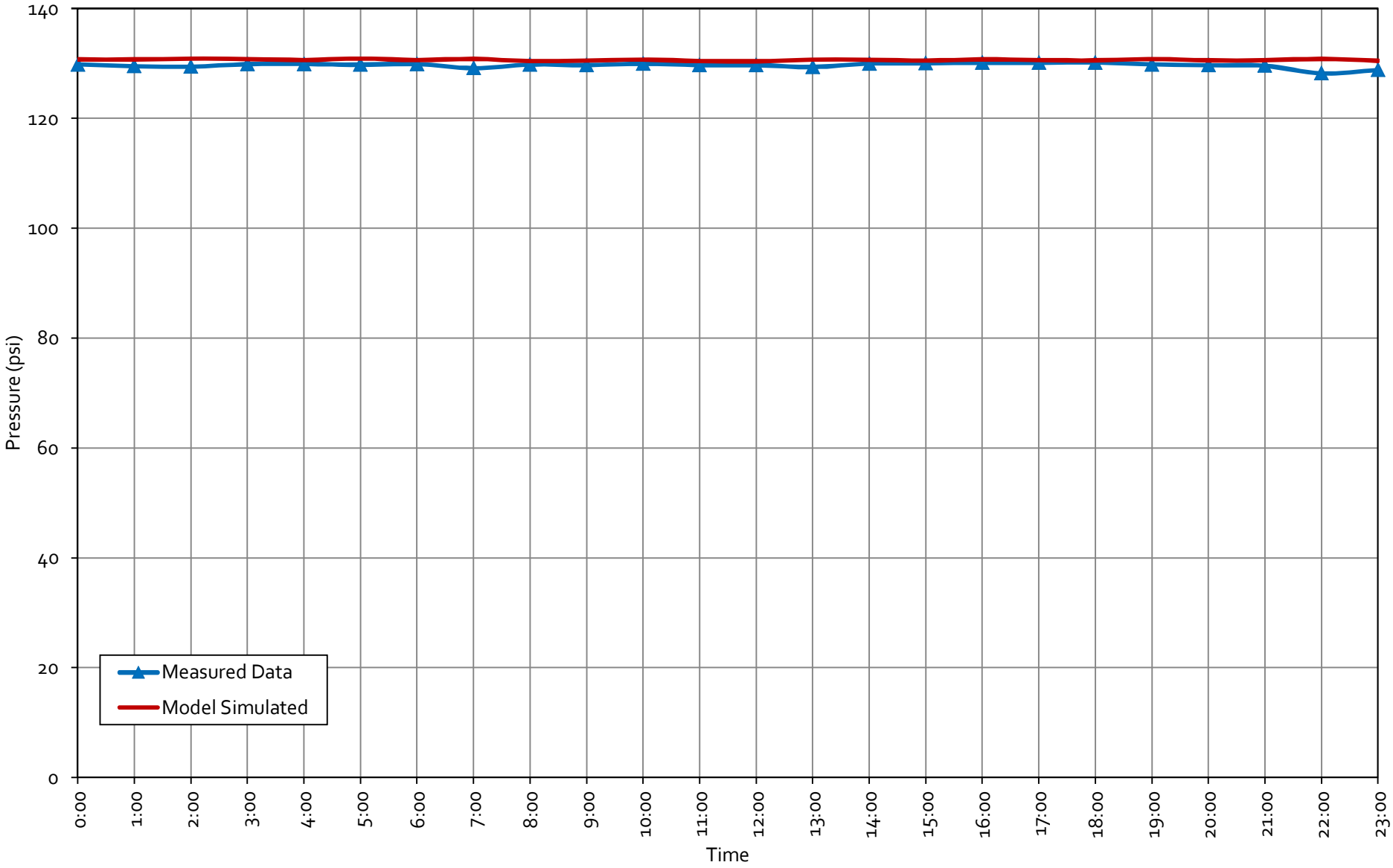
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 34



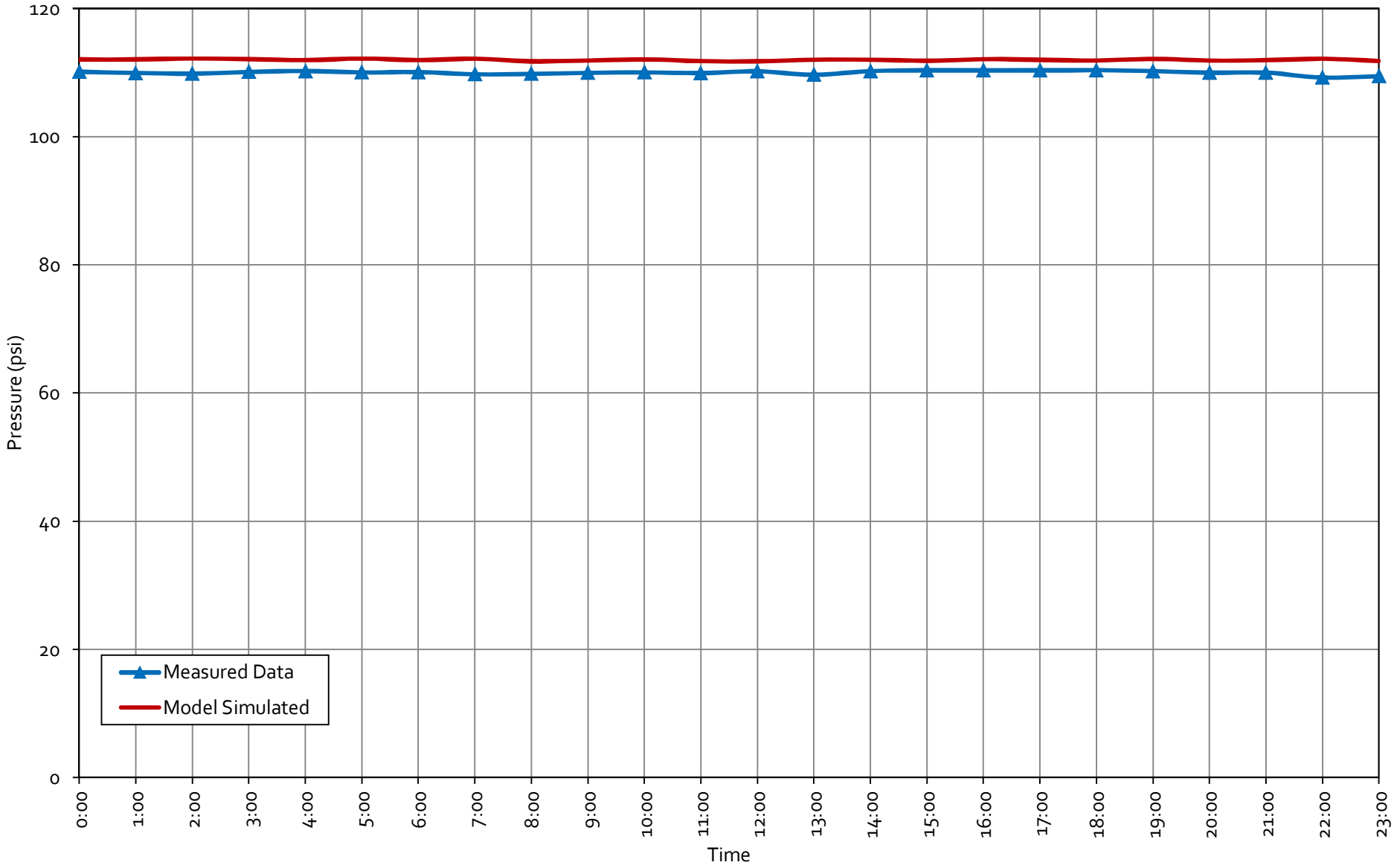
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 35



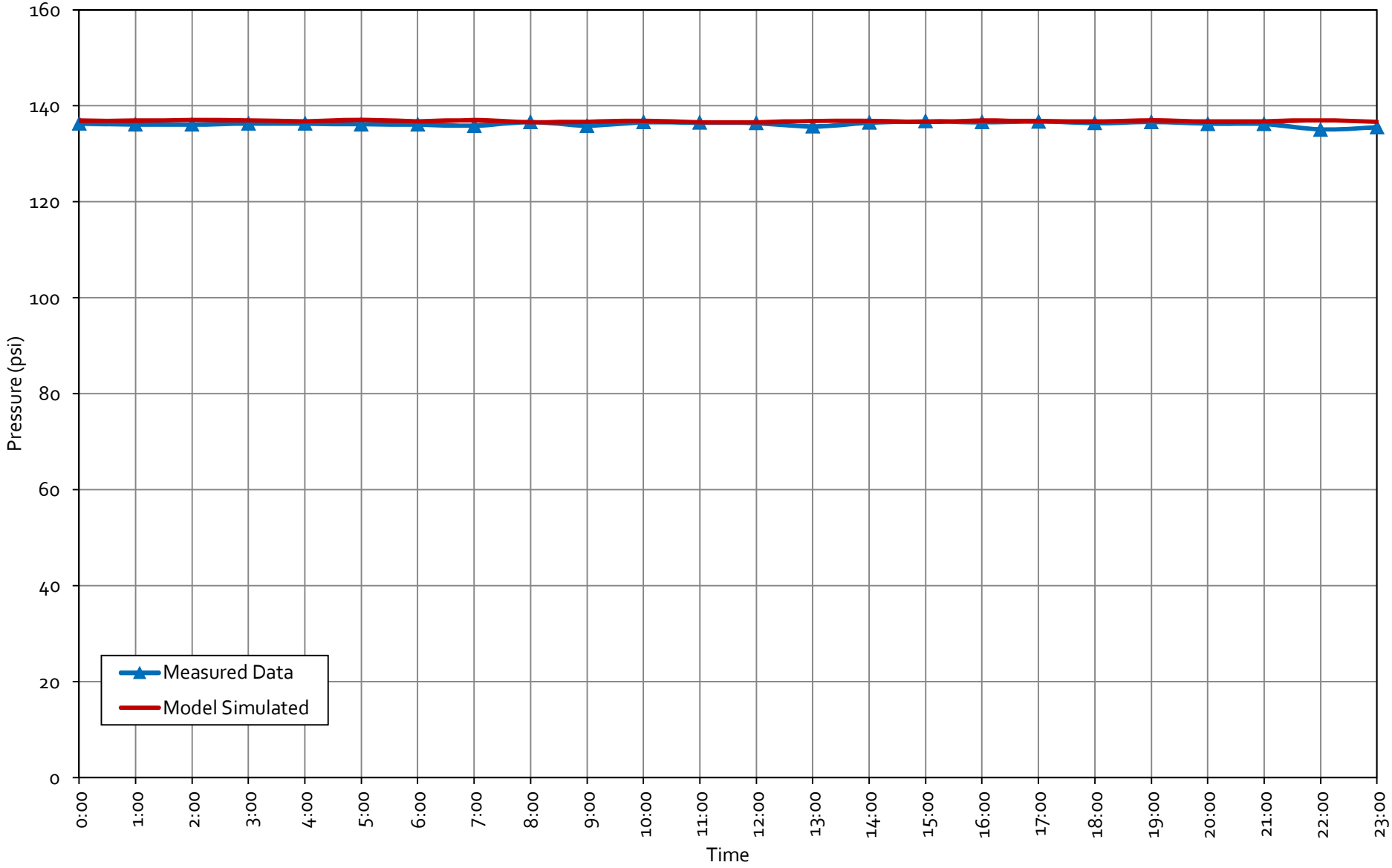
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 36



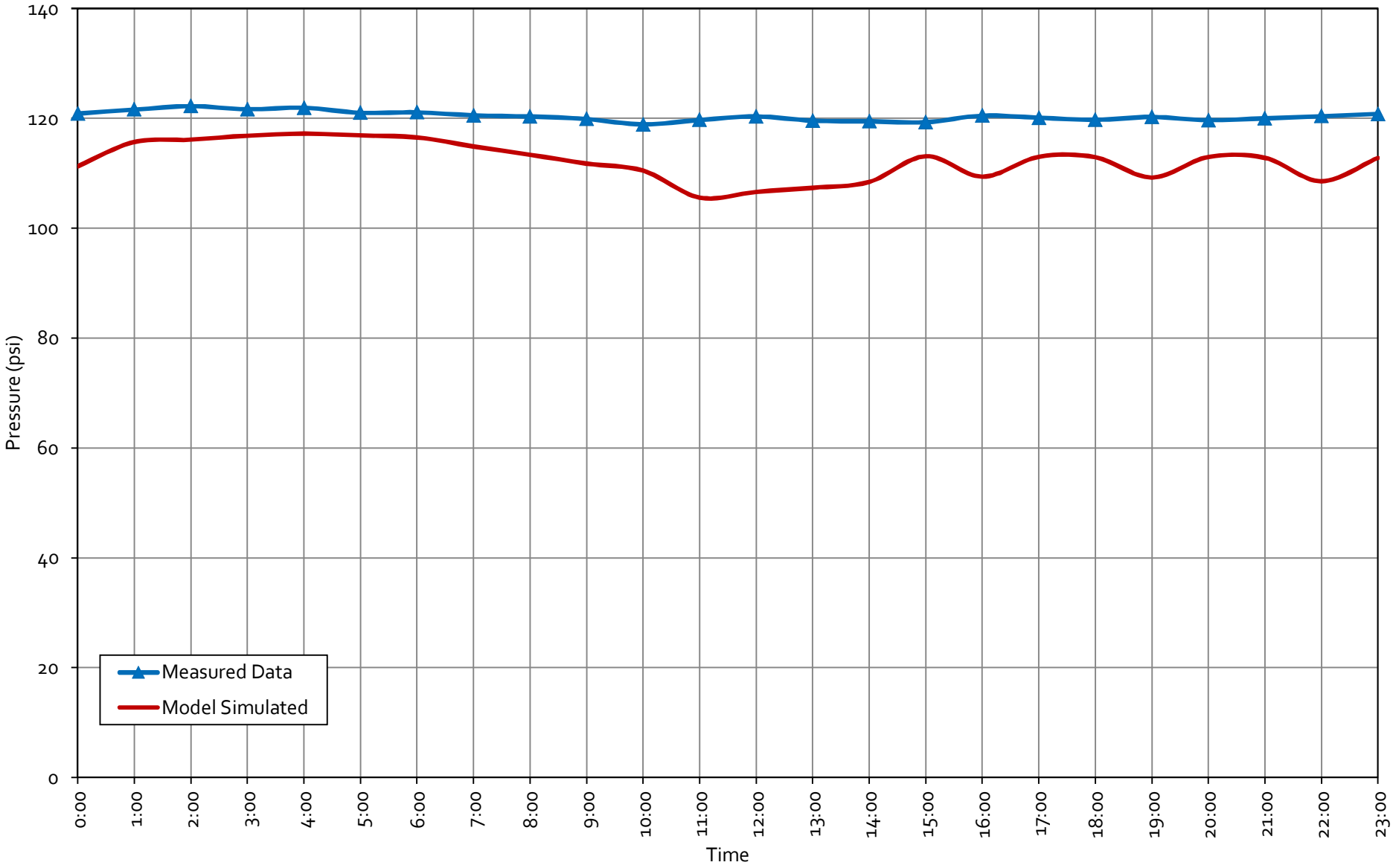
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 37



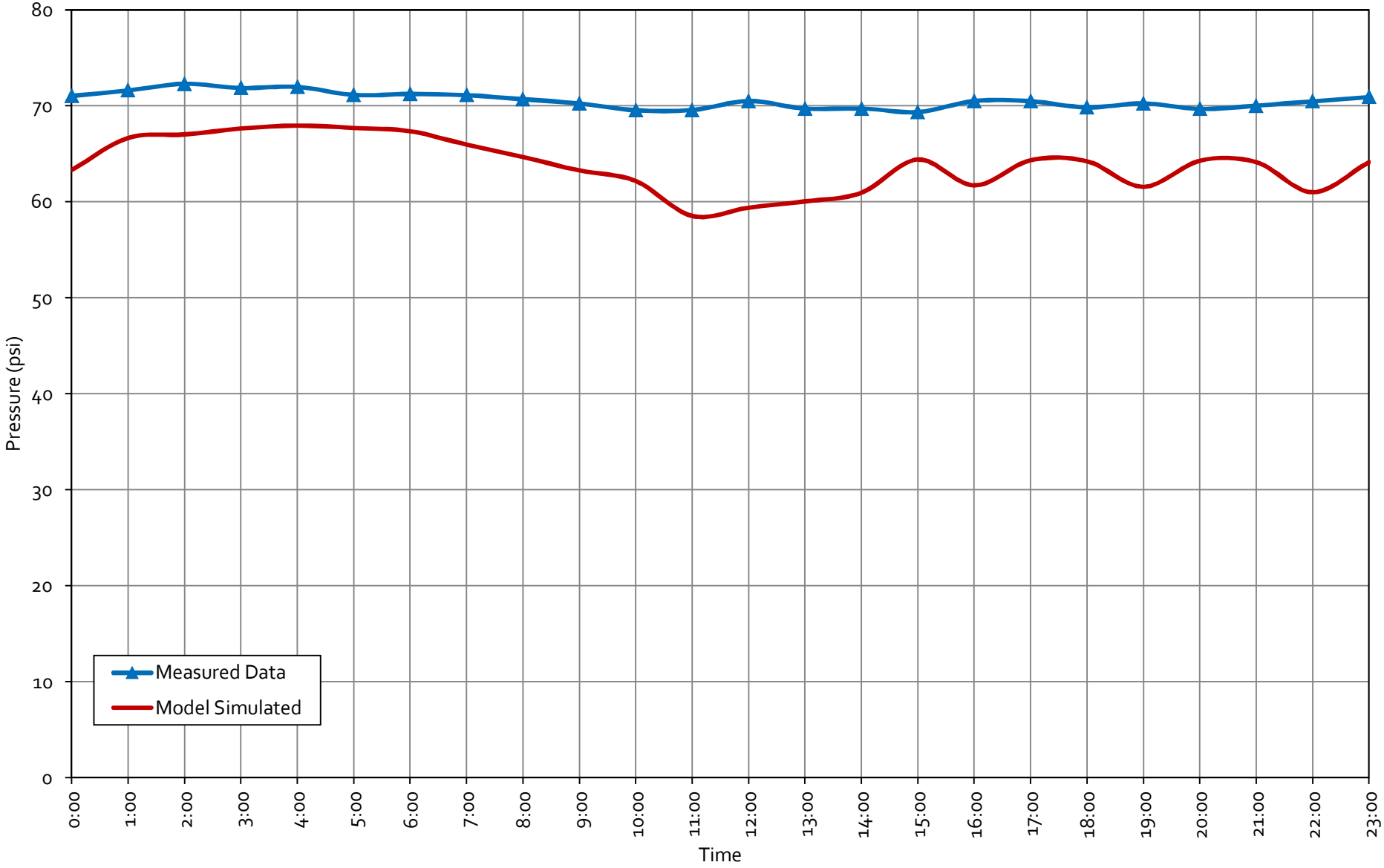
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 38



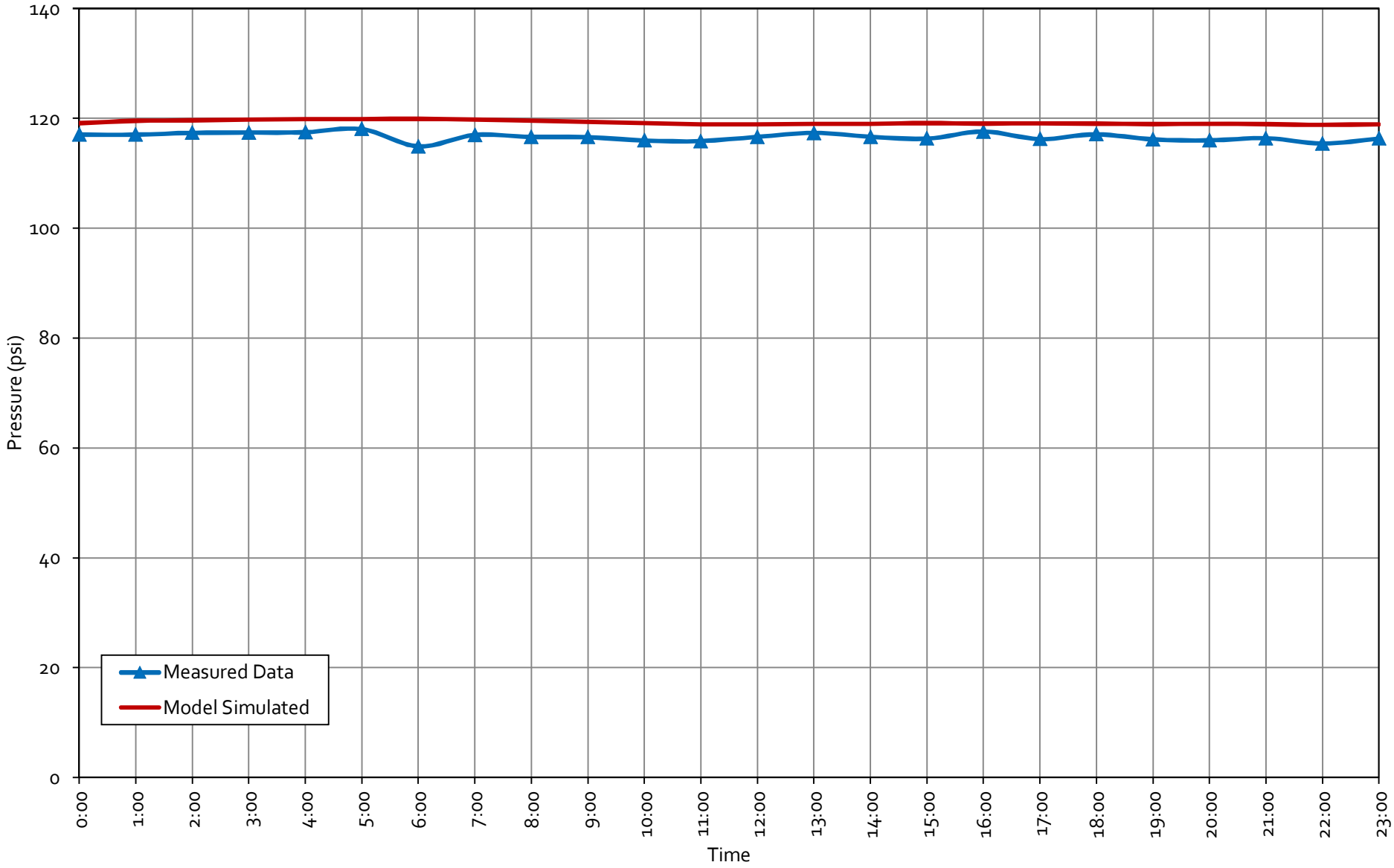
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 39



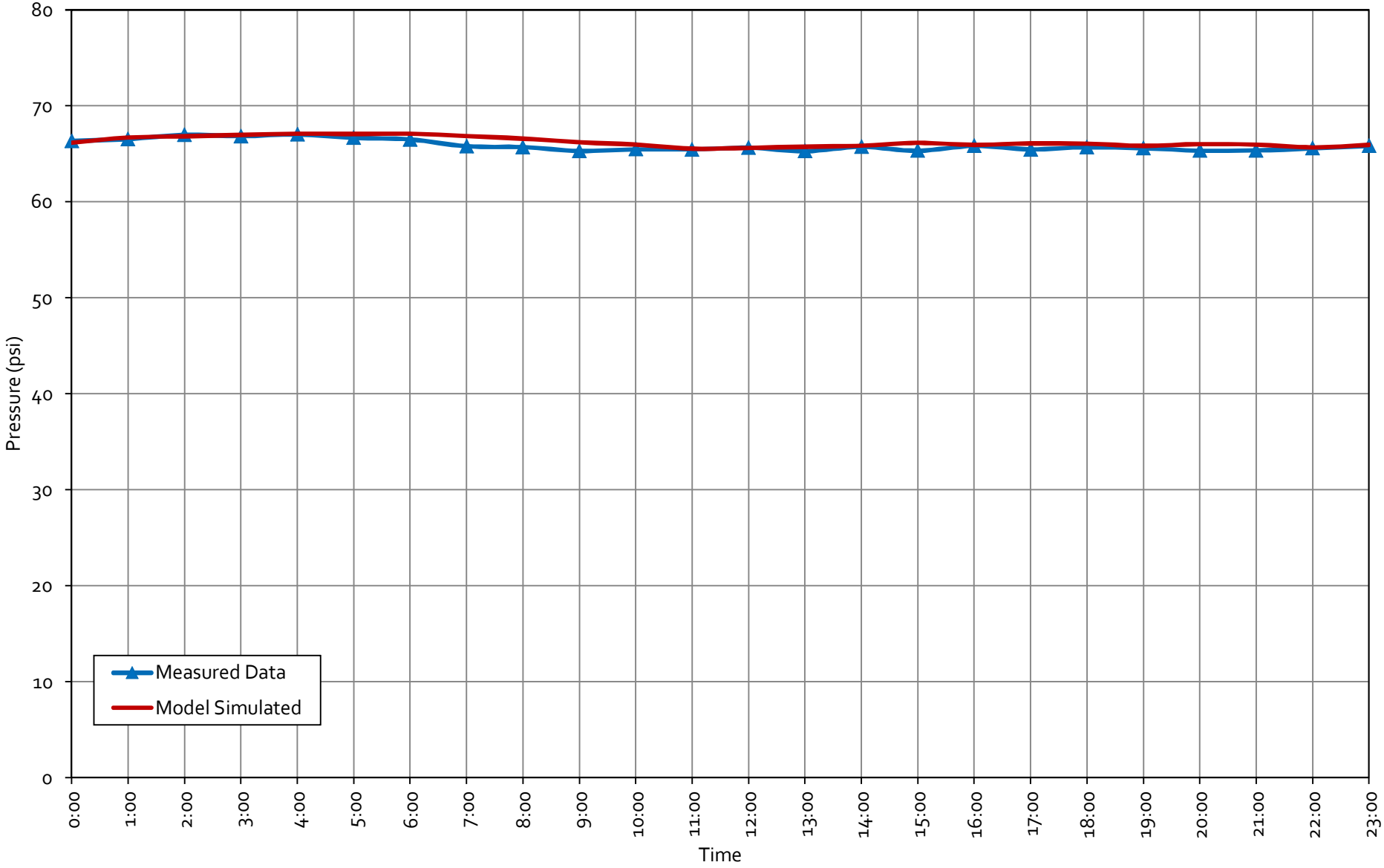
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 40



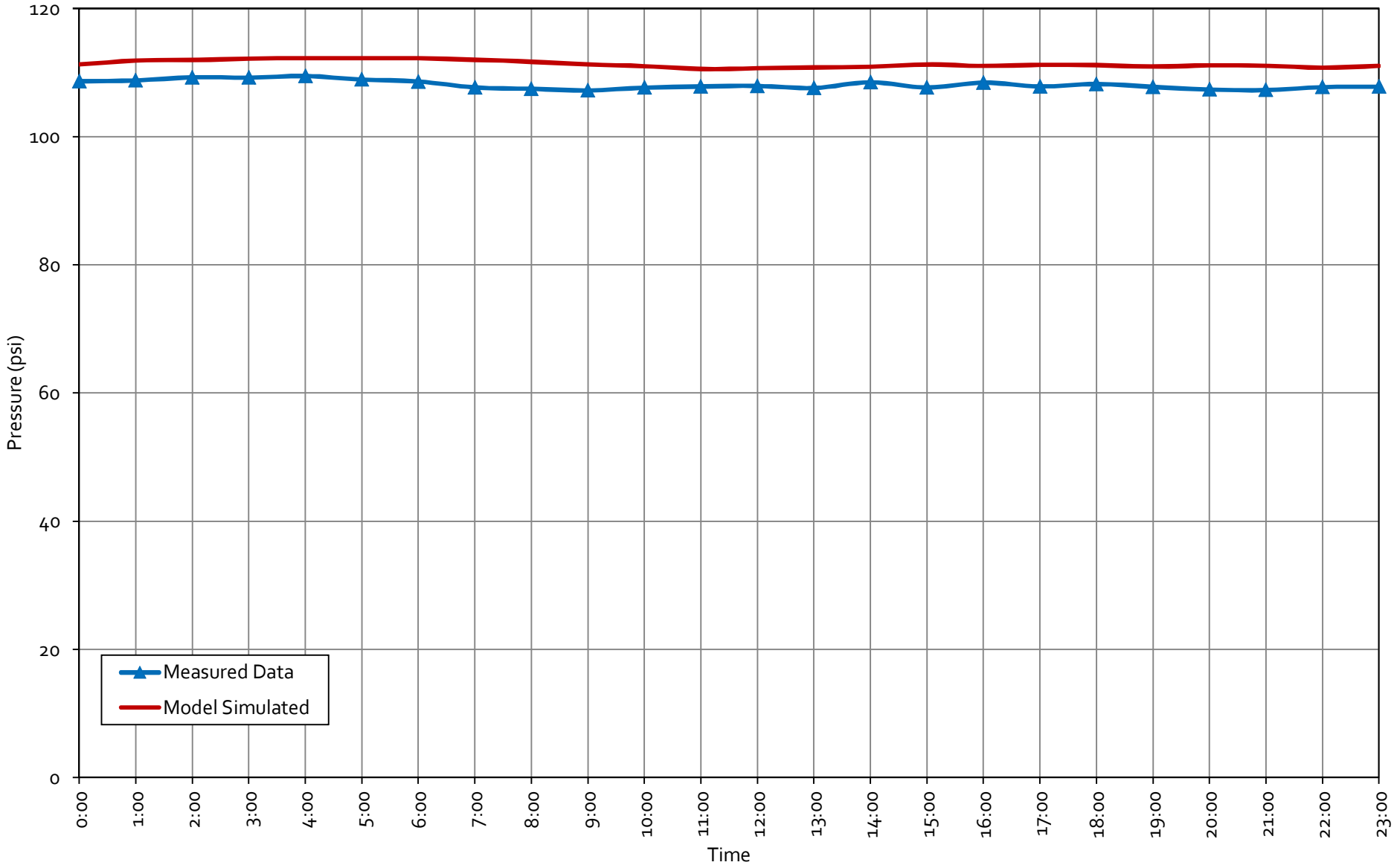
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 41



One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 42



One Water Morro Bay

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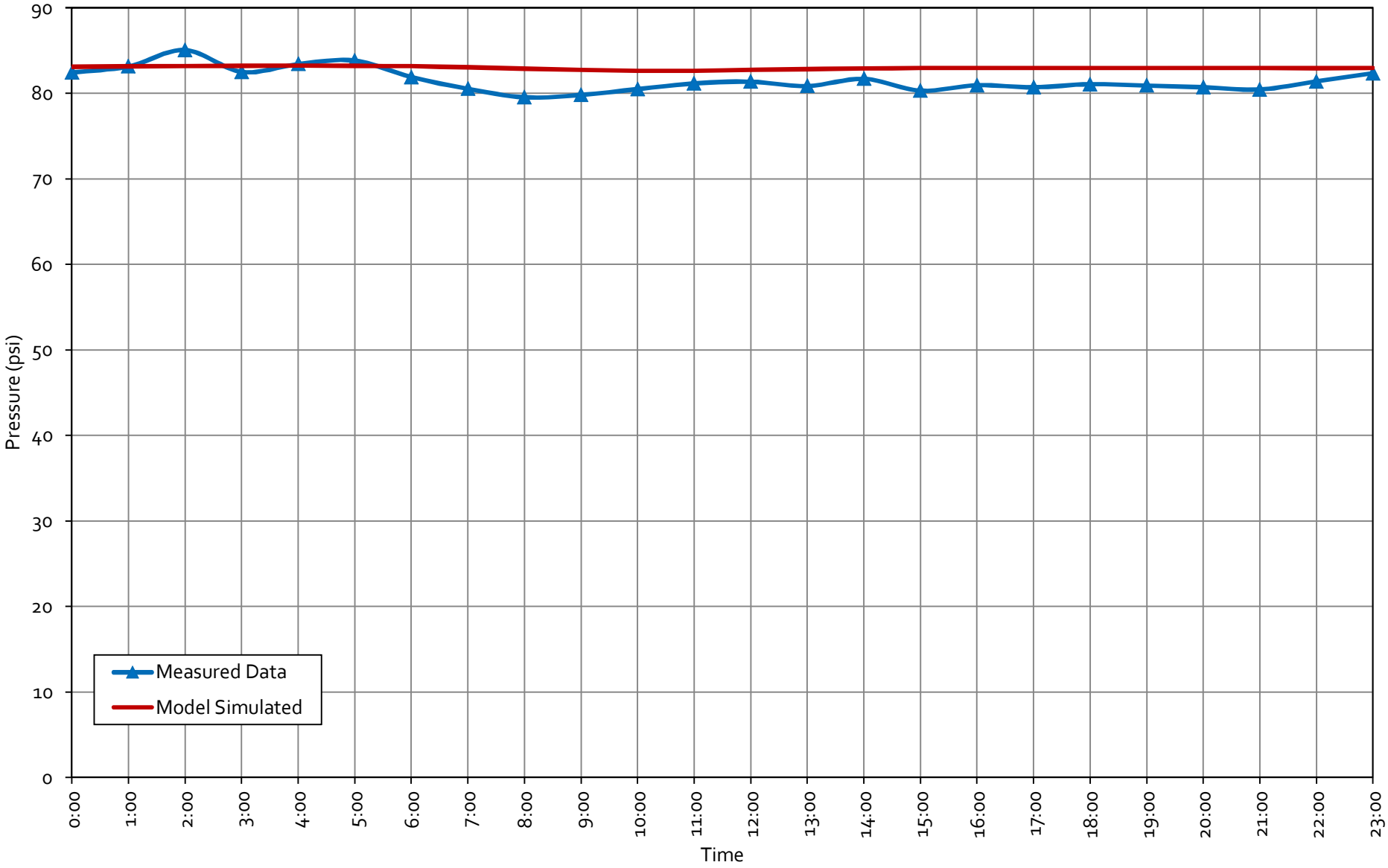
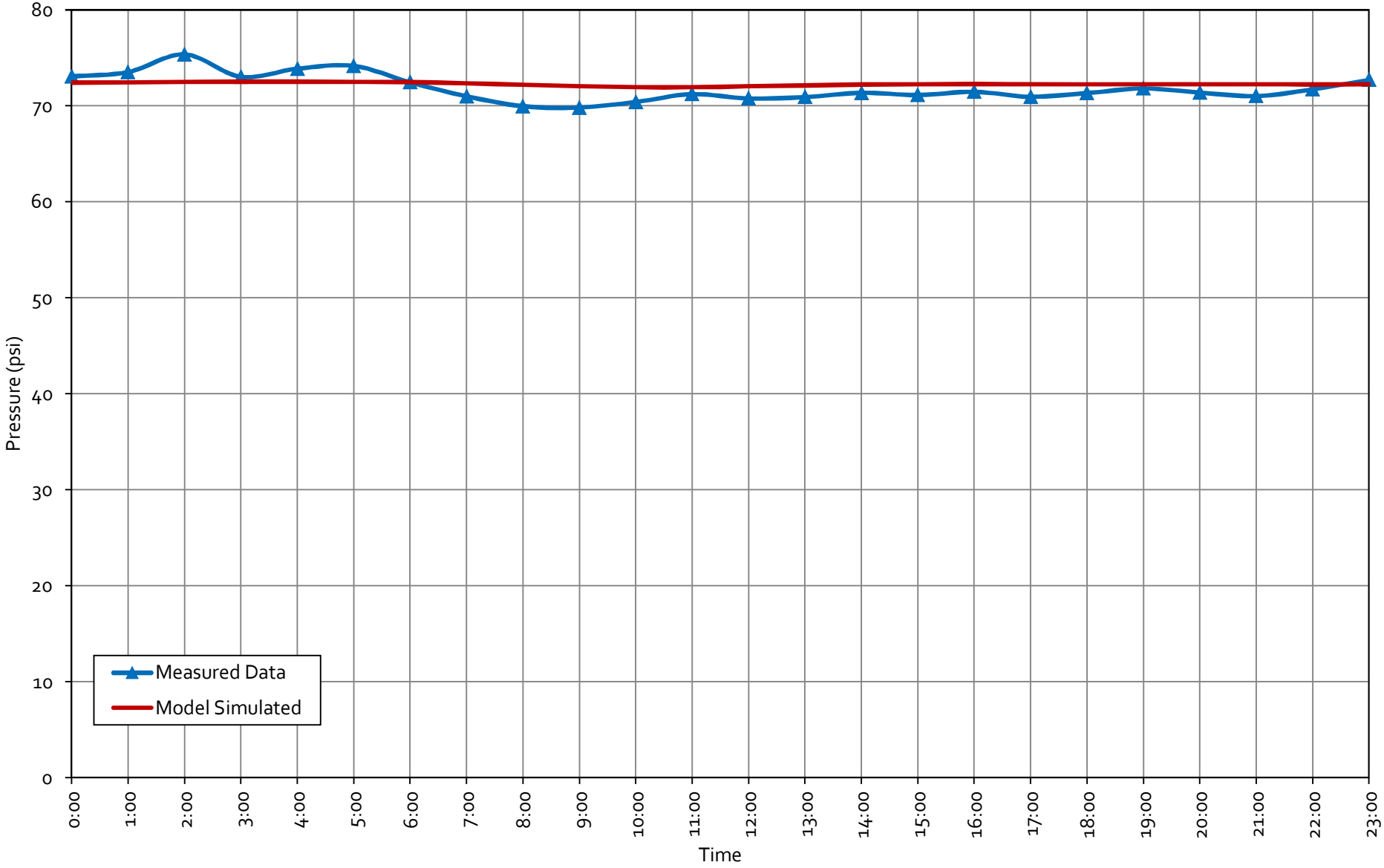


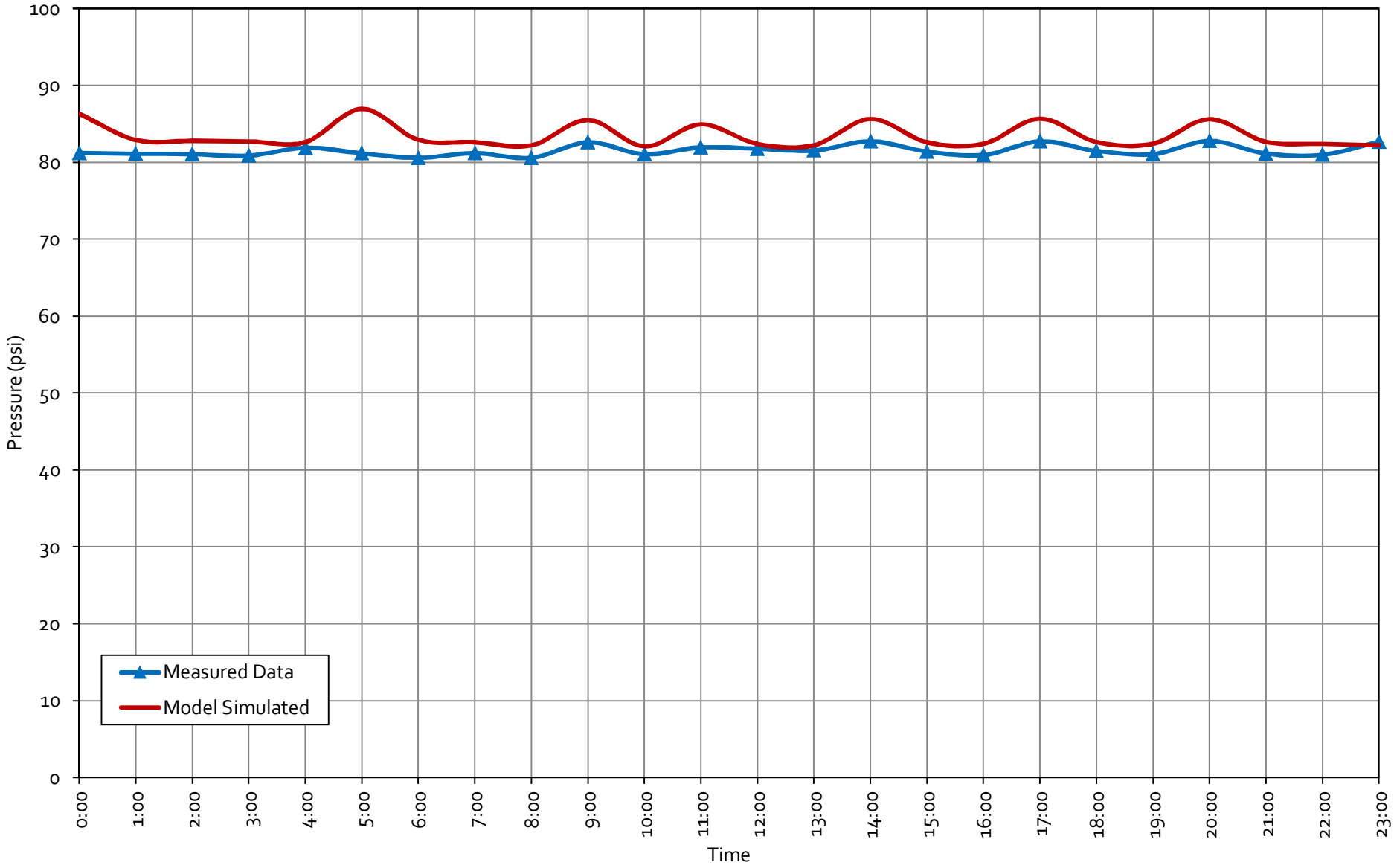
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 44



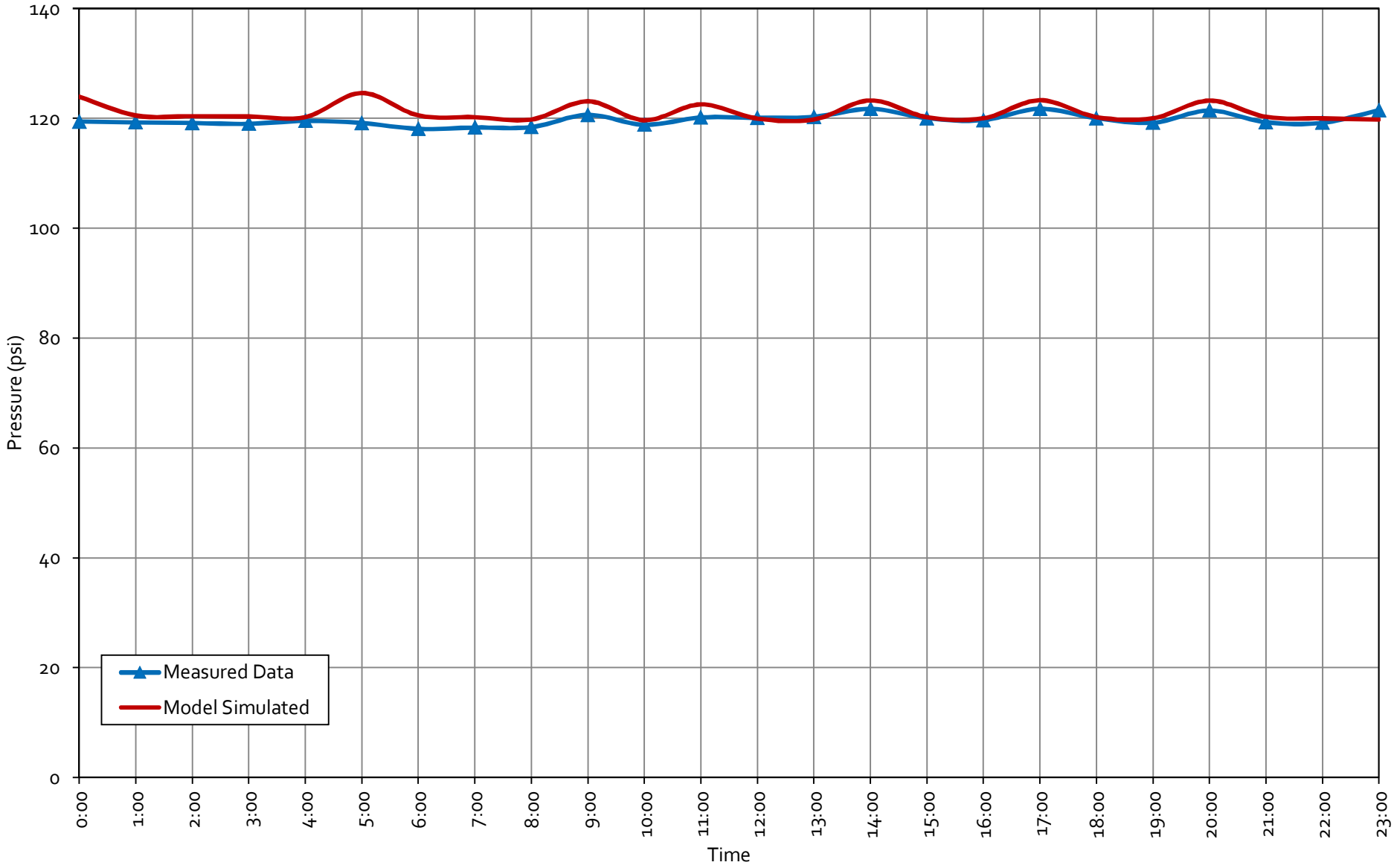
One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 45



One Water Morro Bay

EPS CALIBRATION - PRESSURE LOGGER 46



One Water Morro Bay

FIRE FLOW CALIBRATION (TEST DATE 7/7/17)

Test Site	Test Location (Pressure Zone)	Time	Hydrant Type	Field Measured Data			Model Simulated Data		Percent Difference		Pressure Drop			Notes
				Hydrant Flow (gpm)	Hydrant Pressure (psi)		Hydrant Pressure (psi)				Measured (psi)	Modeled (psi)	Difference	
					Static	Residual	Static	Residual	Static	Residual				
1	Tide Ave and Oahu St (Blanca)	8:00	Flowing	850	68	60	68.9	--	1.3%		--	--	--	
			Pressure 1	--	71	66	71.5	67.1	0.7%	1.7%	5	4.4	-1	
			Pressure 2	--	51	43	50.9	46.5	-0.2%	8.1%	8	4.4	-4	
2	Juniper Ave south of Sequoia St (Nutmeg)	8:16	Flowing	1127	104	60	104.1	--	0.1%		--	--	--	
			Pressure 1	--	104	70	108.1	67.9	3.9%	-3.0%	34	40.2	6	
			Pressure 2	--	116	82	115.3	83.1	-0.6%	1.3%	34	32.2	-2	
3	Elm Ave and Bonita St (Elena)	8:30	Flowing	820	40	30	41.1	--	2.8%		--	--	--	
			Pressure 1	--	46	41	45	40.9	-2.2%	-0.2%	5	4.1	-1	
			Pressure 2	--	48	30	35.1	31.6	**-26.9%**	5.3%	18	3.5	-15	Static pressure appears incorrect
4	Rockview St east of Sunset Rd (Nutmeg)	8:42	Flowing	1000	142	58	142.6	--	0.4%		--	--	--	
			Pressure 1	--	142	68	144.7	68.7	1.9%	1.0%	74	76	2	
			Pressure 2	--	41	41	44.7	44.7	9.0%	9.0%	0	0	0	Hydrant P-2 in Elena Zone (no residual drop)
5	Morro Ave south of Scott St (Lower Kings)	8:57	Flowing	700	58	48	53	--	-8.6%		--	--	--	
			Pressure 1	--	54	48	54	46.4	0.0%	-3.3%	6	7.6	2	
			Pressure 2	--	60	51	61.5	54.1	2.5%	6.1%	9	7.4	-2	
6	Embarcadero Rd (Lower Kings)	~ 9:10	Flowing	930	86	52	83.3	--	-3.1%		--	--	--	
			Pressure 1	--	81	60	81.5	64.9	0.6%	8.2%	21	16.6	-4	
			Pressure 2	--	81	59	83.5	64.8	3.1%	9.8%	22	18.7	-3	
7	Estero Ave south of Pacific St (Upper Kings)	9:27	Flowing	1020	70	60	73.1	--	4.4%		--	--	--	
			Pressure 1	--	76	66	75.6	67.7	-0.5%	2.6%	10	7.9	-2	
			Pressure 2	--	69	57	68.4	57.3	-0.9%	0.5%	12	11.1	-1	
8	Arcadia Ave and Ridgeway St (Ridgeway)	9:40	Flowing	700	73	48	71	--	-2.7%		--	--	--	
			Pressure 1	--	72	52	72.8	53.4	1.1%	2.7%	20	19.4	-1	Pressures off due to discrepancy from EPS calibration
			Pressure 2	--	59	40	60.6	42.6	2.7%	6.5%	19	18	-1	
9	Main St south of Walnut St (Upper Kings)	9:53	Flowing	1300	118	88	114.3	--	-3.1%		--	--	--	
			Pressure 1	--	115	92	114.6	87.4	-0.3%	-5.0%	23	27.2	4	
			Pressure 2	--	115	84	111.5	84.9	-3.0%	1.1%	31	26.6	-4	
10	Teresa Road west of South Bay Blvd (Upper Kings)	10:14	Flowing	720	126	56	99.6	--	**-21.0%**		--	--	--	Static pressure fluctuated significantly during test
			Pressure 1	--	102	36	101.2	33.8	-0.8%	-6.1%	66	67.4	1	
			Pressure 2	--	100	37	99.8	33.7	-0.2%	-8.9%	63	66.1	3	

Appendix I

DWF CALIBRATION SHEETS

Table 1 - Average Dry Weather Flow Calibration Summary

Meter Number	Pipe Diameter (in)	Weekday									Weekend									Overall ADWF		
		Measured Data[1]			Modeled Data[2]			Percent Error[3]			Measured Data[1]			Modeled Data[2]			Percent Error[3]					
		Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Measured (mgd)	Modeled (mgd)	Percent Error (%)
Flume	27	0.89	--	--	0.893	--	--	0.2%	--	--	0.99	--	--	0.99	--	--	0.1%	--	--	0.92	0.92	0.1%
1	11.75	0.18	1.57	3.5	0.183	1.51	3.4	0.9%	-3.3%	-2.7%	0.18	1.66	3.3	0.19	1.52	3.4	2.1%	-8.2%	3.7%	0.18	0.18	1.3%
2	8	0.03	1.09	1.2	0.026	1.19	1.2	0.2%	8.4%	-4.5%	0.03	1.16	1.3	0.03	1.22	1.3	-0.3%	5.2%	-3.3%	0.03	0.03	0.1%
3	15	0.29	1.38	4.3	0.277	1.35	4.4	-2.8%	-2.2%	0.2%	0.32	1.44	4.5	0.31	1.41	4.6	-1.2%	-2.2%	1.4%	0.29	0.29	-2.3%
4	17.5	0.21	1.13	5.4	0.219	0.91	4.3	2.2%	-19.5%	-20.5%	0.25	1.21	5.7	0.25	0.99	4.6	1.3%	-18.4%	-18.8%	0.22	0.23	1.9%
5	8	0.09	2.54	1.7	0.087	2.66	1.5	0.3%	4.5%	-8.8%	0.10	2.78	1.8	0.10	2.78	1.7	0.2%	0.0%	-6.0%	0.09	0.09	0.3%
6	13	0.11	1.39	2.5	0.110	1.31	2.5	0.5%	-5.8%	2.5%	0.12	1.32	2.6	0.12	1.34	2.6	0.3%	1.3%	1.3%	0.11	0.11	0.5%
7	9.75	0.04	0.84	1.9	0.039	0.90	1.7	-0.8%	6.6%	-8.4%	0.04	0.88	2.0	0.04	0.92	1.8	-0.7%	4.8%	-6.7%	0.04	0.04	-0.8%
8	13	0.07	0.41	4.0	0.068	0.44	4.0	0.8%	7.0%	-1.3%	0.07	0.43	4.1	0.08	0.45	4.1	0.8%	3.6%	2.0%	0.07	0.07	0.8%
9	12	0.02	0.48	2.1	0.019	0.46	2.1	-1.8%	-4.1%	1.4%	0.02	0.49	2.3	0.02	0.48	2.2	0.0%	-2.4%	-5.1%	0.02	0.02	-1.3%
10	8	0.07	1.32	2.2	0.071	1.45	2.1	1.7%	9.5%	-5.3%	0.07	1.34	2.2	0.07	1.46	2.1	6.0%	8.9%	-4.2%	0.07	0.07	2.9%
11	8	0.06	1.42	1.9	0.064	1.50	1.9	3.7%	5.8%	-1.0%	0.07	1.48	2.0	0.07	1.53	2.0	2.3%	3.9%	-3.8%	0.06	0.07	3.2%
12	14.375	0.12	0.81	4.9	0.123	0.79	4.8	-0.1%	-2.2%	-1.2%	0.14	0.86	5.1	0.14	0.82	5.0	0.2%	-4.5%	-2.7%	0.13	0.13	0.0%
13	15.5	0.09	0.47	4.6	0.089	0.47	4.7	0.0%	1.6%	1.7%	0.10	0.50	4.7	0.10	0.50	4.9	-0.3%	-0.5%	4.2%	0.09	0.09	-0.1%
14	10	0.09	2.77	1.4	0.089	2.84	1.4	1.3%	2.6%	-6.1%	0.10	3.07	1.5	0.10	2.94	1.4	-0.2%	-4.0%	-3.0%	0.09	0.09	0.8%
15	10	0.04	1.08	1.7	0.043	1.02	1.7	0.1%	-5.4%	-3.4%	0.05	1.14	1.8	0.05	1.08	1.8	-0.1%	-5.9%	-0.8%	0.05	0.05	0.0%
MB-1	12	0.17	1.65	3.9	0.181	1.42	3.6	8.8%	-14.1%	-7.5%	0.17	1.63	4.2	0.19	1.43	3.7	12.3%	-12.0%	-12.7%	0.17	0.18	9.8%
MB-2	6	0.03	1.28	1.6	0.023	1.30	1.1	-18.3%	1.8%	-27.4%	0.03	1.50	1.6	0.03	1.33	1.2	-19.3%	-11.1%	-25.0%	0.03	0.02	-18.6%
MB-3	15	0.29	1.64	5.0	0.275	1.46	4.1	-3.9%	-10.8%	-17.3%	0.32	1.74	5.2	0.31	1.51	4.3	-3.1%	-13.2%	-17.0%	0.30	0.29	-3.7%
MB-4	18	0.21	1.72	2.8	0.221	1.57	3.1	5.5%	-8.8%	8.2%	0.26	1.82	3.1	0.25	1.61	3.3	-1.4%	-11.6%	6.3%	0.22	0.23	3.2%

Table 1 - Average Dry Weather Flow Calibration Summary

Meter Number	Pipe Diameter (in)	Weekday									Weekend									Overall ADWF		
		Measured Data[1]			Modeled Data[2]			Percent Error[3]			Measured Data[1]			Modeled Data[2]			Percent Error[3]					
		Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Measured (mgd)	Modeled (mgd)	Percent Error (%)
LS-1	--	0.04	--	--	0.040	--	--	-0.6%	--	--	0.05	--	--	0.04	--	--	-10.2%	--	--	0.04	0.04	-3.7%
LS-2	--	0.18	--	--	0.214	--	--	17.6%	--	--	0.19	--	--	0.25	--	--	28.3%	--	--	0.18	0.22	20.8%
LS-3	--	0.04	--	--	0.036	--	--	-0.3%	--	--	0.04	--	--	0.04	--	--	0.0%	--	--	0.04	0.04	-0.2%

Notes:

1. Source: V&A Temporary Flow Monitoring Program and City Of Morro Bay SCADA Data
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
3. Percent Difference = (Modeled - Measured)/Measured*100.

