Technical Memorandum

Fair Oaks Sewer Maintenance District Sewer Master Plan

Subject:	Capacity Analysis
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1 Introduction

The Fair Oaks Sewer Maintenance District (FOSMD or District) provides wastewater collection services to an approximate 5-square mile area south of the City of Redwood City in San Mateo County. The District is the largest of the ten wastewater districts operated and maintained by the County of San Mateo Department of Public Works, and serves approximately 7,200 customers in the unincorporated communities of North Fair Oaks and Sequoia Tract, portions of the City of Redwood City, and Towns of Atherton and Woodside. The collection system includes approximately 82 miles of 4- to 33-inch diameter sewer pipelines. The system discharges to the City of Redwood City's collection system at Veterans Boulevard, from where it is conveyed to the Silicon Valley Clean Water (formerly South Bayside System Authority) interceptor system and treatment plant.

The District's last Sewer Master Plan was prepared in 2000. As part of updating the Master Plan, a hydraulic model of the District's trunk sewer system was developed to evaluate the capacity of the system to handle peak wet weather flows. 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 District'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 CS™ by Innovyze, a fully dynamic hydraulic model that has been used for many other collection systems in the Bay Area, including Redwood City (to which the FOSMD system discharges). RMC 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. Note: there are no pump stations in the FOSMD system.
- **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 network for the District includes all pipes 10 inches and larger in diameter and additional 6- and 8-inch lines that were either part of a flow split and could potentially carry flows from a larger diameter pipe, or were considered important because of a significant contributing sewershed. In total, the network includes about 20 miles of pipelines, or about 25 percent of the total length of sewers in the system. The modeled network is shown in **Figure 2-1**.

Flows from sewers owned and operated by the Town of Woodside discharge into the FOSMD sewer system primarily from two areas known as the Town Center Area and the Cañada Sewer Area. Flows from the Town Center Area are pumped into the FOSMD system from the Town's Town Center Pump Station, and flows from the Cañada area discharge by gravity to FOSMD's sewer on Cañada Road. The Town Center Pump Station was not modeled, and information about its current capacity was not available. For purposes of this study, it has been assumed that the pump station has or will have sufficient capacity to deliver all flows from its tributary area to District sewers. Under an agreement with FOSMD dated May 8, 2001, the Town may convey up to 100,000 gpd (0.1 mgd) annualized daily average flow through the FOSMD system, although current and projected flows (on an average annual basis) are likely much lower.

All flow from the District's sewer system is collected in two pipelines that join at the Interceptor Flow Metering Station at the downstream end of the system. The Interceptor Flow Metering Station is scheduled for rehabilitation and flow data is not yet available. Flow entering the Interceptor Flow Metering station is discharged through two 30-inch sewers owned by Redwood City.

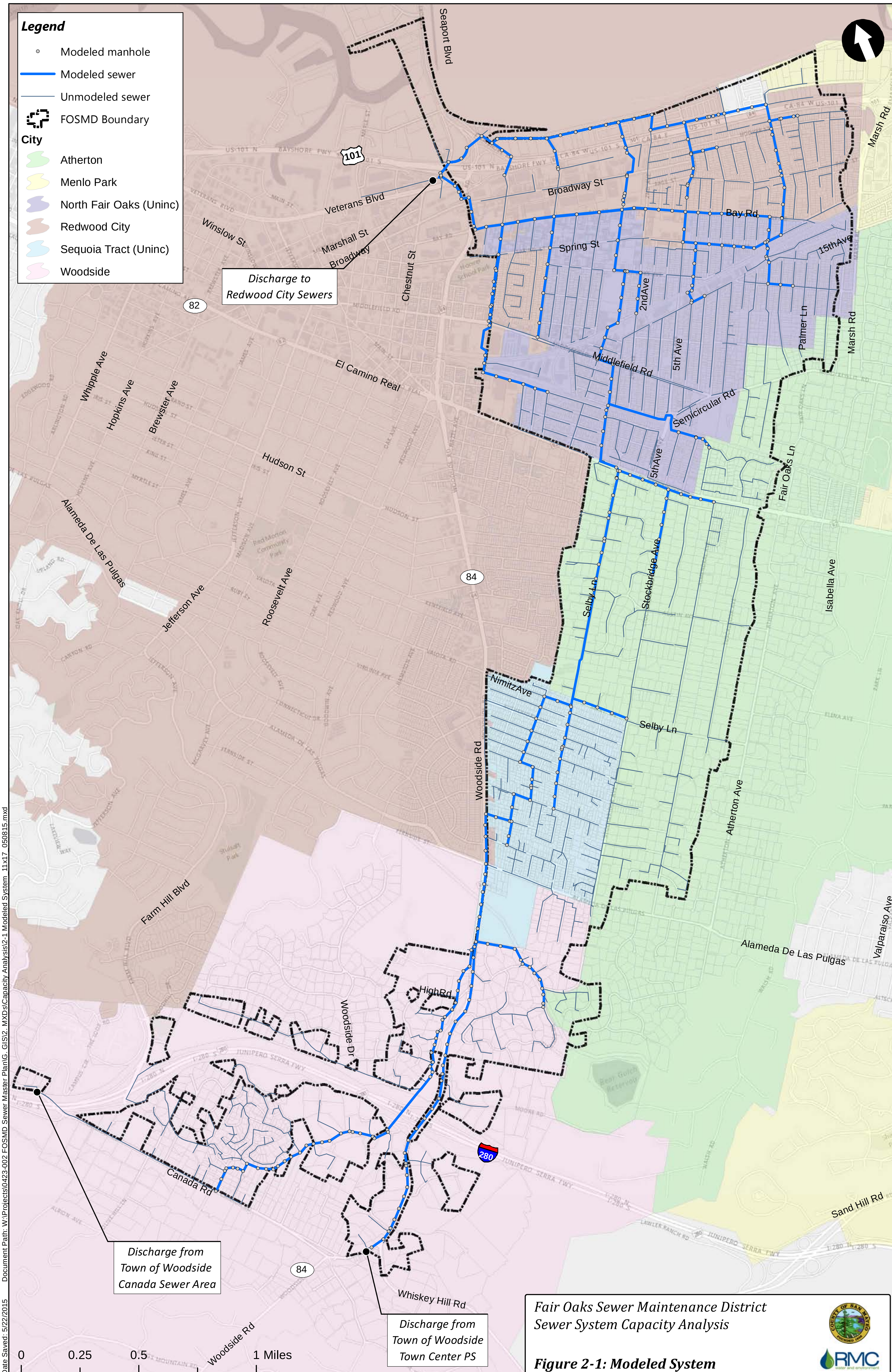

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Figure 2-1: Modeled System

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The District's existing and potential future service area was divided into 311 subcatchments with an overall average size of about 10 acres. Each subcatchment "loads" to a manhole in the modeled network.

2.2.1 Model Network Construction and Validation

The data used to define the FOSMD model network was provided by the County in the form of GIS shapefiles of the sewer system pipelines (SWR_MAINS_FOSMD.shp) and manholes (SWR_NODES_FOSMD.shp). 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 rims, inverts, and pipeline sizes, were refined from the information in the GIS shapefiles based on the following data sources:
    - Pipeline sizes were extracted from GIS data and generally assumed to be accurate. Elevation data (manhole rim elevations and pipeline inverts) were not available in the GIS data.
    - A HYDRA model developed for the 2000 Fair Oaks Sewer Master Plan included many of the larger diameter trunk sewers. Rim and invert data from this previous model were used, and updated if more current data (as-built or survey) were available.
    - In select locations, record drawings for several pipelines were provided by the County and were used to refine elevation, size, and connectivity information. The following as-built drawings were used:
        - Bay Road and Selby Lane Sanitary Sewer Improvement Project (2001, File No. 1/4622)
        - Dumbarton Avenue, Oakside Avenue, Fair Oaks Avenue, Barron Avenue, Bay Road, 12th Avenue and Spring Street Sanitary Sewer Improvement Project (2005, File No. 1/4659)
        - Fair Oaks Sanitary Sewer Improvement (SF700) (2004, File No. 1/4661)
        - Interceptor Sewer and Metering Station (1964) – information in these drawings matched data in the 2000 Hydra model. After model development, the meter was modified to improve meter operations. These modifications were not incorporated in the model, but the changes should not impact model conclusions.
    - Where invert elevation data were missing or inconsistent with nearby elevations, and not determined through as-built or survey information, interpolated values between known values were used as appropriate.
    - Elevation data in the HYDRA model and in the as-builts were adjusted as needed to the NAVD 88 datum.
    - For manholes and pipelines that were not included in the original HYDRA model, and for which no as-built information was available, survey data were collected. Rim elevations

for 211 manholes were surveyed by the County's survey consultant, BKF Engineers, and depths to each pipe invert were measured.

- While the surveyed elevations also used the NAVD 88 datum, discrepancies were identified at manholes for which both as-built and survey information was available. To ensure consistency with the rest of the model, survey data was adjusted to match the as-built elevations. Pipe slopes were calculated based on survey data.

- 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 County, and an approach to resolve the discrepancy was identified.
- The sources of model data (e.g., GIS, record drawings, field verification) 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 are modeled assuming a Manning's n of 0.013.

Figure 2-2 shows the location of surveyed manholes and where as-built information was available.

2.3 Flow Monitoring Program

To support the development of the hydraulic model and flow projections for the Master Plan, a temporary flow monitoring program was conducted as part of this study during the 2013/2014 wet weather season. V&A Consulting Engineers, under sub-contract to RMC, conducted the monitoring at 15 sites on trunk sewers tributary to the District's interceptor system. In addition, three recording rain gauges were also installed. The location of the flow monitoring sites and rain gauges are shown in **Figure 2-3**. The figure also shows the associated tributary area (basin) for each flow meter. The locations of the flow meters relative to each other and to flow splits within the collection system are shown schematically in **Figure 2-4**. Note that eight of the meters were located downstream of other meters; therefore, the tributary areas shown for each of these meters are the "incremental" areas between the flow meter and tributary basins of the upstream flow meters. **Table 2-1** lists the flow meter locations, pipe diameters, and upstream meters.

The purpose of the flow monitoring program was to quantify the flows in the system 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 in various areas of the system. The meters and rain gauges were installed for a 1 1/2-month period from late February through early April 2014 to capture the flow from the tributary areas.

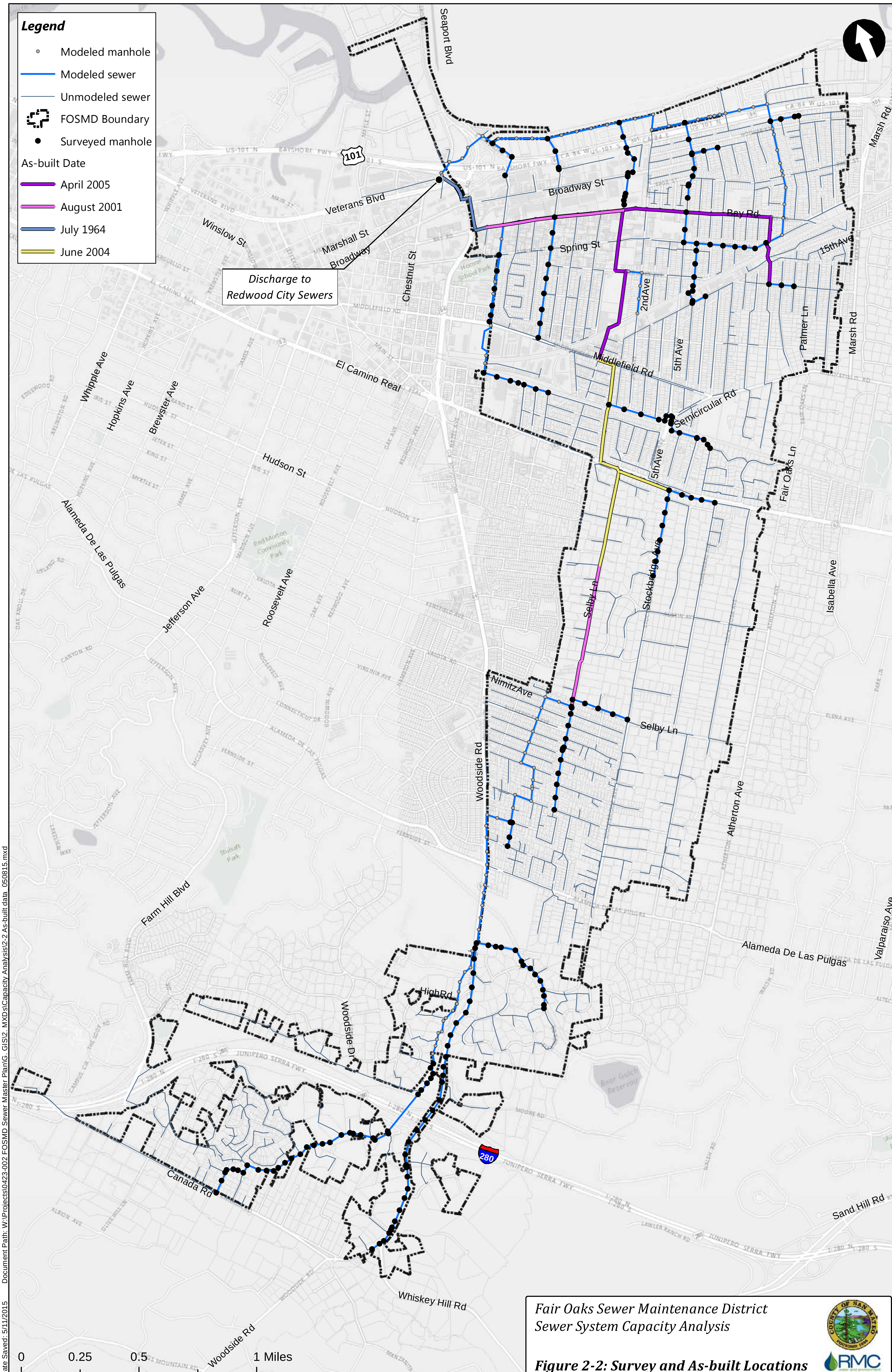

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Figure 2-2: Survey and As-built Locations

