TECHNICAL MEMORANDUM

TO: Dan Herrera, City of Petaluma

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DATE: August 10, 2021

RE: South Area Model Development and Capacity Analysis

Table of Contents

1. INTRODUCTION

The City of Petaluma (City or Petaluma) is located in southern Sonoma County and covers approximately 15.5 square miles. The City collects wastewater from customers in Petaluma, portions of unincorporated Sonoma County, and the community of Penngrove. The collection system consists of approximately 196 miles of pipe, ranging from 6 inches to 54 inches in diameter, and 9 pumping stations. The system discharges to the City's Ellis Creek Water Recycling Facility on Cypress Drive. The plant has a design capacity of 6.7 million gallons per day (mgd) of dry weather flow. In the wintertime, treated wastewater is discharged back into the Petaluma River. In the summertime, recycled water is discharged into the City's recycled water system and is used for irrigation of agricultural lands, two golf courses, and a vineyard. Annually, the City produces about 600 million gallons of recycled water.

The City has experienced capacity issues and sanitary sewer overflows (SSOs) in the area south of the Petaluma River upstream of the C Street Pump Station (C Street PS), known as the South Area. A hydraulic model of the City's trunk sewer system was developed to evaluate the capacity of the system to handle peak wet weather flows in the South Area and to help identify the causes of the SSOs. This Technical Memorandum (TM) describes the process and assumptions used in developing the hydraulic model, the criteria used to assess system performance, and the results of the capacity analysis.

2. HYDRAULIC MODEL DEVELOPMENT

This section describes the development of the hydraulic model that was used to assess the capacity of the City's sewer system. The section provides an overview of the model development process, including descriptions of the modeled sewer network and subcatchments, the flow monitoring program conducted for this study, the basis for estimating wastewater flows, and the calibration of the model.

The modeling software used for the Master Plan was InfoWorks ICM™ by Innovyze, a fully dynamic hydraulic model that has been used for many other collection systems in the Bay Area, including Santa Rosa and Novato. W&C used its own licenses of InfoWorks for this work.

2.1 Modeling Terminology

Key modeling terms are defined below.

- **Network** refers to the representation of the physical facilities being modeled. Modeled network components include pipes, manholes, and pump stations.
- **Nodes** are primarily manholes, but also include pump station wet wells and outfalls (discharge points from the modeled system). Key data associated with nodes include manhole ground elevations and pump station wet well elevations and cross-sectional areas.
- **Pipes** or **conduits** are connections between nodes and include both gravity sewers and force mains. Key data associated with pipes are upstream and downstream node IDs, pipe length, diameter, roughness factor, and upstream and downstream invert elevations.
- **Pumps** are modeled individually, connecting pump station wet wells with the upstream node of associated force mains. Data associated with pumps include type (e.g., fixed or variable speed), on and off levels, pump capacities, and pump discharge curves.
- **Subcatchments** (also called **sewersheds**) are areas that contribute flow to the modeled sewer network and represent the unmodeled sewers in the collection system. Data associated with subcatchments include sanitary

flow (computed based on population, water use, or other available data), type of diurnal sanitary flow profile (which is a function of land use), infiltration/inflow (I/I) parameters, and the node at which the flow from the subcatchment enters the modeled system.

- **Model loads** are the flows entering the modeled sewer system from each subcatchment. Model loads include residential and commercial sanitary or base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent I/I (RDI/I). As a sum, they represent the total wastewater flow applied to the model.
- **Models** are the combination of a modeled network, its associated subcatchments and loads, and other data (e.g., rainfall, diurnal profiles, inflows from other areas, etc.) that comprise a specific model scenario.

2.2 Modeled System

The model trunk network for the City developed for this study includes pipes upstream of C Street PS that are 10 inches and larger in diameter and additional 6- and 8-inch lines that are either outlet pipes from a flow split and could potentially carry flows from a larger diameter pipe or were considered important because of a significant contributing sewershed. The network also includes the gravity sewers downstream of C Street, Wilmington, and Copeland Pump Stations, and the trunk sewers extending downstream to the PIPS. In total, the network includes about 12.7 miles of pipelines, or about 6.5 percent of the total length of sewers in the entire collection system, or about 30 percent of the total sewer length in the South Area. The modeled network is shown in **Figure 2-1**.

The City receives flow from approximately 548 equivalent single family dwelling units (ESDs) in the Penngrove, an unincorporated Sonoma County community situated between Petaluma and Cotati. The Penngrove collection system, called the Penngrove Sanitation Zone, is owned and operated by the Sonoma County Water Agency (a.k.a. Sonoma Water). Flow from Penngrove is collected at the Penngrove Pumping Station and pumped to Petaluma via a 6-inch diameter force main that discharges upstream of Wilmington Pump Station.

All flow from the City's sewer system discharges to the Primary Pond Influent Pump Station (PIPS) and is pumped to the Ellis Creek Water Recycling Facility via a 2.5-mile long, 36-inch diameter force main. The City is in the process of designing a parallel force main to increase system reliability and flow redundancy, as the existing force main is over 46 years old. The new parallel force main is anticipated to be constructed and operational by 2023.

The City's existing service area was divided into subcatchments; each subcatchment "loads" to a manhole in the modeled network. There are approximately 70 subcatchments in the South Area, ranging in size from less than 1 acre to 30 acres. Subcatchments for other parts of the system were also developed to accurately represent flows in the trunk sewer downstream of C Street PS, but generally represent larger sewersheds draining to the modeled manholes downstream of the major pump stations.

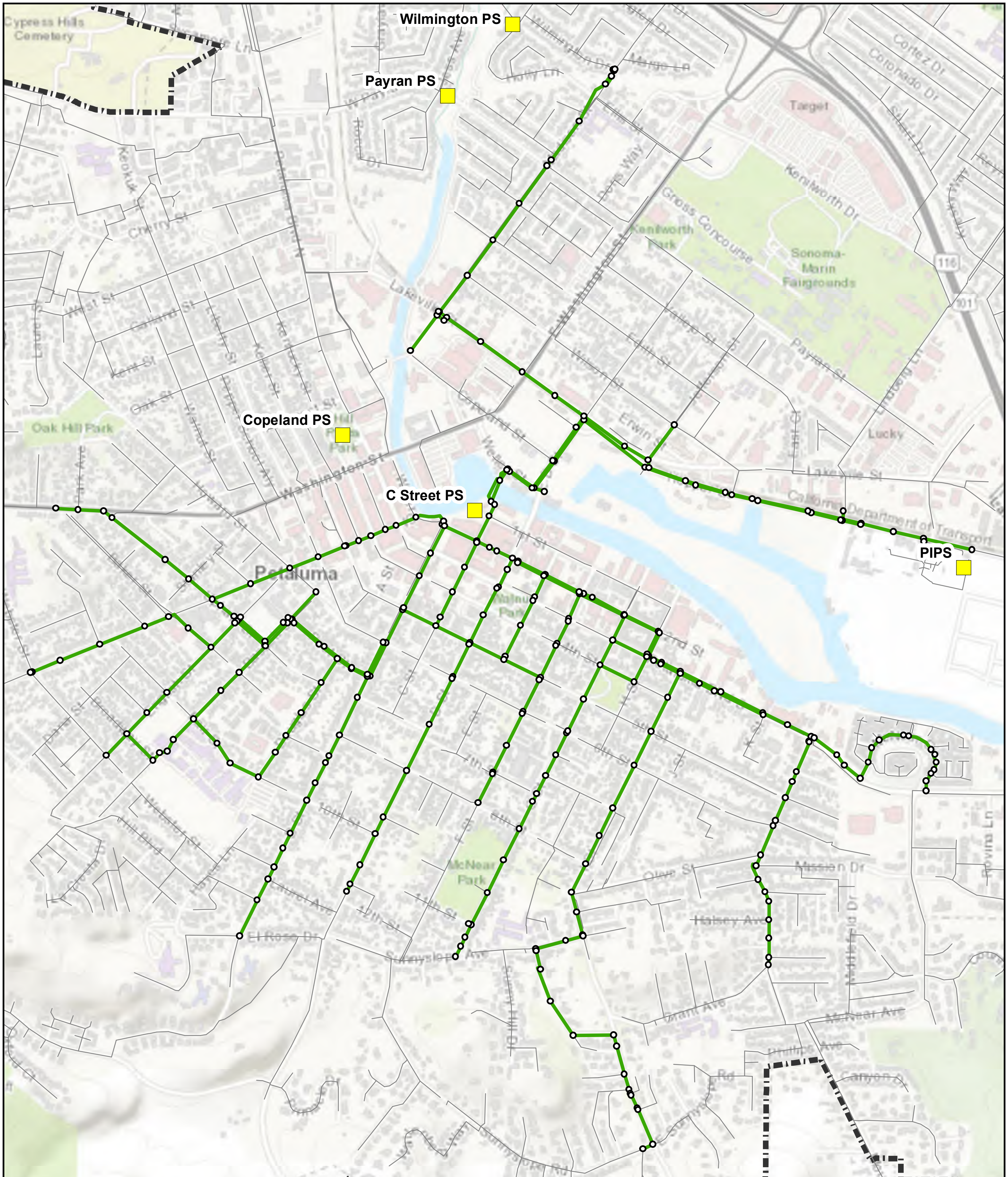

Figure 2-1 Modeled Network

South Area Model Development and Capacity Analysis

Project #: 0011462.00
Map Created: November 2020

2.2.1 Model Network Construction and Validation

The data used to define the Petaluma model network was provided by the City in the form of a PDF map (ssinvert.pdf) and a steady state AutoCAD Civil 3D Storm Sewer Analysis (based on EPA SWMM) model developed by the City which included a small portion of the sewer system pipelines and manholes in the South Area. The pipes and manholes to be included in the modeled network, described previously, were then extracted out of those datasets; these files were imported into the modeling environment in InfoWorks.

The model construction and validation process included the following:

- The modeled network was checked for connectivity, i.e., verifying that the correct upstream/downstream manholes were identified for each pipe and that there were no missing links in the network.
- Model loading manholes were assigned to all subcatchments.
- Manhole and pipeline network data, including rim and invert elevations and pipeline sizes, were refined from the information in the PDF map based on the following data sources:
  - An AutoCAD Storm Sewer Analysis model developed by the City for previous capacity analyses included many of the larger diameter trunk sewers. Rim and invert data from this previous model were used, and updated if more current data (e.g., as-built drawings) were available.
  - In select locations, record drawings for several pipelines were provided by the City and were used to refine elevation, size, and connectivity information. The following as-built drawings were used:
    - 'C' Street Pump Station Upgrade (2012, Project No. C00500205)
    - Copeland Lift Station Rehabilitation Project (2016, Project No. C66501501)
    - Payran Street Pump Station Replacement (1988, Project No. 9684)
    - Wilmington Pump Station (2013, Project No. C00501400)[1]
    - B Street Sewer Replacement (B St, E St, 5th St, & Hinman St) (2019, Project No. C66401941)
  - Where invert elevation data were missing or inconsistent with nearby elevations, and not determined through as-built information, interpolated values between known values were used as appropriate.
  - Elevation data in the PDF map and in the as-builts were adjusted as needed to the NAVD 88 datum.
- Based on the data provided by the sources above, profiles were plotted for each series of pipe segments in the modeled network to visually check for missing or suspect data. Where data indicated a discrepancy (e.g., reverse slope), record drawings or other information were requested from the City, and an approach to resolve the discrepancy was identified.
- The sources of model data (e.g., PDF map, InfoSWMM model, as-built/record drawings,) were documented using "flags" in the model database.
- Subcatchments were delineated to define areas tributary to the modeled pipe network. Each subcatchment was assigned to a manhole in the modeled system to define where the model load from that subcatchment enters the modeled sewer system.
- All gravity pipelines were modeled assuming a Manning's n of 0.013.

[1] Bid set drawings, not as-builts.

2.3 Flow Monitoring Program

To support the development of the hydraulic model and flow projections for the capacity analysis, a temporary flow monitoring program was conducted as part of this study during the 2019/2020 wet weather season. ADS Environmental Services (ADS), under sub-contract to Woodard & Curran, conducted the monitoring at six sites on trunk sewers in the South Area. In addition, one recording rain gauge was also installed by ADS. The City also provided rainfall data from eight gauges located throughout Petaluma. The location of the flow monitoring sites and rain gauges are shown in **Figure 2-2.** The figure also shows the associated tributary area (basin) for each flow meter. Note that meters were located on separate trunk sewers discharging to C Street PS and were not located downstream of other meters. Therefore, none of the meter tributary areas are "incremental" (areas between the flow meter and tributary basins of the upstream flow meters). **Table 2-1**Table 2-1: Flow Meter Locations lists the flow meter locations and pipe diameters.

The purpose of the flow monitoring program was to quantify system flows in the South Area to provide data with which to calibrate the hydraulic model (discussed later in this TM), and to quantify the I/I response to storm events within various subareas of the South Area. The meters and rain gauges were installed for a two-and-a-half-month period from early January through late March 2020 to capture the flow from the tributary areas.

In addition to the temporary flowmeters and rain gauge, the City also provided pump station flow data for the C Street, Wilmington, Copeland, and Pond Influent Pump Stations, as well as rainfall data from several rain gauges distributed throughout the City. Data was provided from December 2019 through March 2020, as well as for an earlier event on January 16-17, 2019. Data for periods prior to the flow monitoring period were used for validating and adjusting the calibration, as discussed in **Section 2.5.2.**

Table 2-1: Flow Meter Locations

Flow Meter ID (FM ID)	Manhole ID	Diameter (in)[a]	Location
PET-01	SWJ04000	12	1027 Petaluma Blvd. South (east of Mountain View Ave.)
PET-02	SWH06000	12	395 Mountain View Ave. (south of Petaluma Blvd. South)
PET-03	SWE01000	12	390 I St. (western sewer at 4th St.)
PET-04	SWC06000	18	F St. at 5th St.
PET-05	SWB04000A	10	D St. at 5th St.
PET-06	SWA04000	18	B St. south of Petaluma Blvd. North (eastern sewer)

a. Actual measured diameter used for meter flow calculations may be slightly different than pipe nominal diameter.

Rainfall was recorded at the one ADS gauge (PET-RG01) and at gauges located at eight weather stations throughout the City (operated as part of Sonoma County's Flood Alert system), as shown on **Figure 2-2**. During the flow monitoring period, there was one fairly significant rainfall event that occurred on January 16, 2020, and a somewhat smaller event that occurred on March 15, 2020.

In addition to the events during the flow monitoring period, rainfall and pump station flow data were provided for several smaller events in December 2019 and the large event on January 16-17, 2019. For the large event on January 16-17, 2019, rainfall data in 1-hour increments was provided for the D Street gauge. In addition, rainfall data in 5-minute increments was downloaded for the January 16, 2020 and January 16-17, 2019 event from the Weather Underground website for station KCAPETAL94, which is located at the corner of D Street and Petaluma Boulevard.

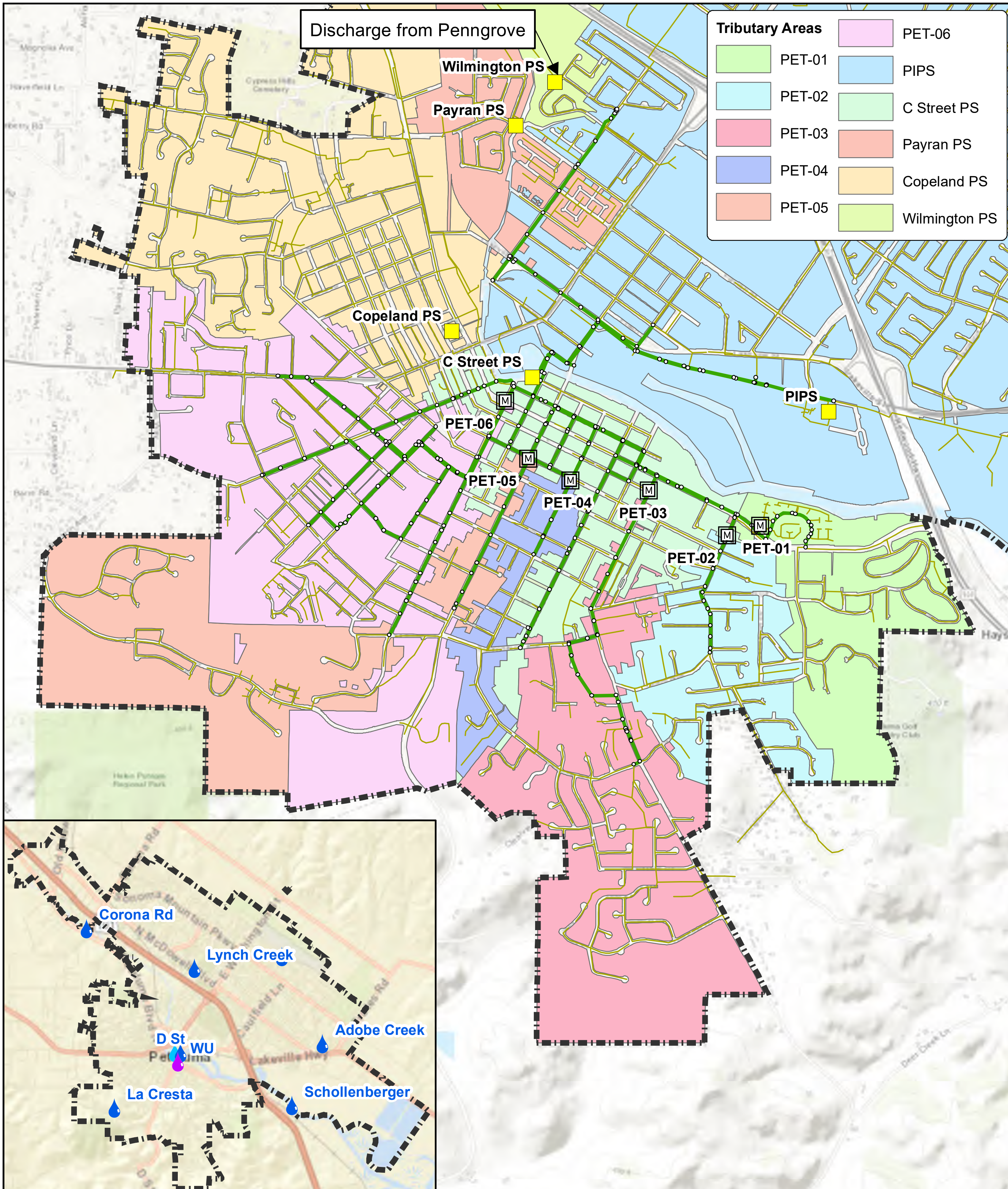

Figure 2-2
Flow Monitoring Sites and Rain Gauges

South Area Model Development and Capacity Analysis

Legend

	Pump Stations		Petaluma Boundary
	Flow Meters		Modeled Manhole
	County Rain Gauge		Modeled Sewer
	ADS Rain Gauge		Unmodeled Sewer
	Weather Underground Station		0 0.125 0.25 0.5 Miles

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Map Created: November 2020

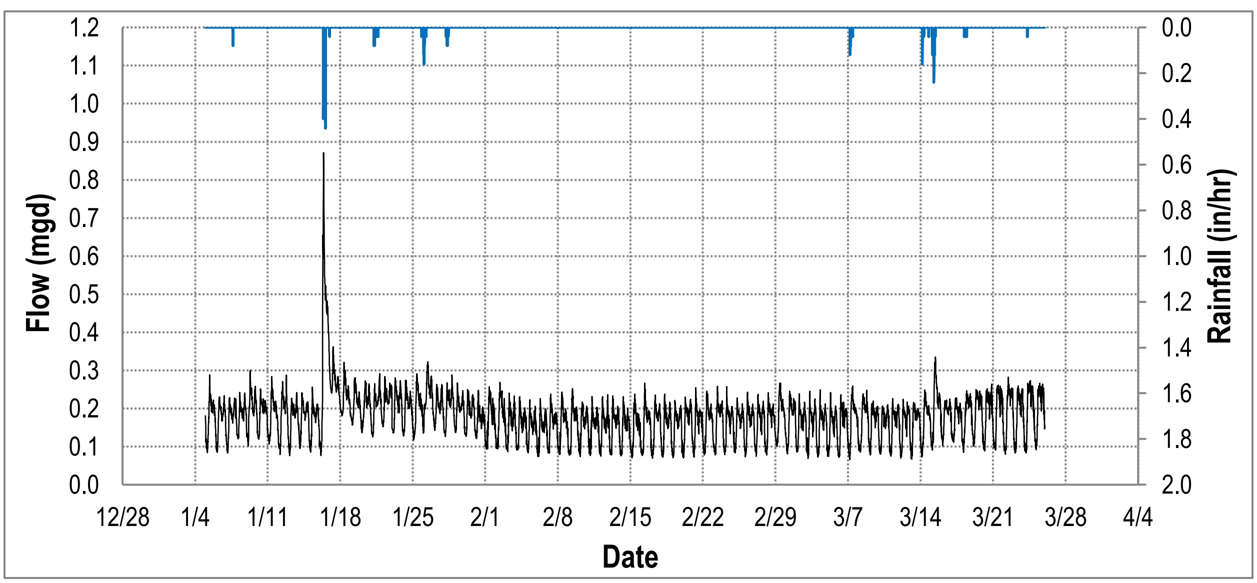
Rainfall data recorded at PET-RG01 was also compared to recorded data from the nearby County gauge (D Street); it was found that PET-RG01 recorded substantially lower rainfall, both for peak intensity and total event depth than County rain gauges for the rainfall event on January 16, 2020, although recorded rainfall for the March 15, 2020 event was similar. The reason for the difference is unclear. This issue, and how it was addressed in calibration, is discussed further in **2.5.2.** Rainfall for key events is summarized in **Table 2-1**. **Figure 2-3** shows a typical plot of measured flow and rainfall for one flow meter. **Appendix A** includes plots of the rainfall and flow data for all of the meters.

Table 2-2: Rainfall Events

Rain Gauge	January 16, 2019		January 16, 2020		March 15, 2020	
	Total 24-Hour Rainfall (in)	Peak 1-hr Intensity (in/hr)	Total 24-Hour Rainfall (in)	Peak 1-hr Intensity (in/hr)	Total 24-Hour Rainfall (in)	Peak 1-Hour Intensity (in/hr)
PET-RG01 (C Street PS)	N/A	N/A	0.44	0.15	0.47	0.18
D Street	2.13	0.40	1.10	0.55	0.40	0.16
Adobe Creek	N/A	N/A	1.10	0.51	0.31	0.08
Corona Road	N/A	N/A	1.02	0.47	0.31	0.15
E Wash Creek	N/A	N/A	1.26	0.59	0.40	0.16
La Cresta	N/A	N/A	1.06	0.59	0.43	0.16
Liberty Road	N/A	N/A	1.18	0.47	0.24	0.08
Lynch Creek	N/A	N/A	1.34	0.59	0.43	0.19
Schollenberger	N/A	N/A	1.03	0.51	0.27	0.12
WU Station KCAPETAL94[1]	1.90	0.43	N/A	N/A	N/A	N/A

a. N/A = Data not available

Figure 2-3: Plot of Typical Flow Data for Flow Monitoring Period (PET-02)



[1] https://www.wunderground.com/weather/us/ca/petaluma/KCAPETAL94

2.4 Flow Estimating Methodology

2.4.1 Wastewater Flow Components

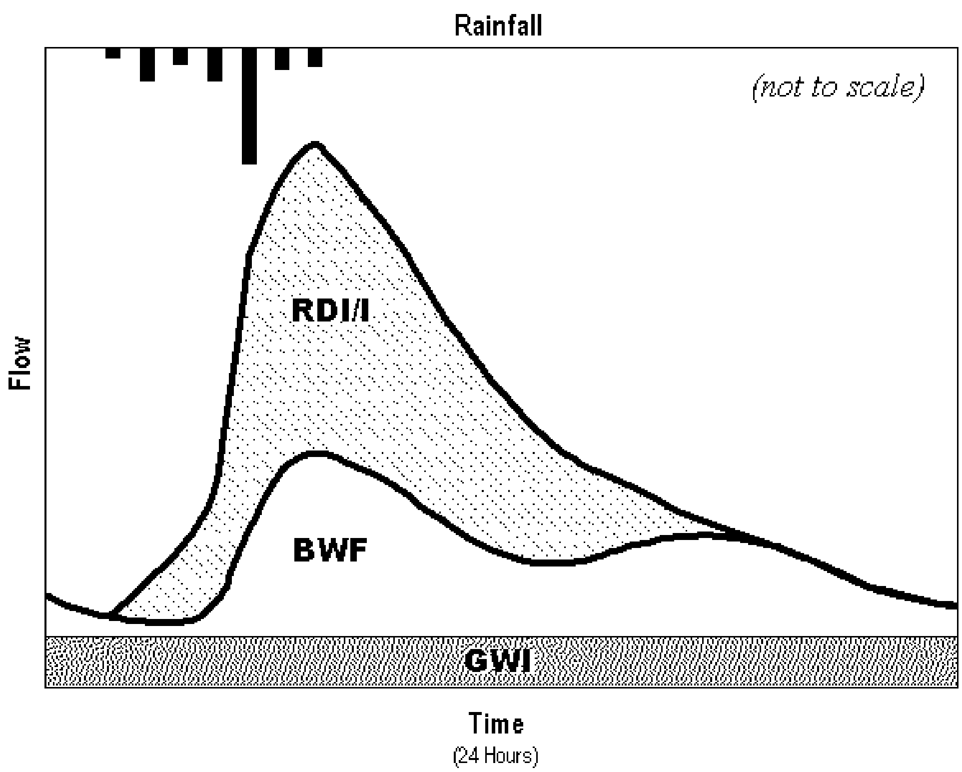
Wastewater flows include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I), as illustrated conceptually in **Figure 2-4.**

BWF represents the sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system. BWF varies throughout the day, but typically follows predictable diurnal patterns depending on the type of land use.

GWI represents groundwater that infiltrates into defects in sewer pipes and manholes, particularly in winter and springtime in low-lying areas. GWI is typically seasonal in nature and remains relatively constant during specific periods of the year. Rainfall typically has long-term impacts on GWI rates, as evidenced by measurable increases in GWI after prolonged periods of rainfall.

RDI/I represents storm water inflow and infiltration that enter the system in direct response to rainfall events, either through direct connections such as holes in manhole covers or illegally connected roof leaders or area drains, or, more commonly, through defects in sewer pipes, manholes, and service laterals. RDI/I typically results in short term peak flows that recede relatively quickly after the rainfall ends. The magnitude of RDI/I flows are related to the intensity and duration of the rainfall, the relative soil moisture at the time of the rainfall event, and the condition of the sewers.

Figure 2-4: Wastewater Flow Components



2.4.2 Base Wastewater Flow

Existing residential and non-residential base wastewater flows were estimated using information compiled at the parcel level (approximately 20,500 parcels) and then aggregated into the 269 model subcatchments. The total residential and non-residential BWF for each model subcatchment were calculated by summing the BWF for all parcels within that subcatchment.

Existing BWF Loads

Existing BWF was determined based on water billing data provided by the City. Metered water use during the winter months most closely approximates wastewater generation, since outdoor water use is at a minimum. Therefore, meter readings averaged over winter months (January, February, March, April) from 2017 through 2019 were used as the basis for estimating residential and non-residential BWF.

All water billing records were geocoded according to parcel APN or to address where parcel APN did not match between the meter shapefile and the water billing data. The geocoded consumption data was assigned a customer type (commercial or residential) based on the Use Code in the water billing data. A visual assessment of the City's water meter locations and parcels using GIS confirmed that data were available for most significantly developed parcels. Water use records were not available for residential parcels in Penngrove. Residential sewer flows from Penngrove were estimated by multiplying the number of parcels discharging to Petaluma's sewer system (376) by the average residential BWF per single family parcel calculated from the Petaluma water billing data (140 gpd). Therefore, BWF from Penngrove was estimated at approximately 80,000 gpd (0.08 mgd).

Diurnal Profiles

BWF varies throughout the day in a typical way, generally peaking early in the morning in most predominantly residential areas. Typical hourly peaks from residential areas tend to be about twice the average flow. Higher peaks can occur on atypical days of the year (e.g., on major holidays such as Thanksgiving or at halftime on Super Bowl Sunday). For Petaluma, typical diurnal profiles were developed for residential and commercial/industrial (non-residential) wastewater flow, for both weekend and weekday conditions. The profiles are applied to the subcatchment BWF in the model. The residential profiles were developed based on monitored flows for primarily residential meter areas, and the non-residential profile is based on typical non-residential flow profiles for similar areas. The diurnal profiles used in the model are shown in **Figure 2-6**. **Figure 2-5** shows the geocoded water billing data by customer type.

Water use records were not available for residential parcels in Penngrove. Residential sewer flows from Penngrove were estimated by multiplying the number of parcels discharging to Petaluma's sewer system (376) by the average residential BWF per single family parcel calculated from the Petaluma water billing data (140 gpd). Therefore, BWF from Penngrove was estimated at approximately 80,000 gpd (0.08 mgd).

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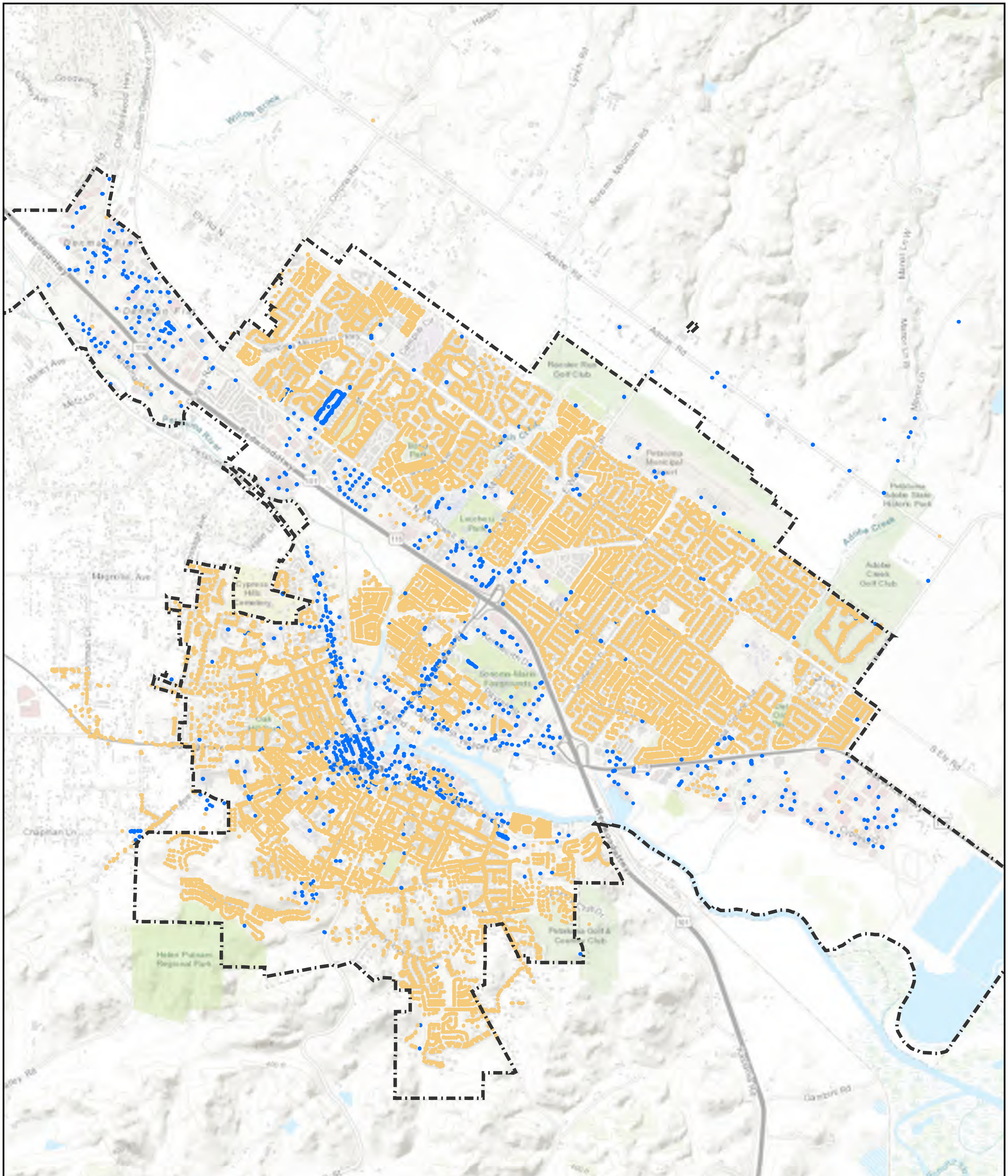

Figure 2-5 Geocoded Water Billing Data

South Area Model Development and Capacity Analysis

Legend

Water Use Data

- Residential
- Non-Residential
- Petaluma Boundary

0 0.175 0.35 0.7 Miles

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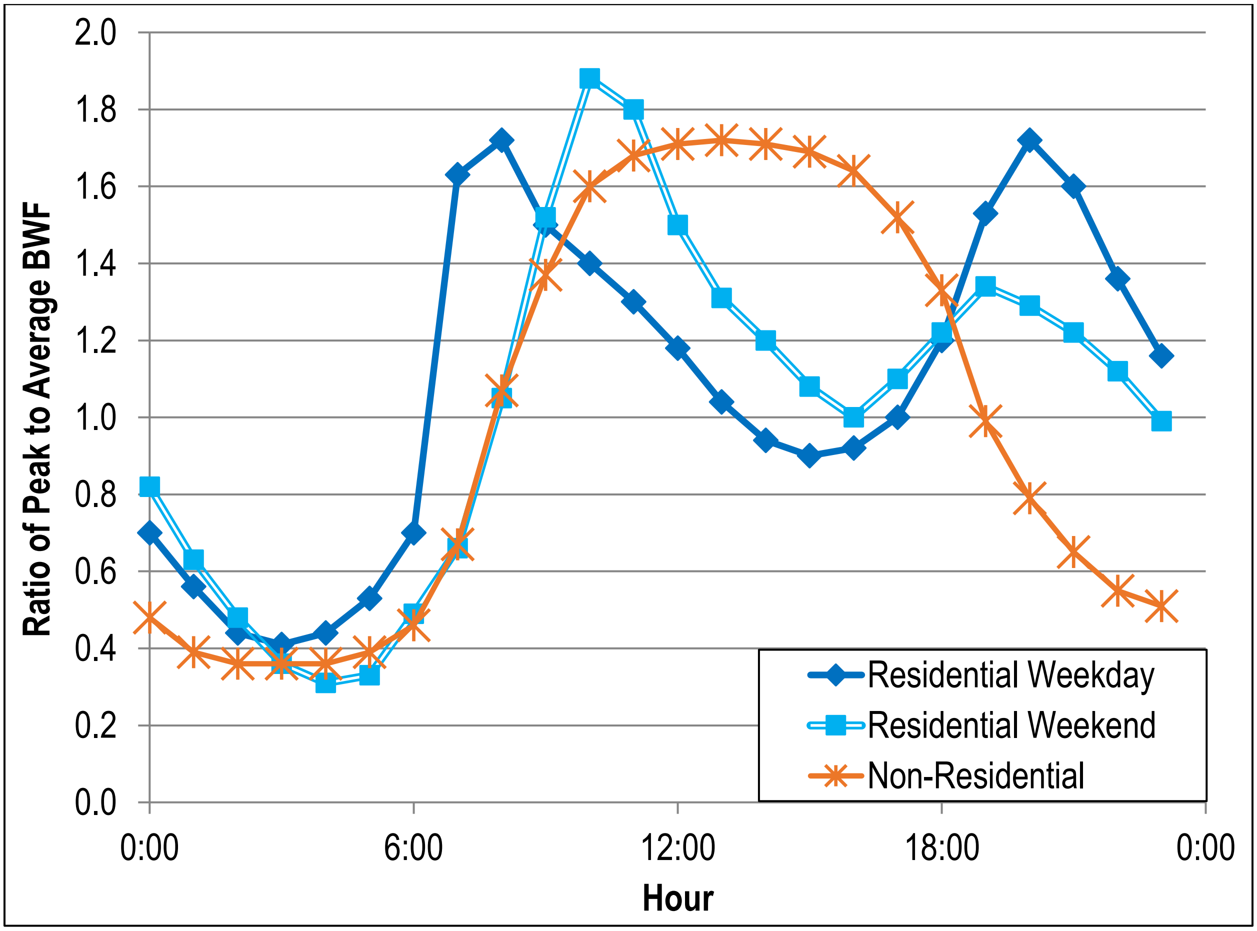
Figure 2-6: Diurnal Profiles



2.4.3 Groundwater Infiltration

GWI is typically applied in the model as a constant load in addition to the BWF. The amount of GWI in any particular area is determined during model calibration by comparing the modeled flows to actual observed dry weather (non-rainfall period) flows at points in the system where flow meter data are available. Where modeled BWF is less than monitored dry weather flow, the difference is assumed to represent GWI. The GWI determined at the monitoring location is then distributed to the meter tributary area on a per-acre basis. Note that because GWI is seasonal in nature, the modeled GWI is intended to represent a typical GWI rate during the wet weather season rather than a dry season (summertime) GWI.

