KING CITY
CALIFORNIA

FINAL

Collection System MASTER PLAN

September 2017

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COLLECTION SYSTEM MASTER PLAN

September 2017

FINAL

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KING CITY

COLLECTION SYSTEM MASTER PLAN

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LIST OF ABBREVIATIONS

AACE	Association for the Advancement of Cost Engineering
AAF	average annual flow
ac	acre
ADWF	average dry weather flow
BWF	base wastewater flow
Carollo	Carollo Engineers, Inc.
CCI	Construction Cost Index
CCTV	closed circuit television
CEQA	California Environmental Quality Act
CIP	Capital Improvement Plan
City	King City
cfs	cubic feet per second
d/D	depth to pipe diameter ratio
DWF	dry weather flow
ENR	Engineering News Record
EPA	Environmental Protection Agency
ºF	degrees Fahrenheit
ft/s	feet per second
GIS	geographic information system
gpd	gallons per day
GUI	graphical user interface
GWI	groundwater infiltration
HGL	hydraulic grade line
I/I	infiltration and inflow
in	inches

LF	linear feet
Little Bear CSD	Little Bear Community Services District
M	million
Master Plan	King City Collection System Master Plan
mgd	million gallons per day
MMR	monthly monitoring report
MSL	mean sea level
NRCS	Natural Resource Conservation Service
PWWF	peak wet weather flow
RDII	rainfall derived infiltration and inflows
ROW	Right of Way
sq ft	square feet
SSO	sanitary sewer overflows
SWMM	Storm Water Management Model
V&A	V&A Consulting Engineers
WaPUG	Wastewater Planning Users Group
WWF	wet weather flow
WWTP	wastewater treatment plant

EXECUTIVE SUMMARY

This executive summary presents a brief background of King City's (City's) wastewater collection system, the need for this master plan, proposed improvements to mitigate existing system deficiencies, and proposed expansion projects. A summary of capital improvement project costs is included at the end of this chapter.

1.1 INTRODUCTION

The City is located in the southern Salinas Valley of Central California, 45 miles south of Salinas in Monterey County. It was incorporated in 1911 with 699 residents, and has since grown to cover 3.8 square miles with a population of 13,580[1]. The City maintains a special sense of community and small-town living with beautiful mountain views, tree-lined streets, and a charming historical downtown. Agriculture continues to be the heart of the City's economic and cultural life.

The City owns, maintains, and operates gravity sewer pipelines, force mains, sewer lift stations, and the King City Wastewater Treatment Plant (WWTP). The City collects wastewater from residential, commercial, institutional, and industrial customers within its service area.

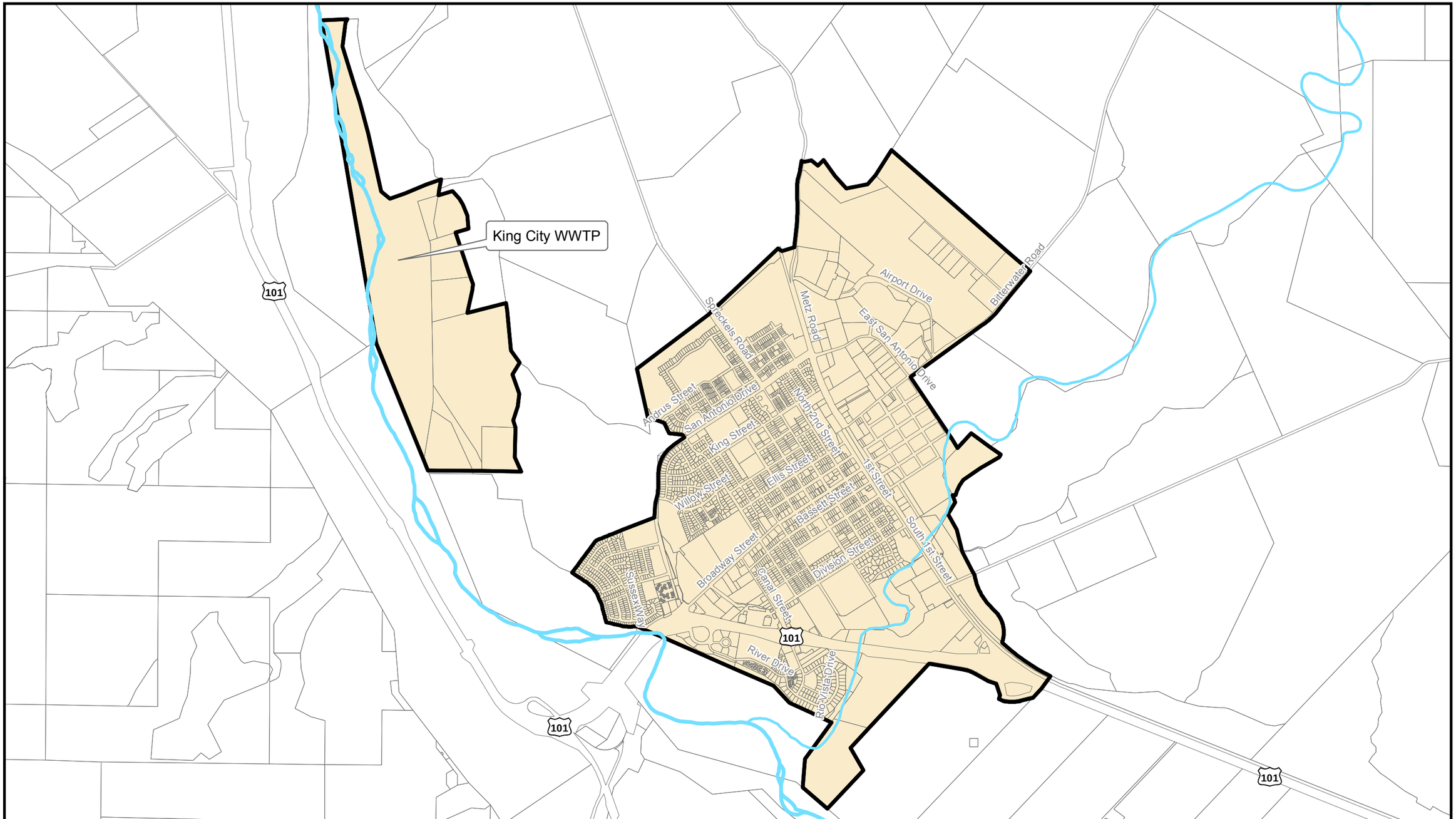
1.2 STUDY AREA

The area serviced by the City is characterized by residential, commercial, and industrial uses within the City limits. The City's economy is based largely on agriculture. What started out as a shipping point for wheat and cattle, the City has grown to become a vegetable center, shipping to all over the nation.

The City is located next to US Highway 101, approximately 45 miles south of Salinas. There are many open space areas within the City, including San Lorenzo Park. The major water bodies located near the City include the Salinas River, which runs along the west side of the City and the San Lorenzo Creek, which runs along the southeast side of the City, discharging to the Salinas River.

The study area boundary for this Master Plan consists generally of the City limits. In addition, the City is considering specific annexations in the future that will extend beyond the current City limits. This Master Plan is intended as the guiding document to plan and implement sewer system improvements to accommodate future growth to build out of the General Plan. Figure 1.1 shows the study area boundary.

[1] http://www.kingcity.com/



Legend

- Study Area
- Parcels
- River or Stream

Miles
0 0.25 0.5

SANITARY SEWER SERVICE AREA

FIGURE 1.1

KING CITY
COLLECTION SYSTEM MASTER PLAN

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The land use assumptions in this Master Plan were based on the City's General Plan Land Use GIS shapefiles and projected future developments within the City. Should future planning conditions change from the assumptions stated in this Master Plan (i.e., accelerated growth, more intense developments, etc.), revisions and adjustments to the Master Plan recommendations would be necessary.

1.3 SEWER SERVICE AREA OVERVIEW

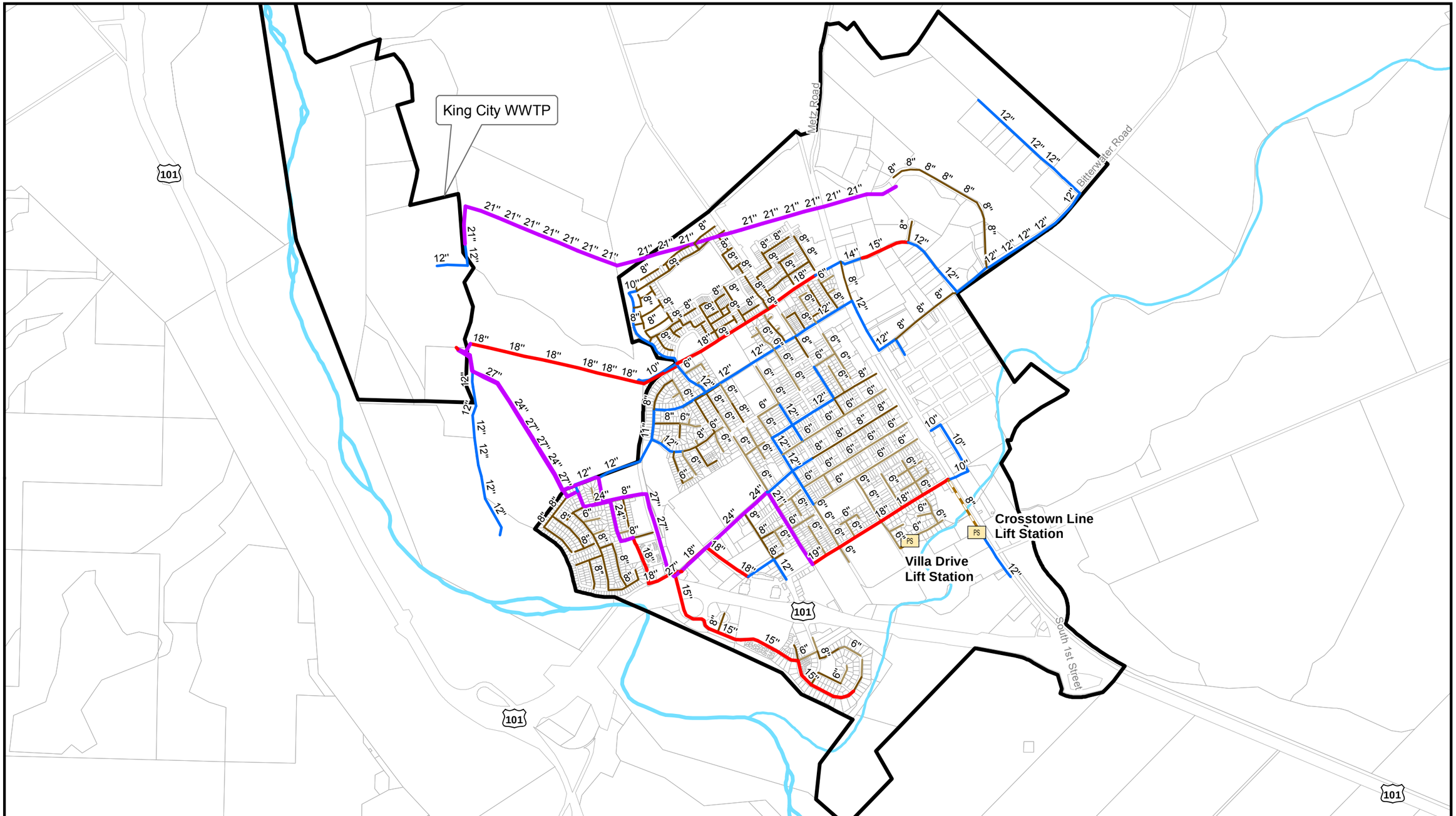
The City manages and maintains approximately 32 miles of gravity sewer lines up to 27-inches in diameter, two lift stations, and associated force mains. All wastewater generated within the sewer service area is conveyed to the City's WWTP for treatment. The City also operates a separate 21-inch industrial sewer line that historically conveyed food process wastewater. Currently, this line accepts distilled water discharges from the Calpine Cogeneration Power Plant. This flow is treated and disposed of separately from the domestic wastewater. Figure 1.2 presents the City's existing collection system, including sewer diameters and lift station locations.

1.4 WASTEWATER FLOWS

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 base wastewater flow (BWF) generated by the City's users, plus dry weather groundwater infiltration (GWI).

Peak wet weather flow (PWWF) is the highest observed hourly flow that occurs following the design storm event. The City's sewers were evaluated based on their capacity to convey the "design flow ("design flow" is synonymous to PWWF in this study).

A summary of the existing and future ADWF and the design flow is presented in Table 1.1. The City's ADWF is projected to almost double from 0.86 mgd to 1.56 mgd by Phase 2 (2037), whereas the PWWF is projected to increase from 4.36 mgd to about 5.67 mgd by Phase 2 (an increase of approximately 30 percent). Therefore, the City's PWWF to ADWF peaking factor is projected to decrease from roughly 5.1 to 3.6 by Phase 2, which is typical for sanitary sewer collection systems.



Legend

- Study Area
- Parcels
- River or Stream
- City Owned Lift Station
- Force Main

Gravity Sewer by Diameter

- 6"
- 8"
- 10"-14"
- 15"-18"
- 21" and Larger

N

0 0.25 0.5 Miles

EXISTING SANITARY SEWER COLLECTION SYSTEM

FIGURE 1.2

KING CITY
COLLECTION SYSTEM MASTER PLAN

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KING CITY CALIFORNIA

Table 1.1	Current and Projected Wastewater Flow Summary Collection System Master Plan King City		
Year	**Average Dry Weather Flow (mgd)**	**Design Flow (mgd)**	**Peaking Factor**
Existing (2017)	0.86	4.36	5.07
Phase 1 (2027)	1.18	5.36	4.54
Phase 2 (2037)	1.56	5.67	3.63
Phase 3 (Post 2037)	3.01	10.03	3.33

1.5 CAPACITY EVALUATION AND PROPOSED IMPROVEMENTS

The capacity analysis identified areas in the sewer system where flow restrictions occur or where pipe capacity is insufficient to convey design flows. Sewers that lack sufficient capacity to convey design flows create bottlenecks in the collection system that can potentially cause sanitary sewer overflows (SSOs).

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) exceeded 90 percent of the pipe diameter (maximum d/D greater than 0.9) were identified.

In general, the City's collection system has sufficient capacity to convey current PWWFs without exceeding the established flow depth criterion. However, there is one area where capacity restrictions lead to flow depths that exceed allowable levels. This is the 8-inch diameter gravity sewer on Bitterwater Road (from San Antonio Drive to Metz Road). Following the completion of the existing system analysis, improvement projects and alternatives were identified in order to mitigate existing system pipeline capacity deficiencies.

The analysis of the future systems (Phase 1, 2, and 3) 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 PWWFs.

The recommended improvements to correct existing deficiencies are summarized below.

- **Project 1 – Smoke Testing**: The majority of flow through contributing to the deficient 8-inch diameter gravity pipe on Bitterwater Road is industrial flow. Analysis of the flow monitoring data indicated that the peak flows in this area are due to inflow. It is possible that there may be a drain connected to the collection system causing spikes in flow. Because of this, smoke testing is recommended along Industrial Way, Airport

Road, and Bitterwater Road to identify the inflow source that is leading to high rates of infiltration and inflow (I/I) in the system.

- **Project 2 – Bitterwater Road Sewer and Reclaimed Water Main**: This project consists of replacing approximately 1,470 feet of existing 8-inch diameter sewer on Bitterwater Road from San Antonio Drive to Metz Road with a new 12-inch diameter sewer. This project is required to mitigate surcharged conditions on Bitterwater Road for existing PWWFs. However, this project may not be necessary if the source of inflow is identified during the smoke testing and is resolved. Although this recommended improvement is included in the capital improvement program presented in Chapter 7, it is recommended that this improvement be reevaluated once the smoke testing is completed to determine if it is still required.

  Because the City wants to take advantage of future pipeline projects, this project also includes the installation of approximately 1,470 feet of 10-inch diameter reclaimed water main.

- **Project 3 – Small Diameter Pipeline Replacement:** This project consists of replacing the City's existing small diameter sewers (6-inch diameter and smaller) with 8-inch diameter sewers. There is a total of 36,010 linear feet (LF) of 6-inch diameter gravity sewers in the collection system (approximately 21 percent of the collection system). A replacement program of 30 years would equate to replacing approximately 1,200 LF per year.

Two future system improvements were identified:

- **Project 4 – Little Bear/San Bernabe Sewer**: The hydraulic evaluation indicated that the existing 15-inch diameter sewer on Broadway Street from River Drive to San Antonio Drive has sufficient capacity to convey Phase 2 flows from Little Bear. However, as flows increase beyond the 20-year planning period (Phase 3), the existing 15-inch diameter sewer experienced surcharging. It was assumed for Phase 2, that Little Bear would be connected directly to the existing 15-inch diameter gravity sewer. However, for Phase 3, it is recommended that the Little Bear and San Bernabe developments connect to the 27-inch Crosstown sewer at Broadway Street and San Antonio Drive with approximately 1,250 LF of new 21-inch diameter gravity sewer.

- **Project 5 – Smith Monterey/Silva Sewer (Phase 3):** Under Phase 3 PWWF conditions, much of the existing 12-inch, 15-inch, and 18-inch diameter sewers on Bitterwater Road, Metz Road, King Street, Mildred Avenue, and San Antonio Drive to the treatment plant experienced surcharging and several overflows. This additional flow also causes surcharging and overflows upstream on Airport Road and Industrial Way. It is recommended that a parallel 15 to 21-inch diameter gravity sewer (approximately 13,380 LF) be installed to serve the new developments. The proposed pipeline extends from the intersection of Bitterwater Road and Industrial Way, along Bitterwater Road, Metz Road, then along San Antonio Drive to the treatment plant.

Figure 1.3 illustrates the improvements recommended to mitigate capacity deficiencies in the existing sewer collection system and improvements to accommodate future growth as identified by the hydraulic analysis.

1.5.1 Existing Versus Future Improvement

An existing deficiency is one where the existing facility's capacity is insufficient to meet the planning criteria (e.g., pipeline upgrades required to prevent severe surcharging during the design wet weather event) for existing users. If a project was proposed to correct an existing deficiency exclusively, then existing users were assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs.

A significant portion of the recommended Master Plan improvements will serve future users, even when an improvement calls for the upgrade of an existing facility. In these cases, an existing sewer may have sufficient capacity to convey current PWWFs, but as growth continues and more users are added to the system, the increased flow results in capacity deficiencies. These projects, as well as new trunk sewers to extend wastewater collection system service to future growth areas, are future improvements. Future users were assigned 100 percent of the future project's benefit and 100 percent of the costs.

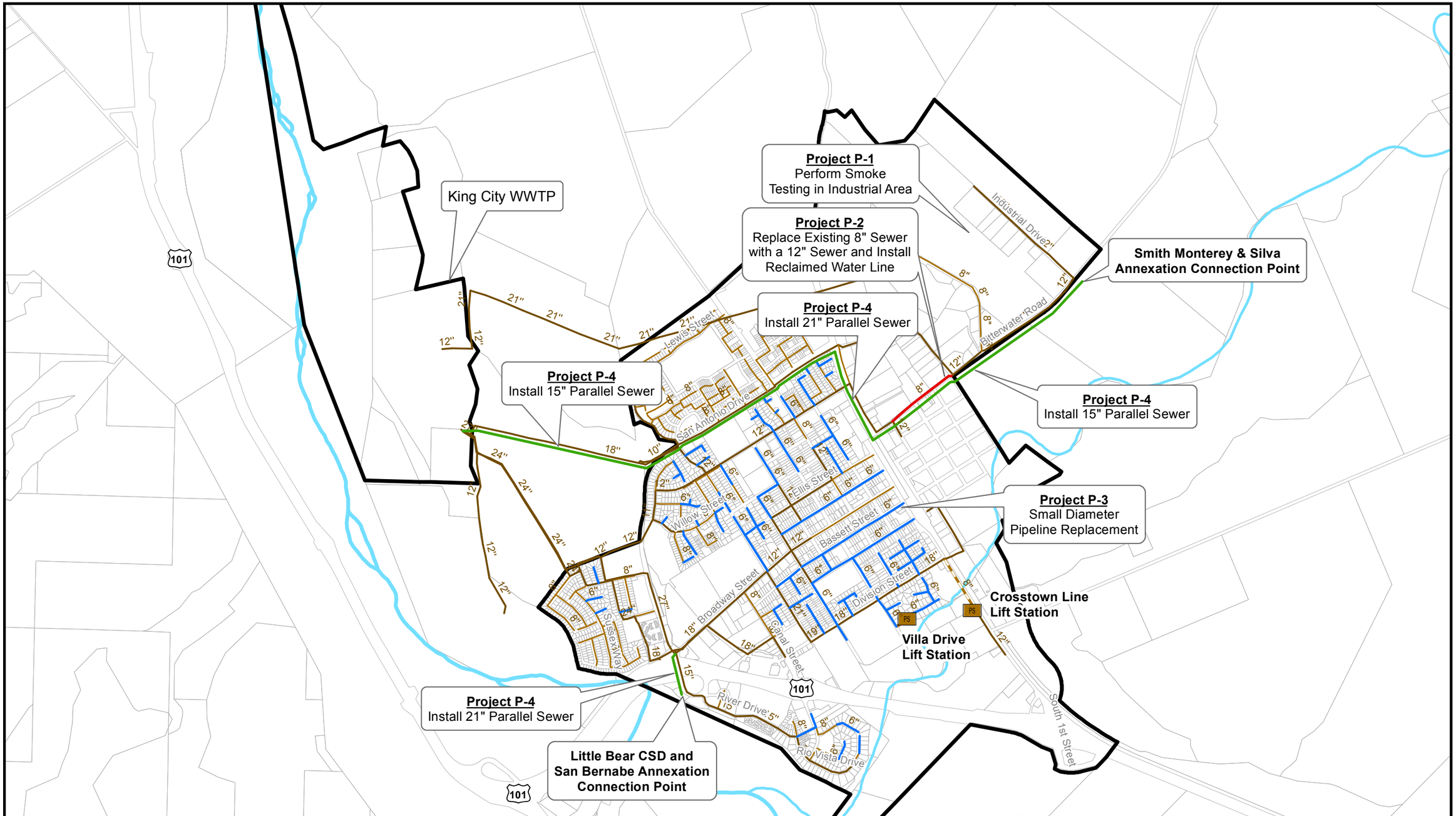
In some cases, a project is needed to correct an existing capacity deficiency, but is sized to accommodate additional flows from future development. In these cases, the hydraulic modeling results were used to determine the cost breakdown between existing and future users based on the ratio of existing and build out average dry weather flows.

1.5.2 Project Prioritization

A small portion of the improvements identified as part of this Master Plan are driven by future development, which consist of new sewers that serve future growth or improvements to existing facilities that are needed to serve future growth. When fully implemented, the capital projects will allow the conveyance of PWWFs to the treatment plant under Phase 3 (build out) conditions.

Prioritizing the required capital improvements for the City's sewer system is an important aspect of this study. The improvement projects were prioritized based on the following factors:

- Upgrading existing facilities to mitigate current capacity deficiencies and to serve future users
- Building the new trunks necessary to serve future users



Legend

- Study Area
- Parcels
- River or Stream
- City Owned Lift Station
- Force Main

Gravity Sewer by Diameter

- 8" and Smaller
- 10" and Larger

Proposed Improvements

- Phase 1 (2018-2027)
- Phase 2 (2028-2037)
- Phase 3 (Post 2037)
- Small Diameter Pipeline Replacement

Miles
0 0.25 0.5

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COLLECTION SYSTEM IMPROVEMENTS

FIGURE 1.3

KING CITY
COLLECTION SYSTEM MASTER PLAN

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CALIFORNIA

Improvements to existing facilities will provide sufficient capacity to mitigate existing issues and to convey increased flows resulting from future growth. Future development will require the construction of sewers to serve new users. The projects were grouped into the following phases:

- Phase 1: Years 2018 through 2027
- Phase 2: Years 2028 through 2037
- Phase 3: Beyond 2037

The projects were phased based on the best available information for how the City will develop moving forward. The actual implementation of the improvements serving future users ultimately depends on growth. The priorities presented below are estimates, and changes in the City's planning assumptions or growth projections could increase or decrease the priority of each improvement.

- **Phase 1 Projects (2018-2027)**: The highest priority project for the existing system is the smoke testing of the industrial area (Project 1). This is important to identify the source of high inflow rates, which may allow the City to avoid upsizing the 8-inch diameter gravity sewer on Bitterwater Road (Project 2). Another project targeted to begin in Phase 1 is the small diameter sewer replacement program (Project 3).
- **Phase 2 (2028-2037)**: No new projects were identified for Phase 2. The small diameter sewer replacement program (Project 3) is a long-term project which extends through Phase 2.
- **Phase 3 (Beyond 2037)**: The third phases target new sewers to serve future planned developments, including Little Bear and San Bernabe (Project 4) and Smith-Monterey and Silva (Project 5). As previously noted, the actual rate of growth within the City will dictate when these new pipelines will need be constructed.

1.6 CAPITAL IMPROVEMENT PLAN

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

The Association for the Advancement of Cost Engineering (AACE) defines an Order of Magnitude Estimate, deemed appropriate for master plan studies, as an approximate estimate made without detailed engineering data. It is normally expected that an estimate of this type would be accurate within plus 50 percent to minus 30 percent.

The CIPs are prioritized based on their urgency to mitigate existing deficiencies and for servicing anticipated growth. It is recommended that improvements to mitigate existing deficiencies be constructed as soon as possible. The improvements proposed in this study either benefit existing users and/or are required to serve new development and future users. A summary of the existing and future user cost share for the proposed projects by phase is summarized in Table 1.2. Recommended improvements for Phase 3 were not included in Table 1.2 or the CIP because it is beyond the planning period of this Master Plan.

Table 1.2 Summary of Capital Improvement Costs
Collection System Master Plan
King City

	Implementation Phase		
Reimbursement Category	**Phase 1 (2018-27) ($,M)**	**Phase 2 (2028-37) ($,M)**	**Total ($,M)**
Existing User	$1.91	$2.23	**$4.14**
Future User	$0.48	$-	**$0.48**
Total	**$2.39**	**$2.23**	**$4.62**

Notes:
(1) Costs are based on ENR CCI 20-City Average of 10,530 (December 2016).

Chapter 2

BACKGROUND AND STUDY AREA DESCRIPTION

This chapter presents a brief summary of the wastewater collection system service area, the need for this Collection System Master Plan (Master Plan) and the objectives of the study. It also presents a description of the study area, defines the planning horizon for the study, and summarizes the land use classifications.

2.1 BACKGROUND

King City (City) is located in the southern Salinas Valley of Central California, 45 miles south of Salinas in Monterey County. Figure 2.1 presents a location map of the City. Salinas Valley is one of the most productive agricultural regions in California. The City covers 3.8 square miles and lies approximately 155 miles south of San Francisco and 277 miles north of Los Angeles. The City is located between the Santa Lucia Mountains on the west and the Gabilan and Diablo Ranges on the east.

The City was incorporated in 1911 with 699 residents and has since grown to a population of 13,580[1]. The City maintains a special sense of community and small-town living with beautiful mountain views, tree-lines streets, and a charming historical downtown. Agriculture continues to be the heart of the City's economic and cultural life.

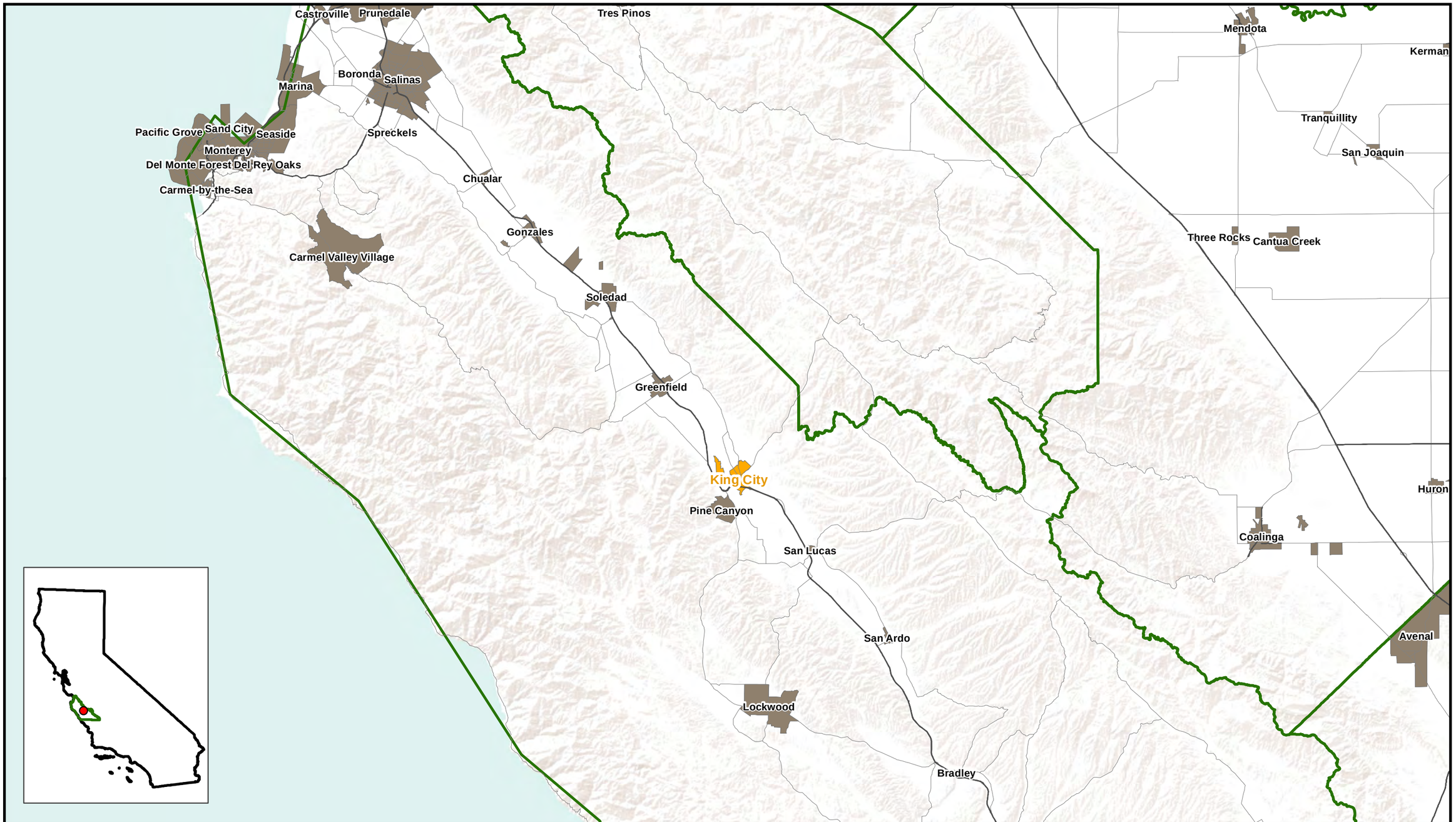
The City owns, maintains, and operates gravity sewer pipelines, force mains, sewer lift stations, and the King City Wastewater Treatment Plant (WWTP). The City collects wastewater from residential, commercial, institutional, and industrial customers within its service area.

2.1.1 Sewer Service Area

Figure 2.2 illustrates the City’s current wastewater service area. The City manages and maintains approximately 32 miles of gravity sewer lines up to 27-inches in diameter, two lift stations, and associated force mains. All wastewater generated within the sewer service area is conveyed to the City’s WWTP for treatment. The City also operates a separate 21-inch industrial sewer line that historically conveyed food process wastewater. Currently, this line accepts distilled water discharges from the Calpine Cogeneration Power Plant. This flow is treated and disposed of separately from the domestic wastewater.

The land use assumptions in this Master Plan were based on the City's General Plan Land Use GIS shapefiles and projected future developments within the City. Should future planning conditions change from the assumptions stated in this Master Plan (i.e., accelerated growth, more intense developments, etc.), revisions and adjustments to the Master Plan recommendations would be necessary.

[1] http://www.kingcity.com/



Legend

Cities

- Neighboring Community
- King City

- Major Highways
- Major Roads
- County Boundaries

N

0 5 10 Miles

REGIONAL LOCATION MAP

FIGURE 2.1

KING CITY
COLLECTION SYSTEM MASTER PLAN

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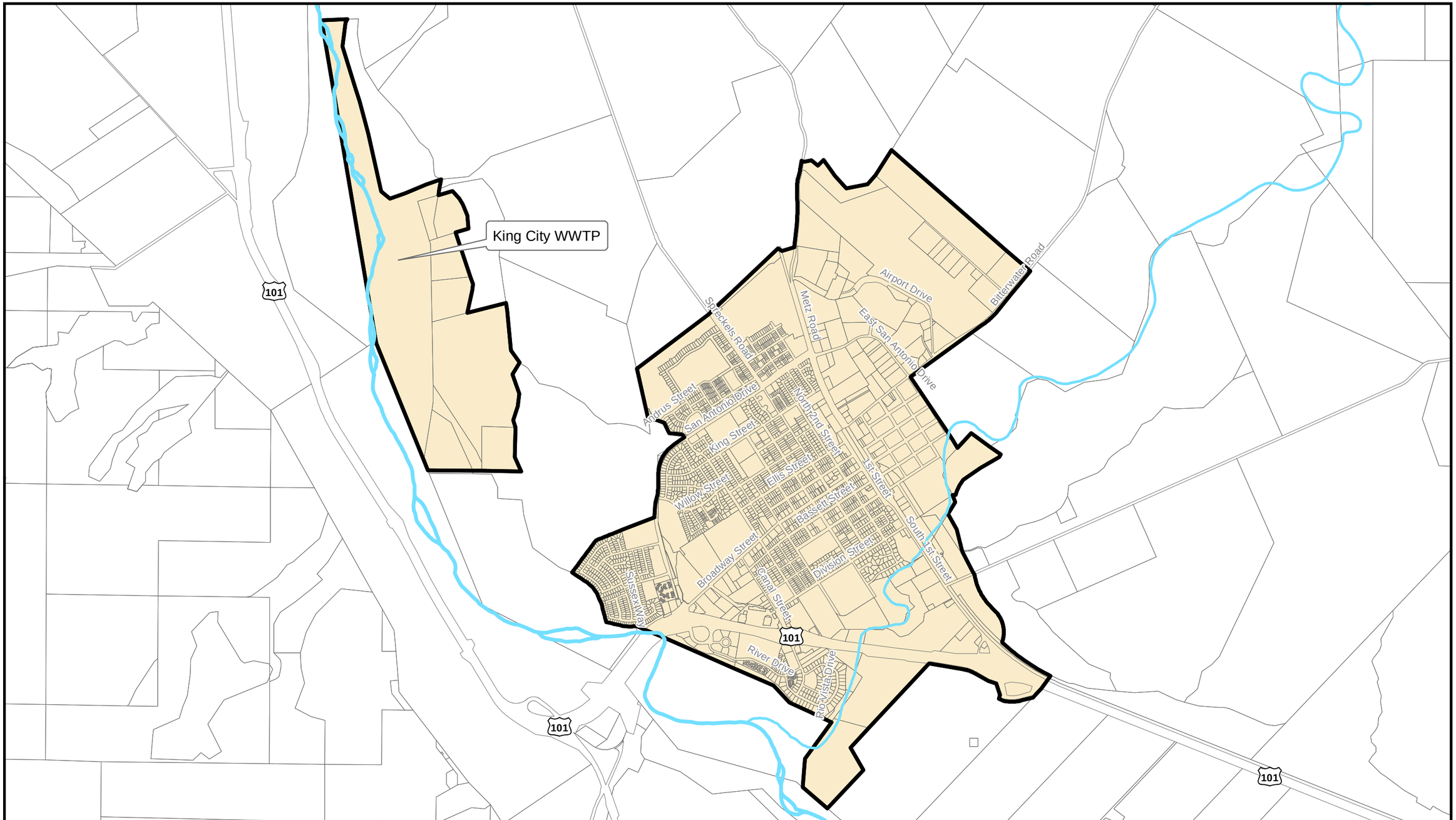

Legend

- Study Area
- Parcels
- River or Stream

Miles
0 0.25 0.5

SANITARY SEWER SERVICE AREA

FIGURE 2.2

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COLLECTION SYSTEM MASTER PLAN

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2.1.2 Previous Master Plan

The City completed a Sewer System Master Plan in 1992 (Carollo, 1992). The report was prepared to inventory the existing sewers and identify the improvements needed to accommodate future growth.

The Sewer System Master Plan (Carollo, 1992) was updated by the Wastewater Master Plan (Boyle, 2007). While this report was never finalized, it did include the creation of a Sewer CAD model and a capacity evaluation. The report indicates that the City's sewer system had been input into a geographic information system (GIS), which was used to create the hydraulic model.

The 2007 model was calibrated to average daily flows. Once calibrated, a capacity analysis of the modeled collection system was performed for both existing and build out peak hour flows. Capital projects were then developed to mitigate the model simulated capacity deficiencies. In total, the 2007 Wastewater Master Plan Update recommended $8,023,900 in capital improvements (in 2007 dollars) to the collection system, and an additional $16,444,500 to expand services to areas planned for annexation.

2.1.3 Scope and Authorization

The purpose of this Master Plan is to update the 2007 Master Plan and to identify capacity deficiencies in the wastewater collection system, develop feasible alternatives to correct these deficiencies, and plan the infrastructure that will serve future development projected by the King City General Plan. In September 2016, the City approved a professional service agreement with Carollo Engineers, Inc. (Carollo), to prepare this Master Plan Update for the wastewater collection system. The professional services agreement, included the following main tasks:

- Task 1 - Project Management
- Task 2 - Data Collection and Background Review
- Task 3 - Wastewater Collection System Evaluation
- Task 4 - Capital Projects
- Task 5 - Report Preparation and Adoption

2.1.4 Acknowledgments

Carollo Engineers wishes to acknowledge and thank Mr. Steven Adams, City Manager; Mr. Octavio Hurtado, Contract City Engineer; and Mr. Sal Morales, Public Works Superintendent. Their cooperation and courtesy in obtaining a variety of necessary information were valuable components in completing and producing this report.

2.2 STUDY AREA DESCRIPTION

The area serviced by the City is characterized by residential, commercial, and industrial uses within the City limits. The City's economy is based largely on agriculture. What started out as a shipping point for wheat and cattle, the City has grown to become a vegetable center, shipping to all over the nation.

The City is located next to US Highway 101, approximately 45 miles south of Salinas. There are many open space areas within the City, including San Lorenzo Park. The major water bodies located near the City include the Salinas River, which runs along the west side of the City and the San Lorenzo Creek, which runs along the southeast side of the City, discharging to the Salinas River.

The study area boundary for this Master Plan consists generally of the City limits. In addition, the City is considering specific annexations in the future that will extend beyond the current City limits. This Master Plan is intended as the guiding document to plan and implement sewer system improvements to accommodate future growth to build out of the General Plan. Figure 2.2 shows the study area boundary.

2.2.1 Planning Period

The following planning years have been established as part of this Master Plan:

- **Existing (Year 2016):** The capacity of the collection system was analyzed under current peak flow conditions to identify improvement projects that would be necessary to mitigate existing system deficiencies.
- **Phase 1 (Year 2026):** Phase 1 includes known developments that are expected to come online and connect to the City's collection system within the next 10-years.
- **Phase 2 (Year 2036):** Phase 2 includes additional known developments that are expected to occur within the 20-year planning horizon that was used to size the City's proposed wastewater treatment plant (WWTP) upgrades.
- **Phase 3 (Build-Out):** Phase 3 includes complete build-out of the City's existing service area.

2.2.2 Study Area Climate

The City’s study area is characterized by a semi-arid climate with cool, wet winters and very warm, mostly dry summers. Table 2.1 summarizes the maximum and minimum monthly temperatures as well as the average monthly precipitation. January is the City’s coldest and wettest month, with an average high temperature of 64.1 degrees Fahrenheit (°F), an average low temperature of 35 °F, and 2.47 inches of precipitation. July is the City’s hottest month, with an average high temperature of 86.8 °F and an average low temperature of 51 °F. Approximately 92 percent of the annual rainfall occurs between November and April, with an average annual rainfall of 11.25 inches.

Table 2.1	Study Area Climate Collection System Master Plan King City		
Month	**Average Maximum Temperature (ºF)**	**Average Minimum Temperature (ºF)**	**Average Monthly Rainfall (inches)**
January	64.1	35	2.47
February	66.7	37.9	2.32
March	70.3	39.6	1.97
April	74.8	41.5	0.79
May	79.1	45.1	0.25
June	83.8	48.5	0.06
July	86.8	51	0.01
August	86.4	51.1	0.02
September	86.2	49	0.14
October	81.3	44.1	0.45
November	72	38.1	0.98
December	65.3	34.6	1.8
Annual	**76.4**	**43**	**11.25**

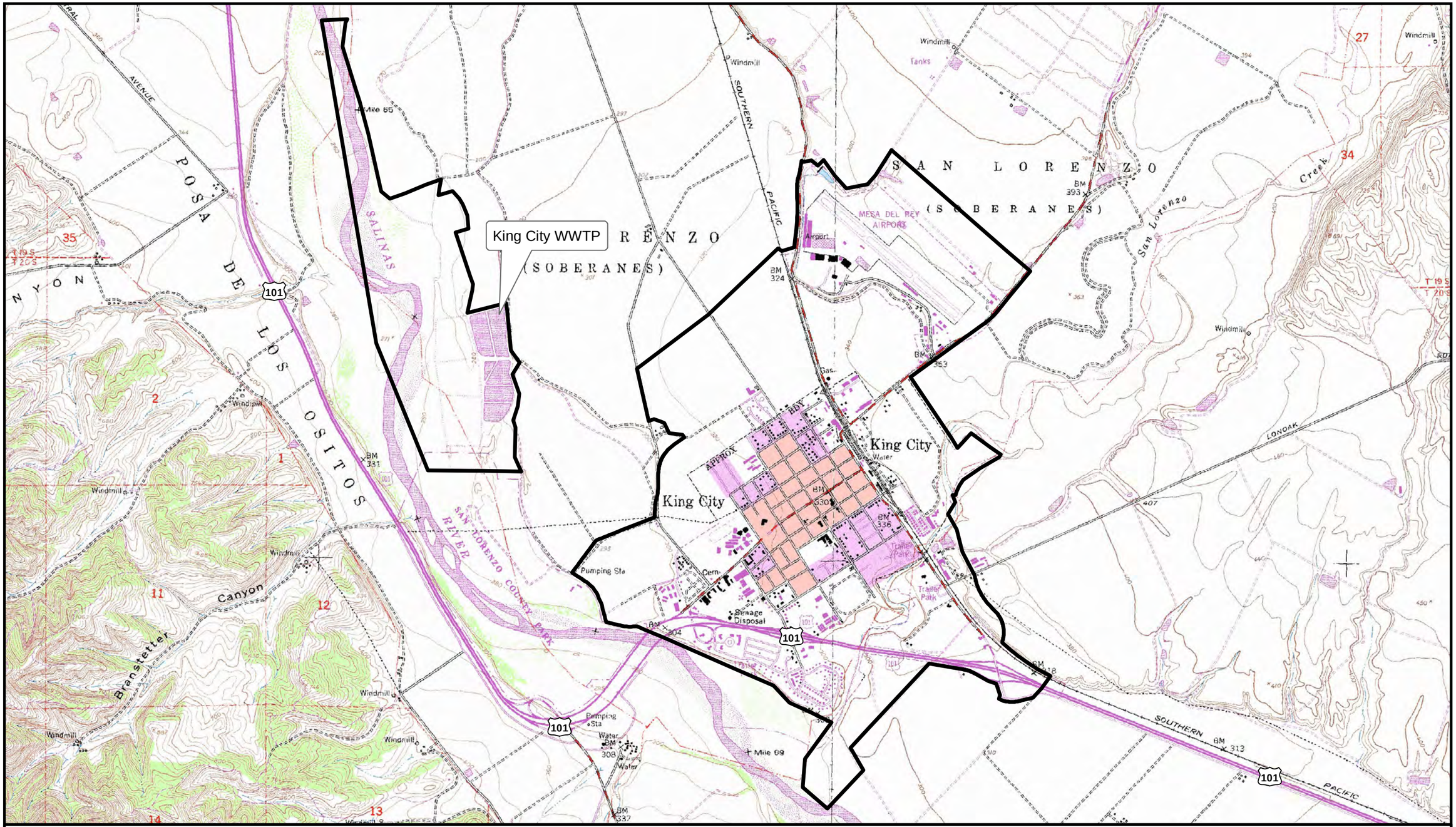
Notes:
(1) Source: Western Regional Climate Center, King City Station 044555, www.wrcc.dri.edu.

2.2.3 Topography

The City is located in the center of San Joaquin Valley near the eastern fringe of the Sierra Nevada mountain range. The terrain is flat except for the bluffs near the San Joaquin River. The majority of the City is relatively flat with an average elevation of approximately 335 feet above mean sea level (MSL). Figure 2.3 shows the general topography of the study area.

2.3 LAND USE AND FUTURE DEVELOPMENTS

Land use information is an integral component in determining the amount of wastewater generated within the City. The type of land use in an area will affect the volume and characteristics of the wastewater generation. Therefore, adequately estimating the generation of wastewater from various land use types is important in sizing and maintaining effective sanitary sewer system facilities.



Legend

Study Area

0 0.25 0.5 Miles

STUDY AREA TOPOGRAPHY

FIGURE 2.3

KING CITY
COLLECTION SYSTEM MASTER PLAN

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An important tool for determining land use projections is the City's General Plan, which guides development within the study area and establishes long-range development policies. Land use assumptions used in this study are consistent with those for existing and proposed development as published in the General Plan. Figure 2.4 shows the City's General Plan land use.

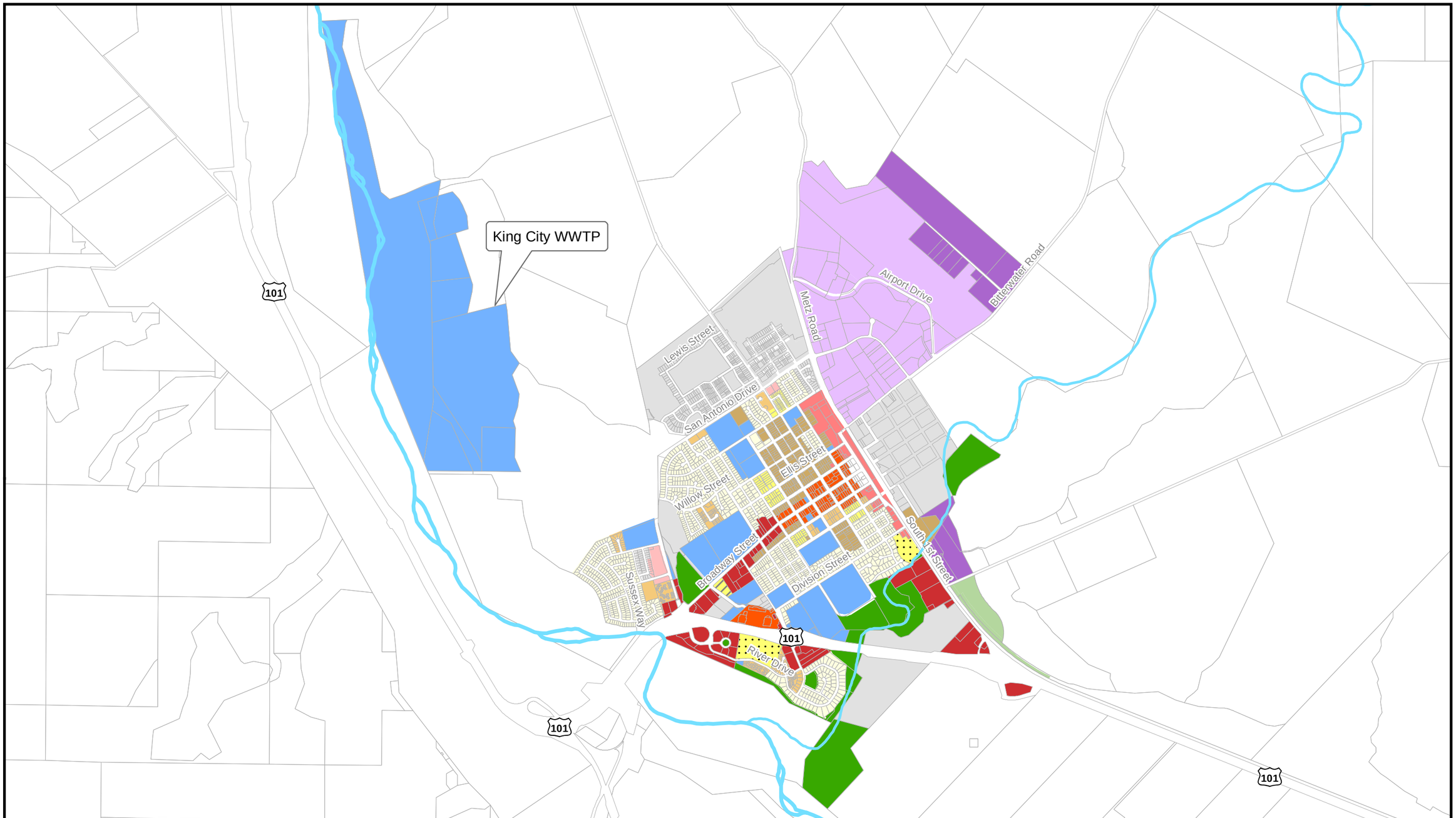
2.3.1 Existing Service Area Land Use

The City provides wastewater collection service to residents, businesses, and other institutions within its service area, which currently encompasses an area of approximately 976 acres. Table 2.2 summarizes the existing sewer service land area by existing land use designation.

Table 2.2 Existing Land Use
Collection System Master Plan
King City

Land Use Classification	Current Sewer Service Area Land Use: Land Use Area (acre)	Current Sewer Service Area Land Use: Percent of Total (%)
Residential		
Low Density	195.2	20.0
Medium Density	18.9	1.9
Medium Density/Mobile Home Park	2.2	0.2
Medium High Density	27.0	2.8
High Density	58.6	6.0
Subtotal	**301.9**	**30.9**
Commercial		
General	21.6	2.2
Highway Service	40.9	4.2
Neighborhood	5.9	0.6
Retail	31.7	3.3
Retail/Transitional	3.4	0.3
Subtotal	**103.5**	**10.6**
Industrial		
Light Industrial	324.6	33.3
General Industrial	17.2	1.8
Subtotal	**341.8**	**35.0**
Other		
Ag	0.0	0.0
Open Space	0.3	0.0
Planned Development	64.5	6.6
Planned Development/Mobile Home Park	19.4	2.0
Public/Quasi-Public Facility	144.3	14.8
Subtotal	**228.6**	**23.4**
Total	**975.7**	**100.0**

Notes:
(1) Land use totals include developed parcels that are connected to the collection system.



Legend

- Parcels
- River or Stream

General Plan Land-Use

- LDR
- MDR
- PD/MHP
- MDR/MHP
- MHDR
- HDR
- PD
- AG
- OS
- NC
- GC
- RC
- HSC
- PQ
- LI
- GI

0 0.25 0.5 Miles

GENERAL PLAN LAND USE

FIGURE 2.4

KING CITY
COLLECTION SYSTEM MASTER PLAN

As shown in Table 2.2, the largest land use category is industrial, which accounts for approximately 35 percent of the total current service area acreage. Residential land uses make up approximately 31 percent and commercial land uses make up approximately 11 percent of the total. Public land uses account for approximately 15 percent of the service area. Other land uses, such as planned developments account for the remaining 9 percent of the total service area.

2.3.2 Build-Out Land Use

At build out of the General Plan, the City will encompass approximately 2,162. Build-out is defined as complete development of the City limits. Table 2.3 summarizes the General Plan land uses for the entire City limits.

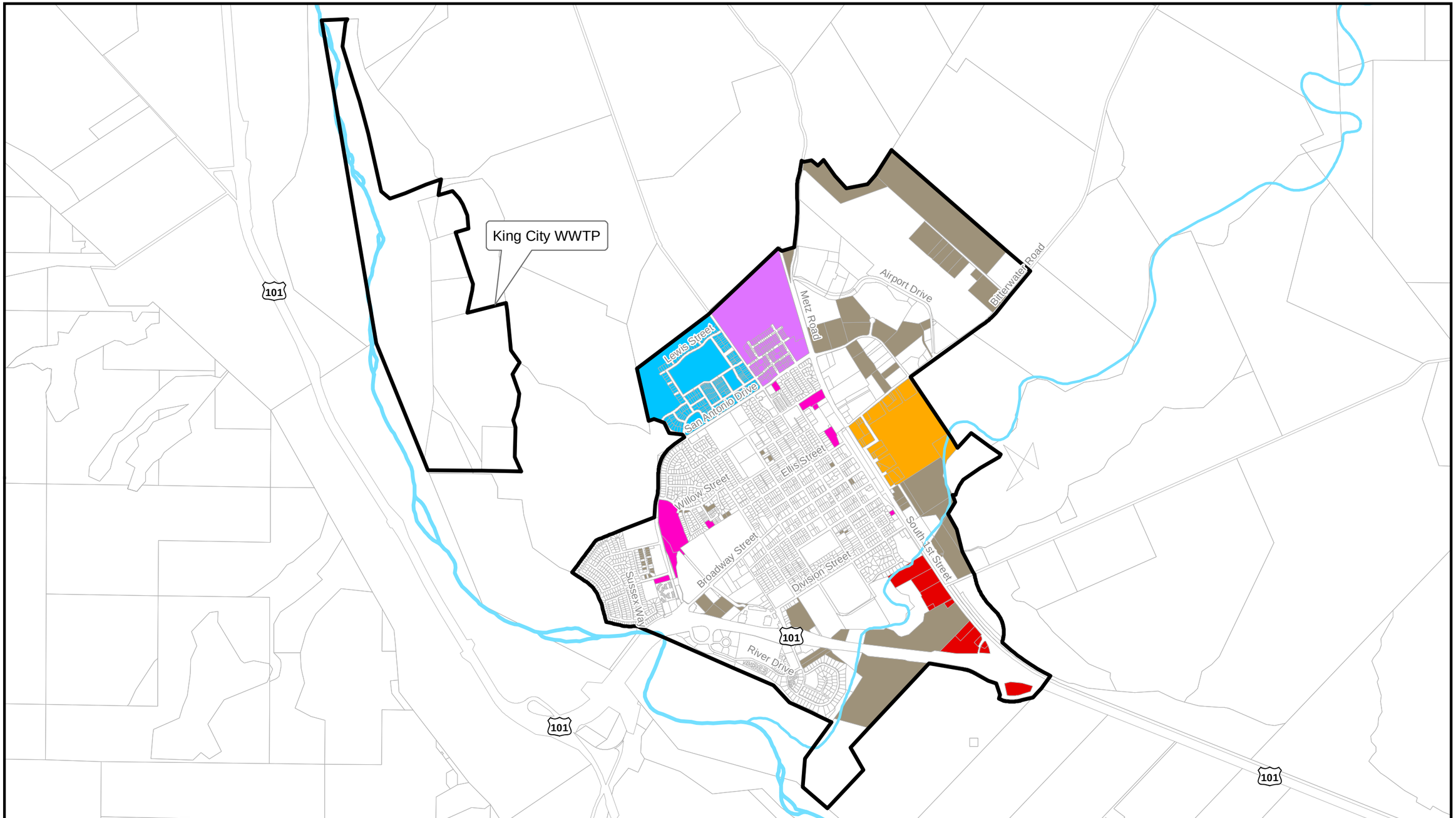
Table 2.3 Build Out Land Use
Collection System Master Plan
King City

Land Use Classification	Current Sewer Service Area Land Use: Land Use Area (acre)	Current Sewer Service Area Land Use: Percent of Total (%)
Residential		
Low Density	196.9	9.1
Medium Density	47.1	2.2
Medium Density/Mobile Home Park	2.2	0.1
Medium High Density	27.0	1.2
High Density	59.3	2.7
Subtotal	**332.5**	**15.4**
Commercial		
General	32.3	1.5
Highway Service	87.2	4.0
Neighborhood	8.3	0.4
Retail	32.1	1.5
Retail/Transitional	3.4	0.2
Subtotal	**163.2**	**7.6**
Industrial		
Light Industrial	381.8	17.7
General Industrial	126.7	5.9
Subtotal	**508.5**	**23.5**
Other		
Ag	20.6	1.0
Open Space	142.1	6.6
Planned Development	360.0	16.7
Planned Development/Mobile Home Park	19.4	0.9
Public/Quasi-Public Facility	615.3	28.5
Subtotal	**1,157.6**	**53.5**
Total	**2,161.8**	**100.0**

2.3.3 Known Developments, Annexations, and Infill

In general terms, the study area can be grouped into two mains categories in regards to development/land use. These are “Existing Developed Areas” and “Developable Areas and Annexations” as summarized below:

- **Existing Developed Areas:** All lands that are already developed and connected to the wastewater collection system are defined as existing developed areas for the purposes of this Master Plan. Wastewater flows associated with existing developed areas are assumed to remain unchanged throughout the planning horizon of this master plan.
- **Developable Areas and Annexations:** These areas will contribute to a net increase in wastewater flows into the collection system. These areas are further broken down subcategories, as described below:
  - Planned Developments and Annexations: City staff identified specific planned developments that could potentially connect to the City's wastewater collection system within the 20-year planning horizon, or by build-out (See Figure 2.5).
    - **Creek Bridge:** Creek Bridge is a residential development that is currently approximately 60 to 70-percent built out. An additional 170 units are expected to develop within this area within the next 10-years.
    - **Mills Ranch:** Mills Ranch is another existing residential development, which is roughly 10-percent built out. An addition 368 dwelling units are expected to be developed in this area within the next 10-years.
    - **Downtown Addition:** Downtown addition is a future development located south of Bitterwater Road and east of 1st Street. It will consist of a mixture of single and multifamily residential units. In total, approximately 650 new units are expected. Based on input from City staff, this area is expected to develop in Phase 2 (within the next 20-years).
    - **Highway 101 Commercial:** An approximately 35 acre piece of land located between Highway 101 and 1st Street is planned for commercial development in the future. Based on input from City staff, this area could develop in Phase 2 (within the next 20-years).
    - **Industrial Growth:** There is approximately 156 acres of undeveloped industrial land available for growth in the area east of Metz Road and west of Bitterwater Road. The City expects an influx in industrial development from medical marijuana production facilities in the coming years. For the purposes of this master plan, it was assumed that the industrial area would develop in Phase 2 (within the next 20-years).



Legend

- Study Area
- Parcels
- River or Stream

Developable Land

- Creekbridge
- Downtown Addition
- Mills Ranch
- Highway 101 Commercial
- Housing Element Infill
- Res., Com., and Ind. Infill

N

0 0.25 0.5 Miles

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DEVELOPABLE LAND WITHIN THE CITY SERVICE AREA

FIGURE 2.5

KING CITY
COLLECTION SYSTEM MASTER PLAN

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- **Annexations:** Three annexation areas may potentially connect to the City's system in the long term (after 20-years). These include the Silva area (2,400 units), the Smith Monterey area (320 units), and the San Bernabe area.

– Little Bear Community Services District (CSD): Little Bear CSD could potentially convey wastewater flows to the City in the future. For the purposes of this plan, the Little Bear CSD was assumed to connect initially in Phase 2 (within the next 20 years) and continue to expand through build-out. A total of 569 units are assumed to connect in Phase 2, and an additional 966 units could connect at build-out.

– Vacant Infill: For the purposes of this Master Plan, infill is defined as development of existing vacant parcels that lie within the existing wastewater service area. The City's housing element identified several infill parcels that could accommodate 396 dwelling units in total. In addition, there are approximately 109 acres of additional residential land use areas and 10 acres of commercial land use areas that could develop. Infill of these areas was assumed to occur in Phase 1 (within the next 10-years).

Wastewater flow projections for the City's developable land and annexation areas are provided in Chapter 4.

Chapter 3

FLOW MONITORING PROGRAM

This chapter defines the typical components of wastewater in a collection system and the temporary flow monitoring program conducted as part of this study. The data and results from the flow monitoring program are summarized and discussed.

3.1 FLOW MONITORING SITES AND RAIN GAUGES

As part of the Scope of Services for this Collection System Master Plan, Carollo Engineers, Inc. (Carollo) contracted with V&A Consulting Engineers (V&A) to conduct a temporary flow monitoring program within the King City (City) wastewater collection system. The purpose of the flow monitoring program was to assist in the development of design flow criteria, and to correlate actual collection system flows to the hydraulic model predicted flows. Flow monitoring data was also used to calibrate the collection system hydraulic model for dry weather and wet weather flow, and to help to identify areas of the system with the highest rates of infiltration/inflow (I/I). The temporary flow monitoring program was conducted for a period of almost 7 weeks from November 14, 2016 to December 29, 2016.

The “Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, May 2017” prepared by V&A summarizes the flow monitoring program. A copy of the report is included in Appendix A.

3.1.1 Flow Monitoring Sites and Tributary Areas

A total of ten open-channel flow meters were installed at locations selected by Carollo and the City. The meter sites were selected to best isolate and model the critical areas and subareas within the sewer system. The ten flow monitoring locations, as well as the area tributary to each site, are shown on Figure 3.1. Table 3.1 lists the flow monitoring locations and the diameters for the sewers where the meters were installed. Figure 3.2 provides a schematic illustration of the flow monitoring locations.

3.1.2 Flow Meter Installation and Flow Calculation

Hach Sigma 904 flow meters with submerged AV sensors were used for this project. Sigma 904 meters use a pressure transducer to collect depth readings and an ultrasonic Doppler sensor 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 flow meters. 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.

Table 3.1	Flow Monitoring Locations Collection System Master Plan King City		
Monitor Site	**Manhole ID**	**Pipe Diameter (in.)**	**Location**
1	N12	18	San Antonio Drive (west of North Mildred Avenue)
1B	MH300	10	San Antonio Drive and North Mildred Avenue
2	--	30	Near the WWTP
3A	T7 (North inlet)	12	Sandringham Street and Willow Street
3B	T7 (East inlet)	12	Sandringham Street and Willow Street
4	S17	18	Broadway Street (northeast of San Antonio Drive)
5	W2	16	River Drive (southeast of Broadway Circle)
6	MH520	18	Division Street and South Mildred Avenue
7	N16	18	San Antonio Drive (west of Van Etten Avenue)
8	MH439	14	San Antonio Drive (east of Metz Road)

V&A conducted an analysis of the data retrieved from each flow meter, and made adjustments as needed for calibration based on the field measurements, and to account for any sediment build up. The flow at each meter was then calculated at 5-minute intervals based on the continuity equation:

$$Q = V \times A$$

where,

Q = Pipeline flow rate, cfs

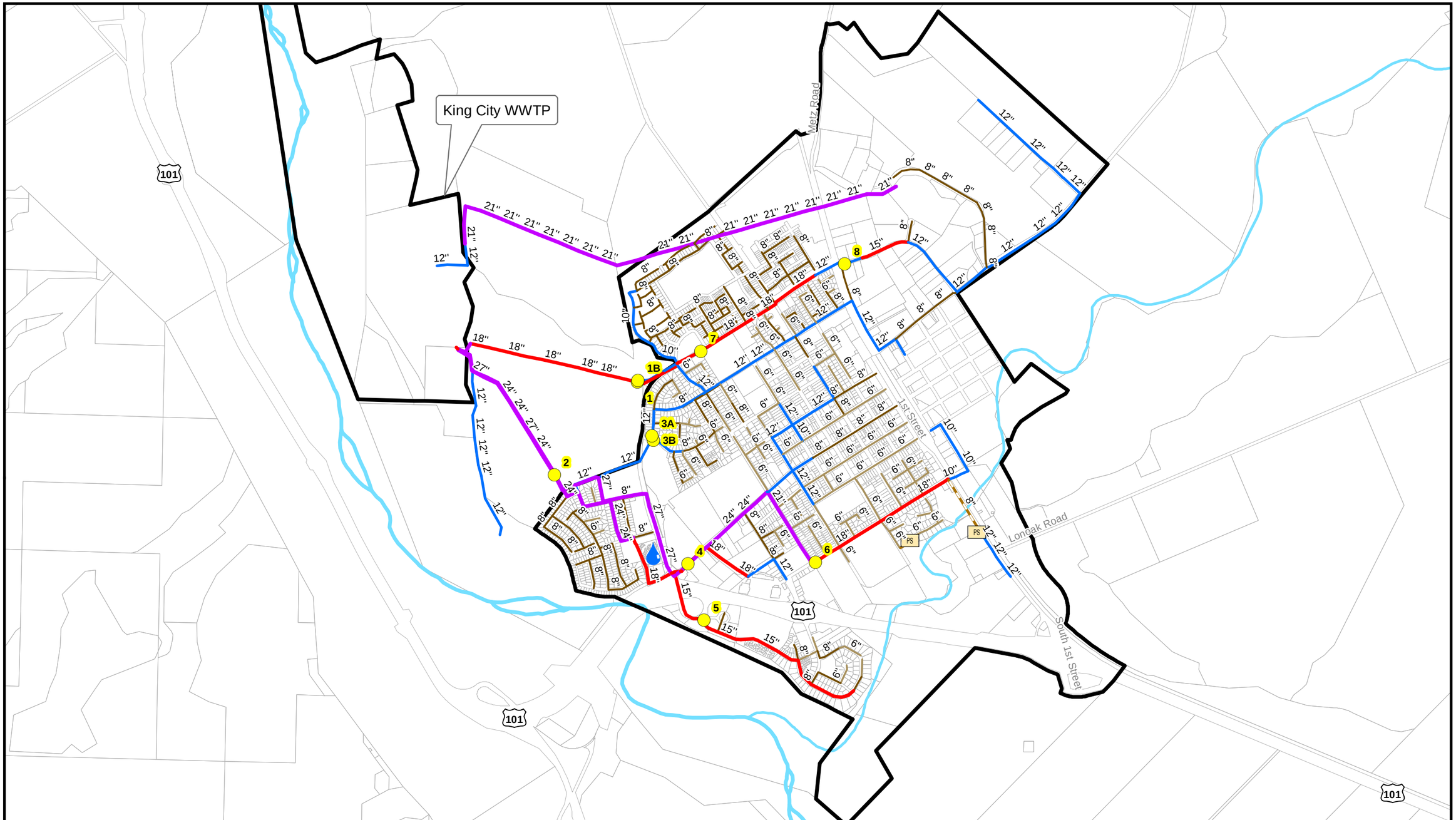
V = Average velocity, ft/s

A = Cross sectional flow area, sq ft

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

3.1.3 Rain Gauges

One rain gauge was installed by V&A as part of the flow monitoring program to capture rainfall that occurred throughout the study area. The rain gauge was installed near the intersection of San Antonio Drive and Amherst Drive, as shown on Figure 3.1.



Legend

- Study Area
- Parcels
- River or Stream
- Rain Gauge
- Flow Meter
- City Owned Lift Station
- Force Main

Gravity Sewer by Diameter

- 6"
- 8"
- 10"-14"
- 15"-18"
- 21" and Larger

N

0 0.25 0.5 Miles

TENPORARY FLOW MONITORING LOCATIONS

FIGURE 3.1

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COLLECTION SYSTEM MASTER PLAN

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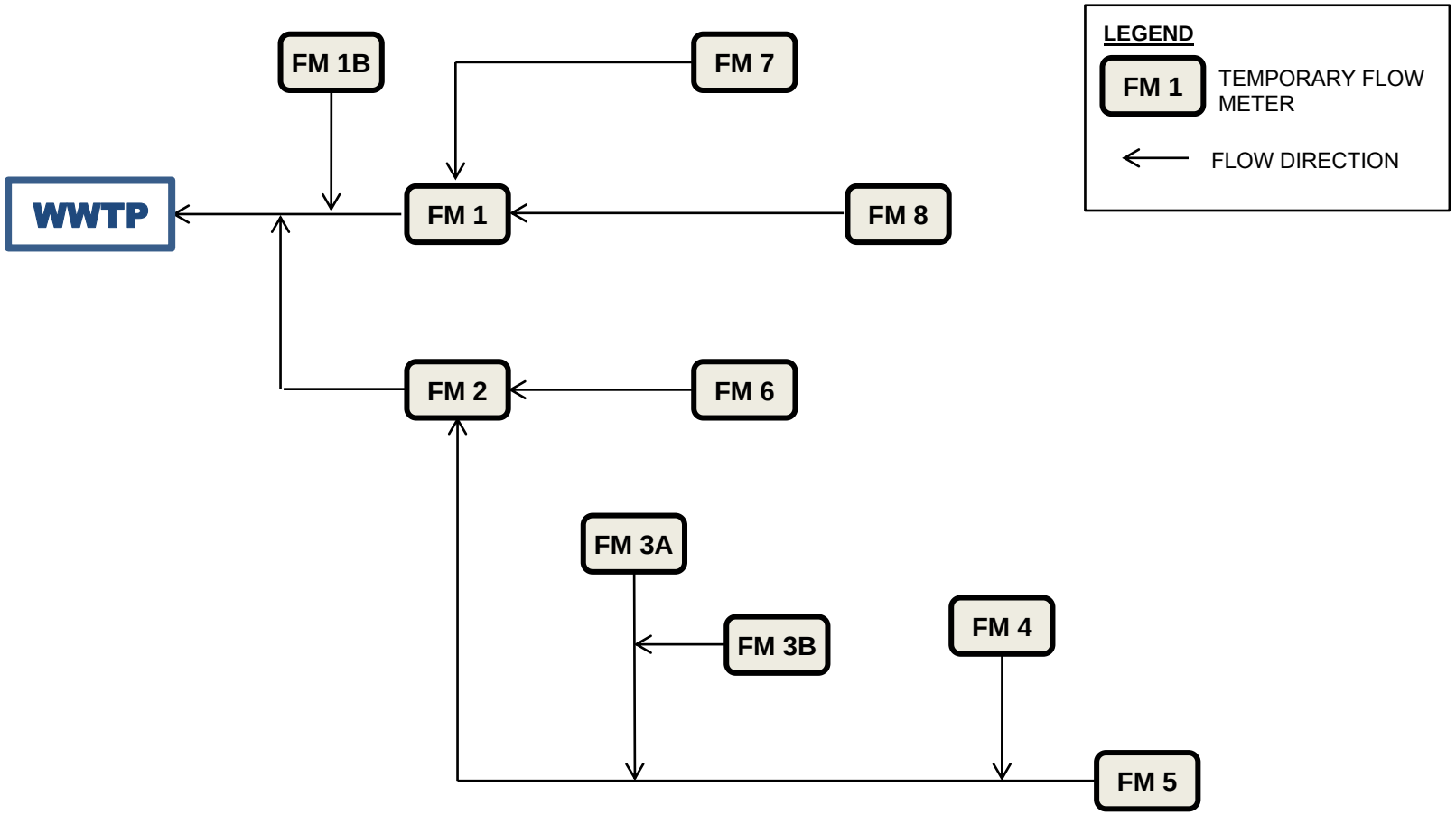

FLOW MONITORING SCHEMATIC

FIGURE 3.2

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COLLECTION SYSTEM MASTER PLAN

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3.2 WASTEWATER FLOW COMPONENTS

As a way to help the reader understand the 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. In general, wastewater consists of dry weather flow (DWF) and wet weather flow (WWF). DWF (or base flow) is flow generated by routine water usage in the residential, commercial, business and industrial sectors of the collection system.

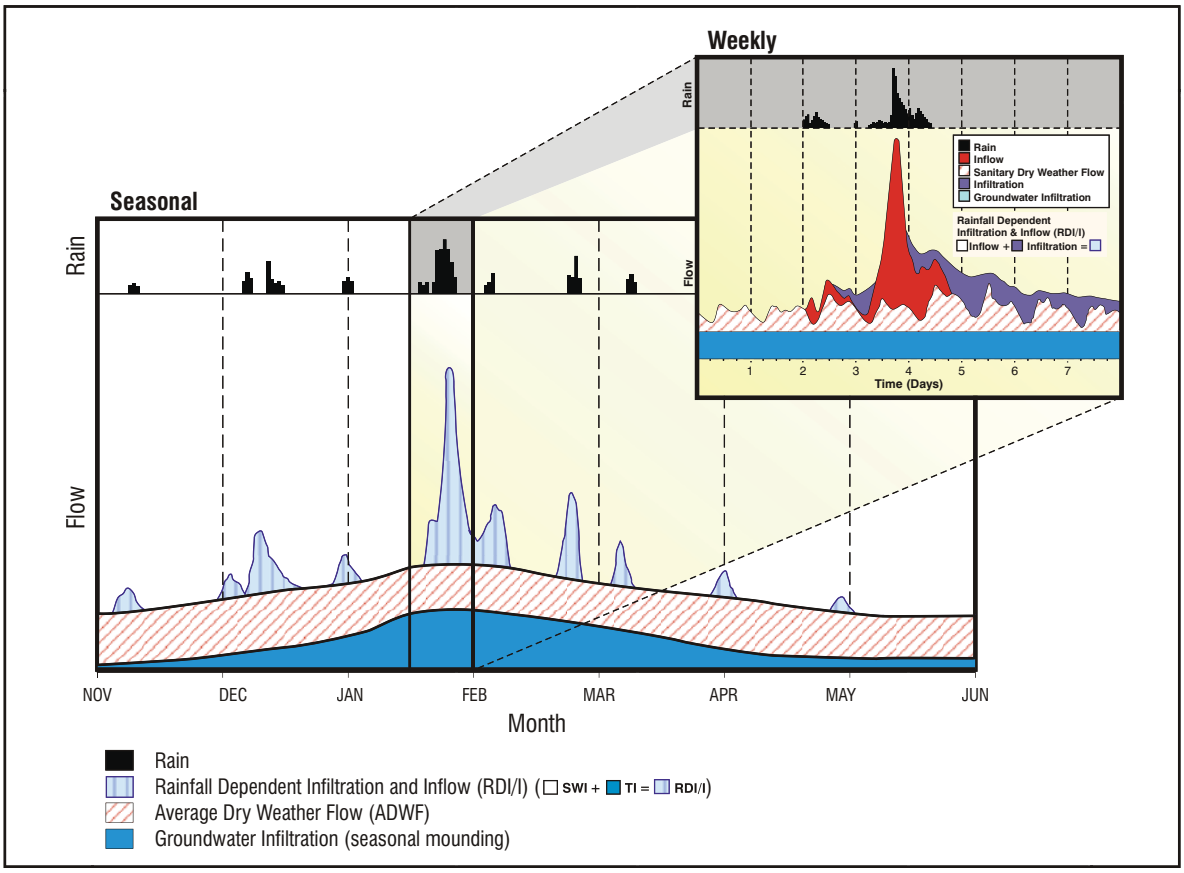
The other component of DWF is the contribution of dry weather groundwater infiltration (GWI) into the collection system. Dry weather GWI will enter the sewer system when the relative depth of the groundwater table is higher than the depth of the pipeline and when the susceptibility of the sanitary sewer pipe allows infiltration through defects such as cracks, misaligned joints, and broken pipelines.

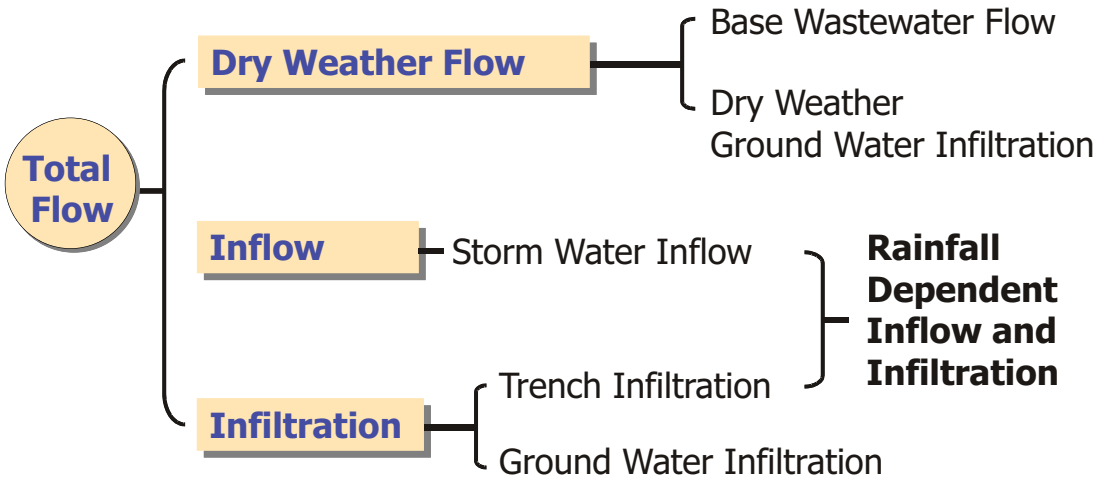
WWF includes storm water inflow, trench infiltration, and GWI. The storm water inflow and trench infiltration comprise the WWF component termed I/I. The response in the sewer system to rainfall is seen immediately (as with inflow) or within hours after the storm (as with infiltration).

The third element of WWF is GWI, which is not specific to a single rainfall event, but rather to the effects on the sewer system over the entire wet weather season. The depth of the groundwater table rising above the pipe invert elevation causes GWI. Sewer pipes within close proximity to a body of water can be greatly influenced by groundwater effects. As the groundwater table fluctuates over the wet weather season, this fluctuation is seen as a mounding effect in flow monitoring data. Figure 3.3 illustrates the various flow components, which are described in detail in the following sections.

3.2.1 Base Wastewater Flow

The base wastewater flow (BWF) is the flow generated by the City's customers. 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.





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

TYPICAL WASTEWATER FLOW COMPONENTS

FIGURE 3.3

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COLLECTION SYSTEM MASTER PLAN

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3.2.2 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.

3.2.3 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.

3.2.4 Groundwater Infiltration

GWI, one of the components of I/I, is associated with 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.

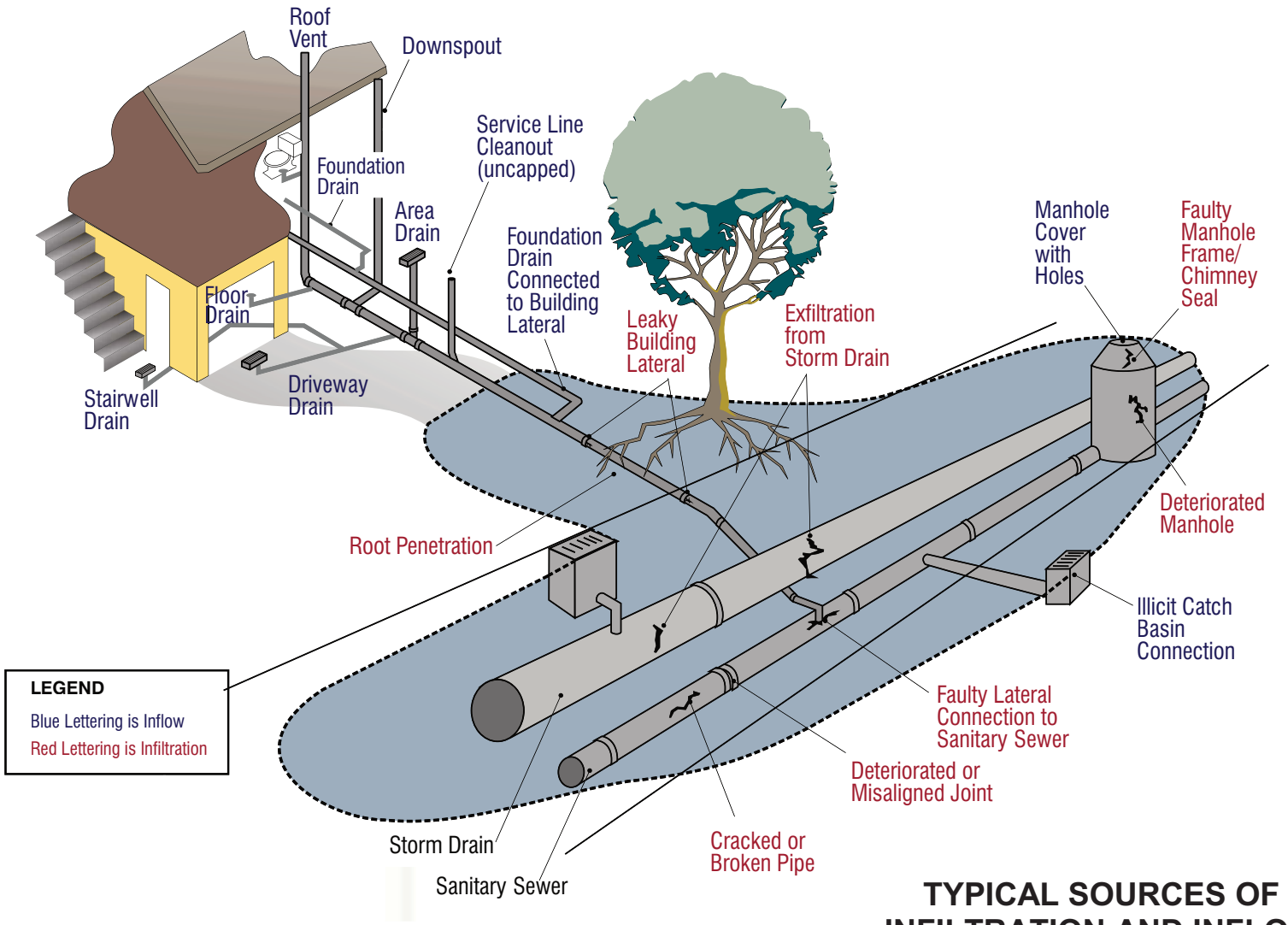
3.2.5 Infiltration and Inflow

All wastewater collection systems have some I/I, although the characteristics and severity vary by region and individual collection system. Some of the most common sources of I/I are shown on Figure 3.4. 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 a 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.

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 3.5. If too much I/I enters the sewer system such that the sewer system is operating at or above its capacity, sanitary sewer overflows (SSOs) could occur.

3.2.6 Peak Wet Weather Flow (Design 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 and lift stations. Therefore, the PWWF and the "Design Flow" are synonymous and will be used interchangeably throughout this report.



TYPICAL SOURCES OF INFILTRATION AND INFLOW

FIGURE 3.4

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COLLECTION SYSTEM MASTER PLAN

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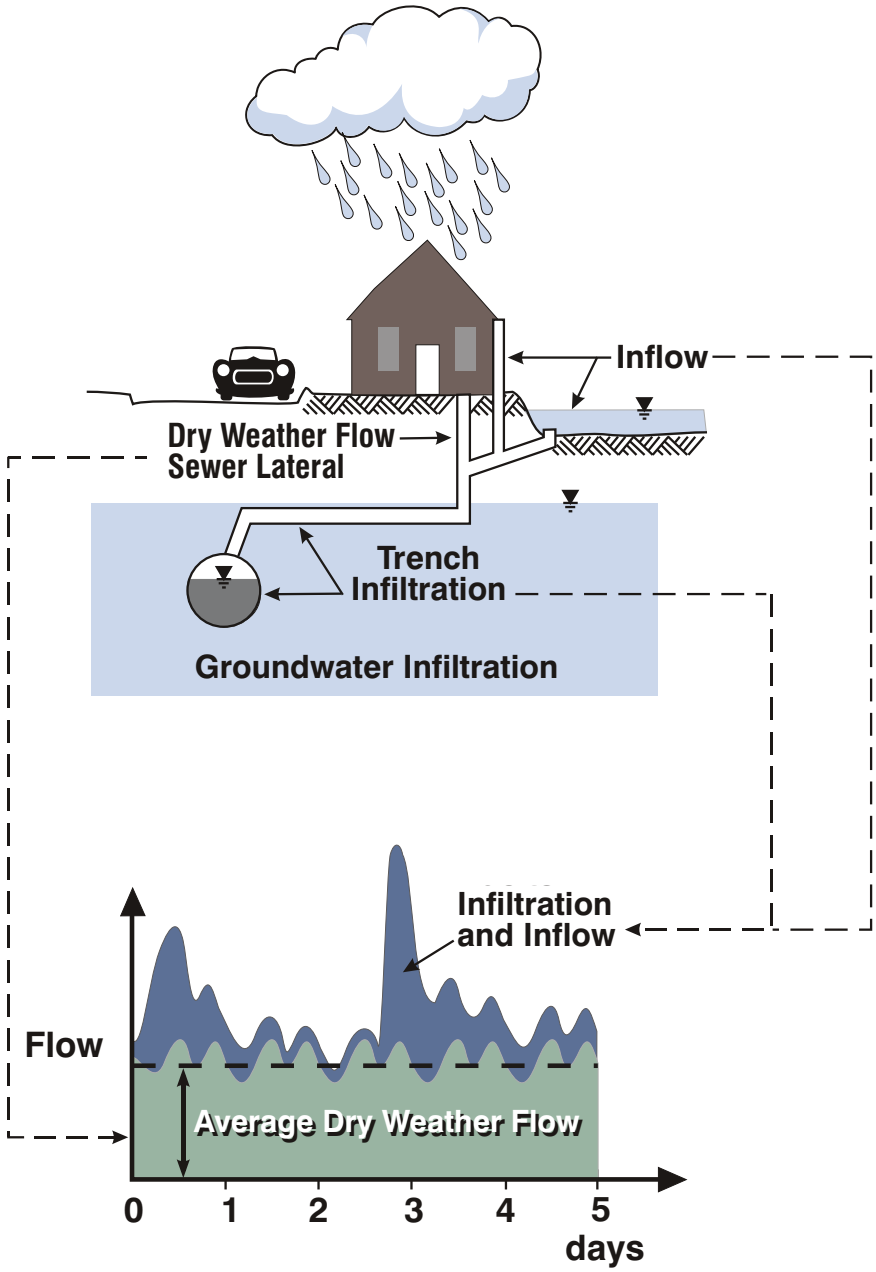

TYPICAL EFFECTS OF INFILTRATION AND INFLOW

FIGURE 3.5

KING CITY
COLLECTION SYSTEM MASTER PLAN

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3.3 FLOW MONITORING RESULTS

This section summarizes the results of the flow monitoring program, including dry weather flow data, rainfall data, and wet weather flow data. Data collected from Meter 2 is presented throughout this and other chapters as an example of the type of data and the results from the flow monitoring program. Refer to Appendix A for additional data summaries and other information associated with the remaining meter sites.

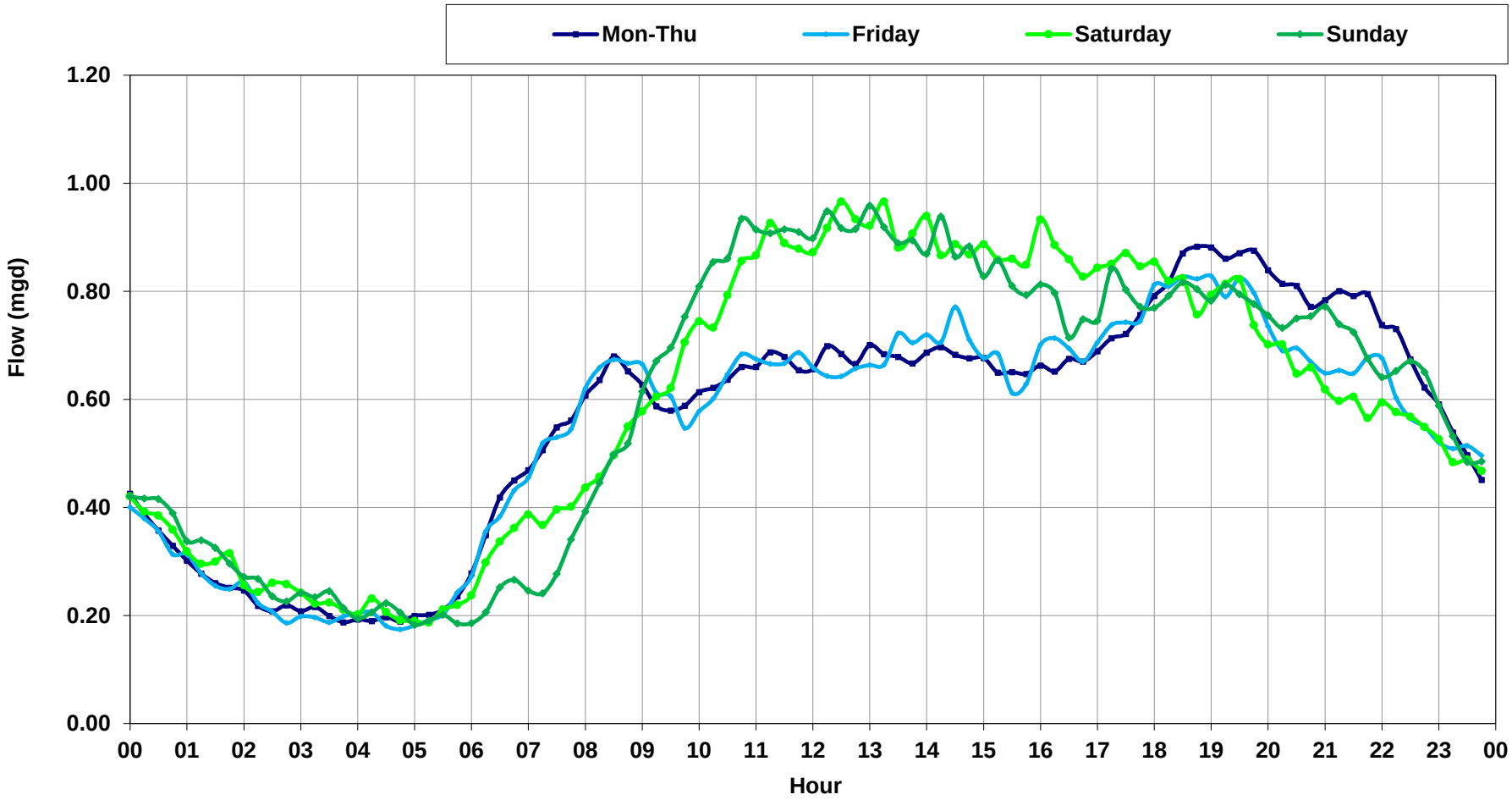
3.3.1 Dry Weather Flow Data

During the flow monitoring period, depth and velocity data were collected at each meter at 5-minute intervals. The 5-minute data was then aggregated to 15-minute data by V&A. 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.

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 dry weather flows. 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 3.6 illustrates a typical variation of weekday and weekend flow in the City, which is based on the data collection from Meter 2. Similar graphics associated with the remaining sites are included in Appendix A. Table 3.2 summarizes the dry weather flows at each meter.

3.3.2 Rainfall Data

There was one main rainfall event that occurred during the course of the flow monitoring period, as well as a few other very minor events. Figure 3.7 illustrates the total accumulation of rainfall over the course of the flow monitoring period. The total rainfall recorded over the duration of the flow monitoring period was 1.01-inches. The most significant rainfall event occurred on November 20, 2016 with a total of 0.52-inches of rain in a 24-hour period. The flow monitoring report prepared by V&A (Appendix A) classified this event as less than a 1-year, 24-hour event.



TYPICAL WEEKDAY VS. WEEKEND DRY WEATHER FLOW VARIATION (METER 2)

FIGURE 3.6

KING CITY
COLLECTION SYSTEM MASTER PLAN

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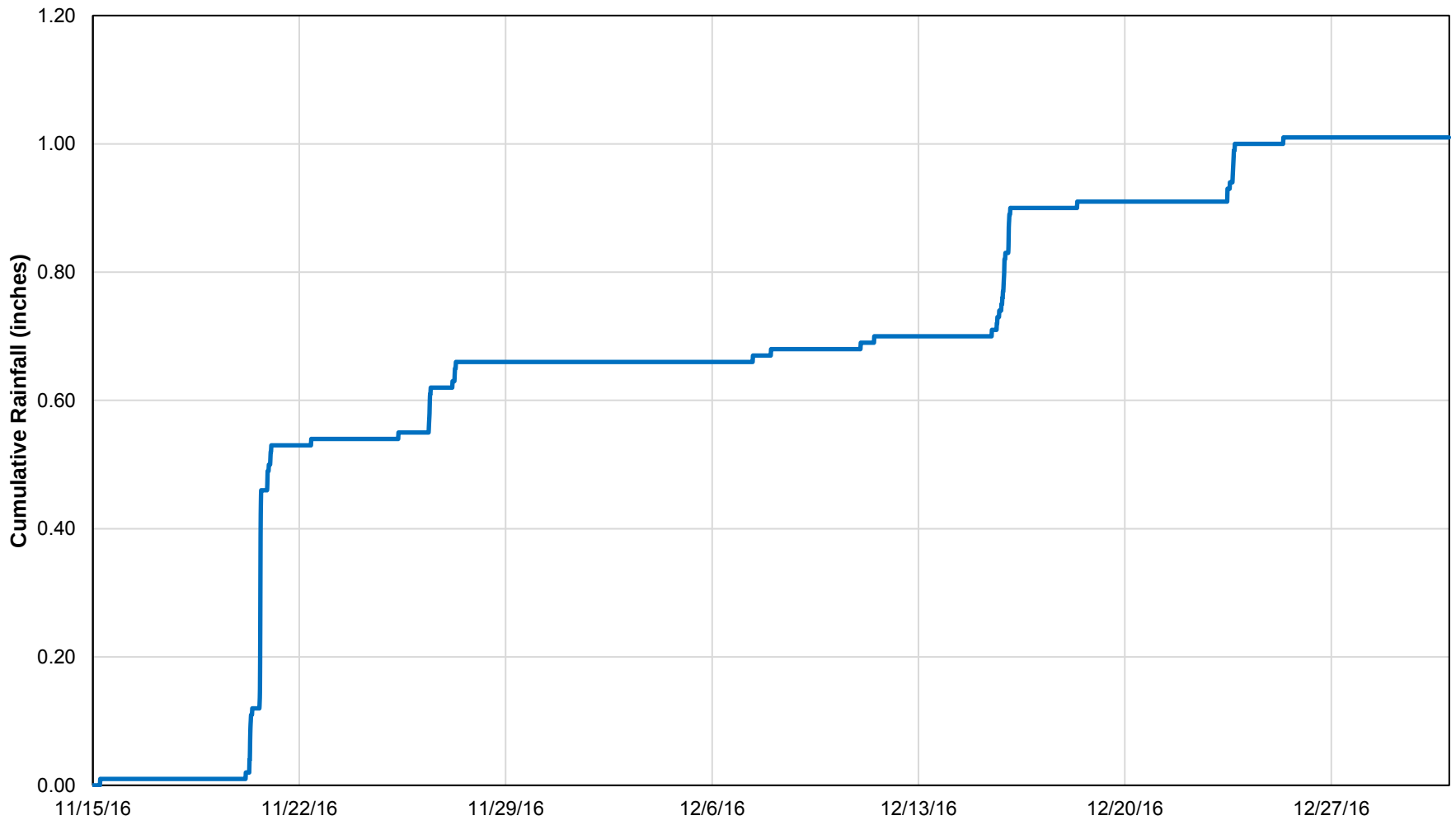

RAINFALL ACCUMULATION PLOT

FIGURE 3.7

KING CITY
COLLECTION SYSTEM MASTER PLAN

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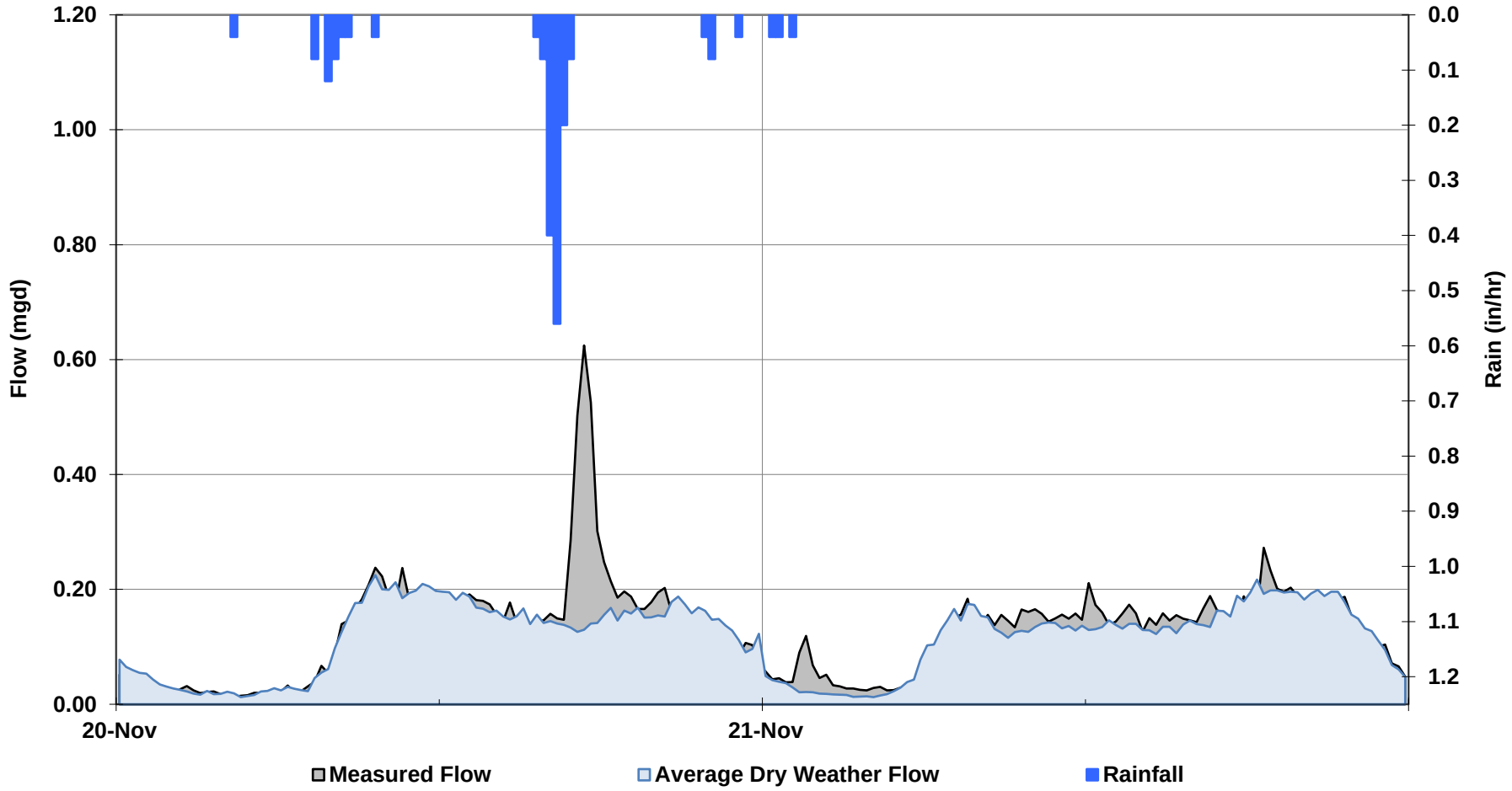
Table 3.2	Dry Weather Flow Summary Collection System Master Plan King City			
Monitor Site	**Weekday Dry Weather Flow (mgd)**	**Weekend Dry Weather Flow (mgd)**	**Overall Dry Weather Flow (mgd)**	**Weekend Weekday Ratio**
1	0.115	0.123	0.117	1.07
1B	0.017	0.019	0.018	1.12
2	0.560	0.598	0.571	1.07
3A	0.062	0.062	0.062	0.99
3B	0.040	0.044	0.041	1.09
4	0.078	0.073	0.076	0.94
5	0.107	0.106	0.107	0.99
6	0.094	0.097	0.095	1.03
7	0.059	0.047	0.055	0.79
8	0.010	0.004	0.008	0.34

Notes:
(1) Source: Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study (V&A, March 2017).
(2) Overall Dry Weather Flow = (5 x Weekday + 2 x Weekend)/7.

3.3.3 Wet Weather Flow Data

The flow monitoring data was also evaluated to determine how the collection system responds to wet weather events. As mentioned above, the flow monitoring program captured one significant rainfall event. The rainfall event that occurred on November 20, 2016 was associated with the largest I/I response during the flow monitoring period, and was the most appropriate to be used for I/I analysis and model calibration.

Figure 3.8 shows an example of the wet weather response at Meter 2 during the November 20, 2017 rainfall event. Figure 3.8 illustrates the volume of I/I that entered the system from the collection system upstream of Site 2. The light blue area is the base sanitary flow while the gray area is the measured flow from the flow monitoring period. As can be seen in the figure, discernible amounts of I/I do enter the system during wet weather events. Similar graphs were generated for the remaining monitoring sites can be found in Appendix A. In general, the City's collection system exhibited a very minor I/I response with the exception of the industrial area of the system. The flow monitoring data from sites 1, 7, and 8 all show a significant inflow response following the November 20, 2016 rainfall event. These meters also showed a similar spike in flow during dry weather conditions as well. It is possible that an industrial customer has an uncovered drain or some other wash down facility that is connected to the sewer system that is responsible for these spikes in flow. Additional field reconnaissance and/or smoke testing in the industrial area is recommended to identify the source of the inflow into the system.



EXAMPLE WET WEATHER FLOW RESPONSE

FIGURE 3.8

KING CITY
COLLECTION SYSTEM MASTER PLAN

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An important metric to quantify the severity of the system's I/I response is the peak measured flow rate. As shown in Table 3.3, the measured peak flow rate to dry weather flow ratio ranged from 1.69 in Meter 2 to 4.13 in Meter 8. It should be noted, however, that the peak flow rates presented in Table 3.3 are for the November 20, 2016 event only, which as previously mentioned, is classified as less than a 2-year event. Therefore, the peak flow rate during the design storm event will be higher.

Table 3.3 Flow Monitoring Summary
Collection System Master Plan
King City

Site Name	Diameter (in)	ADWF (mgd)	Peak Flow (mgd)	Peaking Factor
Site 1	18	0.117	0.225	1.92
Site 1b	10	0.018	0.038	2.11
Site 2	27	0.571	0.966	1.69
Site 3a	12	0.062	0.111	1.79
Site 3b	12	0.041	0.081	1.98
Site 4	18	0.076	0.161	2.12
Site 5	15	0.107	0.208	1.94
Site 6	18	0.095	0.194	2.04
Site 7	18	0.055	0.153	2.78
Site 8	15	0.008	0.033	4.13

PLANNING CRITERIA AND DESIGN FLOWS

The capacity of King City's (City's) wastewater collection system was evaluated based on the planning criteria defined in this chapter. The planning criteria address the collection system capacity, gravity sewer pipe slopes, and maximum allowable depth of flow within a sewer. This chapter also summarizes the existing and future design flows.

4.1 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 flow conditions, 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.

4.1.1 Manning Coefficient (n)

The manning 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 coefficient typically ranges between 0.011 and 0.017, with 0.013 being a representative value used for system planning purposes. For this study, a manning "n" factor of 0.013 was assigned to all existing sewer collection system lines in the hydraulic model, and then refined as necessary during model calibration to accurately simulated field measured levels and velocities.

4.1.2 Flow Depth Criteria (d/D)

The primary criterion used to identify capacity deficient sewers or to size new sewer improvements is the maximum flow depth to pipe diameter ratio (d/D). The d/D value is defined as the depth of flow (d) in a pipe during peak (design) flow conditions divided by the pipe's diameter (D). Based on Carollo Engineers, Inc.'s (Carollo's) experience, City staff input, and industry standards, the following criteria were used:

- **Flow Depth 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 d/D ratio 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, water levels (hydraulic grade line) were allowed to rise to a maximum depth (d/D) of 75 percent during average dry weather flow (ADWF) conditions and 90 percent during peak wet weather flow (PWWF) conditions.

A capacity deficient sewer (i.e., system bottleneck) raises 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 for New Sewers:** When designing sewer pipelines, it is common practice to adopt variable 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. Table 4.1 summarizes the criteria for the evaluation of existing sewers and for sizing new trunk lines. For pipelines less than 12-inches in diameter the maximum d/D value is 0.5 or 50 percent of the pipeline depth. For pipelines 12- to 18-inches in diameter, the maximum d/D is 0.67, and for pipelines larger than 18-inches in diameter the maximum d/D value is 0.75.

Table 4.1 Maximum Flow Depth Criteria
Collection System Master Plan
King City

Maximum Flow Depth for Existing Sewers	
Average Dry Weather Flow:	Maximum d/D = 0.75
Peak Wet Weather Flow:	Maximum d/D = 0.9
Maximum d/D for New Sewers	
Pipe Diameter (inches)	Maximum d/D Ratio (during Peak Flows)
Less than 12	0.50
12 to 18	0.67
Larger than 18	0.75

4.1.3 Design Velocities and Minimum Slopes

In order to minimize the settlement of sewage solids, it is standard practice in the design of gravity sewers to specify that a minimum velocity of 2 feet per second (ft/s) be maintained when the pipeline is half-full. At this velocity, the sewer flow will typically provide self-cleaning for the pipe. Due to hydraulics of a circular conduit, velocity of half-full flow in pipes approaches the velocity of nearly full flow in pipes.

Table 4.2 lists the recommended minimum slopes and their corresponding maximum flows for maintaining self-cleaning velocities (equal to or greater than 2 ft/s) when the pipe is flowing at its maximum depth (d/D ratio).

Table 4.2 Minimum Slope for New Pipes Collection System Master Plan King City

Pipe Diameter (inches)	Minimum Slope[(1)(2)] (feet/feet)	Calculated Flow at Maximum d/D[(2)(3)]	
		d/D	Maximum Flow (mgd)
6	0.0050	0.50	0.127
8	0.0033	0.50	0.226
10	0.0025	0.50	0.353
12	0.0019	0.67	0.796
15	0.0014	0.67	1.24
18	0.0011	0.67	1.79
21	0.0009	0.75	2.84
24	0.0008	0.75	3.70
27	0.0007	0.75	4.68
30	0.0006	0.75	5.79
36	0.0006	0.75	9.65
42	0.0006	0.75	14.56

Notes:
(1) Recommended minimum slope for flows at a velocity greater than or equal to 2 ft/s.
(2) Manning's n = 0.013.
(3) Calculated flow is determined using the minimum slope and maximum allowable d/D from Table 4.1.

4.1.4 Changes in Pipe Size

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the 0.8 depth point of both sewers at the same elevation. For planning purposes and designing new pipes, and in the absence of field data, sewer crowns were matched at the manholes.

4.2 LIFT STATIONS AND FORCE MAINS

Industry standard practice is to require that sewage lift stations have sufficient capacity to pump the PWWF with the largest pump out of service (firm capacity).

Force main piping should be sized to provide a minimum velocity of 3 ft/s at the design flow rate of the lift station and no more than 8 ft/s. For the determination of head loss, the Hazen Williams Equation is used with a C-factor of 120. These factors are typical for sewer system master planning purposes.

4.3 DESIGN FLOWS

This section summarizes the historic flows measured at the City's Wastewater Treatment Plant (WWTP) and presents the calculation of the design flows used to model the existing and future sewer collection system.

4.3.1 Historic WWTP Flows

In addition to the flow monitoring program, summarized in Chapter 3, this project reviewed historical influent flow data at the WWTP from 2005 to 2016 (the last year in which a full years' worth of data was available) to help establish wastewater flow criteria.

Flow data from 2008 through December 2016 are summarized in Table 4.3. The historical average daily flows measured at the WWTP are reported in the City's monthly monitoring reports (MMRs) as both influent and effluent flow. From January 2008 through October 2016, the data was analyzed separately for non-drought years (2008 through 2011) and drought years (2012 through 2016). As expected, the non-drought years generally indicate a higher Average Annual Flow (AAF) than in the drought years, which was likely due to water conservation efforts. Based on a review of the historical plant flow data, it was determined that the City's Average Dry Weather Flow (ADWF), which was calculated as the average flow from May through September, was very similar to the AAF (about 2.5-percent higher on average). Therefore, for the purposes of this Master Plan, AAF and ADWF are essentially equal, and the two terms will be used synonymously throughout the remainder of this report.

As shown in Table 4.3, the City's AAF for years 2008-2011 (non-drought years) was determined to be 0.86 million gallons per day (mgd). This flow value was assumed to be the existing ADWF for the purposes of this Master Plan.

Table 4.3	Historical Flow Analysis Collection System Master Plan King City	
	Year	**AAF, mgd**
	2008	0.861
	2009	0.871
	2010	0.877
	2011	0.830
	2012	0.848
	2013	0.861
	2014	0.871
	2015	0.826
	2016	0.828
Non-Drought (2008-2011) Average		**0.860**
Drought (2012-2016) Average		**0.847**

4.3.2 Wastewater Flow Factors

In order to develop wastewater flow projections and allocate future flows to the collection system, relationships between land use and wastewater generation were developed. These relationships, called wastewater flow factors are established based on the average wastewater flow generated for each existing land use type. These wastewater flow factors were established to project the estimated average dry weather flow through build out of the City's service area.

Unit flow factors provide a means to estimate flow per acre for each land use category. Wastewater unit flow factors, expressed in gallons per day per acre (gpd/ac), are applied to land use acreage for calculating average day flow generated from a particular land use type. A unit flow factor was developed for each of the City's existing land use classifications. The resulting flow was entered in the sewer system hydraulic model.

Flow factors for residential areas typically range between 250 to 4,000 gallons per day per acre (gpd/ac), and commercial and industrial areas may range from 600 to 3,000 gpd/ac or higher. Open space and agriculture land use types were assumed to generate negligible amounts of wastewater flow. For King City, the developed flow factors ranged from 550 gpd/ac for existing industrial land uses to 2,000 gpd/ac for multi-family land uses, as shown on Table 4.4.

Table 4.4 Wastewater Flow Factors Collection System Master Plan King City			
Land Use Type	**Existing Developed Acreage**	**Wastewater Flow Factor, gpd/ac**	**Wastewater Flow, mgd**
Single Family	327	1,200	0.39
Multi Family	59	2,000	0.12
Commercial	103	750	0.08
Industrial	342	550	0.19
Institutional/Governmental	144	600	0.08
Total	**975**	**-**	**0.86**

4.3.3 Existing and Projected Average Dry Weather Flow

Developing an accurate estimate of the future quantity of wastewater generated at build out of the collections system is an important step in maintaining and sizing sewer system facilities, for both existing conditions and future developments. Flow projections were developed starting from current flows and anticipated community growth information based on current and future land use information from the following sources:

- King City General Plan (King City, 1998)
- Housing Element 2015-2023 (King City, 2015)
- Wastewater Master Plan (Boyle, 2007)
- Information provided by staff at Little Bear CSD
- Aerial analysis via GIS

Table 4.5 summarizes the analysis of flow projection by growth area. As shown in Table 4.5, the City's ADWF is projected to reach 1.6 mgd within the next 20-years, and 3.01 mgd through build out.