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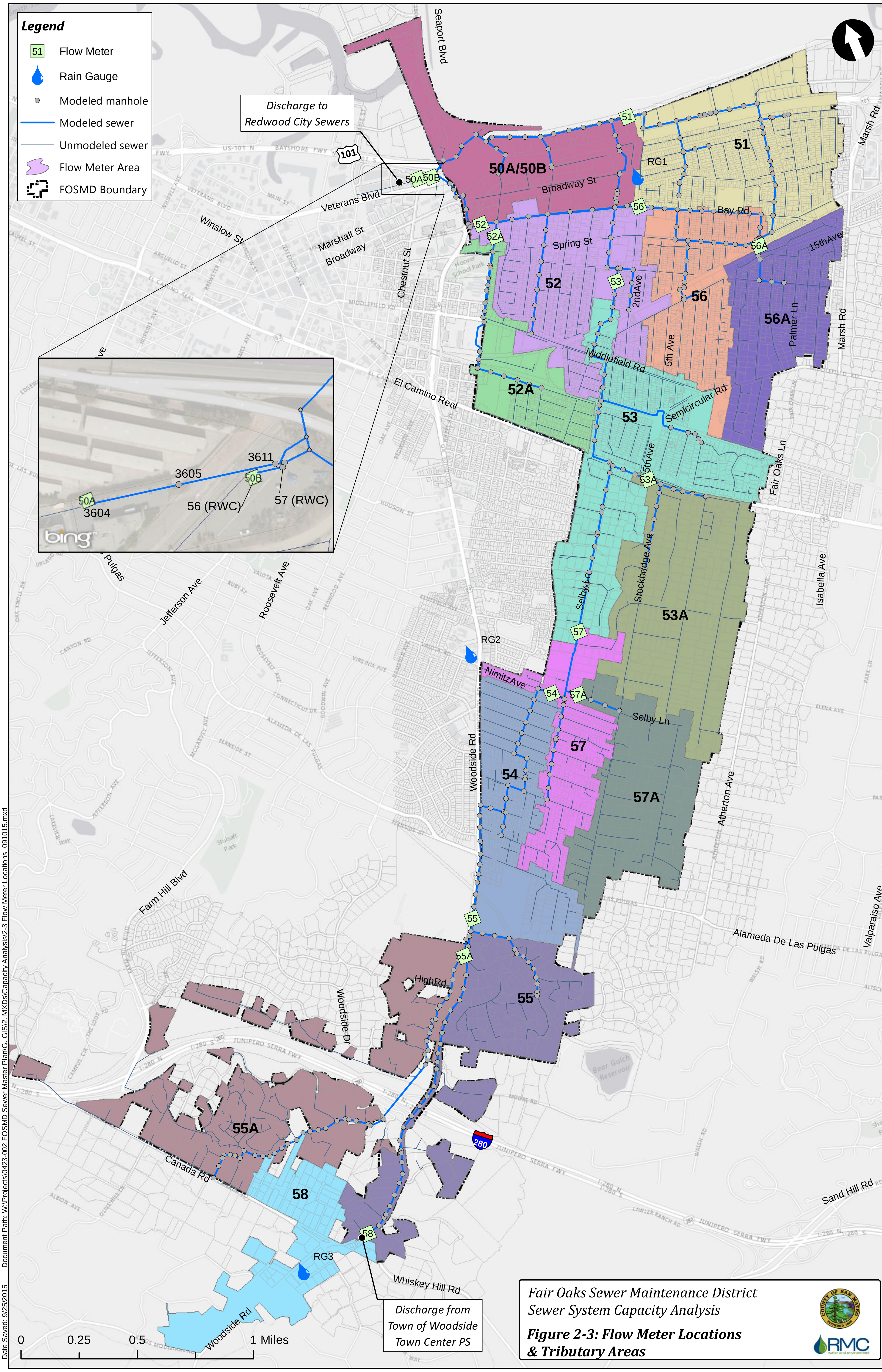

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Figure 2-3: Flow Meter Locations & Tributary Areas

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Figure 2-4: Flow Meter and Flow Split Schematic

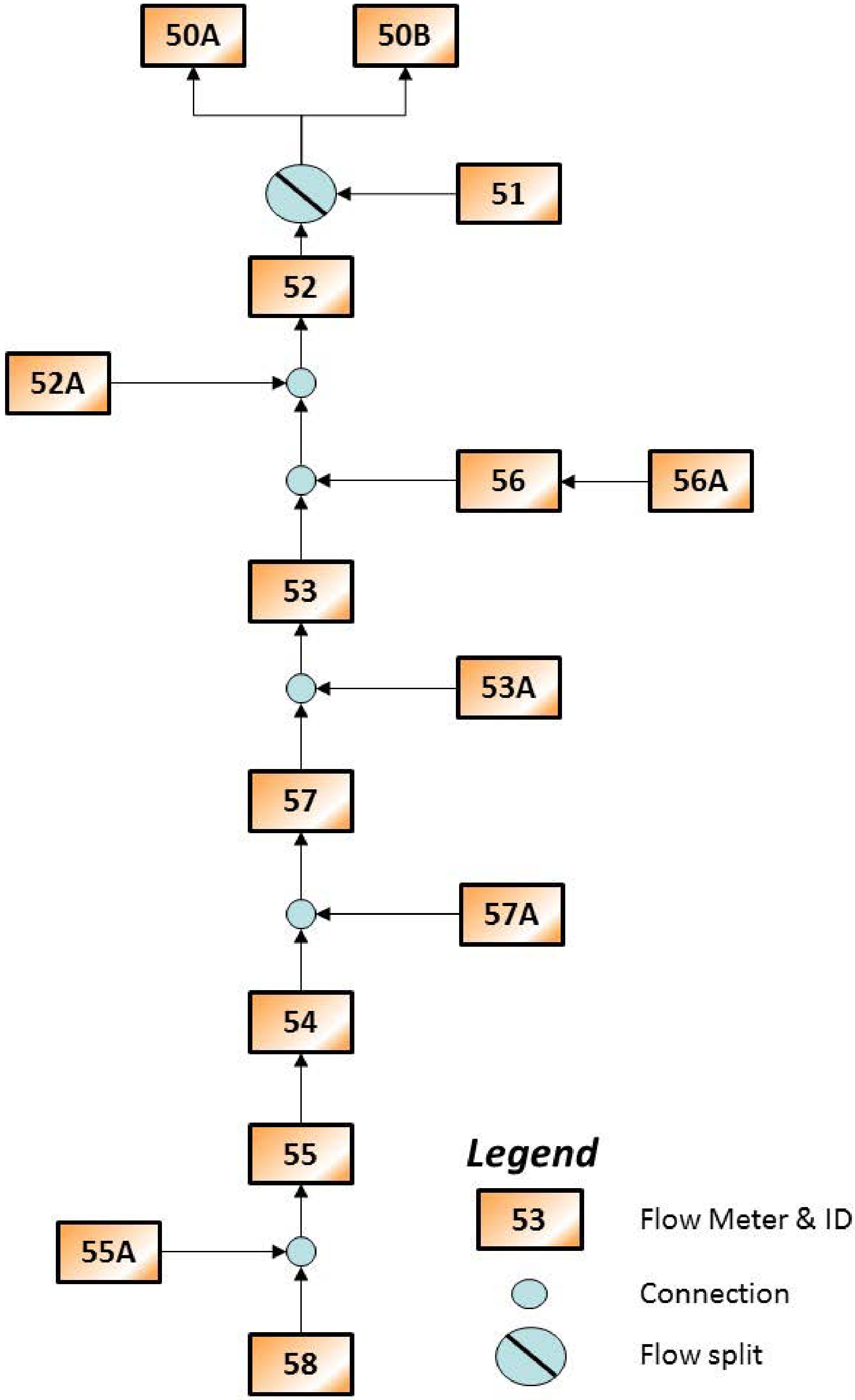


Table 2-1: Flow Meter Locations

Flow Meter ID (FM ID)	Manhole ID	Diameter (in)[a]	Downstream Meters	Upstream Meters
50A[b]	3604[c]	30		51, 52
50B[b]	MH ID 56[c]	30		
51	3877	18	50A, 50B	
52	3670	30	50A, 50B	52A, 53, 56
52A	3676	10	52	
53	3834	24	52	53A, 57
53A	5300	14	53	
54	5351	15	57	55
55	4947	10	54	55A, 58
55A	4956	10	55	
56	3991	18	52	56A
56A	4139	15	56	
57	6057	24	53	57A, 54
57A	5359	10	57	
58[d]	5810	6	55	

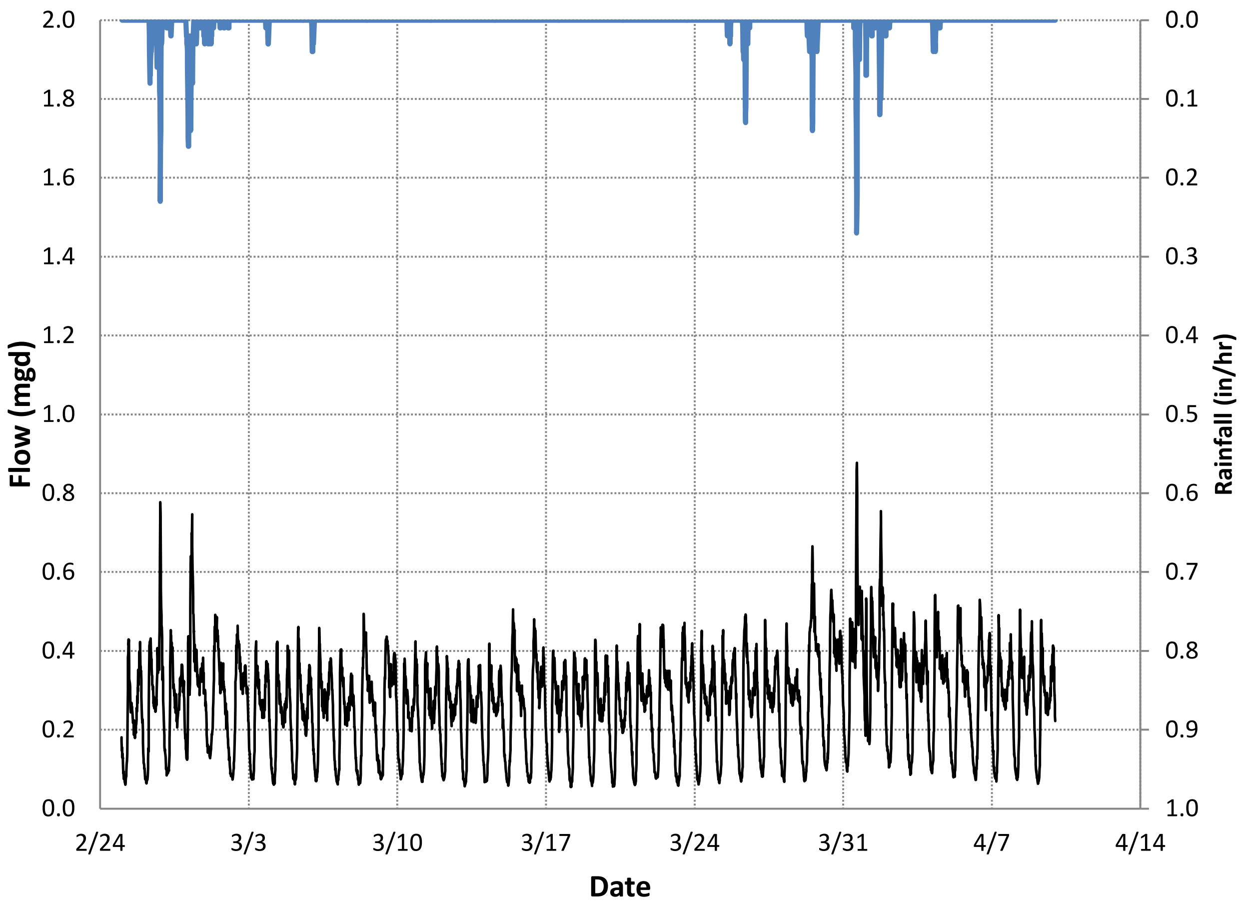
a. Actual measured diameter used for meter flow calculations may be slightly different than pipe nominal diameter.
b. Meter location is in Redwood City; however, all flow to meter originates in FOSMD.
c. Flow Meter 50A and 50B are located on pipes downstream of the Interceptor Flow Metering Station.
d. Flow discharged from Town of Woodside's Town Center Pump Station.