2.4.4 Rainfall-Dependent I/I

RDI/I flows result from rainfall events that produce infiltration and inflow of storm water runoff into the sewer system. RDI/I can be quantified as the difference between the total flow during and immediately following a storm event and the non-rainfall “base flow” (BWF plus GWI) that is estimated to have occurred during the storm period. The magnitude of the resulting RDI/I response is typically described by the percentage of the rainfall volume (called the “R value”) represented by the volume of the RDI/I hydrograph. The R value can vary from storm to storm, depending on such factors as the degree of soil saturation (due to antecedent rainfall) prior to the storm event.

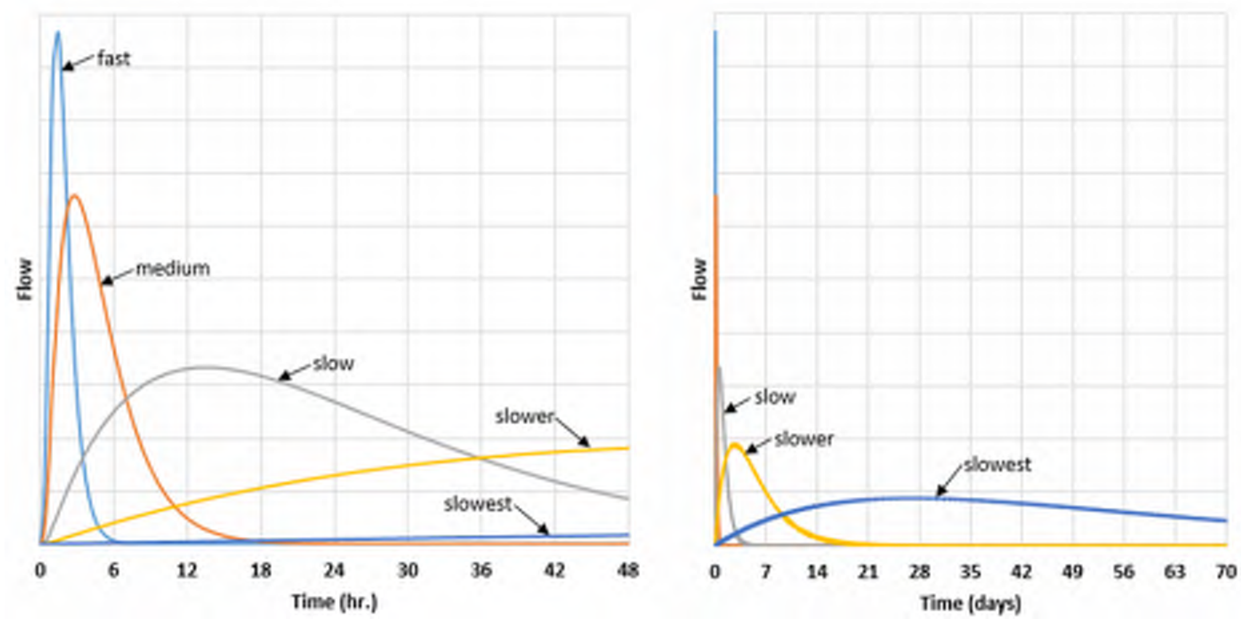
The shape of the RDI/I hydrograph is also important in determining the peak RDI/I response. The RDI/I hydrograph shape is often defined by separating the total RDI/I hydrograph volume into components, representing different response times to rainfall. Up to three or more response patterns may be used, as illustrated in **Figure 2-7**

Figure 2-7. The slowest component may result in a wet weather response several weeks or even months after the rainfall. Alternately, this component could be considered to be a gradual increase in GWI as a result of increased soil saturation and higher groundwater levels after storm events.

Summing all of the component hydrographs for the duration of the rainfall events results in the total RDI/I hydrograph for that area. In most sewer systems, the "fast" component of the hydrograph usually has the biggest impact on the magnitude of the peak wet weather flow response, while the slower components can contribute significantly to the total volume of the RDI/I response. These parameters, when applied to a different rainfall pattern, can be used to estimate the RDI/I response to that particular rainfall event.

The model parameters defining the RDI/I flows to the system within a given meter area are determined by comparing modeled wastewater flow at the meter location to the measured wastewater flow during one or more rainfall events, as discussed in the model calibration section later in this chapter. The same calibrated parameters are generally applied to all subcatchments within each meter area.

Figure 2-7: RDI/I Hydrograph Components



2.5 Model Calibration

2.5.1 Dry Weather Calibration

The 14-day dry period from February 24 to March 9, 2020 was used as the dry weather calibration period for comparing flow data to the model results. This period was selected because it was not impacted by previous rainfall and a majority of the meters showed consistent readings.

The primary focus of the dry weather calibration was to confirm that the calculated average BWF based on winter water consumption was consistent with the measured flows at the meter locations. The other objectives of the dry weather calibration were to confirm the flow routing in the system, particularly in areas where flow can be diverted in more than one direction (flow splits), as well as to confirm the diurnal profiles used to represent the hourly variations in BWF. The diurnal curves shown in **Figure 2-6** were developed based on the calibration.

GWI was added when the observed (metered) dry weather hydrographs were greater than the model-simulated hydrographs by a relatively constant value throughout the day. GWI was applied in three of six flow meter areas: estimated rates of 350, 300, and 120 gpd/acre were applied in flow meter areas PET-02, PET-04, and PET-06, respectively. It should be noted that it may be difficult to assess the actual amount of GWI, as the relative accuracy of the flow monitoring data, water consumption data, and other model assumptions will affect the amount of flow attributed to GWI. However, this methodology is considered adequate for modeling purposes.

Table 2-3 compares the model versus meter average dry weather flow at each meter location, and **Figure 2-8** and **Figure 2-9** show plots of model versus metered dry weather flow for the total flow at the model outfall (PIPS) and C Street PS, respectively. In this graph, the green line represents the monitored (observed) flow, and the red line represents the model-simulated flow. As indicated in **Table 2-3**, the dry weather model calibration resulted in a reasonably good match of modeled to metered flow (within 10 percent at most locations), and to within 2 percent at the model outfall (PIPS).

Table 2-3: Dry Weather Flow Calibration Results

Meter Basin	Meter Avg. Flow (mgd)	Model Avg. Flow (mgd)	Difference (mgd)	Percent Difference
PET-01	0.068	0.074	0.006	9%
PET-02	0.164	0.140	-0.023	-14%
PET-03	0.134	0.125	-0.009	-7%
PET-04	0.087	0.072	-0.015	-17%
PET-05	0.112	0.102	-0.009	-9%
PET-06	0.383	0.367	-0.015	-4%
C Street PS	0.921	1.08	0.154	17%
PIPS	4.70	4.77	0.068	1%
Wilmington PS	1.10	1.27	0.169	15%
Copeland PS [a]	0.272	0.373	0.101	37%

a. Flow meters were only placed in the South Area for this study and therefore, observed flow data to validate Copeland flows were limited. Because observed and predicted flows matched well at the model outfall (PIPS), and wet weather calibration was relatively close, further adjustments were not made to the dry weather calibration.

Figure 2-8: Dry Weather Calibration Graph (PIPS)

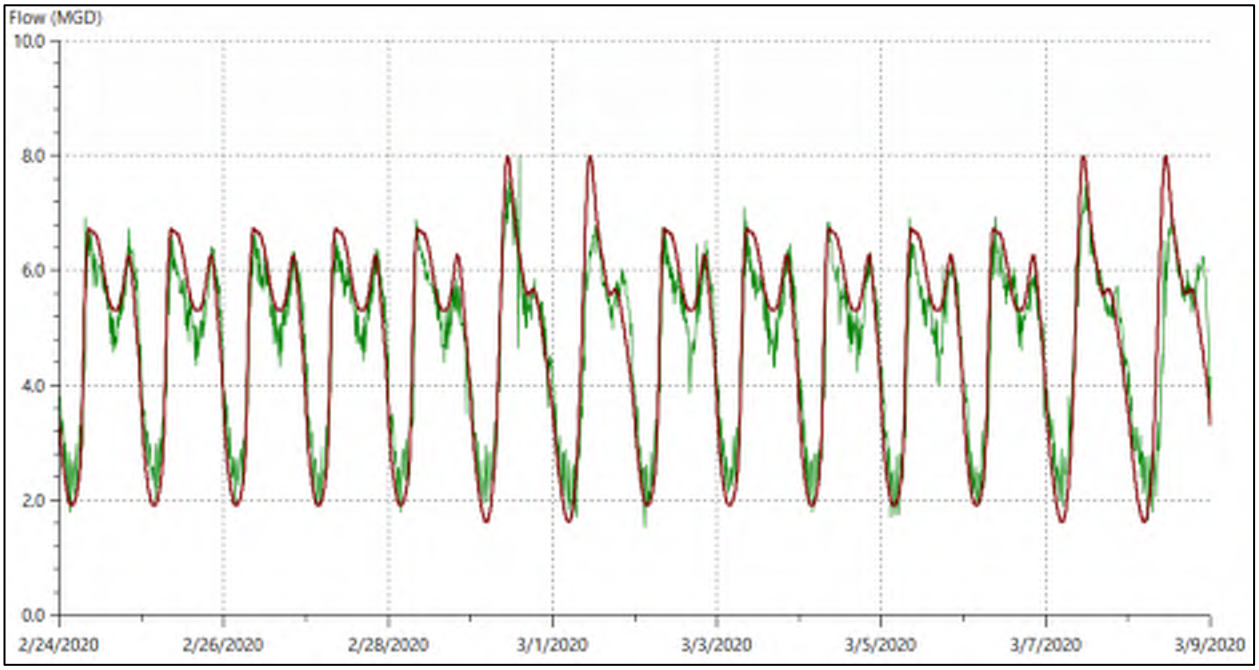


Figure 2-9: Dry Weather Calibration Graph (C Street PS)

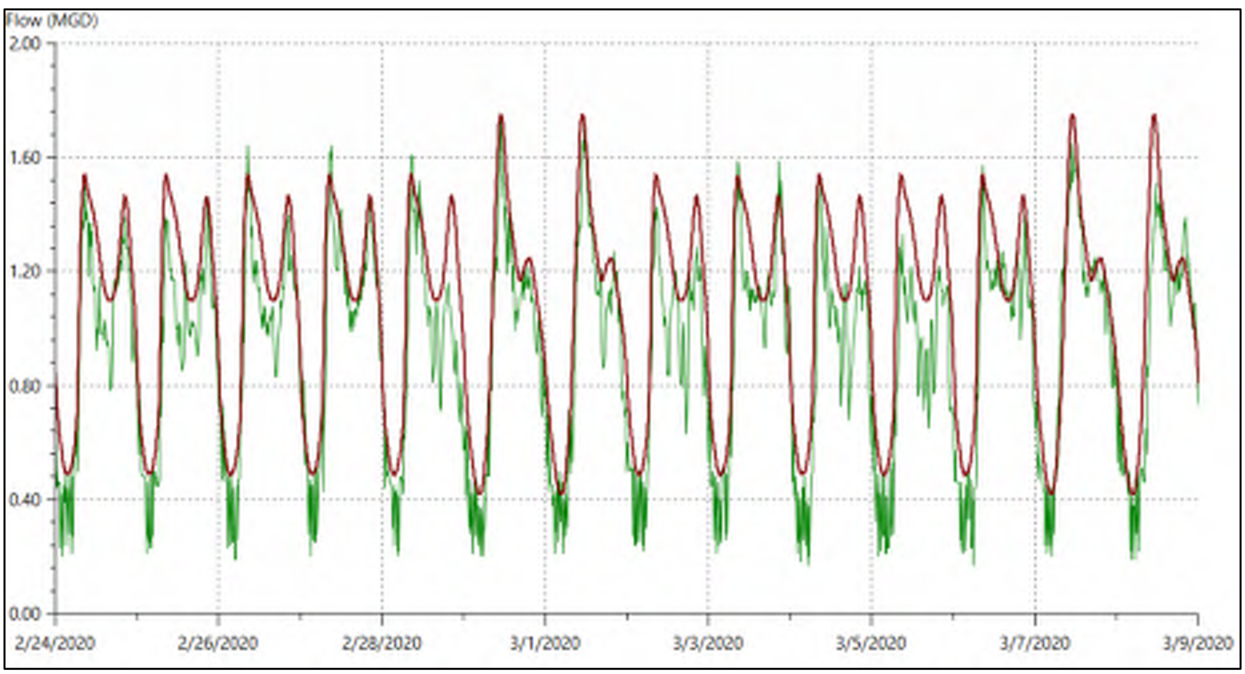


Table 2-4 summarizes the total estimated dry weather flow (DWF) in the entire Petaluma sewer system, including the South Area, based on the model calibration and the existing loads described previously.

Table 2-4: Dry Weather Flow Summary

Flow Component	Flow (mgd)
Residential BWF	2.96
Non-Residential BWF	1.50
Total Average BWF	**4.46**
Estimated GWI[a]	0.12
Total Average DWF	**4.58**

a. Calculated for the South Area based on difference between metered non-rainfall period flows and estimated BWF calculated from winter water use data.

2.5.2 Wet Weather Calibration

During wet weather calibration, parameters are adjusted to simulate the volume and timing of RDI/I for monitored storm events. Rainfall was assigned to subcatchments outside of the South Area using data from the closest of 7 rain gauges maintained by the County. Through the wet weather calibration process, RDI/I hydrograph parameters were developed for each metered area.

The wet weather calibration was performed in two phases:

(1) An initial calibration based on the January 16, 2020 and March 15, 2020 events, which was used to estimate I/I parameters for the areas upstream of the temporary flow meters and upstream of the pump stations.

(2) Verification, based on a series of storms in December 2019, as well as a storm event that occurred on January 16, 2019. For the January 16, 2019 storm, only flow data at the C Street PS was available.

As noted earlier, the rainfall recorded by the PET-RG01 meter during the January 16, 2020 event was significantly less than data recorded by other County meters. For the initial calibration, the PET-RG01 data (when available) was used over data from the County's D Street gauge in the South Area, as the gauge had been recently calibrated and used a 0.01-inch tipping bucket rather than the 0.04-inch tipping bucket provided by the County rain gauges. Further, using the PET-RG01 rain gauge rather than the County's D Street rain gauge provided the most conservative estimate of RDI/I response (greater RDI/I volume per unit volume of recorded rainfall). Initial calibration parameters were primarily based on the January 16, 2020 event; the model somewhat overpredicts flow at most meters during the March 15, 2020 storm event, likely due to the long dry period prior to the event. Verification storms used the County's D Street rain gauge. The most significant finding was that, while the model accurately predicted flows to the C Street PS during the January 16, 2020 event as well as the December 2019 events, peak flows were substantially underpredicted during the January 16, 2019 event. A few potential explanations include:

- **Sources of I/I activated only during large events and/or high tidal conditions.** A possible explanation could be that rainfall was significant enough and the tidal conditions high enough during this storm such that additional sources of I/I (related to flooding and high surface and groundwater levels) were activated that did not occur during the December 2019 and January 2020 storms and are thus not reflected in the model's I/I parameters. High tidal conditions occurring during a large storm event could effectively limit the ability of the storm drain system to drain the area in the vicinity of the Petaluma River, potentially increasing the size and duration of I/I sources into the sanitary sewer system.

- **Relatively dry antecedent conditions.** The January 16, 2020 and March 15, 2020 calibration events occurred under relatively dry antecedent conditions. The storm event on January 16, 2019 occurred after a week of moderately wet conditions. Wetter antecedent conditions could increase the observed infiltration, as well as exacerbate the conditions during large events described above. Given the relatively good calibration during the December 2019 period, which was also under relatively wet conditions, antecedent conditions alone are likely insufficient to fully explain the discrepancy.
- **Accuracy of C Street PS flow meter.** During the January 16, 2019 event, the flow meter at C Street PS reached approximately 12.5 mgd and stayed at that level for several hours. This flowrate is consistent with our estimates of pump capacity (see **Section 3**). Furthermore, the data for the C Street PS flow meter during the January 16, 2020 event was consistent with the temporary flow meter data upstream of the pump station. Therefore, we do not have reason to believe there are significant inaccuracies in the C Street PS flow meter data.

To investigate the potential tidal influence further, tidal conditions during the January 16, 2019 and January 16, 2020 events were reviewed (See **Figure 2-10**). Tidal conditions were several feet higher during and before the January 16, 2019 event compared to the January 16, 2020 event (peak rainfall for the January 16, 2020 event occurred during the low tide, while the January 16, 2019 event rainfall generally occurred during the high tide period), which may have contributed to the apparent higher I/I response. Note that high tidal conditions are common during the January period, and high tide conditions during large storms would not be unusual. However, the relative infrequency of the storm events coinciding with high tide conditions does make quantifying and calibrating the combination of effects challenging.

Based on the relatively fast response to rainfall observed for the January 16, 2019 event, and based on discussions with City staff, it was concluded that the most likely explanation is inflow or rapid infiltration sources in the vicinity of the C Street PS. Therefore, additional I/I was added in the model to the area between Petaluma Boulevard and the Petaluma River to better calibrate to the January 2019 event. This additional I/I does result in somewhat overprediction of flows at C Street PS during the smaller events that occurred during the 2019/2020 wet season (particularly for the early December storms) but does not change the calibrated I/I response for the temporary meters, as the additional I/I is added downstream of those meters. It should be noted that this approach to calibration assumes the high tide conditions and relatively wet antecedent conditions that occurred for the January 16, 2019 event.

Based on these findings, there is some uncertainty in the calibrated flows. This uncertainty has been considered in developing project recommendations, as described in **Section 3**. The estimates of overall I/I in the South Area are discussed further in **Section 3.3**.

Figure 2-11 shows the plot of modeled versus metered wet weather flow for the total flow from Petaluma (at the PIPS), and **Table 2-5** summarizes the results of the wet weather calibration in terms of the R values assigned to each flow meter basin. **Appendix B** contains copies of wet weather calibration graphs for all meters and pump stations. Observed and predicted flows match very well at all six flow meters for the January 16, 2020 storm. **Figure 2-12** and **Figure 2-13** present the calibration results at C Street PS for the January 16, 2019 and January 16, 2020 events, including a comparison of calibration results with and without the additional I/I added to the vicinity of C Street PS.

It should also be noted that calibration of flows at the PIPS and at the Wilmington and Copeland Pump Stations currently rely on the accuracy of the meters at those pump stations. While recorded flows from those stations are generally consistent with expected flows based on water consumption data, temporary flow meters upstream or downstream of those pump stations would provide additional confidence in the accuracy of the estimated wet weather I/I rates.

Figure 2-10: Tidal Conditions During January 16, 2019 and January 16, 2020 Events

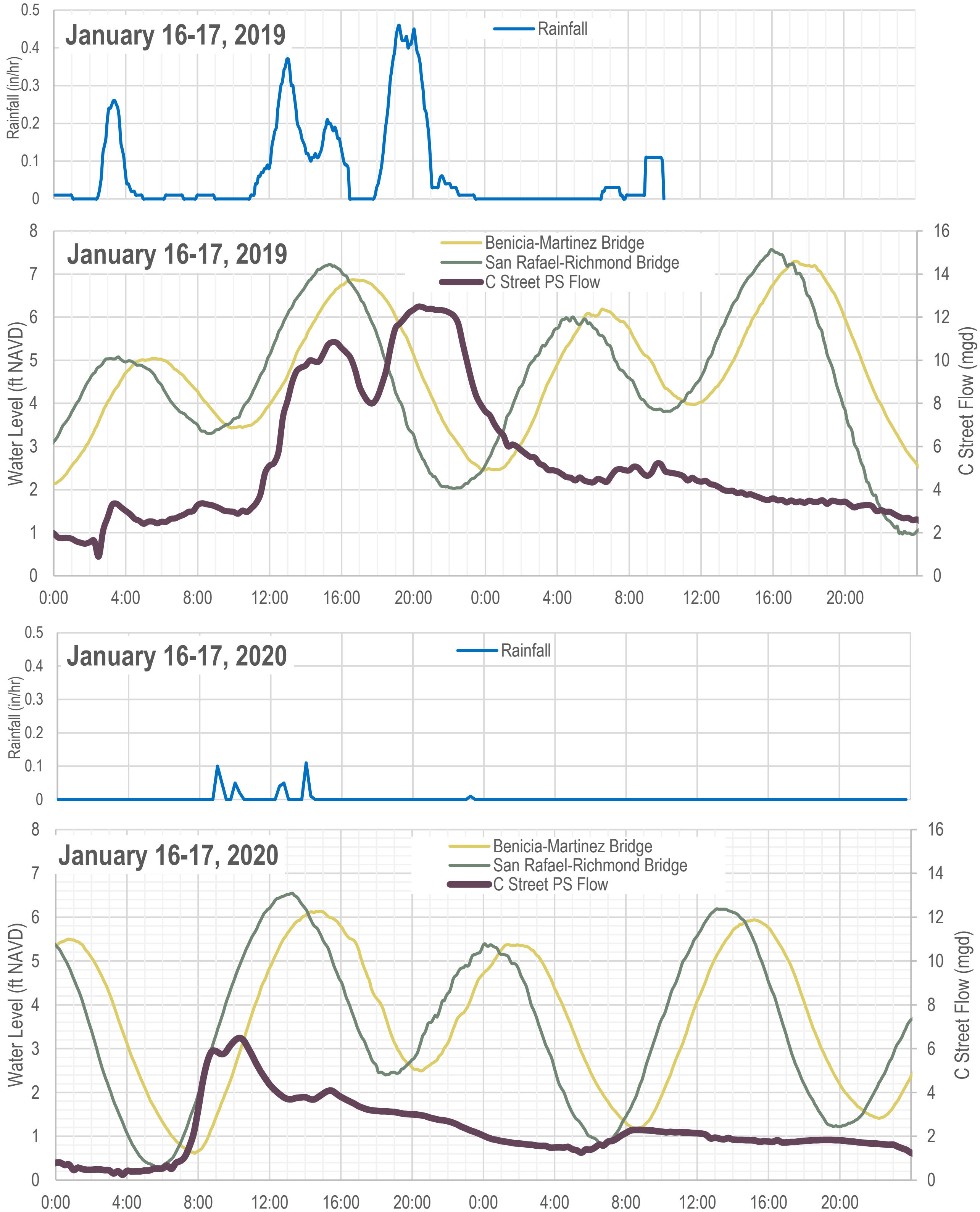


Figure 2-11: Wet Weather Calibration January 16, 2020 Graph (PIPS)

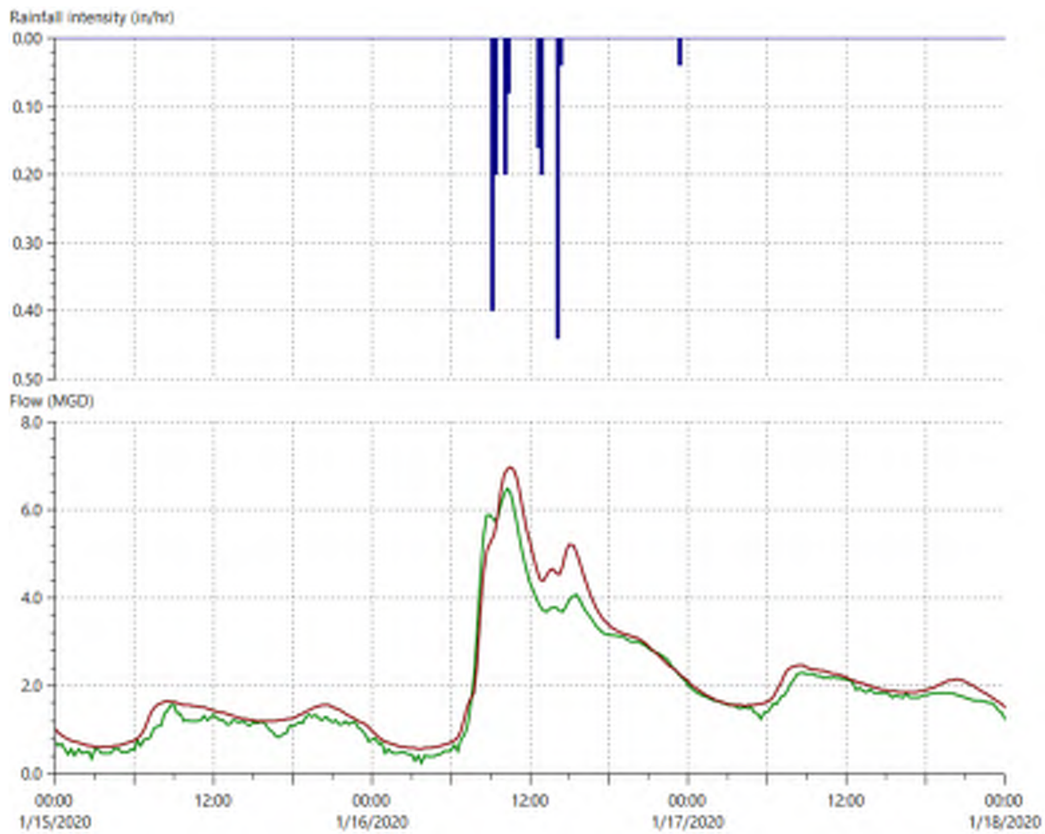


Table 2-5: Wet Weather Calibration Results

Meter Basin	R1 RDI/I Vol. (%)	R2 RDI/I Vol. (%)	R3 RDI/I Vol. (%)	R4 RDI/I Vol. (%)	R5 RDI/I Vol. (%)	Total R Vol. (%)
PET-01	0.3	0	1.8	0	0	2.1
PET-02	2.2	2.8	5.0	16	4.0	30
PET-03	0.1	0.5	2.5	6.0	0	9.1
PET-04	2.8	2.8	10.0	8.0	0	24
PET-05	1.2	1.5	5.0	6.0	4.0	18
PET-06	2.5	3.5	4.0	1.0	0	11
C Street PS High I/I [a]	30	50	5.0	6.0	0	91
C Street PS Remainder [b]	1.4	1.6	5.8	6.7	1.3	17
Copeland PS	1.0	2.0	2.0	0	0	5.0
Payran PS	1.2	1.5	5.0	6.0	0	14
Wilmington PS	0.1	1.0	0	0	0	1.1
PIPS	1.2	1.5	5.0	6.0	0	14

a. Includes the area between Petaluma Blvd. and Petaluma River.
b. Includes the remaining C Street PS area; RDI/I factors are based on an average of PET-03, PET-04, and PET-05, which have similar development characteristics.

Figure 2-12: Wet Weather Calibration January 16, 2020 Graph (C Street PS)

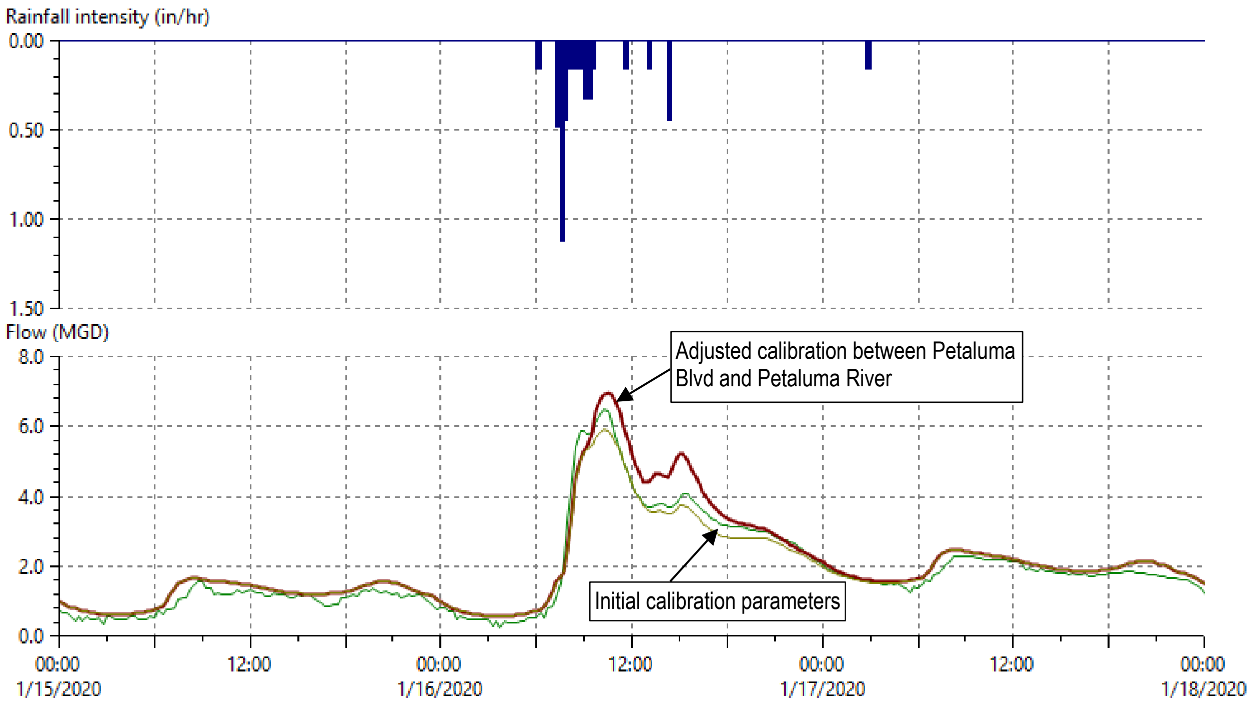
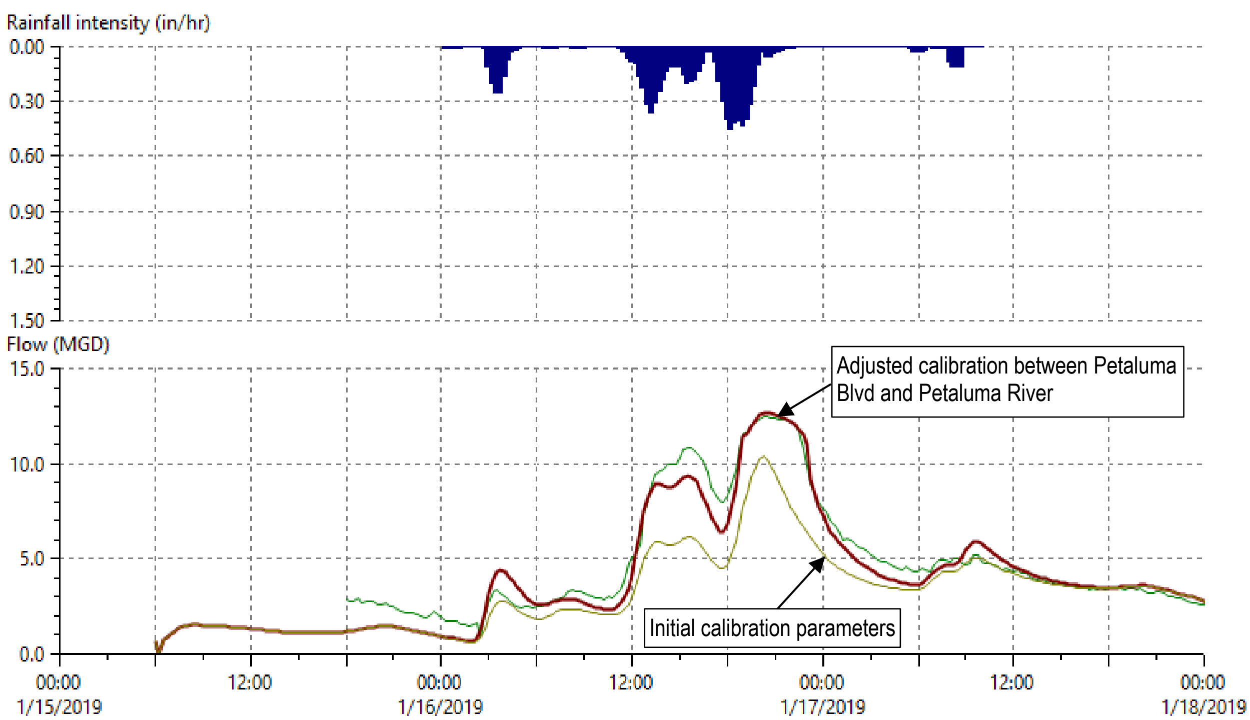


Figure 2-13: Wet Weather Calibration January 16, 2019 Graph (C Street PS)



3. CAPACITY ANALYSIS

The capacity performance of the system and potential need for capacity improvements were evaluated using the calibrated hydraulic model described above. This section discusses the criteria on which the capacity assessment was based and presents the model results.