4.3.4 Design Storm

Design storms are rainfall events used to analyze the performance of a collection system under extreme wet weather events. 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.

Table 4.5 Flow Projections
Collection System Master Plan
King City

Category	Known Residential Development			Other Developments			Flow Summary (mgd)	Phasing			Flows by Phase (mgd)		
	No. of Dwelling Units	Flow Factor (gpd/DU)	Flow (mgd)	Aerial Analysis (acres)	Flow Factor (gpd/ac)	Flow (mgd)		Phase 1 (2017-27)	Phase 2 (2027-37)	Phase 3 (Post 2037)	Phase 1 (2017-27)	Phase 2 (2027-37)	Phase 3 (Post 2037)
Existing	--	--	--	--	--	--	**0.862**	**--**	--	--	**0.862**	**0.000**	**0.000**
Existing Development	--	--	--	--	--	--	0.862	x			0.862	0.000	0.000
Infill Development	**396**	**--**	**0.075**	118.1	--	**0.137**	**0.213**	--	--	--	**0.213**	**0.000**	**0.000**
Housing Element, R-3	13	190	0.002	--	--	--	0.002	x	--	--	0.002	--	--
Housing Element, R-4	334	190	0.063	--	--	--	0.063	x	--	--	0.063	--	--
Housing Element, C-N	16	190	0.003	--	--	--	0.003	x	--	--	0.003	--	--
Housing Element, C-2	33	190	0.006	--	--	--	0.006	x	--	--	0.006	--	--
Additional Residential (P-D)	--	--	--	108.5	1,200	0.130	0.130	x	--	--	0.130	--	--
Commercial	--	--	--	9.6	750	0.007	0.007	x	--	--	0.007	--	--
Creek Bridge (Including Arboleta)	**170**	**--**	**0.032**	--	--	--	0.029	--	--	--	**0.032**	**0.000**	**0.000**
Single Family	153	190	0.029	--	--	--	0.003	x	--	--	0.029	--	--
Multi Family	17	180	0.003	--	--	--	0.070	x	--	--	0.003	--	--
Mills Ranch	368	--	0.070	--	--	--	0.062	**--**	--	--	**0.070**	**0.000**	**0.000**
Single Family	328	190	0.062	--	--	--	0.007	x	--	--	0.062	--	--
Multifamily	40	180	0.007	--	--	--	0.029	x	--	--	0.007	--	--
Downtown Addition	650	--	0.120	--	--	--	0.120	--	--	--	**0.000**	**0.120**	**0.000**
Single Family	175	190	0.033	--	--	--	0.033	--	x	--	--	0.033	--
Multifamily	346	180	0.062	--	--	--	0.062	--	x	--	--	0.062	--
Multifamily Rental	8	180	0.001	--	--	--	0.001	--	x	--	--	0.001	--
Condominium	121	190	0.023	--	--	--	0.023	--	x	--	--	0.023	--
Highway 101 Commercial	--	--	--		--		0.026	--		--	**0.000**	**0.026**	**0.000**
Highway 101 Commercial	--	--	--	35.3	750	0.026	0.026	--	x	--	--	0.026	--
Undeveloped Industrial	**--**	**--**	**--**		**--**		**0.156**	--		--	**0.000**	**0.156**	**0.000**
General	--	--	--	109.5	1,000	0.110	0.110	--	x	--	--	0.110	--
Light	--	--	--	46.3	1,000	0.046	0.046	--	x	--	--	0.046	--
Annexations	**2,720**	**--**	**1.317**	--	--	--	--	--	--	--	**0.000**	**0.000**	**1.317**
Silva	2,400	190	0.456	--	--	--	--	--	--	x	--	--	0.456
Smith-Monterey	320	190	0.061	--	--	--	--	--	--	x	--	--	0.061
San Bernabe	--	--	0.800	--	--	--	--	--	--	x	--	--	0.800
Little Bear CSD	**1,535**	**--**	**0.215**	--	--	--	--	--	--	--	**0.000**	**0.080**	**0.135**
Pine Canyon (Existing)	398	140	0.056	--	--	--	--	--	x	--	--	0.056	--
Pine Canyon (Future Planned)	54	140	0.008	--	--	--	--	--		x	--	--	0.008
Royal Estates (Existing)	82	140	0.011	--	--	--	--	--	x	--	--	0.011	--
Royal Estates (Septic)	76	140	0.011	--	--	--	--	--	x	--	--	0.011	--
Royal Estates (Will Serve)	13	140	0.002	--	--	--	--	--	x	--	--	0.002	--
Morisoli (Future Planned)	198	140	0.028	--	--	--	--	--	--	x	--	--	0.028
Lot 71 (Future Planned)	714	140	0.100	--	--	--	--	--	--	x	--	--	0.100
Total													
Subtotal by Phase											**1.176**	**0.382**	**1.452**
Grand Total by Phase											**1.176**	**1.558**	**3.010**

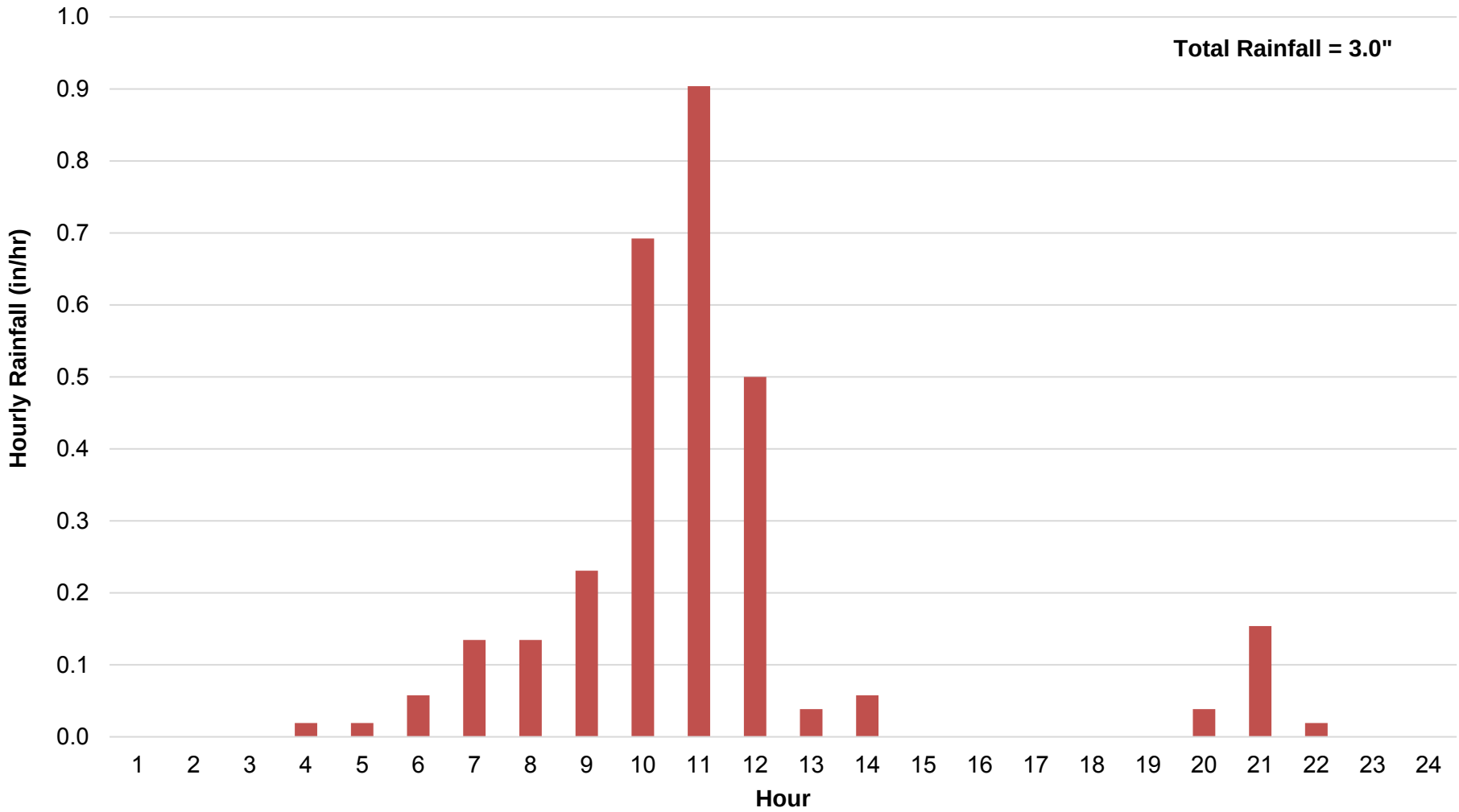
Duration is the length of time in which the rainfall occurs. It is industry standard in California to use the 10-year, 24-hour design storm for analyzing wastewater collection system performance during PWWF conditions. A 10-year, 24-hour storm in King City would have a total volume of 3 inches. Once 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 the duration of the storm. This can be accomplished either by using a synthetic rainfall distribution (developed by the Natural Resource Conservation Service (NRCS)) or using the rainfall distribution for a large historic event of similar volume and duration. The City's design storm was distributed using a rainfall pattern similar to that of the November 18, 2010 storm event, which had a peak intensity of approximately 0.9 inches/hour. Figure 4.1 shows the 10-year, 24-hour design storm.

4.3.5 Existing and Projected Peak Wet Weather Flow

The PWWF is the highest observed hourly flow that occurs following the design storm event. Wet weather infiltration and inflow (I/I), which occurs during and after rainfall events, increases flows in the collection system. PWWF is typically used for designing sewers and lift stations. Therefore, the PWWF is the design flow for the purposes of this study. The City's sewers and lift stations were evaluated based on their capacity to convey the PWWF.

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. Detailed information regarding the calibration of the City's hydraulic model is provided in Chapter 5. 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 I/I rate of 1,000 gallons per day per acre (gpd/ac). For the planned annexations and Little Bear Community Services District (Little Bear CSD), a peaking factor of 3.0 was applied to the projected ADWF.

Table 4.6 presents a summary of the existing and future ADWF and PWWF. In addition to the build out ADWF, Table 4.6 includes the existing and build out PWWFs. As shown in Table 4.6, City's ADWF is projected to increase 3.5 times from 0.86 mgd to 3.01 mgd by build out, whereas the PWWF is projected to increase from 4.36 mgd to about 10.03 mgd by build out (an increase of approximately 2.3 times). Therefore, the City's PWWF to ADWF peaking factor is projected to decrease from roughly 5.1 to 3.3, which is typical for sanitary sewer collection systems. Newer sewers tend to have less I/I 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.



10-YEAR 24-HOUR DESIGN STORM

FIGURE 4.1

KING CITY
COLLECTION SYSTEM MASTER PLAN

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CALIFORNIA

Table 4.6	**Current and Projected Wastewater Flows Collection System Master Plan King City**		
Year	**ADWF(1)**	**PWWF(2)**	**Peaking Factor**
Existing (2017)	0.86	4.36	5.07
Phase 1 (2027)	1.18	5.36	4.54
Phase 2 (2037)	1.56	5.67	3.63
Phase 3 (Post 2037)	3.01	10.03	3.33

Notes:
(1) ADWF = Average Dry Weather Flow.
(2) PWWF = Peak Wet Weather Flow.

COLLECTION SYSTEM FACILITIES AND HYDRAULIC MODEL

This chapter describes the development and calibration of King City's (City's) wastewater collection system hydraulic model. A description of the City's previous hydraulic model, the advantages of the newer modeling software being used for the Master Plan, and an outline of the steps used to build the model are provided. A detailed summary of the hydraulic model calibration steps, standards, and results for both dry weather and wet weather conditions is also provided.

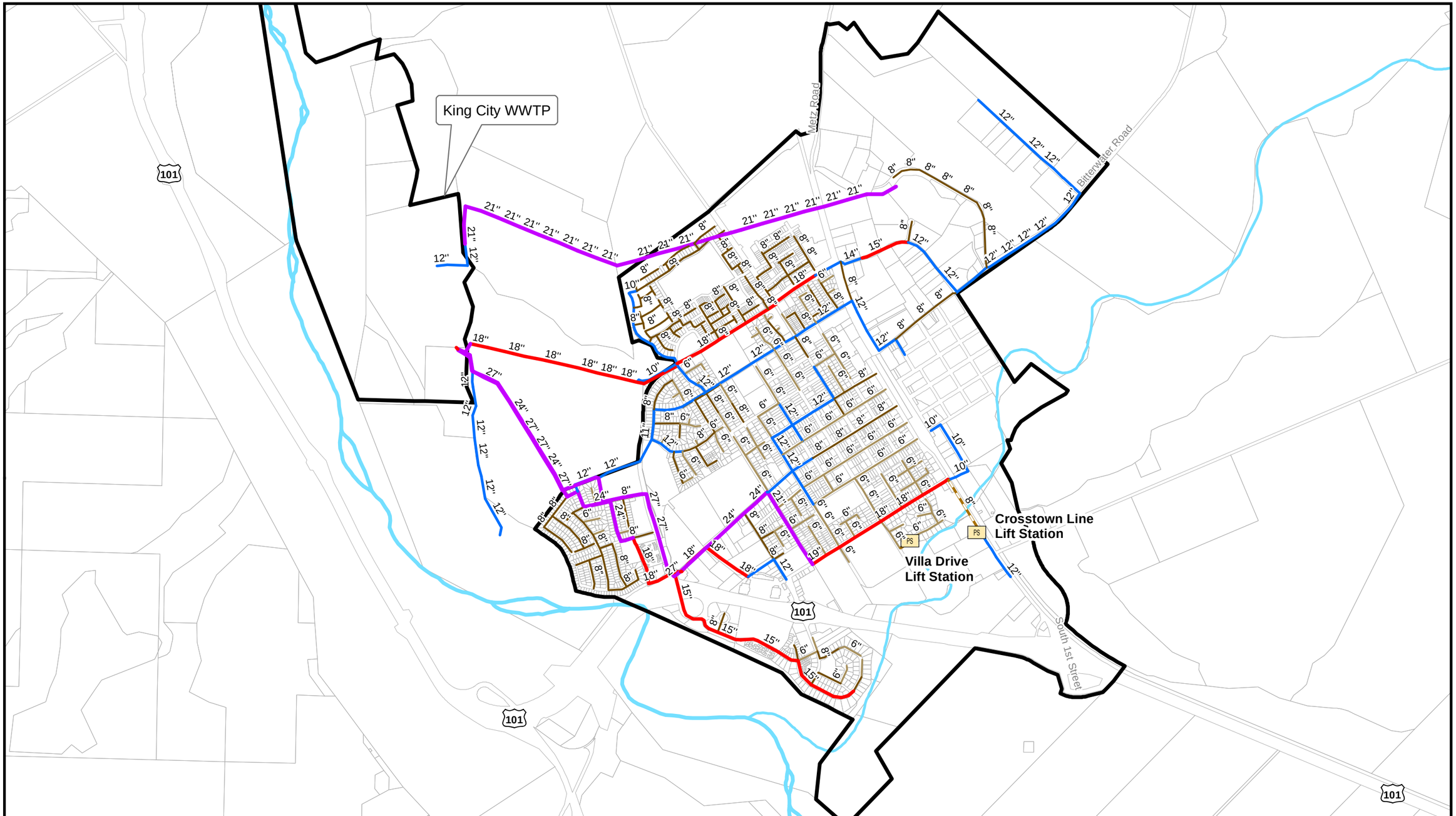
5.1 COLLECTION SYSTEM FACILITIES

The City's collection system consists of sewer mains, trunk sewers, lift stations, and force mains that collect and convey wastewater to the City's Wastewater Treatment Plant (WWTP), which is located along the east bank of the Salinas River on Cemetery Road. Figure 5.1 presents the City's existing wastewater collection system.

5.1.1 Gravity Collection System

The City's existing wastewater collection system is comprised of approximately 32.2 miles of gravity collection system pipe ranging in size from 6-inches to 27-inches in diameter. Table 5.1 presents a summary by diameter of the known sewers in the collection system. As shown in Table 5.1, roughly 52 percent of the system is 8-inches in diameter and smaller, with the majority of the system (approximately 31 percent) being 8-inches in diameter.

The City recently completed construction of the new Crosstown Line project, which consists of approximately 2.9 miles of 12-inch to 27-inch diameter gravity main, a new lift station, and 1,220 linear feet (LF) of 8-inch diameter force main. The Crosstown line will convey future flow from the southeast corner of the City limits northwest to the WWTP, as well as conveying flows from existing customers within the current service area.



Legend

- Study Area
- Parcels
- River or Stream
- City Owned Lift Station
- Force Main

Gravity Sewer by Diameter

- 6"
- 8"
- 10"-14"
- 15"-18"
- 21" and Larger

Miles
0 0.25 0.5

EXISTING SANITARY SEWER COLLECTION SYSTEM

FIGURE 5.1

KING CITY
COLLECTION SYSTEM MASTER PLAN

carollo

KING CITY CALIFORNIA

Table 5.1 Collection System Gravity Pipeline Summary Collection System Master Plan King City		
Diameter (inches)	**Length (feet)**	**Percent of System (by length)**
6	36,010	21.2
8	52,460	30.9
10	4,970	2.9
12	29,600	17.4
14	450	0.3
15	6,010	3.5
18	13,540	8.0
21	11,110	6.5
24	8,290	4.9
27	7,430	4.4
Total (feet)	**169,870**	**100.0**
Total (miles)	**32.2**	**100.0**

Notes:
(1) Source: King City GIS database.

5.1.2 Lift Stations and Force Mains

The City operates and maintains two wastewater lift stations. In addition, there are several private lift stations that discharge into the City's collection system. Figure 5.1 shows the locations of the City owned lift stations. A brief summary of each lift station is presented below:

- **Villa Drive**: This lift station is located on Villa Drive, just east of South Vanderhurst Avenue. The lift station consists of two submersible pumps with an unknown rated capacity. The Villa Drive lift station conveys raw wastewater west via an 6-inch diameter force main approximately 253 feet to a manhole located at the intersection of South Vanderhurst Avenue and Villa Drive. According to City staff, this lift station is in good condition, and the pumps were recently replaced. The rated capacity of the replacement pumps is unknown. This lift station only serves a few homes, and is not expected to serve any additional growth in the future. The City has not experienced any capacity issues with the pumps to date, and none are expected to occur in the future.
- **Crosstown Line Lift Station**: The Crosstown Line Lift Station was constructed in 2016 and is located on South First Street, northwest of Lone Oak Road. The lift station consists of an 8-foot diameter wet well with a depth of 19 feet and two-

1,000 gpm submersible pumps. The Crosstown Line Lift Station is not currently in service, as there are no customers connected to the lift station yet. In the future, this lift station will convey raw wastewater northwest along South First Street via an 8-inch diameter force main approximately 1,220 feet to a manhole located at the corner of Division Street and South First Street.

5.2 HYDRAULIC MODEL DEVELOPMENT

A wastewater collection system model is a simplified representation of the real collection system. Collection system models can assess the conveyance capacity for a collection system. They are also used to perform “what if” scenarios to assess the impacts of future developments and land use changes. The City’s collection system hydraulic model was constructed using a multi-step process utilizing data from a variety of sources. This section summarizes the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled collection system, the hydraulic model elements, and the model creation process.

5.2.1 Previous Hydraulic Computer Model

The City’s previous collection system hydraulic model was developed using SewerCAD, developed by Bentley. The SewerCAD model routes flow through the collection system to evaluate the capacity of existing pipes and to determine where capacity constraints occur using the Kinematic Wave, standard step method. This method is a simplified version of the Saint Venant, one dimensional equations of fluid flow.

The SewerCAD model was assembled as part of the 2007 Wastewater Master Plan Update project using the City’s Geographic Information System (GIS) data, lift station drawings, supplemental survey work, and input from City Staff. A copy of the City's SewerCAD hydraulic model was not available, therefore a new hydraulic model was constructed as part of this project.

5.2.2 Selected Hydraulic Modeling Software

In the nine years since the previous hydraulic model was developed, significant improvements have been made to the hydraulic modeling software available on the market. Some examples of the improvements that have been made include modifications to the hydraulic routing engine as well as an enhanced graphical user interface (GUI), model output reports, and GIS compatibility. This Master Plan Update provided the City an opportunity to reexamine the software available on the market today and make a decision about continuing the use of SewerCAD or converting the model to one of the newer software packages.

In the early stages of this Master Plan, Carollo conducted an evaluation of the major wastewater collection system hydraulic modeling software applications on the market today. The results of the evaluation are presented in a technical memorandum, which is provided

in Appendix B for reference. This technical memorandum summarizes the major software vendors, briefly explains software features, compares the advantages and disadvantages of each software program, and provides a software program recommendation for the City.

Based on the results of the evaluation, it was agreed that InfoSWMM, by Innovyze (formerly MWH Soft), 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.

The latest version (v 14.5) of InfoSWMM was used to assemble the InfoSWMM hydraulic model (InfoSWMM model).

5.2.3 Modeled Collection System

The modeled sewer system consists of approximately 32 miles of sanitary sewer pipelines ranging in diameter from 6-inches to 27-inches. This includes the separate 21-inch industrial pipeline. The two City owned and operated lift stations were included in the hydraulic model:

- Villa Drive Lift Station
- Crosstown Line Lift Station

The remaining lift stations, which are privately owned, were excluded from the hydraulic model. Private lift stations were also excluded from the model. Exclusion of very small lift stations is common in collection system master planning, because the flows pumped through these lift stations is insignificant and does not affect model accuracy for the trunk sewer system.

5.2.4 Elements of the Hydraulic Model

The following provides a brief overview of the major elements of the hydraulic 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 wastewater collection 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 wastewater collection system modeling, an outfall typically represents the connection to the influent pump station at 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.
  - RDII. Rainfall Derived Infiltration and Inflows (RDII) 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.

5.2.5 Wastewater Load Allocation

Determining the quantity of dry weather wastewater flows generated by a municipality and how they are distributed throughout the collection system is an important 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 wastewater collection system facilities, both for existing and future conditions. Baseline wastewater

loads were allocated (assigned to specific nodes) in the hydraulic model based on land use data provided by the City and wastewater flow factors developed for each land use type (these are described in detail in Chapter 4). The flow factors and land use data provides a means to transform a specific land use category into an average dry weather flow, as described below:

- **Step 1**: The City's service area was broken up into 300 individual loading polygons. Each loading polygon represents the geographic area that contributes flows into a single model node (i.e., trunk system manhole). In a skeletonized model, a loading polygon will usually encompass a particular subdivision or grouping of lots. However, in an all pipe model, such as the City's hydraulic model, a loading polygon could be as small as a few parcels.
- **Step 2**: The loads were calculated for each loading polygon using GIS by multiplying the appropriate wastewater flow factor by the land use acreage.
- **Step 3**: The hydraulic model's load allocation tool assigned the calculated average dry weather flow to the appropriate node in the sewer system model.
- **Step 4**: The allocated loads were adjusted as necessary during the dry weather flow calibration process (see Section 5.3) to closely match the actual measured dry weather flows recorded during the flow monitoring period.

5.2.6 Model Construction

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 six steps, as described below:

- **Step 1**: The first step involved in the model conversion process was to review the City's GIS shapefiles for possible data errors and format to match the format accepted by the InfoSWMM software.
- **Step 2**: The collection system layer shapefiles were then imported into InfoSWMM using the "GIS Exchange" functionality of InfoSWMM.
- **Step 3**: Once imported to InfoSWMM, the City's wastewater collection system was reviewed to identify data gaps and/or new facilities that needed to be included in the hydraulic model. New facilities were added directly into the hydraulic model. Site plans and record drawings were provided for areas with missing information. Manholes were identified where data was still missing or where there were data conflicts to have City staff measure the invert depth below rim (or "dip"). Dips were also measured at random manholes to confirm the information provided in the GIS layers.

- **Step 4**: Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the model data was input correctly and that the flow direction, size, and layout of the modeled pipelines were logical. Additionally, the modeled lift stations were also checked to verify that they operated correctly.
- **Step 5**: Dry weather wastewater flows were then allocated to the appropriate model junctions.
- **Step 6**: The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run dates, time steps, reporting parameters, output units, and flow routing method. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

5.3 HYDRAULIC MODEL CALIBRATION

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

For this project, both dry and wet weather flow monitoring were conducted at 10 meter sites for a period of approximately 7 weeks at the end of 2016. Dry weather flow (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. The amount of I/I is essentially the difference between the WWF and DWF components.

5.3.1 Calibration Standards

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

- **Dry Weather Calibration Standards**: Dry weather calibration should be carried out for two dry weather days and the modeled flows and depths should be compared to the field measured flows and depths. Both the modeled and field measured flow hydrographs should closely follow each other in both shape and magnitude.

  In addition to the shape, the flow hydrographs should also meet the following criteria as a general guide:

  – The timing of flow peaks and troughs should be within 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 and depths should be compared to the field measured flows and depths. The flow hydrographs 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.

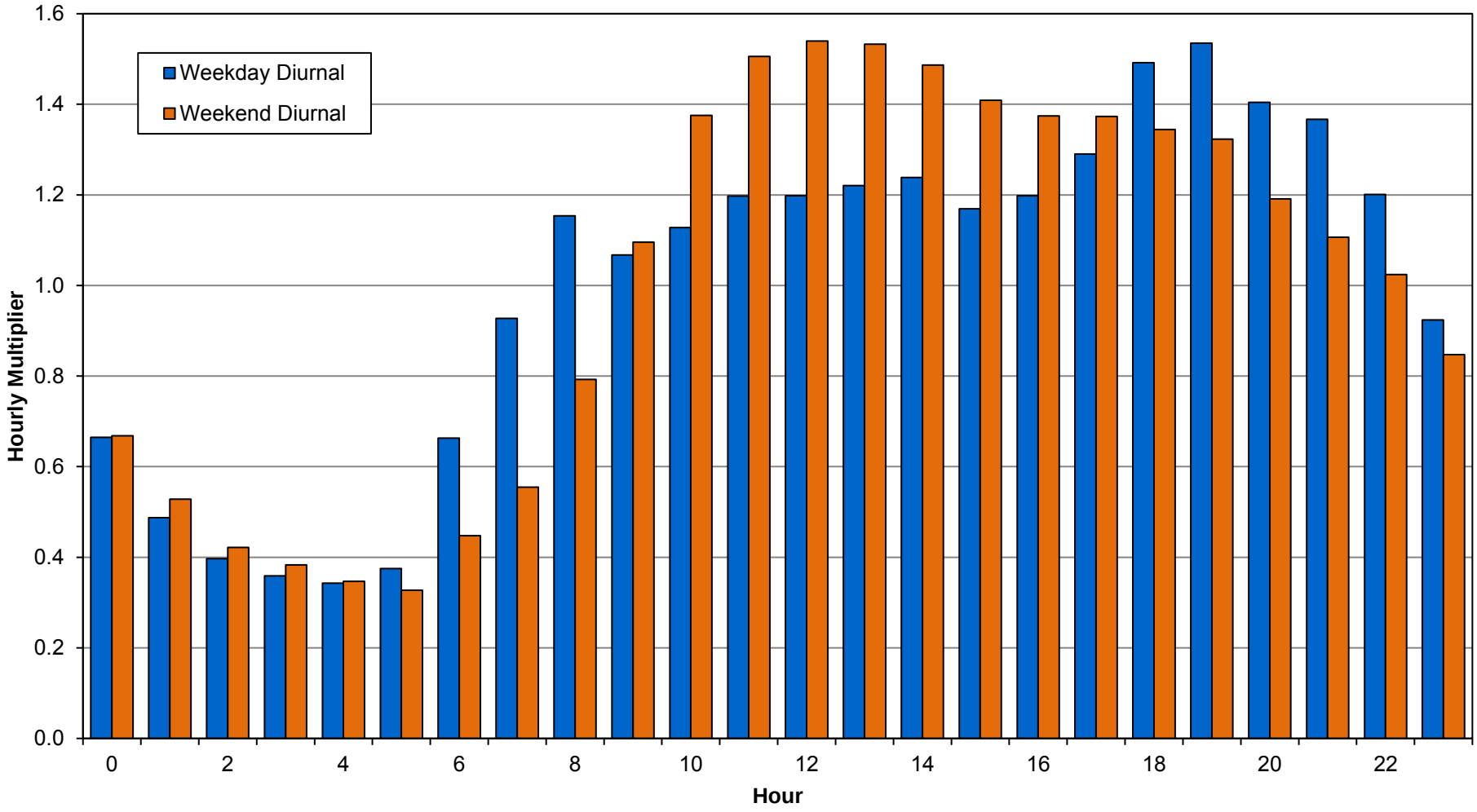
5.3.2 Dry Weather Flow Calibration

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

- **Divide the system into areas tributary to each flow meter**. The first step in the calibration process was to divide the City into flow meter tributary areas. Ten tributary areas were created, one for each flow meter from the temporary flow monitoring program. A map showing the locations of each flow monitoring site and their associated tributary area are provided in Chapter 3 along with a schematic of the flow meters.

- **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 closely matched the field measured flows. Figure 5.2 shows the calibrated weekday and weekend diurnal patterns for the area tributary to Site 2. Similar diurnal curves were developed for each of the meters and its tributary area. These additional curves are available in Appendix C.



EXAMPLE DIURNAL PATTERNS (METER 2)

FIGURE 5.2

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- **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 build up 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 which 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.

Table 5.2 provides a summary of the dry weather flow calibration using the average and daily peak flow results for both weekday and weekend conditions. As shown on Table 5.2, the model simulated average and peak flows for both weekday and weekend DWF were all within 10 percent. In general, the percent difference between the overall modeled and measured DWF ranged between -8.7 and 3.6 percent.

Table 5.2	Dry Weather Flow Calibration Summary Collection System Master Plan King City			
Monitor Site	**Pipe Diameter (in.)**	**Measured Dry Weather Flow(1),(2) (mgd)**	**Modeled Dry Weather Flow (mgd)**	**Percent Difference(3)**
Site 1	18	0.12	0.12	3.6%
Site 1B	10	0.02	0.02	0.1%
Site 2	30	0.57	0.52	-8.7%
Site 3A	12	0.06	0.06	-3.7%
Site 3B	12	0.04	0.04	-1.6%
Site 4	18	0.08	0.08	0.5%
Site 5	16	0.11	0.11	0.1%
Site 6	18	0.09	0.09	-0.2%
Site 7	18	0.06	0.06	3.6%
Site 8	14	0.01	0.01	-1.7%

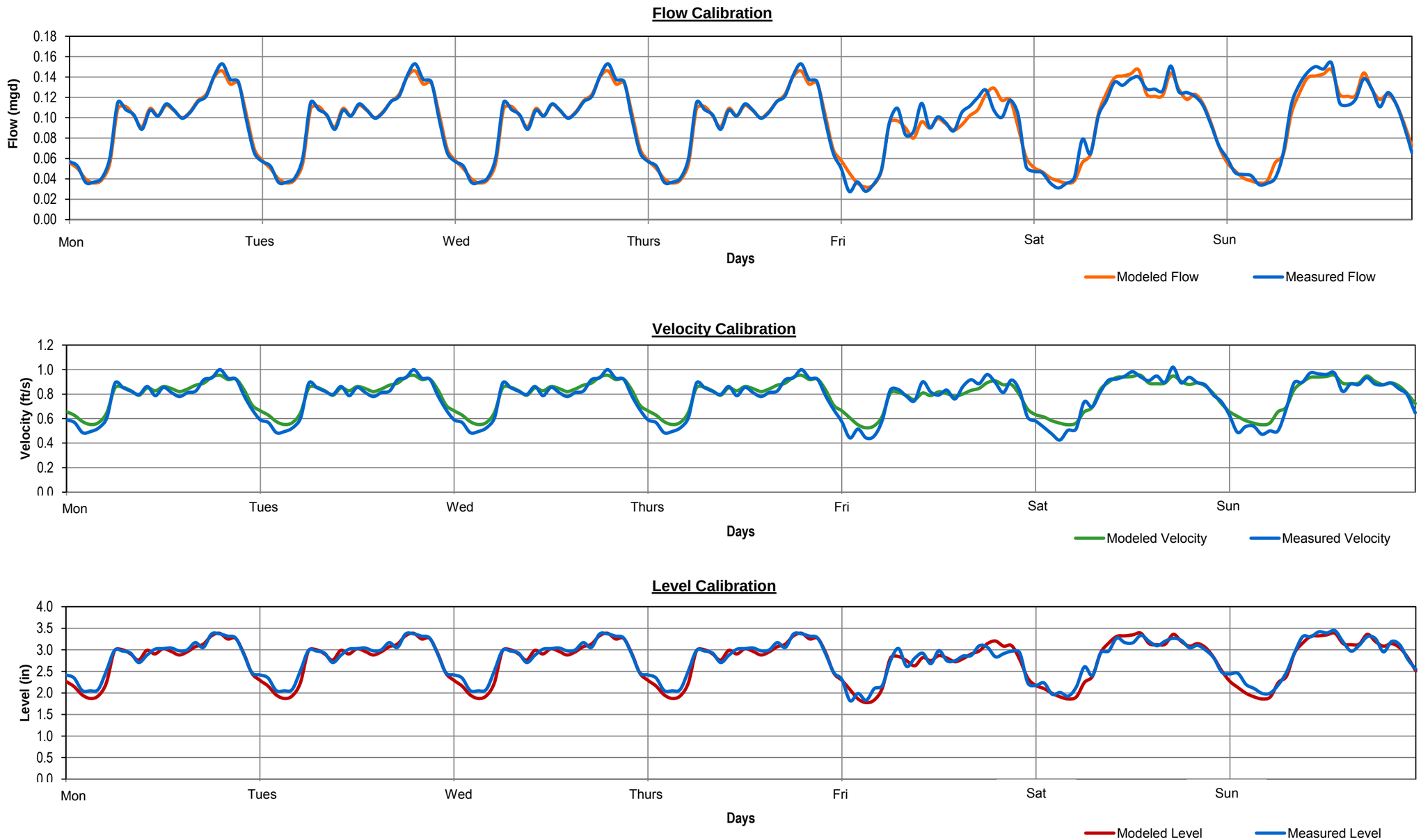
Notes:
(1) Source: King 2016 Temporary Flow Monitoring Program, V&A Consulting Engineers.
(2) Dry Weather Flow = (5 x Weekday Average + 2 x Weekend Average)/7.
(3) Percent Difference = (Modeled – Measured)/Measured x 100.

Appendix C contains a detailed dry weather flow calibration summary sheet for each of the ten meter sites. Each calibration sheet provides plots that compare the model simulated and field measured flow, velocity, and level data for both weekday and weekend conditions. An example of the dry weather calibration for Site 6 is shown on Figure 5.3. As shown on Figure 5.3 and in Appendix C, there is excellent overall correlation of the field measured data to the model output results. The modeled flows, levels, and velocities at each site were within the generally accepted calibration tolerances, and therefore the model was considered calibrated.

5.3.3 Wet Weather Flow Calibration

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

- **Identify calibration rainfall events**. The WWF calibration process consists of running model simulations of historic rainfall events based on data collected as part of the temporary flow monitoring program. The goal of any wet weather flow monitoring program is to capture and characterize a system's response to a significant rainfall event, preferably during wet antecedent moisture conditions.



EXAMPLE DRY WEATHER FLOW CALIBRATION (SITE 6)

FIGURE 5.3

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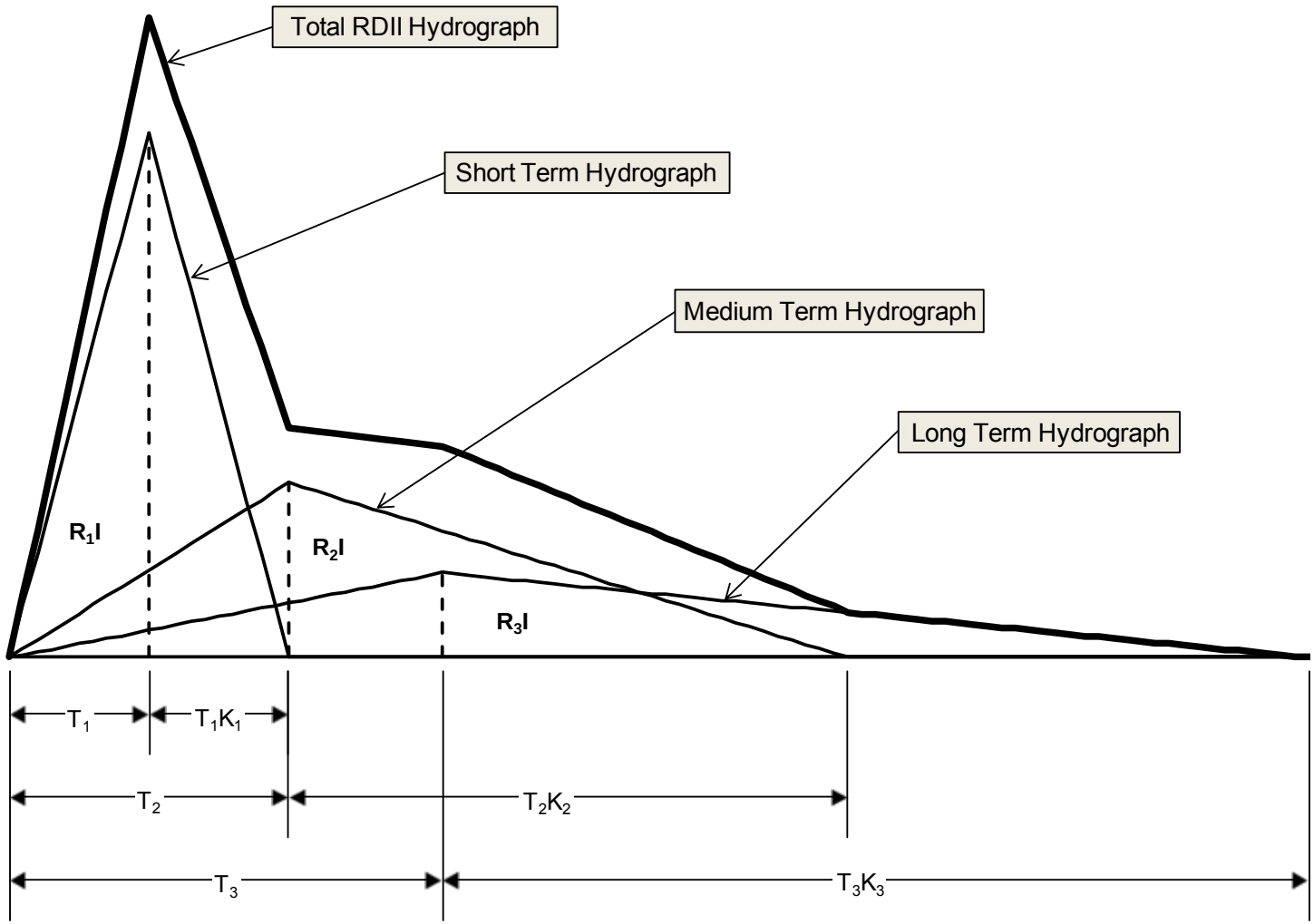
The selection of a particular calibration storm or group of storms is based on a review of the flow and rainfall data. Only one significant rainfall event (total of 0.52 inches of rainfall) occurred during the flow monitoring program on November 20, 2016. However, flow data was only available for five out of ten of the flow monitoring sites during this storm event. A less significant storm event occurred on December 15, 2016, however, this did not generate significant wet weather flows. Therefore, the five monitoring locations and the total flow at the plant were calibrated for the November 20, 2016 storm event. Two additional historical storm events were also simulated to confirm the wet weather calibration:

- January 18, 2010
- January 20, 2017

In order to run a model simulation for the November 20, 2016 rainfall events, the rainfall data, in 15-minute increments, were input into the model.

- **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, excluding any large vacant, open space, or other areas in the system which are not expected to contribute to I/I into the collection system. 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 involves creating custom unit hydrographs for each flow monitoring tributary 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 shed that enters the collection system; T represents the time to peak of the hydrograph; and K represents the ratio of time to recession to the time to peak. Therefore, there are a total of nine separate variables associated with each unit hydrograph. Figure 5.4 shows the shape of an example unit hydrograph.

  The hydrographs utilize 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.



EXAMPLE RDII HYDROGRAPH

FIGURE 5.4

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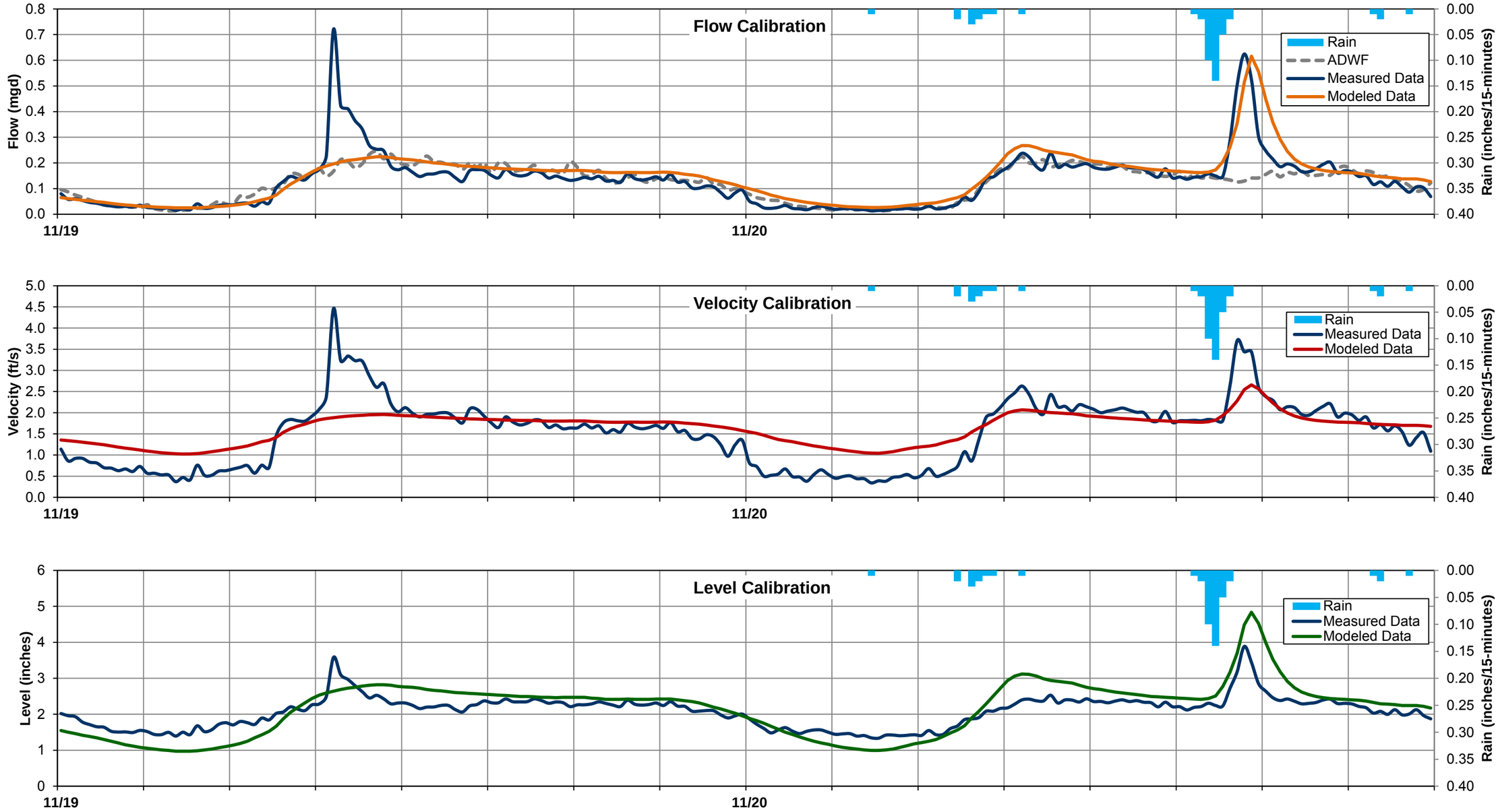
As with the dry weather calibration, the wet weather calibration process compared the meter data with the model output. Comparisons were made for average and peak flows as well as the temporal distribution of flow until flows returned to their baseline levels. According to the WaPUG, a hydraulic model is generally considered to be satisfactorily calibrated to WWF conditions if the modeled peak flows are within +25 percent to -15 percent of the field measured data, and if the average modeled flows are within +20 percent to -10 percent of the field measured data.

Because wet weather flow data was only available at five flow monitoring sites, reasonable I/I parameters, based on overall flow to the plant, were assumed for the remaining five flow monitoring sites. The I/I parameters were adjusted until the simulated flow at the plant closely matched the measured flow. The January 2010 and January 2017 storm events were also simulated to confirm the wet weather parameters and calibration.

- **Refine model variables to match field measured velocity and flow depths**. After the model was considered to be satisfactorily calibrated for wet weather flows, the model simulated velocities and flow depths were checked against the field measured velocities and flow depths during the calibration storms. Refinements were made to the various model parameters so that the modeled and measured velocity and depth closely matched one another. If any adjustments were made to Manning's n-values or other parameters, the DWF calibration was rechecked as well to make sure that the flow depth and velocities still matched well under DWF conditions.

Appendix D contains a detailed wet weather flow calibration summary sheet for each of the flow monitoring sites with wet weather data. Each calibration sheet provides plots that compare the model simulated and field measured flow, velocity, and level data for the calibration storm. An example of the wet weather calibration for Site 1 is shown on Figure 5.5. Table 5.3 provides a summary of the wet weather flow calibration using the average and peak flow results. As shown on Table 5.3, the model simulated average and peak flows at all meter sites were within the acceptable tolerances for the November 20, 2016 storm event. Comparison of flows against the historical rainfall events also shows good correlation (graphs provided in Appendix D, and therefore the model was considered calibrated and ready to use for capacity analysis purposes.



EXAMPLE WET WEATHER FLOW CALIBRATION (SITE 1)

FIGURE 5.5

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Table 5.3 Wet Weather Flow Calibration Summary
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King City

Monitor Site	Field Measured Flow[1][2]		Model Simulated Flow[2]		Percent Difference[3]	
	Average (mgd)	Peak (mgd)	Average (mgd)	Peak (mgd)	Average (%)	Peak (%)
Site 1	0.134	0.625	0.161	0.616	19.8	-1.4
Site 1B[4]	NA	NA	NA	NA	NA	NA
Site 2	0.613	1.059	0.571	1.015	-6.9	-4.1
Site 3A[4]	NA	NA	NA	NA	NA	NA
Site 3B[4]	NA	NA	NA	NA	NA	NA
Site 4	0.076	0.137	0.078	0.135	2.2	-1.6
Site 5[4]	NA	NA	NA	NA	NA	NA
Site 6[4]	NA	NA	NA	NA	NA	NA
Site 7	0.059	0.309	0.070	0.304	19.6	-1.6
Site 8	0.011	0.128	0.013	0.130	16.5	1.7
WWTP	0.872	1.950	1.020	2.142	17.0	9.8

Notes:

(1) Source: King 2016 Temporary Flow Monitoring Program, V&A Consulting Engineers
(2) Average flows are measured over the duration of the storm event. Peak flows represent hourly average peak flows.
(3) Percent Difference = (Modeled – Measured)/Measured x 100
(4) Field measured data not available during the November 20, 2016 storm event. Circle charts provided for the WWTP were used to calibrate remaining flowmeters.

CAPACITY EVALUATION AND PROPOSED IMPROVEMENTS

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

6.1 CAPACITY EVALUATION

Following the dry and wet weather flow calibration, which is summarized in detail in Chapter 5, a capacity analysis of the existing and future collection system was performed. The capacity analysis entailed identifying areas in the sewer system where flow restrictions occur or where pipe capacity is insufficient to convey 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 4.

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

6.1.1 Existing System

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) exceeded 90 percent of the pipe diameter (maximum d/D greater than 0.9) were identified.

In general, the City's collection system has sufficient capacity to convey current PWWFs without exceeding the established flow depth criterion. However, there is one area where capacity restrictions lead to flow depths that exceed allowable levels. This is the 8-inch diameter gravity sewer on Bitterwater Road (from San Antonio Drive to Metz Road).

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 6.2. In accordance with the established planning criteria, new sewer pipelines were sized such that the maximum flow depth to pipe diameter ratio (d/D) did not exceed the values summarized in Chapter 4. In other words, flows in recommended improvements were not allowed to surcharge during PWWF conditions.

6.1.2 Future System

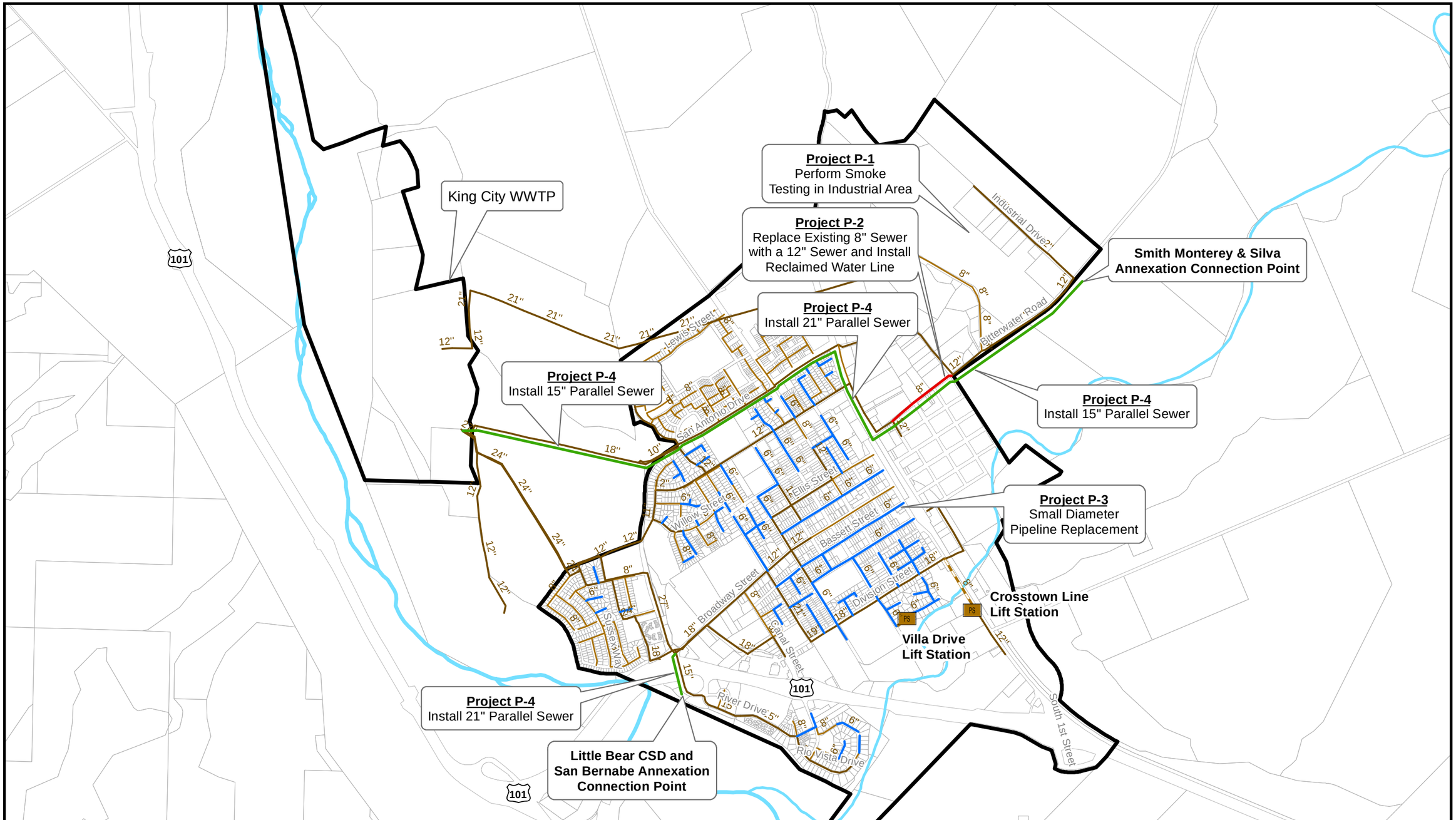
The analysis of the future systems (Phase 1, 2, and 3) was performed in a manner similar to the existing system analysis. The purpose of the future system evaluation is to verify that the existing system improvements were appropriately sized to convey future PWWFs, and to identify the locations of sewers that are adequately sized to convey existing PWWFs, but

cannot convey future PWWFs. The following summarizes the evaluation of the future phases:

- **Phase 1 (10 Years)**: Phase 1 includes the development of all residential and commercial infill and the build out of the Creek Bridge and Mills Ranch developments. No new deficiencies were identified based on the Phase 1 PWWF evaluation.
- **Phase 2 (20 Years)**: Phase 2 includes the development of industrial infill, the Downtown Addition, Highway 101 Commercial area, and a portion of the Little Bear annexation. No new deficiencies were identified based on the Phase 2 PWWF evaluation.
- **Phase 3 (Beyond 20 Years)**: Phase 3 includes flow from several annexations (Smith-Monterey, Silva, San Bernabe, and Little Bear). At Phase 3, the City's wastewater flows are expected to increase by 3.5 times (compared to existing flows). As such, there are several areas of the existing system that cannot convey the Phase 3 PWWF without surcharging and/or overflows. The additional flow from Smith-Monterey and Silva create additional surcharging and overflows from where it connects (Bitterwater Road and Industrial Way) all the way to the treatment plant. The additional flow from Little Bear and San Bernabe also causes surcharging in the 15-in diameter gravity sewer on River Drive and Broadway Street.
- **Lone Oak Sewer Extension**: The Lone Oak sewer was evaluated to determine the available capacity under Phase 3 conditions. Based on the maximum flow depth criteria for new sewers (maximum d/D of 0.75), the available capacity of the Lone Oak sewer is approximately 0.238 mgd average flow or 0.715 mgd peak flow (assuming a peaking factor of 3). If the less conservative maximum flow depth criteria of 90 percent were used, then the available capacity would be approximately 0.305 mgd (average flow) and 0.916 mgd (peak flow).

6.2 COLLECTION SYSTEM IMPROVEMENTS

Figure 6.1 illustrates the proposed sewer improvements required to correct existing deficiencies and to serve future users. When an increase to capacity is required, existing sewers can be upgraded or a parallel or relief sewer can be constructed. For the purposes of this study, unless otherwise stated, we assumed that a capacity deficient sewer would be upgraded to a larger diameter. The upgraded pipeline generally followed the same slope as the existing pipeline, with the exception where survey data revealed negative or flat slopes in an existing alignment.



Legend

- Study Area
- Parcels
- River or Stream
- City Owned Lift Station
- Force Main

Gravity Sewer by Diameter

- 8" and Smaller
- 10" and Larger

Proposed Improvements

- Phase 1 (2018-2027)
- Phase 2 (2028-2037)
- Phase 3 (Post 2037)
- Small Diameter Pipeline Replacement

0 0.25 0.5 Miles

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COLLECTION SYSTEM IMPROVEMENTS

FIGURE 6.1

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In essence, there are two alternatives for every trunk sewer project, but the decision to replace or construct a parallel sewer should be made during the preliminary design phase. During the preliminary design phase, the existing sewer should be inspected by closed circuit television (CCTV) to determine its structural condition. If severely deteriorated, the existing sewer should be upgraded. If moderately deteriorated, slip lining or cured-in-place pipe lining can rehabilitate the existing sewer.

The proposed improvements that will serve future users are sized for Phase 3 (build out) conditions. As the City continues to grow, it is recommended that the proposed pipeline diameters be constructed so that the facilities have sufficient capacity for build out conditions. Building a smaller interim project with the plans of upsizing in the future to account for further growth is not recommended due to the extended useful life of the improvements proposed herein. The proposed pipe diameter represents the ultimate diameter for build out conditions.

6.2.1 Existing versus Future Improvement

An existing deficiency is one where the existing facility's capacity is insufficient to meet the planning criteria (e.g., pipeline upgrades required to prevent severe surcharging during the design wet weather event) for existing users. If a project was proposed to correct an existing deficiency exclusively, then existing users were assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs.

Other recommended improvements will serve future users, even when an improvement calls for the upgrade of an existing facility. In these cases, an existing sewer or lift station may have sufficient capacity to convey current PWWFs, but as growth continues and more users are added to the system, the increased flow results in capacity deficiencies. These projects, as well as new trunk sewers to extend wastewater collection system service to future growth areas, are future improvements. Future users were assigned 100 percent of the future project's benefit and 100 percent of the costs.

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

6.2.2 Existing System Improvements

For the most part, the existing wastewater collection system contains sufficient capacity to convey the current PWWF without exceeding the capacity criteria discussed Chapter 4. There is one exception where existing sewer will need to be replaced by larger diameter

sewers, or parallel sewers will need to be constructed to bypass flow around hydraulically deficient sewers. The recommended improvements to correct existing deficiencies are summarized below.

- **Project 1 – Smoke Testing**: The majority of flow through contributing to the deficient 8-inch diameter gravity pipe on Bitterwater Road is industrial flow. Analysis of the flow monitoring data indicated that the peak flows in this area are due to inflow. It is possible that there may be a drain connected to the collection system causing spikes in flow. Because of this, smoke testing is recommended along Industrial Way, Airport Road, and Bitterwater Road to identify the inflow source that is leading to high rates of I/I in the system.
- **Project 2 – Bitterwater Road Sewer and Reclaimed Water Main**: This project consists of replacing approximately 1,470 feet of existing 8-inch diameter sewer on Bitterwater Road from San Antonio Drive to Metz Road with a new 12 inch diameter sewer. This project is required to mitigate surcharged conditions on Bitterwater Road for existing PWWFs. However, this project may not be necessary if the source of inflow is identified during the smoke testing and is resolved. Although this recommended improvement is included in the capital improvement program presented in Chapter 7, it is recommended that this improvement be reevaluated once the smoke testing is completed to determine if it is still required.

  Because the City wants to take advantage of future pipeline projects, this project also includes the installation of approximately 1,470 feet of 10-inch diameter reclaimed water main.
- **Project 3 – Small Diameter Pipeline Replacement**: This project consists of replacing the City's existing small diameter sewers (6-inch diameter and smaller) with 8-inch diameter sewers. There is a total of 36,010 linear feet (LF) of 6-inch diameter gravity sewers in the collection system (approximately 21 percent of the collection system). A replacement program of 30 years would equate to replacing approximately 1,200 LF per year.

6.2.3 Future System Improvements

The following discussion summarizes the new trunk sewers that will serve future users. The locations of the new trunk sewers are conceptual and are likely to change during the design phase. The locations shown are possible alignments based on available information and are intended to assist in the development of probable construction costs. No investigation into the feasibility of these alignments has been conducted. However, an attempt was made to place new trunk sewer alignments within existing streets or other feasible pipeline alignments.

- **Project 4 – Little Bear/San Bernabe Sewer**: The hydraulic evaluation indicated that the existing 15-inch diameter sewer on Broadway Street from River Drive to San Antonio Drive has sufficient capacity to convey Phase 2 flows from Little Bear.

However, as flows increase beyond the 20-year planning period (Phase 3), the existing 15-inch diameter sewer experienced surcharging. It was assumed for Phase 2, that Little Bear would be connected directly to the existing 15-inch diameter gravity sewer. However, for Phase 3, it is recommended that the Little Bear and San Bernabe developments connect to the 27-inch Crosstown sewer at Broadway Street and San Antonio Drive with approximately 1,250 LF of new 21-inch diameter gravity sewer.

- **Project 5 – Smith Monterey/Silva Sewer**: Under Phase 3 PWWF conditions, much of the existing 12-inch, 15-inch, and 18-inch diameter sewers on Bitterwater Road, Metz Road, King Street, Mildred Avenue, and San Antonio Drive to the treatment plant experienced surcharging and several overflows. This additional flow also causes surcharging and overflows upstream on Airport Road and Industrial Way. It is recommended that a parallel 15 to 21-inch diameter gravity sewer (approximately 13,380 LF) be installed to serve the new developments. The proposed pipeline extends from the intersection of Bitterwater Road and Industrial Way, along Bitterwater Road, Metz Road, then along San Antonio Drive to the treatment plant.

6.2.4 Project Prioritization

A small portion of the improvements identified as part of this Master Plan are driven by future development, which consist of new sewers that serve future growth or improvements to existing facilities that are needed to serve future growth. When fully implemented, the capital projects will allow the conveyance of PWWFs to the treatment plant under Phase 3 (build out) conditions.

Prioritizing the required capital improvements for the City's sewer system is an important aspect of this study. The improvement projects were prioritized based on the following factors:

- Upgrading existing facilities to mitigate current capacity deficiencies and to serve future users
- Building the new trunks necessary to serve future users

Improvements to existing facilities will provide sufficient capacity to mitigate existing issues and to convey increased flows resulting from future growth. Future development will require the construction of sewers to serve new users. The projects were grouped into the following phases:

- Phase 1: Years 2018 through 2027
- Phase 2: Years 2028 through 2037
- Phase 3: Beyond 2037

The projects were phased based on the best available information for how the City will develop moving forward. The actual implementation of the improvements serving future users ultimately depends on growth. The priorities presented below are estimates, and changes in the City's planning assumptions or growth projections could increase or decrease the priority of each improvement.

- **Phase 1 Projects (2018-2027)**: The highest priority project for the existing system is the smoke testing of the industrial area (Project 1). This is important to identify the source of high inflow rates, which may allow the City to avoid upsizing the 8-inch diameter gravity sewer on Bitterwater Road (Project 2). Another project targeted to begin in Phase 1 is the small diameter sewer replacement program (Project 3).
- **Phase 2 (2028-2037)**: No new projects were identified for Phase 2. The small diameter sewer replacement program (Project 3) is a long-term project which extends through Phase 2.
- **Phase 3 (Beyond 2037)**: The third phases target new sewers to serve future planned developments, including Little Bear and San Bernabe (Project 4) and Smith-Monterey and Silva (Project 5). As previously noted, the actual rate of growth within the City will dictate when these new pipelines will need be constructed. Because Phase 3 projects are outside the planning period, they are not included in the CIP.

CAPITAL IMPROVEMENT PLAN

This chapter presents the recommended Capital Improvement Plan (CIP) for the King City (City) collection system and a summary of the capital costs. This chapter is organized to assist the City in making financial decisions, and is based on the evaluation of the City's sewer system.

7.1 CAPITAL IMPROVEMENT PROJECT COSTS

The capacity upgrades set the foundation for the City's capacity related sewer system CIP. 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 10,530 (December 2016).

7.2 COST ESTIMATING ACCURACY

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

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

7.3 CONSTRUCTION UNIT COSTS

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

7.3.1 Gravity Sewer Unit Costs

Sewer pipeline improvements range in size from 12-inches to 21-inches in diameter in this study. Pipe casings up to 42-inches in diameter are included for major crossings (e.g., creeks, canals, highways, and railroad) of the trunk sewers. Unit costs for the construction of pipelines and appurtenances (e.g., manholes) are shown in Table 7.1.

Table 7.1 Gravity Pipeline Unit Costs Collection System Master Plan King City	
Pipe Diameter (inches)	**Unit Cost ($/linear foot)(1)**
8	115
10	143
12	172
15	215
18	234
21	273
24	312
27	351
30	390
33	430
36	469
39	508
42	547
Pipeline Casing for Major Crossings(2)	
15/30	1,367
18/30	1,367
21/42	1,913
24/42	1,913
30/48	2,187
36/48	2,187

Notes:
(1) Costs are based on ENR CCI 20-City Average of 10,530 (December 2016).
(2) Size represents pipeline diameter and associated casing size.

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.

7.4 PROJECT COSTS AND CONTINGENCIES

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

7.4.1 Baseline Construction Cost

This is the total estimated construction cost, in dollars, of the proposed improvement for pipelines and lift stations. Baseline construction costs were calculated by multiplying the estimated length by the unit cost.

7.4.2 Estimated Construction Cost

Contingency costs must be reviewed on a case-by-case basis because they will vary considerably with each project. Consequently, it is appropriate to allow for uncertainties associated with the preliminary layout of a project. Such factors 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 25 percent contingency was applied to the Baseline Construction Cost to account for unforeseen events and unknown conditions. This contingency accounts for unknown site conditions such as poor soils, unforeseen conditions, environmental mitigations, and other unknowns and is typical for master planning projects. The Estimated Construction Cost for the proposed sewer system improvement consists of the Baseline Construction Cost plus the 25 percent construction contingency.

7.4.3 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 investigations 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 such items as construction management, engineering services, materials testing, and inspection during construction. Finally, there are project administration costs, which cover such items 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 25 percent of the estimated construction cost.

As shown in the following sample calculation of the capital improvement cost, the total cost of all project construction contingencies (construction, engineering services, construction

management, and project administration) is 56.3 percent of the baseline construction cost. Calculation of the 56.3 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 (25%)	$250,000
Estimated Construction Cost	**$1,250,000**
Engineering Cost + Construction Management + Project Administration (25%)	$312,500
Capital Improvement Cost	**$1,562,500**

A summary of the capital project costs is presented in Table 7.2. This table identifies the projects, provides a brief description of the project, identifies facility size (e.g., pipe diameter and length), and the capital improvement cost. The table also shows the probable phase in which the project would be implemented. The implementation timeframe was based on the priority of each project to correct existing deficiencies or to serve future users.

7.4.4 Capital Improvement Implementation

As discussed in Chapter 6, recommended CIP projects are prioritized based on their urgency to mitigate existing deficiencies and for servicing anticipated growth. It is recommended that improvements to mitigate existing deficiencies be constructed as soon as possible. The deficiencies in the future system have a significant total capital cost that is best distributed based on the order in which the City develops.

The implementation phases are separated into 10-year increments. Each project is itemized by phase in Table 7.2 and a summary by phase is provided in Table 7.3.

7.4.5 Existing Versus Future Users Cost Share

The improvements proposed in this study either benefit existing users and/or are required to serve new development and future users. Some of the projects provide benefit to both existing and future users. An opinion of benefit to future users, based on preliminary project information, is included in Table 7.2. A summary of the existing and future user cost share for the proposed projects by phase is summarized in Table 7.3.

Table 7.2 Capital Improvement Projects
Collection System Master Plan
King City

Project No.	Type of Improv.	Description/ Street	Description / Limits	Project Length/Size and Cost: Ex. Size/ Diam. (in)	New Size/ Diam. (in)	Replace/ New	Length (ft)	Capital Improvement Cost[(1),(2)] ($)	Capital Improvement Phasing: Phase 1 2018-27 ($)	Phase 2 2028-37 ($)	Future Users Benefit (%)	Reimbursement Category: Existing Improv. ($)	Future Improv. ($)
1	Other	Smoke Testing	Bitterwater Road, Airport Road, Industrial Way	--	--	N/A		$ 25,000	$ 25,000	$ -	0%	$ 25,000	$ -
2A	Pipe	Bitterwater Road Sewer	San Antonio Drive and Metz Road	8	12	Replace	1,470	$ 411,000	$ 411,000	$ -	77%	$ 94,530	$ 316,470
2B	Pipe	Bitterwater Road Reclaimed Water Main[(3)]	San Antonio Drive and Metz Road	--	10	New	1,470	$ 163,000	$ 163,000	$ -	100%	$ -	$ 163,000
3	Pipe	Small Diameter Pipe Replacement[(4)]	Varies	6	8	Replace	36,010	$ 6,703,000	$ 1,787,467	$ 2,234,333	0%	$ 4,021,800	$ -
							CIP Total	**$ 7,302,000**	**$ 2,386,467**	**$ 2,234,333**	**--**	**$ 4,141,330**	**$ 479,470**

Notes:

1. Baseline Construction Cost plus 25% to account for unforeseen events and unknown conditions.
2. Estimated Construction Cost plus 30% to cover other costs including Engineering, Construction Management, and Program Implementation.
3. Installation of new reclaimed water main assumed to be included with proposed sewer projects (per City's request). Assumed cost of $68 per LF of 10-inch diameter reclaimed water main (being installed at same time as sewer).
4. Project assumed to be spread out over 30 years (approximately 1,200 LF per year) starting in 2020.
5. Costs are based on the Engingeering News Record Construction Cost Index 20-city average of 10,530 (December 2016).

Table 7.3 Summary of Capital Improvement Costs
Collection System Master Plan
King City

Reimbursement Category	Implementation Phase		Total ($,M)
	Phase 1 (2018-27) ($,M)	Phase 2 (2028-37) ($,M)	
Existing User	$1.91	$2.23	**$4.14**
Future User	$0.48	$-	**$0.48**
Total	**$2.39**	**$2.23**	**$4.62**

Notes:
(1) Costs are based on ENR CCI 20-City Average of 10,530 (December 2016).

APPENDIX A – KING CITY TEMPORARY FLOW MONITORING REPORT

City of King City

Sewer Flow Monitoring and Inflow/Infiltration Study

Prepared for:

Carollo Engineers
1 East Liberty, Suite 424
Reno, NV 89501

Date:

May 2017

Prepared by:

V&A consulting engineers

<V&A Project No. 16-0200>

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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 was retained by Carollo Engineers (Carollo) to perform sanitary sewer flow monitoring and inflow and infiltration (I/I) analysis within the City of King City (City) at 8 site locations. There were discrepancies between the sewer maps and actual field conditions, including the existence of recently installed truck sewer line not indicated on the maps; thus, six of the eight sites were installed on November 14, 2016 (Sites 1, 2, 4, 5, 7 and 8). V&A conducted further reconnaissance to determine actual field conditions and expanded its scope to include additional flow meters; Sites 1B, 3A, 3B, and 6 were installed on December 8, 2016. Meters were removed on December 29, 2016. There were three general purposes of this study.

1. Establish the baseline sanitary sewer flows at the flow monitoring sites.
2. Estimate available sewer capacity.
3. Isolate I/I response and perform I/I analysis.

Flow and Rainfall Monitoring Sites

Flow monitoring sites are identified by the manholes where the flow monitors were secured and the pipelines where the flow sensors were placed. The flow monitoring sites are shown in Table ES-1 and illustrated in Figure ES-1. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in Appendix A. The flow monitoring sites were selected and approved by Carollo and the City.

Table ES-1. List of Flow Monitoring and Rain Gauge Locations

Site	Pipe Monitored	Dia. (in)	Location
Site 1	East Inlet	18	San Antonio Drive between Mildred Avenue and Cemetery
Site 1B	North Inlet	9.625	Mildred Avenue and San Antonio Drive
Site 2	South Inlet	30	King City Sewage Treatment Plant
Site 3A	North Inlet	11.5	Willow Street at Sandringham Street
Site 3B	East Inlet	11.5	Willow Street at Sandringham Street
Site 4	NE Inlet	18	Broadway Street east of San Antonio Drive, west of Franciscan Way
Site 5	East Inlet	16	River Dr south of Broadway St, west of Rio Plaza Mobile Home Estates
Site 6	East Inlet	19	Mildred Avenue and Division Street
Site 7	East Inlet	18	San Antonio Drive west of Van Etten Avenue
Site 8	East Inlet	14	San Antonio Drive east of Metz Road
Rain Gauge Location:			1286 E. San Antonio Dr.

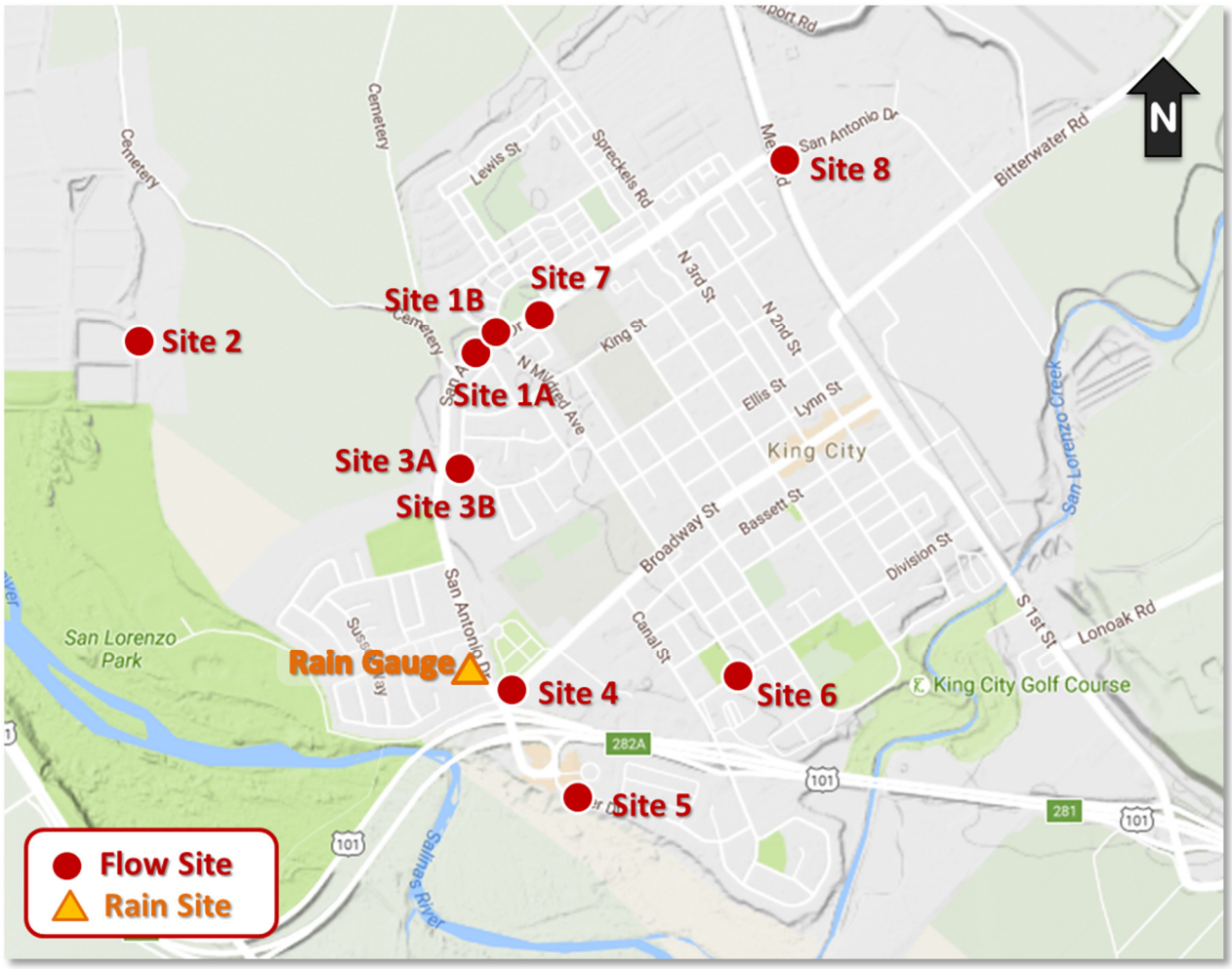


Figure ES-1. Map of Flow Monitoring Sites

Rainfall Monitoring

There were four small rainfall events during the monitoring period. The November 20 – 21, 2016 rainfall event totaled 0.53 inches and caused the greatest I/I response in the City collection system. The November 20 – 21, 2016 rainfall event was classified as a 1.4-year, 1-hour storm event.

Site Flow Monitoring and Capacity Results: Peak d/D Ratio and Peaking Factors

Peak measured flows and the consequent hydraulic grade line data are important to understand the capacity limitations of a collection system. 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, flow attenuation, flow restrictions, characteristics of I/I entering the collection system, and hydraulic features such as pump stations.
- **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. Standards for d/D ratio vary from agency to agency, but typically range between d/D ≤ 0.5 and d/D ≤ 0.75.

Sites 2 and 8 had unique backflow events that caused a surcharged condition. The Site 2 backflow was assumed to be caused by treatment plant operations. The Site 8 surcharge appeared to be caused by a downstream pipe blockage or restriction. Site 2 and 8 hydraulic conditions have been listed for either the backflow and consequent "flushing" event, or per normal operating conditions.

Table ES-2 summarizes the peak recorded flows, maximum 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 site locations; hydraulic conditions in other areas of the collection system will differ.

Table ES-2. Capacity Analysis Summary

Metering Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Pipe Diameter, *D* (in)	Max Depth, *d* (in)	*d/D* Ratio	Surcharge above Pipe Crown (ft)
Site 1	0.117	0.976	8.3	18	4.5	0.25	-
Site 1B	0.018	0.071	4.0	9.625	2.3	0.24	-
Site 2 [A]	0.571	1.076	1.9	30	6.3	0.21	-
- Flushing Event		*1.251*	*2.2*		*35.1*	*1.2*	*0.4*
Site 3A	0.062	0.137	2.2	11.5	3.0	0.26	-
Site 3B	0.041	0.101	2.5	11.5	4.0	0.35	-
Site 4	0.077	0.241	3.1	18	1.9	0.11	-
Site 5	0.107	0.269	2.5	16	4.6	0.29	-
Site 6	0.095	0.233	2.5	19	4.2	0.22	-
Site 7	0.056	0.465	8.4	18	4.3	0.24	-
Site 8 [A]	0.009	0.152	17.8	14	4.2	0.30	-
- Flushing Event		*0.199*	*23.3*		*55.4*	*4.0*	*3.4*

[A] Peak values as recorded during "normal" operation, not during a unique backflow, blockage or flushing event.

The following capacity analysis results are noted:

- **Peaking Factor:** Sites 1, 7 and 8 had peaking factors greater than 8.0 during normal operating conditions. Sites 1, 7 and 8 are connected; Site 8 flow are conveyed to Site 7 and Site 7 flows are conveyed to Site 1.
- **d/D Ratio:**
  - During normal operation, none of the flow monitoring sites had a maximum *d/D* ratio that exceeded a d/D value of 0.75.
  - Sites 2 and 8 had backflow events that caused a surcharged condition. The Site 2 backflow was assumed to be caused by treatment plant operations. The Site 8 surcharge was assumed to be caused by a downstream pipe blockage or restriction.

Figure ES-2 illustrates a flow schematic showing the peak flows per site with the corresponding flow depths (note: during normal operation, does not include backflow or flushing events)

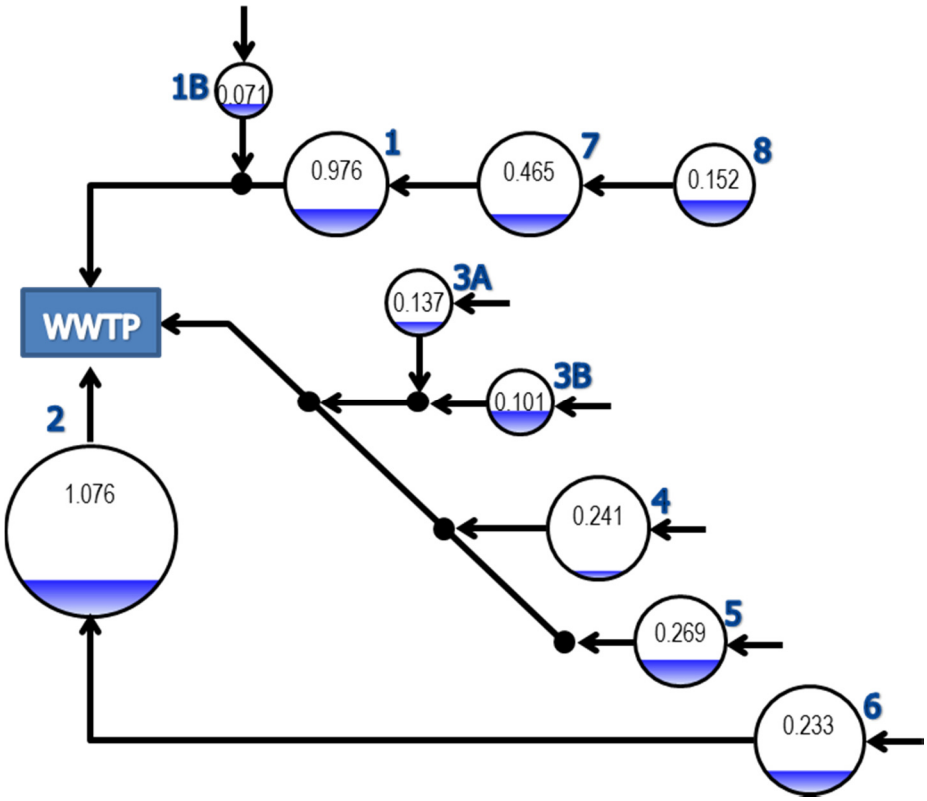


Figure ES-2. Peak Flows, Flow Schematic (normal operation)

Infiltration and Inflow Analysis

Table ES-3 summarizes the flow monitoring and inflow/infiltration (I/I) results for this study.

Table ES-3. I/I Analysis Summary

Site	Inflow Analysis Discussion
Site 1	Site 1 and Basin 1 had a definitive and strong inflow response.
Site 1B	There was no definitive I/I response observed for Site 1B.
Site 2	Site 2 had a definitive but relatively minimal inflow response.
Site 3A	There was no definitive I/I response observed for Site 3A.
Site 3B	There was no definitive I/I response observed for Site 3B.
Site 4	There was no definitive I/I response observed for Site 4.
Site 5	There was a suggestion of inflow observed for Site 5 (Basin 5)
Site 6	There was a suggestion of inflow observed for Site 6 (Basin 6)
Site 7	Site 7 and Basin 7 had a definitive and strong inflow response.
Site 8	Site 8 and Basin 8 had a definitive and strong inflow response.

Additional I/I results are summarized per the data captured in this study:

- I/I flows returned very quickly back to baseline flow levels, indicating minimal rain dependent infiltration and minimal groundwater infiltration.
- Inflow is the stronger component within the City of King City

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 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 greatest inflow problems. The highest inflow occurs in the in basins upstream from Sites 1, 7, and 8.
   b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems. During this project, no rain dependent infiltration was detected. This may be due to the low rainfall amounts that occurred during the study.
2. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
   a. Smoke testing
   b. Mini-basin flow monitoring
   c. Nighttime reconnaissance work to (1) investigate and determine direct point sources of inflow and (2) determine the areas and pipe reaches responsible for high levels of infiltration contribution.
3. **I/I Reduction Cost-Effectiveness Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow and 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 was retained by Carollo Engineers (Carollo) to perform sanitary sewer flow monitoring and inflow and infiltration (I/I) analysis within the City of King City (City) at 8 site locations. There were discrepancies between the sewer maps and actual field conditions, including the existence of recently installed truck sewer line not indicated on the maps; thus, six of the eight sites were installed on November 14, 2016 (Sites 1, 2, 4, 5, 7 and 8). V&A conducted further reconnaissance to determine actual field conditions and expanded its scope to include additional flow meters; Sites 1B, 3A, 3B, and 6 were installed on December 8, 2016. Meters were removed on December 29, 2016. There were three general purposes of this study.

1. Establish the baseline sanitary sewer flows at the flow monitoring sites.
2. Estimate available sewer capacity.
3. Isolate I/I response and perform I/I analysis.

1.2 Flow Monitoring Sites and Rain Gauge

Flow monitoring sites are identified by the manholes where the flow monitors were secured and the pipelines where the flow sensors were placed. Flow monitoring site data may include the flows of one or many drainage basins. Capacity and flow rate information is presented on a site-by-site basis.

The flow monitoring sites are shown in Table 1-1 and illustrated in Figure 1-1. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in Appendix A. The flow monitoring sites were selected and approved by Carollo and the City.

Table 1-1. List of Flow Monitoring and Rain Gauge Locations

Site	Pipe Monitored	Dia. (in)	Location
Site 1	East Inlet	18	San Antonio Drive between Mildred Avenue and Cemetery
Site 1B	North Inlet	9.625	Mildred Avenue and San Antonio Drive
Site 2	South Inlet	30	King City Sewage Treatment Plant
Site 3A	North Inlet	11.5	Willow Street at Sandringham Street
Site 3B	East Inlet	11.5	Willow Street at Sandringham Street
Site 4	NE Inlet	18	Broadway Street east of San Antonio Drive, west of Franciscan Way
Site 5	East Inlet	16	River Dr south of Broadway St, west of Rio Plaza Mobile Home Estates
Site 6	East Inlet	19	Mildred Avenue and Division Street
Site 7	East Inlet	18	San Antonio Drive west of Van Etten Avenue
Site 8	East Inlet	14	San Antonio Drive east of Metz Road
Rain Gauge Location:			1286 E. San Antonio Dr.

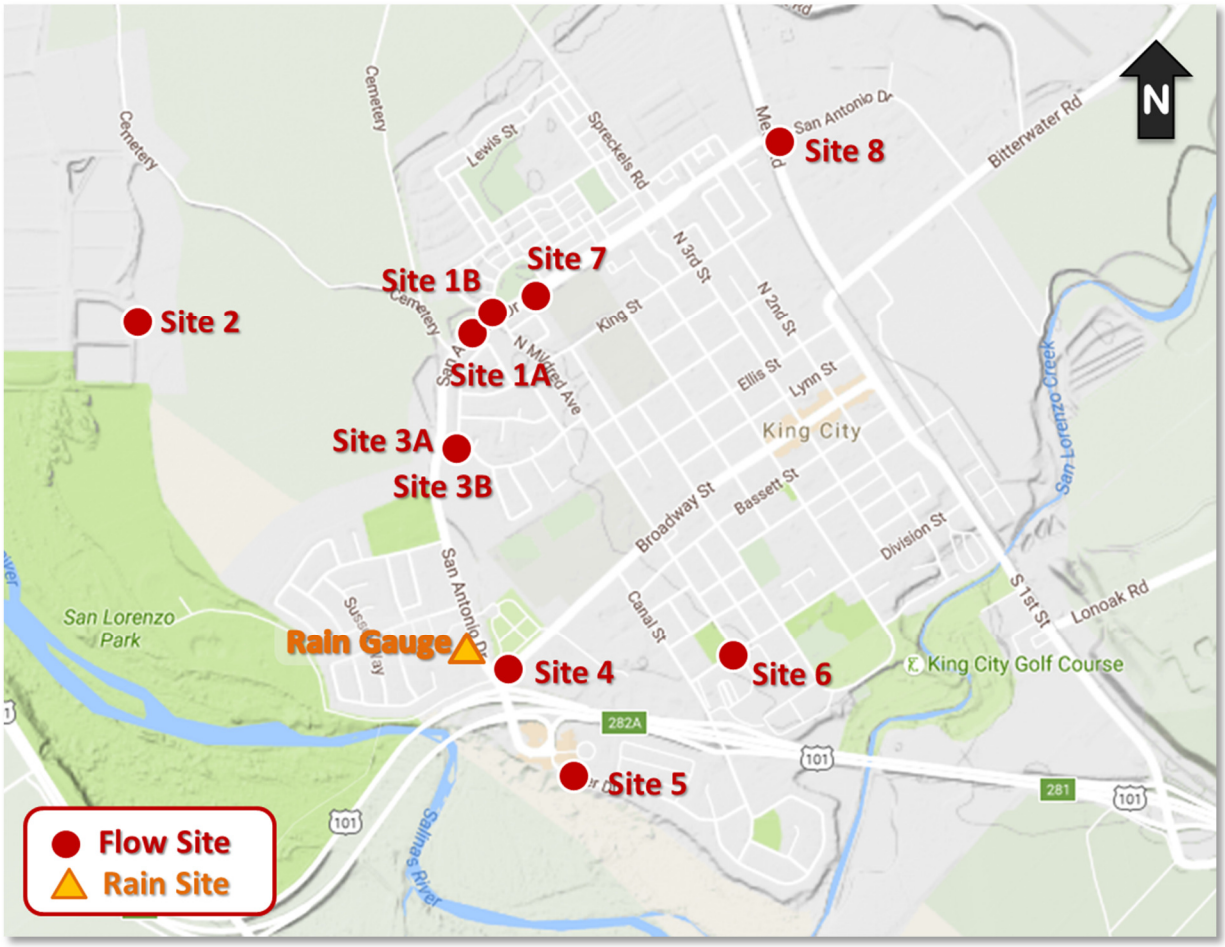


Figure 1-1. Map of Flow Monitoring Sites

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

V&A installed 10 Isco 2150 area-velocity flow meters for temporary metering within the collection system. Isco 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency (500 kHz) 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 compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. Figure 2-1 shows a typical installation for a flow meter with a submerged sensor.

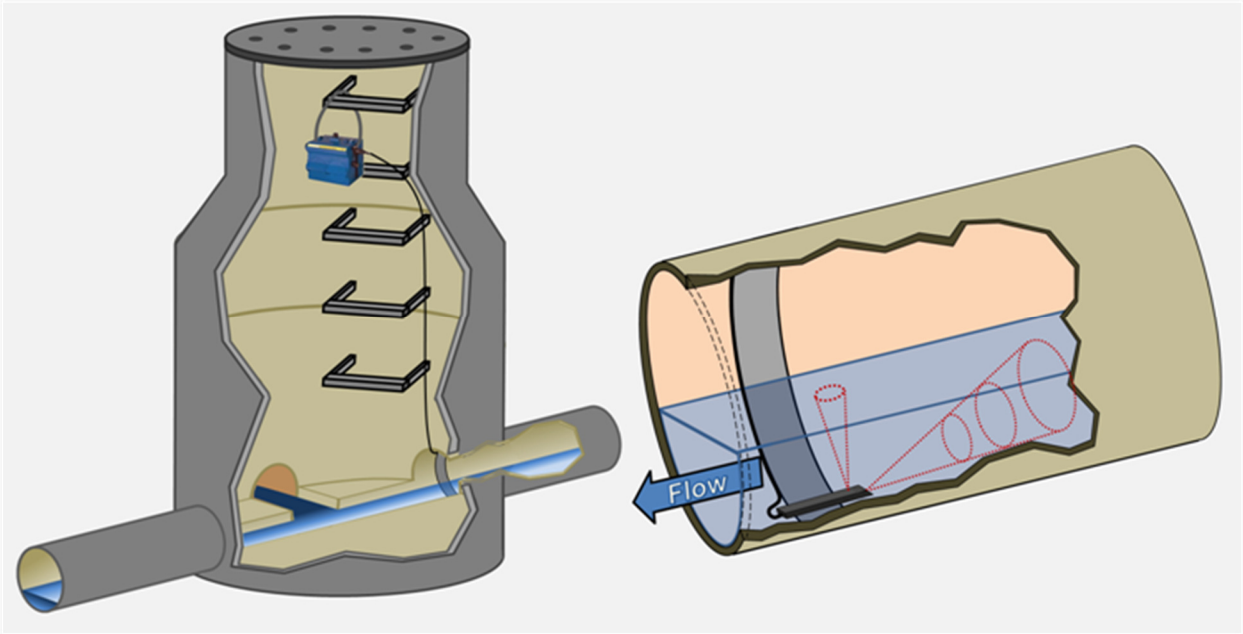


Figure 2-1. Typical Installation for Flow Meter with Submerged Sensor

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.

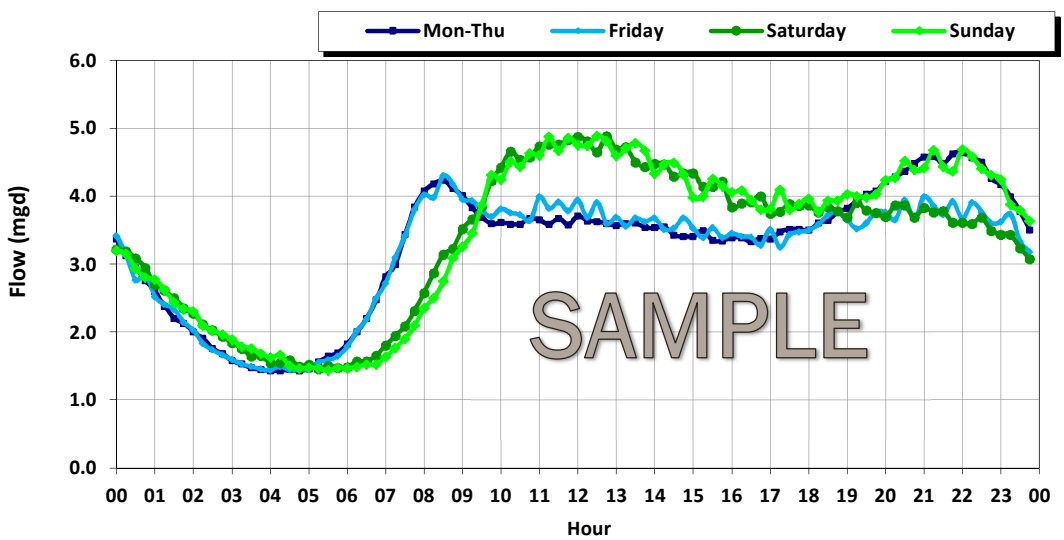


Figure 2-2. Sample ADWF Diurnal Flow Patterns

ADWF curves are taken from "Dry Days", when RDI had the least impact on the baseline flow. 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.

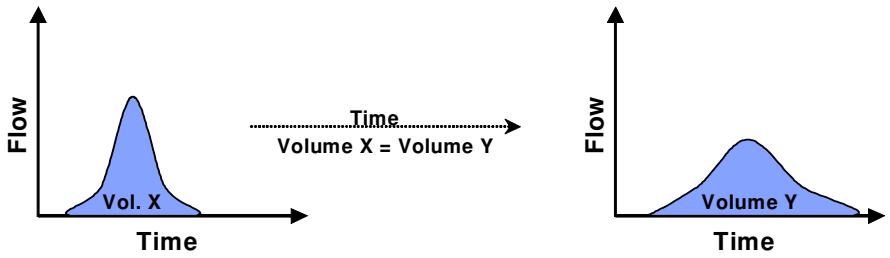


Figure 2-3. Attenuation Illustration

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. An attenuation factor of 0.9 per 10,000 feet of pipe was used for this study.

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.

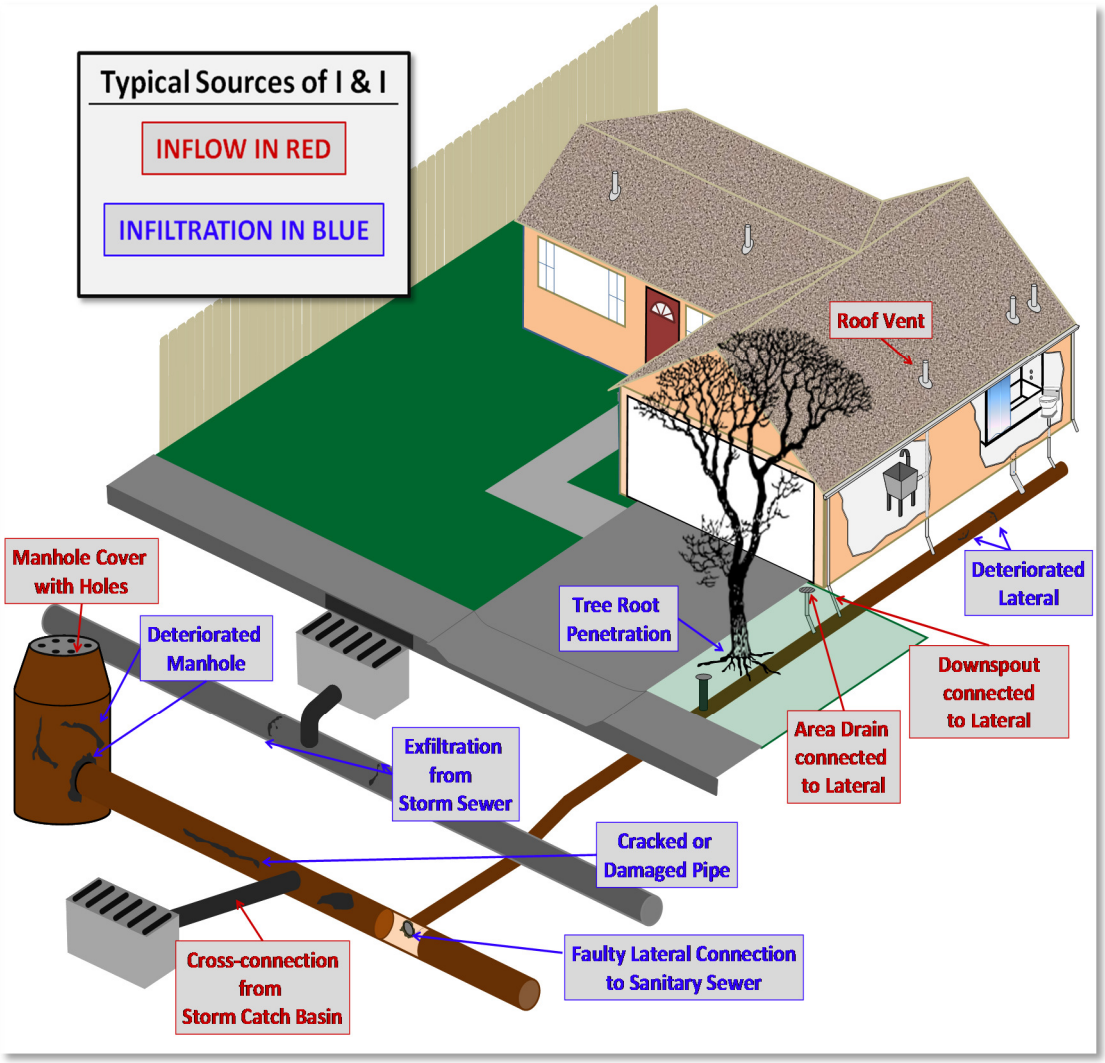


Figure 2-4. Typical Sources of Infiltration and Inflow

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*.

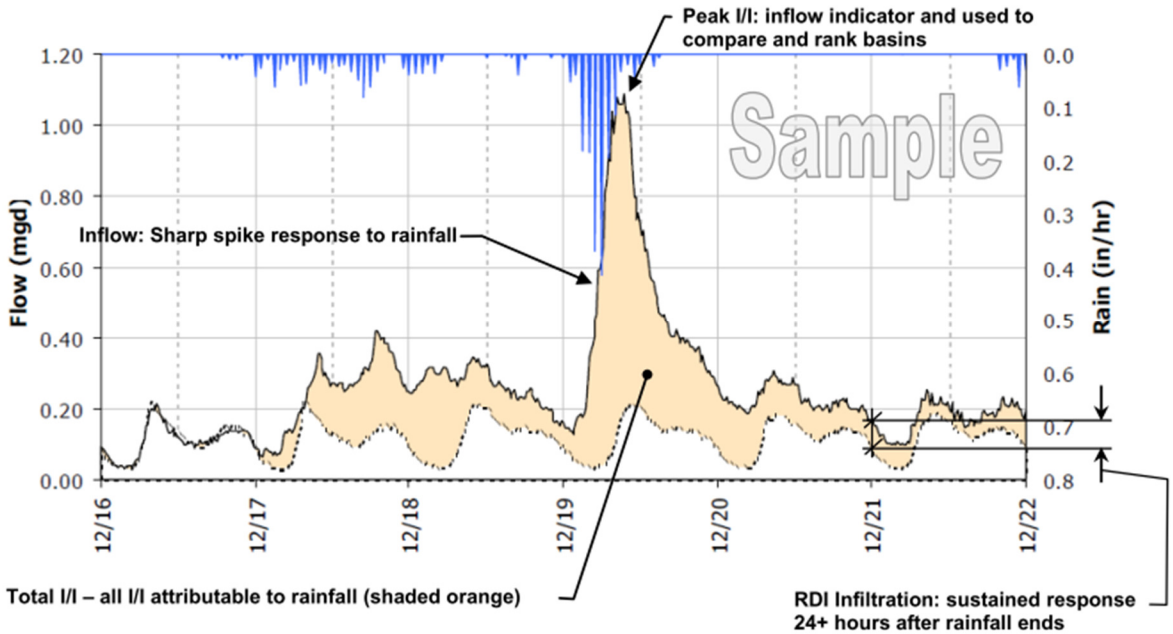


Figure 2-5. Sample Infiltration and Inflow Isolation Graph

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[1].
- **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.

[1] 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.

- **Rainfall-Dependent Infiltration:** Infiltration occurring after the conclusion of a storm event is classified as rainfall-dependent infiltration (RDI). RDI Analysis is conducted by looking at the infiltration rates at set periods after the conclusion of a storm event. Depending on the particular collection system and the time required for flows to return to ADWF levels, different set 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 method in this report.

3 Rainfall Results

3.1 Rainfall Monitoring

V&A installed one rain gauge for the duration of the project. There were four small storms that occurred over the course of the flow monitoring period, summarized in Table 3-1 and illustrated in Figure 3-1. The November 20 – 21, 2016 storm caused the greatest I/I response in the City collection system.

Table 3-1. Rainfall Events Used for I/I Analysis

Rainfall Event	Rainfall (in)
November 20 – 21, 2016	0.53
November 26, 2016	0.13
December 15 – 16, 2016	0.21
December 23, 2016	0.10
Total over Monitoring Period	*1.07*

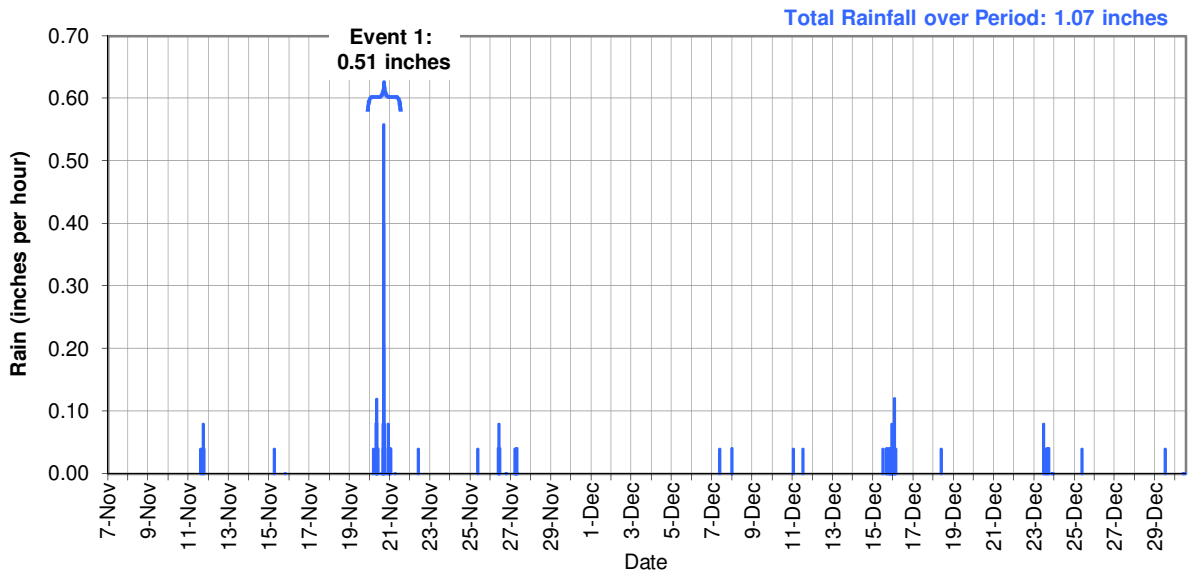


Figure 3-1. Rainfall Activity over Monitoring Period

Figure 3-2 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[2] in King City during this project duration. Rainfall totals for King City were approximately 42% of normal levels during this time period.

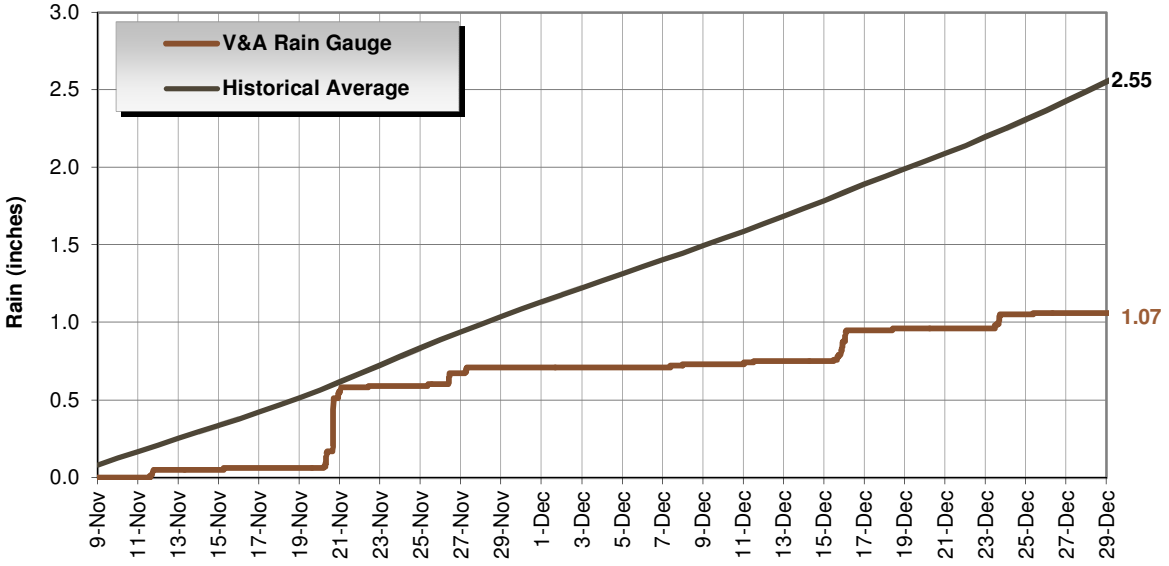


Figure 3-2. Accumulated Precipitation

[2] Historical data taken from the WRCC (triangulated from Station 44555 in King City and 49792 at Pinnacles NM): 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 NOAA Rainfall Frequency Atlas shown in Figure 3-3 (NOAA Western U.S. Precipitation Frequency Maps Atlas 2, 1973: http://www.wrcc.dri.edu/pcpnfreq.html) classifies a 10-year, 24-hour storm event at King City rain gauge as 3.17 inches. This means that in any given year, at this specific location, there is a 10% chance that 3.17 inches of rain will fall in any 24-hour period.

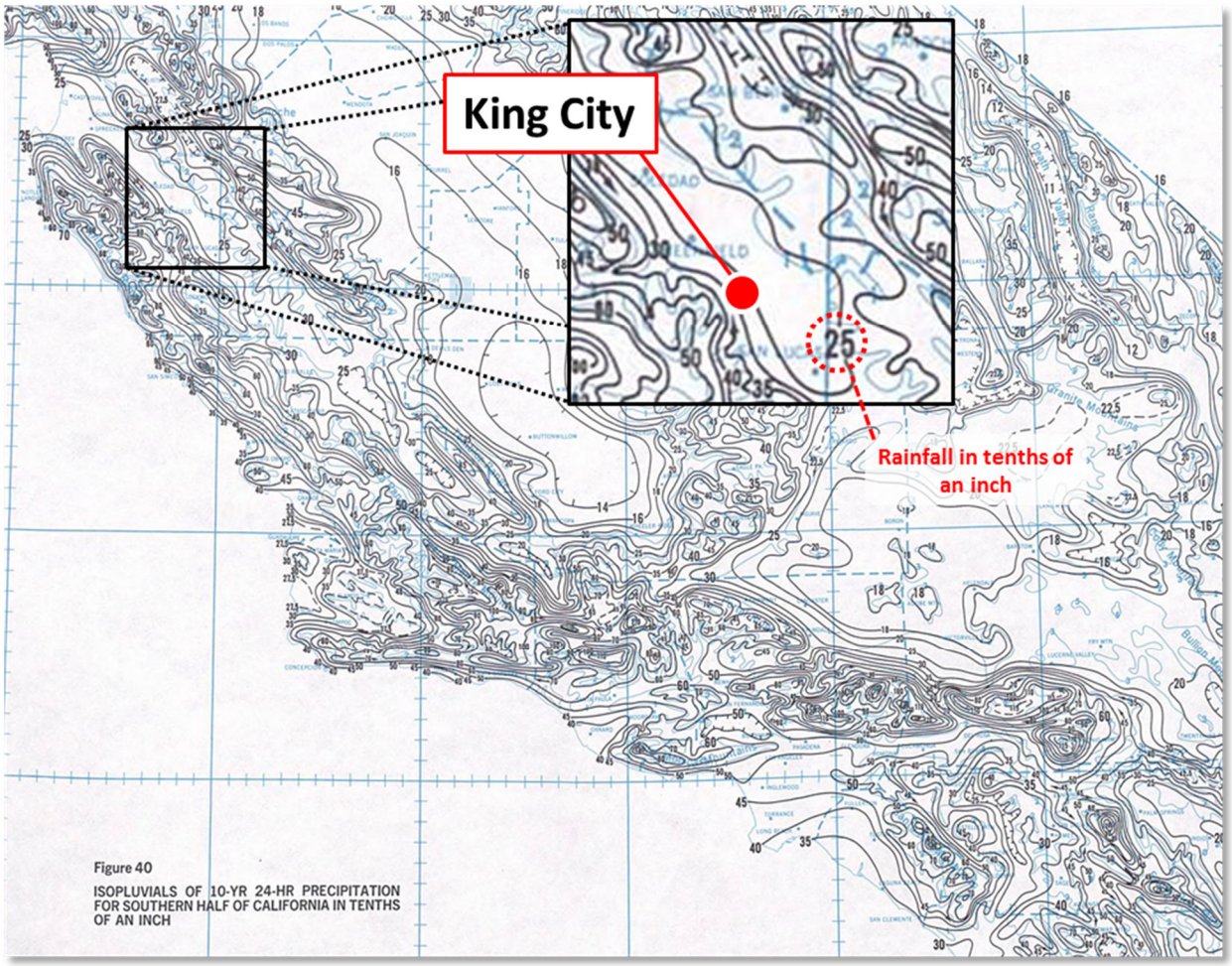


Figure 3-3. NOAA Northern California Rainfall Frequency Map (10-Year, 24-Hour IDF)

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 all the rainfall events on the rain event frequency plot determines the classification of the storm events, shown in Figure 3-4 for the one hour duration and in Figure 3-5 for the 24-hour duration. Only the November 20/21 rainfall event was analyzed.