Rainfall was recorded at three sites in the District as indicated on **Figure 2-3**. Rainfall for the largest storm event, on March 31 through April 1, 2014, is summarized in **Table 2-2**. **Figure 2-5** 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: March 31 - April 1, 2014 Rainfall Event

Rain Gauge	Total 24-Hour Rainfall (in)	Peak Hour Rainfall (in)
Site RG1 (Fire Station #11)	0.54	0.22
Site RG2 (St. Pius School)	0.59	0.24
Site RG3 (Fire Station #7)	0.83	0.32

Figure 2-5: Plot of Typical Flow Data for Flow Monitoring Period (FM 56)



Note: mgd = million gallons per day

2.4 Flow Estimating Methodology

2.4.1 Wastewater Flow Components

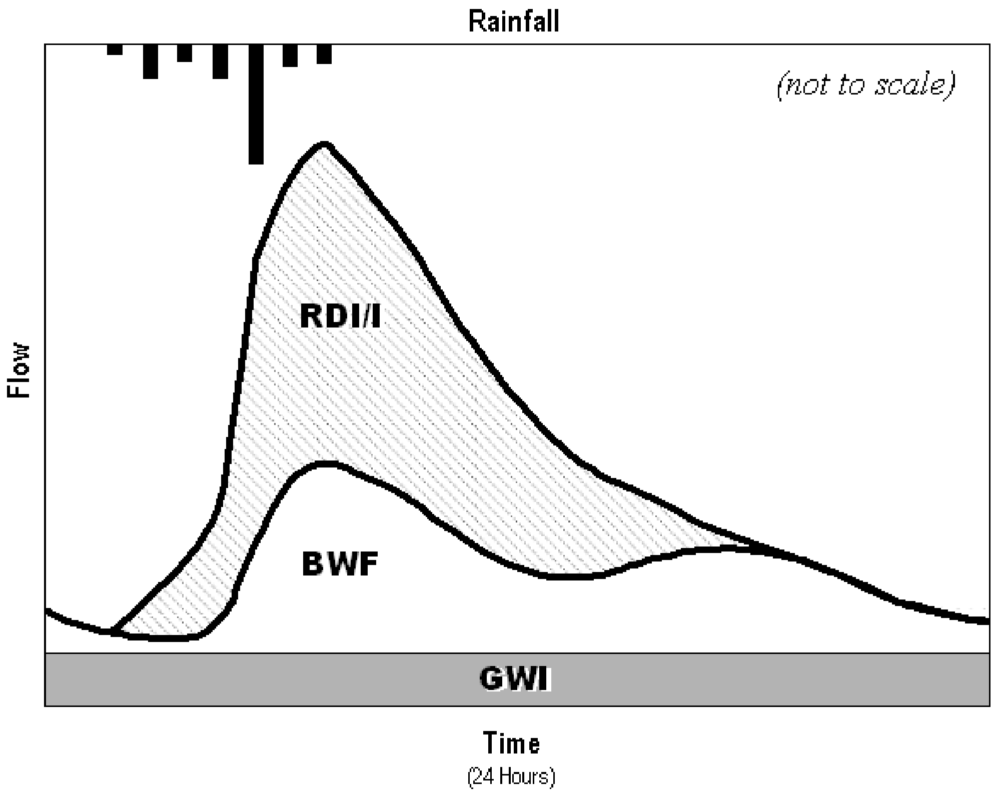
Wastewater flows include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I), as illustrated conceptually in **Figure 2-6**.

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 is 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. However, rainfall typically has long-term impacts on GWI rates, as evidenced by measurable increases in GWI after prolonged periods of rainfall.

RDI/I is 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 2-6: 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 7,700 parcels) and then aggregated into the 311 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 County, as well as sewer billing information. 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 from 2012 through 2014 were used as the basis for estimating residential and non-residential BWF. Several months during these years had very little rainfall; these months were excluded from the analysis as the lack of rainfall likely resulted in additional irrigation water use. Winter months used included January 2012, March 2012, February 2014, and March 2014. A sewer return rate of 80 percent was assumed, based on comparison of water to wastewater flow rates during model calibration.

All water billing records were geocoded according to parcel APN and assigned a customer type (commercial or residential) based on the Use Code in the sewer billing data. A visual assessment of the City using aerial photos confirmed that data were available for most significant developed parcels. **Figure 2-7** shows the geocoded water billing data by customer type.

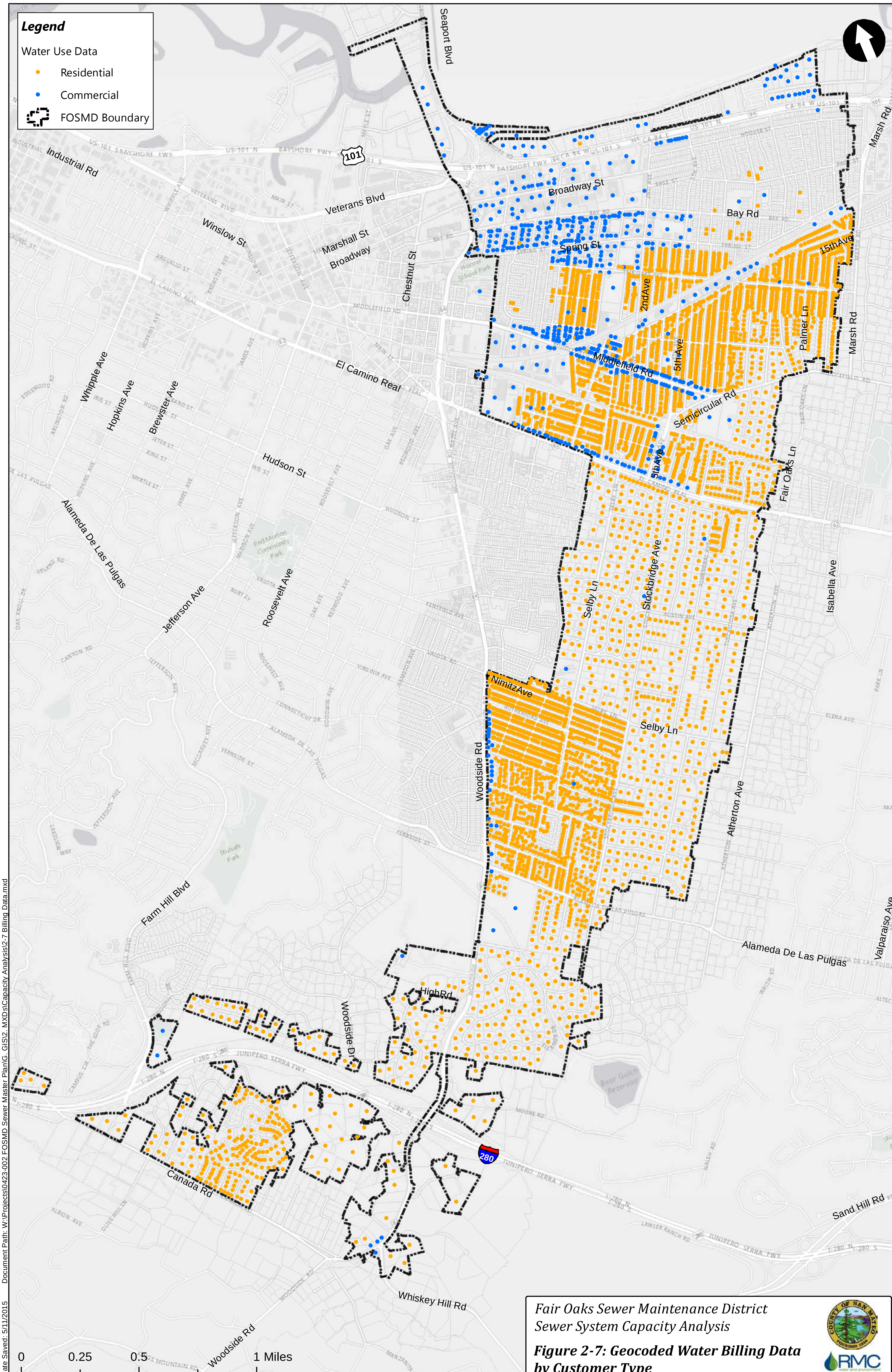


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Figure 2-7: Geocoded Water Billing Data by Customer Type

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Water use records were not available for residential parcels in Redwood City. Also, comparison of water use data with flow data in Atherton indicated that residential water use was not consistent with sewer flows; many of these parcels have large landscaped areas, which may have been watered during winter months. Residential sewer flows in Redwood City and Atherton were therefore estimated using the Equivalent Residential Units (ERUs) used for sewer billing. Based on comparison with wastewater flow rates from the flow meter data during model calibration (see discussion later in this TM), a flow rate of 160 gallons per day per ERU was used to calculate BWF from Redwood City and Atherton residential parcels.

As described previously, two portions of the Town of Woodside drain into the District, but are outside of the District boundaries and administered by the Town Center Sewer Assessment District (TCSAD): the Town Center Area and the Cañada Sewer Area. The Town of Woodside reimburses the District for these connections. Flows from the Town Center Area are pumped by the Town Center Pump Station into the District's sewer system at Whiskey Hill Road and Woodside Road. Based on flow from a temporary meter (Meter 58) located downstream of the pump station, the average dry weather flow from the Town Center Area is currently about 22,000 gpd. Recent payment information from the Town of Woodside to the District indicates that the TCSAD includes 116 residential units, as well as 24 non-residential connections. Based on a BWF rate of 160 gallons per day per ERU, residential flow contributes about 18,600 gpd, and the remaining 3,400 gpd comes from non-residential sources. The Cañada Sewer Area connects to District sewers on Cañada Road near Godetia Drive. Based on recent payment information, the Cañada Sewer Area includes 26 connected residential units, corresponding to a residential flowrate of 4,160 gpd. The locations of each of these areas are indicated in **Figure 2-8**.

Future BWF Loads

Based on discussions with County staff, very little development is expected to occur within the District boundaries; therefore, no future loads have been included within the District. Growth is also not anticipated in the Town of Woodside areas that drain into the District. However; there are several areas within the Town of Woodside that are currently on privately maintained septic systems that could be connected to the District's sewer system in the future. These areas were identified in the 2010 Town of Woodside Sanitary Sewer Master Plan and include parcels within the Redwood Creek Assessment District (but outside the District's current boundary) and other parcels identified as potential future connections. These parcels are indicated in **Figure 2-8**. A total of 224 potential parcel connections were identified, 77 of which are not within the District boundaries or the boundaries of the Town Center or Redwood Creek Assessment Districts; these parcels do not currently have connection rights to the District and would need to obtain capacity rights before connection. Based on an aerial review of the area, most parcels are single family residential. A standard BWF rate of 160 gallons per day per parcel was assumed for these potential future connections.

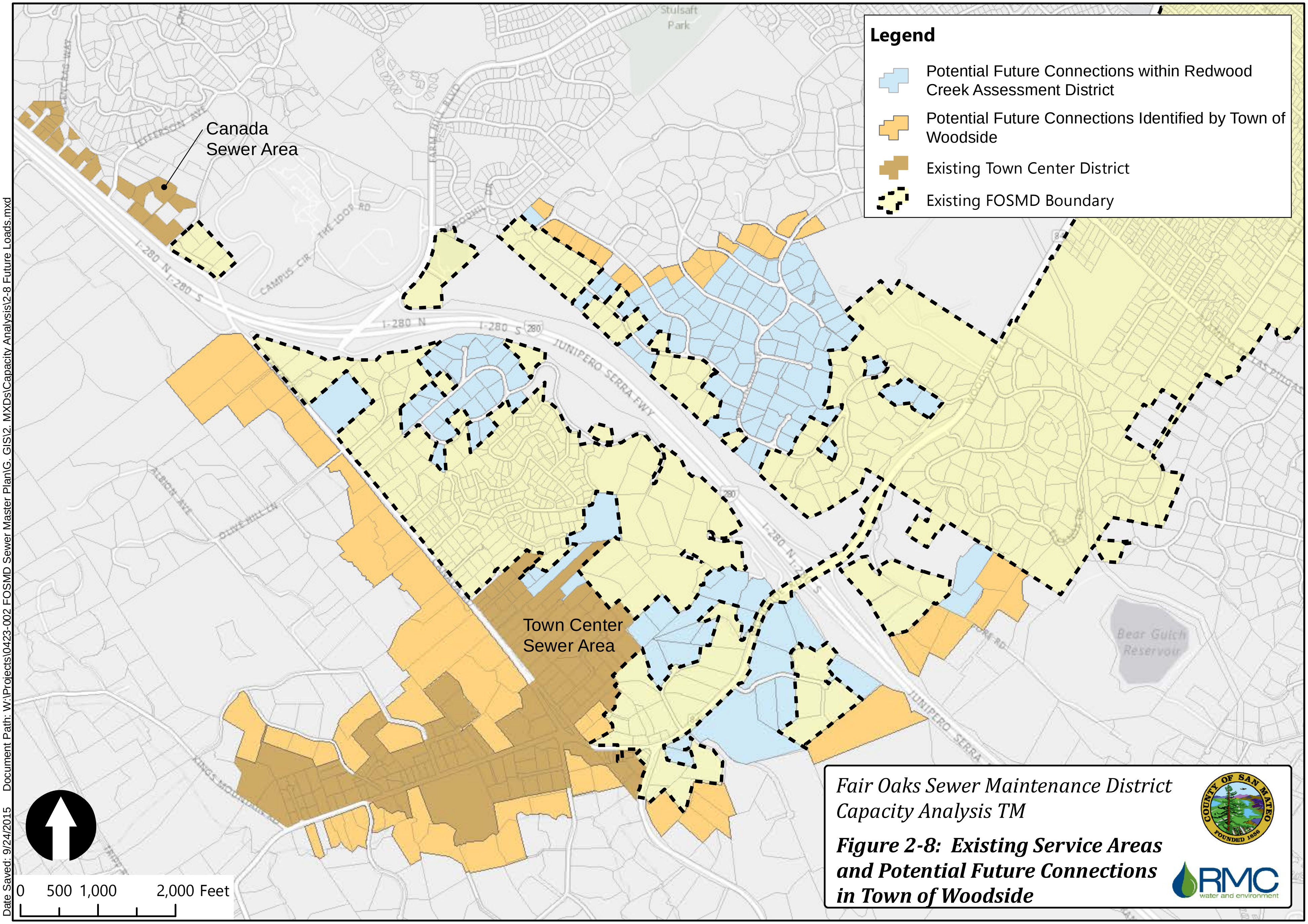

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Figure 2-8: Existing Service Areas and Potential Future Connections in Town of Woodside