3.1 Design Flow and Performance Criteria

Sewer system capacity is assessed with respect to the system's performance under a design flow condition. The subsections below define the design flow criteria proposed for the Petaluma capacity assessment and the criteria for assessing system performance and identifying system capacity deficiencies.

3.1.1 Design Storm Condition

The use of wet weather design events as the basis for sewer capacity evaluation is a well-accepted practice. The approach is to first calibrate a hydraulic model of the system to match wet weather flows from observed storm(s), and then apply the calibrated model to a design rainfall event to identify capacity deficiencies and size improvement projects. The design event may be synthesized from rainfall statistics or may be an actual historical rainfall event of appropriate duration and intensity. There is no regulatory standard for design return periods for wastewater collection systems; however, the majority of Bay Area agencies that have adopted a specific return period have selected return periods of 5 or 10 years. Several storm events that could be used as the design event are described below. All design events considered were developed using rainfall volumes from NOAA Atlas 14 point precipitation frequency estimates[1] at the PETALUMA FS2 station. **Table 3-1** summarizes the total volume and peak intensity for each of these potential design events.

- A 5-year, 24-hour design event developed using the SCS Type IA (SCS-IA) distribution[2].
- A 10-year, 24-hour design event developed using the SCS Type IA (SCS-IA) distribution.
- A 5-year, 24-hour design event developed using a nested distribution[3].
- A 10-year, 24-hour design event developed using a nested distribution. This the most conservative design storm considered and was recently adopted by the Sonoma Water Flood Management Manual.

Table 3-1: Potential Design Storm Characteristics

Frequency	Distribution	Volume (in)[a]	Duration (hrs)	Peak Hour Intensity (in/hr)
5-yr, 24-hr	SCS-IA	3.28	24	0.51
10-yr, 24-hr	SCS-IA	3.88	24	0.61
5-yr, 24-hr	Nested	3.28	24	0.67
10-yr, 24-hr	Nested	3.88	24	0.79

a. Rainfall volume at the PETALUMA FS2 station (NOAA Atlas 14).

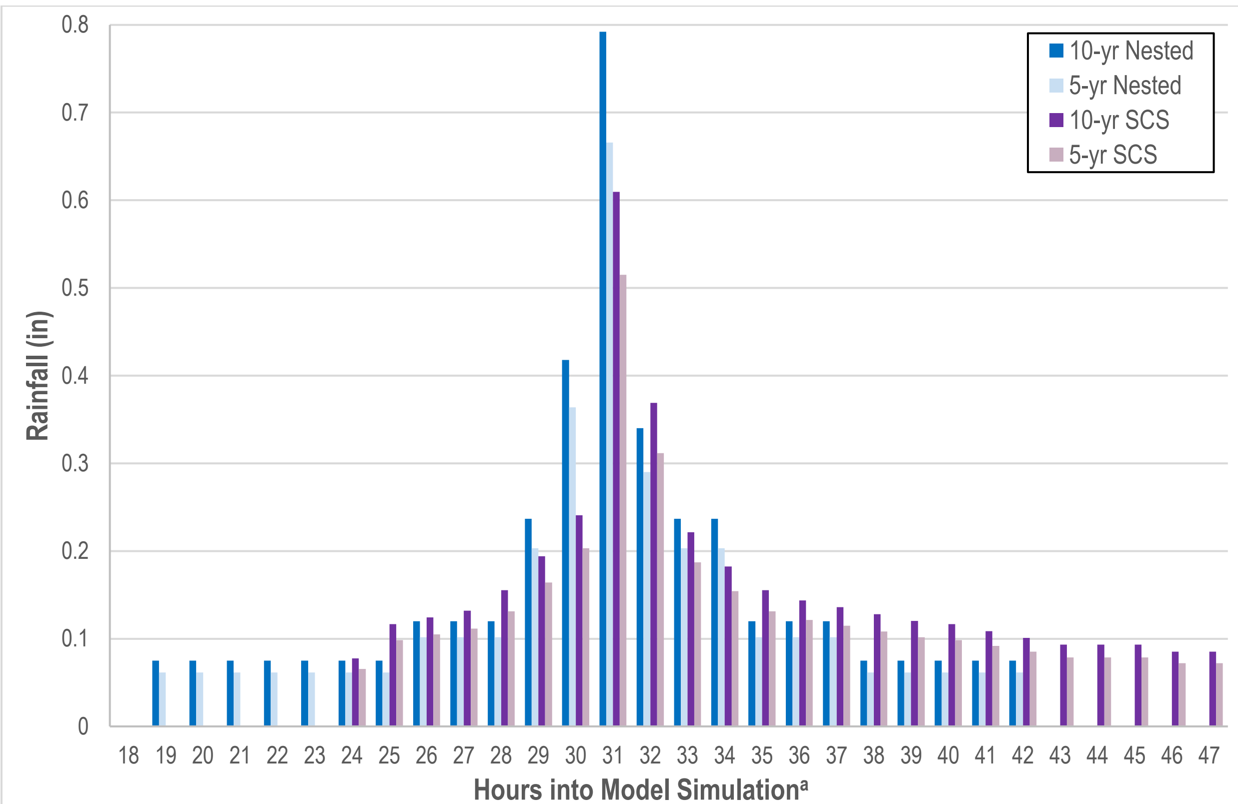
[1] NOAA Atlas 14 Volume 6 Version 2.0 data available at: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca

[2] SCS Standard Rainfall Distributions from the Natural Resources Conservation Service (NRCS) available at: https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?cid=stelprdb1044959

[3] A nested distribution represents a synthetic storm distribution that is generated by placing the highest rainfall intensity at the center of the storm. Lower intensities are placed on alternating sides of the peak, until a complete curve is developed. This distribution is referred to as a nested storm because depths are nested inside each other.

Figure 3-1 shows how the rainfall distributions (volume and intensity) compare for the different storm events considered and indicates that the 10-year, 24-hour nested design event is the most intense. The timing of the design storm also affects the resulting peak wastewater flows. The design storms considered were all timed to generate peak RDI/I at roughly the same time as peak BWF ("peak-on-peak"). The peak-on-peak timing generates a higher total peak wet weather flow than if the peak RDI/I generated by the design storm was adjusted to occur at the time of the average or minimum BWF. Timing the storm to produce peak-on-peak results is generally thought to create a wastewater flow return period that is greater than the return period of the design rainfall event itself (e.g., the peak flow during a 10-year storm event occurring at the same time as peak BWF would occur less often than a 10-year storm occurring at any other time during the day).

Figure 3-1: Comparison of Potential Design Storms



a. Model simulation begins the day before the rainfall event. Hour 31 represents 7 a.m. on a typical BWF diurnal profile.

The City selected the 10-year, 24-hour nested design rainfall event for this study, as it is the most conservative event. Based on this design storm, peak wet weather flow (PWWF) at C Street PS, once all upstream capacity deficiencies in the collection system are relieved, is estimated to be approximately 24 mgd (approximated by doubling the size of all sewers). For comparison, the firm capacity and total capacity at C Street PS are 11.5 mgd and 13.2 mgd, respectively.

It should be noted that using the 10-year, 24-hour nested design event in combination with the relatively conservative calibration approach described in **Section 2.5** potentially results in very high predicted flows, particularly at C Street PS. Additional verification of potential flows is therefore recommended prior to implementation for many of the improvement needs identified.

3.1.2 Capacity Deficiency Criteria

Capacity deficiency or performance criteria are used to determine when the capacity of a sewer pipeline is exceeded to the extent that a capacity improvement project (e.g., a relief sewer or larger replacement sewer) is required. Capacity deficiency criteria are sometimes called "trigger" criteria in that they trigger the need for a capacity improvement project. These criteria may differ from "design criteria" that are applied to determine the size of a new facility, which may be more conservative than the performance criteria.

It is important that the capacity deficiency criteria be coordinated with the peak design flow criteria. For example, if the peak design flow considers only peak dry weather flow and little or no I/I, the deficiency criteria should be conservative (e.g., require pipes to flow less than full under dry weather flow to allow capacity for I/I that may increase the flow under a wet weather condition). On the other hand, if the peak design flow includes I/I from a large, relatively infrequent design storm event, it is appropriate to allow the sewers to flow full or even surcharged to some extent, since the peak flows will be infrequent and brief in duration.

For this study, a capacity deficiency was identified under the following conditions:

- Any modeled surcharging under PDWF
- Any modeled overflows or surcharge reaching within 5 feet of manhole rims under design storm PWWF

In comparison, Sonoma Water allows some surcharging, up to within 5 feet of the manhole rim under design storm PWWF conditions. Exceptions to the 5-foot limit can also be allowed for limited surcharging in shallow pipes that do not impact connecting sewers. However, if an improvement project is developed, the improvement project is sized to eliminate all surcharging at the capacity deficiency location.

As the model is a calibrated fully-dynamic model, the design condition represents a relatively infrequent storm event. Therefore, as many of Petaluma's larger diameter sewers are relatively deep, a criterion similar to Sonoma Water's was applied, with surcharging up to 5 feet of the manhole rims considered acceptable under the 10-year nested design storm PWWF.

For master planning, pump stations are typically considered capacity deficient if the peak design flow exceeds the station's estimated firm capacity (capacity with largest pump out of service). Force mains are considered to be deficient if velocity under peak design flow exceeds 8 to 10 feet per second (fps).

3.2 Capacity Analysis Results

The calibrated model was run for existing conditions (using the RDI/I factors shown in **Table 2-5**) to identify areas of the system that fail to meet the specified performance criteria under design storm PWWF.

3.2.1 Gravity Sewer System Deficiencies

No capacity deficiencies in the system were identified for dry weather conditions. The location of model-predicted surcharged sewers and potential overflows during design storm PWWF conditions are summarized in **Table 3-2** and shown in **Figure 3-2**. The figure identifies five main locations in the South Area where capacity deficiencies were predicted by the model based on the City's criteria. Throttle conditions indicate that the full flow capacity of the pipe is less than the predicted peak flow. It should also be noted that the location of model-predicted surcharge or overflows may not reflect actual physical conditions (e.g., root intrusion or debris) that are not reflected in the model, or system storage that is available in the smaller diameter, unmodeled pipes. Hydraulic profiles of the locations identified are included in **Appendix C**.

Several of the predicted capacity deficiencies (Locations 1 and 2 as identified in **Figure 3-2**) are located in the vicinity of C Street PS, where additional I/I has been applied. However, there is significant uncertainty in both the quantity of I/I in this area, as well as the location of I/I sources. In addition, this area is heavily impacted by backup surcharge from C Street PS. It is recommended that additional I/I investigations or I/I reduction measures and subsequent flow monitoring are performed.

The other predicted capacity deficiencies are in the PET-06 meter tributary area. As discussed in **Section 3.3**, relatively high I/I was observed for this area, which is resulting in the predicted capacity deficiencies in this area. However, PET-06 is a relatively large sewershed. The general approach to applying I/I in the model distributes the I/I evenly to the entire sewershed upstream of the flow meter; however, it is likely that some portions of the sewershed are generating higher I/I than other parts. Therefore, additional flow monitoring is recommended to better isolate the I/I source locations and confirm the model-predicted capacity deficiencies.

As noted above, predicted surcharge in a particular pipe does not necessarily indicate a capacity deficiency at that particular location, as flows can back up due to a downstream capacity deficiency and cause extensive surcharging or even overflows upstream due to backwater effects. However, relieving upstream deficiencies can also create additional or more severe capacity deficiencies downstream of the relieved pipe, and therefore these downstream areas would also require relief (such as Location 3). These effects were considered in developing the capacity improvement projects described in **Section 3.4.**

Table 3-2: Model-Predicted Capacity Deficiencies

Location	Description	US Manhole	DS Manhole	Comments
1	C Street PS and force mains	--	--	C Street PS has reached capacity for several hours during storm events. Existing force mains run at very high velocities - in excess of 12 fps.
2	C Street PS to Mountain View Ave (via C St, 2nd St, H St, Petaluma Blvd S)	SSMH-3768	SW000425	Throttle conditions resulting in several predicted (and observed) overflows. Capacity deficiencies in this area are likely due to high I/I in the vicinity of C Street PS and are exacerbated by backup from C Street PS.
3	B St (South of Post St)	SWA27000	SWA21000	Throttle condition where 10" transitions to 12".
4	Bodega Ave to Bassett St (via Upham St)	SWA56000	SWA30040	Throttle condition from high I/I, resulting in 1 predicted overflow.
5	Webster St to Post St (via Western Ave, Baker St, English St, Upham St, Bassett St)	SWA47000	SWA22000	Throttle condition from high I/I, resulting in 1 predicted overflow.

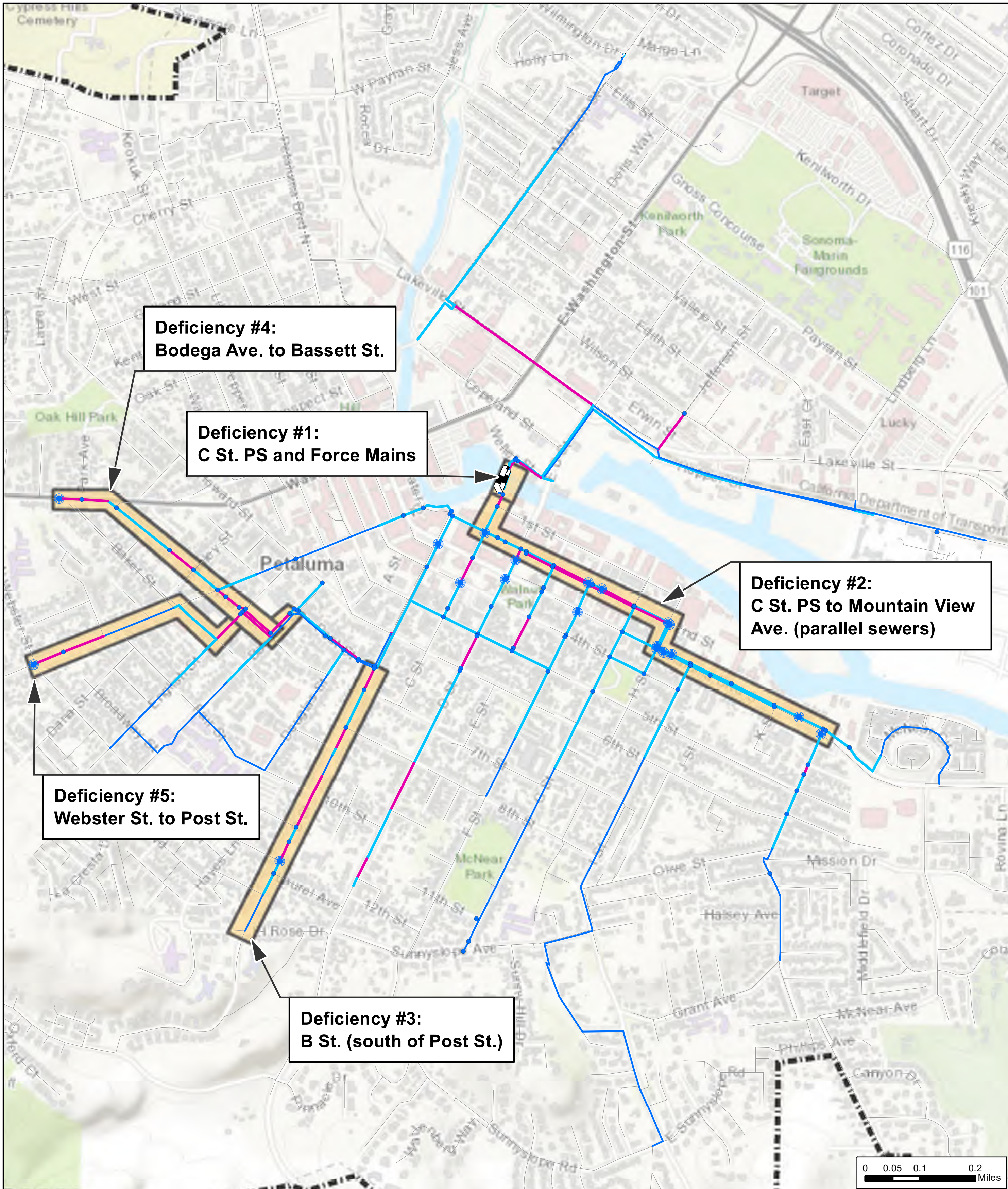


Figure 3-2
Model-Predicted Capacity Deficiencies

South Area Model Development and Capacity Analysis

Legend

Modeled Sewer
- Not Surcharged
- Backwater Surcharge
- Throttle Surcharge
- Model Freeboard < 5 ft
- Model Predicted Overflow
- Unmodeled Sewer
- Force Main
- Capacity Deficiency Area
- Petaluma Boundary

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PETALUMA 1858

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Map Created: November 2020

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3.2.2 C Street Pump Station

During wet weather events, the City has observed overflows in the sewers upstream of C Street PS. During the January 2019 event, C Street PS reached capacity and stayed at capacity for approximately 2 hours. C Street PS has a total capacity of approximately 12.6 mgd from its three (3) pumps; firm capacity is 11.5 mgd with two (2) pumps running. The capacity benefit of the third pump is limited due to friction losses in the two force mains crossing the river.

The existing force mains are running at very high velocities - in excess of 12 feet per second (fps) when the pump station is flowing at capacity. Due to the age of the force mains and the high velocities, the force mains may be nearing the end of their useful lives. The high velocities increase the chance of force main failure due to pipe wall rupture or pulling apart at the joints. A broken force main would result in raw wastewater spilling directly into the Petaluma River. In this scenario the pump station would have to operate with a single force main, further reducing the pump station's capacity, or a temporary floating pipe bridge would need to be installed from the C Street PS to an existing manhole on the north side of the Petaluma River. This temporary force main would likely be needed to convey flow from C Street PS if the break occurred during the winter (which is more likely because the force mains experience the highest velocities during this time of year) to avoid the pump station throttling and potentially overtopping the existing sewer network upstream of the station. To reduce the capacity limitations, high velocities, and significant consequence of failure present in the current system, force main improvements are recommended, as discussed in **Section 3.4**.

3.3 Infiltration and Inflow Discussion

RDI/I was analyzed for each flow meter area based on modeled flows generated for the design storm. Refer to **Section 2.3** for a discussion of the flow meter program and meter locations.

There are various methods for characterizing the relative contributions of RDI/I from different areas of the sewer system. Since the critical issue with respect to RDI/I is the impact of the peak flows that are generated in the system, the focus is on characterizing peak RDI/I in particular. Potential approaches to quantifying peak RDI/I include: the ratio of PWWF to ADWF, referred to as the wet weather peaking factor, for the design storm; peak RDI/I per acre of contributing area; and peak RDI/I per foot of pipe. The RDI/I response in each meter basin based on these parameters is summarized in **Table 3-3.**

As noted in **Table 3-3**, meter basins PET-02, PET-04, and PET-06 all have relatively high I/I rates, using any of the metrics described above. Furthermore, it is expected that I/I may not be evenly distributed throughout the sewersheds; portions of the sewersheds would likely have higher I/I than the overall sewershed, while other portions would be lower. Therefore, it may be useful to perform additional monitoring to better isolate the areas of higher and lower I/I. PET-04 is already a very small sewershed, and a relatively small contributor to overall peak flows at C Street PS. PET-06 is significantly larger, and the currently assumed uniform distribution of I/I upstream of the meter location results in three modeled capacity deficiencies. As noted above, a follow-up flow monitoring program is recommended to further isolate the I/I in this area and confirm the need for any capacity improvement projects. PET-02 is also a relatively large area and should also be considered for follow-up monitoring.

The area upstream of C Street PS, between Petaluma Boulevard South and the Petaluma River, is indicated to have an extremely high wet weather peaking factor, as shown in **Table 3-3**. The I/I estimate for this area should be considered very approximate, as described in **Section 2.5**. Unfortunately, there is no effective way to isolate this area to provide more accurate estimates of I/I. Therefore, other source detection methods may be necessary to better understand potential I/I from this area.

Once an area of high I/I is identified, a number of I/I source detection methods may be used to identify the specific defects that are causing the high I/I. These methods are summarized in **Appendix D**. For the area in the vicinity of C Street PS suspected of having high I/I, smoke testing is likely to be the most cost-effective method of identifying any significant sources, although review of CCTV and manhole inspection records is also recommended. In addition, this area is low lying, and any buildings with basements are likely to have sump pumps. It is possible that some sump pumps may be directly connected to the sewer system, rather than the storm system. Connected sump pumps would not be identified by any of the source detection methods described but building inspectors may have more information about whether these types of connections exist in the area.

Table 3-3: Peak I/I by Flow Meter Area

Flow Meter Basin	Contributing Area (ac)[b]	Length of Contributing Sewers (ft)	ADWF (mgd)[a,c]	Peak RDI/I (mgd)[d]	PWWF (mgd)[e]	Wet Weather Peaking Factor[h]	Unit Peak RDI/I Rate (gpd/ac)[f]	Unit Peak RDI/I Rate (gpd/ft)[g]
PET-01	148	24,266	0.07	0.23	0.34	5	2,288	14
PET-02	144	29,246	0.14	2.09	2.28	17	15,813	78
PET-03	285	46,071	0.12	0.87	0.87	7	3,050	19
PET-04	66	12,263	0.07	1.17	1.27	18	19,267	104
PET-05	151	31,414	0.10	1.27	1.42	14	8,375	40
PET-06	413	74,697	0.35	6.78	7.30	21	16,444	91
C Street PS High I/I	48	17,329	0.07	9.60	9.69	146	8,380	23
C Street PS Remainder	137	31,717	0.12	1.15	1.33	11	8,376	36

a. Sum of basin flows; does not reflect flow routing through system.
b. Net area of developed parcels.
c. Average dry weather flow. Includes groundwater infiltration during non-rainfall periods, representing approximately 18 percent of overall ADWF (may be higher in some basins and negligible in others).
d. Peak rainfall-dependent I/I flow for design storm. Represents sum of peak flows for individual subcatchments within each basin.
e. Peak wet weather flow for design storm. Represents sum of peak flows for individual subcatchments within each basin; does not reflect flow routing through the system (which would typically reduce the peak flows).
f. Peak RDI/I per contributing acre.
g. Peak RDI/I per foot of sewer.
h. Ratio of PWWF to ADWF.

3.3.1 Recommended Infiltration and Inflow Investigations

Based on the findings of this study, some recommendations related to I/I are recommended and are discussed below.

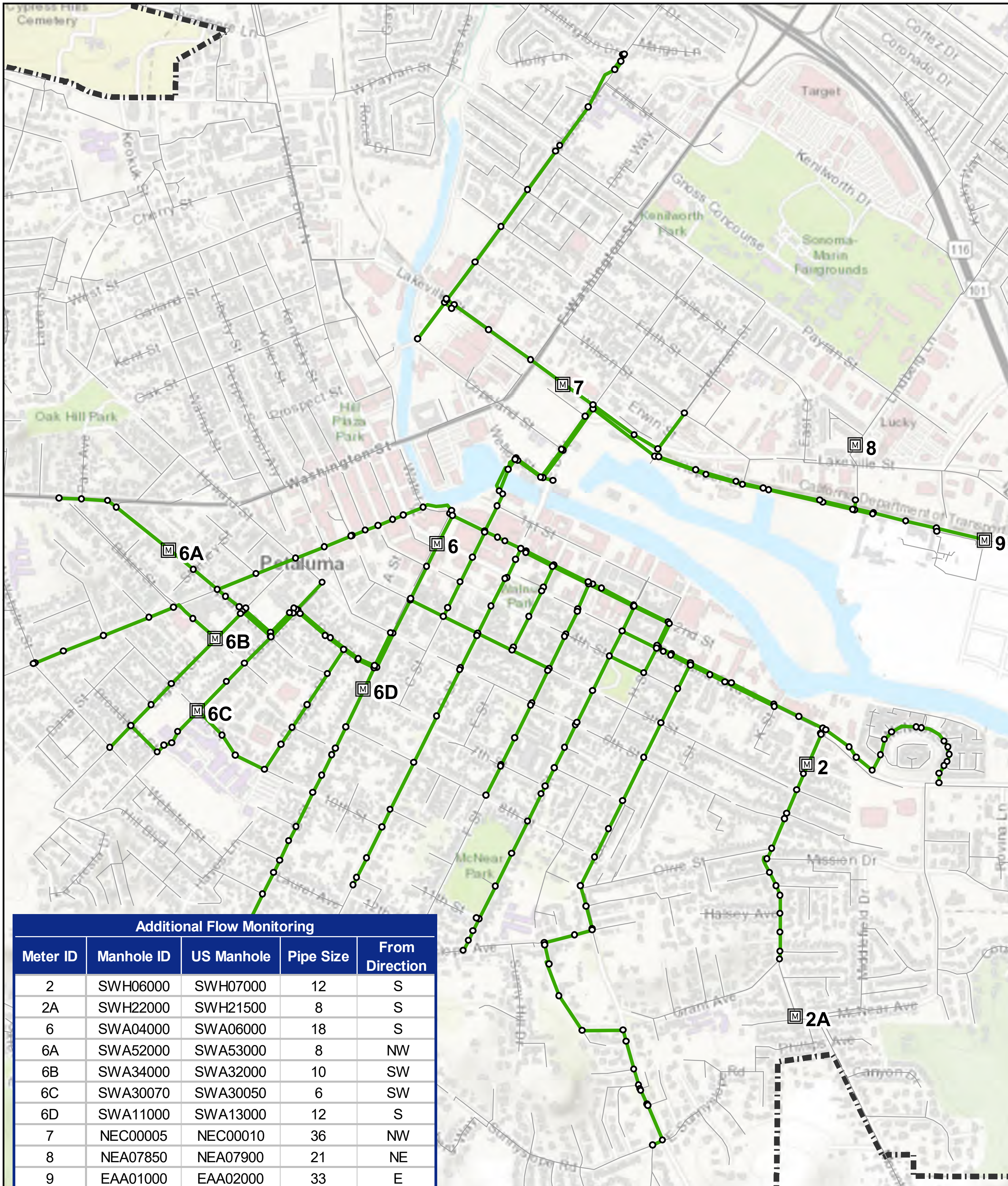
3.3.1.1 Additional Flow Monitoring

Additional flow monitoring is recommended to better isolate areas of the system where significant I/I was observed during the 2019/2020 flow monitoring period, to confirm the need for projects in the locations identified as capacity deficiencies, and to confirm estimated flows in the sewers downstream of the C Street PS. The recommended flow monitoring program would include installation of approximately 10 flow meters and 1 rain gauge. Of the 10 flow meters proposed, 7 would be placed in meter tributary areas PET-02 and PET-06 to better isolate flows, and the remaining 3 will be placed to measure flows into PIPS. The proposed flow monitoring plan has been included in **Figure 3-3**.

These meters could also be included as part of a larger city-wide flow monitoring program.

3.3.1.2 Smoke Testing

To isolate potential direct inflow sources that may be contributing to the high flows observed at C Street PS, it is recommended that the City conduct smoke testing in the downtown area upstream of the pump station. A recommended smoke testing program is shown in **Figure 3-4**. Smoke testing is used to identify potential sources of I/I and is performed by isolating a portion of the sewer system and forcing smoke through the sewer lines. Potential direct inflow sources or indirect connections through drainage paths in the soil are identified by observing where smoke exits the system through drainage connections (e.g., catch basins, area drains, or roof downspouts) or from the ground above potential sewer or lateral defects. The recommended smoke testing program includes approximately 16,000 linear feet of pipe ranging from 6-inches to 30-inches in diameter.



Additional Flow Monitoring				
Meter ID	**Manhole ID**	**US Manhole**	**Pipe Size**	**From Direction**
2	SWH06000	SWH07000	12	S
2A	SWH22000	SWH21500	8	S
6	SWA04000	SWA06000	18	S
6A	SWA52000	SWA53000	8	NW
6B	SWA34000	SWA32000	10	SW
6C	SWA30070	SWA30050	6	SW
6D	SWA11000	SWA13000	12	S
7	NEC00005	NEC00010	36	NW
8	NEA07850	NEA07900	21	NE
9	EAA01000	EAA02000	33	E

Figure 3-3 Additional Flow Monitoring Locations

South Area Model Development and Capacity Analysis

Legend

- Proposed Flow Monitoring Sites
- Modeled Manhole
- Modeled Sewer
- Unmodeled Sewer
- Petaluma Boundary

0 0.05 0.1 0.2 Miles

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PETALUMA 1858

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Map Created: November 2020

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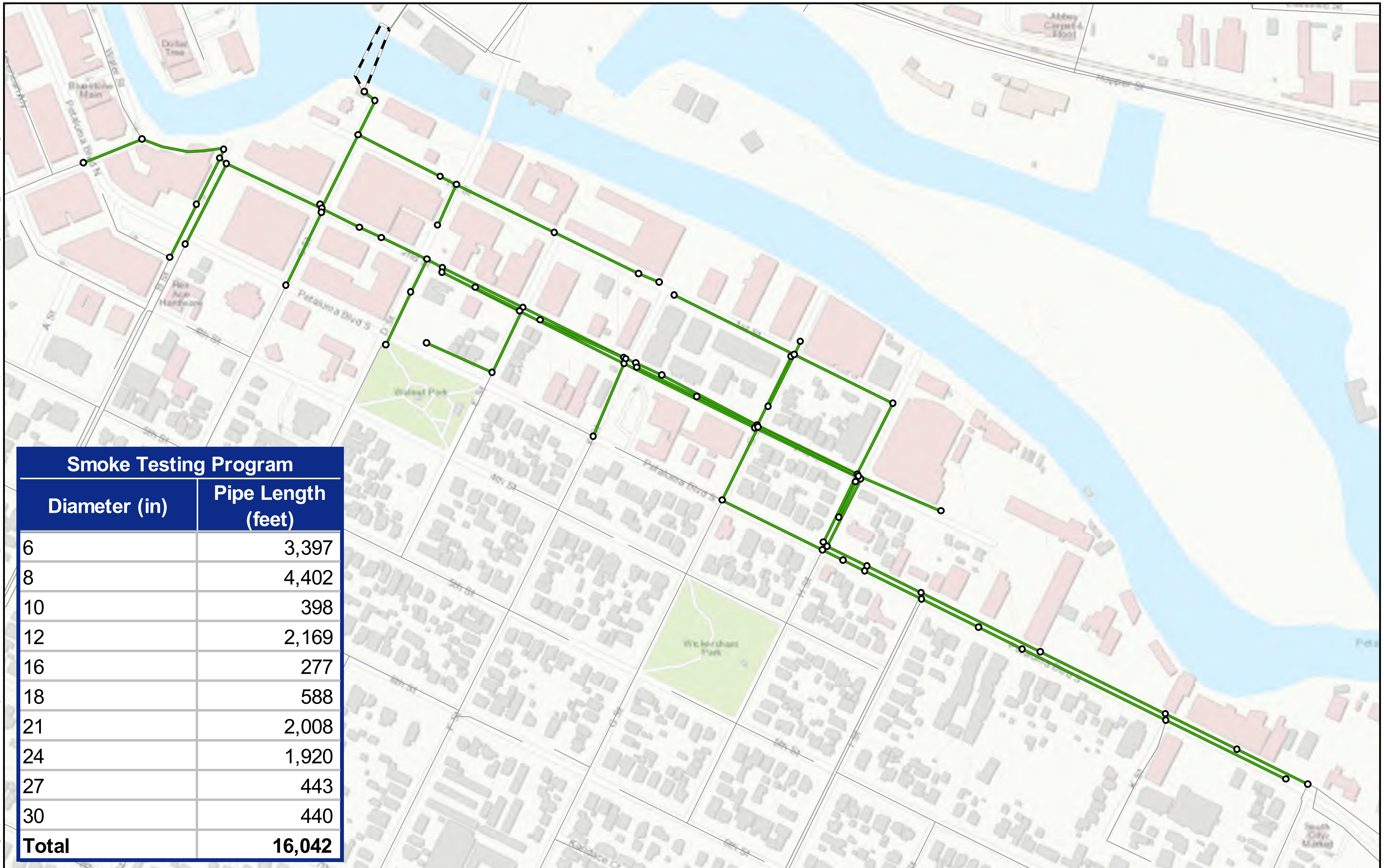

Smoke Testing Program	
Diameter (in)	**Pipe Length (feet)**
6	3,397
8	4,402
10	398
12	2,169
16	277
18	588
21	2,008
24	1,920
27	443
30	440
Total	**16,042**

Figure 3-4
Smoke Testing

South Area Model Development and Capacity Analysis

Legend

- Manholes (Smoke Testing)
- Sewers (Smoke Testing)
- Other Sewers
- Existing Force Main

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Map Created: November 2020

3.4 C Street Pump Station and Force Main Project Recommendations

This section describes the recommendations that the City can move forward with based on results of the initial capacity analysis. These recommendations are discussed in **Section 3.3** and include additional flow monitoring, smoke testing, and initial phasing of upgrading the C Street PS and its discharge force mains.