Flow Monitoring Site Flume, Dry Weather Flow Calibration

Location: WWTP 27-inch Pakmer-Bowlus Flume
Pipeline Diameter: 27"
City Manhole ID: OFALL-10
Model Pipe ID: CDT-43

Flow Monitor Location

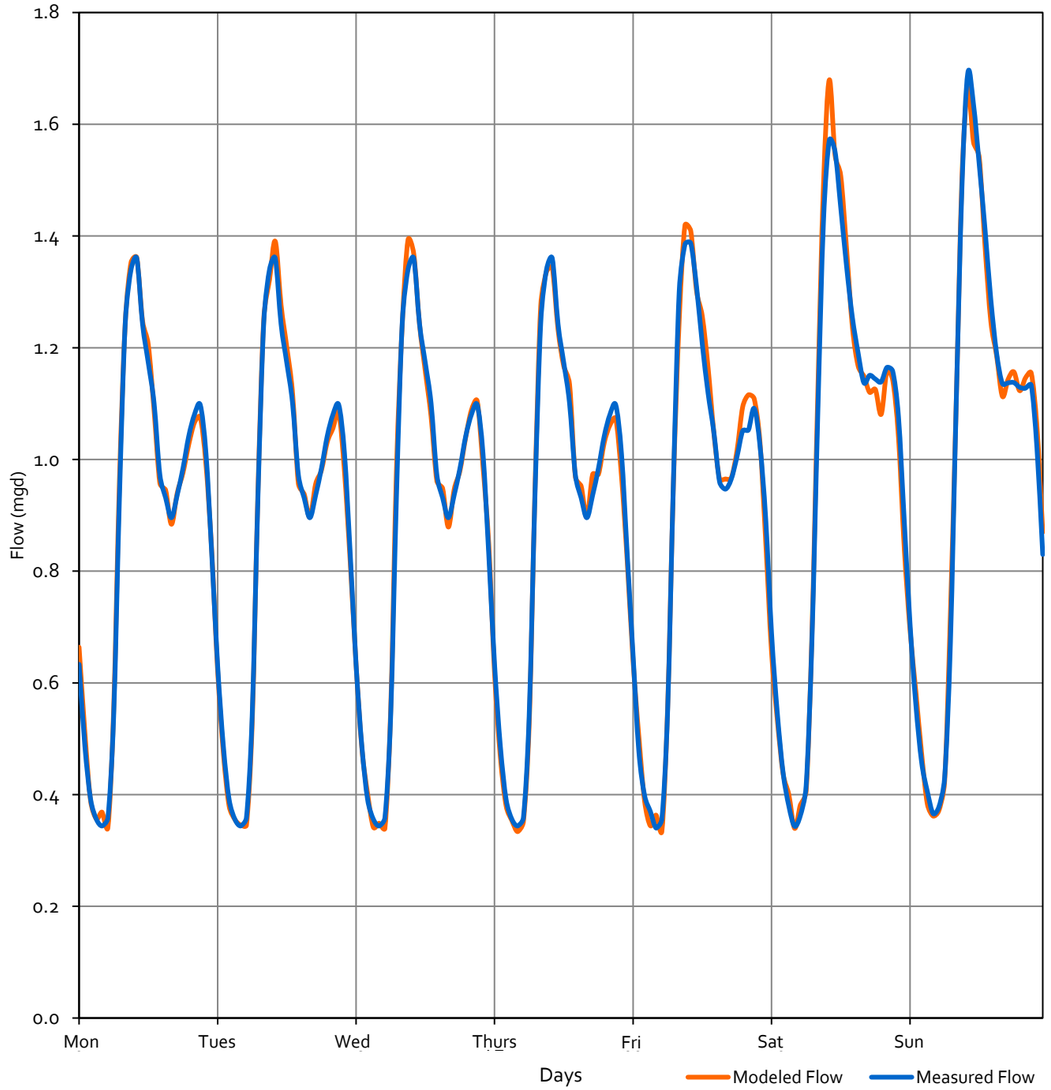
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.89	1.36	--	--	0.89	1.39	--	--	0.0%	2.4%	--	--
Tues.	0.89	1.36	--	--	0.89	1.39	--	--	0.0%	2.4%	--	--
Wed.	0.89	1.36	--	--	0.89	1.39	--	--	0.0%	2.4%	--	--
Thur.	0.89	1.36	--	--	0.89	1.39	--	--	0.0%	2.4%	--	--
Fri.	0.91	1.39	--	--	0.92	1.42	--	--	0.9%	2.3%	--	--
Sat.	0.98	1.57	--	--	0.98	1.68	--	--	0.2%	6.9%	--	--
Sun.	1.00	1.69	--	--	1.00	1.66	--	--	0.1%	-1.6%	--	--
Summary												
Weekday	0.89	--	--	--	0.89	--	--	--	0.2%	--	--	--
Weekend	0.99	--	--	--	0.99	--	--	--	0.1%	--	--	--
ADWF[4]	0.92	--	--	--	0.92	--	--	--	0.1%	--	--	--

Notes:

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

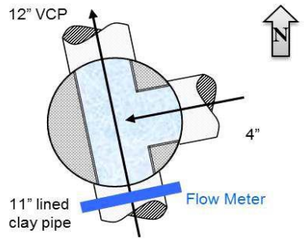
Flow Calibration



Flow Monitoring Site 1, Dry Weather Flow Calibration

Location: Main St. ~225-feet south of Errol St.
Pipeline Diameter: 11.75"
City Manhole ID: 13.38
Model Pipe ID: 1010

Flow Monitor Location



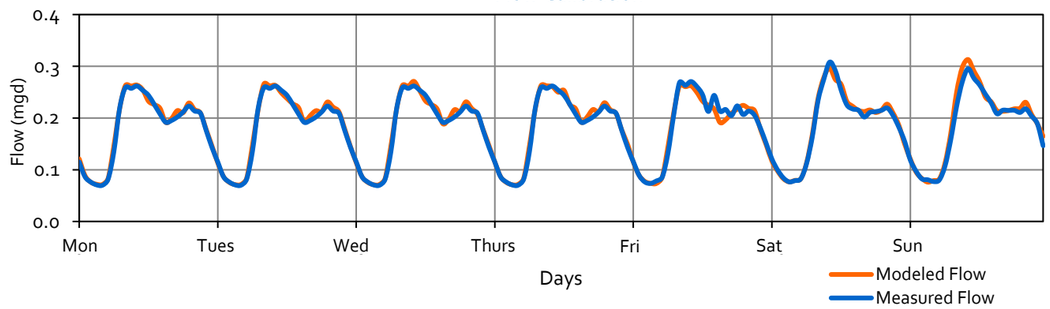
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.18	0.26	3.5	1.57	0.18	0.27	3.4	1.51	1.5%	3.4%	-2.8%	-3.5%
Tues.	0.18	0.26	3.5	1.57	0.18	0.27	3.4	1.51	1.5%	3.4%	-2.8%	-3.5%
Wed.	0.18	0.26	3.5	1.57	0.18	0.27	3.4	1.51	1.5%	3.4%	-2.8%	-3.5%
Thur.	0.18	0.26	3.5	1.57	0.18	0.27	3.4	1.51	1.5%	3.4%	-2.8%	-3.5%
Fri.	0.18	0.27	3.5	1.56	0.18	0.26	3.4	1.52	-1.3%	-2.9%	-2.4%	-2.8%
Sat.	0.18	0.31	3.3	1.66	0.19	0.30	3.4	1.52	0.6%	-3.6%	3.2%	-8.5%
Sun.	0.18	0.30	3.3	1.66	0.19	0.31	3.5	1.53	3.6%	6.0%	4.3%	-7.9%
Summary												
Weekday	0.18	--	3.5	1.57	0.18	--	3.4	1.51	0.9%	--	-2.7%	-3.3%
Weekend	0.18	--	3.3	1.66	0.19	--	3.4	1.52	2.1%	--	3.7%	-8.2%
ADWF[4]	0.18	--	3.4	1.59	0.18	--	3.4	1.52	1.3%	--	-0.9%	-4.8%

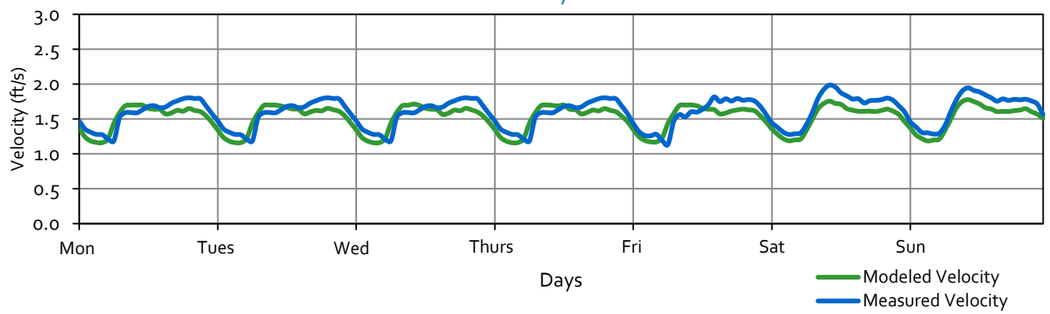
Notes:

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

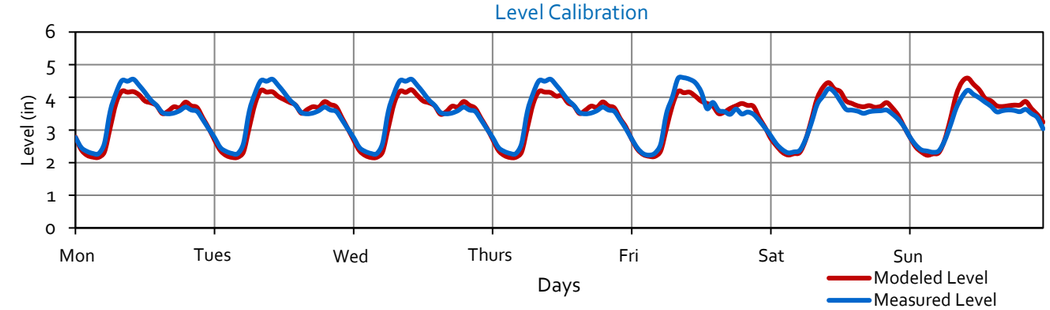
Flow Calibration



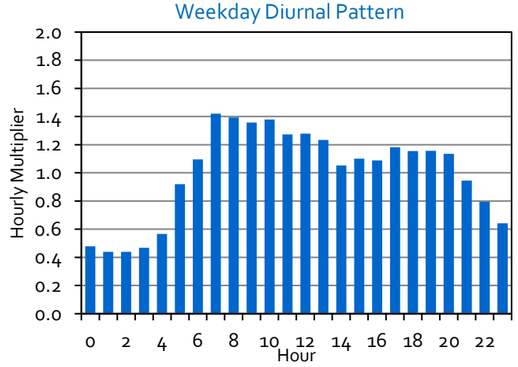
Velocity Calibration



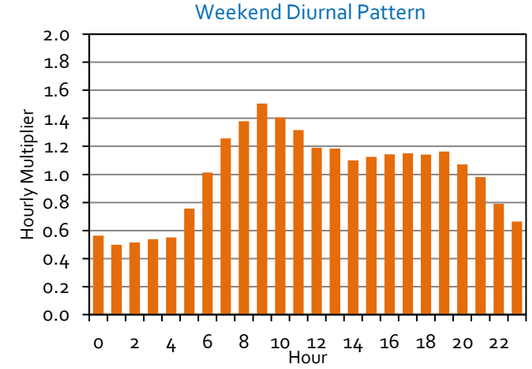
Level Calibration



Weekday Diurnal Pattern



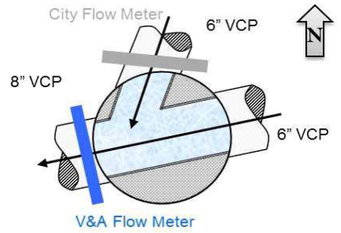
Weekend Diurnal Pattern

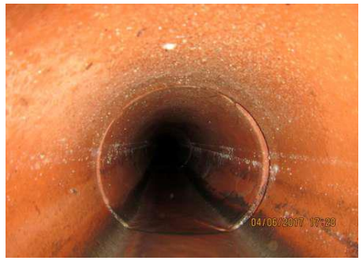


Flow Monitoring Site 2, Dry Weather Flow Calibration

Location: Atascadero Rd. ~85-feet east of Sunset Ave.
Pipeline Diameter: 8"
City Manhole ID: 5.19
Model Pipe ID: 1118

Flow Monitor Location





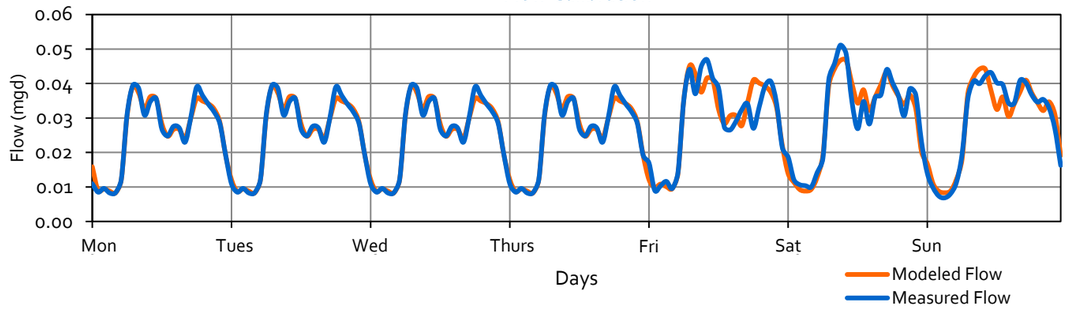
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.03	0.04	1.2	1.08	0.03	0.04	1.2	1.18	0.2%	-0.6%	-4.9%	9.3%
Tues.	0.03	0.04	1.2	1.08	0.03	0.04	1.2	1.18	0.2%	-0.6%	-4.9%	9.3%
Wed.	0.03	0.04	1.2	1.08	0.03	0.04	1.2	1.18	0.2%	-0.6%	-4.9%	9.3%
Thur.	0.03	0.04	1.2	1.08	0.03	0.04	1.2	1.18	0.2%	-0.6%	-4.9%	9.3%
Fri.	0.03	0.05	1.3	1.17	0.03	0.05	1.3	1.22	0.3%	-3.9%	-2.8%	4.9%
Sat.	0.03	0.05	1.3	1.20	0.03	0.05	1.3	1.23	-0.4%	-8.7%	-2.1%	2.5%
Sun.	0.03	0.04	1.3	1.12	0.03	0.04	1.2	1.21	-0.2%	2.3%	-4.5%	8.2%
Summary												
Weekday	0.03	--	1.2	1.09	0.03	--	1.2	1.19	0.2%	--	-4.5%	8.4%
Weekend	0.03	--	1.3	1.16	0.03	--	1.3	1.22	-0.3%	--	-3.3%	5.2%
ADWF[4]	0.03	--	1.3	1.11	0.03	--	1.2	1.20	0.1%	--	-4.2%	7.5%

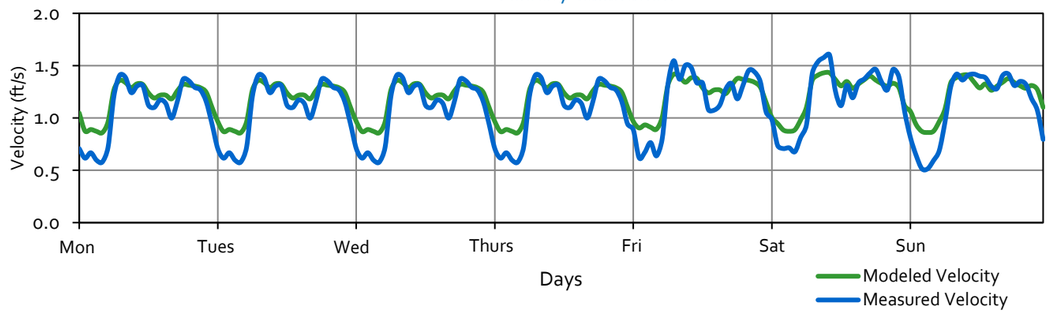
Notes:

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

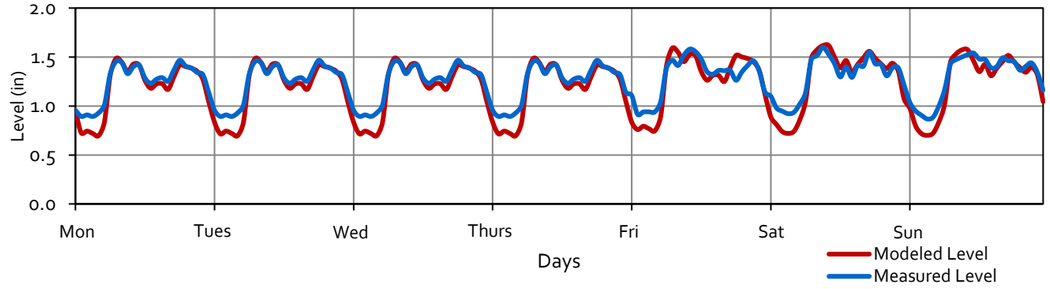
Flow Calibration



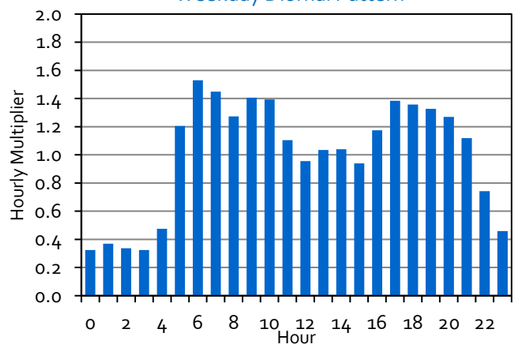
Velocity Calibration



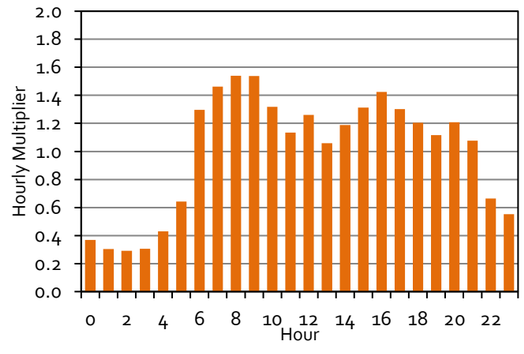
Level Calibration



Weekday Diurnal Pattern



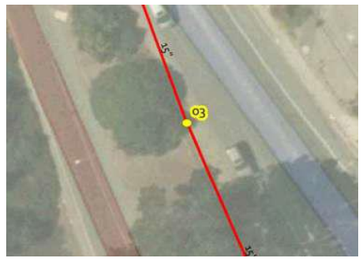
Weekend Diurnal Pattern

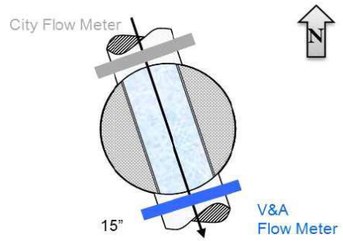


Flow Monitoring Site 3, Dry Weather Flow Calibration

Location: Island between Main St. and Hwy 1 on-ramp, ~135-feet north of Atascadero Rd.
Pipeline Diameter: 15"
City Manhole ID: 5.25A
Model Pipe ID: 1119

Flow Monitor Location





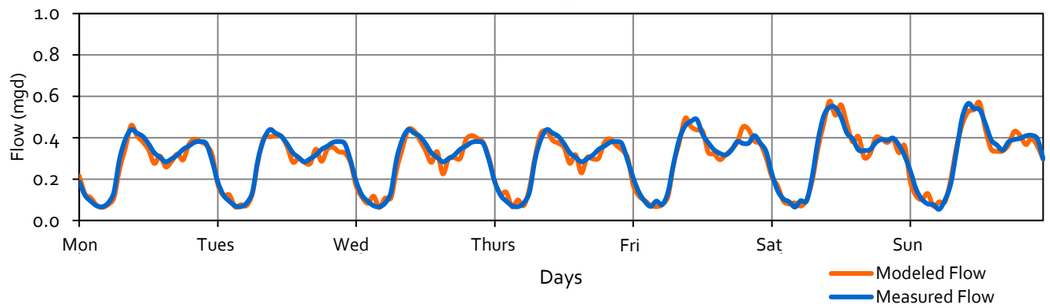
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.28	0.44	4.3	1.37	0.27	0.46	4.3	1.35	-3.0%	4.7%	-0.1%	-1.7%
Tues.	0.28	0.44	4.3	1.37	0.27	0.46	4.3	1.35	-3.0%	4.7%	-0.1%	-1.7%
Wed.	0.28	0.44	4.3	1.37	0.27	0.46	4.3	1.35	-3.0%	4.7%	-0.1%	-1.7%
Thur.	0.28	0.44	4.3	1.37	0.27	0.46	4.3	1.35	-3.0%	4.7%	-0.1%	-1.7%
Fri.	0.30	0.49	4.4	1.43	0.30	0.49	4.5	1.38	-2.0%	1.2%	1.3%	-3.9%
Sat.	0.32	0.55	4.5	1.45	0.32	0.58	4.6	1.41	0.1%	5.3%	2.2%	-2.6%
Sun.	0.32	0.57	4.5	1.43	0.31	0.57	4.6	1.41	-2.4%	0.7%	0.7%	-1.9%
Summary												
Weekday	0.29	--	4.3	1.38	0.28	--	4.4	1.35	-2.8%	--	0.2%	-2.2%
Weekend	0.32	--	4.5	1.44	0.31	--	4.6	1.41	-1.2%	--	1.4%	-2.2%
ADWF[4]	0.29	--	4.4	1.40	0.29	--	4.4	1.37	-2.3%	--	0.5%	-2.2%

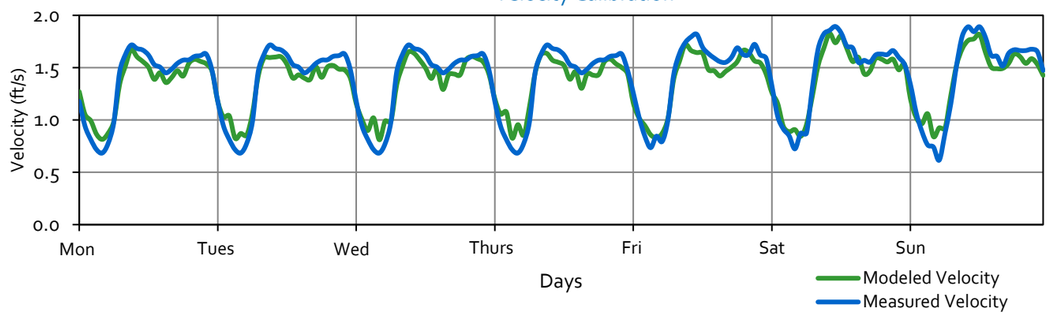
Notes:

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

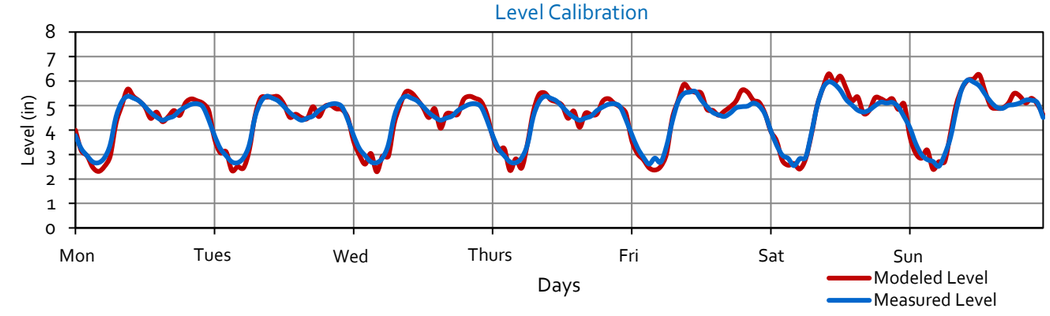
Flow Calibration



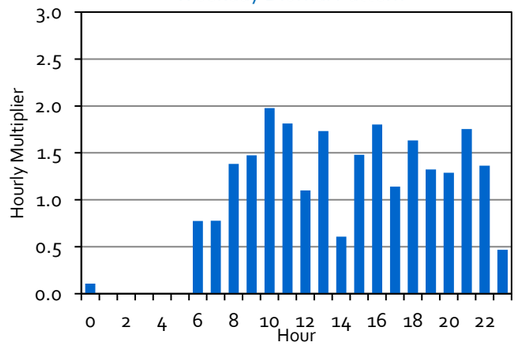
Velocity Calibration



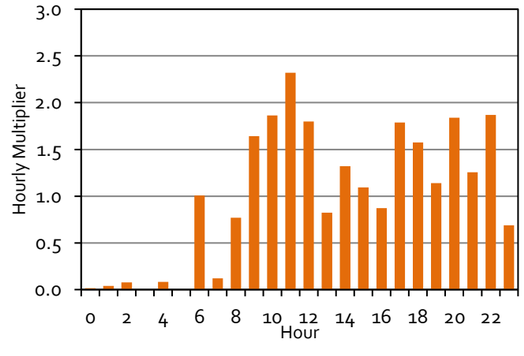
Level Calibration



Weekday Diurnal Pattern



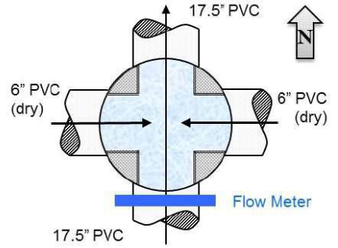
Weekend Diurnal Pattern



Flow Monitoring Site 4, Dry Weather Flow Calibration

Location: Embarcadero, ~685-feet north of Colman Dr.
Pipeline Diameter: 17.5"
City Manhole ID: 14.2
Model Pipe ID: 38

Flow Monitor Location



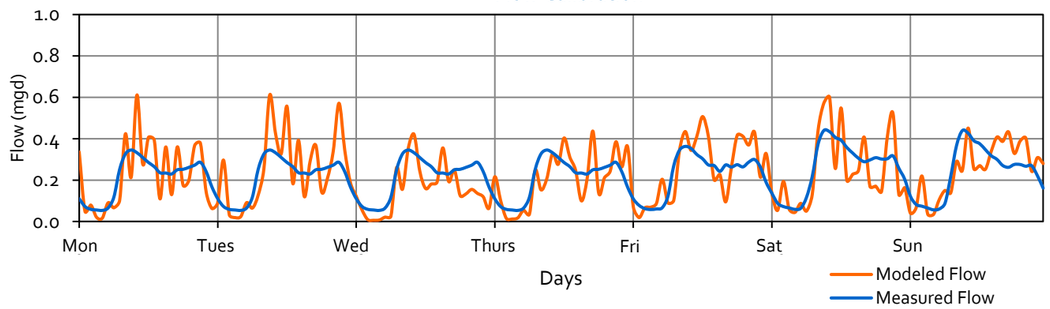
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.21	0.35	5.4	1.13	0.21	0.61	4.2	0.88	-0.7%	77.0%	-21.1%	-22.1%
Tues.	0.21	0.35	5.4	1.13	0.21	0.61	4.2	0.88	-0.7%	77.0%	-21.1%	-22.1%
Wed.	0.21	0.35	5.4	1.13	0.21	0.61	4.2	0.88	-0.7%	77.0%	-21.1%	-22.1%
Thur.	0.21	0.35	5.4	1.13	0.21	0.61	4.2	0.88	-0.7%	77.0%	-21.1%	-22.1%
Fri.	0.23	0.36	5.5	1.13	0.25	0.51	4.5	1.03	13.0%	39.8%	-18.1%	-9.2%
Sat.	0.26	0.44	5.7	1.21	0.25	0.60	4.6	0.95	-2.9%	36.5%	-19.0%	-21.4%
Sun.	0.24	0.44	5.7	1.21	0.25	0.45	4.6	1.02	5.9%	2.8%	-18.5%	-15.4%
Summary												
Weekday	0.21	--	5.4	1.13	0.22	--	4.3	0.91	2.2%	--	-20.5%	-19.5%
Weekend	0.25	--	5.7	1.21	0.25	--	4.6	0.99	1.3%	--	-18.8%	-18.4%
ADWF[4]	0.22	--	5.5	1.15	0.23	--	4.4	0.93	1.9%	--	-20.0%	-19.2%

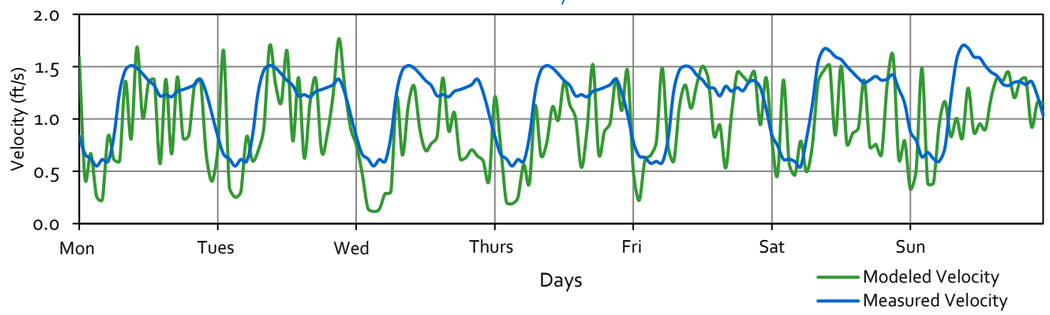
Notes:

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

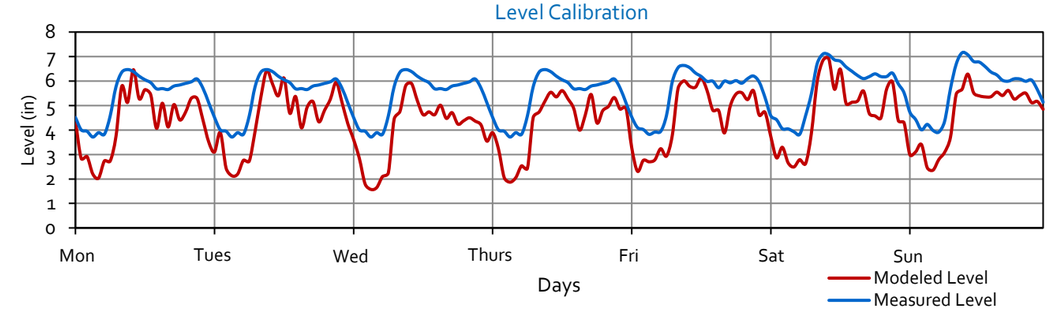
Flow Calibration



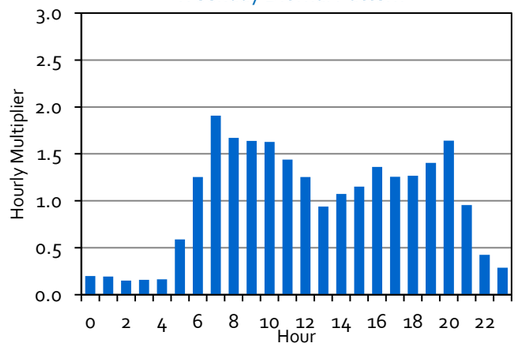
Velocity Calibration



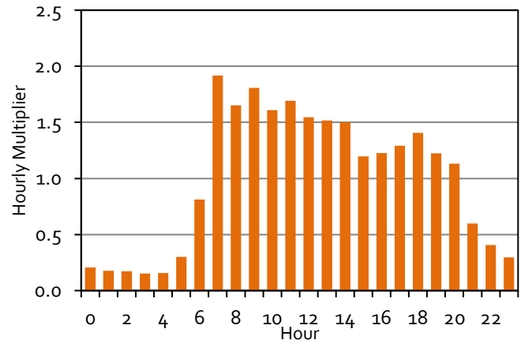
Level Calibration



Weekday Diurnal Pattern



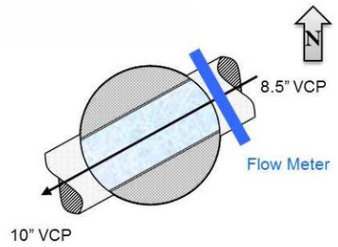
Weekend Diurnal Pattern



Flow Monitoring Site 5, Dry Weather Flow Calibration

Location: 371 Avalon St.
Pipeline Diameter: 8"
City Manhole ID: 4.89
Model Pipe ID: 525

Flow Monitor Location



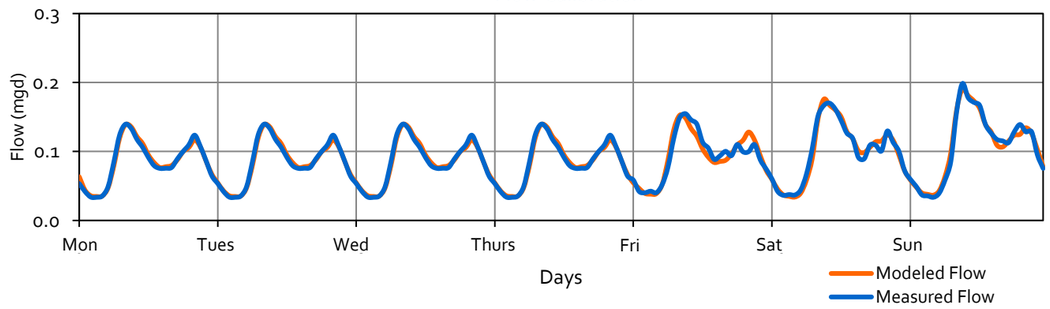
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.08	0.14	1.7	2.52	0.09	0.14	1.5	2.64	0.5%	-0.5%	-9.0%	5.0%
Tues.	0.08	0.14	1.7	2.52	0.09	0.14	1.5	2.64	0.5%	-0.5%	-9.0%	5.0%
Wed.	0.08	0.14	1.7	2.52	0.09	0.14	1.5	2.64	0.5%	-0.5%	-9.0%	5.0%
Thur.	0.08	0.14	1.7	2.52	0.09	0.14	1.5	2.64	0.5%	-0.5%	-9.0%	5.0%
Fri.	0.09	0.15	1.7	2.64	0.09	0.15	1.6	2.71	-0.3%	-1.7%	-7.8%	2.5%
Sat.	0.10	0.17	1.7	2.75	0.10	0.18	1.6	2.75	0.2%	3.3%	-6.6%	0.0%
Sun.	0.11	0.20	1.8	2.81	0.11	0.19	1.7	2.81	0.1%	-4.1%	-5.5%	0.0%
Summary												
Weekday	0.09	--	1.7	2.54	0.09	--	1.5	2.66	0.3%	--	-8.8%	4.5%
Weekend	0.10	--	1.8	2.78	0.10	--	1.7	2.78	0.2%	--	-6.0%	0.0%
ADWF[4]	0.09	--	1.7	2.61	0.09	--	1.6	2.69	0.3%	--	-8.0%	3.1%

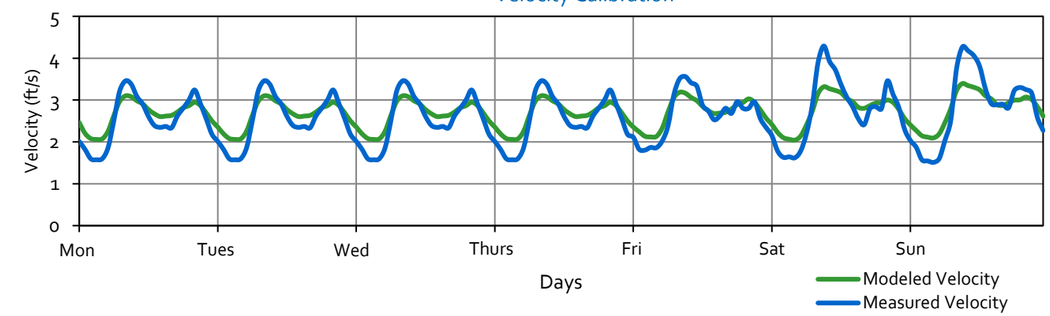
Notes:

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

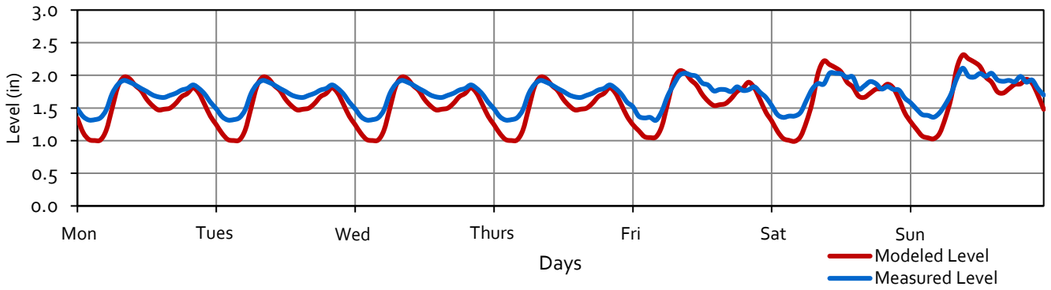
Flow Calibration



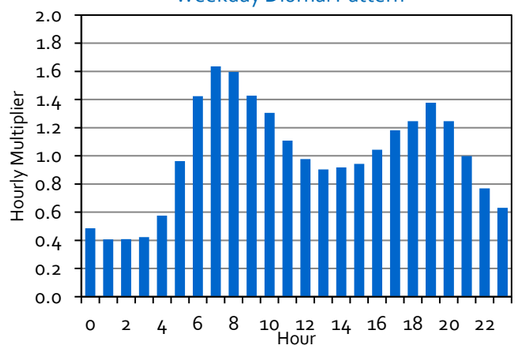
Velocity Calibration



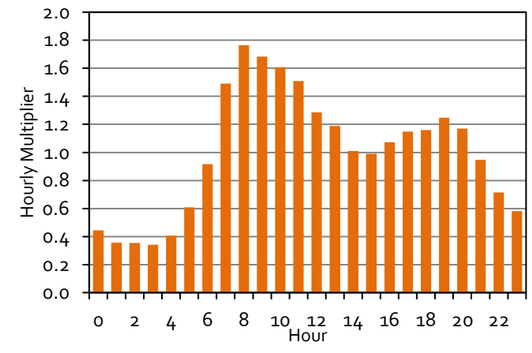
Level Calibration



Weekday Diurnal Pattern



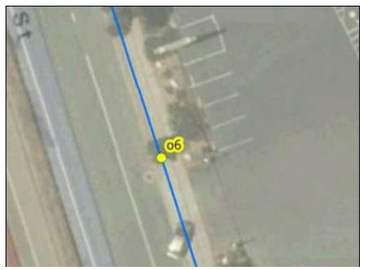
Weekend Diurnal Pattern

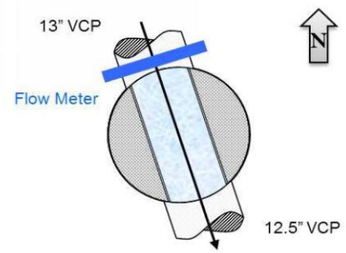


Flow Monitoring Site 6, Dry Weather Flow Calibration

Location: Main St. ~290-feet south of Elena St.
Pipeline Diameter: 13"
City Manhole ID: 3.58
Model Pipe ID: 140

Flow Monitor Location





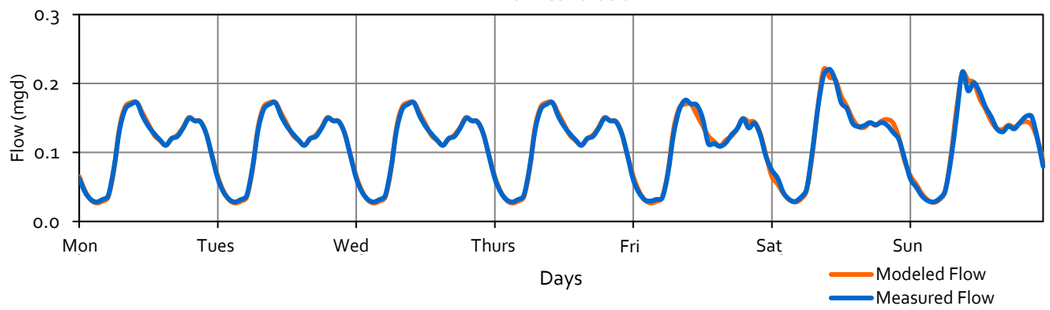
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.11	0.17	2.5	1.39	0.11	0.17	2.5	1.31	0.6%	0.3%	3.1%	-5.3%
Tues.	0.11	0.17	2.5	1.39	0.11	0.17	2.5	1.31	0.6%	0.3%	3.1%	-5.3%
Wed.	0.11	0.17	2.5	1.39	0.11	0.17	2.5	1.31	0.6%	0.3%	3.1%	-5.3%
Thur.	0.11	0.17	2.5	1.39	0.11	0.17	2.5	1.31	0.6%	0.3%	3.1%	-5.3%
Fri.	0.11	0.18	2.5	1.42	0.11	0.17	2.5	1.31	0.5%	-3.3%	0.3%	-7.9%
Sat.	0.12	0.22	2.6	1.39	0.12	0.22	2.6	1.34	0.3%	-0.1%	2.8%	-3.2%
Sun.	0.12	0.22	2.6	1.25	0.12	0.21	2.6	1.33	0.4%	-0.9%	-0.3%	6.3%
Summary												
Weekday	0.11	--	2.5	1.39	0.11	--	2.5	1.31	0.5%	--	2.5%	-5.8%
Weekend	0.12	--	2.6	1.32	0.12	--	2.6	1.34	0.3%	--	1.3%	1.3%
ADWF[4]	0.11	--	2.5	1.37	0.11	--	2.6	1.32	0.5%	--	2.2%	-3.9%

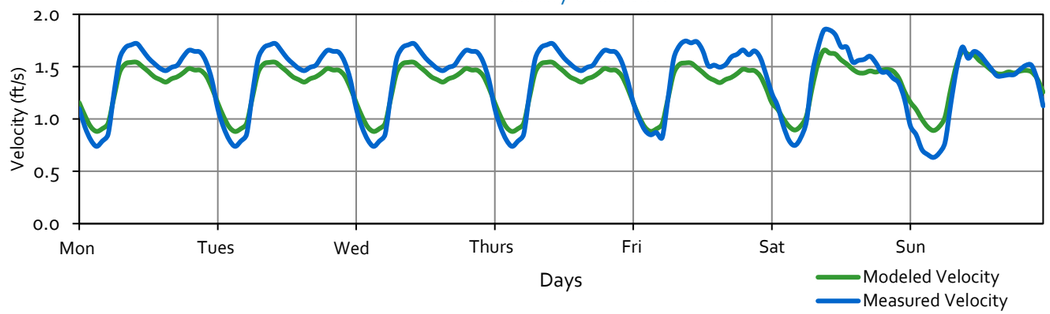
Notes:

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

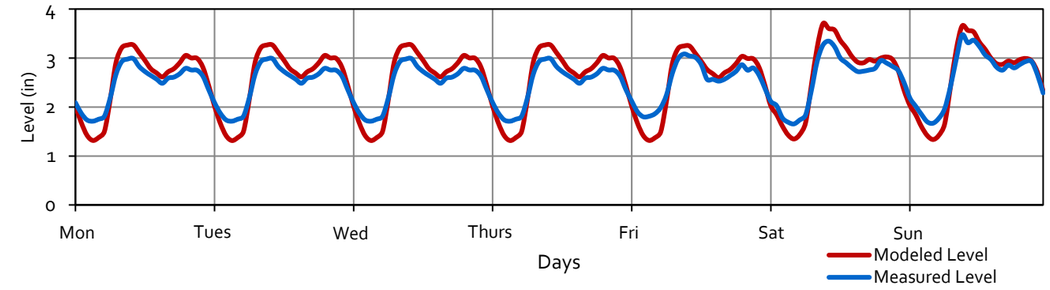
Flow Calibration



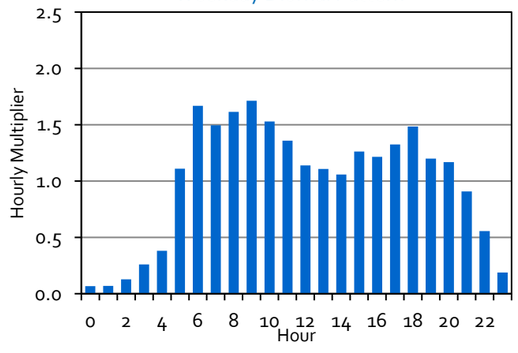
Velocity Calibration



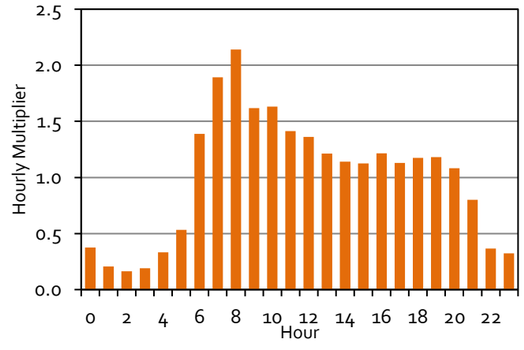
Level Calibration



Weekday Diurnal Pattern



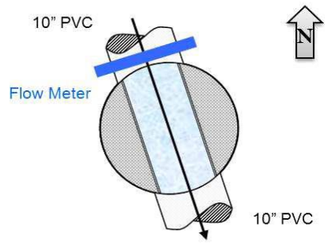
Weekend Diurnal Pattern



Flow Monitoring Site 7, Dry Weather Flow Calibration

Location: Coral Ave. ~630' north of Emerald Cir.
Pipeline Diameter: 9.75"
City Manhole ID: 2A.14
Model Pipe ID: 84

Flow Monitor Location



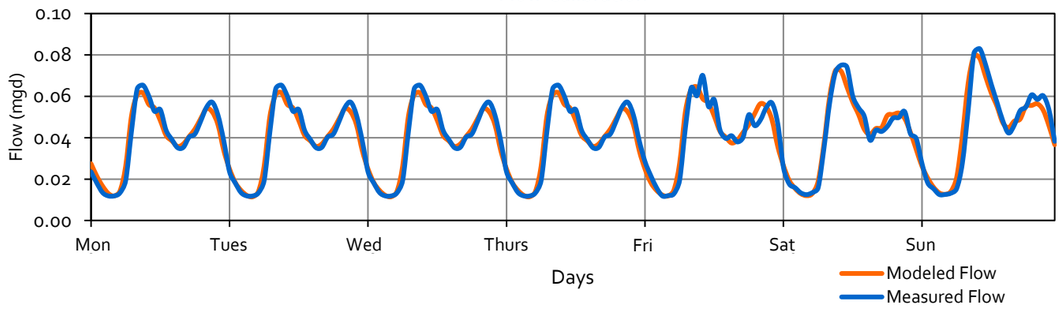
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.04	0.07	1.9	0.85	0.04	0.06	1.7	0.90	-0.7%	-5.5%	-8.6%	6.2%
Tues.	0.04	0.07	1.9	0.85	0.04	0.06	1.7	0.90	-0.7%	-5.5%	-8.6%	6.2%
Wed.	0.04	0.07	1.9	0.85	0.04	0.06	1.7	0.90	-0.7%	-5.5%	-8.6%	6.2%
Thur.	0.04	0.07	1.9	0.85	0.04	0.06	1.7	0.90	-0.7%	-5.5%	-8.6%	6.2%
Fri.	0.04	0.07	1.9	0.84	0.04	0.06	1.8	0.91	-1.4%	-8.0%	-7.7%	8.4%
Sat.	0.04	0.08	1.9	0.87	0.04	0.07	1.8	0.91	-0.8%	-3.4%	-8.1%	4.9%
Sun.	0.04	0.08	2.0	0.89	0.04	0.08	1.9	0.93	-0.6%	-4.3%	-5.3%	4.7%
Summary												
Weekday	0.04	--	1.9	0.84	0.04	--	1.7	0.90	-0.8%	--	-8.4%	6.6%
Weekend	0.04	--	2.0	0.88	0.04	--	1.8	0.92	-0.7%	--	-6.7%	4.8%
ADWF[4]	0.04	--	1.9	0.85	0.04	--	1.8	0.91	-0.8%	--	-7.9%	6.1%

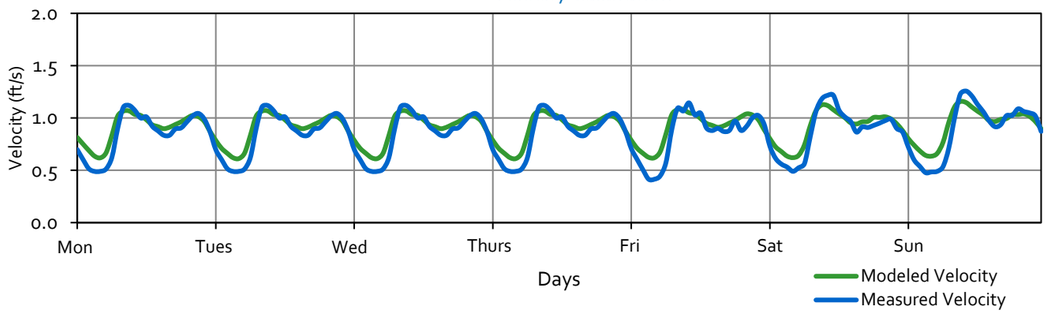
Notes:

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

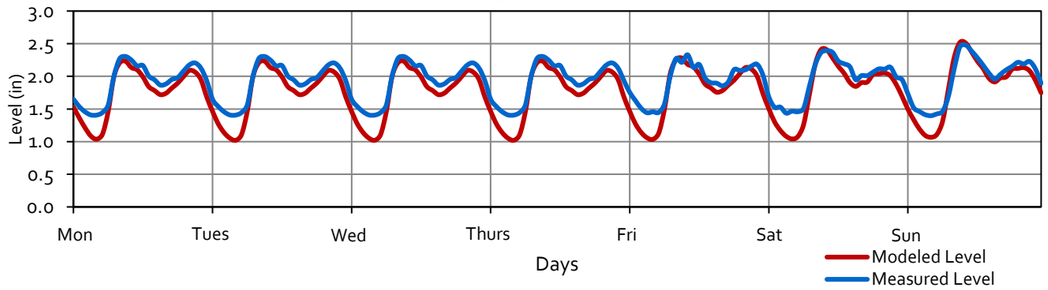
Flow Calibration



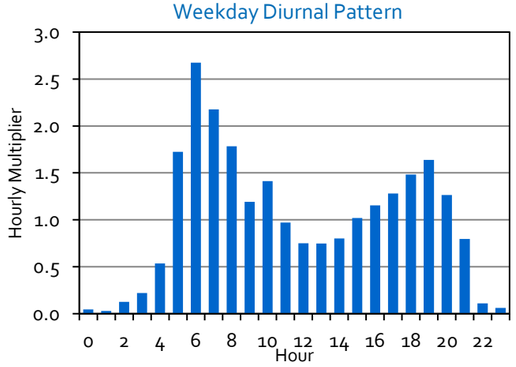
Velocity Calibration



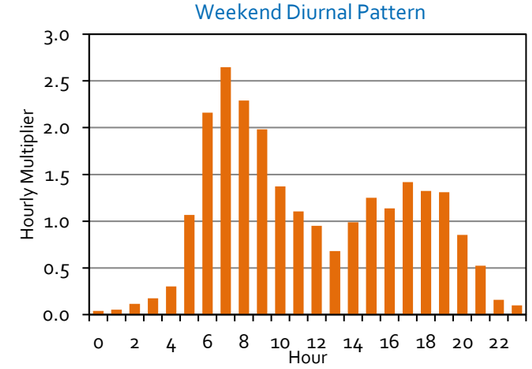
Level Calibration



Weekday Diurnal Pattern



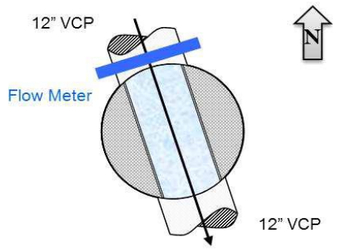
Weekend Diurnal Pattern



Flow Monitoring Site 8, Dry Weather Flow Calibration

Location: Main St. ~400-feet south of Island St.
Pipeline Diameter: 13"
City Manhole ID: 3.1
Model Pipe ID: 1126

Flow Monitor Location



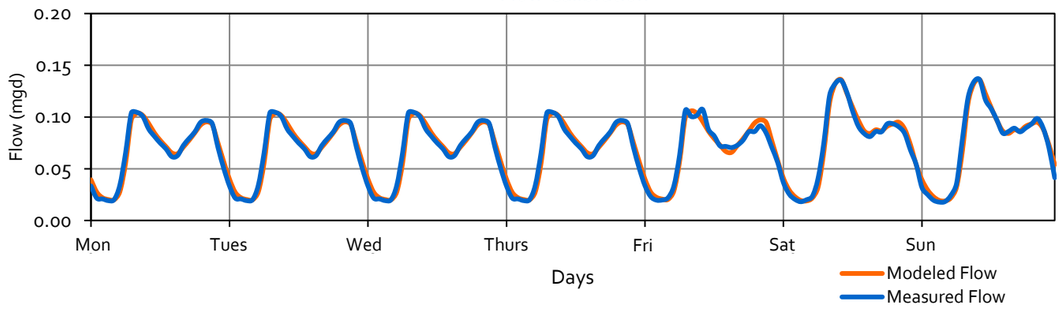
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.07	0.10	4.0	0.41	0.07	0.10	3.9	0.44	0.8%	-0.6%	-1.6%	7.2%
Tues.	0.07	0.10	4.0	0.41	0.07	0.10	3.9	0.44	0.8%	-0.6%	-1.6%	7.2%
Wed.	0.07	0.10	4.0	0.41	0.07	0.10	3.9	0.44	0.8%	-0.6%	-1.6%	7.2%
Thur.	0.07	0.10	4.0	0.41	0.07	0.10	3.9	0.44	0.8%	-0.6%	-1.6%	7.2%
Fri.	0.07	0.11	4.0	0.41	0.07	0.11	4.0	0.44	0.7%	-1.3%	0.1%	6.5%
Sat.	0.07	0.14	4.1	0.43	0.08	0.14	4.1	0.45	0.8%	0.6%	2.0%	3.3%
Sun.	0.07	0.14	4.0	0.43	0.08	0.14	4.1	0.45	0.7%	0.1%	2.1%	3.9%
Summary												
Weekday	0.07	--	4.0	0.41	0.07	--	4.0	0.44	0.8%	--	-1.3%	7.0%
Weekend	0.07	--	4.1	0.43	0.08	--	4.1	0.45	0.8%	--	2.0%	3.6%
ADWF[4]	0.07	--	4.0	0.42	0.07	--	4.0	0.44	0.8%	--	-0.3%	6.0%

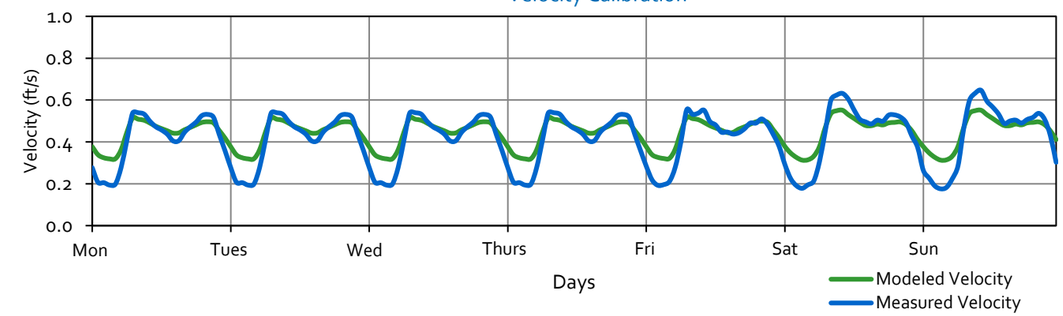
Notes:

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

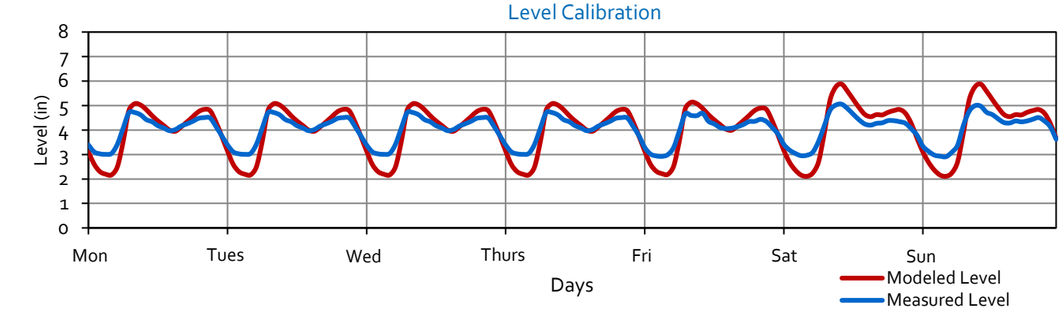
Flow Calibration



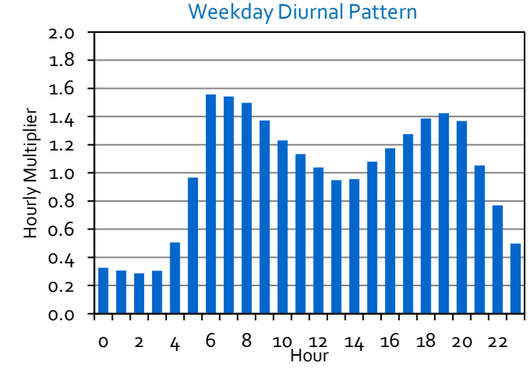
Velocity Calibration



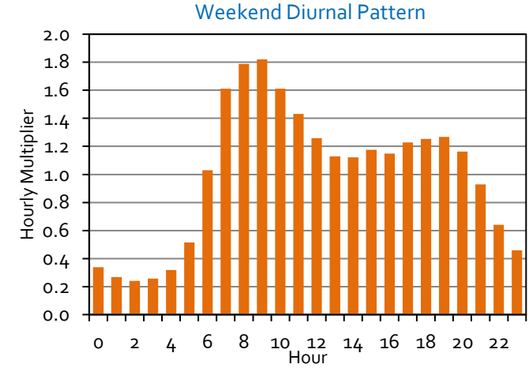
Level Calibration



Weekday Diurnal Pattern



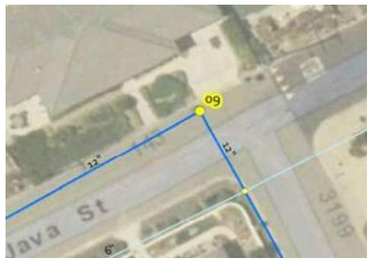
Weekend Diurnal Pattern

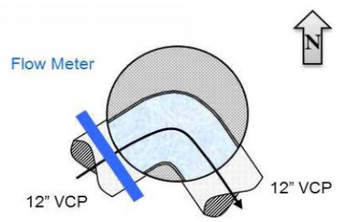


Flow Monitoring Site 9, Dry Weather Flow Calibration

Location: 125 Java St. (near intersection of Sandalwood Ave.)
Pipeline Diameter: 12"
City Manhole ID: 2.28
Model Pipe ID: 177

Flow Monitor Location





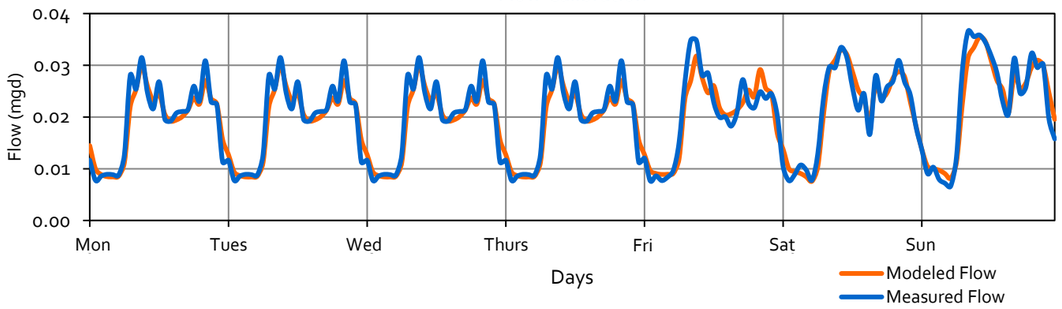
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.02	0.03	2.1	0.48	0.02	0.03	2.1	0.46	-2.0%	-5.2%	1.0%	-4.6%
Tues.	0.02	0.03	2.1	0.48	0.02	0.03	2.1	0.46	-2.0%	-5.2%	1.0%	-4.6%
Wed.	0.02	0.03	2.1	0.48	0.02	0.03	2.1	0.46	-2.0%	-5.2%	1.0%	-4.6%
Thur.	0.02	0.03	2.1	0.48	0.02	0.03	2.1	0.46	-2.0%	-5.2%	1.0%	-4.6%
Fri.	0.02	0.03	2.1	0.48	0.02	0.03	2.2	0.47	-1.2%	-8.6%	2.6%	-2.0%
Sat.	0.02	0.03	2.3	0.50	0.02	0.03	2.2	0.48	0.6%	-1.6%	-5.3%	-4.4%
Sun.	0.02	0.04	2.3	0.49	0.02	0.04	2.2	0.49	-0.5%	-3.1%	-4.8%	-0.4%
Summary												
Weekday	0.02	--	2.1	0.48	0.02	--	2.1	0.46	-1.8%	--	1.4%	-4.1%
Weekend	0.02	--	2.3	0.49	0.02	--	2.2	0.48	0.0%	--	-5.1%	-2.4%
ADWF[4]	0.02	--	2.2	0.48	0.02	--	2.2	0.46	-1.3%	--	-0.6%	-3.6%

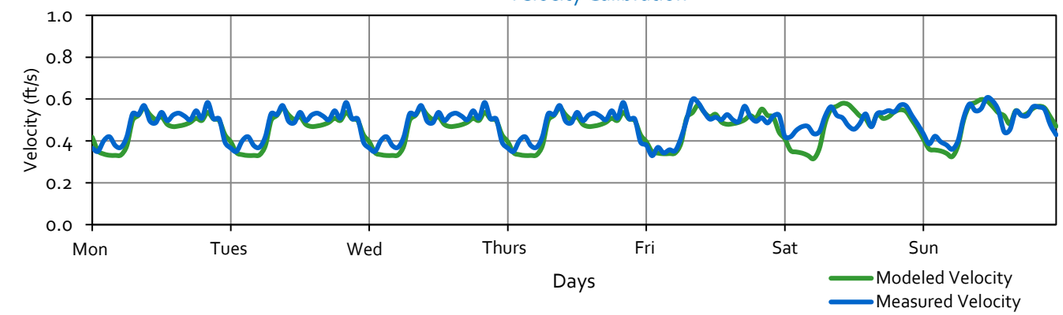
Notes:

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

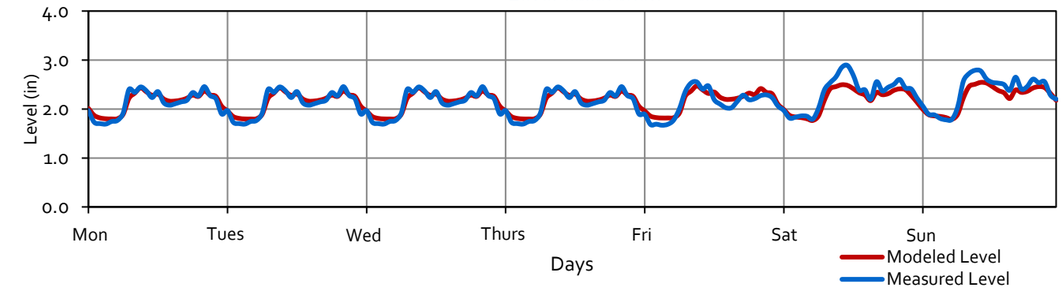
Flow Calibration



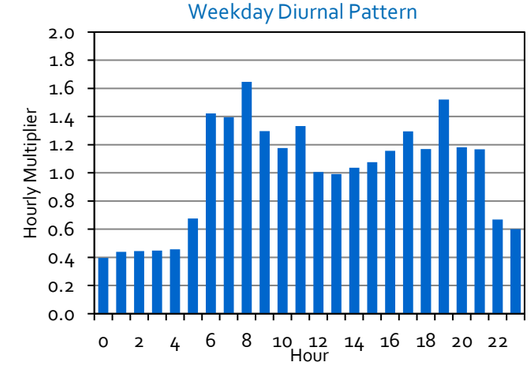
Velocity Calibration



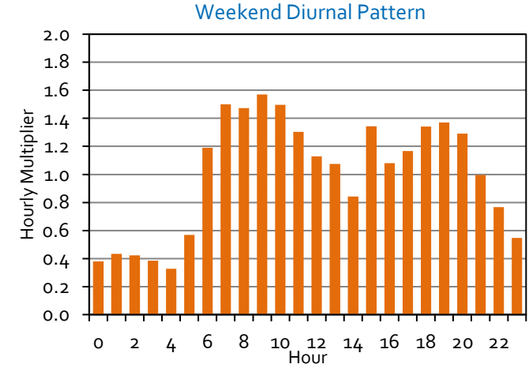
Level Calibration



Weekday Diurnal Pattern



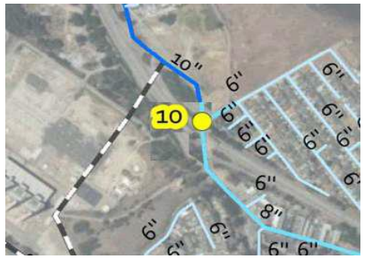
Weekend Diurnal Pattern

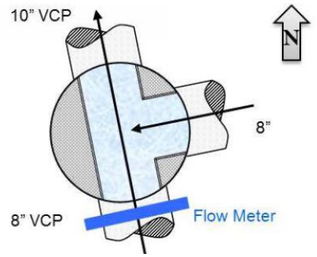


Flow Monitoring Site 10, Dry Weather Flow Calibration

Location: Main St. just north of intersection with Radcliff Ave.
Pipeline Diameter: 8"
City Manhole ID: 13.25
Model Pipe ID: 475

Flow Monitor Location





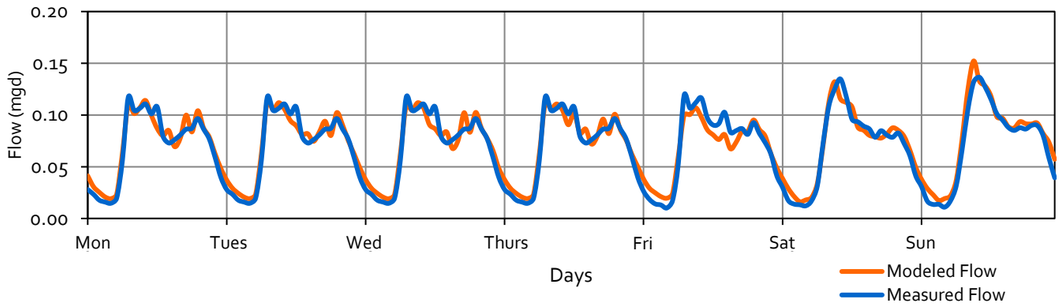
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.07	0.12	2.2	1.32	0.07	0.11	2.1	1.45	2.8%	-2.6%	-4.3%	10.1%
Tues.	0.07	0.12	2.2	1.32	0.07	0.11	2.1	1.45	2.8%	-2.6%	-4.3%	10.1%
Wed.	0.07	0.12	2.2	1.32	0.07	0.11	2.1	1.45	2.8%	-2.6%	-4.3%	10.1%
Thur.	0.07	0.12	2.2	1.32	0.07	0.11	2.1	1.45	2.8%	-2.6%	-4.3%	10.1%
Fri.	0.07	0.12	2.3	1.33	0.07	0.11	2.1	1.42	-2.9%	-10.1%	-9.2%	7.1%
Sat.	0.07	0.13	2.2	1.33	0.07	0.13	2.1	1.43	3.0%	-1.9%	-5.9%	7.3%
Sun.	0.07	0.14	2.2	1.34	0.08	0.15	2.2	1.49	8.9%	11.5%	-2.5%	10.5%
Summary												
Weekday	0.07	--	2.2	1.32	0.07	--	2.1	1.45	1.7%	--	-5.3%	9.5%
Weekend	0.07	--	2.2	1.34	0.07	--	2.1	1.46	6.0%	--	-4.2%	8.9%
ADWF[4]	0.07	--	2.2	1.33	0.07	--	2.1	1.45	2.9%	--	-5.0%	9.3%

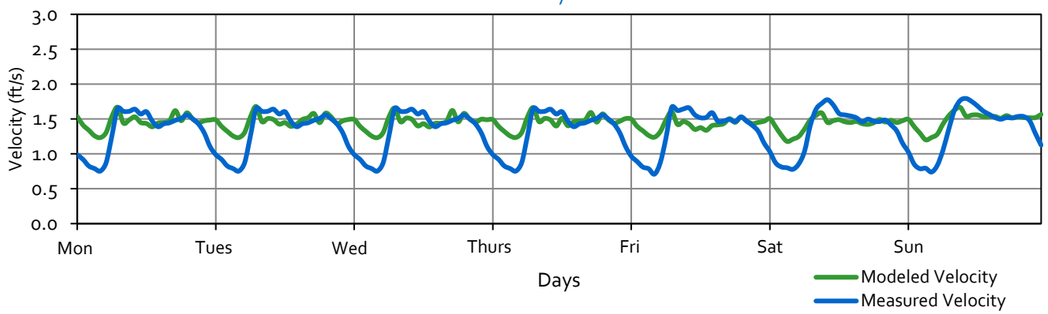
Notes:

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

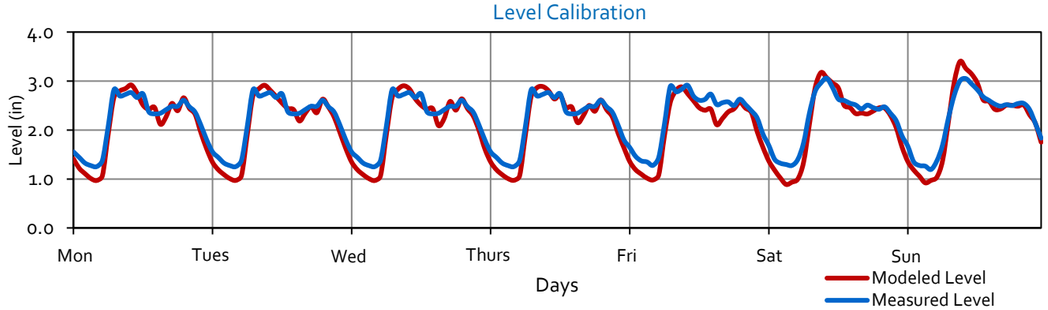
Flow Calibration



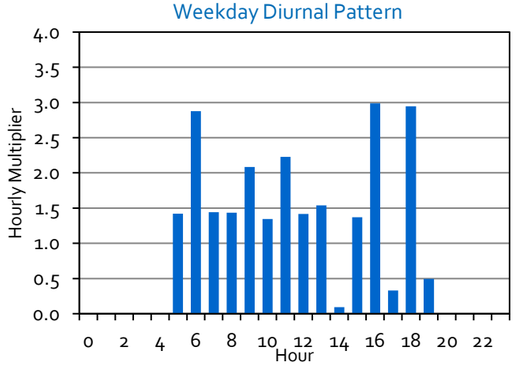
Velocity Calibration



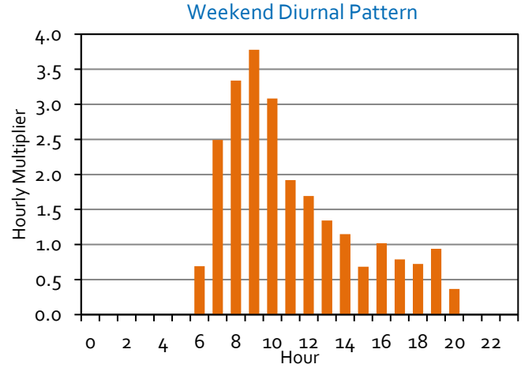
Level Calibration



Weekday Diurnal Pattern



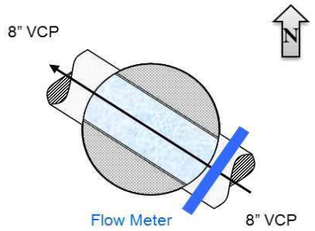
Weekend Diurnal Pattern



Flow Monitoring Site 11, Dry Weather Flow Calibration

Location: 785 Quintana Rd.
Pipeline Diameter: 8"
City Manhole ID: 8.4
Model Pipe ID: 775

Flow Monitor Location



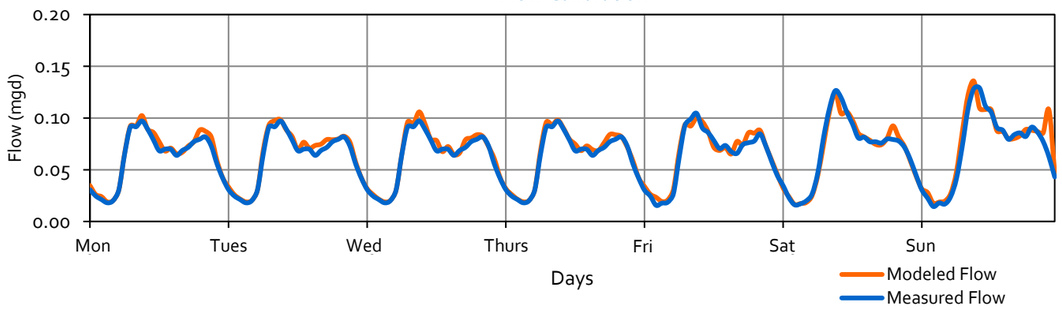
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.06	0.10	1.9	1.41	0.06	0.11	1.9	1.50	3.8%	9.0%	-0.4%	6.1%
Tues.	0.06	0.10	1.9	1.41	0.06	0.11	1.9	1.50	3.8%	9.0%	-0.4%	6.1%
Wed.	0.06	0.10	1.9	1.41	0.06	0.11	1.9	1.50	3.8%	9.0%	-0.4%	6.1%
Thur.	0.06	0.10	1.9	1.41	0.06	0.11	1.9	1.50	3.8%	9.0%	-0.4%	6.1%
Fri.	0.06	0.10	1.9	1.44	0.06	0.10	1.9	1.51	3.0%	-4.5%	-3.0%	4.7%
Sat.	0.07	0.13	2.0	1.47	0.07	0.12	1.9	1.51	-0.7%	-1.5%	-5.8%	2.5%
Sun.	0.07	0.13	2.0	1.48	0.07	0.14	2.0	1.56	5.2%	5.2%	-1.8%	5.3%
Summary												
Weekday	0.06	--	1.9	1.42	0.06	--	1.9	1.50	3.7%	--	-1.0%	5.8%
Weekend	0.07	--	2.0	1.48	0.07	--	2.0	1.53	2.3%	--	-3.8%	3.9%
ADWF[4]	0.06	--	1.9	1.44	0.07	--	1.9	1.51	3.2%	--	-1.8%	5.3%

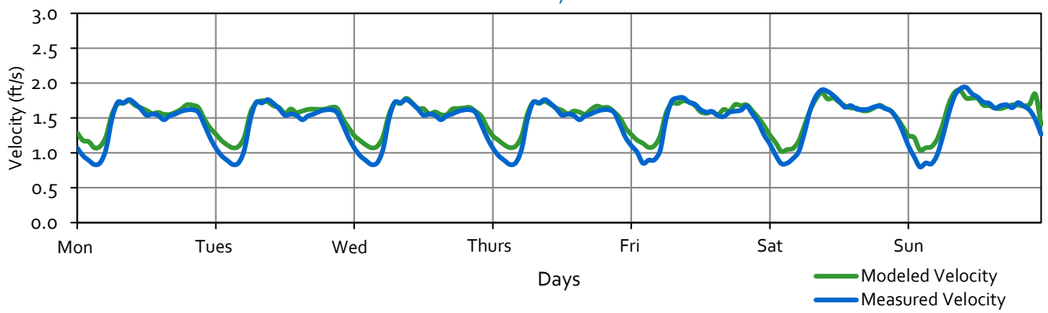
Notes:

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

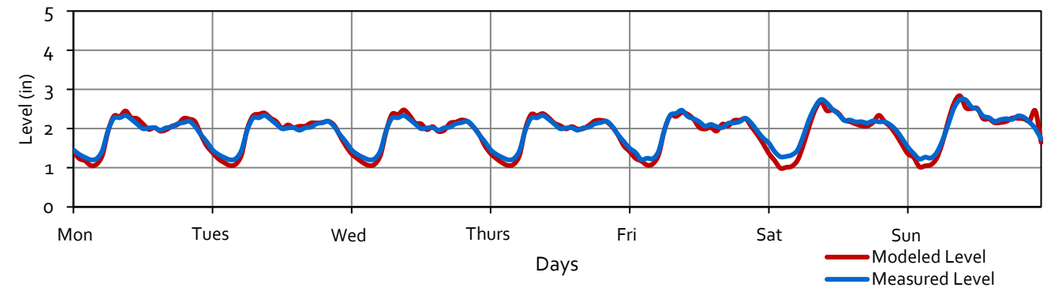
Flow Calibration



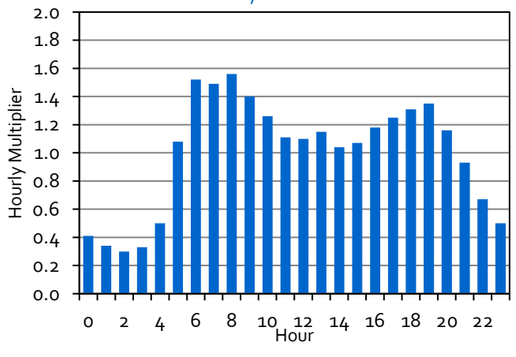
Velocity Calibration



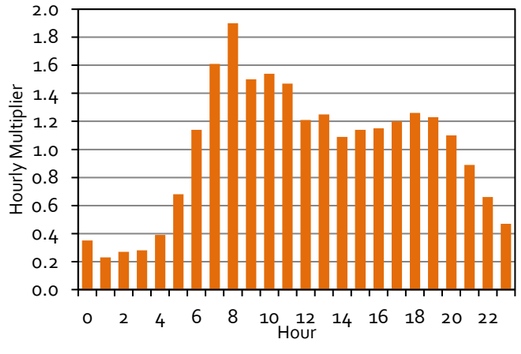
Level Calibration



Weekday Diurnal Pattern



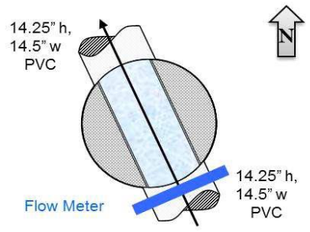
Weekend Diurnal Pattern



Flow Monitoring Site 12, Dry Weather Flow Calibration

Location: Parking lot of 1154 Embarcadero
Pipeline Diameter: 14.375"
City Manhole ID: 12.46
Model Pipe ID: 965

Flow Monitor Location



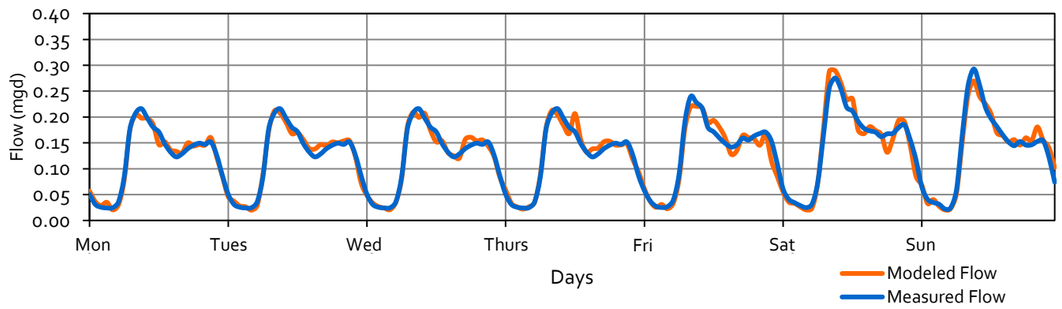
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.12	0.22	4.8	0.81	0.12	0.21	4.8	0.79	0.7%	-1.5%	-1.1%	-2.5%
Tues.	0.12	0.22	4.8	0.81	0.12	0.21	4.8	0.79	0.7%	-1.5%	-1.1%	-2.5%
Wed.	0.12	0.22	4.8	0.81	0.12	0.21	4.8	0.79	0.7%	-1.5%	-1.1%	-2.5%
Thur.	0.12	0.22	4.8	0.81	0.12	0.21	4.8	0.79	0.7%	-1.5%	-1.1%	-2.5%
Fri.	0.13	0.24	5.0	0.82	0.13	0.22	4.9	0.81	-3.1%	-7.9%	-1.6%	-1.3%
Sat.	0.15	0.28	5.1	0.86	0.15	0.29	5.1	0.83	0.0%	5.2%	-1.6%	-3.0%
Sun.	0.14	0.29	5.1	0.87	0.14	0.27	5.0	0.82	0.5%	-7.9%	-3.7%	-5.9%
Summary												
Weekday	0.12	--	4.9	0.81	0.12	--	4.8	0.79	-0.1%	--	-1.2%	-2.2%
Weekend	0.14	--	5.1	0.86	0.14	--	5.0	0.82	0.2%	--	-2.7%	-4.5%
ADWF[4]	0.13	--	4.9	0.83	0.13	--	4.9	0.80	0.0%	--	-1.6%	-2.9%

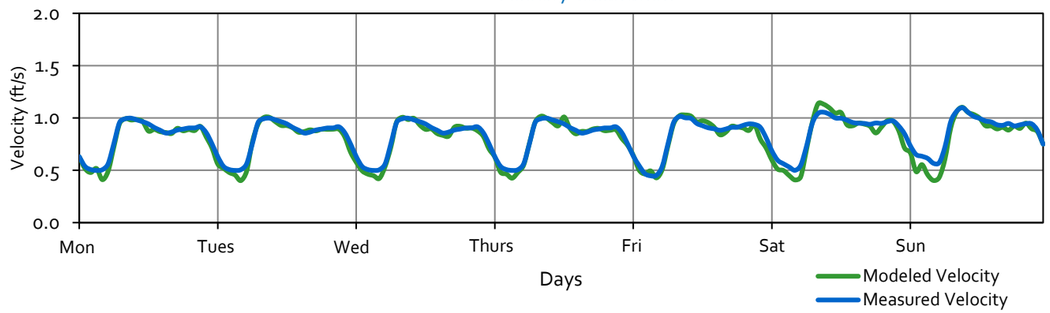
Notes:

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

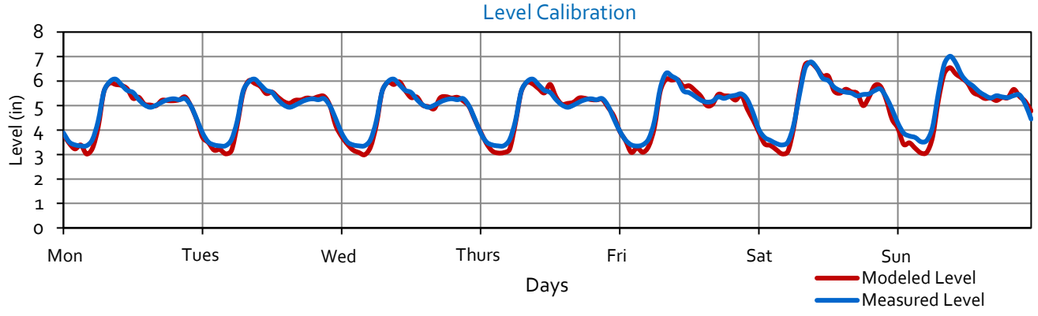
Flow Calibration



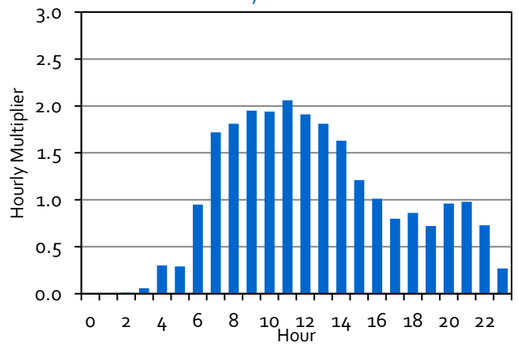
Velocity Calibration



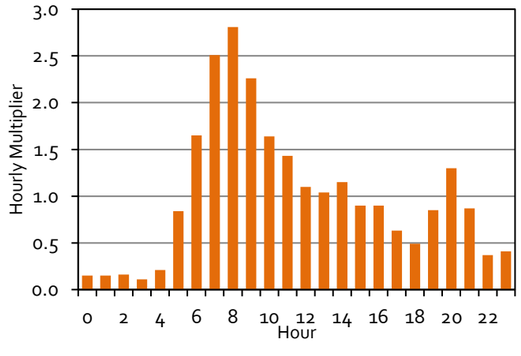
Level Calibration



Weekday Diurnal Pattern



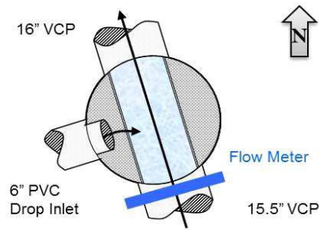
Weekend Diurnal Pattern



Flow Monitoring Site 13, Dry Weather Flow Calibration

Location: Sidewalk of 910 Embarcadero
Pipeline Diameter: 15.5"
City Manhole ID: 12.11
Model Pipe ID: 1125

Flow Monitor Location



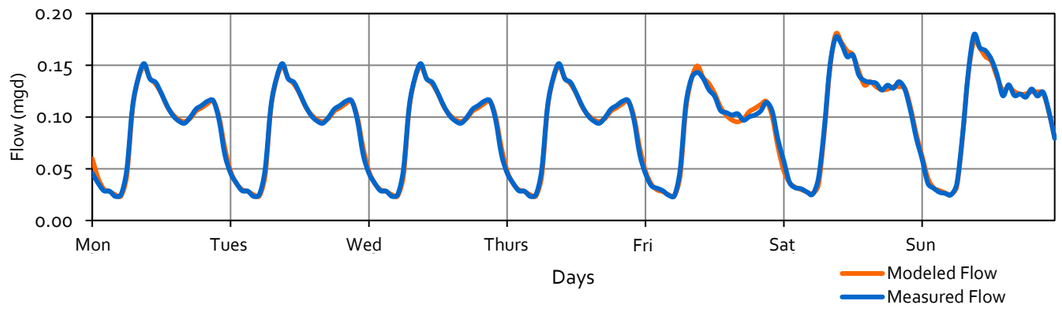
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Tues.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Wed.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Thur.	0.09	0.15	4.6	0.46	0.09	0.15	4.7	0.47	0.0%	-1.7%	1.4%	1.8%
Fri.	0.09	0.14	4.5	0.47	0.09	0.15	4.7	0.47	0.1%	4.3%	3.0%	0.6%
Sat.	0.11	0.18	4.7	0.51	0.10	0.18	4.9	0.50	-0.6%	1.8%	3.8%	-1.9%
Sun.	0.10	0.18	4.7	0.49	0.10	0.17	4.9	0.49	-0.1%	-3.3%	4.6%	0.9%
Summary												
Weekday	0.09	--	4.6	0.47	0.09	--	4.7	0.47	0.0%	--	1.7%	1.6%
Weekend	0.10	--	4.7	0.50	0.10	--	4.9	0.50	-0.3%	--	4.2%	-0.5%
ADWF[4]	0.09	--	4.6	0.47	0.09	--	4.7	0.48	-0.1%	--	2.5%	0.9%

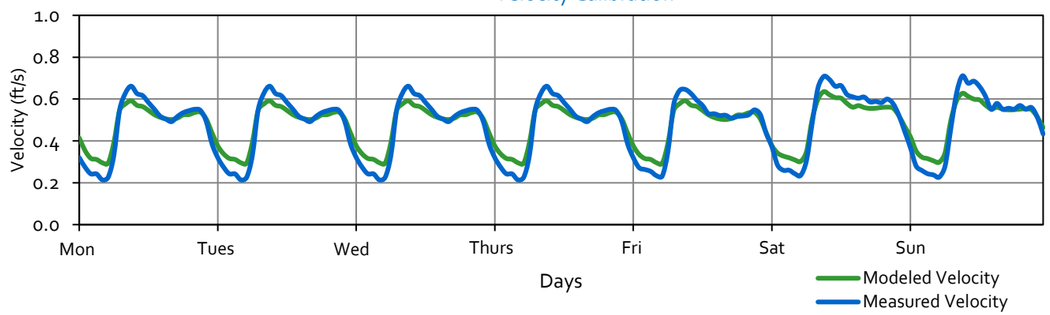
Notes:

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

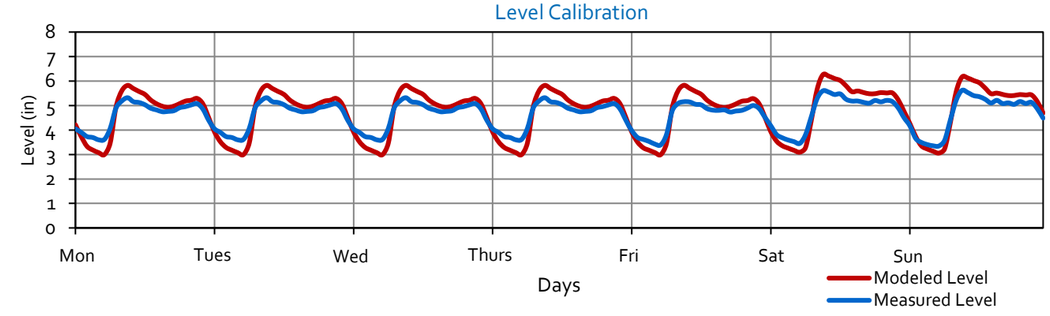
Flow Calibration



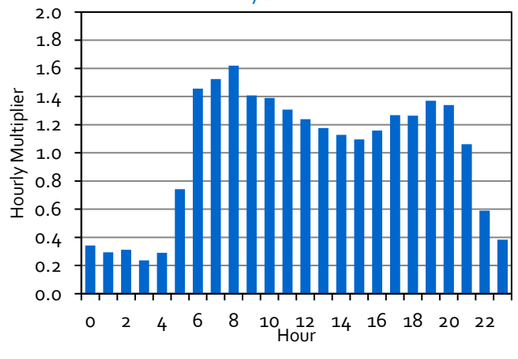
Velocity Calibration



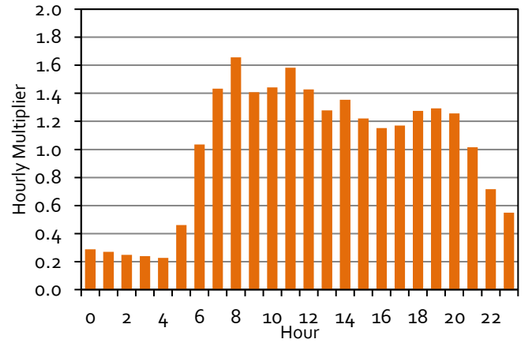
Level Calibration



Weekday Diurnal Pattern



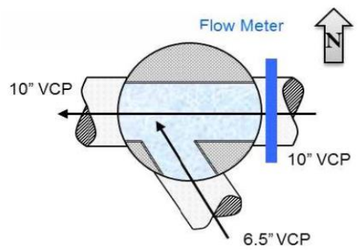
Weekend Diurnal Pattern



Flow Monitoring Site 14, Dry Weather Flow Calibration

Location: Intersection of Main St. and Marina St.
Pipeline Diameter: 10"
City Manhole ID: 11.59
Model Pipe ID: 915

Flow Monitor Location



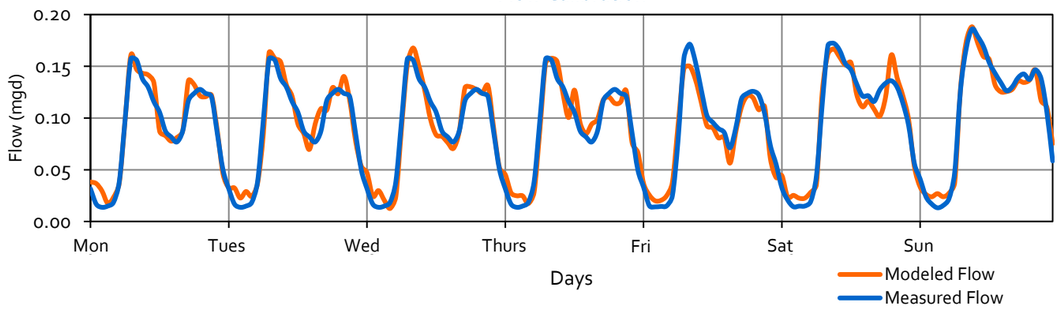
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.09	0.16	1.4	2.79	0.09	0.17	1.4	2.86	2.7%	6.8%	-5.5%	2.4%
Tues.	0.09	0.16	1.4	2.79	0.09	0.17	1.4	2.86	2.7%	6.8%	-5.5%	2.4%
Wed.	0.09	0.16	1.4	2.79	0.09	0.17	1.4	2.86	2.7%	6.8%	-5.5%	2.4%
Thur.	0.09	0.16	1.4	2.79	0.09	0.17	1.4	2.86	2.7%	6.8%	-5.5%	2.4%
Fri.	0.09	0.17	1.4	2.69	0.08	0.15	1.3	2.78	-4.6%	-12.4%	-8.8%	3.4%
Sat.	0.10	0.17	1.5	3.02	0.10	0.17	1.4	2.91	0.4%	-3.1%	-3.5%	-3.4%
Sun.	0.11	0.19	1.5	3.11	0.11	0.19	1.5	2.97	-0.8%	1.3%	-2.5%	-4.5%
Summary												
Weekday	0.09	--	1.4	2.77	0.09	--	1.4	2.84	1.3%	--	-6.1%	2.6%
Weekend	0.10	--	1.5	3.07	0.10	--	1.4	2.94	-0.2%	--	-3.0%	-4.0%
ADWF[4]	0.09	--	1.5	2.86	0.09	--	1.4	2.87	0.8%	--	-5.2%	0.6%

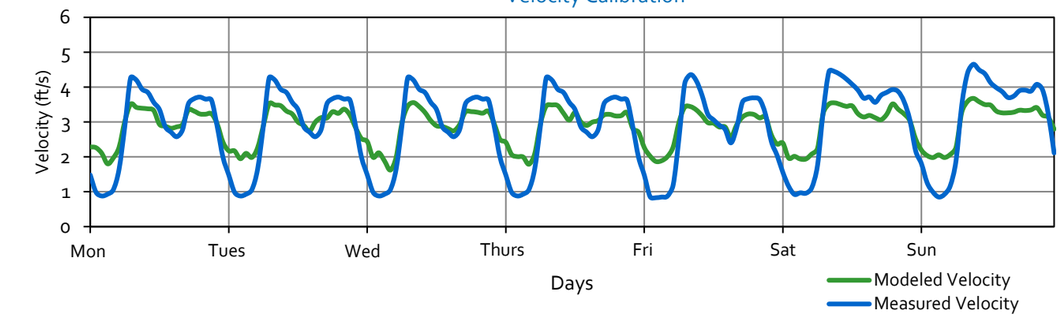
Notes:

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

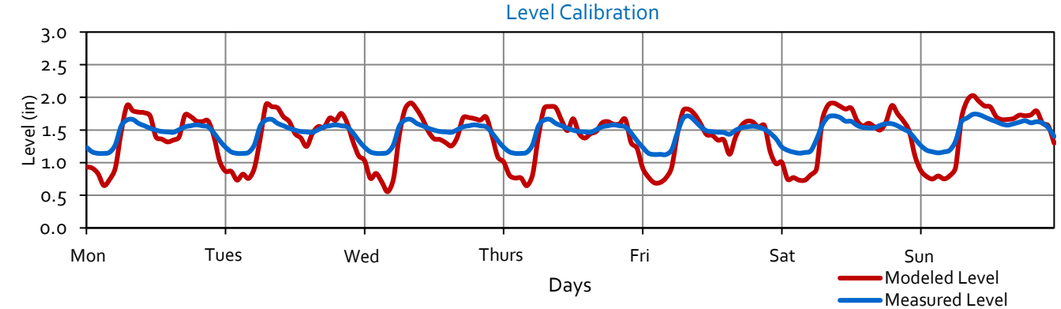
Flow Calibration



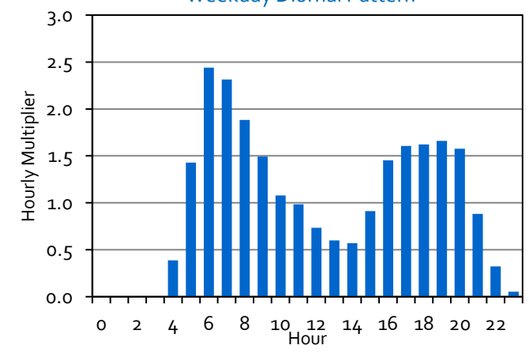
Velocity Calibration



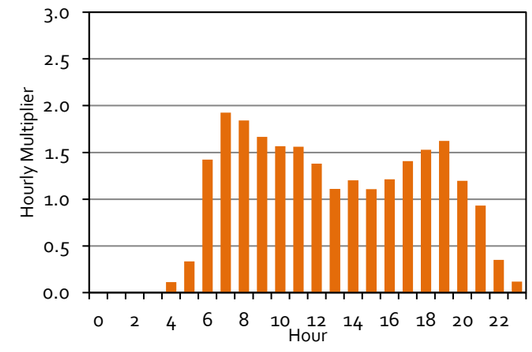
Level Calibration



Weekday Diurnal Pattern



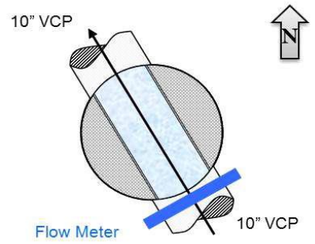
Weekend Diurnal Pattern



Flow Monitoring Site 15, Dry Weather Flow Calibration

Location: 301 Embarcadero ~90-feet north of Morro Bay Launching Facility
Pipeline Diameter: 10"
City Manhole ID: 11.79
Model Pipe ID: 1129

Flow Monitor Location



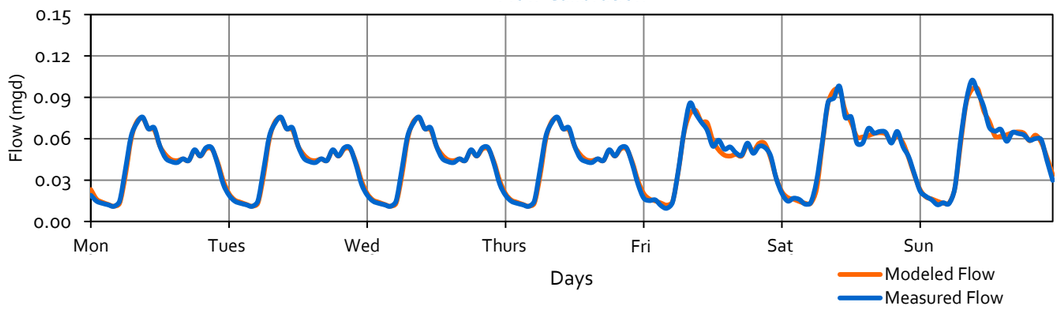
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.04	0.08	1.7	1.08	0.04	0.07	1.7	1.02	0.2%	-1.0%	-3.7%	-5.5%
Tues.	0.04	0.08	1.7	1.08	0.04	0.07	1.7	1.02	0.2%	-1.0%	-3.7%	-5.5%
Wed.	0.04	0.08	1.7	1.08	0.04	0.07	1.7	1.02	0.2%	-1.0%	-3.7%	-5.5%
Thur.	0.04	0.08	1.7	1.08	0.04	0.07	1.7	1.02	0.2%	-1.0%	-3.7%	-5.5%
Fri.	0.05	0.09	1.8	1.09	0.05	0.08	1.7	1.04	-0.5%	-6.0%	-2.3%	-4.7%
Sat.	0.05	0.10	1.8	1.15	0.05	0.10	1.8	1.07	-0.2%	-2.7%	-0.7%	-6.4%
Sun.	0.05	0.10	1.8	1.14	0.05	0.10	1.8	1.08	0.1%	-6.2%	-1.0%	-5.5%
Summary												
Weekday	0.04	--	1.7	1.08	0.04	--	1.7	1.02	0.1%	--	-3.4%	-5.4%
Weekend	0.05	--	1.8	1.14	0.05	--	1.8	1.08	-0.1%	--	-0.8%	-5.9%
ADWF[4]	0.05	--	1.8	1.10	0.05	--	1.7	1.04	0.0%	--	-2.6%	-5.5%

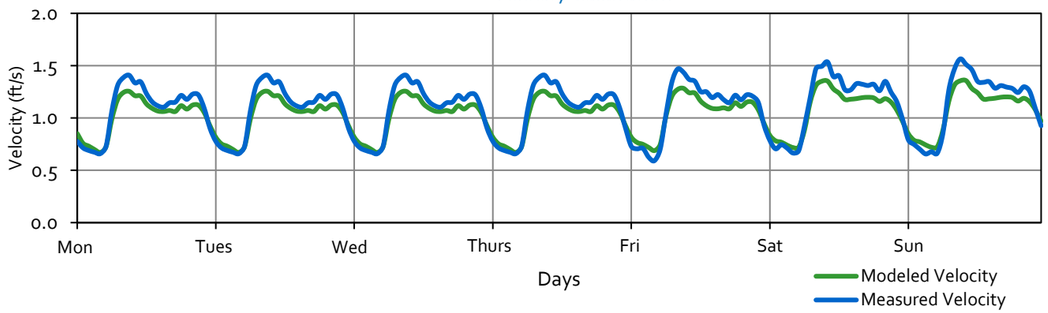
Notes:

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

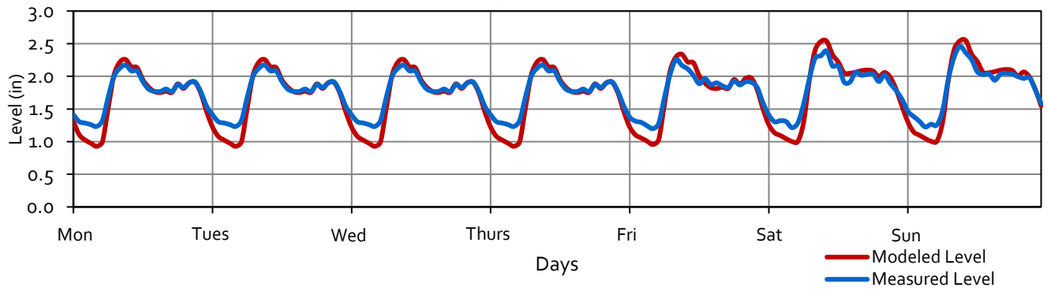
Flow Calibration



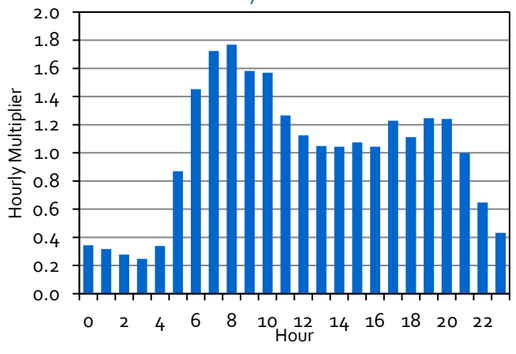
Velocity Calibration



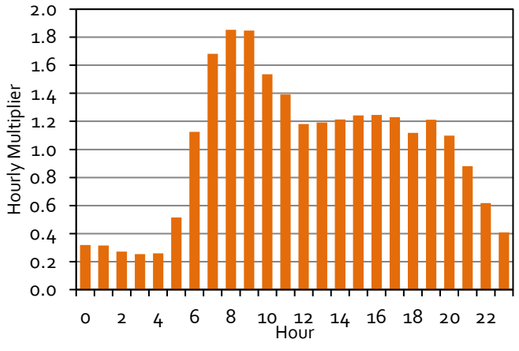
Level Calibration



Weekday Diurnal Pattern



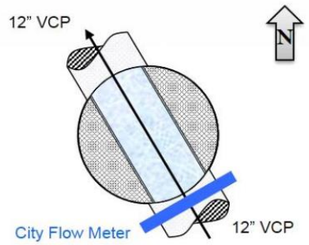
Weekend Diurnal Pattern



Flow Monitoring Site MB-1, Dry Weather Flow Calibration

Location: Main St. ~225-feet south of Errol St.
Pipeline Diameter: 12"
City Manhole ID: 13.36
Model Pipe ID: 1009

Flow Monitor Location



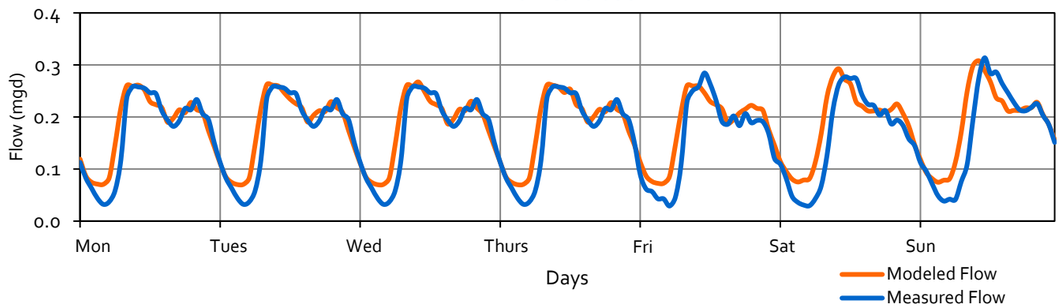
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.17	0.26	3.9	1.64	0.18	0.27	3.6	1.42	7.7%	3.5%	-6.7%	-13.4%
Tues.	0.17	0.26	3.9	1.64	0.18	0.27	3.6	1.42	7.7%	3.5%	-6.7%	-13.4%
Wed.	0.17	0.26	3.9	1.64	0.18	0.27	3.6	1.42	7.7%	3.5%	-6.7%	-13.4%
Thur.	0.17	0.26	3.9	1.64	0.18	0.27	3.6	1.42	7.7%	3.5%	-6.7%	-13.4%
Fri.	0.16	0.28	4.1	1.70	0.18	0.26	3.6	1.42	13.4%	-8.6%	-10.6%	-16.6%
Sat.	0.16	0.28	4.2	1.64	0.18	0.29	3.7	1.43	15.1%	5.7%	-12.6%	-13.1%
Sun.	0.17	0.31	4.2	1.61	0.19	0.31	3.7	1.44	9.7%	-1.8%	-12.8%	-10.8%
Summary												
Weekday	0.17	--	3.9	1.65	0.18	--	3.6	1.42	8.8%	--	-7.5%	-14.1%
Weekend	0.17	--	4.2	1.63	0.19	--	3.7	1.43	12.3%	--	-12.7%	-12.0%
ADWF[4]	0.17	--	4.0	1.64	0.18	--	3.6	1.42	9.8%	--	-9.1%	-13.5%

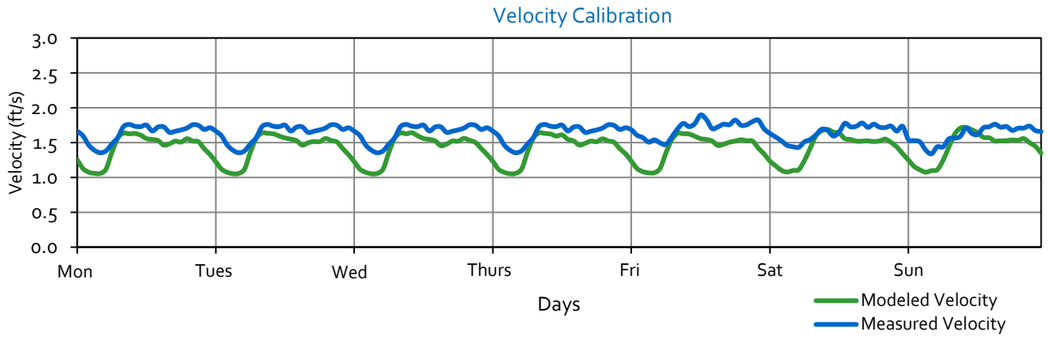
Notes:

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

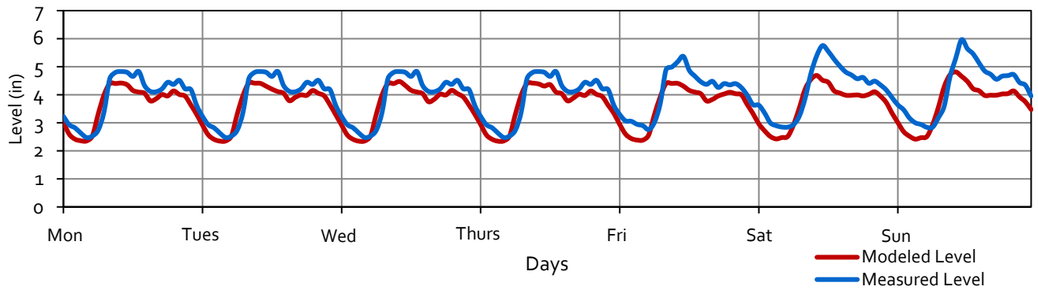
Flow Calibration



Velocity Calibration



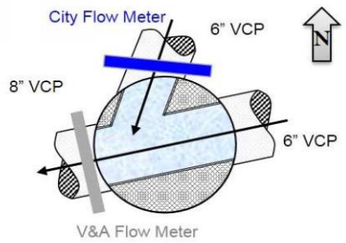
Level Calibration



Flow Monitoring Site MB-2, Dry Weather Flow Calibration

Location: Atascadero Rd. ~85-feet east of Sunset Ave.
Pipeline Diameter: 6"
City Manhole ID: 5.19
Model Pipe ID: 660

Flow Monitor Location



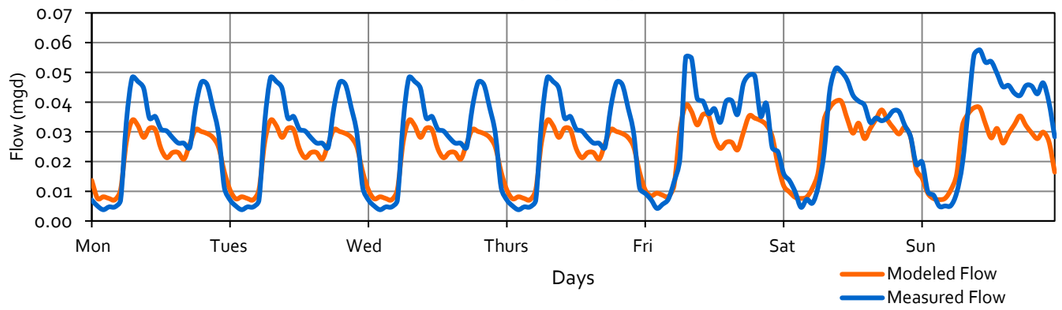
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.03	0.05	1.5	1.26	0.02	0.03	1.1	1.29	-18.2%	-29.8%	-27.4%	2.6%
Tues.	0.03	0.05	1.5	1.26	0.02	0.03	1.1	1.29	-18.2%	-29.8%	-27.4%	2.6%
Wed.	0.03	0.05	1.5	1.26	0.02	0.03	1.1	1.29	-18.2%	-29.8%	-27.4%	2.6%
Thur.	0.03	0.05	1.5	1.26	0.02	0.03	1.1	1.29	-18.2%	-29.8%	-27.4%	2.6%
Fri.	0.03	0.06	1.6	1.36	0.03	0.04	1.2	1.34	-18.8%	-29.4%	-27.4%	-1.3%
Sat.	0.03	0.05	1.6	1.48	0.03	0.04	1.2	1.34	-10.2%	-21.5%	-24.7%	-9.1%
Sun.	0.03	0.06	1.6	1.52	0.02	0.04	1.2	1.33	-27.0%	-33.9%	-25.3%	-12.9%
Summary												
Weekday	0.03	--	1.6	1.28	0.02	--	1.1	1.30	-18.3%	--	-27.4%	1.8%
Weekend	0.03	--	1.6	1.50	0.03	--	1.2	1.33	-19.3%	--	-25.0%	-11.1%
ADWF[4]	0.03	--	1.6	1.34	0.02	--	1.1	1.31	-18.6%	--	-26.7%	-2.3%