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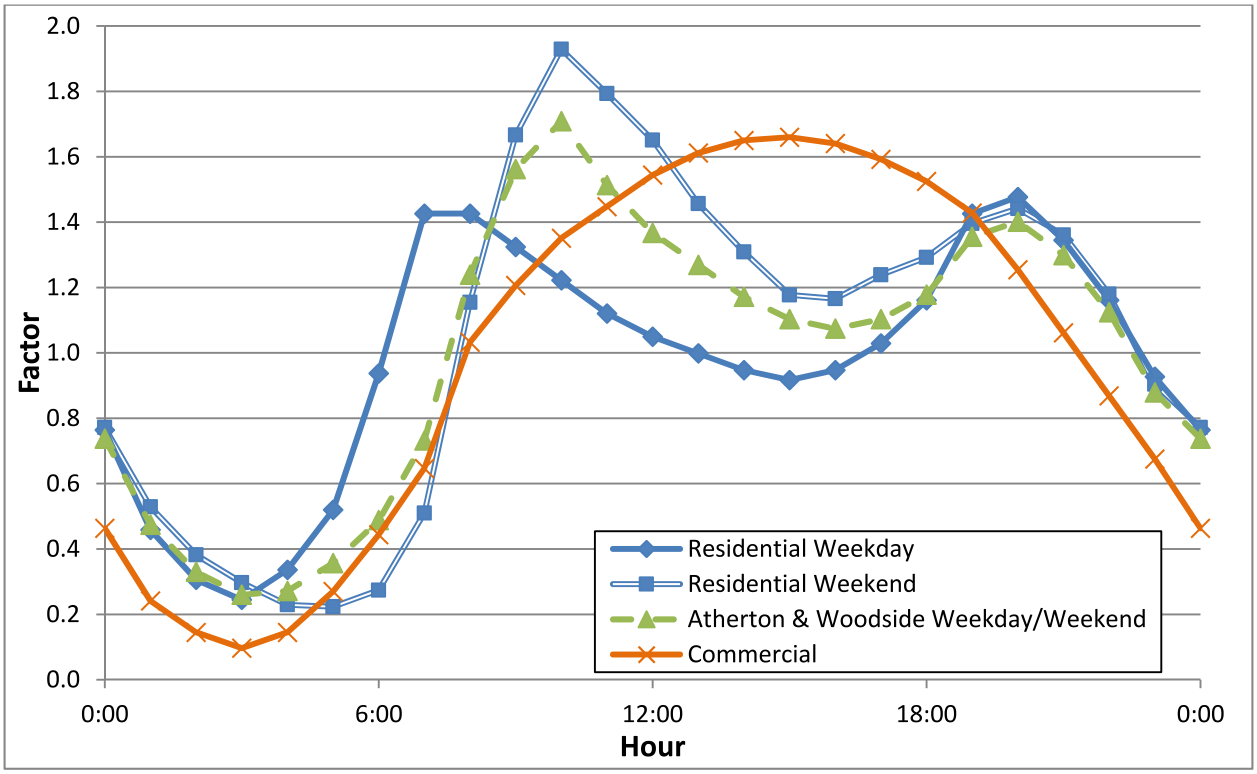
Diurnal Profiles

BWF varies throughout the day in a typical way; generally peaking early in the morning in upstream sewers, and later and less sharply in larger downstream sewers. Typical hourly peaks from small residential areas tend to be about twice the average flow, whereas peak flows further downstream may be less than 1.5 times average flows due to flow attenuation in the collection system. 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 FOSMD, 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 smaller, primarily residential meter areas, and the non-residential profile is based on typical non-residential flow profiles for similar areas. During calibration, it was noted that residential flows in the Atherton and Woodside service areas exhibited a slightly different diurnal pattern. An alternate residential pattern (same for weekday and weekend) was developed for these areas. The diurnal profiles used in the model are shown in **Figure 2-9**.

Figure 2-9: Diurnal Profiles



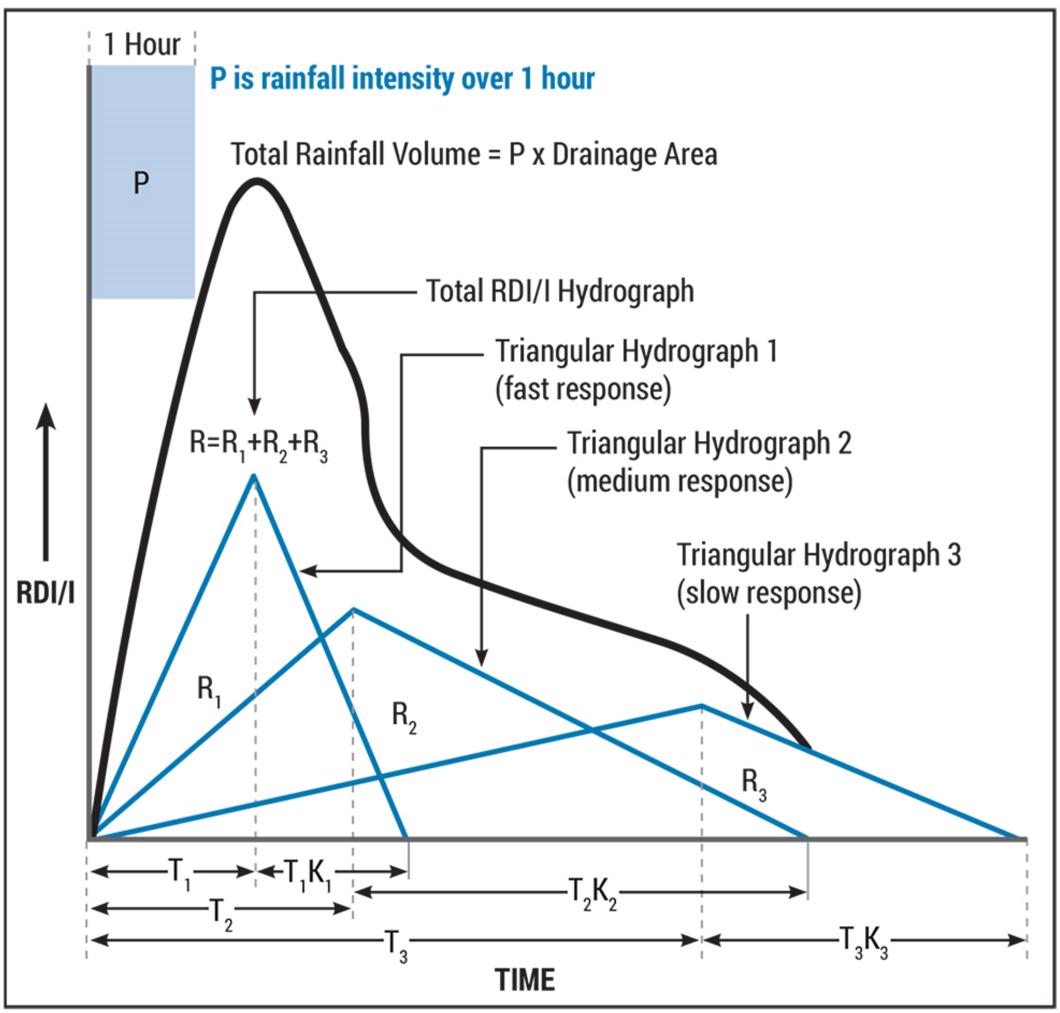
2.4.3 Groundwater Infiltration

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 flows are defined by the magnitude, shape, and timing of the RDI/I response. RDI/I varies depending on many factors, including the magnitude and intensity of the storm event, area topography, type of soil, and the condition of the sewers, manholes, and sewer service laterals. In a dynamic model, RDI/I is typically computed as a percentage of the rainfall (sometimes referred to as the "R value") falling on the contributing area of a subcatchment for each of three or more hydrograph components, representing different response times to rainfall, e.g., fast, medium, and slow, as illustrated in **Figure 2-10**. (The contributing area is assumed to be the sum of the area of all developed parcels, except for large open areas such as parks and parking lots.) Summing all of the component hydrographs for the entire duration of the rainfall event results in the total RDI/I hydrograph for the event for that subcatchment. Note that although the "slow" RDI/I component can contribute significantly to the total RDI/I volume, the "fast" component has the biggest impact on the magnitude of the peak wet weather flow.

Figure 2-10: RDI/I Hydrograph Components



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. For areas currently on septic systems, no additional RDI/I was assumed, as the areas are relatively small and are expected to have new, relatively tight sewers when they are eventually connected.

2.5 Model Calibration

2.5.1 Dry Weather Calibration

The 14-day dry period from March 10 to March 24, 2014 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 2-9** 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 only three of the flow meter areas: estimated rates of 350, 180, and 260 gpd/acre were applied in flow meter areas 50A/50B, 51, and 52A, respectively, which are the areas closest to the bay. 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 2-11** shows a plot of model versus metered dry weather flow for the total flow from FOSMD (sum of meters 50A and 50B). In this line, the green line represents the monitored (observed) flow and the red line is the model-simulated flow. As indicated in the table, 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 3 percent at the model outfall (combined Meters 50A/B).

Table 2-3: Dry Weather Flow Calibration Results

Meter	Meter Avg. Flow (mgd)	Model Avg. Flow (mgd)	Difference (mgd)	Percent Difference
50A/B	1.90	1.84	-0.059	-3.1%
51	0.346	0.403	0.057	16.5%
52	1.33	1.29	-0.038	-2.9%
52A	0.160	0.151	-0.009	-5.6%
53	0.609	0.679	0.069	11.3%
53A	0.050	0.050	0.001	2.0%
54	0.272	0.281	0.010	3.7%
55	0.158	0.156	-0.001	-0.6%
55A	0.082	0.085	0.003	3.7%
56	0.245	0.222	-0.023	-9.4%
56A	0.079	0.085	0.006	7.6%
57	0.407	0.408	0.001	0.2%
57A	0.043	0.045	0.002	4.7%

Figure 2-11: Dry Weather Calibration Graph (Meters 50A+50B)

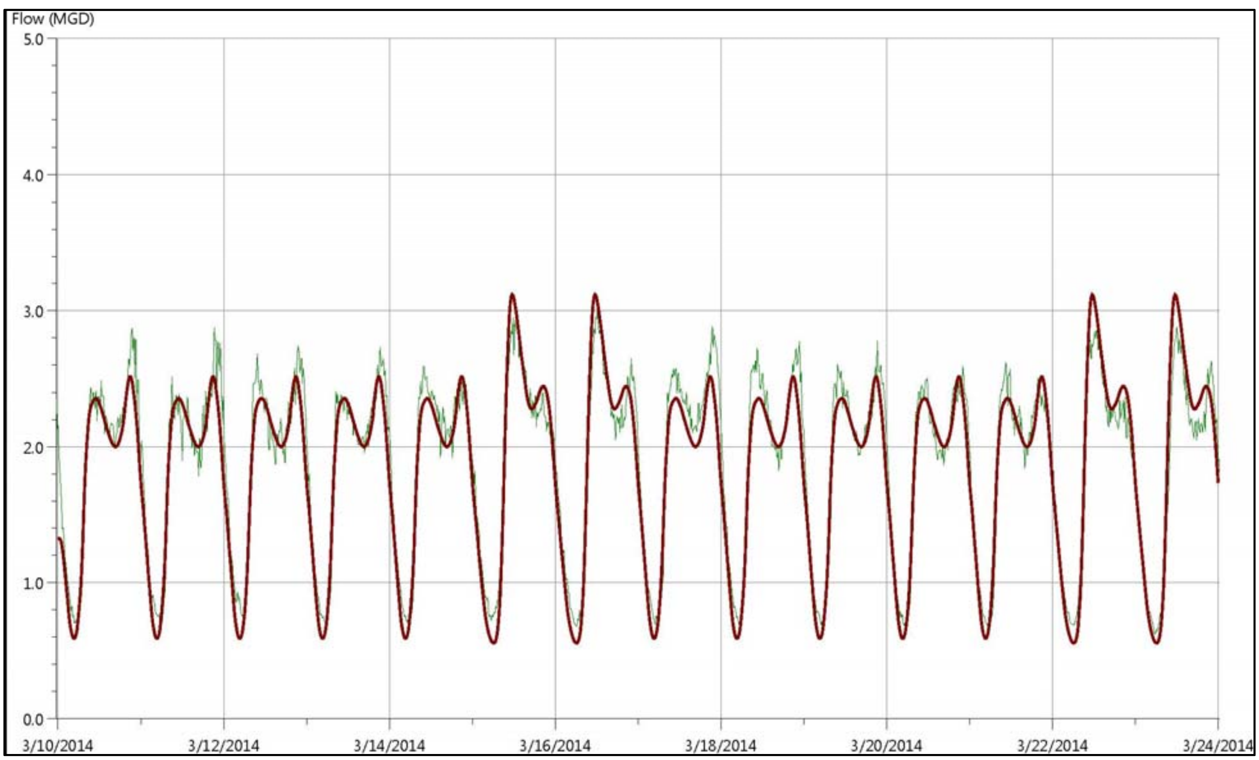


Table 2-4 summarizes the estimated dry weather flow (DWF) in the FOSMD sewer system based on the model calibration and estimated future loads described previously.

Table 2-4: Dry Weather Flow Summary

Flow Component	Flow (mgd)	
	Existing	Future
Residential BWF	1.46	1.49
Non-Residential BWF	0.29	0.29
Total Average BWF	**1.75**	**1.78**
Estimated GWI[a]	0.10	0.10
Total Average DWF	**1.85**	**1.88**

a. Calculated 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 using data from the closest of three rain gauges maintained by V&A during the monitoring period. Through the wet weather calibration process, RDI/I hydrograph parameters were developed for each metered area.

Figure 2-12 shows the plot of model versus metered wet weather flow for the total flow from FOSMD (sum of meters 50A and 50B), 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 of the meters.