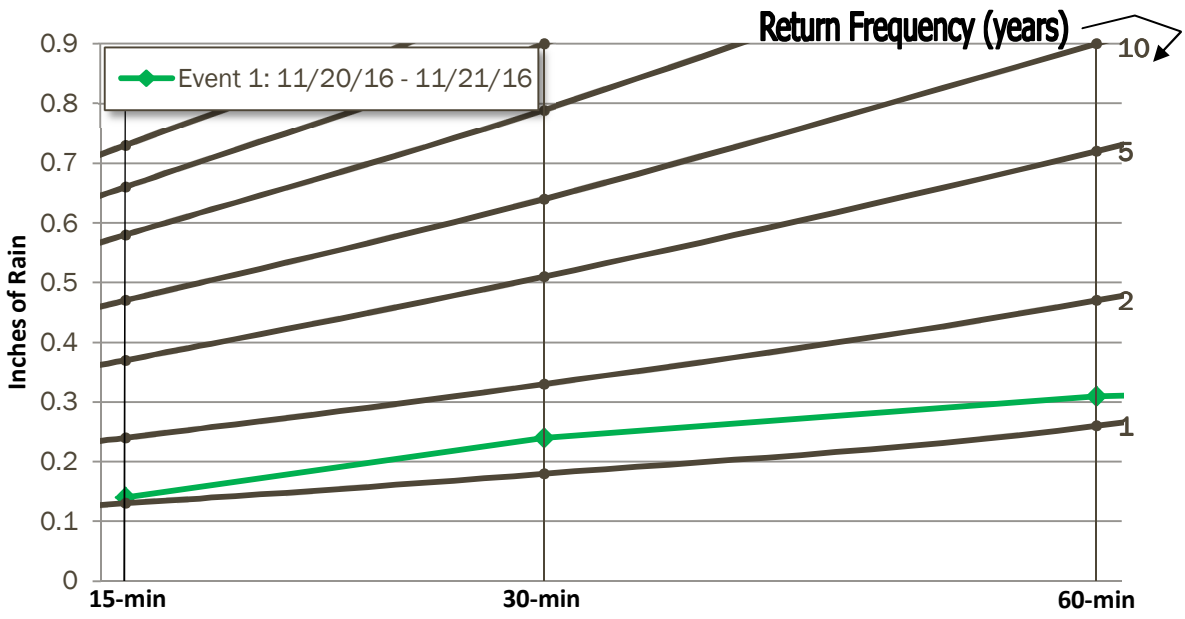


Figure 3-4. Rainfall Event Classification – 1 Hour

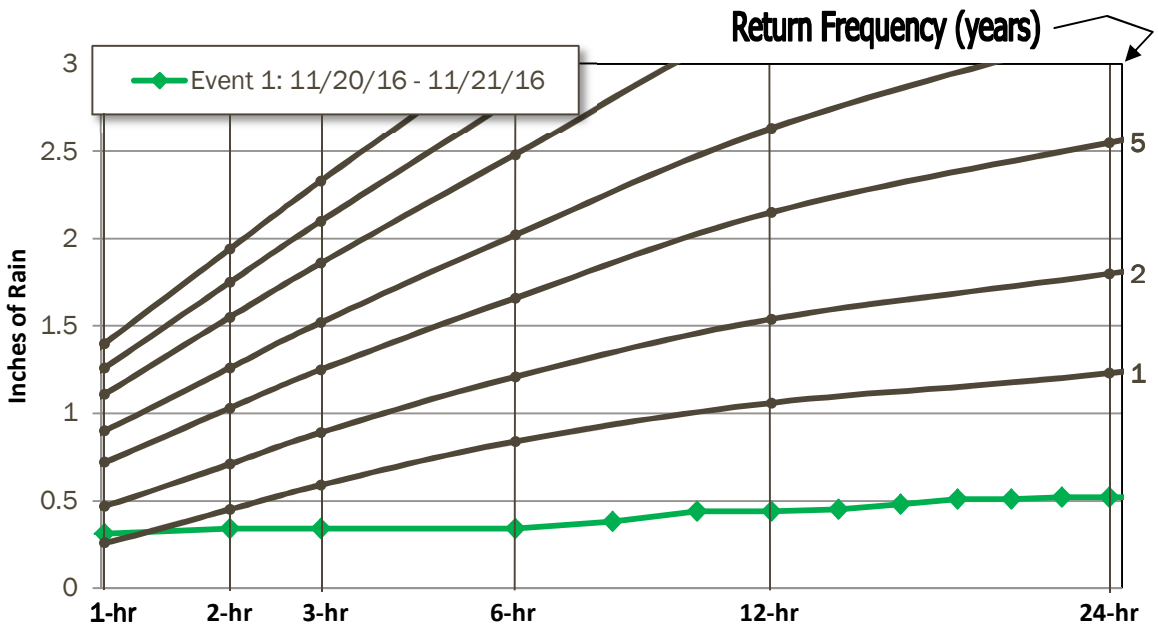


Figure 3-5. Rainfall Event Classification – 24 Hour

The November 20/21 rainfall event was classified as a 1.4-year, 30-minute storm event and a 1.2-year, 1-hour storm event.

4 Flow Monitoring Results

4.1 Average Flow Analysis

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 schematic diagram of the average dry weather flows and flow levels.

Table 4-1. Dry Weather Flow Summary

Monitoring Site	Mon- Thu ADWF (mgd)	Friday ADWF (mgd)	Saturday ADWF (mgd)	Sunday ADWF (mgd)	Overall ADWF (mgd)
Site 1	0.117	0.107	0.128	0.118	0.117
Site 1B	0.017	0.017	0.020	0.018	0.018
Site 2	0.563	0.546	0.597	0.599	0.571
Site 3A	0.064	0.054	0.061	0.062	0.062
Site 3B	0.040	0.039	0.044	0.043	0.041
Site 4	0.078	0.077	0.074	0.072	0.077
Site 5	0.108	0.104	0.105	0.107	0.107
Site 6	0.096	0.085	0.097	0.097	0.095
Site 7	0.062	0.047	0.047	0.046	0.056
Site 8	0.011	0.008	0.003	0.004	0.009

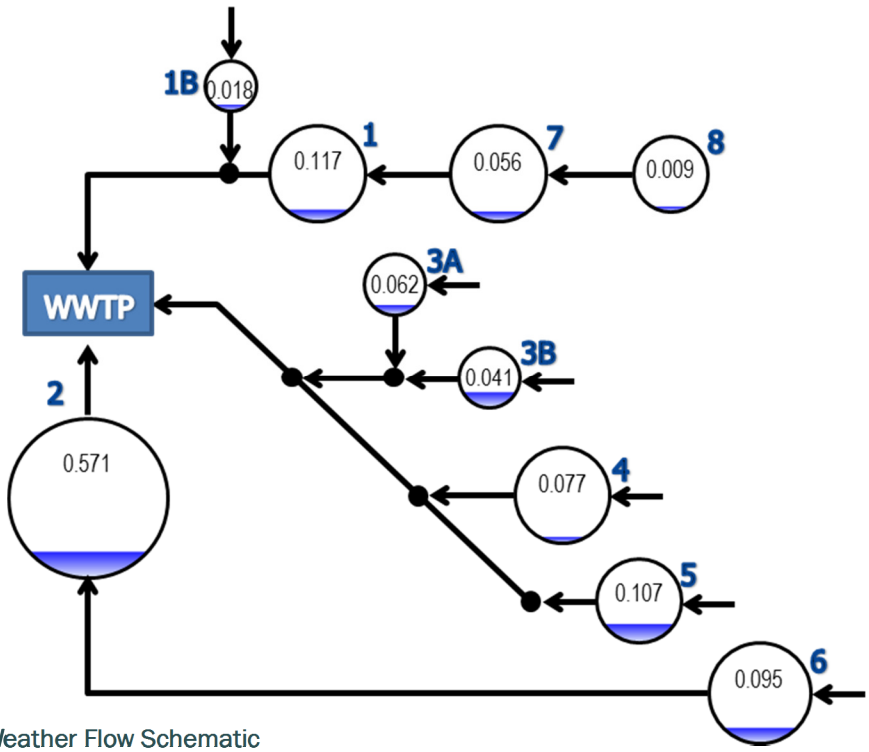


Figure 4-1. Dry Weather Flow Schematic

4.2 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.

The following capacity analysis terms are defined as follows:

- **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, proximity to pump stations, and the amount and characteristics of I/I entering the collection system. Flow attenuation and flow restrictions will also affect the peaking factor. A peaking factor threshold value of 3.0 is commonly used for sanitary sewer design of new pipe; however, it is noted that this value is variable and subject to attenuation and the size of the upstream collector area. The City should follow its own standards and criteria when examining peaking factors.
- **d/D Ratio:** The d/D ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). Standards for d/D ratio vary from agency to agency, but typically range between d/D ≤ 0.5 and d/D ≤ 0.75. The d/D ratio for each site was computed based on the maximum depth of flow for the flow monitoring study.

Backflow and Flush Condition

Sites 2 and 8 both had a "backflow and flush" condition occur during the course of the flow monitoring study

- Site 2, November 30, 2016. For approximately 2.5 hours, there was a backflow condition with steadily increasing levels and reduced velocities. Site 2 is located near the treatment facility; it is assumed this event was due to treatment plant operations. When relieved, the resulting "flush" of flow resulted in the peak measured flow for this site of 1.25 mgd. A similar event occurred on December 16, 2016 for 6 hours.
- Site 8, December 2 – 3, 2016: For approximately 24 hours, flows levels steadily increased and surcharged to 3.4 feet above the pipe crown. During this period velocity measurements were at zero, indicating stagnant flows. It is estimated that there was a downstream blockage in the pipeline. When the blockage was cleared, the responding "flush" of flow resulted in the peak measured flow during the period of 0.199 mgd.
- These flows were not considered to be flows measured during normal or routine dry or wet weather conditions. Refer to Figure 4-2.

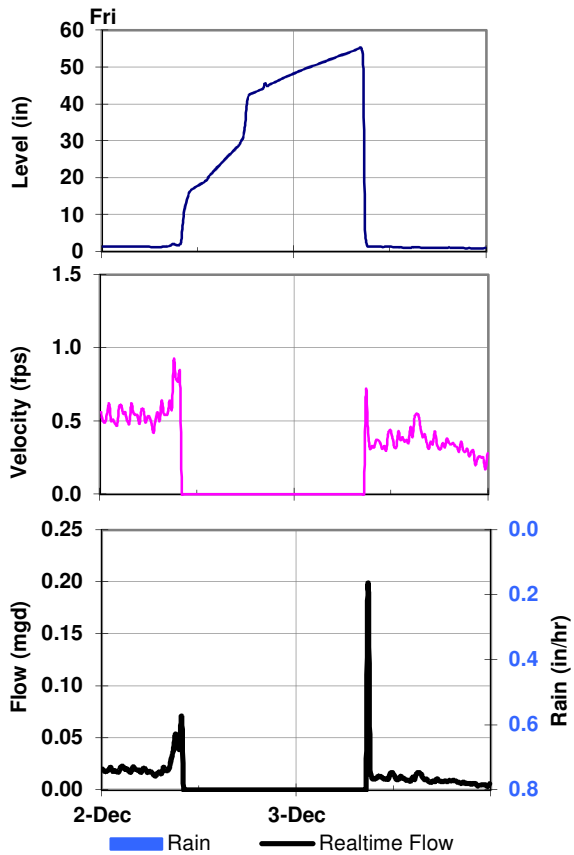


Figure 4-2. Site 2 Hydrographs, Dec 2 – 3, 2016

Table 4-2 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 are 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 4-2. Capacity Analysis Summary

Metering Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Pipe Diameter, *D* (in)	Max Depth, *d* (in)	*d/D* Ratio	Surcharge above Pipe Crown (ft)
Site 1	0.117	0.976	8.3	18	4.5	0.25	-
Site 1B	0.018	0.071	4.0	9.625	2.3	0.24	-
Site 2 [A]	0.571	1.076	1.9	30	6.3	0.21	-
- Flushing Event		*1.251*	*2.2*		*35.1*	*1.2*	*0.4*
Site 3A	0.062	0.137	2.2	11.5	3.0	0.26	-
Site 3B	0.041	0.101	2.5	11.5	4.0	0.35	-
Site 4	0.077	0.241	3.1	18	1.9	0.11	-
Site 5	0.107	0.269	2.5	16	4.6	0.29	-
Site 6	0.095	0.233	2.5	19	4.2	0.22	-
Site 7	0.056	0.465	8.4	18	4.3	0.24	-
Site 8 [A]	0.009	0.152	17.8	14	4.2	0.30	-
- Flushing Event		*0.199*	*23.3*		*55.4*	*4.0*	*3.4*

[A] Peak values as recorded during "normal" operation, not during a unique backflow, blockage or flushing event.

The following capacity analysis results are noted:

- **Peaking Factor:** Sites 1, 7 and 8 had peaking factors greater than 8.0 during normal operating conditions. Sites 1, 7 and 8 are connected; Site 8 flow are conveyed to Site 7 and Site 7 flows are conveyed to Site 1.
- **d/D Ratio:**
  - During normal operation, none of the flow monitoring sites had a maximum *d/D* ratio that exceeded a d/D value of 0.75.
  - Sites 2 and 8 had backflow events that caused a surcharged condition. The Site 2 backflow was assumed to be caused by treatment plant operations. The Site 8 surcharge was assumed to be caused by a downstream pipe blockage or restriction.

Figure 4-3 and Figure 4-4 show bar graphs of the capacity results. Figure 4-5 illustrates a flow schematic showing the peak flows per site with the corresponding flow depths (note: during normal operation, does not include backflow or flushing events).

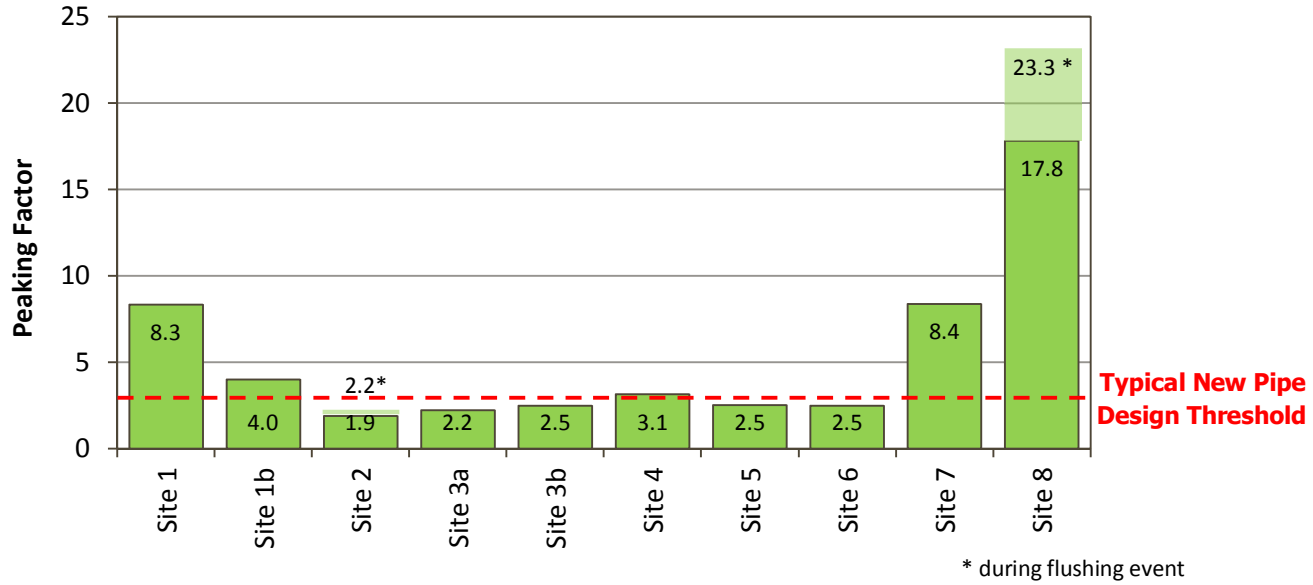


Figure 4-3. Capacity Summary: Peaking Factors

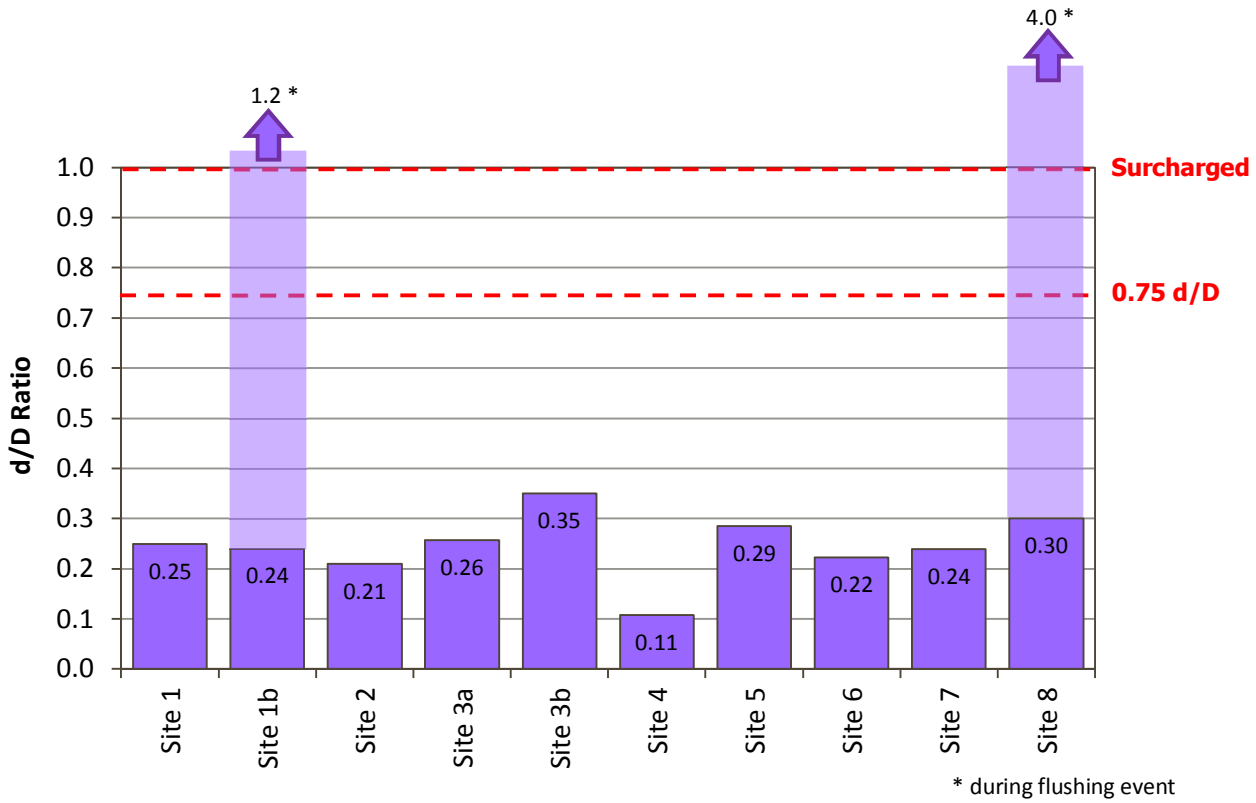


Figure 4-4. Capacity Summary: Max d/D Ratios

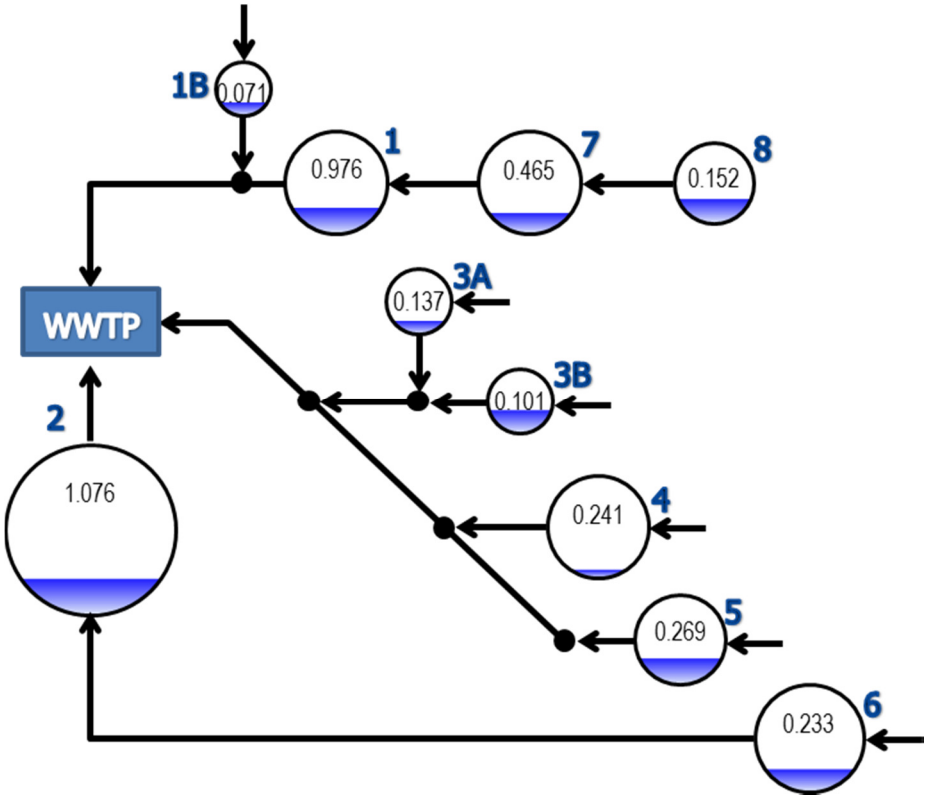


Figure 4-5. Peak Flows, Flow Schematic (normal operation)

5 Inflow and Infiltration Results

5.1 Preface

Due to the low rainfall totals captured in King City, the timing of the installation of the flow metering devices and the characteristics of the I/I response curves, the I/I analyses discussed will be handled as a site-by-site discussion of inflow and infiltration. I/I analyses typically separates the I/I response curves into different components signifying different types of response:

- **Inflow:** Storm water discharged into the sewer system from direct connections such as downspouts, yard and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins. Graphically signified by a spike in flows that mirrors the shape of the rainfall intensity and with quick return to baseline flows.
- **Rainfall-Dependent Infiltration:** Storm 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. Graphically signified by a gradual increase in flow levels, and also a gradual decline to baseline flows.
- **Groundwater Infiltration:** Groundwater infiltration that enters the system by the same means as rainfall dependent infiltration, but via the groundwater table. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.

For the flows measured in this study, the rain-dependent infiltration and groundwater infiltration components were negligible and not analyzed. For example, the I/I isolation response curve for Site 2 is illustrated in Figure 5-1.

Site 2 had the largest measured flows and largest collection area of the 10 flow monitoring sites. The I/I response curve for the largest rainfall event shows that there was a definitive and measurable I/I response (shown by the red line). However, the flows attributable to I/I return very quickly back to baseline flow levels, indicating minimal rain dependent infiltration.

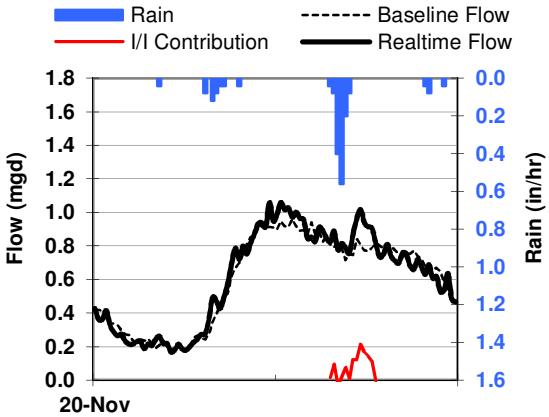


Figure 5-1. I/I Isolation Curve, Site 2

The discussions within this chapter are focused primarily on inflow.

5.2 Site-by-Site I/I Discussion

The I/I discussion will review the observed I/I responses for the November 20, 2016 rainfall event (classified as a 1.2-year, 1-hour storm event) and for the December 15/16, 2016 rainfall event.

5.2.1 November 20, 2016 Rainfall

For the November 20, 2016 event, Sites 1, 2, 4, 7 and 8 were installed and operational. Given the short-duration intensity of the rainfall, this event should identify sites with high inflow contribution.

Site 8

Site 8 is discussed first; it is upstream from both Sites 1 and 7 and many of the discussion topics for Site 8 relate directly to Sites 1 and 7. Figure 5-2 illustrates the flow hydrograph during the week of November 16 – 23, 2016. The following items are noted:

- There appeared to be a clear inflow response to the November 20 rainfall event.
- There were several other peak discharge events of similar magnitude not related to rainfall.

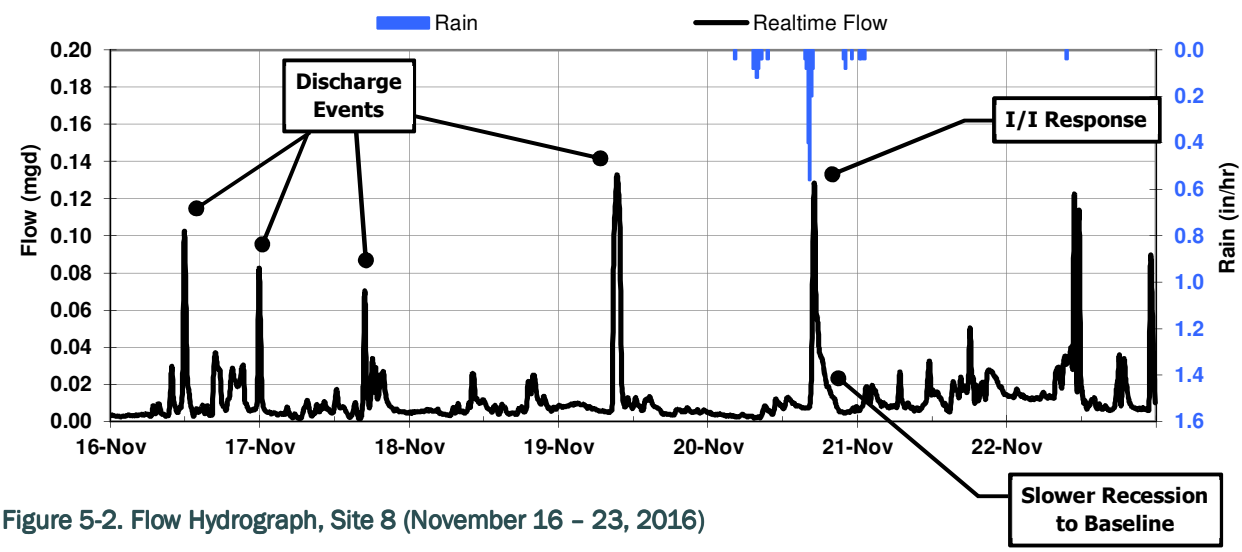


Figure 5-2. Flow Hydrograph, Site 8 (November 16 – 23, 2016)

V&A following bullet items are presented to review the possibility that the apparent I/I response could be misinterpreted as a peak discharge event:

- The November 20 spike occurred at a 15 to 30 minute delay with the high intensity rainfall. This delay is typical and expected for flow monitoring sites, and corresponds in timing to other flow monitoring sites for this rainfall event.
- For the November 20 spike, the shape of the post-rainfall recession was more indicative of inflow response; the peak discharge events had a much sharper and more immediate decline to baseline flows.
- There were 22 peak discharge events (not relating to rainfall) of magnitude 0.06 mgd or higher over the course of the flow monitoring study. Using this as a relative potential frequency of occurrence, the odds of a peak discharge event aligning with the timing of the rainfall are 0.5%.
- V&A's opinion is that the November 20 spike was inflow response to the rainfall event and not a coincidental peak discharge event.

Site 8 had a clear and definitive inflow response, peaking at 0.124 mgd. The ratio of peak I/I to ADWF was calculated as 14.5. This is a high I/I to ADWF ratio and suggests that there is a strong inflow component within Basin 8.

Site 7

Site 7 was located downstream from Site 8; the peak discharge event items discussed within Site 8 apply to Site 7.

Site 7 had a clear and definitive inflow response, peaking at 0.262 mgd. The ratio of peak I/I to ADWF was calculated as 4.7. This is a high I/I to ADWF ratio and suggests that there is a strong inflow component for the basin upstream from Site 7. The peak I/I response increased from Site 8 to Site 7, from 0.124 to 0.262 mgd respectively, indicating that over half of the peak I/I is generated within Basin 7 (independent from Basin 8). Basin 7 has a strong inflow component.

Site 1

Site 1 was located downstream from Site 7 and Site 8; the peak discharge event items discussed within Site 8 apply to Site 1.

Site 1 had a clear and definitive inflow response, peaking at 0.495 mgd. The ratio of peak I/I to ADWF was calculated as 4.2. This is a high I/I to ADWF ratio and suggests that there is a strong inflow component for the basin upstream from Site 1. The peak I/I response increased from Site 7 to Site 1, from 0.262 to 0.495 mgd respectively, indicating that nearly half of the peak I/I is generated within Basin 1 (independent from Basins 7 and 8). Basin 1 has a strong inflow component.

Site 2

Site 2 had a clear and definitive inflow response, peaking at 0.22 mgd (refer to Figure 5-1). The ratio of peak I/I to ADWF was calculated as 0.38. This is a relatively low ratio and suggests the following items in relation to inflow response relating to Site 2:

- There was a relatively minimal inflow response within the basin upstream from Site 2.
- It is possible the effects of attenuation (Chapter 2.5) may have slightly dampened the peak I/I response.

Site 4

Site 4 did not have a clear and definitive inflow response (refer to Figure 5-3). The realtime flow less the baseline flow defined the I/I Contribution and it is observed that the red line response at the time of the storm event is similar to the noise observed at time prior to the rain event.

- There was minimal or negligible inflow response within the basin upstream from Site 4 for the November 20, 2016 rainfall event.

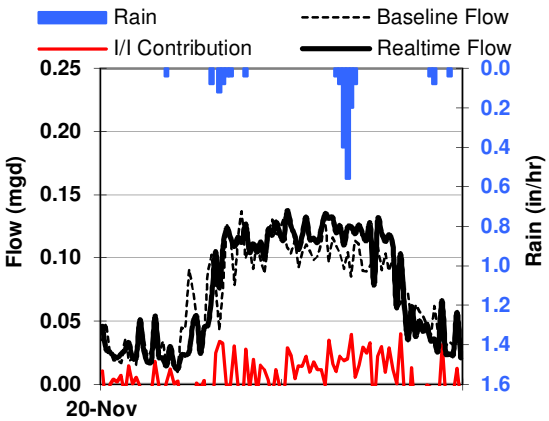


Figure 5-3. I/I Isolation Curve, Site 4

5.2.2 December 15/16, 2016 Rainfall

The other rainfall event that could be analyzed was the December 15/16 rainfall event. While this event did not have considerable rainfall, the most intense portions of the event occurred during the early morning hours when sanitary sewer flows are the most predictable; it is possible that real I/I response could be differentiated from the random noise that is observed during peak flow hours.

All sites were operational for this event, though the analyses for Site 1, 2, 4, 7 and 8 are considered only as supplementary to the preferred November 20, 2016 rainfall event analysis.

Sites 1B, 3A, 3B, 5 6

Site 1B: There was no definitive I/I response observed for Site 1B.

Site 3A: There was no definitive I/I response observed for Site 3A.

Site 3B: There was no definitive I/I response observed for Site 3B.

Site 5: There appeared to be an I/I response coinciding rain event during the early morning hours of December 16 (Figure 5-4), suggesting the potential for inflow in the basin upstream from Site 5. Additional flow monitoring during a larger rainfall event would be required to confirm this suggestion.

Site 6: There appears to be a lift station upstream from Site 6; the spikes in flow occur several times per day in patterns consistent with lift stations. The spike that occurred immediately after the rainfall event in the early morning hours of December 16 was had a stronger magnitude and was longer in duration that other typical spikes, suggesting possible inflow in the basin upstream from Site 6 (Figure 5-4). Additional flow monitoring during a larger rainfall event would be required to confirm this suggestion.

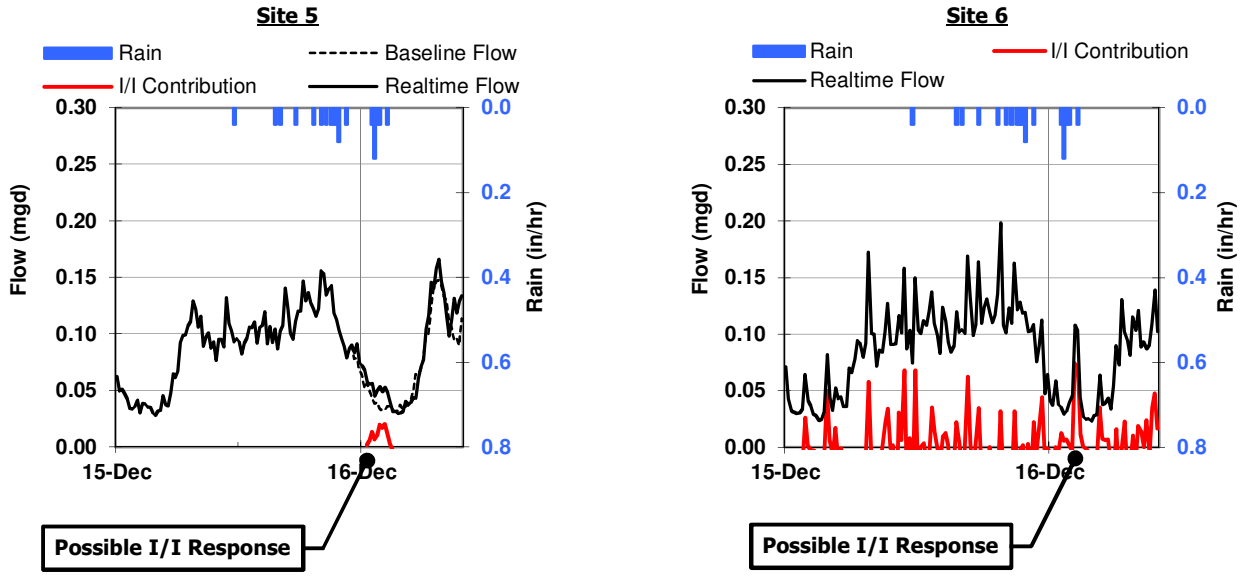


Figure 5-4. I/I Isolation Curves, Sites 5 and 6 (December 15/16, 2016)

Sites 1, 2, 4 7, 8

Site 1: The I/I response observed for Site 1 confirms the inflow discussion items from the November 20 rainfall event. Figure 5-5 shows the I/I response for Site 1 during the early morning hours of December 16, suggesting inflow response even during smaller rainfall events.

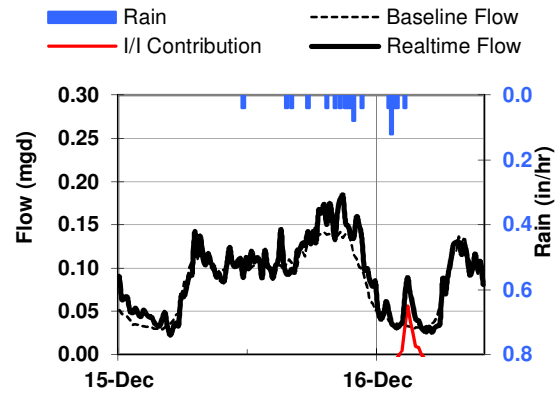


Figure 5-5. I/I Isolation Curve, Site 1

Site 2: Site 2 could not be analyzed; treatment facility operations had flows through this site in a backflow condition and I/I response could not be measured.

Site 4: Site 4 did not have a clear and definitive inflow response, in alignment with the discussion items from the November 20 rainfall event.

Site 7: The I/I response observed for Site 7 confirms the inflow discussion items from the November 20 rainfall event.

Site 8: The I/I response observed for Site 7 confirms the inflow discussion items from the November 20 rainfall event.

Table 5-1 summarizes the inflow analysis items discussed in this chapter.

Table 5-1. Inflow Analysis Summary

Site	Inflow Analysis Discussion
Site 1	Site 1 and Basin 1 had a definitive and strong inflow response.
Site 1B	There was no definitive I/I response observed for Site 1B.
Site 2	Site 2 had a definitive but relatively minimal inflow response.
Site 3A	There was no definitive I/I response observed for Site 3A.
Site 3B	There was no definitive I/I response observed for Site 3B.
Site 4	There was no definitive I/I response observed for Site 4.
Site 5	There was a suggestion of inflow observed for Site 5 (Basin 5)
Site 6	There was a suggestion of inflow observed for Site 6 (Basin 6)
Site 7	Site 7 and Basin 7 had a definitive and strong inflow response.
Site 8	Site 8 and Basin 8 had a definitive and strong inflow response.

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 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 greatest inflow problems. The highest inflow occurs in the in basins upstream from Sites 1, 7, and 8.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems. During this project, no rain dependent infiltration was detected. This may be due to the low rainfall amounts that occurred during the study.
2. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
    a. Smoke testing
    b. Mini-basin flow monitoring
    c. Nighttime reconnaissance work to (1) investigate and determine direct point sources of inflow and (2) determine the areas and pipe reaches responsible for high levels of infiltration contribution.
3. **I/I Reduction Cost-Effectiveness Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow and 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 King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 1

Location: San Antonio Drive between Mildred Avenue and Cemetery

Data Summary Report

Vicinity Map: Site 1

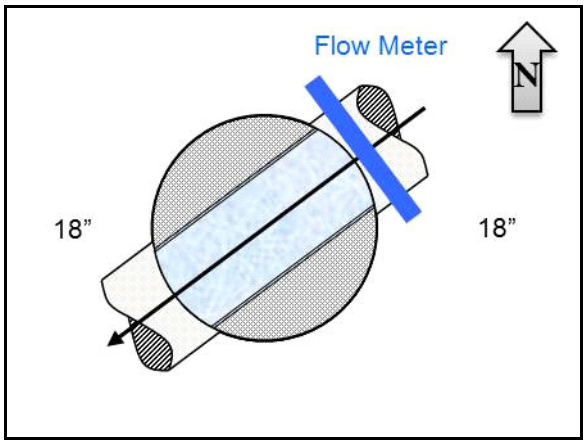
SITE 1

Site Information

Location:	San Antonio Drive between Mildred Avenue and Cemetery
Coordinates:	121.1390° W, 36.2158° N
Rim Elevation (Earth):	318 feet
Pipe Diameter:	18 inches
ADWF:	0.117 mgd
Peak Measured Flow:	0.976 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

Plan View

SITE 1

Additional Site Photos

Effluent Pipe

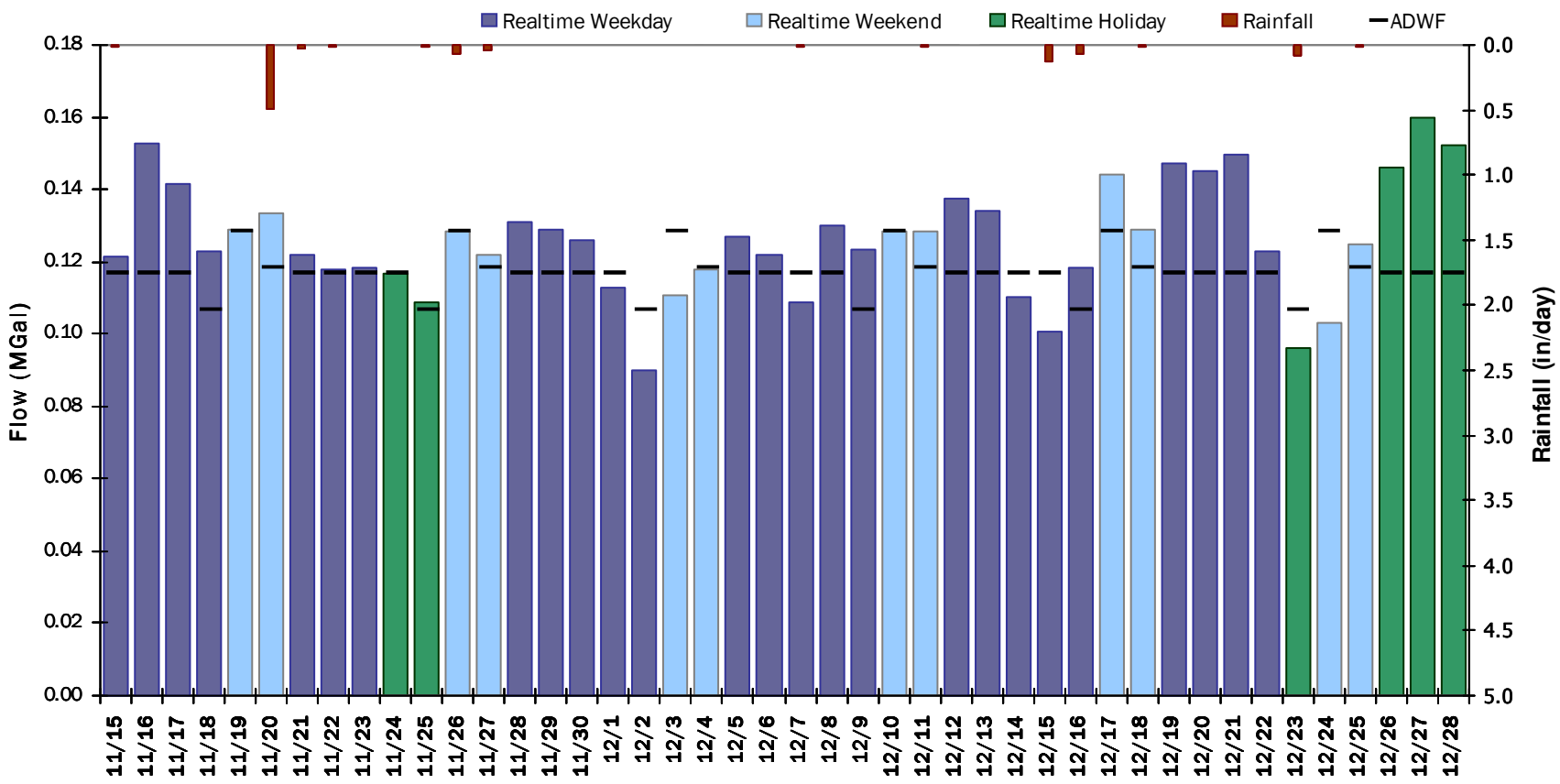
Influent Pipe

SITE 1

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.126 MGal **Peak Daily Flow: 0.160 MGal** **Min Daily Flow: 0.090 MGal**

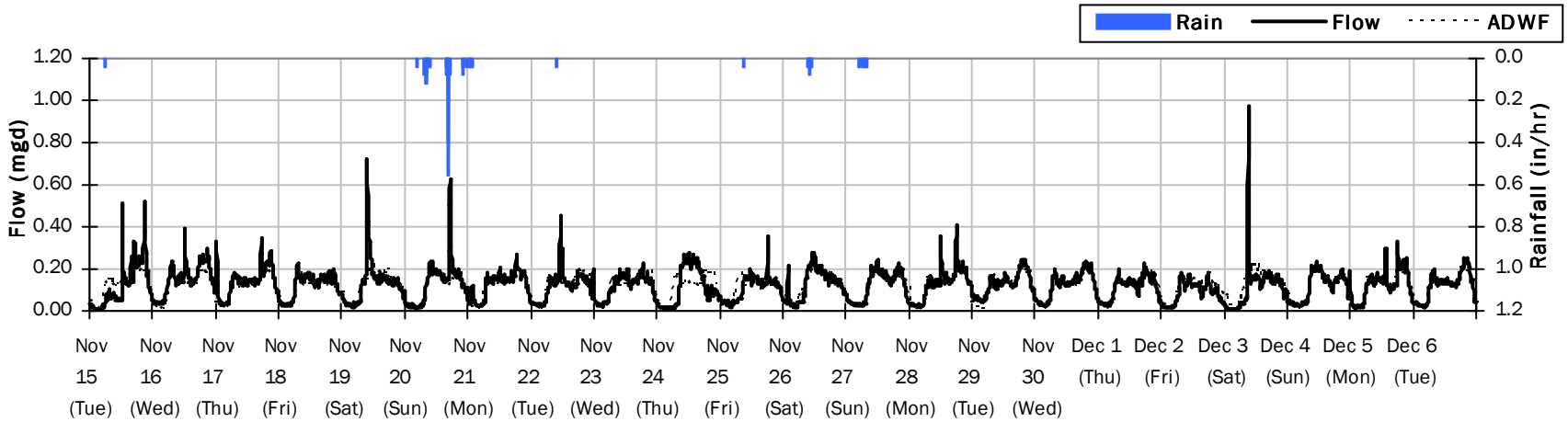
Total Period Rainfall: 1.01 inches

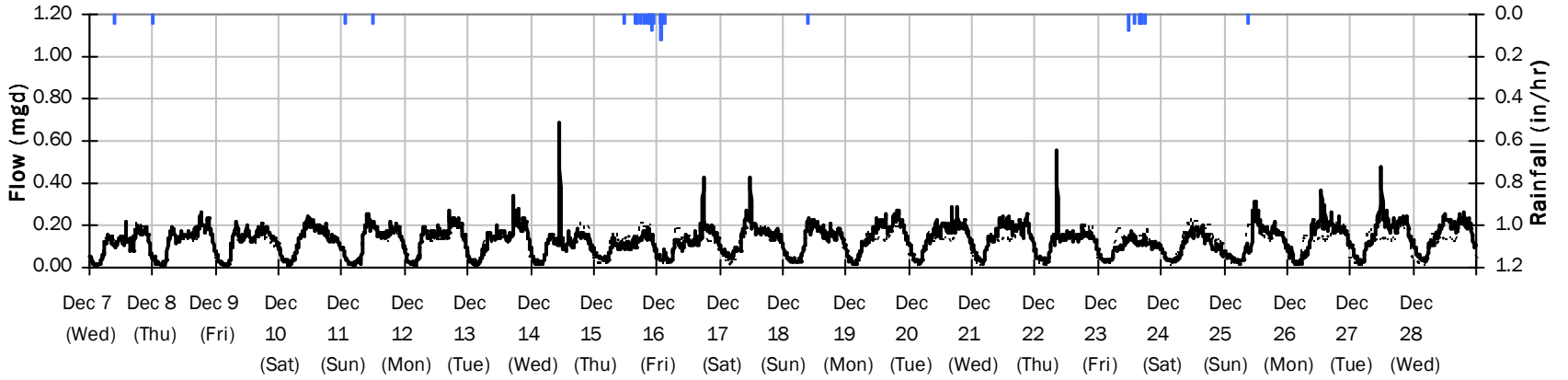


SITE 1

Flow Summary: 11/15/2016 to 12/28/2016

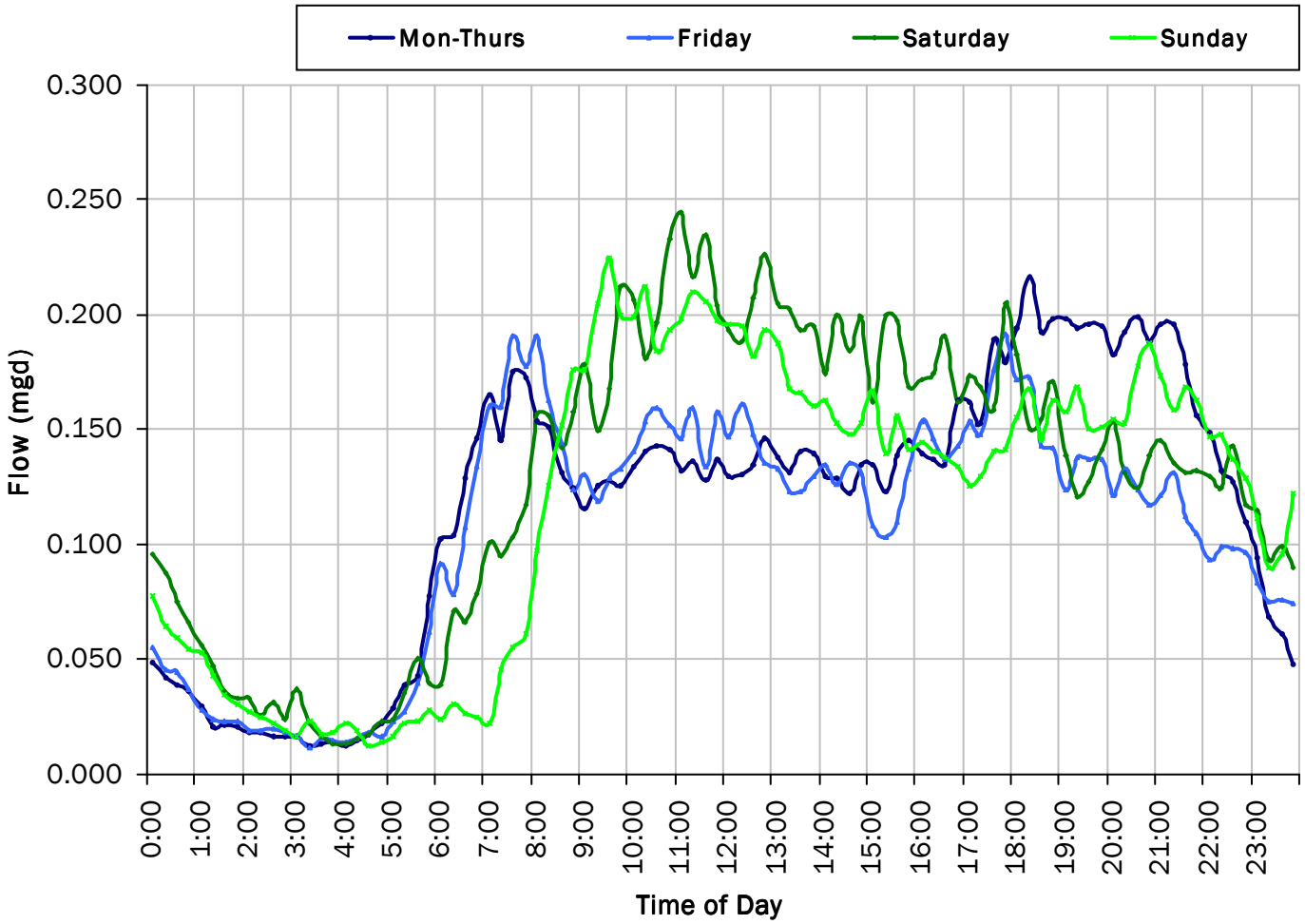
Total Period Rainfall: 1.01 inches **Avg Flow: 0.126 mgd** **Peak Flow: 0.976 mgd** **Min Flow: 0.007 mgd**





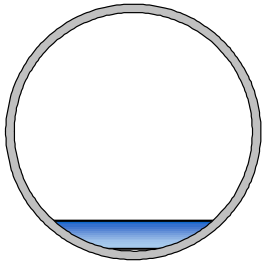
SITE 1

Average Dry Weather Flow Hydrographs



ADWF:

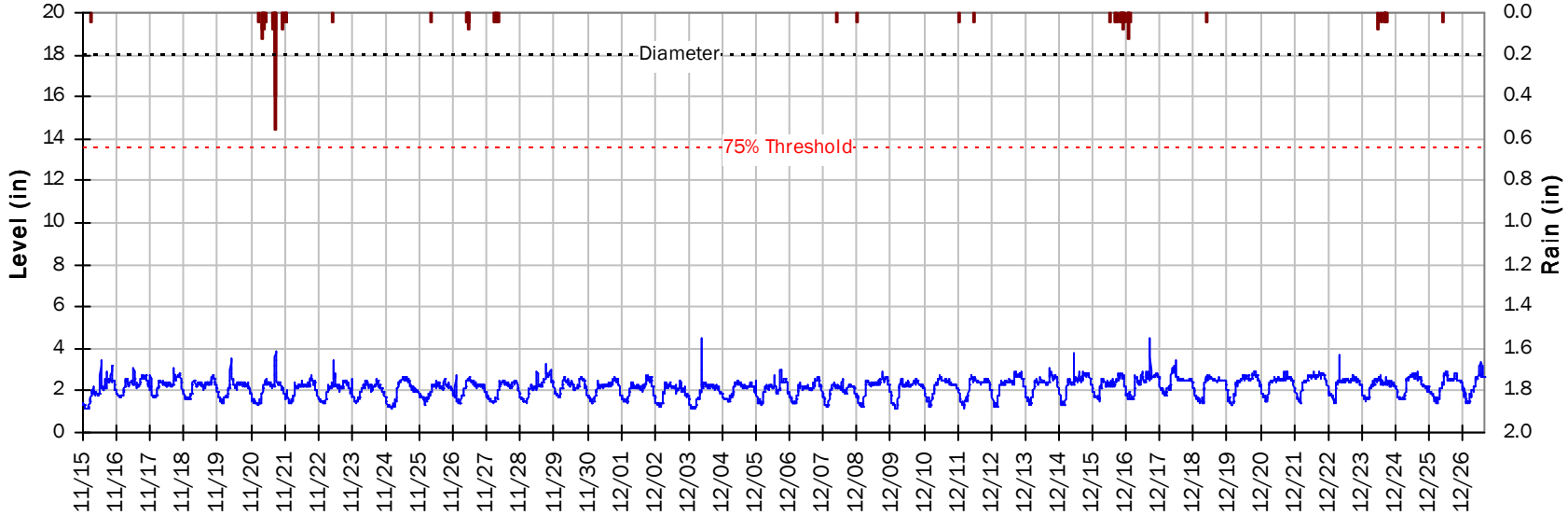
0.117 *mgd*

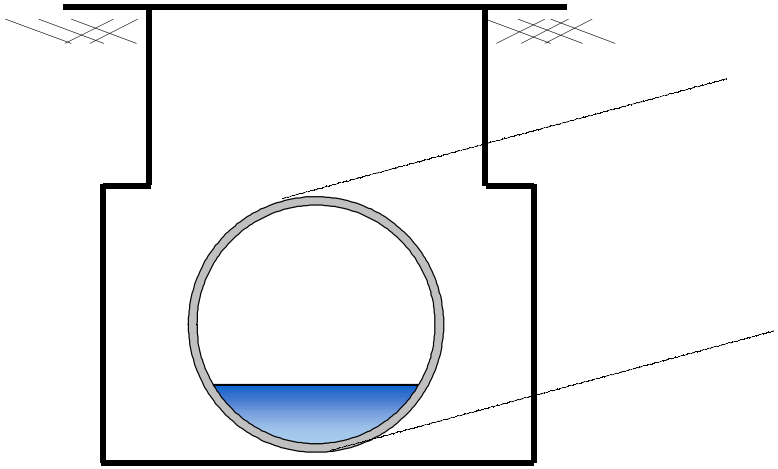


SITE 1

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



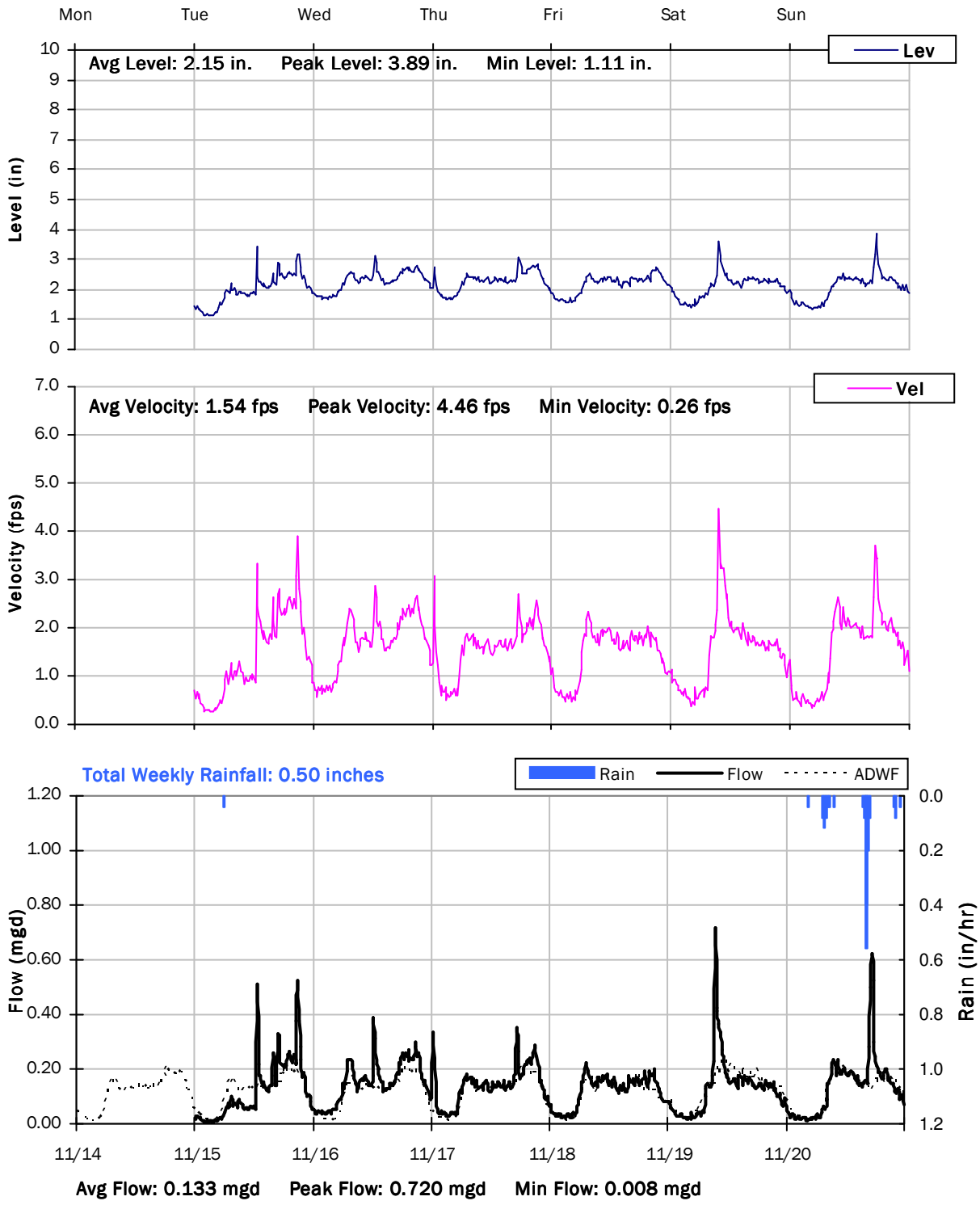


Pipe Diameter:	18	*inches*
Peak Measured Level:	4.5	*inches*
Peak d/D Ratio:	0.25	
Dry Weather Design Threshold Level:	13.5	*inches*

SITE 1

Weekly Level, Velocity and Flow Hydrographs

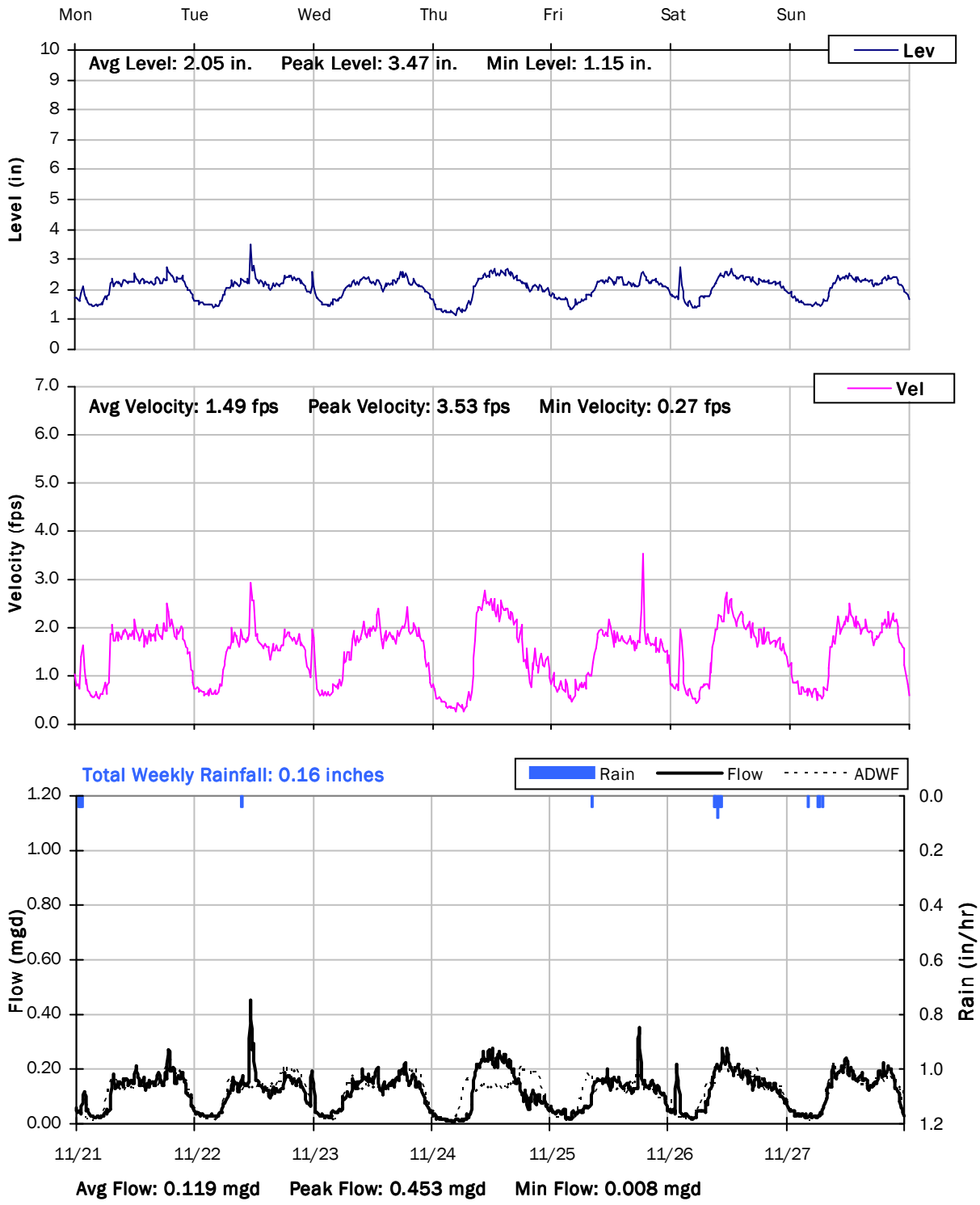
11/14/2016 to 11/21/2016



SITE 1

Weekly Level, Velocity and Flow Hydrographs

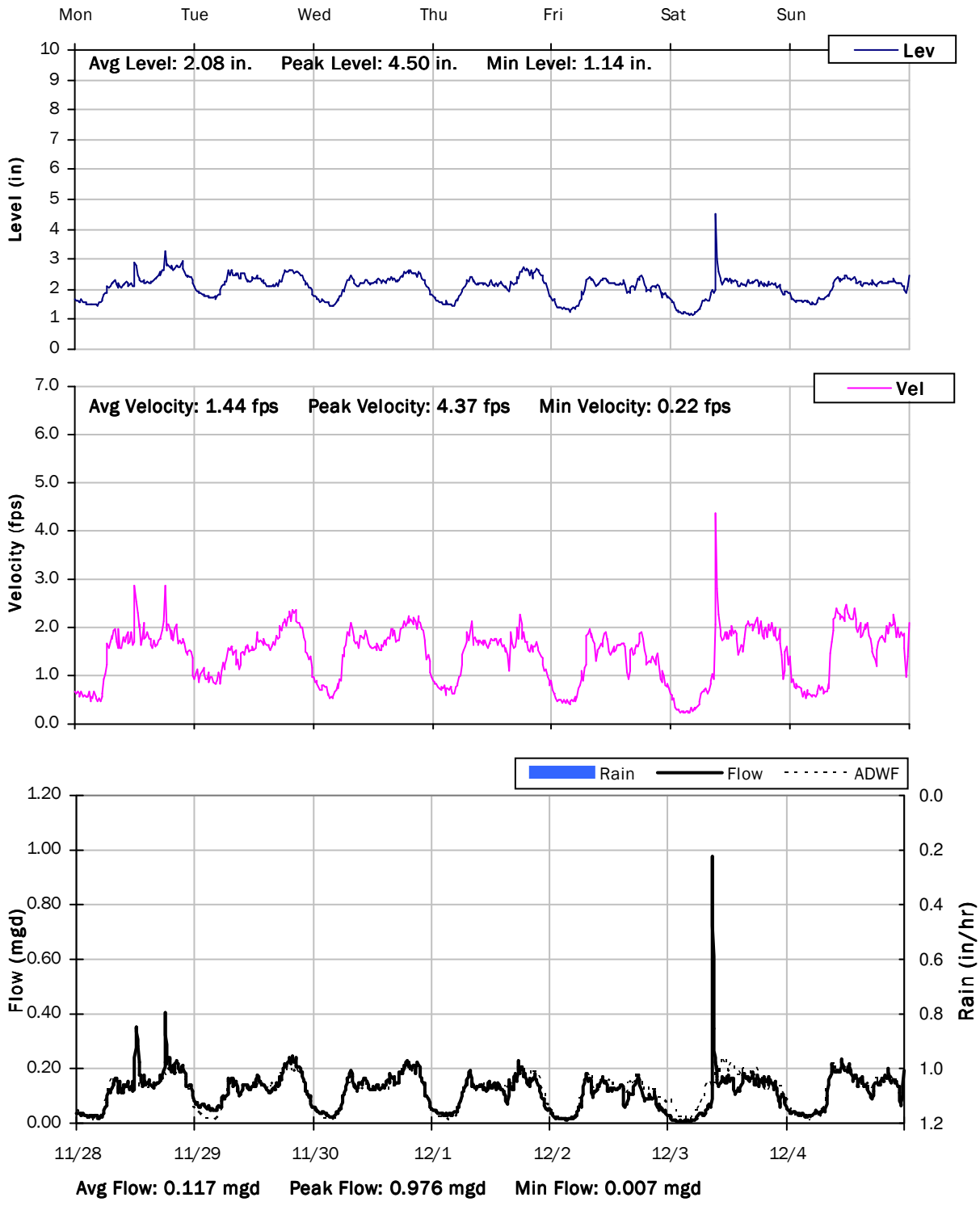
11/21/2016 to 11/28/2016



SITE 1

Weekly Level, Velocity and Flow Hydrographs

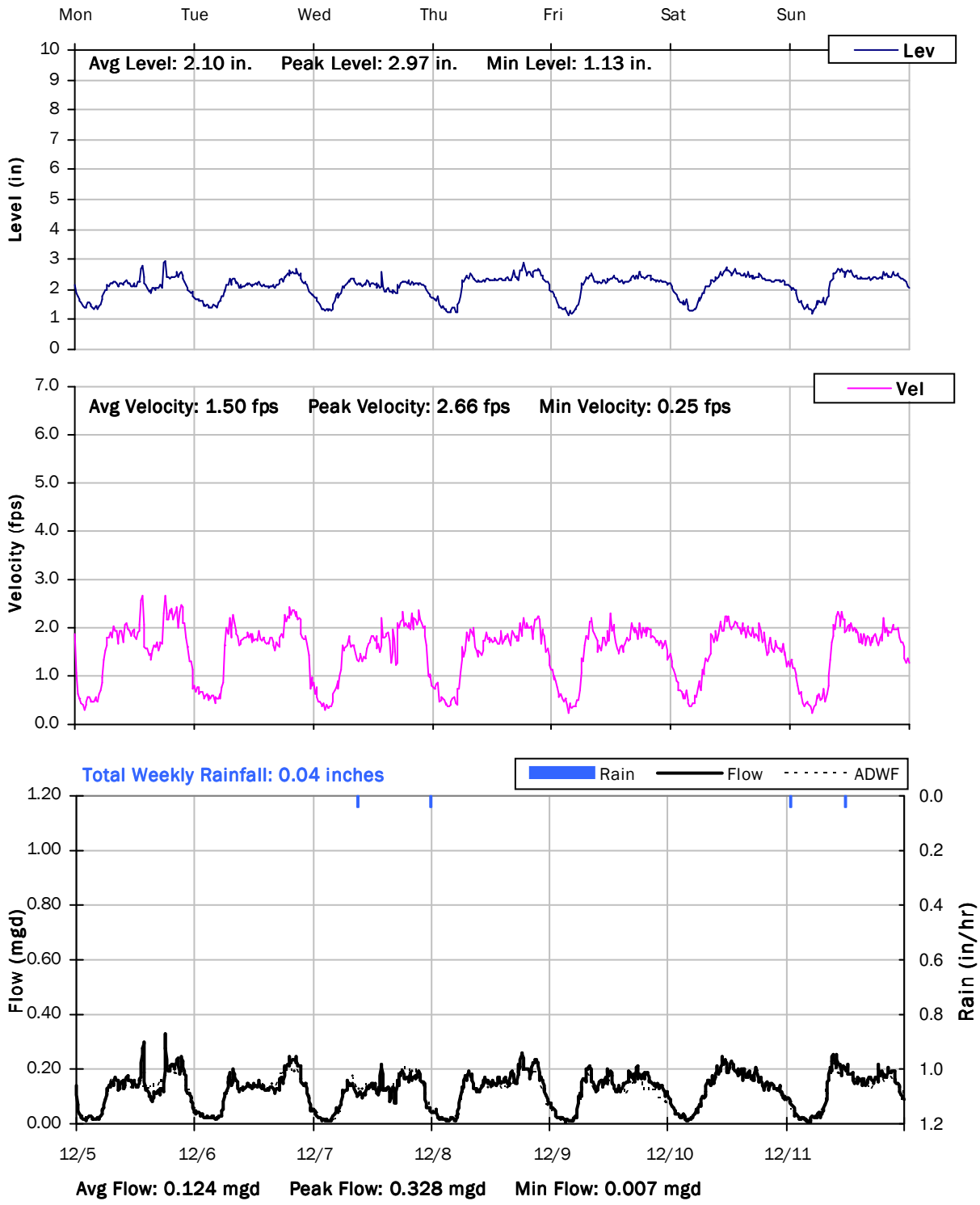
11/28/2016 to 12/5/2016



SITE 1

Weekly Level, Velocity and Flow Hydrographs

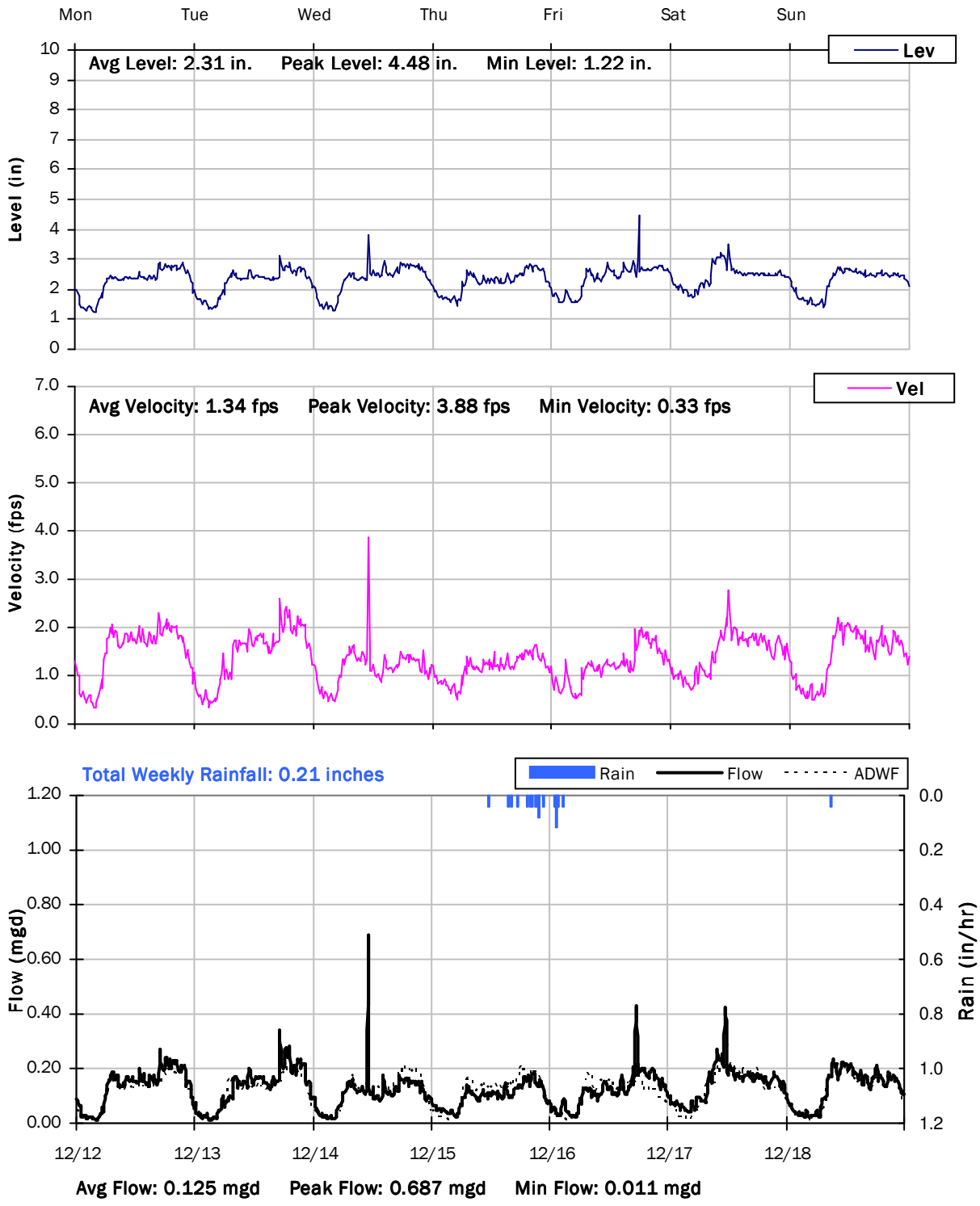
12/5/2016 to 12/12/2016



SITE 1

Weekly Level, Velocity and Flow Hydrographs

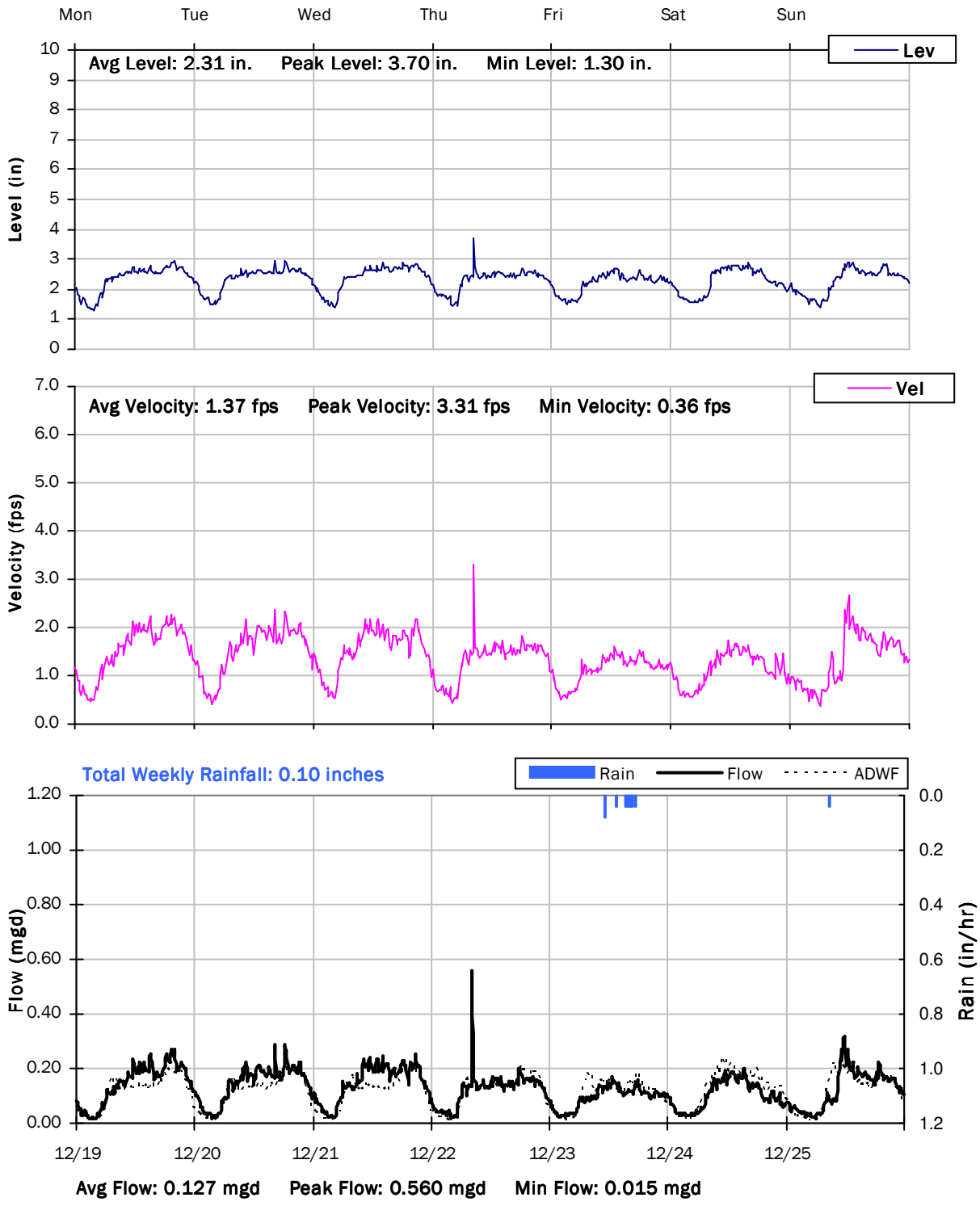
12/12/2016 to 12/19/2016



SITE 1

Weekly Level, Velocity and Flow Hydrographs

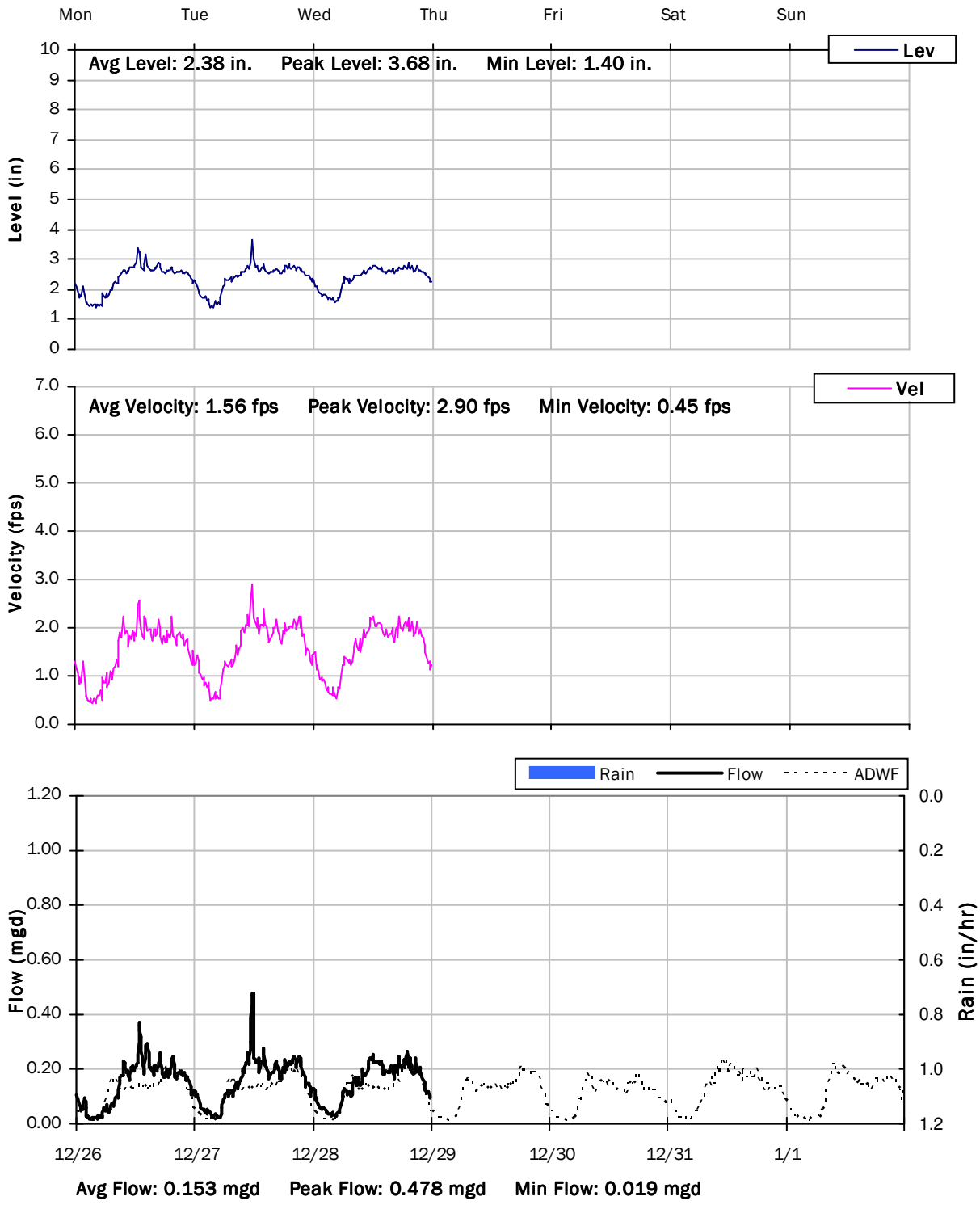
12/19/2016 to 12/26/2016



SITE 1

Weekly Level, Velocity and Flow Hydrographs

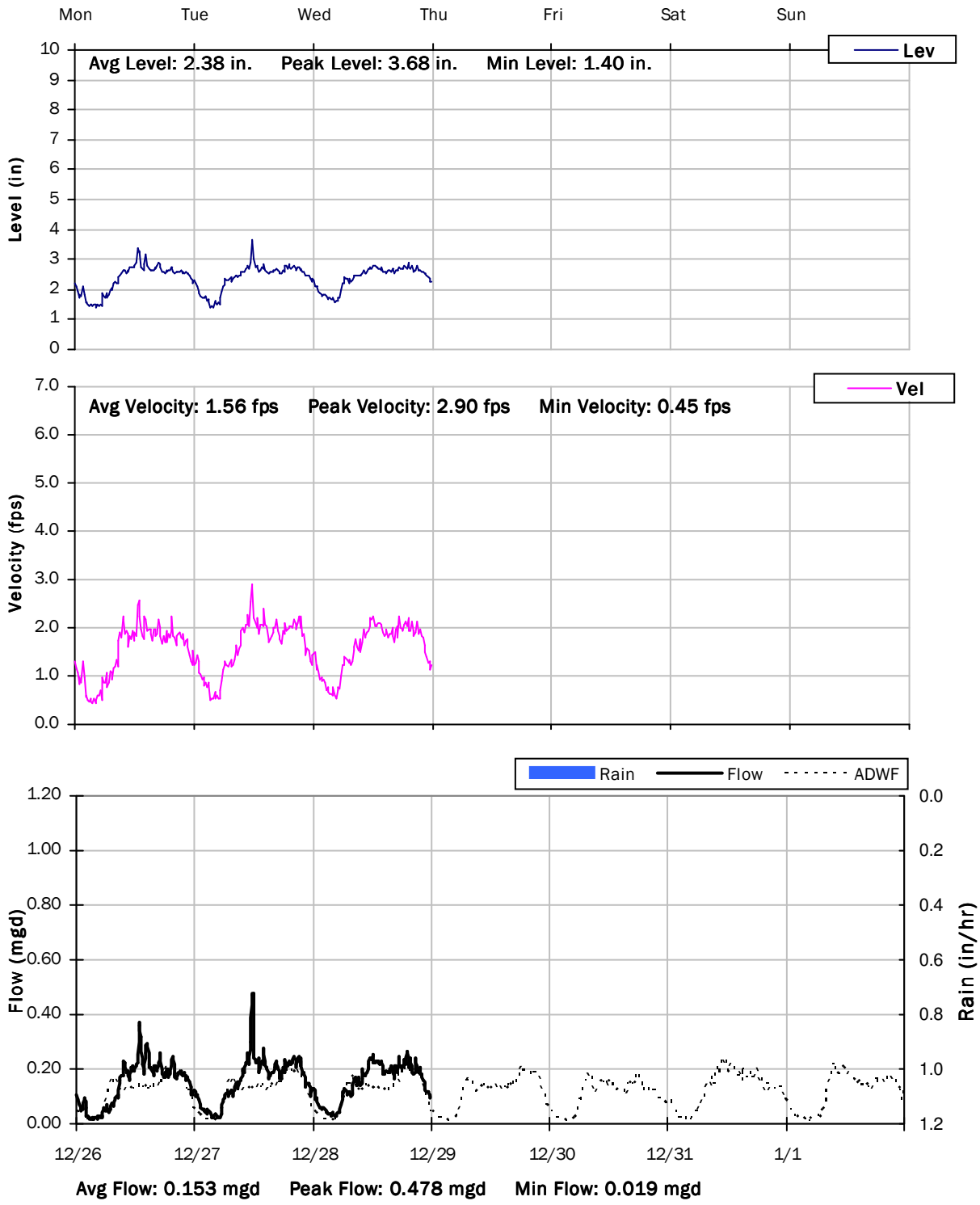
12/26/2016 to 1/2/2017



SITE 1

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 1b

Location: Mildred Avenue and San Antonio Drive

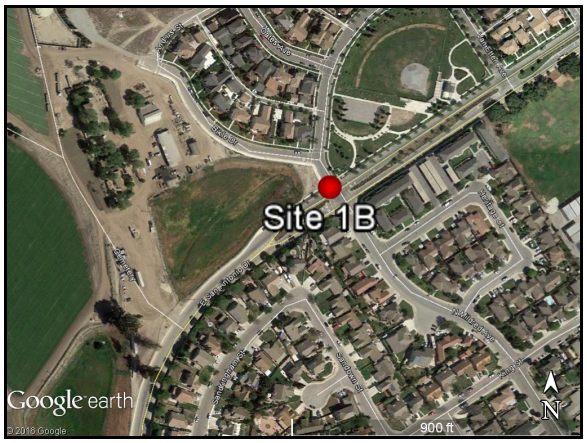
Data Summary Report

Vicinity Map: Site 1b

SITE 1B

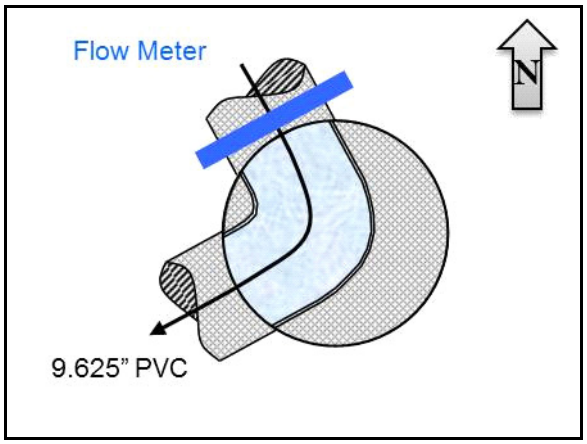
Site Information

Location:	Mildred Avenue and San Antonio Drive
Coordinates:	121.1381° W, 36.2164° N
Rim Elevation (Earth):	319 feet
Pipe Diameter:	9.625 inches
ADWF:	0.018 mgd
Peak Measured Flow:	0.071 mgd



Satellite Map

Sewer Map



Flow Sketch

Street View

Plan View

SITE 1B

Additional Site Photos

Effluent Pipe

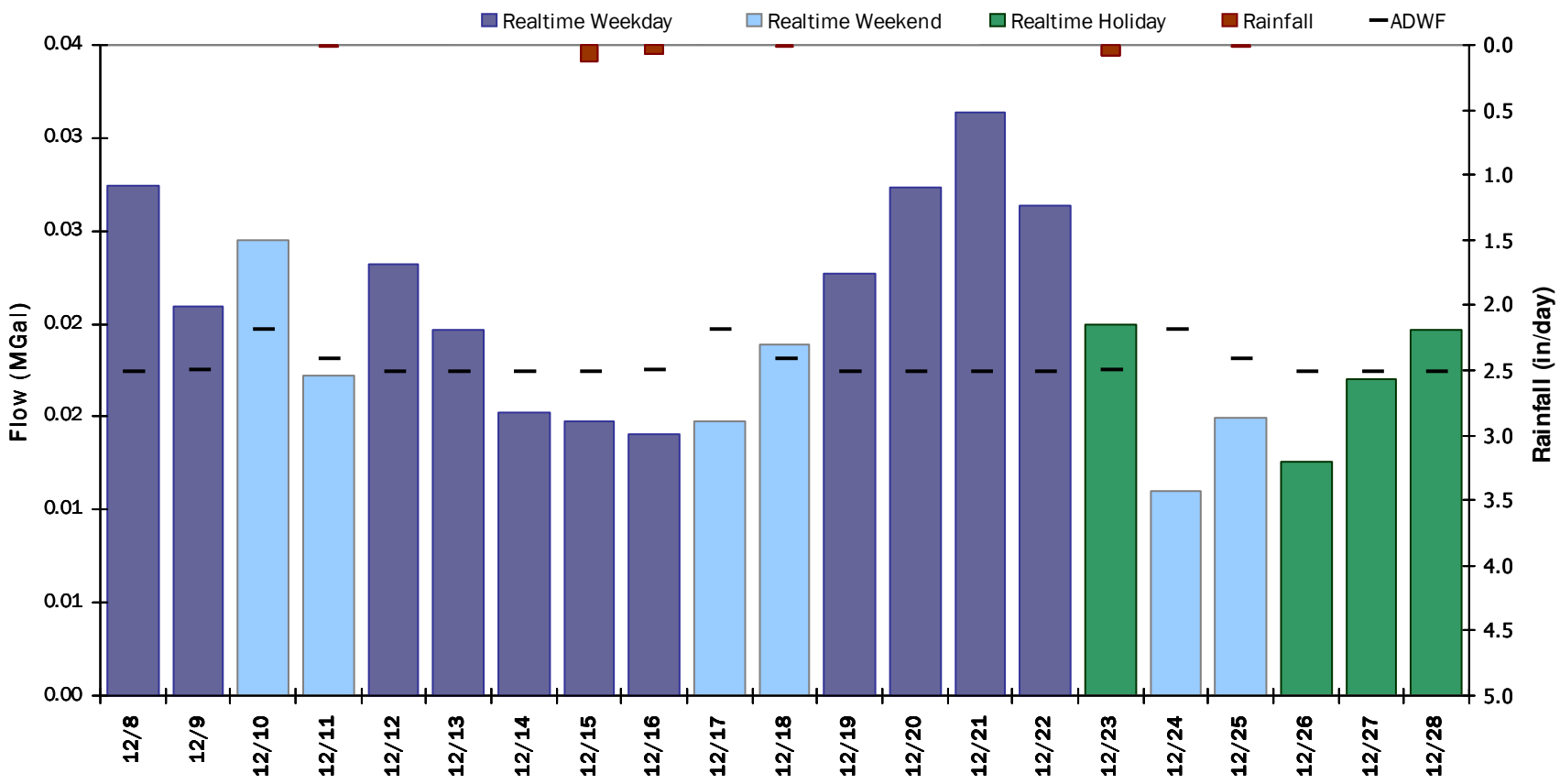
Influent Pipe

SITE 1B

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.020 MGal **Peak Daily Flow: 0.031 MGal** **Min Daily Flow: 0.011 MGal**

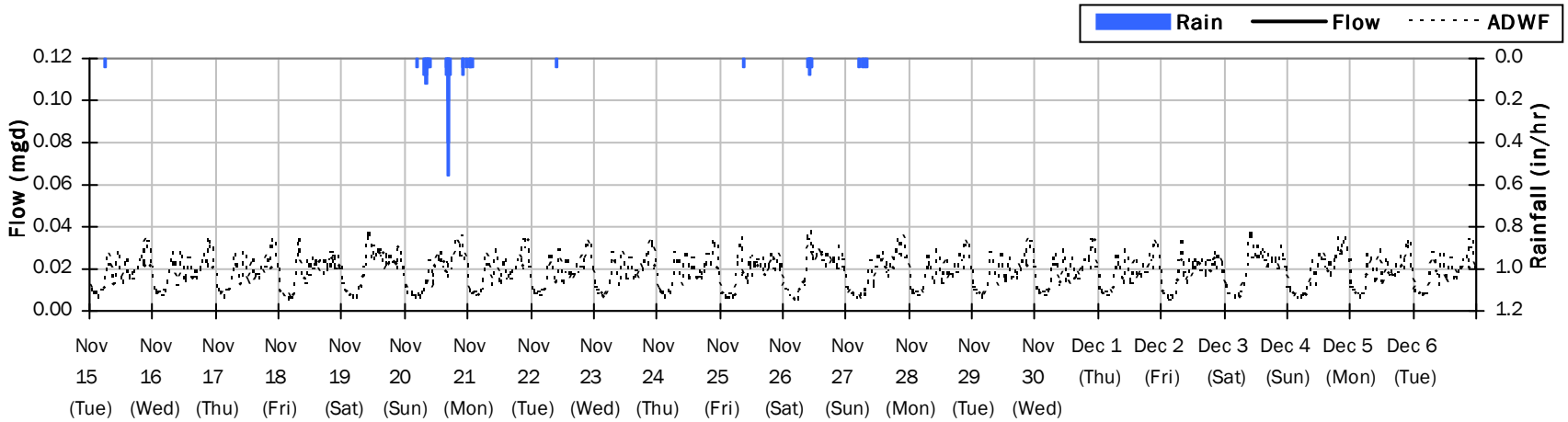
Total Period Rainfall: 0.33 inches

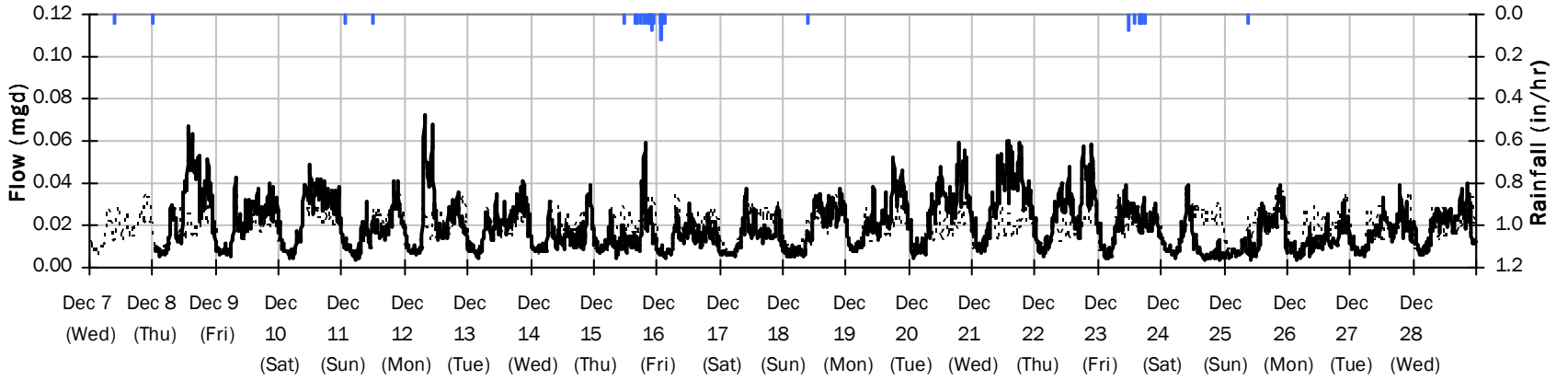


SITE 1B

Flow Summary: 11/15/2016 to 12/28/2016

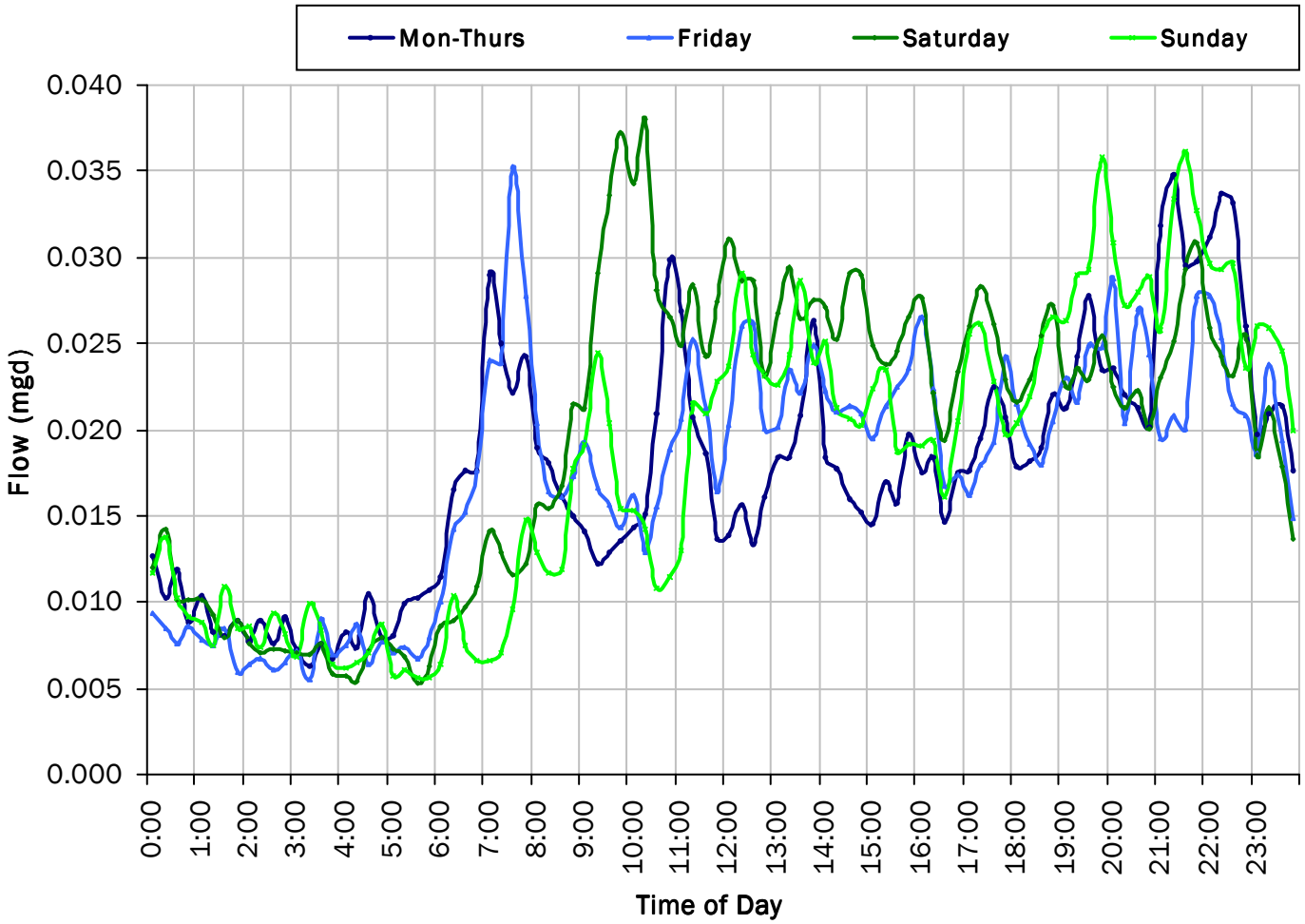
Total Period Rainfall: 1.01 inches **Avg Flow: 0.020 mgd** **Peak Flow: 0.071 mgd** **Min Flow: 0.003 mgd**





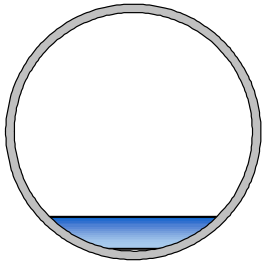
SITE 1B

Average Dry Weather Flow Hydrographs



ADWF:

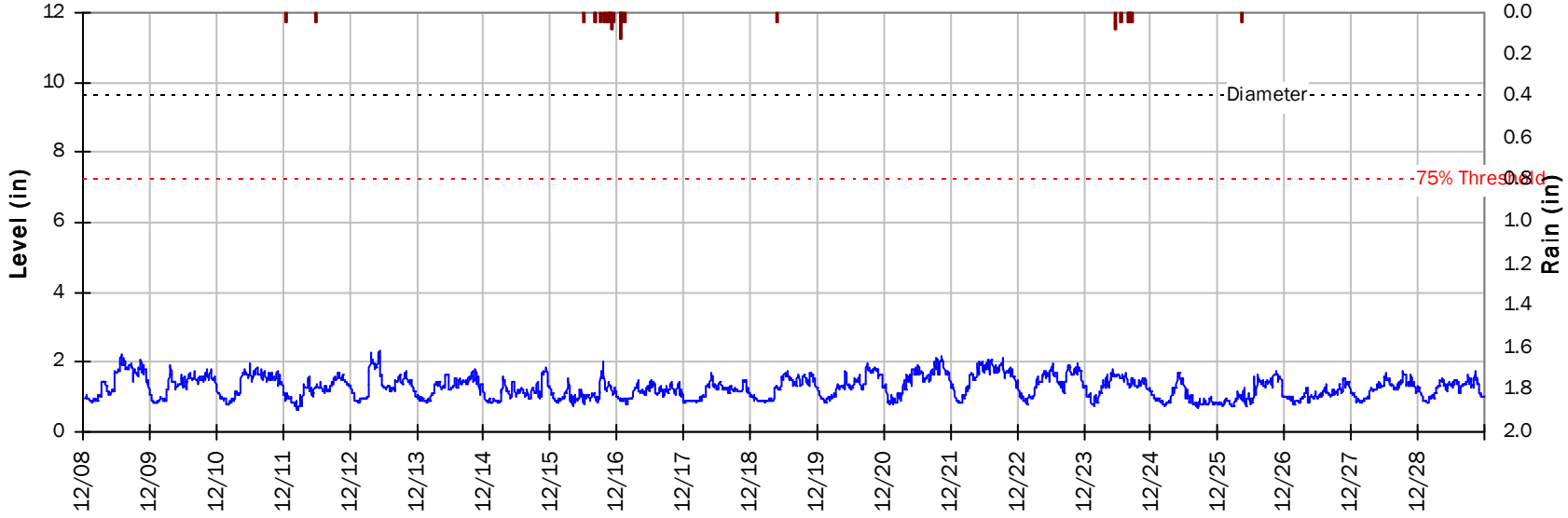
0.018 *mgd*

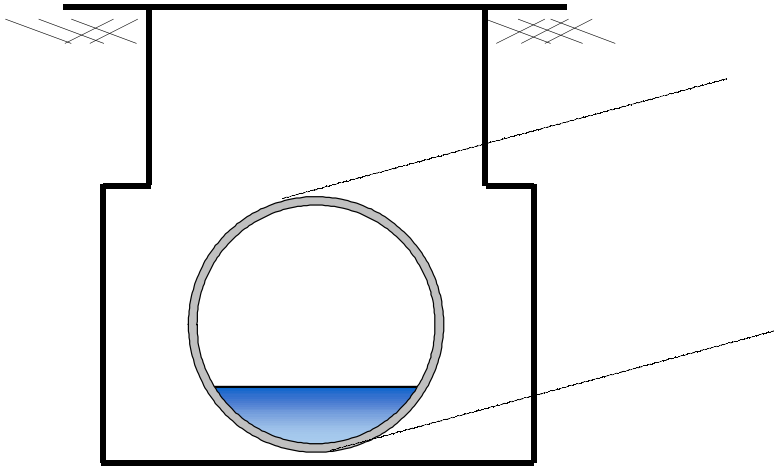


SITE 1B

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



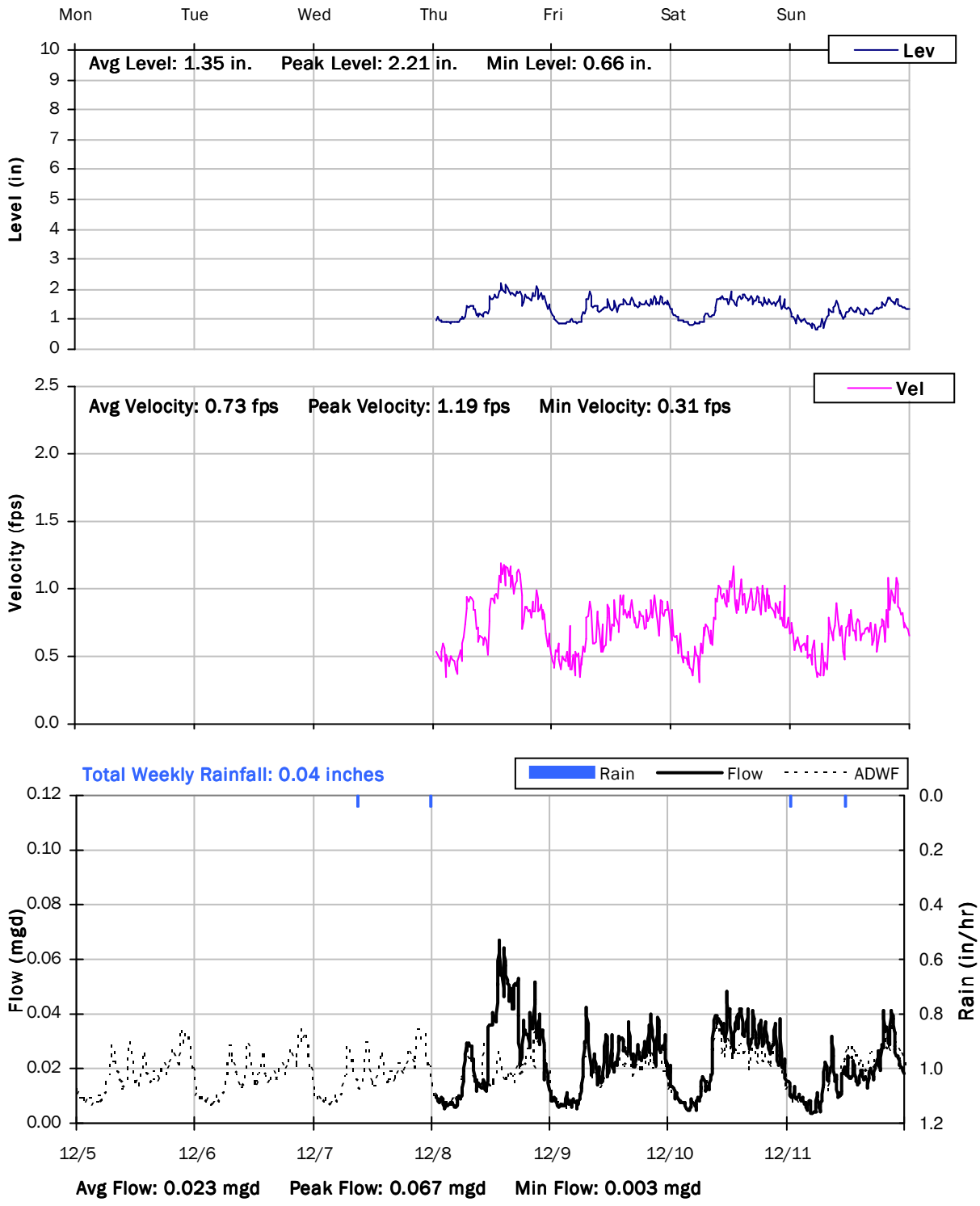


Pipe Diameter:	9.63	*inches*
Peak Measured Level:	2.32	*inches*
Peak d/D Ratio:	0.24	
Dry Weather Design Threshold Level:	4.81	*inches*