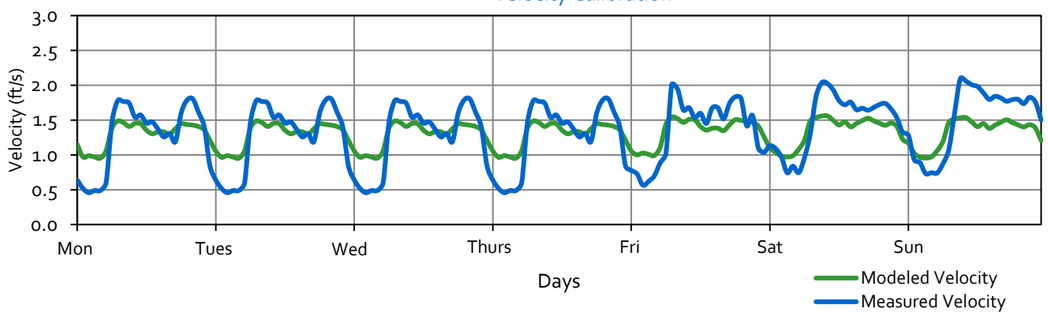
Notes:

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

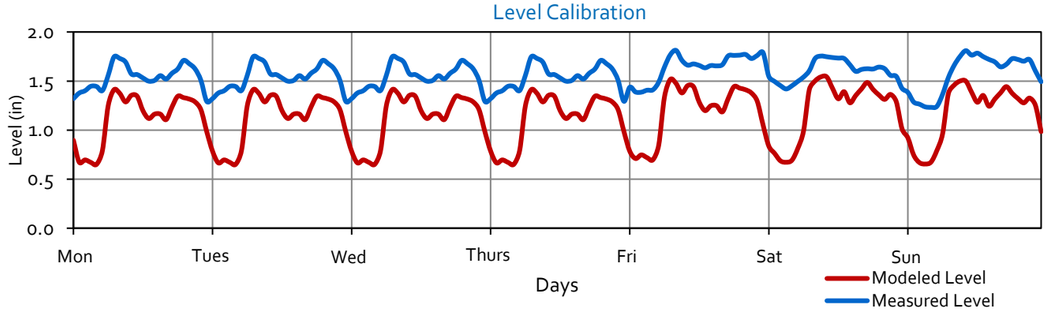
Flow Calibration



Velocity Calibration



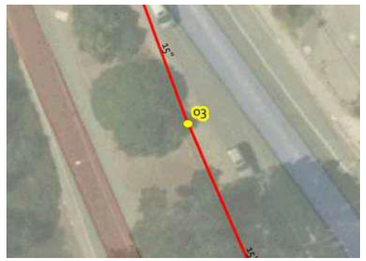
Level Calibration

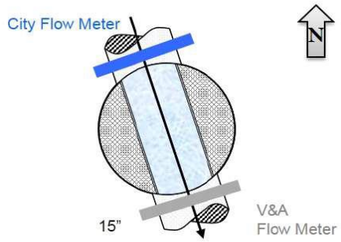


Flow Monitoring Site MB-3, Dry Weather Flow Calibration

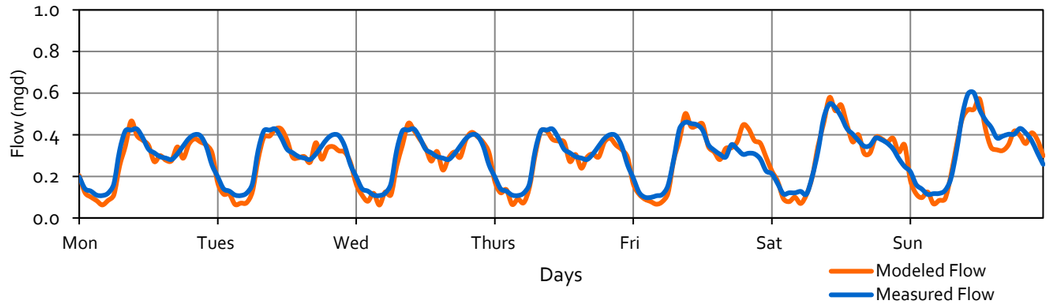
Location: Island between Main St. and Hwy 1 on-ramp, ~135-feet north of Atascadero Rd.
Pipeline Diameter: 15"
City Manhole ID: 5.25A
Model Pipe ID: 675

Flow Monitor Location

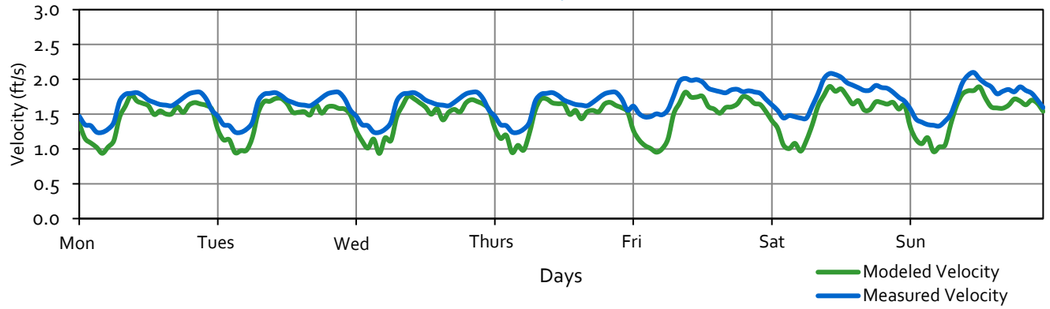




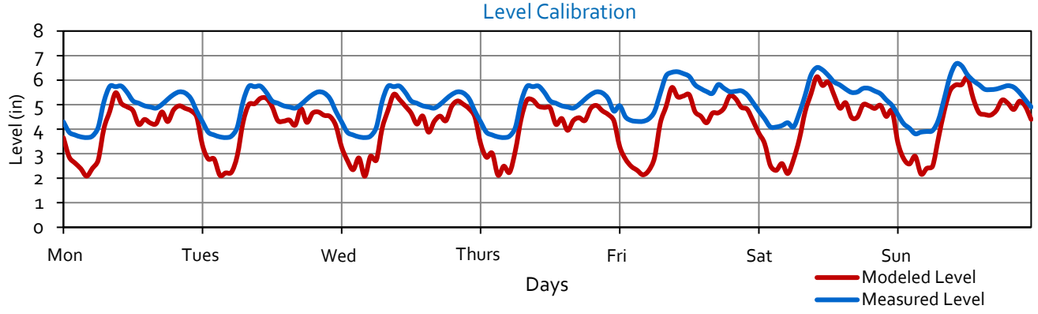
Flow Calibration



Velocity Calibration



Level Calibration



Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.29	0.43	4.9	1.60	0.27	0.47	4.1	1.45	-6.0%	9.0%	-16.1%	-9.5%
Tues.	0.29	0.43	4.9	1.60	0.27	0.47	4.1	1.45	-6.0%	9.0%	-16.1%	-9.5%
Wed.	0.29	0.43	4.9	1.60	0.27	0.47	4.1	1.45	-6.0%	9.0%	-16.1%	-9.5%
Thur.	0.29	0.43	4.9	1.60	0.27	0.47	4.1	1.45	-6.0%	9.0%	-16.1%	-9.5%
Fri.	0.28	0.46	5.4	1.76	0.29	0.50	4.2	1.48	4.4%	9.0%	-21.6%	-15.8%
Sat.	0.31	0.55	5.3	1.77	0.31	0.58	4.4	1.51	1.2%	5.7%	-17.1%	-14.3%
Sun.	0.33	0.60	5.2	1.71	0.31	0.57	4.3	1.51	-7.1%	-4.9%	-17.0%	-12.1%
Summary												
Weekday	0.29	--	5.0	1.64	0.27	--	4.1	1.46	-3.9%	--	-17.3%	-10.8%
Weekend	0.32	--	5.2	1.74	0.31	--	4.3	1.51	-3.1%	--	-17.0%	-13.2%
ADWF[4]	0.30	--	5.0	1.67	0.29	--	4.2	1.47	-3.7%	--	-17.2%	-11.5%

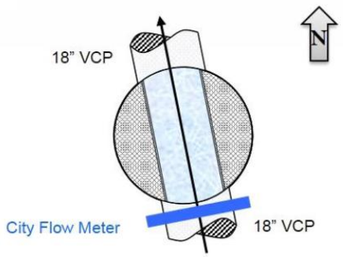
Notes:

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

Flow Monitoring Site MB-4, Dry Weather Flow Calibration

Location: Embarcadero ~300-feet south of entrance to Morro Dunes mobile home park
Pipeline Diameter: 18"
City Manhole ID: 14.5
Model Pipe ID: 61

Flow Monitor Location



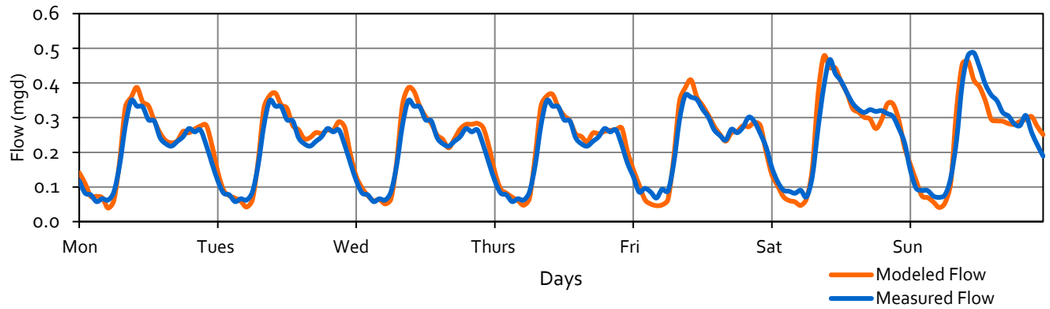
Model Calibration Summary

Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.21	0.35	2.8	1.70	0.22	0.39	3.1	1.57	6.8%	11.0%	8.5%	-8.1%
Tues.	0.21	0.35	2.8	1.70	0.22	0.39	3.1	1.57	6.8%	11.0%	8.5%	-8.1%
Wed.	0.21	0.35	2.8	1.70	0.22	0.39	3.1	1.57	6.8%	11.0%	8.5%	-8.1%
Thur.	0.21	0.35	2.8	1.70	0.22	0.39	3.1	1.57	6.8%	11.0%	8.5%	-8.1%
Fri.	0.23	0.36	2.9	1.77	0.23	0.41	3.1	1.57	0.7%	12.0%	7.0%	-11.6%
Sat.	0.26	0.47	3.1	1.83	0.26	0.48	3.3	1.62	-0.3%	2.1%	7.1%	-11.5%
Sun.	0.25	0.49	3.1	1.81	0.25	0.46	3.2	1.60	-2.5%	-5.1%	5.4%	-11.7%
Summary												
Weekday	0.21	--	2.8	1.72	0.22	--	3.1	1.57	5.5%	--	8.2%	-8.8%
Weekend	0.26	--	3.1	1.82	0.25	--	3.3	1.61	-1.4%	--	6.3%	-11.6%
ADWF[4]	0.22	--	2.9	1.75	0.23	--	3.1	1.58	3.2%	--	7.6%	-9.6%

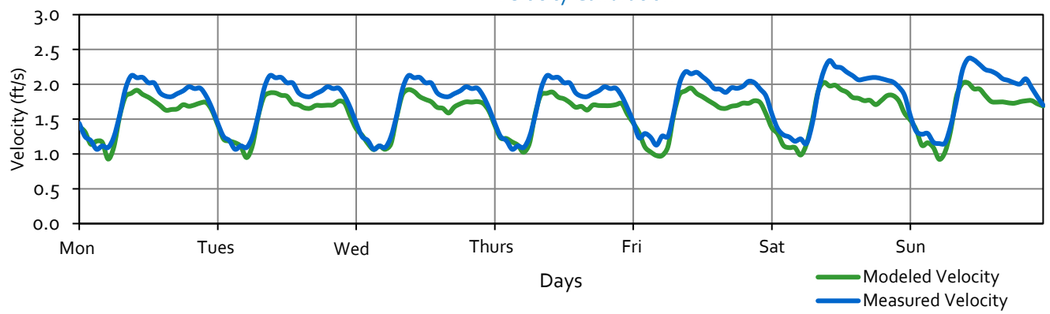
Notes:

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

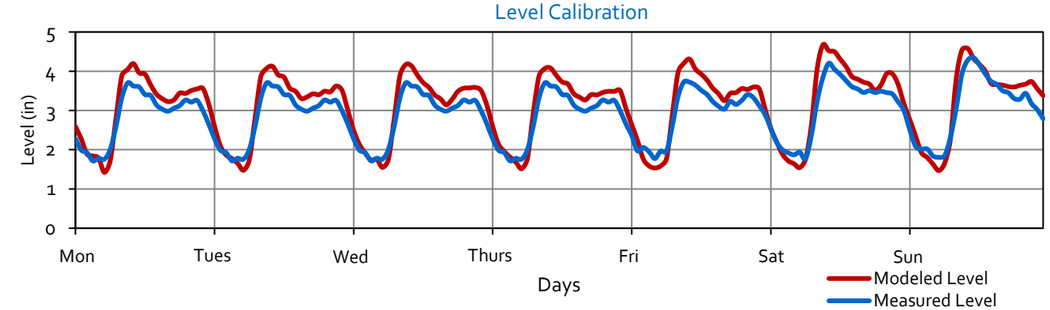
Flow Calibration



Velocity Calibration



Level Calibration



Flow Monitoring Site Lift Station 1, Dry Weather Flow Calibration

Location:
Pipeline Diameter: 6"
Model Wet Well ID: LS.1
Model Pipe ID: FM-9

Flow Monitor Location

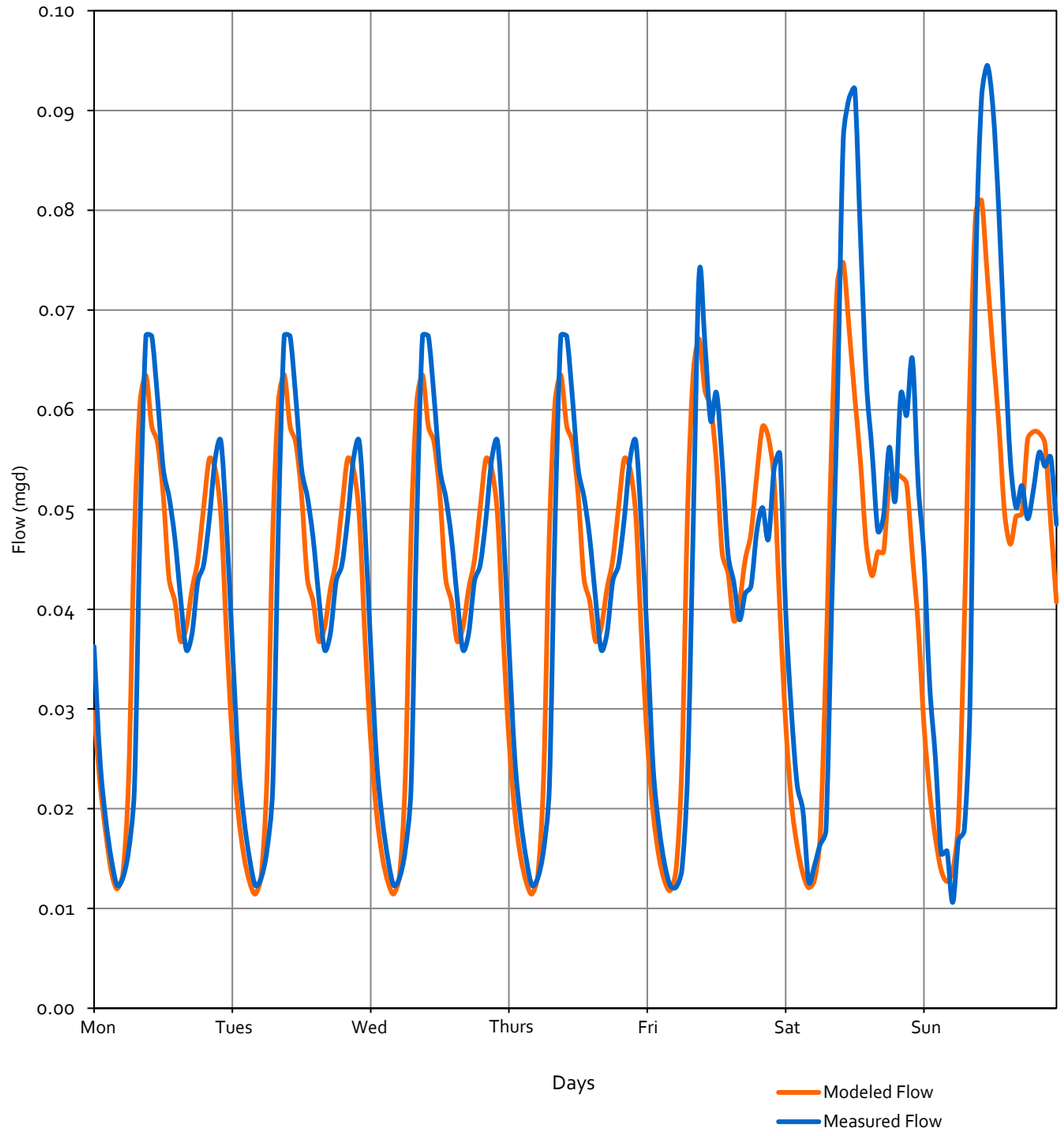
Model Calibration Summary

Day	Measured Data[1] Avgerage Influent Flow (mgd)	Measured Data[1] Peak Influent Flow[2] (mgd)	Modeled Data Avgerage Influent Flow (mgd)	Modeled Data Peak Influent Flow[2] (mgd)	Percent Error[3] Avgerage Influent Flow (%)	Percent Error[3] Peak Influent Flow (%)
Mon.	0.04	0.07	0.04	0.06	-1.3%	-5.8%
Tues.	0.04	0.07	0.04	0.06	-1.3%	-5.8%
Wed.	0.04	0.07	0.04	0.06	-1.3%	-5.8%
Thur.	0.04	0.07	0.04	0.06	-1.3%	-5.8%
Fri.	0.04	0.07	0.04	0.07	2.0%	-8.9%
Sat.	0.05	0.09	0.04	0.07	-13.7%	-18.9%
Sun.	0.05	0.09	0.05	0.08	-6.7%	-14.3%
Summary						
Weekday	0.04	--	0.04	--	-0.6%	--
Weekend	0.05	--	0.04	--	-10.2%	--
ADWF[4]	0.04	--	0.04	--	-3.7%	--

Notes:

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

Influent Flow Calibration

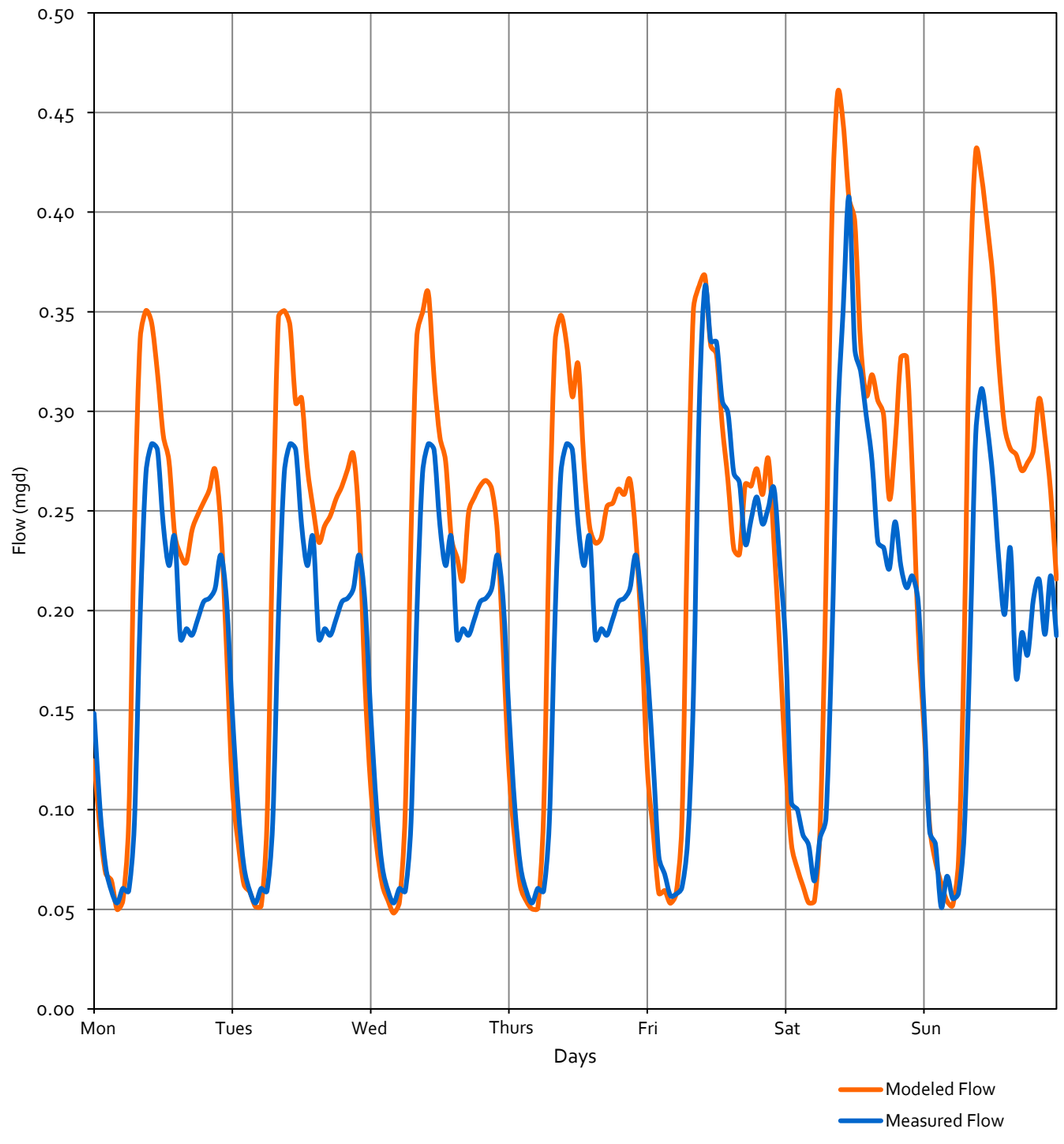


Flow Monitoring Site Lift Station 2, Dry Weather Flow Calibration

Location:
Pipeline Diameter: 12"
Model Wet Well ID: LS_2_WW
Model Pipe ID: FM-15

Flow Monitor Location

Influent Flow Calibration



Model Calibration Summary

Day	Measured Data[1] Avgerage Influent Flow (mgd)	Measured Data[1] Peak Influent Flow[2] (mgd)	Modeled Data Avgerage Influent Flow (mgd)	Modeled Data Peak Influent Flow[2] (mgd)	Percent Error[3] Avgerage Influent Flow (%)	Percent Error[3] Peak Influent Flow (%)
Mon.	0.17	0.28	0.21	0.36	21.5%	26.7%
Tues.	0.17	0.28	0.21	0.36	21.5%	26.7%
Wed.	0.17	0.28	0.21	0.36	21.5%	26.7%
Thur.	0.17	0.28	0.21	0.36	21.5%	26.7%
Fri.	0.21	0.36	0.22	0.37	4.7%	1.5%
Sat.	0.21	0.41	0.25	0.46	20.2%	12.7%
Sun.	0.17	0.31	0.24	0.43	38.0%	38.3%
Summary						
Weekday	0.18	--	0.21	--	17.6%	--
Weekend	0.19	--	0.25	--	28.3%	--
ADWF[4]	0.18	--	0.22	--	20.8%	--

Notes:

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

Flow Monitoring Site Lift Station 3, Dry Weather Flow Calibration

Location:
Pipeline Diameter: 8"
Model Wet Well ID: LS.3
Model Pipe ID: FM-6

Flow Monitor Location

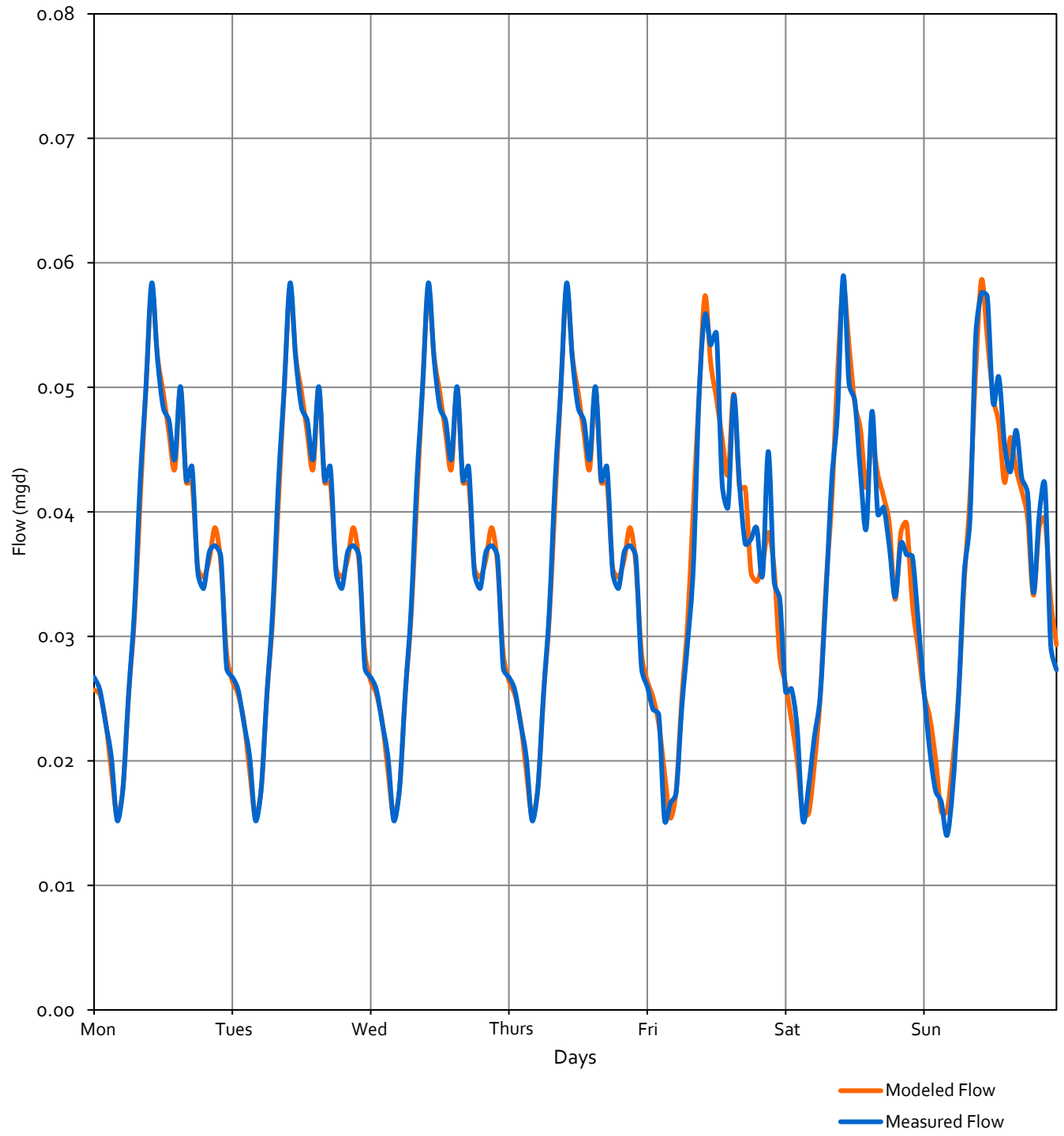
Model Calibration Summary

Day	Measured Data[1] Avgerage Influent Flow (mgd)	Measured Data[1] Peak Influent Flow[2] (mgd)	Modeled Data Avgerage Influent Flow (mgd)	Modeled Data Peak Influent Flow[2] (mgd)	Percent Error[3] Avgerage Influent Flow (%)	Percent Error[3] Peak Influent Flow (%)
Mon.	0.04	0.06	0.04	0.06	-0.4%	-0.8%
Tues.	0.04	0.06	0.04	0.06	-0.4%	-0.8%
Wed.	0.04	0.06	0.04	0.06	-0.4%	-0.8%
Thur.	0.04	0.06	0.04	0.06	-0.4%	-0.8%
Fri.	0.04	0.06	0.04	0.06	0.0%	2.7%
Sat.	0.04	0.06	0.04	0.06	0.3%	-1.5%
Sun.	0.04	0.06	0.04	0.06	-0.4%	1.8%
Summary						
Weekday	0.04	--	0.04	--	-0.3%	--
Weekend	0.04	--	0.04	--	0.0%	--
ADWF[4]	0.04	--	0.04	--	-0.2%	--

Notes:

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

Influent Flow Calibration



Appendix J

WWF CALIBRATION SHEETS

Table 1 - Wet Weather Calibration Event 1 Summary

Meter Number	Pipe Diameter (in)	Storm 1 (1/3/2017-1/6/2017)											
		Measured Data[1]				Modeled Data[2]				Percent Error[3]			
		Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (%)	Peak Flow (%)	Avg. Level (%)	Avg. Velocity (%)
WWTP Flume	27	1.73	10.08	--	--	1.68	5.45	--	--	-2.8%	-45.9%	--	--
Greyline Site 1	12	0.27	0.91	4.00	1.7	0.31	0.98	4.76	1.7	15.1%	7.3%	19.2%	-5.1%
Greyline Site 2	6	0.05	0.17	1.71	1.5	0.05	0.18	1.65	1.6	10.8%	3.9%	-3.3%	6.4%
Greyline Site 3	15	0.71	2.54	6.08	2.1	0.66	2.54	6.18	1.9	-8.0%	-0.1%	1.7%	-9.9%
Greyline Site 4	18	0.27	1.02	3.12	1.8	0.29	1.01	3.49	1.7	6.6%	-1.6%	11.9%	-9.4%
Lift Station 1 Influent	--	0.13	0.68	--	--	0.13	0.61	--	--	-4.8%	-11.3%	--	--
Lift Station 2 Influent	--	0.21	1.18	--	--	0.28	0.91	--	--	31.0%	-23.4%	--	--
Lift Station 3 Influent	--	0.04	0.08	--	--	0.05	0.10	--	--	13.1%	21.6%	--	--

Notes:

1. Source: City of Morro Bay 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
3. Percent Difference = (Modeled - Measured)/Measured*100.

Table 2 - Wet Weather Calibration Event 2 Summary

Meter Number	Pipe Diameter (in)	Storm 2 (1/7/2017-1/17/2017)											
		Measured Data[1]				Modeled Data[2]				Percent Error[3]			
		Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (%)	Peak Flow (%)	Avg. Level (%)	Avg. Velocity (%)
WWTP Flume	27	1.57	4.31	--	--	1.56	5.00	--	--	-0.5%	16.1%	--	--
Greyline Site 1	12	0.29	0.73	4.37	1.7	0.32	0.84	4.88	1.7	10.8%	15.1%	11.7%	1.4%
Greyline Site 2	6	0.05	0.15	1.69	1.7	0.05	0.16	1.65	1.6	-6.0%	7.2%	-2.6%	-3.6%
Greyline Site 3	15	0.63	2.26	5.94	2.1	0.59	2.42	6.04	1.9	-6.8%	6.9%	1.7%	-9.4%
Greyline Site 4	18	0.25	0.69	3.01	1.8	0.27	0.77	3.44	1.7	11.6%	12.0%	14.0%	-6.6%
Lift Station 1 Influent	--	0.11	0.49	--	--	0.11	0.57	--	--	-2.5%	15.4%	--	--
Lift Station 2 Influent	--	0.21	0.54	--	--	0.26	0.71	--	--	27.6%	31.9%	--	--
Lift Station 3 Influent	--	0.04	0.08	--	--	0.05	0.08	--	--	6.6%	2.8%	--	--

Notes:

1. Source: City of Morro Bay 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
3. Percent Difference = (Modeled - Measured)/Measured*100.

Table 3 - Wet Weather Calibration Event 3 Summary

Meter Number	Pipe Diameter (in)	Storm 3 (1/18/2017-1/25/2017)											
		Measured Data[1]				Modeled Data[2]				Percent Error[3]			
		Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (%)	Peak Flow (%)	Avg. Level (%)	Avg. Velocity (%)
WWTP Flume	27	1.75	4.89	--	--	1.73	4.21	--	--	-1.3%	-13.8%	--	--
Greyline Site 1	12	0.32	0.93	4.79	1.7	0.35	0.70	5.14	1.8	9.0%	-24.3%	7.4%	4.6%
Greyline Site 2	6	0.06	0.14	1.76	1.8	0.06	0.14	1.82	1.7	1.3%	3.4%	3.1%	-4.3%
Greyline Site 3	15	0.76	2.37	6.49	2.2	0.70	2.03	6.66	2.0	-8.4%	-14.1%	2.7%	-10.7%
Greyline Site 4	18	0.23	0.88	2.98	1.8	0.28	0.62	3.51	1.7	20.3%	-28.8%	17.9%	-4.7%
Lift Station 1 Influent	--	0.14	0.62	--	--	0.13	0.42	--	--	-5.7%	-32.3%	--	--
Lift Station 2 Influent	--	0.20	0.63	--	--	0.27	0.60	--	--	38.2%	-4.4%	--	--
Lift Station 3 Influent	--	0.05	0.09	--	--	0.05	0.08	--	--	7.1%	-1.4%	--	--

Notes:

1. Source: City of Morro Bay 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
3. Percent Difference = (Modeled - Measured)/Measured*100.

Table 4 - Wet Weather Calibration Event 4 Summary

Meter Number	Pipe Diameter (in)	Storm 4 (2/5/2017-2/12/2017)											
		Measured Data[1]				Modeled Data[2]				Percent Error[3]			
		Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (%)	Peak Flow (%)	Avg. Level (%)	Avg. Velocity (%)
WWTP Flume	27	1.73	3.07	--	--	1.54	3.41	--	--	-11.0%	11.1%	--	--
Greyline Site 1	12	0.32	0.71	4.81	1.7	0.31	0.54	4.82	1.7	-3.3%	-24.5%	0.1%	0.3%
Greyline Site 2	6	0.06	0.12	1.77	1.9	0.05	0.10	1.69	1.6	-18.3%	-10.9%	-4.6%	-14.3%
Greyline Site 3	15	0.79	1.71	7.38	2.1	0.60	1.82	6.19	1.9	-23.2%	6.7%	-16.2%	-11.1%
Greyline Site 4	18	0.25	0.60	3.07	1.8	0.27	0.54	3.43	1.7	7.6%	-11.0%	11.7%	-7.8%
Lift Station 1 Influent	--	0.13	0.37	--	--	0.11	0.37	--	--	-15.7%	-0.2%	--	--
Lift Station 2 Influent	--	0.19	0.43	--	--	0.26	0.51	--	--	34.8%	17.0%	--	--
Lift Station 3 Influent	--	0.05	0.08	--	--	0.05	0.08	--	--	-0.5%	-9.1%	--	--

Notes:

1. Source: City of Morro Bay 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
3. Percent Difference = (Modeled - Measured)/Measured*100.

Table 5 - Wet Weather Calibration Event 5 Summary

Meter Number	Pipe Diameter (in)	Storm 5 (2/16/2017-2/22/2017)											
		Measured Data[1]				Modeled Data[2]				Percent Error[3]			
		Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Level (in)	Avg. Velocity (ft/s)	Avg. Flow (%)	Peak Flow (%)	Avg. Level (%)	Avg. Velocity (%)
WWTP Flume	27	1.82	3.87	--	--	1.85	4.44	--	--	1.3%	14.6%	--	--
Greyline Site 1	12	0.36	0.87	5.31	1.7	0.38	0.75	5.32	1.8	3.3%	-13.5%	0.3%	5.5%
Greyline Site 2	6	0.07	0.16	1.81	2.1	0.06	0.18	1.88	1.7	-8.8%	15.8%	3.5%	-17.4%
Greyline Site 3	15	0.76	1.97	6.60	2.2	0.76	2.16	6.90	2.0	-0.3%	9.7%	4.6%	-6.7%
Greyline Site 4	18	0.26	0.63	3.12	1.9	0.29	0.71	3.54	1.7	10.2%	12.0%	13.6%	-8.7%
Lift Station 1 Influent	--	0.13	0.48	--	--	0.15	0.47	--	--	9.3%	-0.7%	--	--
Lift Station 2 Influent	--	0.20	0.45	--	--	0.28	0.67	--	--	38.6%	48.4%	--	--
Lift Station 3 Influent	--	0.05	0.12	--	--	0.05	0.09	--	--	-4.2%	-21.8%	--	--

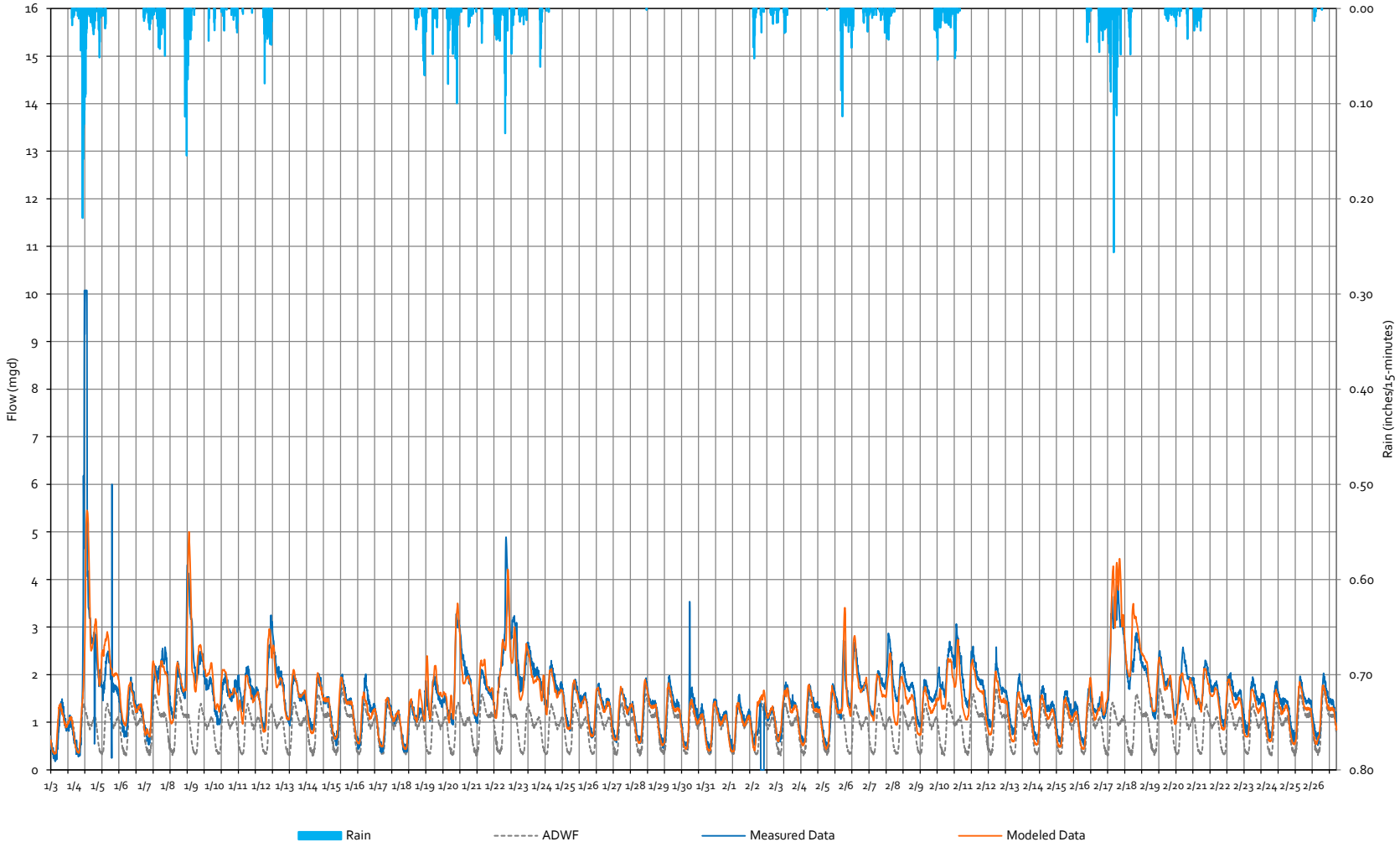
Notes:

1. Source: City of Morro Bay 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
3. Percent Difference = (Modeled - Measured)/Measured*100.

Flume Wet Weather Calibration

Location: WWTP 27-inch Pakmer-Bowlus Flume
Pipeline diameter: 27"
City Manhole ID:
Model Pipe ID: CDT-43
Silt Level at Site: "

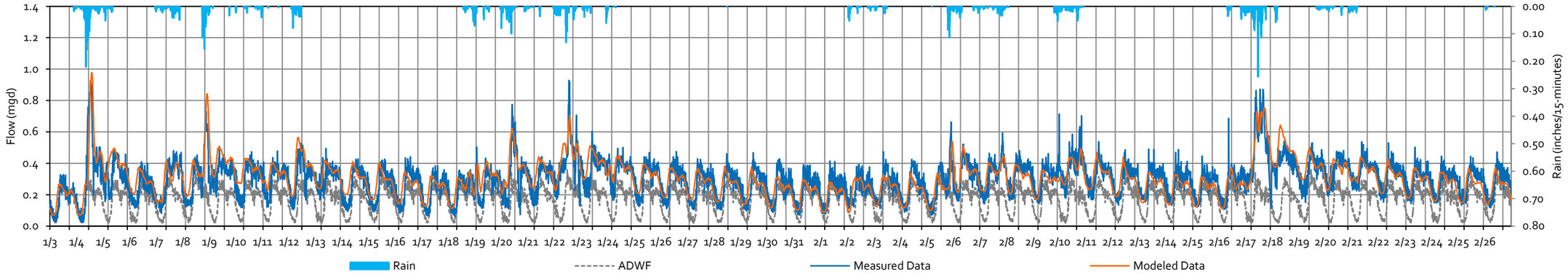
Flow Calibration



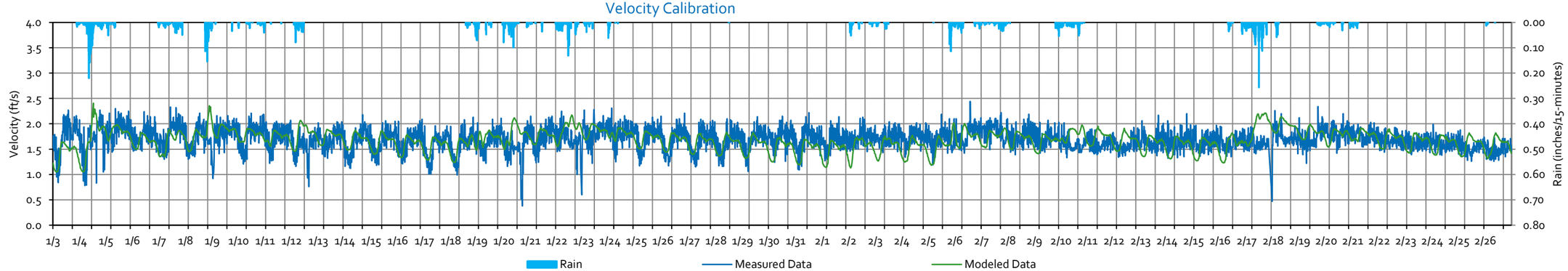
MB-1 Wet Weather Calibration

Location: Main St. ~225-feet south of Errol St.
Pipeline diameter: 12"
City Manhole ID: 13.36
Model Pipe ID: 1009
Silt Level at Site: 0"

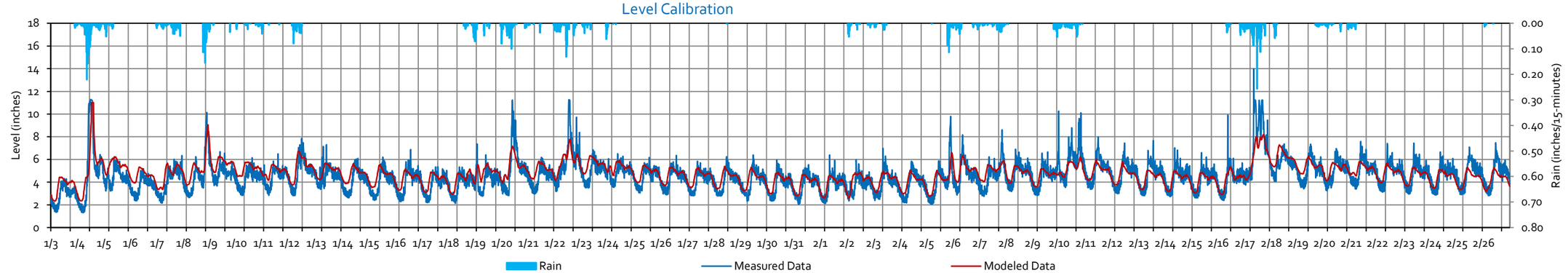
Flow Calibration



Velocity Calibration



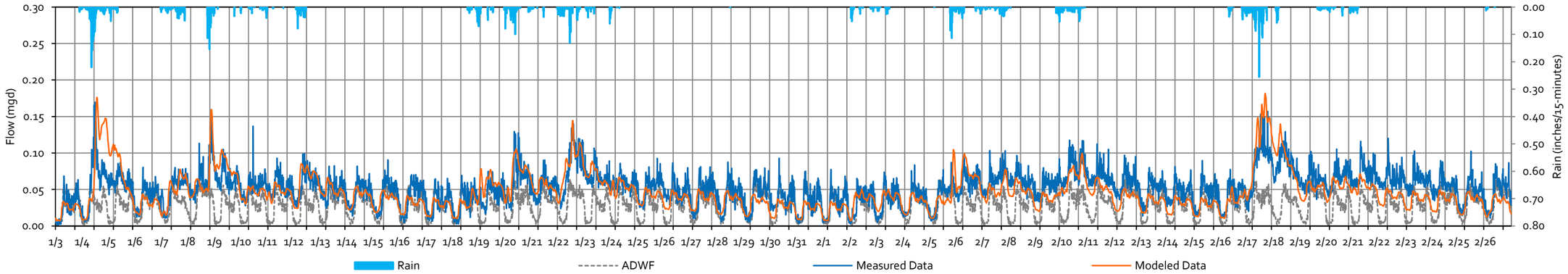
Level Calibration



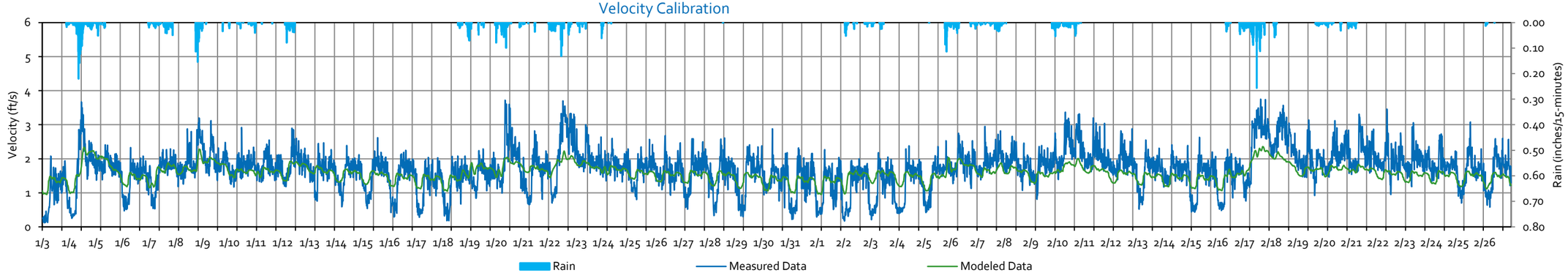
MB-2 Wet Weather Calibration

Location: Atascadero Rd. ~85-feet east of Sunset Ave.
Pipeline diameter: 6"
City Manhole ID: 5.19
Model Pipe ID: 660
Silt Level at Site: 0"

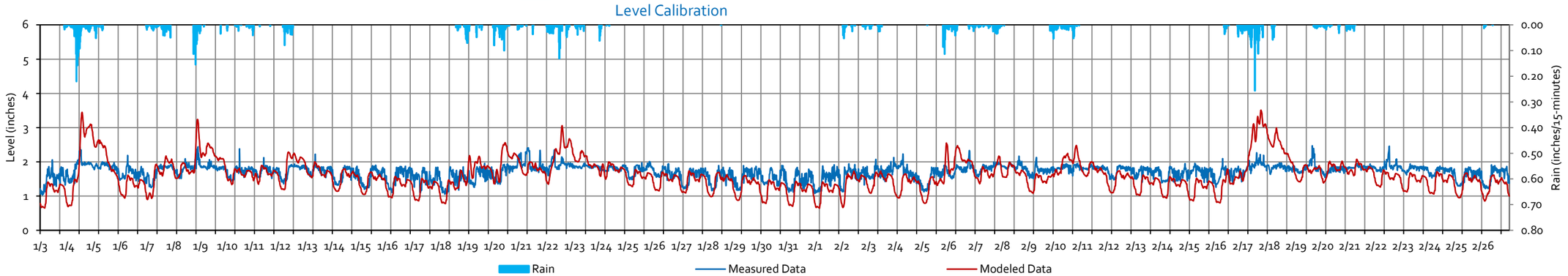
Flow Calibration



Velocity Calibration



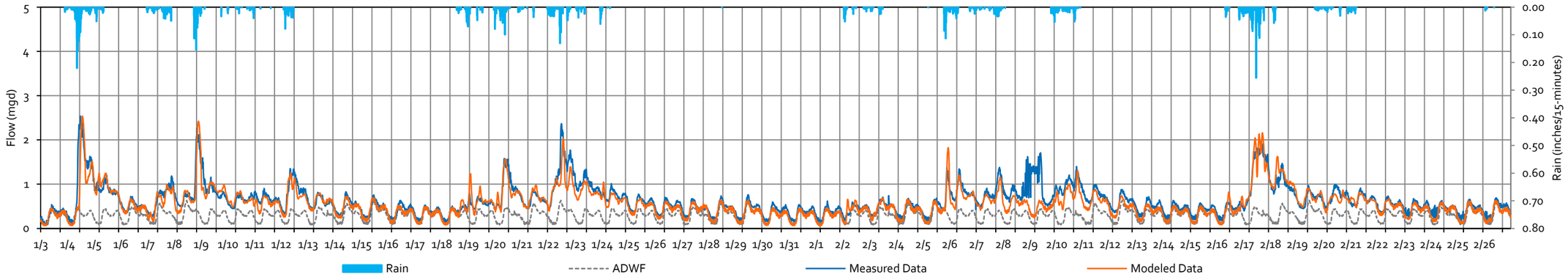
Level Calibration



MB-3 Wet Weather Calibration

Location: Island between Main St. and Hwy 1 on-ramp, ~135-feet north of Atascadero Rd.
Pipeline diameter: 15"
City Manhole ID: 5.25A
Model Pipe ID: 675
Silt Level at Site: 0"

Flow Calibration



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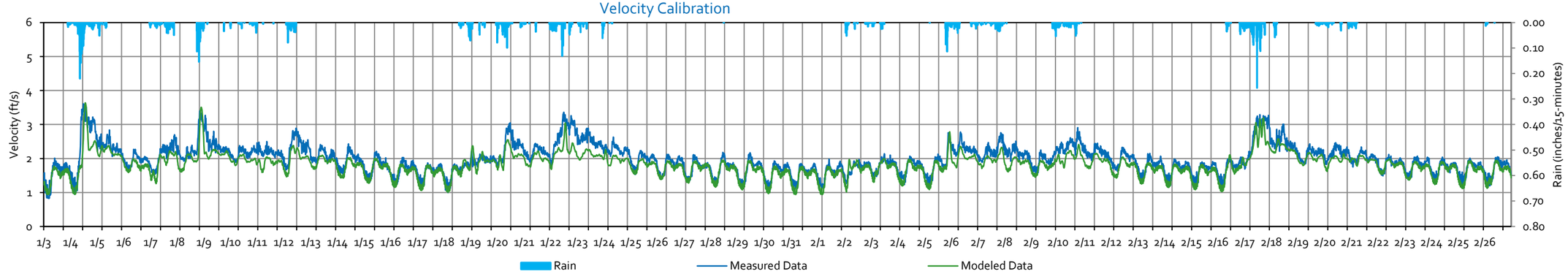

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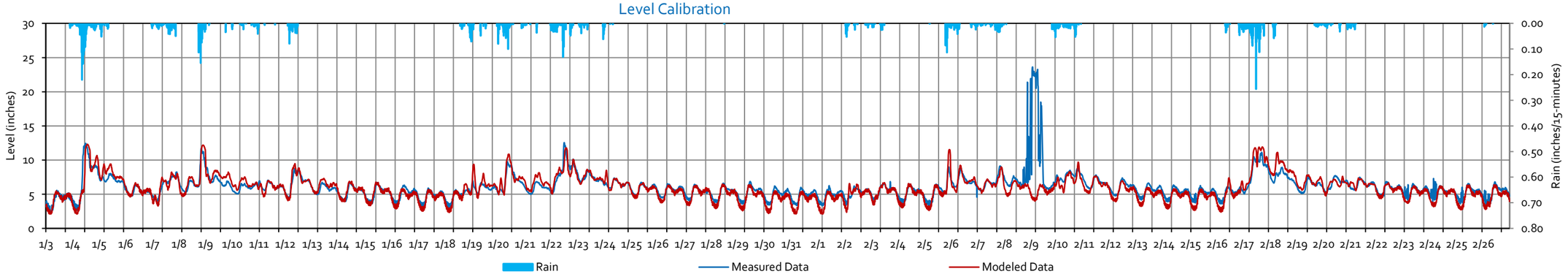
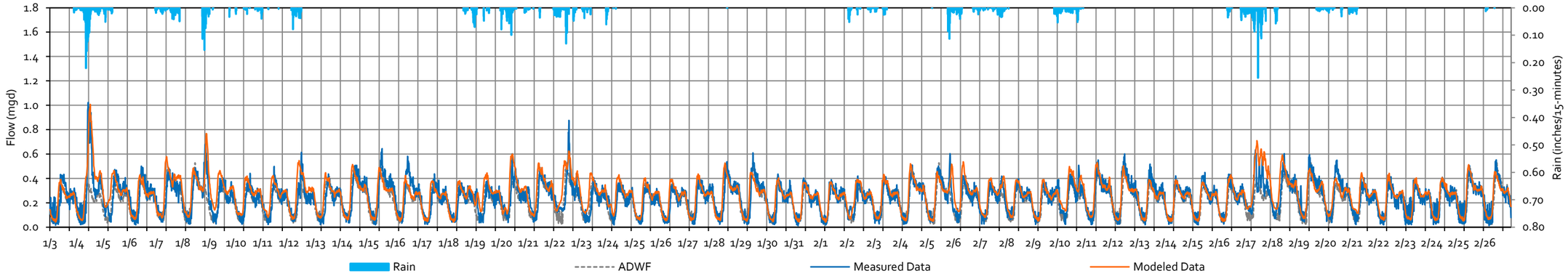