3.4.1 C Street Pump Station Improvements

Several alternatives have been considered for improvements to the C Street PS, to address both the capacity limitations of the pump station as well as the reliability of the force mains. As described in **Section 3.2**, the existing 8- and 14-inch force mains in particular represent both a significant risk of failure, as well as a significant contributor to capacity limitations at the pump station. For the purpose of this TM, an initial project to install a new 24-inch force main across the river is proposed as shown in **Figure 3-5**Error! Reference source not found., which should provide the lowest cost immediate solution.

The proposed 24-inch pipe could be constructed using directional drilling or with a large diameter microtunneling shaft. Both installation methods are costly but are the only feasible technologies for trenchless pipeline installation given the significant presence of groundwater under the Petaluma River. Microtunneling was recommended based on preliminary analysis, due to its shorter length and comparable cost. However, to reach the depth needed and the minimum bend radius required for the 24-inch pipeline, a directional drill pipeline would need to be approximately 900 feet in length and extend at least 215 feet north of the Petaluma River, which is significantly longer than the existing force mains. This installation would impact an existing under-utilized parcel north of Weller Street. Because this parcel is planned for development, obtaining an easement for this parcel would be difficult; and therefore, directional drilling was not considered as a feasible alternative.

As an alternative to directional drilling, a large diameter microtunneling bore could be performed from Weller Street to C Street Pump Station through the existing sewer easement used by the current force mains. The microtunneling bore requires that a deep jacking and receiving shaft be constructed to ensure a 20-foot clearance from the steel casing installed by the microtunneling machine and the bottom of the Petaluma River. An additional benefit of microtunneling is that several force mains could be installed inside the steel casing once the tunneling is complete. This could allow for operation flexibility and allow for a smaller diameter pipe to be used during dry weather flow. While the preliminary locations of the microtunneling shafts seem feasible, a more detailed investigation into how the pipeline would be installed and operated would be required during a future design phase. Three alternatives for the proposed microtunneling installation, including the recommended alternative shown in **Figure 3-5**, are included in **Appendix F**.

The large variation in flow between dry weather and peak wet weather conditions, however, would make operation of the new 24-inch force main challenging. Under dry weather conditions, the force main would generally not achieve a 2 feet per second minimum cleaning velocity under normal operation. There are three options to address this limitation:

- The force main could be installed with valves to remotely open and close during storm events. Dry weather flows would then need to be conveyed through a smaller pipe – options are discussed further below.
- The pump station could be turned off and allowed to back up during low flow periods (a minimum of 10 minutes of runtime is recommended). This would allow the station to reach minimum cleansing velocities on a daily basis. Note that this could result in the pumps operating at relatively low operating heads during low flow periods, which may be outside the pumps operating regime; pump operation should be confirmed during pre-design. The pre-design should also consider whether the cleansing velocity could be maintained for long enough to clear settled material.

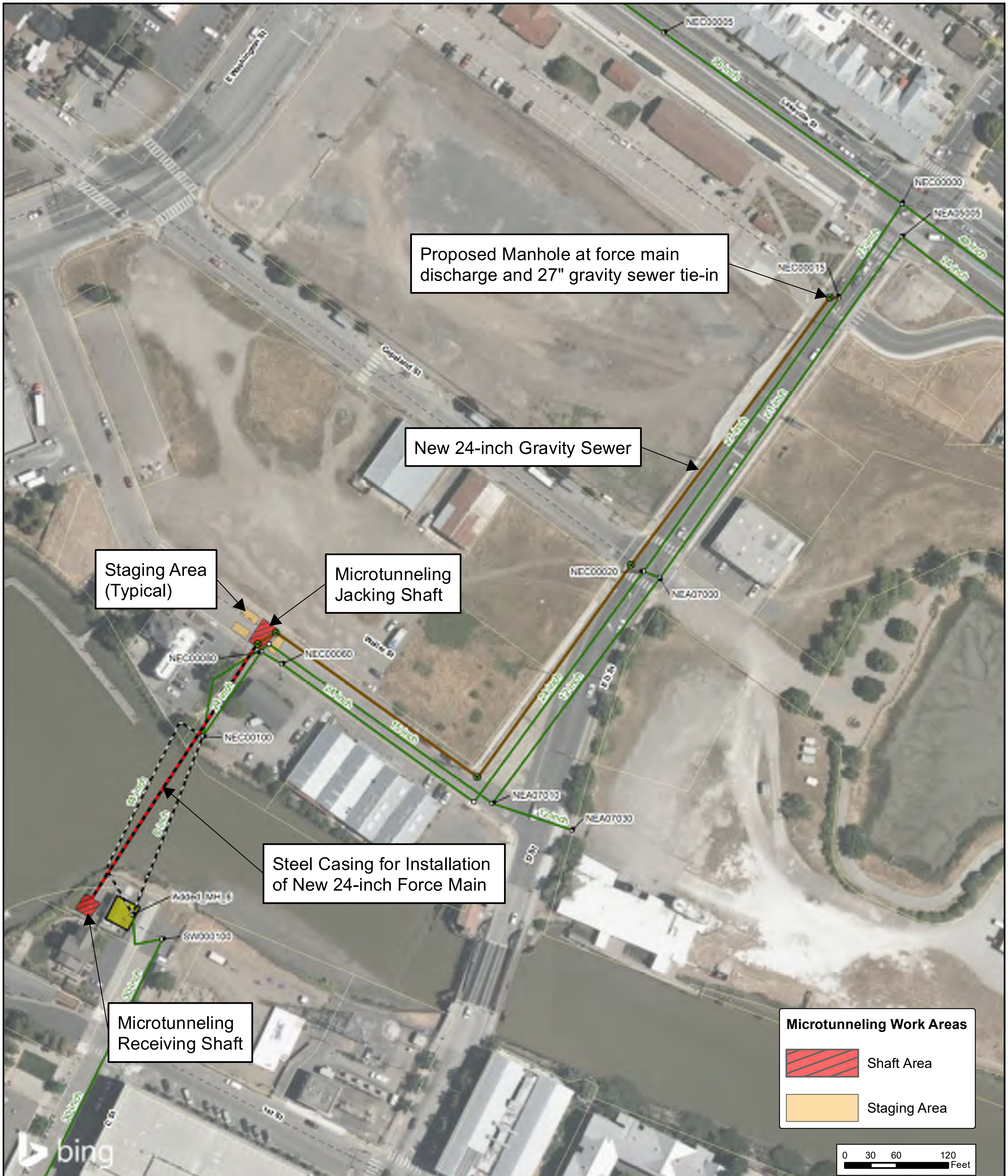

Figure 3-5
Proposed 24-inch Force Main

South Area Model Development and Capacity Analysis

Legend

- New 27" Gravity Sewer
- New 24" Force Main
- Existing Force Main
- Proposed Manhole
- Existing Manhole
- Modeled Sewer
- Unmodeled Sewer
- Parcels
- C Street PS

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WOODARD & CURRAN

Project #: 0011462.00
Map Created: November 2020

- If microtunneling is determined to be feasible it is recommended that two additional 12-inch force mains are installed inside the 54-inch steel casing to provide redundancy and/or a dry weather flow pipe which would meet the 2 feet per second minimum cleaning velocity under normal operation. This also provides redundancy ideally avoiding any need for additional force mains until the end of the pipeline's useful life.

Based on the existing pump performance curves provided by the City, the new force main would allow the existing C Street PS to convey approximately 18.1 mgd with just the 24-inch force main in operation, or 20.5 mgd with both the existing 14-inch and the new 24-inch force mains in operation. As there are uncertainties in the projected flow to C Street PS, if I/I reduction activities in the South Area are effective, this may be sufficient to alleviate the capacity limitations. Pump and system curves for the existing and proposed force main operations are included in **Appendix F**.

The existing force mains discharge into a pair of parallel 15-inch and 24-inch sewers on the north side of the Petaluma River, along Weller Street and E D Street. The 15-inch sewer eventually connects to a 24-inch sewer at Lakeville Street, which connects to a 48-inch sewer on Lakeville Street. Under design storm PWWF, based on model runs with increased C Street PS capacity, the 15-inch and 24-inch sewers can convey up to 10.3 mgd before a model-predicted surcharge reaching within 5 feet of manhole rims is observed and up to 17.8 mgd before a model-predicted overflow is observed. Due to capacity limitations in these sewers, a new parallel gravity sewer main would need to extend to either the 48-inch sewer on Lakeville Street, near manhole NEC00000, or to the 27-inch sewer on East D Street near manhole NEC00015 (which connects to the 48-inch sewer) from manhole NEC00080. Currently a parallel 24-inch gravity sewer is proposed to convey flow from manhole NEC00080 on Weller Street to manhole NEC00000 on Lakeville Street.

Based on review of the model, the additional flow from the added capacity at C Street PS is not predicted to result in any capacity deficiencies in the 48-inch gravity sewer, although the sewer is projected to flow full (no significant surcharge). The model does project a potential capacity shortfall at the PIPS (with the C Street PS improvements, flow to the PIPS would increase to about 50 mgd during the design event compared to an existing design flow of 45 mgd without improvements). The PIPS was constructed in 1972 and was designed to handle a PWWF of 35 mgd, although the 1996 Sewer System Infiltration/Inflow Study[1] estimated that the theoretical firm capacity (with 3 pumps running) ranges from 29 to 34 mgd. The station was designed with 4 pumps in total and has room for 2 additional pumps. The City is currently in the process of designing a parallel force main to convey flow from the PIPS to the Ellis Creek Water Recycling Facility. Construction of a parallel force main was recommended to increase reliability and resiliency; however, the additional force main may also increase pump station capacity if both force mains are in operation. Capacity at the PIPS should be investigated further in coordination with a future systemwide master plan update.

3.4.1.1 Dry Weather Operation

The condition of the existing force mains is not currently known. With the installation of a new 24-inch force main, the 8-inch existing force main would no longer be necessary and could be deactivated or abandoned.

The existing 14-inch force main could be used for dry weather operation, and to provide some additional capacity when necessary during extreme wet weather events. However, because of its age and because it has been subjected to high velocities, a condition assessment of the force main is recommended. If appropriate, cured-in-place pipe (CIPP) lining may effectively rehabilitate the force main.

If the existing 14-inch force main is not suitable for rehabilitation, a new parallel 12-inch force main would be recommended. If the preferred installation method of microtunneling is selected it is recommended that one or two additional 12-inch force mains are installed inside the steel casing installed by the tunneling machine. Due to the high

[1] Sewer System Infiltration/Inflow Study, May 1996, Winzler & Kelly/Montgomery Watson

cost of the steel casing building future redundancy and capacity into a microtunneling project is recommended. If directional drilling is selected as the installation method, a 12-inch force main could be installed using a new directional drill parallel to the proposed 24-inch force main. This force main could be installed in a later phase.

The existing 14-inch (or new 12-inch) pipe could be integrated with a set of valves controlled by actuators and a SCADA system that would switch flow from the existing 14-inch (or new 12-inch) pipe to the proposed 24-inch force main once the pressure in the force main exceeded a set operational point (when velocities in the pipe would exceed a value chosen during design). The larger force main would convey most storm events, but for significant storms, both force mains could be programed to be active and convey flow simultaneously. This system would reduce the strain on each of the force mains and give the City operational flexibility to handle a variety of storm events and dry weather conditions.

3.4.1.2 Improvements to the Pump Station

The operation of the system could also be improved through either a new pump station, or improvements to the existing C Street PS. A new pump station might be preferred for a number of reasons, such as if the existing station is in poor condition (pump station condition has not been evaluated in this study), or susceptibility to flooding due to climate change. The existing C Street PS location is low lying, and in an area that could be subject to significant impacts associated with sea level rise.

Either a new pump station or improvements to the existing pump station could address some of the operational difficulties of the wide range of flow. For example, different dry weather and wet weather pumps could be installed at the existing C Street PS, with the smaller, dry weather pumps connected to the smaller force main and the larger, wet weather pumps connected to the new 24-inch force main. SCADA and valving improvements would also provide a simpler method of operation.

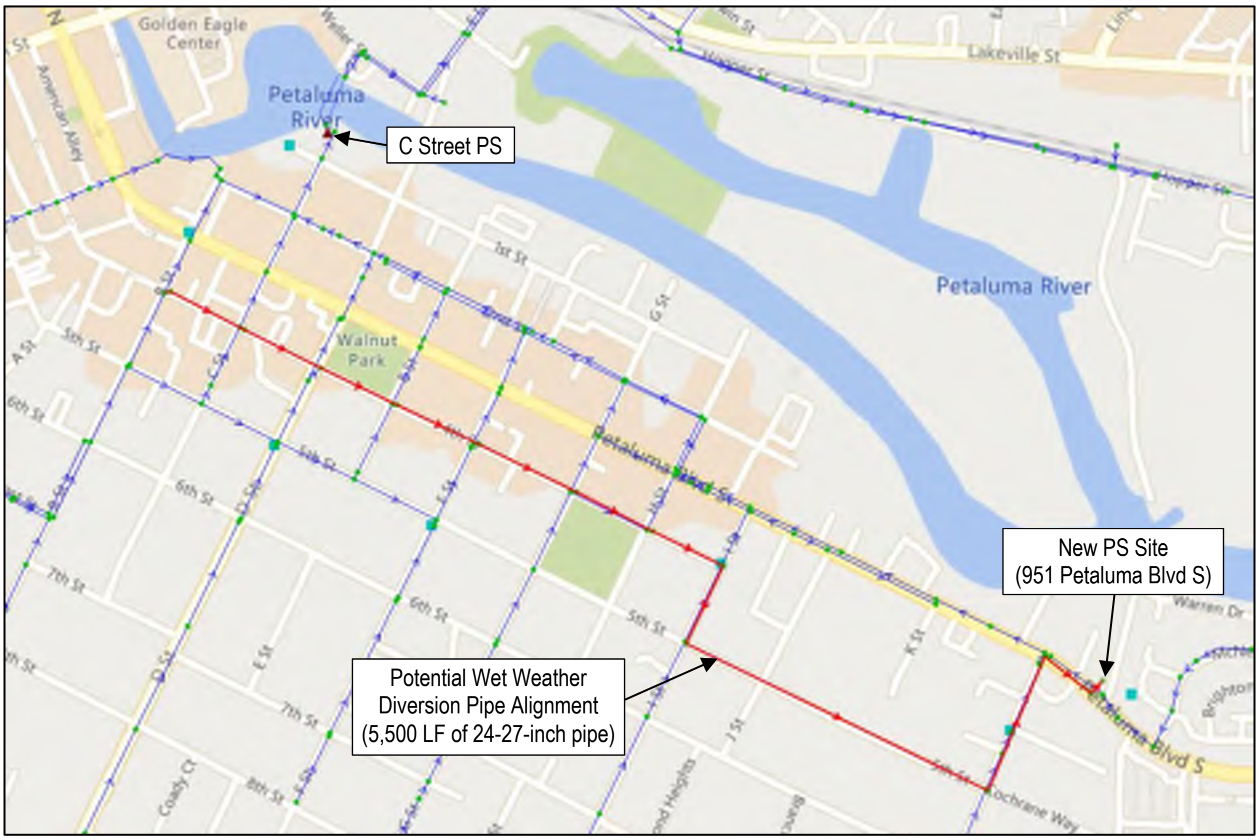
In addition to improvements within the pump station, placing a grit trap upstream of the pump station could limit potential sediment from entering the pump station. Grit can accumulate in force mains, which can reduce the effective capacity of the pump station and contribute to failures. Due to low anticipated cleansing velocities in the force mains under dry weather flows, it is recommended that the City further evaluate if grit accumulation has been an issue at C Street PS before installing a grit trap. A grit trap is typically a passive system using an “oversized manhole” (see **Appendix E** for a detailed description and figure of a grit trap) that would allow influent sediment to settle out of the wastewater as it passes through the grit trap. This type of system has been used effectively by the Oro Loma Sanitary District upstream of inverted siphons (which have similar grit issues) and only requires periodic cleaning using a vacuum truck. If the system is not effective or odors become problematic, the bottom of the grit trap could be filled in and it would act like a traditional manhole.

3.4.1.3 Alternatives for a New Pump Station

In lieu of increasing capacity at C Street PS, the City could install a wet weather diversion pipe to carry surcharge flows to a new pump station that could pump directly to PIPS when C Street PS reaches capacity. The new pump station could be constructed at a City-owned parcel located at 951 Petaluma Boulevard South, which is situated adjacent to the Petaluma River, north of Petaluma Boulevard South, and east of Mountain View Avenue. **Figure 3-6** shows the City-owned parcel where the new pump station could be constructed and a potential alignment for a 24-inch to 27-inch wet weather diversion pipe. The wet weather diversion pipe could be installed along Mountain View Avenue, 5th Street, I Street, and 4th Street, to limit the amount of construction required along Petaluma Boulevard South. Although the modeled wet weather diversion pipe alignment could eliminate model-predicted overflows upstream of C Street PS under the design storm PWWF, it would only convey sewer backups from C Street PS once the capacity of the PS is exceeded and therefore would not reduce the high velocities in the existing C Street PS force mains. Thus, a new C Street PS discharge force main is still be recommended even if the City elects to install a wet weather diversion pipe to carry surcharge flows.

As discussed in **Section 2.5.2**, there is some uncertainty in the model-calibrated sewer flows in the downtown area between Petaluma Boulevard and the Petaluma River. Additional I/I added to the model to better calibrate to the January 2019 event was assumed to be distributed evenly throughout the downtown area but, in reality, may be more concentrated to specific subareas. The recommended smoke testing and additional flow monitoring discussed in **Section 3.3.1** would help identify and pinpoint specific areas of higher I/I within the downtown area, which would better inform the wet weather diversion pipe alignment.

Figure 3-6: Wet Weather Diversion Pipe Alignment



Alternatively, a new pump station could be constructed on the north side of the river to replace the existing C Street PS. This would require either an inverted siphon crossing the river, or a drop structure, a microtunneled gravity sewer river crossing, and a deep wet well for the new pump station. This would allow pump station to be located in a more industrial area and, because it would significantly reduce future flood risk, could potentially open up additional funding opportunities through federal and state programs that provide funding for climate adaptation and resiliency projects (e.g., FEMA Hazard Mitigation Assistance Grants).

3.4.2 Opinion of Probable Capital Cost

An opinion of probable construction costs for the proposed 54-inch steel casing with a 24-inch and two 12-inch force mains and 1,200 ft of 24-inch gravity sewer is presented in **Table 3-4**. A more detailed cost breakdown is included in **Appendix F**. Costs were estimated based on Woodard & Curran's experience with similar projects and include planning, design, and construction. Allowances added to the baseline construction cost include mobilization/demobilization and project-specific costs for installing a force main using a combination of a microtunneling and open cut. Costs for a new SCADA capabilities and valving at the existing station discussed in **Section 3.4.1.1** have not been included in this cost estimate. Costs for a grit trap have also not been included in this estimate as discussed in **Section 3.4.1**. A 30 percent allowance for contingencies for unknown conditions was also included for the project, as well as an allowance of 25 percent of construction cost for engineering, administration, and legal costs.

Table 3-4: 24-inch Force Main Opinion of Probable Cost

Item	Cost
Site Work & Special Construction [a]	$2,617,000
Steel Casing, Force Mains, and Gravity Mains	$1,742,000
Mechanical Equipment [b]	$250,000
Subtotal	$4,609,000
Mobilization (10%)	$461,000
Construction Subtotal	$5,070,000
Contingencies (30%)	$1,521,000
Construction Cost	$6,591,000
Engineering, Administration, Legal (25%)	$1,648,000
Capital Improvement Cost	**$8,239,000**

a. Includes the Jacking and Receiving Shafts
b. Includes valve vault and new discharge piping

4. CONCLUSIONS AND RECOMMENDATIONS

The recommendations of this study are summarized in **Table 4-1**. More detailed improvement recommendations to reduce the risk of overflows due to insufficient capacity during PWWF will be developed after the additional flow monitoring and smoke testing programs, and subsequent analyses to refine model calibration and identify potential I/I reduction efforts, are conducted. Improvement recommendations would be developed to address areas in which predicted peak flows exceed the City's capacity deficiency criteria. Planning-level construction and capital cost estimates will be developed as part of the future improvement recommendations.

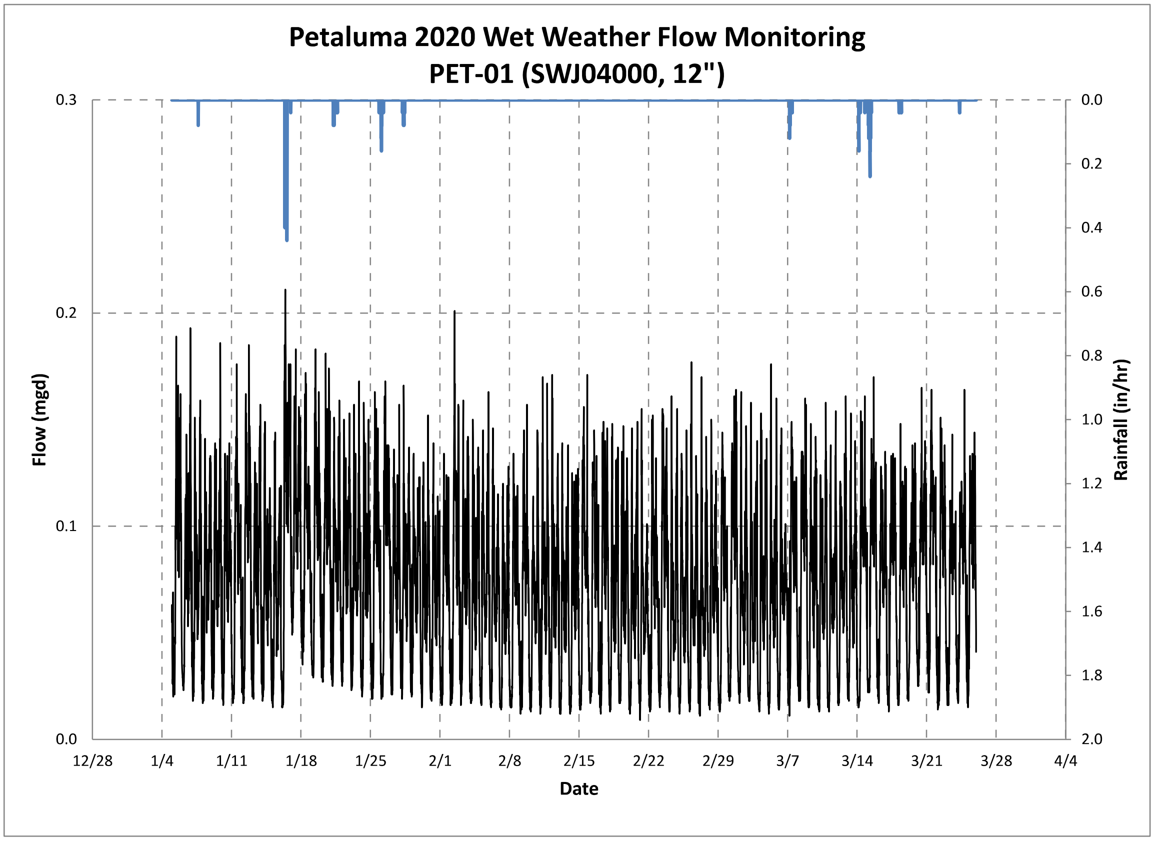
Table 4-1: Summary of Recommendations

Recommendation	Description	Estimated Capital Cost	Comments
Additional Flow Monitoring	Conduct 2020/2021 wet weather flow monitoring program including installation of ~10 flow meters and 1 rain gauge	$ 80,000	Cost includes flow monitoring only and does not include recalibration and subsequent reassessment of the system.
Smoke Testing	Conduct smoke testing of ~16,000 linear feet of pipe ranging from 6-inches to 30-inches in diameter	$ 30,000	Does not include rehabilitation/repair of any defects identified.
Force Main Installation -- C Street PS Improvements	Install a new force main via microtunneling and additional gravity sewers for the C Street PS.	$ 8,239,000	Cost includes 350 linear feet (LF) of force main installed via microtunneling under the Petaluma River and 1,200 LF of 24-inch gravity sewer. Refer to **Appendix F** for details.

In addition, this study has identified a potential capacity deficiency at PIPS during the design event, based on PIPS capacity estimated as part of prior studies. However, the City is currently working on the design of a new parallel 24-inch force main for PIPS, which could increase station capacity if both the existing and new force main are operated in parallel during design events. Based upon the potential impacts of the C-Street Pump Station on the PIPS and the upstream gravity sewer, City staff have decided that a system-wide study to evaluate the impacts and explore potential solutions would be warranted.

APPENDICES

APPENDIX A - PLOTS OF FLOW MONITORING DATA


Petaluma 2020 Wet Weather Flow Monitoring
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Flow (mgd)
Rainfall (in/hr)
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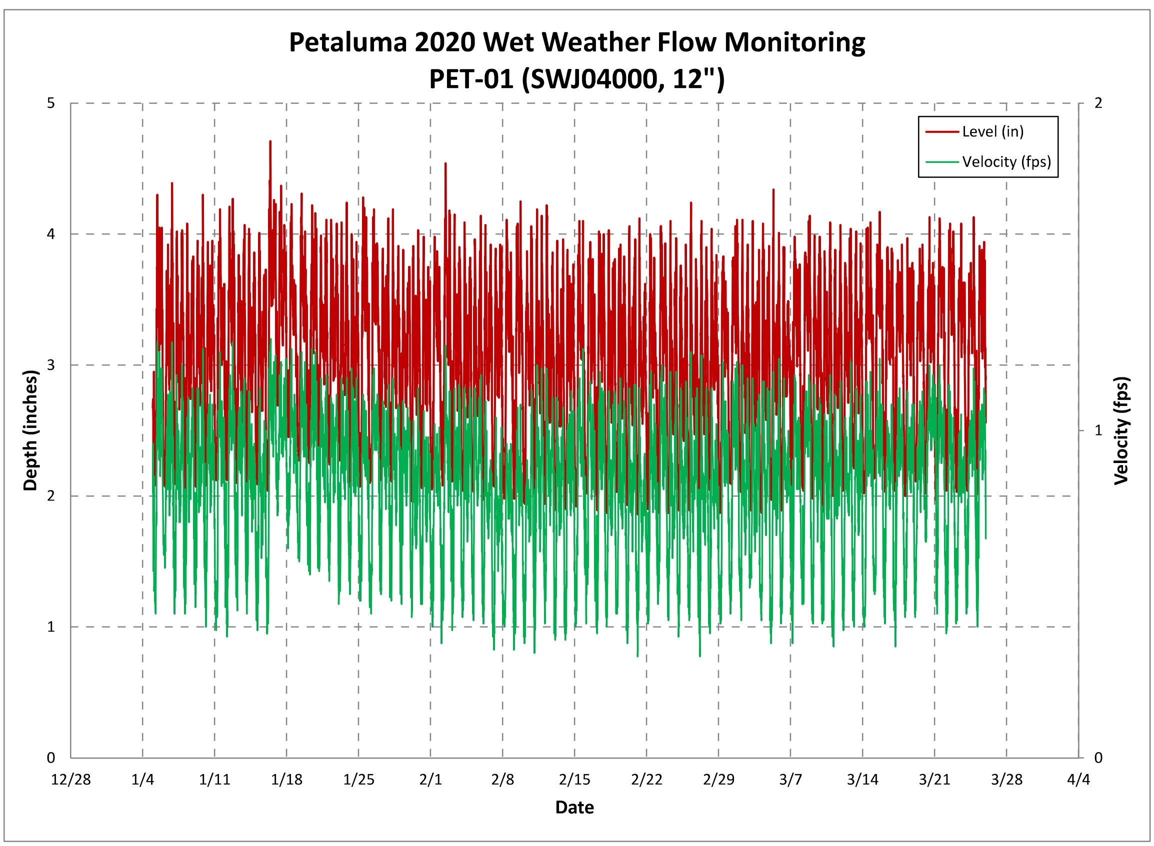
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Velocity (fps)
Depth (inches)
Velocity (fps)
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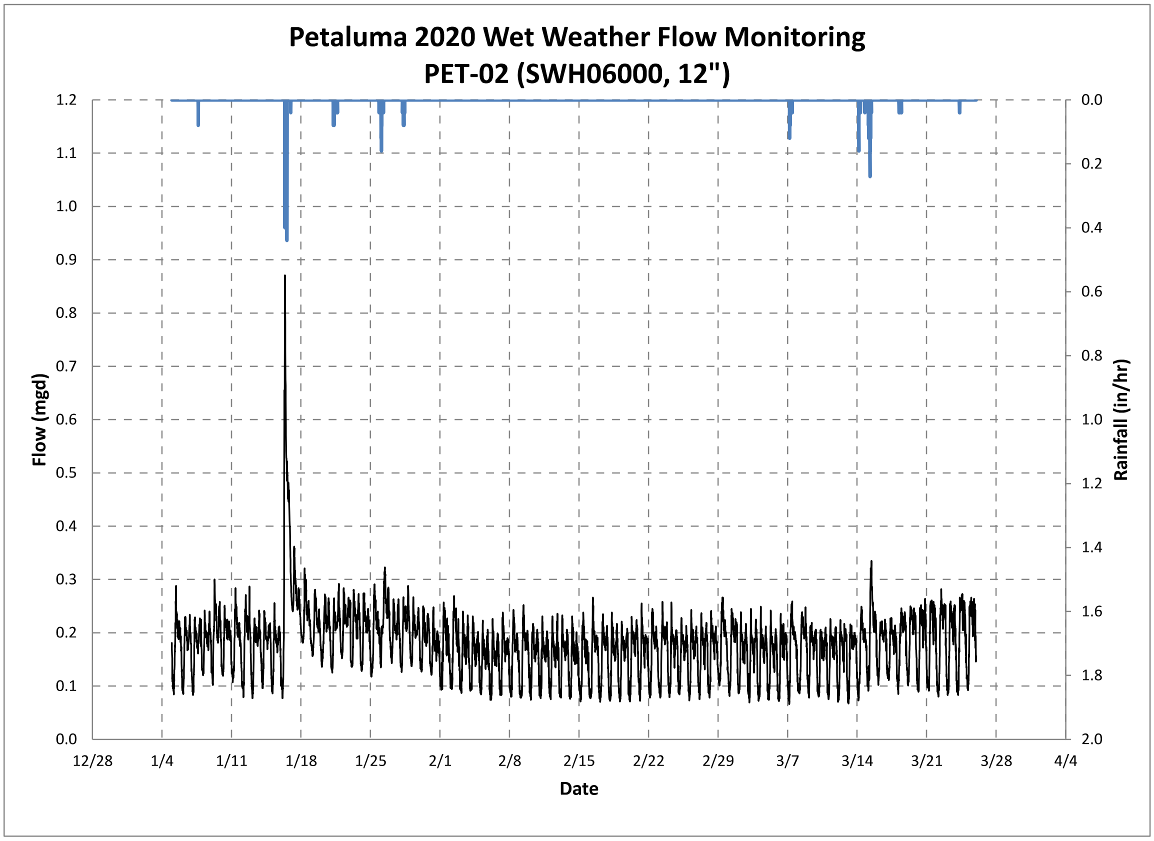
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Flow (mgd)
Rainfall (in/hr)
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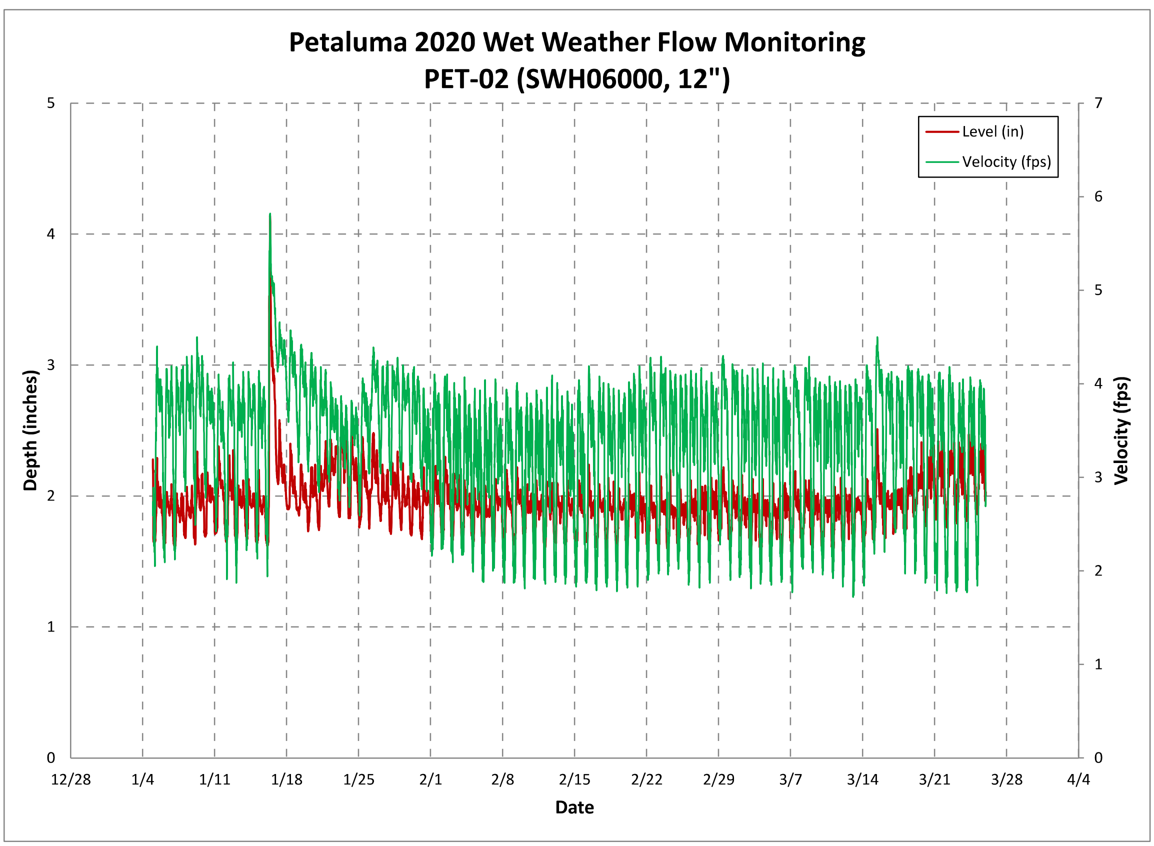
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Velocity (fps)
Depth (inches)
Velocity (fps)
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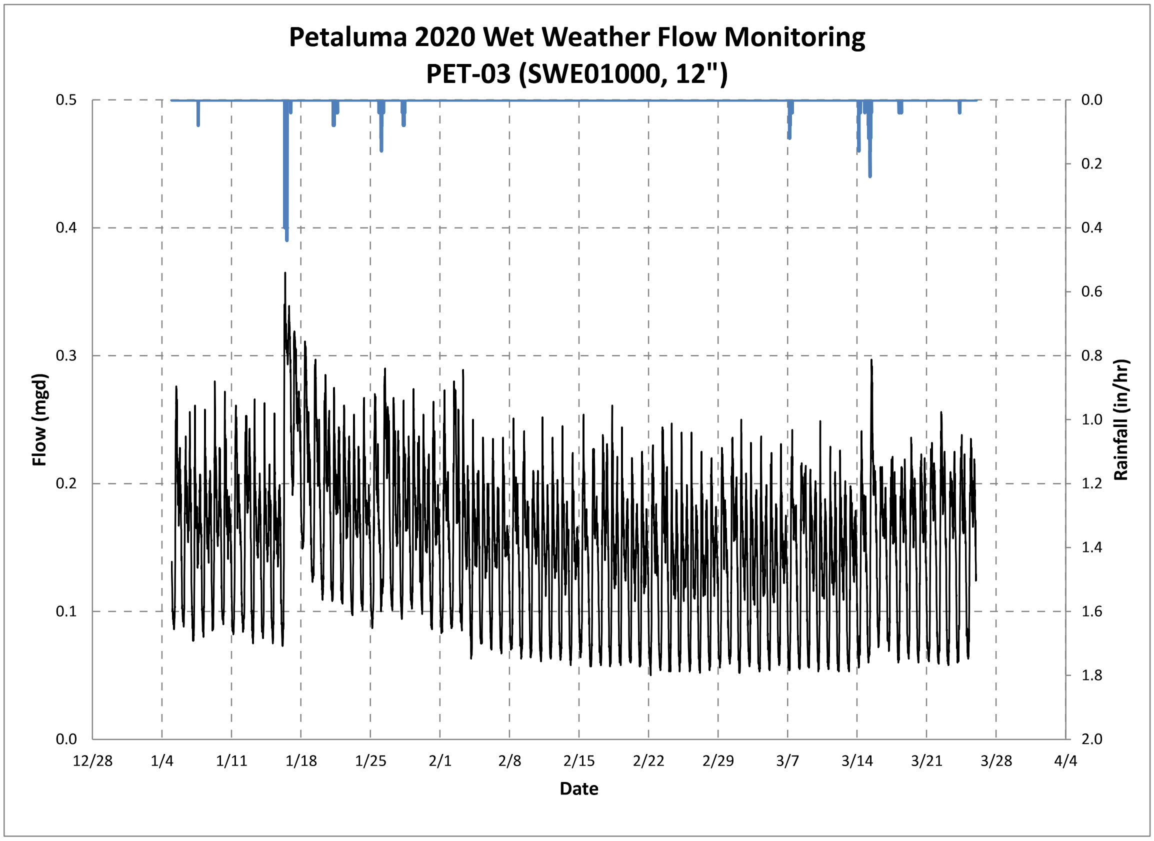
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Rainfall (in/hr)
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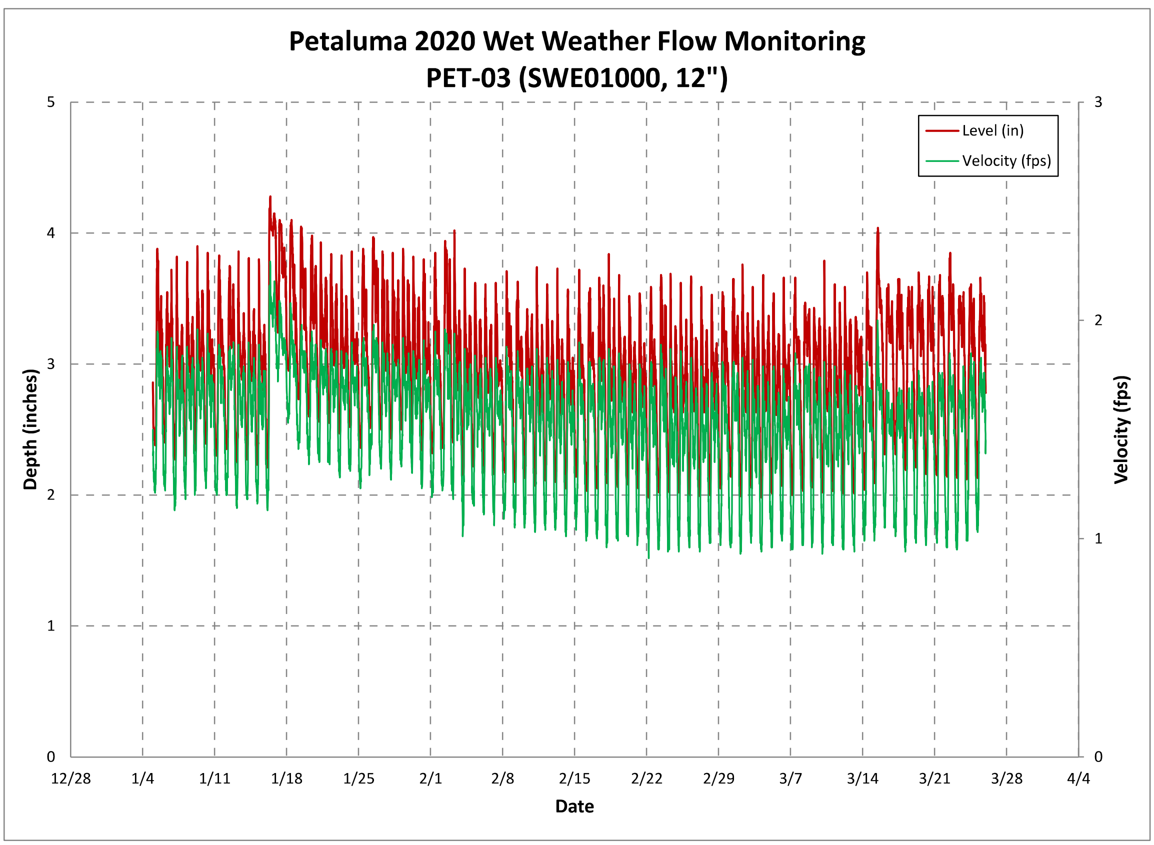
Petaluma 2020 Wet Weather Flow Monitoring
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Velocity (fps)
Depth (inches)
Velocity (fps)
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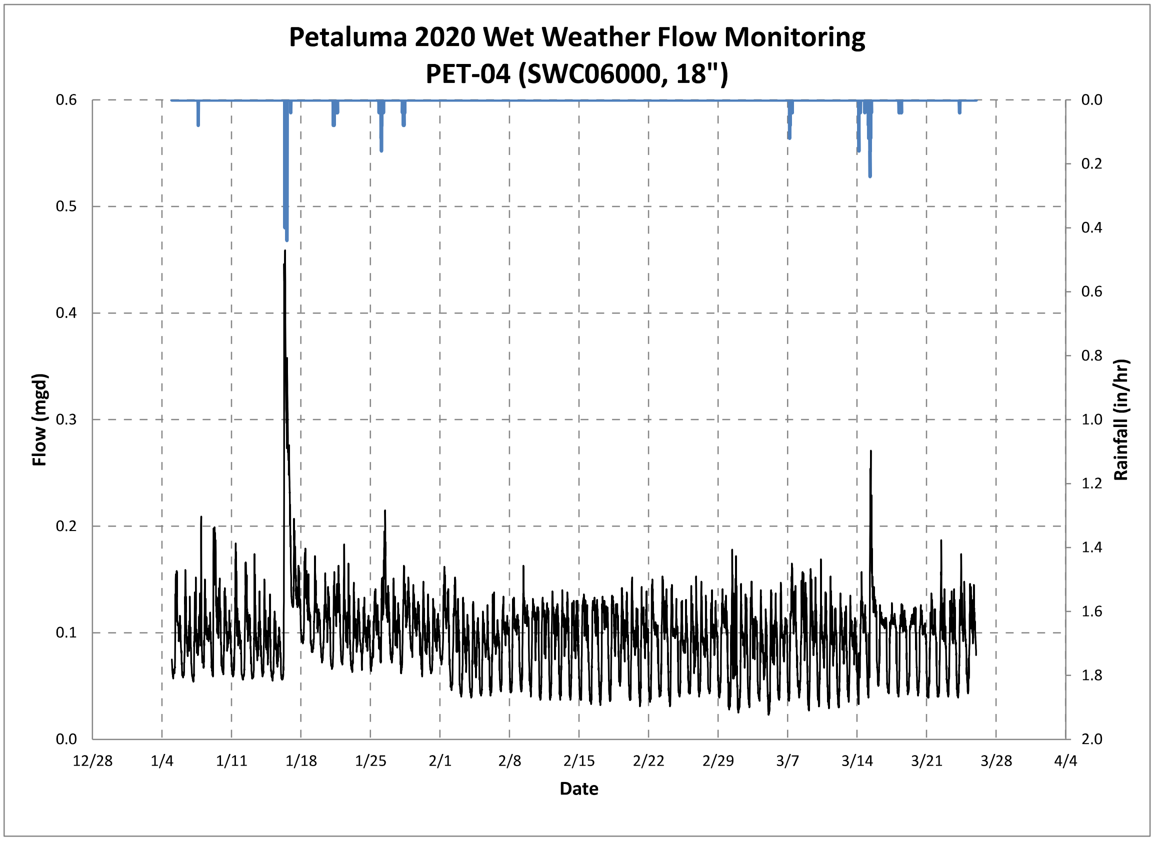