Figure 2-12: Wet Weather Calibration Graph (Meters 50A+50B)

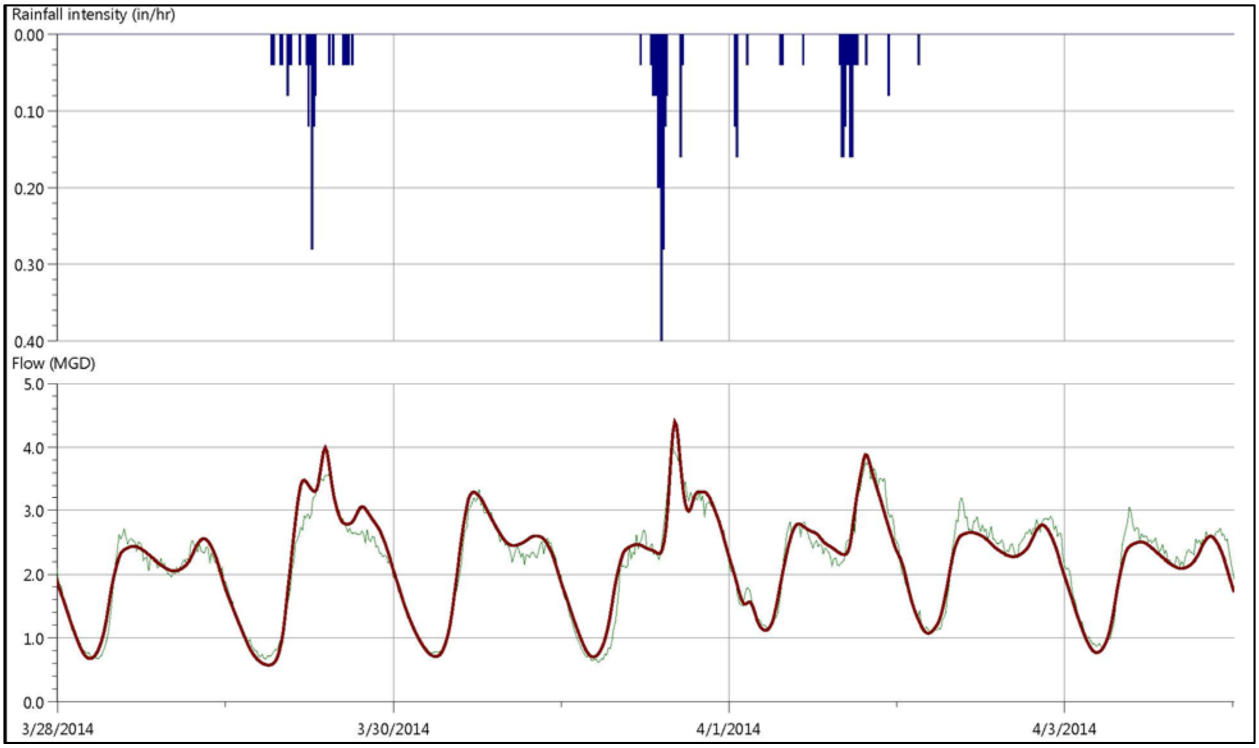


Table 2-5: Wet Weather Calibration Results

Meter Basin[a]	R1 RDI/I Vol. (%)	R2 RDI/I Vol. (%)	R3 RDI/I Vol. (%)	Total R (%)
50A/50B	1.4	1.5	3.5	6.4
51	1.4	1.5	3.5	6.4
52	0.3	0.5	0	0.8
52A	1.5	1.1	0	2.6
53	0.1	0	0	0.1
53A	0.1	0.1	0.1	0.3
54	1.5	1.3	1	3.8
55	0.1	0.1	1	1.2
55A	0.4	0.2	0.7	1.3
56	3.1	3	2	8.1
56A	0.4	0.5	0	0.9
57	0.1	0.7	1	1.8
57A	0.6	0.8	0.5	1.9

a. For meters with upstream basins, represents the incremental meter basin area, as shown on **Figure 2-3**.

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 FOSMD 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.

- A 6-hour duration, 5-year return period design event based on an actual storm that occurred on January 18, 1998. This event was used in the 2000 FOSMD Master Plan. The 2000 Master Plan describes this event as similar to a 5-year design storm in terms of intensity, duration, and volume. The 2000 Master Plan notes that minor adjustments were made to the actual storm rainfall to account for volume differences between the actual storm and the 5-year design rainfall, but does not describe those adjustments or document the 5-year design storm volume that was used. Based on figures in the 2000 Master Plan report showing the "typical" storm rainfall pattern for this event, this storm appeared to have a relatively high peak hour intensity.
- An 8-hour duration, 5-year return period "synthetic" design event developed from the Redwood City's Standard IDF (intensity-duration-frequency) curves. This design storm was used for Redwood City's 2008 Sewer Master Plan. This design storm is a "nested" event, in that it also includes the 5-year return period rainfall intensities for durations less than 8 hours.
- A 10-year, 24-hour design event developed using the SCS Type I (SCS-I) distribution (as defined in the USDA guidance document Urban Hydrology for Small Watersheds TR-55), and rainfall volumes from the NOAA rainfall atlas. This design event was used for Redwood City's 2013 Sewer Master Plan Update, based on the 10-year rainfall volume for the Redwood City station.
- A 10-year, 24-hour design event developed using the SCS Type IA (SCS-IA) distribution and rainfall volumes from the NOAA rainfall atlas. A Type IA distribution was used for the 2013 Sewer Master Plan for San Carlos based on the requirements of a Consent Decree with San Francisco Baykeeper. A 10-year, 24-hour SCS Type IA design storm is also specified in a similar Consent Decree for the County's Burlingame Hills Sewer Maintenance District.
- 5-year, 24-hour design events developed using the SCS-I or IA distributions and rainfall volumes from the NOAA rainfall atlas.

Note that the TR-55 guidelines show the areas of California where a SCS-I vs. a SCS-IA rainfall distribution is best suited. However, the San Francisco Bay Area is located approximately at the boundary of those areas, so either storm distribution may be appropriate.

Table 3-1 summarizes the total volume and peak intensity for each of these potential design events. **Figure 3-1** shows how the storm rainfall volumes compare for different storm durations (note that the design storms from the 2000 FOSMD and 2008 Redwood City Master Plans were only 6- and 8-hour duration events, respectively). **Figure 3-1** indicates that the January 18, 1998 event is nearly as intense as the SCS-I 10 year design event for very short durations, but is more similar to the 5-year SCS-I event for durations longer than 2 hours. The 5-year event developed from Redwood City's Standard IDF curves is similar in intensity to a 10-year SCS-IA storm at 1- to 4-hour durations but less intense at longer durations than the other potential design events. The SCS-IA distribution used by San Carlos is significantly less intense at durations less than 12 hours than the Type 1 distribution, although the total 24-hour volume is the same.

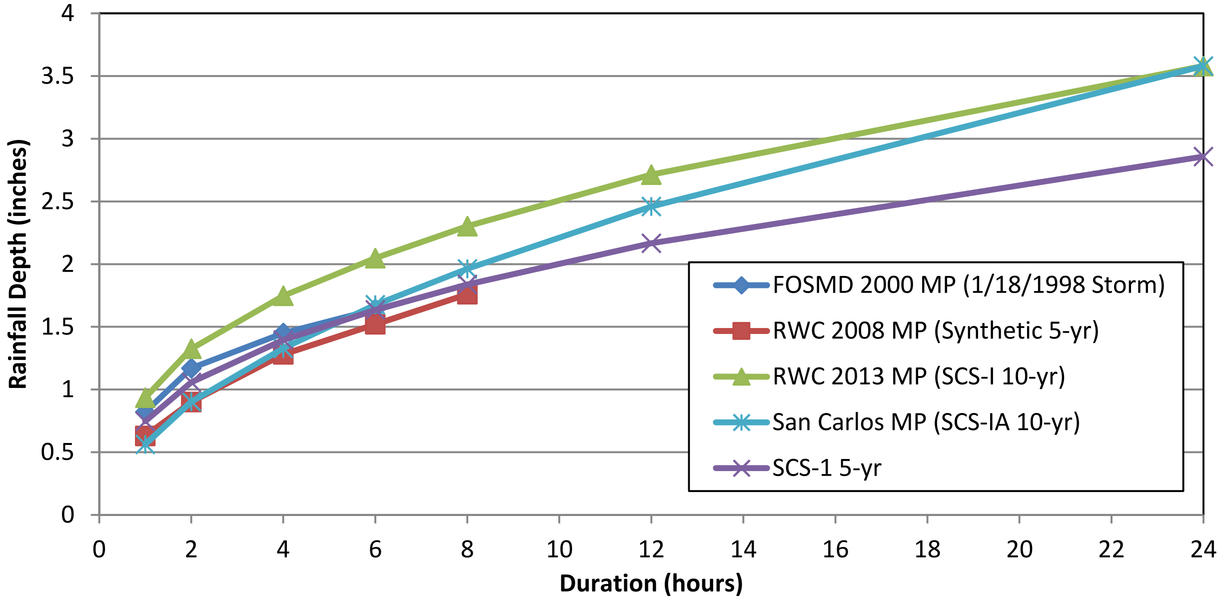
As shown in **Table 3-1** and **Figure 3-1**, the most intense periods of rainfall in all of these potential design storms occur within about a 6-hour period. Generally, travel times (time of concentration) within typical medium size sewer systems are also on the order of a few hours at most. Therefore, using a 24-hour design storm as opposed to a shorter-duration storm may have only a small impact on the peak flow in the system, but rather a greater impact on the volume of overflows, should there be any.

Table 3-1: Potential Design Storm Characteristics

Volume Source	Frequency & Distribution	Volume (in)	6 hour Volume (in)	Duration (hrs)	Peak Intensity (in/hr)
1/18/1998 Rainfall Data[a]		1.64	1.64	6	0.82
5-yr Redwood City Standard IDF Curves[b]		1.80	1.52	8	0.63
NOAA Redwood City Station	10-yr/24-hr SCS-I[c]	3.58	2.04	24	0.93
NOAA Redwood City Station	10-yr/24-hr SCS-IA[d]	3.58	1.67	24	0.56
NOAA Redwood City Station	5-yr/24-hr SCS-I	3.02	1.63	24	0.75

a. Design storm used for FOSMD 2000 Master Plan (adjusted to 6-hour, 5-year return period volume).
b. Design storm used in Redwood City's 2008 Sewer Master Plan.
c. Design storm used in Redwood City's 2013 Sewer Master Plan Update.
d. Design storm frequency and distribution used for 2013 San Carlos Master Plan.

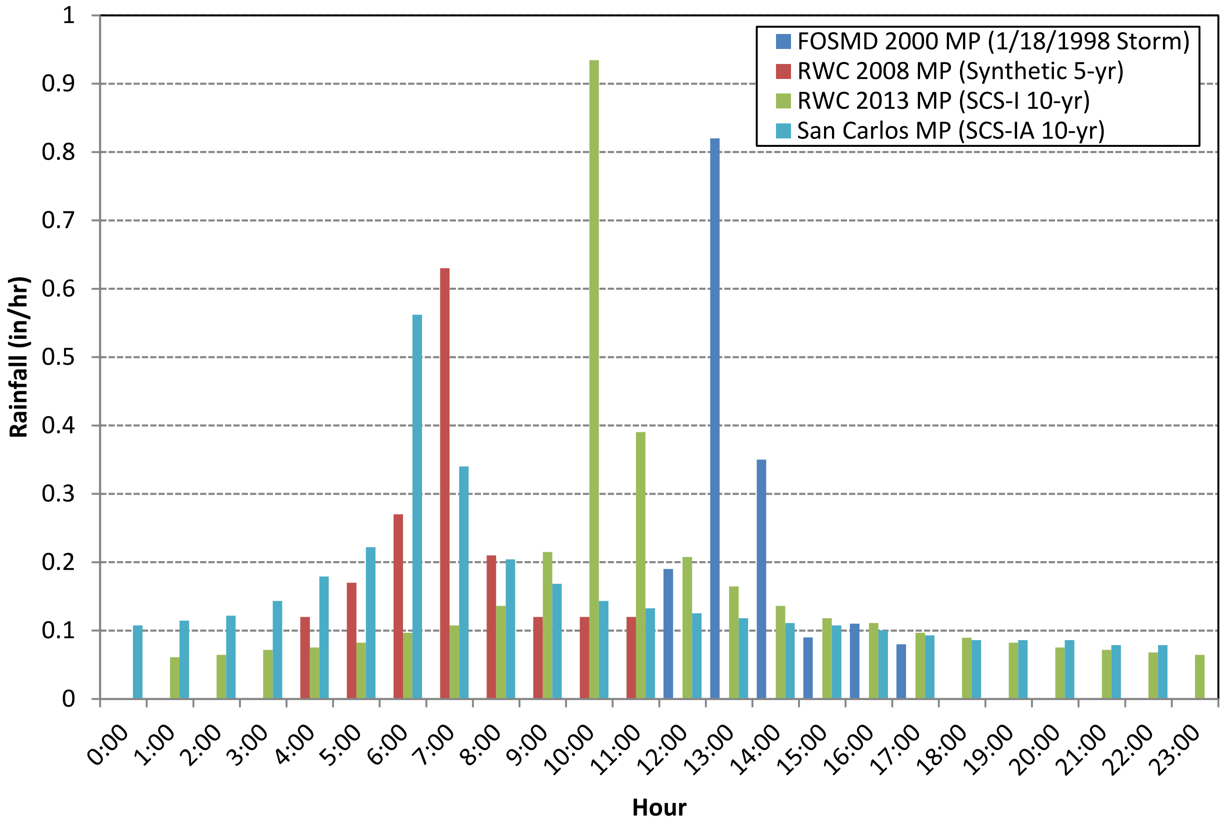
Figure 3-1: Comparison of Potential Design Storms



The timing of the design storm also affects the resulting peak wastewater flows. If the design storm is timed to cause peak RDI/I at roughly the same time as peak BWF ("peak-on-peak"), the total peak wet weather flow will be higher than if the peak RDI/I generated by the design storm occurs 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 3-2 shows the rainfall distributions used for the 2008 FOSMD, 2008 Redwood City Master Plan, the 2013 Redwood City Master Plan Update and the 2013 San Carlos Master Plan over a 24-hour period. As shown in the figure, the peak rainfall for the 2000 FOSMD Master Plan design storm (based on the January 18, 1998 event) occurs several hours after the diurnal peak flow (although it is possible that the storm was "shuffled" to coincide with the peak diurnal flow by the HYDRA model used for that Master Plan). The peak rainfall for the 10-year SCS-I storm used for the 2013 Redwood City Master Plan Update was set at about 10 a.m., also later than the diurnal peak. However, for the 2008 Redwood City Master Plan, the rainfall was timed to generate close to a peak-on-peak flow. The San Carlos Master Plan also timed the peak rainfall to occur at or near the time of peak BWF flow. If using a synthetic IDF-derived or SCS storm distribution, peak rainfall can be timed to occur at any time of day, depending on the desired level of conservatism.