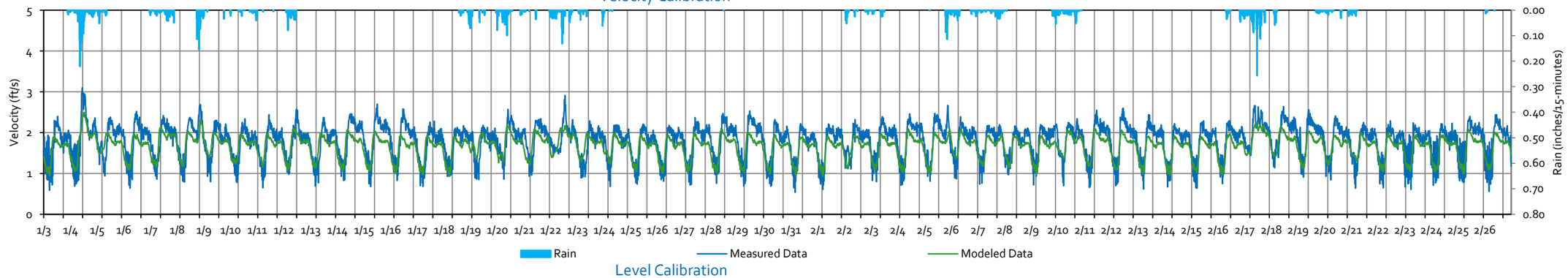
MB-4 Wet Weather Calibration

Location: Embarcadero ~300-feet south of entrance to Morro Dunes mobile home park
Pipeline diameter: 18"
City Manhole ID: 14.5
Model Pipe ID: 61
Silt Level at Site: 0"

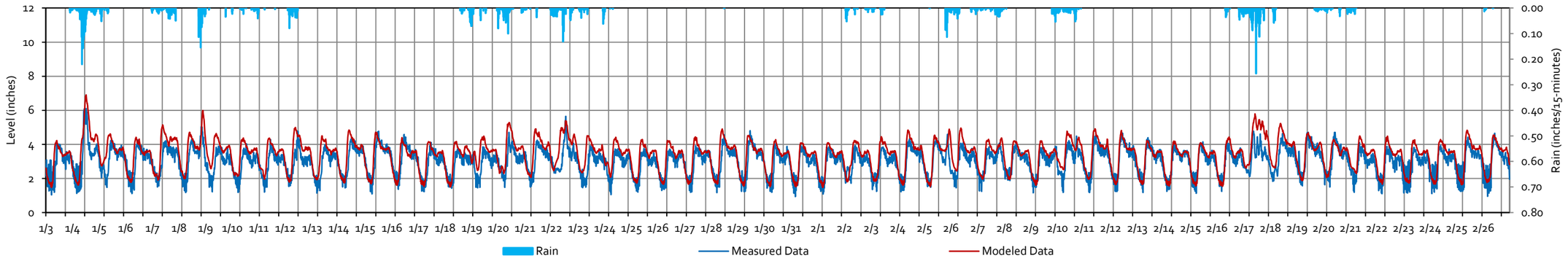
Flow Calibration



Velocity Calibration



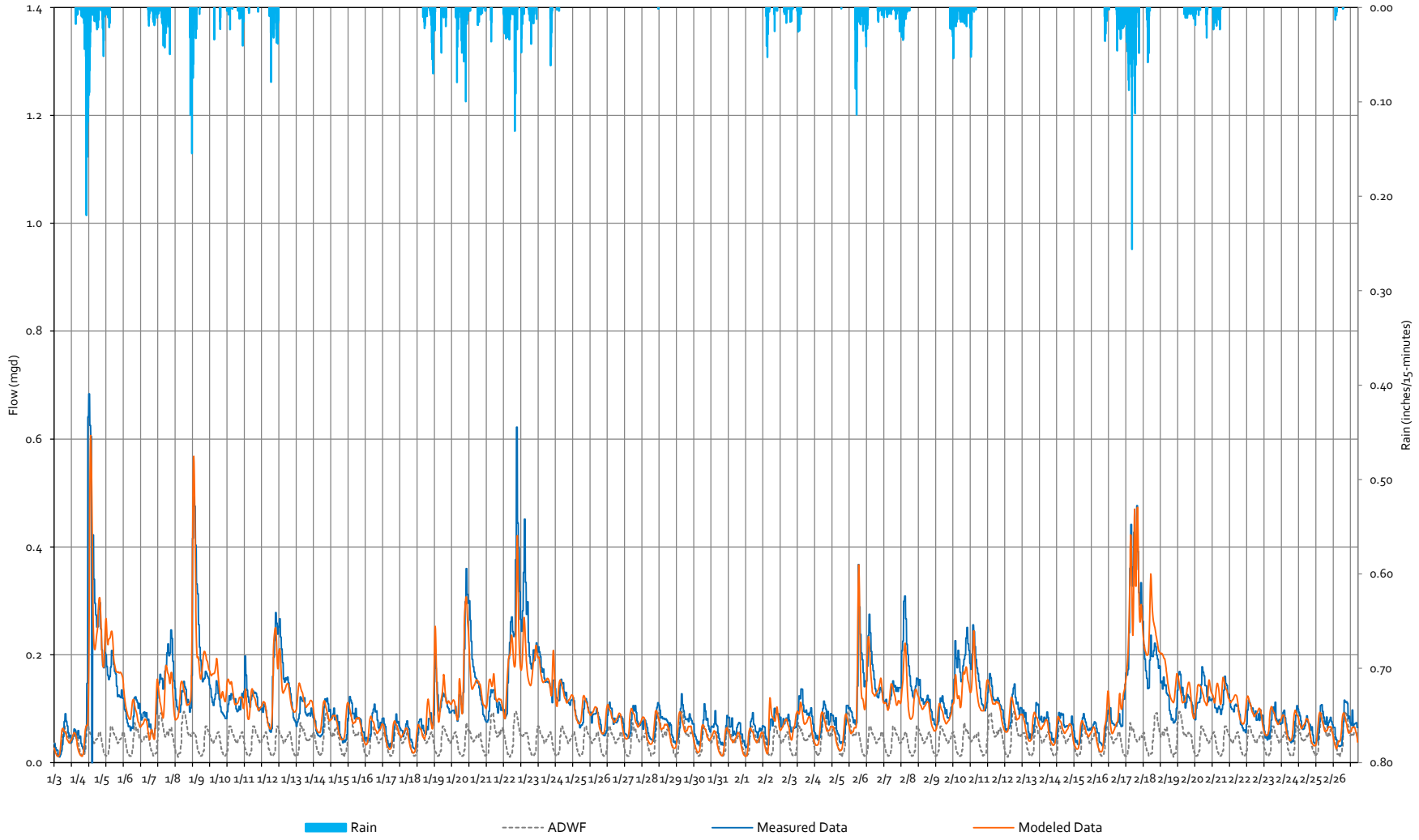
Level Calibration



Lift Station 1 Wet Weather Calibration

Location:
Pipeline diameter: 6"
City Manhole ID: LS.1
Model Pipe ID: FM-9
Silt Level at Site: 0"

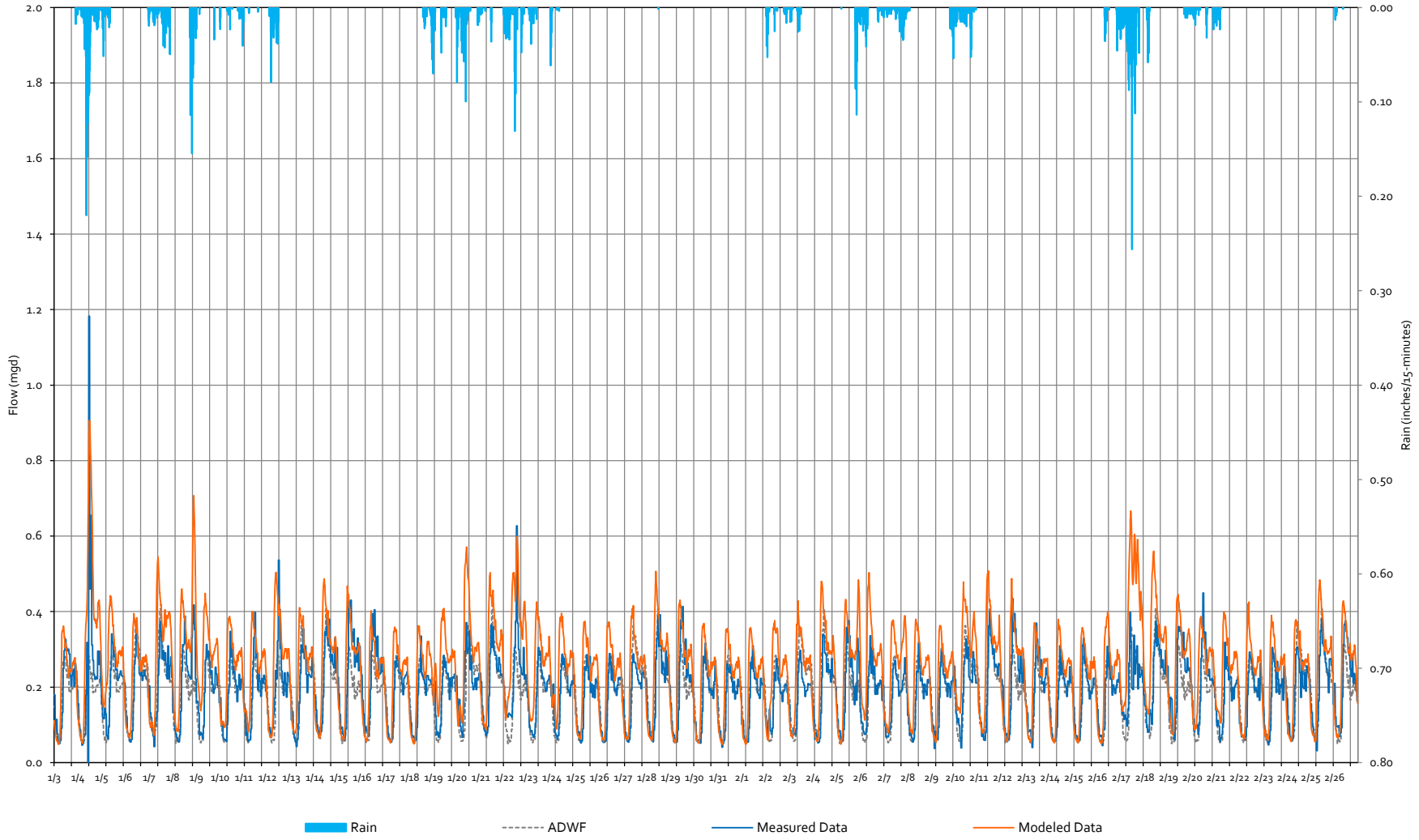
Influent Flow Calibration



Lift Station 2 Wet Weather Calibration

Location:
Pipeline diameter: 12"
City Manhole ID: LS_2_WW
Model Pipe ID: FM-15
Silt Level at Site: 0"

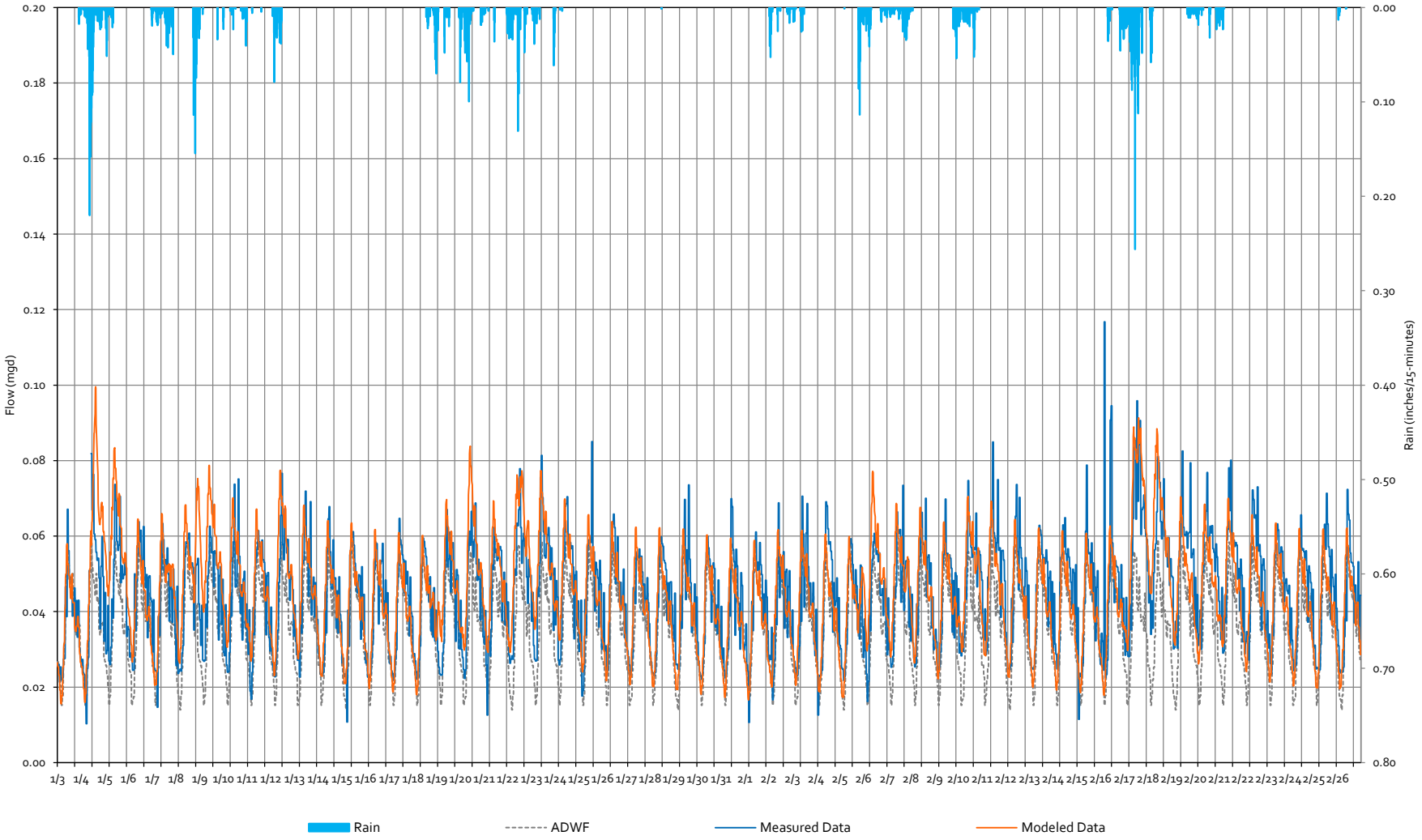
Influent Flow Calibration



Lift Station 3 Wet Weather Calibration

Location:
Pipeline diameter: 8"
City Manhole ID: LS.3
Model Pipe ID: FM-6
Silt Level at Site: 0"

Influent Flow Calibration



Appendix K

WATER DISTRIBUTION SYSTEM IMPROVEMENTS

Project Number: PWFF-1
Project Name: Pipeline along Quintana Road
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 3,000 feet of 8-inch diameter pipeline along Quintana Road, between South Bay Boulevard and La Loma Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressure occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline. ***This project is needed to provide the required fire flows at the City's new WRF site*** .

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	8	10	Replace	3,000	$ 220	$ 660,000	$ 858,000	$ 1,094,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

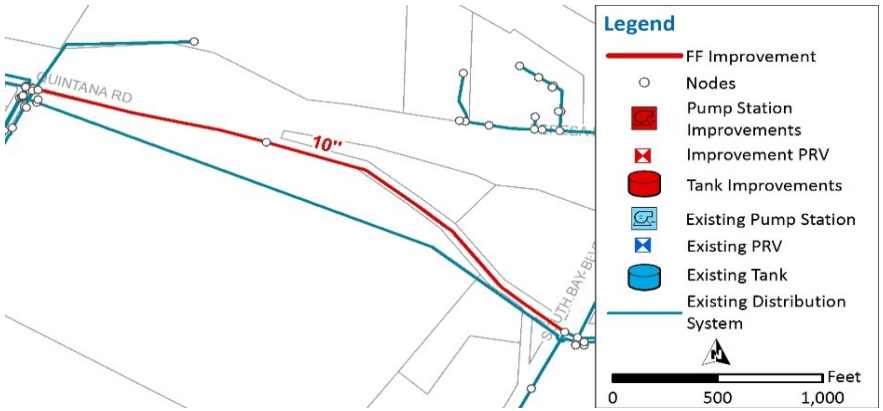
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 1,094,000
Future Users	0%	$ -
Total	100%	$ 1,094,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-2
Project Name: Pipeline along Quintana Road
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 3,300 feet of 8-inch diameter pipeline along Quintana Road, between Morro Bay Boulevard and Quintana Place. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 16-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	8	16	Replace	3,300	$ 300	$ 990,000	$ 1,287,000	$ 1,641,000	2025

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

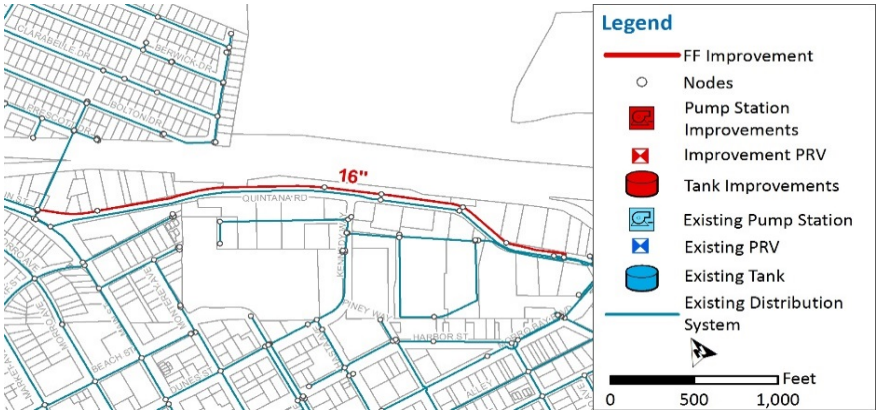
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 1,641,000
Future Users	0%	$ -
Total	100%	$ 1,641,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-3A
Project Name: Pipeline along Main Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 1,650 feet of 8-inch diameter pipeline along Main Street, between Radcliff Avenue and Preston Lane. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	8	12	Replace	1,650	$ 225	$ 371,000	$ 482,000	$ 615,000	2023

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

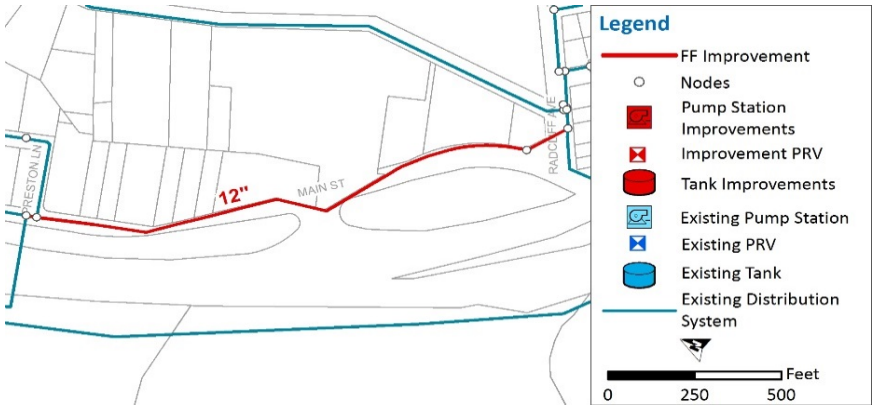
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 615,000
Future Users	0%	$ -
Total	100%	$ 615,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-3B
Project Name: Pipeline along Main Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 600 feet of 8-inch diameter pipeline along Main Street, between Preston Lane and Errol Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	8	10	Replace	600	$ 220	$ 132,000	$ 172,000	$ 219,000	2023

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

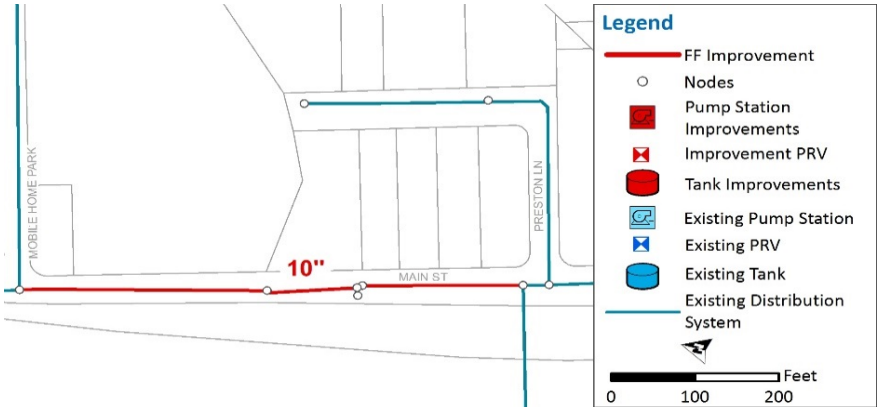
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 219,000
Future Users	0%	$ -
Total	100%	$ 219,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-4
Project Name: Pipeline along Ridgeway Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 830 feet of 6-inch diameter pipeline along Ridgeway Street, between Black Hill Tank and Fairview Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	10	Replace	830	$ 220	$ 183,000	$ 238,000	$ 303,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

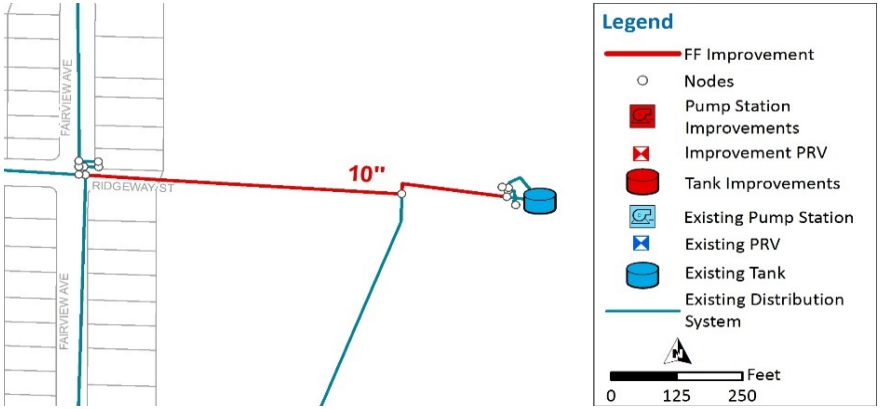
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 303,000
Future Users	0%	$ -
Total	100%	$ 303,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-5A
Project Name: Pipeline along Highway 41 and Main Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 270 feet of 8-inch diameter pipeline along Highway 41, between Sunset Avenue and Main Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that an additional 8-inch diameter pipeline be added to complete the loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	270	$ 175	$ 47,000	$ 61,000	$ 78,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

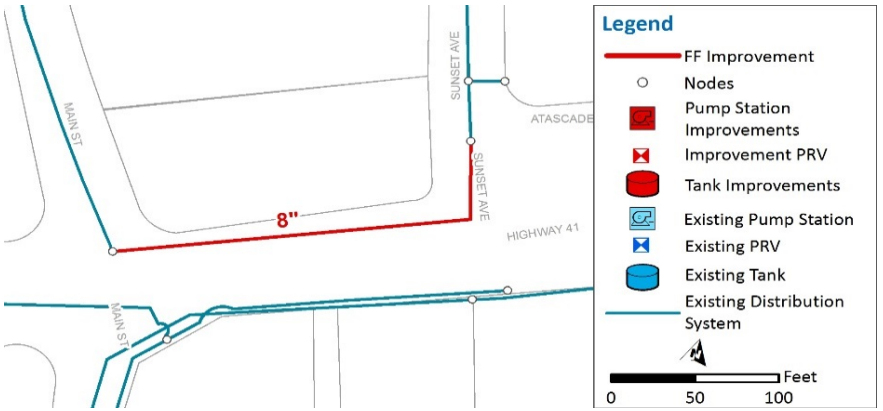
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 78,000
Future Users	0%	$ -
Total	100%	$ 78,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-5B
Project Name: Pipeline along Highway 41 and Main Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 870 feet of 6-inch diameter pipeline along Main Street, between Highway 41 and Hill Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	870	$ 175	$ 152,000	$ 198,000	$ 252,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 252,000
Future Users	0%	$ -
Total	100%	$ 252,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-5C
Project Name: Pipeline along Highway 41 and Main Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 950 feet of 6-inch diameter pipeline along Main Street, between Hill Street and Avalon Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	10	Replace	950	$ 220	$ 209,000	$ 272,000	$ 346,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 346,000
Future Users	0%	$ -
Total	100%	$ 346,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-6A
Project Name: Pipeline along Highway 41 and Main Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 100 feet of 8-inch diameter pipeline along Sunset Avenue, between Highway 41 and Atascadero Road. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that an additional 8-inch diameter pipeline be added to convey water from Upper Kings Zone to Elena Zone.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in/in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe and Interstate Crossing	--	8/20	New	270	$ 470	$ 127,000	$ 165,000	$ 211,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

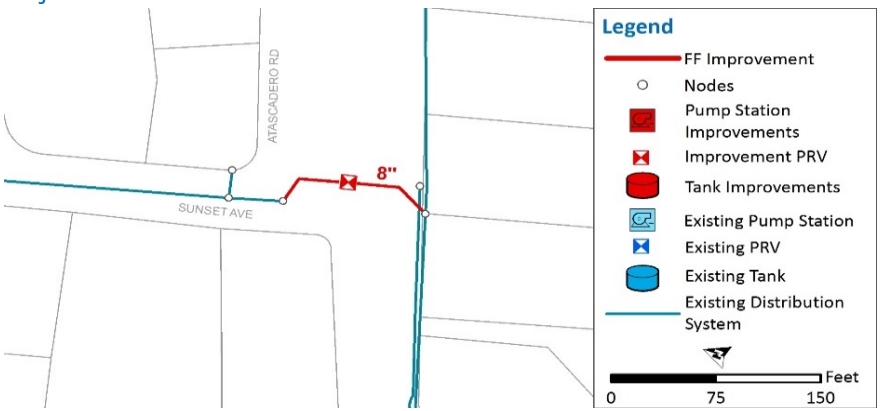
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 211,000
Future Users	0%	$ -
Total	100%	$ 211,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-6B
Project Name: PRV from Upper Kings Zone to Elena Zone
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along improvement project PWFF-6A, between Highway 41 and Atascadero Road. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new pressure reducing valve be added to convey flow from Upper Kings Zone to Elena Zone. ***The pressure setting for this PRV will be low enough so it only opens during fire flow conditions.***

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of PRVs (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

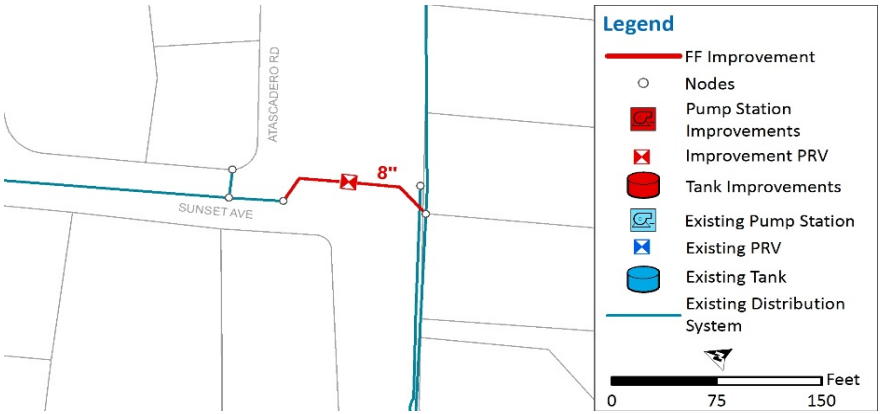
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 312,000
Future Users	0%	$ -
Total	100%	$ 312,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-7
Project Name: Pipeline along Juniper Avenue
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 570 feet of 6-inch diameter pipeline along Juniper Avenue, between Elena Street and San Joaquin Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	570	$ 175	$ 100,000	$ 130,000	$ 166,000	2023

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 166,000
Future Users	0%	$ -
Total	100%	$ 166,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-8A
Project Name: Pipeline along San Joaquin Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 240 feet of 8-inch diameter pipeline along San Joaquin Street, between Juniper Avenue and Ironwood Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	240	$ 175	$ 42,000	$ 55,000	$ 70,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

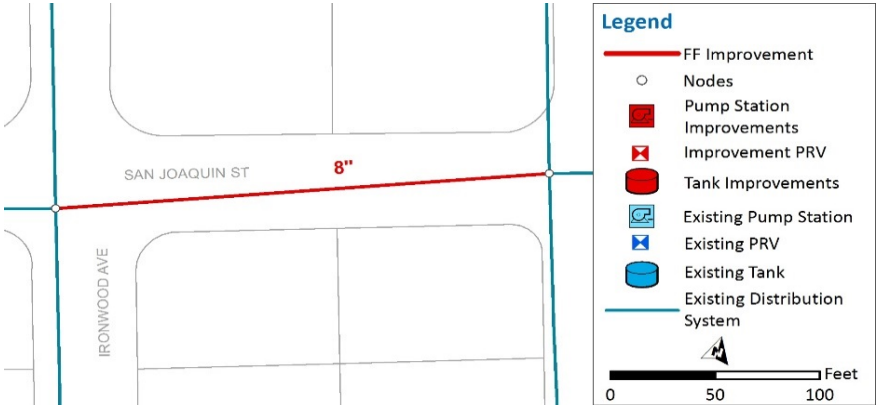
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 70,000
Future Users	0%	$ -
Total	100%	$ 70,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-8B
Project Name: Pipeline along San Joaquin Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 230 feet of 6-inch diameter pipeline along San Joaquin Street between Ironwood Avenue and Hemlock Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	230	$ 175	$ 40,000	$ 52,000	$ 66,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

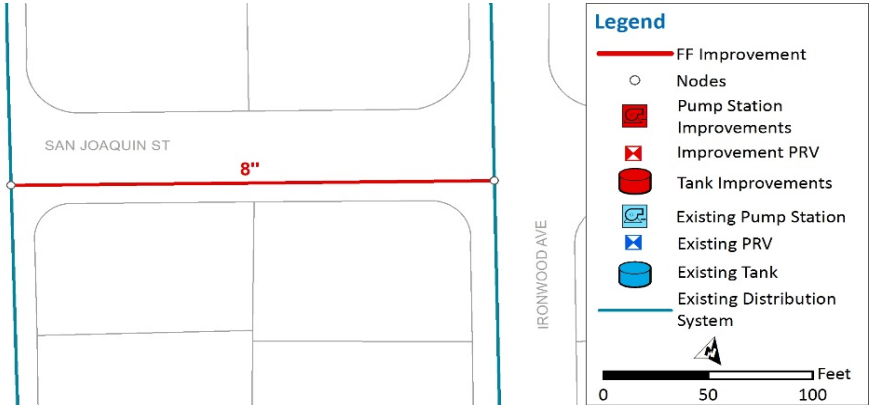
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 66,000
Future Users	0%	$ -
Total	100%	$ 66,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-8C
Project Name: Pipeline along San Joaquin Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 100 feet of 8-inch diameter pipeline along San Joaquin Street, between Hemlock Avenue and Greenwood Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	100	$ 175	$ 18,000	$ 23,000	$ 30,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

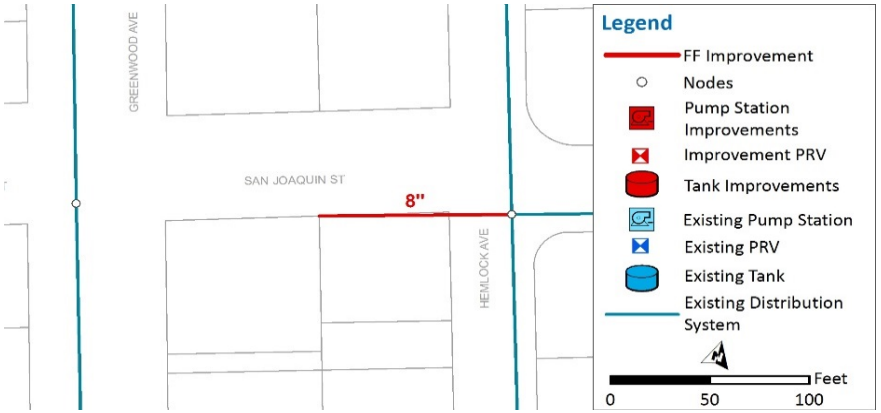
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 30,000
Future Users	0%	$ -
Total	100%	$ 30,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-8D
Project Name: Pipeline along San Joaquin Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 970 feet of 10-inch diameter pipeline along San Joaquin Street, between Hemlock Avenue and Alder Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 10-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	10	New	970	$ 220	$ 213,000	$ 277,000	$ 353,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 353,000
Future Users	0%	$ -
Total	100%	$ 353,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-8E
Project Name: PRV on San Joaquin Street
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along San Joaquin Street, between Hemlock Avenue and Greenwood Avenue (improvement PWFF-8C). The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new pressure reducing valve be added to convey flow from Nutmeg Zone to Elena Zone. ***The pressure setting for this PRV will be low enough so it only opens during fire flow conditions.***

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of PRVs (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 312,000
Future Users	0%	$ -
Total	100%	$ 312,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-9A
Project Name: Pipeline along Greenwood Avenue and San Jacinto Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 640 feet of 6-inch diameter pipeline along Greenwood Avenue between San Joaquin Street and San Jacinto Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	10	Replace	640	$ 220	$ 141,000	$ 183,000	$ 234,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

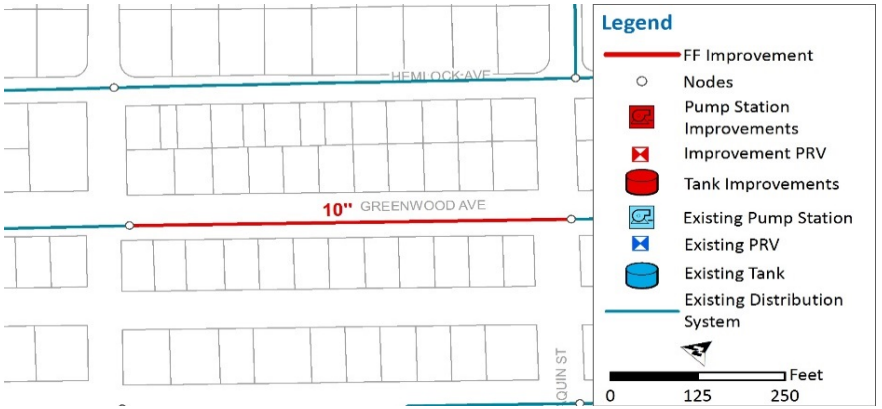
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 234,000
Future Users	0%	$ -
Total	100%	$ 234,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-9B
Project Name: Pipeline along Greenwood Avenue and San Jacinto Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 850 feet of 10-inch diameter pipeline along San Jacinto Street, between Greenwood Avenue and Alder Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 10-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	10	New	850	$ 220	$ 187,000	$ 243,000	$ 310,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

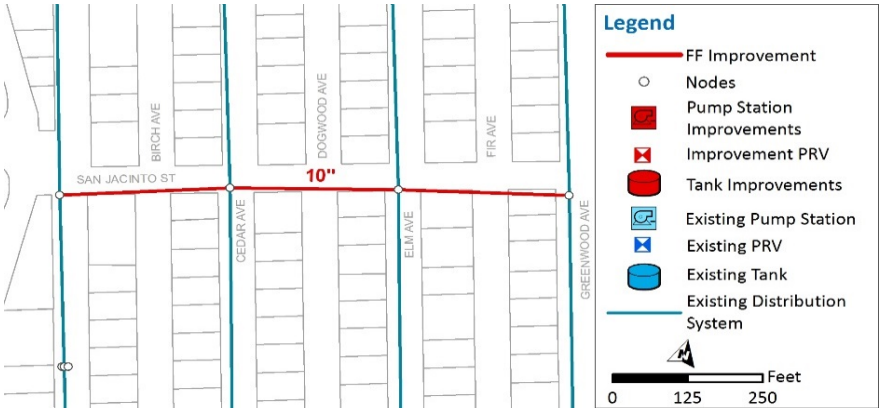
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 310,000
Future Users	0%	$ -
Total	100%	$ 310,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-10
Project Name: Pipeline along Sequoia Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 540 feet of 8-inch diameter pipeline along Sequoia Street between Dogwood Avenue and Main Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	8	10	Replace	540	$ 220	$ 119,000	$ 155,000	$ 197,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

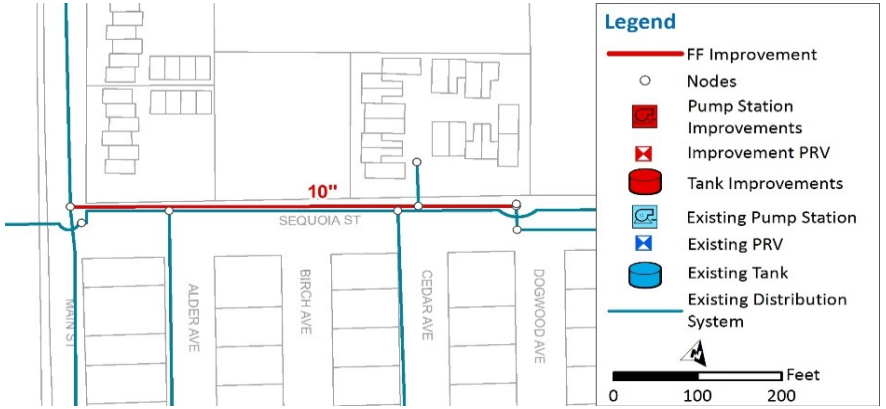
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 197,000
Future Users	0%	$ -
Total	100%	$ 197,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-11
Project Name: Pipeline along Las Vegas Street and Main Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 1,610 feet of 6-inch diameter pipeline along Las Vegas Street between Elm Avenue and Main Street, and along Main Street between Las Vegas Street and Elena Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	1,610	$ 175	$ 282,000	$ 367,000	$ 467,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 467,000
Future Users	0%	$ -
Total	100%	$ 467,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-12
Project Name: Pipeline along Highway 41 and Hill Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 1,290 feet of 8-inch diameter pipeline along Highway 41, between Ironwood Avenue and Hill Street, along Hill Street between Highway 41 and Rockview Street, and along the street east of Hill Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	1,290	$ 175	$ 226,000	$ 294,000	$ 375,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

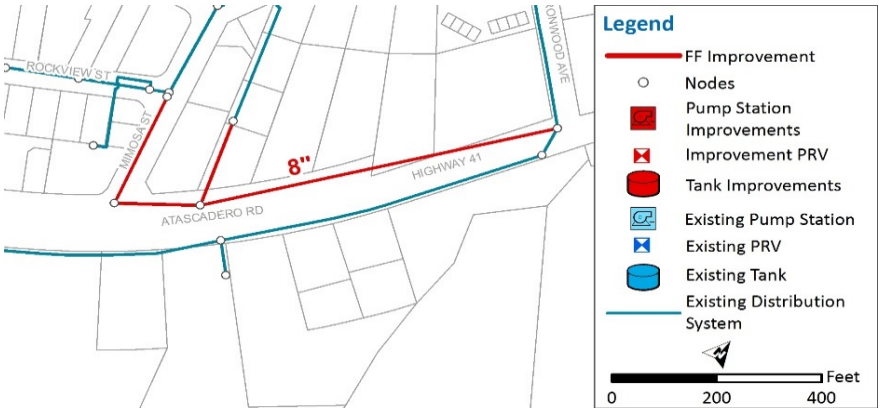
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 375,000
Future Users	0%	$ -
Total	100%	$ 375,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-13
Project Name: Pipeline along Ponderosa Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 910 feet of 6-inch diameter pipeline along the entire length of Ponderosa Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	910	$ 175	$ 159,000	$ 207,000	$ 264,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

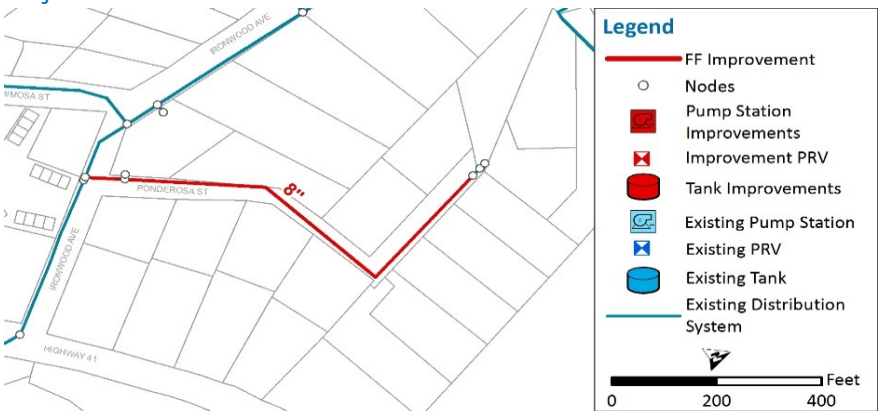
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 264,000
Future Users	0%	$ -
Total	100%	$ 264,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-14
Project Name: Pipeline along Mimosa Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 580 feet of 6-inch diameter pipeline along Mimosa Street, between Ironwood Avenue and the alley northwest of Hill Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	580	$ 175	$ 102,000	$ 133,000	$ 169,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

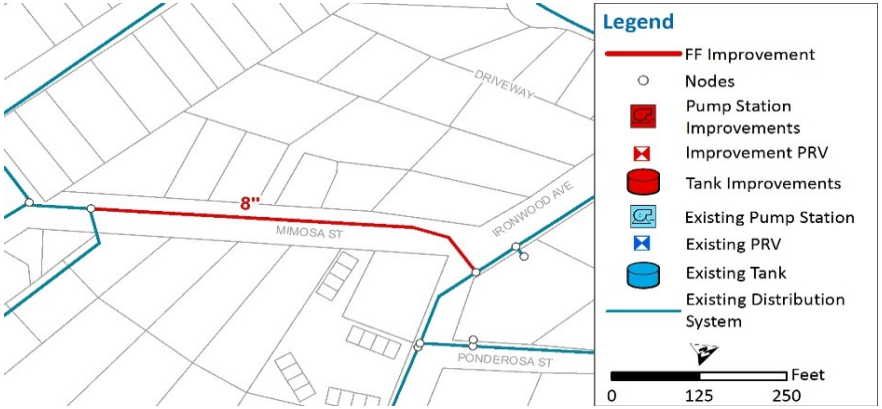
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 169,000
Future Users	0%	$ -
Total	100%	$ 169,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-15A
Project Name: Pipeline along Surf Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 640 feet of 6-inch diameter pipeline along Surf Street, between Market Avenue and west of West Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	640	$ 175	$ 112,000	$ 146,000	$ 186,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 186,000
Future Users	0%	$ -
Total	100%	$ 186,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-15B
Project Name: Pipeline along Surf Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 420 feet of 8-inch diameter pipeline along Surf Street, between Embarcadero Road and west of West Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	420	$ 175	$ 74,000	$ 96,000	$ 123,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

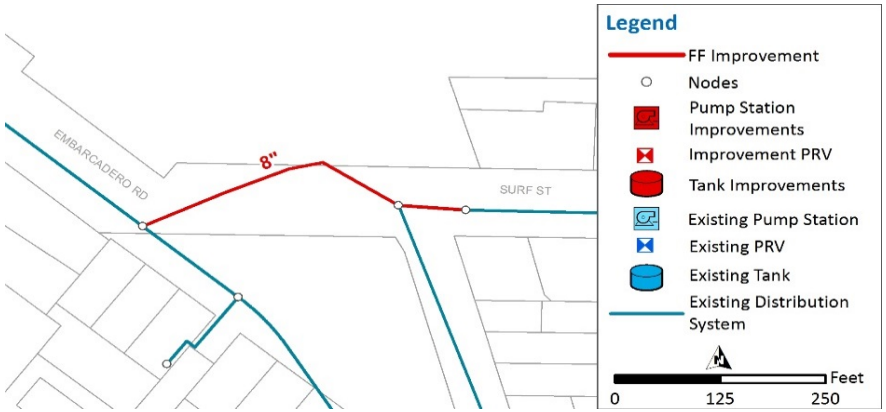
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 123,000
Future Users	0%	$ -
Total	100%	$ 123,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-16
Project Name: Pipeline along Piney Way and Alta Court
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 910 feet of 6-inch diameter pipeline along Piney Way, between Main Street and Alta Court, and along the entire length of Alta Court. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	910	$ 175	$ 159,000	$ 207,000	$ 264,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

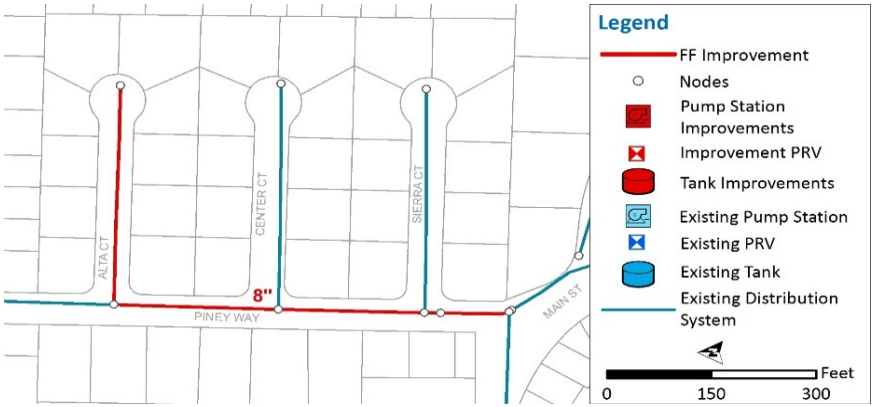
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Improvements	100%	$ 264,000
New Development Service Improvements	0%	$ -
Total	100%	$ 264,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-17
Project Name: Pipeline from Kings Avenue to Arbutus Avenue
System Type: Potable Water

Project Description:

This project includes the addition of approximately 1,050 feet of 8-inch diameter pipeline from the southern end of Fairview Avenue, connecting to Arbutus Avenue to the west (while also connecting to Kings Avenue,Tulare Avenue, and Arcadia Avenue). The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	1,050	$ 175	$ 184,000	$ 239,000	$ 305,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

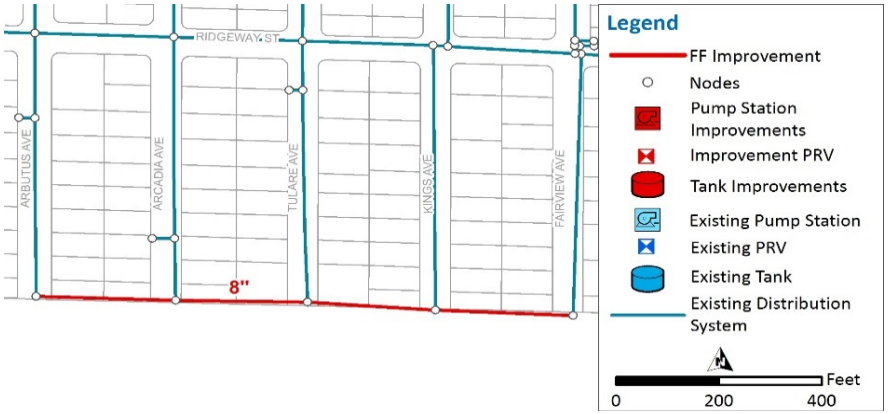
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Improvements	100%	$ 305,000
Future Users	0%	$ -
Total	100%	$ 305,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-18
Project Name: Pipeline along Bolton Drive
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 700 feet of 6-inch diameter pipeline along Bolton Court, between Norwich Avenue and Radcliff Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	700	$ 175	$ 123,000	$ 160,000	$ 204,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

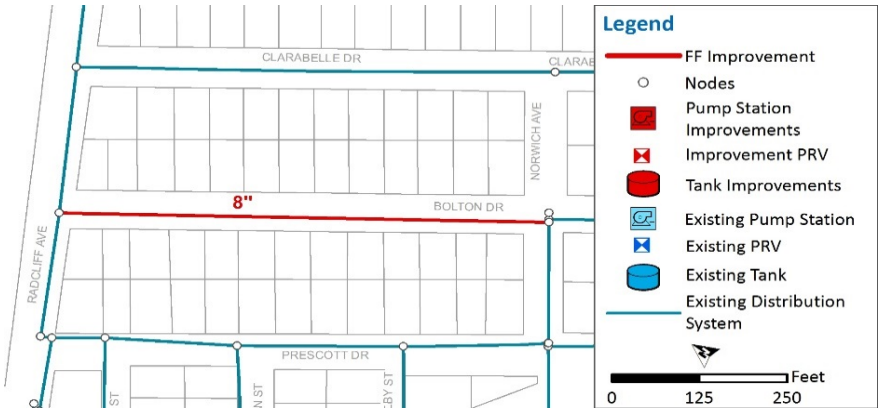
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 204,000
Future Users	0%	$ -
Total	100%	$ 204,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-19
Project Name: Pipeline along Norwich Avenue
System Type: Potable Water

Project Description:

This project includes the addition of approximately 220 feet of 8-inch diameter pipeline along Norwich Avenue, between Clarabell Drive and Bolton Drive. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	220	$ 175	$ 39,000	$ 51,000	$ 65,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

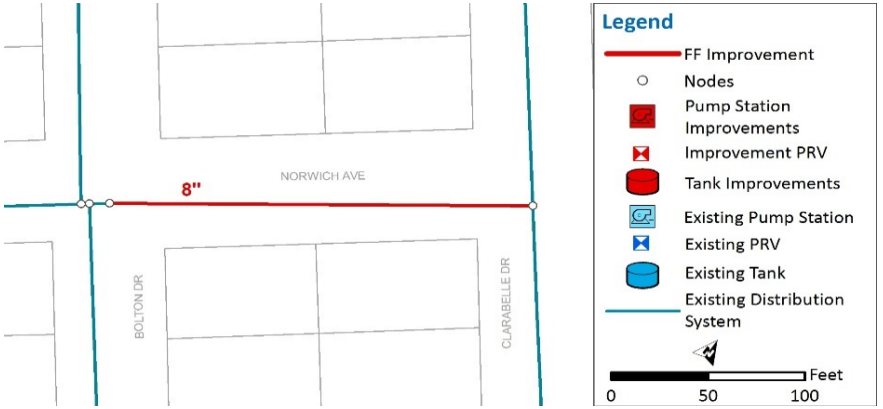
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 65,000
Future Users	0%	$ -
Total	100%	$ 65,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-20
Project Name: Pipeline along Blanca Street and Tuscan Avenue
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 450 feet of 6-inch diameter pipeline along Blanca Street, between the Blana Tanks site and Tuscan Avenue, and along Tuscan Avenue, between Blanca Street and Dawson Street. The residual fire hydrant pressure at a hydrant in this area is below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressure occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	450	$ 175	$ 79,000	$ 103,000	$ 131,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

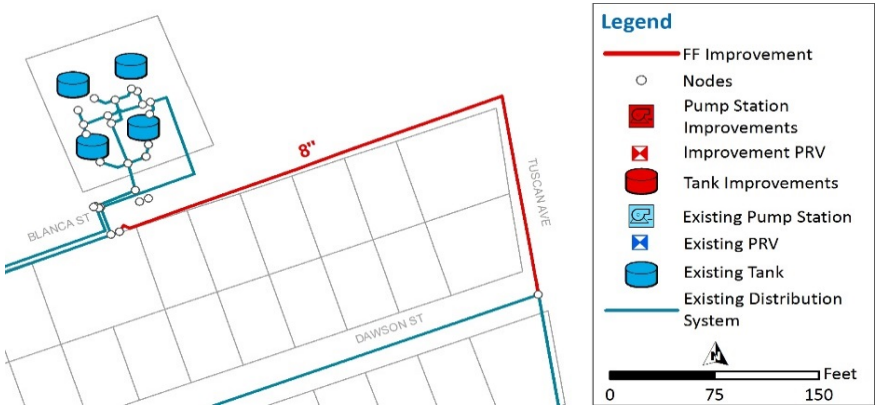
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 131,000
Future Users	0%	$ -
Total	100%	$ 131,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-21
Project Name: Pipeline along Toro Lane
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 800 feet of 6-inch diameter pipeline along Toro Lane, northwest of Yerba Buena Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	800	$ 175	$ 140,000	$ 182,000	$ 232,000	2027

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

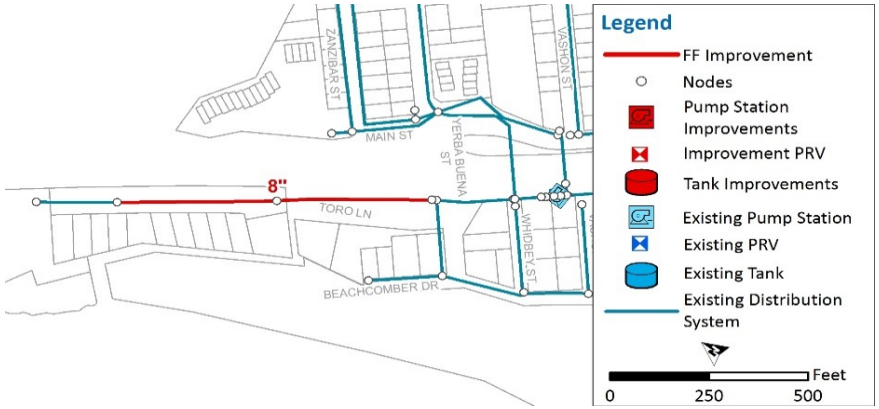
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 232,000
Future Users	0%	$ -
Total	100%	$ 232,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-22
Project Name: Pipeline along Sequoia Court and Ironwood Avenue
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 385 feet of 6-inch diameter pipeline along Ironwood Avenue, between Sequoia Street and Sequoia Court, and along Sequoia Court, from Ironwood Avenue to east of Ironwood. The residual fire hydrant pressure at a hydrant in this area is below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressure occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	385	$ 175	$ 67,000	$ 87,000	$ 111,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

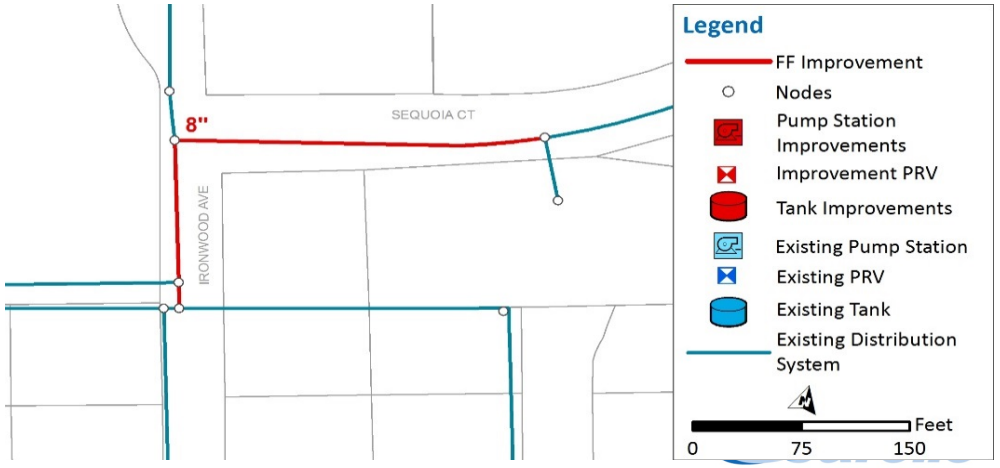
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 111,000
Future Users	0%	$ -
Total	100%	$ 111,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-23
Project Name: Pipeline along Paula Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 200 feet of 8-inch diameter pipeline along Paula Street, between Hemlock Avenue and Greenwood Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	200	$ 175	$ 35,000	$ 46,000	$ 58,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