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Rainfall (in/hr)
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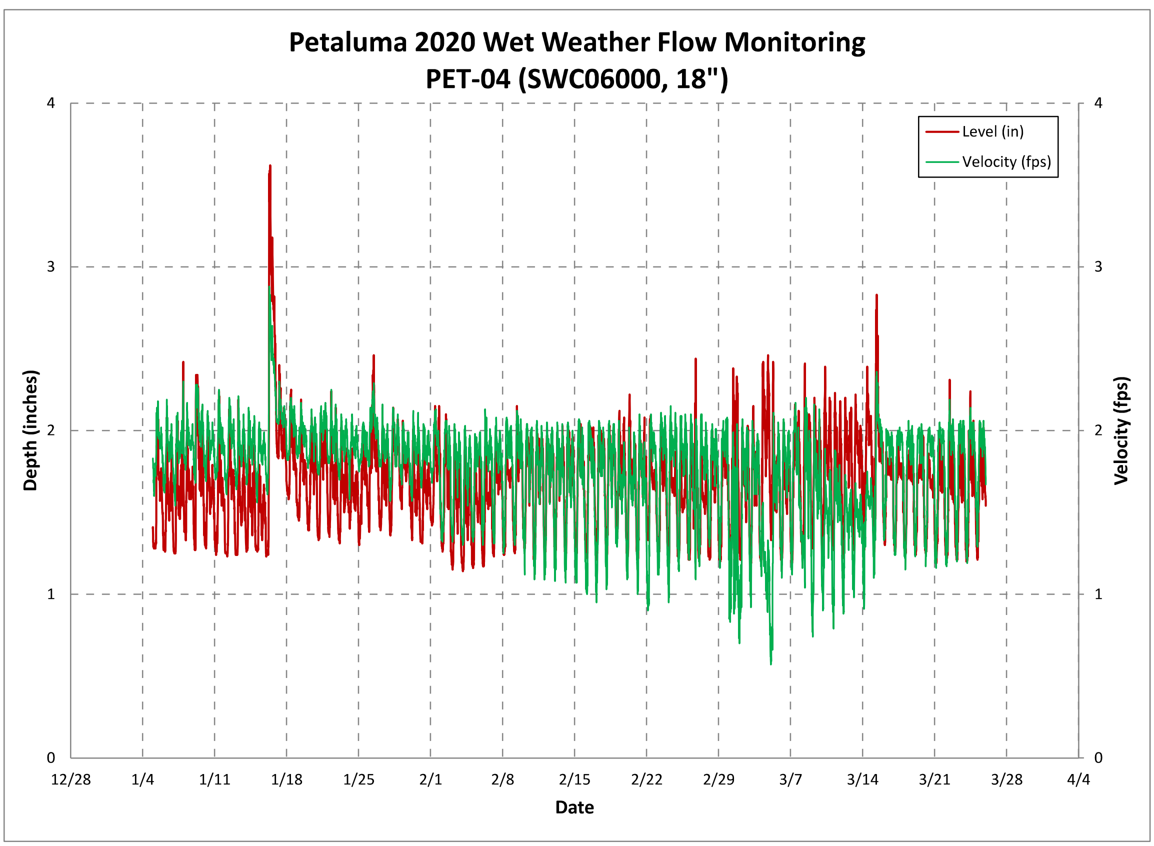

Petaluma 2020 Wet Weather Flow Monitoring
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Velocity (fps)
Depth (inches)
Velocity (fps)
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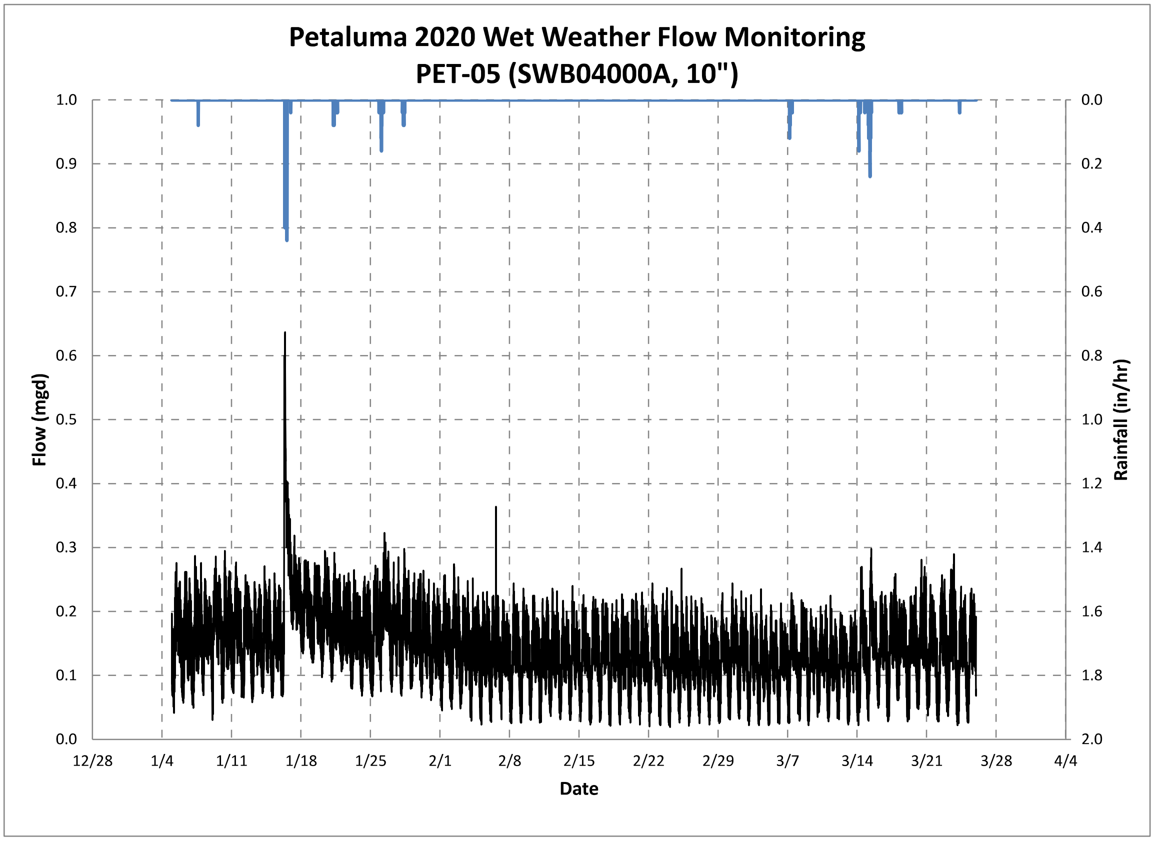
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Rainfall (in/hr)
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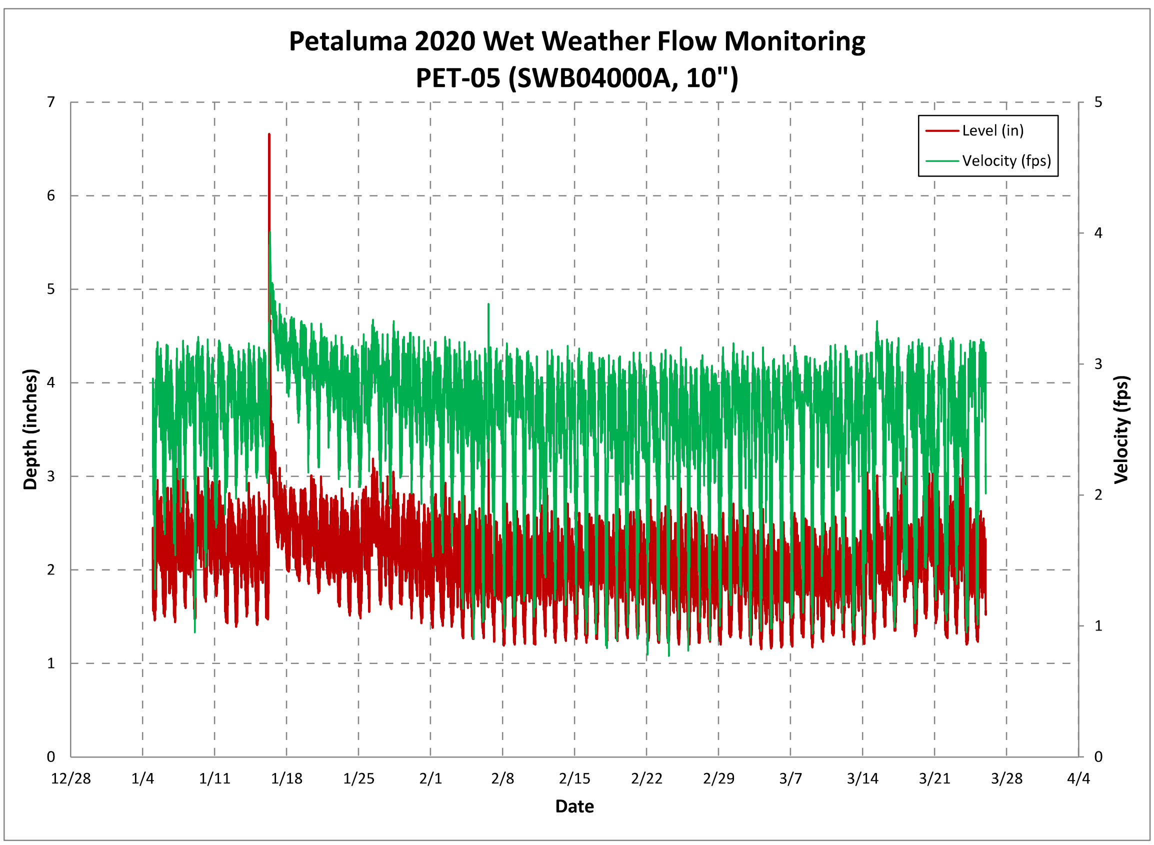
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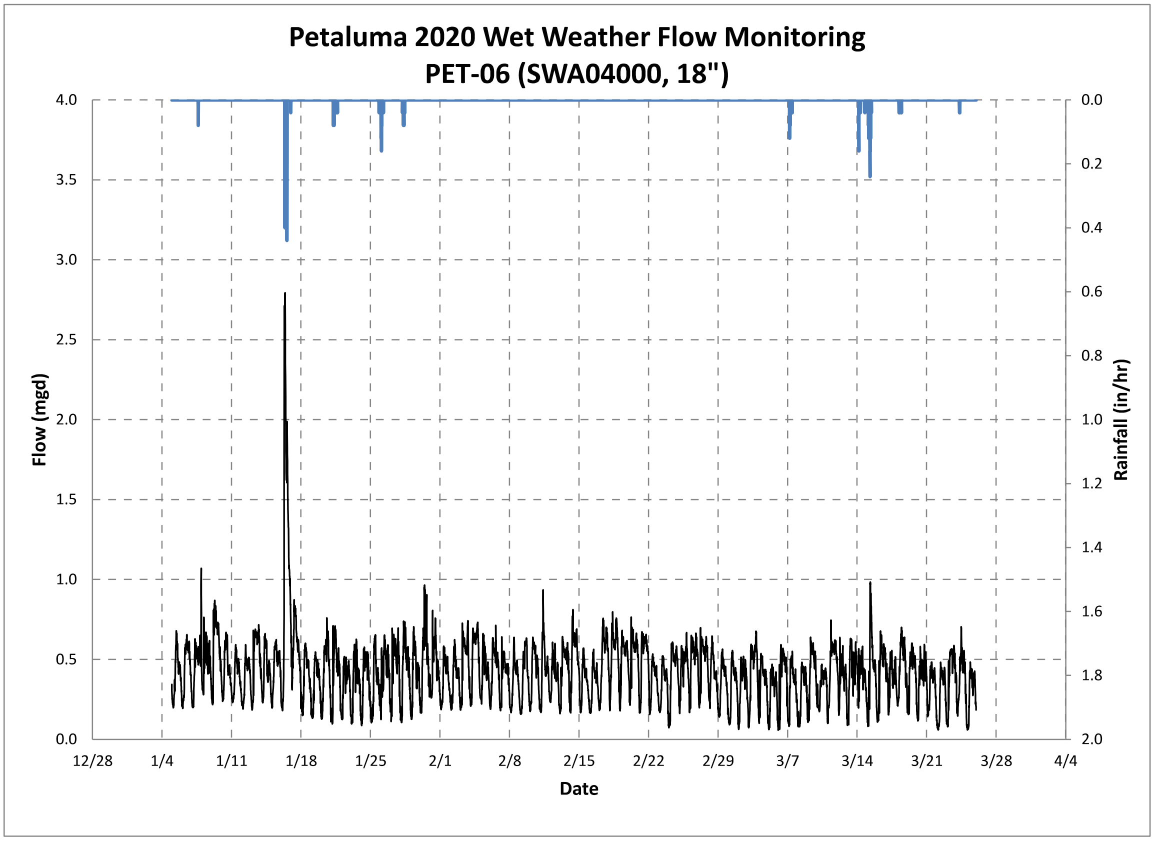
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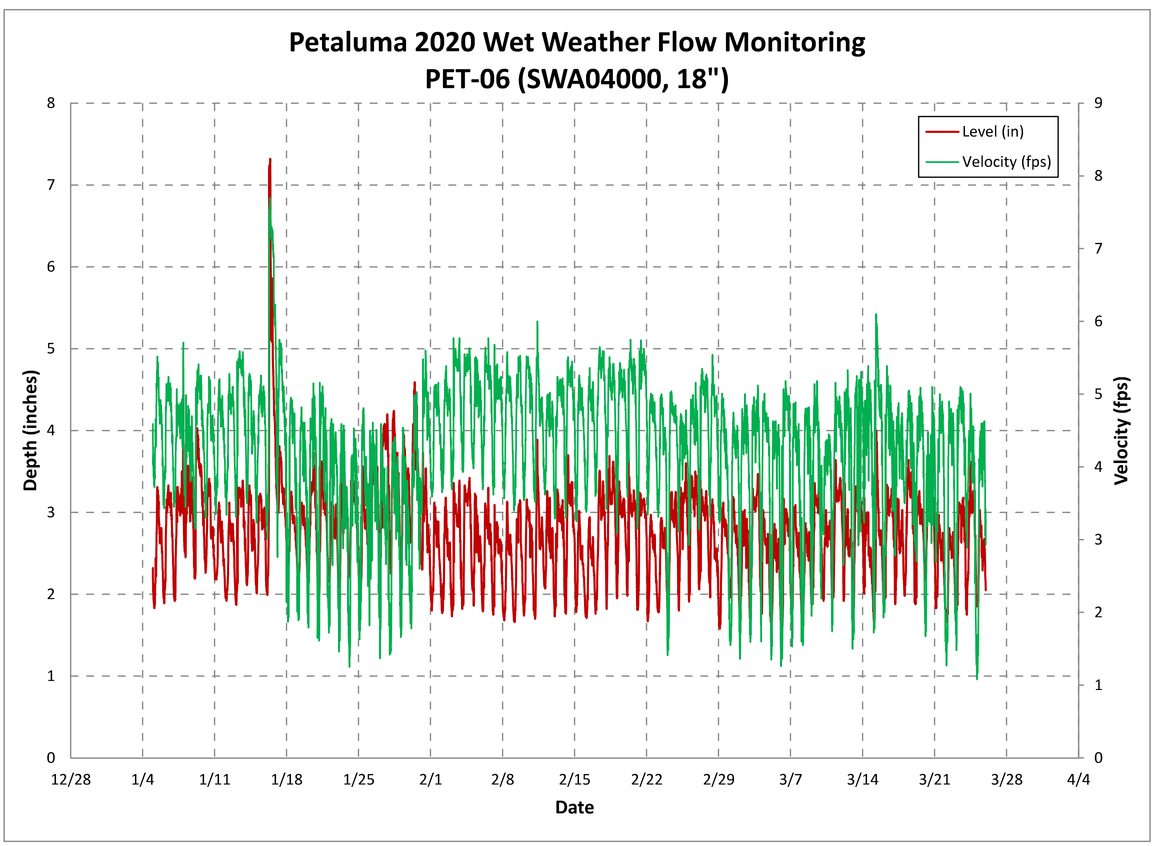
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APPENDIX B - MODEL CALIBRATION GRAPHS

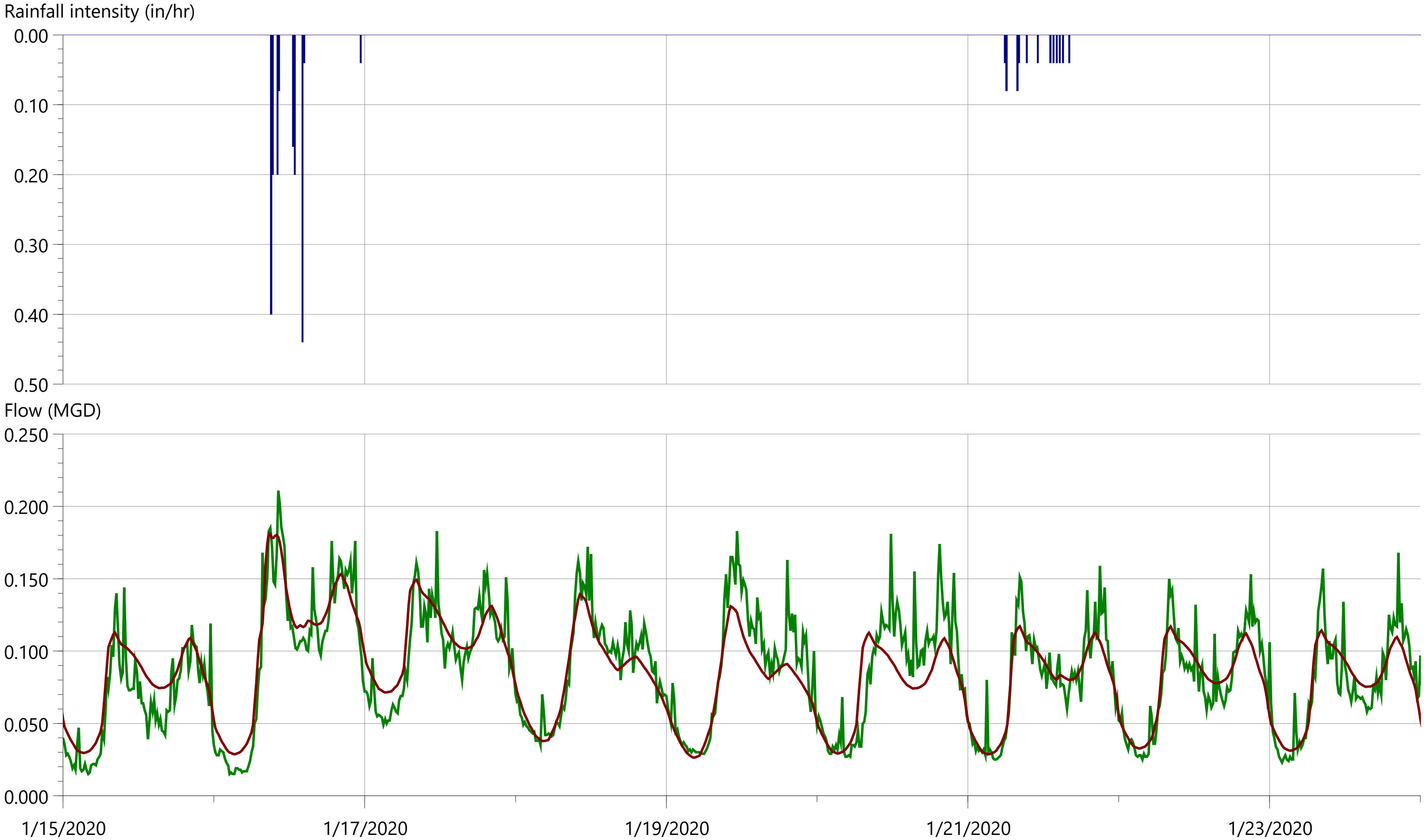
Flow Meter Graphs
January 16, 2020 Storm

PET-01
PET-02
PET-03
PET-04
PET-05
PET-06

PET-01

Observed / Predicted Report Produced by shubli (12/8/2020 5:31:04 PM) Page 1 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...