Figure 3-2: Design Storm Rainfall Timing



For consistency with Redwood City's 2013 Sewer Master Plan Update, the 10-year SCS Type I design rainfall event and rainfall timing was selected for this FOSMD Sewer Master Plan. 10-year event rainfall volumes were based on NOAA estimates at the rain gage locations used for the 2014 FOSMD flow monitoring program, to incorporate variations in design storm intensity across the service area, as shown in **Table 3-2** below.

Table 3-2: NOAA 10-year 24-hour Rainfall Volume at FOSMD Rain Gage Sites

Location	Rainfall Volume (in)
RG1 (lower basin)	3.11
RG2 (middle basin)	3.58
RG3 (upper basin)	4.97

Based on this design storm, peak wet weather flow (PWWF) from the FOSMD system, once all capacity deficiencies are relieved, is estimated to be approximately 10.3 mgd. For comparison, Redwood City's 2013 Sewer Master Plan Update predicted a future PWWF of 9.6 mgd from the FOSMD system.

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.

The 2000 FOSMD Sewer Master Plan did not specifically allow for any surcharging. According to the report:

> *Modeled flow is compared to the theoretical capacity of each pipe segment…If capacity deficiencies were detected, (HYDRA) was used to size the appropriate relief and/or replacement sewer size.*

In comparison, the Redwood City’s 2013 Master Plan, as well as well as master plans for other nearby communities such as San Carlos, allow some surcharging, up to within 4 to 5 feet of the manhole rim. Exceptions to the 5 foot limit can also be allowed for limited surcharging on 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, and many of FOSMD’s larger diameter sewers are relatively deep, a criterion similar to Redwood City’s was applied, with surcharging up to 5 feet of the manhole rims considered acceptable under 10-year design storm PWWF.

3.2 Capacity Analysis Results

The calibrated model was run for existing and future conditions to identify areas of the system that fail to meet the specified performance criteria under design storm PWWF.

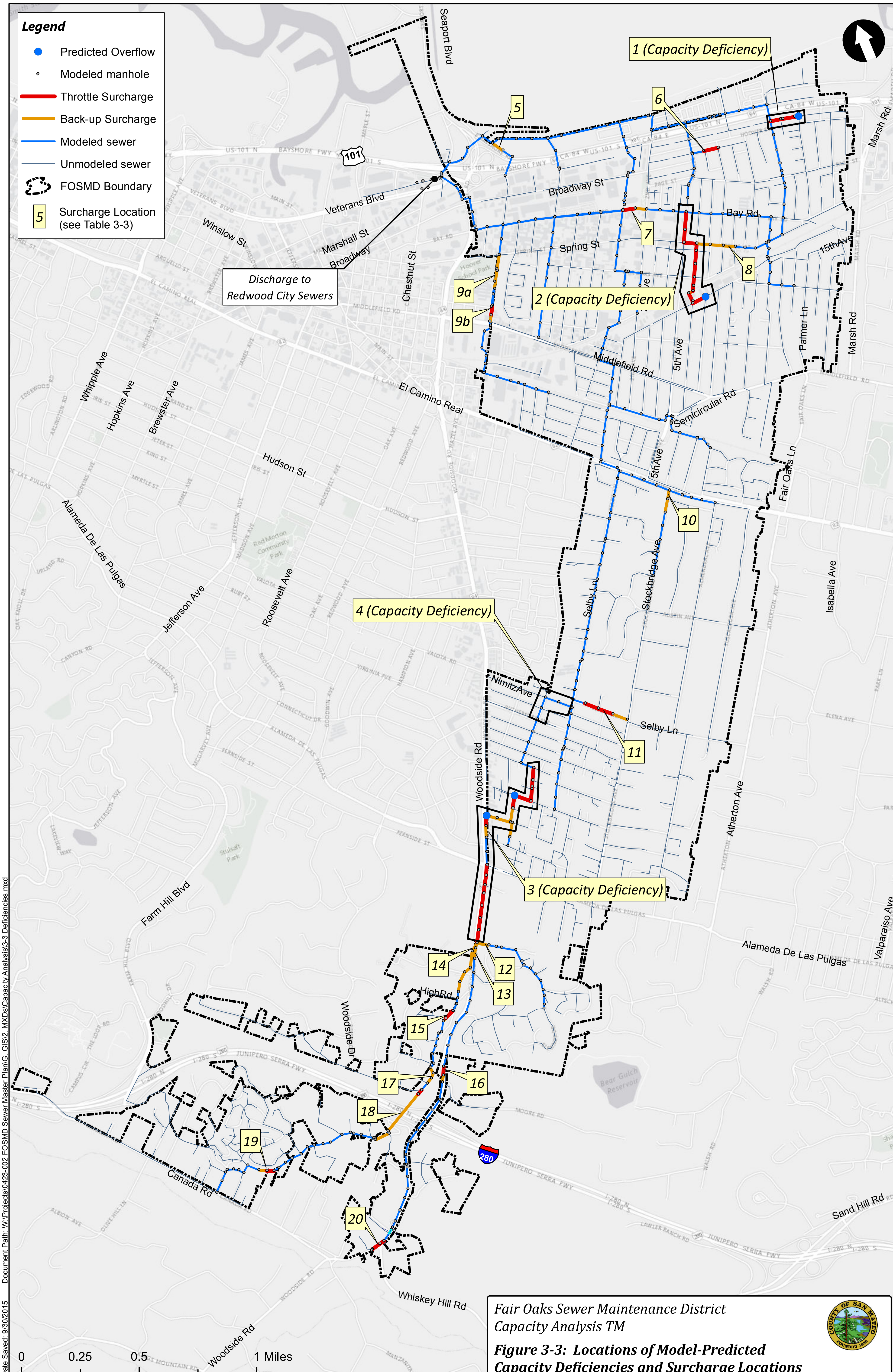
3.2.1 Sewer System Deficiencies

No capacity deficiencies in the system were identified for dry weather conditions. The location of model-predicted surcharged sewers and potential overflows during future design storm PWWF conditions are summarized in **Table 3-3** and shown in **Figure 3-3**. The figure indicates four locations that exceed (violate) the District’s capacity criteria. Hydraulic profiles of the locations indicated are included in **Appendix C**. Note that Location 4 exceeds criteria only after the capacity deficiency at Location 3 is relieved. Locations 5 through 20 do not exceed District capacity criteria and no projects are recommended for these locations.

In the figure, pipes shown in red are predicted to surcharge due to “throttle” conditions, indicating that the full pipe capacity of the pipe is less than the predicted peak flow. Pipes shown in orange are predicted to surcharge due to backwater from a downstream throttle condition. The locations of model-predicted overflows during the design storm are shown as blue circles in the figure. It should be noted that the location of model-predicted overflows may not reflect the actual 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.

Table 3-3: Locations of Model-Predicted Surcharge and Potential Capacity Deficiencies

Location	US MH	DS MH	Category (Throttle or Backup Surcharge)	Recommendation	Comments
1	3959	3931	Throttle	Capacity Project 1	Exceeds criteria
2	4235	4067	Throttle	Capacity Project 2	Exceeds criteria
3	4949	4825	Throttle	Capacity Project 3	Exceeds criteria
4	4823	5351	Throttle	Capacity Project 4	Exceeds criteria when Location 3 deficiency is relieved (backup surcharge from MH 4641 to 4823)
5	3635	3633	Backup	No changes	Backup surcharge at connection to trunk (Freeboard > 10 feet)
6	3889	3960	Throttle	No changes	Surcharge is < 0.2 feet due to flat pipe segment
7	3991	3976	Throttle	No changes	Freeboard is > 5 feet Surcharge is < 1 foot
8	4221	4097	Backup	No changes	Backup surcharge due to deficiency at Location 2
9A	3642	3720	Backup	No changes	Parallel line is higher and not surcharged
9B	3652	3648	Throttle	No changes	
10	5396	5305	Backup	No changes	Backup surcharge due to reverse slope pipe segment between MH 5304 and MH 5305 (Freeboard > 5 feet)
11	5613	5359	Throttle	No changes	Freeboard is > 6 feet Surcharge is < 1 foot
12	4951	4949	Backup	No changes	Backup surcharge due to deficiency at location 3
13	4955	4949	Backup	No changes	Backup surcharge due to deficiency at location 3
14	4954	4949	Backup	No changes	Backup surcharge due to deficiency at location 3
15	4975	4978	Throttle	No changes	Freeboard is > 10 feet Surcharge is < 1 foot
16	5004	4995	Throttle	No changes	Freeboard is > 6 feet Surcharge is < 1 foot
17	6158	4994	Throttle	No changes	Freeboard is > 6 feet Surcharge is < 1 foot
18	5751	4797	Throttle	No changes	Freeboard is > 6 feet Surcharge is < 1 foot
19	5746	5755	Throttle	No changes	Freeboard is > 10 feet Surcharge is < 1 foot
20	5810	5808	Throttle	No changes	Freeboard is > 7 feet Surcharge is < 1 foot



Fair Oaks Sewer Maintenance District
Capacity Analysis TM

Figure 3-3: Locations of Model-Predicted Capacity Deficiencies and Surcharge Locations

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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 4). These effects were considered in developing the capacity improvement projects described below.

3.2.2 Capacity Improvement Projects

This section describes the sewer improvement projects that would be needed to reduce the risk of overflows in the collection system due to insufficient capacity for design peak wet weather flows. These improvement projects have been developed to address areas in which predicted peak flows would exceed the District's capacity deficiency criteria. For each identified gravity sewer capacity deficiency, a project was developed to replace the existing pipe with a larger pipe or, alternatively in some cases, install a new pipeline in a different alignment. None of the predicted capacity deficiencies were located near existing sewers with available capacity; therefore, diversion to another existing sewer was not feasible.

The assumptions that were used to define the projects are discussed below. **Table 3-4** summarizes the recommended projects, and **Figure 3-4** shows the locations the projects. Detailed maps and project information sheets that provide project details, key considerations, and planning-level construction and capital cost estimates are included in **Appendix D**.

Project Sizing Criteria

For gravity sewer capacity improvement projects identified as part of this Master Plan, replacement or new pipes were sized to convey the future design storm PWWF with no surcharge. Existing pipe slopes and depths were preserved when upsizing sewers in-place. Model runs with all capacity projects in place were made to determine the impact of increased capacity from upstream projects on peak flows in pipes downstream of those projects to verify that no additional collection system capacity deficiencies would result.

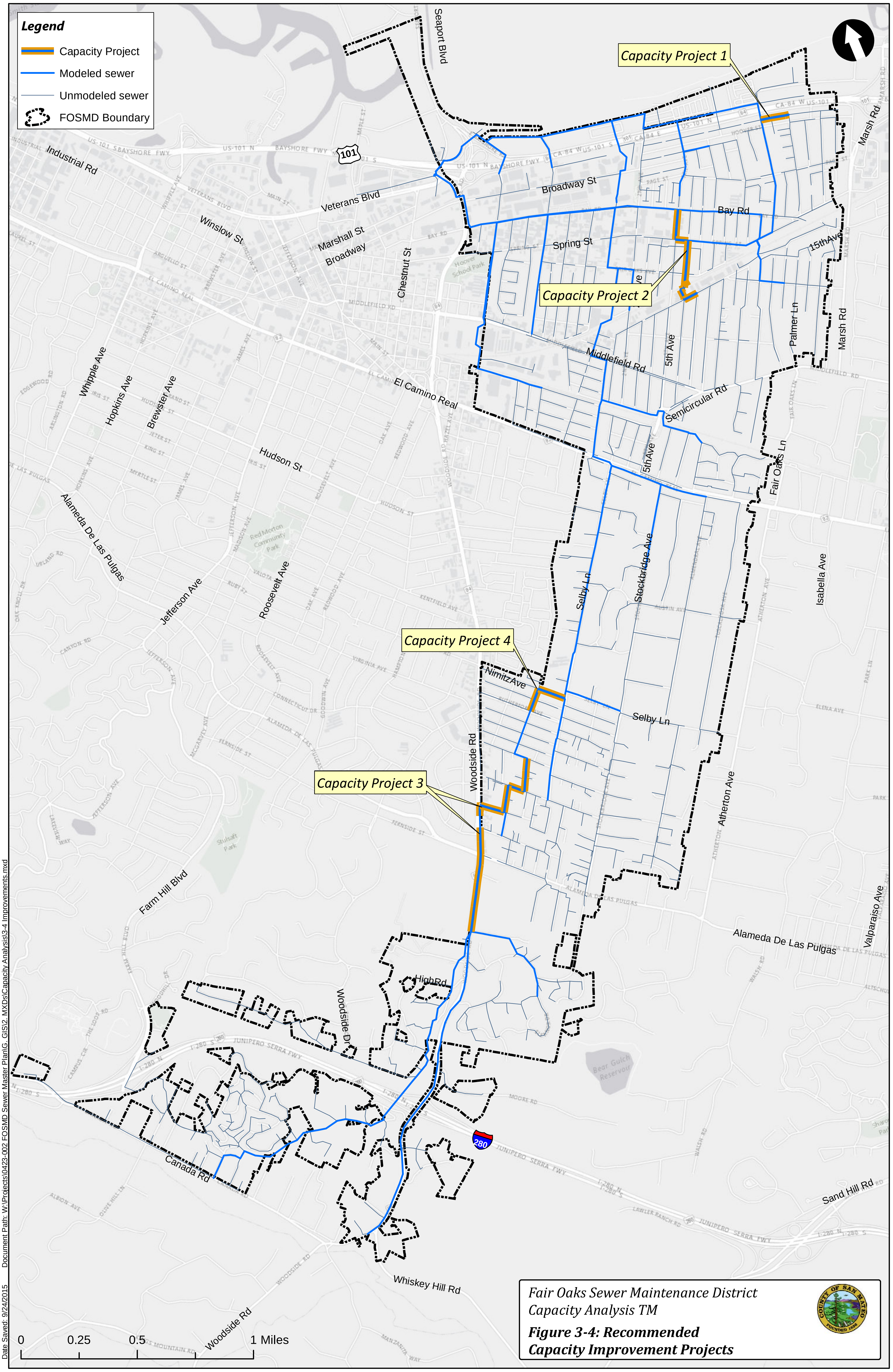
Cost Criteria

Costs for capacity improvement projects were estimated based on RMC's experience with similar projects and recent project bids provided by the County. These cost estimates are planning or conceptual level estimates, and are considered to have an estimated accuracy range of -30 to +50 percent. This level of accuracy corresponds to an "order of magnitude" or "Class 5" cost estimate as defined by the American Association of Cost Estimators. These estimates are suitable for use for budget forecasting, CIP development, and project evaluations, with the understanding that refinements to the project details and costs would be necessary as projects proceed into the design and construction phases. All costs have been adjusted to an Engineering News Record Construction Cost Index (ENR CCI) of approximately 11,178, which represents the February 2015 ENR CCI for the San Francisco Area.