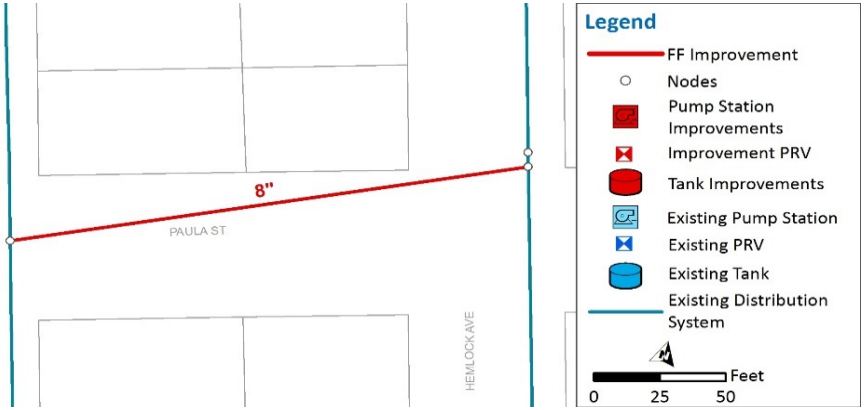
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 58,000
Future Users	0%	$ -
Total	100%	$ 58,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-24
Project Name: Pipeline along Reno Court
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 450 feet of 6-inch diameter pipeline along the entire length of Reno Court. The residual fire hydrant pressure at a hydrant in this area is below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressure occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	450	$ 175	$ 79,000	$ 103,000	$ 131,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

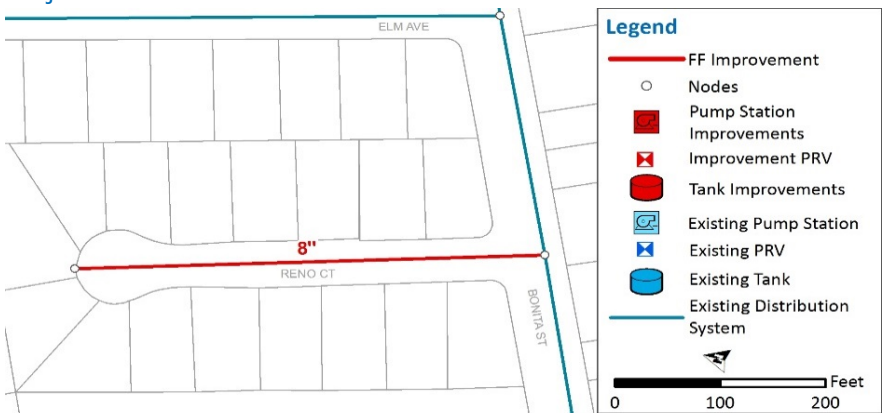
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 131,000
Future Users	0%	$ -
Total	100%	$ 131,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-25
Project Name: Pipeline along Errol Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 545 feet of 4-inch diameter pipeline along the entire length of Errol Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	4	8	Replace	545	$ 175	$ 95,000	$ 124,000	$ 157,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

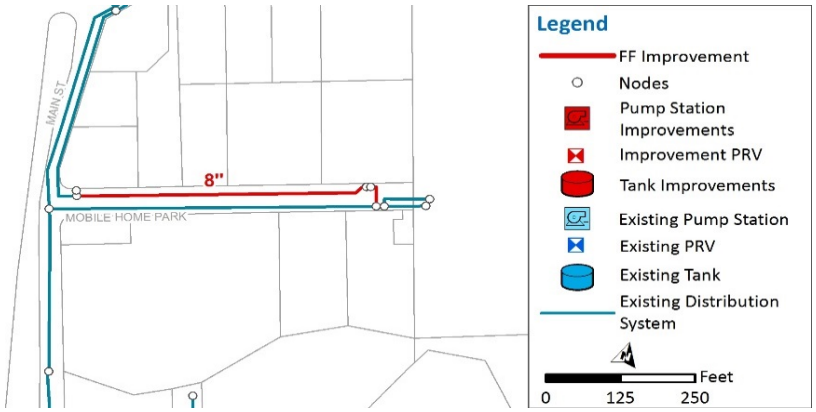
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 157,000
Future Users	0%	$ -
Total	100%	$ 157,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-26
Project Name: Pipeline along Napa Avenue
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 210 feet of 6-inch diameter pipeline along Napa Avenue, north of Dunes Street. The residual fire hydrant pressure at a hydrant in this area is below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressure occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	210	$ 175	$ 37,000	$ 48,000	$ 61,000	2025

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

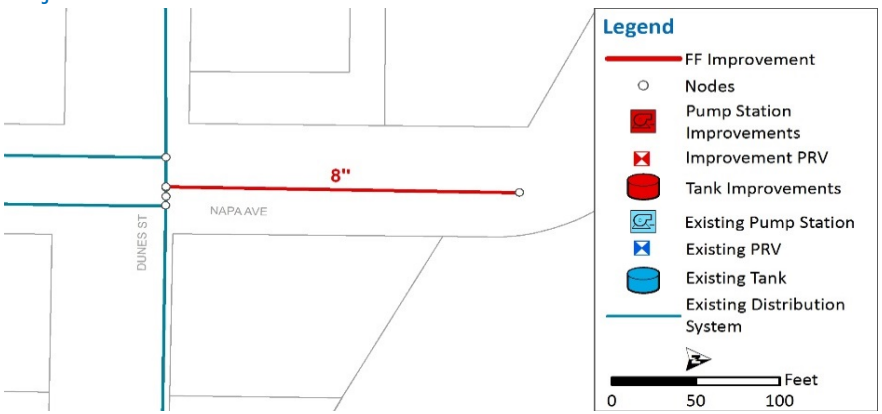
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 61,000
Future Users	0%	$ -
Total	100%	$ 61,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-27A
Project Name: Pipeline along Marina Street
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 220 feet of 2-inch diameter pipeline along Marina Street, between Main Street and Morro Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	2	8	Replace	220	$ 175	$ 39,000	$ 51,000	$ 65,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

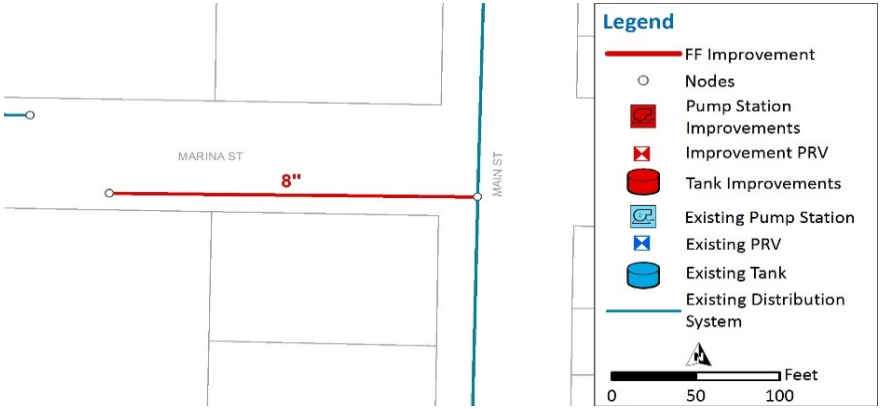
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 65,000
Future Users	0%	$ -
Total	100%	$ 65,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-27B
Project Name: Pipeline along Marina Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 115 feet of 8-inch diameter pipeline along Marina Street, between Morro Avenue and Main Street. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	115	$ 175	$ 20,000	$ 26,000	$ 33,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

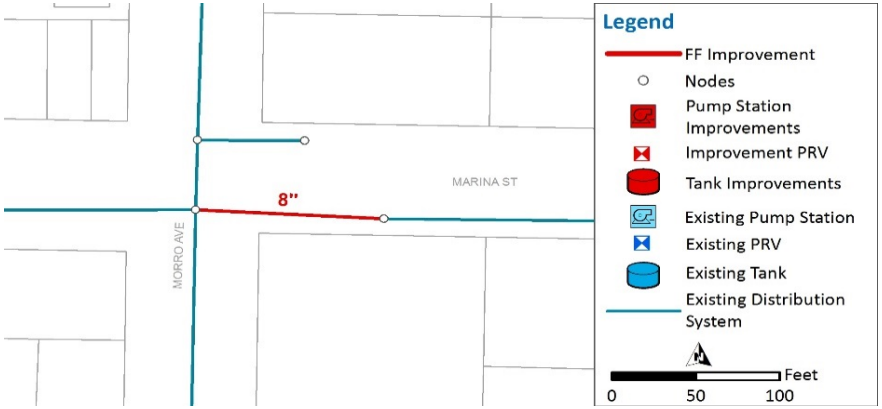
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 33,000
Future Users	0%	$ -
Total	100%	$ 33,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-28
Project Name: Pipeline along Driftwood Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 370 feet of 8-inch diameter pipeline along Driftwood Street, between Morro Avenue and Embarcadero Road. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	370	$ 175	$ 65,000	$ 85,000	$ 108,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

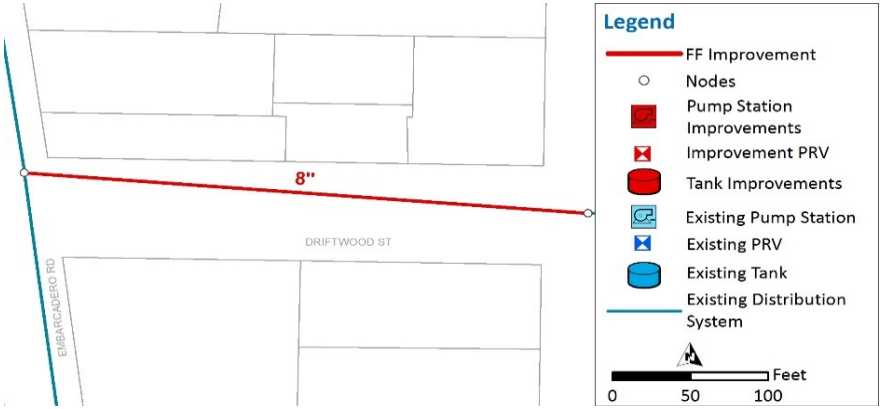
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 108,000
Future Users	0%	$ -
Total	100%	$ 108,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-29
Project Name: Pipeline along South Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 300 feet of 8-inch diameter pipeline along South Street, between Morro Avenue and Embarcadero Road. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new 8-inch diameter pipeline be added to create a loop.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	300	$ 175	$ 53,000	$ 69,000	$ 88,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

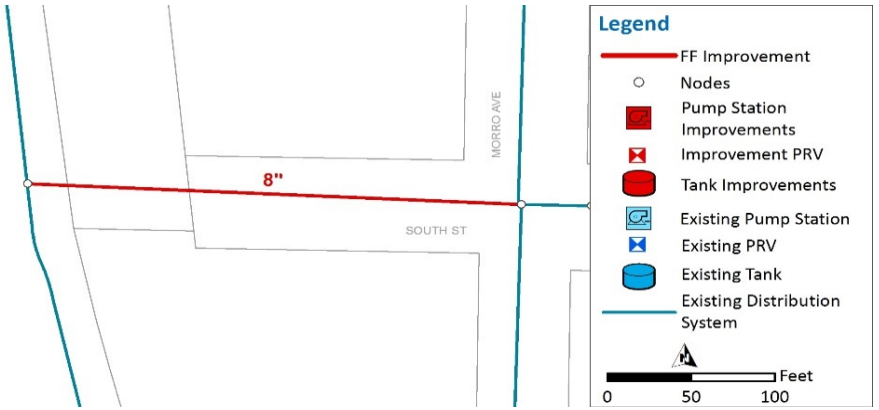
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 88,000
Future Users	0%	$ -
Total	100%	$ 88,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-30
Project Name: Pipeline along Piney Lane
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 370 feet of 6-inch diameter pipeline along the entire length of Piney Lane. The residual fire hydrant pressure at a hydrant in this area is below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressure occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	370	$ 175	$ 65,000	$ 85,000	$ 108,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

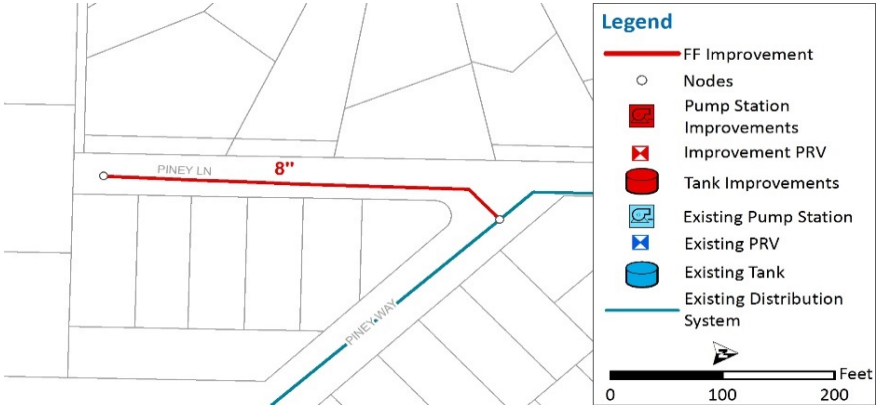
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 108,000
Future Users	0%	$ -
Total	100%	$ 108,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-31
Project Name: PRV on Avalon Street and Ironwood Avenue
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along Avalon Street at the corner of Ironwood Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new pressure reducing valve be added to convey flow from Nutmeg Zone to Elena Zone. ***The pressure setting for this PRV will be low enough so it only opens during fire flow conditions.***

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of Valves (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2025

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

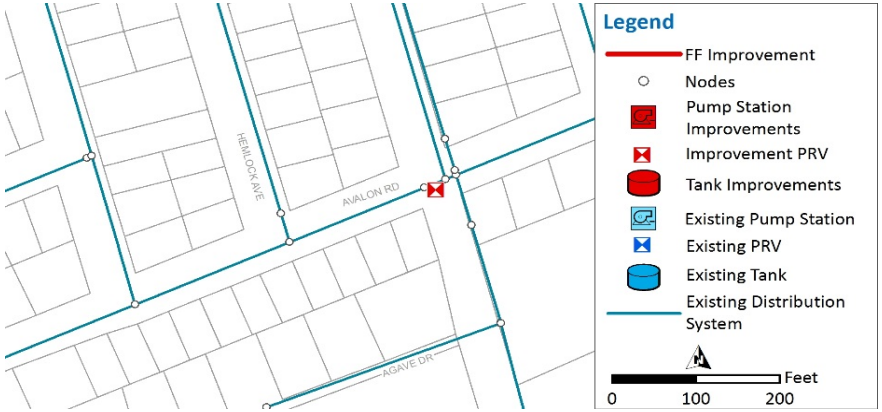
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 312,000
Future Users	0%	$ -
Total	100%	$ 312,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-32
Project Name: PRV on Highway 41 and Ironwood Avenue
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along Highway 41 at the corner of Ironwood Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new pressure reducing valve be added to convey flow from Upper Kings Zone to Elena Zone.***The pressure setting for this PRV will be low enough so it only opens during fire flow conditions.***

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of Valves (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 312,000
Future Users	0%	$ -
Total	100%	$ 312,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWFF-33
Project Name: PRV on Dunes Street and Shasta Avenue
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along Dunes Street at the corner of Shasta Avenue. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new pressure reducing valve be added to convey flow from Upper Kings Zone to Lower Kings Zone. ***The pressure setting for this PRV will be low enough so it only opens during fire flow conditions.***

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of Valves (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

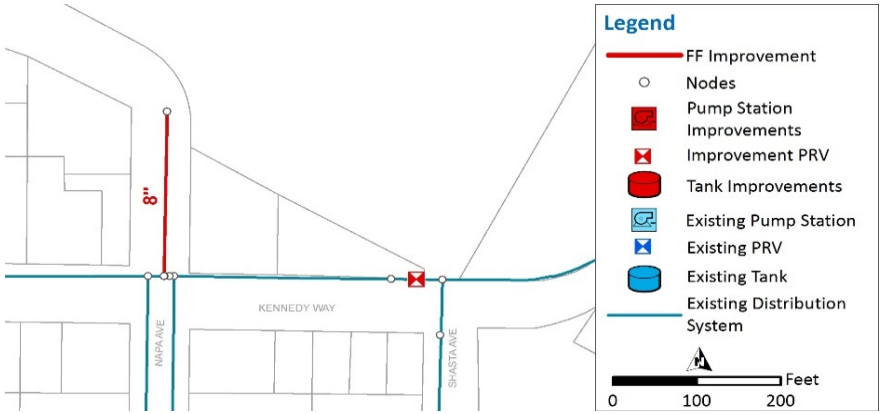
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 312,000
Future Users	0%	$ -
Total	100%	$ 312,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-34
Project Name: PRV on Main Street and Quintana Place
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along Main Street at the corner of Quintana Place. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that a new pressure reducing valve be added to convey flow from Upper Kings Zone to Lower Kings Zone. ***The pressure setting for this PRV will be low enough so it only opens during fire flow conditions.***

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of Valves (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

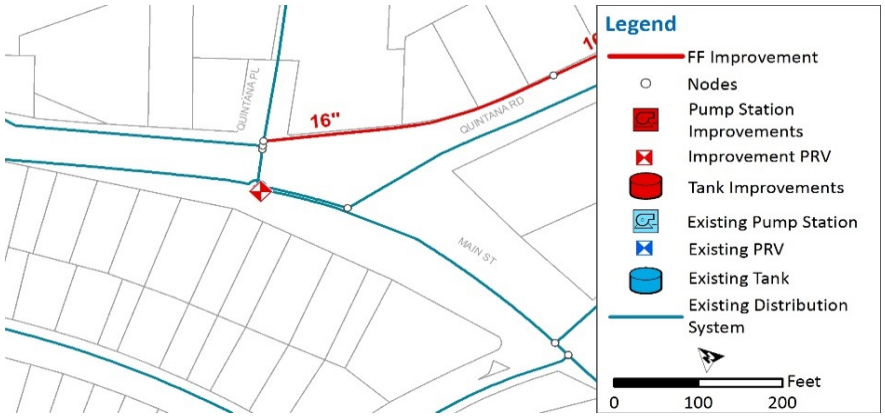
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 312,000
Future Users	0%	$ -
Total	100%	$ 312,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWFF-35
Project Name: Pipeline along Sunset Road
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 2,000 feet of 6-inch diameter pipeline along Sunset Road, between Highway 41 and Hill Street, and along Hill Street, between Sunset Road and Main Street.. The residual fire hydrant pressures in this area are below 20 psi under MDD plus fire flow conditions. To mitigate the low residual pressures occuring under MDD plus fire flow conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	6	8	Replace	2,000	$ 175	$ 350,000	$ 455,000	$ 580,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

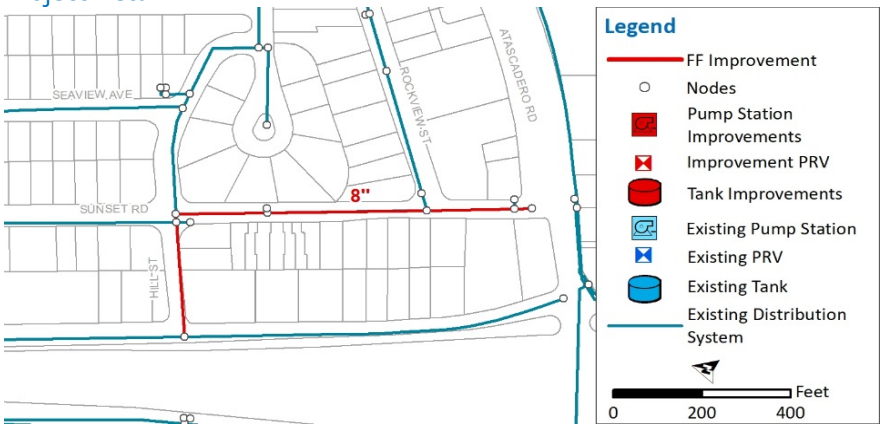
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 580,000
Future Users	0%	$ -
Total	100%	$ 580,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWP-1
Project Name: Fill line for Blanca Tanks
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 2,350 feet of 4-inch diameter pipeline from Vashon Booster Pump Station to the Blanca Tanks. The headloss within this pipeline are approaching the maximum limit under PHD conditions. To mitigate the high headlosses occuring under PHD conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	4	8	Replace	2,210	$ 175	$ 387,000	$ 503,000	$ 641,000	2020
Pipe and Interstate Crossing	--	8/20	Replace	140	$ 470	$ 66,000	$ 86,000	$ 109,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 735,000
Future Users	2%	$ 15,000
Total	100%	$ 750,000

Notes on Cost Estimation:

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

Project Detail:

Project Number: PWP-2
Project Name: Fill line for Nutmeg Tank
System Type: Potable Water

Project Description:

This project includes the replacement of approximately 1,280 feet of 4-inch diameter pipeline from pipe segment upstream of Elena Booster Pump Station to the Nutmeg Tank. The headloss within this pipeline exceed the maximum limit under PHD conditions. To mitigate the high headlosses occuring under PHD conditions, it is recommended that the existing pipeline be replaced with an 8-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	4	8	Replace	1,280	$ 175	$ 224,000	$ 291,000	$ 371,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 364,000
Future Users	2%	$ 7,000
Total	100%	$ 371,000

Notes on Cost Estimation:

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

Project Detail:

Project Number: PWP-3
Project Name: Fill line for Elena Tanks
System Type: Potable Water

Project Description:

This project includes the addition of approximately 1,910 feet of 10-inch diameter pipeline along Main Street, between La Jolla Street and Elena Street. The parallel pipeline will serve as a feed line for the Elena Tanks.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	10	New	1,730	$ 220	$ 381,000	$ 495,000	$ 632,000	2022
Pipe and Interstate Crossing	--	10/24	New	180	$ 520	$ 94,000	$ 122,000	$ 156,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

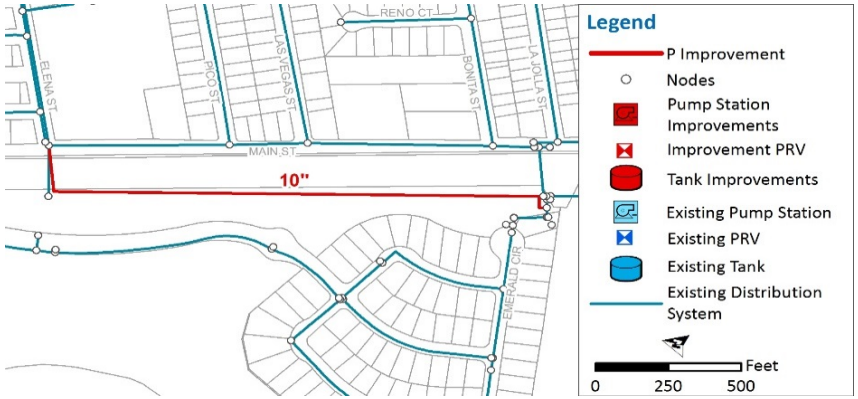
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	92%	$ 725,000
Future Users	8%	$ 63,000
Total	100%	$ 788,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: PWP-4
Project Name: Parallel pipeline on Juniper Avenue
System Type: Potable Water

Project Description:

This project includes the addition of approximately 2,050 feet of 8-inch diameter pipeline along Juniper Avenue, between Elena Street and Sequoia Street. The parallel pipeline will serve as a feed line from the Nutmeg pressure zone to the Blanca Pressure Zone.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	2,050	$ 175	$ 359,000	$ 467,000	$ 595,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 583,000
Future Users	2%	$ 12,000
Total	100%	$ 595,000

Notes on Cost Estimation:

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

Project Detail:

Project Number: PWP-5
Project Name: Pipeline on Sequoia Street
System Type: Potable Water

Project Description:

This project includes the addition of approximately 530 feet of 8-inch diameter pipeline along Sequoia Street, between Dogwood Avenue and Main Street. The parallel pipeline will serve as a feed line from the Nutmeg pressure zone to the Blanca Pressure Zone.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pipe	--	8	New	530	$ 175	$ 93,000	$ 121,000	$ 154,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

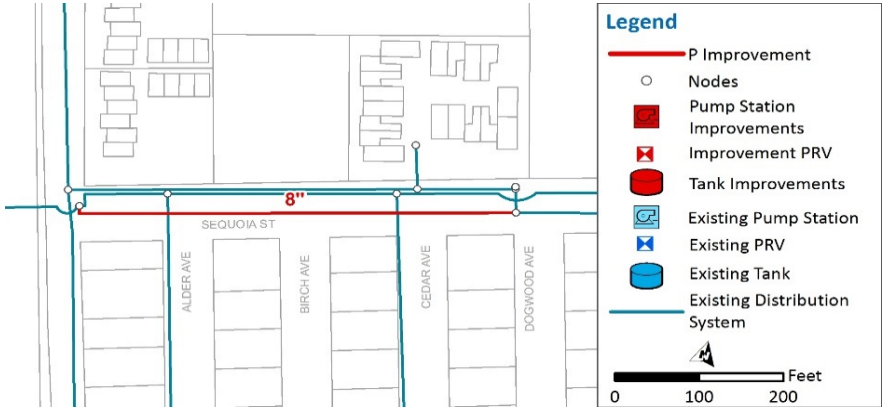
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 151,000
Future Users	2%	$ 3,000
Total	100%	$ 154,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: PWPS-1
Project Name: Elena Booster Pump Station Upgrade
System Type: Potable Water

Project Description:

This project includes the replacement of the two pumps at Elena Booster Pump Station. The existing pump station does not have the capacity to fill Nutmeg Tank under PHD conditions. To mitigate the capacity deficiency occuring under PHD conditions, it is recommended that the booster pump station replace the existing pumps with two larger pumps with a firm capacity of 0.50 mgd.

Project Details:

Project Element	Existing Firm Capacity (mgd)	Proposed Firm Capacity (mgd)	Replace/ New	Total HP	Unit Cost[1] ($/HP)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pump	0.44	0.50	Replace	70	$ 5,000	$ 350,000	$ 455,000	$ 580,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

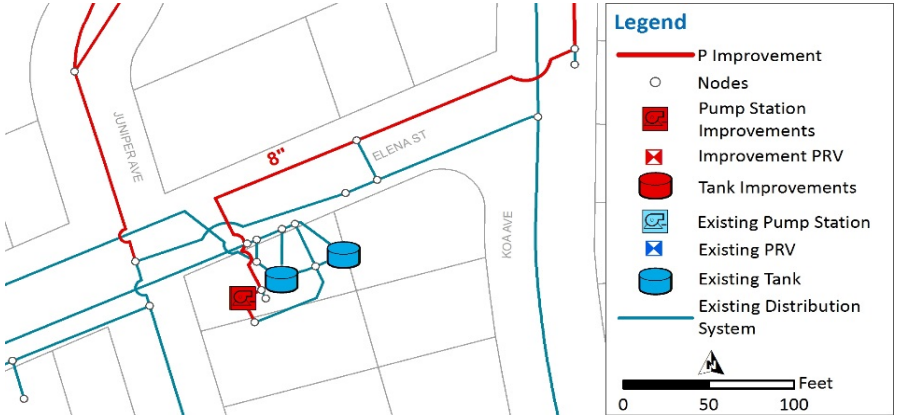
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 568,000
Future Users	2%	$ 12,000
Total	100%	$ 580,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: PWPS-2
Project Name: Kings Booster Pump Station Upgrade
System Type: Potable Water

Project Description:

This project includes the addition of the one fire flow pump at Kings Booster Pump Station. The Ridgeway zone does not have enough storage under MDD plus fire flow conditions. To mitigate the storage deficiency occuring under MDD plus fire flow conditions, it is recommended that a 2.16 mgd fire flow booster pump be added.

Project Details:

Project Element	FF Pump Capacity (mgd) (mgd)	Proposed FF Pump Capacity (mgd)	Replace/ New	Total HP	Unit Cost[1] ($/HP)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pump	--	2.16	New	90	$ 5,000	$ 450,000	$ 585,000	$ 746,000	2023

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

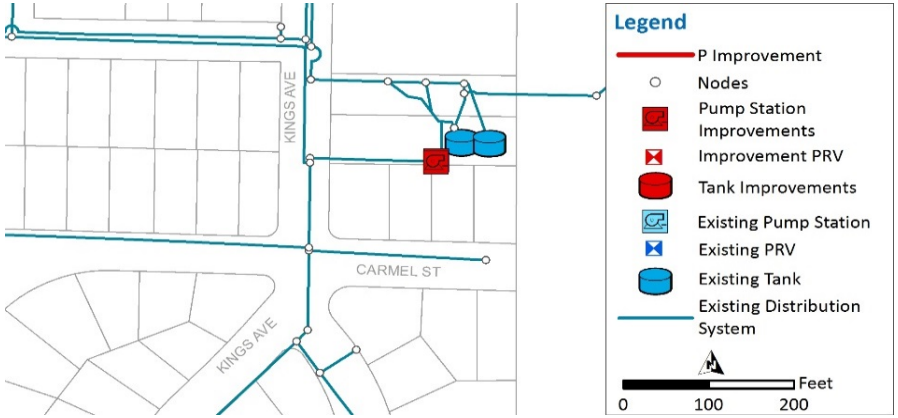
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 746,000
Future Users	0%	$ -
Total	100%	$ 746,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWS-1
Project Name: Nutmeg Tank Upgrade
System Type: Potable Water

Project Description:

This project includes the replacement of Nutmeg Tank. The existing storage tank does not have the capacity to supply Nutmeg Pressure Zone under MDD conditions. To mitigate the capacity deficiency occuring under MDD conditions, it is recommended that the existing storage tank is replaced with a 1.0 MG storage tank.

Project Details:

Project Element	Existing Capacity (MG)	Proposed Capacity (MG)	Replace/ New	Proposed Capacity (gal)	Unit Cost[1] ($/gal)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Storage Tank	0.14	1.0	Replace	1,000,000	$ 2.00	$ 2,000,000	$ 2,600,000	$ 3,315,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

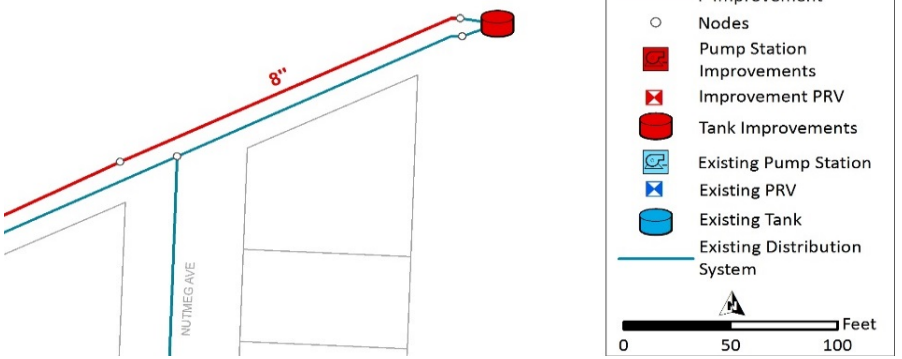
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 3,249,000
Future Users	2%	$ 66,000
Total	100%	$ 3,315,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: PWS-2
Project Name: Elena Tank Upgrade
System Type: Potable Water

Project Description:

This project includes the replacement of the Elena Tanks. The existing storage tanks do not have the capacity to supply Elena Pressure Zone under MDD conditions. To mitigate the capacity deficiency occuring under MDD conditions, it is recommended that the existing storage tanks are replaced with one 0.15 MG storage tank.

Project Details:

Project Element	Existing Capacity (MG)	Proposed Capacity (MG)	Replace/ New	Proposed Capacity (gal)	Unit Cost[1] ($/gal)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Storage Tank	0.12	0.15	Replace	150,000	$ 2.50	$ 375,000	$ 488,000	$ 622,000	2021
On Site Piping	--	--	--	--	--	$ 100,000	$ 130,000	$ 166,000	2021
On Site Civil	--	--	--	--	--	$ 50,000	$ 65,000	$ 83,000	2021
Tank Demolition	--	--	--	--	--	$ 100,000	$ 130,000	$ 166,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

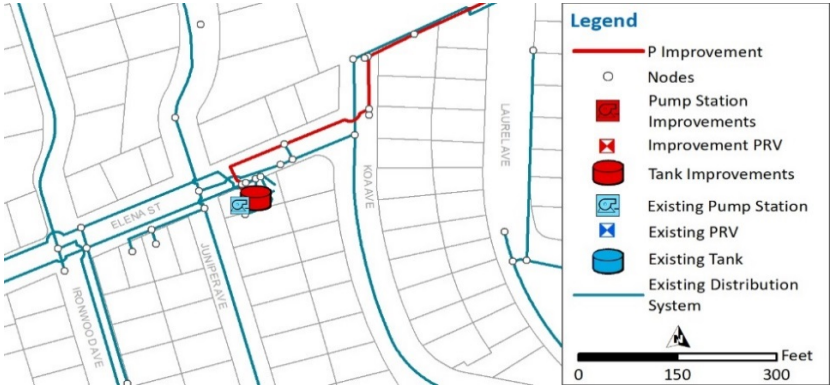
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	92%	$ 954,000
Future Users	8%	$ 83,000
Total	100%	$ 1,037,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: PWV-1
Project Name: PRV on Juniper Avenue
System Type: Potable Water

Project Description:

This project includes the addition of a pressure reducing valve along improvement project Junpier Avenue, between Elena Street and San Joaquin Street. The existing Blanca zone recieves water from Nutmeg zone through a bleed valve that is manually opened when necessary. To eliminate this manually opened bleed valve , it is recommended a pressure reducing valve be installed to convey water from Nutmeg zone to Blanca zone.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	No. of PRVs (Units)	Unit Cost[1] ($/PRV)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Pressure Reducing Valve	--	8	New	1	$ 187,500	$ 188,000	$ 244,000	$ 312,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

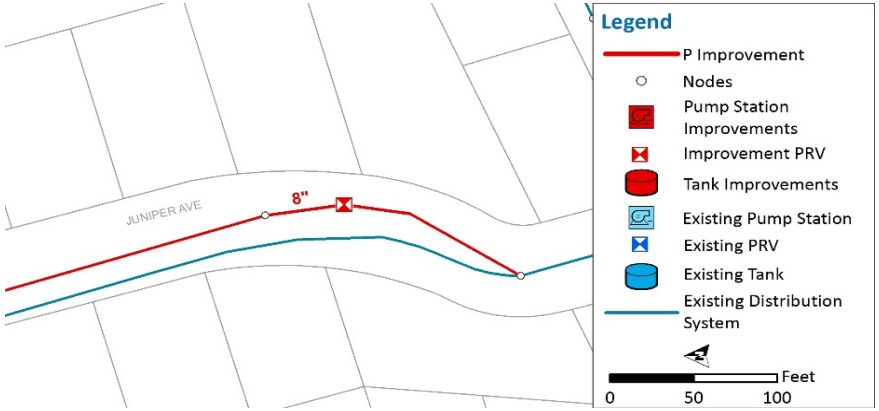
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	98%	$ 306,000
Future Users	2%	$ 6,000
Total	100%	$ 312,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: PWRR-1
Project Name: Pipeline R&R Program
System Type: Potable Water

Project Description:

This project includes the rehabilitation or replacement of pipelines. This Master Plan assumes an allowance of $100,000 a year on rehabilitation or replacement related projects starting in 2019.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Annual Cost ($/yr)	Total Capital Improvement Cost[3] ($)	Project Schedule
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2019
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2020
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2021
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2022
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2023
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2024
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2025
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2026
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2027
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 100,000	2028
Pipe R&R Program	1-16	> 6	R&R	Varies	$ 100,000	$ 1,200,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated c

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 2,200,000
Future Users	0%	$ -
Total	100%	$ 2,200,000

Notes on Cost Estimation:

As an existing R&R project, current users are assigned 100-percent of the project's cost.

Project Number: PWO-1
Project Name: Water Master Plan Update
System Type: Potable Water

Project Description:

It is recommended that the City undergoes a Water Master Plan Update every 5-years to evaluate wastewater collection system.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Water Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2023
Water Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2028
Water Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2029-2040
Water Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 600,000
Future Users	0%	$ -
Total	100%	$ 600,000

Notes on Cost Estimation:

As a Water Master Plan Update, current users are assigned 100-percent of the project's cost.

Project Number: PWO-2
Project Name: Re-Skin Desalination Plant Building
System Type: Potable Water

Project Description:

This project includes the re-skinning of the desalination plant building. This consists of the material and labor necessary to remove the existing corrugated metal building siding and replace with new.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Desalination Plant Building Re-Skinning	--	--	New	--		$ 36,300	$ 47,000	$ 60,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

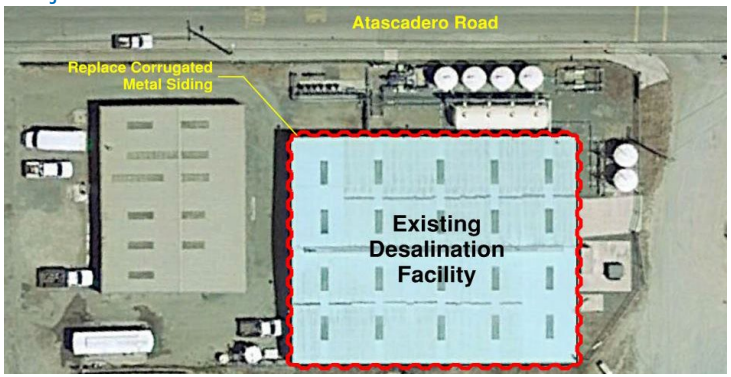
(2) Estimated Construction Cost includes a 30% contingency of the baseline construction cost.

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 60,000
Future Users	0%	$ -
Total	100%	$ 60,000

Project Detail:



Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Number: PWO-3
Project Name: Control Upgrades at Desalination Plant
System Type: Potable Water

Project Description:

This project includes control upgrades at the Desalination plant.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Desalination Plant Control Upgrades	--	--	New	--		$ 888,000	$ 1,154,000	$ 1,472,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 1,472,000
Future Users	0%	$ -
Total	100%	$ 1,472,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Number: PWO-4
Project Name: Add Screening at Desalination Plant
System Type: Potable Water

Project Description:

This project includes the addition of screening at the calcium carbonate tanks at the Desalination plant. The screening will be accomplished by covering the existing tanks with trees planted adjanent to the sidewalk.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Visual Screening at Calcium Carbonate Tanks	--	--	New	--		$ 10,000	$ 13,000	$ 17,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

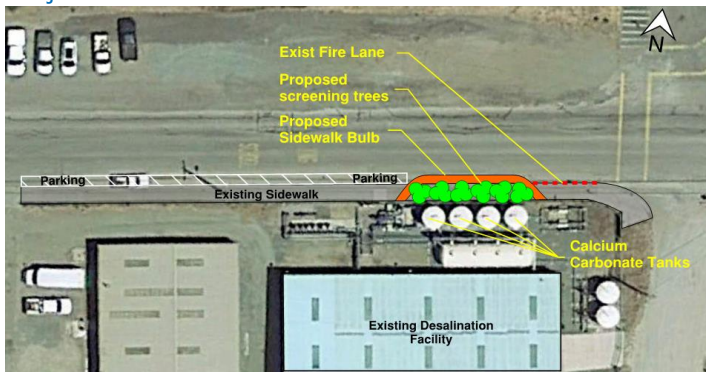
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 17,000
Future Users	0%	$ -
Total	100%	$ 17,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Project Number: PWO-5
Project Name: Chlorination Upgrades
System Type: Potable Water

Project Description:

This project includes chlorination upgrades at the Desalination plant and at the Kings tanks. All finished water from the existing desalination facility is sent to the Kings Tank therefore, the chlorination upgrades will be added at the Kings Tank Site.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Chlorination Upgrades for Desalination Plant	--	--	New	--		$ 450,000	$ 585,000	$ 746,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 746,000
Future Users	0%	$ -
Total	100%	$ 746,000

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:

Project Number: PWO-6
Project Name: Desalination Plant Relocation
System Type: Potable Water

Project Description:

This project includes the relocation of the Desalination plant. This work includes relocating the desalination facility outside of the flood plain and completely replacing the existing treatment system with a brand new treatment facility.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Desalination Plant Relocation	--	--	New	--		--	--	--	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

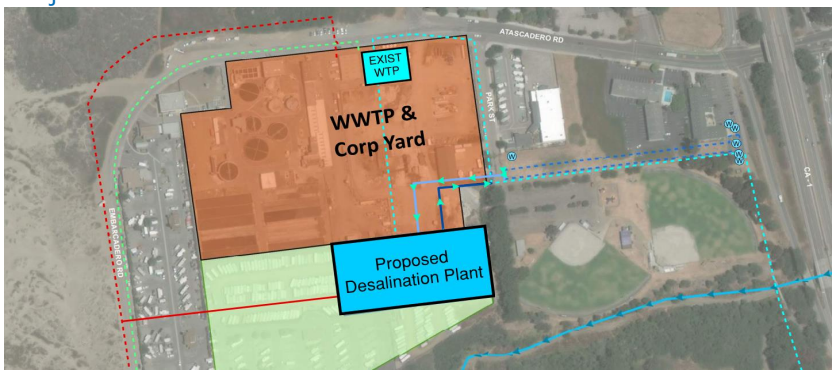
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ -
Future Users	0%	$ -
Total	100%	$ -

Notes on Cost Estimation:

This project is an existing improvement therefore 100% of cost are assigned to existing users.

Project Detail:



Appendix L

WASTEWATER COLLECTION SYSTEM IMPROVEMENTS

Project Number: WWGM-1
Project Name: Gravity Main along Atascadero Road
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,000 feet of 18-inch diameter pipeline along Atascadero Road, between Main Street and Park Street. The flow levels of the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 27-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	27	Replace	1,000	$ 320	$ 320,000	$ 416,000	$ 530,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	87%	$ 463,000
Future Users	13%	$ 67,000
Total	100%	$ 530,000

Notes on Cost Estimation:

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

Project Detail:

Project Number: WWGM-2
Project Name: Gravity Main along Main Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 2,900 feet of 15-inch diameter pipeline along Main Street, between Atascadero Road and Las Vegas Street. The flow levels within the gravity sewer cause existing manholes to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	15	24	Replace	2,900	$ 285	$ 827,000	$ 1,075,000	$ 1,371,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	87%	$ 1,194,000
Future Users	13%	$ 177,000
Total	100%	$ 1,371,000

Notes on Cost Estimation:

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

Project Detail:

Project Number: WWGM-3
Project Name: Gravity Main along Main Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,600 feet of 12-inch diameter pipeline along Main Street, between Las Vegas Street and San Joaquin Street. The flow levels within the gravity sewer cause upstream manholes to surcharge within 5 feet of the rim during PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with an 18-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	18	Replace	1,600	$ 205	$ 328,000	$ 426,000	$ 544,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 544,000
Future Users	0%	$ -
Total	100%	$ 544,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:

Project Number: WWGM-4A
Project Name: Gravity Main along San Joaquin Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of 6-inch diameter pipelines approximately 150 feet along San Joaquin Street, between Main Street and Alder Avenue. The flow levels within the gravity sewer cause upstream manholes to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 12-inch diameter pipelines.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	12	Replace	150	$ 175	$ 26,000	$ 34,000	$ 43,000	2023

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

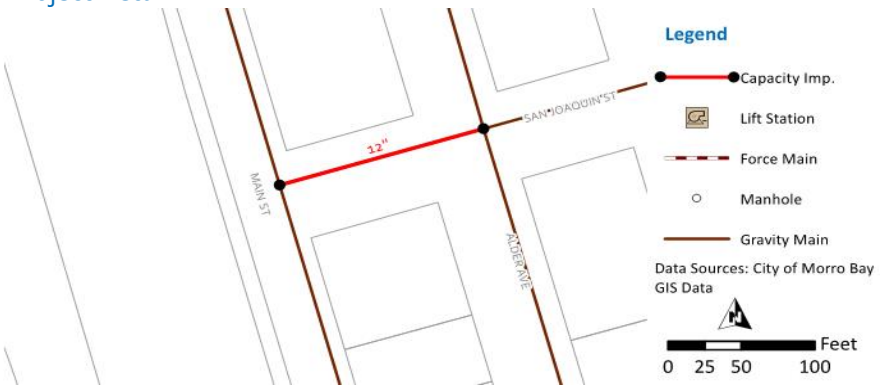
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	95%	$ 41,000
Future Users	5%	$ 2,000
Total	100%	$ 43,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: WWGM-4B
Project Name: Gravity Main along Alder Avenue, and San Jacinto Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of 6-inch diameter pipelines approximately 650 feet along Alder Avenue, between San Joaquin Street and San Jacinto Street and 550 feet along San Jacinto Street, between Alder Avenue and Elm Avenue. The flow levels within the gravity sewer cause the upstream manholes to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 10-inch diameter pipelines.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	10	Replace	1,200	$ 165	$ 198,000	$ 257,000	$ 328,000	2024

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	92%	$ 301,000
Future Users	8%	$ 27,000
Total	100%	$ 328,000

Notes on Cost Estimation:

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

Project Detail:

Project Number: WWGM-5
Project Name: Gravity Main along Greenwood Avenue
System Type: Wastewater Collection System

Project Description:

This project includes the replacement approximately 1,850 feet of 6-inch diameter pipeline along Greenwood Avenue, between Avalon Street and Elena Street. The flow levels within the gravity sewers cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	12	Replace	1,850	$ 175	$ 324,000	$ 421,000	$ 537,000	2025

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 537,000
Future Users	0%	$ -
Total	100%	$ 537,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:

Project Number: WWGM-6
Project Name: Gravity Main along Coral Avenue
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 1,900 feet of 10-inch diameter pipeline along Coral Avenue, between Lift Station 1 and Azure Street. The flow levels in the gravity sewer cause upstream manholes to surcharge within 5 feet of the rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	10	12	Replace	1,900	$ 175	$ 333,000	$ 433,000	$ 552,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

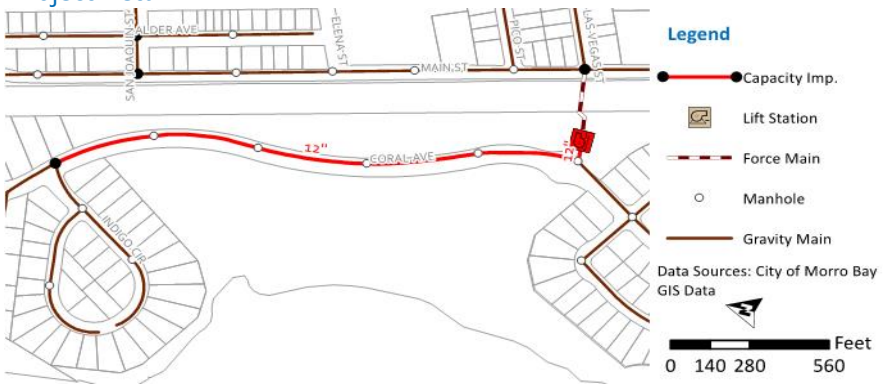
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 552,000
Future Users	0%	$ -
Total	100%	$ 552,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:



Project Number: WWGM-7
Project Name: Gravity Main along Sienna Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 250 feet of 6-inch diameter pipeline along Sienna Street, between Sandalwood Avenue and Beachcomber Drive. The flow levels within the gravity sewer cause upstream manholes to surcharge within 3 feet of the manhole rim under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	10	Replace	250	$ 165	$ 41,000	$ 53,000	$ 68,000	2026

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

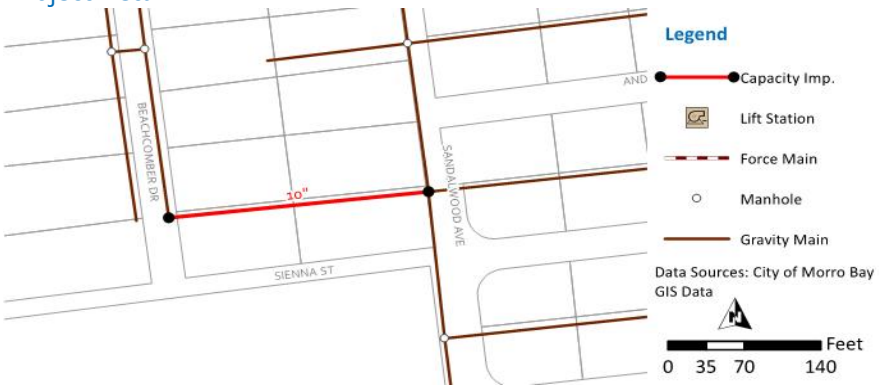
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 68,000
Future Users	0%	$ -
Total	100%	$ 68,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:



Project Number: WWGM-8A
Project Name: Gravity Main along Main Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 2,600 feet of 10-inch and 12-inch diameter pipeline along Main Street, between Radcliff Avenue and Atascadero Road. The flow levels within the gravity sewer cause upstream manholes to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 15-inch diameter pipelines.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	10 & 12	15	Replace	2,600	$ 190	$ 494,000	$ 642,000	$ 819,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

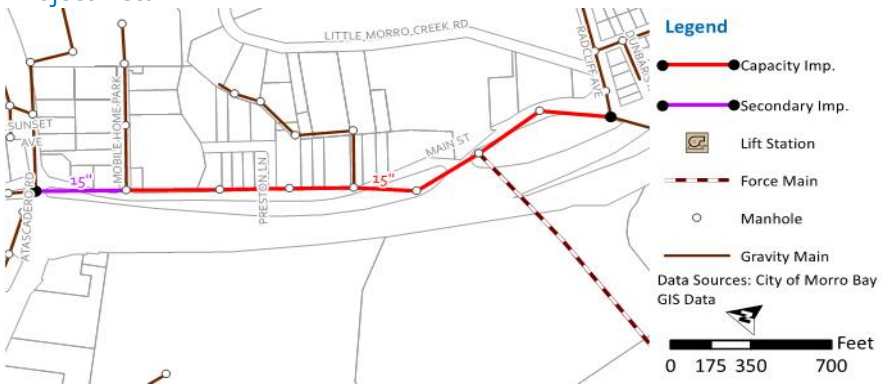
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	89%	$ 728,000
Future Users	11%	$ 91,000
Total	100%	$ 819,000

Notes on Cost Estimation:

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

Project Detail:



Project Number: WWGM-8B
Project Name: Gravity Main along Main Street
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of approximately 400 feet of 8-inch and 10-inch diameter pipeline along Main Street south of Radcliff Avenue. The flow levels within the gravity sewer cause existing manholes to overflow under PWWF conditions. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with 12-inch diameter pipelines.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	8 & 10	12	Replace	400	$ 175	$ 70,000	$ 91,000	$ 116,000	2022

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

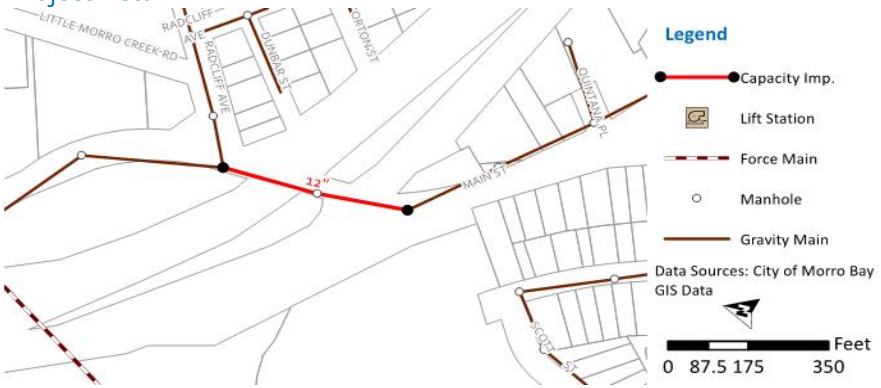
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 116,000
Future Users	0%	$ -
Total	100%	$ 116,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:



Project Number: WWGM-9
Project Name: Gravity Main under Highway 41
System Type: Wastewater Collection System

Project Description:

City Staff indicated that there is a belly in the existing 6-inch diameter main that crosses Highway 41, which has created ongoing maintenance issues. It is recommended that the existing 6-inch diameter pipe crossing under Highway 41 be replaced with a new 8-inch diameter main. This pipeline would be installed in a 20-inch diameter casing.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main and Interstate Crossing	6	8/20	Replace	110	$ 470	$ 52,000	$ 68,000	$ 86,000	2021

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

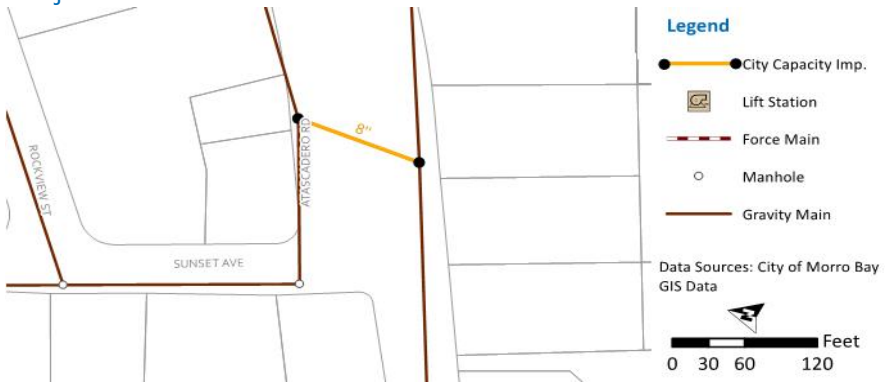
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 86,000
Future Users	0%	$ -
Total	100%	$ 86,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:



Project Number: WWLS-1
Project Name: Lift Station 1 Replacement
System Type: Wastewater Collection System

Project Description:

This project includes the replacement of the existing lift station. The existing influent flow exceeds the existing firm pumping capacity under PWWF conditions. To mitigate the risk of a SSO occurring during PWWF conditions, it is recommended that the new lift station have a firm pumping capacity of 1.10 mgd and a total capacity of 1.65 mgd.

Project Details:

Project Element	Existing Total Capacity (mgd)	Proposed Total Capacity (mgd)	Replace/ New	No. of Pumps (Units)	Unit Cost ($)	Baseline Construction Cost[1] ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Lift Station	0.98	1.65	Replace	3	--	$ 868,000	$ 1,128,000	$ 1,439,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

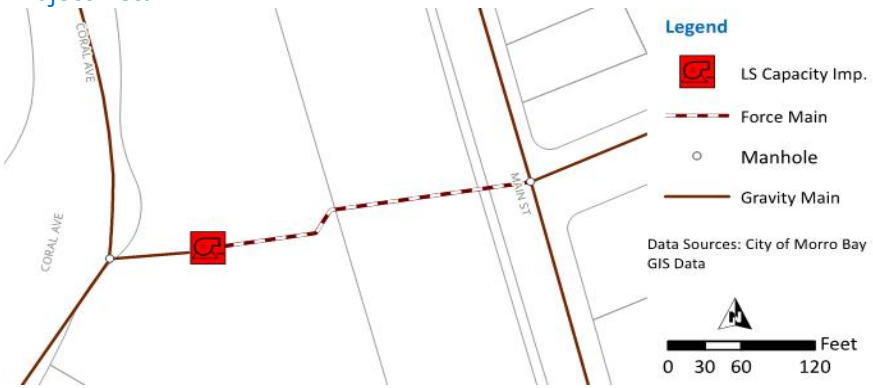
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 1,439,000
Future Users	0%	$ -
Total	100%	$ 1,439,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost. Costs based on lift station cost curve.