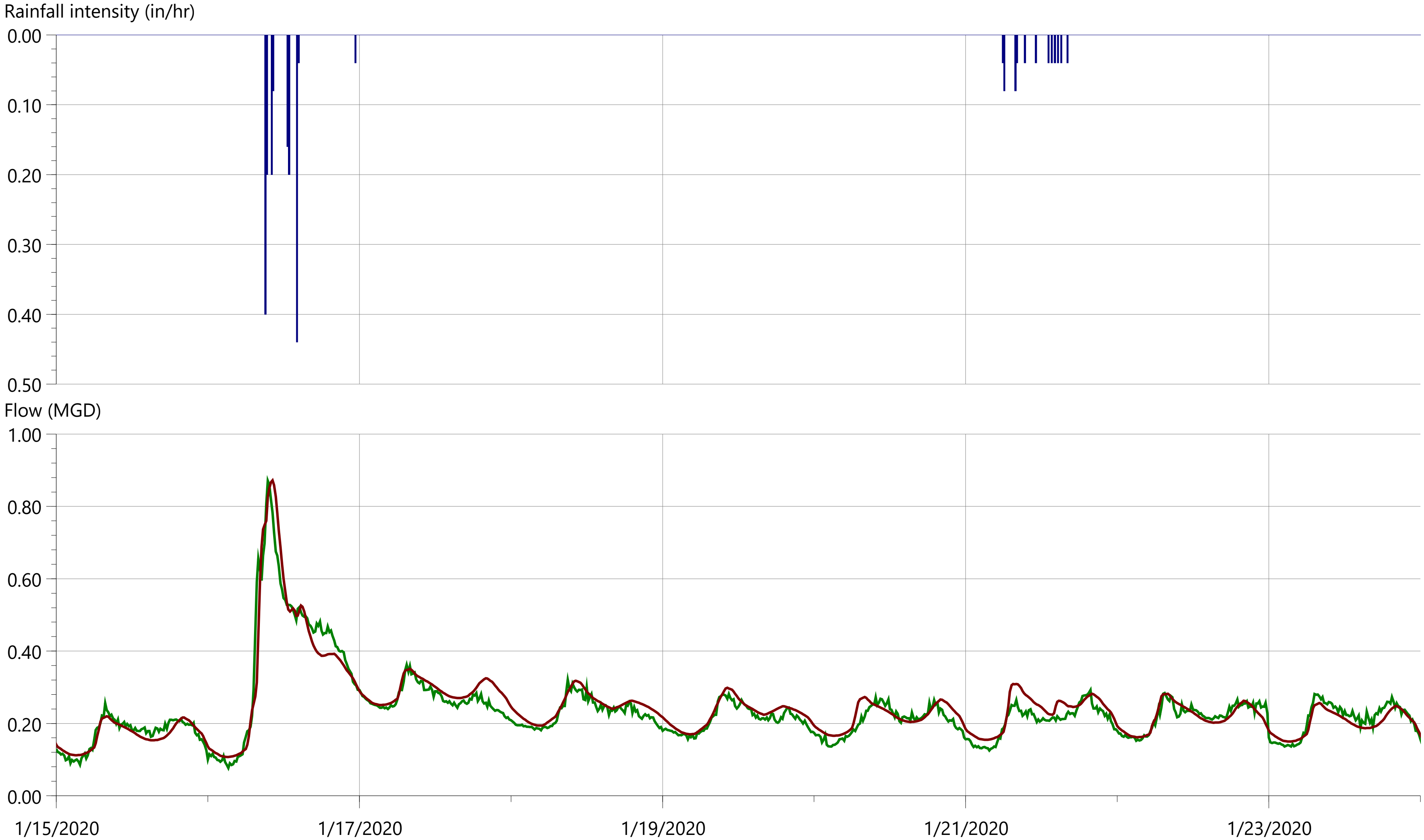
Flow Survey Location (Obs.) PET-01, Model Location (Pred.) D/S SWJ04000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.580	0.440	0.003			
Observed				0.015	0.211	0.772
...ls - With ADS gauge				0.026	0.181	0.758

PET-02

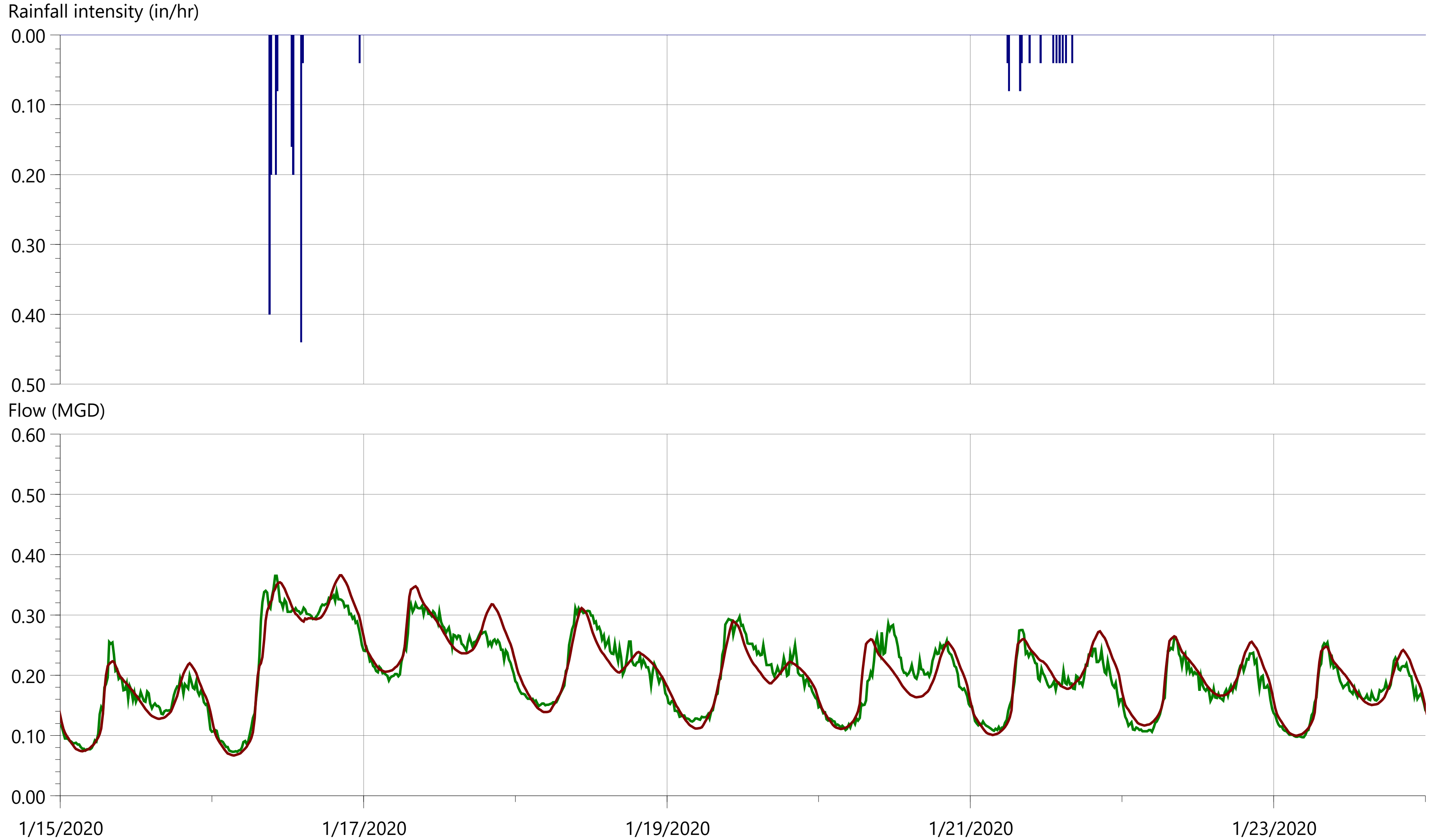
Observed / Predicted Report Produced by shubli (12/8/2020 5:31:04 PM) Page 2 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-02, Model Location (Pred.) D/S SWH07000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.580	0.440	0.003			
Observed				0.077	0.871	2.117
...ls - With ADS gauge				0.107	0.872	2.185

PET-03

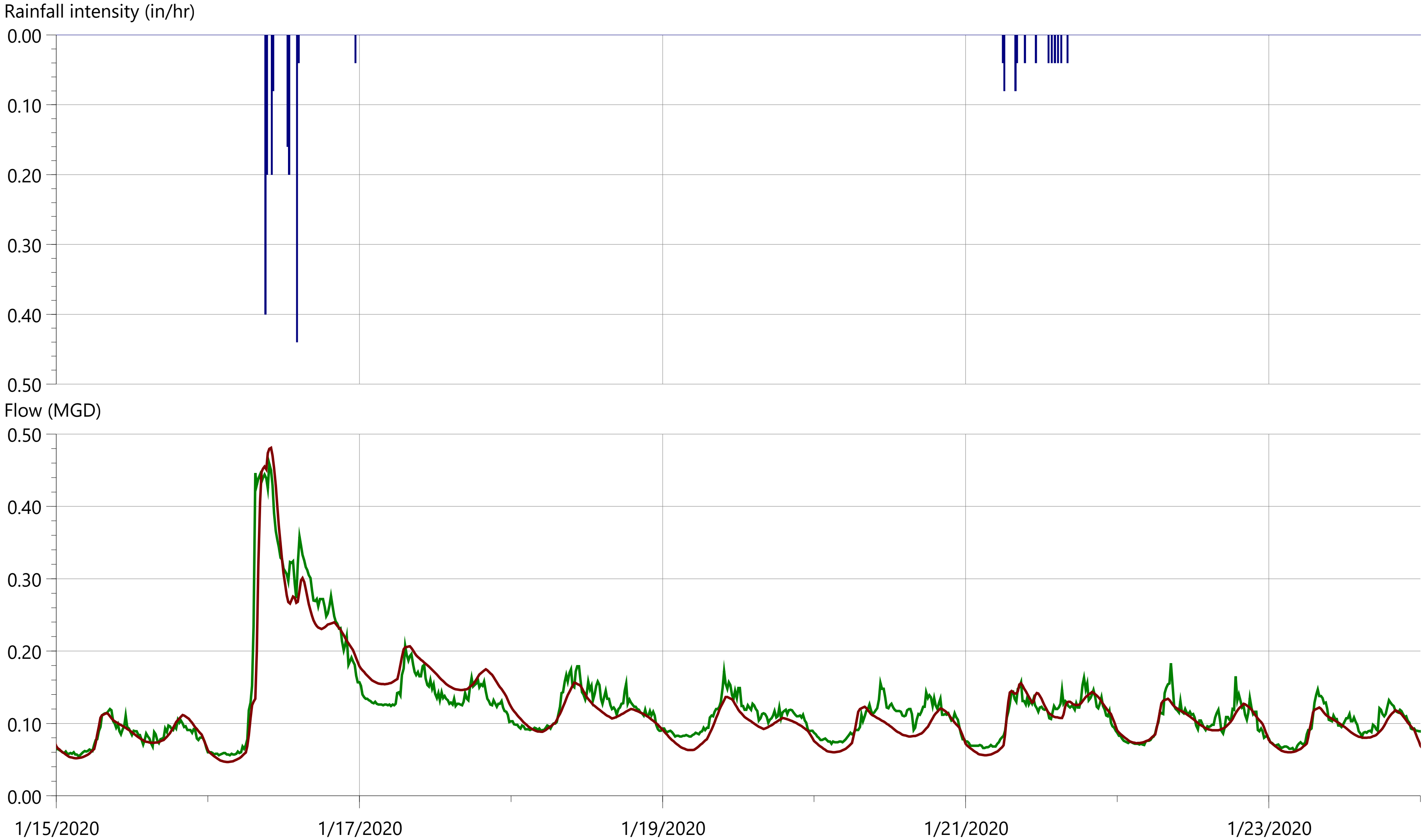
Observed / Predicted Report Produced by shubli (12/8/2020 5:31:04 PM) Page 3 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-03, Model Location (Pred.) D/S SWE02000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.580	0.440	0.003			
Observed				0.073	0.365	1.784
...ls - With ADS gauge				0.067	0.366	1.801

PET-04

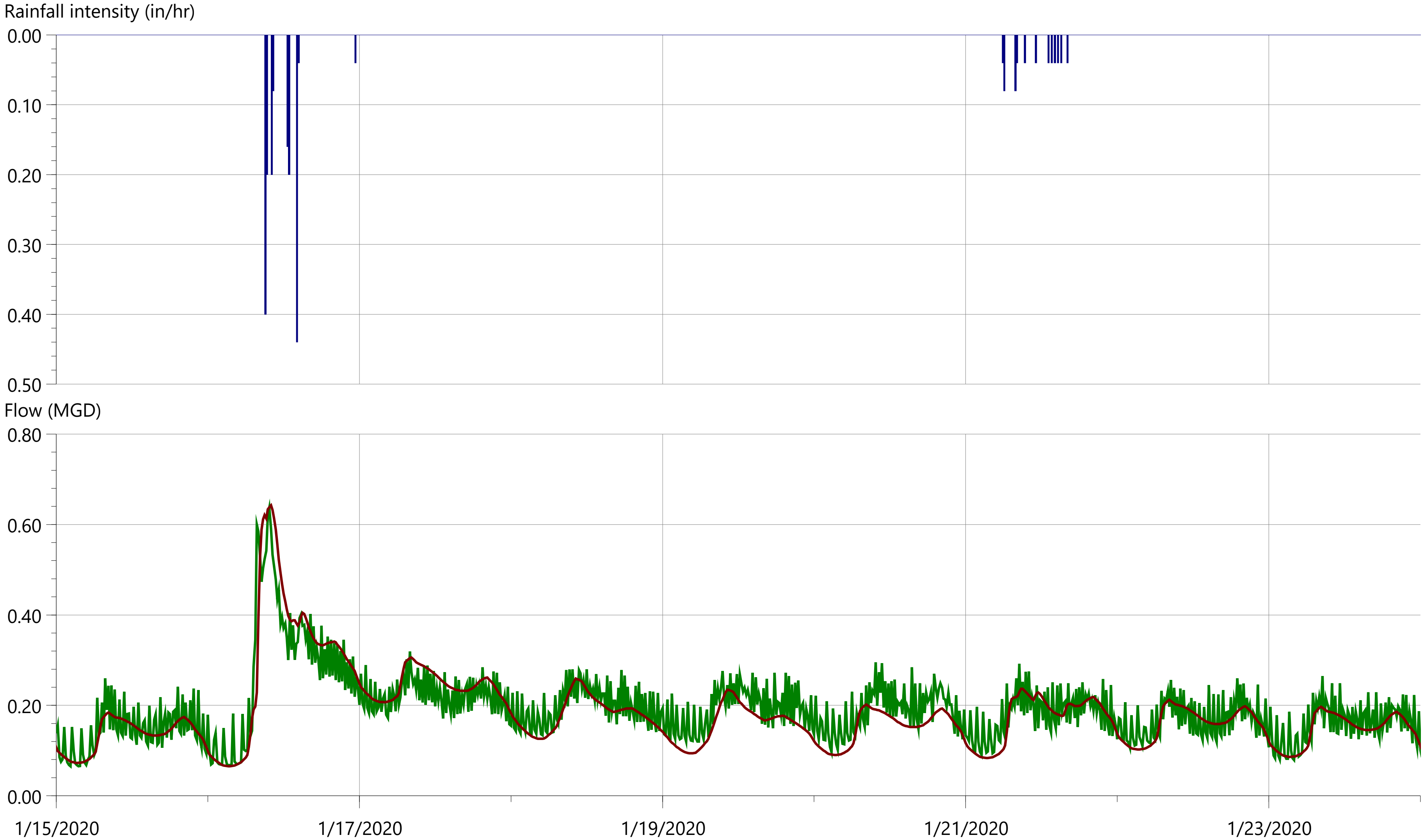
Observed / Predicted Report Produced by shubli (12/8/2020 5:31:04 PM) Page 4 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-04, Model Location (Pred.) D/S SWC07000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.580	0.440	0.003			
Observed				0.055	0.459	1.106
...ls - With ADS gauge				0.047	0.481	1.065

PET-05

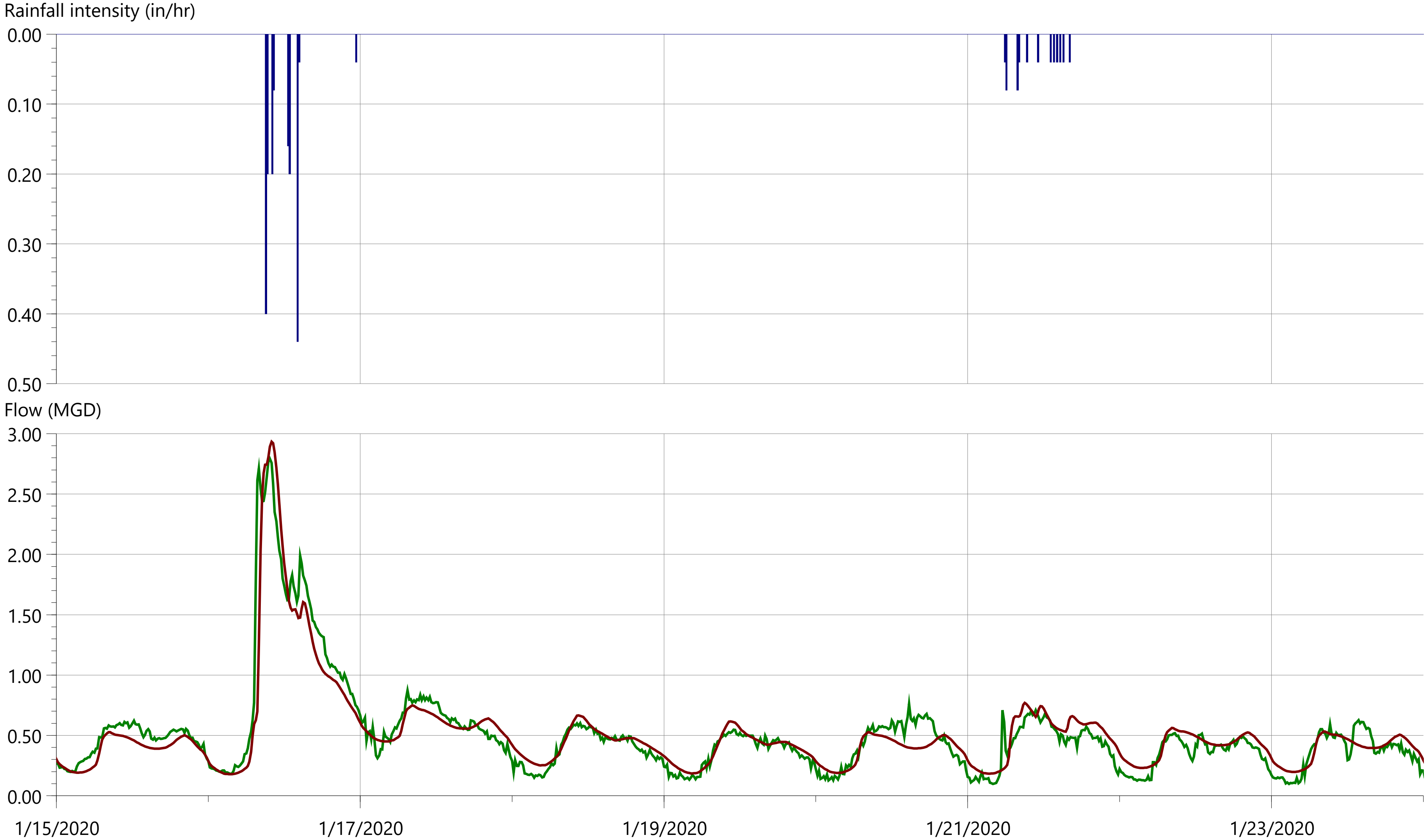
Observed / Predicted Report Produced by shubli (12/8/2020 5:31:04 PM) Page 5 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-05, Model Location (Pred.) D/S MH_Added_32.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.580	0.440	0.003			
Observed				0.065	0.637	1.749
...ls - With ADS gauge				0.065	0.643	1.648

PET-06

Observed / Predicted Report Produced by shubli (12/8/2020 5:31:04 PM) Page 6 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-06, Model Location (Pred.) D/S SWA06000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.580	0.440	0.003			
Observed				0.097	2.793	4.558
...ls - With ADS gauge				0.178	2.934	4.548

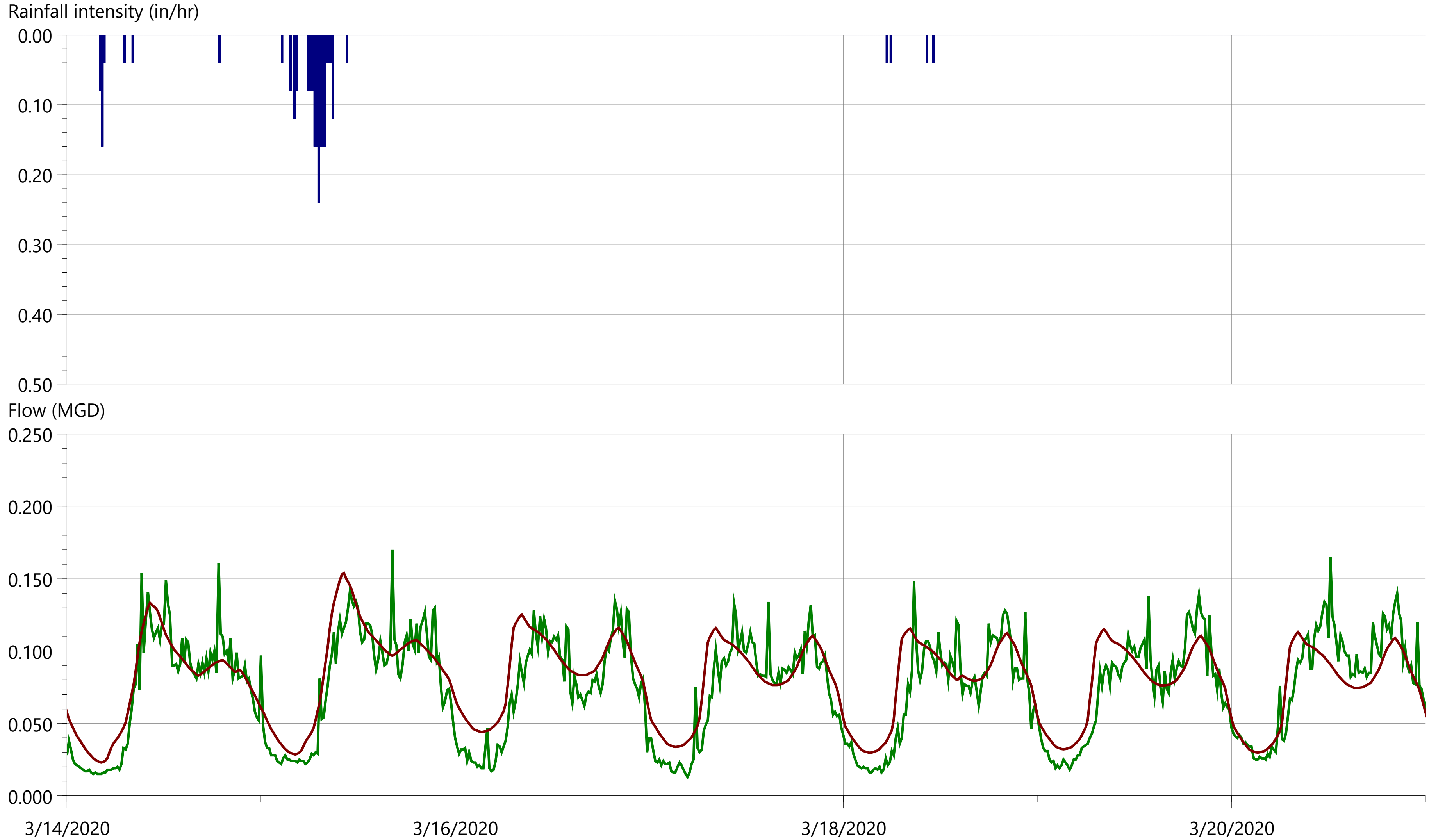
Flow Meter Graphs
<u>March 15, 2020 Storm</u>

PET-01
PET-02
PET-03
PET-04
PET-05
PET-06

PET-01

Observed / Predicted Report Produced by shubli (12/8/2020 5:27:44 PM) Page 1 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...

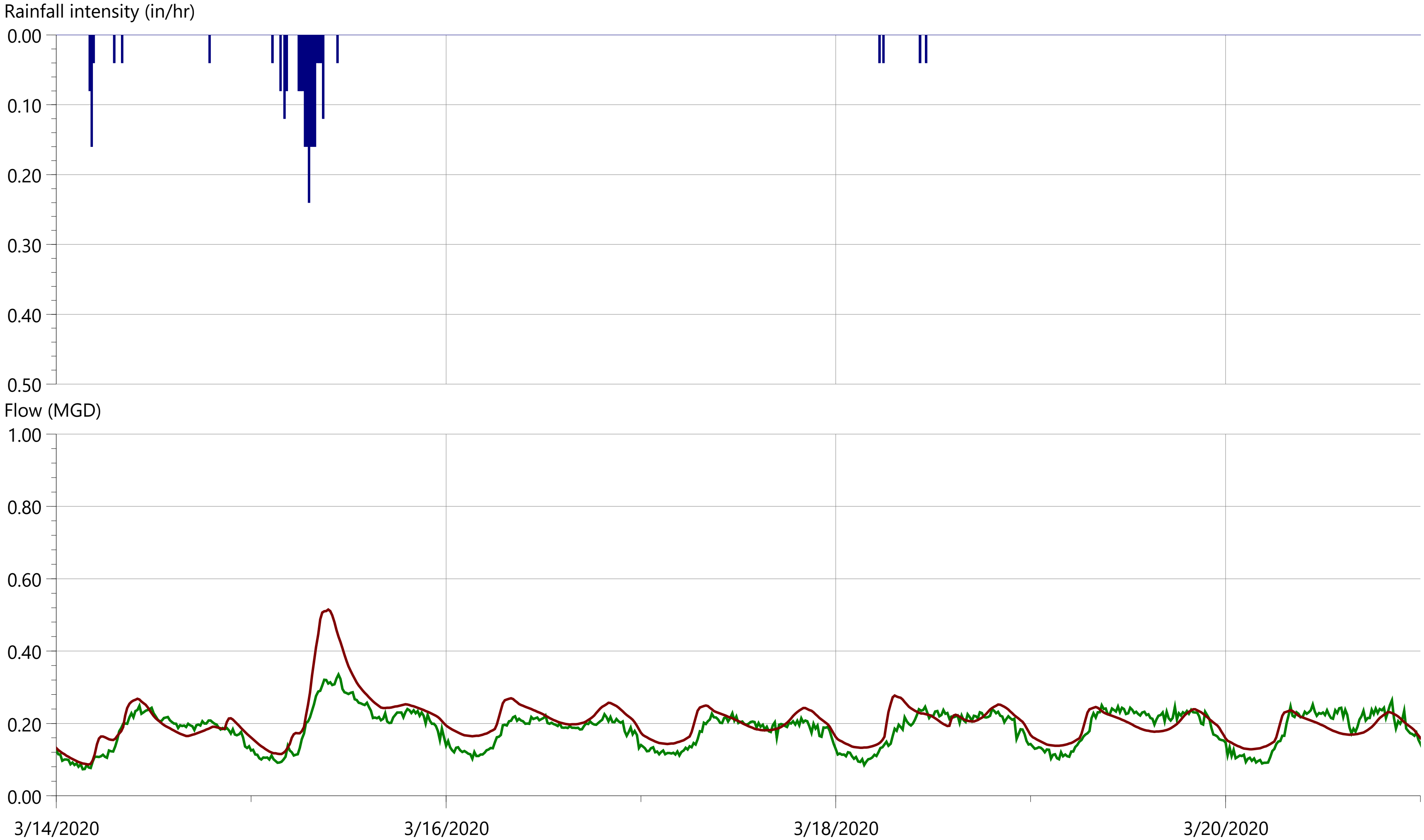
Flow Survey Location (Obs.) PET-01, Model Location (Pred.) D/S SWJ04000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.610	0.240	0.004			
Observed				0.013	0.170	0.524
...ls - With ADS gauge				0.023	0.154	0.561

PET-02

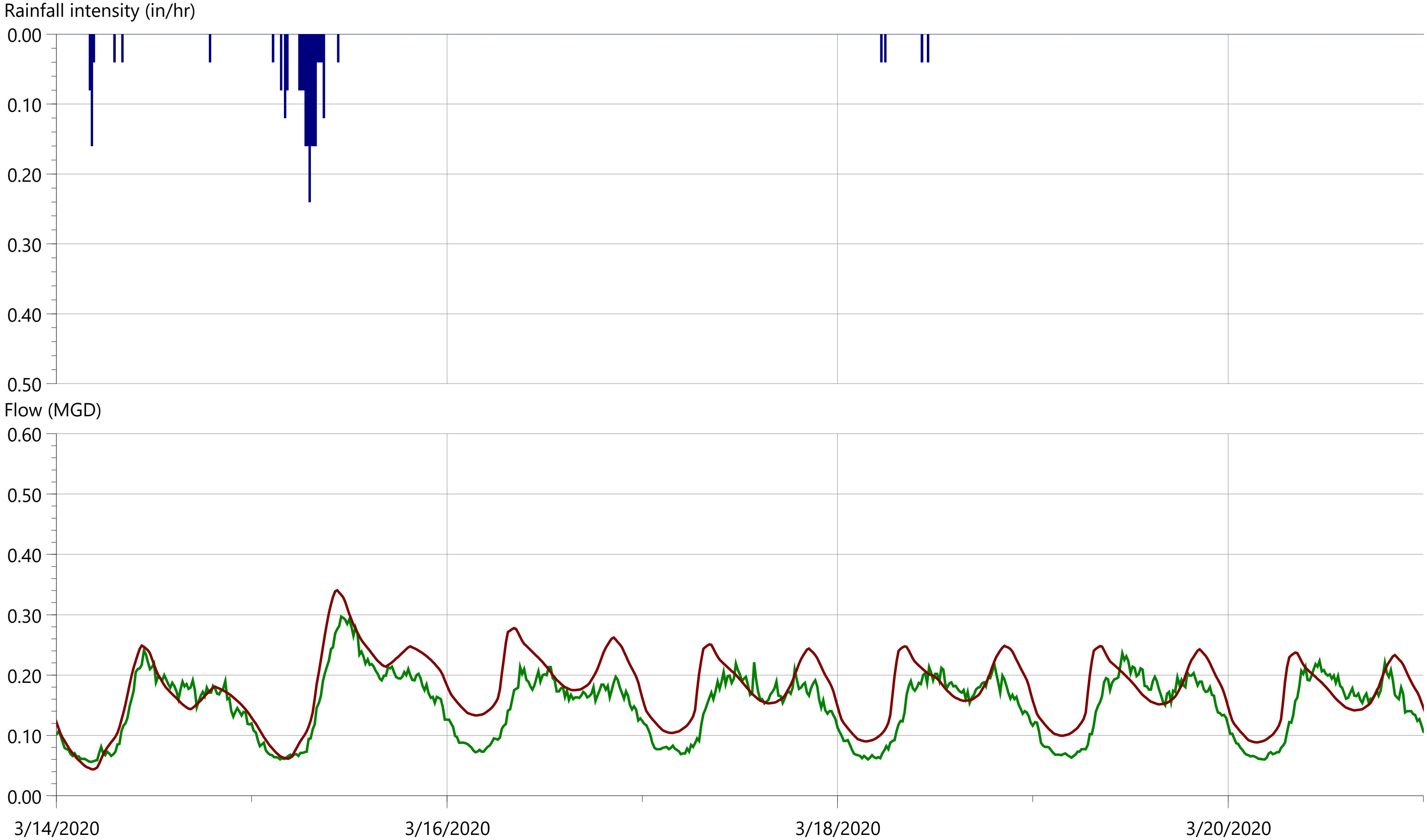
Observed / Predicted Report Produced by shubli (12/8/2020 5:27:44 PM) Page 2 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-02, Model Location (Pred.) D/S SWH07000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.610	0.240	0.004			
Observed				0.073	0.335	1.289
...ls - With ADS gauge				0.086	0.515	1.423

PET-03

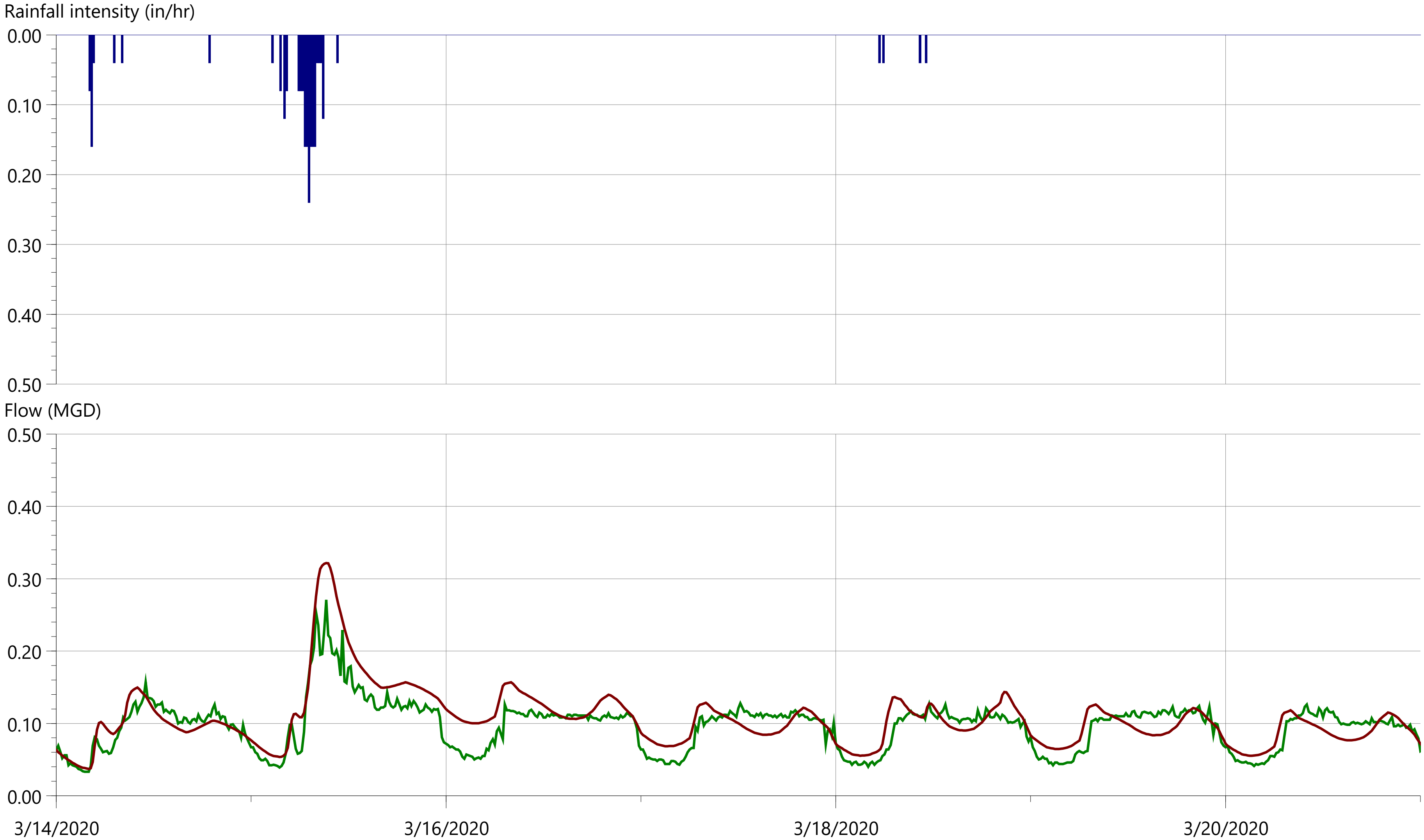
Observed / Predicted Report Produced by shubli (12/8/2020 5:27:44 PM) Page 3 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-03, Model Location (Pred.) D/S SWE02000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.610	0.240	0.004			
Observed				0.056	0.297	1.040
...ls - With ADS gauge				0.044	0.340	1.227

PET-04

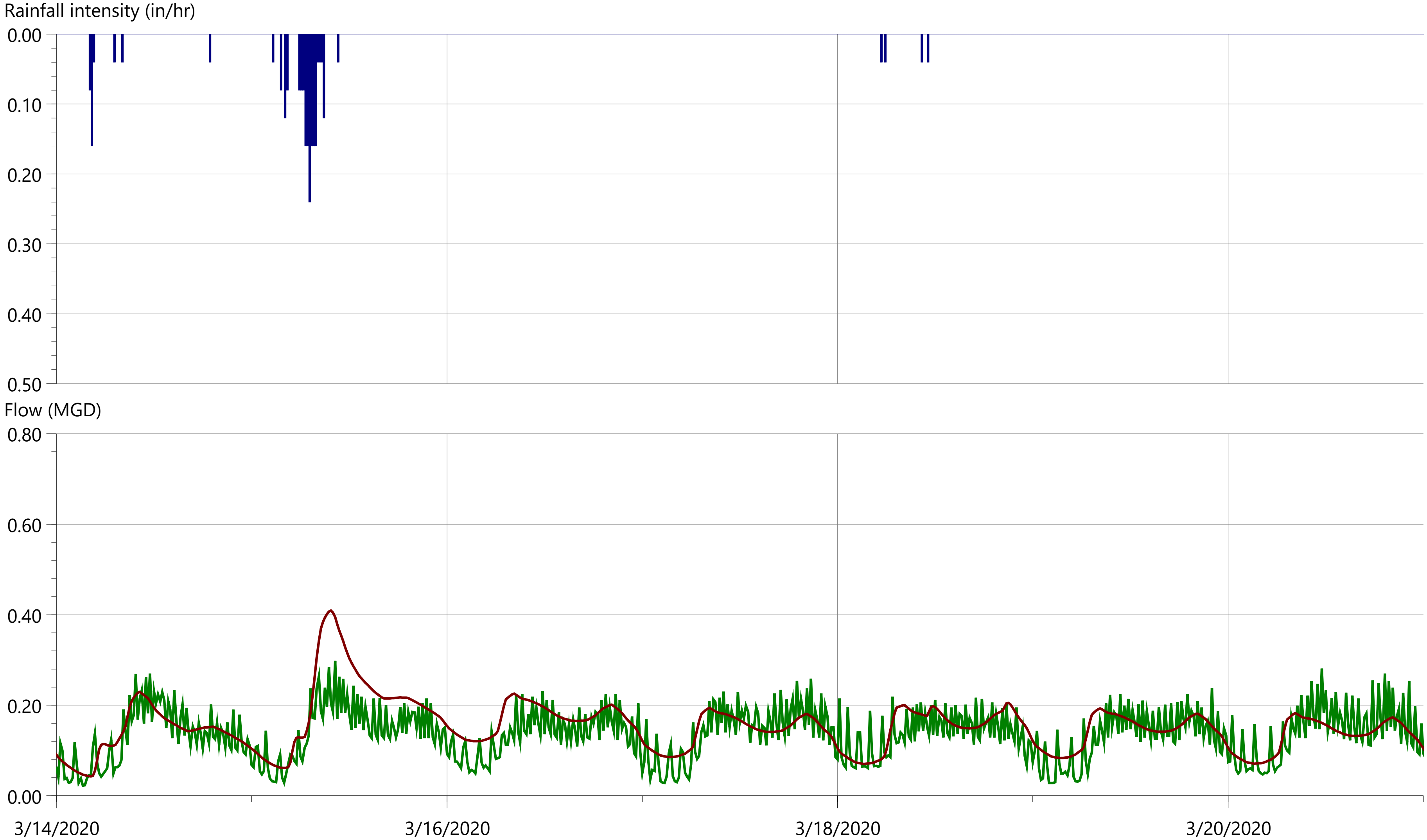
Observed / Predicted Report Produced by shubli (12/8/2020 5:27:44 PM) Page 4 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-04, Model Location (Pred.) D/S SWC07000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.610	0.240	0.004			
Observed				0.033	0.271	0.676
...ls - With ADS gauge				0.037	0.322	0.744