Cost criteria include baseline unit construction costs for gravity sewers using open-cut and trenchless (e.g., pipe bursting) methods. Pipe bursting is assumed for most projects that involve upsizing existing sewers to 15-inch diameter or smaller; construction of new sewers or pipes larger than 15 inches assumes open cut construction, except where trenchless construction would be required for major crossings (e.g., railroad crossings). Costs for gravity trunk sewers vary with pipe diameter and depth (in the case of open-cut construction), and include lateral reconnections and insertion trenches (in the case of pipe bursting). Allowances added to the baseline construction cost include mobilization/demobilization and project-specific costs for bypass pumping for pipe bursting and remove and replace construction and traffic control for work in roadways. A 30 percent allowance for contingencies for unknown conditions was also included for all projects, as well as an allowance of 25 percent of construction cost for engineering, administration, and legal costs.

Table 3-4: Recommended Capacity Improvement Projects

Project No.	Project Name	U/S MHID	D/S MHID	Description	Estimated Capital Cost
1	Hoover Street Easement	3959	3931	Replace 645 feet of 6" sewer with 10" pipe in an alley easement between Hoover Street and Rollison Road, east of Haven Avenue.	$ 274,000
2	Edison Way to Bay Road	4235	4067	Replace approximately 2,000 feet of 8" and 10" pipe with 12" and 15" pipe from Edison Way and 7th Avenue along 6th Avenue to Bay Road and 5th Avenue. Install approximately 360 feet of new 12" pipe from Edison Way to Fair Oaks Avenue, including new railroad crossing.	$ 1,270,000
3	Woodside Road to Sequoia Avenue	4949	4825	Replace approximately 4,900 feet of 10" pipe with 12" and 15 inch pipe from Woodside Road near Churchill Avenue to Sequoia Avenue and Milton Street (along Hull Avenue, Santa Clara Avenue and Milton Street).	$ 1,891,000
4	Himmel Avenue	4641	5353	Replace 1,200 feet of 15" pipe with 18" pipe in Himmel Avenue from Rutherford Avenue to Nimitz Avenue and in Nimitz Avenue to Shelby Lane.	$ 626,000
				Total	**$ 4,061,000**



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4 Infiltration and Inflow Analysis

RDI/I was analyzed for each flow meter area based on modeled flows generated for the design storm. Refer to Section 2 for a discussion of the flow meter program and meter locations. Note that some of these areas were metered directly (no upstream flow meter) and others represent the "incremental" area between upstream and downstream meters.

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. These approaches are discussed in more detail below. The RDI/I response in each meter basin is summarized in **Table 4-1.**

Wet Weather Peaking Factor

The wet weather peaking factor is the ratio of PWWF to ADWF. The peaking factor provides a good intuitive sense of the significance of RDI/I at a location in the system. Peaking factors as recorded at flow meters are not necessarily a good way to identify which areas are contributing most significantly to RDI/I. The peaking factor at downstream meters depends on flow from its entire tributary meter – that is, if an upstream area has very high RDI/I, but a downstream portion of the meter tributary area (the incremental area) does not, the peaking factor at the downstream meter may still be high because of the influence of the contributing upstream area on the flows. Furthermore, attenuation can also cause dampening of peak flows as flow travels downstream.

As described in Section 2, flow loads to the hydraulic model were developed for each sewer subcatchment. For each subcatchment, the model includes the estimates of average BWF and GWI, as well as the RDI/I hydrograph components (see Figure 2-10) that characterize the subcatchment response to rainfall. Based on these components, it is possible to calculate the estimated PWWF generated by each subcatchment and calculate a total PWWF for each individual flow meter area (including incremental areas) by summing the values for the subcatchments that comprise that area. It is important to note that this may be a conservative estimate, as the sum of the subcatchment flows does not reflect the routing of flow hydrographs in the system, but is still a useful and reasonable way of characterizing the relative peak flow contributions from various areas of the system.

To compute a wet weather peaking factor, the flow meter area PWWF is divided by the ADWF (sum of ADWF for its associated subcatchments).

Figure 4-1 shows the range of wet weather peaking factors for the flow meter areas, based on the modeled design storm event, which range from under 4 to over 12. Note that a high peaking factor may also be a reflection of a low BWF, for example, areas of low density development may have lower BWF than similar size areas of higher density development, whereas their RDI/I contributions may be the same, resulting in a higher computed wet weather peaking factor for the low density area. Therefore, peaking factors should only be used as a general indicator of peak RDI/I in the system.

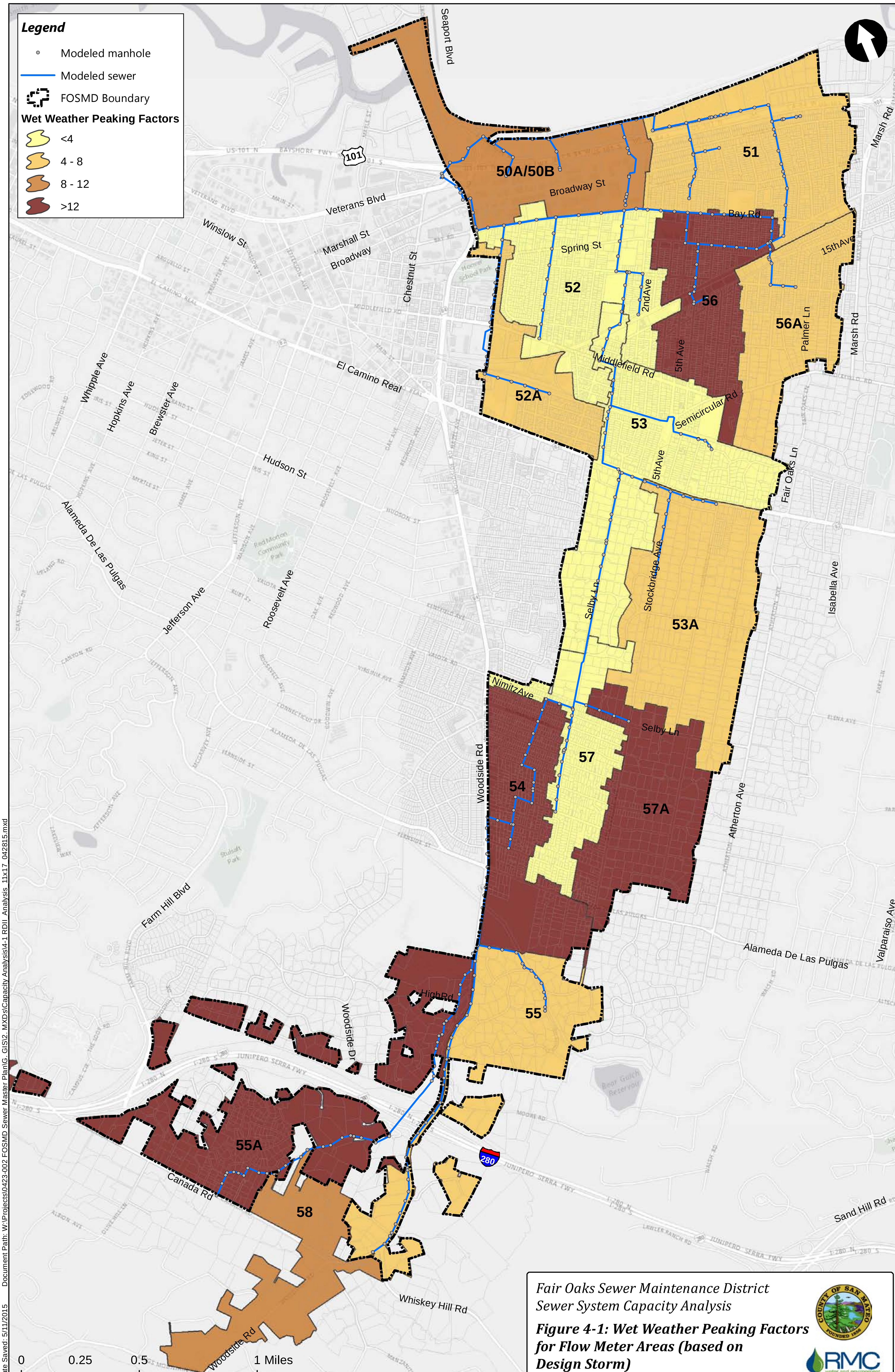
R Value

As discussed in Chapter 4, the R value is the percentage of rainfall volume entering the system as I/I. As shown in Figure 2-10, the total RDI/I volume can be characterized by three components representing different response times to rainfall. The first RDI/I component, R1, characterizes the most rapid response and therefore has the greatest impact on the peak RDI/I flow. However, the R1 value is a measure of volume rather than flow rate, so it does not specifically equate to a peak flow. Furthermore, the magnitude of the R values is dependent on the estimated sewered or contributing area of the subcatchment. Areas of the system with larger lots may have larger contributing areas, but not necessarily in proportion to the amount of RDI/I generated since the density of sewers may be lower.

Table 4-1: Peak I/I by Flow Meter Area

Flow Meter Basin[a]	Contributing Area (ac)[b]	ADWF (mgd)[c]	Peak RDI/I (mgd)[d]	PWWF (mgd)[e]	Unit Peak RDI/I Rate (gpd/ac)[f]	Unit Peak RDI/I Rate (gpd/ft)[g]	Wet Weather Peaking Factor[h]
50A/50B	115	0.11	0.84	1.04	7,300	43	9.4
51	221	0.35	1.61	2.22	7,300	34	6.4
52	183	0.24	0.30	0.70	1,600	6	2.9
52A	77	0.14	0.58	0.81	7,500	28	5.9
53	251	0.21	0.12	0.47	500	2	2.2
53A	276	0.05	0.30	0.37	1,100	10	7.4
54	172	0.13	1.49	1.69	8,600	47	13.3
55	197	0.04	0.16	0.23	800	6	5.1
55A	301	0.08	0.92	1.04	3,100	16	12.2
56	130	0.16	2.04	2.30	15,700	62	14.0
56A	146	0.08	0.31	0.45	2,100	11	5.3
57	124	0.08	0.17	0.27	1,400	7	3.4
57A	193	0.04	0.71	0.76	3,700	29	17.7
58[i]	142	0.02	0.23	0.26	1,600	15	11.9
Total	2,528	1.75	9.76	12.61[j]	3,900	22	7.2

a. For meters with upstream basins, represents the incremental meter basin area, as shown on Figure 2-3.
b. Net area of developed parcels.
c. Average dry weather flow. Includes groundwater infiltration during non-rainfall periods, representing approximately 6 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.
i. Basin includes Town of Woodside sewers (tributary to Town Center Pump Station) only.
j. Sum of basin flows; does not reflect flow routing through system. Total estimated PWWF discharged to Redwood City without sewer system capacity improvements is 9.2 mgd; with capacity improvements, estimated PWWF is 10.3 mgd (future connections do not significantly affect PWWF).



Fair Oaks Sewer Maintenance District
Sewer System Capacity Analysis

Figure 4-1: Wet Weather Peaking Factors for Flow Meter Areas (based on Design Storm)

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Peak RDI/I per Unit Area or Length of Pipe

The peak RDI/I flow generated per unit area, e.g., gallons per day (gpd) per acre, or length of pipe (e.g., gpd/foot) provides a measure of which areas of the system contribute the highest peak RDI/I flows on a unit basis. The peak RDI/I per unit area or length of pipe is a better indicator than the peaking factor or R value of where RDI/I reduction efforts could potentially be most successful in reducing peak flows and alleviating downstream capacity deficiencies. This is because the peak RDI/I per unit area or length of pipe is not affected by the magnitude of base wastewater flows, and indicates the peak flow rate rather than RDI/I volume. Peak RDI/I per length of pipe is considered a better representation of the relative contribution of peak RDI/I rather than peak RDI/I per area for the reasons discussed above with respect to differing contributing areas and their impact on R values.

Figure 4-2 shows the peak RDI/I flows generated in the model for the design storm for each flow meter area, in gpd per foot of sewer pipe (based on mainline footage, not including laterals). Note that an alternate method of expressing length of pipe is sometimes used for these calculations that also incorporates the pipe diameter (e.g., gpd/inch-diameter-mile), assuming that larger diameter pipes contribute more I/I due to larger pipe wall surface and joint area. However, since the majority of the sewers in the system are small diameter 6- and 8-inch pipes, normalizing based on the total pipe footage can be used for representing relative unit RDI/I contributions.