Project Detail:



Project Number: WWFM-1
Project Name: Lift Station 1 Force Main
System Type: Wastewater Collection System

Project Description:

The purpose of this project is to mitigate the high velocity (> 8 fps) that the existing force main experiences following WWLS-1 under existing conditions. It is recommended that an 8-inch diameter force main be constructed parallel to the existing 6-inch diameter force main.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Force Main	6	8	New	300	$ 160	$ 48,000	$ 62,000	$ 80,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

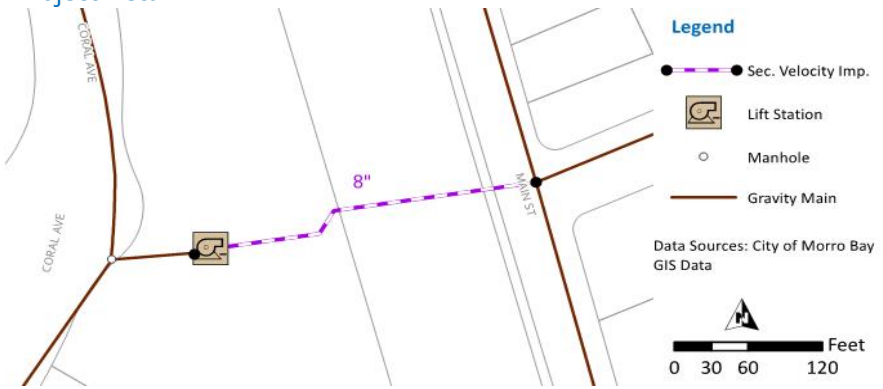
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 80,000
Future Users	0%	$ -
Total	100%	$ 80,000

Notes on Cost Estimation:

As an existing deficiency, current users are assigned 100-percent of the project's cost.

Project Detail:



Project Number: WWRR-1A
Project Name: Gravity Main Beachcomber Drive
System Type: Wastewater Collection System

Project Description:

This project includes the rehabilitation or replacement of approximately 570 feet of 12-inch diameter pipeline along Beachcomber Drive between Java Street and north of Mindoro Street.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	12	Replace	570	$ 175	$ 100,000	$ 130,000	$ 166,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

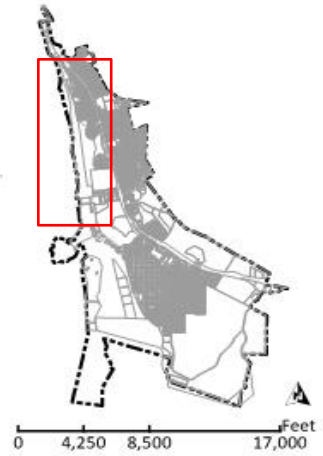
Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 166,000
Future Users	0%	$ -
Total	100%	$ 166,000

Notes on Cost Estimation:

As an R&R project, current users are assigned 100-percent of the project's cost.

Project Detail:

Project Number: WWRR-1B
Project Name: Gravity Main Beachcomber Drive
System Type: Wastewater Collection System



Project Description:

This project includes the rehabilitation or replacement of approximately 960 feet of 10-inch diameter pipeline along Beachcomber Drive between north of Mindoro Street and north of Sicily Street.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	10	10	Replace	960	$ 165	$ 158,000	$ 205,000	$ 262,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

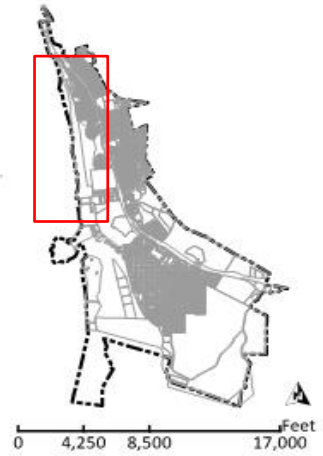
Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 262,000
Future Users	0%	$ -
Total	100%	$ 262,000

Notes on Cost Estimation:

As an R&R project, current users are assigned 100-percent of the project's cost.

Project Detail:

Project Number: WWRR-1C
Project Name: Gravity Main Beachcomber Drive
System Type: Wastewater Collection System



Project Description:

This project includes the rehabilitation or replacement of approximately 950 feet of 8-inch diameter pipeline along Beachcomber Drive between north of Sicily Lane and Toro Lane.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	8	8	Replace	950	$ 160	$ 152,000	$ 198,000	$ 252,000	2019

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 252,000
Future Users	0%	$ -
Total	100%	$ 252,000

Notes on Cost Estimation:

As an R&R project, current users are assigned 100-percent of the project's cost.

Project Detail:

Project Number: WWRR-2
Project Name: Pipe R&R Program Upstream of LS-1
System Type: Wastewater Collection System

Project Description:

This project includes the rehabilitation or replacement of pipelines that are upstream of Lift Station 1 not already a recommended improvement project. This equates to 21,400 feet of 6-inch, 2,000 feet of 8-inch, 900 feet of 10-inch and 2,600 feet of 12-inch diameter pipeline R&R projects between 2020 through 2028. The City should use existing inspection data to prioritize pipelines and choose the best rehabilitation or replacement method. It is recommended that pipelines less than 8-inches be upsized to a minimum of 8-inches in diameter.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Annual Length (ft/yr)	Unit Cost[1] ($/ft)	Baseline Construction Cost[1] ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2020
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2021
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2022
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2023
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2024
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2025
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2026
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2027
Gravity Main	6-12	8-12	R&R	3,000	Varies	$ 363,000	$ 472,000	$ 602,000	2028

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 5,418,000
Future Users	0%	$ -
Total	100%	$ 5,418,000

Notes on Cost Estimation:

As an R&R project, current users are assigned 100-percent of the project's cost.

Project Number: WWRR-3
Project Name: I/I Reduction Projects
System Type: Wastewater Collection System

Project Description:

This project includes the rehabilitation or replacement of pipelines and structures that contribute significant amount of I/I. These projects will be identified during the I/I Reduction Program's inspection phases. Inspection data will be used to prioritize projects and determine best rehabilitation method or replacement. This Master Plan assumes an allowance of $100,000 a year on I/I reduction related projects. The annual budget may change as the I/I reduction program is implemented. It is recommended that pipelines less than 8-inches be upsized to a minimum of 8-inches in diameter.

Project Details:

Project Element	Existing Diameter	Proposed Diameter	Replace/ New	Length (ft)	Annual Cost ($/yr)	Capital Improvement Cost[3]	Project Schedule
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2019
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2020
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2021
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2022
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2023
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2024
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2025
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2026
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2027
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 100,000	2028
I/I Reduction Projects	4-18	> 6	R&R	Varies	$ 100,000	$ 1,200,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

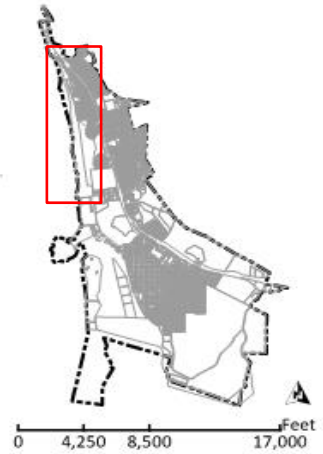
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 2,200,000
Future Users	0%	$ -
Total	100%	$ 2,200,000

Notes on Cost Estimation:

As an existing R&R project, current users are assigned 100-percent of the project's cost. This CIP line item was developed so the City can budget for R&R Projects that could come from the I/I Reduction Program's inspection phases.

Project Number: WWRR-4
Project Name: Cap Replacement Upstream of LS-1
System Type: Wastewater Collection System



Project Description:

This project consists of replacing caps and/or plugs with manholes upstream of Lift Station 1 due to high amounts of inflow entering the system through the existing end caps. This project includes the replacement of 31 caps with manholes between 2019 and 2020.

Project Details:

Project Element	Existing Structure	Proposed Structure	Replace/ New	No.	Unit Cost(1) ($/MH)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Manhole Replacement	Cap	Manhole	Replace	15	$ 10,000	$ 150,000	$ 195,000	$ 249,000	2019
Manhole Replacement	Cap	Manhole	Replace	16	$ 10,000	$ 160,000	$ 208,000	$ 265,000	2020

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 514,000
Future Users	0%	$ -
Total	100%	$ 514,000

Notes on Cost Estimation:

As an existing R&R project, current users are assigned 100-percent of the project's cost. Unit manhole costs developed using the City's bid tabs.

Project Number: WWRR-5
Project Name: Cap Replacement Program
System Type: Wastewater Collection System

Project Description:

This project consists of replacing the remaining cap and/or plug structures in the City's collection system with manholes. These projects do not include caps and plugs replaced upstream of Lift Station 1. This project consists of replacing 121 caps with manholes.

Project Details:

Project Element	Existing Structure	Proposed Structure	Replace/ New	No.	Unit Cost[1] ($/MH)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Manhole Replacement	Cap	Manhole	Replace	121	$ 10,000	$ 1,210,000	$ 1,573,000	$ 2,006,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 2,006,000
Future Users	0%	$ -
Total	100%	$ 2,006,000

Notes on Cost Estimation:

As an existing R&R project, current users are assigned 100-percent of the project's cost. Unit manhole costs developed using the City's bid tabs.

Project Number: WWO-1
Project Name: Sewer Master Plan Update
System Type: Wastewater Collection System

Project Description:

It is recommended that the City undergoes a Sewer Master Plan Update every 5-years to evaluate wastewater collection system.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2023
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2028
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2029-2040
Sewer Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

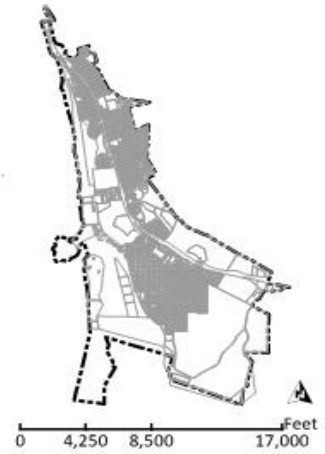
Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 600,000
Future Users	0%	$ -
Total	100%	$ 600,000

Notes on Cost Estimation:

As a Sewer Master Plan Update, current users are assigned 100-percent of the project's cost.

Project Number: WWO-2
Project Name: Flow Monitoring Program
System Type: Wastewater Collection System



Project Description:

It is recommended that the City conduct a flow monitoring program every 5-years to evaluate the I/I Reduction Programs performance and to aid with the Sewer Master Plan Updates (WWO-1). It is assumed that each program will consist of 15 flow meters that collects data for four weeks. Flow monitoring should be conducted during the wet season.

Project Details:

Project Element	Existing Diameter (in)	Duration (weeks)	Replace/ New	No.	Unit Cost[1] ($/meter-week)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Flow Monitoring Program	--	4 weeks	--	15	$ 1,000	--	--	$ 60,000	2023
Flow Monitoring Program	--	4 weeks	--	15	$ 1,000	--	--	$ 60,000	2028
Flow Monitoring Program	--	4 weeks	--	15	$ 1,000	--	--	$ 60,000	2029-2040
Flow Monitoring Program	--	4 weeks	--	15	$ 1,000	--	--	$ 60,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Cost Allocation:

Reimbursement Category	Percent	Cost ($)
Existing Users	100%	$ 240,000
Future Users	0%	$ -
Total	100%	$ 240,000

Notes on Cost Estimation:

As an existing inspection program, current users are assigned 100-percent of the programs's cost.

Appendix M

STORM DRAINAGE IMPROVEMENTS

Project Number: SD-01A
Project Name: Pipeline along Kern Street and Anchor Street
System Type: Storm Water Collection System

Project Description:

This project includes the addition of approximately 125 feet of 27-inch diameter pipeline along Kern Street, between Pecho Street and Anchor Street. The flow in this area flows to the northeast, and causes flooding in the 10-year storm conditions near the roundabout at Morro Bay Boulevard and Quintana Road. To mitigate flooding it is recommended this 27-inch pipeline is added to convey the flow west.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	27	New	125	$ 320	$ 40,000	$ 52,000	$ 66,000	2019

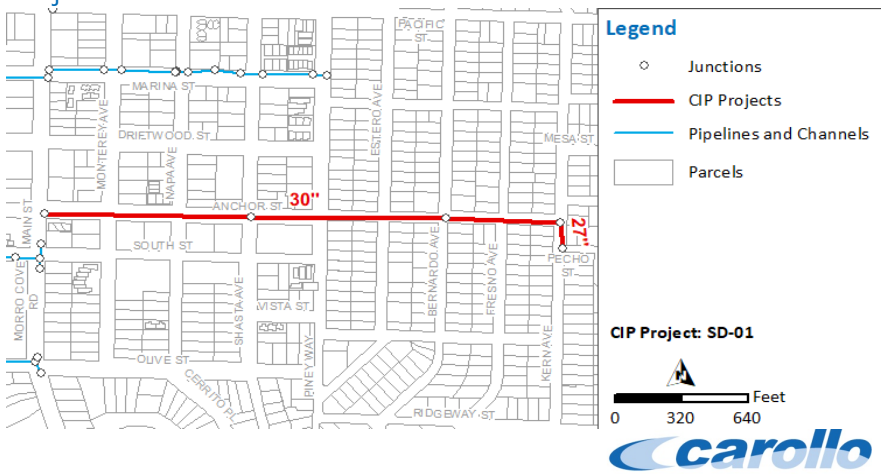
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-01B
Project Name: Pipeline along Kern Street and Anchor Street
System Type: Storm Water Collection System

Project Description:

This project includes the addition of approximately 2,600 feet of 30-inch diameter pipeline along Anchor Street, between Kern Avenue and Main Street. The flow in this area flows to the northeast, and causes flooding in the 10-year and 100-year storm conditions near the roundabout at Morro Bay Boulevard and Quintana Road. To mitigate flooding it is recommended this 30-inch pipeline is added to convey the flow west.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	30	New	2,600	$ 355	$ 923,000	$ 1,200,000	$ 1,530,000	2019

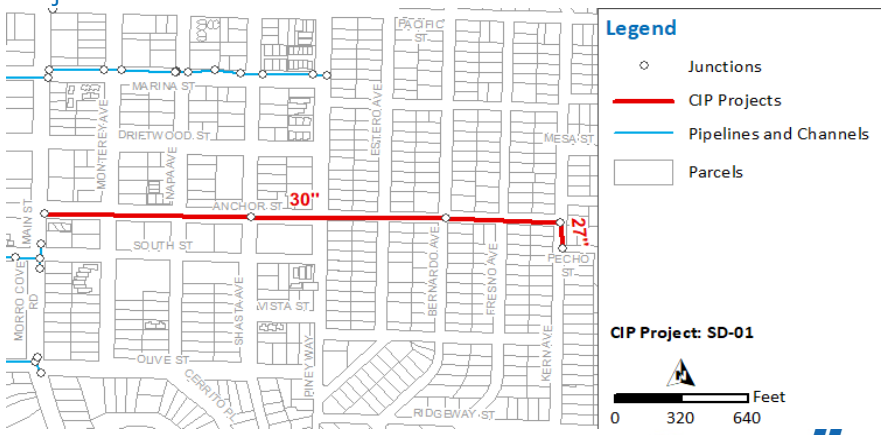
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-02A
Project Name: Pipeline along Marina Street
System Type: Storm Water Collection System

Project Description:

This project includes the addition of approximately 365 feet of 12-inch diameter pipeline along Marina Street, between Piney Way and Shasta Avenue. The flow in this area flows to the northeast, and causes flooding in the 10-year and 100-year storm conditions near the roundabout at Morro Bay Boulevard and Quintana Road. To mitigate flooding it is recommended this 12-inch pipeline is added to convey the flow west.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	12	New	365	$ 175	$ 64,000	$ 83,000	$ 106,000	2023

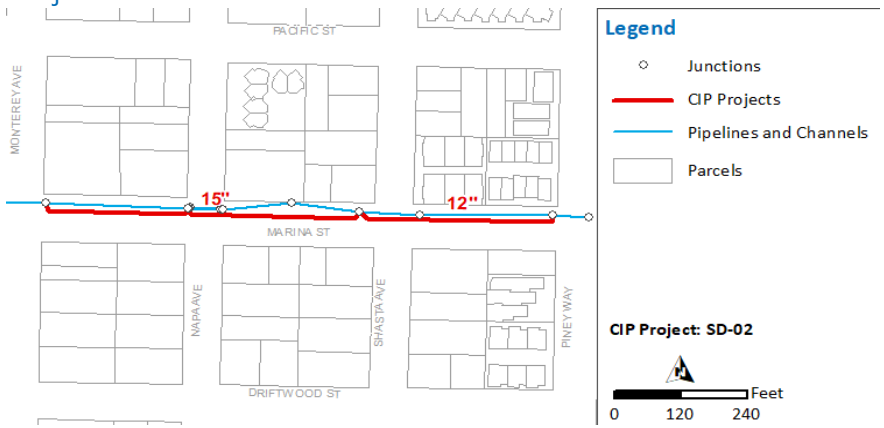
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-02B
Project Name: Pipeline along Marina Street
System Type: Storm Water Collection System

Project Description:

This project includes the addition of approximately 600 feet of 15-inch diameter pipeline along Marina Street, between Shasta Avenue and Monterey Avenue. The flow in this area flows to the northeast, and causes flooding in the 10-year and 100-year storm conditions near the roundabout at Morro Bay Boulevard and Quintana Road. To mitigate flooding it is recommended this 15-inch pipeline is added to convey the flow west.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	15	New	600	$ 190	$ 114,000	$ 148,000	$ 189,000	2023

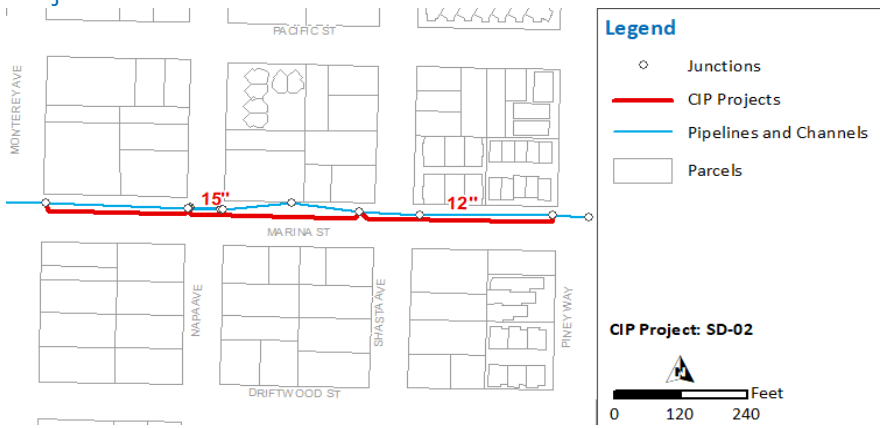
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-03
Project Name: Pipeline along Harbor Street
System Type: Storm Water Collection System

Project Description:

This project includes abandoning the existing 24-inch diameter pipeline along Main Street, and the addition of approximately 890 feet of 24-inch diameter pipeline along Harbor Street, between Main Street and Centennial Parkway. The flow, when directed from Harbor Street to Main Street, causes flooding in the 10-year storm conditions. To mitigate flooding it is recommended this 24-inch diameter line is added to convey water along Harbor Street.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	24	New	890	$ 285	$ 254,000	$ 330,000	$ 421,000	2023

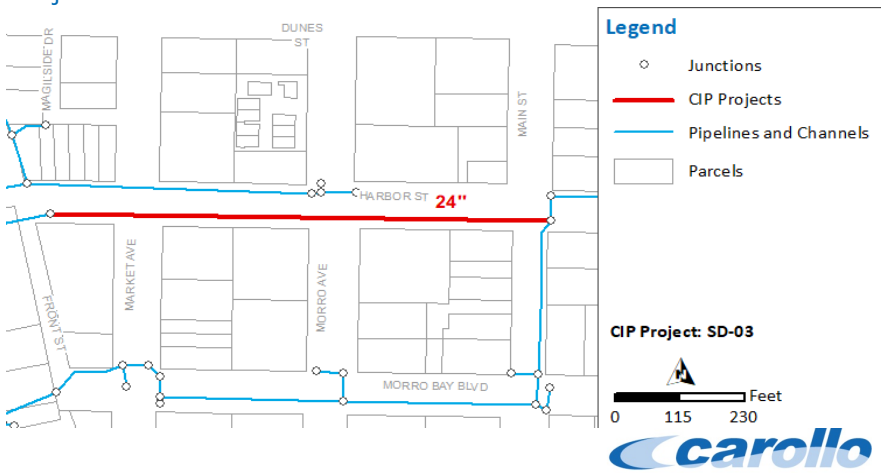
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-04
Project Name: Pipeline at Morro Bay Boulevard and Quintana Road
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 130 feet of 12-inch diameter pipelines along the north side of the roundabout at Morro Bay Boulevard and Quintana Road. The flow levels within the pipeline cause flooding under 10-year storm conditions. To mitigate the flooding, it is recommended that the existing pipeline be replaced with 18-inch diameter pipelines.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	18	Replace	130	$ 205	$ 27,000	$ 35,000	$ 45,000	2021

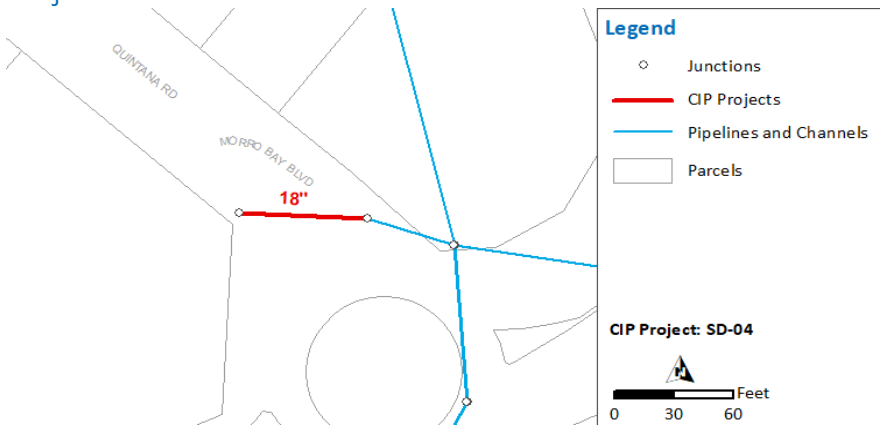
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-05
Project Name: Pipeline along Beach Street
System Type: Storm Water Collection System

Project Description:

This project includes the addition of approximately 1,000 feet of 24-inch diameter pipeline along Beach Street, between Morro Avenue and Embarcadero Road. In this area, the storm water flows through storm drains to the intersection at Morro Avenue and Beach Street, and then resurfaces and flows along the street. To reduce overland flow, it is recommended a 24-inch diameter pipeline be added.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost(1) ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost(2) ($)	Capital Improvement Cost(3) ($)	Project Schedule
Gravity Main	--	24	New	1,000	$ 285	$ 285,000	$ 371,000	$ 472,000	2021

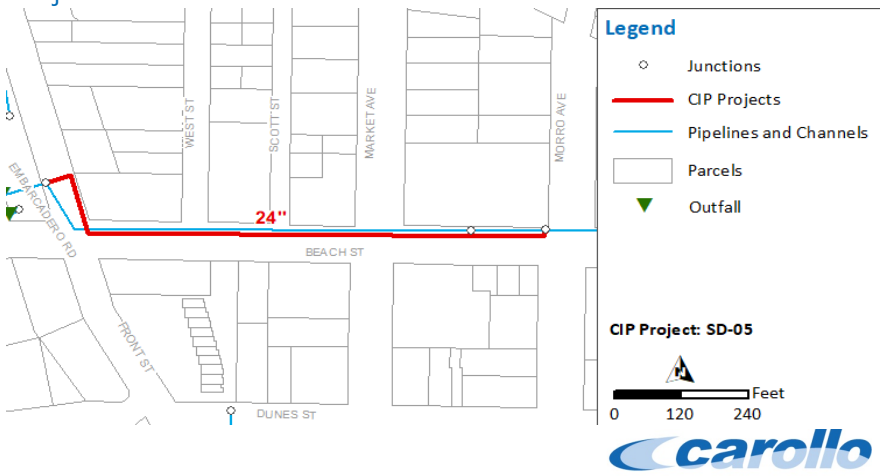
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-06A
Project Name: Pipeline along Cabrillo Highway
System Type: Storm Water Collection System

Project Description:

This project includes the replacement approximately 380 feet of open channel along Cabrillo Highway, west of Main Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing open channel be replaced with a triple barrel 48-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	48	Replace	1,140	$ 600	$ 684,000	$ 889,000	$ 1,134,000	2020

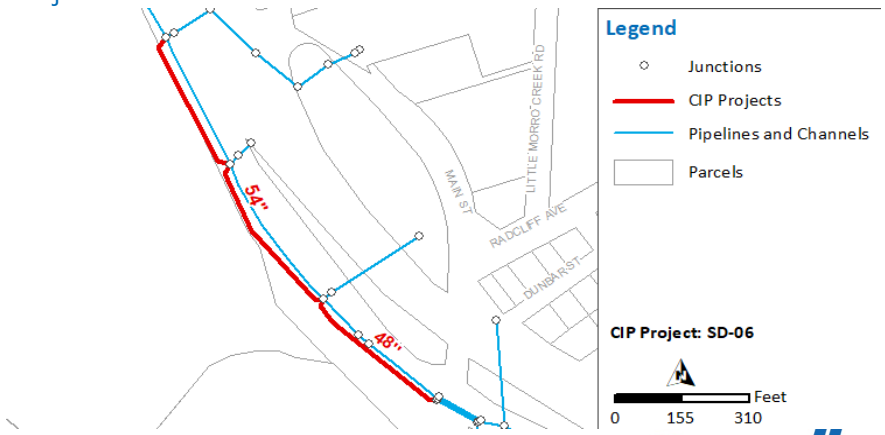
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-06B
Project Name: Pipeline along Cabrillo Highway
System Type: Storm Water Collection System

Project Description:

This project includes the replacement approximately 780 feet of open channel along Cabrillo Highway, west of Main Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing open channel be replaced with a triple barrel 54-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	54	Replace	2,340	$ 640	$ 1,498,000	$ 1,947,000	$ 2,483,000	2020

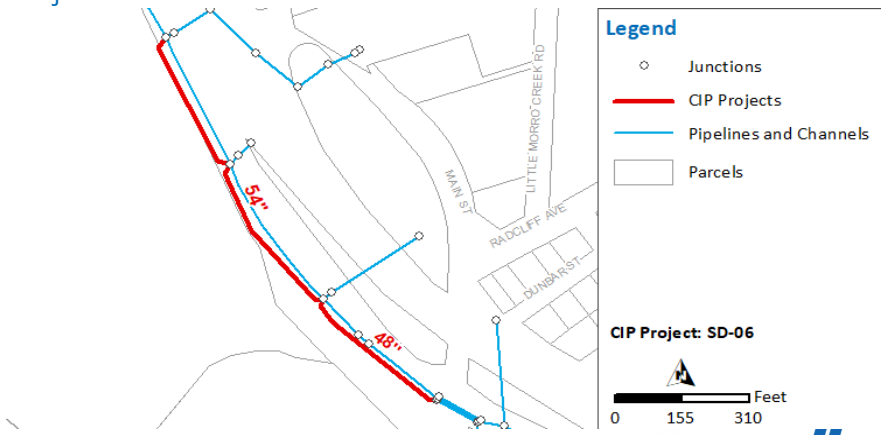
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-07A
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 360 feet of 12-inch diameter pipeline along Main Street, between Preston Lane and Azure Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	15	Replace	360	$ 190	$ 68,000	$ 88,000	$ 113,000	2022

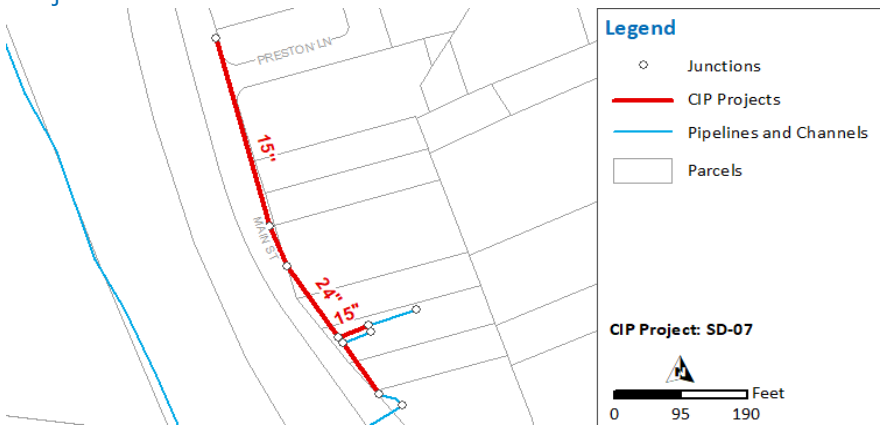
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-07B
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 230 feet of 12-inch diameter pipeline along Main Street, between Preston Lane and Azure Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	24	Replace	230	$ 285	$ 66,000	$ 86,000	$ 109,000	2022

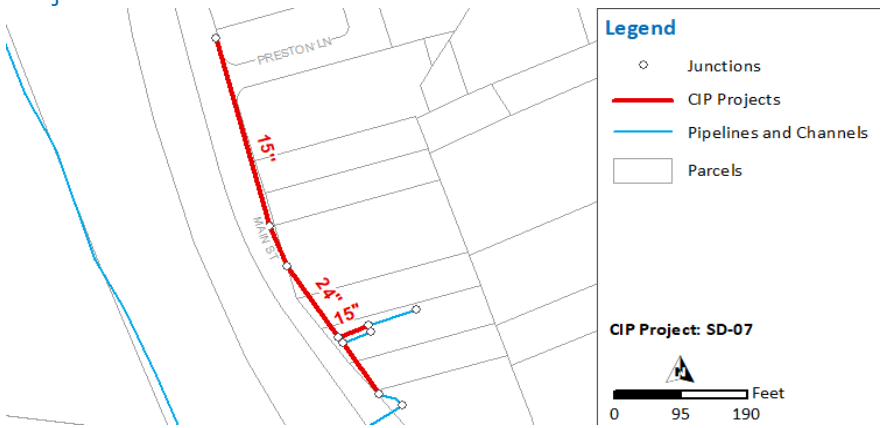
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-07C
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 50 feet of 12-inch diameter pipeline along Main Street, between Preston Lane and Azure Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	15	Replace	50	$ 190	$ 10,000	$ 13,000	$ 17,000	2022

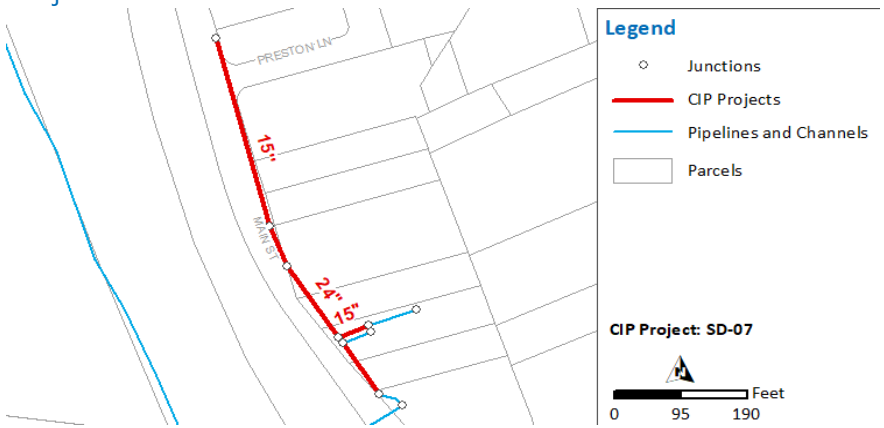
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-08
Project Name: Pipeline between Main Street and Cabrillo Highway
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 190 feet of 18-inch diameter pipeline between Main Street and Cabrillo Highway, northwest of the Main Street onramp. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 27-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	27	Replace	190	$ 320	$ 61,000	$ 79,000	$ 101,000	2024

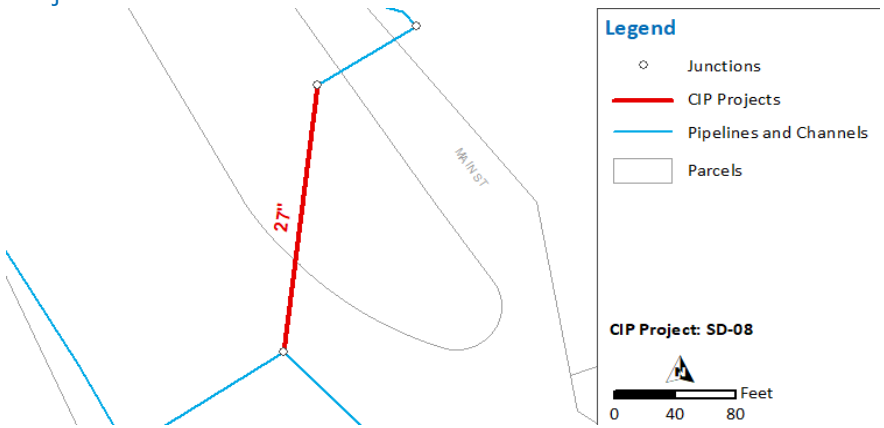
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-09
Project Name: Pipeline at Main Street and Atascadero Road
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 85 feet of 22-inch by 13-inch culvert at the intersection of Main Street and Atascadero Road. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing culvert be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	24	Replace	85	$ 285	$ 24,000	$ 31,000	$ 40,000	2025

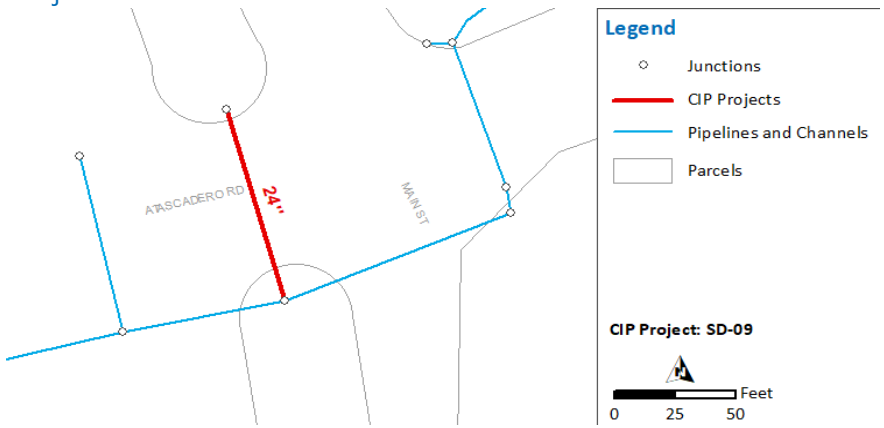
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-10
Project Name: Pipeline at Cabrillo Highway and Atascadero Road
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 80 feet of 22-inch by 13-inch culvert at the intersection of Main Street and Atascadero Road. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	24	Replace	80	$ 285	$ 23,000	$ 30,000	$ 38,000	2026

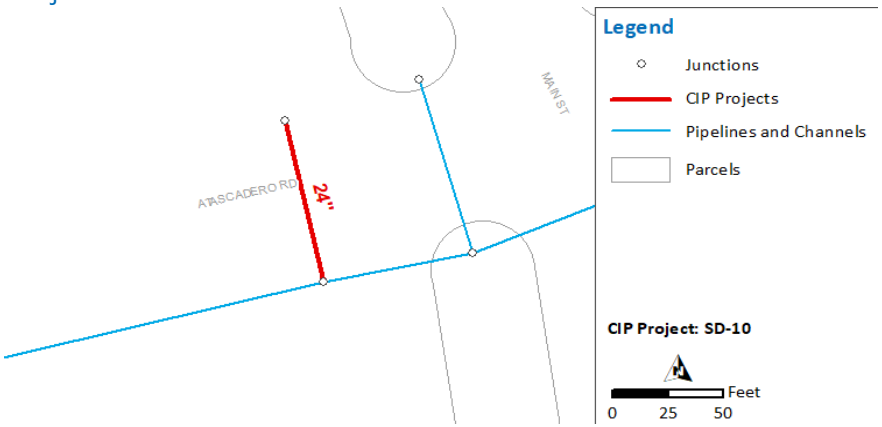
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-11
Project Name: Pipeline along Laurel Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 80 feet of 6-inch diameter pipeline along Laurel Street, south of Avalon Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	12	Replace	80	$ 175	$ 14,000	$ 18,000	$ 23,000	2022

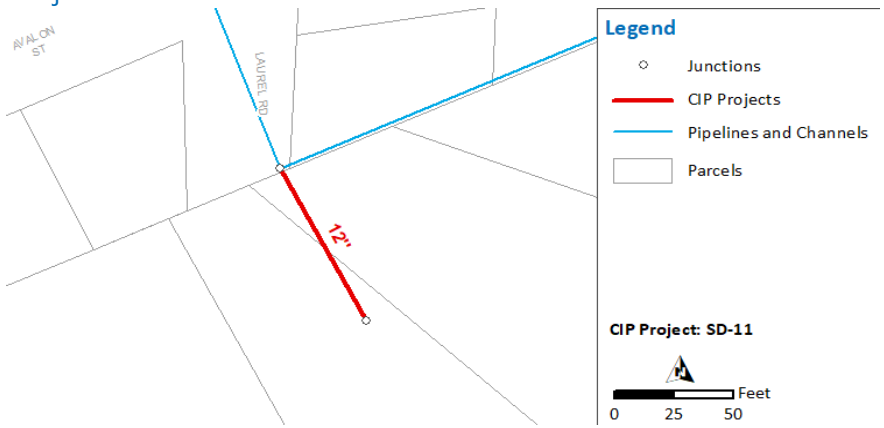
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-12
Project Name: Pipeline along Avalon Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 200 feet of 16-inch diameter pipeline along Avalon Street, between Ironwood Avenue and Hemlock Avenue. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	16	24	Replace	200	$ 285	$ 57,000	$ 74,000	$ 94,000	2022

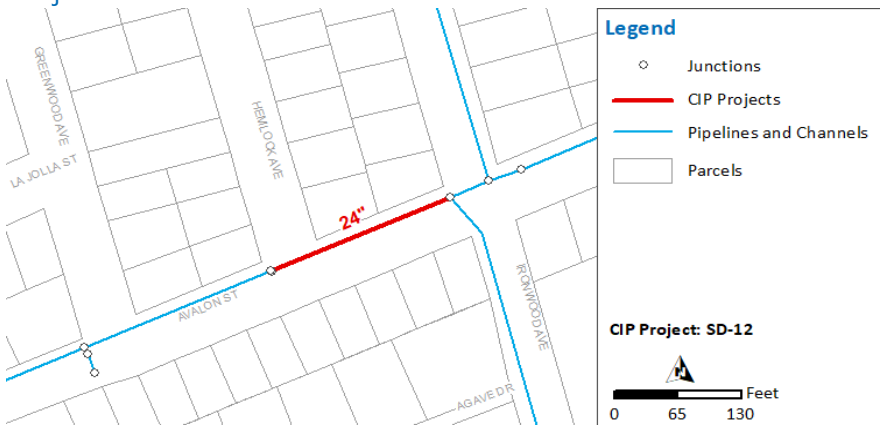
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-13
Project Name: Pipeline between Sunset Avenue and Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 260 feet of 20-inch diameter pipeline between Sunset Avenue and Main Street, starting at the corner of Hillview Street and Sunset Avenue. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 42-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	20	42	Replace	260	$ 550	$ 143,000	$ 186,000	$ 237,000	2027

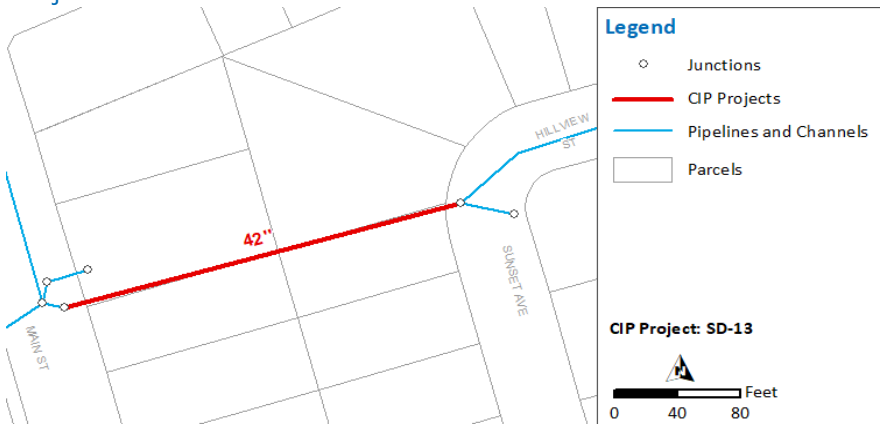
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-14
Project Name: Pipeline along Sienna Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 450 feet of 24-inch diameter pipeline along Sienna Street, between Sandalwood Avenue and west of Beachcomber Drive. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 36-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	36	Replace	450	$ 440	$ 198,000	$ 257,000	$ 328,000	2025

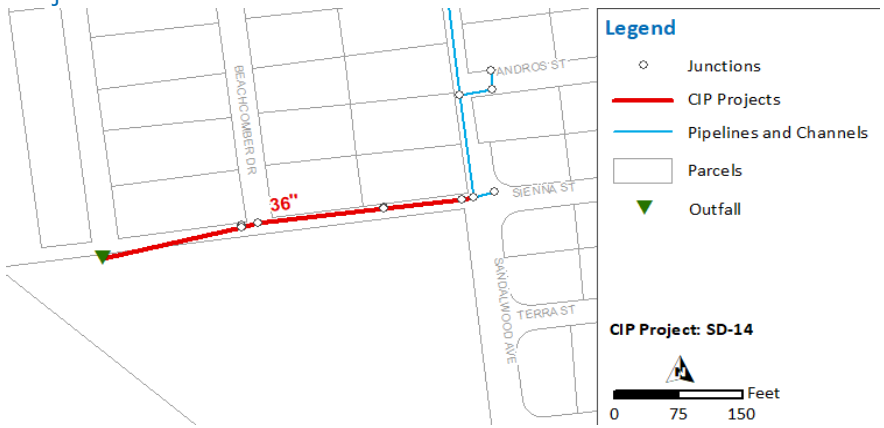
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-15
Project Name: Pipeline perpendicular to Oceanside Drive and Driftwood Avenue
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 240 feet of 16-inch by 42-inch culvert perpendicular to Driftwood Avenue and Oceanside Drive, near Orca Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 48-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	48	Replace	240	$ 600	$ 144,000	$ 187,000	$ 239,000	2026

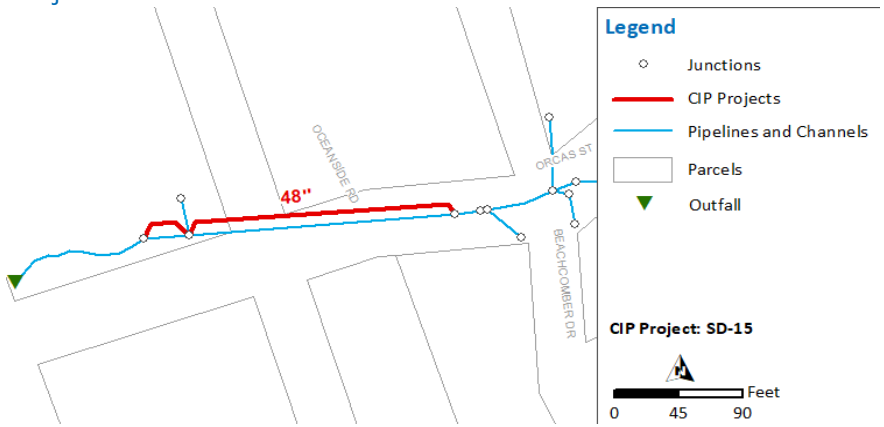
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-16
Project Name: Pipeline along Elena Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 450 feet of 24-inch diameter pipeline along Elena Street, between Cedar Avenue and Main Street. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 36-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	36	Replace	450	$ 440	$ 198,000	$ 257,000	$ 328,000	2022

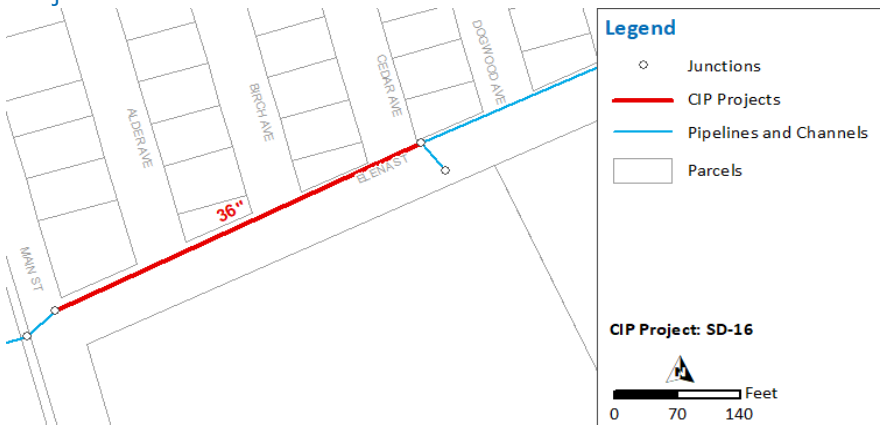
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-17
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 120 feet of 18-inch diameter pipeline along Main Street, just north of Sequoia Street. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	24	Replace	120	$ 285	$ 34,000	$ 44,000	$ 56,000	2028

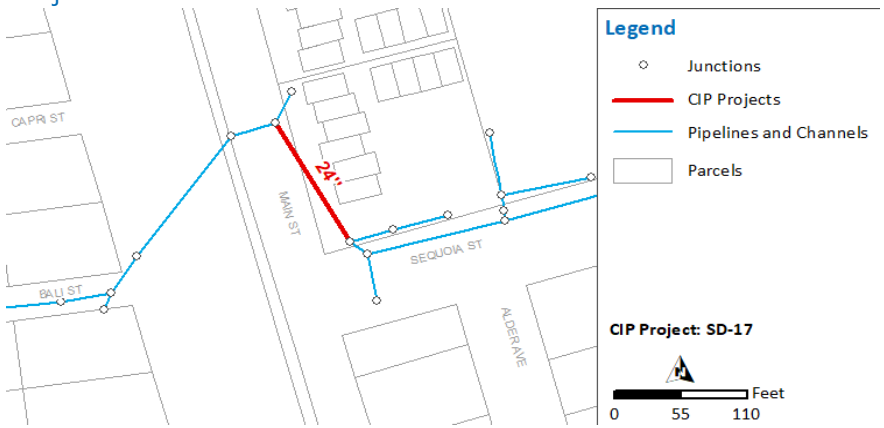
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-18A
Project Name: Pipeline along Sandalwood Avenue and Sienna Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 320 feet of 24-inch diameter pipeline along Sandalwood Avenue, between Bali Street and Sienna Street. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 30-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	30	Replace	320	$ 355	$ 114,000	$ 148,000	$ 189,000	2028

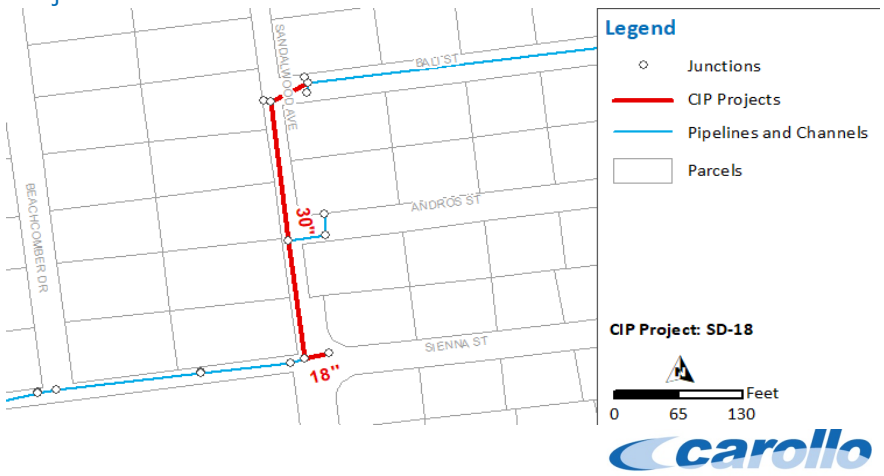
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-18B
Project Name: Pipeline along Sandalwood Avenue and Sienna Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 25 feet of 12-inch diameter pipeline along Sienna Street, east of Sandalwood Avenue. The flow caused flooding to occur under 10-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 18-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	18	Replace	25	$ 205	$ 5,000	$ 7,000	$ 8,000	2028

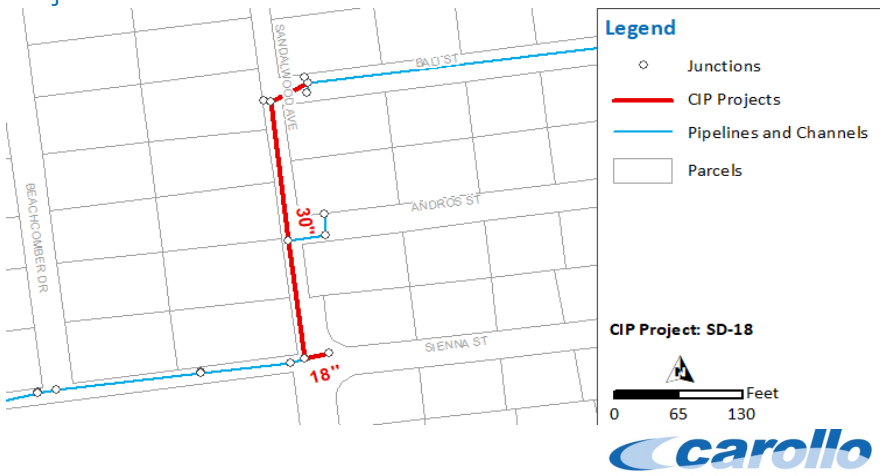
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-19
Project Name: Pipeline along Elena Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 500 feet of 24-inch diameter pipeline along Elena Street, between Greenwood Avenue and Cedar Avenue. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 30-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	30	Replace	500	$ 355	$ 178,000	$ 231,000	$ 295,000	2029-2040

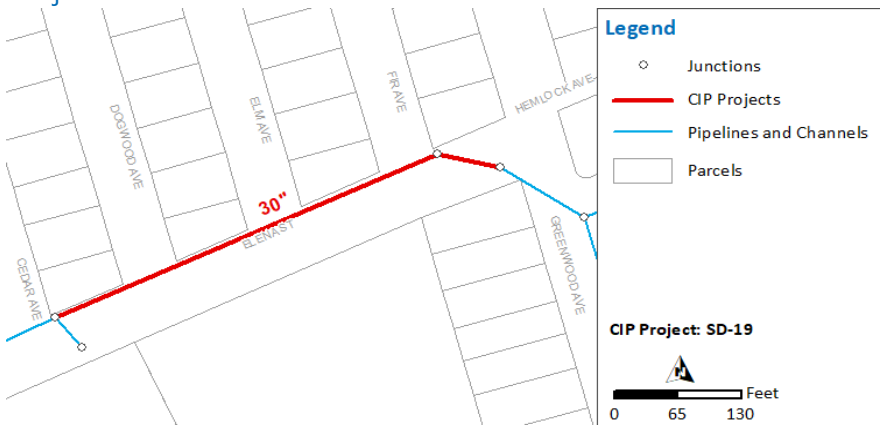
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-20
Project Name: Pipeline along Elena Street (under Cabrillo Highway)
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 175 feet of 24-inch diameter pipeline along Elena Street, from Main Street and under Cabrillo Highway. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 36-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	36	Replace	360	$ 1,000	$ 360,000	$ 468,000	$ 597,000	2029-2040

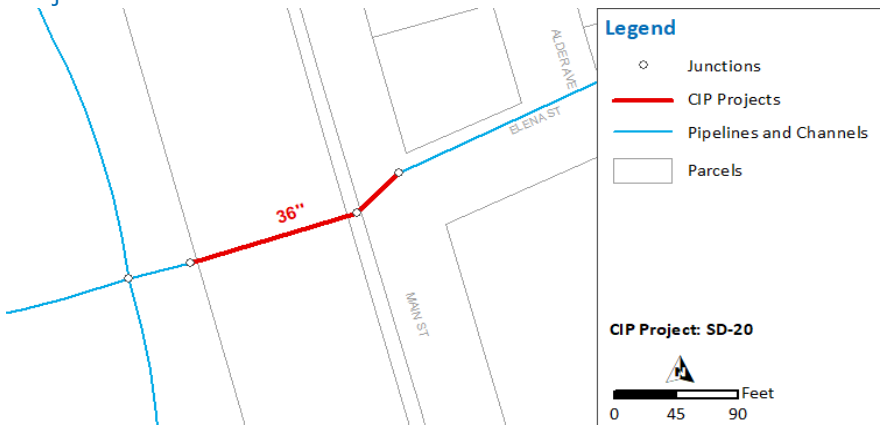
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-21
Project Name: Pipeline at Avalon Street and Ironwood Avenue
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 80 feet of 16-inch diameter pipeline at the intersection of Avalon Street and Ironwood Avenue. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	16	24	Replace	80	$ 285	$ 23,000	$ 30,000	$ 38,000	2029-2040

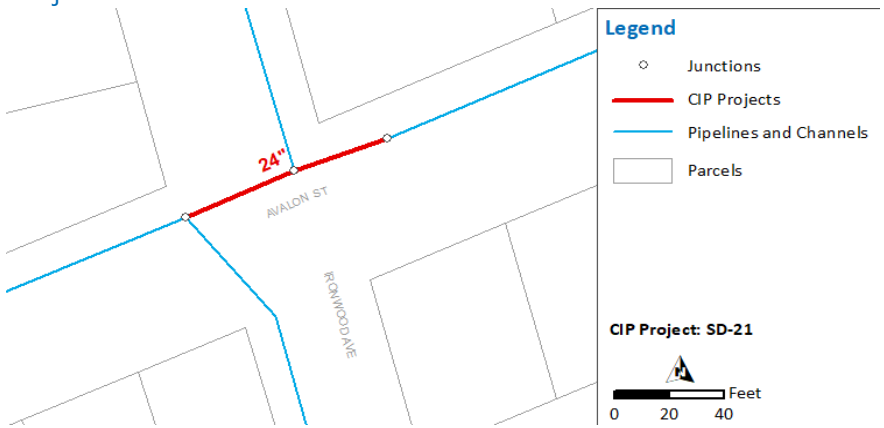
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-22
Project Name: Pipeline along Bayview Avenue
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 210 feet of 18-inch diameter pipeline along Bayview Avenue, from Hillview Street to the south. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	24	Replace	210	$ 285	$ 60,000	$ 78,000	$ 99,000	2029-2040