PET-05

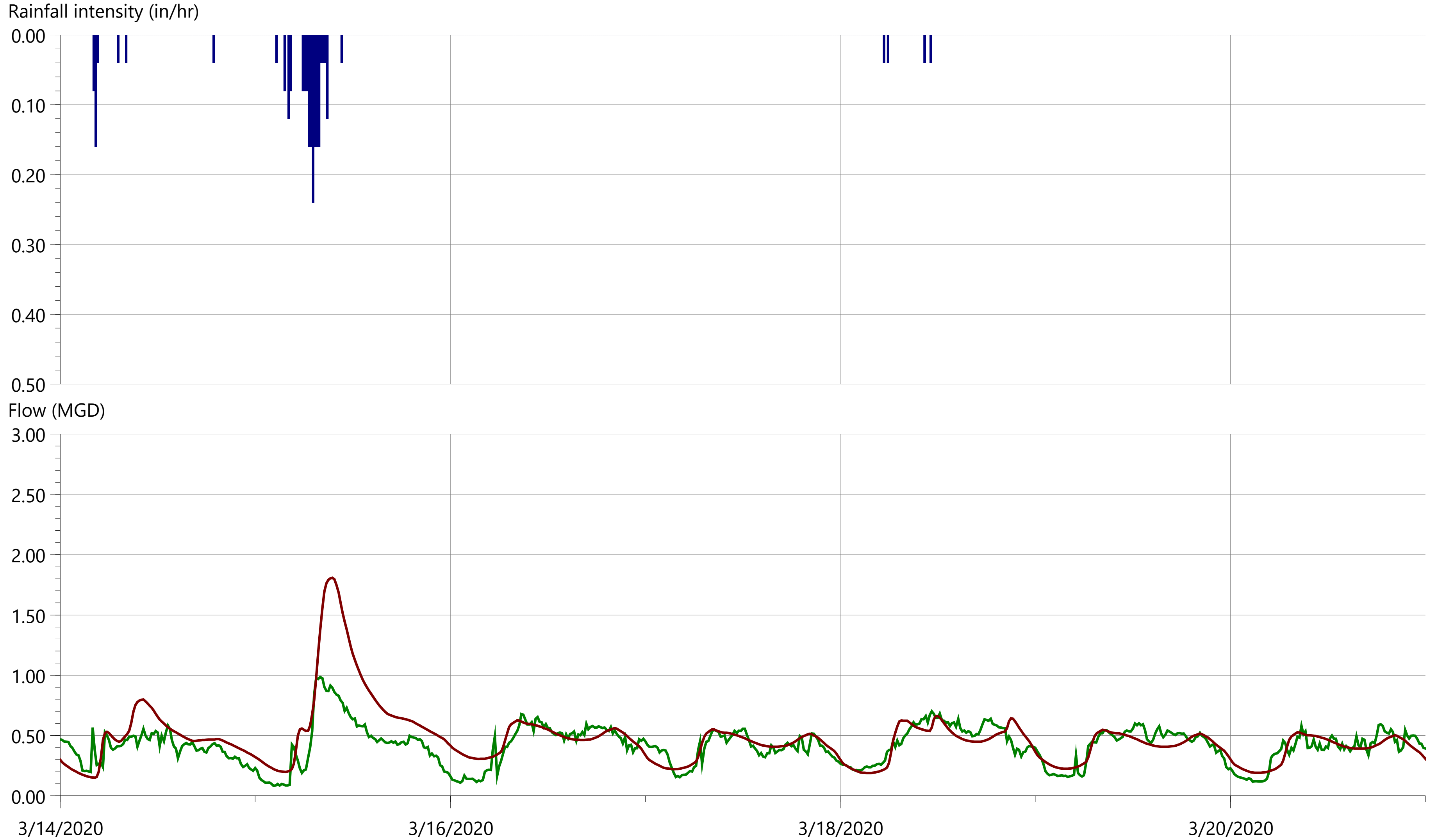
Observed / Predicted Report Produced by shubli (12/8/2020 5:27:44 PM) Page 5 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-05, Model Location (Pred.) D/S MH_Added_32.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.610	0.240	0.004			
Observed				0.022	0.298	0.959
...ls - With ADS gauge				0.043	0.409	1.077

PET-06

Observed / Predicted Report Produced by shubli (12/8/2020 5:27:44 PM) Page 6 of 6
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Flow Meter Data>2020 Flow Meters (4/20/2020 12:05:15 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (12/1/20-3/26/20)>Calibration V2 -with PS operating levels - With ADS g...
Flow Survey Location (Obs.) PET-06, Model Location (Pred.) D/S SWA06000.1, Rainfall Profile: PET-RG01



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.610	0.240	0.004			
Observed				0.082	0.984	2.907
...ls - With ADS gauge				0.149	1.809	3.300

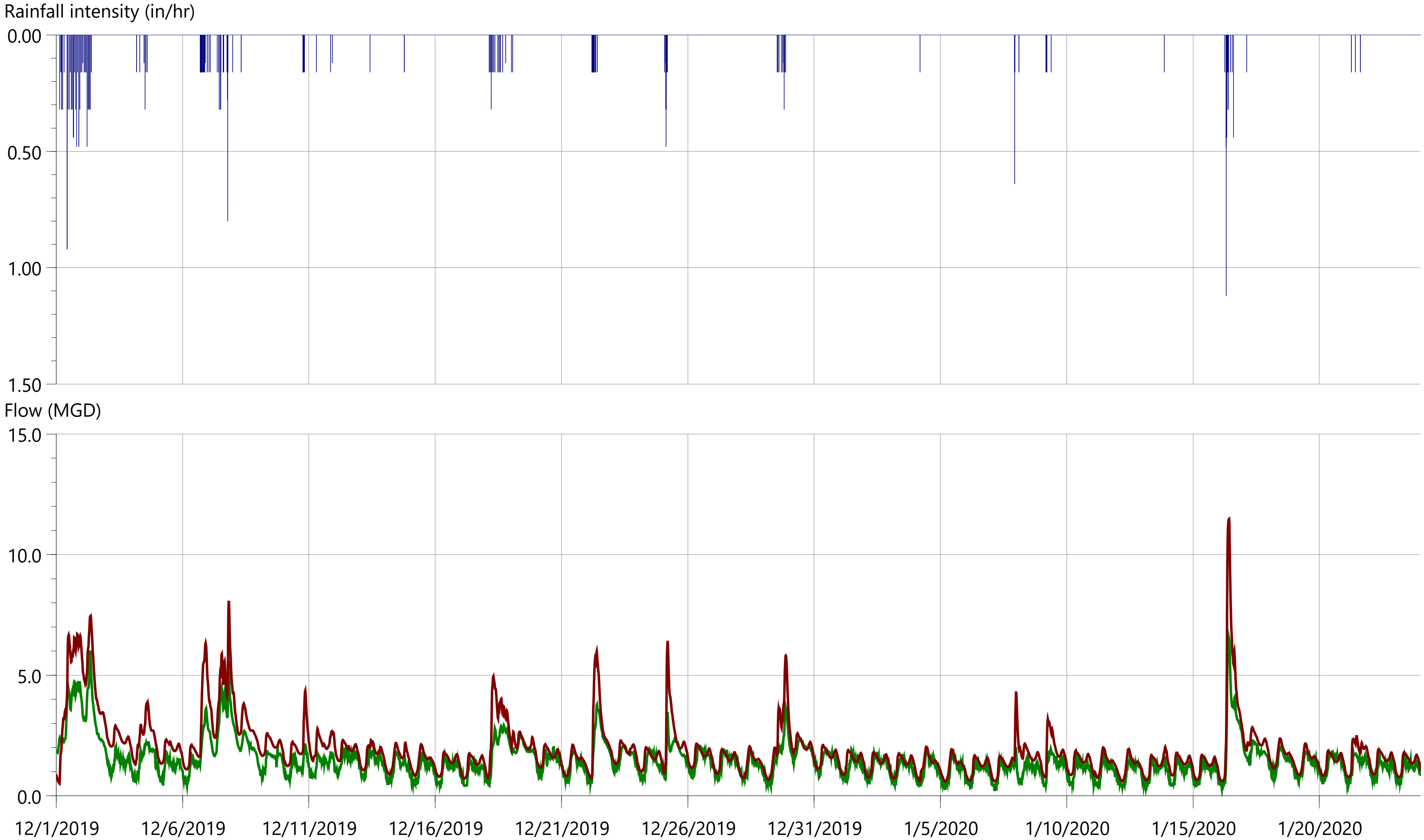
2019/2020 Pump Station Graphs

C Street
Copeland
PIPS
Wilmington

C Street PS

Observed / Predicted Report Produced by shubli (12/8/2020 5:37:27 PM) Page 1 of 4
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Pump Station Data>SCADA (15-min) (7/9/2020 1:14:05 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (11/30/19-3/26/20)_cvl>Calibration V2 -with PS operating levels Rainfall ...

Flow Survey Location (Obs.) C Street, Model Location (Pred.) D/S SWD00002.1, Rainfall Profile: D St



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	8.070	1.120	0.006			
Observed				0.200	6.480	79.735
... without ADS gauge				0.510	11.474	105.222

Copeland PS

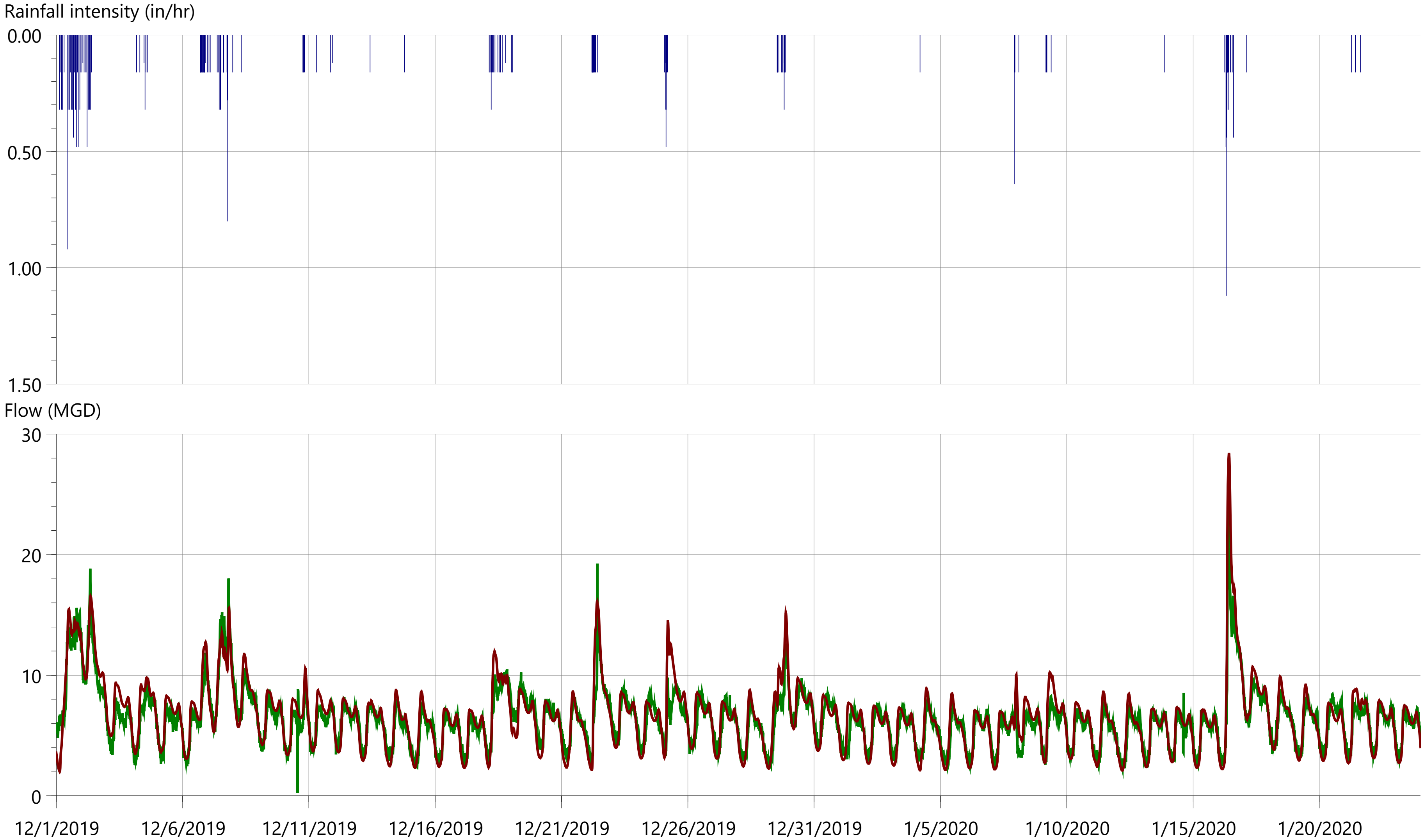
Observed / Predicted Report Produced by shubli (12/8/2020 5:37:27 PM) Page 2 of 4
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Pump Station Data>SCADA (15-min) (7/9/2020 1:14:05 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (11/30/19-3/26/20)_cvl>Calibration V2 -with PS operating levels Rainfall ...
Flow Survey Location (Obs.) Copeland, Model Location (Pred.) D/S NEC01030.1, Rainfall Profile: D St

	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	8.070	1.120	0.006			
Observed				0.070	2.120	21.528
... without ADS gauge				0.113	2.079	25.061

PIPS

Observed / Predicted Report Produced by shubli (12/8/2020 5:37:27 PM) Page 3 of 4
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Pump Station Data>SCADA (15-min) (7/9/2020 1:14:05 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (11/30/19-3/26/20)_cvl>Calibration V2 -with PS operating levels Rainfall ...

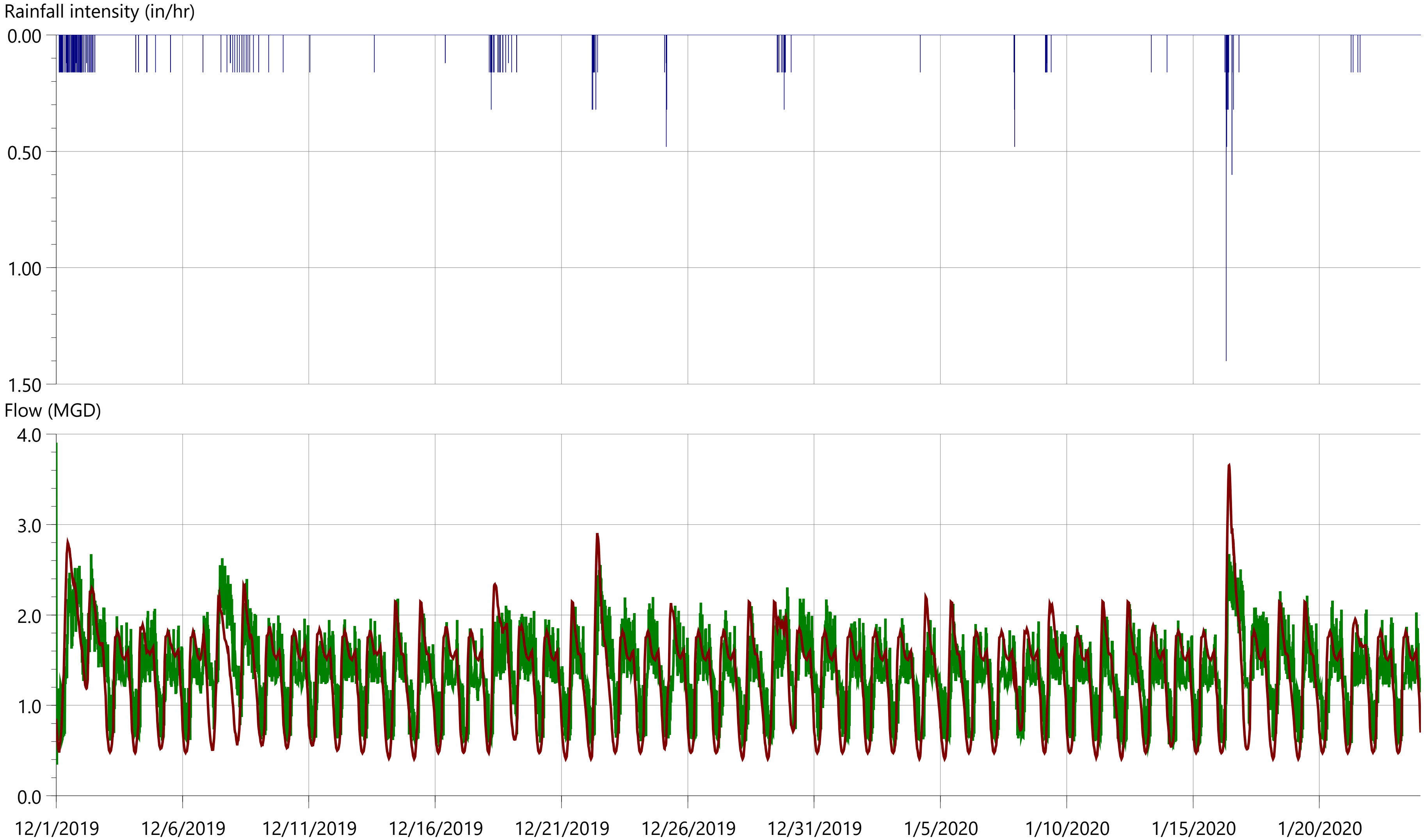
Flow Survey Location (Obs.) PIPS, Model Location (Pred.) D/S NEA00000.1, Rainfall Profile: D St



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	8.070	1.120	0.006			
Observed				0.250	25.720	344.176
... without ADS gauge				1.990	28.349	356.768

Wilmington PS

Observed / Predicted Report Produced by shubli (12/8/2020 5:37:27 PM) Page 4 of 4
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Pump Station Data>SCADA (15-min) (7/9/2020 1:14:05 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (11/30/19-3/26/20)_cvl>Calibration V2 -with PS operating levels Rainfall ...
Flow Survey Location (Obs.) Wilmington, Model Location (Pred.) D/S Added_MH_8_1.1, Rainfall Profile: Lynch Creek



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	7.130	1.400	0.006			
Observed				0.347	3.905	71.971
... without ADS gauge				0.406	3.654	71.957

Pump Station Graph

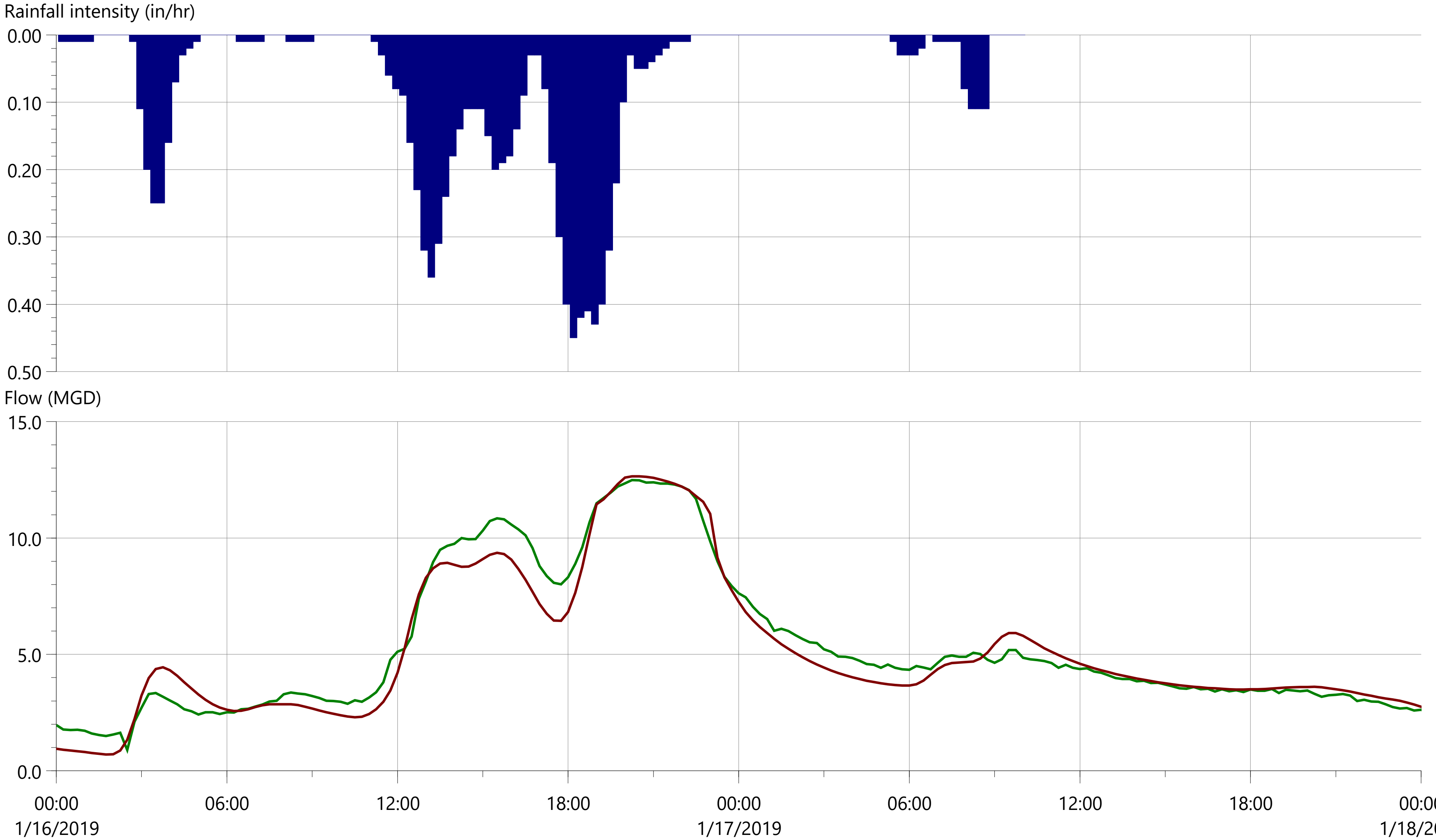
January 16, 2019 Storm

C Street

C Street PS

Observed / Predicted Report Produced by shubli (12/8/2020 5:36:09 PM) Page 1 of 1
Flow survey: >South Area Sewer Model>03 DATA>A Flow Data>Pump Station Data>1/16/19 SCADA-15min (7/9/2020 1:14:05 PM)
Sim: >South Area Sewer Model>05 Runs>Calibration Runs>Wet Weather Flow>Wet Weather Flow #2 (1/16/19 storm)>Calibration V2 -with PS operating levels 1/16/19 Rainfall...

Flow Survey Location (Obs.) C Street, Model Location (Pred.) D/S SWD00002.1, Rainfall Profile: WU



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	2.332	0.450	0.068			
Observed				0.892	12.487	10.652
...WU 5-min) adjusted				0.695	12.654	10.247

APPENDIX C - MODEL HYDRAULIC PROFILES

APPENDIX D – SOURCE DETECTION METHODS

Source Detection Methods

Smoke Testing

Smoke testing has historically been one of the most commonly used methods for I&I source detection. Smoke testing involves blowing a non-toxic smoke into the sewers at selected manholes (typically spaced at 600 to 800-foot intervals) and observing and documenting the locations where the smoke emerges from the surface. These locations, called "smoke returns," are assumed to be locations where rainwater can enter the sewer system. These may be surface connections where stormwater runoff can enter the system directly ("inflow"), such as directly connected roof downspouts, driveway and area drains, open cleanouts, holes in manhole covers, and storm drain inlets where there is a piped connection from the storm drain to the sanitary system. Most cases, however, are found to be indirect connections where rainwater seeping into the ground enters the sewer system through cracks or leaky joints in sewer pipes, manhole walls, defective cleanouts, and service laterals ("infiltration"). The smoke returns are normally documented by photographs, sketches, and other information including their location (address), type of source, smoke intensity, and the estimated tributary drainage area of the I&I source.

In California, smoke testing is normally conducted during the summer or fall months under dry soil conditions to maximize the amount of smoke that can pass through the soil. However, it may not be an effective method for identifying infiltration sources in areas with year-round high groundwater. While a fairly effective method for identifying direct inflow sources, smoke testing is not necessarily conclusive for identifying sources of infiltration, as submerged defects will not be identified, and detection is not possible if the smoke cannot reach the ground surface, for example due to surface pavement or deep sewers.

Smoke testing is considered relatively inexpensive compared to other source detection methods. However, considerable public outreach and notification is usually required.

Dye Testing/Dye Flooding

Dye testing and dye flooding are used to identify potential cross connections between storm and sanitary sewers. Dye testing typically involves introducing a fluorescent dye into catch basins, storm drain manholes, or suspected directly connected area drains or roof downspouts and observing downstream manholes to detect if the dye has entered the sewer system. Dye water flooding involves plugging the ends of a section of storm drain and filling it with dyed water. Dye testing or flooding is often used as a verification method after a suspected storm drain cross connection is detected by smoke testing. CCTV inspection used in conjunction with dye testing or dye flooding can identify exact locations of cross connections between storm drain and sewer system and can sometimes detect indirect (infiltration) connections where water from a storm drain exfiltrates from defects in the storm drain pipe into defects in a sanitary sewer located at a lower elevation.

Closed-Circuit Television (CCTV) Inspection

CCTV is the primary method for evaluating the internal condition of sanitary sewer pipelines. CCTV inspection involves running a remotely controlled camera through the pipe from manhole to manhole and documenting observations of construction features (e.g., lateral connections) and defects. Documentation includes a video recording, still image photographs of observations, and other information input into a database using one of several available CCTV software programs. Over the past 10+ years, CCTV observation coding has become widely standardized using the National Association of Sewer Service Company (NASSCO) Pipeline Assessment Certification Program (PACP).

Commonly observed defects identified through CCTV include structural problems such as cracks, offset joints, corrosion, and sags; as well as maintenance-related issues such as root intrusion, debris, and grease. Active I&I can sometimes be observed (typically groundwater infiltration); however, rainfall-induced infiltration may also be seen if the inspection is conducted during or immediately following a rainfall event and the pipe is not overly full. Observed infiltration may range from a wet surface ("seeper") to a significant flow discharge ("gusher"). CCTV inspection, however, cannot observe active infiltration if the infiltration is entering the pipe below the water surface.

CCTV inspection is not usually done for the sole purpose of actually observing active infiltration. Rather, the defects observed during the inspection, such as cracks, open joints, defective lateral connections, and root intrusion. are assumed to be locations where infiltration could enter the sewer pipe under rainfall or high groundwater conditions in areas where flow monitoring has documented the presence of such extraneous flows.

While most CCTV inspection is done on sewer mains, CCTV of laterals is also possible, either through the use of "push cameras" inserted into lateral cleanouts, or sometimes through cameras that can be "launched" up the lateral from mainline during the mainline CCTV inspection.

Manhole Inspection

Visual inspections of manholes can be performed to identify structural, construction, or maintenance defects that may allow entry of I&I. As with CCTV inspection, NASSCO has developed standard codes and data format for manhole inspections under its Manhole Assessment Certification Program (MACP). Manhole inspections can be conducted from "topside" without entering the manhole (MACP Level 1) or a more detailed inspection by man-entry (MACP Level 2). The inspections can also be conducted using video camera similar to CCTV inspection of the pipe.

Focused Electrode Leak Location

Focused electrode leak location, or electro-scanning, is a proprietary method to detect defects in a non-metallic pipe (such as clay, concrete, or plastic) by measuring the electrical resistivity of the pipe wall. A pipe location that allows water to penetrate or leak also allows electrical current to escape. To detect pipe defects, electro-scanning involves filling the sewer pipe with water, then setting a fixed voltage between an electrode sent through the pipe (a "sonde") and an electrode at the surface. Current measurements are taken continuously while pulling the sonde through the pipe at a speed of 30 feet per minute. When the sonde comes within 20 to 30 mm of a pipe defect, electric current escapes through the defect, causing the current measurement to increase, peaking when the center of the sonde is aligned with the defect. The current measurements relative to distance along the pipe are recorded. The grade of the defect can be rated (e.g., "small," "medium," or "large,") based on a sonde current "threshold" established from comparison studies between previous electro-scan data and joint pressure tests. The electro-scan information can be used to prioritize pipes for further inspection and to guide a CCTV camera to the location of the leak to identify the type and size of the defect (if the defect can be seen). Focused electrode leak location can also be used to find leaks associated with non-visual defects such as cracked joint sealing material.

Pressure Testing

Pressure testing of sewer mains is used to determine the integrity of joints, which if leaky, are potential entry points for infiltration. Joint pressure testing uses an in-pipe packer to isolate the joint and then apply air to test the joint under pressure. Sometimes chemical grout is then injected to seal off any failed joints. CCTV is typically used as part of this process to locate the joints and monitor the testing and sealing procedure.

Service laterals can also be pressure tested using air or water (exfiltration test) by plugging the lateral at the connection to the main and/or at property line or building cleanouts (depending on which portion of the lateral is tested). A failed test indicates that the lateral is leaky, although the specific defects would still need to be identified by CCTV inspection.

Alternately, the lateral can be deemed to have “failed” the test and the entire pipe considered to be a discrete I&I source.

Rainfall Simulation

This method is typically used to identify and quantify potential infiltration from leaking service laterals. A rainfall event is simulated by applying water on an area along the lateral alignment using lawn sprinklers. The water is applied for several hours at a known rate in order to saturate the area. CCTV inspection is then performed in the sewer main to which the lateral is connected to observe and estimate the rate of flow from the lateral into the sewer.

Other Methods

Other I&I field investigation methods include temperature or wastewater strength sampling to assess the relative amount of non-sanitary flow in the wastewater stream, conductivity (salinity) monitoring to assess potential infiltration of seawater in tidal areas, as well as hydrogeological investigations to assess groundwater levels. These methods are more properly characterized as flow isolation approaches, as they may identify areas with I&I but not specific sources (defects).

APPENDIX E – GRIT TRAP DETAILS

Inverted Siphons: Reduced Maintenance by Using Grit Traps

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ABSTRACT
In collection systems with difficult to clean inverted siphons that frequently fill with sediment, grit/gravel traps are a solution to save time and money to maintain adequate flow of wastewater. In California, the Oro Loma Sanitary District (OLSD) and City of Santa Clara installed grit/gravel traps to address inverted siphon cleaning challenges in their respective collection systems. In both cases, the traps have substantially reduced required cleaning time of inverted siphons, mitigated potential sanitary sewer overflows, and simplified maintenance.

KEYWORDS
grit traps, inverted siphons, grit, gravel, sediment, stormwater, sanitary sewer, collection system maintenance

INTRODUCTION

Inverted siphons are critical in wastewater collection systems in which the flow needs to travel under streams, highway cuts, or other obstacles in the ground (*see Figure 1*). Due to the wide range of solids being conveyed in a typical wastewater collection system, not all solids are conveyed across a siphon. Over time, sediment builds up in the inverted siphon and must be cleaned. A typical siphon cleaning operation uses a combination sewer cleaning truck positioned at the downstream outlet structure and hydroflushing passes are used to mobilize the sediment to the outlet structure where the sediment is vacuumed out of the system. Multiple passes with the hydroflushing nozzle are required as large gravel is difficult to mobilize upwards through the upleg of the siphon.

The Oro Loma Sanitary District (OLSD) collects sewage from approximately 46,000 households and 1,000 businesses in Alameda County, California. One of their inverted siphons was difficult to clean due to poor access of the outlet structure; the combination unit had to back up a long, narrow path to leave the outlet site (*see Figures 2-3*). Approximately 20 years ago, OLSD opted to install a grit trap upstream from the inverted siphon to catch the sediment before it reached the siphon. Installing this trap substantially reduced sediment buildup in the siphon, therefore frequent cleaning was no longer necessary.