SITE 1B

Weekly Level, Velocity and Flow Hydrographs

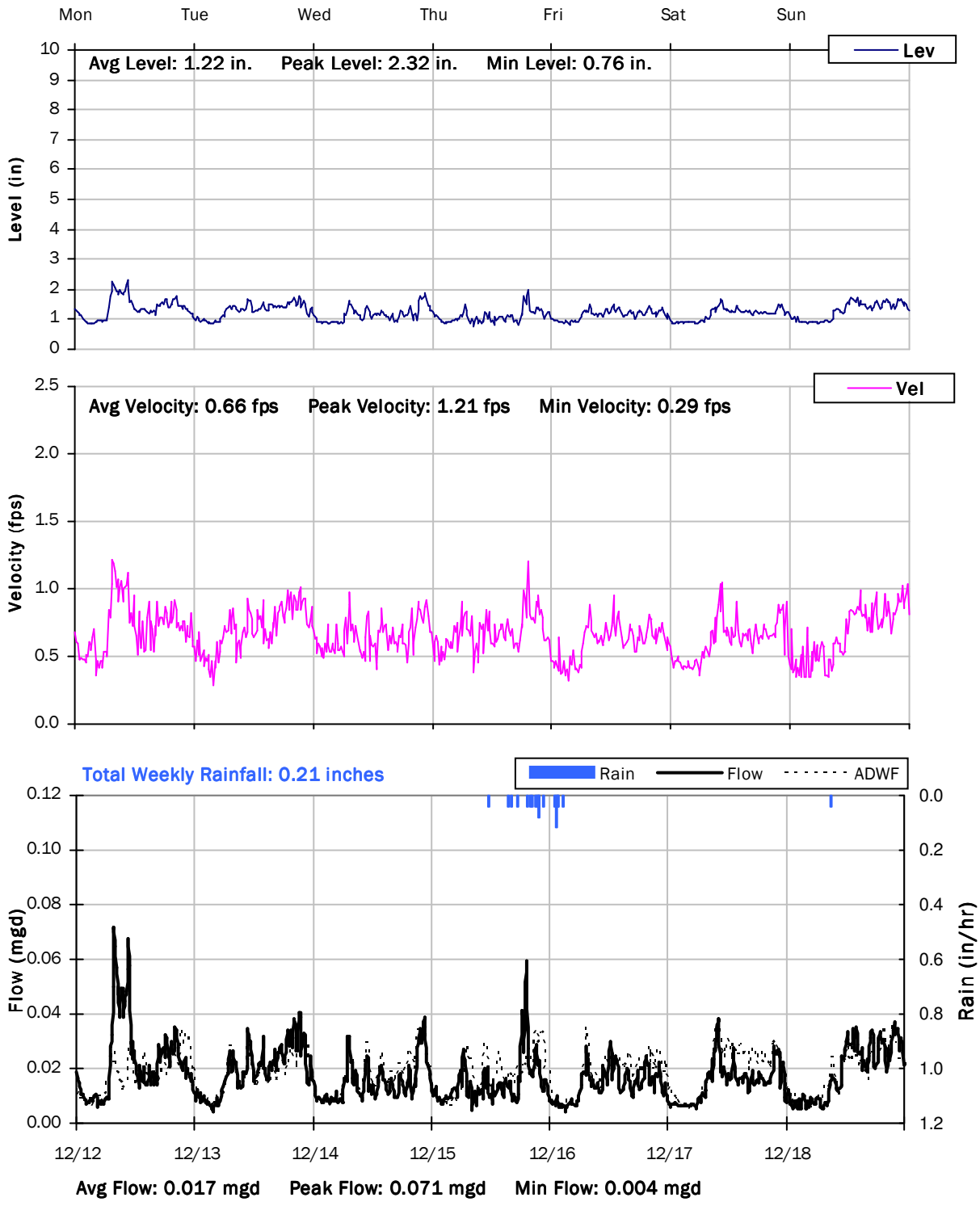
12/5/2016 to 12/12/2016



SITE 1B

Weekly Level, Velocity and Flow Hydrographs

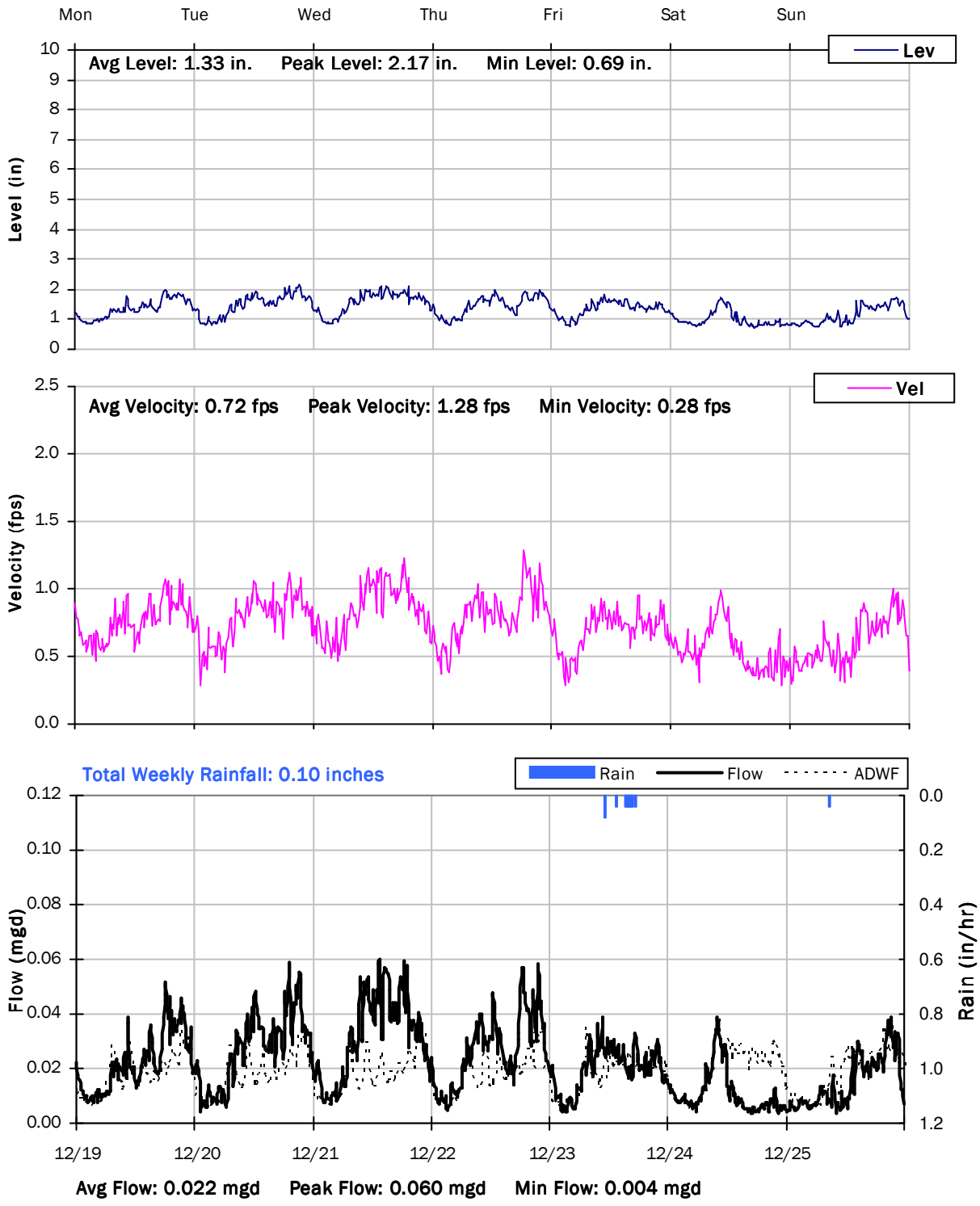
12/12/2016 to 12/19/2016



SITE 1B

Weekly Level, Velocity and Flow Hydrographs

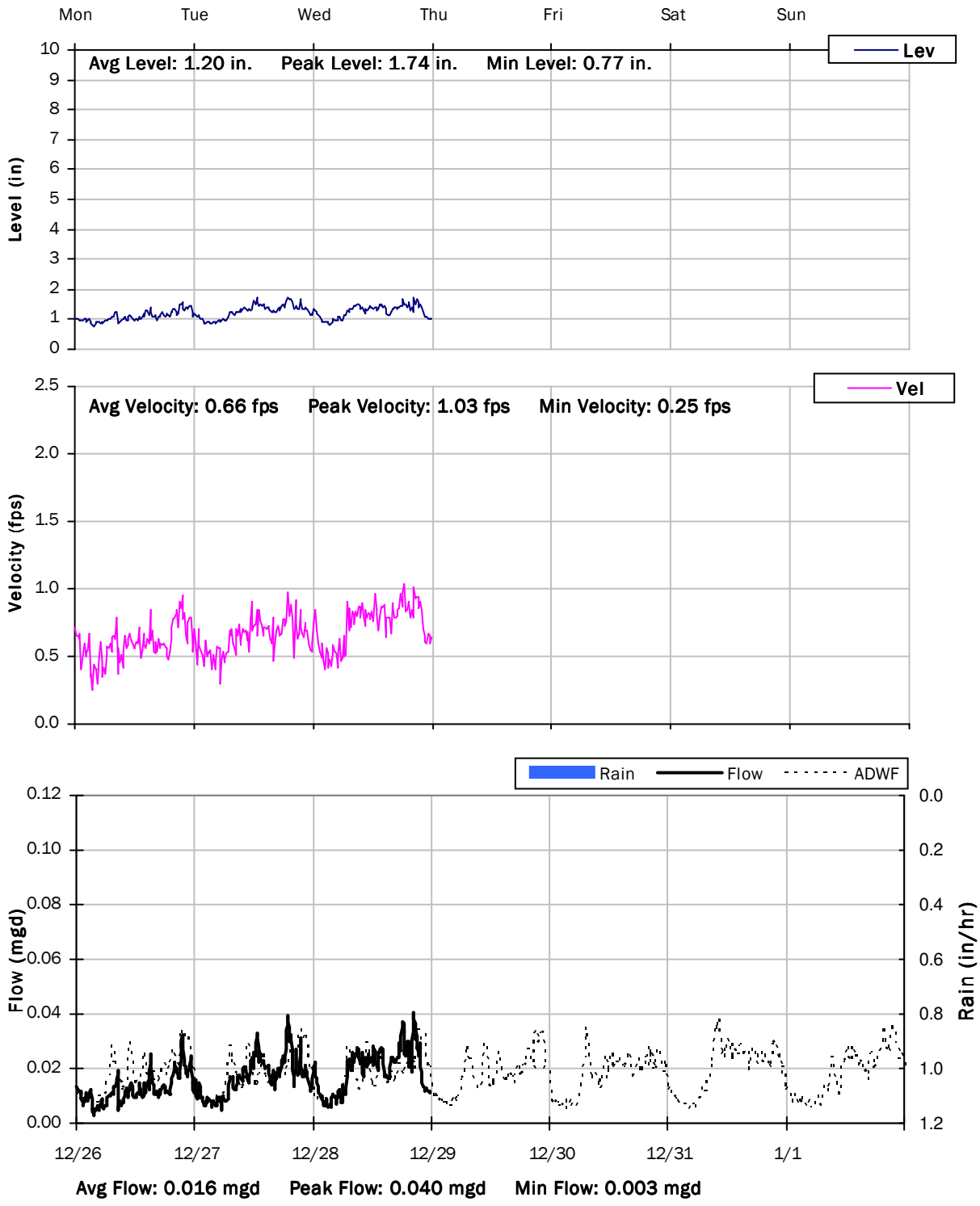
12/19/2016 to 12/26/2016



SITE 1B

Weekly Level, Velocity and Flow Hydrographs

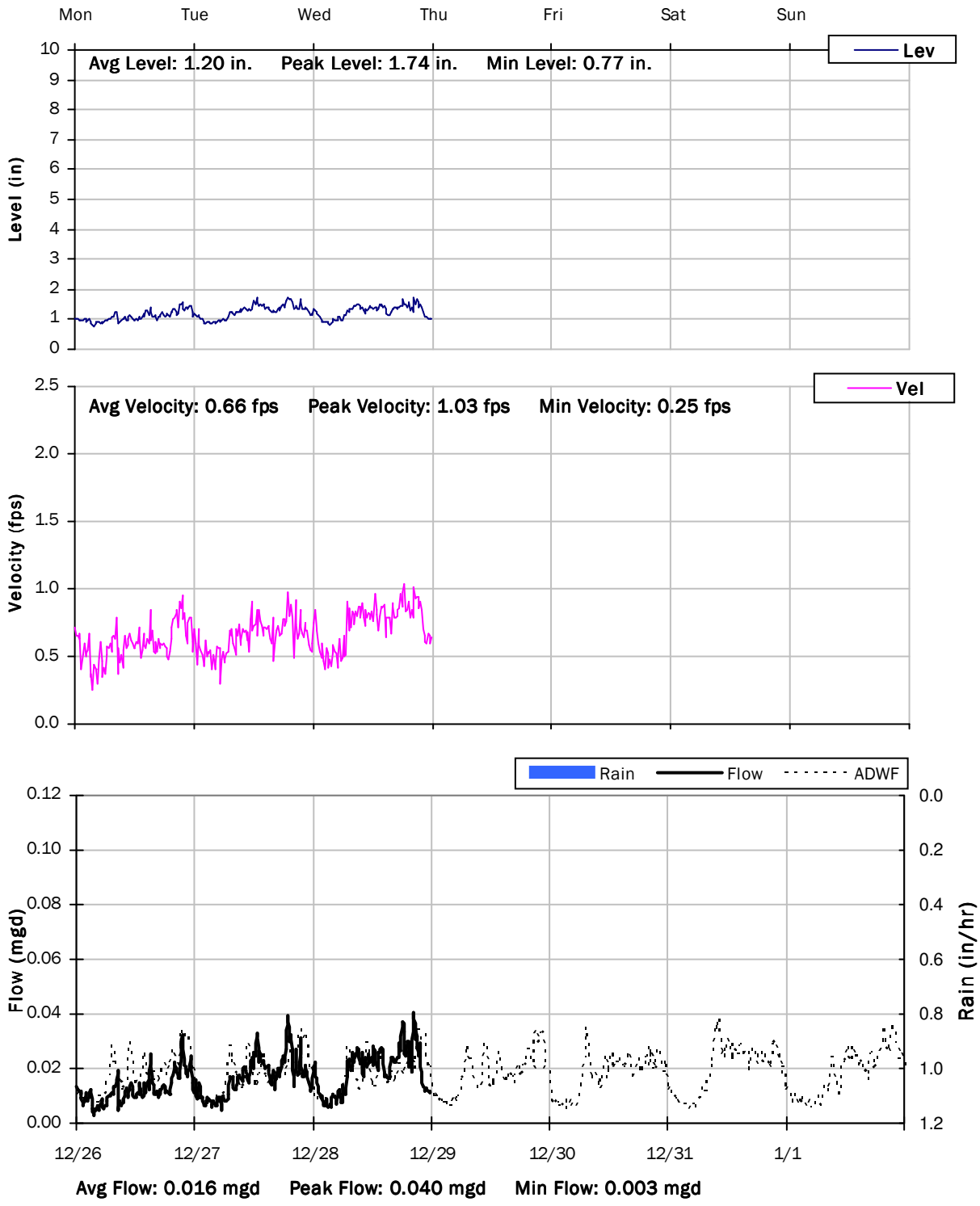
12/26/2016 to 1/2/2017



SITE 1B

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 2

Location: King City Sewage Treatment Plant

Data Summary Report

Vicinity Map: Site 2

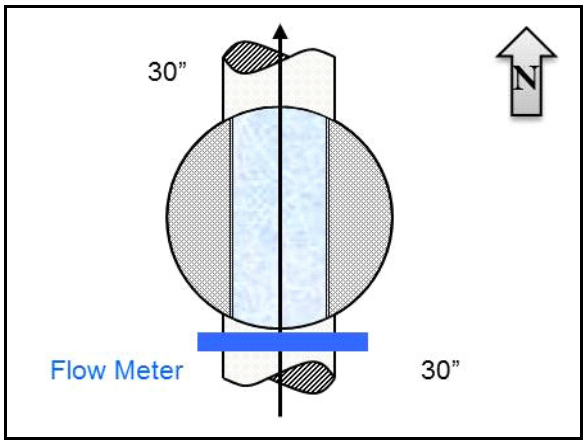
SITE 2

Site Information

Location:	King City Sewage Treatment Plant
Coordinates:	121.1517° W, 36.2165° N
Rim Elevation (Earth):	288 feet
Pipe Diameter:	30 inches
ADWF:	0.571 mgd
Peak Measured Flow:	1.251 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

Plan View

SITE 2

Additional Site Photos

Effluent Pipe

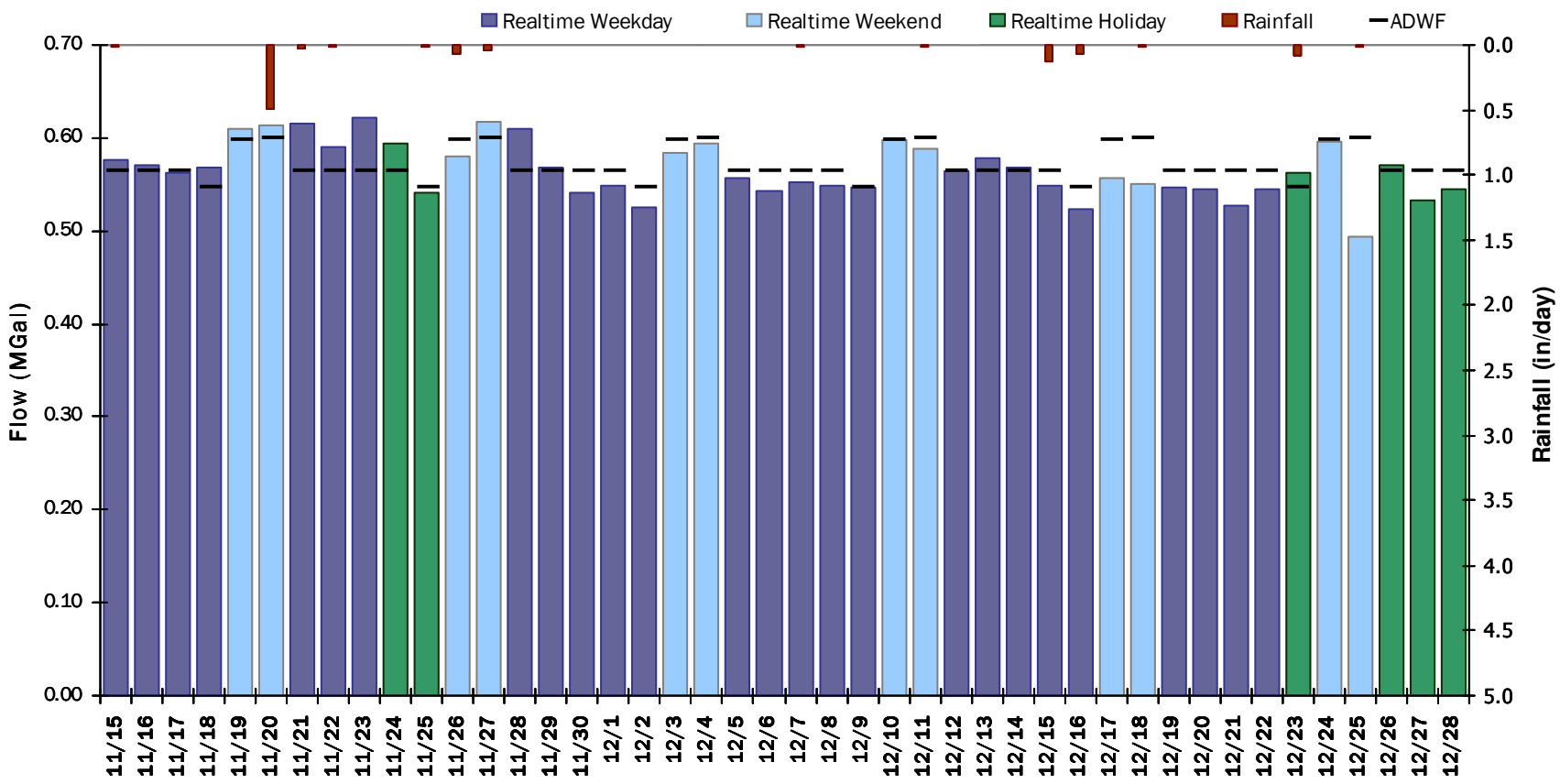
Influent Pipe

SITE 2

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.566 MGal **Peak Daily Flow: 0.621 MGal** **Min Daily Flow: 0.494 MGal**

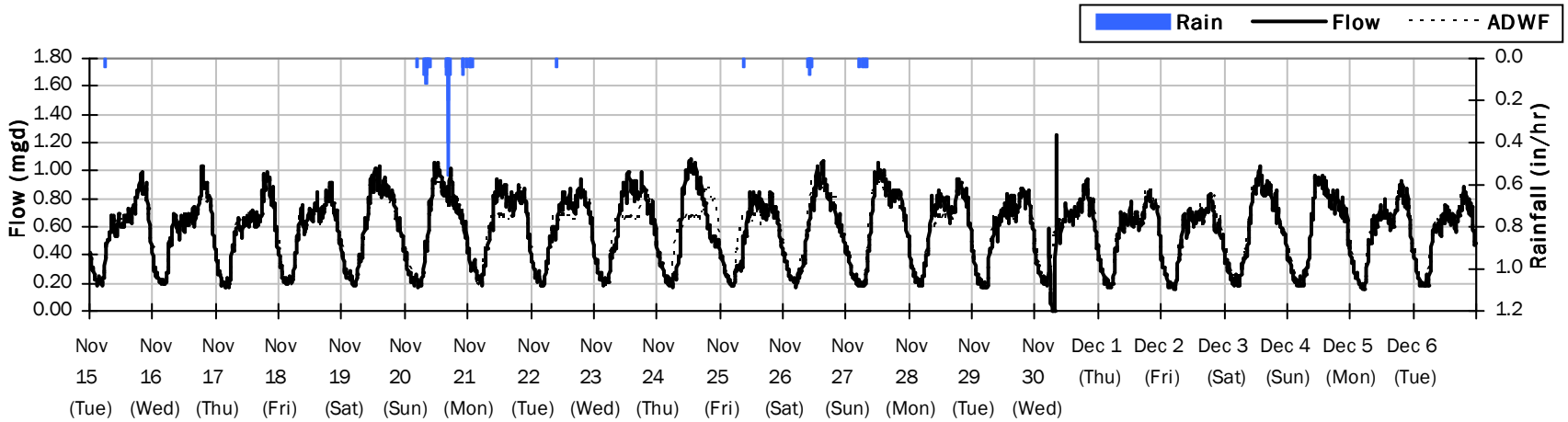
Total Period Rainfall: 1.01 inches

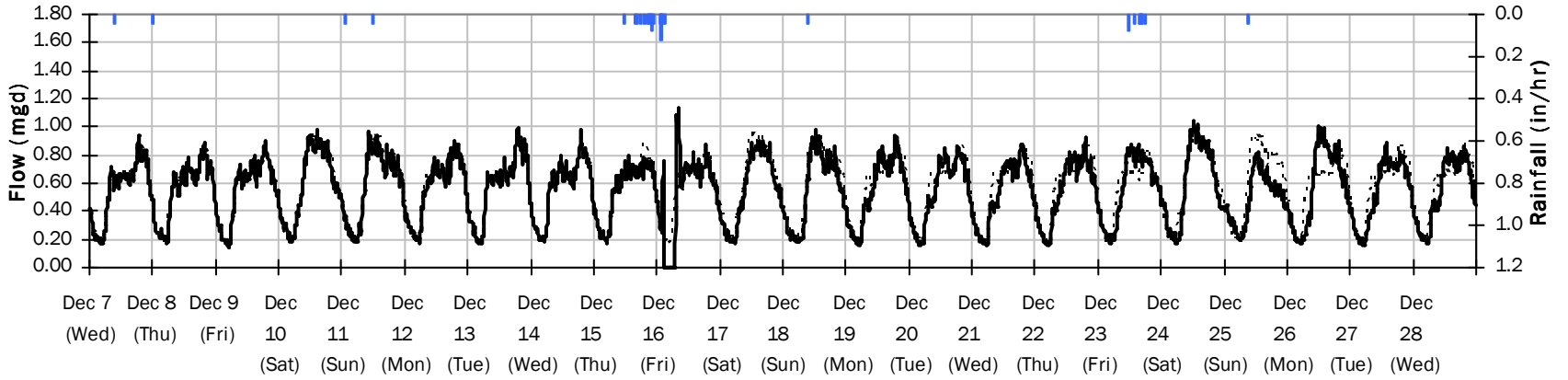


SITE 2

Flow Summary: 11/15/2016 to 12/28/2016

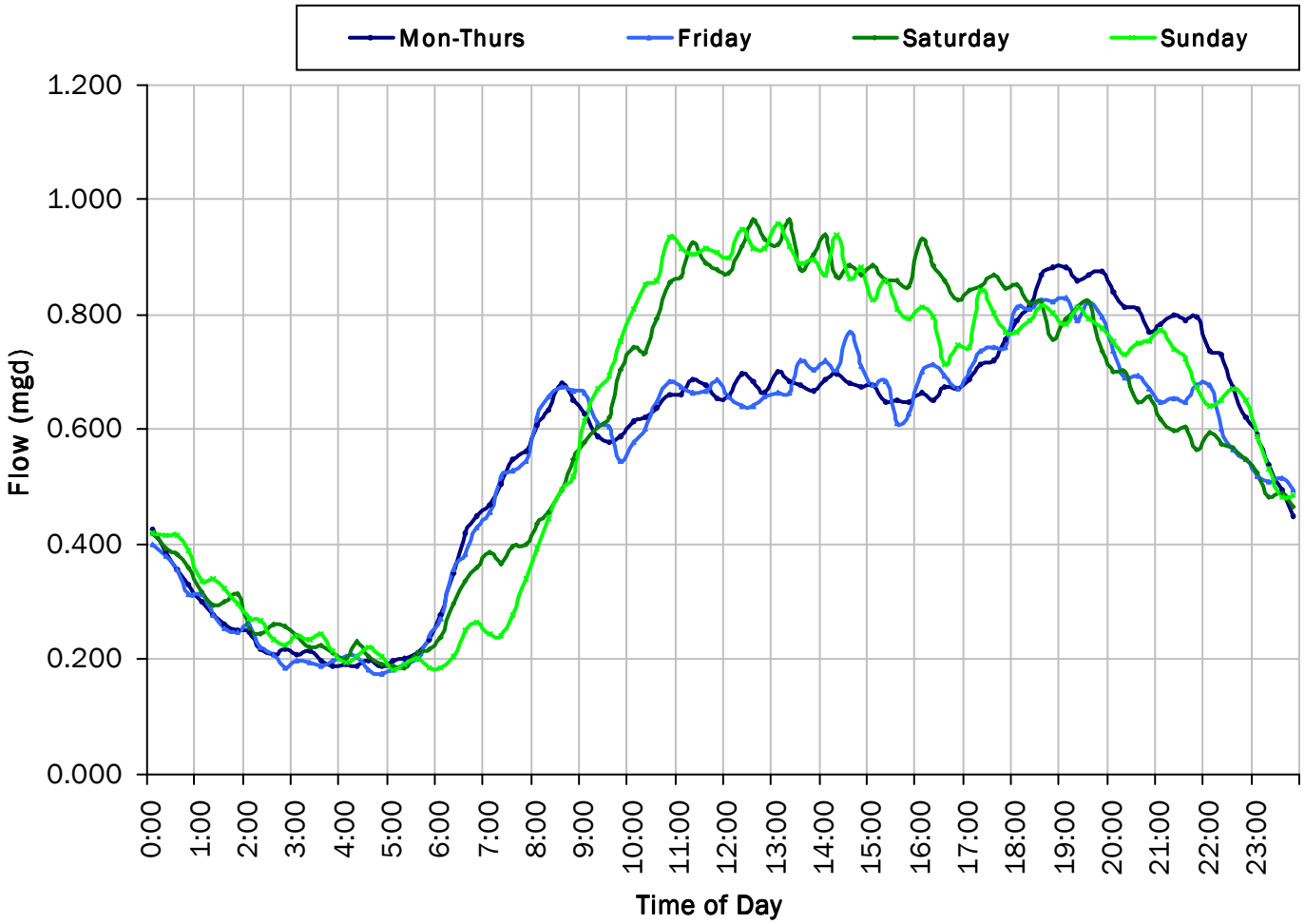
Total Period Rainfall: 1.01 inches **Avg Flow: 0.566 mgd** **Peak Flow: 1.251 mgd** **Min Flow: 0.000 mgd**





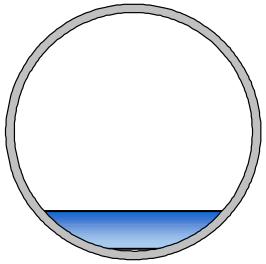
SITE 2

Average Dry Weather Flow Hydrographs



ADWF:

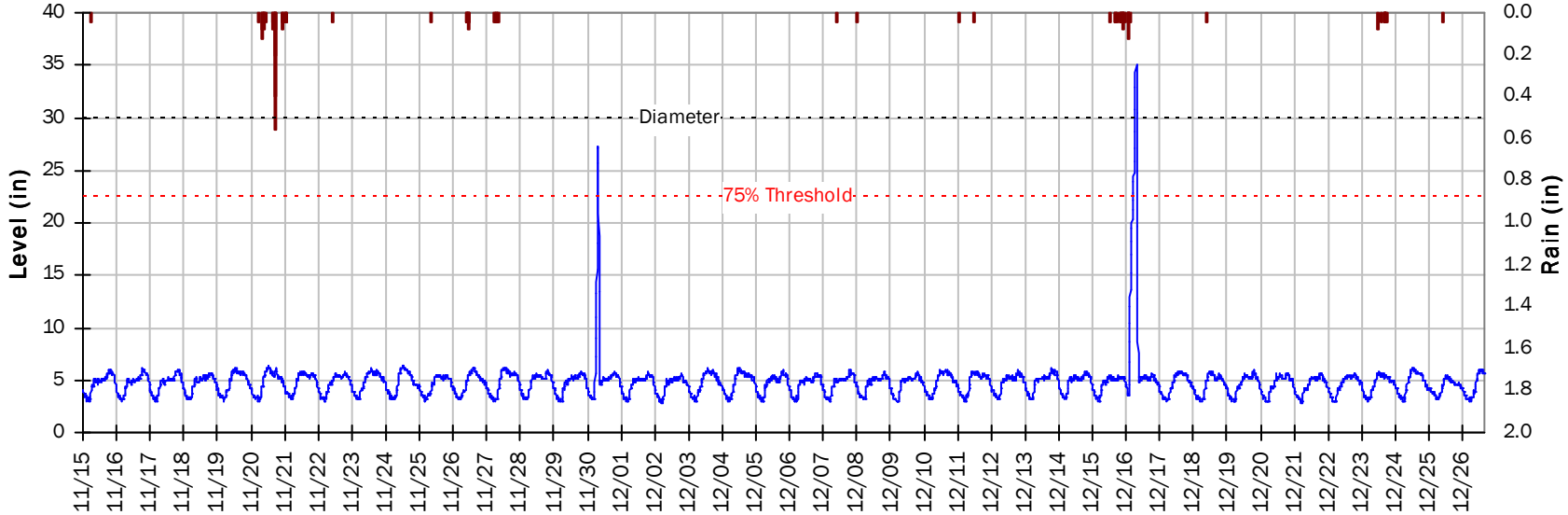
0.571 *mgd*

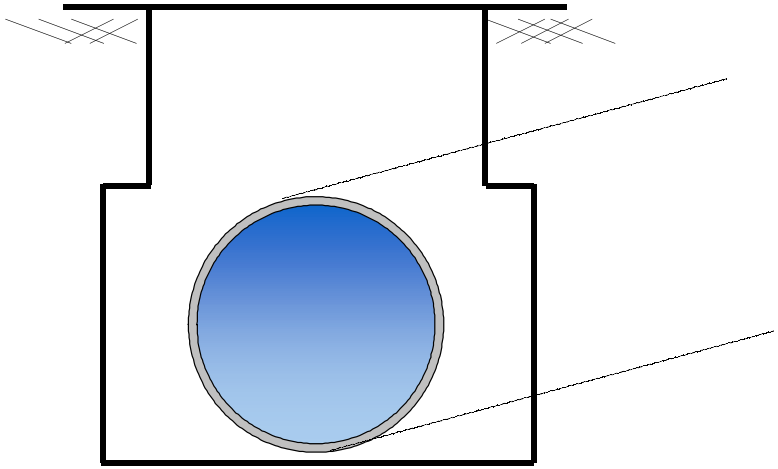


SITE 2

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event





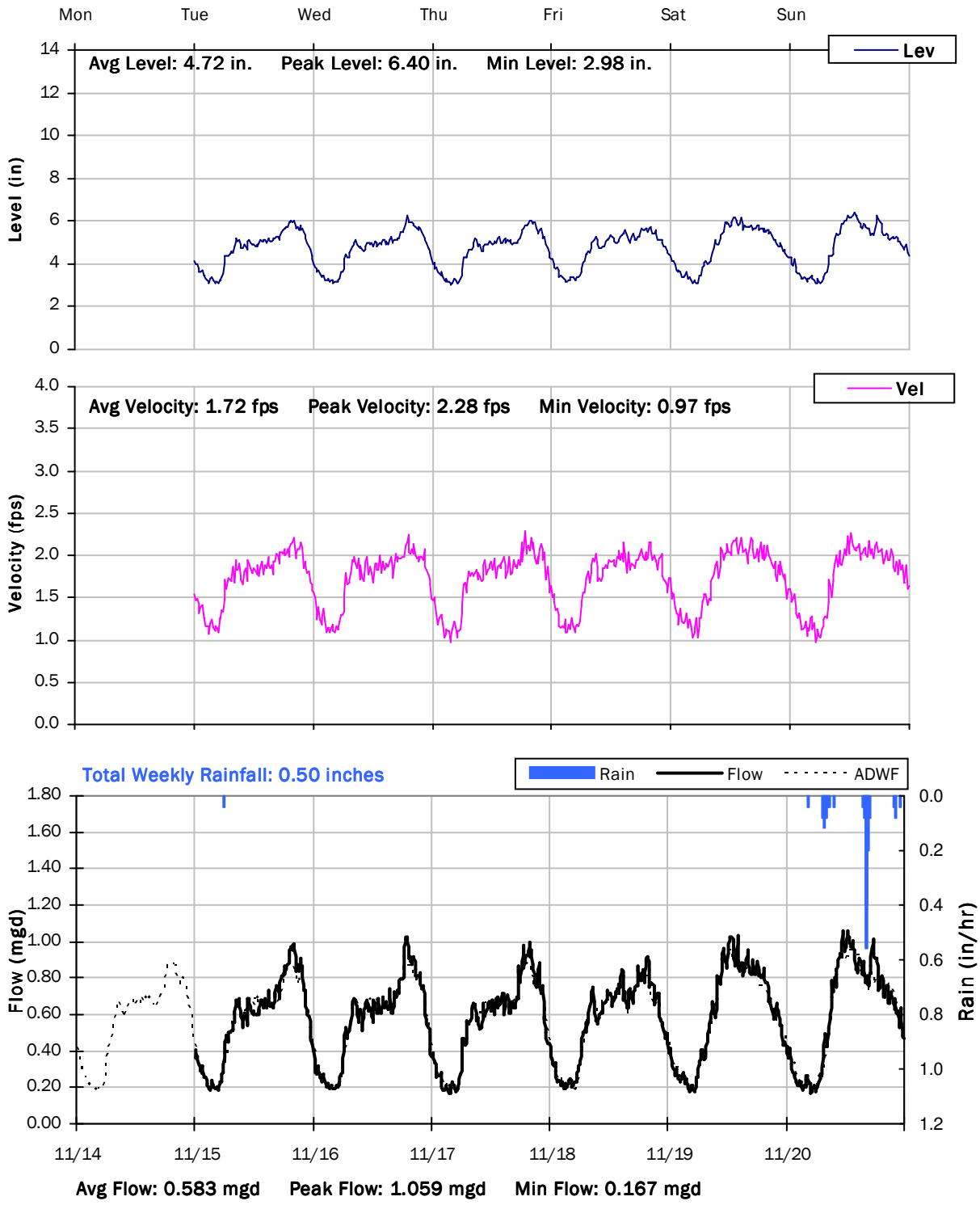
Pipe Diameter:	30	*inches*
Peak Measured Level:	35.1	*inches*
Peak d/D Ratio:	1.17	
Dry Weather Design Threshold Level:	22.5	*inches*

Surcharged 5.1 inches over crown

SITE 2

Weekly Level, Velocity and Flow Hydrographs

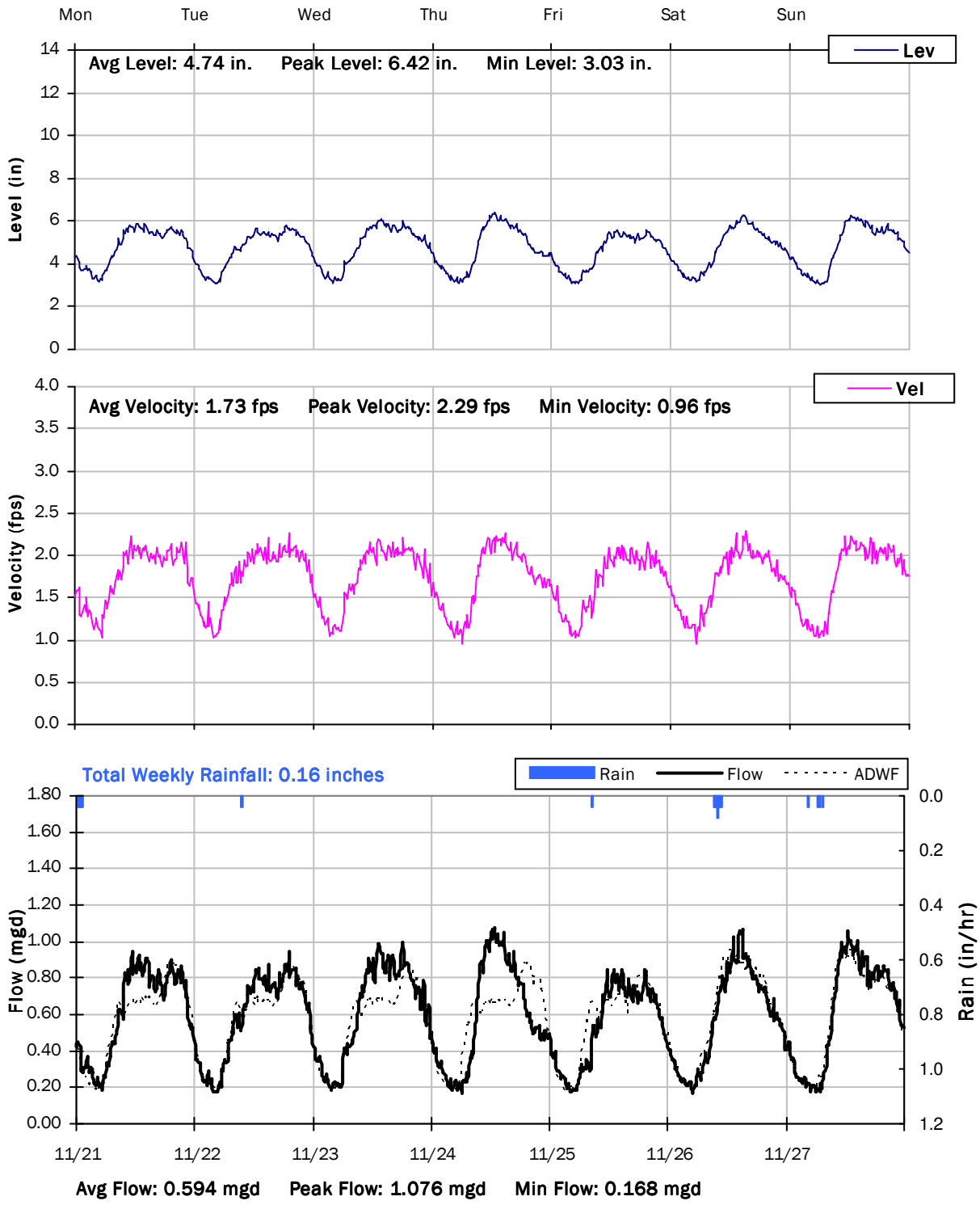
11/14/2016 to 11/21/2016



SITE 2

Weekly Level, Velocity and Flow Hydrographs

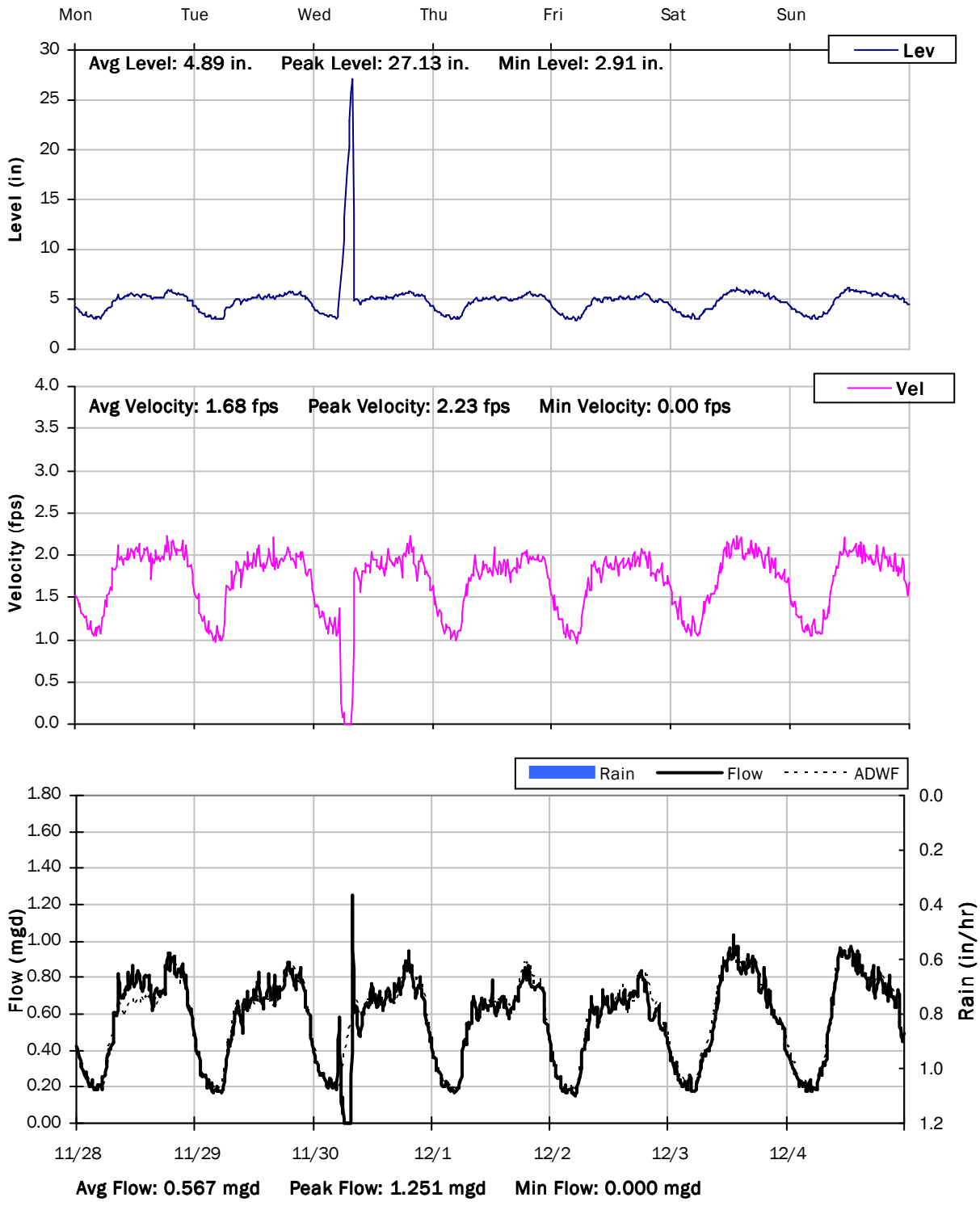
11/21/2016 to 11/28/2016



SITE 2

Weekly Level, Velocity and Flow Hydrographs

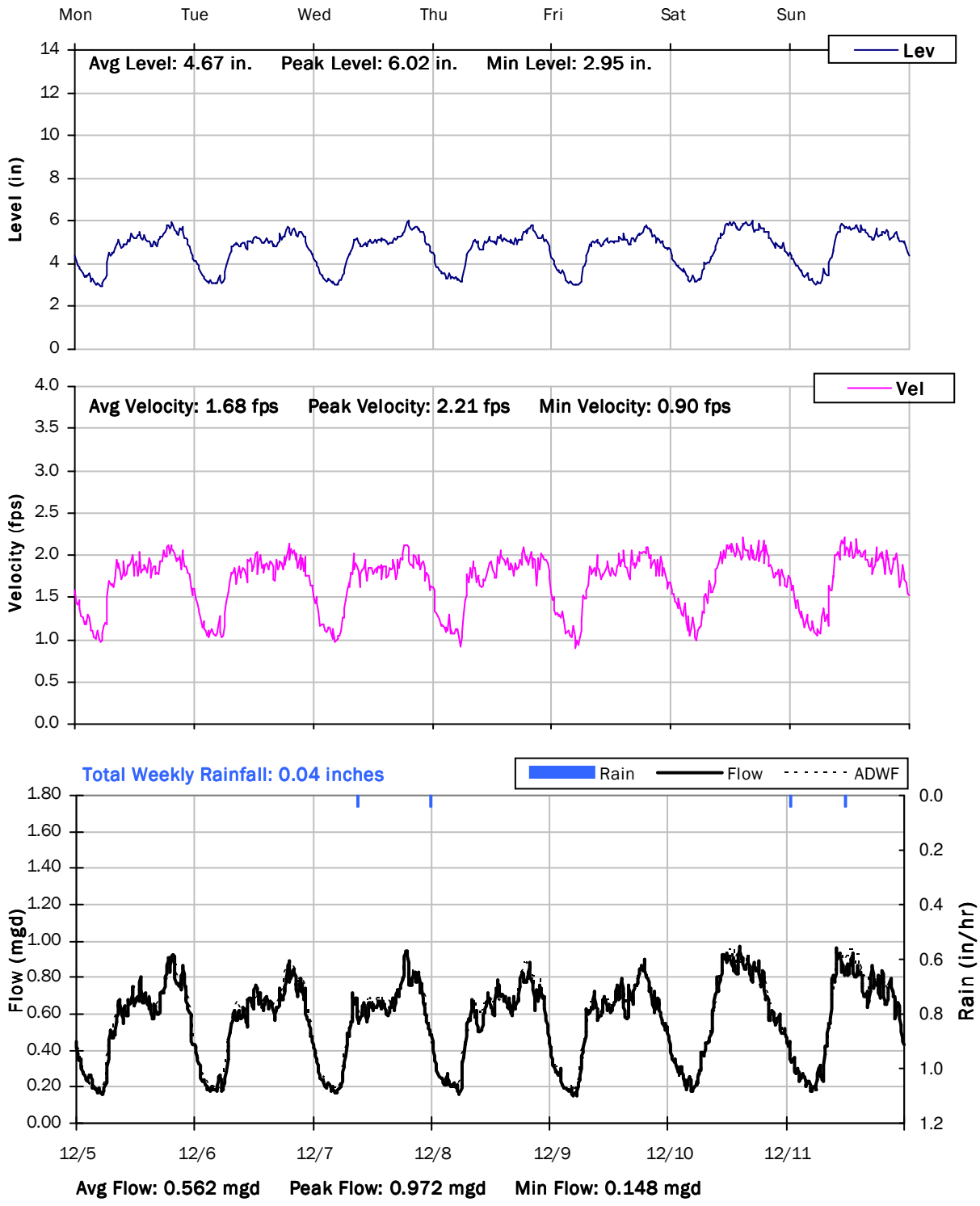
11/28/2016 to 12/5/2016



SITE 2

Weekly Level, Velocity and Flow Hydrographs

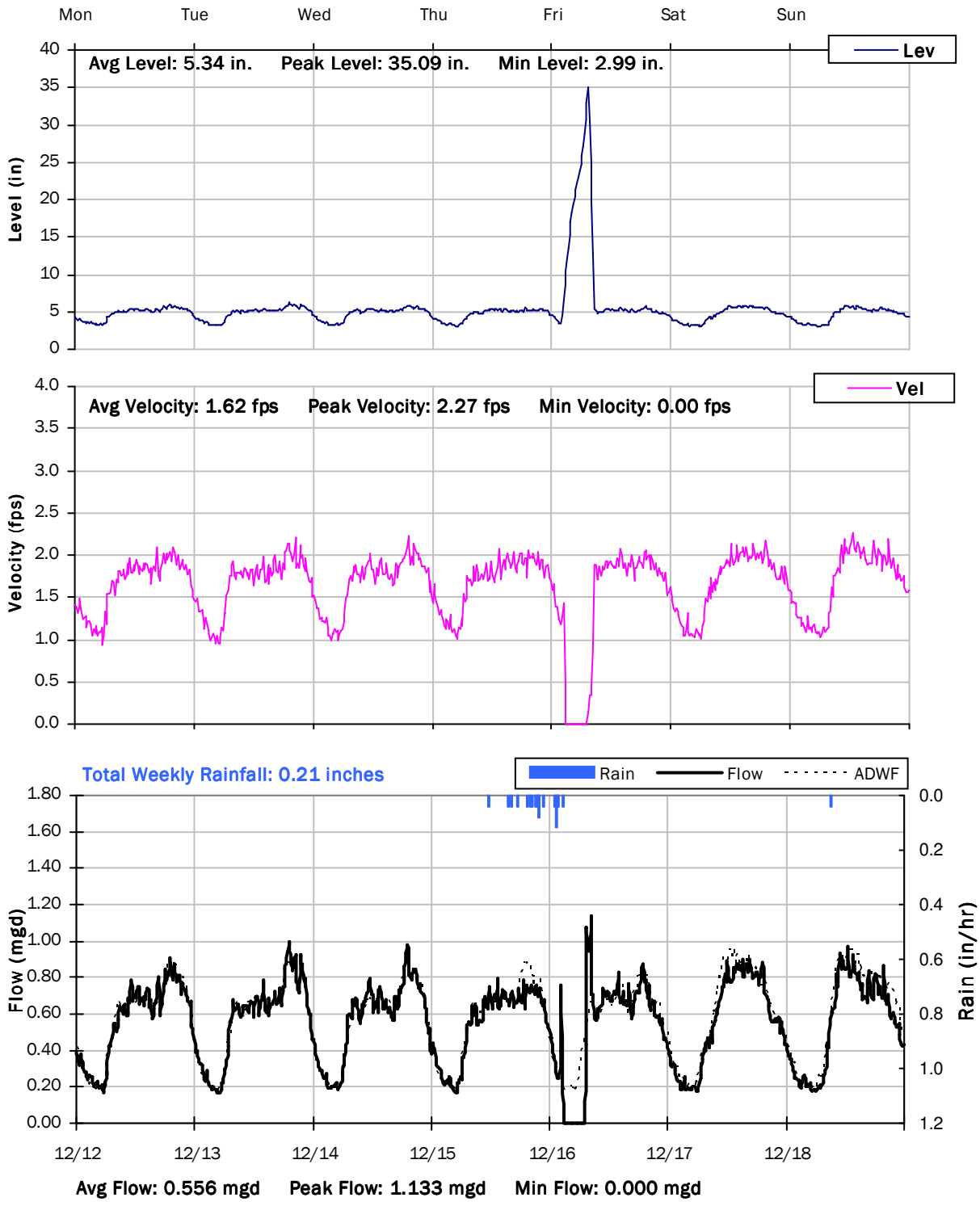
12/5/2016 to 12/12/2016



SITE 2

Weekly Level, Velocity and Flow Hydrographs

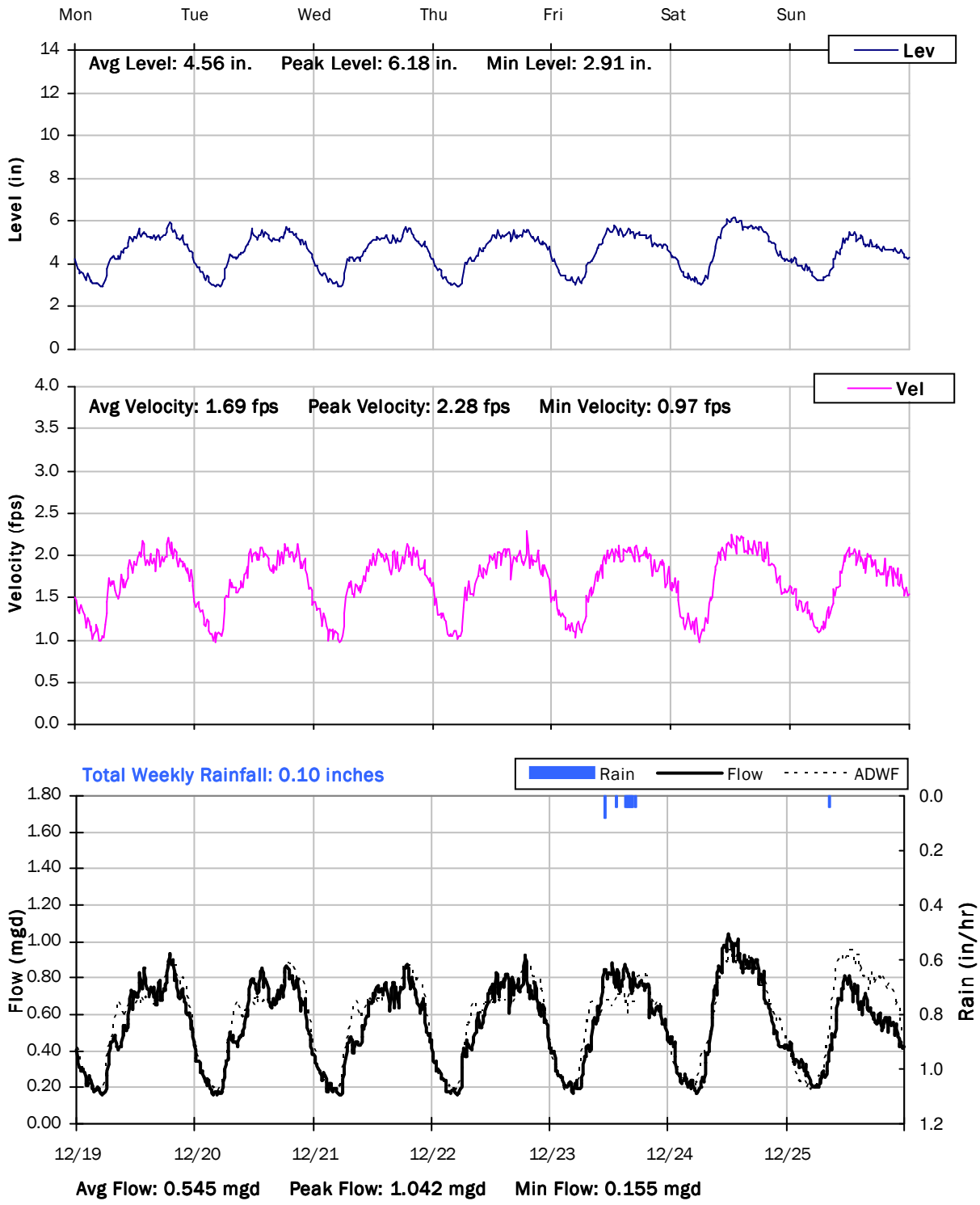
12/12/2016 to 12/19/2016



SITE 2

Weekly Level, Velocity and Flow Hydrographs

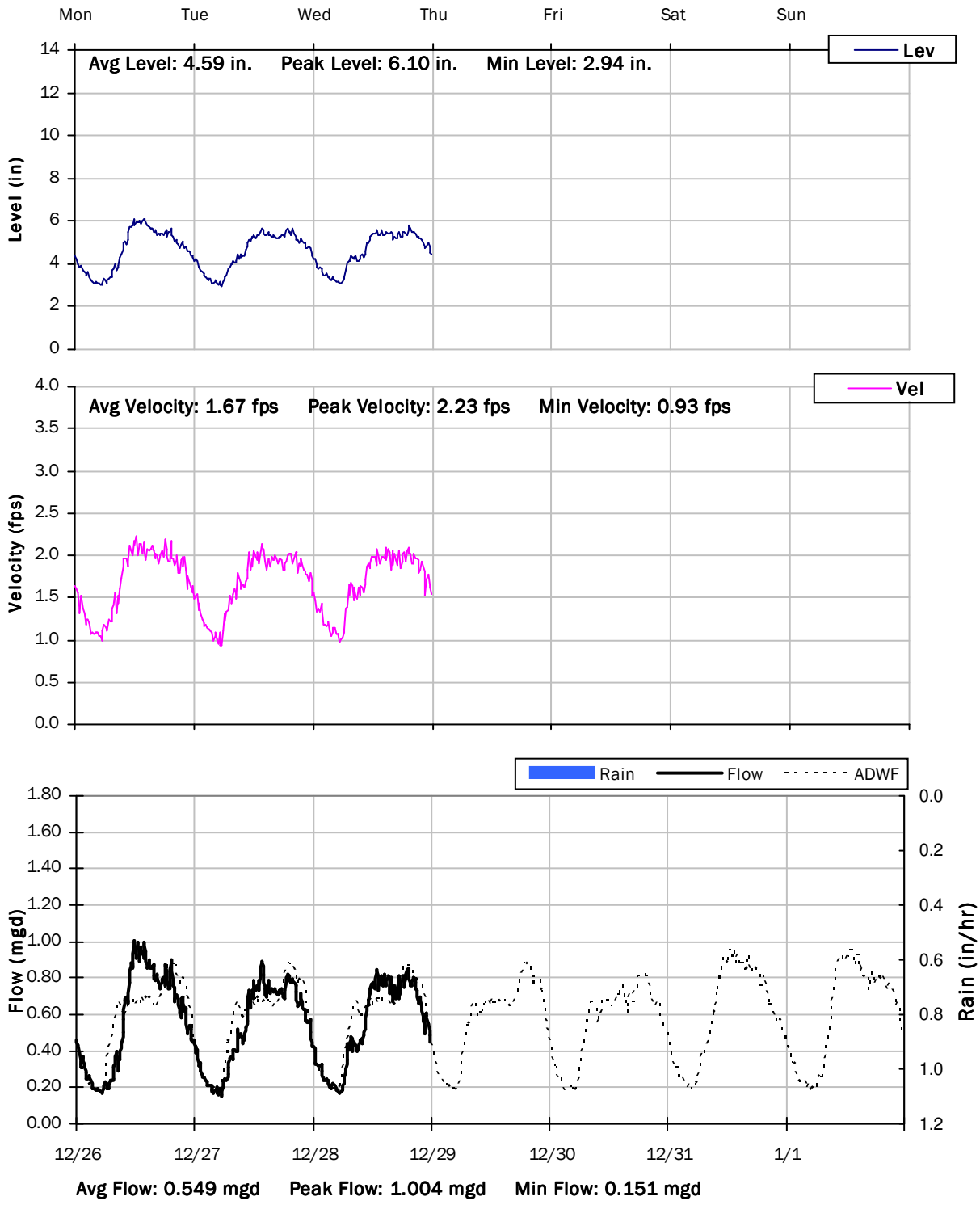
12/19/2016 to 12/26/2016



SITE 2

Weekly Level, Velocity and Flow Hydrographs

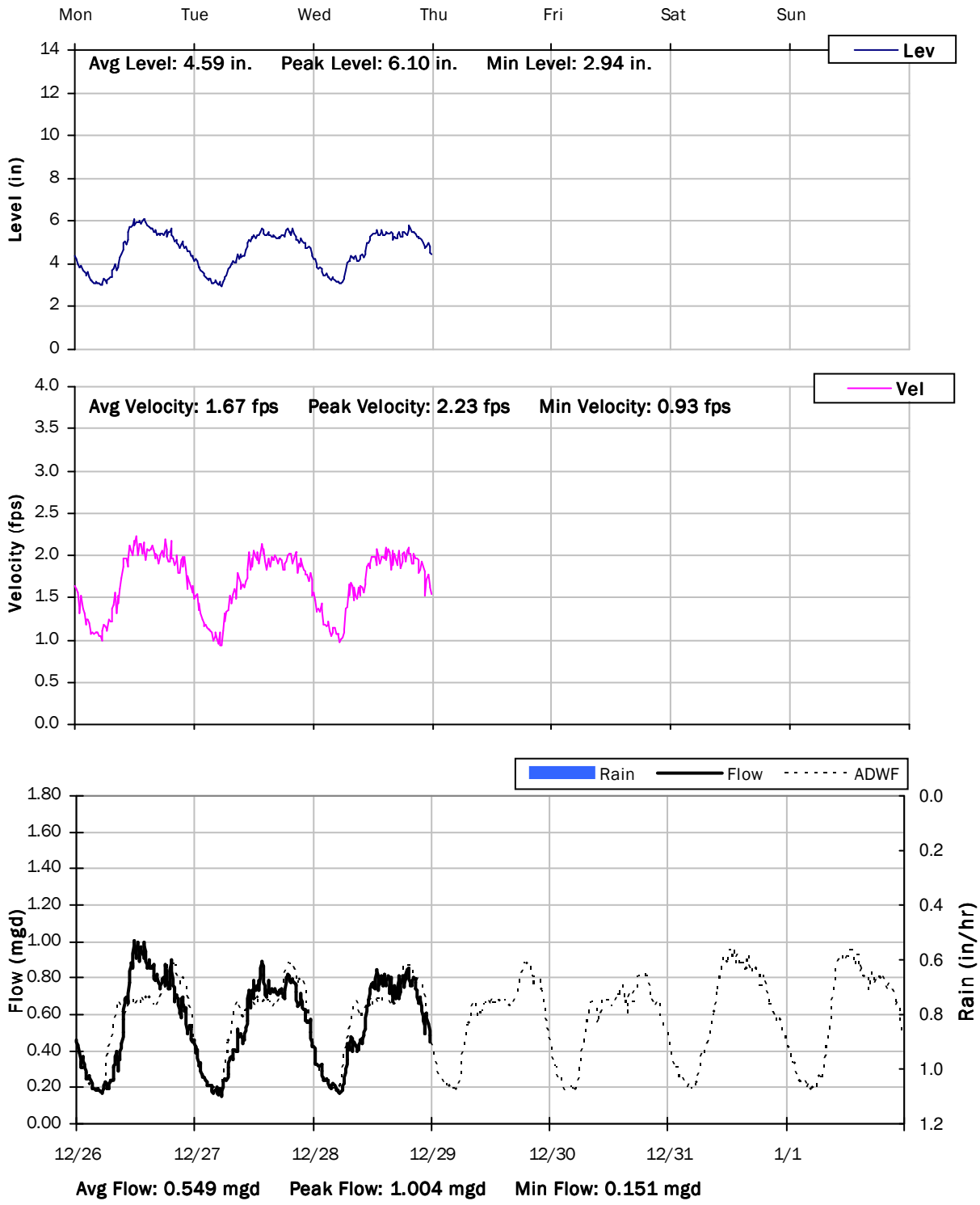
12/26/2016 to 1/2/2017



SITE 2

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 3a

Location: Willow Street at Sandringham Street

Data Summary Report

Vicinity Map: Site 3a

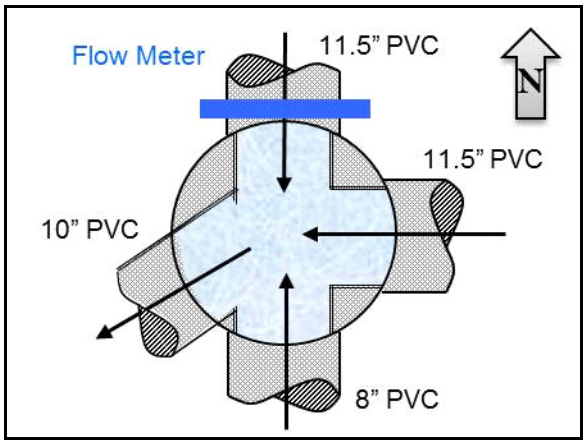
SITE 3A

Site Information

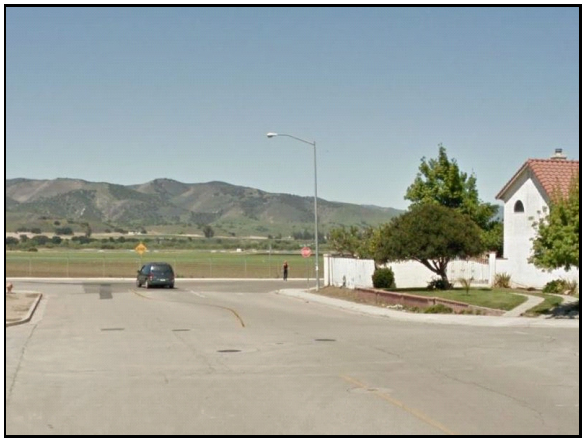
Location:	Willow Street at Sandringham Street
Coordinates:	121.1397° W, 36.2123° N
Rim Elevation (Earth):	316 feet
Pipe Diameter:	11.5 inches
ADWF:	0.062 mgd
Peak Measured Flow:	0.137 mgd

Satellite Map

Sewer Map



Flow Sketch



Street View

Plan View

SITE 3A

Additional Site Photos

Effluent Pipe

North Influent Pipe

SITE 3A

Additional Site Photos

East Influent Pipe

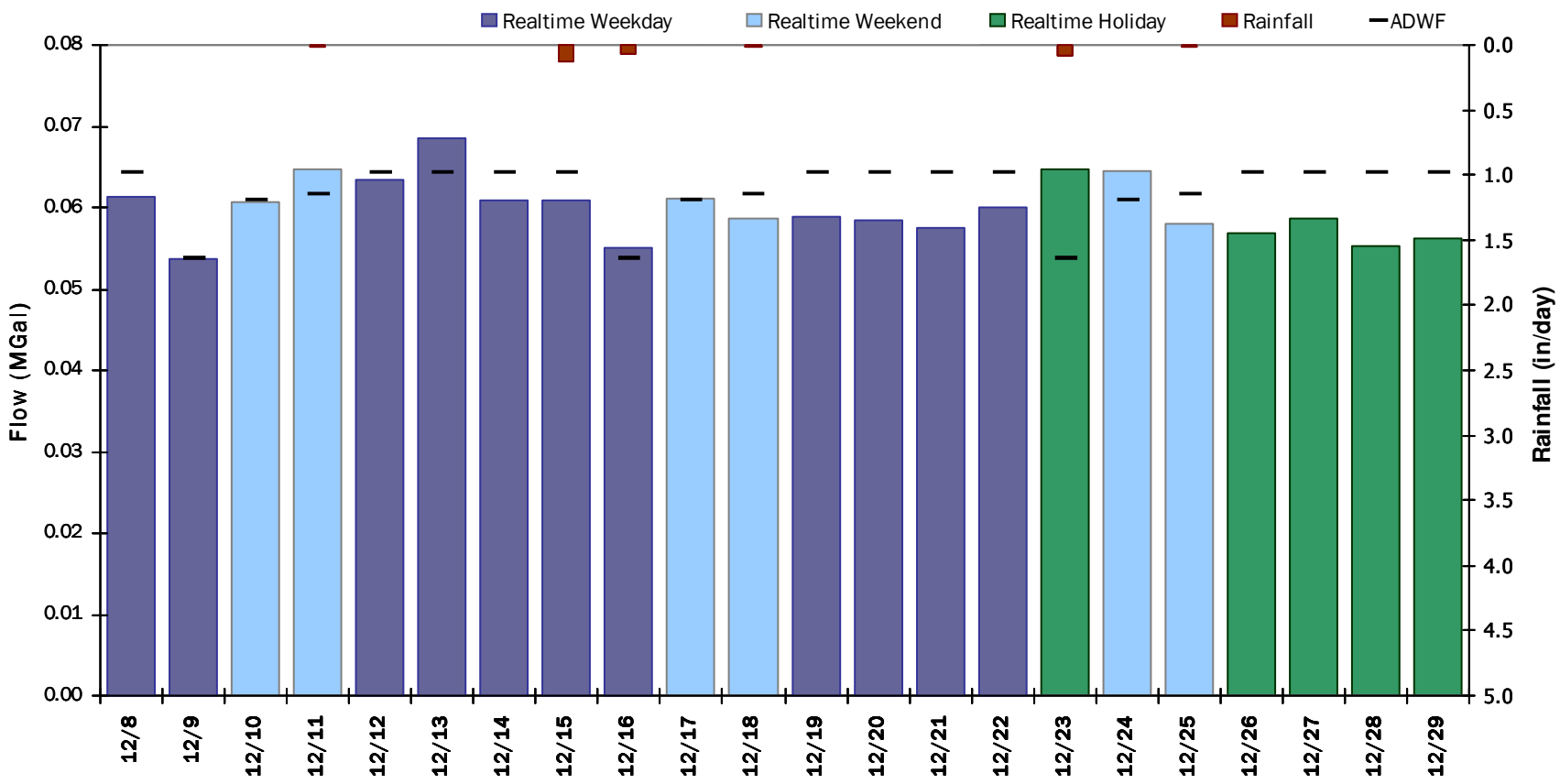
South Influent Pipe

SITE 3A

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.060 MGal **Peak Daily Flow: 0.069 MGal** **Min Daily Flow: 0.054 MGal**

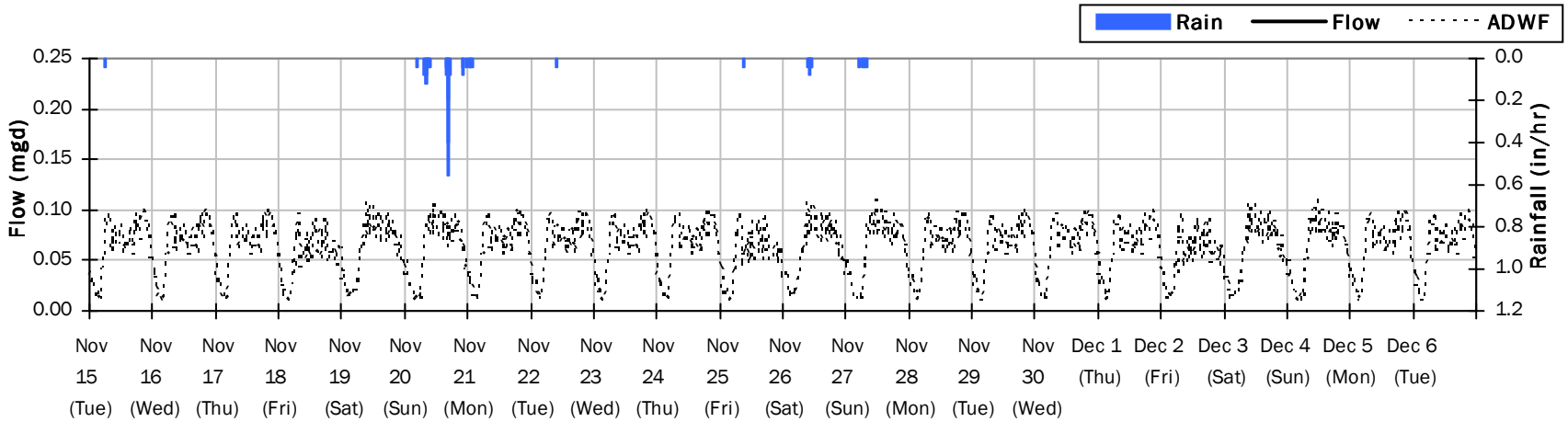
Total Period Rainfall: 0.33 inches

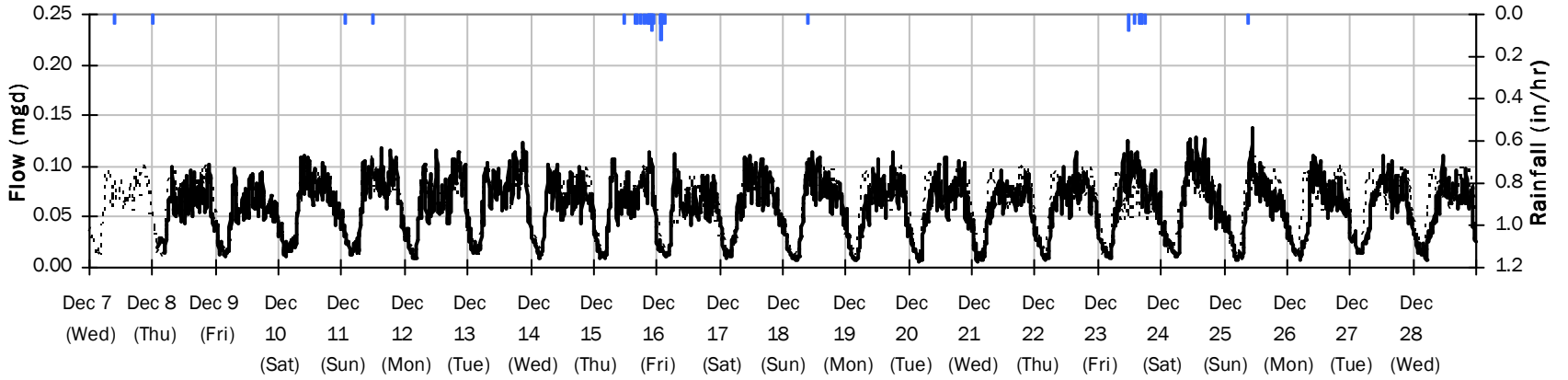


SITE 3A

Flow Summary: 11/15/2016 to 12/28/2016

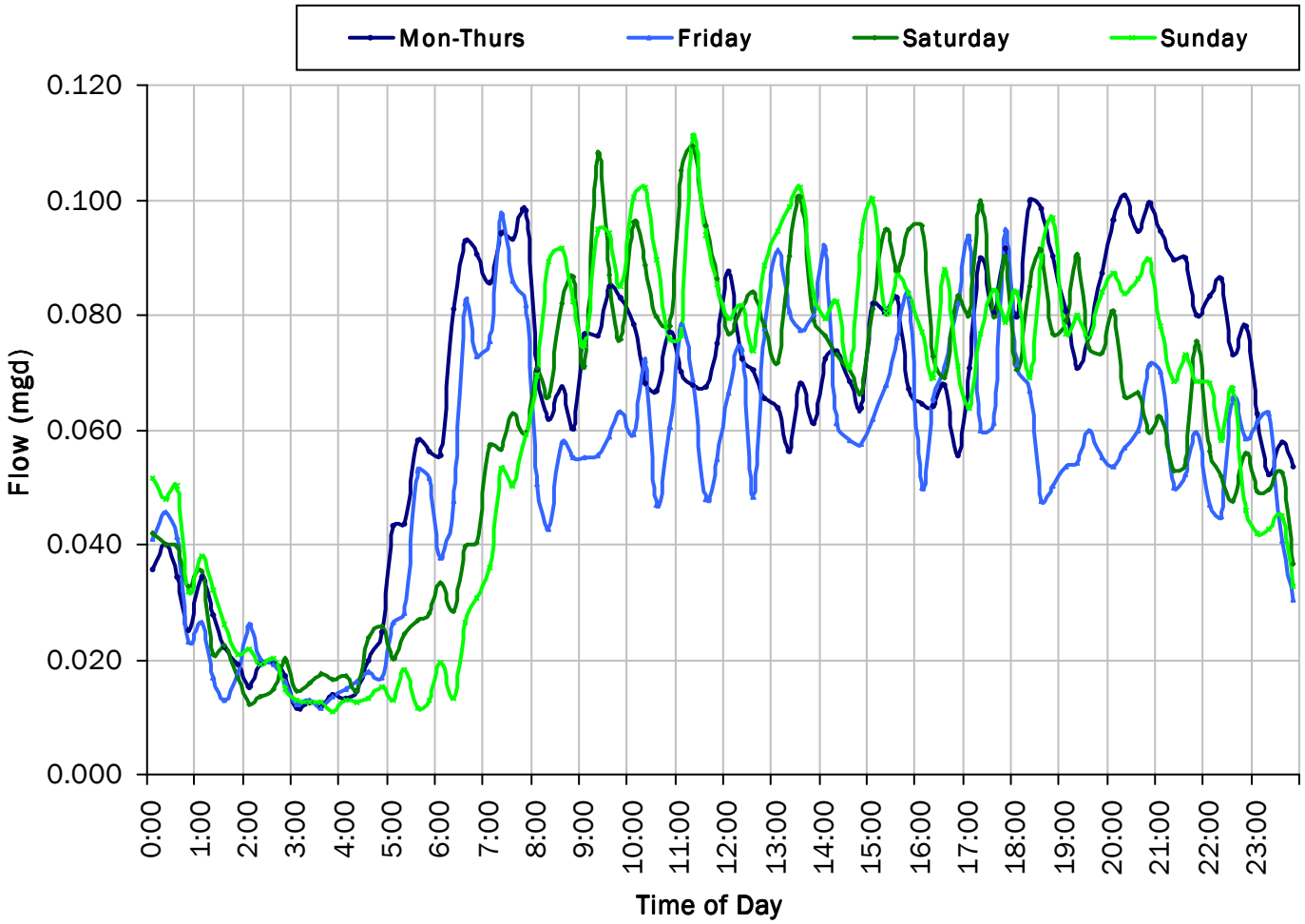
Total Period Rainfall: 1.01 inches **Avg Flow: 0.060 mgd** **Peak Flow: 0.137 mgd** **Min Flow: 0.005 mgd**





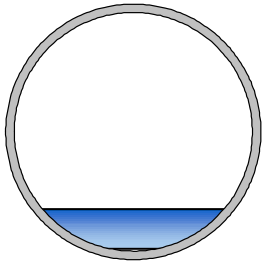
SITE 3A

Average Dry Weather Flow Hydrographs



ADWF:

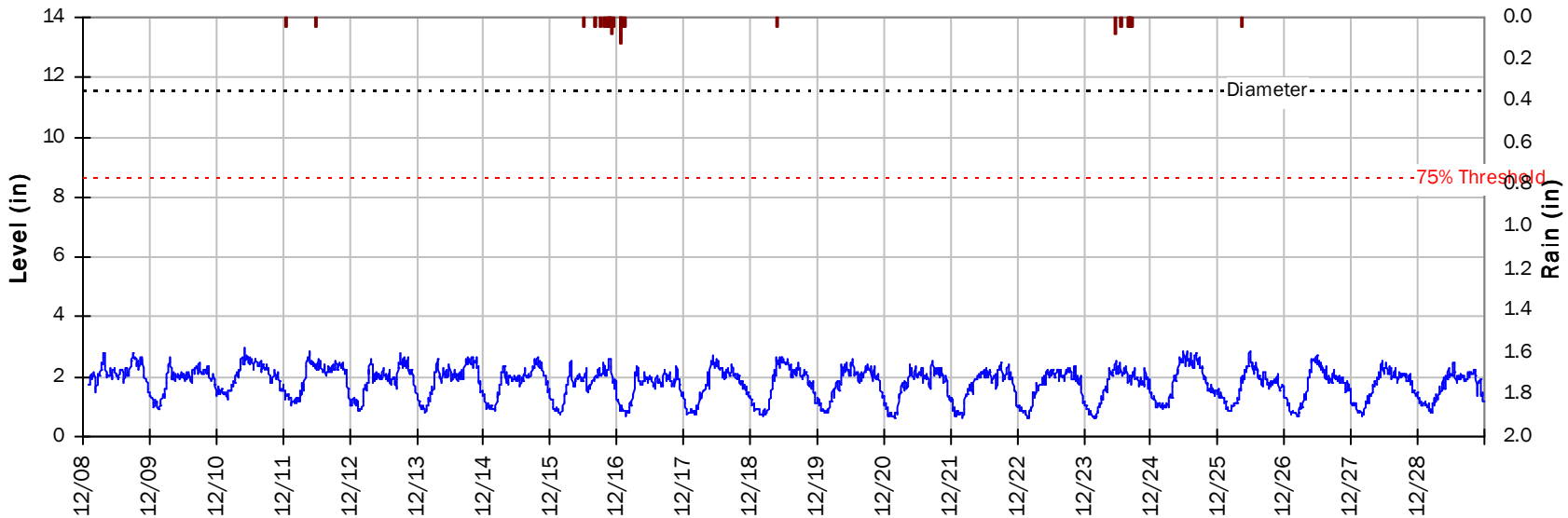
0.062 *mgd*

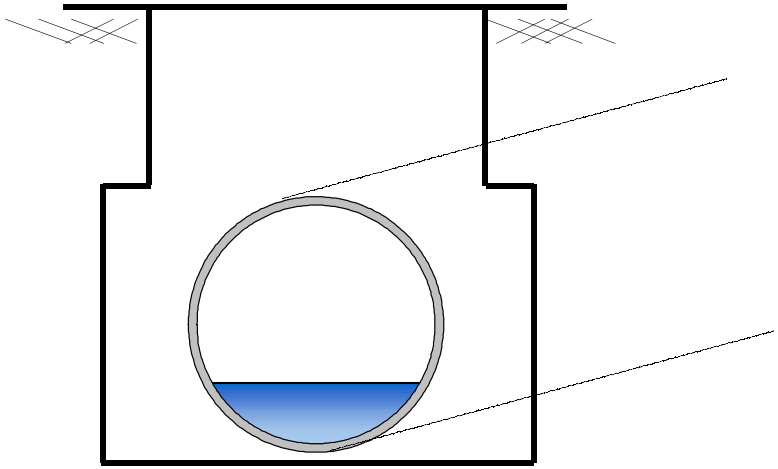


SITE 3A

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



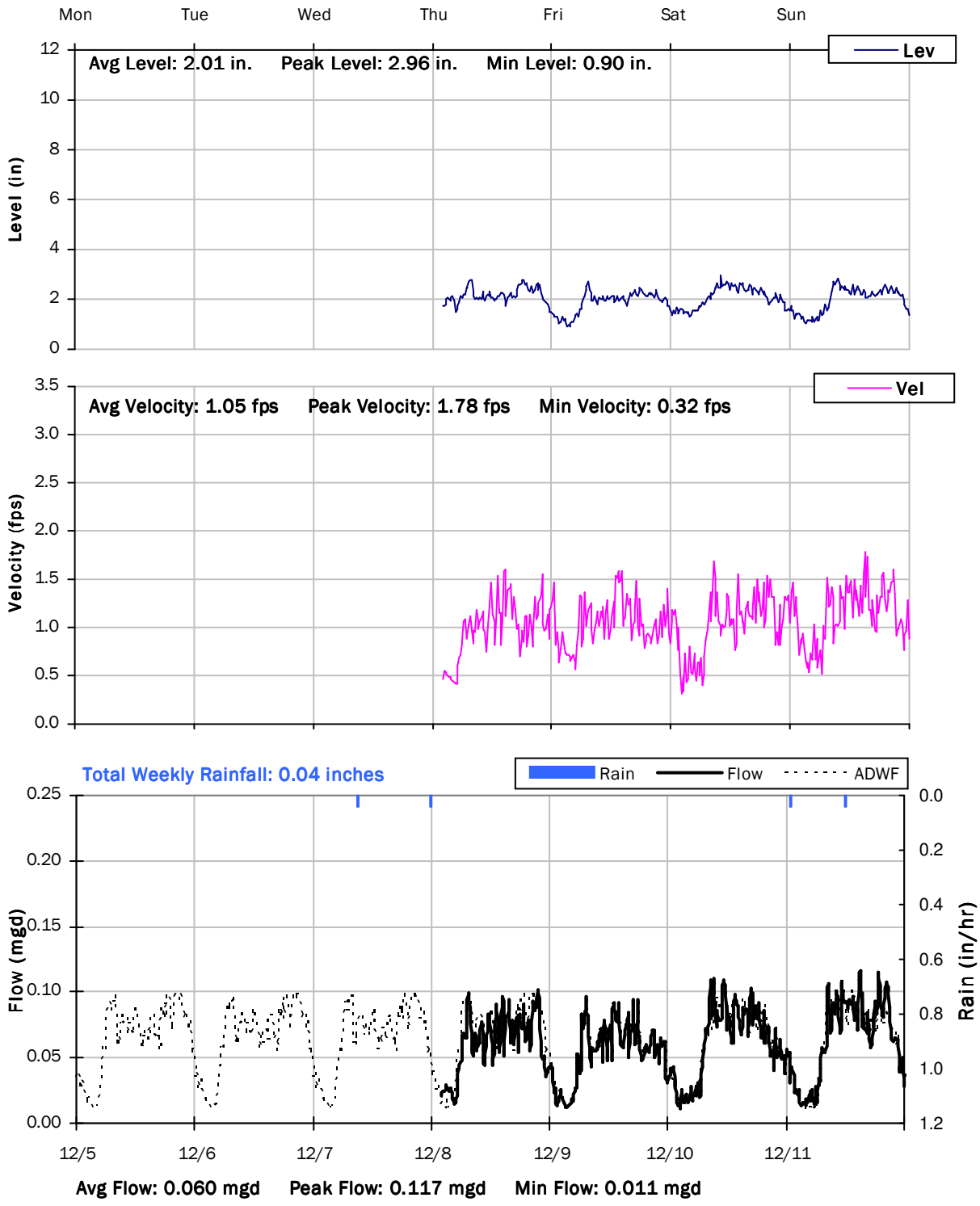


Pipe Diameter:	11.5	*inches*
Peak Measured Level:	2.96	*inches*
Peak d/D Ratio:	0.26	
Dry Weather Design Threshold Level:	5.75	*inches*

SITE 3A

Weekly Level, Velocity and Flow Hydrographs

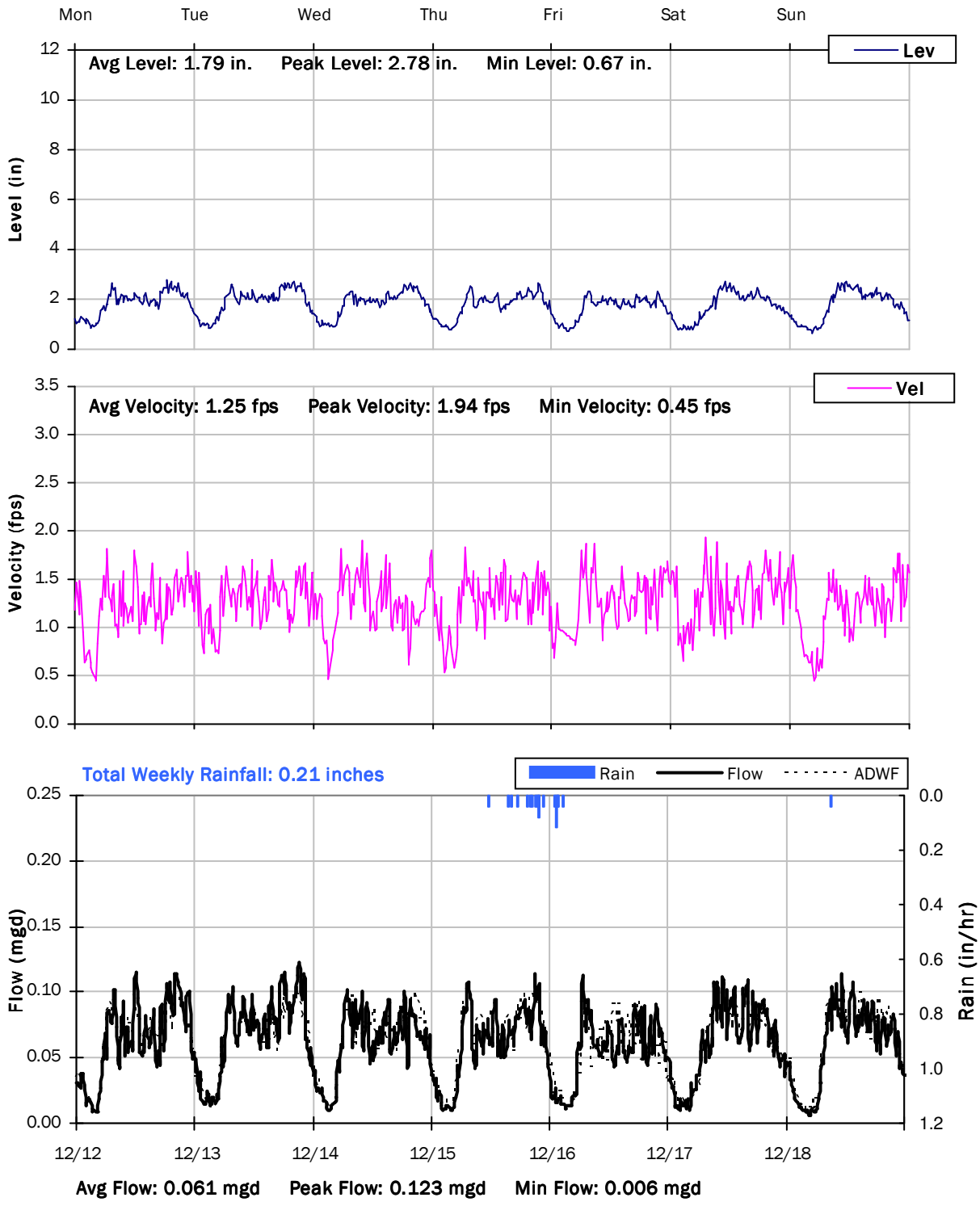
12/5/2016 to 12/12/2016



SITE 3A

Weekly Level, Velocity and Flow Hydrographs

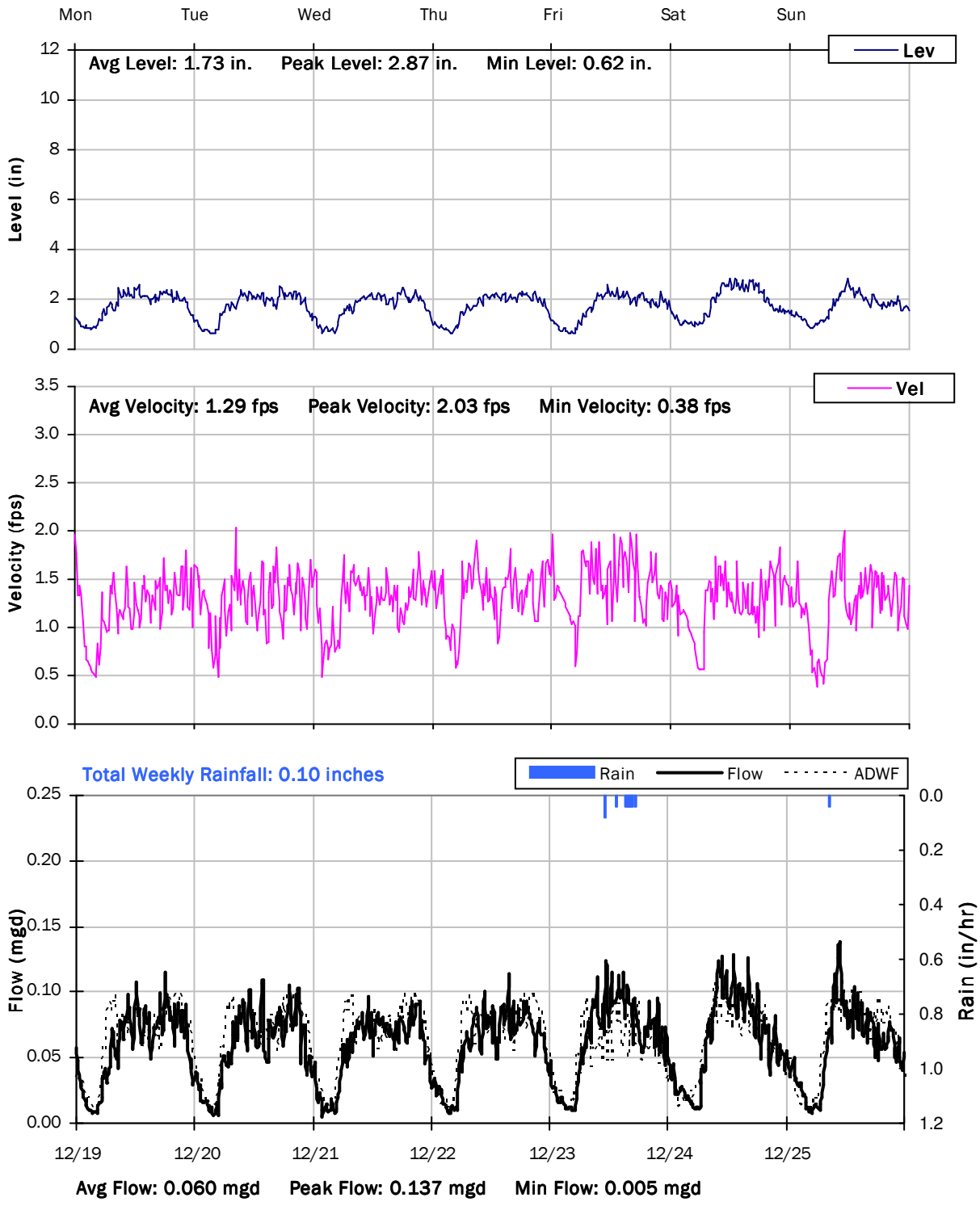
12/12/2016 to 12/19/2016



SITE 3A

Weekly Level, Velocity and Flow Hydrographs

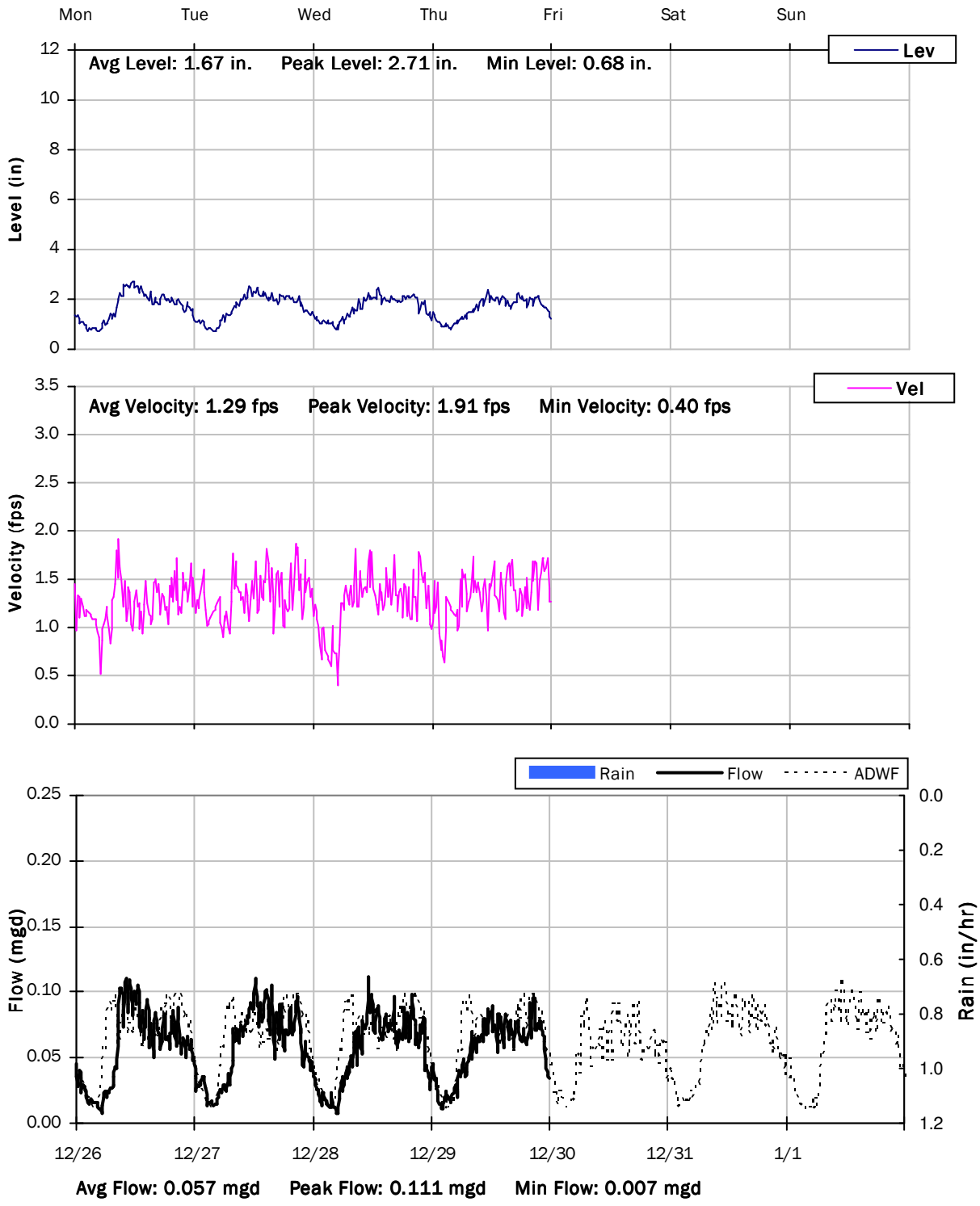
12/19/2016 to 12/26/2016



SITE 3A

Weekly Level, Velocity and Flow Hydrographs

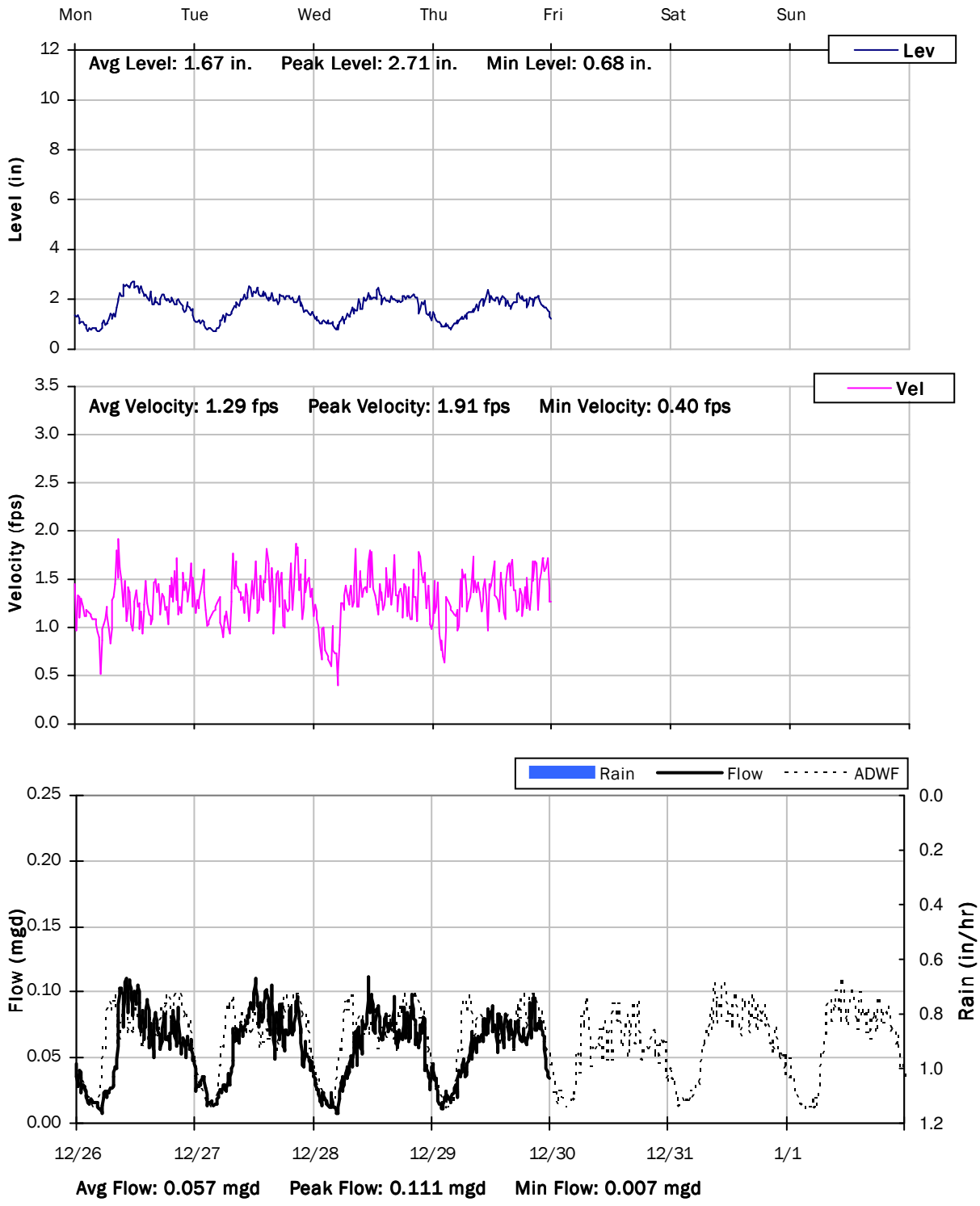
12/26/2016 to 1/2/2017



SITE 3A

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 3b

Location: Willow Street at Sandringham Street

Data Summary Report

Vicinity Map: Site 3b

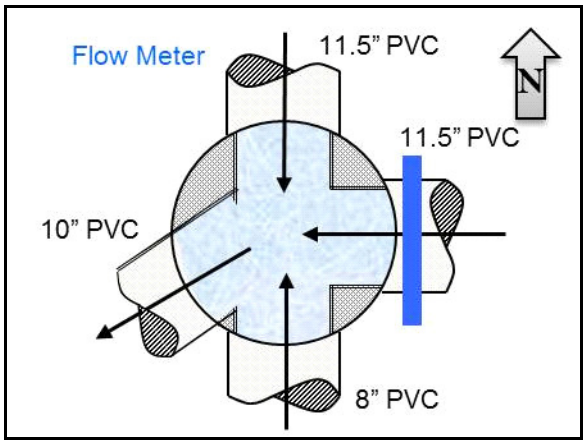
SITE 3B

Site Information

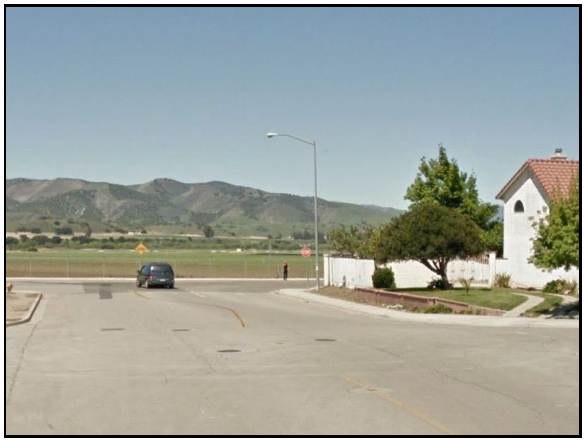
Location:	Willow Street at Sandringham Street
Coordinates:	121.1397° W, 36.2123° N
Rim Elevation (Earth):	316 feet
Pipe Diameter:	11.5 inches
ADWF:	0.041 mgd
Peak Measured Flow:	0.101 mgd

Satellite Map

Sewer Map



Flow Sketch



Street View

Plan View

SITE 3B

Additional Site Photos

Effluent Pipe

North Influent Pipe

SITE 3B

Additional Site Photos

East Influent Pipe

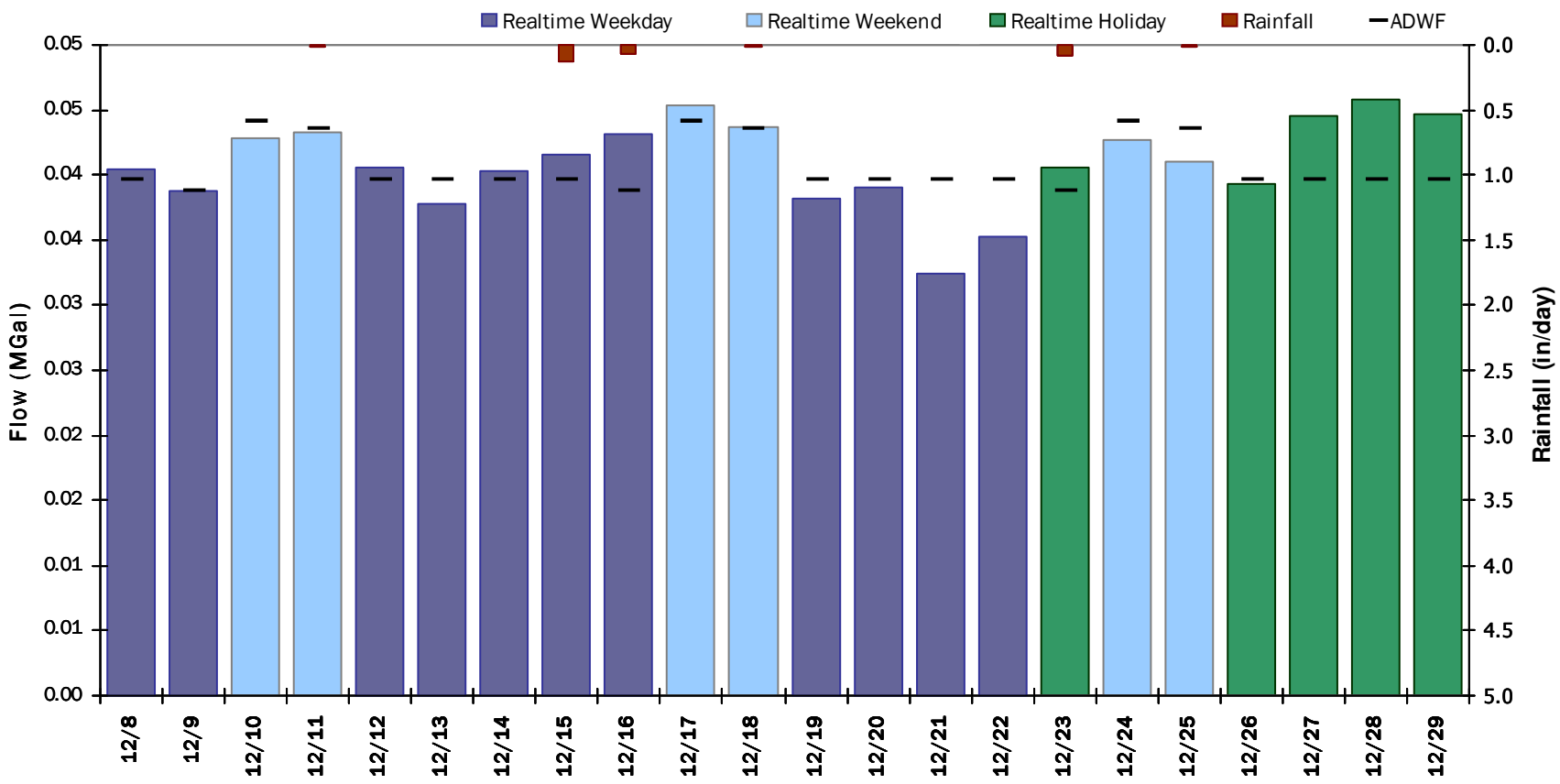
South Influent Pipe

SITE 3B

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.041 MGal **Peak Daily Flow: 0.046 MGal** **Min Daily Flow: 0.032 MGal**

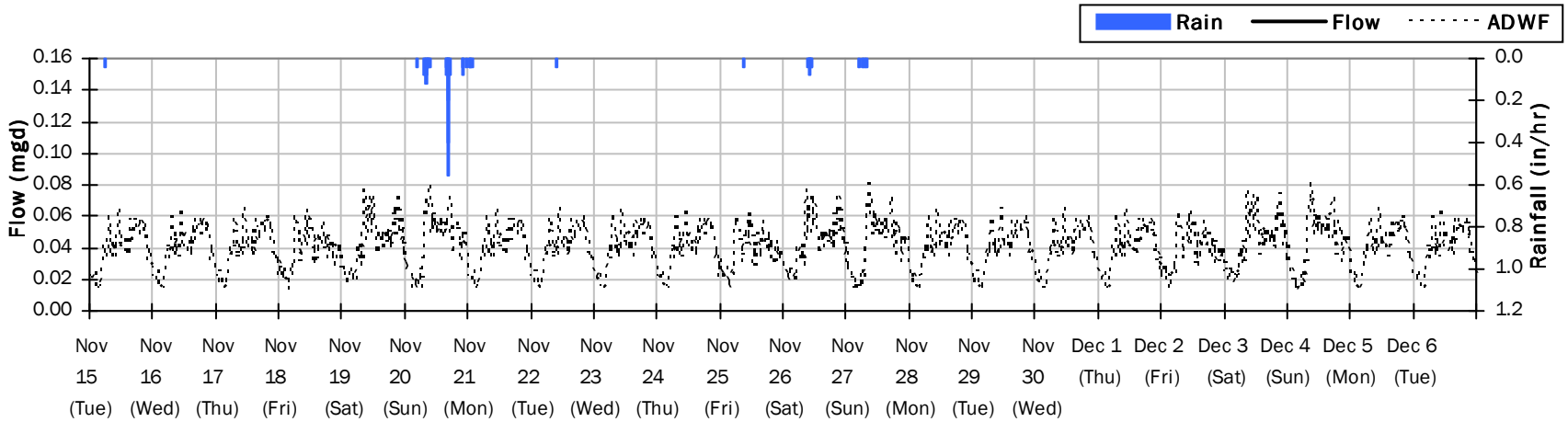
Total Period Rainfall: 0.33 inches

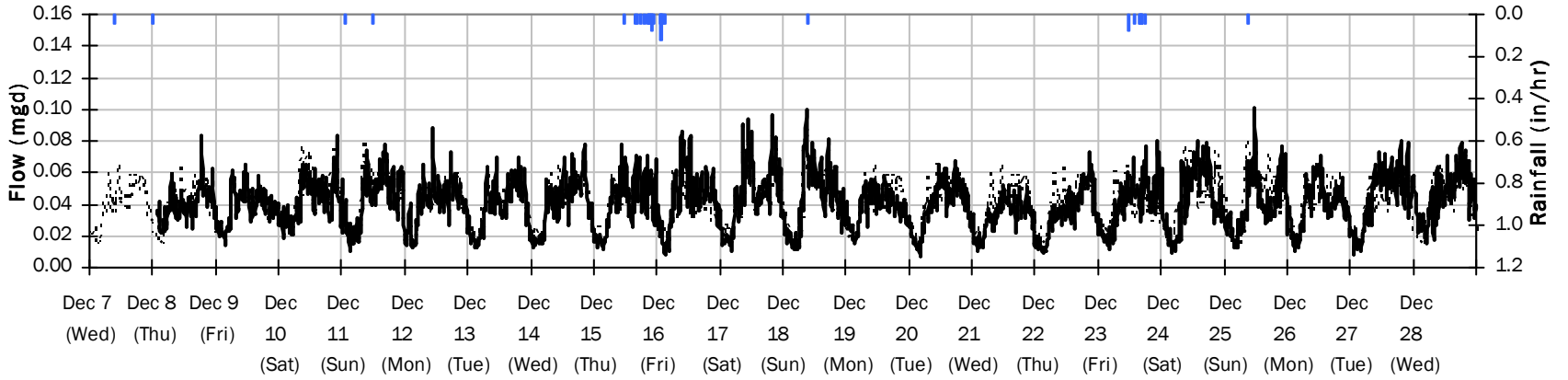


SITE 3B

Flow Summary: 11/15/2016 to 12/28/2016

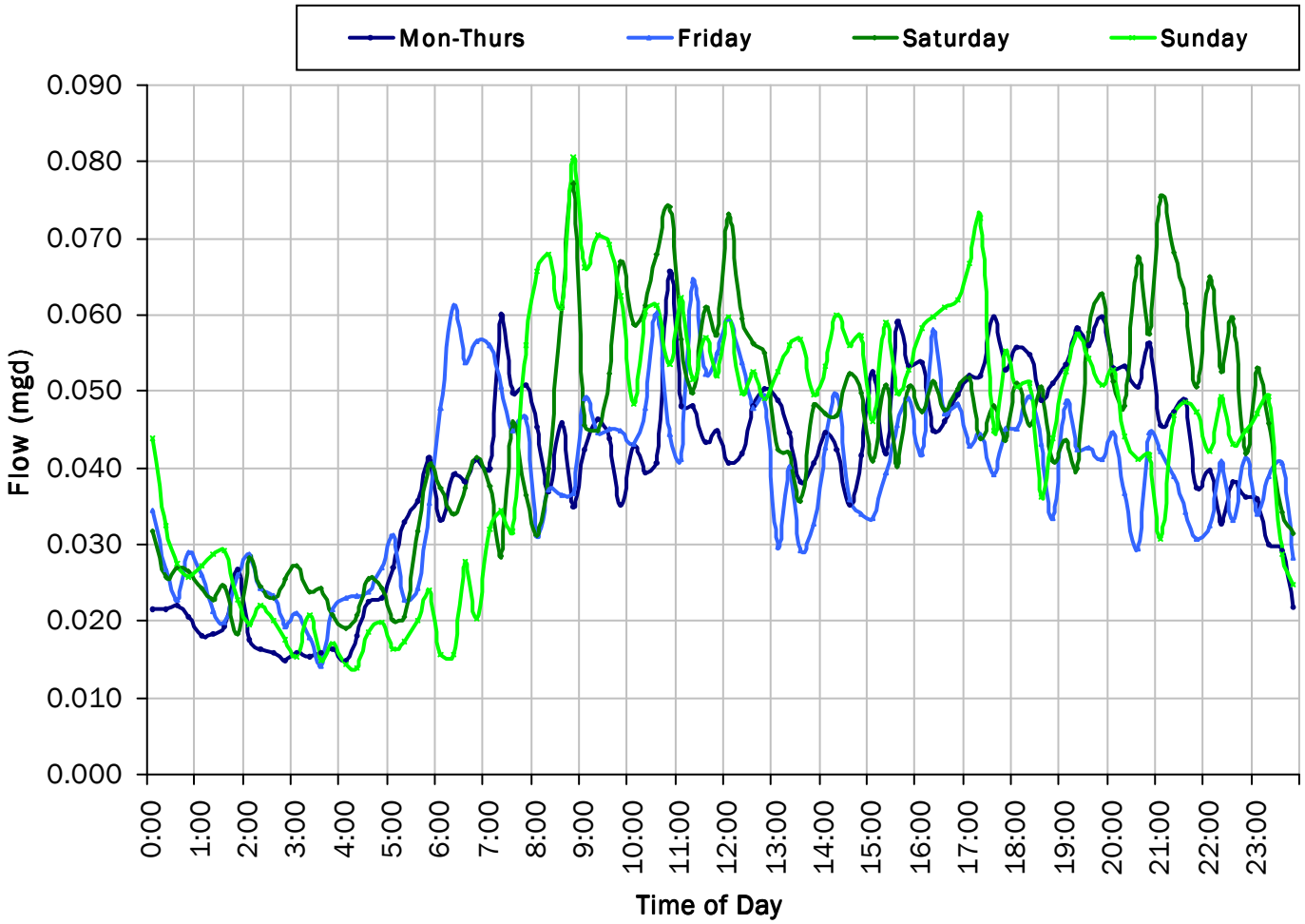
Total Period Rainfall: 1.01 inches **Avg Flow: 0.041 mgd** **Peak Flow: 0.101 mgd** **Min Flow: 0.006 mgd**