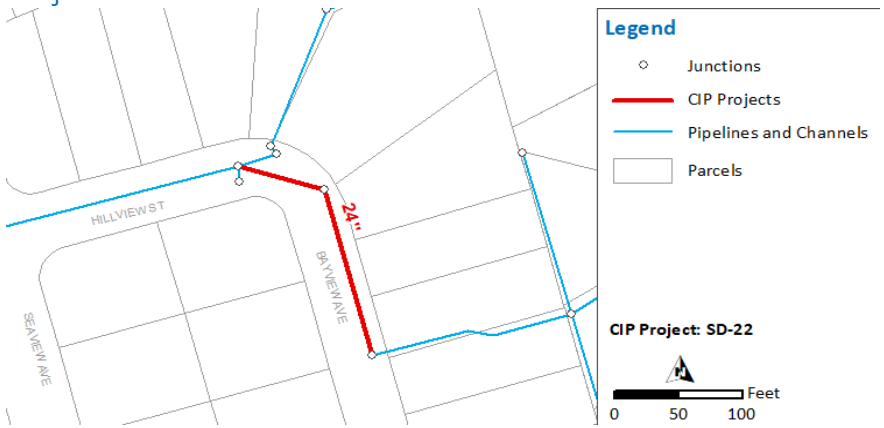
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-23
Project Name: Pipeline along Hillview Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 260 feet of 30-inch diameter pipeline along Hillview Street, between Seaview Avenue and Sunset Avenue. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 36-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	30	36	Replace	260	$ 440	$ 114,000	$ 148,000	$ 189,000	2029-2040

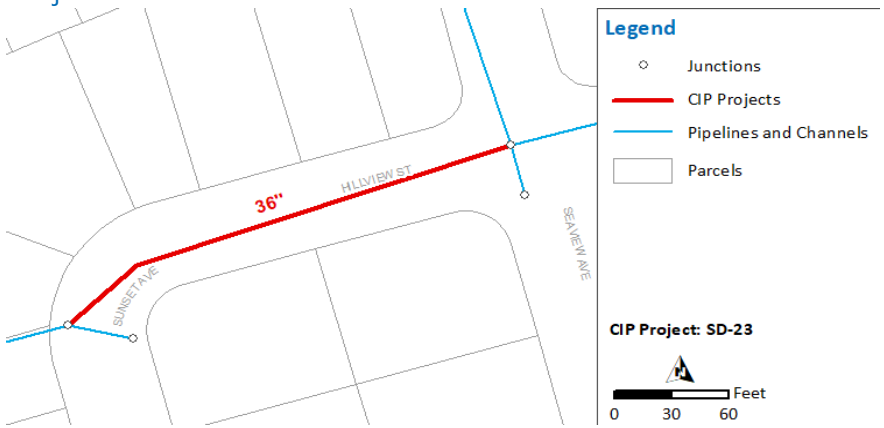
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-24
Project Name: Pipeline near Ponderosa Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 90 feet of 10-inch diameter pipeline near Ponderosa Street, parallel to Ironwood Avenue. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	10	15	Replace	90	$ 190	$ 17,000	$ 22,000	$ 28,000	2029-2040

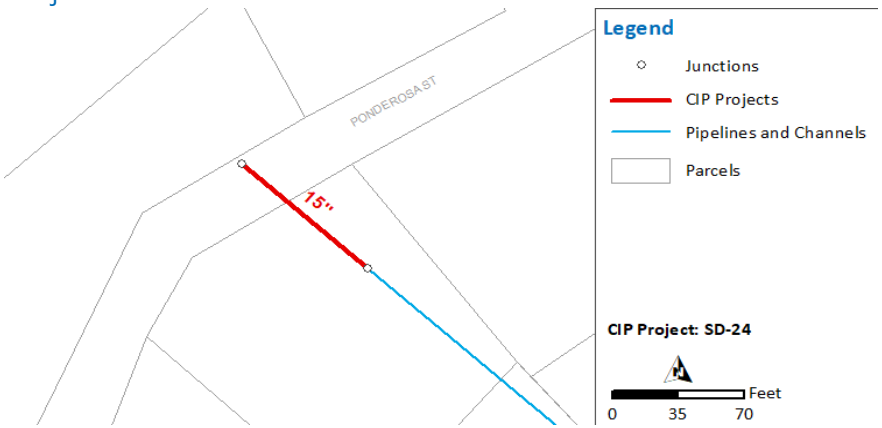
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-25
Project Name: Pipeline near Mimosa Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 105 feet of 12-inch diameter pipeline crossing Mimosa Street, north of the intersection of Hill Street and Mimosa Street. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	15	Replace	105	$ 190	$ 20,000	$ 26,000	$ 33,000	2029-2040

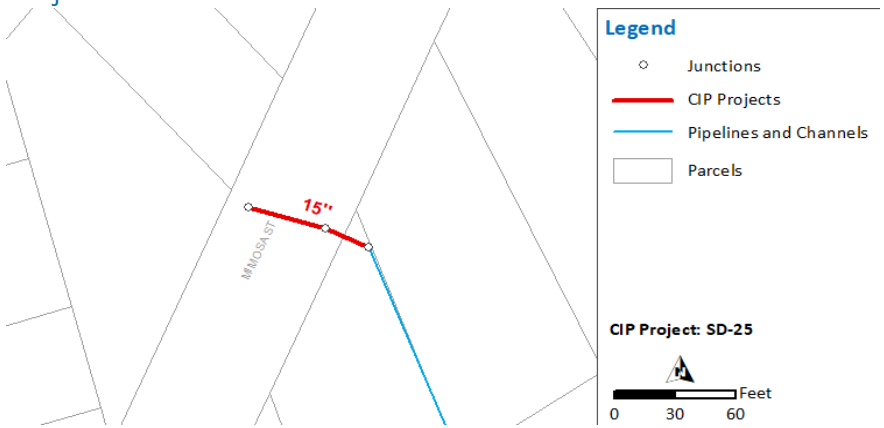
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-26A
Project Name: Pipeline near Miner's Ace Hardware and Morro Creek
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 90 feet of 12-inch diameter pipeline near Morro Creek, southeast of the Miner's Ace Hardware parking lot. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	15	Replace	90	$ 190	$ 17,000	$ 22,000	$ 28,000	2029-2040

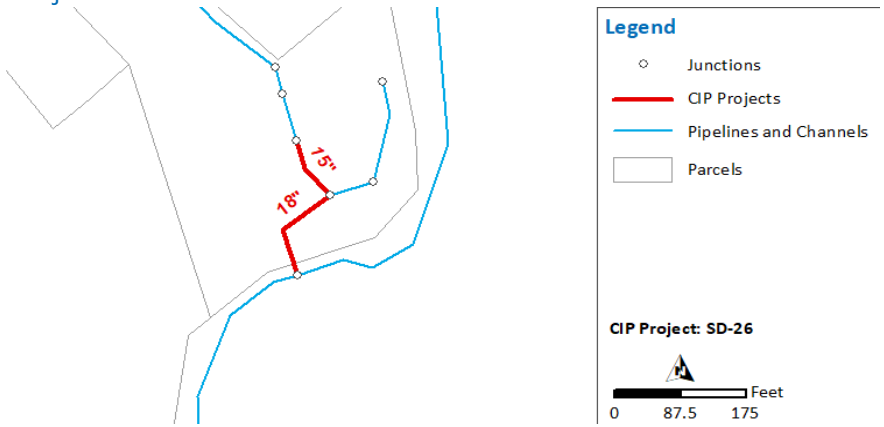
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-26B
Project Name: Pipeline near Miner's Ace Hardware and Morro Creek
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 150 feet of 12-inch diameter pipeline near Morro Creek, southeast of the Miner's Ace Hardware parking lot. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 18-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	18	Replace	150	$ 205	$ 31,000	$ 40,000	$ 51,000	2029-2040

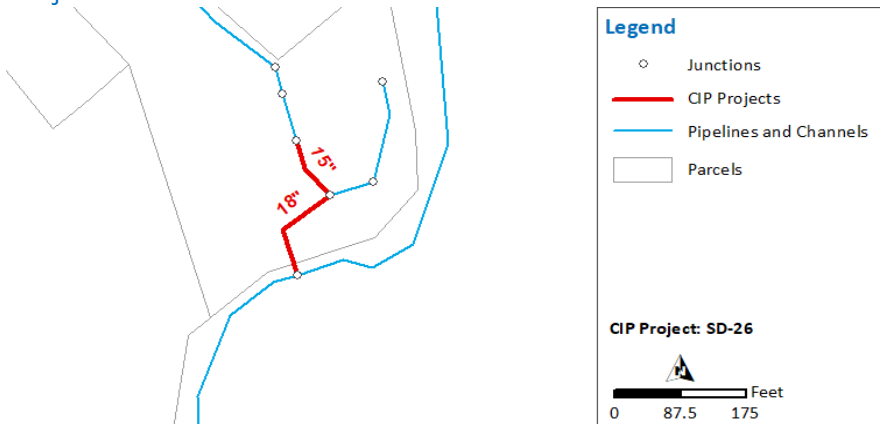
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-27
Project Name: Pipeline at Sunset Avenue and Atascadero Road
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 60 feet of 18-inch diameter pipeline along Atascadero Road, at the intersection of Sunset Avenue and Atascadero Road. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	24	Replace	60	$ 285	$ 17,000	$ 22,000	$ 28,000	2029-2040

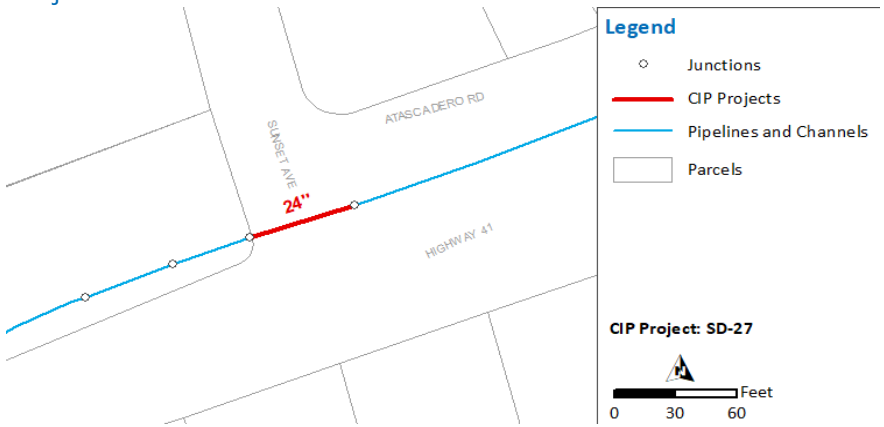
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-28
Project Name: Pipeline along Atascadero Road (underpass)
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 430 feet of 24-inch diameter pipeline along Atascadero Road, under the underpass near Main Street. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 36-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	36	Replace	430	$ 440	$ 189,000	$ 246,000	$ 313,000	2029-2040

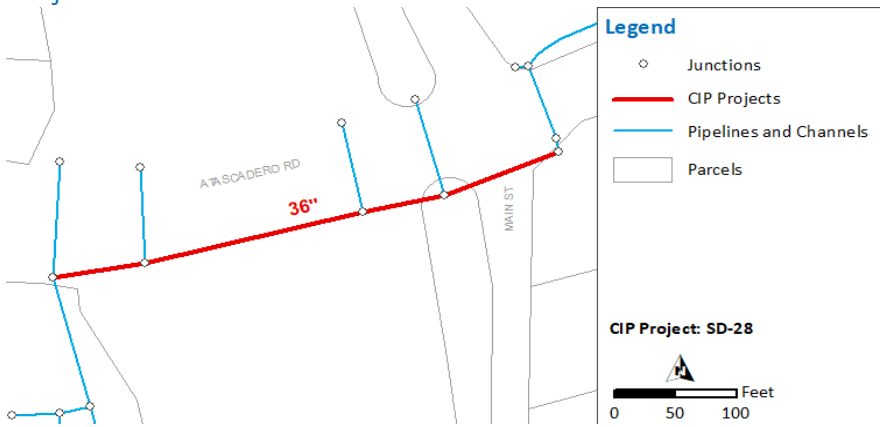
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-29
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 170 feet of 12-inch diameter pipeline along Main Street, between Errol Street and Morro Creek. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 18-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	18	Replace	170	$ 205	$ 35,000	$ 46,000	$ 58,000	2029-2040

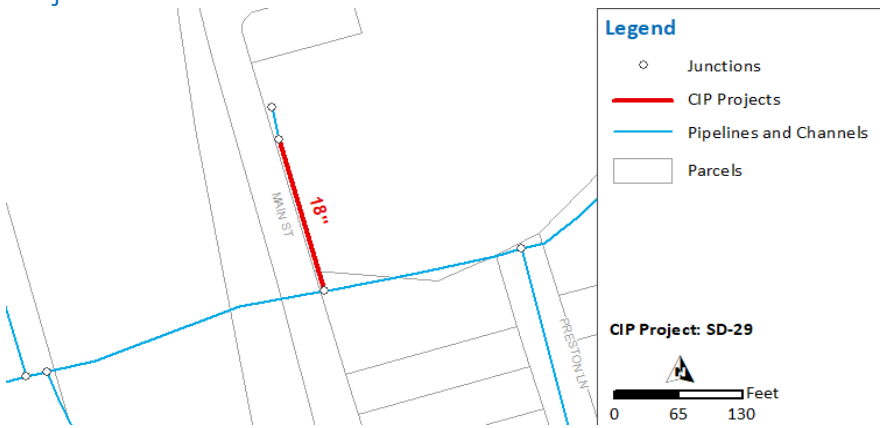
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-30A
Project Name: Pipeline along Preston Lane
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 40 feet of 6-inch diameter pipeline along Preston Lane. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	15	Replace	40	$ 190	$ 8,000	$ 10,000	$ 13,000	2029-2040

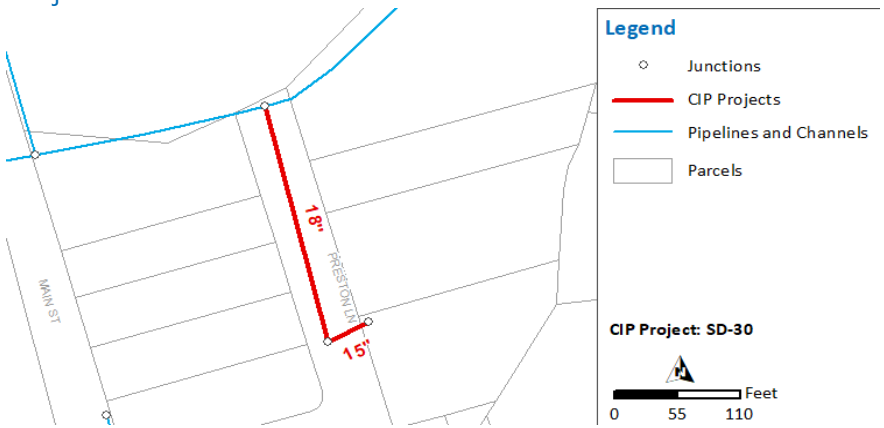
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-30B
Project Name: Pipeline along Preston Lane
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 220 feet of 6-inch diameter pipeline along Preston Lane. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with an 18-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	6	18	Replace	220	$ 205	$ 45,000	$ 59,000	$ 75,000	2029-2040

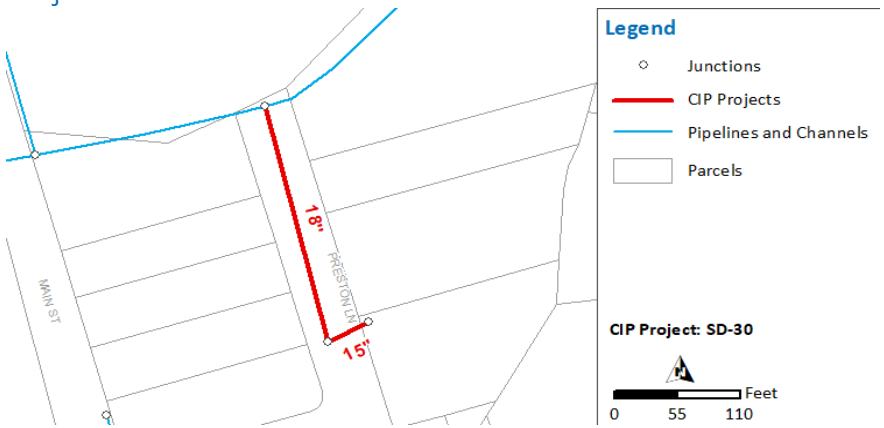
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-31
Project Name: Pipeline along Main Street (near Main Street on ramp)
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 80 feet of 18-inch diameter pipeline crossing Main Street, near the Main Street on ramp to Cabrillo Highway. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	24	Replace	80	$ 285	$ 23,000	$ 30,000	$ 38,000	2029-2040

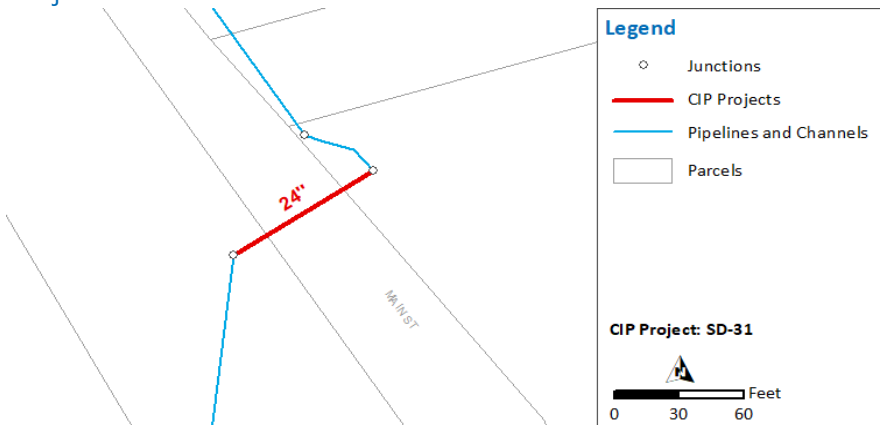
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-32A
Project Name: Pipeline near parking lot west of Surf Street and West Avenue
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 60 feet of 12-inch diameter pipeline in the parking lot located west of the intersection of Surf Street and West Avenue. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with an 18-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	12	18	Replace	60	$ 205	$ 12,000	$ 16,000	$ 20,000	2029-2040

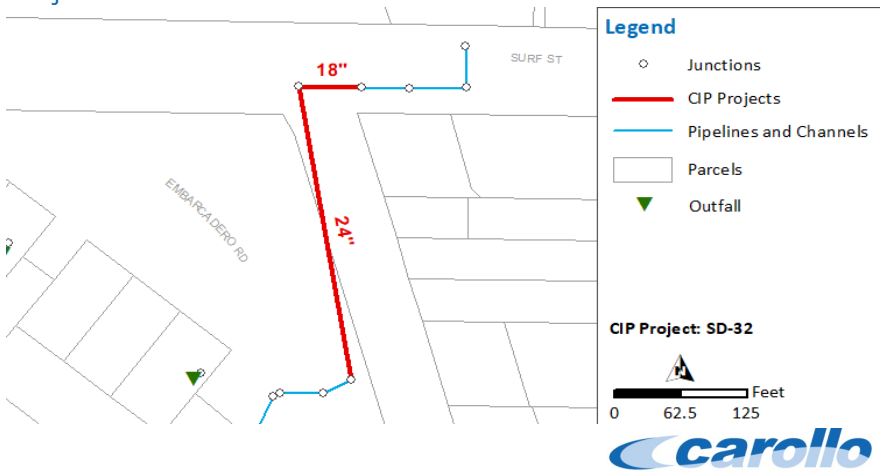
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-32B
Project Name: Pipeline near parking lot west of Surf Street and West Avenue
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 300 feet of 18-inch diameter pipeline in the parking lot located west of the intersection of Surf Street and West Avenue. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	18	24	Replace	300	$ 285	$ 86,000	$ 112,000	$ 143,000	2029-2040

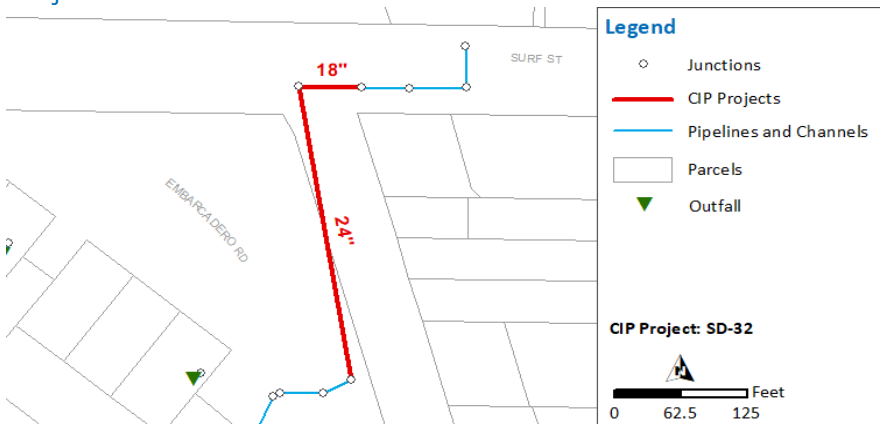
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-33
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 140 feet of 24-inch diameter pipeline located east of the roundabout at Morro Bay Boulevard and Quintana Road. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 30-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	30	Replace	140	$ 355	$ 50,000	$ 65,000	$ 83,000	2029-2040

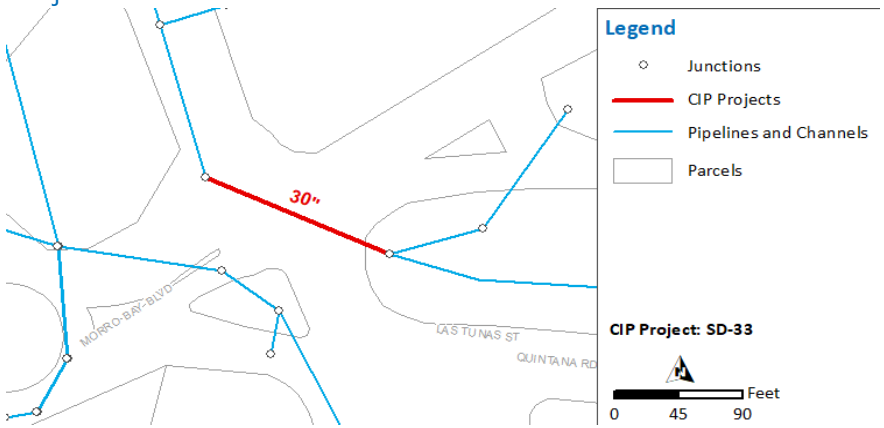
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-34
Project Name: Pipeline at Bonita Street and Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 100 feet of 24-inch diameter pipeline at the intersection of Bonita Street and Main Street. The flow caused flooding to occur under 100-year storm conditions. To mitigate the risk of flooding, it is recommended that the existing pipeline be replaced with a 30-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	30	Replace	100	$ 355	$ 36,000	$ 47,000	$ 60,000	2029-2040

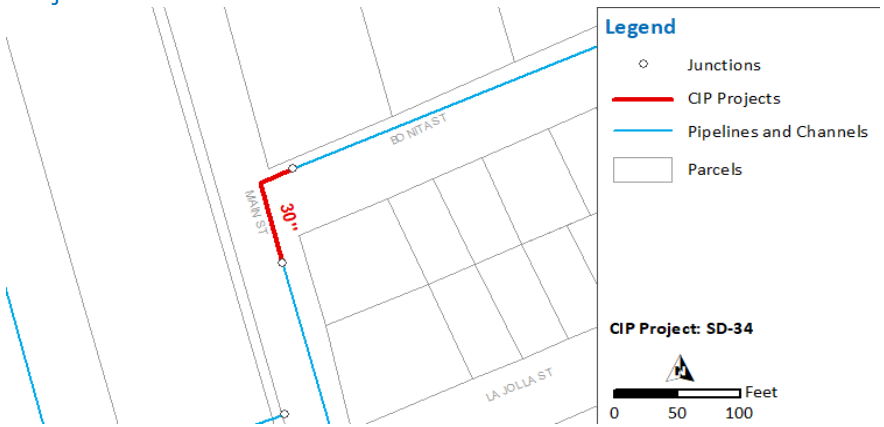
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 100-year design storm.

Project Number: SD-35
Project Name: Pipeline along Main Street
System Type: Storm Water Collection System

Project Description:

This project includes the addition of approximately 270 feet of 15-inch diameter pipeline along Main Street, between Quintana Place and Cabrillo Highway. In this area, the flow travels on the street and crosses over Main Street. To mitigate the risk of flooding, it is recommended that this 15-inch diameter pipeline be added.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	--	15	New	270	$ 190	$ 51,000	$ 66,000	$ 85,000	2027

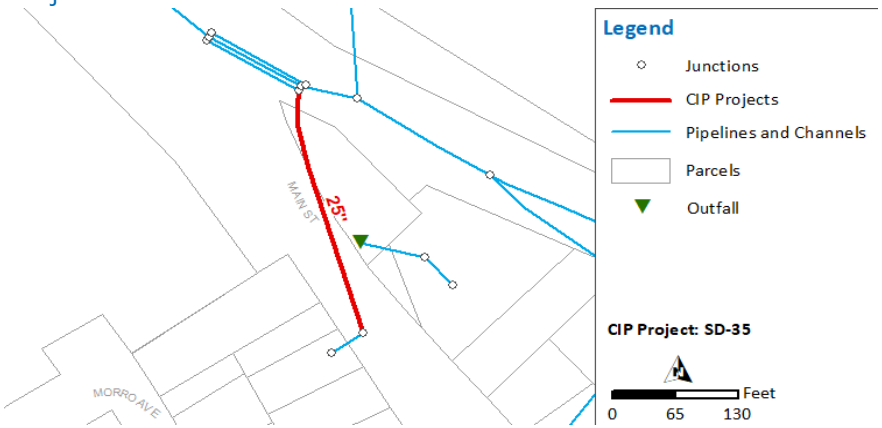
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SD-36
Project Name: Pipeline near Morro Bay Boulevard off ramp
System Type: Storm Water Collection System

Project Description:

This project includes the replacement of approximately 150 feet of 24-inch diameter pipeline near the Morro Bay Boulevard off ramp that goes under Cabrillo Highway. This pipe has failed, and currently has no bottom. It is recommended that the existing failed pipeline be replaced with a 24-inch diameter pipeline.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Gravity Main	24	24	Replace	150	$ 835	$ 125,000	$ 163,000	$ 207,000	2026

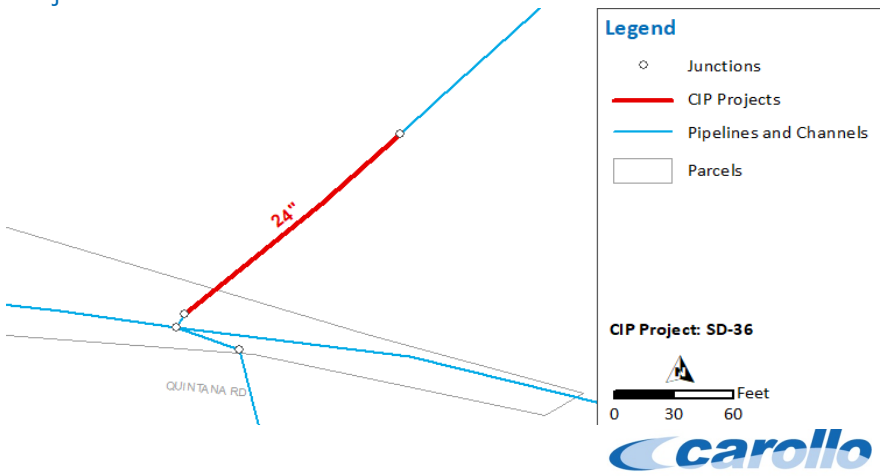
Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Project Detail:



Notes on Improvement:

This project is an improvement for the 10-year design storm.

Project Number: SDO-1
Project Name: Storm Drain Master Plan Update
System Type: Storm Water Collection System

Project Description:

It is recommended that the City undergoes a Storm Drain Master Plan Update every 5-years to evaluate wastewater collection system.

Project Details:

Project Element	Existing Diameter (in)	Proposed Diameter (in)	Replace/ New	Length (ft)	Unit Cost[1] ($/ft)	Baseline Construction Cost ($)	Estimated Construction Cost[2] ($)	Capital Improvement Cost[3] ($)	Project Schedule
Storm Drain Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2023
Storm Drain Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2028
Storm Drain Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2029-2040
Storm Drain Master Plan Update	--	--	--	--	--	--	--	$ 150,000	2029-2040

Notes:

(1) ENR 20 City Average Construction Cost Index for February 2018 is 10,889.

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

(3) Total project costs includes a 10% markup for engineering, a 10% markup for construction management and a 7.5% markup for project administration of the estimated construction cost.

Notes on Cost Estimation:

As a Storm Drain Master Plan Update, current users are assigned 100-percent of the project's cost.

Appendix N

WATER SUPPLY ALTERNATIVE COST ESTIMATES

Alternative 2A: Morro Well Field BWRO Treamtent

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
Saltwater Water Pipeline	12	900	$ 180.00	$ 162,000.00	$ 243,000.00	$ 1,620.00	
			Subtotal For Pipelines	*$ 522,000.00*	*$ 783,000.00*	*$ 5,220.00*	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 59,000.00
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 59,000.00
			Subtotal for Water Pump Station	$ 550,000.00	$ 826,000.00	$ 11,000.00	$ 118,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ -	$ -	$ 400.00	$ 20,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	0.519	mgd	$ 7,000,000.00	$ 3,633,000.00	$ 5,450,000.00	$ 145,320.00	$ 153,000.00
			Subtotal for Treatment Facilities	$ 3,633,000.00	$ 5,450,000.00	$ 145,320.00	$ 153,000.00
TOTAL				**$ 4,705,000.00**	**$ 7,059,000.00**	**$ 161,940.00**	**$ 291,000.00**

Net Yield (AFY)	**581**
Amoritized Value w/ O&M ($)	**$607,498.27**
Amoritized Value w/ O&M & Energy ($)	**$898,498.27**
Energy (kWh/AF)	**3339.07**
Unit Cost ($/AF)	**$ 1,546.47**

Alternative 2B: Morro Well Field Nitrate Treamtent

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	12	3000	$ 180.00	$ 540,000.00	$ 810,000.00	$ 5,400.00	
			Subtotal For Pipelines	$ 540,000.00	$ 810,000.00	$ 5,400.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 550,000.00	$ 826,000.00	$ 11,000.00	$ 98,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ -	$ -	$ 400.00	$ 20,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	360	gpm	$ 3,000,000.00	$ 3,000,000.00	$ 4,500,000.00	$ 120,000.00	$ 78,000.00
			Subtotal for Treatment Facilities	$ 3,000,000.00	$ 4,500,000.00	$ 120,000.00	$ 78,000.00
TOTAL				**$ 4,090,000.00**	**$ 6,136,000.00**	**$ 136,800.00**	**$ 196,000.00**

Net Yield (AFY)	**581**
Amoritized Value w/ O&M ($)	**$522,038.37**
Amoritized Value w/ O&M & Energy ($)	**$718,038.37**
Energy (kWh/AF)	**2249.00**
Unit Cost ($/AF)	**$ 1,235.87**

Alternative 2C: Morro + Chorro Well Fields BWRO Treamtent

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
Potable Water Pipeline	10	18000	$ 150.00	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
Saltwater Water Pipeline	12	900	$ 180.00	$ 162,000.00	$ 243,000.00	$ 1,620.00	
			Subtotal For Pipelines	*$ 3,546,000.00*	*$ 5,319,000.00*	*$ 35,460.00*	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	140	hp	$ 5,500.00	$ 770,000.00	$ 1,155,000.00	$ 15,400.00	$ 137,000.00
<100 hp	140	hp	$ 5,500.00	$ 770,000.00	$ 1,155,000.00	$ 15,400.00	$ 137,000.00
<100 hp	40	hp	$ 5,500.00	$ 220,000.00	$ 330,000.00	$ 4,400.00	$ 39,000.00
<100 hp	40	hp	$ 5,500.00	$ 220,000.00	$ 330,000.00	$ 4,400.00	$ 39,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 1,980,000.00	$ 2,970,000.00	$ 39,600.00	$ 352,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 400.00	$ 20,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	1.54	mgd	$ 7,000,000.00	$ 10,780,000.00	$ 16,170,000.00	$ 431,200.00	$ 453,000.00
			Subtotal for Treatment Facilities	$ 10,780,000.00	$ 16,170,000.00	$ 431,200.00	$ 453,000.00
TOTAL				**$ 18,306,000.00**	**$ 27,459,000.00**	**$ 506,660.00**	**$ 825,000.00**

Net Yield (AFY)	**1723.5**	*=581 AFY + 1142 AFY*
Amoritized Value w/ O&M ($)	**$2,187,315.51**	
Amoritized Value w/ O&M & Energy ($)	**$3,012,315.51**	
Energy unit costs (kWh/AF)	**3191.18**	
Unit Cost ($/AF)	**$ 1,747.79**	

Alternative 2C: Morro + Chorro Well Fields BWRO Treamtent (Chorro Offline)

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
Potable Water Pipeline	10	18000	$ 150.00	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
Saltwater Water Pipeline	12	900	$ 180.00	$ 162,000.00	$ 243,000.00	$ 1,620.00	
			Subtotal For Pipelines	*$ 3,546,000.00*	*$ 5,319,000.00*	*$ 35,460.00*	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	140	hp	$ 5,500.00	$ 770,000.00	$ 1,155,000.00	$ 15,400.00	$ 137,000.00
<100 hp	140	hp	$ 5,500.00	$ 770,000.00	$ 1,155,000.00	$ 15,400.00	$ 137,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 1,980,000.00	$ 2,970,000.00	$ 30,800.00	$ 274,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 400.00	$ 20,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	1.540	mgd	$ 7,000,000.00	$ 10,780,000.00	$ 16,170,000.00	$ 431,200.00	$ 153,000.00
			Subtotal for Treatment Facilities	$ 10,780,000.00	$ 16,170,000.00	$ 431,200.00	$ 153,000.00
TOTAL				**$ 18,306,000.00**	**$ 27,459,000.00**	**$ 497,860.00**	**$ 447,000.00**

Net Yield (AFY)	**581** *=581 AFY + 1142 AFY*
Amoritized Value w/ O&M ($)	**$2,178,515.51**
Amoritized Value w/ O&M & Energy ($)	**$2,625,515.51**
Energy unit costs (kWh/AF)	**5129.09**
Unit Cost ($/AF)	**$ 4,518.96**

Alternative 2D: Morro + Chorro Well Fields Nitrate Treamtent

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
			Subtotal For Pipelines	$ 684,000.00	$ 1,026,000.00	$ 6,840.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	70	hp	$ 5,500.00	$ 385,000.00	$ 578,000.00	$ 7,700.00	$ 69,000.00
<100 hp	70	hp	$ 5,500.00	$ 385,000.00	$ 578,000.00	$ 7,700.00	$ 69,000.00
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ 20,000.00
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ 20,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 990,000.00	$ 1,486,000.00	$ 19,800.00	$ 178,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 2,000.00	$ 100,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	1500	gpm	$ 8,000,000.00	$ 8,000,000	$ 12,000,000	$ 320,000.00	$ 327,000.00
			Subtotal for Treatment Facilities	$ 8,000,000.00	$ 12,000,000.00	$ 320,000.00	$ 327,000.00
TOTAL				**$ 11,674,000.00**	**$ 17,512,000.00**	**$ 348,640.00**	**$ 605,000.00**

Net Yield (AFY)	**1723.5**	*=581 AFY + 1142 AFY*
Amoritized Value w/ O&M ($)	**$1,436,389.71**	
Amoritized Value w/ O&M & Energy ($)	**$2,041,389.71**	
Energy unit costs (kWh/AF)	**2340.20**	
Unit Cost ($/AF)	**$ 1,184.44**	

Alternative 2D: Morro + Chorro Well Fields Nitrate Treamtent (Chorro Offline)

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
			Subtotal For Pipelines	$ 684,000.00	$ 1,026,000.00	$ 6,840.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	70	hp	$ 5,500.00	$ 385,000.00	$ 578,000.00	$ 7,700.00	$ 69,000.00
<100 hp	70	hp	$ 5,500.00	$ 385,000.00	$ 578,000.00	$ 7,700.00	$ 69,000.00
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ -
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ -
			Subtotal for Water and Recycled Water Pump Station	$ 990,000.00	$ 1,486,000.00	$ 19,800.00	$ 138,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 2,000.00	$ 100,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	1500	gpm	$ 8,000,000.00	$ 8,000,000	$ 12,000,000	$ 320,000.00	$ 78,000.00
			Subtotal for Treatment Facilities	$ 8,000,000.00	$ 12,000,000.00	$ 320,000.00	$ 78,000.00
TOTAL				**$ 11,674,000.00**	**$ 17,512,000.00**	**$ 348,640.00**	**$ 316,000.00**

Net Yield (AFY)	**581** *=581 AFY + 1142 AFY*
Amoritized Value w/ O&M ($)	**$1,436,389.71**
Amoritized Value w/ O&M & Energy ($)	**$1,752,389.71**
Energy unit costs (kWh/AF)	**3625.93**
Unit Cost ($/AF)	**$ 3,016.16**

Alternative 2E: Morro + Chorro Well Fields BWRO + Nitrate Treamtent

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
Saltwater Water Pipeline	12	900	$ 180.00	$ 162,000.00	$ 243,000.00	$ 1,620.00	
			Subtotal For Pipelines	$ 846,000.00	$ 1,269,000.00	$ 8,460.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ 20,000.00
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ 20,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 770,000.00	$ 1,156,000.00	$ 15,400.00	$ 138,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 2,000.00	$ 100,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	1000	gpm	$ 8,000,000.00	$ 8,000,000.00	$ 12,000,000.00	$ 320,000.00	$ 218,000.00
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	0.519	mgd	$ 7,000,000.00	$ 3,633,000.00	$ 5,450,000.00	$ 145,320.00	$ 153,000.00
			Subtotal for Treatment Facilities	$ 11,633,000.00	$ 17,450,000.00	$ 465,320.00	$ 371,000.00
TOTAL				**$ 15,249,000.00**	**$ 22,875,000.00**	**$ 491,180.00**	**$ 609,000.00**

Net Yield (AFY)	**1723.5**	*=581 AFY + 1142 AFY*
Amoritized Value w/ O&M ($)	**$1,927,538.98**	
Amoritized Value w/ O&M & Energy ($)	**$2,536,538.98**	
Energy unit costs (kWh/AF)	**2355.67**	
Unit Cost ($/AF)	**$ 1,471.74**	

Alternative 2E: Morro + Chorro Well Fields BWRO + Nitrate Treamtent (Chorro Offline)

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Purified Water Pipeline	12	2000	$ 180.00	$ 360,000.00	$ 540,000.00	$ 3,600.00	
Saltwater Water Pipeline	12	900	$ 180.00	$ 162,000.00	$ 243,000.00	$ 1,620.00	
			Subtotal For Pipelines	$ 846,000.00	$ 1,269,000.00	$ 8,460.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ -
<100 hp	20	hp	$ 5,500.00	$ 110,000.00	$ 165,000.00	$ 2,200.00	$ -
			Subtotal for Water and Recycled Water Pump Station	$ 770,000.00	$ 1,156,000.00	$ 15,400.00	$ 98,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
Existing Well pumping to Local Facility (annual)	4	Well	$ 5,000.00	$ -	$ -	$ 400.00	$ 20,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 2,000.00	$ 100,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	1000	gpm	$ 8,000,000.00	$ 8,000,000.00	$ 12,000,000.00	$ 320,000.00	$ 78,000.00
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	0.51875	mgd	$ 7,000,000.00	$ 3,631,000.00	$ 5,447,000.00	$ 145,240.00	$ 153,000.00
			Subtotal for Treatment Facilities	$ 11,631,000.00	$ 17,447,000.00	$ 465,240.00	$ 231,000.00
TOTAL				**$ 15,247,000.00**	**$ 22,872,000.00**	**$ 491,100.00**	**$ 429,000.00**

Net Yield (AFY)	**581**	*=581 AFY + 1142 AFY*
Amoritized Value w/ O&M ($)	**$1,927,263.83**	
Amoritized Value w/ O&M & Energy ($)	**$2,356,263.83**	
Energy unit costs (kWh/AF)	**4922.55**	
Unit Cost ($/AF)	**$ 4,055.53**	

Alternative 3A: Chorro Creek Streamflow Augmentation

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	1500	$ 120.00	$ 180,000.00	$ 270,000.00	$ 1,800.00	
Purified Water Pipeline	12	16500	$ 180.00	$ 2,970,000.00	$ 4,455,000.00	$ 29,700.00	
Jack and Bore	30	500	$ 525,000.00	$ 525,000.00	$ 788,000.00	$ 5,250.00	
Streamflow Gage			$ 38,500.00	$ 38,500.00	$ 58,000.00	$ -	
			Subtotal For Pipelines	$ 3,713,500.00	$ 5,571,000.00	$ 36,750.00	
STORAGE TANKS							
	Quantity	**Unit**	**$/Unit**				
<1 million gallons	0.25	MG	$ 2.50	$ 625,000.00	$ 938,000.00	$ 6,250.00	
			Subtotal for Tanks	$ 625,000.00	$ 938,000.00	$ 6,250.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	40	hp	$ 5,500.00	$ 220,000.00	$ 330,000.00	$ 4,400.00	$ 39,000.00
<100 hp	40	hp	$ 5,500.00	$ 220,000.00	$ 330,000.00	$ 4,400.00	$ 39,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 440,000.00	$ 660,000.00	$ 8,800.00	$ 78,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 1,600.00	$ 80,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	0.51	mgd	$ 7,000,000.00	$ 3,570,000.00	$ 5,355,000.00	$ 142,800.00	$ 150,000.00
Title 22 UV Equipment ($/mgd)	1.02	mgd	$ 450,000.00	$ 459,000.00	$ 689,000.00	$ 13,770.00	$ 51,000.00
Microfiltration ($/mgd)	1.02	mgd	$ 2,500,000.00	$ 2,550,000.00	$ 3,825,000.00	$ 76,500.00	$ 43,000.00
			Subtotal for Treatment Facilities	$ 6,579,000.00	$ 9,869,000.00	$ 233,070.00	$ 244,000.00
TOTAL				**$ 13,357,500.00**	**$ 20,038,000.00**	**$ 286,470.00**	**$ 402,000.00**

Net Yield (AFY)	**1142**
Amoritized Value w/ O&M ($)	**$1,497,435.85**
Amoritized Value w/ O&M & Energy ($)	**$1,899,435.85**
Energy (kWh/AF)	**2346.76**
Unit Cost ($/AF)	**$ 1,663.25**

Alternative 3B: Chorro Creek Streamflow Augmentation + Nitrate Treatment

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	8	1500	$ 120.00	$ 180,000.00	$ 270,000.00	$ 1,800.00	
Purified Water Pipeline	12	16500	$ 180.00	$ 2,970,000.00	$ 4,455,000.00	$ 29,700.00	
Potable Water Pipeline	8	2700	$ 120.00	$ 324,000.00	$ 486,000.00	$ 3,240.00	
Jack and Bore	30	500	$ 525,000.00	$ 525,000.00	$ 788,000.00	$ 5,250.00	
Streamflow Gage			$ 38,500.00	$ 38,500.00	$ 58,000.00	$ -	
			Subtotal For Pipelines	$ 4,037,500.00	$ 6,057,000.00	$ 39,990.00	
STORAGE TANKS							
	Quantity	**Unit**	**$/Unit**				
<1 million gallons	0.25	MG	$ 2.50	$ 625,000.00	$ 938,000.00	$ 6,250.00	
			Subtotal for Tanks	$ 625,000.00	$ 938,000.00	$ 6,250.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	40	hp	$ 5,500.00	$ 220,000.00	$ 330,000.00	$ 4,400.00	$ 39,000.00
<100 hp	40	hp	$ 5,500.00	$ 220,000.00	$ 330,000.00	$ 4,400.00	$ 39,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 440,000.00	$ 660,000.00	$ 8,800.00	$ 78,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	4	Well	$ 500,000.00	$ 2,000,000.00	$ 3,000,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
			Subtotal for Groundwater Injection Wells	$ 2,000,000.00	$ 3,000,000.00	$ 1,600.00	$ 80,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	708	gpm	$ 3,600,000.00	$ 3,600,000.00	$ 5,400,000.00	$ 144,000.00	$ 154,000.00
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	0.51	mgd	$ 7,000,000.00	$ 3,570,000.00	$ 5,355,000.00	$ 142,800.00	$ 150,000.00
Title 22 UV Equipment ($/mgd)	1.02	mgd	$ 450,000.00	$ 459,000.00	$ 689,000.00	$ 13,770.00	$ 51,000.00
Microfiltration ($/mgd)	1.02	mgd	$ 2,500,000.00	$ 2,550,000.00	$ 3,825,000.00	$ 76,500.00	$ 43,000.00
			Subtotal for Treatment Facilities	$ 10,179,000.00	$ 15,269,000.00	$ 377,070.00	$ 398,000.00
TOTAL				**$ 17,281,500.00**	**$ 25,924,000.00**	**$ 433,710.00**	**$ 556,000.00**

Net Yield (AFY)	**1142**
Amoritized Value w/ O&M ($)	**$ 2,022,575.09**
Amoritized Value w/ O&M & Energy ($)	**$ 2,578,575.09**
Energy (kWh/AF)	**3245.77**
Unit Cost ($/AF)	**$ 2,257.95**

Alternative 4A: WRF Morro Well Field Injection

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	12	15000	$ 180.00	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
			Subtotal For Pipelines	*$ 2,700,000.00*	*$ 4,050,000.00*	*$ 27,000.00*	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	60	hp	$ 5,500.00	$ 330,000.00	$ 495,000.00	$ 6,600.00	$ 59,000.00
<100 hp	60	hp	$ 5,500.00	$ 330,000.00	$ 495,000.00	$ 6,600.00	$ 59,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 660,000.00	$ 990,000.00	$ 13,200.00	$ 118,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Shallow Injection Well (100 ft)	4	Well	$ 2,050,000.00	$ 8,200,000.00	$ 12,300,000.00	$ 164,000.00	$ 3,500.00
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
			Subtotal for Groundwater Injection Wells	$ 8,200,000.00	$ 12,300,000.00	$ 164,000.00	$ 3,500.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
IPR ($/mgd)	0.737	mgd	$ 8,200,000.00	$ 6,043,000.00	$ 9,065,000.00	$ 241,720.00	$ 149,000.00
			Subtotal for Treatment Facilities	$ 6,043,000.00	$ 9,065,000.00	$ 241,720.00	$ 149,000.00
TOTAL				**$ 17,603,000.00**	**$ 26,405,000.00**	**$ 445,920.00**	**$ 270,500.00**

Net Yield (AFY)	**943**
Amoritized Value w/ O&M ($)	**$1,988,906.26**
Amoritized Value w/ O&M & Energy ($)	**$2,259,406.26**
Energy (kWh/AF)	**1912.34**
Unit Cost ($/AF)	**$ 2,395.98**

Alternative 4B: WRF Morro Well Field Injection with BWRO Treatment

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	12	15000	$ 180.00	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
			Subtotal For Pipelines	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	60	hp	$ 5,500.00	$ 330,000.00	$ 495,000.00	$ 6,600.00	$ 59,000.00
<100 hp	60	hp	$ 5,500.00	$ 330,000.00	$ 495,000.00	$ 6,600.00	$ 59,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 660,000.00	$ 990,000.00	$ 13,200.00	$ 118,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Shallow Injection Well (100 ft)	4	Well	$ 2,050,000.00	$ 8,200,000.00	$ 12,300,000.00	$ 164,000.00	$ 3,500.00
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
			Subtotal for Groundwater Injection Wells	$ 8,200,000.00	$ 12,300,000.00	$ 164,000.00	$ 3,500.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
IPR ($/mgd)	0.737	mgd	$ 8,200,000.00	$ 6,043,000.00	$ 9,065,000.00	$ 241,720.00	$ 149,000.00
Brackish Groundwater Desalination Treatment (BWRO) ($/mgd)	1.071	mgd	$ 7,000,000.00	$ 7,497,000.00	$ 11,246,000.00	$ 299,880.00	$ 315,000.00
			Subtotal for Treatment Facilities	$ 13,540,000.00	$ 20,311,000.00	$ 541,600.00	$ 464,000.00
TOTAL				$ 25,100,000.00	$ 37,651,000.00	$ 745,800.00	$ 585,500.00

Net Yield (AFY)	**1200**
Amoritized Value w/ O&M ($)	**$3,020,354.70**
Amoritized Value w/ O&M & Energy ($)	**$3,605,854.70**
Energy (kWh/AF)	**3252.78**
Unit Cost ($/AF)	**$ 3,004.88**

Alternative 4C: WRF Morro Well Field Injection with Nitrate Treatment

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	12	15000	$ 180.00	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
			Subtotal For Pipelines	$ 2,700,000.00	$ 4,050,000.00	$ 27,000.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	60	hp	$ 5,500.00	$ 330,000.00	$ 495,000.00	$ 6,600.00	$ 59,000.00
<100 hp	60	hp	$ 5,500.00	$ 330,000.00	$ 495,000.00	$ 6,600.00	$ 59,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 660,000.00	$ 990,000.00	$ 13,200.00	$ 118,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Shallow Injection Well (100 ft)	4	Well	$ 2,050,000.00	$ 8,200,000.00	$ 12,300,000.00	$ 164,000.00	$ 3,500.00
Existing Well pumping to Remote (annual)	4	Well	$ 20,000.00	$ -	$ -	$ 1,600.00	$ 80,000.00
			Subtotal for Groundwater Injection Wells	$ 8,200,000.00	$ 12,300,000.00	$ 164,000.00	$ 3,500.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Well Nitrate Treatment ($ per system)	585	gpm	$ 3,600,000.00	$ 3,600,000.00	$ 5,400,000.00	$ 144,000.00	$ 127,000.00
IPR ($/mgd)	0.737	mgd	$ 8,200,000.00	$ 6,043,000.00	$ 9,065,000.00	$ 241,720.00	$ 149,000.00
			Subtotal for Treatment Facilities	$ 9,643,000.00	$ 14,465,000.00	$ 385,720.00	$ 276,000.00
TOTAL				**$ 21,203,000.00**	**$ 31,805,000.00**	**$ 589,920.00**	**$ 397,500.00**

Net Yield (AFY)	**943**
Amoritized Value w/ O&M ($)	**$2,484,184.01**
Amoritized Value w/ O&M & Energy ($)	**$2,881,684.01**
Energy (kWh/AF)	**2810.18**
Unit Cost ($/AF)	**$ 3,055.87**

Alternative 5: Ocean Desalination

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Saltwater Water Pipeline	12	900	$ 180.00	$ 162,000.00	$ 243,000.00	$ 1,620.00	
			Subtotal For Pipelines	$ 162,000.00	$ 243,000.00	$ 1,620.00	
STORAGE TANKS							
	Quantity	**Unit**	**$/Unit**				
<1 million gallons	0.25	MG	$ 2.50	$ 625,000.00	$ 938,000.00	$ 6,250.00	
			Subtotal for Tanks	$ 625,000.00	$ 938,000.00	$ 6,250.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	10	hp	$ 5,500.00	$ 55,000.00	$ 83,000.00	$ 1,100.00	$ 10,000.00
			Subtotal for Water and Recycled Water Pump Station	$ 55,000.00	$ 83,000.00	$ 1,100.00	$ 10,000.00
WELLS							
	Quantity	**Unit**	**$/Unit**				
Well E&I and Infrastructure Updates	6	Well	$ 500,000.00	$ 3,000,000.00	$ 4,500,000.00	$ -	$ -
Existing Well pumping to Remote (annual)	6	Wells	$ 20,000.00	$ -	$ -	$ 2,400.00	$ 120,000.00
			Subtotal for Groundwater Injection Wells	$ 3,000,000.00	$ 4,500,000.00	$ 2,400.00	$ 120,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
Ocean Desalination ($/mgd)	0.576	mgd	$ 12,000,000.00	$ 6,912,000.00	$ 10,368,000.00	$ 276,480.00	$ 387,000.00
Iron Removal Pretreatment ($/mgd)	0.576	mgd	$ 500,000.00	$ 288,000.00	$ 432,000.00	$ 11,520.00	$ -
			Subtotal for Treatment Facilities	$ 7,200,000.00	$ 10,800,000.00	$ 288,000.00	$ 387,000.00
TOTAL				**$ 11,042,000.00**	**$ 16,564,000.00**	**$ 299,370.00**	**$ 517,000.00**

Net Yield (AFY)	**645**
Amoritized Value w/ O&M ($)	**$1,373,823.04**
Amoritized Value w/ O&M & Energy ($)	**$1,890,823.04**
Energy (kWh/AF)	**5343.67**
Unit Cost ($/AF)	**$ 2,931.51**

Alternative 6: Direct Potable Reuse (DPR)

Facility Type	Unit Costs and Quantities			Baseline Construction Cost ($)	Capital Improvement Cost ($)	Annual Operations and Maintenance Cost ($)	Annual Energy Cost ($)
PURIFIED, POTABLE, & SALT WATER PIPELINES							
	Diameter (in)	**Length (ft)**	**Unit Cost ($)**				
Purified Water Pipeline	12	250	$ 180.00	$ 45,000.00	$ 68,000.00	$ 450.00	
Potable Water Pipeline	12	5775	$ 180.00	$ 1,039,500.00	$ 1,559,000.00	$ 10,395.00	
			Subtotal For Pipelines	$ 1,084,500.00	$ 1,627,000.00	$ 10,845.00	
STORAGE TANKS							
	Quantity	**Unit**	**$/Unit**				
<1 million gallons	0.375	MG	$ 2.50	$ 938,000.00	$ 1,407,000.00	$ 9,380.00	
			Subtotal for Tanks	$ 938,000.00	$ 1,407,000.00	$ 9,380.00	
WATER PUMP STATION							
	Quantity	**Unit**	**$/Unit**				
<100 hp	70	hp	$ 5,500.00	$ 385,000.00	$ 578,000.00	$ 7,700.00	$ 69,000.00
<100 hp	70	hp	$ 5,500.00	$ 385,000.00	$ 578,000.00	$ 7,700.00	$ 69,000.00
<100 hp	10	hp	$ 5,500.00	$ 55,000.00	$ 83,000.00	$ 1,100.00	$ 10,000.00
<100 hp	10	hp	$ 5,500.00	$ 55,000.00	$ 83,000.00	$ 1,100.00	$ 10,000.00
<100 hp	50	hp	$ 5,500.00	$ 275,000.00	$ 413,000.00	$ 5,500.00	$ 49,000.00
	Subtotal for Water and Recycled Water Pump Station			$ 1,155,000.00	$ 1,735,000.00	$ 23,100.00	$ 207,000.00
TREATMENT FACILITIES							
	Quantity	**Unit**	**$/Unit**				
DPR ($/mgd)	0.97	mgd	$ 8,000,000.00	$ 7,760,000.00	$ 11,640,000.00	$ 310,400.00	$ 196,000.00
Microfiltration ($/mgd)	0.82	mgd	$ 2,500,000.00	$ 2,061,000.00	$ 3,092,000.00	$ 61,830.00	$ 35,000.00
DPR additional Monitoring (Lump Sum)	1	LS	$ 100,000.00	$ 100,000.00	$ 150,000.00	$ 3,000.00	$ -
Granular Activated Carbon Vessel ($/mgd)	0.82	mgd	$ 300,000.00	$ 247,000.00	$ 371,000.00	$ 49,400.00	$ 49,000.00
			Subtotal for Treatment Facilities	$ 10,168,000.00	$ 15,253,000.00	$ 424,630.00	$ 280,000.00
TOTAL				**$ 13,345,500.00**	**$ 20,022,000.00**	**$ 467,955.00**	**$ 487,000.00**

Net Yield (AFY)	**923**
Amoritized Value w/ O&M ($)	**$1,552,416.14**
Amoritized Value w/ O&M & Energy ($)	**$2,039,416.14**
Energy (kWh/AF)	**3515.84**
Unit Cost ($/AF)	**$ 2,208.50**

=Average WWTP capacity of 0.97 mgd x 85% recovery for advanced treatment