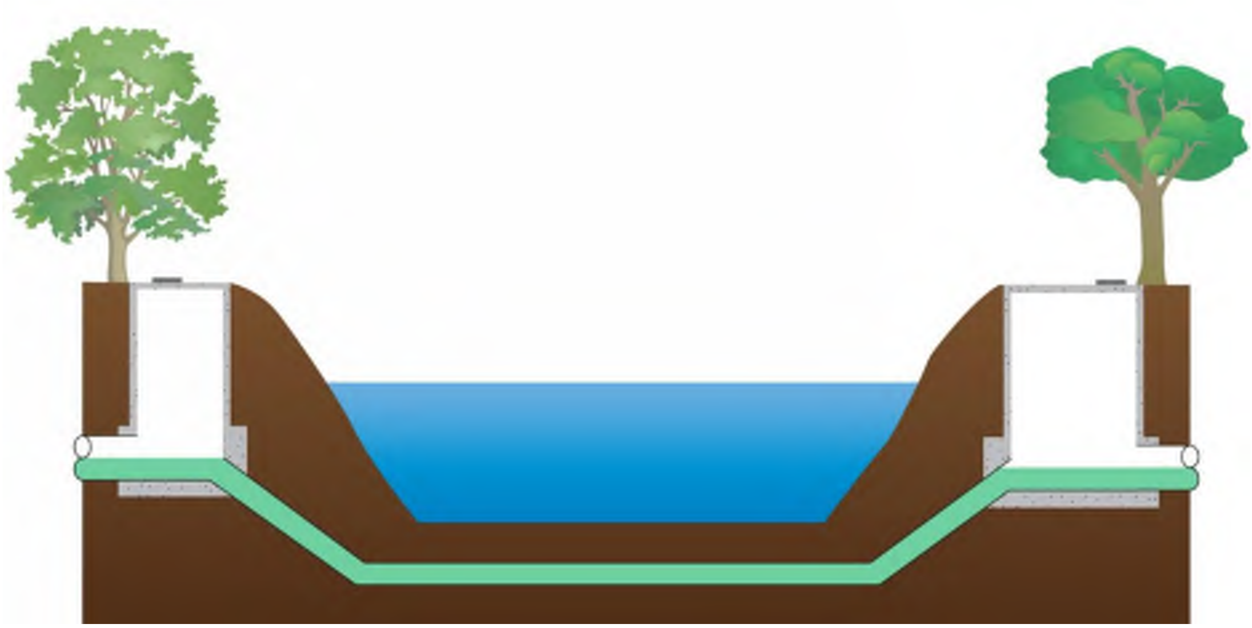


Figure 1: Diagram of Inverted Siphon

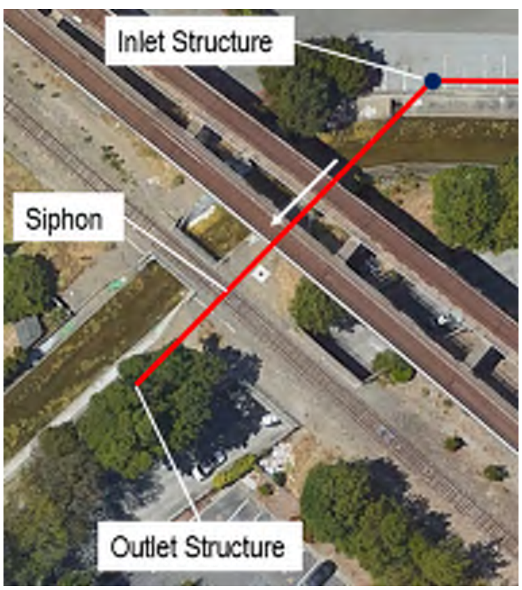


Figure 2: OLSD inverted siphon locale

Figure 3: Poor access to outlet structure

Just southwest of Alameda County, the City of Santa Clara serves the city's population of approximately 130,000 residents. In 2011, when installing a new inverted siphon in its collection system, the City also installed a new grit trap. Due to the success of their first grit trap, a second grit trap was installed in 2015 upstream of an existing inverted siphon. Since the grit traps were installed, the utility has successfully maintained its inverted siphons, finding less sediment buildup, and reduced the potential for sanitary sewer overflows due to a clogged siphon.

METHODOLOGY

Most collection systems have inverted siphons which, at one point or another, require cleaning due to the buildup of sand, grit, gravel, and other materials, including fat, oil, and grease. The frequency of cleaning is site specific and often ranges from monthly to twice a year. The duration necessary to clean an inverted siphon is also site-specific. If a siphon takes a full day to clean, installing a grit trap will reduce maintenance time.

The decision to add a grit trap for OLSD stemmed from the difficult to access siphon outlet structure. It was difficult to navigate the combination sewer cleaning truck to and from the outlet, whereas the upstream manhole in a frequently empty area of a large parking lot was easily accessible by the equipment. To address this issue, the OLSD installed an 2400 mm-diameter by 1500 mm-deep (8-foot-diameter by 5-foot-deep) grit trap upstream of the siphon inlet structure on the 900-mm (36-inch) trunk sewer.

The City of Santa Clara's first grit trap was installed as part of a new inverted siphon included in the Walsh Avenue Sanitary Sewer Improvements project. The grit trap was installed on the new 680 mm (27-inch) diameter trunk sewer twenty feet upstream of the siphon. The City's second grit trap was installed as part of the Trimble Road Trunk Sewer Improvements project. The grit trap was installed on an existing 1200 mm (48-inch) trunk sewer 180 meters (600 feet) upstream of an existing inverted siphon.

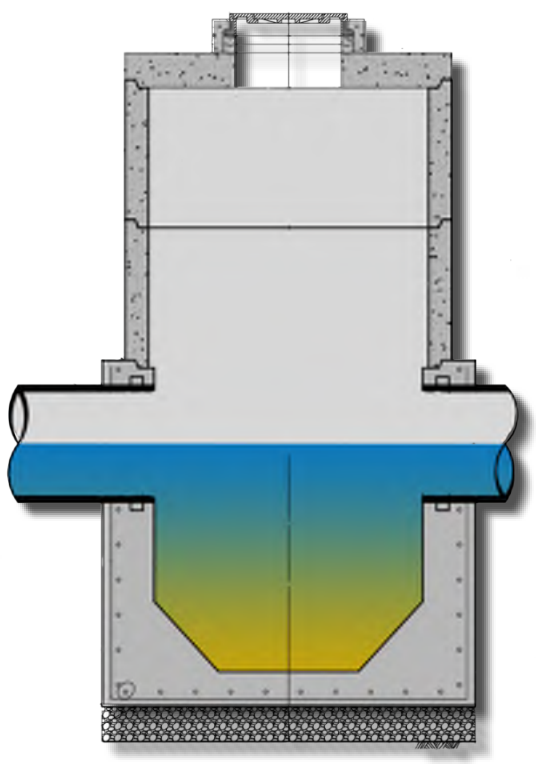


Figure 6: Digital Rendering of Grit Trap

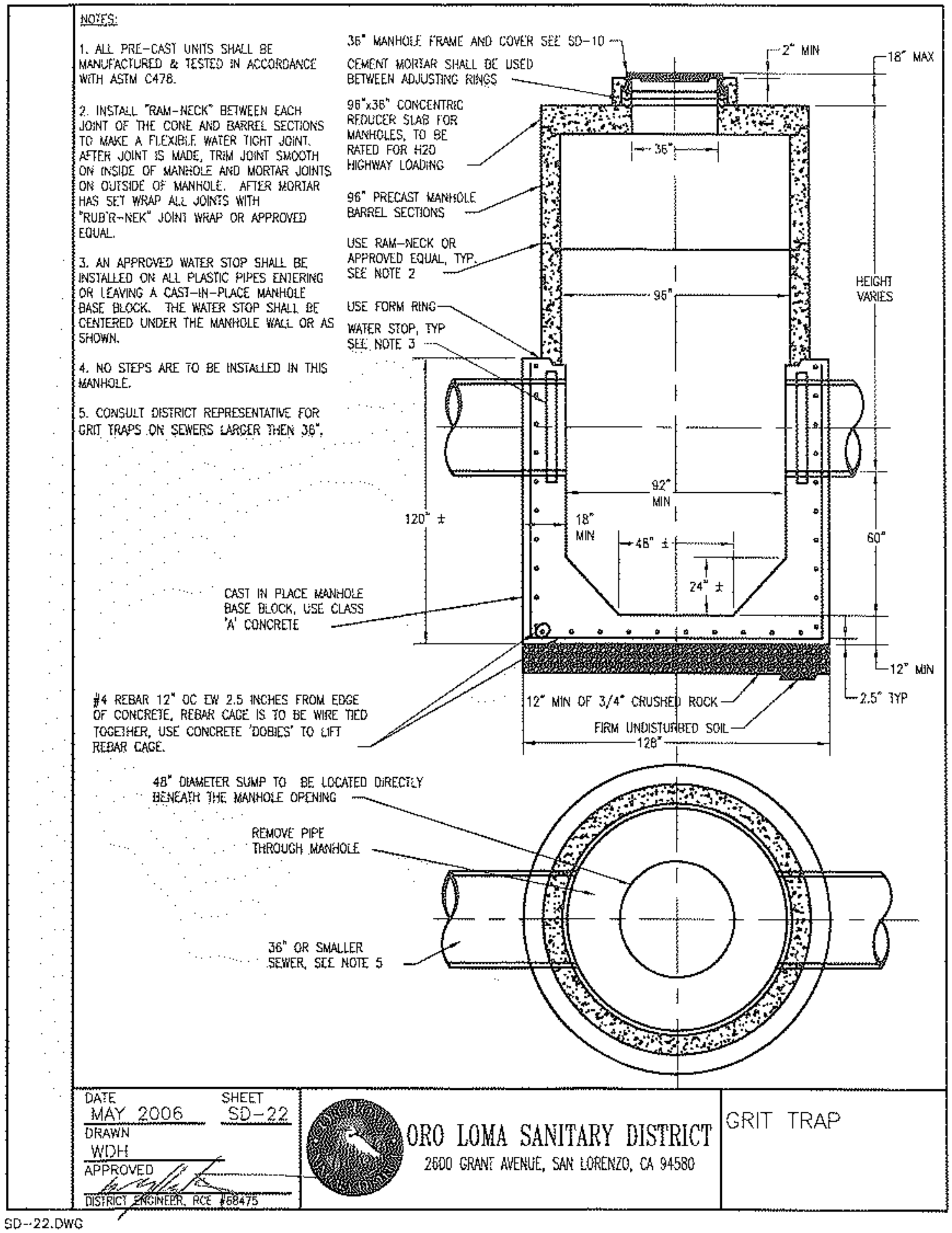


Figure 7: OLSD Grit Trap Standard Detail

RESULTS
In both collection systems, the addition of grit traps has proved successful in keeping inverted siphons clear of large debris and easing the maintenance for operations teams. Rather than clog up the collection system's flow, grit, gravel, sand, and other debris falls into the trap where it settles.

Neither agency has experienced odor issues since the grit traps were installed. If odor issues did occur, and if no other solutions solved the odor issue, a grit trap could be abandoned simply by filling in the trap with concrete and converting the grit trap to a normal manhole.

The sediment load of the sewershed and the capacity of the grit trap dictates how often it needs to be emptied, which unlike the full day necessary to clean out a single inverted siphon, the same crew can clean a grit trap within three hours.

Grit Trap Cleaning Process
As show below in Figures 8 to 13, cleaning a grit trap is more efficient than cleaning an inverted siphon. The steps to do so include:

1) Position the combination sewer cleaning truck at the grit trap so the vacuum boom can easily reach the grit trap and allow workers to safely move around the vehicle.
2) Vacuum tube attachments (aka intake tube attachments) are added to the vacuum boom to match the depth of the grit trap. The OLSD crew has a section fabricated from plexiglass to easily monitor the material being removed so the crew knows when the grit trap is clean. OLSD also uses an adjustable fluidizing tube (aka air adapter fitting) to adjust the amount of air allowed in the tube. As this author understands it, this fitting creates a "mini tornado" inside the tubing, which swirls up a mixture of water, sediment, and air from the bottom of grit trap. There are similar products in the marketplace that achieve the same result, including Vactor's Higbee vacuum tube for example.
3) Prior to lowering the vacuum boom into the grit trap, a member of the crew is using a long rod to break up the sediment settled at the bottom of the grit trap. Due to the myriad of components in raw sewage, the combination of sand, gravel, fats, starches, and proteins can often create a semi-solid mass in the grit trap. Loosening this up will help expedite the cleaning process.
4) The boom is lowered into the grit trap and sediment is vacuumed up. The suction tube will need to be moved so the suction end stays in the sediment
5) As the combination sewer cleaning truck fills with water and sediment, it is important to decant the water from the tank back into the sewer. A drain hose off the back of the truck removes the water and returns it to the sewer collection system downstream from the grit trap. This allows more sediment to be removed before the truck needs to be emptied.

The OLSD cleaning crew fills the 2.7-cubic-meter (3.5 cubic yard) truck in approximately 30 minutes after the cleaning begins (the entire process of setup, cleaning, and dumping takes approximately 4 hours). The crew fills and dumps the truck twice per cleaning event; sediment is dumped at the WWTP. The City of Santa Clara uses a similar process.

Figure 8: OLSD grit trap location (note the easily accessible location)

Figure 9: Grit trap at the beginning of cleaning

Figure 10: Attachments for vacuum (note the clear plexiglass tube)

Figure 11: Cleaning begins

Figure 12: Drain hose decanting water from Combination sewer cleaning truck

Figure 13: Sediment cleaned from grit trap

DISCUSSION

While grit traps are not commonly used in collection systems, installing grit traps upstream of inverted siphons should be considered, especially when access to siphons is difficult. Grit traps minimize maintenance efforts, improve cleaning efficiency, and may result in a cost savings.

The investment in grit traps varies depending on depth to groundwater and other factors. While there is not a large bid database of cost, grit traps are essentially large, deep manholes. The City of Santa Clara opened bids for the Trimble Road Trunk Sewer Improvements grit trap in February 2015. The specs included:

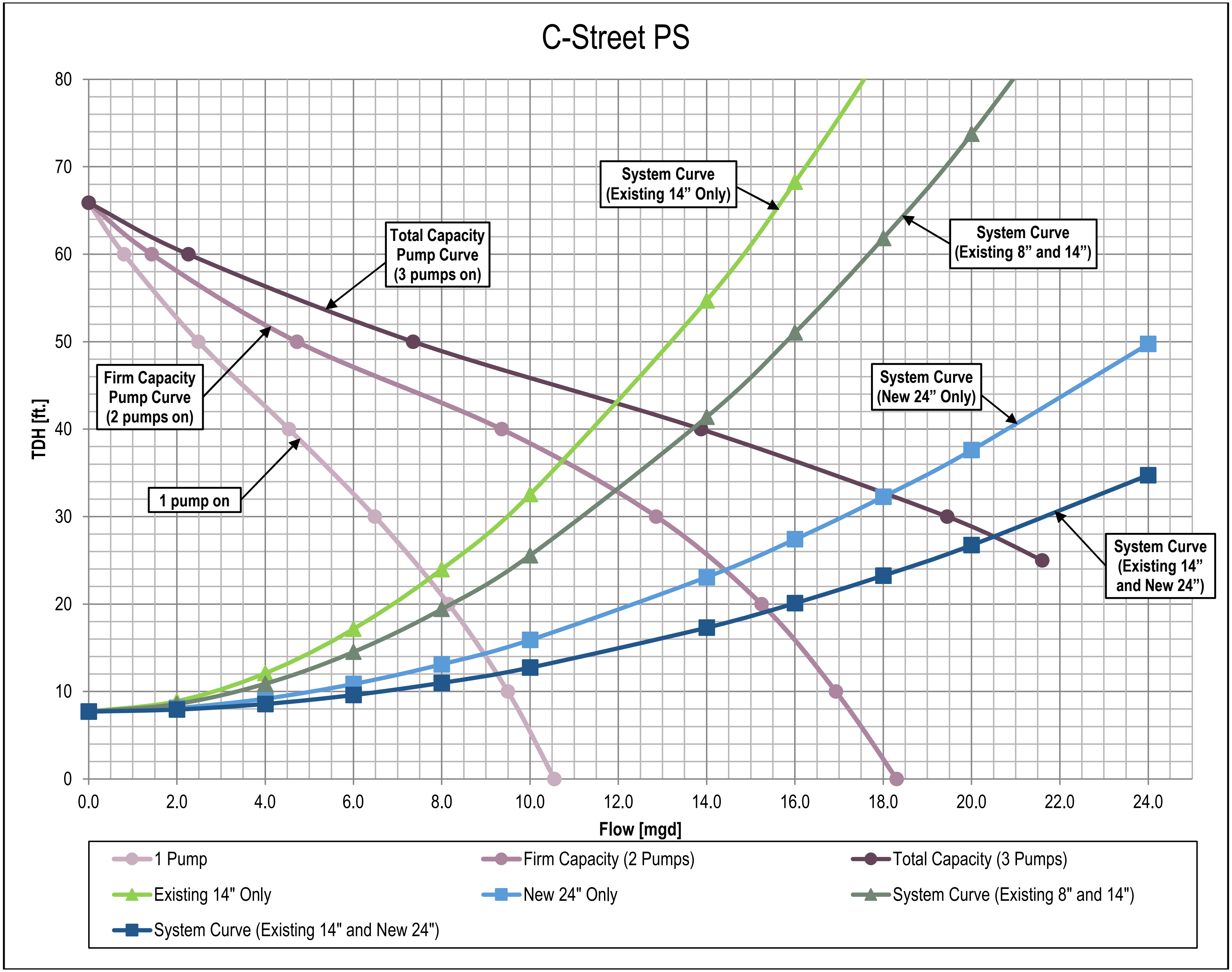
- Diameter of Sewer: 1200 mm (48-inch)
- Depth to Invert of Sewer: 2.4 meters (8-feet)
- Diameter of Grit Trap: 3000 mm (120-inch)
- Depth to Bottom of Grit Trap from Ground Surface: 4 meters (13-feet)
- Bids for Grit Trap: one at $40,000 and two at $85,000

REFERENCES

Oro Loma Sanitary District, June 2014, Oro Loma Sanitary District Standards, Standard Detail 22, Grit Trap.

APPENDIX F – FORCE MAIN DETAILS

Appendix F
System and Pump Curves



Appendix F
Cost Estimate

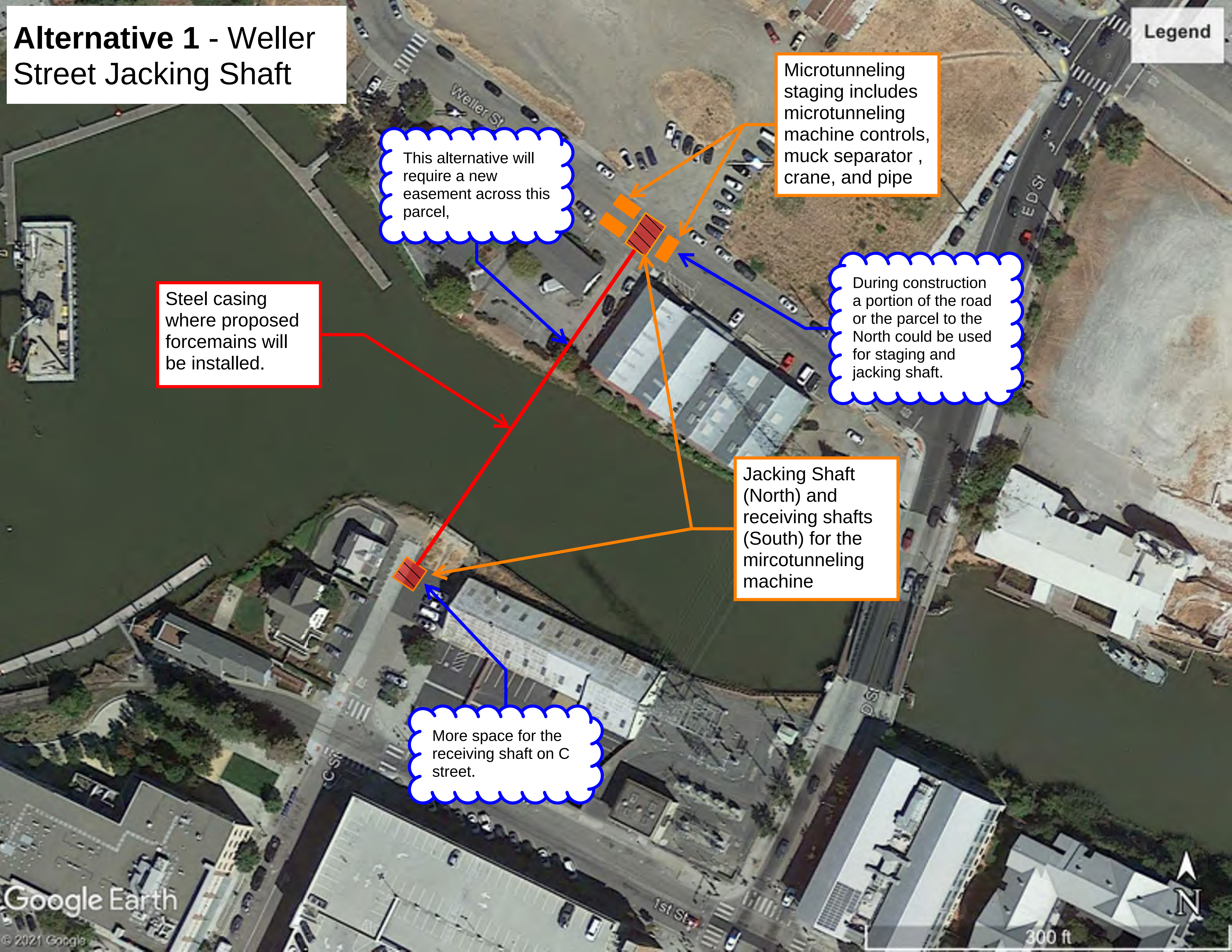
Project 1: 24" Force main Installation -- C Street Pump Station Improvements

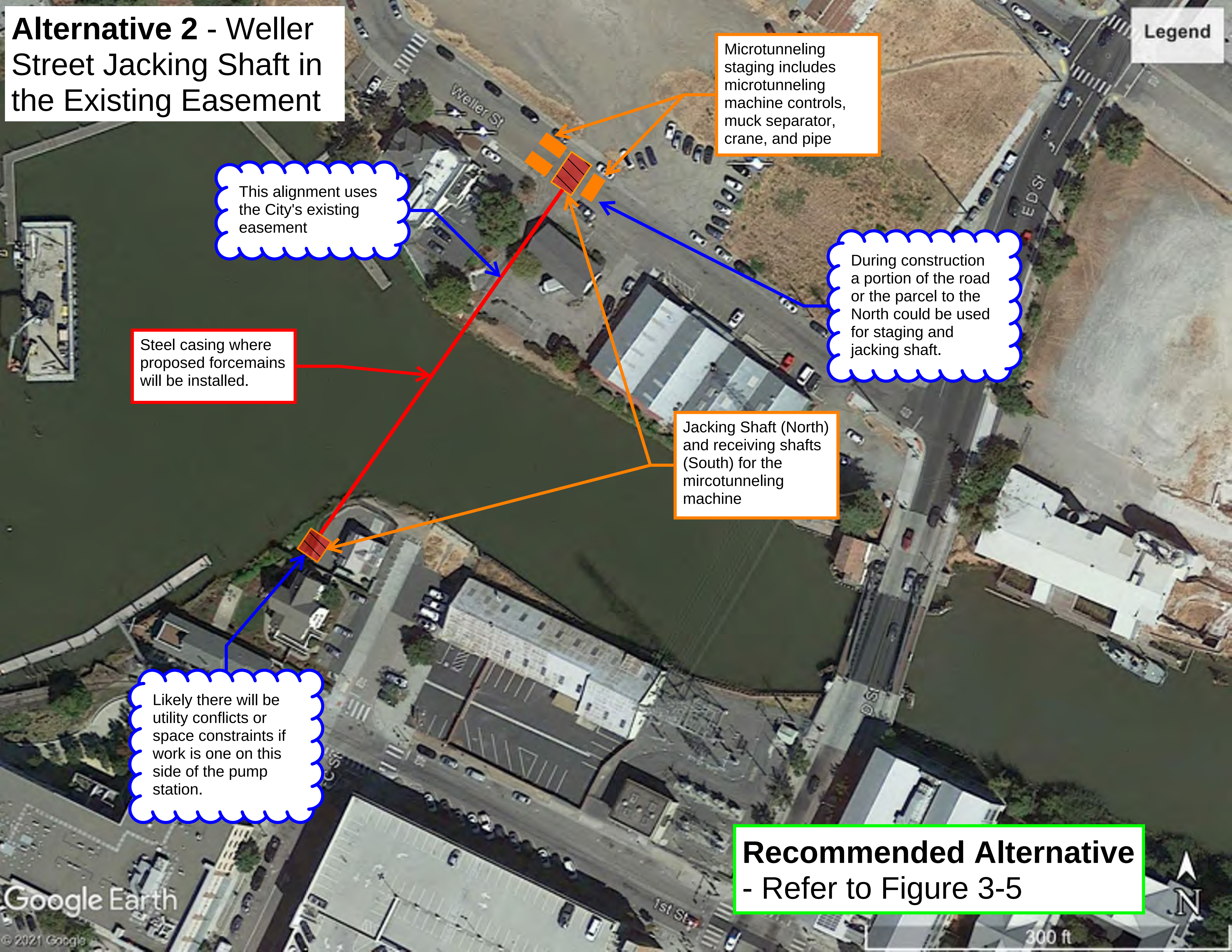
PROJECT DESCRIPTION	
Project ID	South Area Modeling Project
Project Name	24" Force main Installation -- C Street Pump Station Improvements
Description	Install 3 new force mains (2 - 12" forcemains and 1 - 24" forcemain) through a 54" casing installed using the microtunneling technique under the Petaluma River. The casing will be installed from the C Street Pump Station to Weller Street. The new force mains will reduce the risk of the existing force mains failing due to age and operational damage. The new microtunneled casing and forcemains are assumed to be approximately 350 ft in length. The jacking and receiving shafts will be approximately 46 ft deep to maintain 20 ft of clearance from the bottom of the Petaluma river to the top of the casing. Additional allowances for new valve vaults and pipe manifolds will are included. This piping will tie the new force mains into the existing C Street wet well and gravity sewer on Weller Street. An additional 1,200 feet of gravity sewer is required to convey the predicted 17.8 mgd of wet weather flow to the existing 48-inch sewer on Lakeville Street.
Est. Construction Cost	$ 6,591,000
Estimated Capital Imp. Cost	$ 8,239,000
Comments	i) Mobilization/Demobilization included as 10% applied to construction cost.
	ii) Jacking shaft will be 30 ft by 20 ft and 46 ft deep. The microtunneling machine will be installed at the bottom of the shaft. Significant shoring will be required to maintain the integrity of the shaft during construction.
	iii) The receiving shaft will be 20 ft by 20 ft and 46 ft deep.
	iv) The road will be restored where the jacking and receiving shafts are installed after construction is complete.
	v) A 54" steel casing will be installed by the microtunneling machine.
	vi) Two 12" forcemains and a 24" forcemain will be installed inside the 54" steel casing installed by the microtunneling machine.
	vii) A 24" gravity sewer is needed to convey the predicted 17.8 mgd of wet weather flow from the C street pump station with the existing 24" gravity sewer. The parallel gravity sewer will extend from manhole NEC00080 to NEC00000.
	viii) Traffic control will be required on Weller Street. The street will be closed during construction and will have the jacking shaft dug in the street.
	ix) The existing forcemains crossing the Petaluma River will be abandoned.
	x) A valve vault and a discharge pipe manifold and a vertical section of pipe will be added to the discharge of the C Street PS's existing pumps to the steel casing at the bottom of the jacking/receiving shaft. This valve vault will control how flow is distributed between the proposed 12-inch force main and24-inch force main.
	xi) A discharge pipe manifold and vertical section of pipe will be added to the discharge of the proposed force mains on Weller Street. This discharge manifold will control which parallel gravity sewer will receive flow.
	xii) A new discharge manhole and gravity sewer will convey flow from the proposed force mains to a new manhole installed upstream of Manhole NEC00080. Four new manholes will be installed to tie into the existing 48-inch gravity sewer in Lakeville Street.

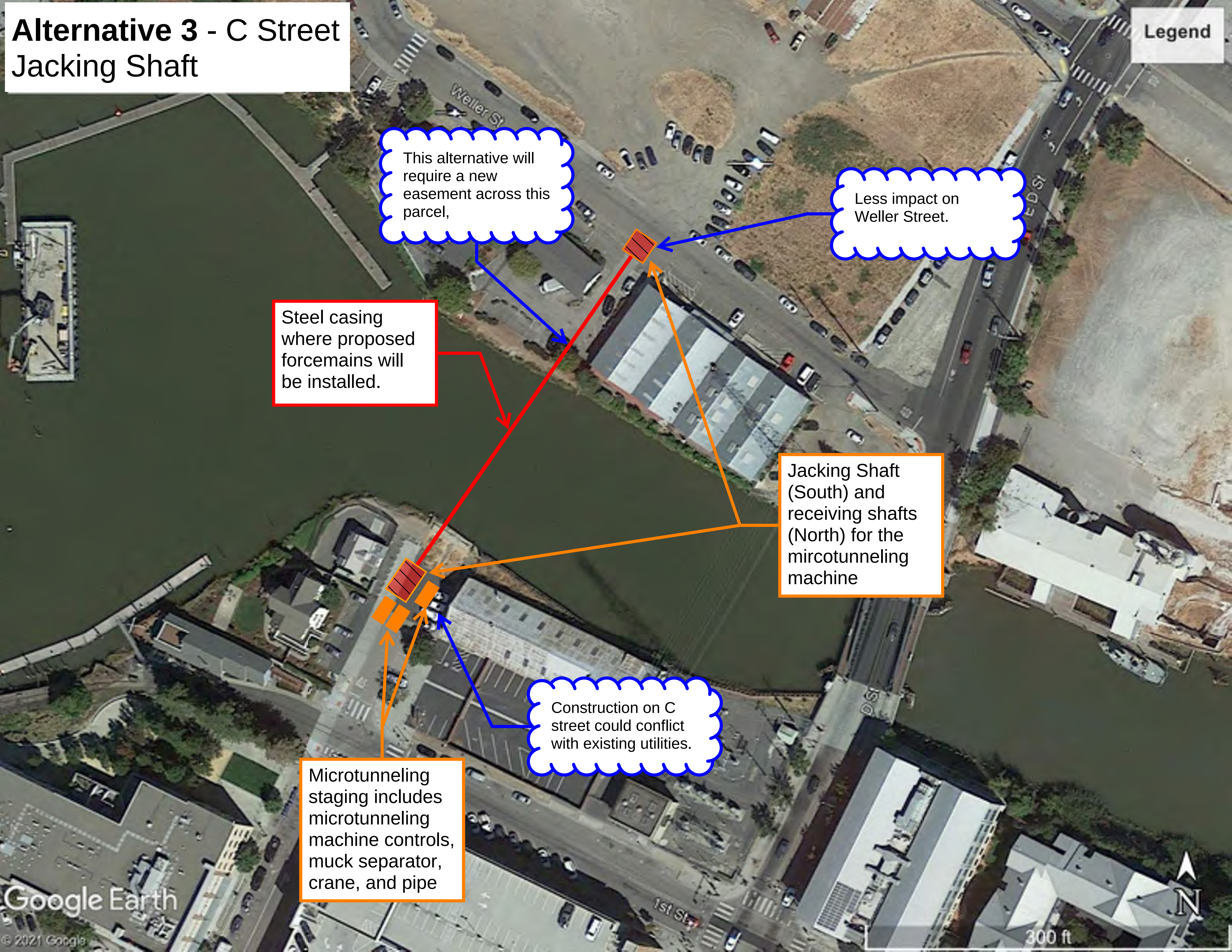
Item	Description	Size	Units	Quantity	Unit	Unit Cost		Total Cost
Site Work & Special Construction								
	Jacking Shaft (*ii*)			1	EA	$2,200,000	$	2,200,000
	Receiving Shaft (*iii*)			1	EA	$400,000	$	400,000
	Road Restoration *(iv)*			1	LS	$17,000	$	17,000
Forcemain								
	54" Steel Casing (*v*)	54	IN	350	LF	$2,800	$	980,000
	12" FM (*vi*)	12	IN	350	LF	$230	$	80,500
	24" FM (*vi*)	24	IN	350	LF	$280	$	98,000
	12" FM (*vi*)	12	IN	350	LF	$230	$	80,500
	24" Gravity Sewer (vii)	24	IN	1,200	LF	$340	$	408,000
	Traffic Control *(viii)*			1	LS	$30,000	$	30,000
	Weller Street Manifold Connection (ix)			1	LS	$60,000	$	60,000
	Pipe Abandonment *(x)*			1	LS	$5,000	$	5,000
Mechanical Equipment								
	C-Street Discharge piping and valve vault (*xi*)			1	LS	$150,000	$	150,000
	New Manholes (*xii*)			5	EA	$20,000	$	100,000
Electrical Equipment								
Subtotal							**$**	**4,609,000**
	Mobilization/Demobilization *(i)*			10	%		$	461,000
Construction Subtotal							**$**	**5,070,000**
	Contingencies			30	%		$	1,521,000
Construction Cost							**$**	**6,591,000**
	Engineering, Administration, Legal			25	%		$	1,648,000
Capital Improvement Cost							**$**	**8,239,000**

APPENDIX A - FORCE MAIN DETAILS

Microtunneling Alternative Alignments


Alternative 1 - Weller Street Jacking Shaft
Legend
Weller St
Microtunneling staging includes microtunneling machine controls, muck separator , crane, and pipe
This alternative will require a new easement across this parcel,
Steel casing where proposed forcemains will be installed.
During construction a portion of the road or the parcel to the North could be used for staging and jacking shaft.
E D St
Jacking Shaft (North) and receiving shafts (South) for the mircotunneling machine
D St
More space for the receiving shaft on C street.
C St
1st St
Google Earth
© 2021 Google
300 ft
N


Alternative 2 - Weller Street Jacking Shaft in the Existing Easement
Legend
Microtunneling staging includes microtunneling machine controls, muck separator, crane, and pipe
This alignment uses the City's existing easement
During construction a portion of the road or the parcel to the North could be used for staging and jacking shaft.
Steel casing where proposed forcemains will be installed.
Jacking Shaft (North) and receiving shafts (South) for the mircotunneling machine
Likely there will be utility conflicts or space constraints if work is one on this side of the pump station.
Recommended Alternative - Refer to Figure 3-5
Weller St
E D St
D St
C St
1st St
N
300 ft
Google Earth
© 2021 Google



Alternative 3 - C Street Jacking Shaft
Legend
Weller St
This alternative will require a new easement across this parcel,
Less impact on Weller Street.
E D St
Steel casing where proposed forcemains will be installed.
Jacking Shaft (South) and receiving shafts (North) for the mircotunneling machine
D St
Construction on C street could conflict with existing utilities.
Microtunneling staging includes microtunneling machine controls, muck separator, crane, and pipe
1st St
Google Earth
© 2021 Google
300 ft
N