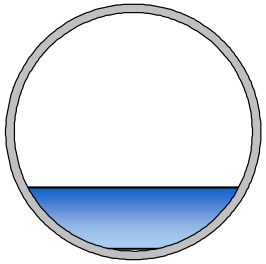
SITE 3B

Average Dry Weather Flow Hydrographs



ADWF:

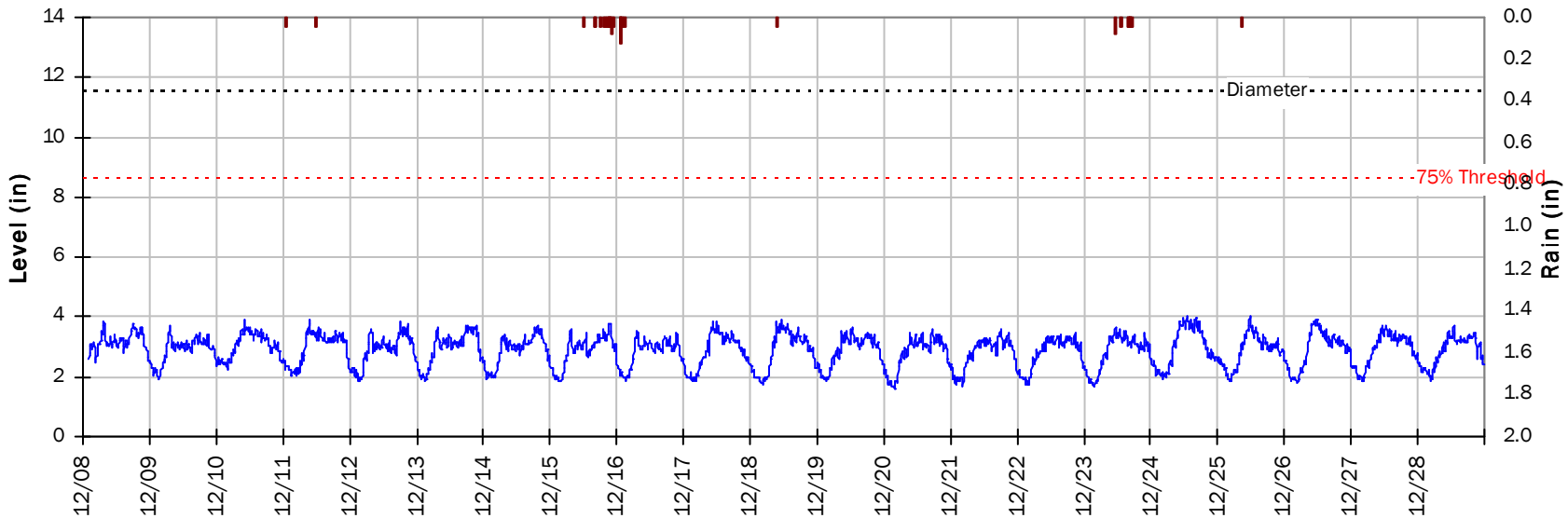
0.041 *mgd*

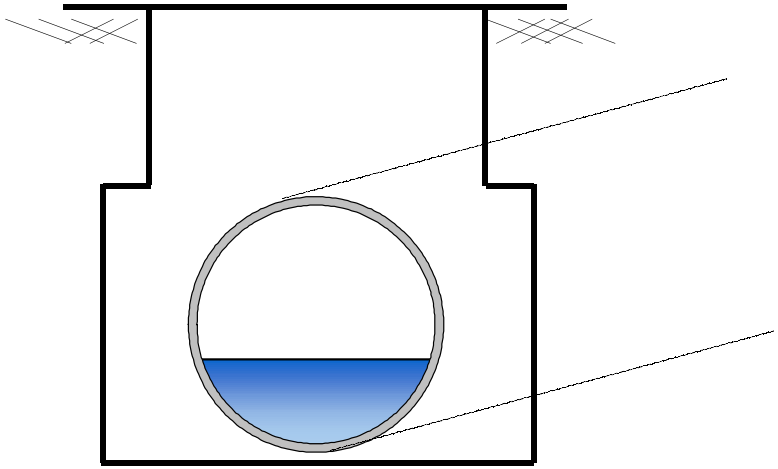


SITE 3B

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



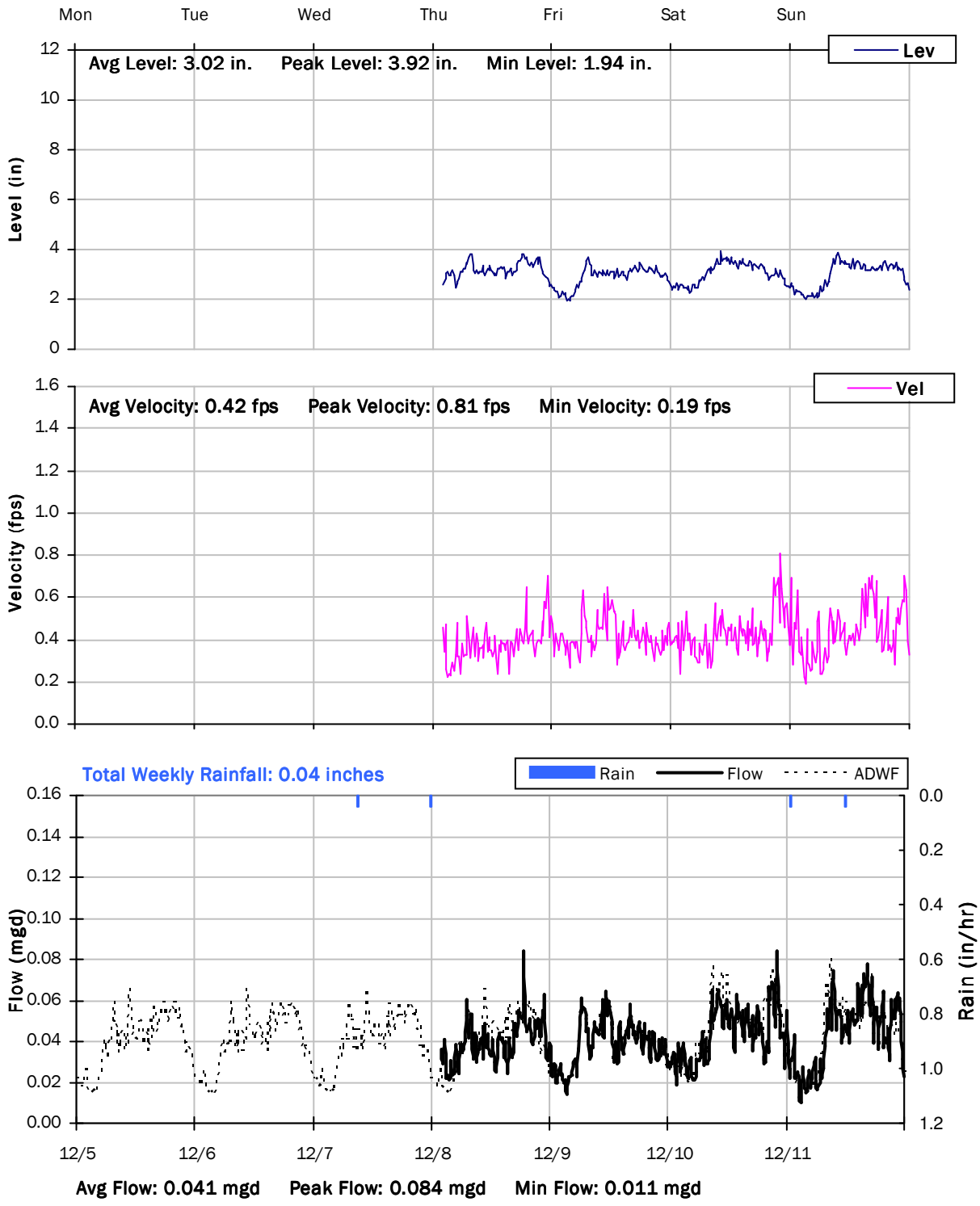


Pipe Diameter:	11.5	*inches*
Peak Measured Level:	4.02	*inches*
Peak d/D Ratio:	0.35	
Dry Weather Design Threshold Level:	5.75	*inches*

SITE 3B

Weekly Level, Velocity and Flow Hydrographs

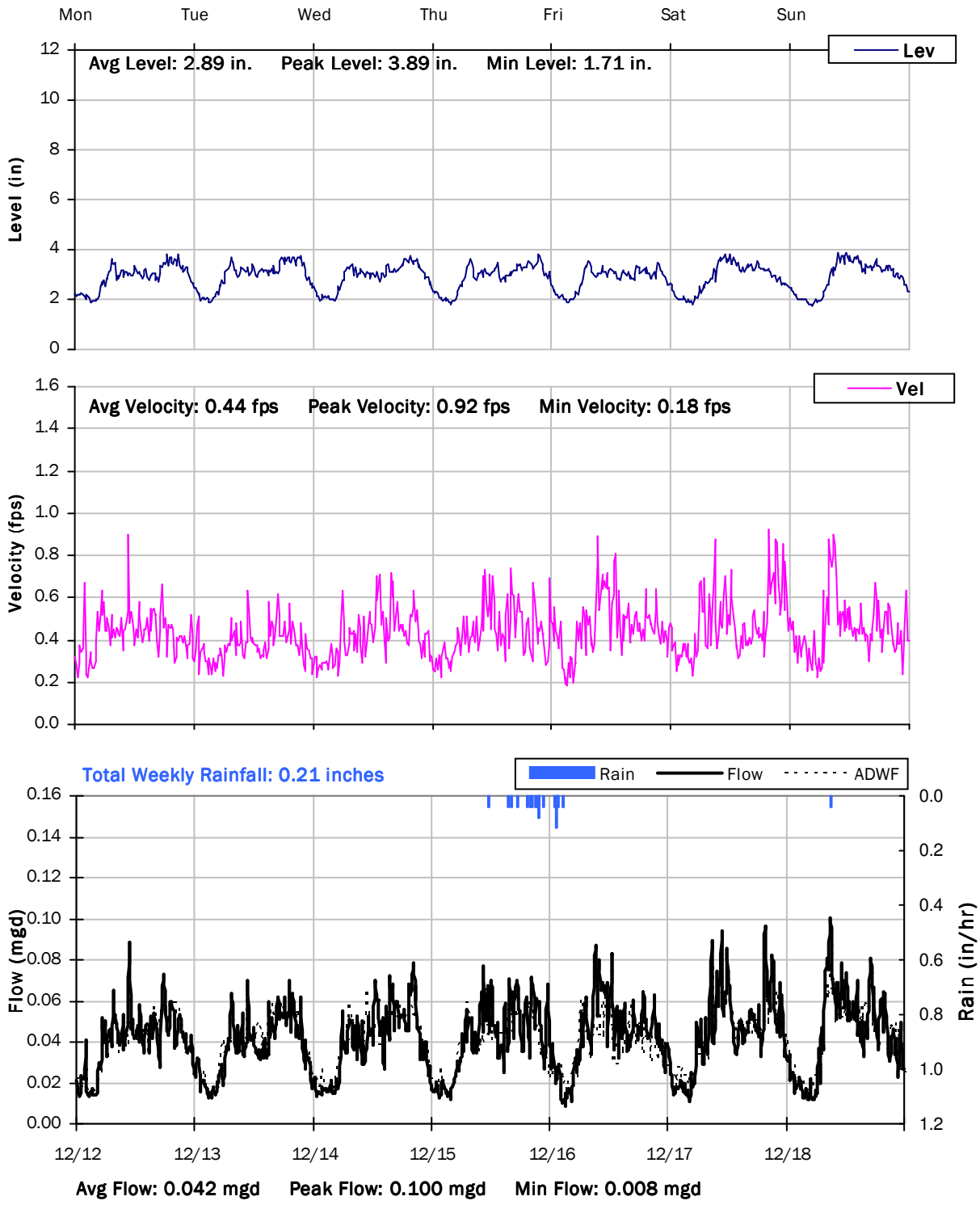
12/5/2016 to 12/12/2016



SITE 3B

Weekly Level, Velocity and Flow Hydrographs

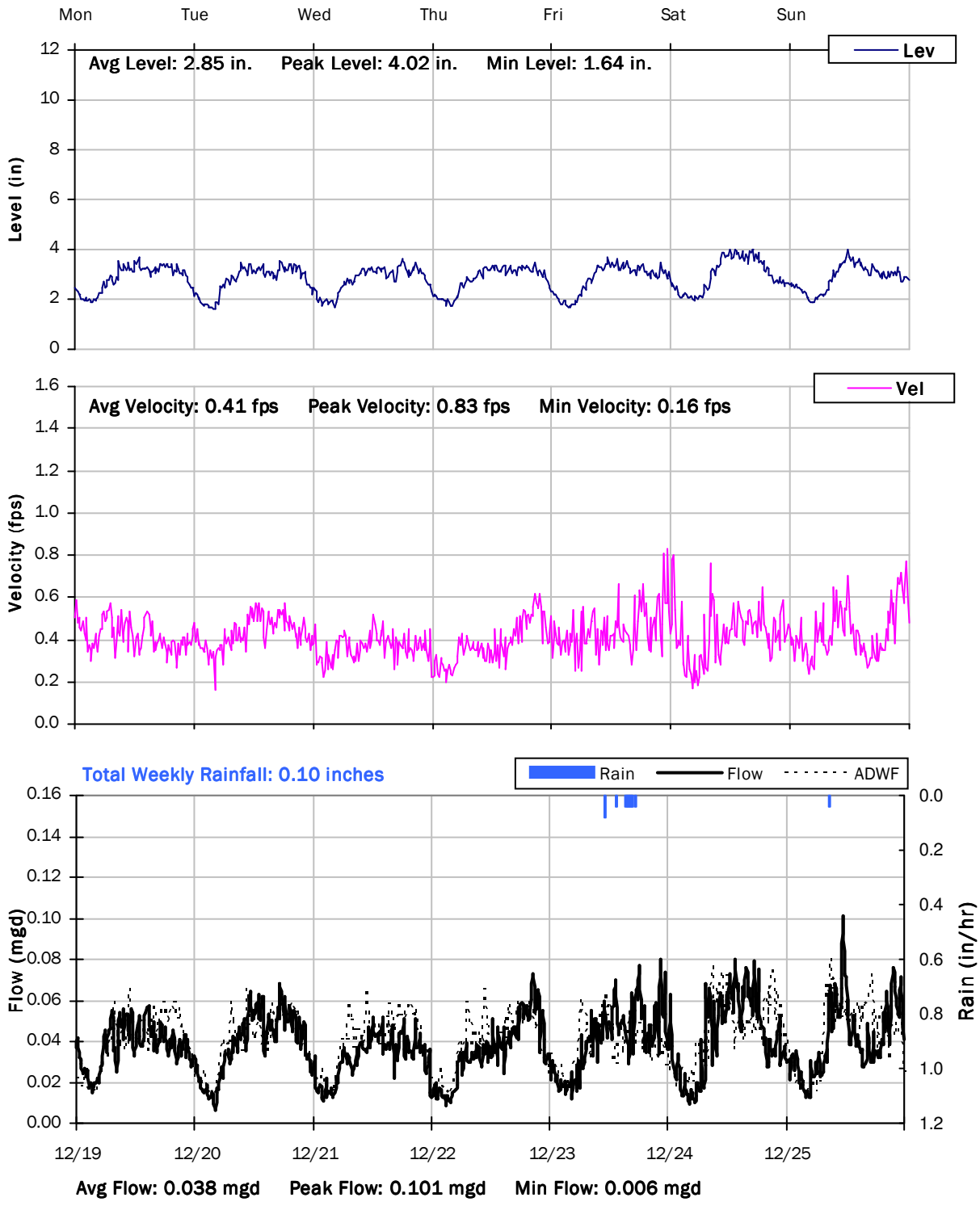
12/12/2016 to 12/19/2016



SITE 3B

Weekly Level, Velocity and Flow Hydrographs

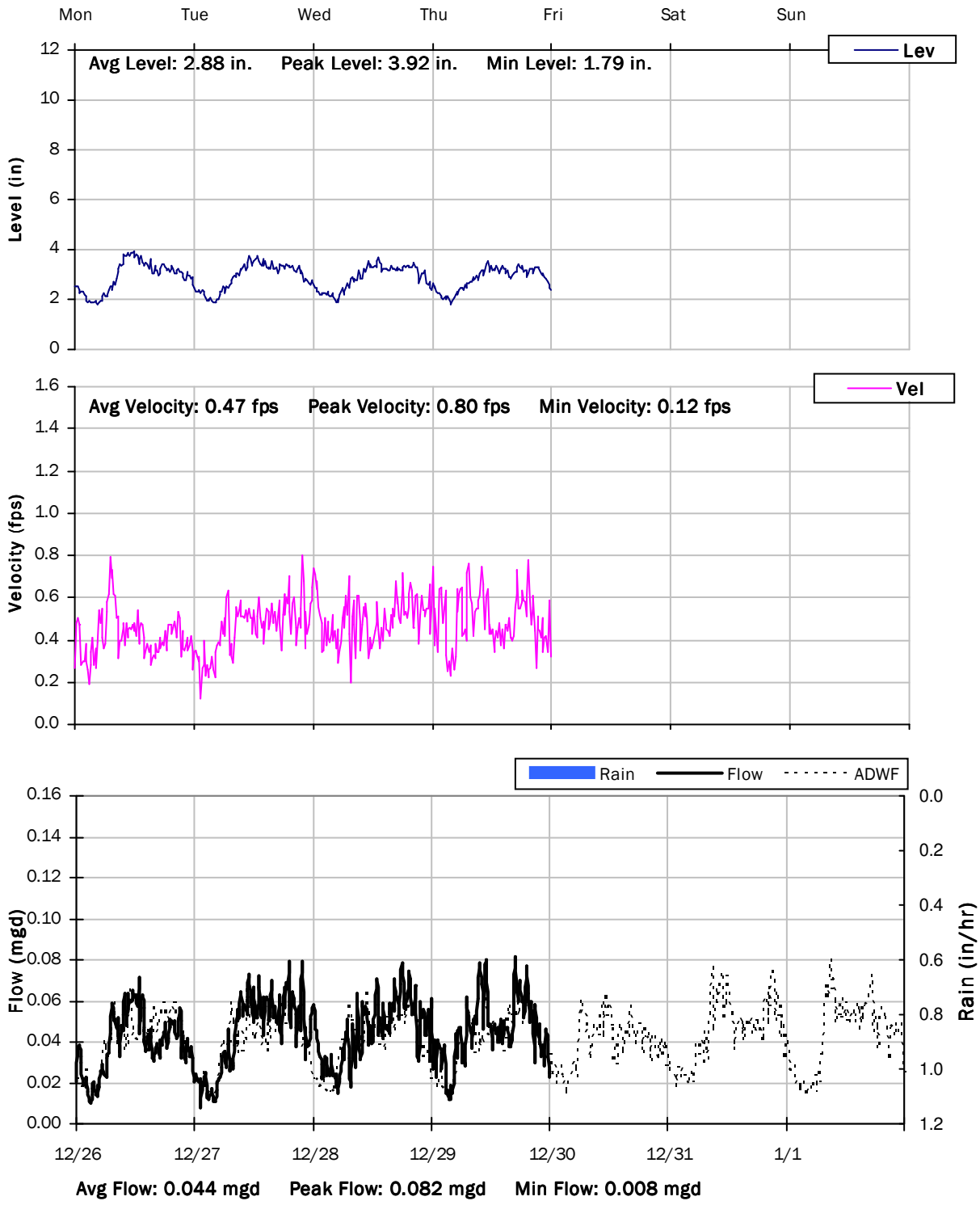
12/19/2016 to 12/26/2016



SITE 3B

Weekly Level, Velocity and Flow Hydrographs

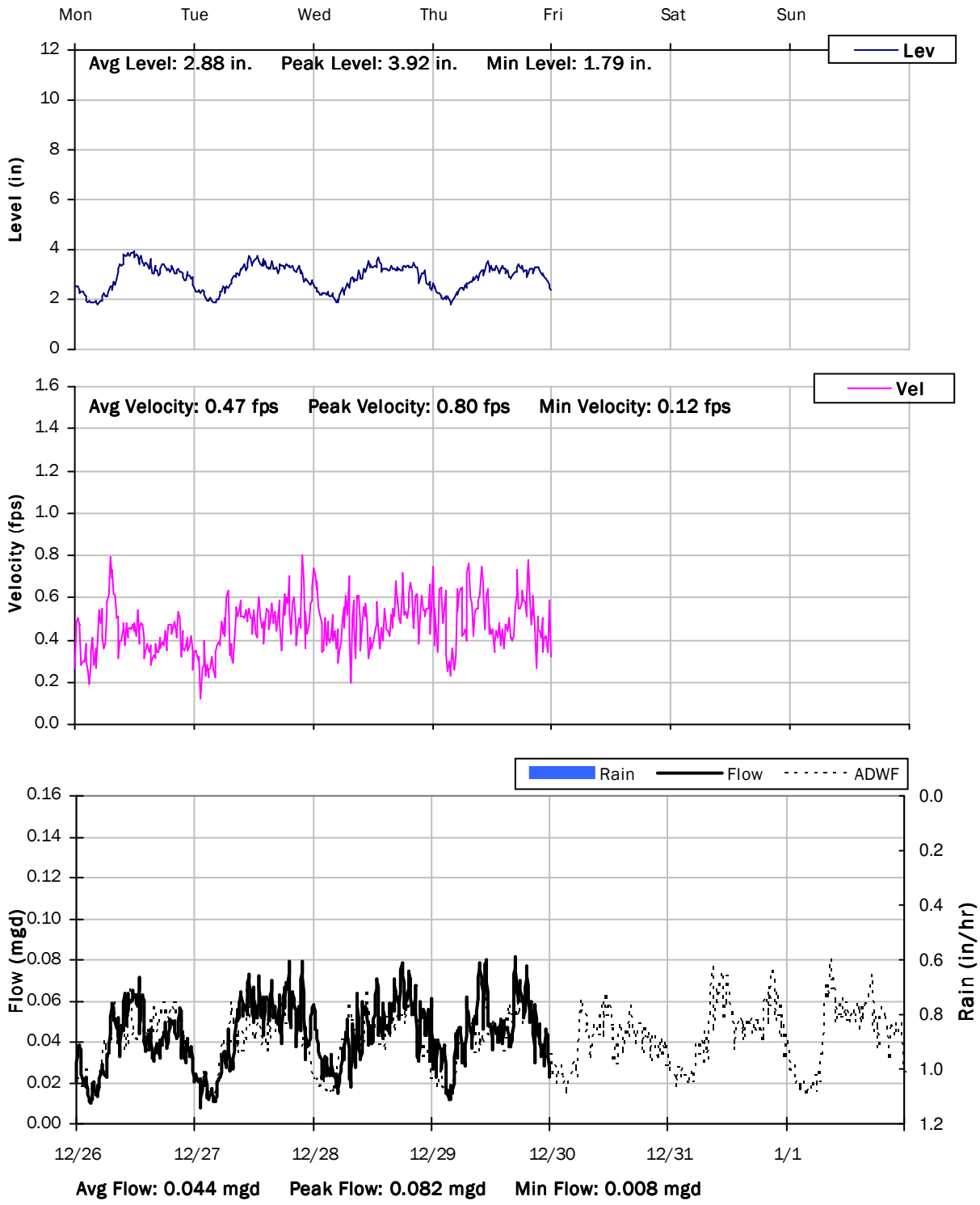
12/26/2016 to 1/2/2017



SITE 3B

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 4

Location: Broadway Street east of San Antonio Drive, west of Franciscan Way

Data Summary Report

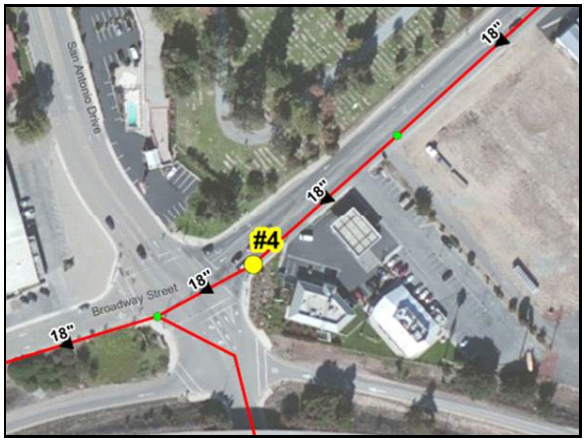
Vicinity Map: Site 4

SITE 4

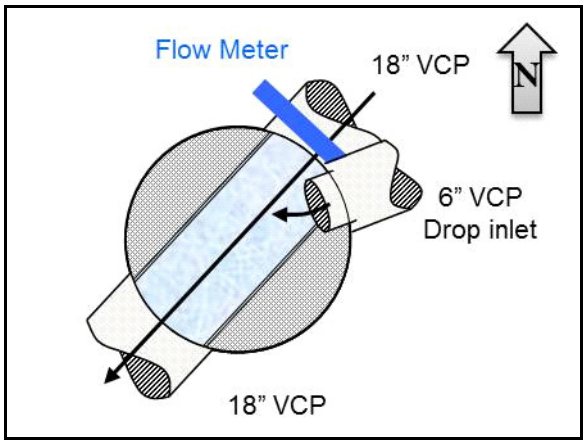
Site Information

Location:	Broadway Street east of San Antonio Drive, west of Franciscan Way
Coordinates:	121.1375° W, 36.2054° N
Rim Elevation (Earth):	304 feet
Pipe Diameter:	18 inches
ADWF:	0.077 mgd
Peak Measured Flow:	0.241 mgd

Satellite Map



Sewer Map



Flow Sketch

Street View

Plan View

SITE 4

Additional Site Photos

Effluent Pipe

Influent Pipe

SITE 4

Additional Site Photos

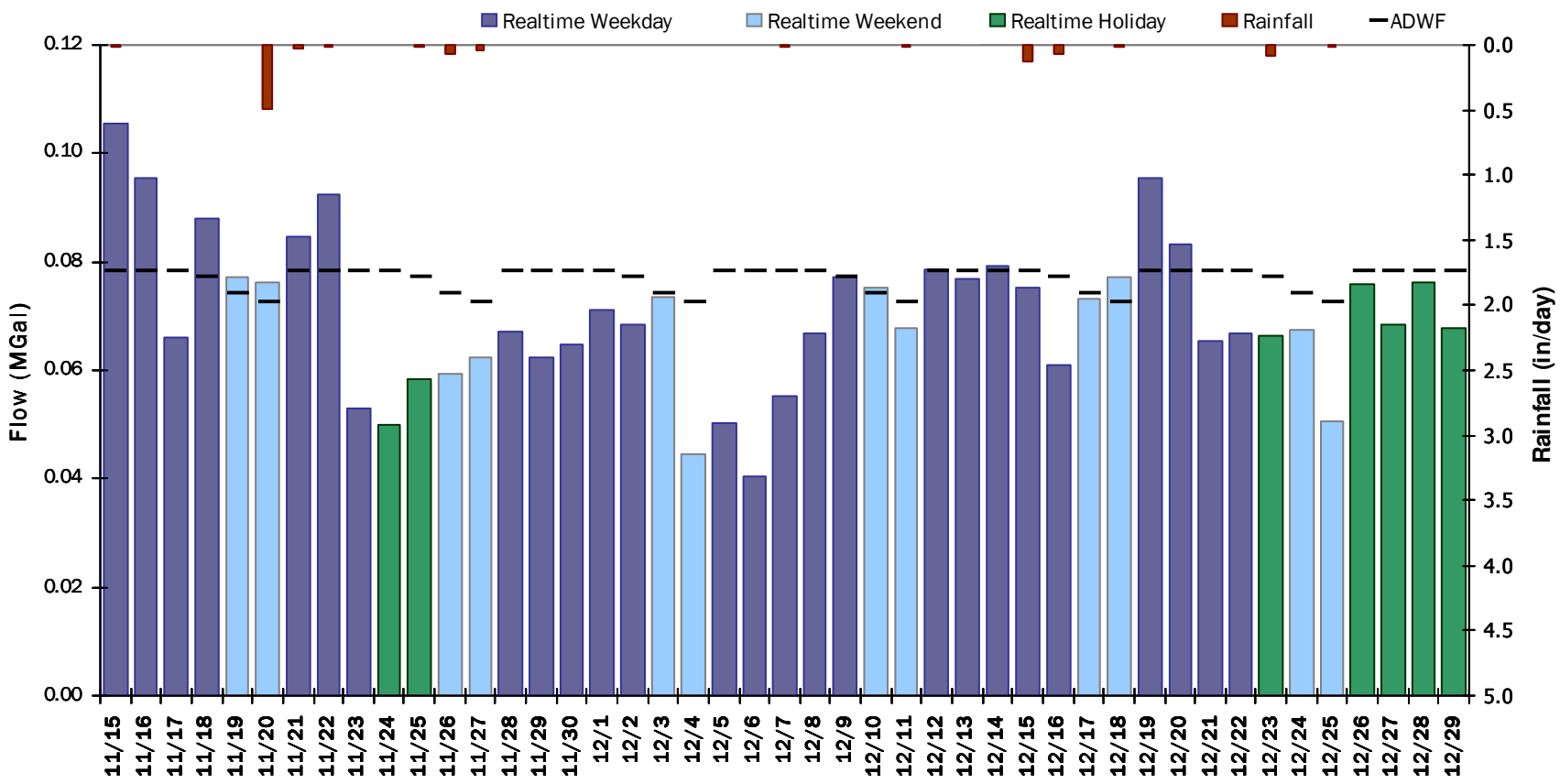
Drop Influent Pipe

SITE 4

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.070 MGal **Peak Daily Flow: 0.105 MGal** **Min Daily Flow: 0.041 MGal**

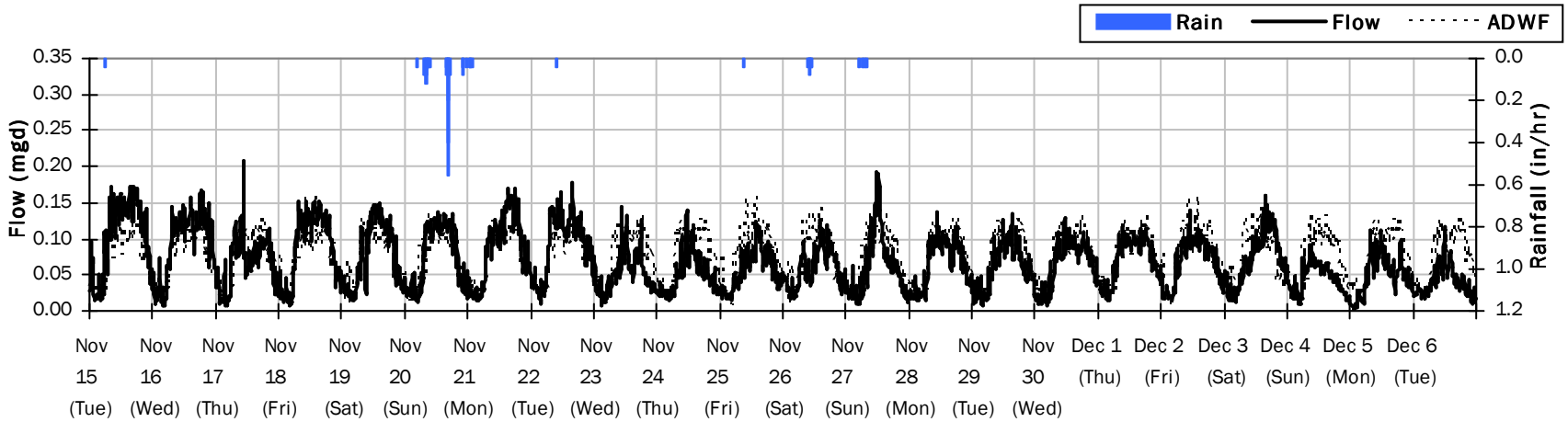
Total Period Rainfall: 1.01 inches

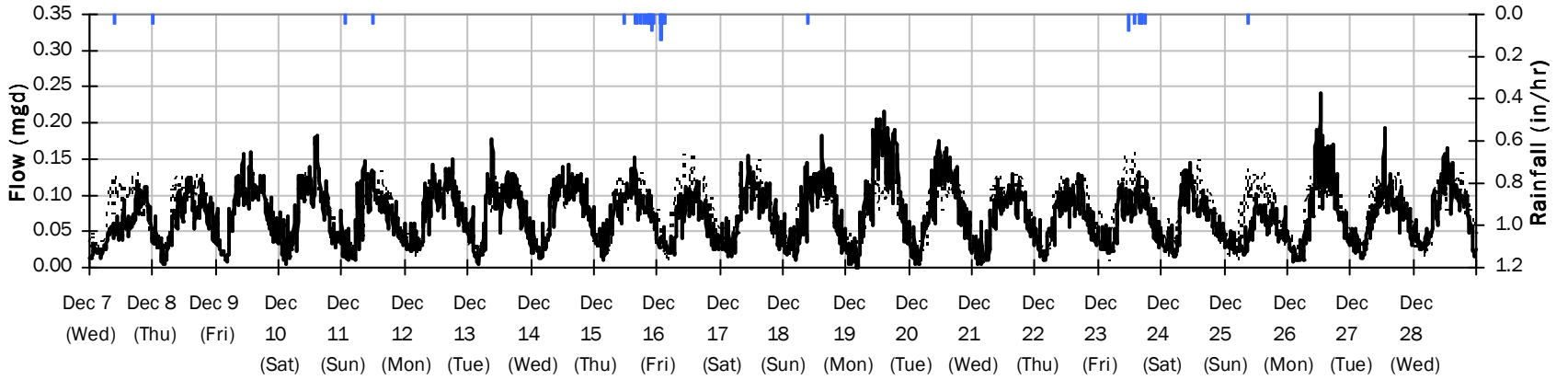


SITE 4

Flow Summary: 11/15/2016 to 12/28/2016

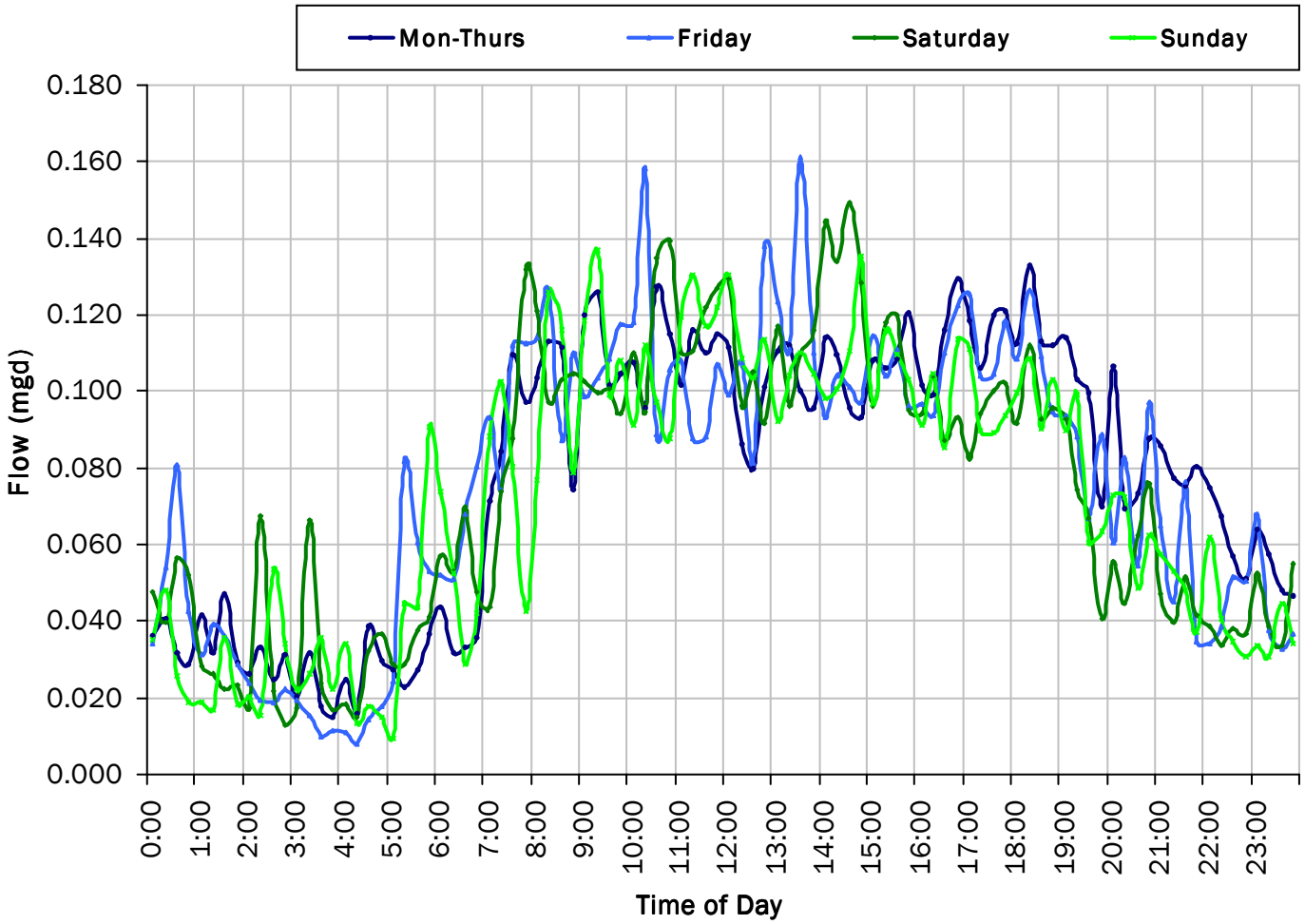
Total Period Rainfall: 1.01 inches **Avg Flow: 0.070 mgd** **Peak Flow: 0.241 mgd** **Min Flow: 0.000 mgd**





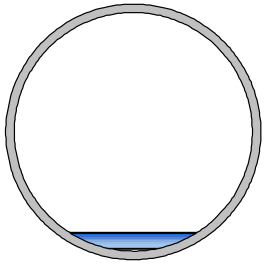
SITE 4

Average Dry Weather Flow Hydrographs



ADWF:

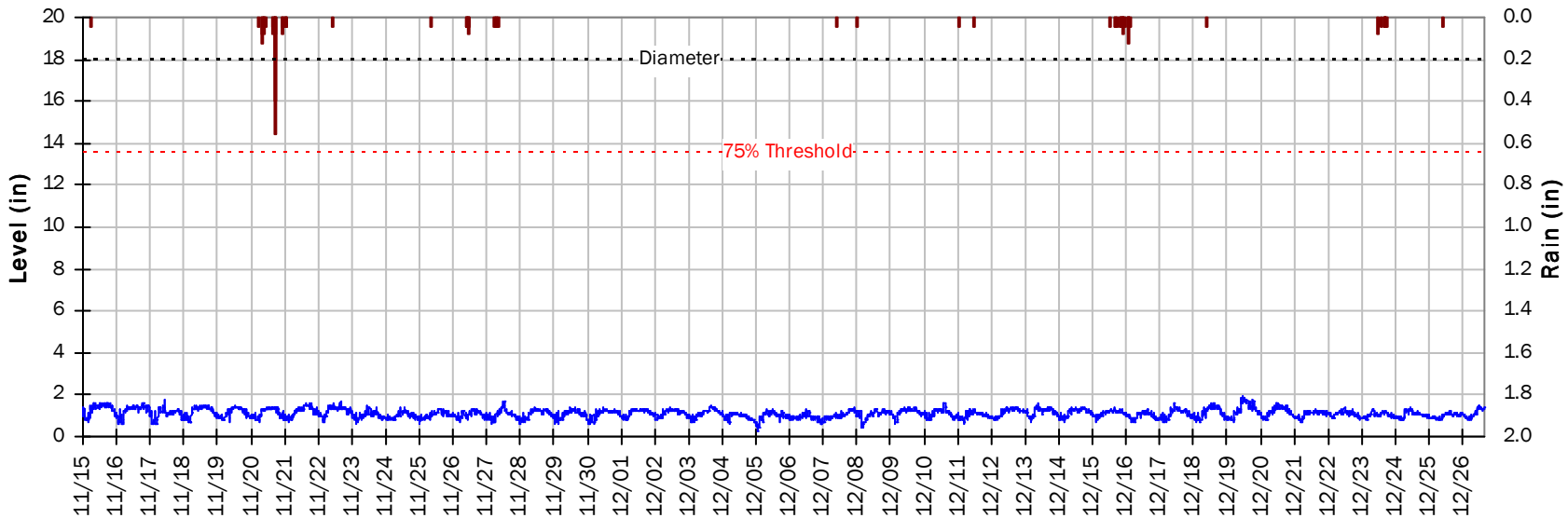
0.077 *mgd*

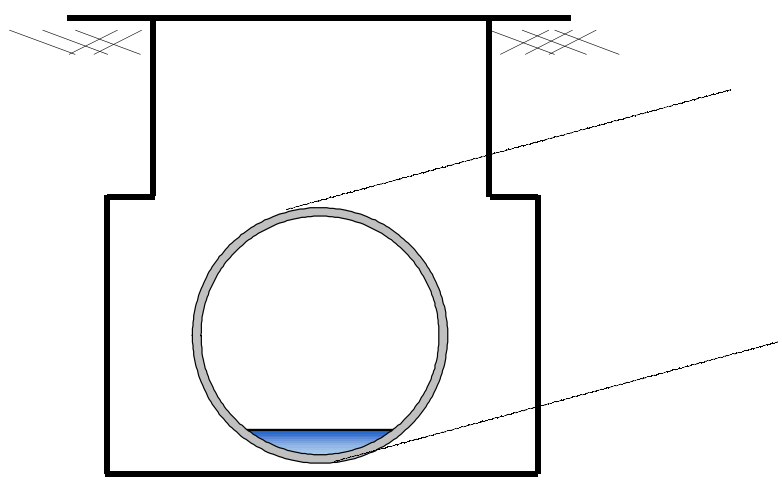


SITE 4

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



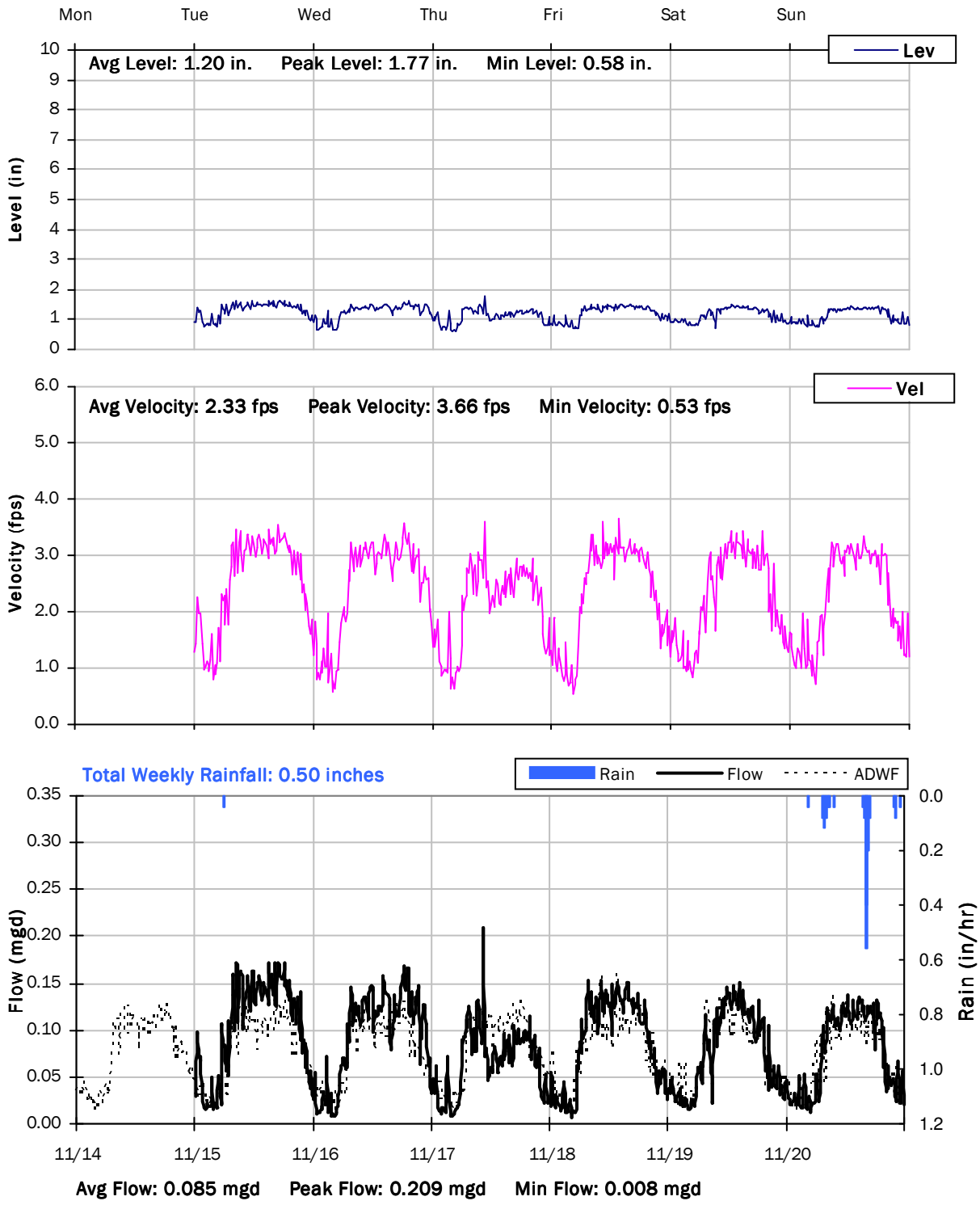


Pipe Diameter:	18	*inches*
Peak Measured Level:	1.94	*inches*
Peak d/D Ratio:	0.11	
Dry Weather Design Threshold Level:	13.5	*inches*

SITE 4

Weekly Level, Velocity and Flow Hydrographs

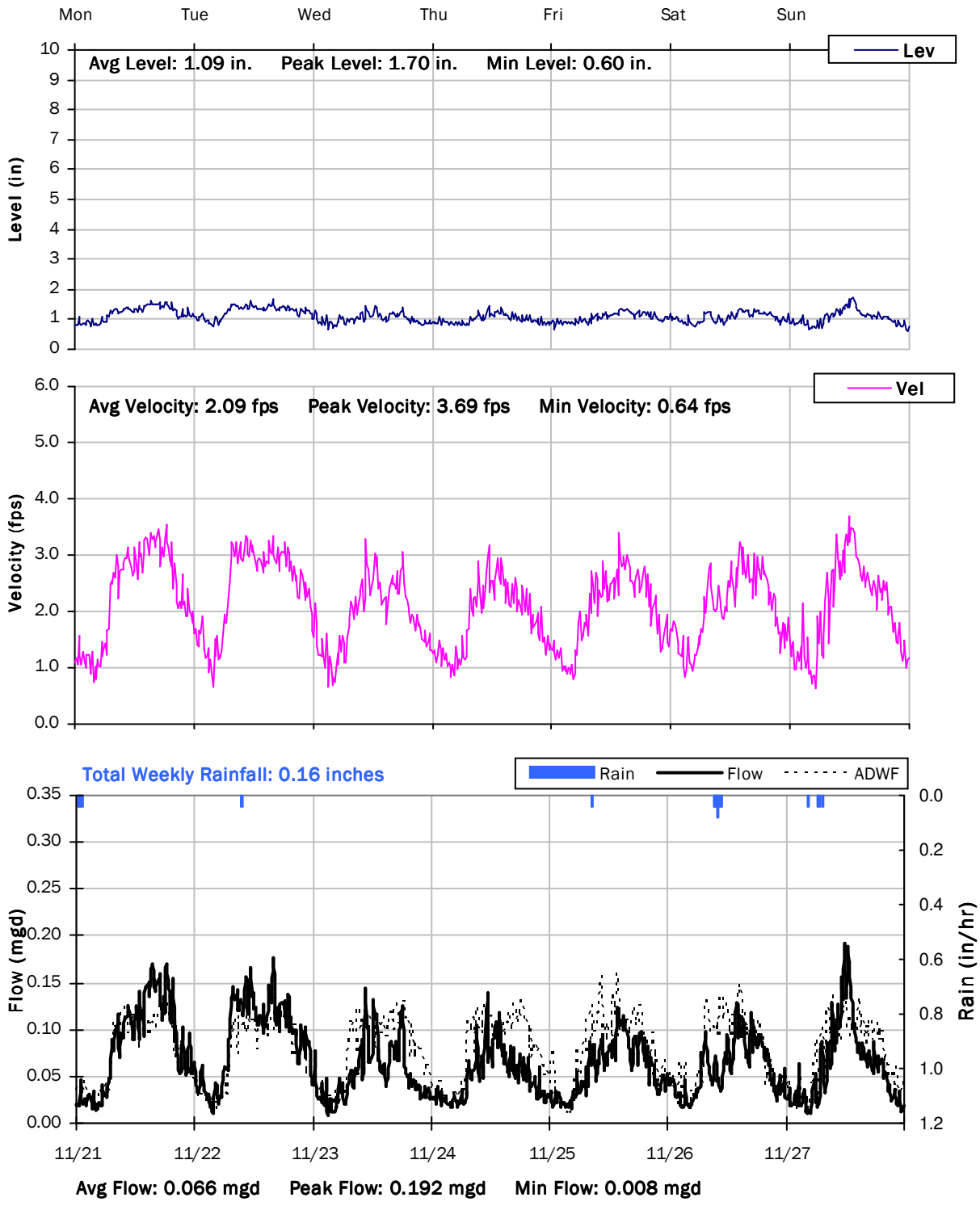
11/14/2016 to 11/21/2016



SITE 4

Weekly Level, Velocity and Flow Hydrographs

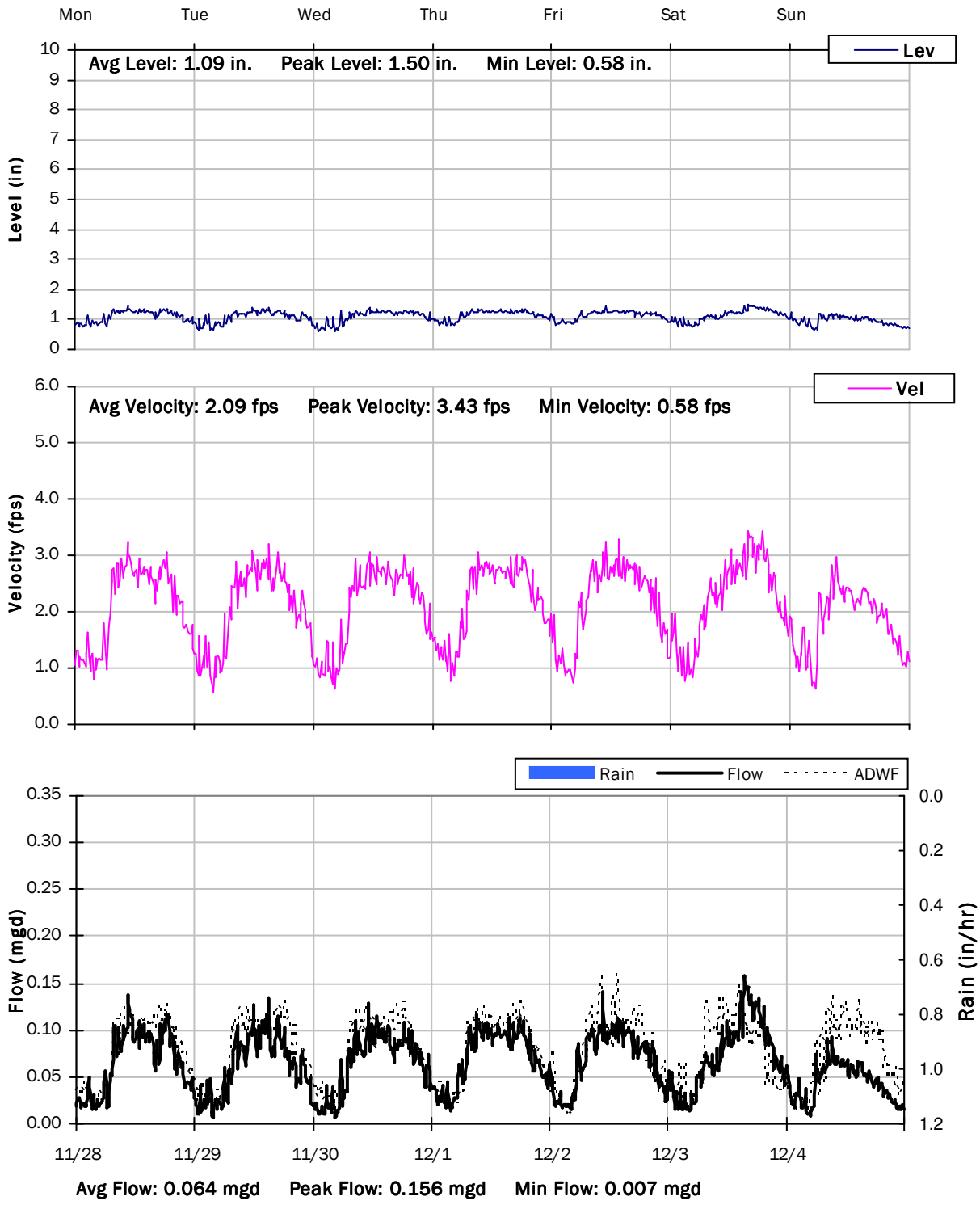
11/21/2016 to 11/28/2016



SITE 4

Weekly Level, Velocity and Flow Hydrographs

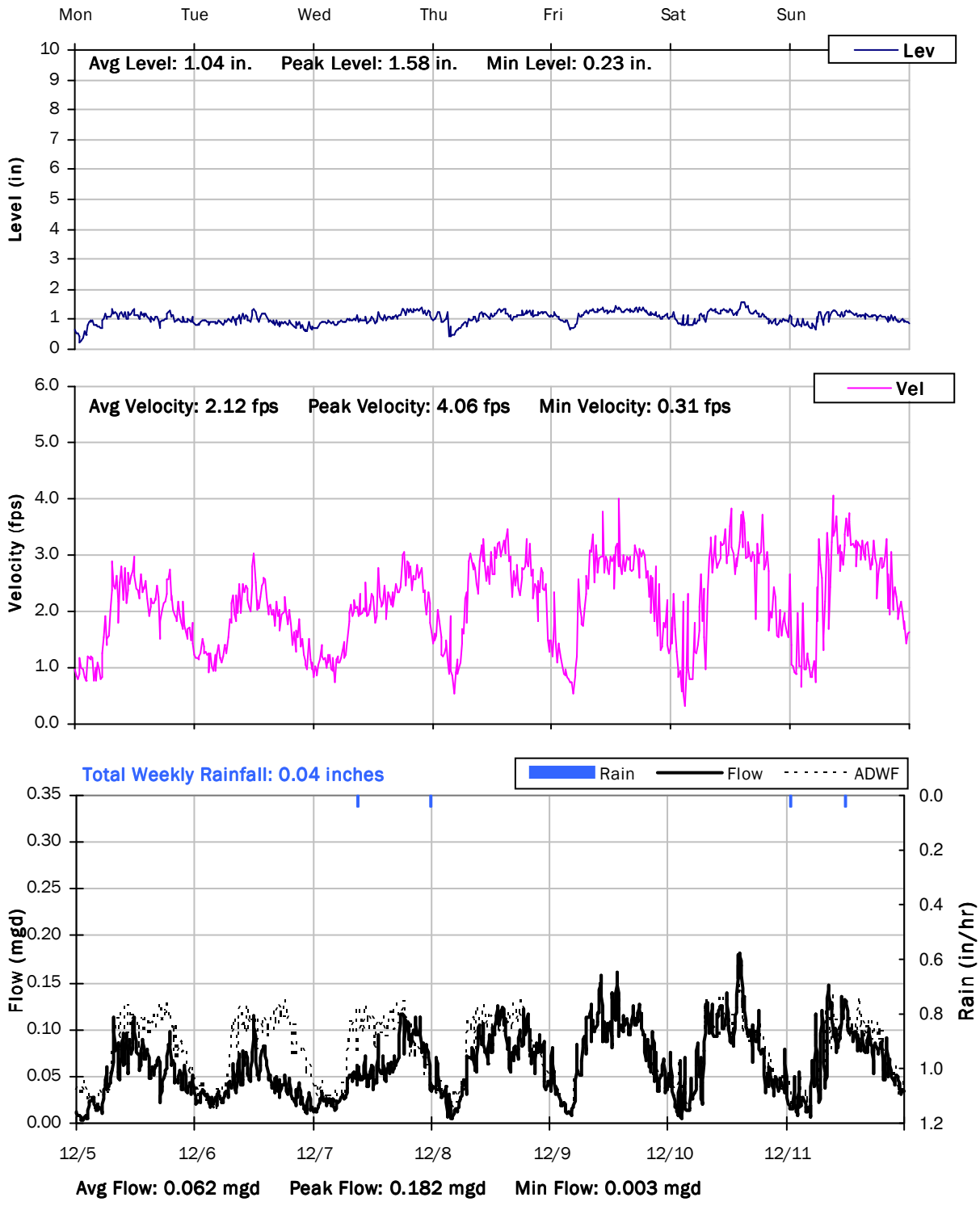
11/28/2016 to 12/5/2016



SITE 4

Weekly Level, Velocity and Flow Hydrographs

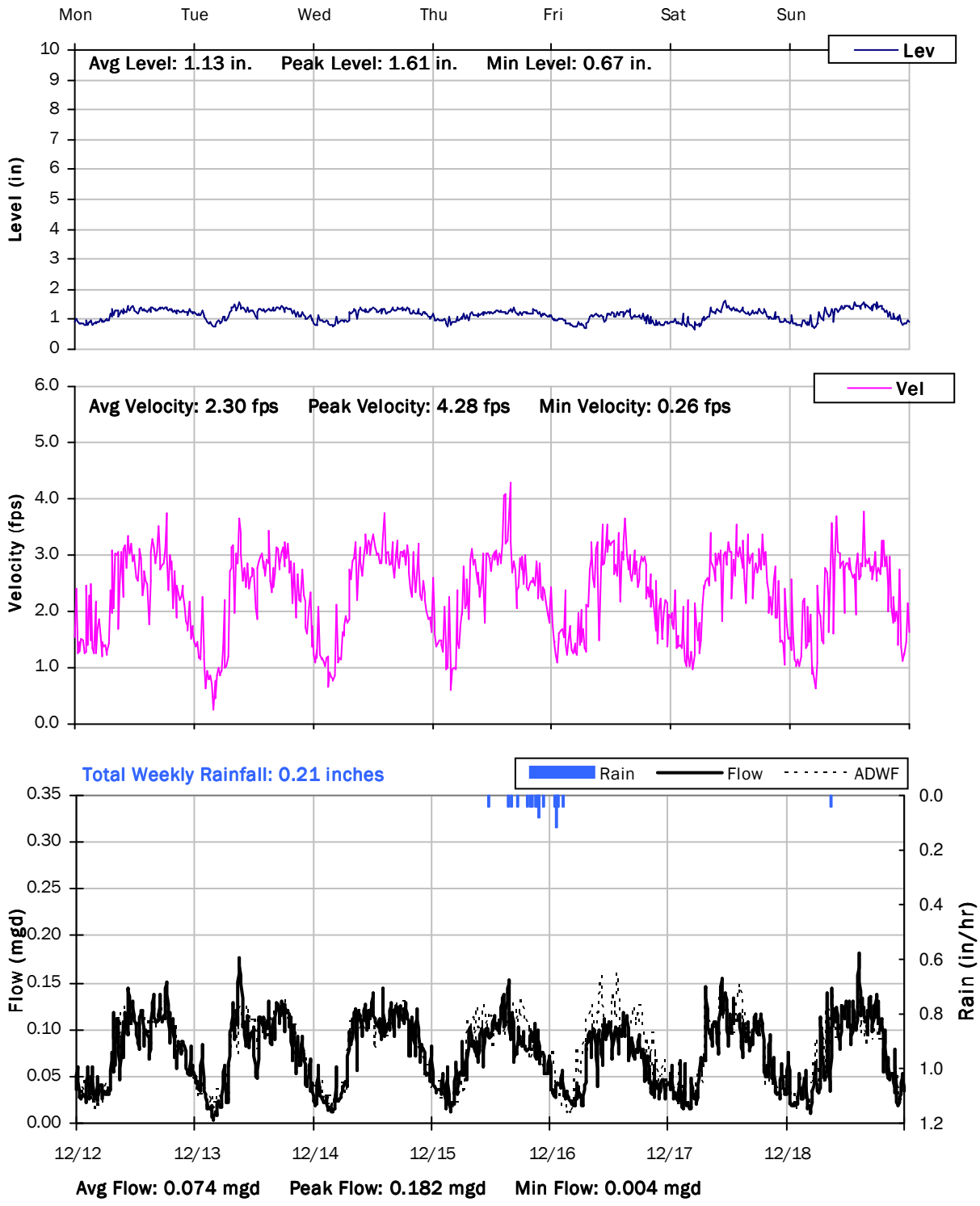
12/5/2016 to 12/12/2016



SITE 4

Weekly Level, Velocity and Flow Hydrographs

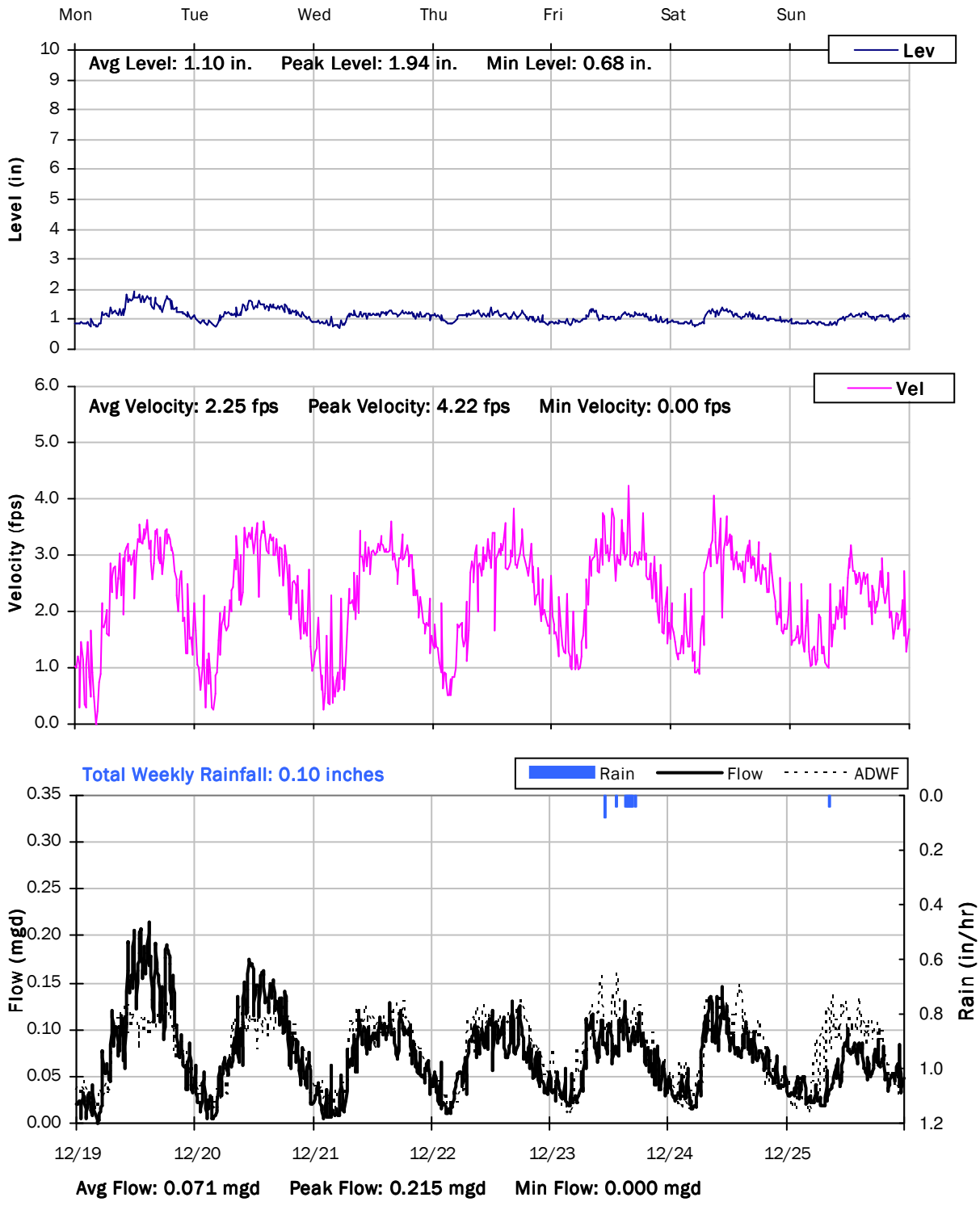
12/12/2016 to 12/19/2016



SITE 4

Weekly Level, Velocity and Flow Hydrographs

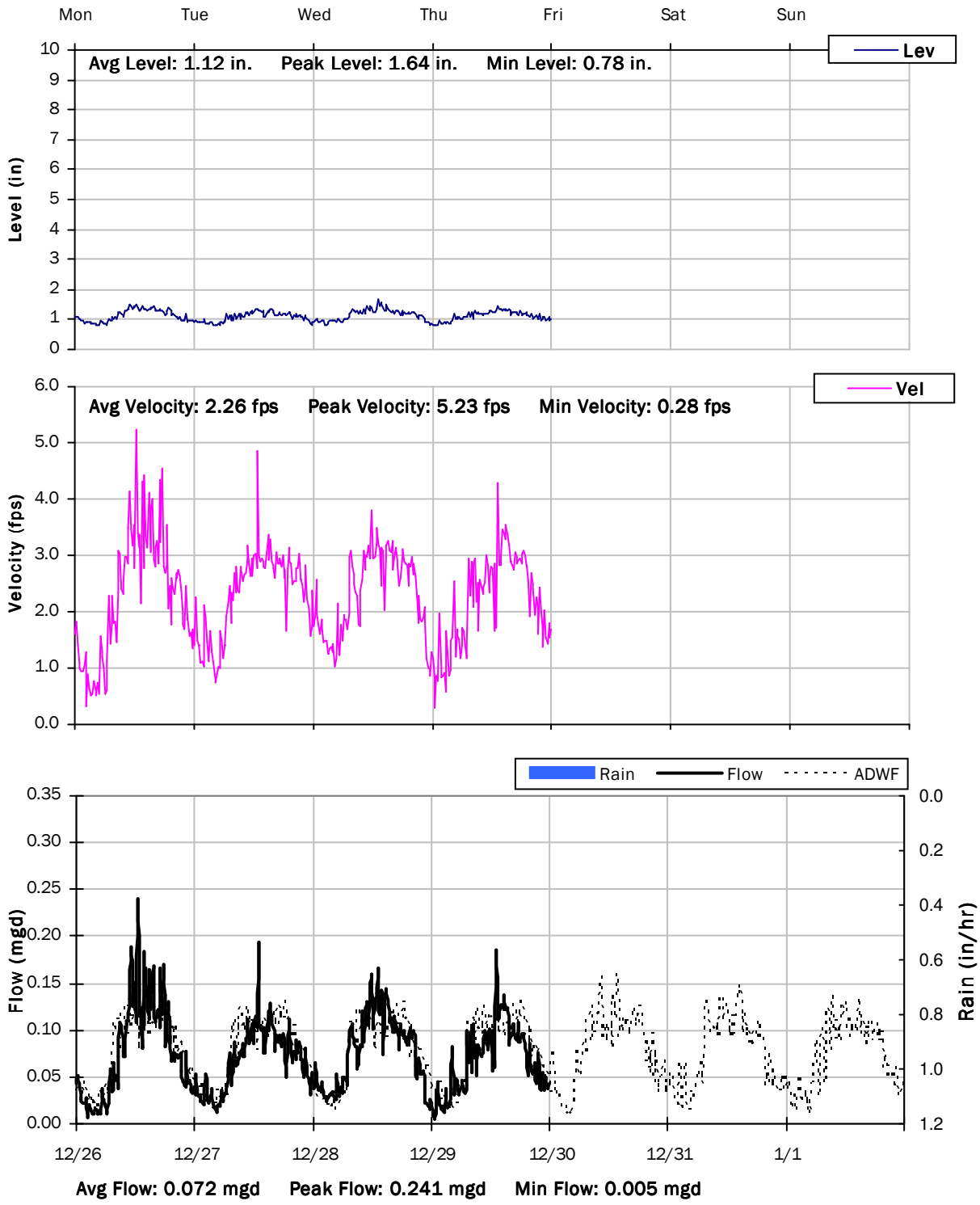
12/19/2016 to 12/26/2016



SITE 4

Weekly Level, Velocity and Flow Hydrographs

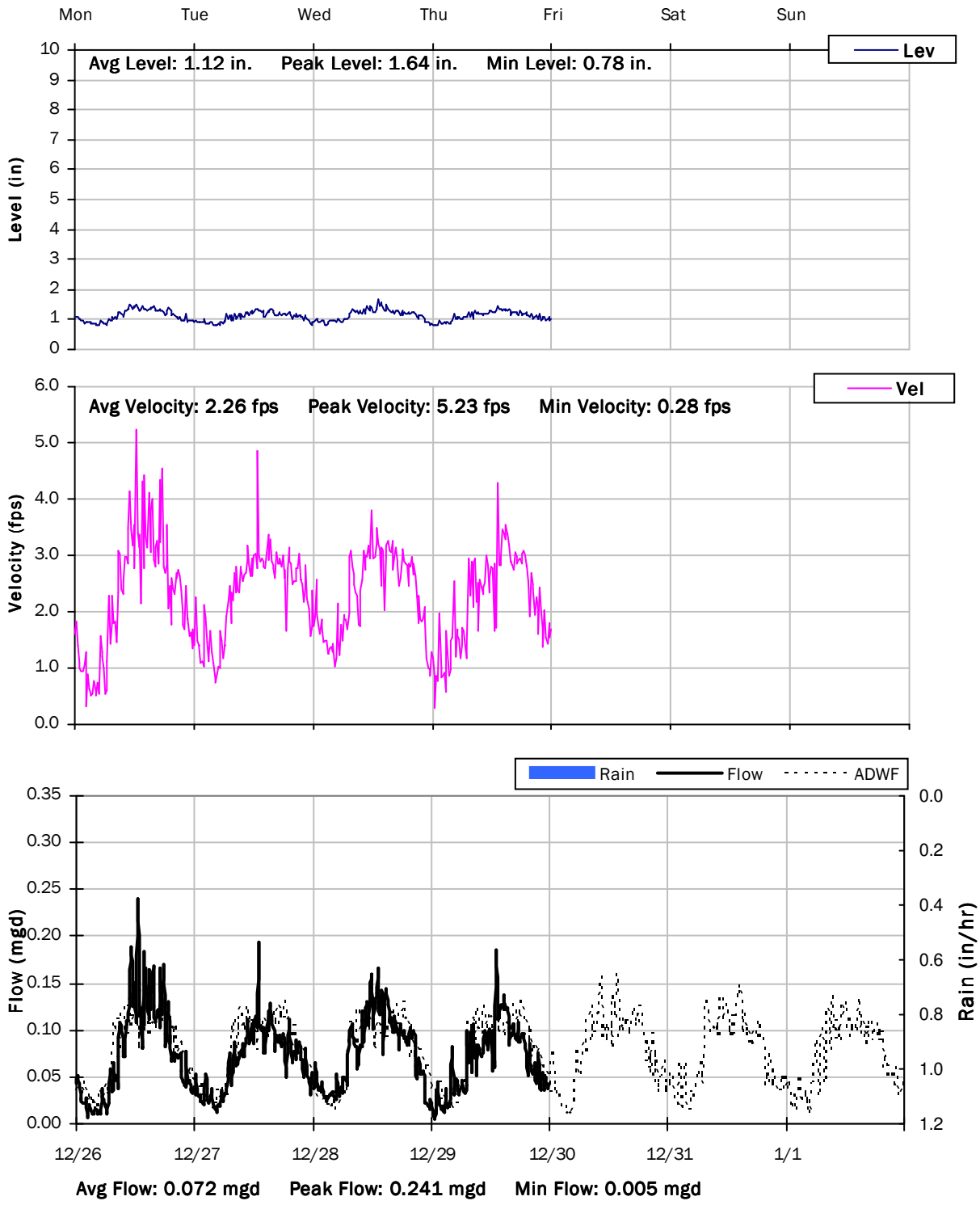
12/26/2016 to 1/2/2017



SITE 4

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

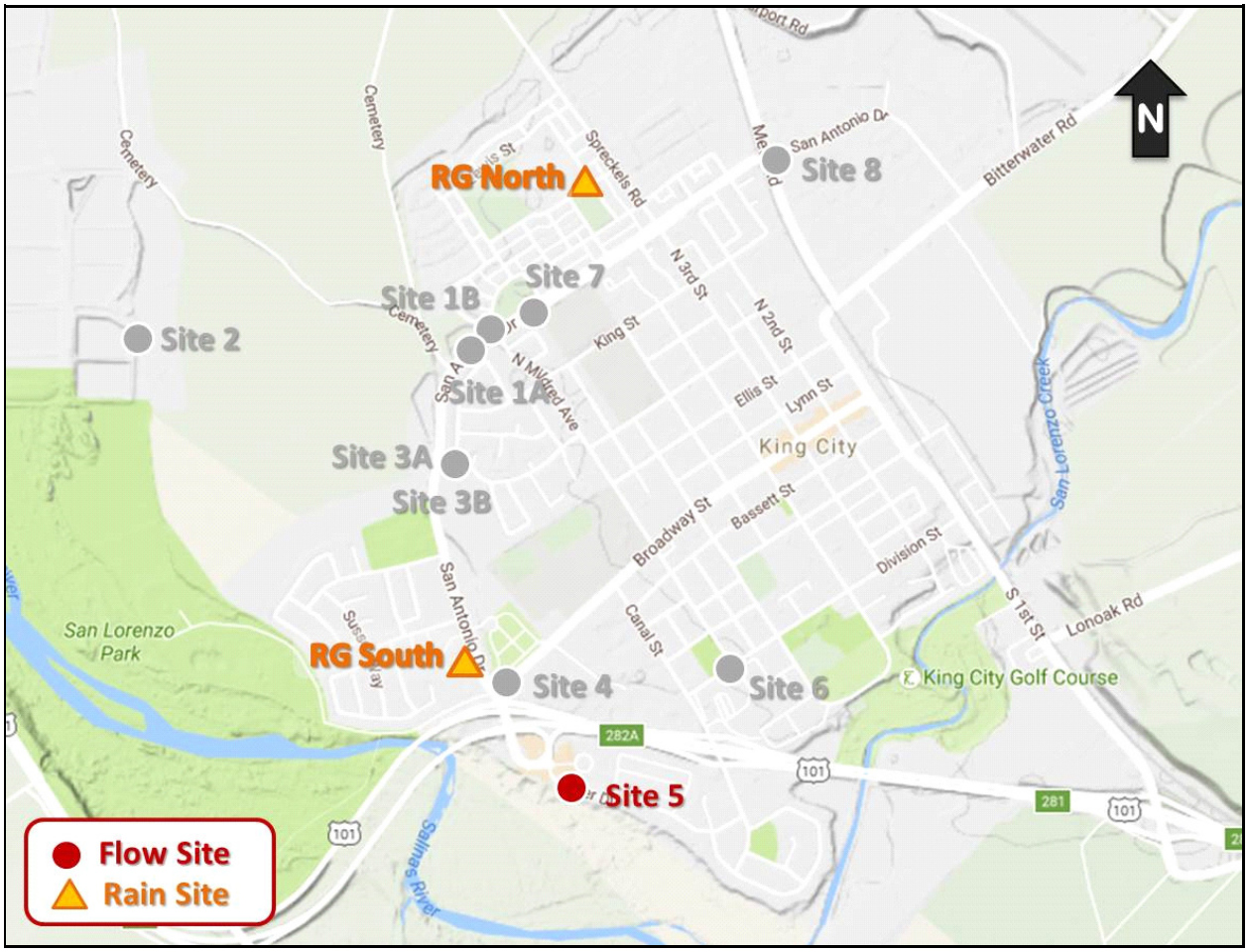
Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 5

Location: River Drive south of Broadway Street, west of Rio Plaza Mobile Home Estates

Data Summary Report



Vicinity Map: Site 5

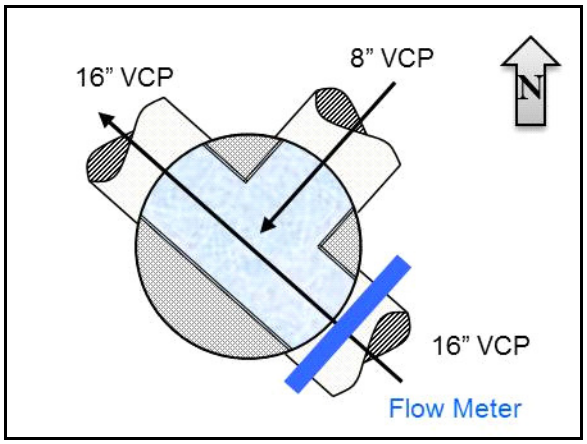
SITE 5

Site Information

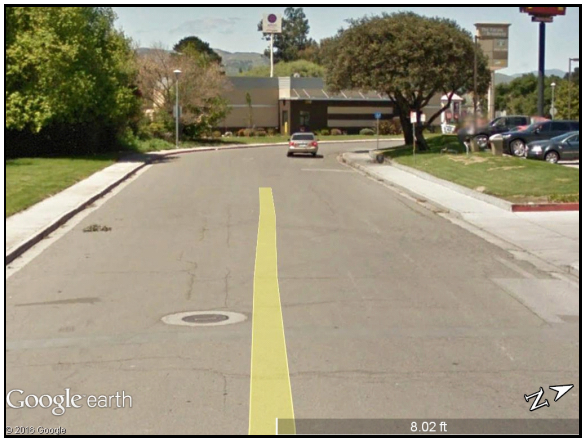
Location:	River Drive south of Broadway Street, west of Rio Plaza Mobile Home Estates
Coordinates:	121.1350° W, 36.2021° N
Rim Elevation (Earth):	305 feet
Pipe Diameter:	16 inches
ADWF:	0.107 mgd
Peak Measured Flow:	0.269 mgd

Satellite Map

Sewer Map



Flow Sketch



Street View

Plan View

SITE 5

Additional Site Photos

Effluent Pipe

Influent Pipe

SITE 5

Additional Site Photos

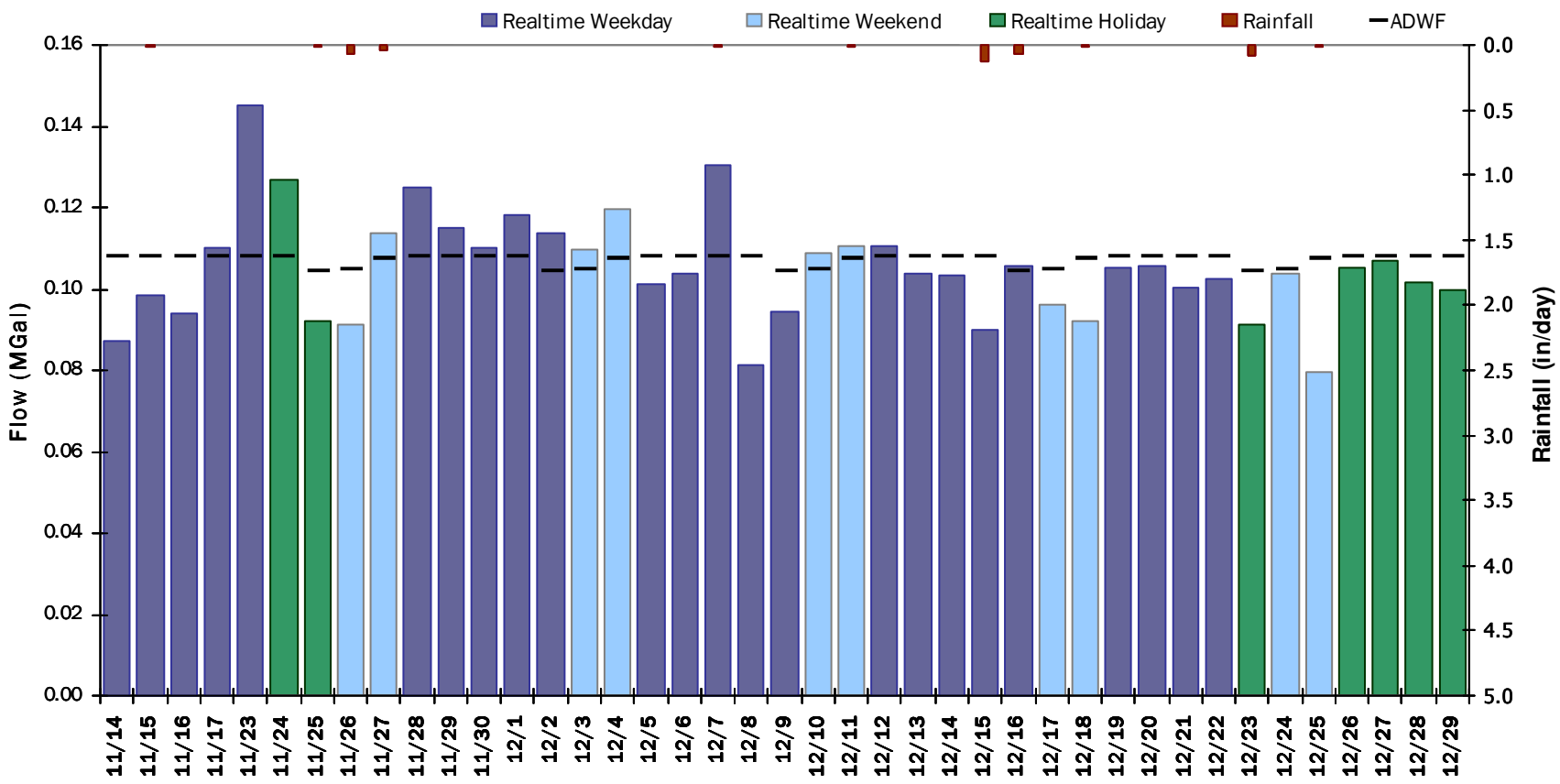
Lateral Pipe

SITE 5

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.105 MGal **Peak Daily Flow: 0.145 MGal** **Min Daily Flow: 0.079 MGal**

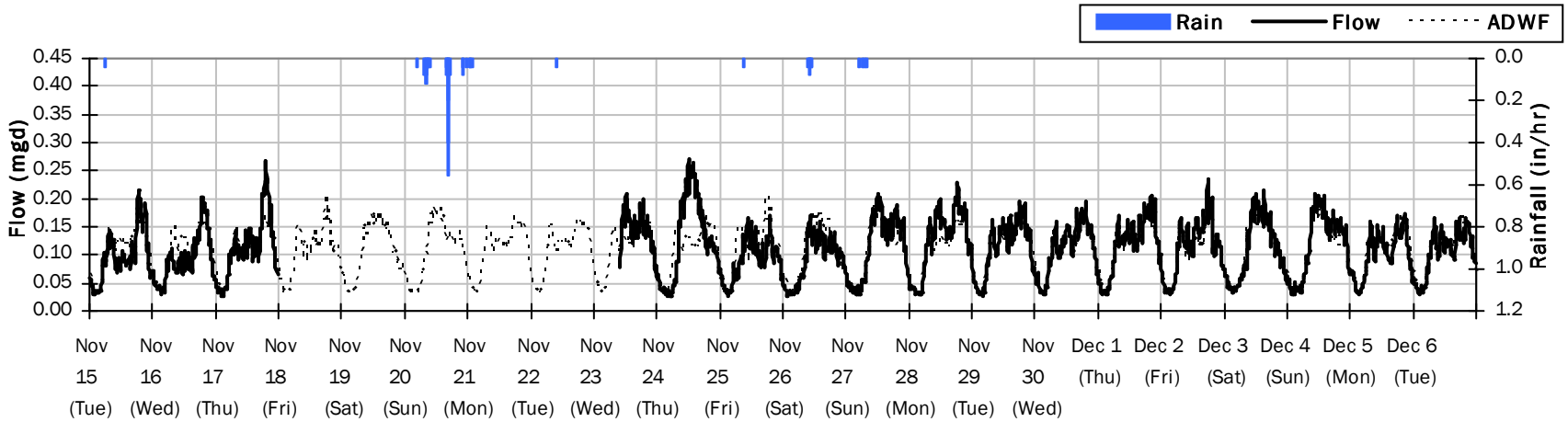
Total Period Rainfall: 0.48 inches

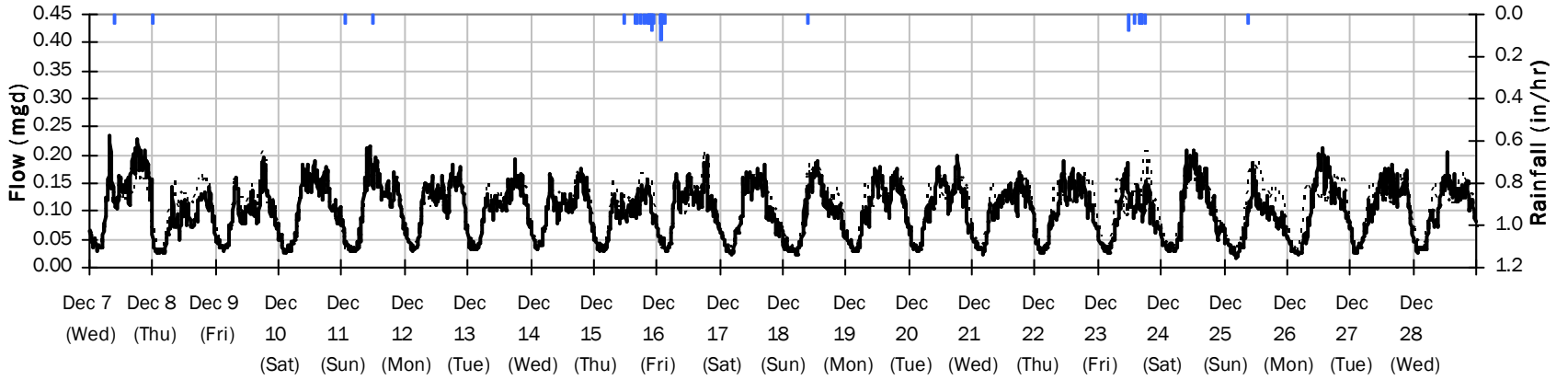


SITE 5

Flow Summary: 11/15/2016 to 12/28/2016

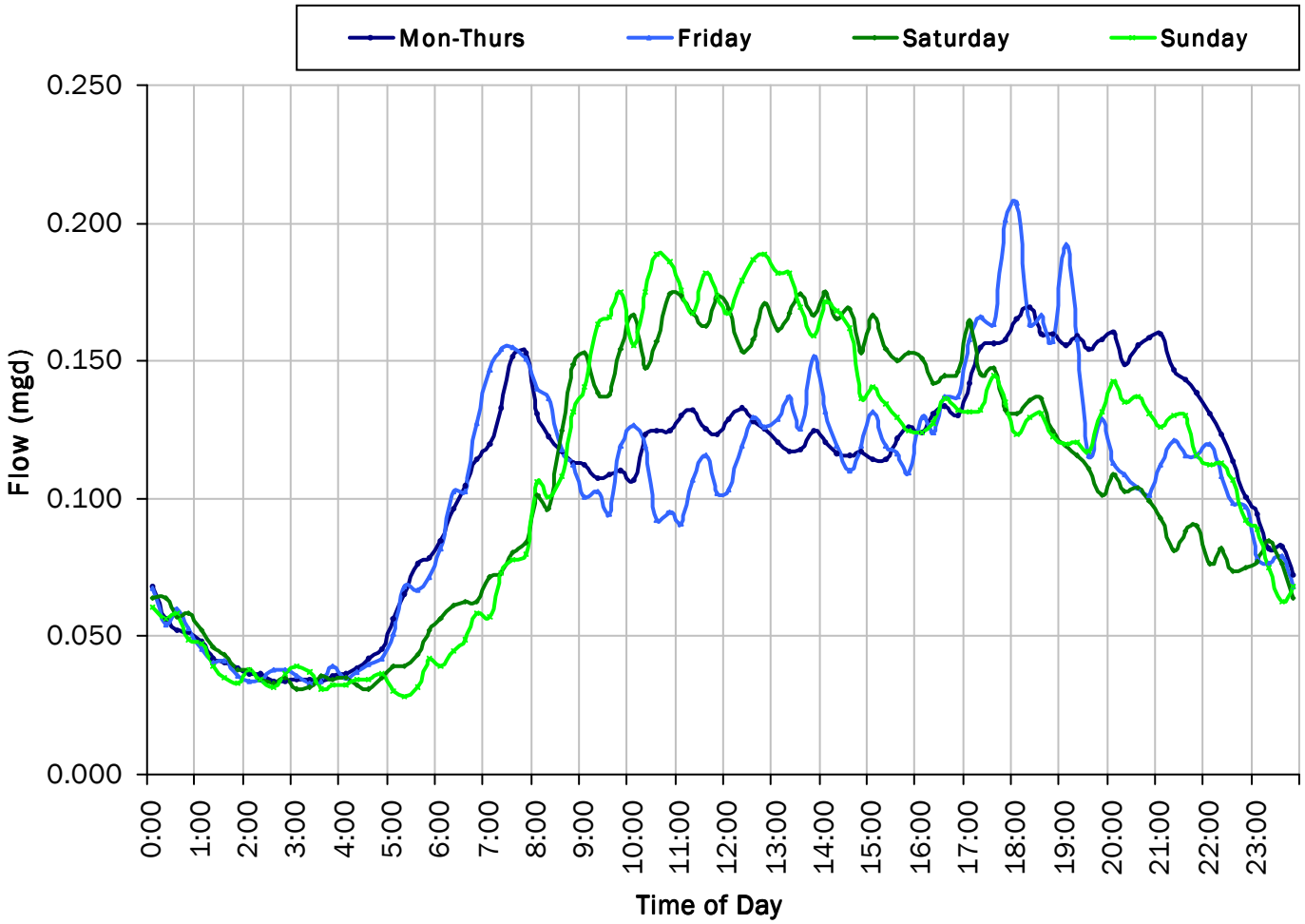
Total Period Rainfall: 1.01 inches **Avg Flow: 0.105 mgd** **Peak Flow: 0.269 mgd** **Min Flow: 0.015 mgd**





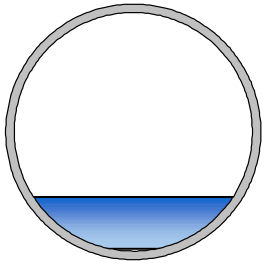
SITE 5

Average Dry Weather Flow Hydrographs



ADWF:

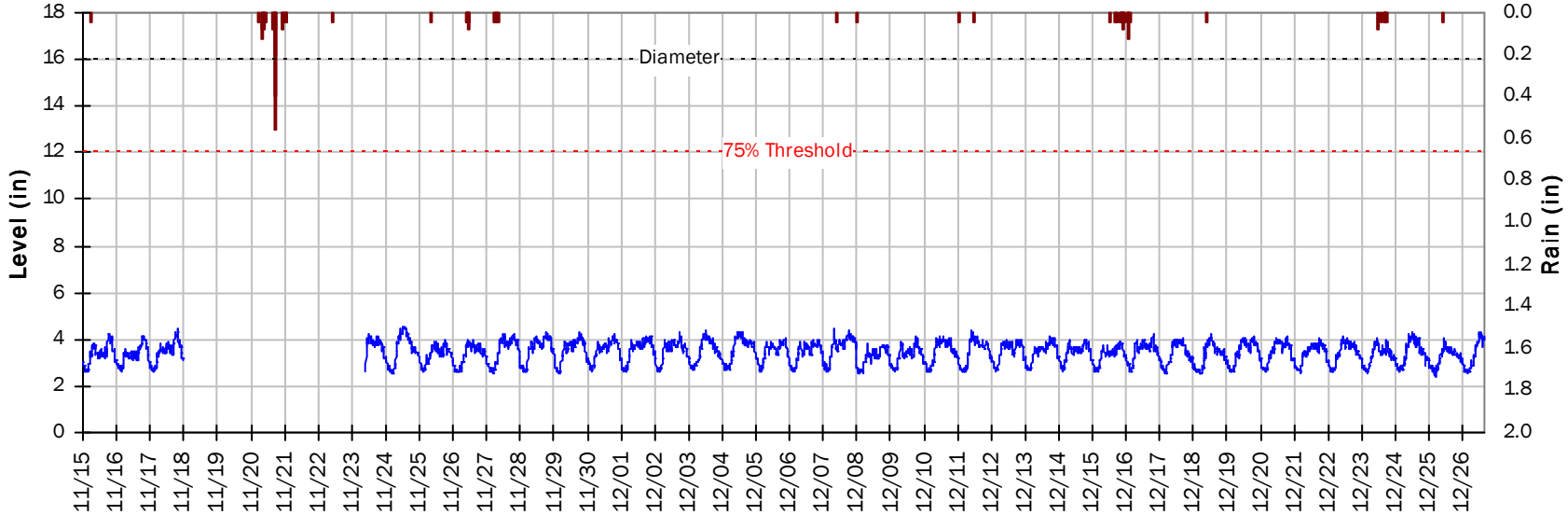
0.107 *mgd*

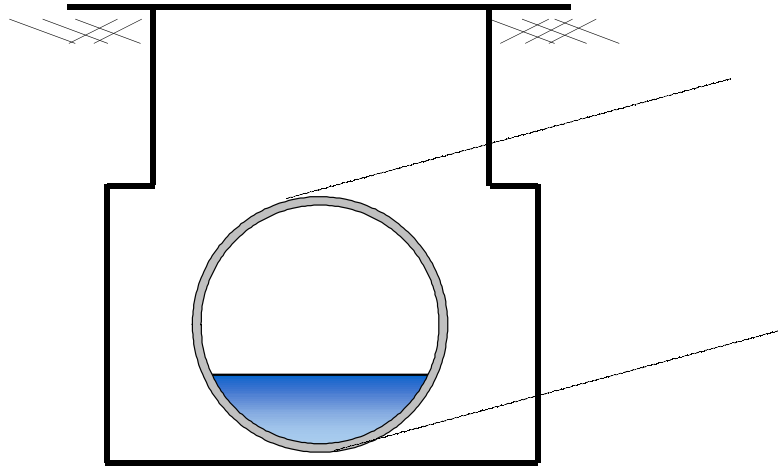


SITE 5

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



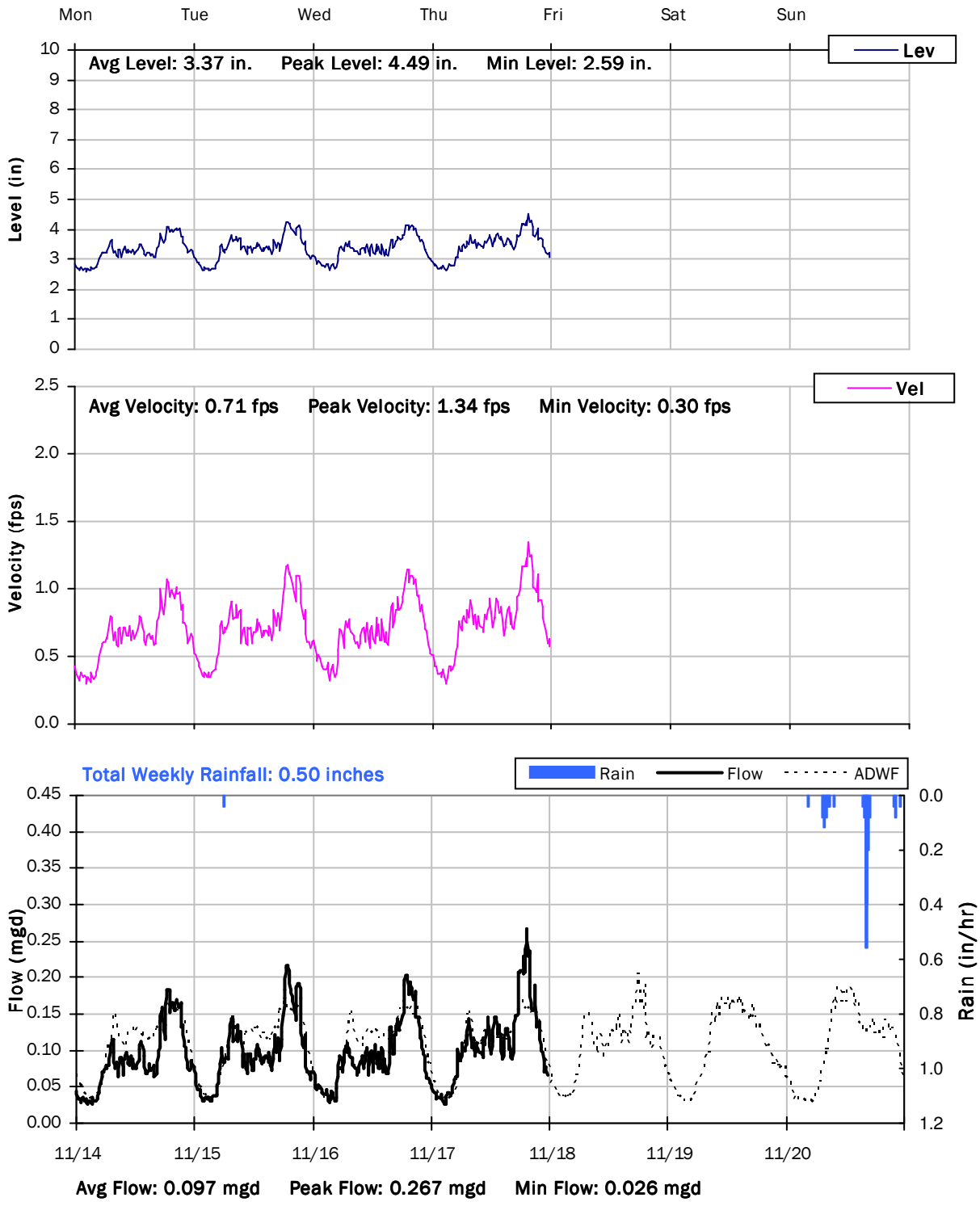


Pipe Diameter:	16	*inches*
Peak Measured Level:	4.57	*inches*
Peak d/D Ratio:	0.29	
Dry Weather Design Threshold Level:	12.0	*inches*

SITE 5

Weekly Level, Velocity and Flow Hydrographs

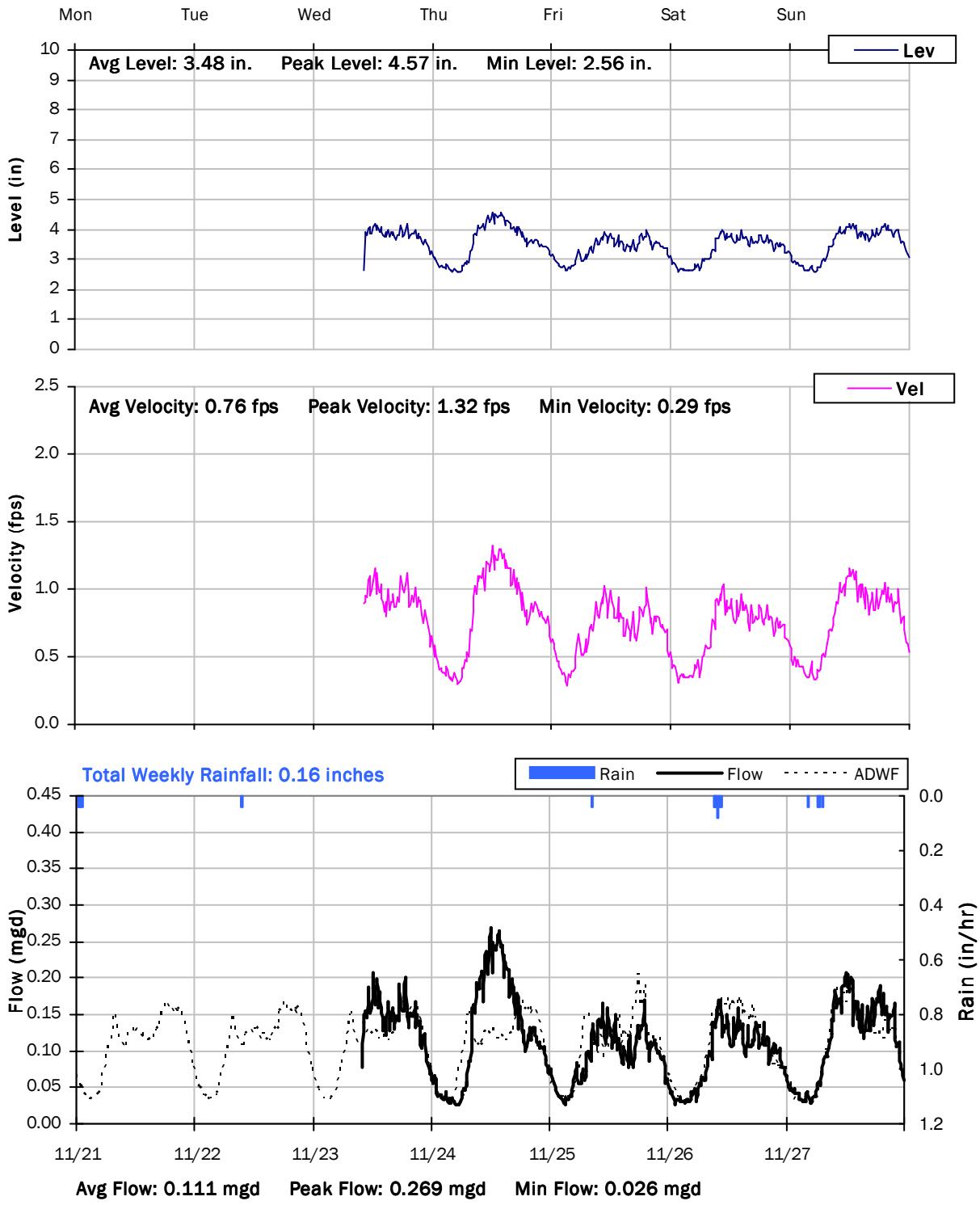
11/14/2016 to 11/21/2016



SITE 5

Weekly Level, Velocity and Flow Hydrographs

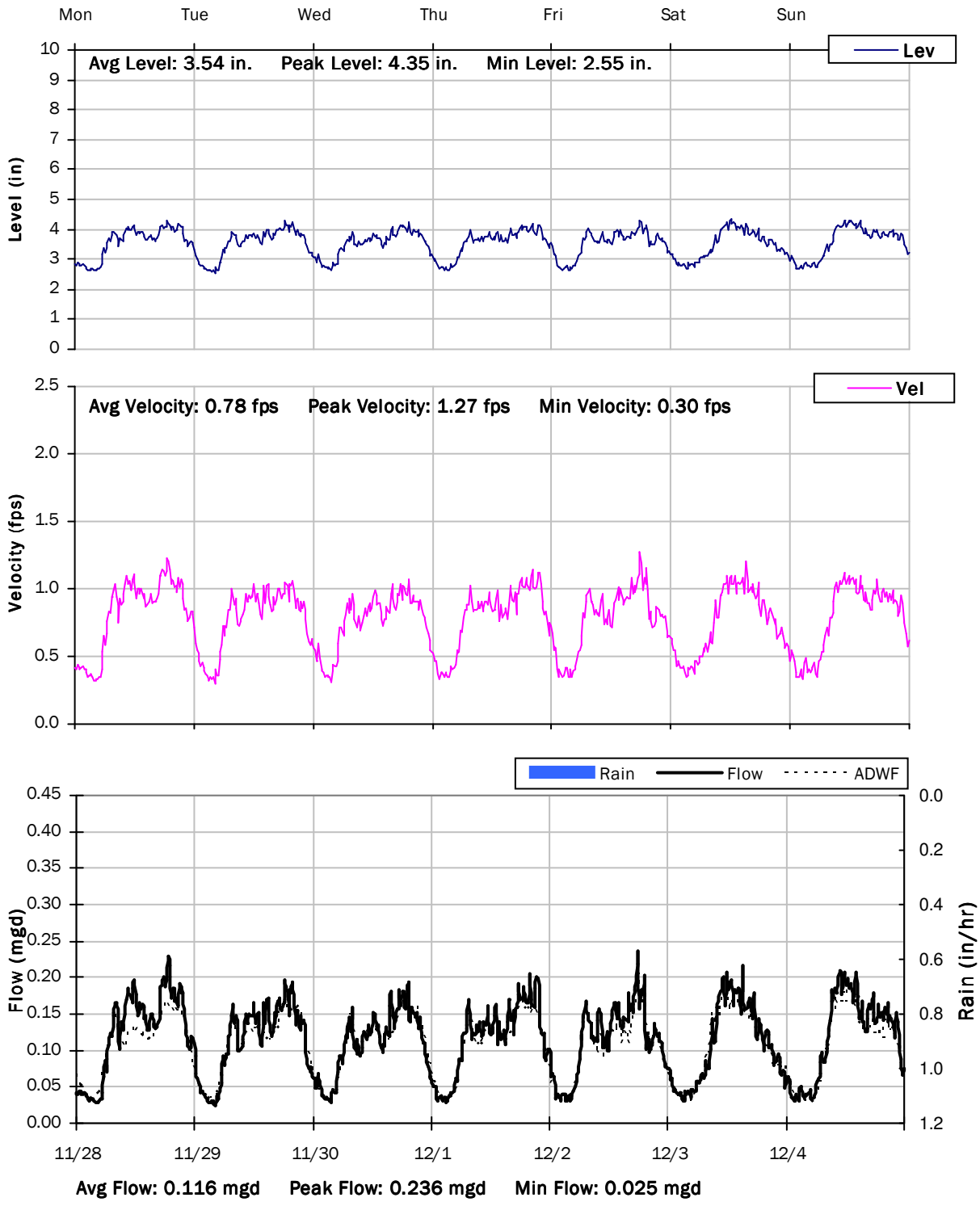
11/21/2016 to 11/28/2016



SITE 5

Weekly Level, Velocity and Flow Hydrographs

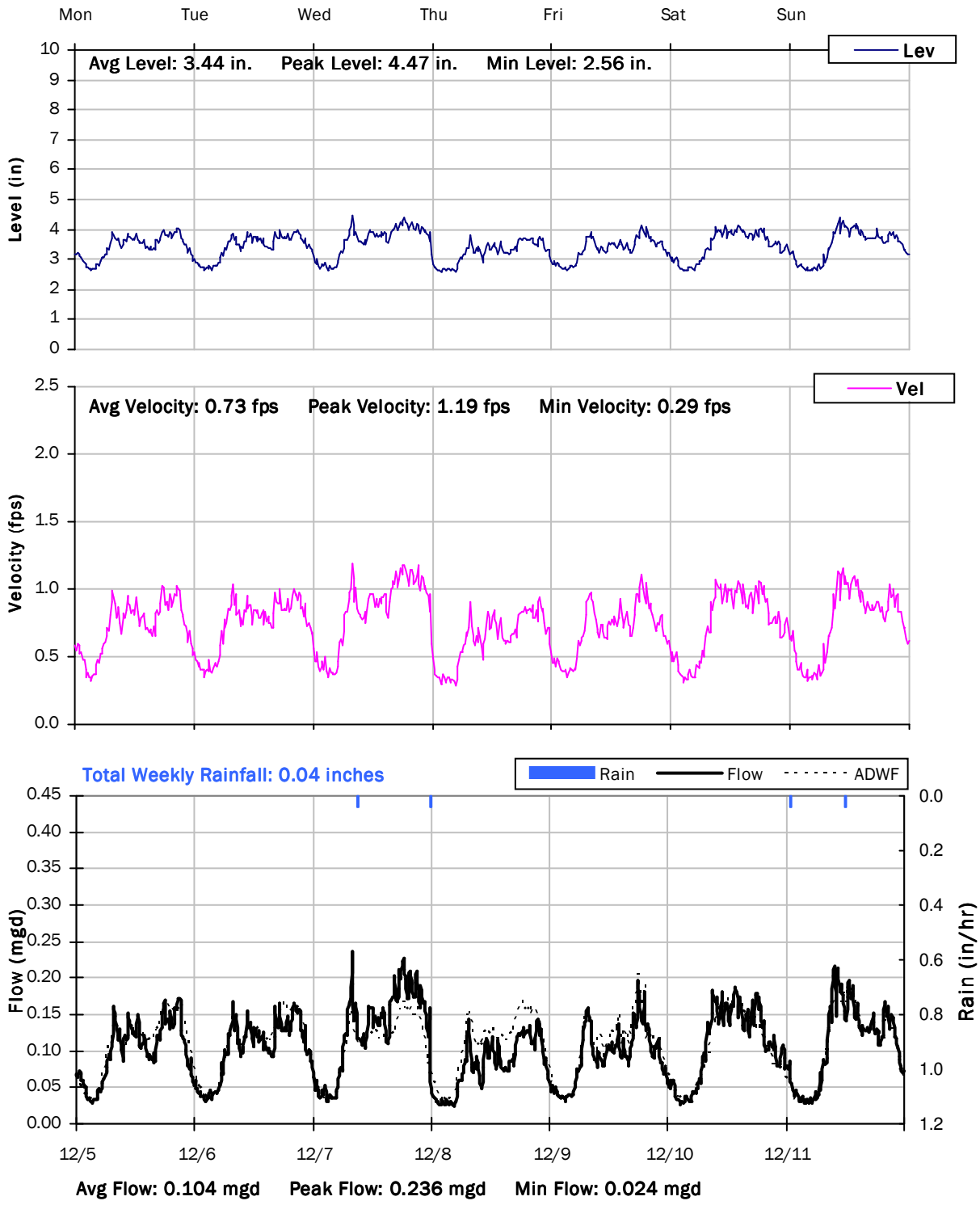
11/28/2016 to 12/5/2016



SITE 5

Weekly Level, Velocity and Flow Hydrographs

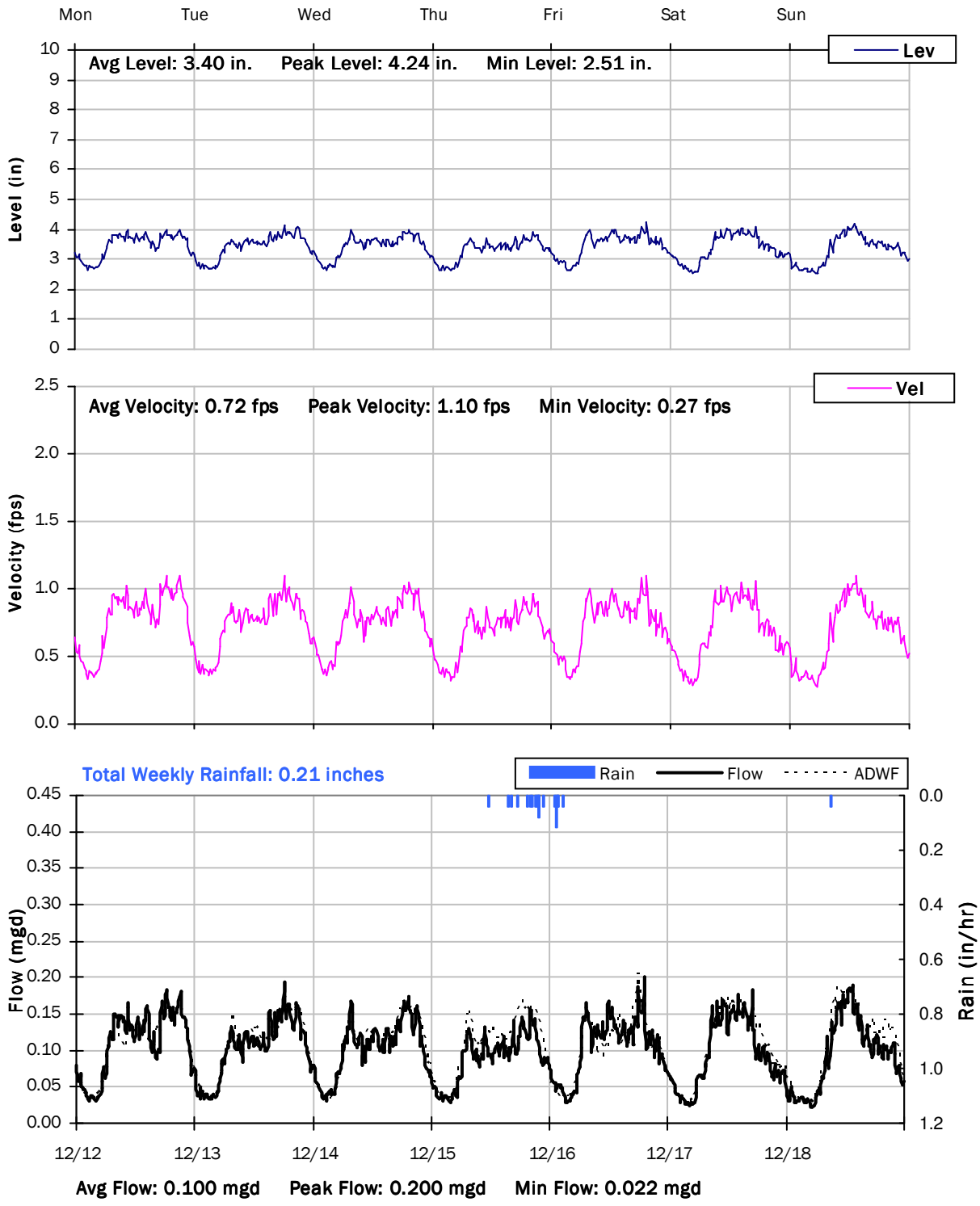
12/5/2016 to 12/12/2016



SITE 5

Weekly Level, Velocity and Flow Hydrographs

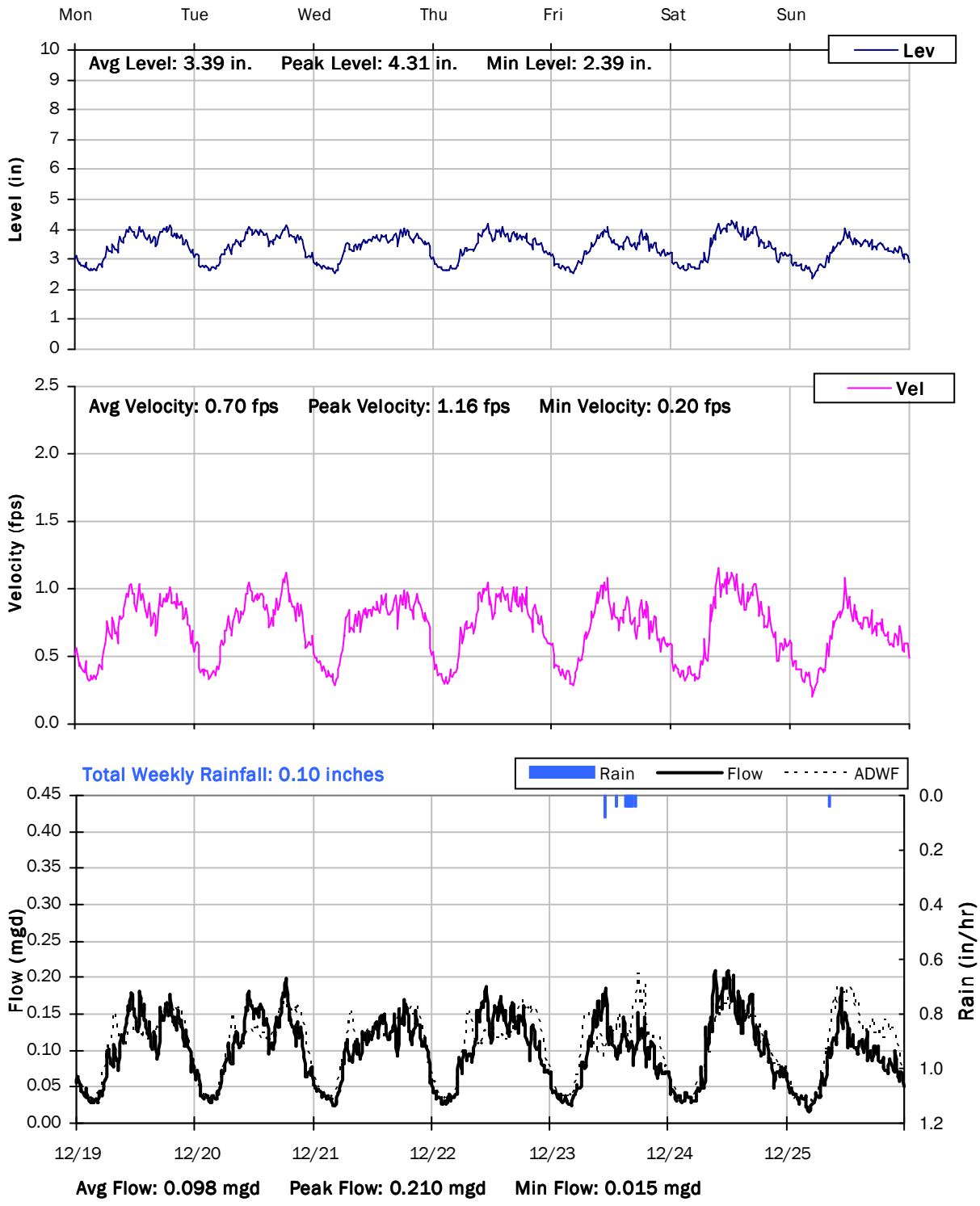
12/12/2016 to 12/19/2016



SITE 5

Weekly Level, Velocity and Flow Hydrographs

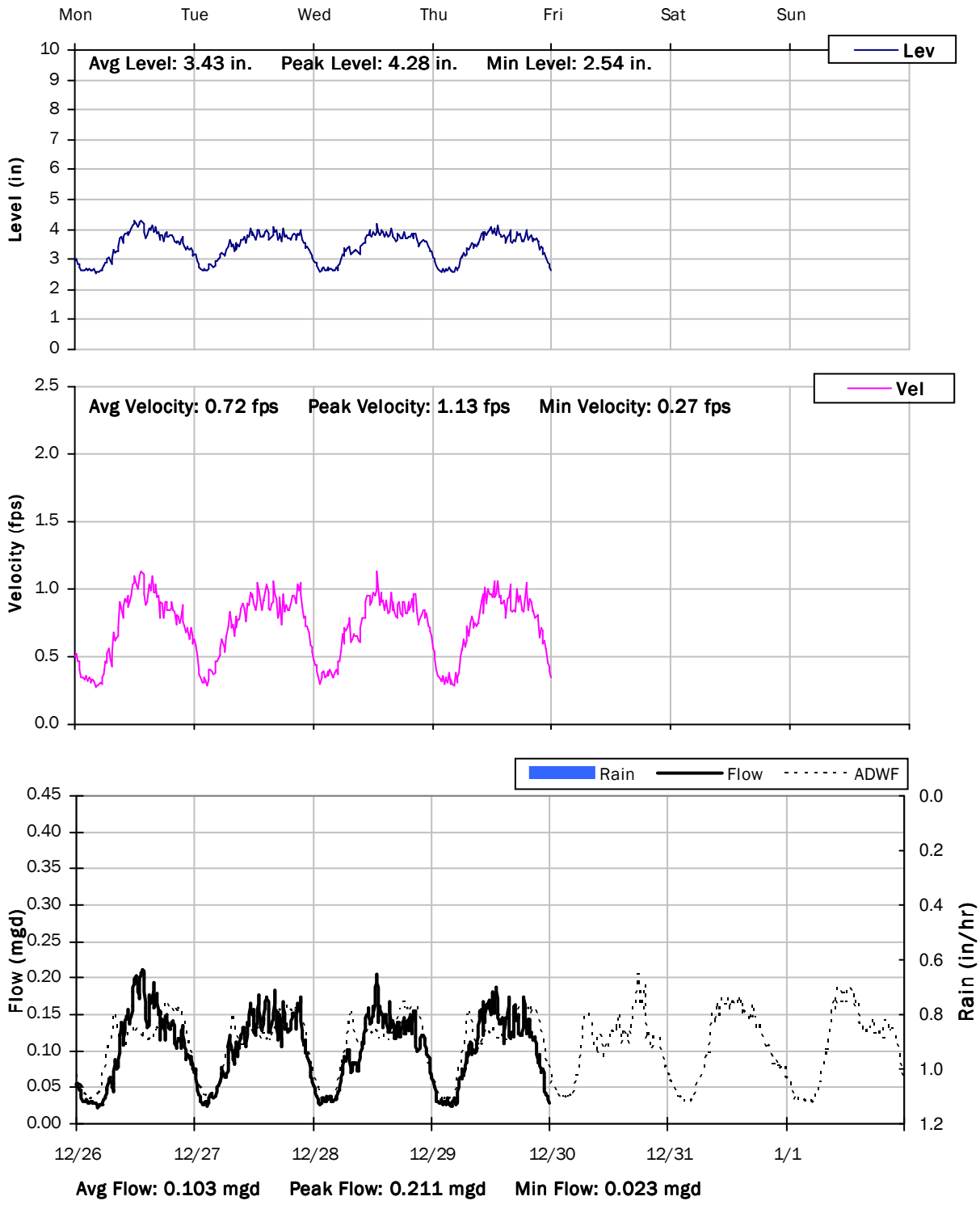
12/19/2016 to 12/26/2016



SITE 5

Weekly Level, Velocity and Flow Hydrographs

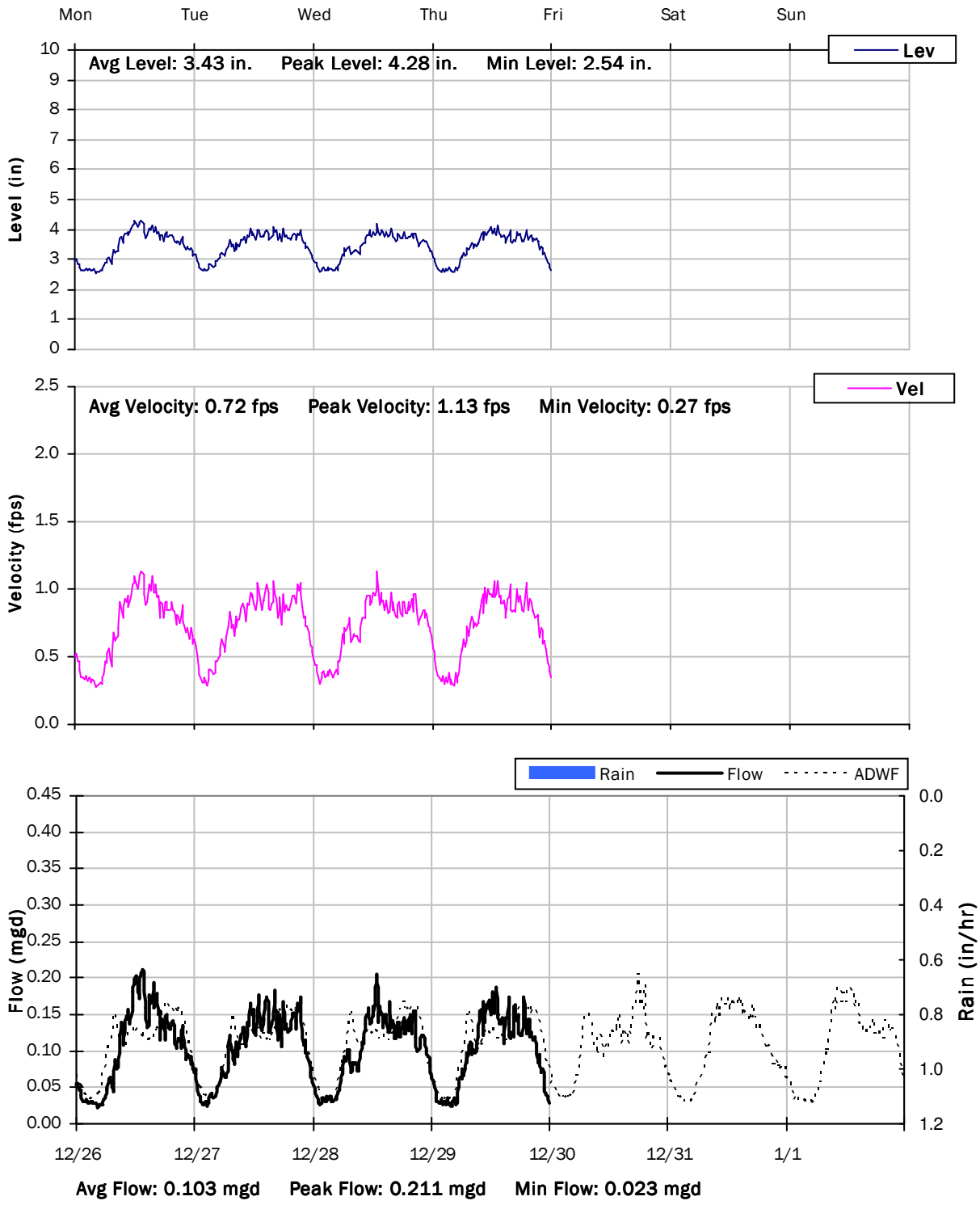
12/26/2016 to 1/2/2017



SITE 5

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 6

Location: Mildred Avenue and Division Street

Data Summary Report

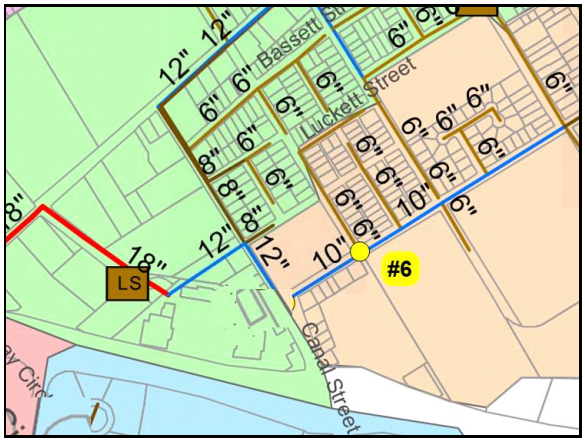
Vicinity Map: Site 6

SITE 6

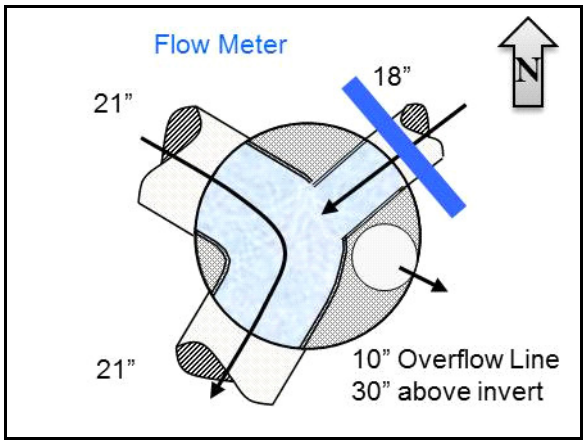
Site Information

Location:	Mildred Avenue and Division Street
Coordinates:	121.1289° W, 36.2059° N
Rim Elevation (Earth):	320 feet
Pipe Diameter:	18 inches
ADWF:	0.095 mgd
Peak Measured Flow:	0.233 mgd

Satellite Map



Sewer Map



Flow Sketch

Street View

Plan View

SITE 6

Additional Site Photos

Effluent Pipe

Influent Pipe

SITE 6

Additional Site Photos

Lateral Pipe

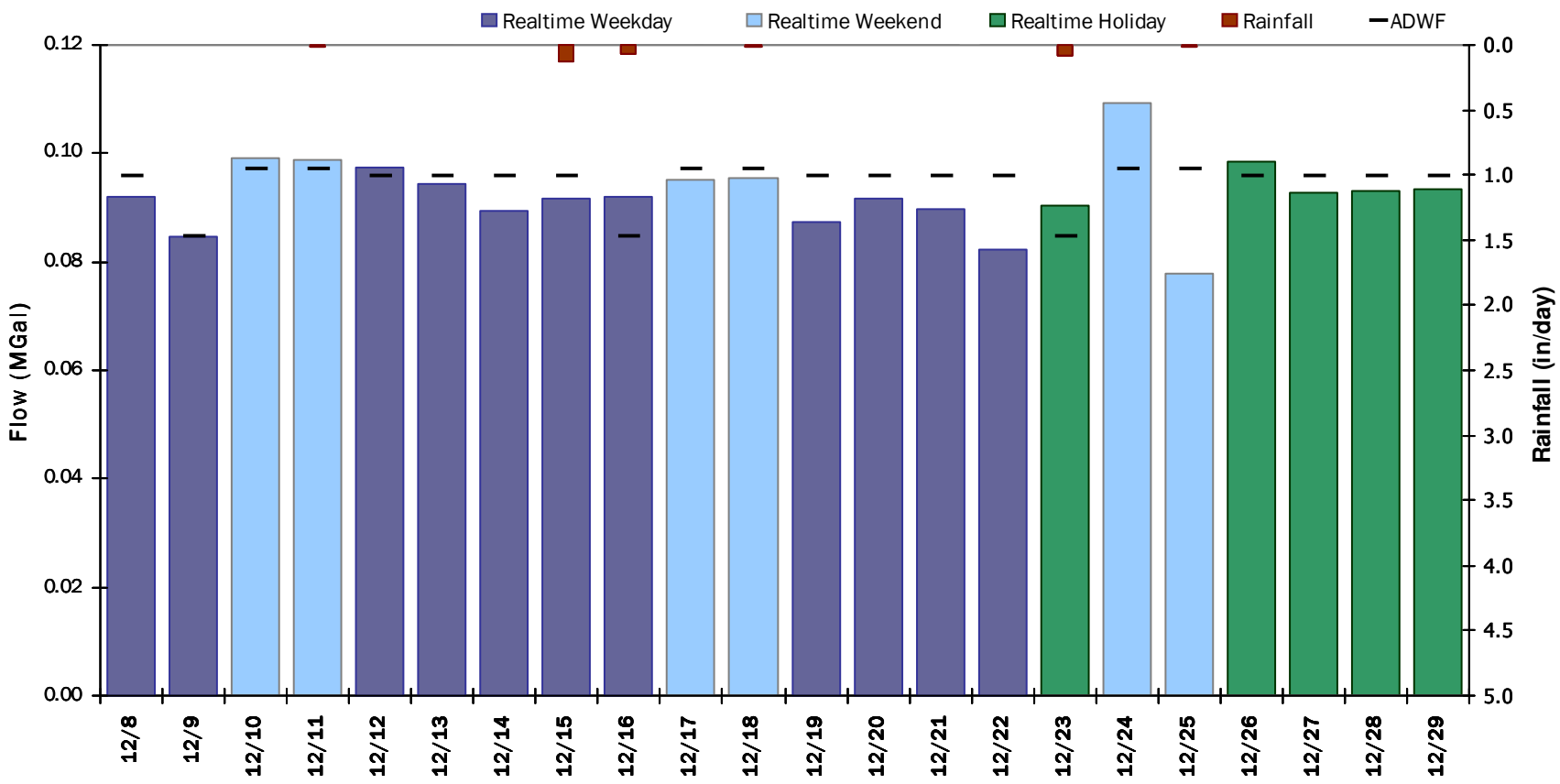
Overflow Pipe

SITE 6

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.093 MGal **Peak Daily Flow: 0.109 MGal** **Min Daily Flow: 0.078 MGal**

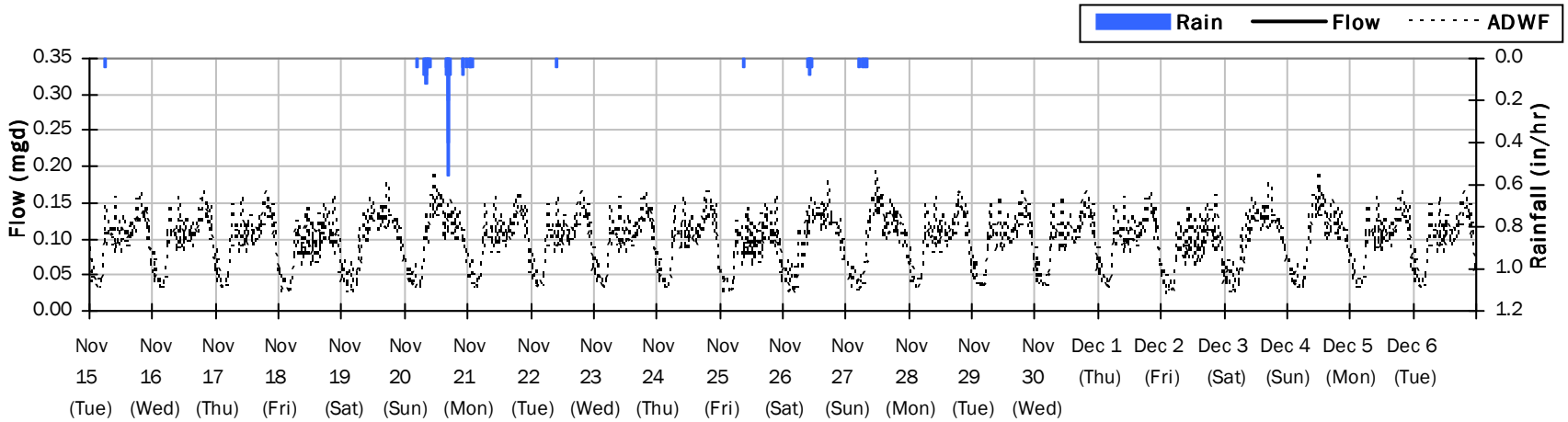
Total Period Rainfall: 0.33 inches

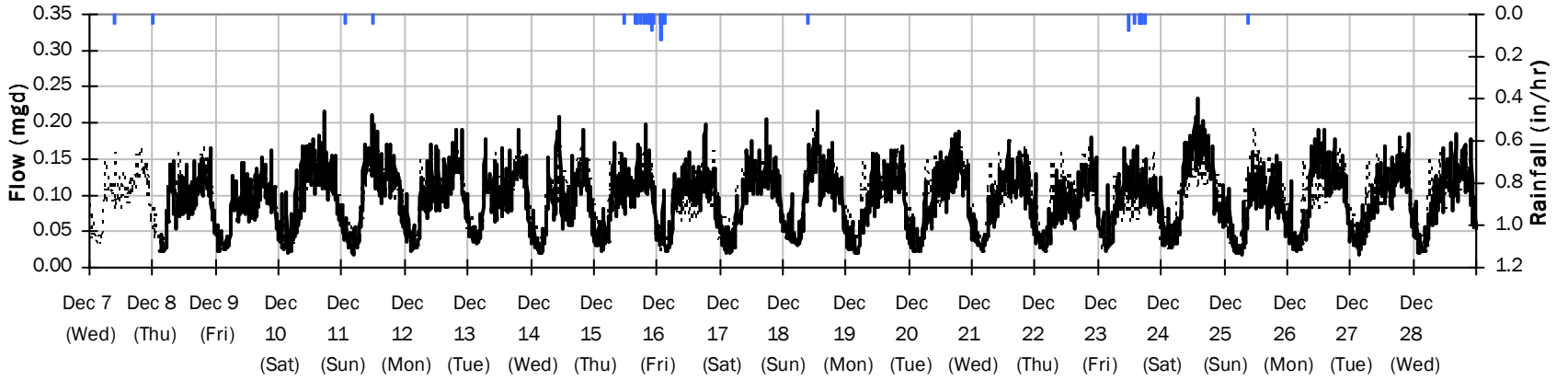


SITE 6

Flow Summary: 11/15/2016 to 12/28/2016

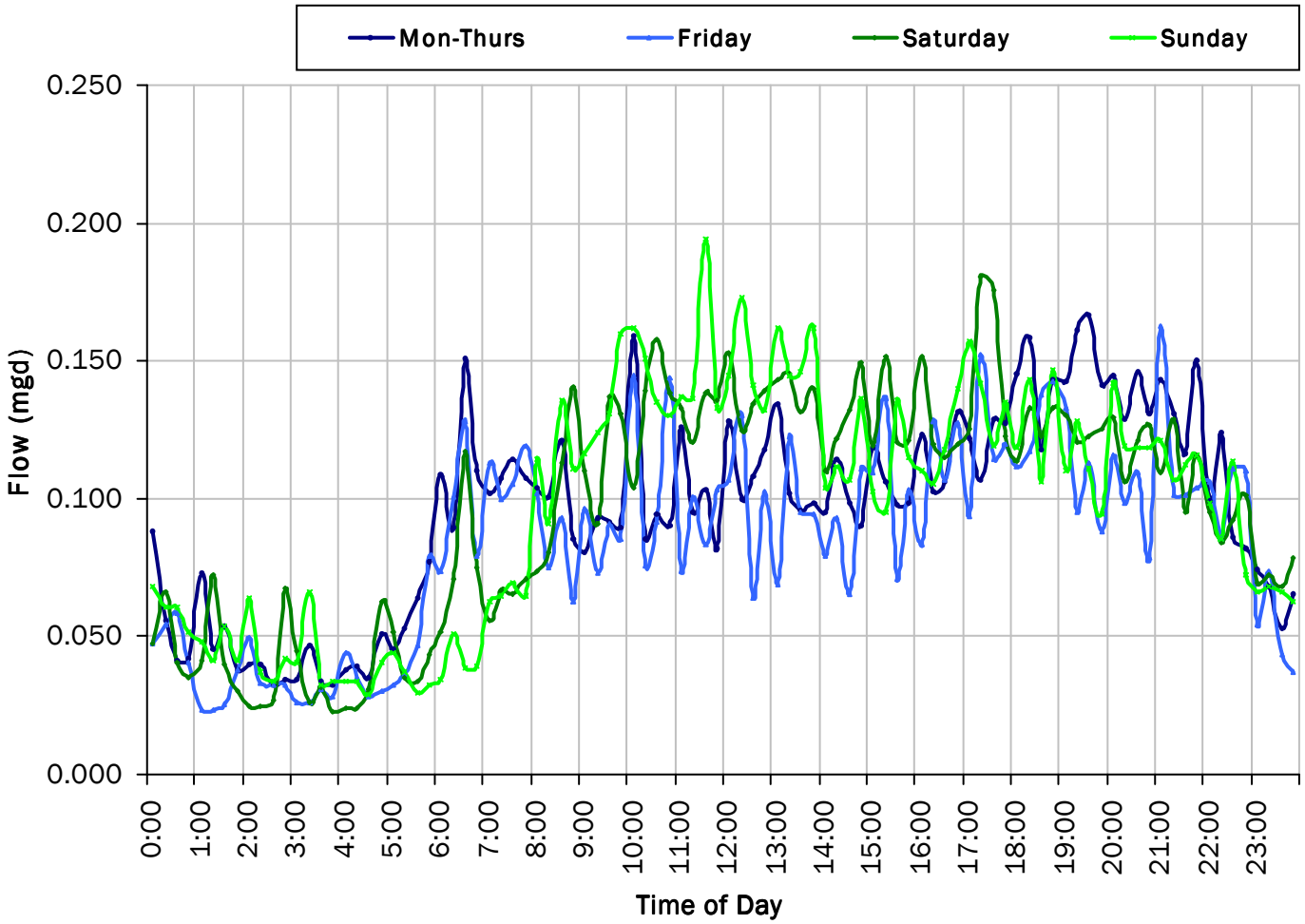
Total Period Rainfall: 1.01 inches **Avg Flow: 0.093 mgd** **Peak Flow: 0.233 mgd** **Min Flow: 0.018 mgd**





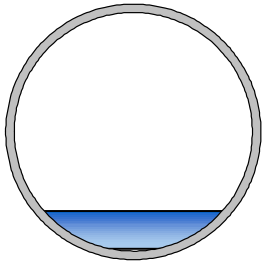
SITE 6

Average Dry Weather Flow Hydrographs



ADWF:

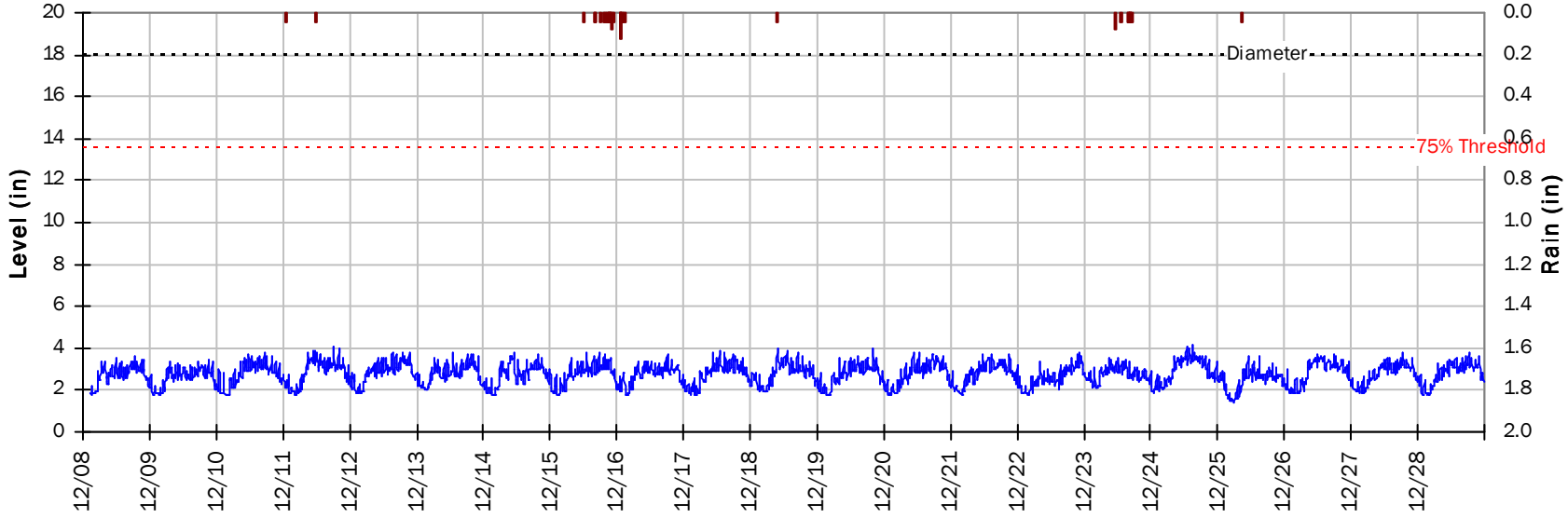
0.095 *mgd*

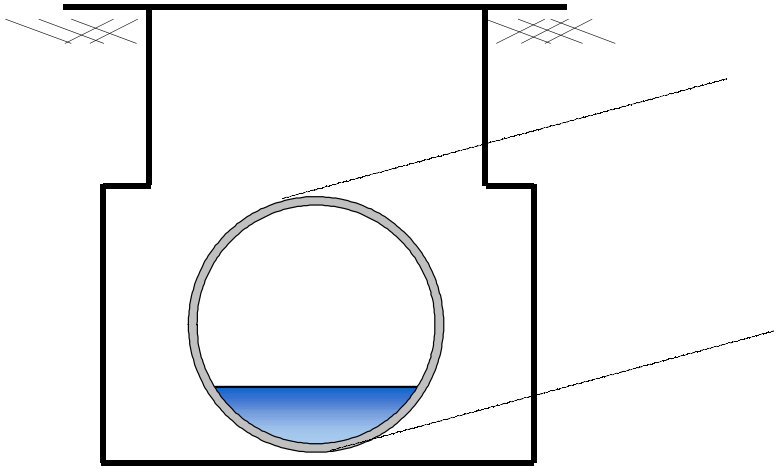


SITE 6

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



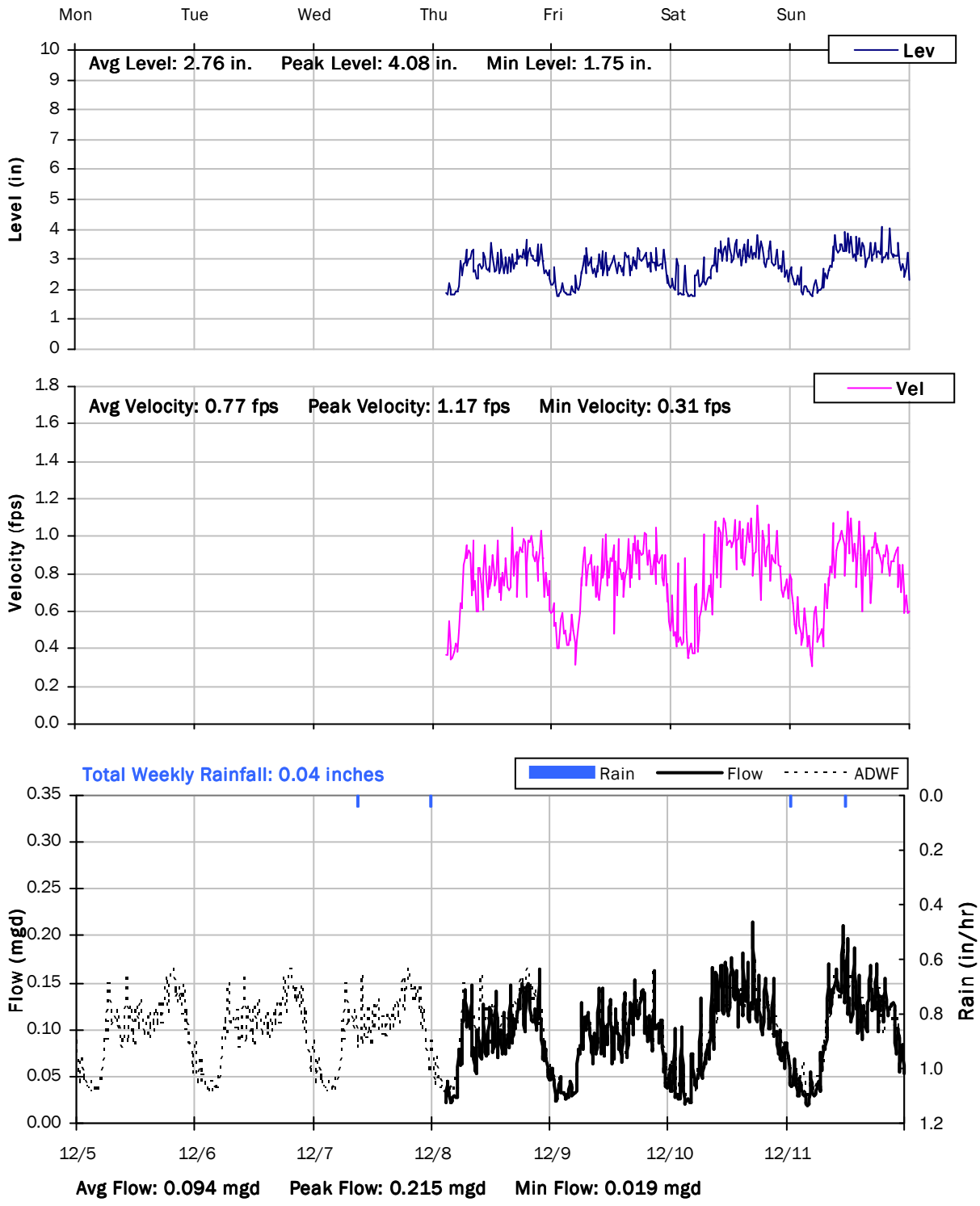


Pipe Diameter:	18	*inches*
Peak Measured Level:	4.24	*inches*
Peak d/D Ratio:	0.24	
Dry Weather Design Threshold Level:	13.5	*inches*

SITE 6

Weekly Level, Velocity and Flow Hydrographs

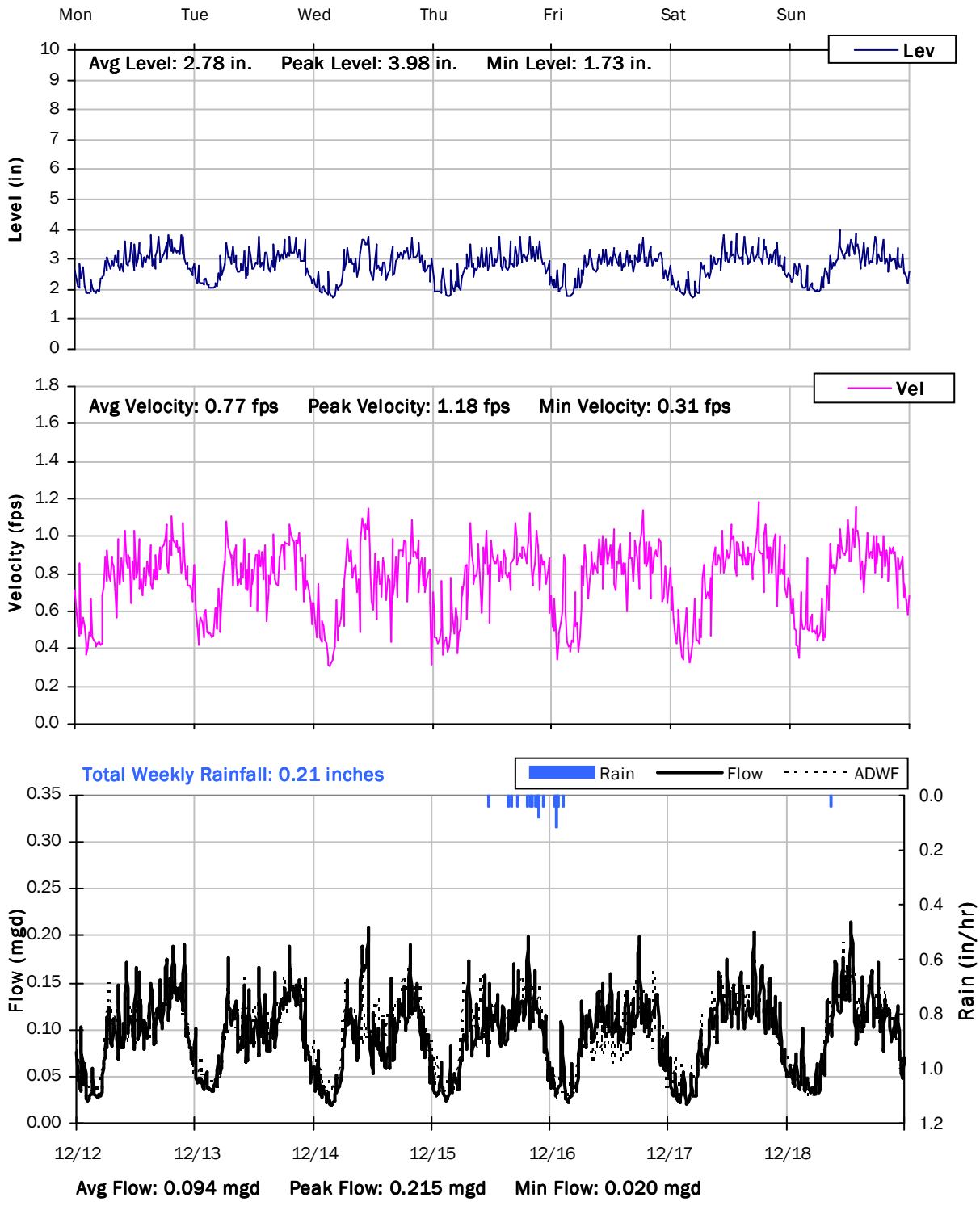
12/5/2016 to 12/12/2016



SITE 6

Weekly Level, Velocity and Flow Hydrographs

12/12/2016 to 12/19/2016



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Weekly Level, Velocity and Flow Hydrographs

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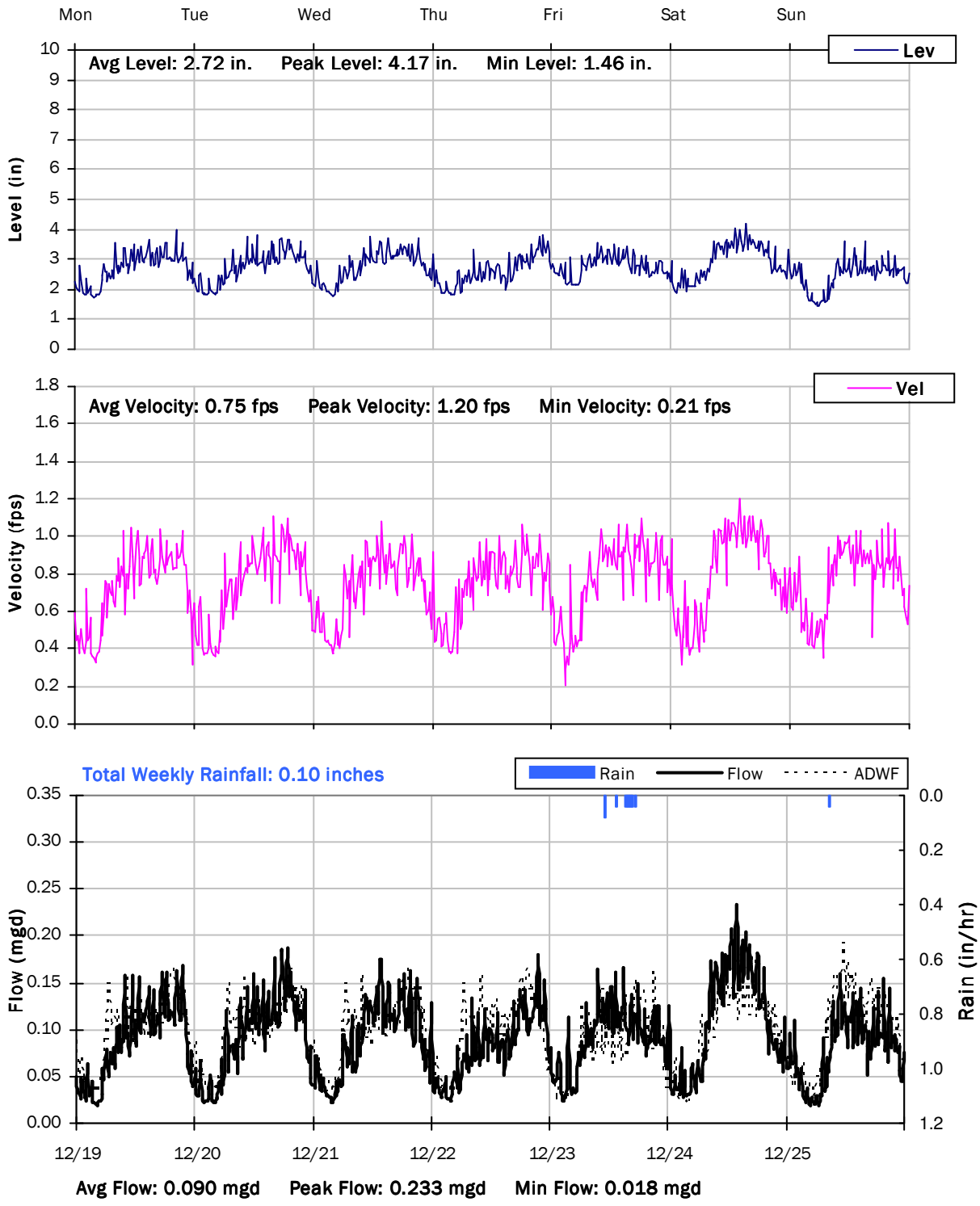
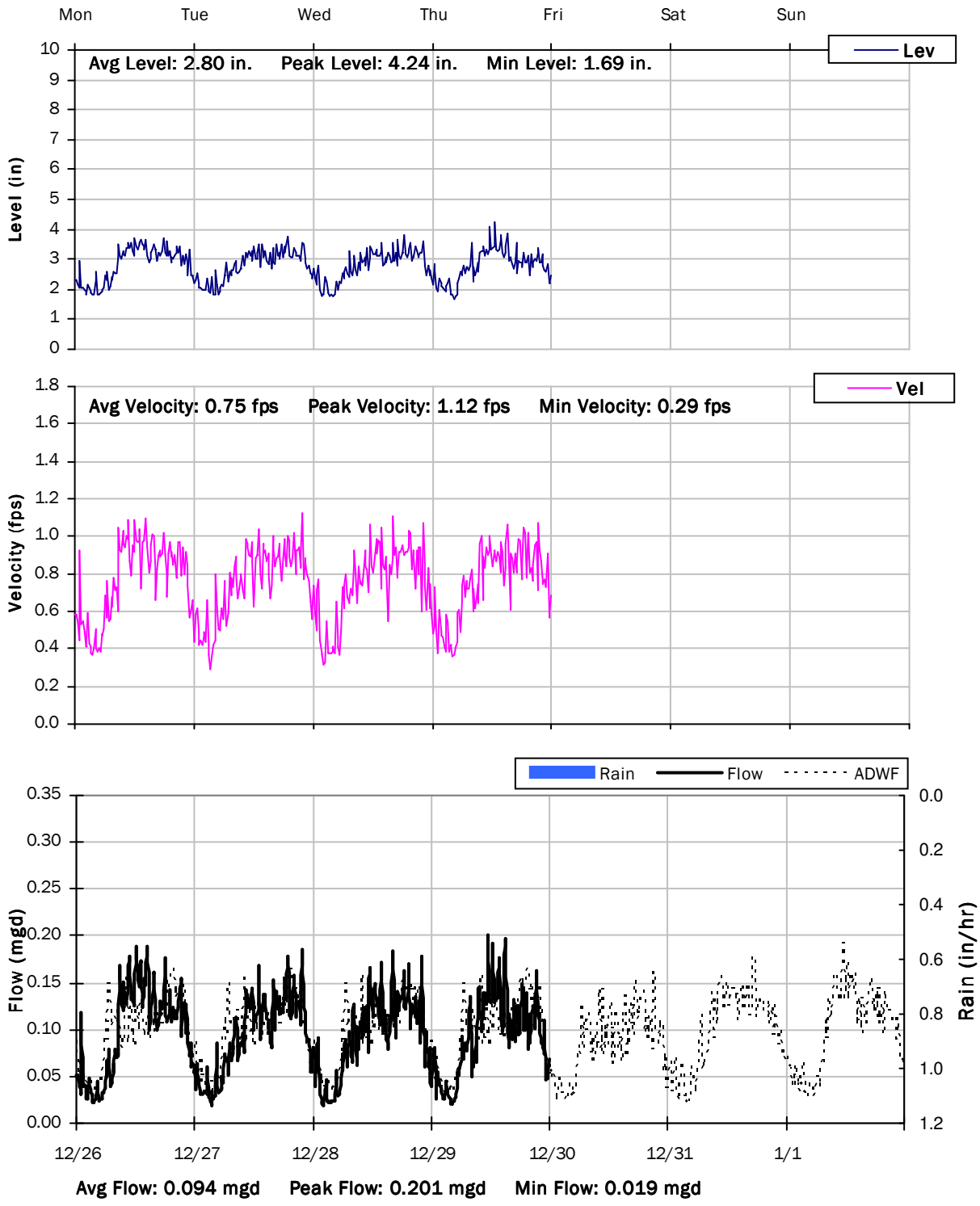


SITE 6

Weekly Level, Velocity and Flow Hydrographs

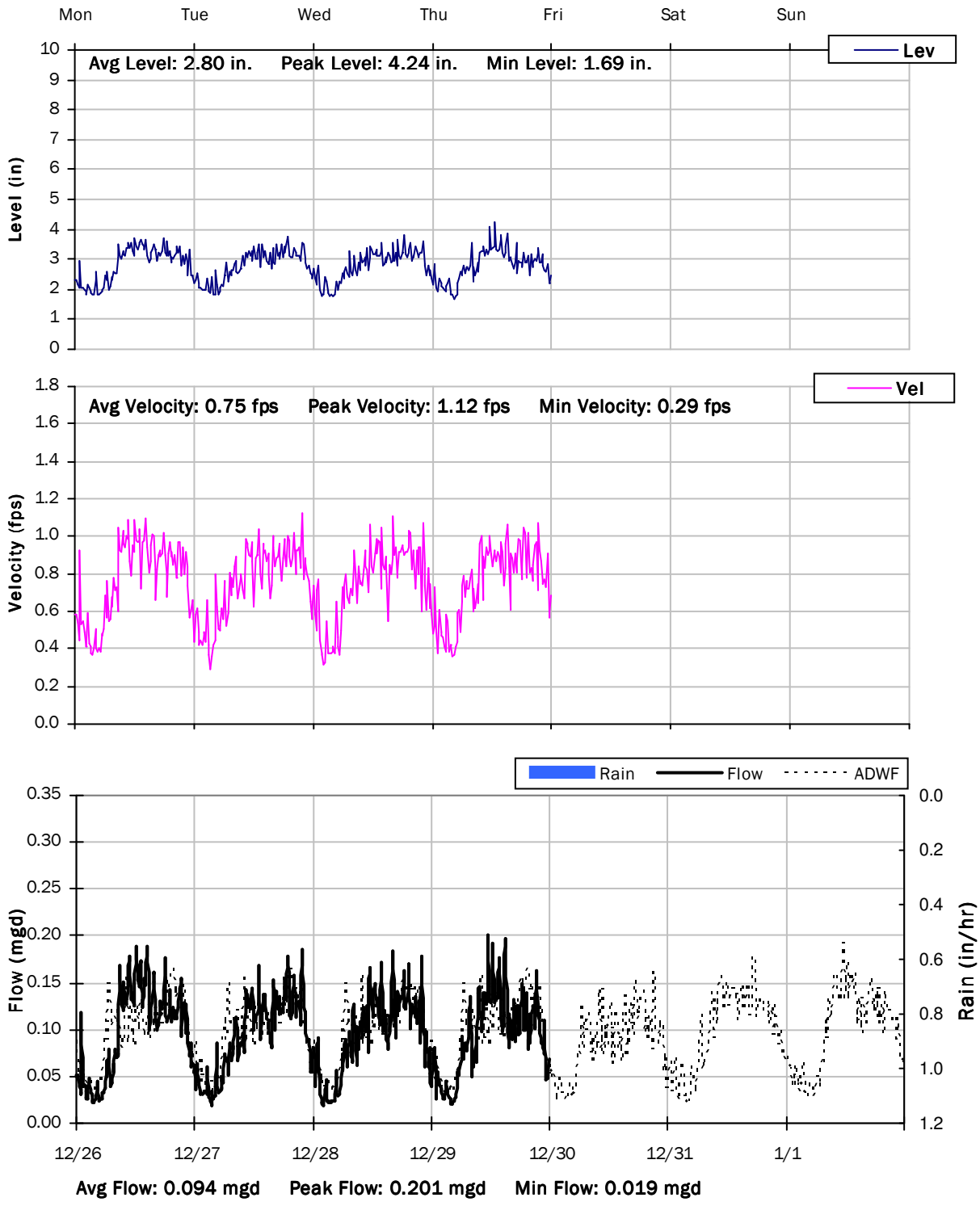
12/26/2016 to 1/2/2017



SITE 6

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 7

Location: San Antonio Drive west of Van Etten Avenue

Data Summary Report

Vicinity Map: Site 7

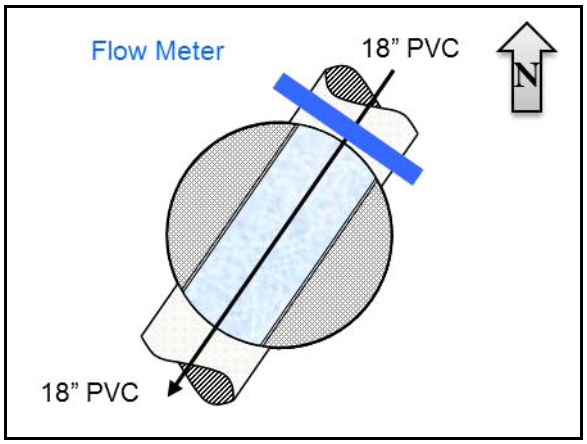
SITE 7

Site Information

Location:	San Antonio Drive west of Van Etten Avenue
Coordinates:	121.1365° W, 36.2171° N
Rim Elevation (Earth):	322 feet
Pipe Diameter:	18 inches
ADWF:	0.056 mgd
Peak Measured Flow:	0.465 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

Plan View

SITE 7

Additional Site Photos

Effluent Pipe

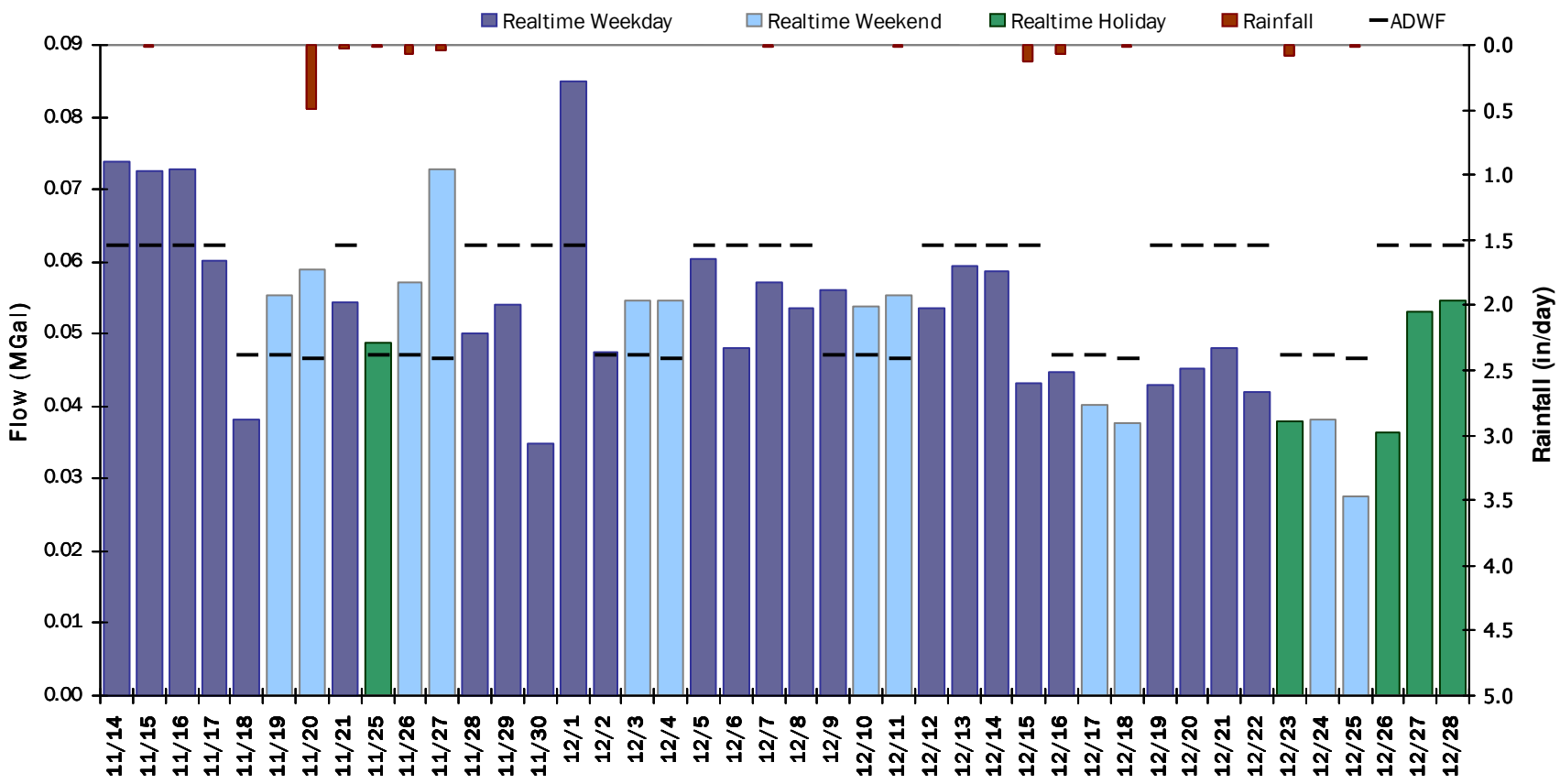
Influent Pipe

SITE 7

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.052 MGal **Peak Daily Flow: 0.085 MGal** **Min Daily Flow: 0.028 MGal**

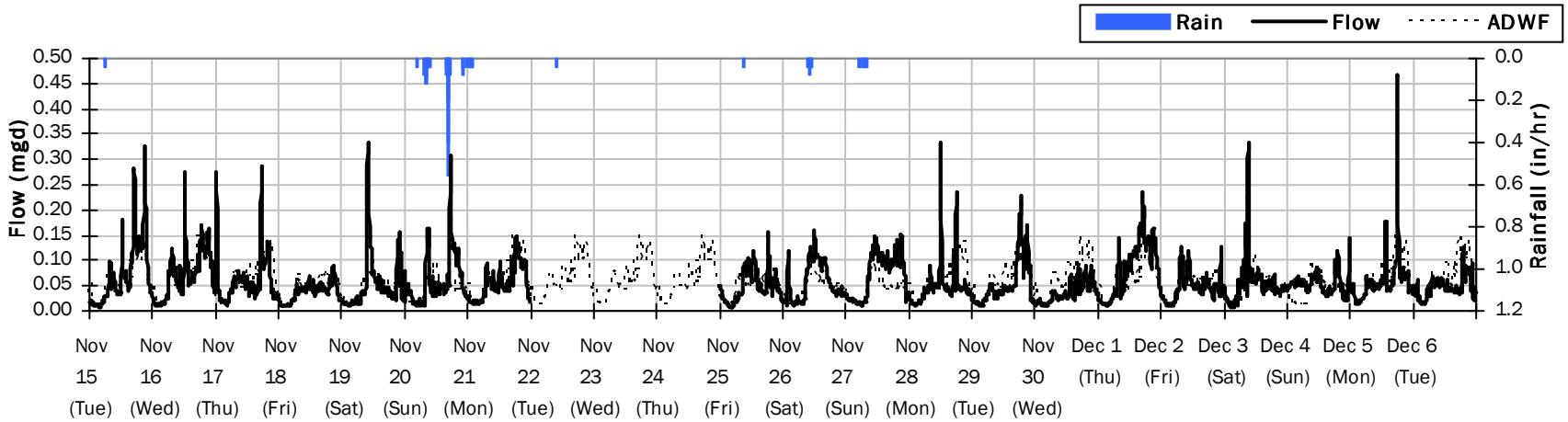
Total Period Rainfall: 1.00 inches

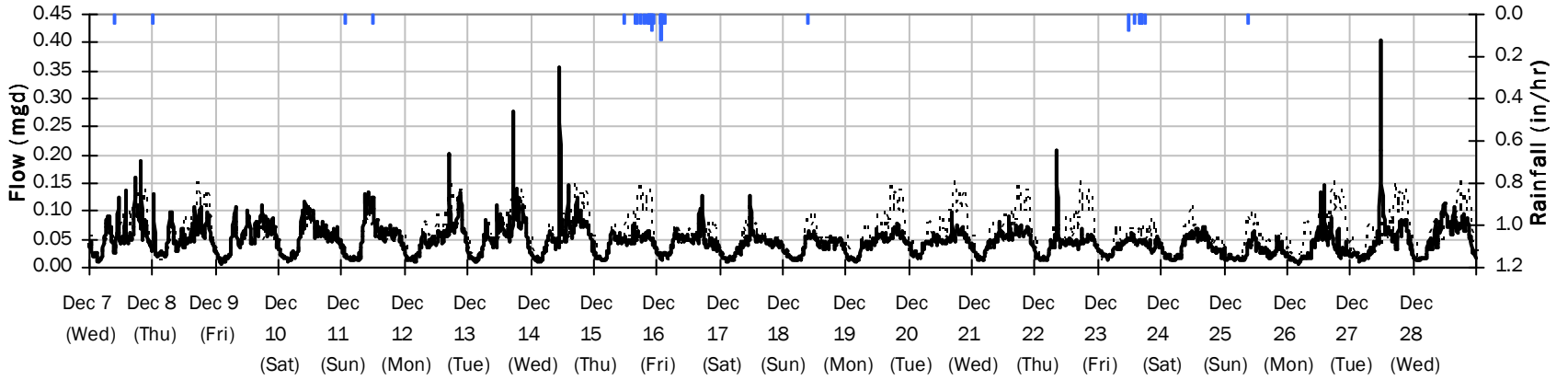


SITE 7

Flow Summary: 11/15/2016 to 12/28/2016

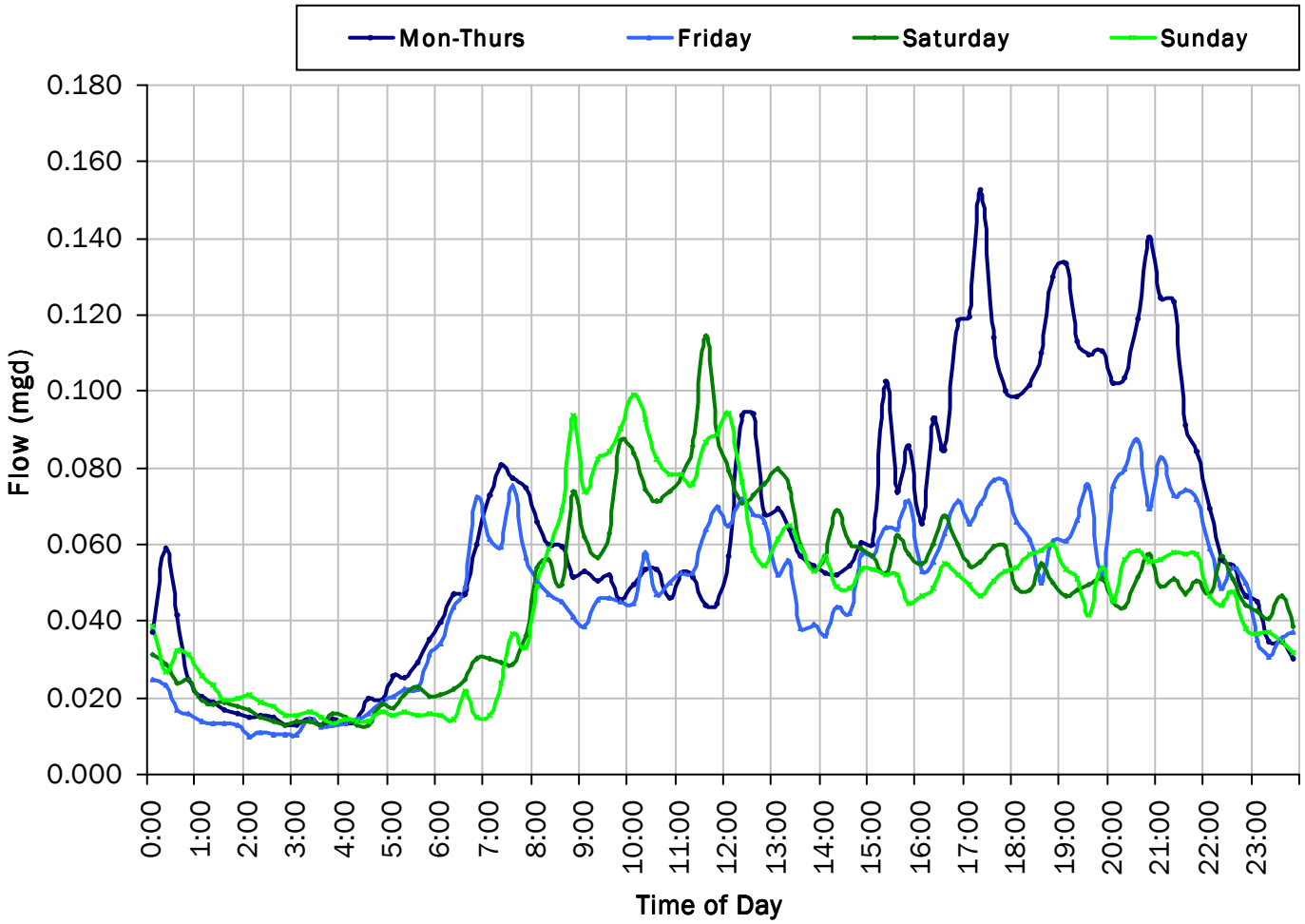
Total Period Rainfall: 1.01 inches **Avg Flow: 0.052 mgd** **Peak Flow: 0.465 mgd** **Min Flow: 0.007 mgd**





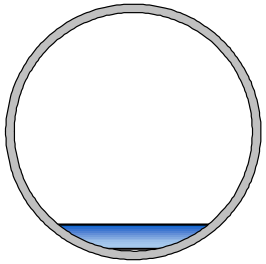
SITE 7

Average Dry Weather Flow Hydrographs



ADWF:

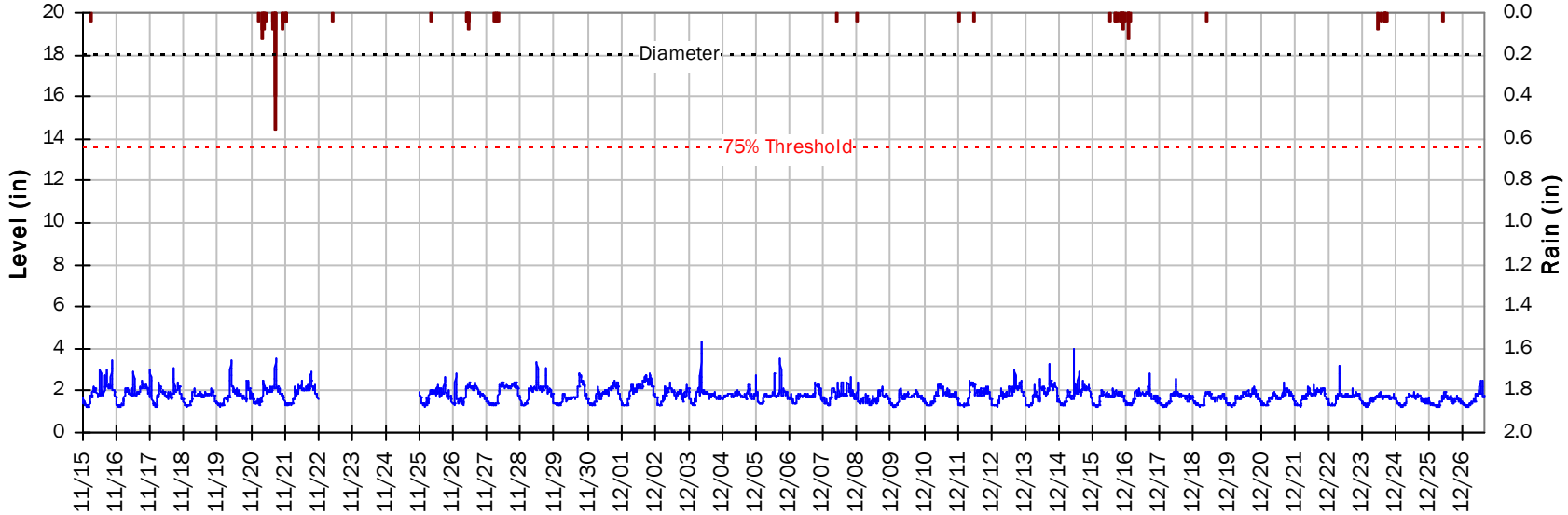
0.056 *mgd*

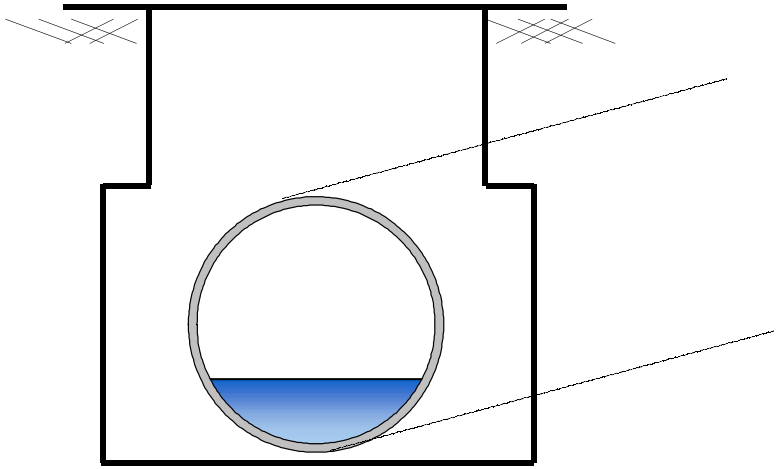


SITE 7

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event



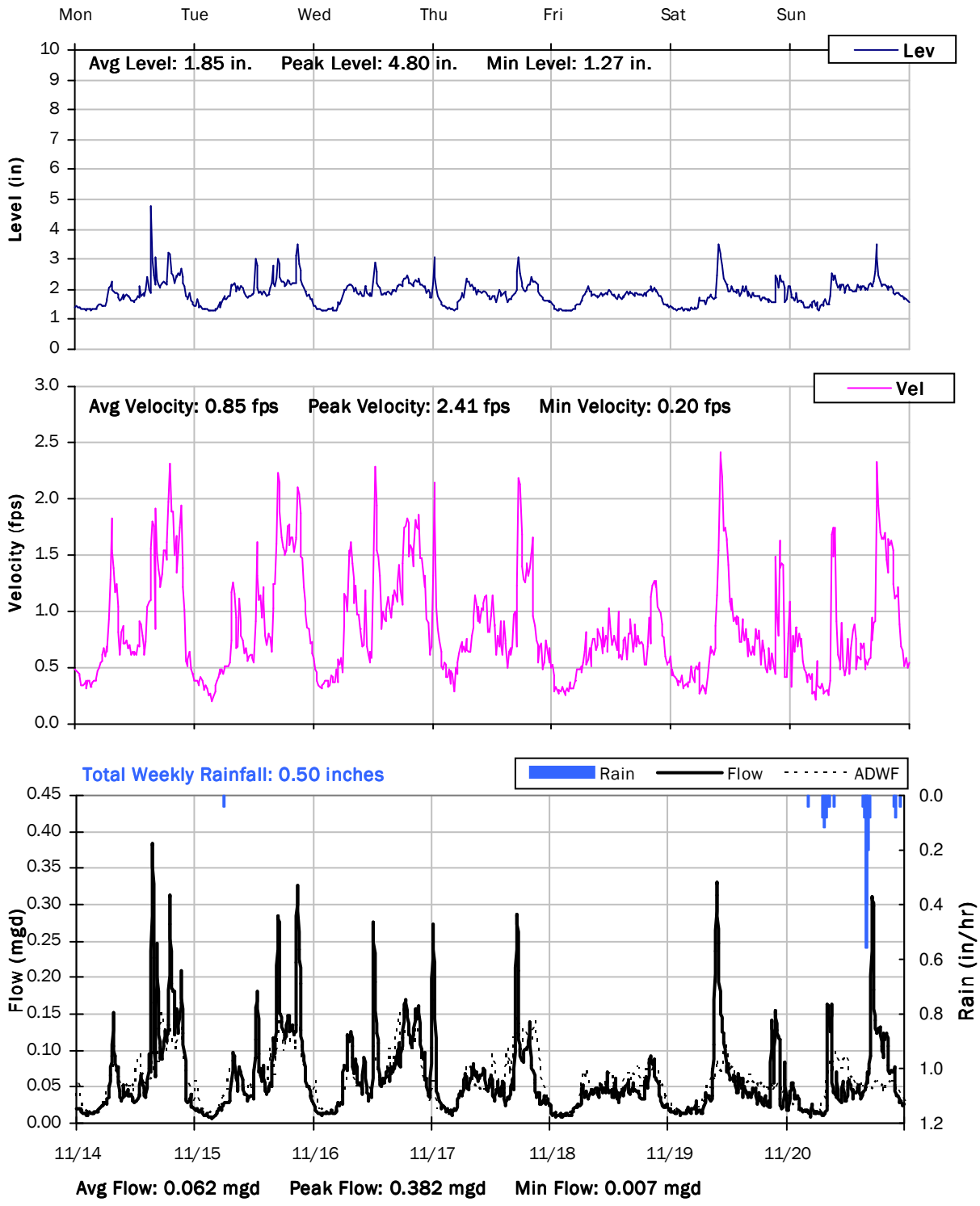


Pipe Diameter:	18	*inches*
Peak Measured Level:	4.8	*inches*
Peak d/D Ratio:	0.27	
Dry Weather Design Threshold Level:	13.5	*inches*

SITE 7

Weekly Level, Velocity and Flow Hydrographs

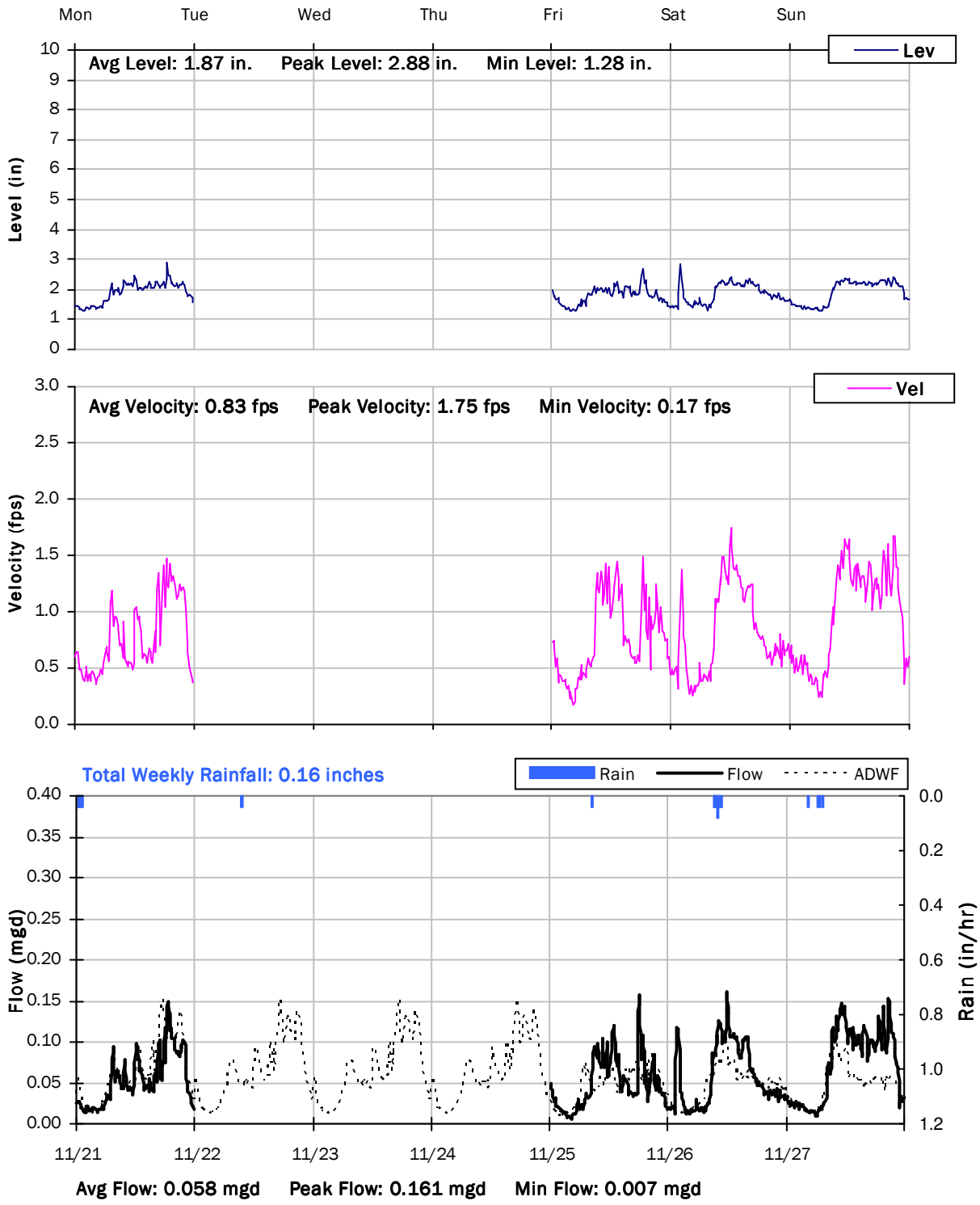
11/14/2016 to 11/21/2016



SITE 7

Weekly Level, Velocity and Flow Hydrographs

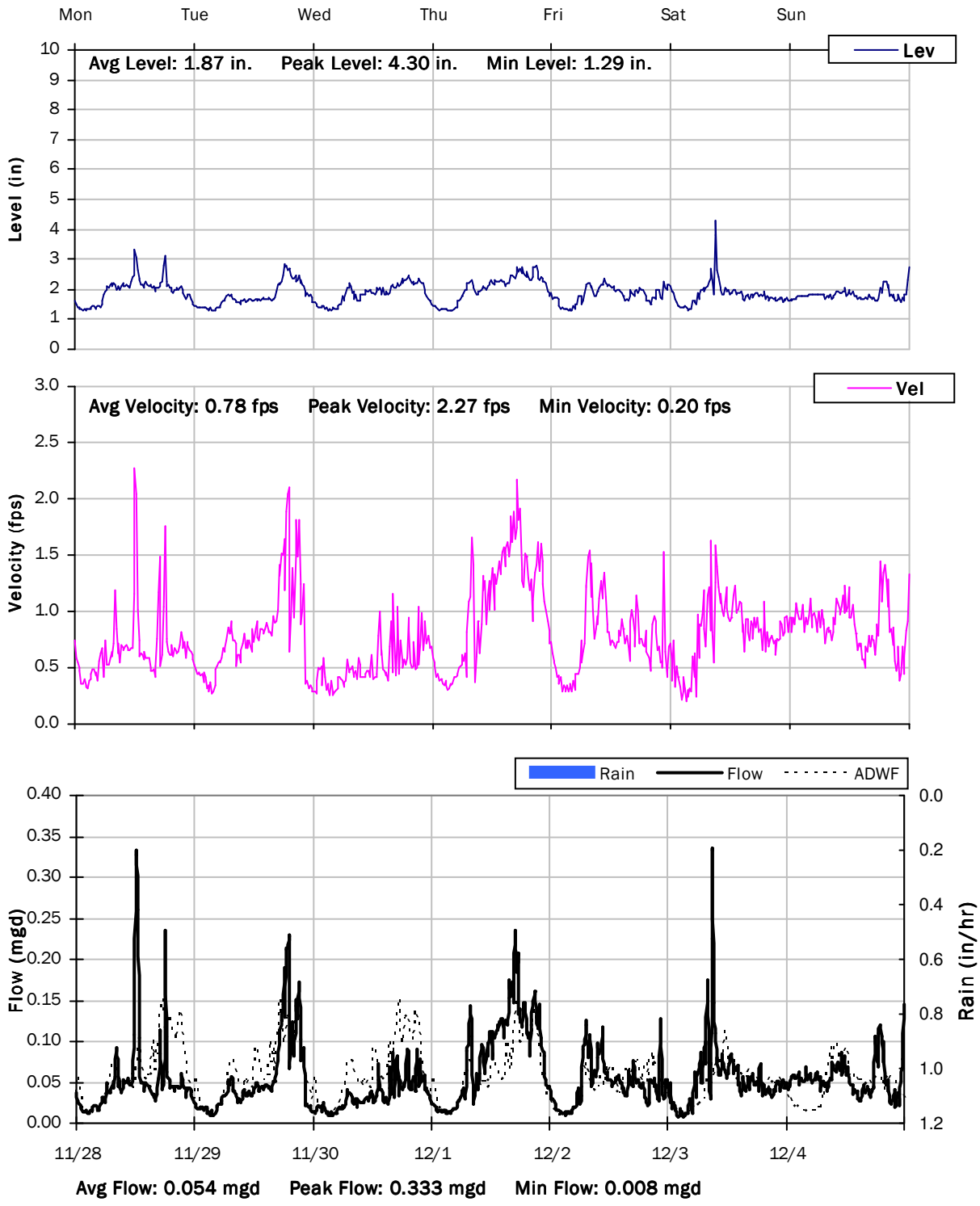
11/21/2016 to 11/28/2016



SITE 7

Weekly Level, Velocity and Flow Hydrographs

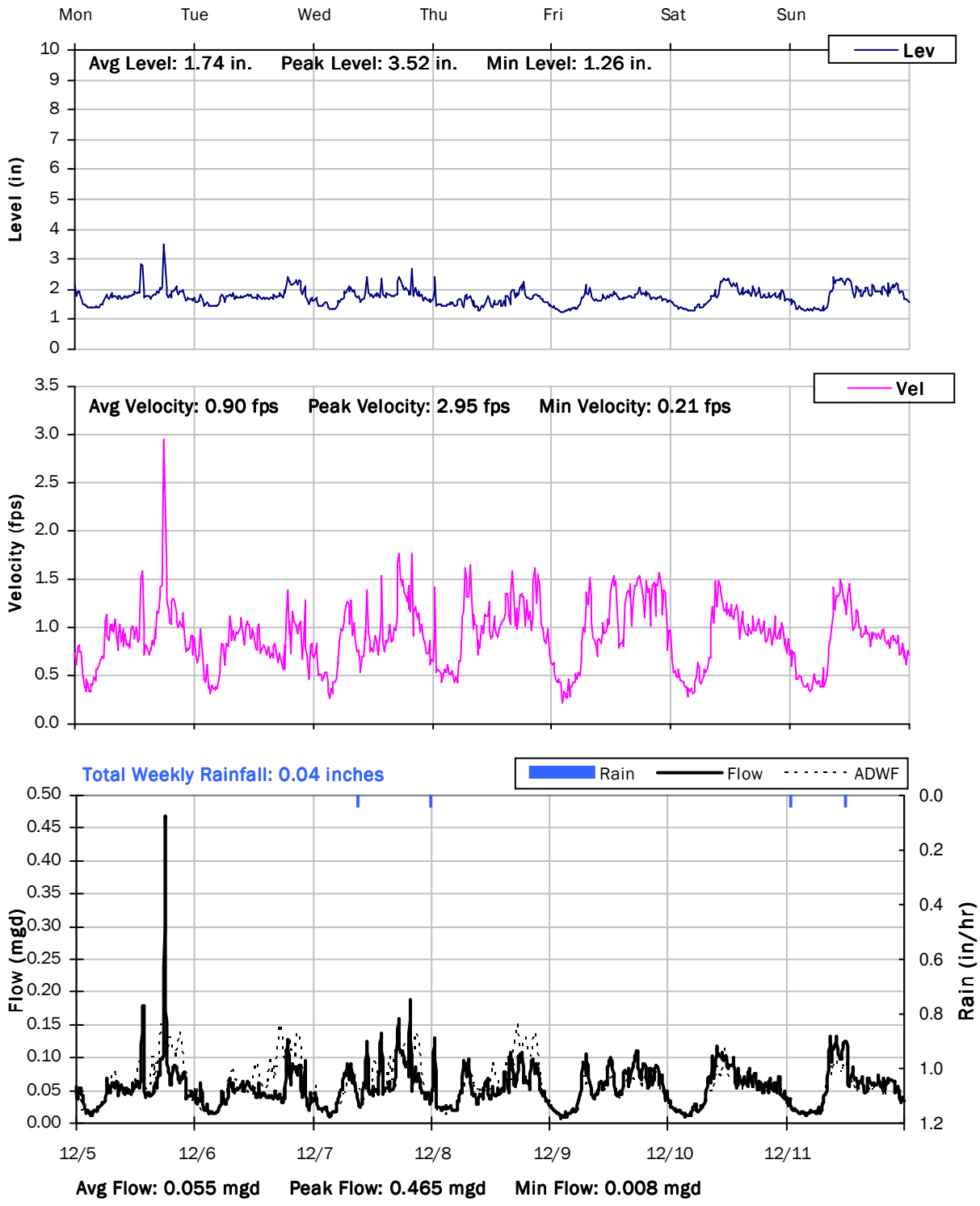
11/28/2016 to 12/5/2016



SITE 7

Weekly Level, Velocity and Flow Hydrographs

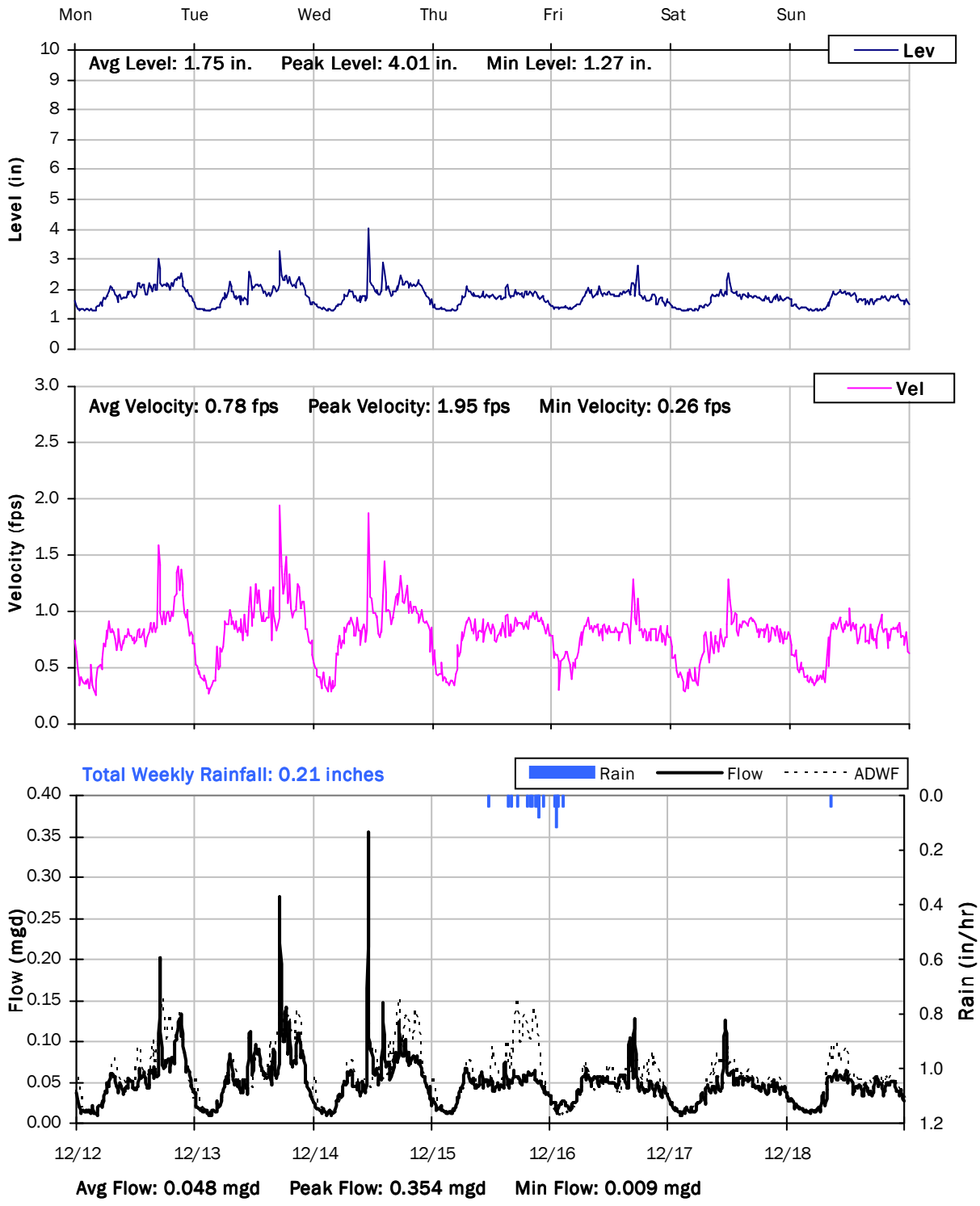
12/5/2016 to 12/12/2016



SITE 7

Weekly Level, Velocity and Flow Hydrographs

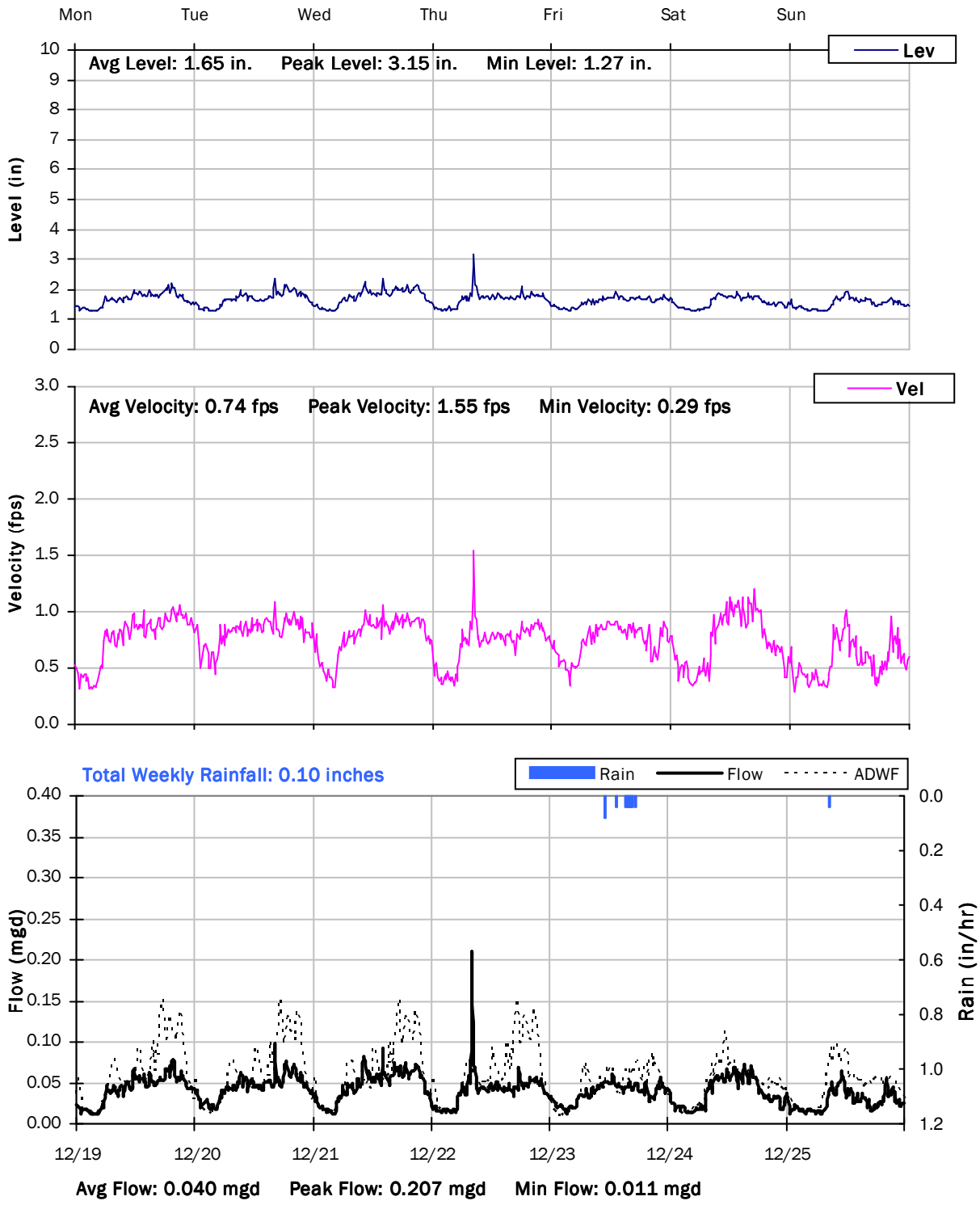
12/12/2016 to 12/19/2016



SITE 7

Weekly Level, Velocity and Flow Hydrographs

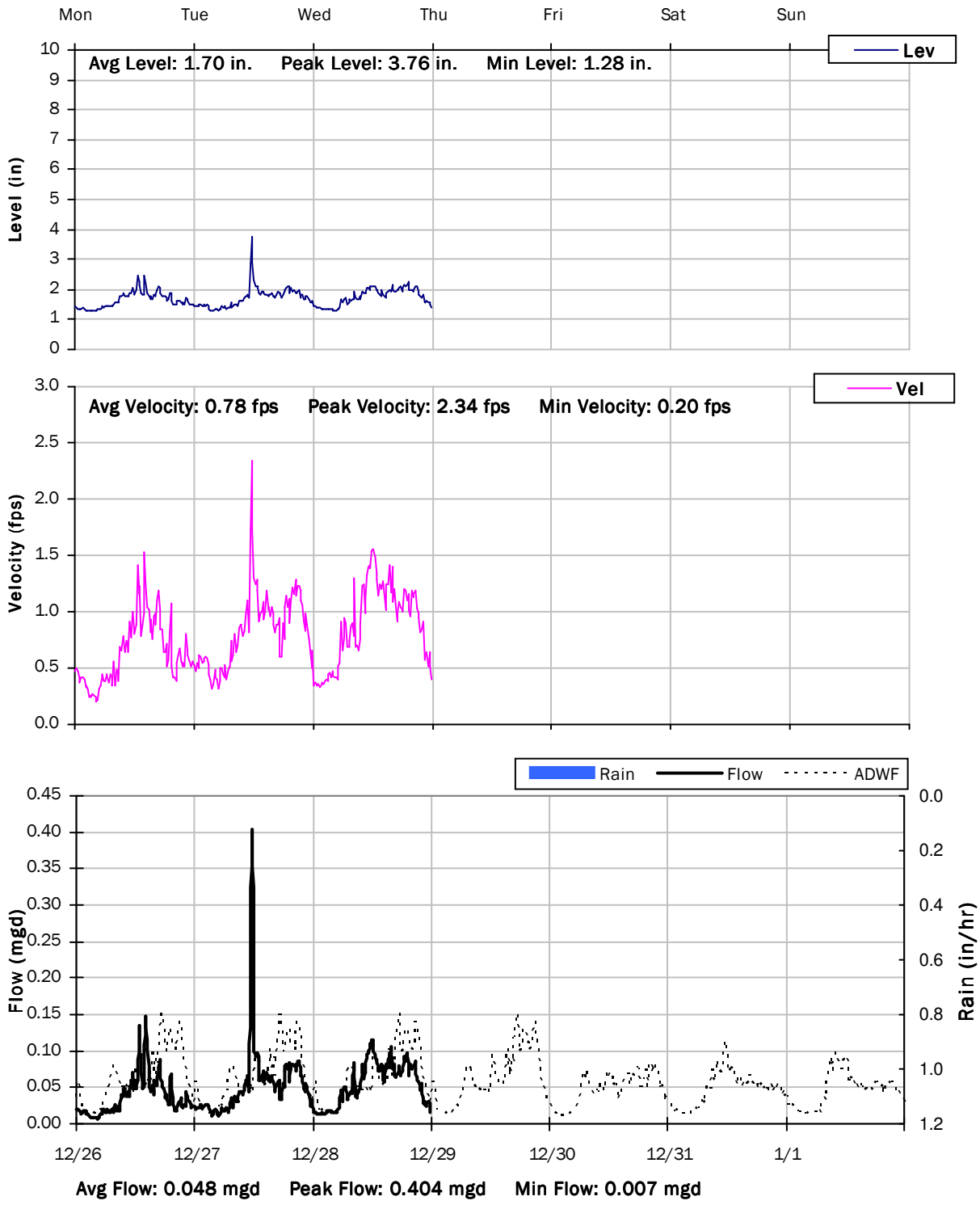
12/19/2016 to 12/26/2016



SITE 7

Weekly Level, Velocity and Flow Hydrographs

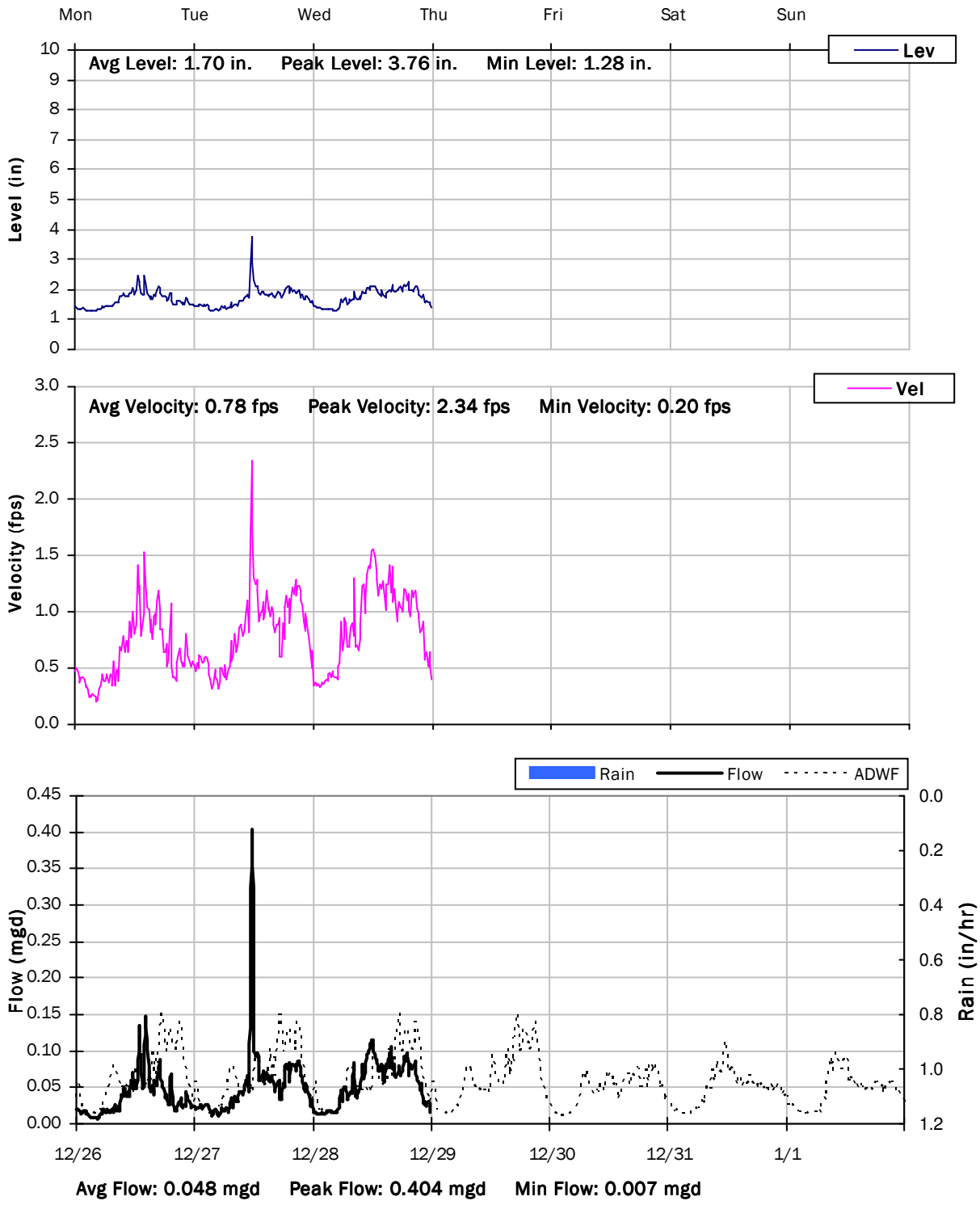
12/26/2016 to 1/2/2017



SITE 7

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



City of King City

Sanitary Sewer Flow Monitoring

Temporary Monitoring: January 29 to April 6, 2016

Monitoring Site: Site 8

Location: San Antonio Drive east of Metz Road

Data Summary Report

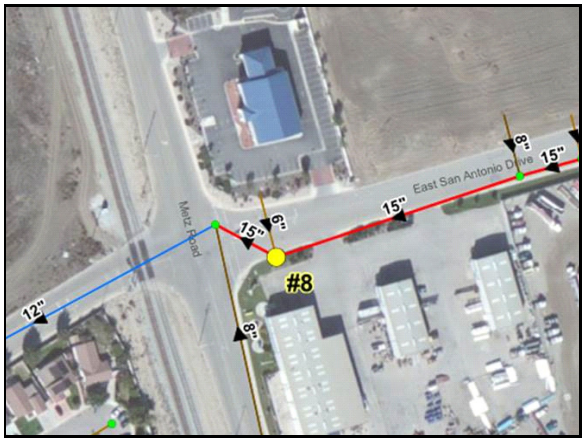
Vicinity Map: Site 8

SITE 8

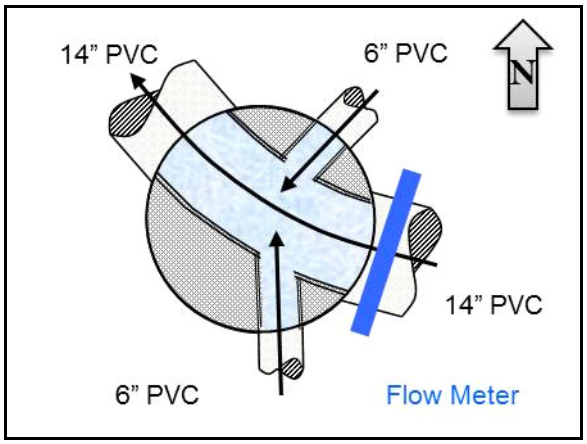
Site Information

Location:	San Antonio Drive east of Metz Road
Coordinates:	121.1272° W, 36.2219° N
Rim Elevation (Earth):	334 feet
Pipe Diameter:	14 inches
ADWF:	0.009 mgd
Peak Measured Flow:	0.199 mgd

Satellite Map



Sewer Map



Flow Sketch

Street View

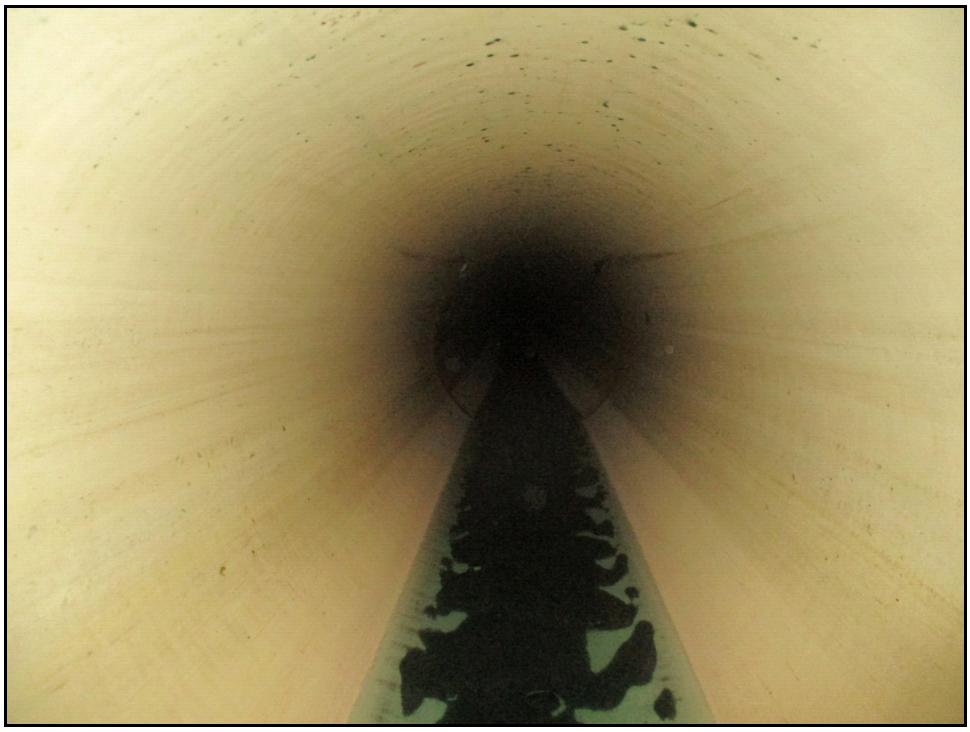
Plan View

SITE 8

Additional Site Photos

Effluent Pipe

Influent Pipe



SITE 8

Additional Site Photos

North Lateral Pipe

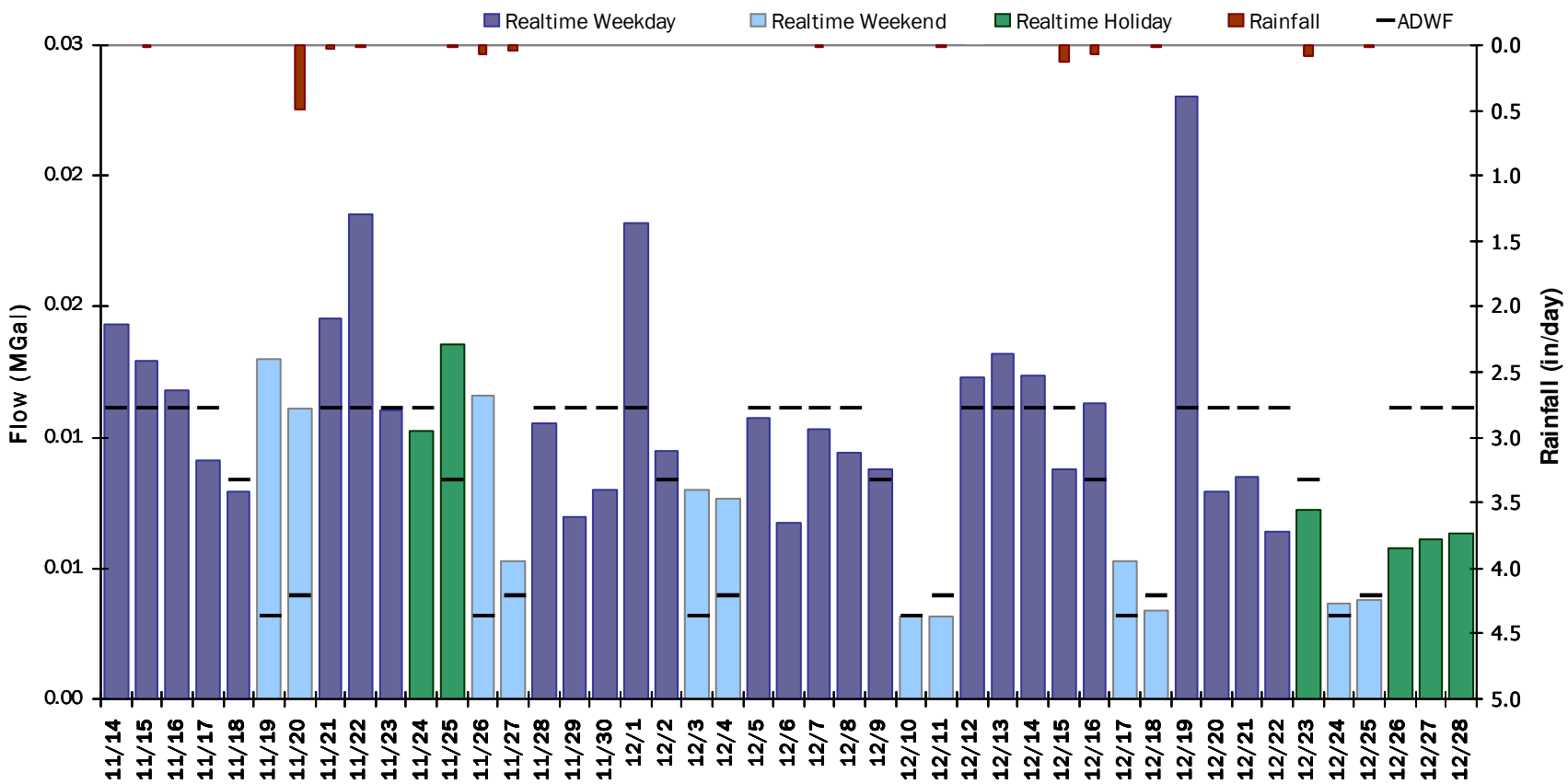
South Lateral Pipe

SITE 8

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.010 MGal **Peak Daily Flow: 0.023 MGal** **Min Daily Flow: 0.003 MGal**

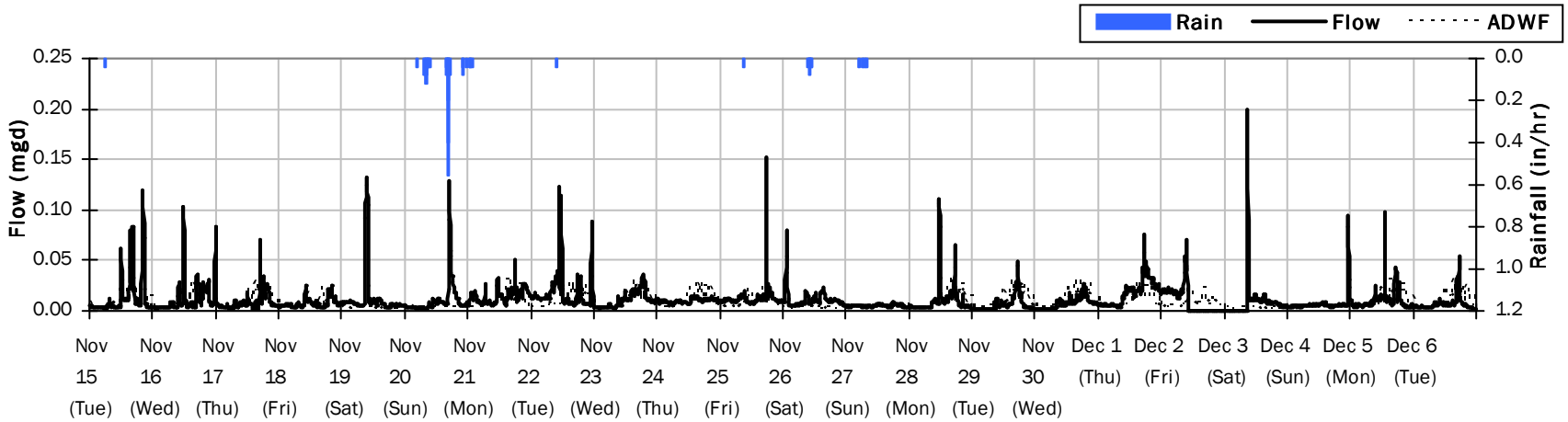
Total Period Rainfall: 1.01 inches

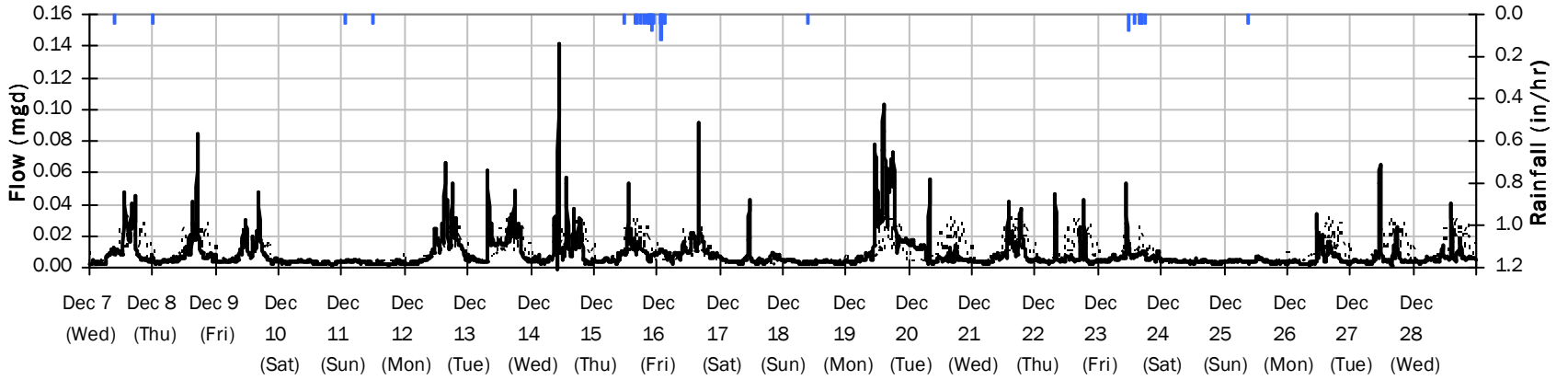


SITE 8

Flow Summary: 11/15/2016 to 12/28/2016

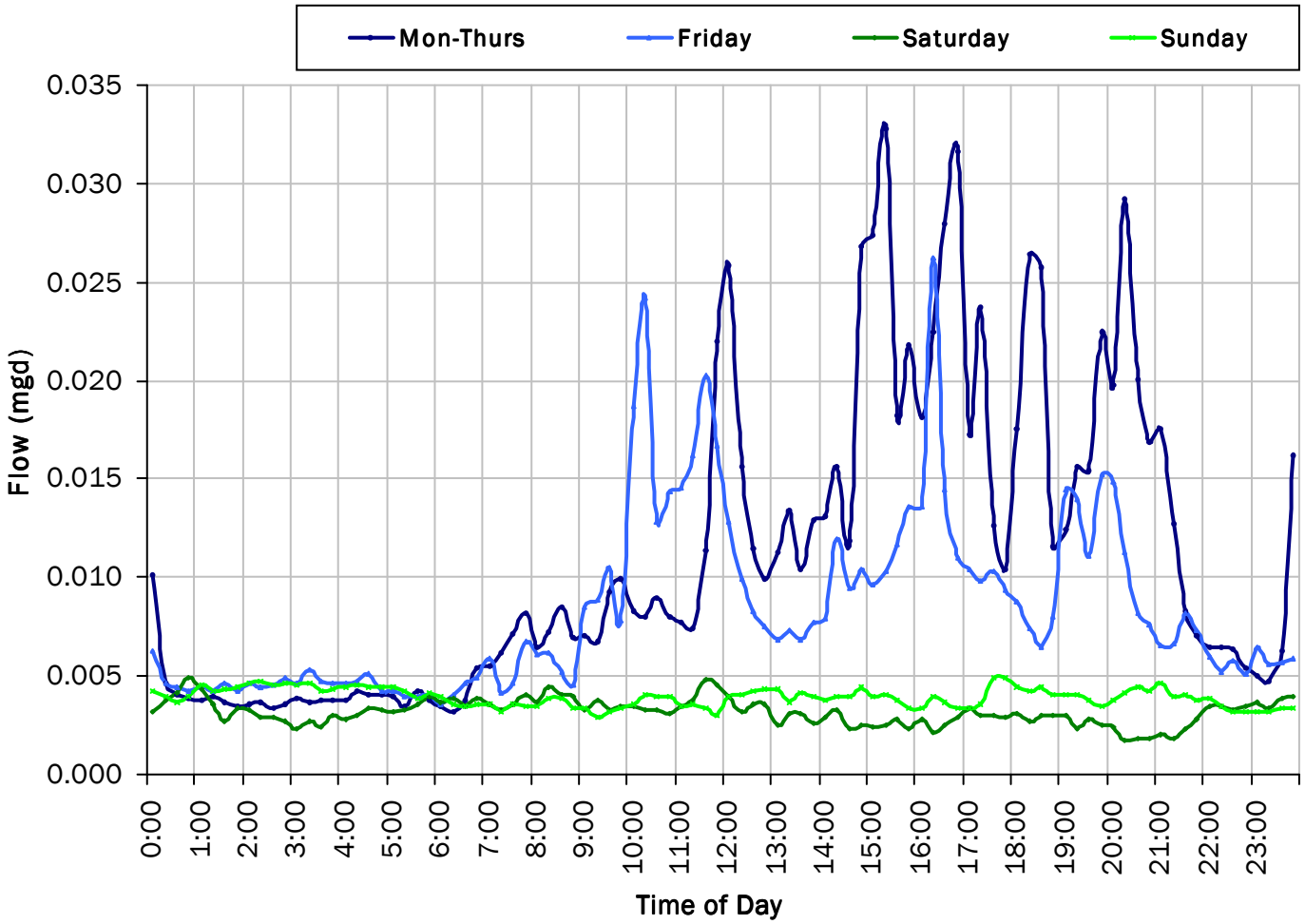
Total Period Rainfall: 1.01 inches **Avg Flow: 0.009 mgd** **Peak Flow: 0.199 mgd** **Min Flow: 0.000 mgd**





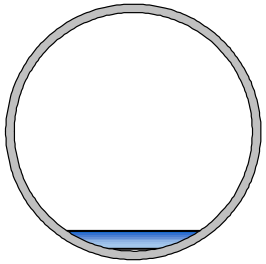
SITE 8

Average Dry Weather Flow Hydrographs



ADWF:

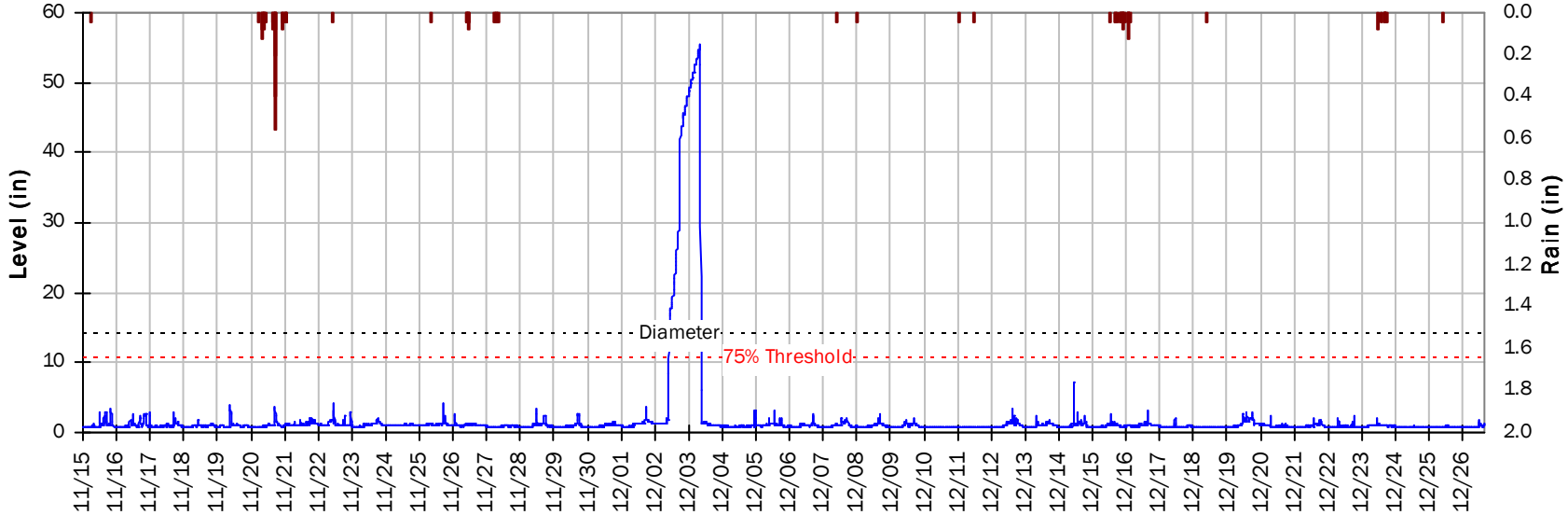
0.009 *mgd*

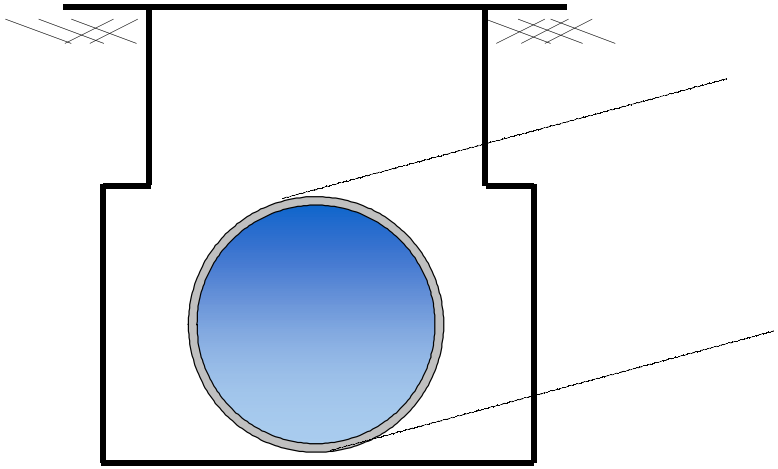


SITE 8

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Rainfall Event





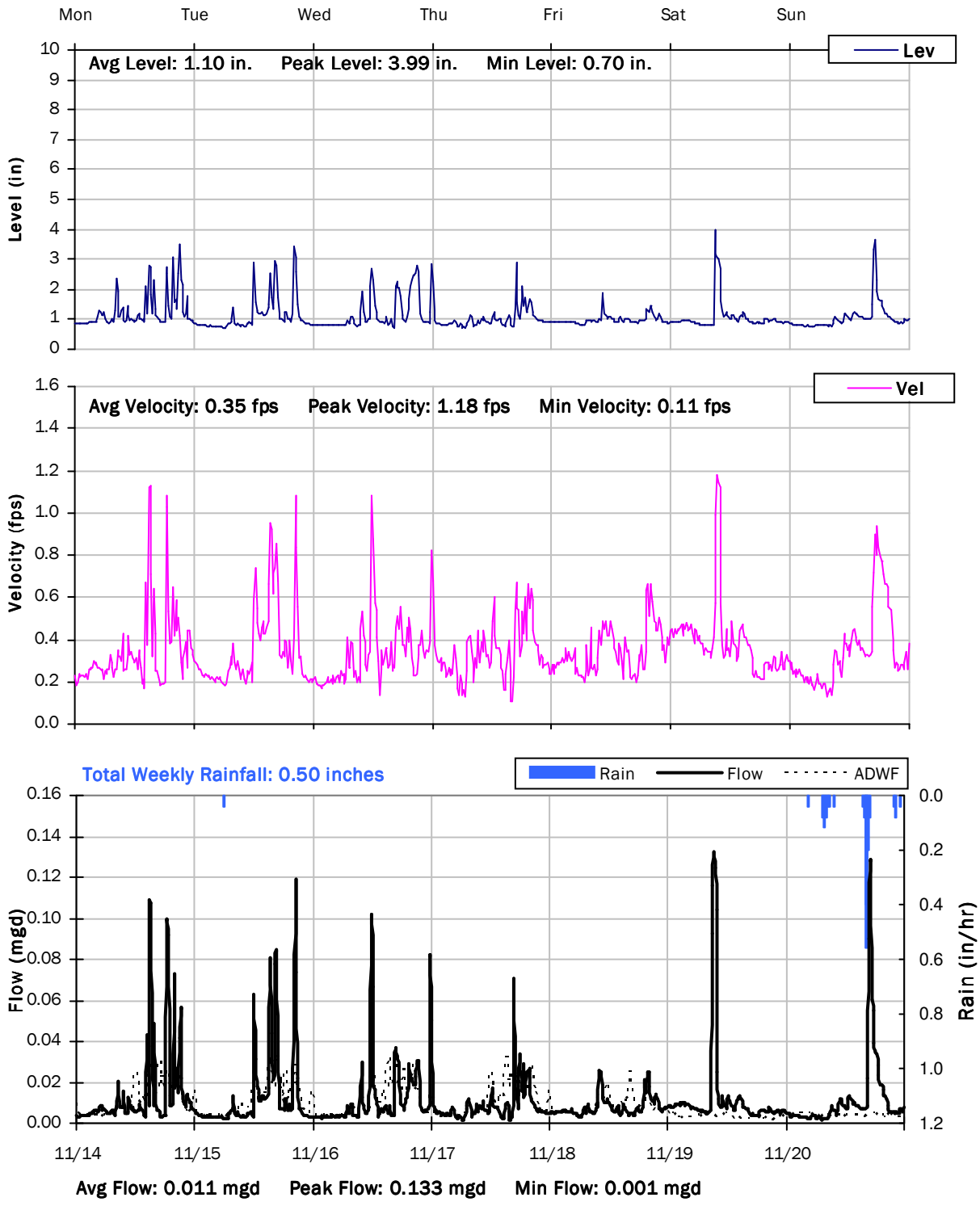
Pipe Diameter:	14	*inches*
Peak Measured Level:	55.4	*inches*
Peak d/D Ratio:	3.96	
Dry Weather Design Threshold Level:	10.5	*inches*

Surcharged 41.4 inches over crown

SITE 8

Weekly Level, Velocity and Flow Hydrographs

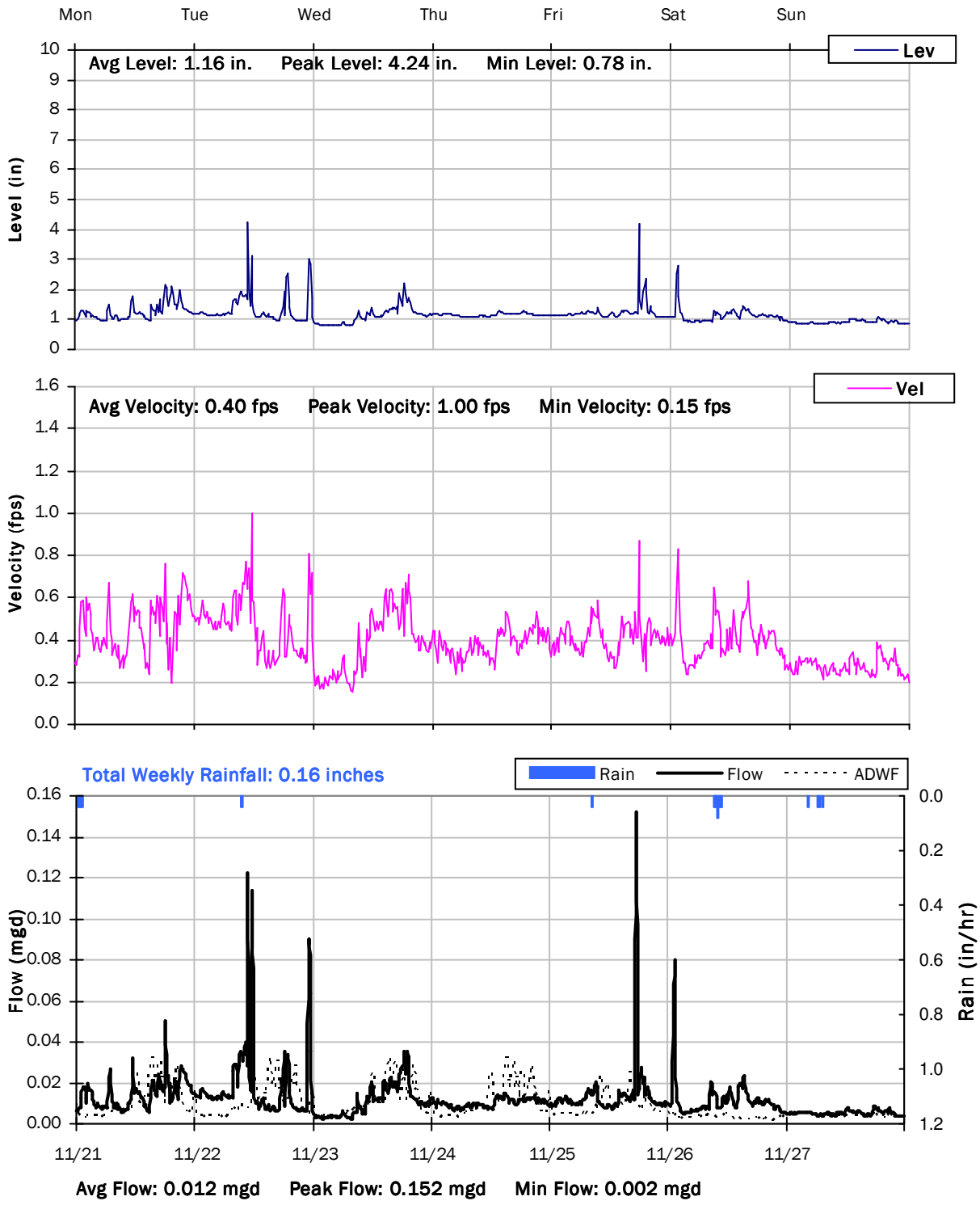
11/14/2016 to 11/21/2016



SITE 8

Weekly Level, Velocity and Flow Hydrographs

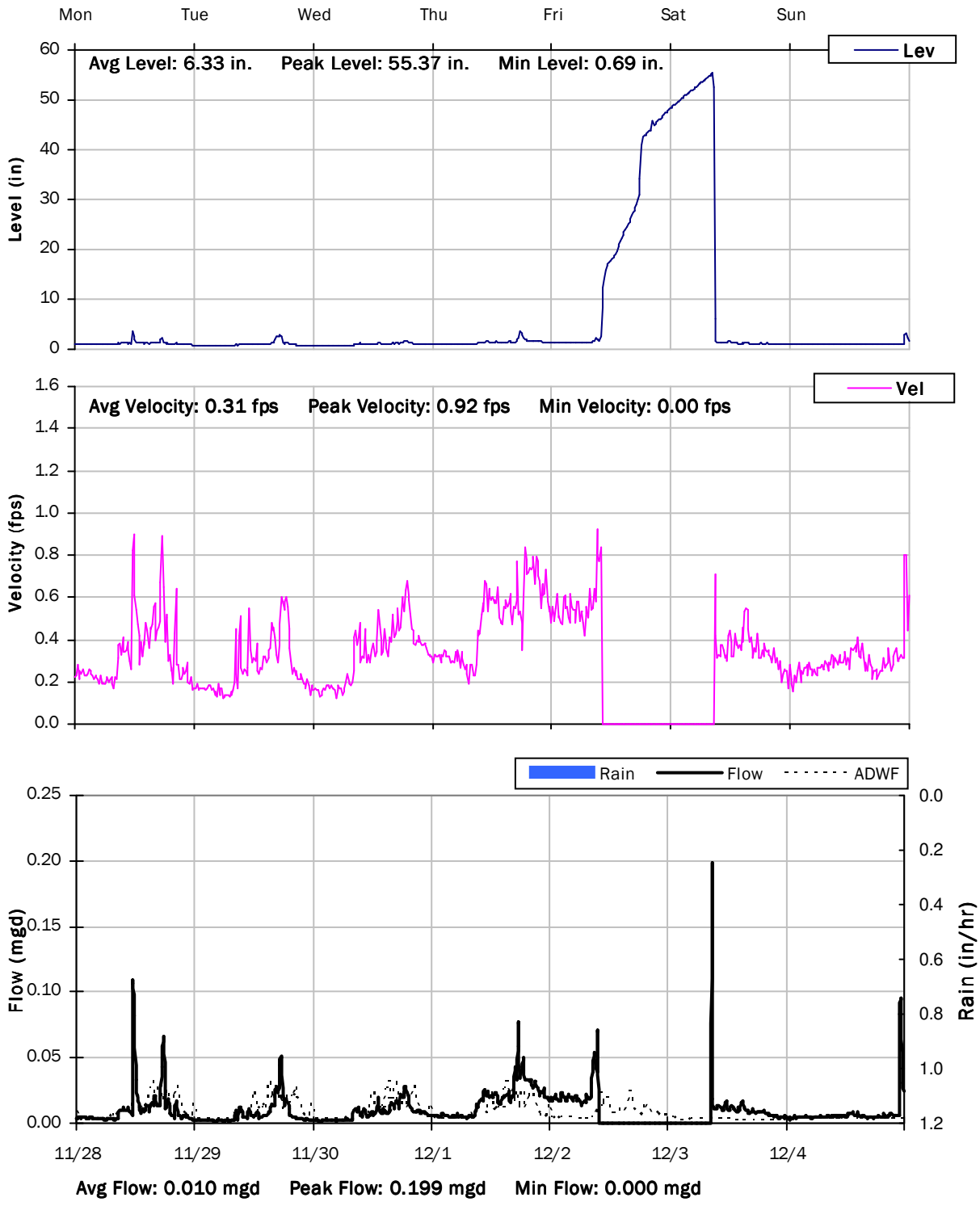
11/21/2016 to 11/28/2016



SITE 8

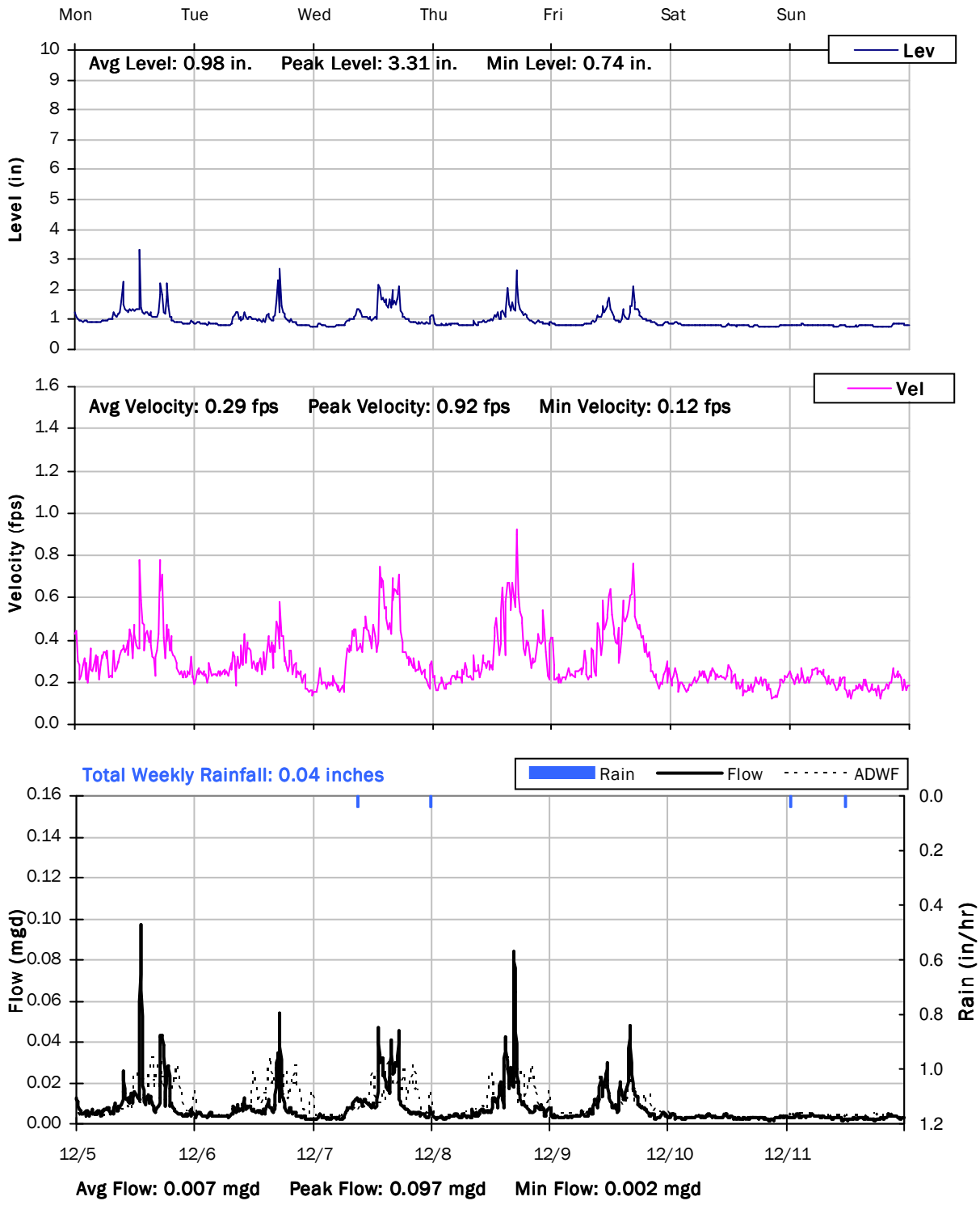
Weekly Level, Velocity and Flow Hydrographs

11/28/2016 to 12/5/2016



SITE 8

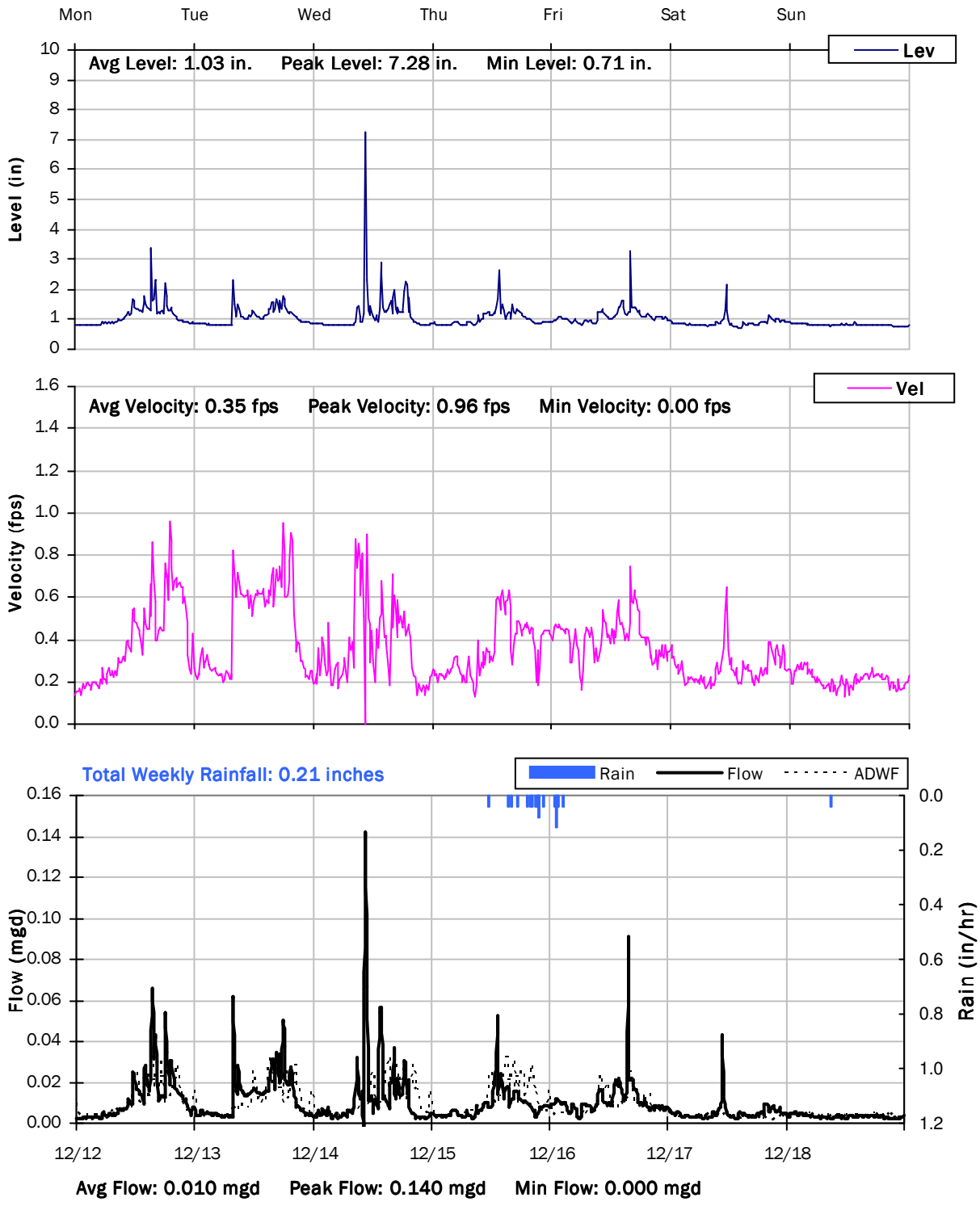
Weekly Level, Velocity and Flow Hydrographs
12/5/2016 to 12/12/2016



SITE 8

Weekly Level, Velocity and Flow Hydrographs

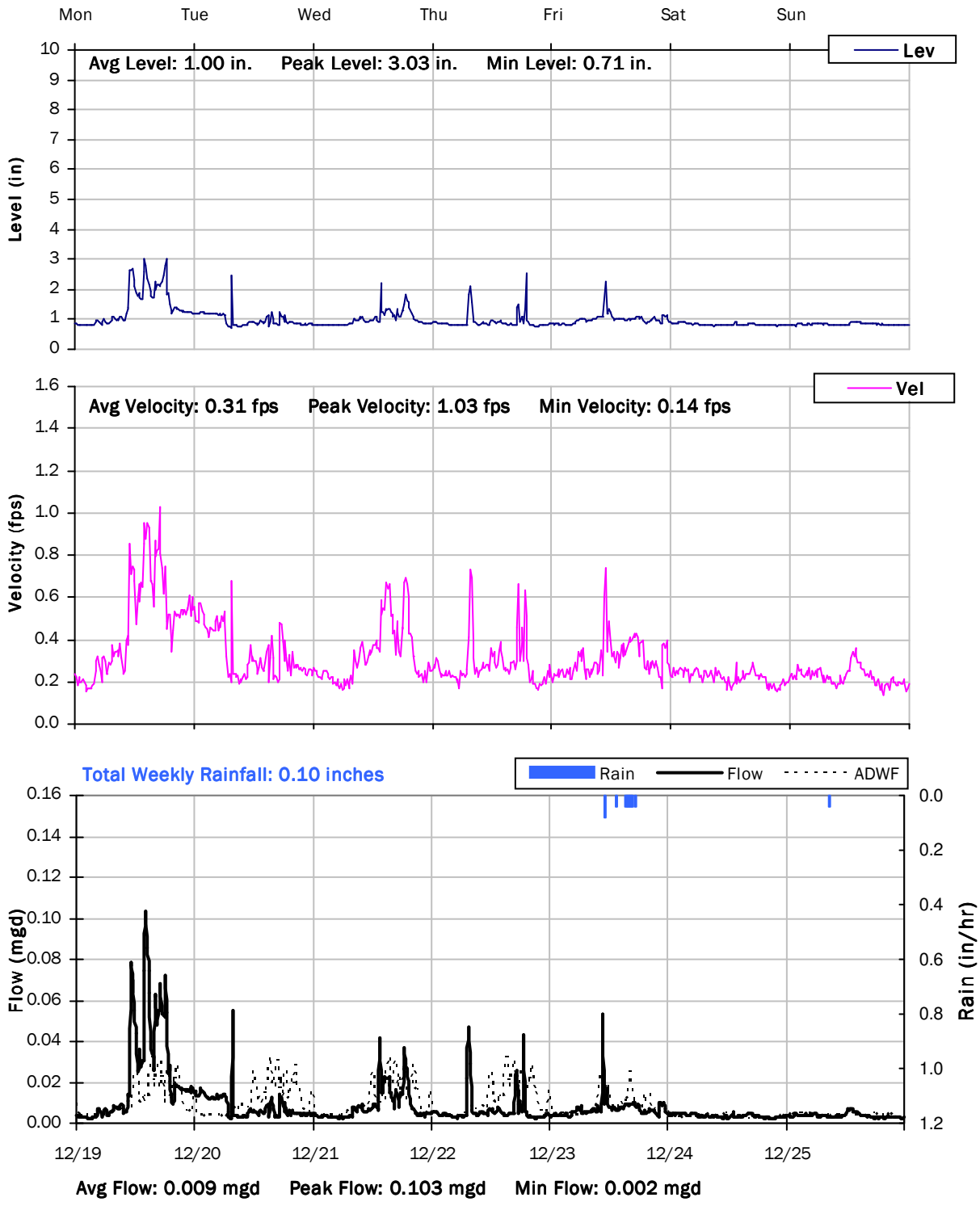
12/12/2016 to 12/19/2016



SITE 8

Weekly Level, Velocity and Flow Hydrographs

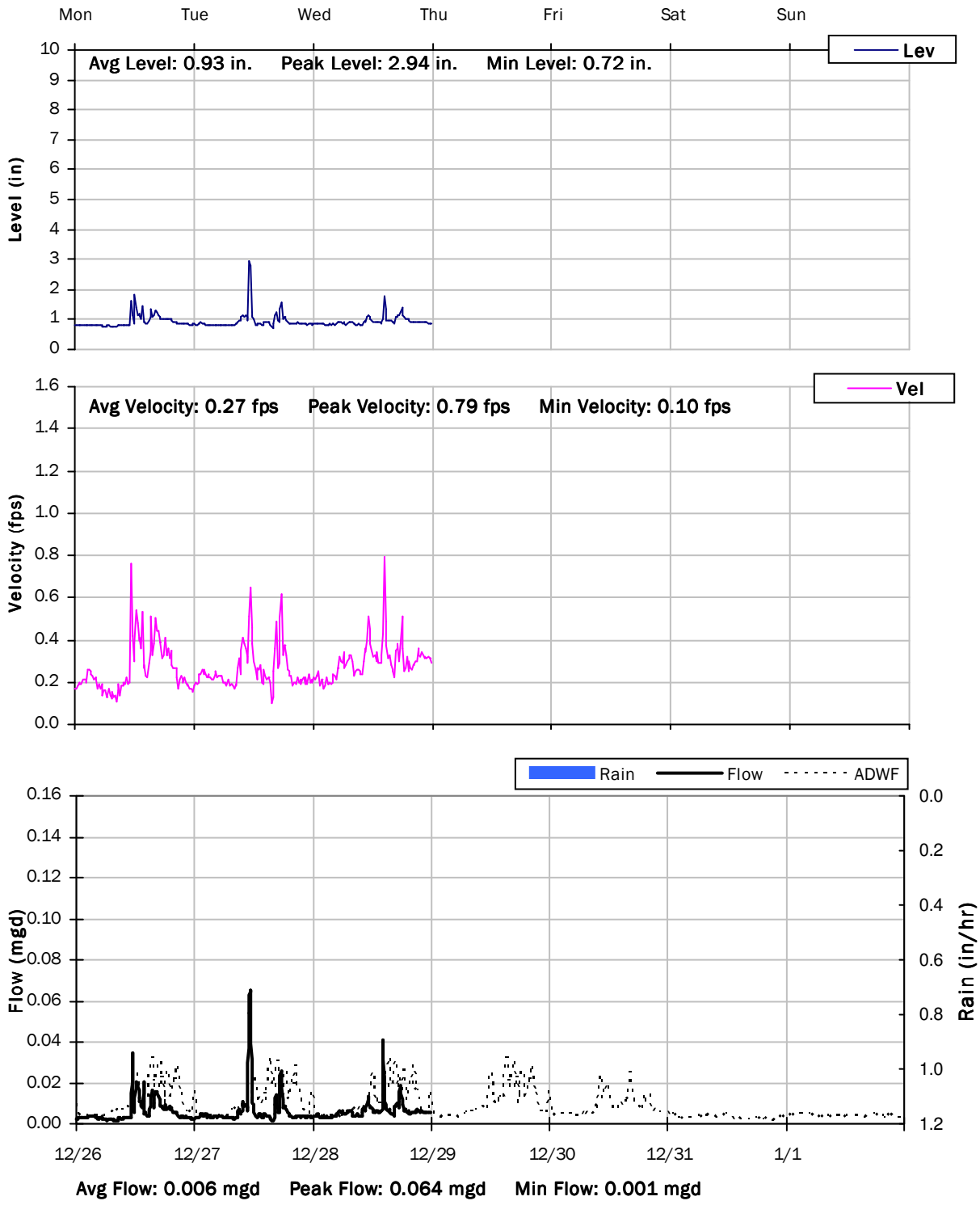
12/19/2016 to 12/26/2016



SITE 8

Weekly Level, Velocity and Flow Hydrographs

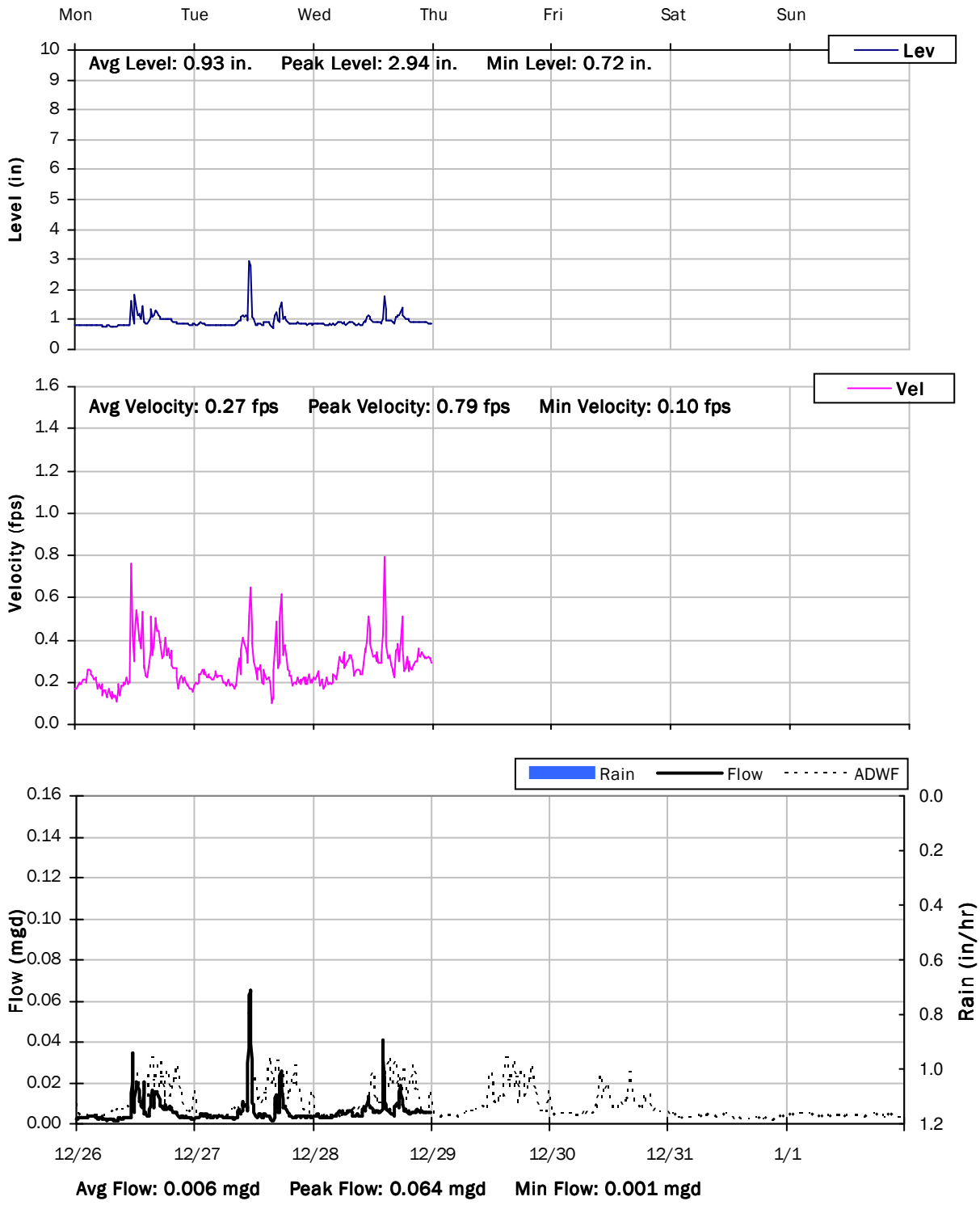
12/26/2016 to 1/2/2017



SITE 8

Weekly Level, Velocity and Flow Hydrographs

12/26/2016 to 1/2/2017



V&A Project No. 16-0200

V&A
consulting engineers
1000 Broadway, Suite 230
Oakland, CA 94607
510.903.6600
510.903.6601, Fax

APPENDIX B – TM NO. 1 – HYDRAULIC MODELING SOFTWARE EVALUATION

This document is released for the purpose of information exchange review and planning only under the authority of Eric T. Casares, September 23, 2016, California 73351

KING CITY

COLLECTION SYSTEM MASTER PLAN

TECHNICAL MEMORANDUM NO. 1

HYDRAULIC MODELING SOFTWARE EVALUATION

DRAFT
September 2016

710 W. PINEDALE AVENUE • FRESNO, CALIFORNIA 93711 • P. 559.436.6616 • F. 559.436.1191

KING CITY

COLLECTION SYSTEM MASTER PLAN

TECHNICAL MEMORANDUM
NO. 1
HYDRAULIC MODELING SOFTWARE EVALUATION

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APPENDIX A SEWER SYSTEM MODELING WHITE PAPER

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HYDRAULIC MODELING SOFTWARE EVALUATION

1.0 INTRODUCTION

King City (City) has contracted Carollo Engineers, Inc. (Carollo) to assist the City in developing its Collection System Master Plan (CSMP) and Wastewater Treatment Plant (WWTP) Master Plan. As a part of this CSMP, a hydraulic computer model of the City's collection system will be developed. The purpose of this technical memorandum (TM) is to compare and evaluate the various modeling software packages available to the City and recommend the modeling software that best meets the City's needs.

In the past decade, improvements have been made to the hydraulic modeling software available on the market. Some examples of the improvements include modifications to the hydraulic routing engine as well as an enhanced graphical user interface (GUI), model output reports, and geographic information systems (GIS) compatibility. This TM presents a summary of the major software vendors, briefly explains software features, compares the advantages and disadvantages of each software program, and provides a recommendation for the software program to be used for preparation of the CSMP.

Appendix A provides a basic overview of what a sewer system hydraulic model is, as well as an explanation of the different computational methods that are used in the hydraulic models available in the marketplace today.

2.0 SOFTWARE VENDORS

Many software packages can potentially address the needs of the City, and vary in their methods of analysis and user friendliness. Sanitary sewer hydraulic modeling software packages from five (5) major software vendors were evaluated and are listed below in alphabetical order:

- **Bentley Systems, Inc.**: Bentley Systems, Inc. (Bentley) is an engineering and architecture software company with corporate headquarters in Exton, Pennsylvania. Bentley added a suite of water, wastewater, and storm water analysis software through its acquisition of Connecticut based Haestad Methods, Inc. in 2004. The company offers two wastewater collection system packages: SewerCAD and SewerGEMS.

- **Computational Hydraulics Institute:** Computational Hydraulics International (CHI) is a consulting engineering firm specializing in stormwater management. CHI is a headquartered in Ontario, Canada with a US office in New York. The company has been providing the PC-SWMM software package since 1984.

- **Danish Hydraulic Institute:** The Danish Hydraulic Institute (DHI) is an international hydraulic consulting and research institution headquartered in Denmark. There are three offices in the United States: Portland, Oregon, St. Petersburg, Florida, and Solana Beach, California. The company's MIKE URBAN software application supports two computational engines for urban hydrology and open channel/closed pipe hydraulics: the Environmental Protection Agencies (EPA) open source SWMM 5 engine, and DHI's proprietary MOUSE computational engine.

- **Environmental Protection Agency (EPA):** The EPA first developed the Storm Water Management Model (SWMM) around 1970, which has undergone several major upgrades since then. The latest version of EPA SWMM, SWMM 5, was released in 2005. The SWMM 5 hydraulic emgine is widely used throughout the industry through the EPA's open source SWMM 5 software application. In addition, several third party software vendors use the SWMM 5 hydraulic engine in their modeling software packages.

- **Innovyze:** Innovyze is headquartered in Broomfield, Colorado, and is a leading provider of software products geared towards hydraulics and hydrology. In 2009, Colorado based MWH Soft and British based Wallingford Software merged into a single company (MWH Soft), which in 2011 was renamed Innovyze. Innovyze offers three main sewer system modeling software packages. These are InfoSewer/H_2OMAP Sewer, InfoSWMM/H_2OMAP SWMM, and InfoWorks. The InfoWorks software is not included in this analysis. Its higher costs and capabilities are not aligned with the City's needs.

The comparison of each of the technical features of the five software vendors and the modeling software that each vendor offers is presented in Table 1. This table allows a side-by-side comparison of similar features in each software package. The features that have the greatest impact on the selection of an appropriate software package are discussed in detail as part of this memo.

3.0 EVALUATION CRITERIA

As a way to evaluate the advantaged and disadvantages of each software package, several criteria are used for a software evaluation, including:

- Dry and Wet Weather Flow Calculation Methods
- Hydraulic Flow Routing Calculation Algorithms
- GIS Interface
- Scenario Management
- Customer Service and Support

Table 1 Model Comparison
Collection System Master Plan
King City

Technical Characteristics	Innovyze			Bentley		CHI	DHI	EPA
	H_2OMAP Sewer/ InfoSewer(1)	H_2OMAP SWMM/ InfoSWMM(1)	InfoWorks	SewerCAD(2)	SewerGEMS(5)	PCSWMM(3)	MIKE URBAN(4)	SWMM 5
Computation Method	Quasi-Dynamic and Steady State	Dynamic Wave and Kinematic Wave	Dynamic Wave	Standard Step	Dynamic Wave and Kinematic Wave	Dynamic Wave and Kinematic	Dynamic Wave	Dynamic Wave and Kinematic Wave
GIS Compatible	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Reads shapefiles directly	Yes	Yes	No	Yes	Yes	Yes	Yes	No
Writes to shapefiles directly	Yes	Yes	No	Yes	Yes	No	No	No
Tools to fix GIS data topology problems	Yes	Yes	Yes	Partial	Yes	Partial	Partial	No
Utilizes Standard Database	Yes	Yes	Yes	Proprietary	Proprietary	Yes	ASCII-based	Yes
Automatically sizes new mains	Yes	Yes	No	Yes	Yes	No	Yes	No
Calculates pipe replacement costs	Yes	Yes	No	Yes	Yes	No	No	No
Calculates loads based on GIS land use	Yes	Yes	Yes	via GIS	via GIS	Yes	Yes	No
Time step	User Defined	User Defined	User Defined	User Defined	User Defined	User Defined	User Defined	User Defined
Scenario manager	Yes	Yes	Yes	Yes	Yes	No	Yes	No
Customizable tabular reports	Yes	Yes	Yes	Yes	Yes	No	Yes	No
Graphically compares the results of multiple simulations	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Displays GIS data layers on	Yes	Yes	Yes	No	Yes	Yes	Yes	No
Export tabular data to excel	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Internet Based Model Network/Output Viewer Available	Yes	Yes	Yes	No	No	Yes	No	No
Single Licenses Cost (1,000 pipe version)	$4,000	$6,000	$13,500	$5,452	$10,909	$1,440 ($2,160)	$7,000	Free
Maintenance and Service (Annual Fee)	$800	$1,200	$2,025	$1,316	$2,620	n/a	$1,960	Free
Water Modeling Software	Yes	Yes	Yes	Yes	Yes	No	Yes	No

Notes:

(1) Costs presented are for the standard fixed seat license. Pricing differs for the floating seat licenses and the suite packages.

(2) SewerCAD evaluation is based on the stand-alone version. SewerCAD with AutoCAD is priced at $8,726 for 1000 pipe version with $2,102 annual support and maintenance fee.

(3) Pricing represents a single user license of PCSWMM Standard. Cost in parentheses is for PCSWMM Professional.

(4) Pricing is for MIKE URBAN supporting the MOUSE (and SWMM5) engine. MIKE URBAN supporting MOUSE also supports the EPANET engine, which means that is doubles as a sewer/storm as well as a water distribution system modeling software package.

(5) SewerGEMS includes SewerCAD and all of its capabilities.

- Cost
- Ease of Use
- Internet Based Model Network/Output Viewer

Although the City has no immediate plans of purchasing the selected software package, it is still interested in these factors should they decide to purchase the software at a later date. For this reason, Carollo recommends the City select a model that is easy to operate, compatible with GIS software and data sources, has the ability to analyze several scenarios with multiple facility options, and is cost effective. City staff may also find it necessary to use the software vendor's customer service and support to troubleshoot operating issues associated with model use. Each of the criteria listed above is briefly discussed herein.

3.1 Dry and Wet Weather Flow Calculations

Many models listed here were first developed not only for sanitary sewers but also for stormwater sewers. Therefore, these models contain modules for hydrologic (or wet weather flow - WWF) calculations as well as hydraulic calculations. When used as a sanitary sewer model, the WWF calculations are used to calibrate the infiltration and inflows (I/I) that enter the system as a result of rainfall events, and then apply these I/I characteristics to other rainfall events. Several of the models also have the ability to project dry weather flows (DWF) based on population, land use data, or parcel level water usage. Estimating accurate DWF and WWF is critical because all hydraulic calculations are based on these flows.

3.1.1 Dry Weather Flows

DWFs can be entered directly into a model, as a series of diurnal flows, or can be generated in the model based on population or land use estimates. Most of the models reviewed can accept a time series of diurnal flows. Certain models also have the ability to generate DWFs based on a population in a basin (in gallons per capita per day) or based on land use (in gallons per acre per day). Once the ADWF is estimated, a diurnal pattern can be applied to the average dry weather flow (AWDF).

3.1.2 Wet Weather Flows

WWFs can be generated using a variety of hydrologic techniques typically applied to stormwater runoff in order to approximate I/I in the collection system. Most models generate an I/I hydrograph by converting rainfall into flow based on the area that contributes I/I flow to the collection system. Unlike stormwater, the area contributing to I/I in a sanitary sewer basin area cannot be directly measured. Therefore, hydraulic models perform runoff calculations based on a percentage of the total sewer basin area, or "effective area," that contributes I/I (e.g. 5 percent).

Simple models usually employ a unit hydrograph type algorithm to generate the WWF hydrograph. This usually includes the use of an “effective area” variable that is sometimes referred to as an R-value. An R-value represents the amount of rainfall that enters a sewer basin as a percent of the total rainfall that fell on the basin (and is usually reported as a percentage). This variable, along with a variable that approximates the time of concentration of the basin is applied to the intensity of rainfall during a storm to calculate the I/I hydrograph. Some models include two or more of these types of hydrographs – one for inflow, and one or more for infiltration. If the system being modeled experiences little I/I, these simplistic routines may be all that is warranted.

Complex models employ a more rigorous estimation of WWFs. These models include more variables to better approximate the peak, volume, and shape of the I/I hydrograph by taking into account soil saturation and near surface groundwater interaction. These routines include linear reservoir, non-linear reservoir, and other combinations of algorithms. The more complex models like MIKE URBAN, InfoWorks and PC-SWMM employ these more complex routines to better approximate I/I hydrographs for collection systems that have significant flow contributions from infiltration and inflow.

3.2 Hydraulic Calculation

There are several differences among sewer models in how hydraulic computations are performed. The most important difference is in how the calculations involve time. There are two primary kinds of hydraulic sewer models:

- **Steady State** models do not account for changes in flow over time, and
- **Dynamic** models involve time in their calculations, most notably in being able to vary flow over time and calculate the associated changes in depth and velocity.

Several other refinements may be used to further differentiate sewer hydraulic models, namely in how they handle changes in flow characteristics over the length of the channel, or spatial changes. Differences in the assumptions in the underlying equations may make an important difference in certain situations, while for others the “simpler is better” dictum may prevail. In other words, it is not always true that complex models are always “better,” it depends on the problem to be solved. Common differences in model calculations are demonstrated in Table 2 and Table 3.

3.2.1 Terminology

To simulate the flow of water in sewers, the equations describing the depth and velocity of flow through the collection system must be solved. Each sewer simulation model solves some form of these equations, known formally as the "1-dimensional Saint Venant equations of open channel flow." The St. Venant equations are comprised of two simultaneous equations: a continuity equation that describes the conservation of mass and momentum equation that describes the conservation of energy. While some models solve

the complete equations with all terms included, other models solve simplifications of the equations to facilitate faster run times or to evaluate specific conditions that do not require the full sophistication of the complete equations.

Technical definitions from the field of open channel hydraulics are helpful to better understand the differences between different model solution techniques and to assist in the model selection process. The terms describe how depth and velocity of open channel flow is computed over the length of the channel over the simulation time period.

Temporal Terms:

Steady State: The flow rate is assumed constant in time at any point along the channel. Flow may change along the length of the channel (i.e., a constant tributary flow rate may be added at a point along the channel, or a diversion may reduce the flow by a constant rate at a certain point along the channel). Steady State model assumptions, the formulas used and the software that employ the specified methods are summarized in Table 2.

Table 2 Steady State Assumptions and Models
Collection System Master Plan
King City

	Spatial Assumptions	**Model or Equation**
Uniform Flow	Water surface is parallel to slope of pipe invert.	Manning Equation
Varied Flow	Water surface may vary in depth along the length of the pipe or channel.	SewerCAD

Dynamic: In dynamic models, also known as "unsteady state" models, the flow rate may change over time. There are four primary solution schemes to the St. Venant equations listed below and the dynamic routing methods, assumptions, and modeling software for each are summarized in Table 3:

1. “Kinematic Wave” assumption. The simplest of the dynamic models.
2. “Diffusion Wave” or “Non inertia” assumption. Both acceleration (inertial) terms are ignored.
3. "Quasi-Dynamic” assumption - Four of five momentum terms are used. Only the local acceleration term is dropped.
4. “Dynamic Wave” - All five terms of the momentum equation are used.

Table 3	Dynamic Assumptions and Models Collection System Master Plan King City	
	Assumptions	**Model or Equation**
Kinematic Wave	Inertia and pressure are ignored, only gravitational and frictional forces are considered. The most simplified dynamic model.	SWMM 5 Option in InfoSWMM Option in SewerGEMS Option in PCSWMM
Diffusion Wave	Pressure force term is included along with gravitational and frictional terms. Inertial acceleration terms are not included.	Option in MIKE URBAN
Quasi-Dynamic	One inertial term is included, along with gravity, friction, and pressure. One inertial acceleration term is ignored.	H_2OMAP Sewer/InfoSewer
Dynamic Wave	All five momentum terms are included: gravity, friction, pressure and two inertial acceleration terms. Computationally time-consuming to solve over large sewer networks.	InfoSWMM, InfoWorks, H_2OMAP SWMM, SewerGEMS, PCSWMM, SWMM 5, MIKE URBAN

In general, the complexity of the routing equation increases from Kinematic Wave to Dynamic Wave, with Dynamic Wave representing the full solution to the 1-Dimensional St. Venant equation. Kinematic Wave simplification allows for faster computational run times, but is not ideal for flat pipeline slopes (<0.002 ft/ft) and cannot calculate backwater effects as accurately as the Dynamic Wave routing equation. In addition to lacking adequate abilities to calculate backwater, the Kinematic Wave equations approximate flow attenuation by mathematical approximations, instead of wave propagation.

The Kinematic Wave equation was very popular, but with increases in computing power and improvements in the user interface, the use of the Dynamic Wave routing equations has become a much more user friendly and accessible software option.

3.3 GIS Interface

Municipal and utility operators use GIS software and databases to control, organize, and catalog system data into easy to access and useable formats. GIS compatibility is an essential element of any infrastructure modeling software. The ability to synchronize system databases with modeling software can result in significant time saving for City staff. Software should be able to display GIS data, such as land use, aerial photos, zoning data, parcels, and growth boundaries on the screen in order to allocate flows, and evaluate new facilities based on planning assumptions.

Often, GIS data has topography flaws that need to be corrected before the modeling software can run. Software packages with data diagnostic tools to identify and correct these topology flaws can save time in the model building and updating process.

3.4 Scenario Management

Typically, a planning level hydraulic model serves several purposes. First, the model is used to analyze the existing system and determine where capacity deficiencies and operational problems exist. The second purpose is to evaluate the system under future flows or land use designations. To be used effectively by City staff, the model will need to be able to create and modify multiple scenarios in order to evaluate the effects of infrastructure changes (e.g. new pipelines, pump stations) and increased flows into the collection system. The ability of a model to create and manage what-if scenarios is a necessary component of hydraulic model construction and analysis.

3.5 Customer Service and Support

Operation of a computer model requires a direct relationship with the software vendor in order to troubleshoot any problems that may arise during model operation. Technical service representatives, on-line help, help files, and operating manuals all factor into the customer service and support evaluation. Customer support should be fast, responsive, and technically qualified to handle the most advanced modeling questions. New and infrequent users usually have many questions regarding the operation of modeling software, and a helpful and responsive customer support department can be an invaluable tool.

An evaluation of customer service and support provided by the software vendors is subjective at best, since the evaluation is influenced by the specific personalities and experiences of both parties. Anecdotal information obtained from other software users is subject to biases as well. However, establishing and maintaining a good working relationship with the vendor can be very helpful to maximize the benefits obtained from the software. Maintaining a good personal relationship with the software vendor is probably the most effective way to obtain extra support and software enhancements when needed.

3.6 Cost

The cost of a software package involves several items. With any software package, the associated costs include single license or network license fees, support and maintenance fees, and additional add-on modules. Current software package costs are given in the information matrices (Table 1). The cost of the software should be appropriate for the intended use by the City, and the complexity of the City's collection system.

3.7 Ease of Use

In order for a model to be an effective tool for City staff in planning and development, it must be user friendly and easy to operate. The operating system must be graphically based and intuitive in its operation. Adding facilities to the existing system and creating scenarios for new improvements should be straightforward and intuitive.

3.8 Internet Based Model Network/Output Viewer

In recent years, some of the software vendors have begun to offer internet based applications where the model network and model simulation results can be accessed without having to actually purchase the software license or be trained on how to use the model. This type of functionality is particularly desirable for clients that do not have the available staff resources and/or need to maintain a hydraulic model, but would still like to be able to view hydraulic model output as the need arises.

4.0 SCREENING LEVEL EVALUATION

Carollo conducted a preliminary evaluation based on the hydraulic calculations, cost, and ease of use criteria. Table 1 displays a comparison between the sanitary sewer modeling software's cost and technical characteristics. Considering the criteria described above, SWMM 5, MIKE URBAN, PCSWMM, and InfoWorks are not recommended for the reasons described below.

SWMM 5 is not recommended since the software lacks the enhanced graphical user interface (GUI) and additional features available through the third party vendors. Some examples of this include a lack of GIS compatibility and the inability to create multiple scenarios within a single model. For SWMM 5, a separate model is needed for each scenario that is created, which greatly complicates the model development and hydraulic analysis process.

MIKE URBAN and PCSWMM are not recommended since these software packages are much less intuitive when compared to other software packages and requires a good knowledge and frequent use of hydraulic models to operate and maintain. These packages would not be the most logical choice for the City because the new or infrequent user will struggle with the operation, maintenance, and model updates.

InfoWorks is not recommended due to the excessive cost of the software license. It is also less user friendly for the novice hydraulic modeler.

5.0 COMPREHENSIVE EVALUATION

After the preliminary evaluation was completed, the remaining software packages, SewerCAD and SewerGEMS (Bentley), H_2OMAP Sewer/InfoSewer, H_2OMAP SWMM/InfoSWMM (Innovyze) were further evalutated.

5.1 Dry and Wet Weather Flow Calculations

The remaining software packages have varying capabilities for modeling dry and wet weather flows. Both the Innovyze and Bentley programs provide flexibility for developing dry weather flows from a variety of sources, such as geocoded billing records, population, land use, and other GIS based methods. All of the software packages also have the capability to load user-defined hydrographs into the model.

5.2 Hydraulic Calculations

Two of the remaining software applications, SewerCAD and H_2OMAP Sewer/InfoSewer have less robust computational engines. This significantly limits the software's ability to model complex systems and low slope conditions.

The software applications that feature fully dynamic flow routing capabilities (H_2OMAP SWMM, InfoSWMM, SewerGEMS) are better suited to simulate backwater conditions, and produce the most accurate results possible. In addition to the increased accuracy associated with the fully dynamic computational engine, these applications do not require the development of diversion curves, which can be difficult to generate accurately. The drawback of models that feature fully dynamic flow routing capabilities is increased computational time (longer "run" time), although for a system the size of the City's, computational time should not be a significant issue.

5.3 GIS Interface

Most of the data in a collection system model comes from GIS databases. These information systems are increasingly becoming the primary repository for spatial infrastructure data. Agencies that have had these systems in place have usually been successful in significantly increasing the quality of this data.

Software programs such as H_2OMAP Sewer and H_2OMAP SWMM operate in a stand-alone mode, however these programs can easily read, write, and manipulate GIS data.

InfoSewer and InfoSWMM run within ESRI's ArcGIS software program, so every user of these modeling programs must also have a copy of the GIS software. These modeling programs are able to use additional GIS functionality. The hydraulic calculations are identical to the corresponding H_2OMAP Sewer and H_2OMAP SWMM software. This software tends to be selected by users who have experience and like working from within the GIS software.

SewerCAD and SewerGEMS users can work in an AutoCAD, MicroStation, or GIS environment when doing model creation and setup tasks. Modeling simulations are often performed in a stand-alone mode. However, these two programs tend to open, load data, and run much slower than the Innovyze programs.

5.4 Scenario Management

H_2OMAP Sewer, H_2OMAP SWMM, InfoSewer, InfoSWMM, SewerGEMS, and SewerCAD offer sophisticated parent child tree scenario creation and management schemes. This feature allows the user to set up multiple what if scenarios based on a variety of model parameters.

The Innovyze software packages also have a facility manager, which enables the model to display only the facilities that are modeled in that simulation. The Innovyze data set manager is very useful in organizing and controlling which facilities and controls are associated with each scenario. In SewerCAD and SewerGEMS, all facilities are displayed for all scenarios. Therefore, facilities that are not present in a particular scenario must be turned off manually.

5.5 Customer Service and Support

Innovyze customer support has been good with timely and supportive response to issues, such as software bugs and technical problems. Innovyze has shown that they are responsive to clients' needs and are able to quickly provide enhancements when needed. Instructional manuals are adequate. Help files can be limited, so e-mail and telephone support is the best means of quickly obtaining solutions.

SewerCAD and SewerGEMS offers several support and maintenance options. Users have the option to pay an annual fee or pay a price for each service contract. Anecdotal information obtained from other users was less complimentary on timely responses and personal service.

It should be noted that, in general, the software vendors tend to issue more frequent updates and new features to the fully dynamic hydraulic modeling software, because the trend in the industry is going towards fully dynamic software engines. In future years, this trend is expected to continue, and therefore it is expected that from a customer service perspective, it would be advantageous to select one of the fully dynamic hydraulic modeling software applications (such as H2OMAP SWMM, InfoSWMM, or SewerGEMS).

5.6 Cost

Software costs are a major factor in the selection of a modeling package. Cost discussed here are for a 1,000-pipe version unless otherwise noted. The cost to model collection systems with more pipes will often be higher. The cost for all software packages are summarized in Table 1.

Both InfoSewer and H_2OMAP Sewer have a single license fee of $4,000. Support and maintenance fees cost $800 annually. Network licenses are available, as well as Pro, Suite, and Pro Suite versions of the software for an additional cost.

InfoSWMM and H_2OMAP SWMM are priced at $6,000 for a single license. Support and maintenance fees are $1,200 annually. Network licenses are available, as well as a Suite and Executive Suite version of the software for an additional cost.

The stand-alone version of SewerCAD is priced at $5,452 with an annual support and maintenance fee of $1,316. SewerGEMS is priced at $10,909 with an annual support and maintenance fee of $2,620.

5.7 Ease of Use

The ease of use of each package is an important factor in the software selection.

The user interface for the Innovyze programs has many features that help the user to quickly see and identify associated facility data and controls. The attribute browser allows the user to click on a facility and view or edit information in the database. Another advantage is that output results are viewed in the same window as the model input. This feature is useful for analysis when focusing on specific sections of the system, such as new facilities or system upgrades. The user interface has a control center that displays GIS layer information as well as operational data, annotation, and map display operations that create an easy means to manipulate operational data and view output results for the entire system.

SewerCAD and SewerGEMS have many features that also have the disadvantage of unneeded complexity for the new or infrequent user. One drawback of the SewerCAD and SewerGEMS software is that they use a proprietary database. In doing so, external databases, such as Microsoft Excel, cannot be used to view or edit model data or output results.

5.8 Internet Based Model Network/Output Viewer

Of the software vendors selected for comprehensive evaluation, only the Innovyze software packages offer the ability to export the model network and model analysis results to an internet based viewing application. Innovyze uses its “NetView” add on module to export the model network and output results to a “.kml file,” which can be easily viewed by downloading a free copy of the Google Earth program. NetView is available as part of Innovyze’s Executive Suite license.

Depending on the City’s intended model usage, however, purchase of the Executive Suite license may or may not be required in order to see the benefits of this feature. For example, if the City does not plan to have one or more of its staff member routinely update and/or use the model, it may be more practical for the City to use the exported NetView .kml file (which

would be exported by Carollo) to view model results. If this is the case, the City could opt to purchase the Base package only (if City staff may occasionally want to run a model scenario) or simply not purchase the software at all, while still having the ability to see peak flows and other model output for any given model element.

6.0 RECOMMENDATION

Based on the City's needs, it is recommended that the City select a software application that features fully dynamic flow routing capabilities. The trend in the industry has been towards the fully dynamic models, and therefore the individual software vendors are more invested in providing more frequent updates and enhancements to this type of modeling software as opposed to a steady state or quasi-dynamic model. Of the fully dynamic models that made it past the initial screening, it is recommended that the City select Innovyze's InfoSWMM software application for the following reasons:

- Superior GIS capabilities.
- Superior or comparable ease of use.
- Excellent scenario manager.
- Best value in terms of features/capabilities to cost.
- Available NetView module allows for model network and output results to be viewed for free through Google Earth.
- Responsive customer service.

APPENDIX A – SEWER SYSTEM MODELING WHITE PAPER

SELECTING THE RIGHT MODEL
TO ANALYZE COLLECTION SYSTEM FLOWS AND HYDRAULICS

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ABSTRACT

There are a wide variety of public domain and commercial sewer system models available today. It can be challenging to select a model that meets the goals of a specific project due to the myriad of features now available in most software packages. The purpose of this paper is to provide a guide for selecting a model to analyze flows and hydraulics within a collection system network. A variety of core model features are detailed including dry and wet weather flow estimation, and hydraulics. The computational algorithms that are used in many models, such as the Saint-Venant equations, are also summarized. Other features are discussed that allow for the efficient management of data and the effective display of results. Various model application levels including gross planning, detailed planning, and design are also addressed.

KEY WORDS

Model, Collection System, Flow Estimation, Hydrology, Hydraulics, Infiltration and Inflow, Commercial Software

INTRODUCTION

The purpose of this paper is to provide a guide for selecting a model to analyze flows and hydraulics within a collection system network. There are a wide variety of public domain and commercial models available; some commercial models were developed by private software companies, while others are simply repackaged versions of public domain models and are sold by software companies with value-added support services. Still others are hybrids, they had their genesis as public domain models but subsequently private software companies have made significant additions and improvements. With the many choices on the market today, it can be challenging to select a model that meets the goals of a specific project and budget.

There are a wide variety of features available in most models today, including sophisticated interfaces, database and Geographic Information Systems (GIS) links and several different computational options. This paper cannot hope to cover details of every model, however the fundamental features in most models can be divided into understandable components. This paper will describe these core components of collection system models, delve into the specifics of generating both dry weather flow (DWF) and wet weather flow (WWF), compare the different hydraulic algorithms used to route flows through a pipe network, and include methods to select the right model for an individual system. This paper identifies specific commercial models, but attempts to compare these models based on only the core hydraulic features without taking into account the myriad of other features that may play into the selection of a specific model.

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WHAT IS A MODEL?

A model is a schematic description of a system, theory, or phenomenon that accounts for its known or inferred properties and may be used for further study of its characteristics (dictionary.com). Frequently the most convenient and useful system models are mathematical in nature, and are solved with computer algorithms in software packages. This is what constitutes a "model" for most water resource engineers, and is the working definition for this paper. Every model is a simplification of the real system. The applicability of a given model depends on the degree of simplification that can be applied while still representing the system in a meaningful way to solve a problem. The accuracy and precision required dictates the amount of simplification the model should compromise from reality for a given project (e.g. planning level vs. design level). A model can be as simple as a spreadsheet with calculations for peaking factors of sanitary sewer flows and Manning's equation for calculations of depth and velocity in a single pipe. Or a model can be a complex, fully dynamic model that accurately simulates a sewer network with thousands of basins and pipes in a sophisticated software suite of programs.

Models can be classified in many different ways. Some of the basic classifications are included as applicable to collection system models. This paper deals strictly with sanitary and combined sewer system models, although many of the model traits are also applicable to storm water models. Some common ways of classifying models include:

- *Hydrologic vs. Hydraulic* – a hydrologic model estimates flow quantity over time, while a hydraulic model estimates properties of flow (typically depth and velocity) as it travels through a channel or a network of pipes. For the purposes of this paper, a hydrologic model will also be referred to as a flow estimation model since sewer modeling usually requires estimates of both DWF and WWF.
- *Deterministic vs. Stochastic* – deterministic models produce identical outputs for a given set of inputs while stochastic models produce different outputs for a given set of inputs (Nix, 1994). Stochastic models contain random variables usually defined by a probability density function, are used for probabilistic hydrologic projections, and are sometimes referred to as statistical models. Stochastic models will not be covered in this paper.
- *Lumped vs. Distributed* – these terms are applied to both hydrologic as well as hydraulic models. A lumped hydrologic model assumes that all characteristics are constant over a watershed or sewer basin, while a distributed hydrologic model accounts for spatial variability as a function of position in a watershed (Nix, 1994). In a lumped hydraulic model, the flow is calculated as a function of time alone at a particular location, while in a distributed hydraulic model, routing of flow is calculated as a function of space and time throughout the system (Chow et al, 1988).
- *Static vs. Dynamic* – a static simulation usually refers to a steady-state analysis of a single flow rate (e.g. a peak flow), while a dynamic simulation refers to time-variable flows typically over at least a day's period for sewer related analyses.
- *Single-Event vs. Continuous* – both single-event and continuous refer to a dynamic simulation (i.e. variation in flow characteristics are estimated over time). However for single-

event simulations, one event is analyzed based on some "event definition". This could include an "event" defined as one day of DWF, or a single WWF event based on a minimum dry antecedent period. Continuous simulation refers to an analysis of both wet and dry periods in chronological sequence; the flow response of dry as well as wet periods is simulated. Continuous simulation is commonly used to estimate the probability of peak flows or volumes.

Most modern collection system models are deterministic and contain both hydrologic (or flow estimation) and hydraulic modules. Most models utilize lumped hydrologic processes, while hydraulic calculations can be either lumped or distributed. The majority of models simulate single-event projections, while some are also able to accommodate continuous simulations. For large collection systems, continuous simulation usually requires a more simplified model than single event analyses warrant, in order to reduce model run times.

WHY USE A MODEL?

It is always important to step back and ask if a model is necessary for a specific project. Many times the answer to this question is "yes", even if that model is a spreadsheet with simple calculations. In fact, deciding on the degree of model sophistication required to solve a problem can be most challenging. All to often it is tempting to use the most sophisticated hydraulic model complete with GIS interfaces and a myriad of other features to model every pipe in a collection system. However the cost of this comprehensive modeling is frequently high, and many problems are more aptly, and cost-effectively accommodated using a more simplified model coupled with decision analysis tools (e.g. cost effectiveness models, uncertainty analysis and multiple objective optimization). This is especially true for planning level analyses. Assuming the project warrants analyses beyond what can be done in a spreadsheet, then some reasons to consider more sophisticated modeling software include:

- *Limitations in flow measurement* – Flow measurements should be included in any collection system modeling effort, but there are two major drawbacks to only using flow measurements for analysis of a collection system. First, you can only measure so much. Accurate flow monitoring is expensive, and unless the smallest project is considered, monitoring cannot fully describe the flows and hydraulics in a system. Second, the sewers and flows change over time. For example, service areas grow, pipes get rerouted, and new pipes age and deteriorate thus causing more infiltration and inflow (I/I) to enter the system. All of these factors could cause measured flows to be out of date within a few years.
- *Ability to predict* – one of the main reasons to model a system is to examine "what-if" scenarios once a model is calibrated and accurately predicting results. A model can provide spatial predictions of future DWF (based on changes in land use or population). A model can predict whether an improvement in a section of pipeline will be adequate. Many of today's commercial models allow for scenario management to allow the modeler to test an almost unlimited amount of alternatives.
- *Stakeholder education* – many models today include impressive graphical displays of information in a variety of formats (e.g. graphs, dynamic "movies", maps, etc.). Model output can be directly used to educate stakeholders at all levels within an organization, including board members, managers, engineers, and even operators. It is very useful to verify

model results with field observations from operators. This may also help in the future by demonstrating to operators system performance during unobserved, infrequent events, e.g. where to look for future Sanitary Sewer Overflows (SSOs), effects of major wet weather events on pump station failures, etc.

Assuming a model needs to be used, the real question then is… How detailed should the analysis be to efficiently and accurately solve the problem? To address this issue, the core components of a model will be examined, followed by more detailed descriptions of the flow estimation and hydraulic algorithms of a collection system model.

COMPONENTS OF A MODEL

The basic components of a model include input data, computational algorithms, output data, and a graphical user interface (GUI). Input data are included to describe the physical geometry of the system, define boundary conditions (i.e. at outfalls), and delineate time series that drive the system (e.g. rainfall, flow rates, etc).

The computational algorithms make up the "engine" that drives a model. Unfortunately, like engines in an automobile, model engines can be easily misunderstood and forgotten about when all that is seen is the "shiny exterior." The computational algorithms will be discussed in detail below in the flow estimation and hydraulics sections. Output are the results of the computational algorithms and may include time series (e.g. flow, velocity, depth for given pipes), scatter graphs (i.e. velocity vs. depth), volumes, hydraulic grade lines (HGL's), pipes that surcharge, nodes that overflow, depth to diameter (d/D) ratios of flow in pipes, and other statistics. Model output can be voluminous, so it is important that a model utilize some type of GUI that helps the user manage large quantities of information efficiently.

The purpose of a GUI is to facilitate efficient data input, execute the model runs, and present the results. It is easy to become enamored with all the features in today's models GUI's, and loose sight of what is happening "inside" the model; the computational algorithms that produce the results. Ultimately the decisions that will be made based on model results are inextricably tied to the features that today's model GUI's provide, which makes it is easier than ever to misapply a model to a given project. Typical features that are common to many model GUI's for management of both input and output are:

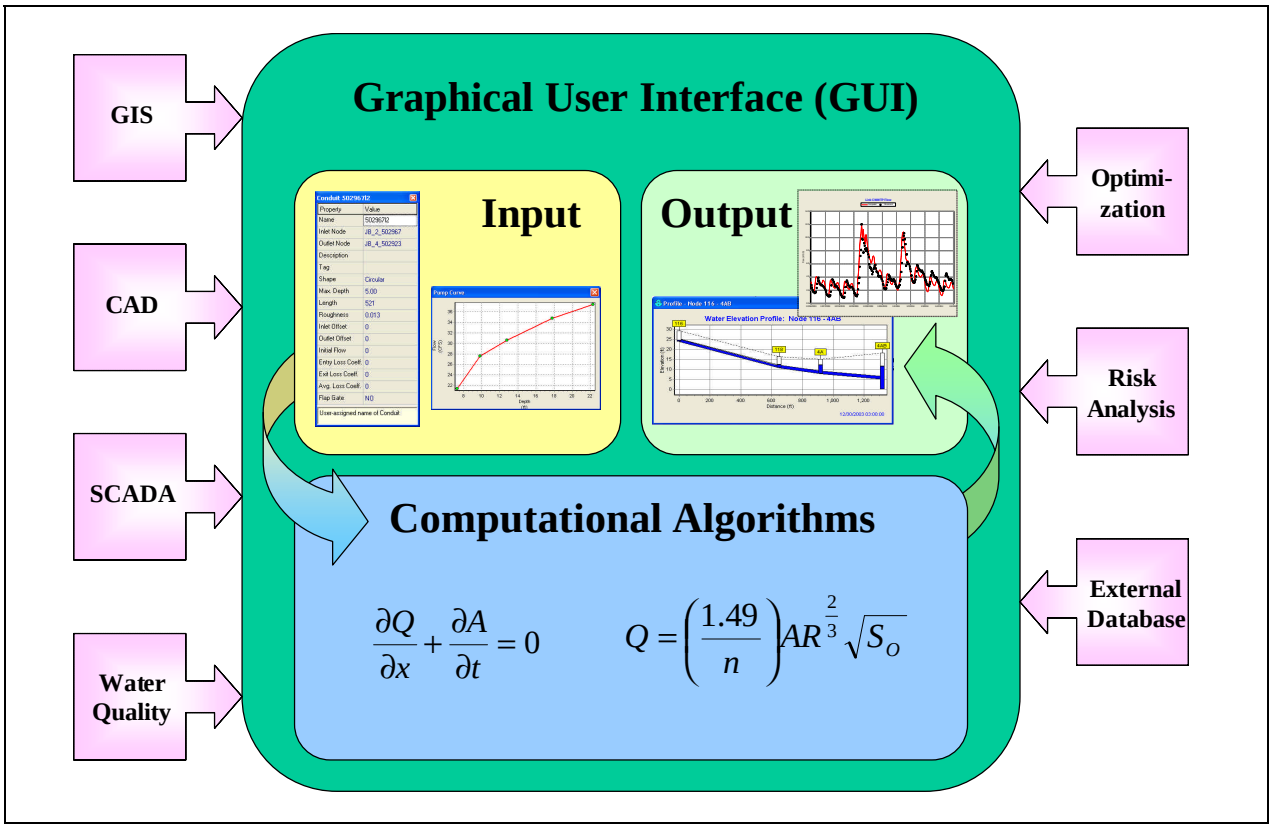
- *Time series database with tabular and graphic capabilities* – used to manage variables (e.g. hydrographs)
- *Spatial database with tabular and mapping capabilities* – used to manage structures and connectivity within the system (e.g. what pipes connect to what junctions)
- *Scenario manager* –manages multiple model runs in a structured format (otherwise model runs need to be managed at a file level by the modeler).
- *Model execution* – allows for starting a run and observing errors.
- *Dynamic displayer* – allows for dynamic display of results in either plan view or profile view. Plan view is typically a map were pipes and nodes change in size or color depending

on certain output variables. A profiler displays a pipeline segment with a varying HGL as it changes throughout the model simulation.

The spatial database feature can either be built into the model, or can be separate Geographic Information System (GIS) software. Many collection system models are moving toward integrating the model with a GIS (e.g. ArcView, ESRI). This provides the advantage of utilizing the graphical and database features of the GIS (without reprogramming them into the model). This structure leverages the many features within GIS and does not require the model to import or export data to a GIS since the data structure is already within the GIS. This feature significantly increases efficiency in inputting data as well as displaying results. The downside of this structure is that the GIS software has to be purchased along with the commercial model.

There are a variety of other model features too numerous to list. However, it's worth noting some linkages that now can be made relatively easily between modeling software and other software to expand the model's capabilities. Certain models now provide linkages to GIS, Computer Aided Design (CAD) software, Supervisory Control And Data Acquisition (SCADA) software, water quality models, optimization and risk analysis software, and external databases. **Figure 1** illustrates a conceptual layout of model components.

Figure 1 – Conceptual Layout of Model Components



FLOW ESTIMATION

Flow estimation is sometimes an overlooked aspect of collection system modeling. However, it is just as important as hydraulics in sizing improvements. If flows are not correctly estimated, the results from the hydraulic calculations will be inappropriate for estimating new or improved pipeline sizes. Flow estimation in a collection system model can be divided into DWF and WWF estimation. DWF is generated by domestic, commercial and industrial wastewater flows throughout a collection system. WWF represents the hydrology of the system and is a factor of the rainfall that enters a collection system either through designed sources, as is the case with combined sewers, or thorough deteriorated facilities or illicit connections, as is the case with sanitary sewers.

Dry Weather Flow

Depending on the complexity of the model, DWF's can either be directly entered into the model or calculated using GIS techniques. A simple approach could consist of calculating an Average Dry Weather Flow (ADWF) for a sewer basin, then applying a peaking factor (typically based on ADWF, sewer basin area, or population) to estimate a peak instantaneous DWF. This single flow value could then be used to calculate the hydraulic capacity required for a pipeline (i.e. steady-state analysis where variability in flow rate is taken into account in the conservativeness of the assumptions underlying ADWF and the peaking factor).

Land use, demographic data (population and employment), and water-use records are typically used to estimate DWF. Population and employment projections may be available as Transportation Analysis Zone (TAZ) data or Census data. Land use information may be available from local planning departments. Water use records may be available from local water agencies (however, water services boundaries may not directly overlap with sewer service boundaries and adjustments must be made). To efficiently utilize this spatial data, especially if it is available at a parcel level, a GIS is a necessity.

Some models, especially those that are integrated with GIS, have the capability of converting this information into Average Dry Weather Flow (ADWF) buy applying unit flow factors (e.g. gallons per capita per day, or gallons per acre per day) directly in the model software. This feature can save significant time and effort especially if multiple years need to be analyzed (e.g. current, buildout, etc.). If a model does not have this capability, the calculations can be done externally in a GIS or a spreadsheet, and then imported into the model. If a dynamic model is used, the ADWF then needs to be converted to a diurnal pattern by applying percentages for each time unit (e.g. every hour). Certain models can also use more than one diurnal pattern to account for day-today changes in the diurnal pattern (e.g. weekday vs. weekend, month by month variations, etc.). This will facilitate a more accurate simulation of DWF patterns over an extended period of time.

DWF's are usually calibrated by adjusting unit flow factors to match modeled volumes to measured volumes and adjusting the diurnal pattern to match the corresponding DWF model hydrograph (e.g. timing, shape, peak flow and minimum flow). Options exist to compare DWF hydrographs graphically, along with goodness-of-fit statistics (e.g. comparison of minimums,

maximums, averages, R^2, etc.). Some modeling software packages calculate and graph these statistics directly. If this feature doesn't exist, then it's the responsibility of the modeler to complete these calculations externally. Automated methods for performing the DWF calibration process are not typically available as a built-in feature in most modeling software. However external optimization software can be used to perform the calibration. In cases where collection systems experience very little I/I, calibration of DWF along with the inclusion of some base I/I may be all that is needed for estimation. However, many systems suffer from significant I/I and require explicit modeling of WWF's.

Wet Weather Flow

Combined sewer systems (CSS's) are designed to accommodate WWF's up to the point of discharging out designed overflow points within the system – or Combined Sewer Overflows (CSO's). In this case WWF may include I/I as well as direct, purposeful stormwater connections. I/I is usually defined as extraneous flows that are not desirable in a sanitary collection system, while CSSs were originally designed to accommodate direct storm water drainage. However, the distinction is blurred, as many sanitary sewers also suffer from excessive I/I that result in SSOs that behave similarly to CSSs. In fact, some coastal systems have experienced wet weather peaking factors (peak wet weather flow divided by average dry weather flow) that approach the peaking factor of some CSS's. Therefore, it is extremely important that WWF be estimated as accurately as possible for analyzing and designing improvements for these types of systems.

I/I is a hydrologic component of both combined and separate sanitary systems. Combined systems are generally dominated by storm water that is in effect the same as sanitary inflow. Separate systems can be dominated by either inflow or infiltration. When portions of a combined sewer system are separated, the storm water contribution is greatly diminished, but infiltration may still be a significant component of flow, and may in turn exhibit characteristics of a sanitary system with significant I/I. In either case (a partially separated combined system or a sanitary system with significant I/I), infiltration can be the most difficult portion of the wet weather hydrograph to model, as well as one of the more challenging components of WWF to remove.

Modeling inflow is usually straightforward because the inflow response is a direct result of the rainfall pattern and the amount of impervious area that is tributary to the sewer system (it behaves very similarly to an urban storm water response to rainfall). Most collection system models adequately model inflow. However, infiltration is much more difficult to model than inflow since it can be an indirect response to rainfall and can be heavily influenced by groundwater conditions, which nearly always introduce a tremendous amount of uncertainty into any hydrologic analysis. Infiltration generally can be categorized in two ways:

- Near surface infiltration
- Groundwater infiltration

Near surface infiltration usually starts a few hours after the beginning a rain event and subsides within generally a day or two as the saturated soils return to normal. Groundwater infiltration is caused by groundwater that raises due to saturated soil conditions to a point where the groundwater table elevation exceeds that of the local collection system. This type of infiltration

can be an issue for coastal communities or those located near lakes and rivers were the groundwater table is normally very close to sewer inverts. Groundwater infiltration will not necessarily influence a sewer system until multiple rainfall events saturate the soils and start to increase the groundwater table, but when it does, it can contribute infiltration that last for weeks or months. Also, certain systems directly adjacent to the ocean or estuaries can experience diurnal groundwater infiltration due to tidal fluctuations that directly affect the water table.

Modeling of WWF generally falls into these categories (Wright and Dent, 2001):

- Rational Method - calculation of an individual peak flow or volume using the simplistic equation Q=CiA, or R-values (which are calculated the same way, but "C" is replaced by an "R")
- Unit Hydrograph – includes SCS curve number methods, the RTK method (triple unit triangular hydrographs), and regression analysis (linear or non-linear)
- Physically based - Non-linear reservoir, multiple non-linear reservoirs (when groundwater is included in the model)

The Rational Method should only be used in the case of calculating inflow for small, highly impervious areas. R-values are easy to calculate but can be applied beyond their accurate limits, and should only be used as a first check to see if sanitary sewer basins suffer from significant I/I. To adequately model wet weather flow hydrographs, a unit hydrograph or physically based method should be applied. These methods are more complex and take more time and experience to apply properly. If groundwater is a significant issue, and if there is the possibility that storage facilities are going to be sized to manage excess I/I, a physically based multiple non-linear reservoir method should be applied. For a more in-depth discussion on modeling wet weather flows, refer to Dent et al (2000) and Wright et al (2000).

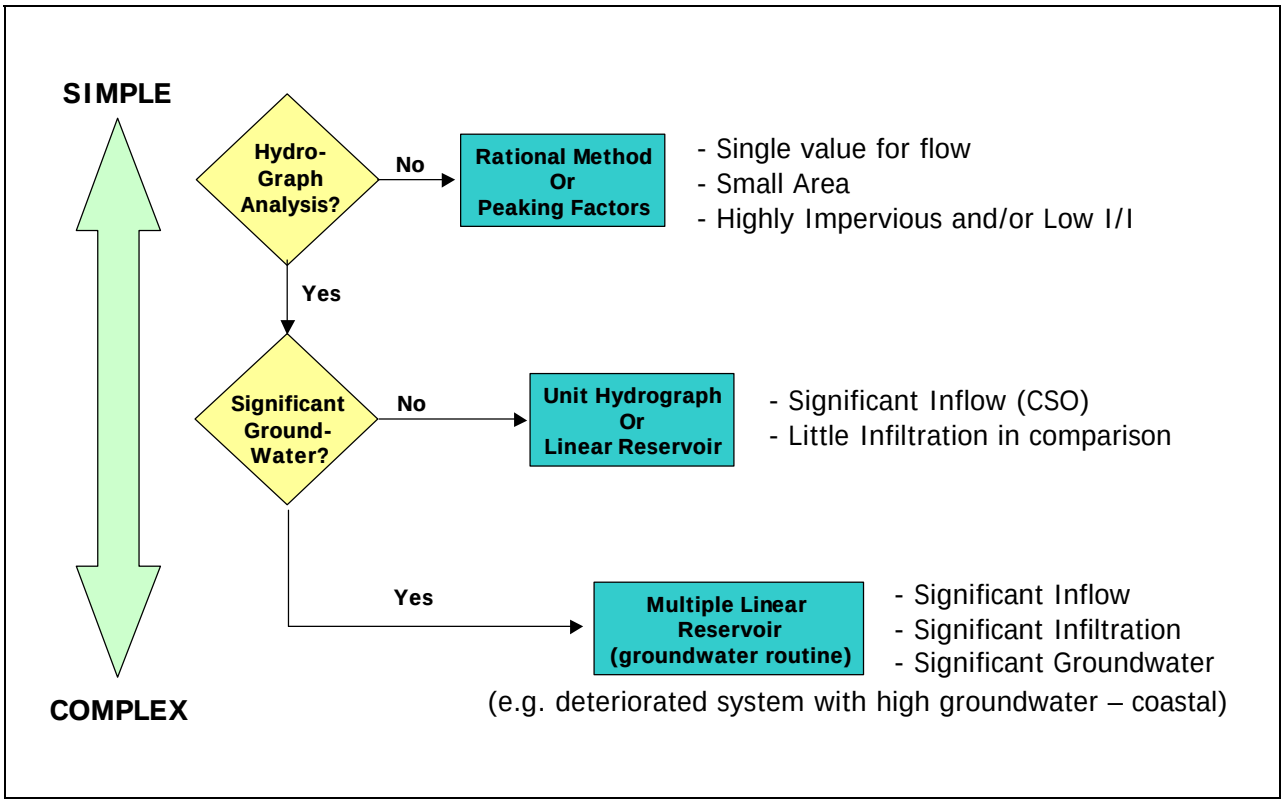
There are usually a variety of wet weather flow estimation options available in most public domain and commercial models, so no specific model or models will be discussed since the topic is too extensive for this paper. However, it is worth noting that automated calibration of WWF's is beginning to be explored in certain commercial collection system models. There also exists the opportunity to link external optimization software to efficiently run iterative calculations for WWF calibration in public domain models (Dent et al, 2004). **Figure 2** illustrates a flowchart to help chose a wet weather flow estimation method.

HYDRAULICS

Hydraulics, and the application of hydraulics, are often misunderstood. Many books are available that explain hydraulic theory and application, but few if any exist that provide a consolidated summary of hydraulic modeling for practical purposes. This explanation of hydraulics hopefully provides this consolidated review.

Due to the wide variety of flow conditions that may exist in an open channel, a complete but sometimes confusing nomenclature has developed to describe the fundamental properties of fluid flow. Yen (2001) provides a simple breakdown of these terms for overland and open-channel flow in terms of variation with time and space.

Figure 2 – Flowchart for Choosing a Wet Weather Flow Estimation Method



Flow variation with time is commonly referred to as either:

- Steady (time invariant), or
- Unsteady (dynamic flow)

A steady state model only uses a single flow within the analysis. A peak flow is a common way to analyze the capacity of a pipe. An unsteady, or dynamic flow model, uses a time series of flows. A hydrograph is commonly used in a dynamic model and can range from several values (e.g. diurnal DWF's over 24 hours) to millions of flow values (e.g. long term continuous simulation). Both steady and unsteady flow will be discussed below as they relate to flow variation in space.

Flow variation in space, or flow along the length of a channel, is referred to as either:

- Uniform, or
- Nonuniform (gradually or rapidly varied)

Uniform flow is the simplest type of open channel flow and the governing equations give unique flow rates as a function of depth. One simple way to envision this type of flow is a pipeline

where the slope of the hydraulic grade line (HGL) is parallel to the slope of the pipe. Under this regime the two forces acting on the flow, gravity and friction, are balanced, and there is no acceleration of the flow in any dimension. Under uniform flow the friction slope is equal to the pipe bed slope (i.e. $S_f = S_0$). By definition, uniform flow must exhibit uniform depth along the length of a pipeline. This depth is commonly referred to as normal depth (Chaudry, 1993).

Steady Flow (Uniform and Nonuniform)

Steady flow models, those where only a single flow is used, are solved for depth and velocity using uniform or nonuniform techniques. When considering flow in time and space, the simplest type of flow is steady uniform flow. The flow rate does not change with time, the depth of flow does not change in space, and the streamlines are all parallel. All acceleration terms are zero, e.g. friction and gravity are in perfect balance. Manning's equation (developed by Robert Manning in 1891) is frequently used to determine the flow rate or velocity for uniform flow because under uniform flow conditions the depth of flow and flow rate are related by a unique rating curve (i.e. the flow rate is a unique function of depth). **Figure 3** illustrates Manning's equation expressed in customary US units.

Figure 3 – Manning's Equation

$$Q = \left(\frac{1.49}{n}\right) A R^{\frac{2}{3}} \sqrt{S_O}$$

where: Q = Flow (cfs)
n = Manning's roughness coefficient (unitless)
A = Area (ft^2)
R = Hydraulic Radius
S_o = Pipe Slope

Manning's equation is an empirical equation (i.e. derived from field and laboratory experimental observations) and is appropriate if the conditions mentioned above for uniform flow exist in the pipeline that is being analyzed. If flow in a sewer pipe approaches a steady uniform condition, then Manning's equation may give reasonable results. However, actual sewer pipelines frequently experience a variety of flow conditions where the uniform flow principle is violated and the more complete nonuniform solution techniques are required.

Nonuniform conditions take into account that a single flow will usually exhibit different depths along a length of pipeline. Nonuniform flow is typically categorized as gradually varied and rapidly varied flow. Rapidly varied flow will not be covered in this paper since sewer flows rarely exhibit this flow condition, and no models covered in this paper solve for this condition. Therefore, from now on, this paper will refer to nonuniform flow only in terms of gradually varied flow.

Gradually varied flow can be either steady or unsteady in time. Steady, gradually varied flow, requires calculations beyond Manning's equation to solve for depth and velocity in a open channel (unsteady gradually varied flow will be addressed in detail later in this paper). The Step Method has been developed to solve the gradually varied flow equation. Numerical integration is necessary because the equations are not generally explicitly soluble (Henderson, 1966). Two common solutions of this method include the Standard and Direct Step Method. These methods can be used to solve for changing depths and velocities for a given flow along the length of an open channel (or pipeline if the flow does not surcharge beyond the crown of the pipe). Two commercial models that utilize the Direct Step Method include SewerCAD and StormCAD both by Haestad Solutions (Bentley Systems).

Unsteady Gradually Varied Flow

The most computationally complex type of one-dimensional open channel flow is unsteady flow, which is, for practical purposes, also nonuniform (Chaudry 1993). Unsteady gradually varied flow can be solved using distributed flow routing models where flow rate and water level are computed as functions of space and time, rather than space alone (such as the Standard Step Method discussed above) or time alone (such as spatially lumped models which are not covered in this paper) – (Chow et al, 1988). Most of the unsteady sewer models on the market today utilize distributed flow routing and solve a form of the Saint-Venant equations. These equations are covered in detail in many publications including, but not limited to, Chow et al (1988), Bedient and Huber (1992), Chaudry (1993), Yen (2001), and Henderson (1966).

The Saint-Venant equations, first developed by Barre de Saint-Venant in 1871, describe one-dimensional unsteady open channel flow where depth and velocity only vary in the longitudinal direction of the channel, the bottom slope of the channel is small, and Manning's equation can be used to describe resistance effects (Chow et al, 1988). So why are these equations so important in distributed flow routing models? One primary reason why these equations are needed are because a wave (a variation in energy in time and space) can are propagated upstream and downstream in unsteady gradually varied flow. Waves cannot be accounted for in the simplified forms of the St. Venant equations that do not consider changes in acceleration over time and space. The Saint-Venant equations generally consist of equations of continuity and momentum and are able to estimate wave celerity (the velocity of a wave along the channel). These partial differential equations are summarized in **Figure 4** (adapted from Chow et al, 1988).

The kinematic wave solution, as shown in Figure 4, balances the gravity and frictional forces and assumes the flow does not accelerate appreciably. The diffusion wave solution is the kinematic solution with the pressure term included which takes into account the change in water pressure with depth along the channel. The Quasi-Dynamic wave solution is the diffusion wave solution with the inclusion of the convective acceleration term which describes the change in momentum due to the change in velocity along the channel. The Fully Dynamic solution includes all terms in the momentum equation including the local acceleration term, which describes the change in momentum due to the change in velocity over time.

Figure 4 – Saint-Venant Equations

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

Momentum Equation:

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + g\frac{\partial y}{\partial x} - g\left(S_o - S_f\right) = 0$$

Local Acceleration	Convective Acceleration	Pressure Force	Gravity & Friction Force

Kinematic Wave

Diffusion Wave

Quasi-Dynamic Wave

Fully Dynamic Wave

where:
Q = Flow
A = Area
t = Time
x = Distance
y = Depth
S_o = Pipe Slope
S_f = Friction Slope (slope of hydraulic grade line)

The Saint Venant equations are partial differential equations that describe one dimensional, unstready, gradually varied flow. Unfortunately they are not amenable to analytical solution methods and therefore must be solved using numerical approximations. A common method for approximating the solution of these equations, although not covered in detail here, is the finite-difference method. This method can employ either an explicit scheme (where unknown values are solved for sequentially) or implicit scheme (where unknowns are solved for simultaneously). The explicit method is iterative and somewhat simpler to implement in a computer program, but can produce unstable results. The implicit method is more complicated from a programming perspective, but generally provides a more stable solution and can provide faster simulation times than the explicit method. Explicit solutions were applied in the past, but implicit methods are more commonly used in today's distributed flow models.

Another mathematical solution issue, which may seem somewhat esoteric, but is necessary to understand because of how certain sewer models are marketed, include the way the equations are

applied to solve for surcharge flow (when flows exceed the crown of a pipe and transition into pressure flow). There are two ways to simulate unsteady surcharged flow in sewer models and include the standard transient flow approach and the hypothetical piezometric open slot approach. The hypothetical piezometric open slot approach, is also known as the Priessmann slot approach. According to Yen (2001), both solution techniques have their own pros and cons and it is not clear whether one method is superior in all cases, regardless of how some commercial software is marketed.

Generally, the more terms of the Saint-Venant equations that are included in the solution, the more computational intensive the model can become and the slower the simulation times become. Excessive run times can still be a problem (even with the computational power of today's computers) if the sewer network becomes large enough. Therefore, some models standard solution routine applies the kinematic wave solution, such as the Hydra software by Pizer, or provides this solution technique as an option, as is the case with the US EPA Storm Water Management Model - SWMM5. This simplification of the Saint-Venant equations will provide very efficient simulation run times. However, the kinematic wave routine does have some significant limitations in that flow is routed only in a downstream direction, only open channel flow (or gravity flow as it is sometimes called) can be simulated (no surcharge), and flow in looped pipes (e.g. cross connections) cannot be directly calculated.

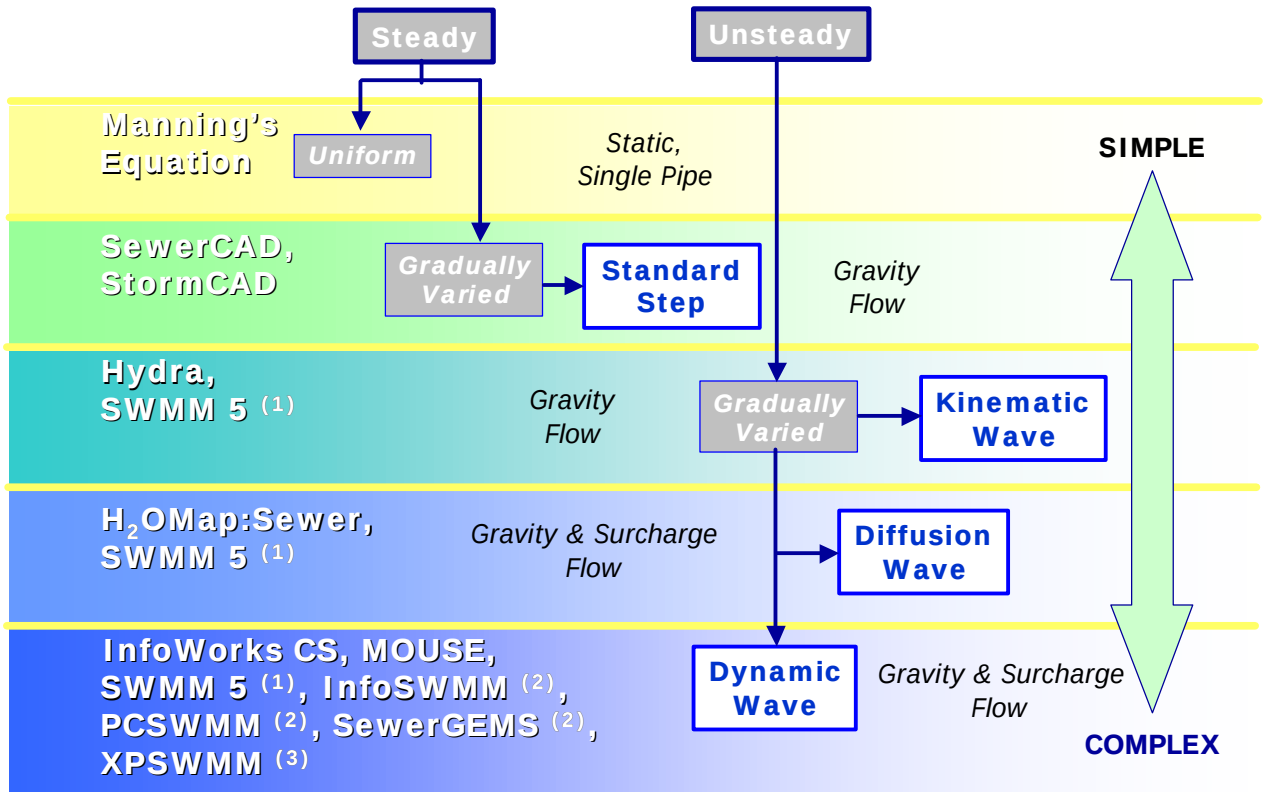
Many distributed flow routing models on the market today include fully dynamic wave solutions as well as options to model only quasi-dynamic or diffusion wave solutions for sewer networks. H2Omap:Sewer (MWH Soft) includes a diffusion wave solution routine. Other models such as SWMM5, MOUSE (Danish Hydraulic Institute – DHI), InfoWorks CS (Wallingford Software), and XPSWMM (XP Software) include fully dynamic wave solutions and may offer options for diffusion and quasi-dynamic wave solutions. PCSWMM (Computational Hydraulic Institute – CHI), InfoSWMM (MWH Soft), and SewerGEMS (Haestad Methods/Bentley Solutions) utilize the SWMM5 engine but provide a more comprehensive GUI than the standard SWMM5 package. **Figure 5** provides a graphical characterization of hydraulic solution methods along with a summary of which models apply which routines.

OTHER MODEL FEATURES

What about the other features that are available in today's public domain or commercial models? The core features of today's models are described above, but it is also necessary to investigate other model features that may influence the choice of selecting one model over another. The following list is by no means comprehensive, but hopefully provides some features that should be investigated before an individual model is chosen or upgraded:

- Who will be the model users (consultant vs. client, full-time vs. part time, experienced with hydrology and hydraulics)
- Cost (both initial purchase as well as yearly service contracts for technical support and upgrades)

Figure 5 – Characterization of Hydraulic Solution Methods



Notes:

(1) SWMM 5 has four solution routine options, which include kinematic wave, diffusion wave, quasi-dynamic wave, and fully dynamic wave. The dynamic wave section of this graphic encompasses both the quasi and fully dynamic wave solutions.

(2) PCSWMM, InfoSWMM and SewerGEMS utilize the SWMM 5 engine.

(3) XPSWMM utilizes a modified SWMM engine.

- Database and Engine structure (ability to work with 3rd party software, ability to easily extract model input/output from model)
- Estimate DWF from population or landuse
- Utilize monitored data for calibration
- Perform automated calibration
- Efficiently complete continuous simulation modeling
- Perform automated pipe sizing (upgrades or parallel pipelines under open channel and surcharge conditions)
- Water quality (i.e. hydrogen sulfide) and sediment transport modeling
- Simulate Real Time Controls (RTC) at pumps, weirs, orifices, etc.
- Link with SCADA software to provide real time modeling capabilities

Many of the options mentioned in the above list can be found in some of the models listed previously in this paper. However, two features that would be extremely helpful in sewer system model, but aren't available (to the authors knowledge) include:

(1) Ability to directly distribute WWF's into unmonitored areas of the system

(2) Ability to analyze capacity of sewer pipes based on a comparison of the HGL to the slope of the sewer pipe

Feature (1) addresses the need to provide a modeler with a mechanism to distribute WWF's throughout sewer basins even when basins are not monitored for flow (which is typically the case). Statistical methods do exist (see Wright et al, 2005) but could be included as an option in software packages.

Feature (2) is related to automated pipe sizing, but seems to be feature that has been overlooked. If a model could illustrate, during a dynamic simulation, which pipes in a network have an HGL equal to, greater than, or less than the slope of the pipe, the modeler would be able to easily identify which pipes need to be upgraded. This is especially true when pipes become surcharged. For example, if a pipe has an HGL slope greater than the slope of the pipe, and portion of the pipe is surcharged, then that pipe has a capacity restriction for the modeled flow and needs be upgraded (e.g. larger diameter or parallel pipeline). Likewise, if a pipe has an HGL slope less than the slope of the pipe, and the pipe is surcharged, the pipe does not represent a restriction (and the restriction is somewhere downstream). This relatively simple feature would provide the modeler with invaluable information to identify deficiencies within a sewer network and help focus capacity improvements.

MODEL APPLICATION

Selection of any model always comes back to what type of project the model is being used for, and what accuracy is needed in the results. Three examples, or levels of sewer system modeling include:

- Gross Planning Level
- Detailed Planning Level
- Design Level

Gross planning level modeling can be accomplished in several ways. One example of gross planning level modeling is the use of hydrologic routines to simulate the overall WWF response of a sewer system. For example, if a municipality wants to identify the I/I response of a sewer system upstream of some critical point (e.g. a pump station or wastewater treatment plant), a model could be constructed by simplifying the entire sewer network upstream of that point into one or more sewer basins. Measured WWF can then be used to calibrate the models I/I response and design flow events can be projected (either using design storms or continuous simulation techniques). This simplified technique, although it does not provide information on the detailed hydraulics of the upstream sewer system, can provide invaluable information regarding the

management of WWF at a pump station or wastewater treatment plant. If a sophisticated model is used to perform this simplified effort (such as SWMM), the model can always be expanded at a later time to include the detailed upstream sewer hydraulics.

Detailed planning level modeling would be considered for existing systems when the flows (both DWF and WWF) are estimated throughout a sewer network, and the flows are routed through the sewer network to a downstream point. This type of modeling is typically performed during master planning projects. In selecting a model for detailed planning level investigations, it is necessary to not only identify the core hydrologic and hydraulic features of a model, but also all the options that might be necessary to provide an efficient analysis of a complex system (see the Other Model Features above). When selecting a model based on core features, the model should simulate what the system is experiencing (i.e. surcharge, looped connections, complex pump stations and forcemains, etc). Also, many times the optional features tend to be the discriminating factors in selecting a model when the core features are very similar (e.g. PCSWMM, InfoSWMM, and SewerGEMS – which all have the same SWMM5 engine, but different GUI's).

Design level models are used to design new facilities, but may not need to be as sophisticated as models used for detailed planning. If new pipes are being designed that do not surcharge, then less sophisticated hydraulic routines may be adequate (e.g. models that utilize the Step Method or the Kinematic wave method). These models are very useful for designing sewers for new developments or even completing detailed planning level modeling if an existing system is not hydraulically complex and any surcharging will be relieved with upsized or paralleled pipelines. However, the results from these types of models are not typically satisfactory for combined sewer systems or sanitary sewer systems that experience extensive I/I (i.e. if surcharging could occur).

CONCLUSIONS

Too simple of a model does not adequately represent reality and can lead to inappropriate decisions. Too complex of a model may be difficult to use (including interpretation of results) and my lead to abandonment. Modeling software packages have a variety of strengths and weaknesses. Each software company has developed its own software package based on its vision, experience, and client base. Therefore, a software package may be stronger for some applications than for others. A model should only be used to assist in informed decision-making. Since all models include some simplification of the actual system, the engineer or operator should make the final decision, not the model.

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APPENDIX C – DRY WEATHER FLOW CALIBRATION

TABLE 1 DRY WEATHER FLOW CALIBRATION
WASTEWATER COLLECTION SYSTEM MASTER PLAN
KING CITY

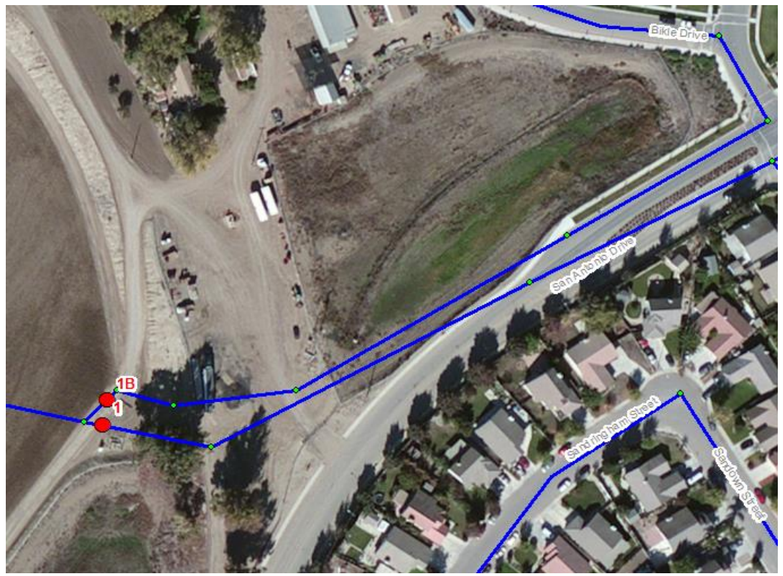
		Weekday									Weekend									Overall ADWF		
		Measured Data[1]			Modeled Data[2]			Percent Error[3]			Measured Data[1]			Modeled Data[2]			Percent Error[3]					
Meter Number	Pipe Diameter (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	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 (%)
1	18	0.11	1.47	2.1	0.119	1.58	2.0	3.9%	7.6%	-5.4%	0.12	1.48	2.2	0.13	1.60	2.0	2.9%	7.8%	-5.1%	0.12	0.12	3.6%
1B	9.625	0.02	0.67	1.2	0.018	0.69	1.2	0.3%	2.6%	-1.8%	0.02	0.69	1.3	0.02	0.70	1.2	-0.4%	2.3%	-1.4%	0.02	0.02	0.1%
2	30	0.56	1.69	4.7	0.513	1.87	4.3	-8.4%	10.2%	-7.9%	0.60	1.72	4.8	0.54	1.89	4.4	-9.6%	9.9%	-8.4%	0.57	0.52	-8.7%
3A	11.5	0.06	1.20	1.9	0.059	1.08	1.9	-4.5%	-10.0%	3.4%	0.06	1.17	1.9	0.06	1.05	2.0	-1.7%	-9.8%	4.0%	0.06	0.06	-3.7%
3B	11.5	0.04	0.41	2.9	0.039	0.42	2.9	-1.6%	1.3%	-2.0%	0.04	0.45	2.9	0.04	0.44	3.0	-1.4%	-2.7%	0.3%	0.04	0.04	-1.6%
4	18	0.08	2.28	1.2	0.078	2.30	1.2	0.4%	0.7%	0.0%	0.07	2.32	1.1	0.07	2.26	1.1	0.6%	-2.8%	3.0%	0.08	0.08	0.5%
5	16	0.11	0.75	3.5	0.107	0.72	3.5	0.3%	-3.9%	1.6%	0.11	0.73	3.4	0.11	0.71	3.5	-0.5%	-3.5%	0.5%	0.11	0.11	0.1%
6	19	0.09	0.77	2.8	0.093	0.79	2.7	-0.4%	2.5%	-1.5%	0.10	0.78	2.8	0.10	0.79	2.8	0.1%	2.1%	-1.1%	0.09	0.09	-0.2%
7	18	0.06	0.87	1.8	0.061	0.85	1.9	3.7%	-2.0%	6.1%	0.05	0.79	1.7	0.05	0.80	1.7	3.2%	1.4%	1.8%	0.06	0.06	3.6%
8	14	0.01	0.34	1.1	0.010	0.33	1.2	-1.8%	-3.1%	9.7%	0.00	0.21	0.8	0.00	0.23	0.8	-1.3%	8.7%	-5.8%	0.01	0.01	-1.7%

Notes:
1. Source: King City 2016 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.

FLOW MONITORING SITE 1 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: San Antonio Drive (west of North Mildred Avenue)
Pipeline diameter: 18"
City Manhole ID: N12
Model Mahhole ID: N12
Silt Level at Site: none"

Flow Monitor Location

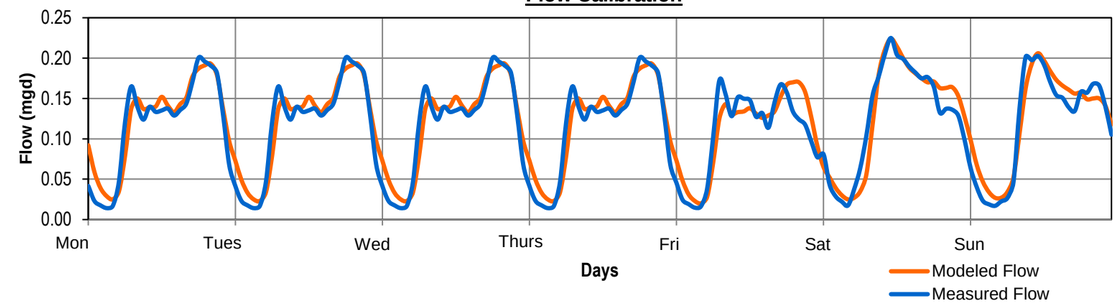


Model Calibration Summary

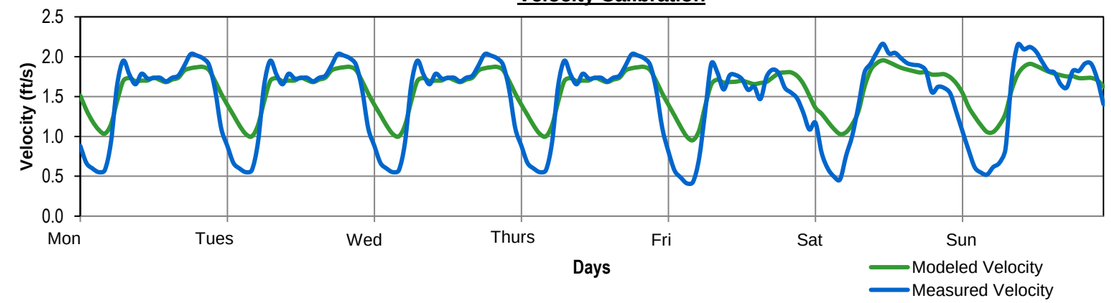
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.12	0.20	2.1	1.50	0.12	0.19	2.02	1.59	3.9%	-3.7%	-5.6%	6.1%
Tues.	0.12	0.20	2.1	1.50	0.12	0.19	2.02	1.59	3.9%	-3.7%	-5.6%	6.1%
Wed.	0.12	0.20	2.1	1.50	0.12	0.19	2.02	1.59	3.9%	-3.7%	-5.6%	6.1%
Thur.	0.12	0.20	2.1	1.50	0.12	0.19	2.02	1.59	3.9%	-3.7%	-5.6%	6.1%
Fri.	0.11	0.17	2.0	1.35	0.11	0.17	1.94	1.54	3.7%	-1.4%	-4.6%	14.4%
Sat.	0.13	0.22	2.2	1.49	0.13	0.22	2.07	1.60	1.5%	-0.4%	-4.8%	7.4%
Sun.	0.12	0.20	2.1	1.47	0.12	0.21	2.03	1.59	4.4%	1.7%	-5.5%	8.2%
Summary												
Weekday	0.11	--	2.1	1.47	0.12	--	2.0	1.58	3.9%	--	-5.4%	7.6%
Weekend	0.12	--	2.2	1.48	0.13	--	2.0	1.60	2.9%	--	-5.1%	7.8%
ADWF[4]	0.12	--	2.1	1.47	0.12	--	2.0	1.58	3.6%	--	-5.3%	7.7%

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

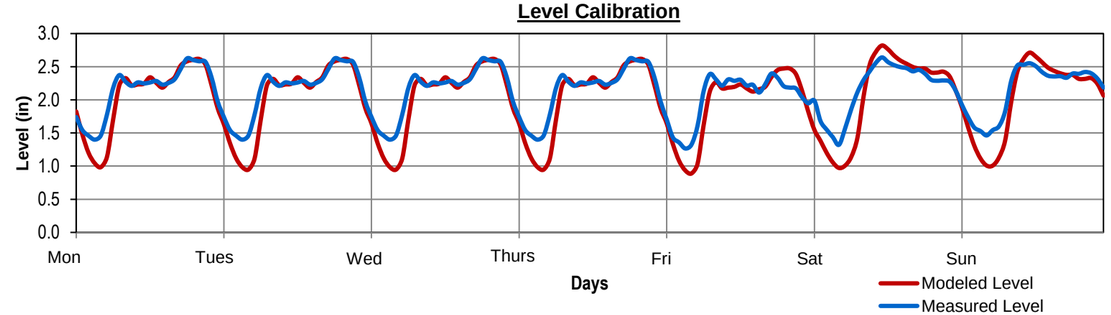
Flow Calibration



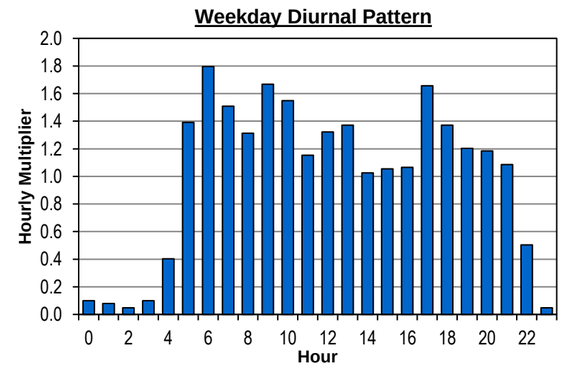
Velocity Calibration



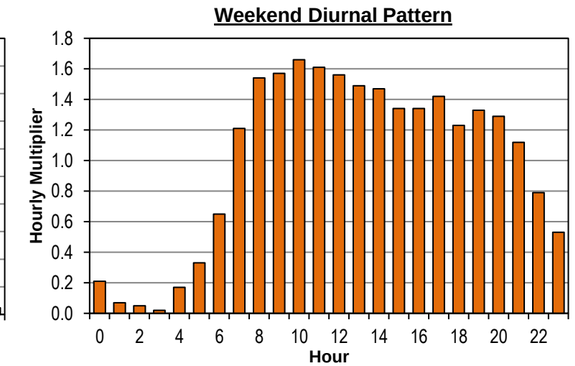
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



FLOW MONITORING SITE 1B DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: San Antonio Drive and North Mildred Avenue
Pipeline diameter: 9.625"
City Manhole ID: MH300
Model Mahhole ID: MH300
Silt Level at Site: none"

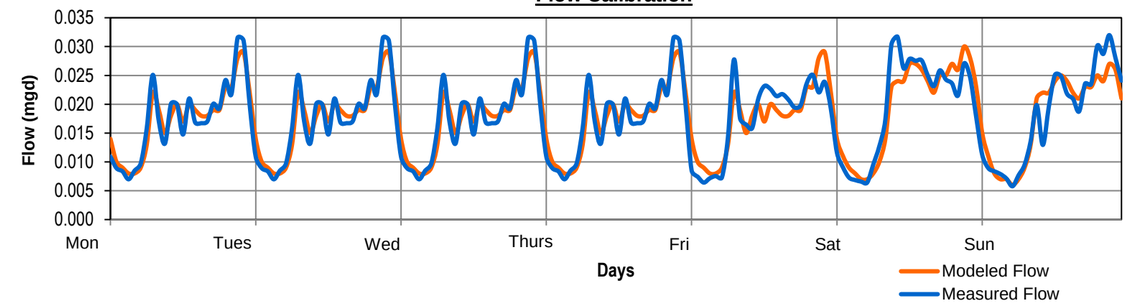
Flow Monitor Location

Model Calibration Summary

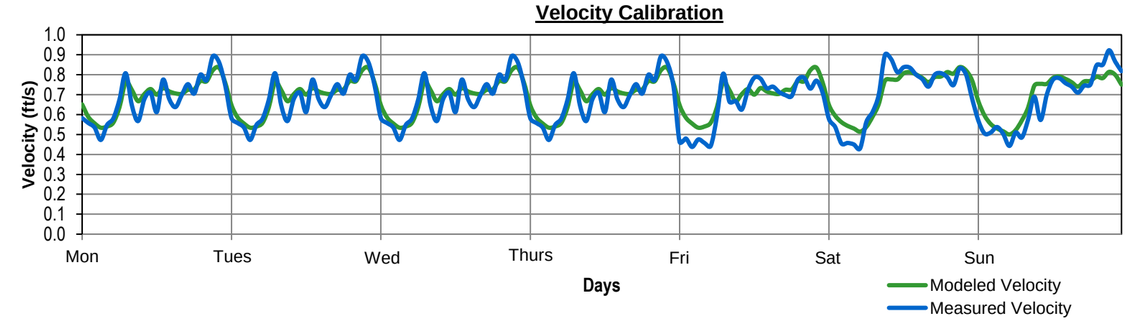
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.02	0.03	1.2	0.68	0.02	0.03	1.21	0.69	0.3%	-7.9%	-1.5%	1.9%
Tues.	0.02	0.03	1.2	0.68	0.02	0.03	1.21	0.69	0.3%	-7.9%	-1.5%	1.9%
Wed.	0.02	0.03	1.2	0.68	0.02	0.03	1.21	0.69	0.3%	-7.9%	-1.5%	1.9%
Thur.	0.02	0.03	1.2	0.68	0.02	0.03	1.21	0.69	0.3%	-7.9%	-1.5%	1.9%
Fri.	0.02	0.03	1.3	0.66	0.02	0.03	1.21	0.69	0.0%	4.5%	-3.1%	5.4%
Sat.	0.02	0.03	1.3	0.70	0.02	0.03	1.26	0.71	-0.6%	-5.4%	-0.8%	1.6%
Sun.	0.02	0.03	1.2	0.67	0.02	0.03	1.22	0.69	-0.2%	-15.6%	-2.1%	3.1%
Summary												
Weekday	0.02	--	1.2	0.67	0.02	--	1.2	0.69	0.3%	--	-1.8%	2.6%
Weekend	0.02	--	1.3	0.69	0.02	--	1.2	0.70	-0.4%	--	-1.4%	2.3%
ADWF[4]	0.02	--	1.2	0.68	0.02	--	1.2	0.69	0.1%	--	-1.7%	2.5%

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

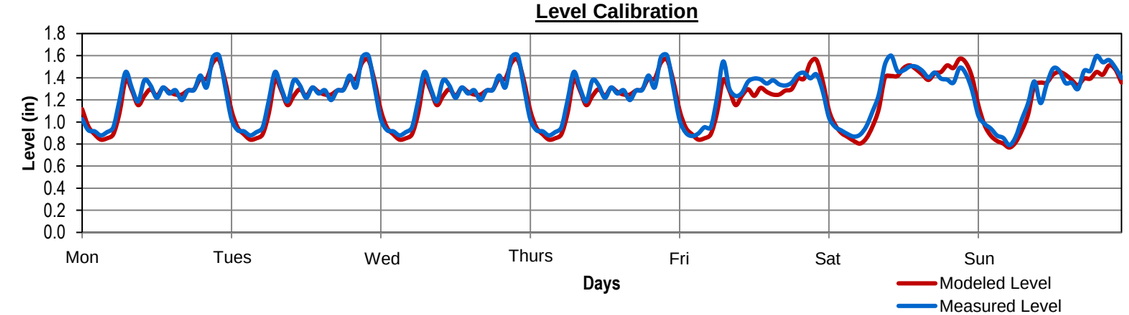
Flow Calibration



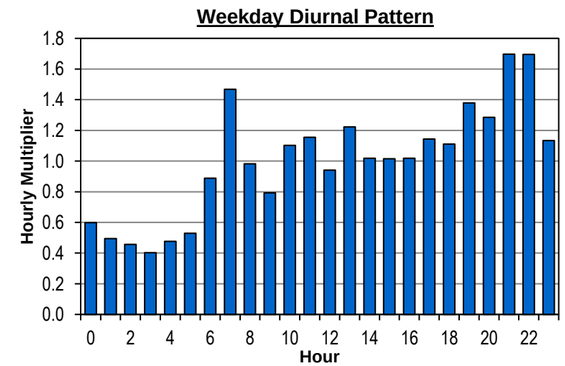
Velocity Calibration



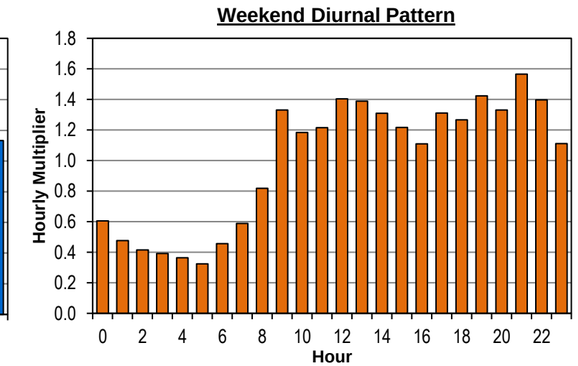
Level Calibration



Weekday Diurnal Pattern



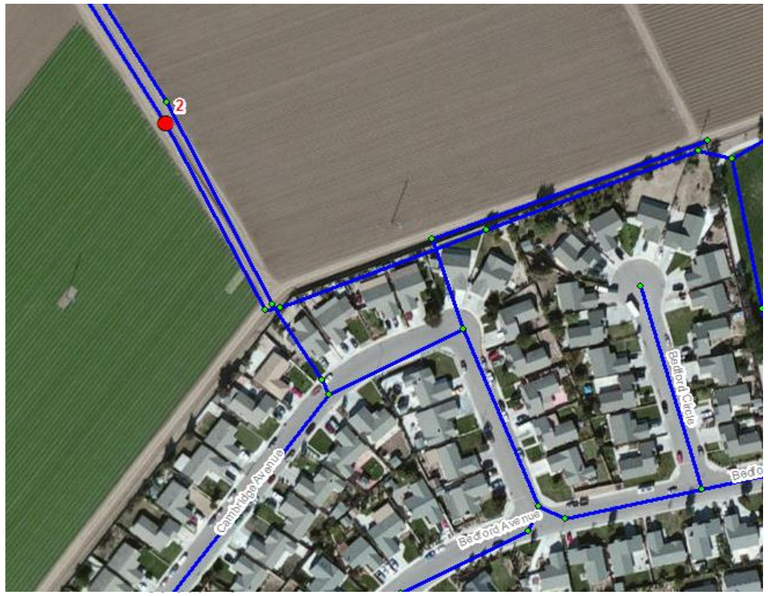
Weekend Diurnal Pattern



FLOW MONITORING SITE 2 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: Upstream of WWTP (south branch)
Pipeline diameter: 30"
City Manhole ID:
Model Mahhole ID:
Silt Level at Site: none"

Flow Monitor Location

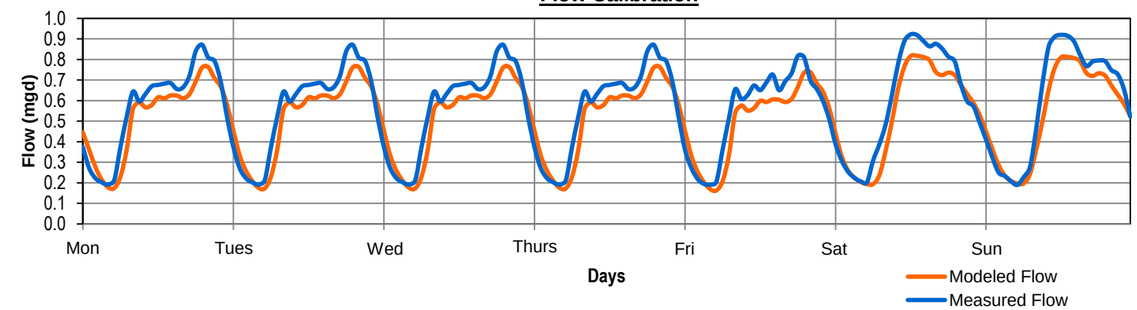


Model Calibration Summary

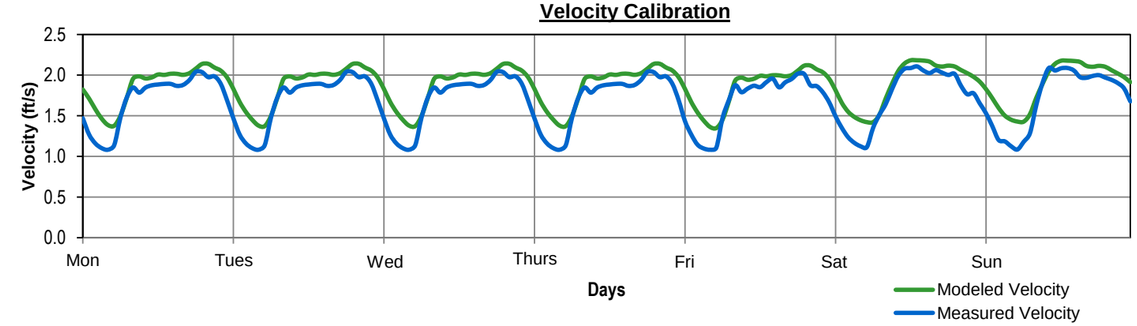
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.56	0.87	4.7	1.70	0.52	0.76	4.30	1.87	-8.3%	-12.5%	-7.8%	10.2%
Tues.	0.56	0.87	4.7	1.70	0.52	0.76	4.30	1.87	-8.3%	-12.5%	-7.8%	10.2%
Wed.	0.56	0.87	4.7	1.70	0.52	0.76	4.30	1.87	-8.3%	-12.5%	-7.8%	10.2%
Thur.	0.56	0.87	4.7	1.70	0.52	0.76	4.30	1.87	-8.3%	-12.5%	-7.8%	10.2%
Fri.	0.55	0.82	4.6	1.68	0.50	0.74	4.22	1.85	-8.6%	-9.8%	-8.2%	10.2%
Sat.	0.60	0.92	4.8	1.72	0.54	0.82	4.36	1.89	-9.6%	-11.3%	-8.5%	9.5%
Sun.	0.60	0.92	4.8	1.71	0.54	0.81	4.38	1.89	-9.6%	-11.7%	-8.3%	10.3%
Summary												
Weekday	0.56	--	4.7	1.69	0.51	--	4.3	1.87	-8.4%	--	-7.9%	10.2%
Weekend	0.60	--	4.8	1.72	0.54	--	4.4	1.89	-9.6%	--	-8.4%	9.9%
ADWF[4]	0.57	--	4.7	1.70	0.52	--	4.3	1.87	-8.7%	--	-8.1%	10.1%

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

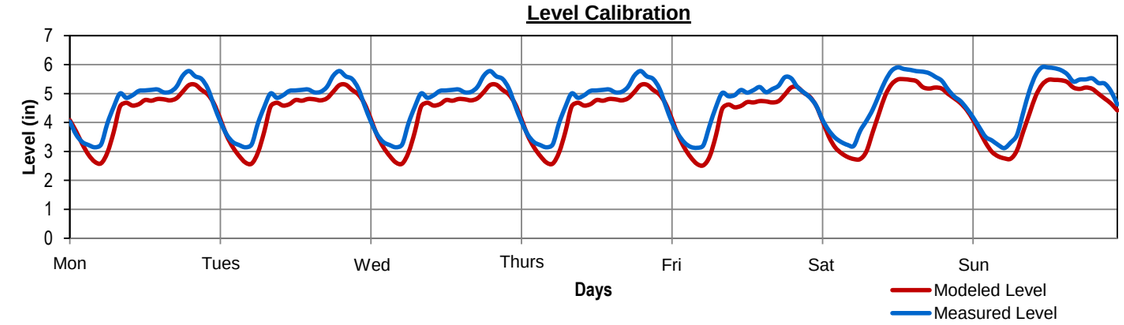
Flow Calibration



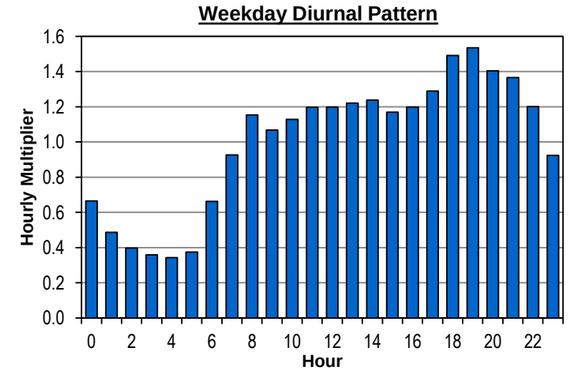
Velocity Calibration



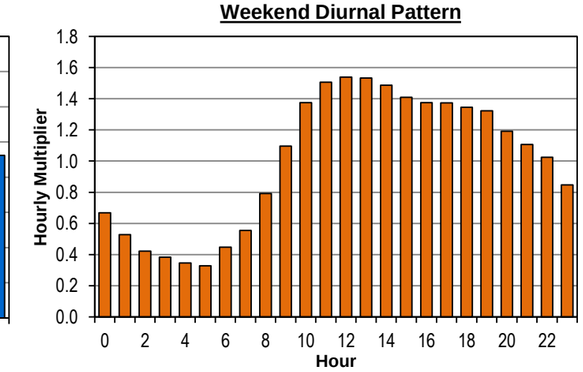
Level Calibration



Weekday Diurnal Pattern



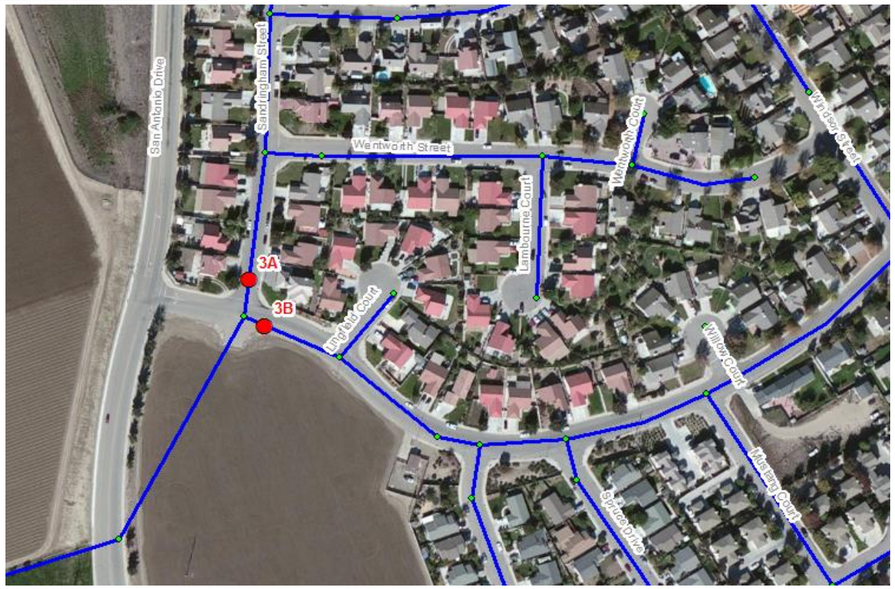
Weekend Diurnal Pattern



FLOW MONITORING SITE 3A DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: Sandringham Street and Willow Street
Pipeline diameter: 11.5"
City Manhole ID: T7 (North inlet)
Model Mahhole ID: T7 (North inlet)
Silt Level at Site: none"

Flow Monitor Location

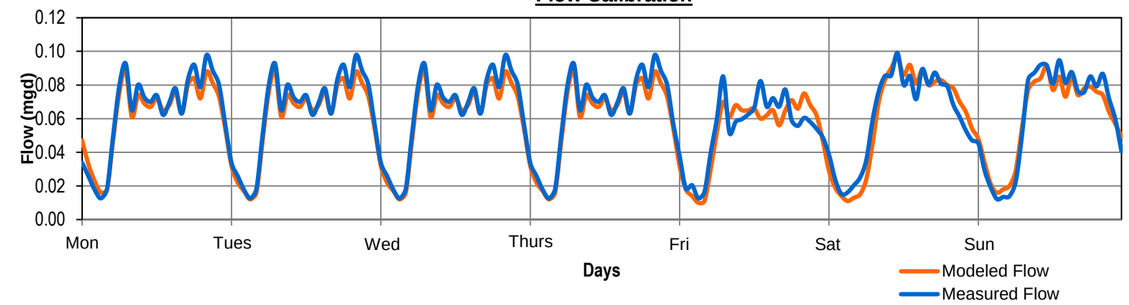


Model Calibration Summary

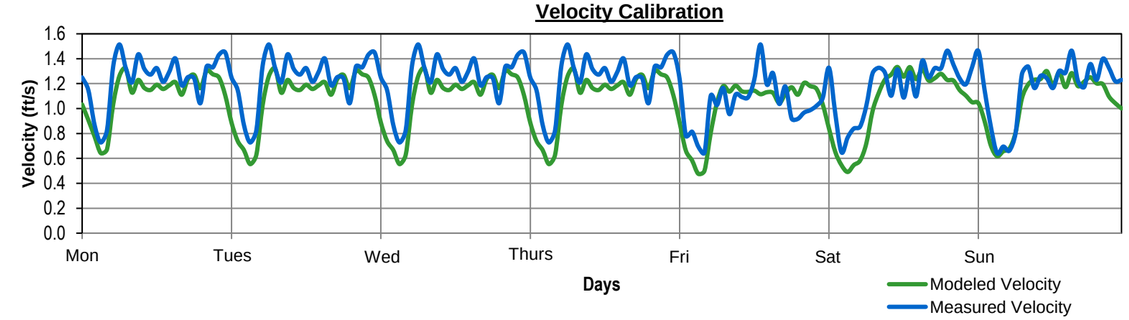
Day	Measured Data(1) Avg. Flow (mgd)	Measured Data(1) Peak Flow(2) (mgd)	Measured Data(1) Avg. Level (in)	Measured Data(1) Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow(2) (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error(3) Avg. Flow (%)	Percent Error(3) Peak Flow (%)	Percent Error(3) Avg. Level (%)	Percent Error(3) Avg. Vel. (%)
Mon.	0.06	0.10	1.9	1.24	0.06	0.09	1.96	1.10	-4.9%	-9.0%	4.5%	-11.4%
Tues.	0.06	0.10	1.9	1.24	0.06	0.09	1.96	1.10	-4.9%	-9.0%	4.5%	-11.4%
Wed.	0.06	0.10	1.9	1.24	0.06	0.09	1.96	1.10	-4.9%	-9.0%	4.5%	-11.4%
Thur.	0.06	0.10	1.9	1.24	0.06	0.09	1.96	1.10	-4.9%	-9.0%	4.5%	-11.4%
Fri.	0.05	0.09	1.9	1.04	0.05	0.08	1.85	1.01	-2.5%	-11.9%	-1.0%	-3.2%
Sat.	0.06	0.10	1.9	1.17	0.06	0.10	1.95	1.04	-1.1%	-3.1%	2.9%	-11.1%
Sun.	0.06	0.09	1.9	1.16	0.06	0.09	1.96	1.07	-2.3%	-4.0%	5.2%	-8.4%
Summary												
Weekday	0.06	--	1.9	1.20	0.06	--	1.9	1.08	-4.5%	--	3.4%	-10.0%
Weekend	0.06	--	1.9	1.17	0.06	--	2.0	1.05	-1.7%	--	4.0%	-9.8%
ADWF(4)	0.06	--	1.9	1.19	0.06	--	1.9	1.07	-3.7%	--	3.6%	-9.9%

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

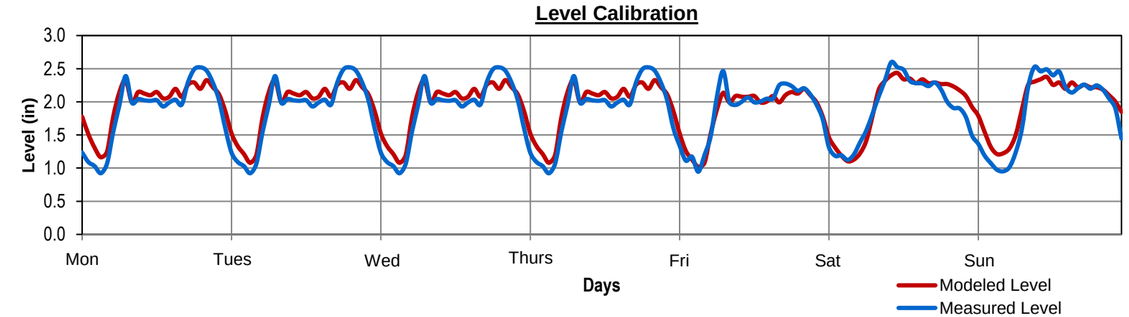
Flow Calibration



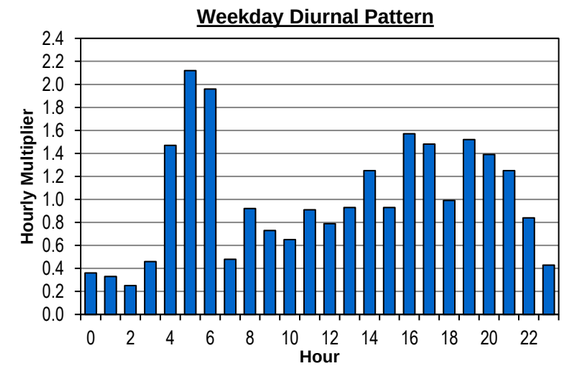
Velocity Calibration



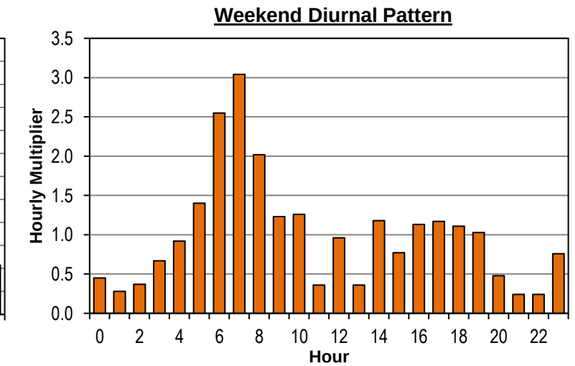
Level Calibration



Weekday Diurnal Pattern



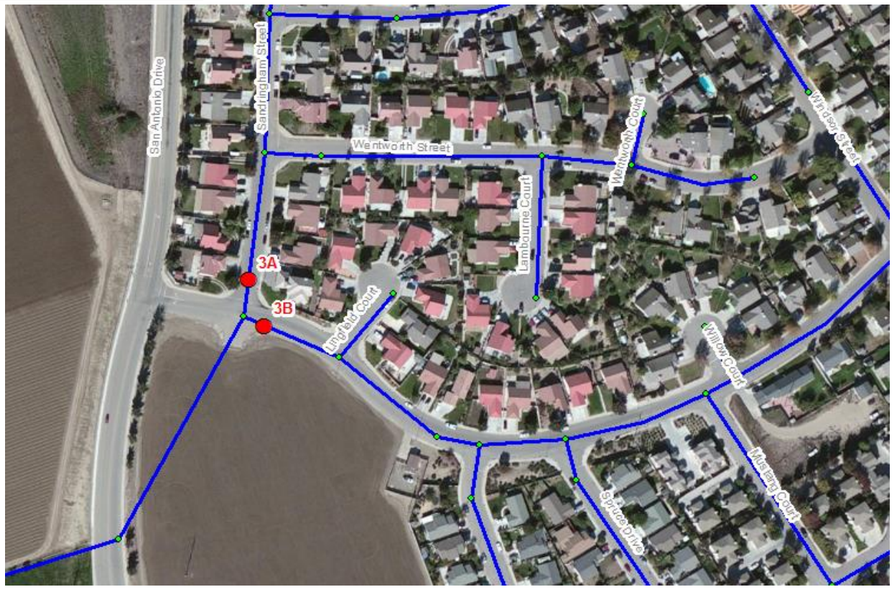
Weekend Diurnal Pattern



FLOW MONITORING SITE 3B DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: Sandringham Street and Willow Street
Pipeline diameter: 11.5"
City Manhole ID: T7 (East inlet)
Model Mahhole ID: T7 (East inlet)
Silt Level at Site: none"

Flow Monitor Location

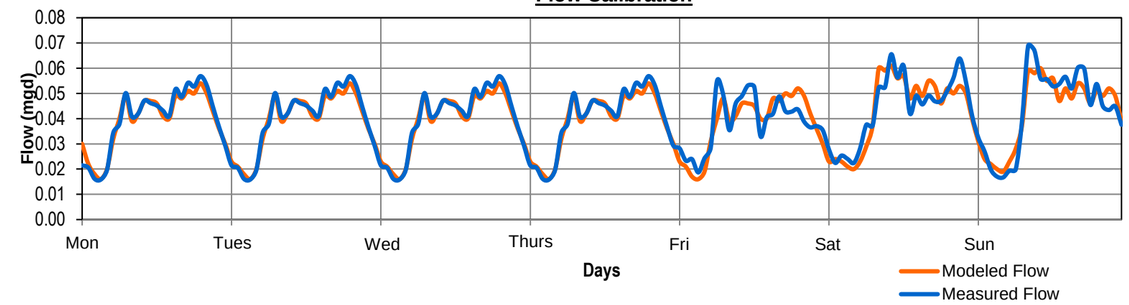


Model Calibration Summary

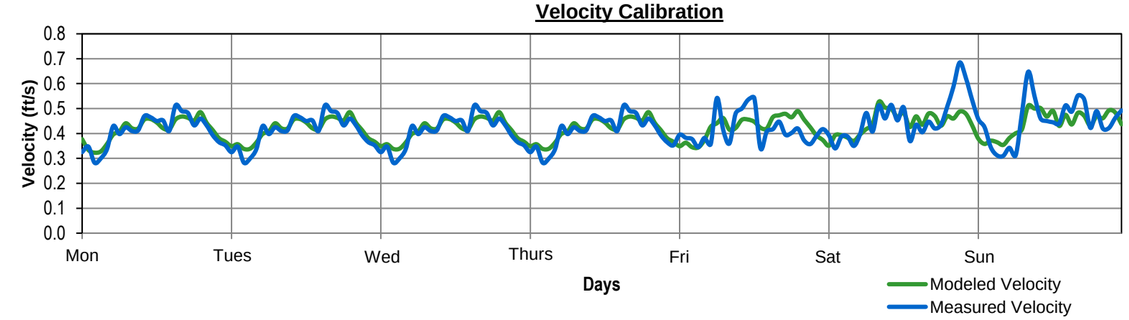
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.04	0.06	2.9	0.41	0.04	0.05	2.88	0.41	-1.7%	-5.0%	-1.6%	1.2%
Tues.	0.04	0.06	2.9	0.41	0.04	0.05	2.88	0.41	-1.7%	-5.0%	-1.6%	1.2%
Wed.	0.04	0.06	2.9	0.41	0.04	0.05	2.88	0.41	-1.7%	-5.0%	-1.6%	1.2%
Thur.	0.04	0.06	2.9	0.41	0.04	0.05	2.88	0.41	-1.7%	-5.0%	-1.6%	1.2%
Fri.	0.04	0.05	2.9	0.42	0.04	0.05	2.80	0.42	-1.5%	-5.2%	-3.2%	1.8%
Sat.	0.04	0.07	3.0	0.46	0.04	0.06	2.95	0.44	-1.6%	-6.9%	-0.2%	-3.4%
Sun.	0.04	0.07	2.9	0.45	0.04	0.06	2.95	0.44	-1.2%	-12.9%	0.8%	-2.0%
Summary												
Weekday	0.04	--	2.9	0.41	0.04	--	2.9	0.42	-1.6%	--	-2.0%	1.3%
Weekend	0.04	--	2.9	0.45	0.04	--	3.0	0.44	-1.4%	--	0.3%	-2.7%
ADWF[4]	0.04	--	2.9	0.42	0.04	--	2.9	0.42	-1.6%	--	-1.3%	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

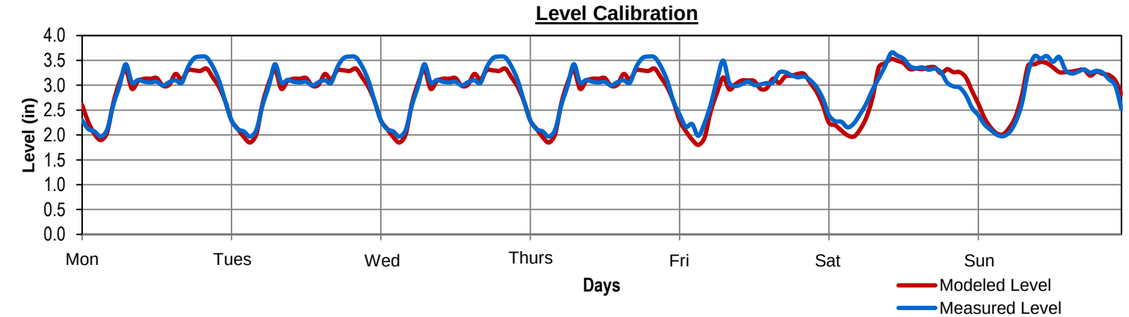
Flow Calibration



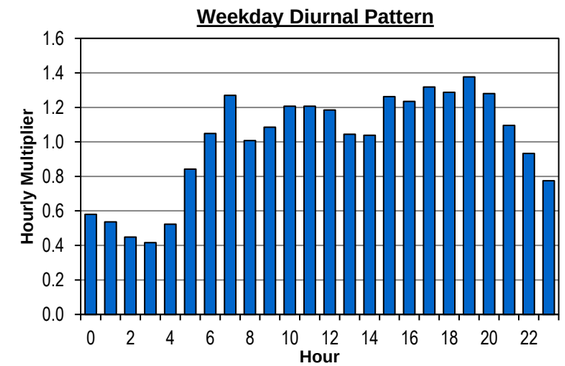
Velocity Calibration



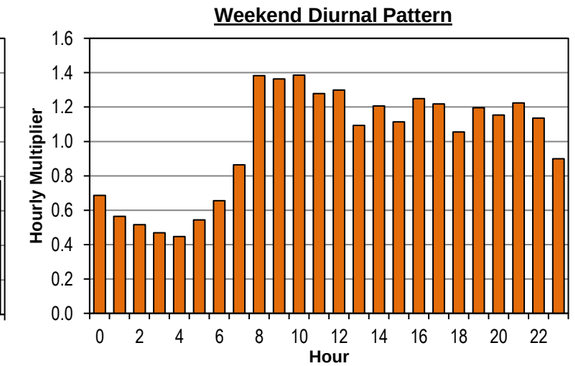
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



FLOW MONITORING SITE 4 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: Broadway Street (northeast of San Antonio Drive)
Pipeline diameter: 18"
City Manhole ID: S17
Model Mahhole ID: S17
Silt Level at Site: none"

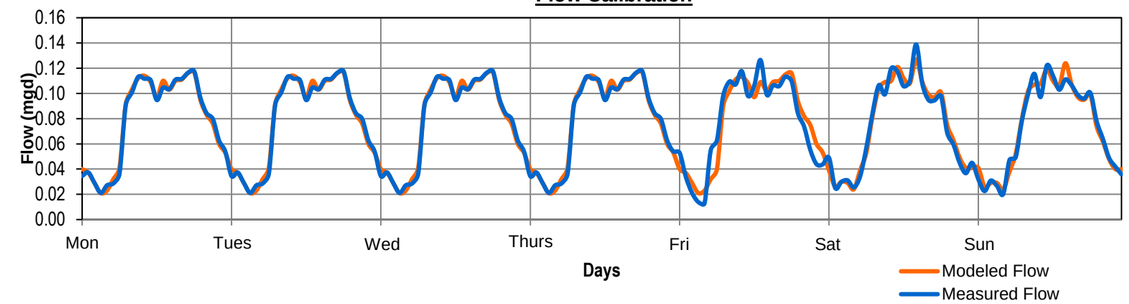
Flow Monitor Location

Model Calibration Summary

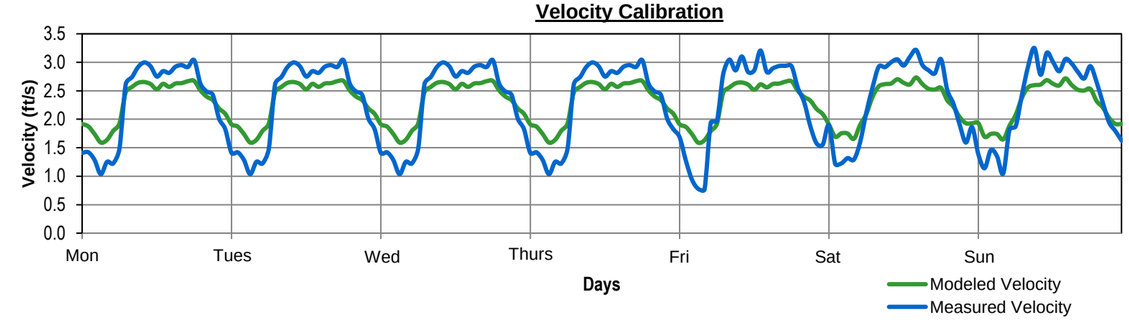
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.08	0.12	1.2	2.29	0.08	0.12	1.18	2.30	0.3%	-0.6%	0.0%	0.6%
Tues.	0.08	0.12	1.2	2.29	0.08	0.12	1.18	2.30	0.3%	-0.6%	0.0%	0.6%
Wed.	0.08	0.12	1.2	2.29	0.08	0.12	1.18	2.30	0.3%	-0.6%	0.0%	0.6%
Thur.	0.08	0.12	1.2	2.29	0.08	0.12	1.18	2.30	0.3%	-0.6%	0.0%	0.6%
Fri.	0.08	0.13	1.2	2.27	0.08	0.12	1.17	2.30	0.9%	-8.3%	0.0%	1.2%
Sat.	0.07	0.14	1.1	2.34	0.07	0.13	1.14	2.26	0.3%	-8.6%	2.9%	-3.3%
Sun.	0.07	0.12	1.1	2.30	0.07	0.12	1.13	2.25	1.0%	1.5%	3.1%	-2.3%
Summary												
Weekday	0.08	--	1.2	2.28	0.08	--	1.2	2.30	0.4%	--	0.0%	0.7%
Weekend	0.07	--	1.1	2.32	0.07	--	1.1	2.26	0.6%	--	3.0%	-2.8%
ADWF[4]	0.08	--	1.2	2.30	0.08	--	1.2	2.29	0.5%	--	0.8%	-0.3%

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

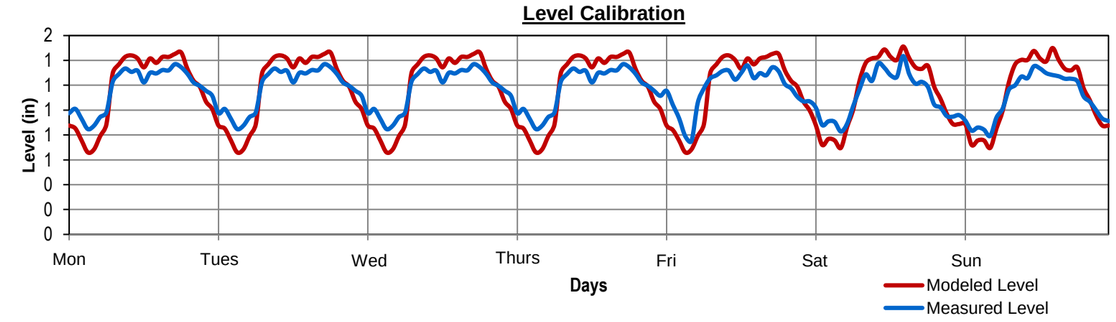
Flow Calibration



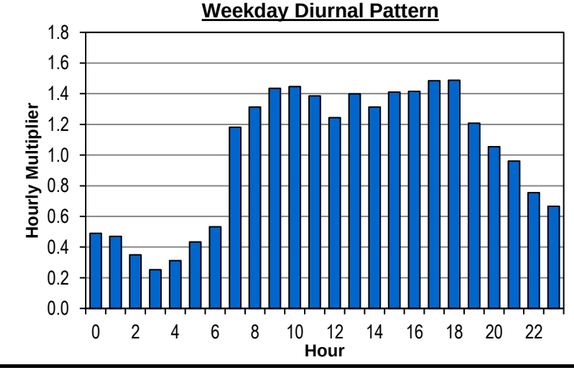
Velocity Calibration



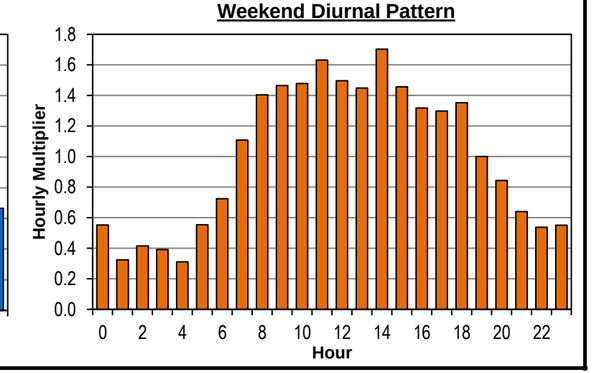
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



FLOW MONITORING SITE 5 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: River Drive (southeast of Broadway Circle)
Pipeline diameter: 16"
City Manhole ID: W2
Model Mahhole ID: W2
Silt Level at Site: 0.5""

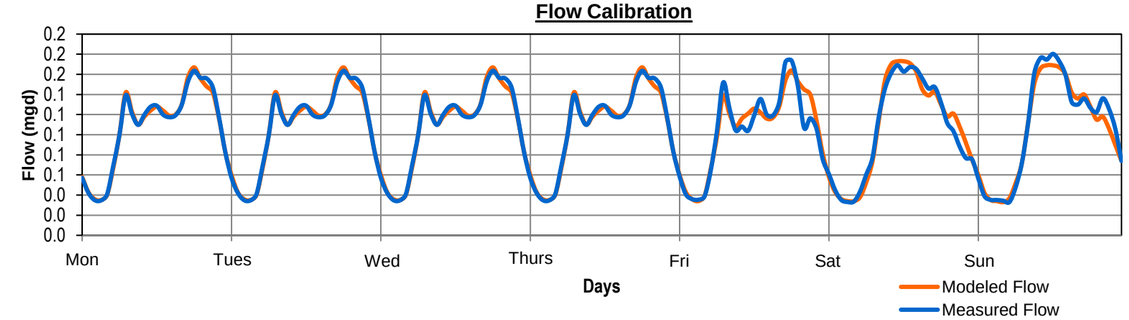
Flow Monitor Location

Model Calibration Summary

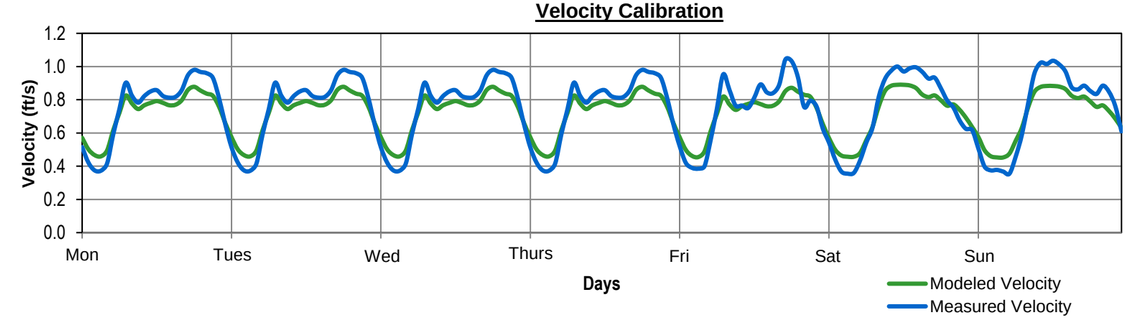
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.11	0.16	3.5	0.75	0.11	0.17	3.53	0.72	0.0%	2.1%	1.6%	-4.0%
Tues.	0.11	0.16	3.5	0.75	0.11	0.17	3.53	0.72	0.0%	2.1%	1.6%	-4.0%
Wed.	0.11	0.16	3.5	0.75	0.11	0.17	3.53	0.72	0.0%	2.1%	1.6%	-4.0%
Thur.	0.11	0.16	3.5	0.75	0.11	0.17	3.53	0.72	0.0%	2.1%	1.6%	-4.0%
Fri.	0.10	0.17	3.4	0.74	0.11	0.16	3.50	0.72	1.8%	-5.5%	1.8%	-3.2%
Sat.	0.10	0.17	3.4	0.73	0.11	0.17	3.48	0.71	1.8%	2.3%	1.2%	-2.8%
Sun.	0.11	0.18	3.5	0.73	0.10	0.17	3.45	0.70	-2.8%	-6.3%	-0.2%	-4.2%
Summary												
Weekday	0.11	--	3.5	0.75	0.11	--	3.5	0.72	0.3%	--	1.6%	-3.9%
Weekend	0.11	--	3.4	0.73	0.11	--	3.5	0.71	-0.5%	--	0.5%	-3.5%
ADWF[4]	0.11	--	3.5	0.74	0.11	--	3.5	0.72	0.1%	--	1.3%	-3.8%

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

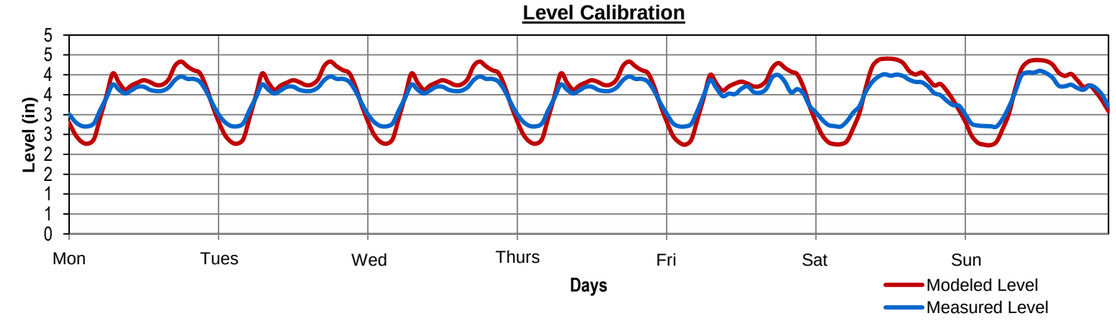
Flow Calibration



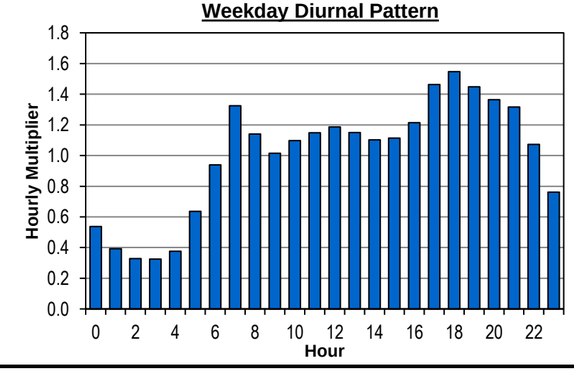
Velocity Calibration



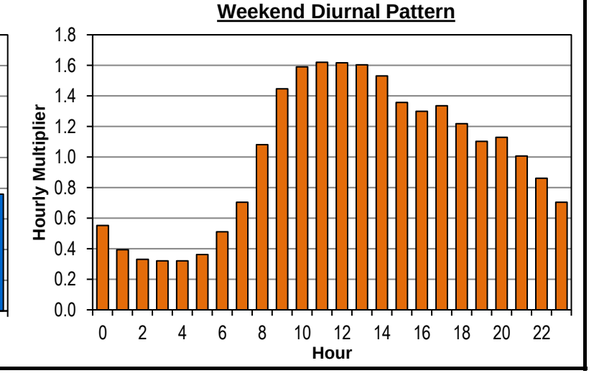
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



FLOW MONITORING SITE 6 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: Division Street and South Mildred Avenue
Pipeline diameter: 19"
City Manhole ID: MH520
Model Mahhole ID: MH520
Silt Level at Site: none"

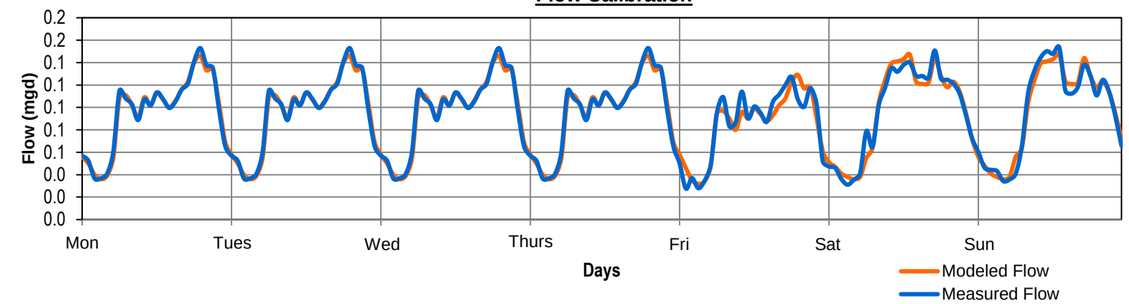
Flow Monitor Location

Model Calibration Summary

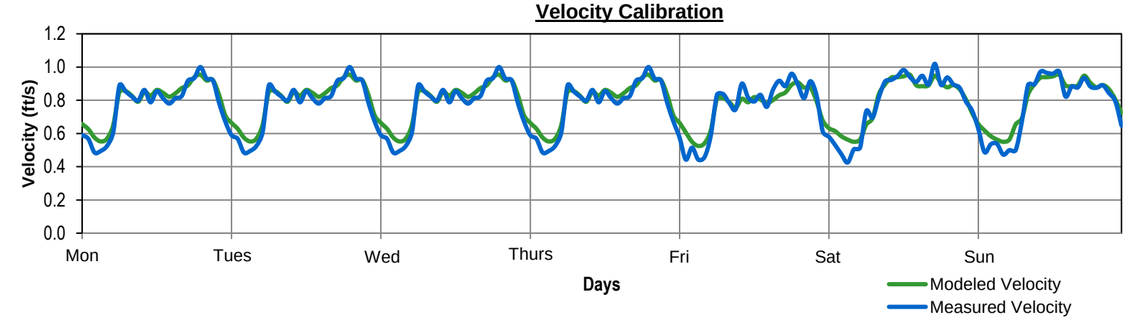
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.10	0.15	2.8	0.77	0.10	0.15	2.77	0.79	-0.4%	-4.5%	-1.9%	2.8%
Tues.	0.10	0.15	2.8	0.77	0.10	0.15	2.77	0.79	-0.4%	-4.5%	-1.9%	2.8%
Wed.	0.10	0.15	2.8	0.77	0.10	0.15	2.77	0.79	-0.4%	-4.5%	-1.9%	2.8%
Thur.	0.10	0.15	2.8	0.77	0.10	0.15	2.77	0.79	-0.4%	-4.5%	-1.9%	2.8%
Fri.	0.08	0.13	2.6	0.75	0.08	0.13	2.64	0.76	-0.4%	1.4%	0.0%	0.9%
Sat.	0.10	0.15	2.8	0.79	0.10	0.15	2.77	0.79	0.0%	-2.5%	-0.2%	0.8%
Sun.	0.10	0.15	2.8	0.77	0.10	0.15	2.78	0.79	0.1%	-4.4%	-2.0%	3.4%
Summary												
Weekday	0.09	--	2.8	0.77	0.09	--	2.7	0.79	-0.4%	--	-1.5%	2.5%
Weekend	0.10	--	2.8	0.78	0.10	--	2.8	0.79	0.1%	--	-1.1%	2.1%
ADWF[4]	0.09	--	2.8	0.77	0.09	--	2.8	0.79	-0.2%	--	-1.4%	2.4%

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

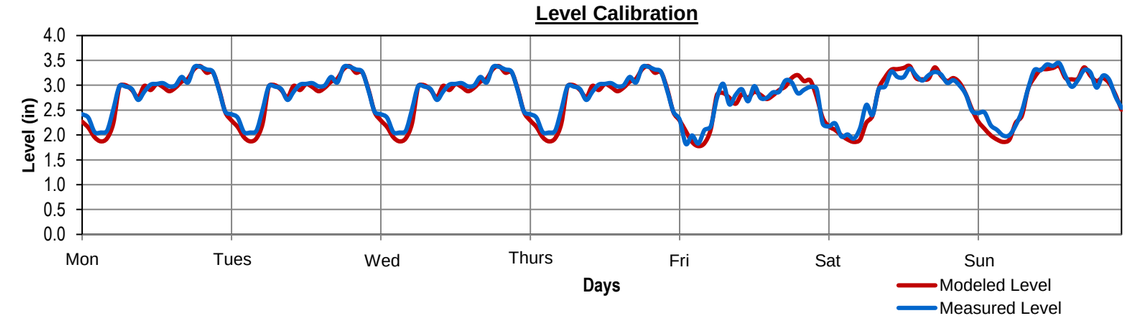
Flow Calibration



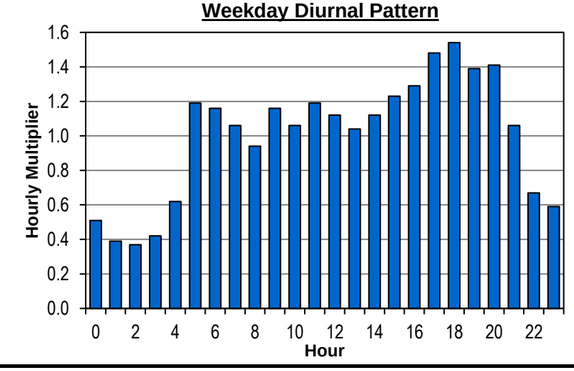
Velocity Calibration



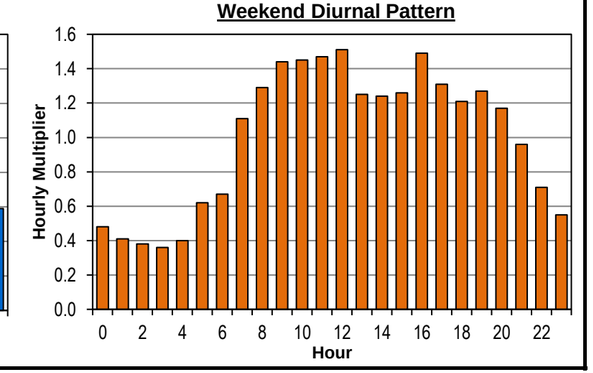
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



FLOW MONITORING SITE 7 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: San Antonio Drive (west of Van Etten Avenue)
Pipeline diameter: 18"
City Manhole ID: N16
Model Mahhole ID: N16
Silt Level at Site: none"

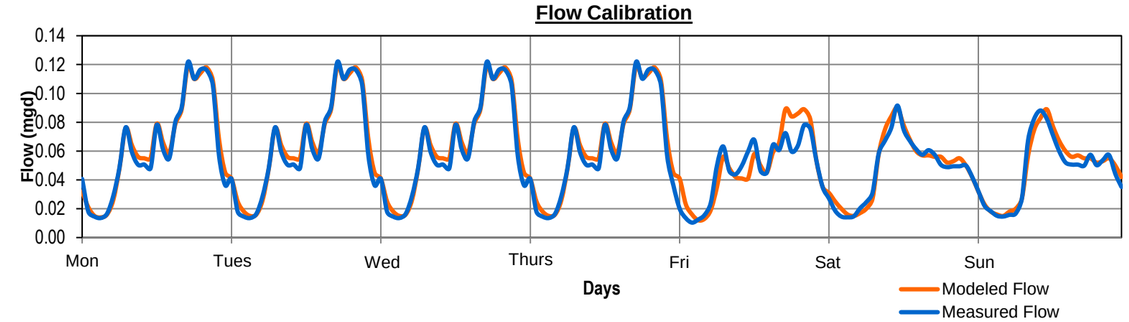
Flow Monitor Location

Model Calibration Summary

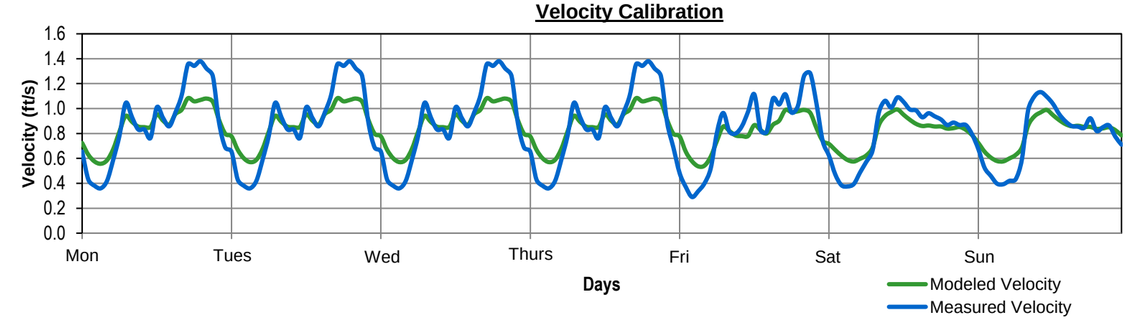
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.06	0.12	1.9	0.88	0.06	0.12	1.97	0.87	3.5%	-2.2%	6.5%	-1.6%
Tues.	0.06	0.12	1.9	0.88	0.06	0.12	1.97	0.87	3.5%	-2.2%	6.5%	-1.6%
Wed.	0.06	0.12	1.9	0.88	0.06	0.12	1.97	0.87	3.5%	-2.2%	6.5%	-1.6%
Thur.	0.06	0.12	1.9	0.88	0.06	0.12	1.97	0.87	3.5%	-2.2%	6.5%	-1.6%
Fri.	0.05	0.08	1.7	0.83	0.05	0.09	1.75	0.80	4.7%	14.1%	4.0%	-3.6%
Sat.	0.05	0.09	1.7	0.80	0.05	0.09	1.74	0.80	3.1%	-1.7%	2.2%	0.1%
Sun.	0.05	0.09	1.7	0.78	0.05	0.09	1.73	0.80	3.4%	1.0%	1.3%	2.7%
Summary												
Weekday	0.06	--	1.8	0.87	0.06	--	1.9	0.85	3.7%	--	6.1%	-2.0%
Weekend	0.05	--	1.7	0.79	0.05	--	1.7	0.80	3.2%	--	1.8%	1.4%
ADWF[4]	0.06	--	1.8	0.85	0.06	--	1.9	0.84	3.6%	--	4.9%	-1.1%

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

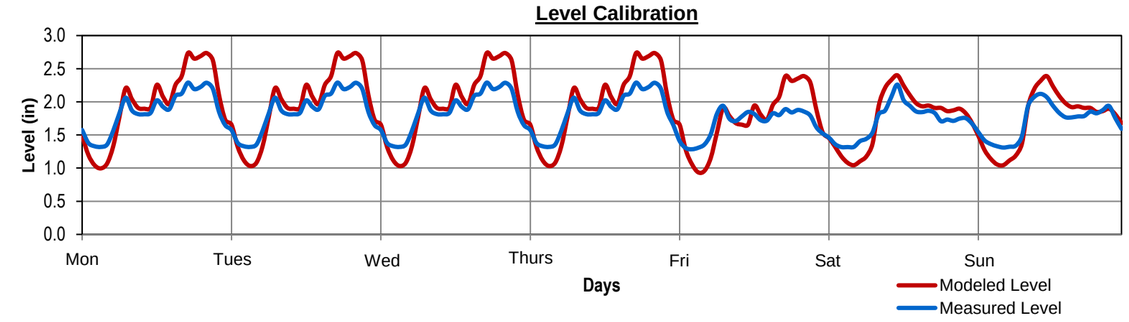
Flow Calibration



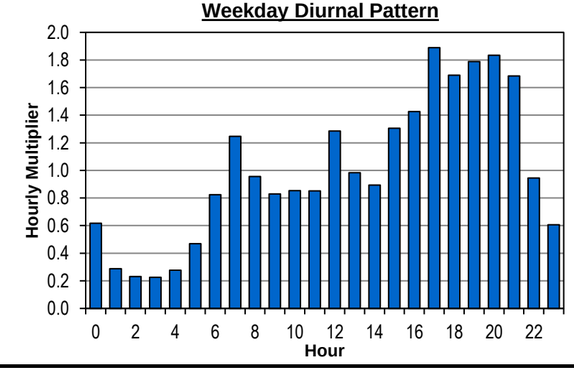
Velocity Calibration



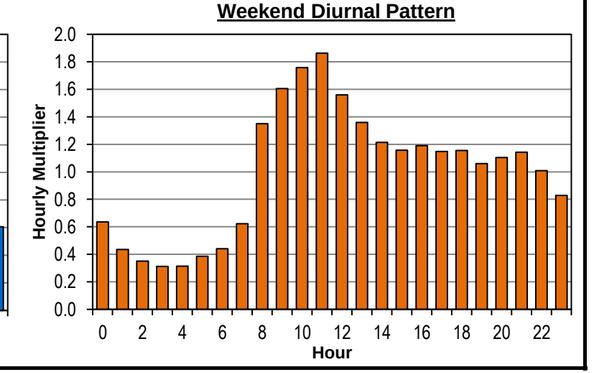
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



FLOW MONITORING SITE 8 DRY WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: San Antonio Drive (east of Metz Road)
Pipeline diameter: 14"
City Manhole ID: MH439
Model Mahhole ID: MH439
Silt Level at Site: none"

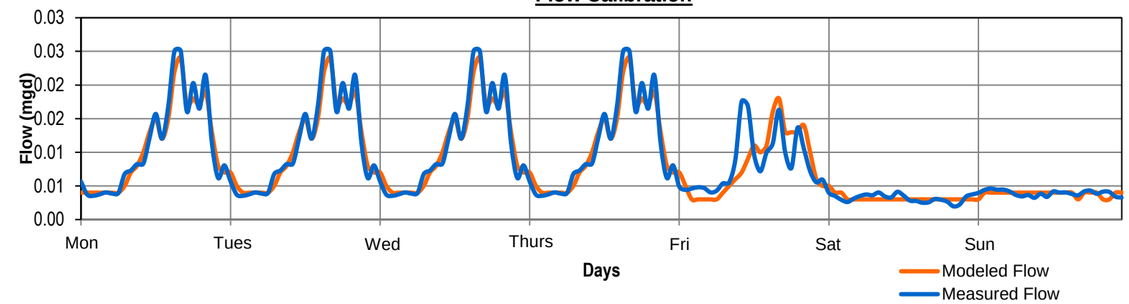
Flow Monitor Location

Model Calibration Summary

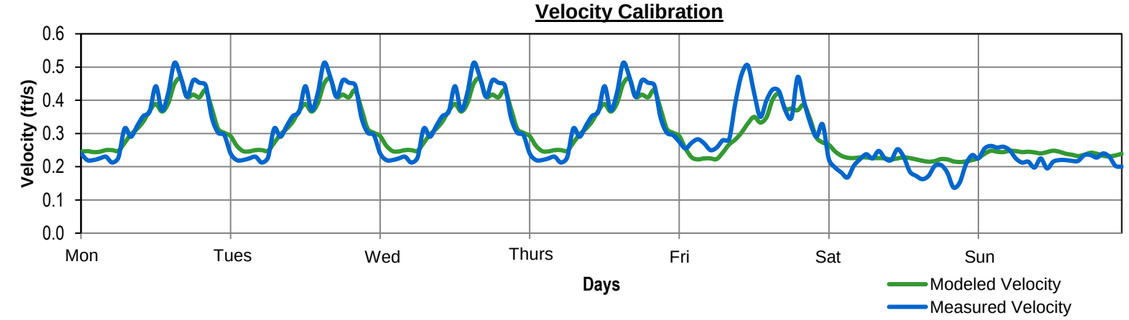
Day	Measured Data[1] Avg. Flow (mgd)	Measured Data[1] Peak Flow[2] (mgd)	Measured Data[1] Avg. Level (in)	Measured Data[1] Avg. Vel. (ft/s)	Modeled Data Avg. Flow (mgd)	Modeled Data Peak Flow[2] (mgd)	Modeled Data Avg. Level (in)	Modeled Data Avg. Vel. (ft/s)	Percent Error[3] Avg. Flow (%)	Percent Error[3] Peak Flow (%)	Percent Error[3] Avg. Level (%)	Percent Error[3] Avg. Vel. (%)
Mon.	0.01	0.03	1.1	0.34	0.01	0.02	1.22	0.34	-1.8%	-4.2%	9.8%	-0.8%
Tues.	0.01	0.03	1.1	0.34	0.01	0.02	1.22	0.34	-1.8%	-4.2%	9.8%	-0.8%
Wed.	0.01	0.03	1.1	0.34	0.01	0.02	1.22	0.34	-1.8%	-4.2%	9.8%	-0.8%
Thur.	0.01	0.03	1.1	0.34	0.01	0.02	1.22	0.34	-1.8%	-4.2%	9.8%	-0.8%
Fri.	0.01	0.02	1.0	0.35	0.01	0.02	1.09	0.31	-1.4%	2.7%	9.3%	-12.1%
Sat.	0.00	0.00	0.8	0.20	0.00	0.01	0.74	0.23	-0.1%	21.1%	-6.9%	11.7%
Sun.	0.00	0.00	0.8	0.23	0.00	0.00	0.80	0.24	-2.3%	-13.3%	-4.7%	6.0%
Summary												
Weekday	0.01	--	1.1	0.34	0.01	--	1.2	0.33	-1.8%	--	9.7%	-3.1%
Weekend	0.00	--	0.8	0.21	0.00	--	0.8	0.23	-1.3%	--	-5.8%	8.7%
ADWF[4]	0.0085	--	1.0	0.31	0.01	--	1.1	0.30	-1.7%	--	6.1%	-0.7%

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

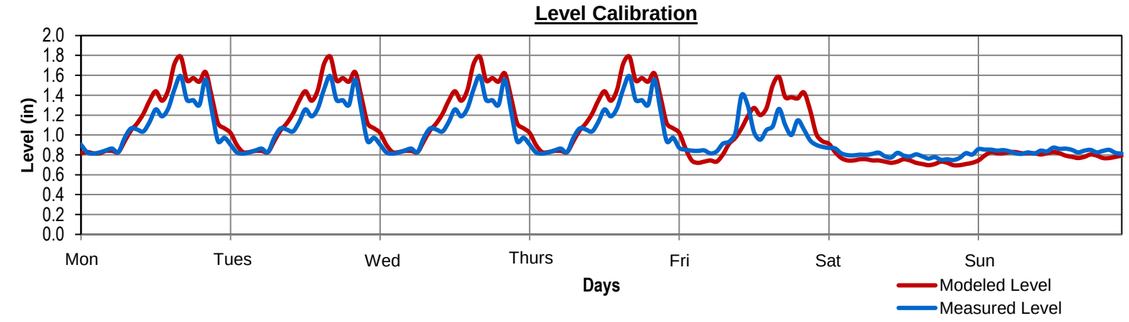
Flow Calibration



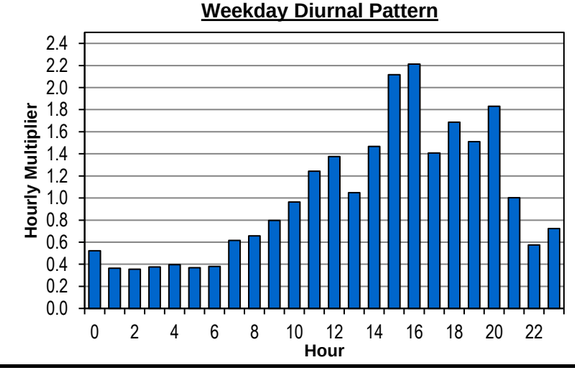
Velocity Calibration



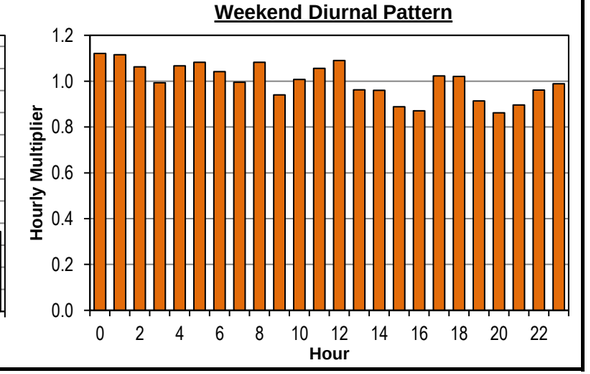
Level Calibration



Weekday Diurnal Pattern



Weekend Diurnal Pattern



APPENDIX D – WET WEATHER FLOW CALIBRATION

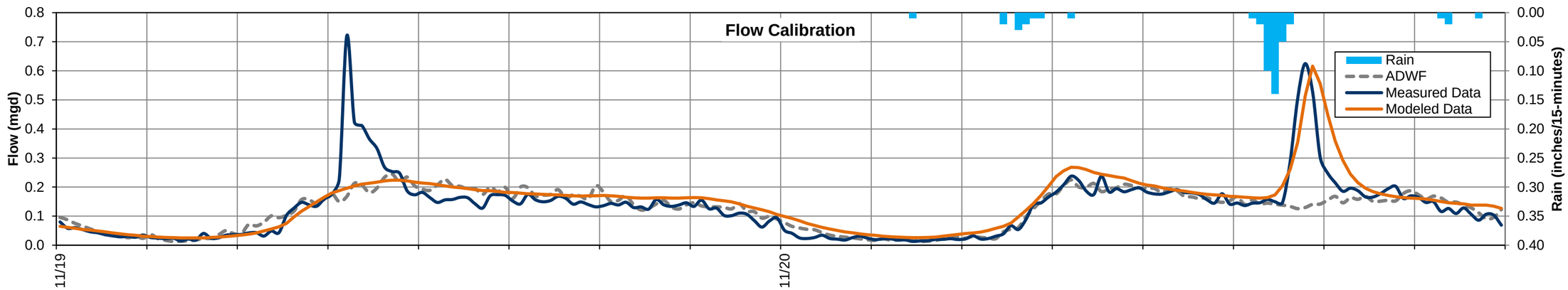
Table 2 WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

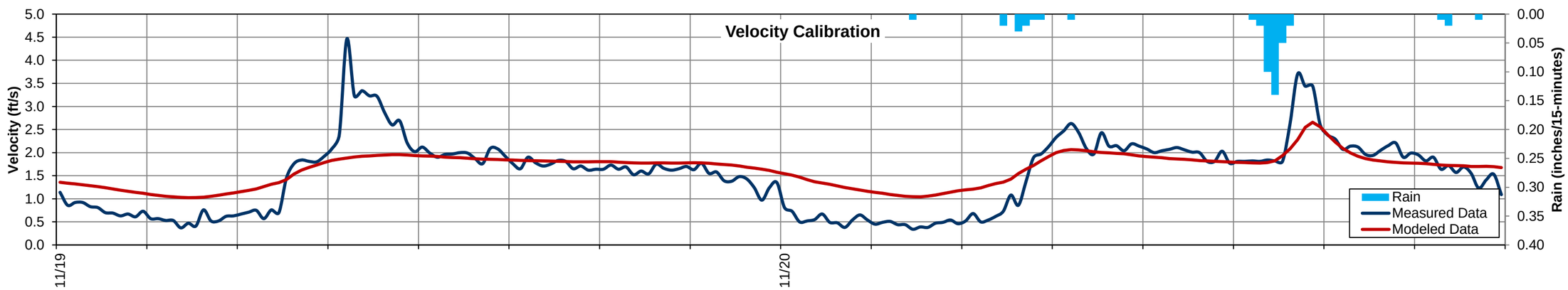
		Storm 1 (11/20/2016-11/20/2016)											
		Measured Data[1]				Modeled Data[2]				Percent Error[3]			
Meter Number	Pipe Diameter (in)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Peak Flow (%)	Avg. Velocity (%)	Avg. Level (%)
1	18	0.134	0.625	1.56	2.1	0.161	0.616	1.69	2.3	19.8%	-1.4%	8.3%	9.7%
1B [4]	9.625	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	30	0.613	1.059	1.72	4.8	0.571	1.015	1.91	4.5	-6.9%	-4.1%	11.2%	-7.1%
3A [4]	11.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3A [4]	11.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	18	0.076	0.137	2.24	1.1	0.078	0.135	2.28	1.2	2.2%	-1.6%	1.4%	1.7%
5 [4]	16	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
6 [4]	19	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
7	18	0.059	0.309	0.80	1.9	0.070	0.304	0.87	2.0	19.6%	-1.6%	7.7%	4.5%
8	14	0.011	0.128	0.35	1.0	0.013	0.130	0.33	1.2	16.5%	1.7%	-7.1%	12.6%
WWTP	18	0.872	1.950	N/A	N/A	1.020	2.142	N/A	N/A	17.0%	9.8%	N/A	N/A

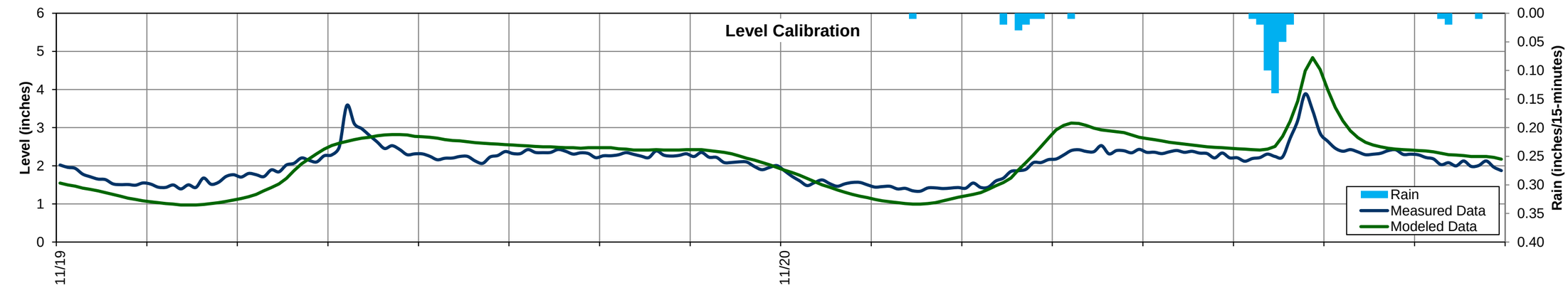
Notes:
1. Source: King City 2016 Temporary Flow Monitoring Program, V&A Consulting Engineers
2. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks. Averages were adjusted to account for data not recorded.
3. Percent Difference = (Modeled - Measured)/Measured*100.
4. Flow monitoring data unavailabe during November 20, 2016 storm event. Flow data received for WWTP (circle charts) were used to calibrate these flow monitoring sites.

FLOW MONITORING SITE 1 WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: San Antonio Drive (west of North Mildred Avenue)
Pipeline diameter: 18"
City Manhole ID: N12
Model Mahhole ID: N12
Silt Level at Site: 0"

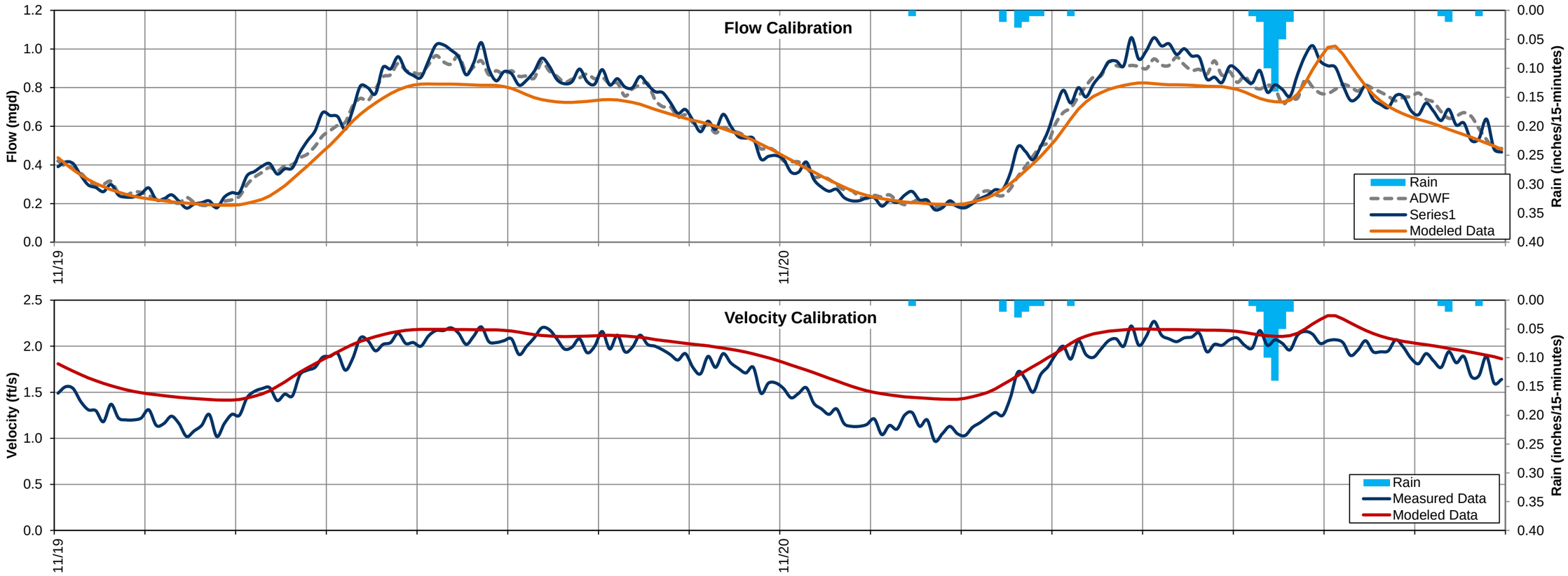






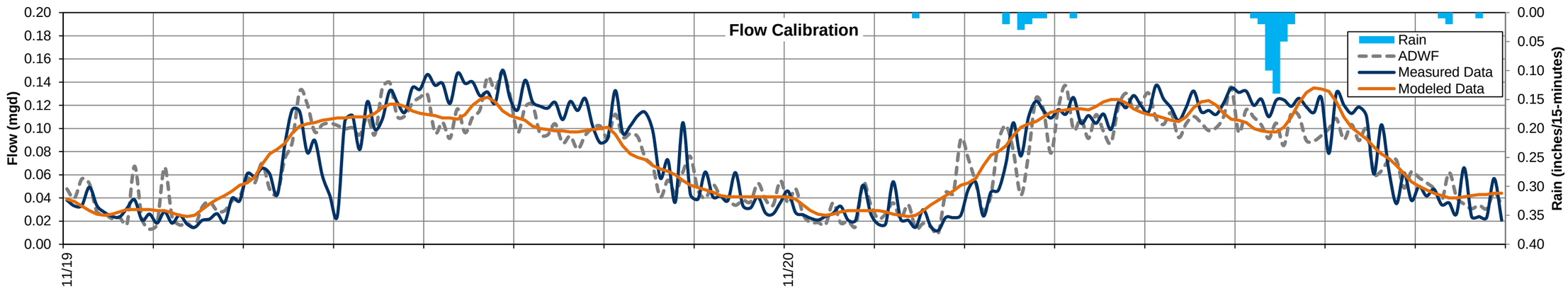
FLOW MONITORING SITE 2 WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

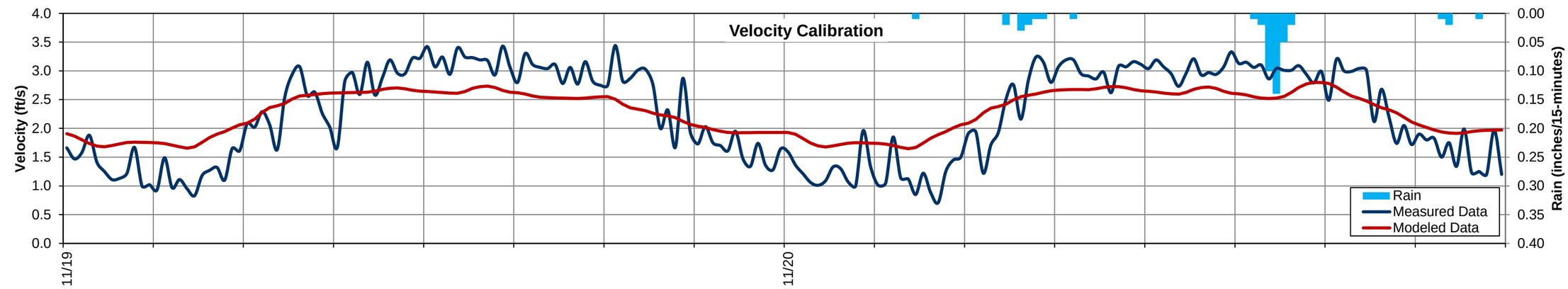
Location: Upstream of WWTP (south branch)
Pipeline diameter: 30"
City Manhole ID:
Model Mahhole ID:
Silt Level at Site: 0"

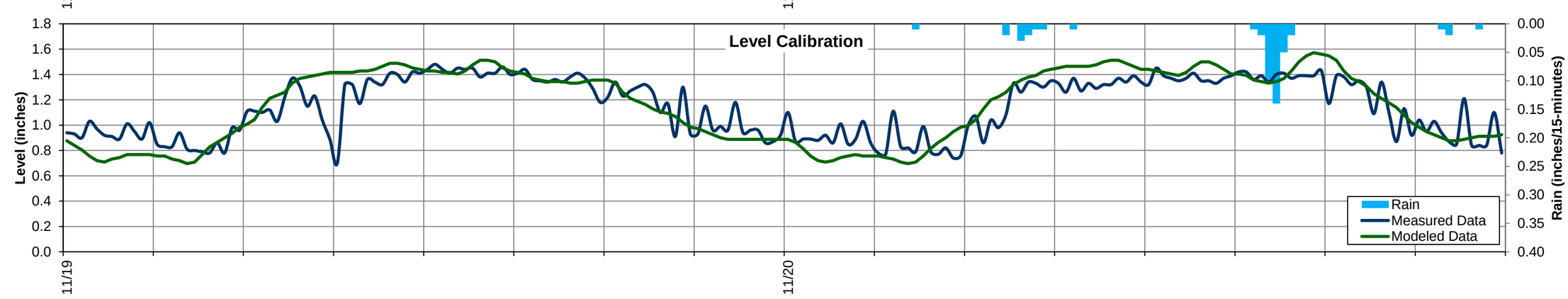


FLOW MONITORING SITE 4 WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: Broadway Street (northeast of San Antonio Drive)
Pipeline diameter: 18"
City Manhole ID: S17
Model Mahhole ID: S17
Silt Level at Site: 0"

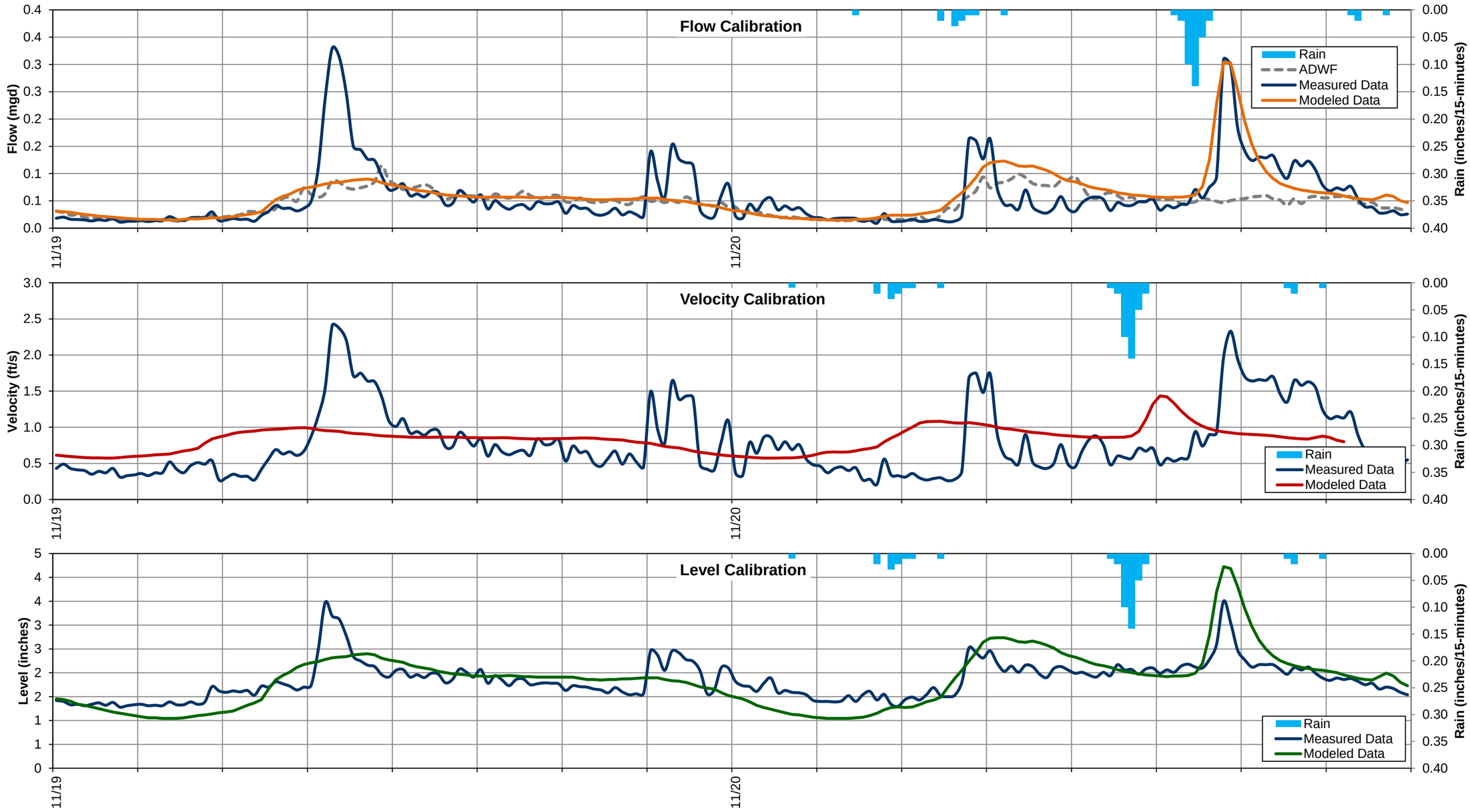






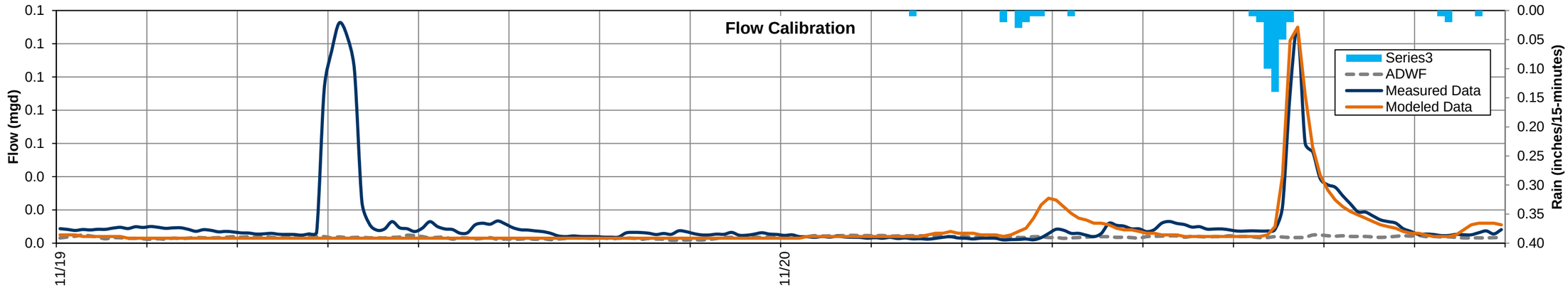
FLOW MONITORING SITE 7 WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

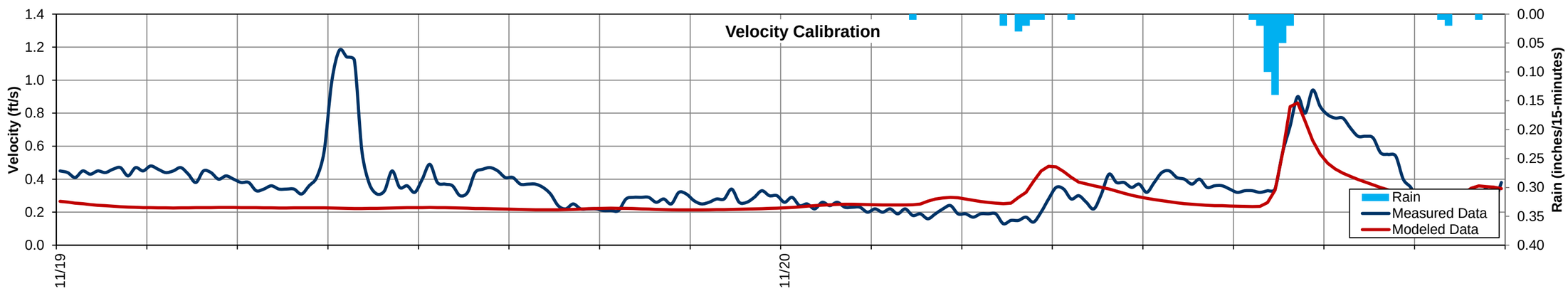
Location: San Antonio Drive (west of Van Etten Avenue)
Pipeline diameter: 18"
City Manhole ID: N16
Model Mahhole ID: N16
Silt Level at Site: 0"

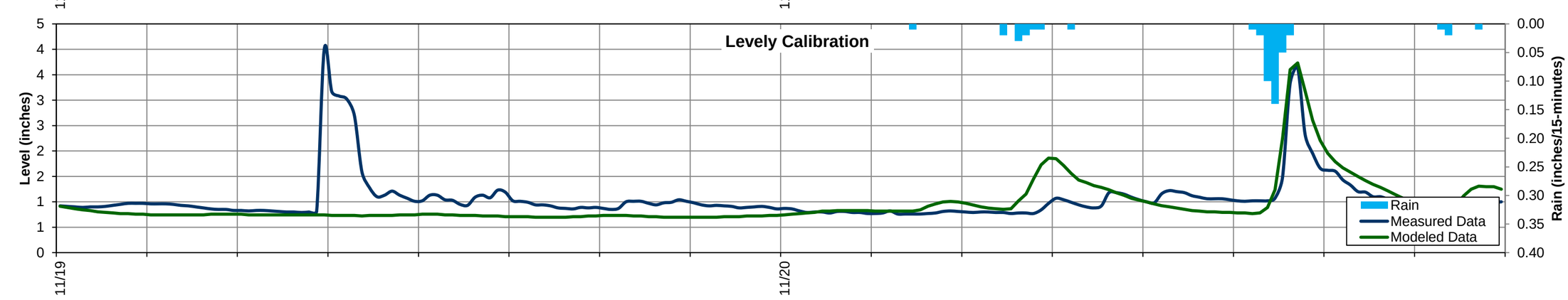


FLOW MONITORING SITE 8 WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: San Antonio Drive (east of Metz Road)
Pipeline diameter: 14"
City Manhole ID: MH439
Model Mahhole ID: MH439
Silt Level at Site: 0"

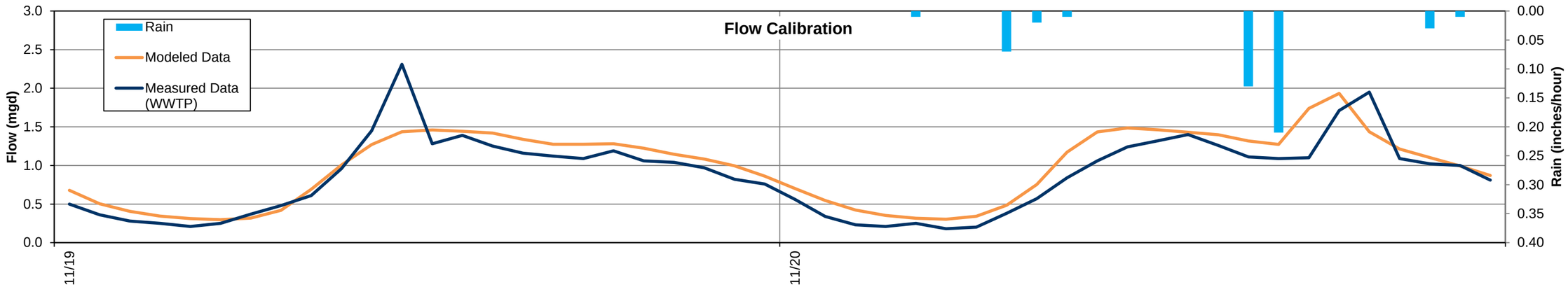


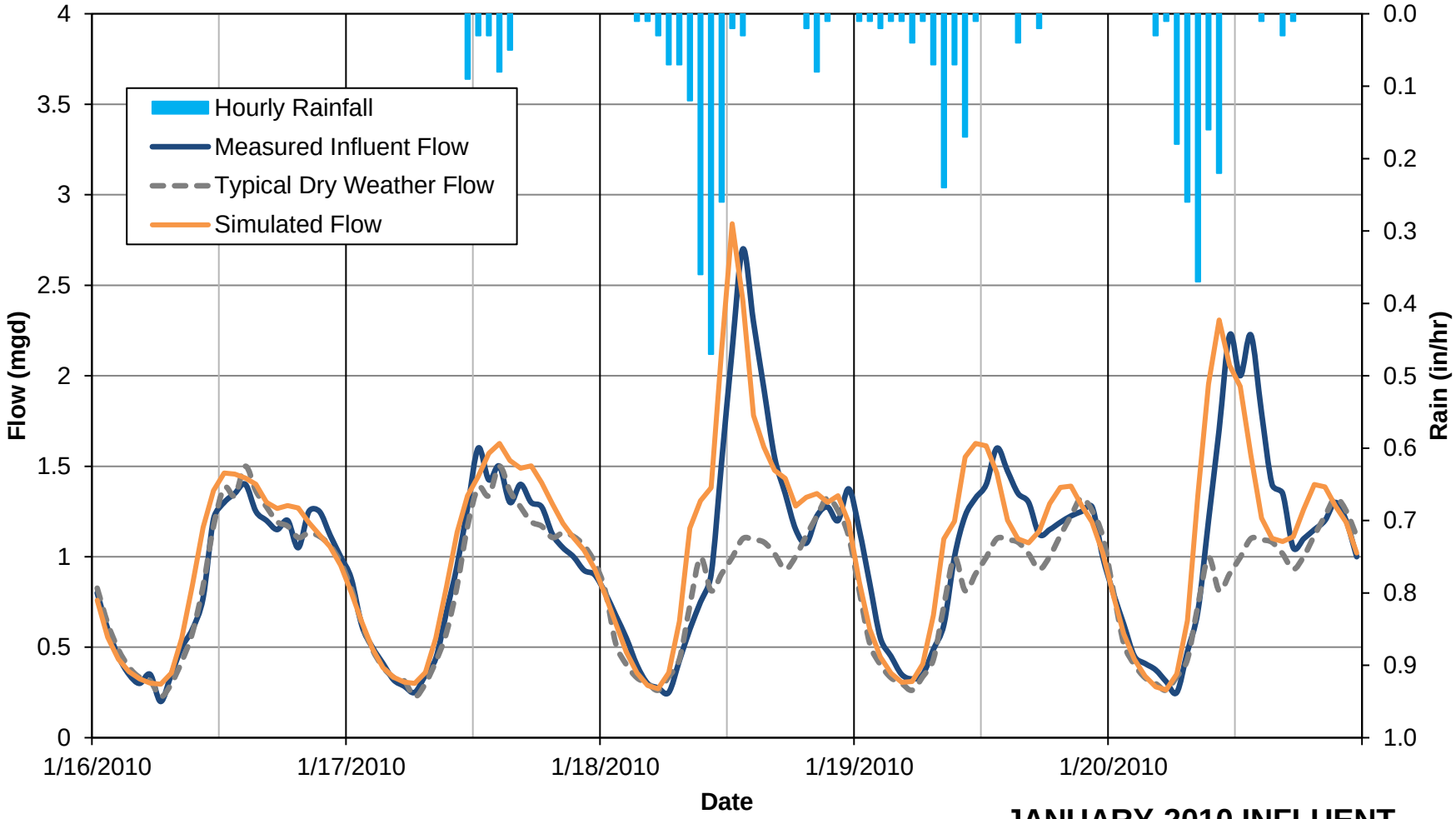




WWTP INFLUENT WET WEATHER FLOW CALIBRATION
COLLECTION SYSTEM MASTER PLAN
KING CITY

Location: WWTP
Pipeline diameter: 18"
City Manhole ID: N/A
Model Mahhole ID: N/A
Silt Level at Site: 0"





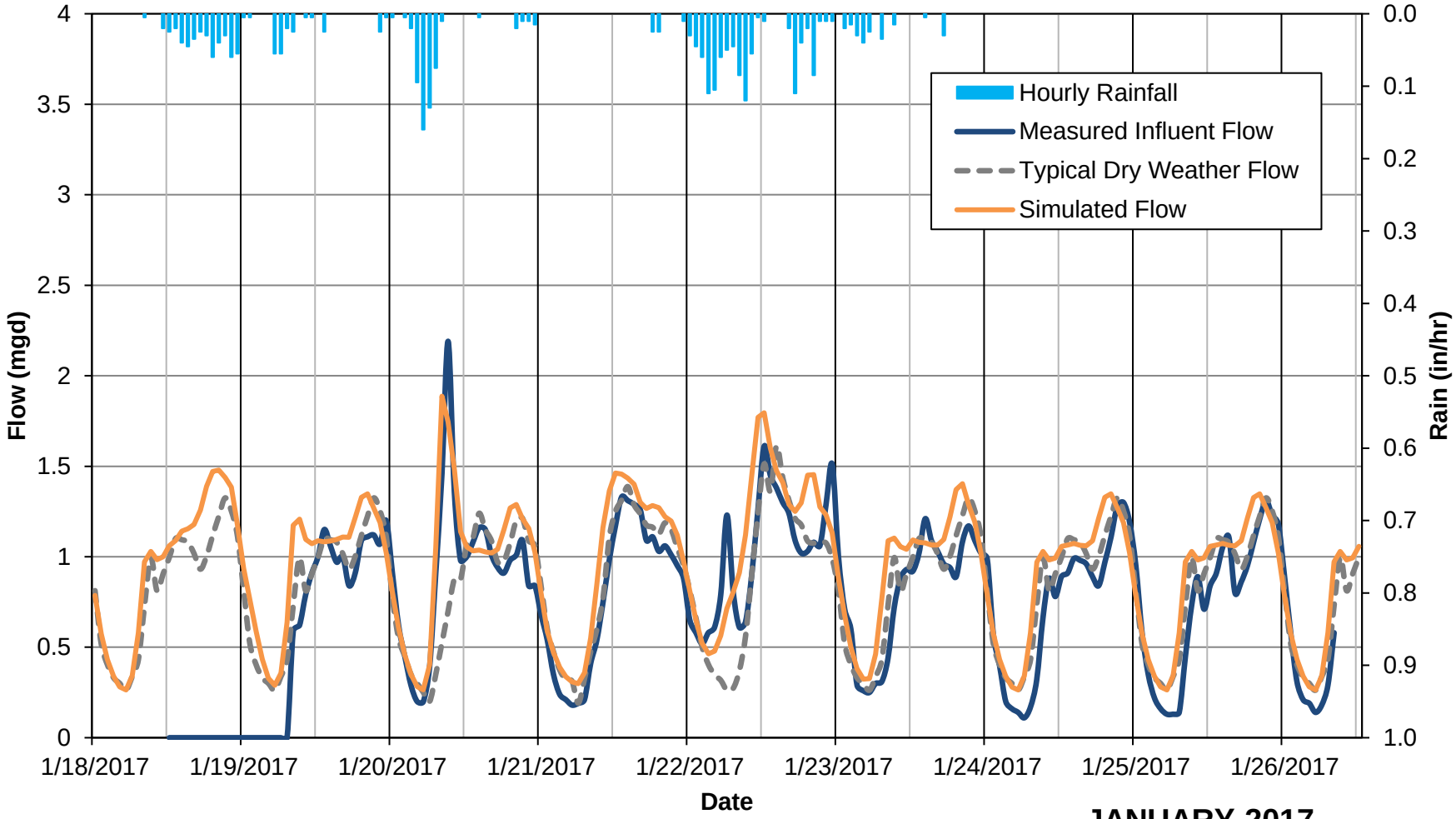
JANUARY 2010 INFLUENT FLOW DATA

FIGURE D.1

KING CITY
COLLECTION SYSTEM MASTER PLAN

carollo

KING CITY
CALIFORNIA



JANUARY 2017
INFLUENT FLOW DATA

FIGURE D.2

KING CITY
COLLECTION SYSTEM MASTER PLAN

carollo

KING CITY
CALIFORNIA