4.1.1 I/I Source Detection and Control Methods

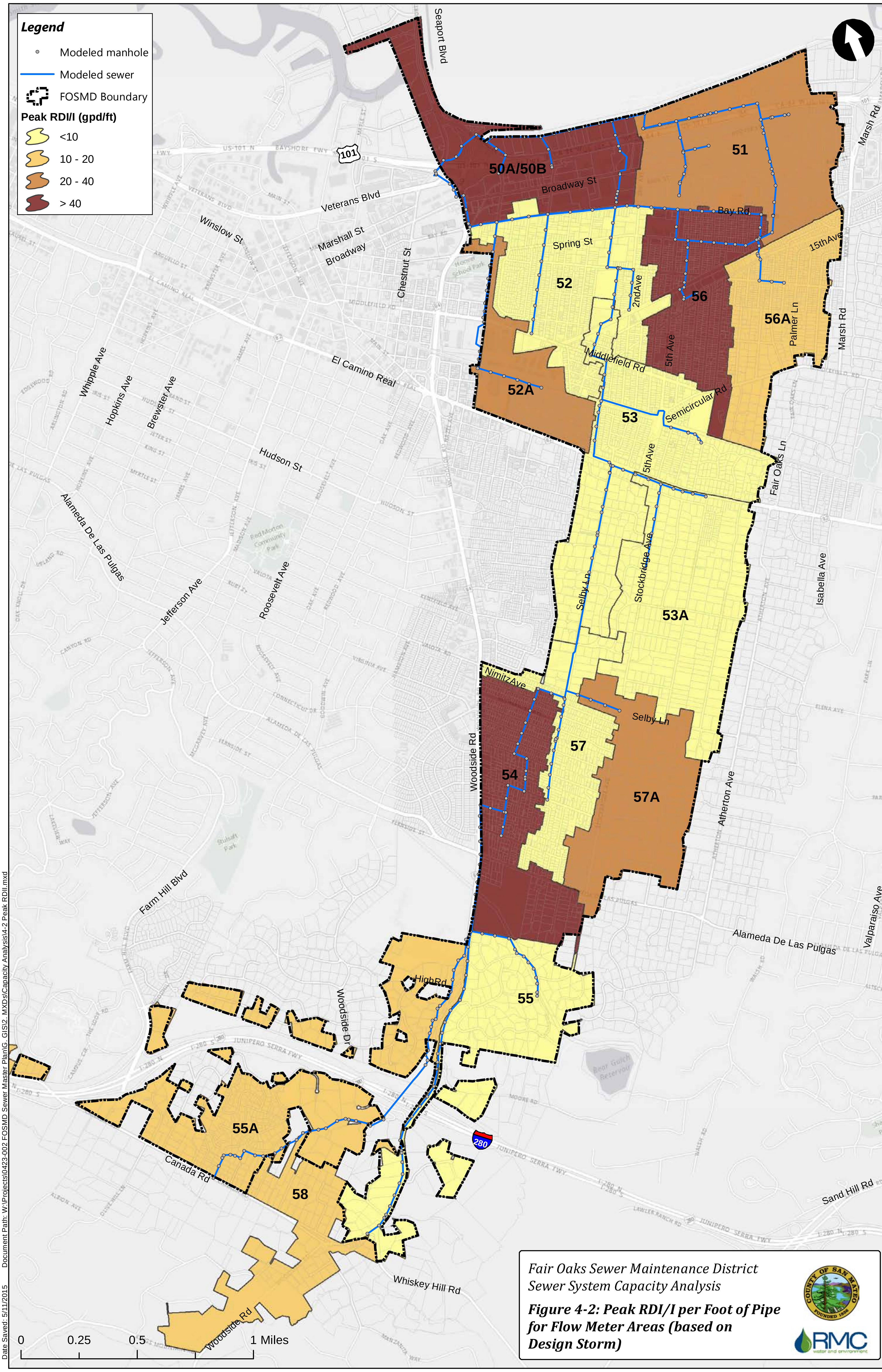
A necessary step in identifying potential I/I control measures is a realistic assessment of the actual sources of I/I in the sewer system. Based on the pattern and magnitude of flows in the District's sewer system, the likely sources of RDI/I flows are defects in sewers and service laterals, and possibly some direct connections (e.g., illegally connected roof and area drains, direct connections from the storm drain system, etc.). Appropriate I/I control methods depend on the type and sources of I/I. Control methods must include detection as well as correction. Potential methods are described in the following paragraphs.

Direct Inflow Sources

Direct inflow sources can contribute significantly to both volume and peak rates of I/I, and have the greatest probability of being cost effective to eliminate. The main methods used to detect and locate direct inflow sources are smoke and dye testing (dye testing is used primarily as a confirmatory test). Smoke testing is considered to be a relatively easy and inexpensive method (cost is approximately $0.50 to $0.60 per foot if a substantial length of pipe is tested), and discovery of just a few direct storm drain cross-connections, for example, can make the effort worthwhile. However, unless there is some indication or knowledge of the existence of direct connections in the system, finding them may require an extensive smoke testing program, which requires public notification measures and access onto private property to document the smoke returns. For this reason, smoke testing is generally targeted at specific areas with high peak RDI/I rates.

Generally the most numerous type of sources found during smoke testing are not direct inflow connections but defects in shallow pipes, primarily laterals. Rehabilitation of laterals may be a challenging institutional issue (see discussion below on correction of private property I/I sources).

Manholes subject to ponding or located in drainage courses may also be sources of direct inflow. The amount of I/I depends on the manhole location, type of manhole cover (number and size of holes), and the condition of the cover and frame. Physical inspection of manholes is the most effective way to identify such conditions, and correction is relatively straightforward (replace cover, realign frame, raise manhole to grade, remove or relocate manhole in watercourse, etc.). Physical inspection can be conducted in conjunction with sewer inspection or routine cleaning work, or as a separate activity.



Fair Oaks Sewer Maintenance District
Sewer System Capacity Analysis

Figure 4-2: Peak RDI/I per Foot of Pipe for Flow Meter Areas (based on Design Storm)

Document Path: W:\Projects\0423-002 FOSMD Sewer Master Plan\G. GIS\2. MXDs\Capacity Analysis\4-2 Peak RDII.mxd

Date Saved: 5/11/2015

Service Layer Credits: Esri, HERE, DeLorme, MapmyIndia, © OpenStreetMap contributors, and the GIS user community

Elimination of direct inflow connections requires disconnection of the source and re-direction of the drainage to an appropriate location. This may simply be to the ground surface (as in the case of roof drains), or connection to a nearby storm drain or street gutter. In general, each identified source needs to be evaluated on a case-by-case basis to identify the appropriate corrective measure.

Infiltration Sources in Sewer Mains and Manholes

Infiltration sources are defects in sewer pipes or manholes caused by defective materials or construction, general deterioration, or damage caused by physical conditions such as ground movement or settlement, traffic loads, or root intrusion. Infiltration sources (defects) are detected by inspection: visual inspection in the case of manholes and CCTV inspection for sewer mains. However, visual observation of active I/I is generally not feasible since the RDI/I generally occurs for only short periods during rainfall events, and the pipes may fill up during those periods, making CCTV inspection difficult or impossible.

Infiltration correction methods involve rehabilitation or replacement of entire pipe segments or manholes or spot repair of localized defects. There are numerous materials and methods used for this type of rehabilitation. In general, however, the cost per unit amount of I/I removed is relatively high, since the defects individually contribute relatively small amounts of flow. It is recognized that infiltration in the sewer system will "migrate" to other nearby defects that are left un-repaired. Therefore, a fairly extensive area of the system may need to be included in the rehabilitation effort in order to achieve substantial flow reduction. Furthermore, reductions greater than about 30 percent can rarely be achieved without also addressing the infiltration from private laterals. Generally, rehabilitation to reduce infiltration is cost effective only if a significant amount of infiltration can be isolated to a relatively small area, or there are extremely costly improvements required downstream to convey, treat, and dispose of the excess flow.

I/I Sources on Private Property

I/I sources on private property are primarily defective laterals, but may also include broken cleanouts or cleanout caps, or directly connected roof and area drains. Smoke testing is the primary method for detecting private property I/I sources. For more aggressive programs, building or property inspections can be conducted, and/or laterals can be CCTV inspected or tested for leaks using air or water pressure tests. These types of inspections and tests generally require that the lateral have cleanout access, ideally at both the connection to the building plumbing and at or near the property line. However, new technologies are now available, such as cameras that can be "launched" up the lateral during CCTV inspection of the mainline, that make it easier to inspect private laterals.

4.2 Smoke Testing

Smoke testing was performed in several areas of the system to identify potential infiltration and inflow sources. Smoke testing 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.

Portions of flow meter areas 53, 54, 56, 56A, and 57 were selected for smoke testing based on location (predominantly unincorporated San Mateo County), County experience (suspected I/I sources), and flow monitoring data. The smoke testing was conducted during late September and early October, 2014, by E2 Consulting Engineers under contract to the County. The smoke testing areas are shown on **Figure 4-3,** and the results are summarized in **Table 4-2**. As is typical of such smoke testing programs, the predominant type of defects observed were service laterals. However, the results identified several smoke returns indicating a potential connection to the storm drain system (at catch basins, a storm drain manhole, and from area drains and roof downspouts) that could be direct inflow connections to the sanitary sewer system.

Table 4-2: Smoke Testing Results

Smoke Test Area	Catch Basin	Storm MH	Sewer Main	Area Drain	Roof Down-spout	Lower Lateral	Upper Lateral	Upper CO	Total
1	3	1	1	2	2	3	12		24
2							8		8
3				3	1	3	8	1	16
Total	3	1	1	5	3	6	28	1	48

The smoke testing areas are also candidates for further condition assessment. The condition assessment could also include dye testing to further isolate and confirm the potential cross-connections identified during smoke testing. The County is planning to conduct manhole and pipe (closed-circuit television) inspection of the sewers in flow meter basins 54 and 57 (smoke test area 1, which had the greatest number of defects identified by the smoke testing) as part of an inspection project scheduled to begin in the summer 2015.

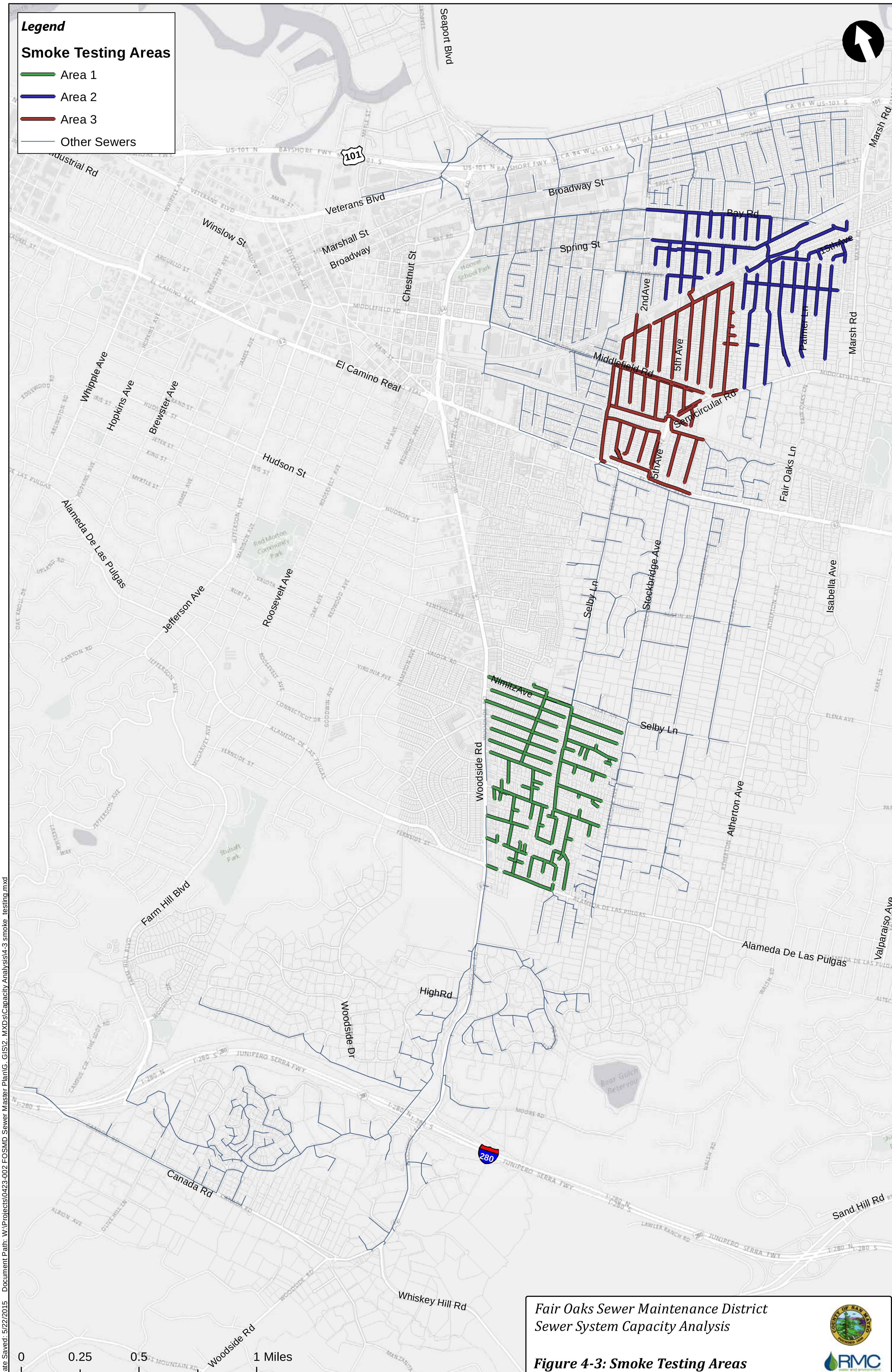


Figure 4-3: Smoke Testing Areas

Date Saved: 5/22/2015 Document Path: W:\Projects\0423-002 FOSMD Sewer Master Plan\G. GIS\2. MXDs\Capacity Analysis\4-3 smoke_testing.mxd

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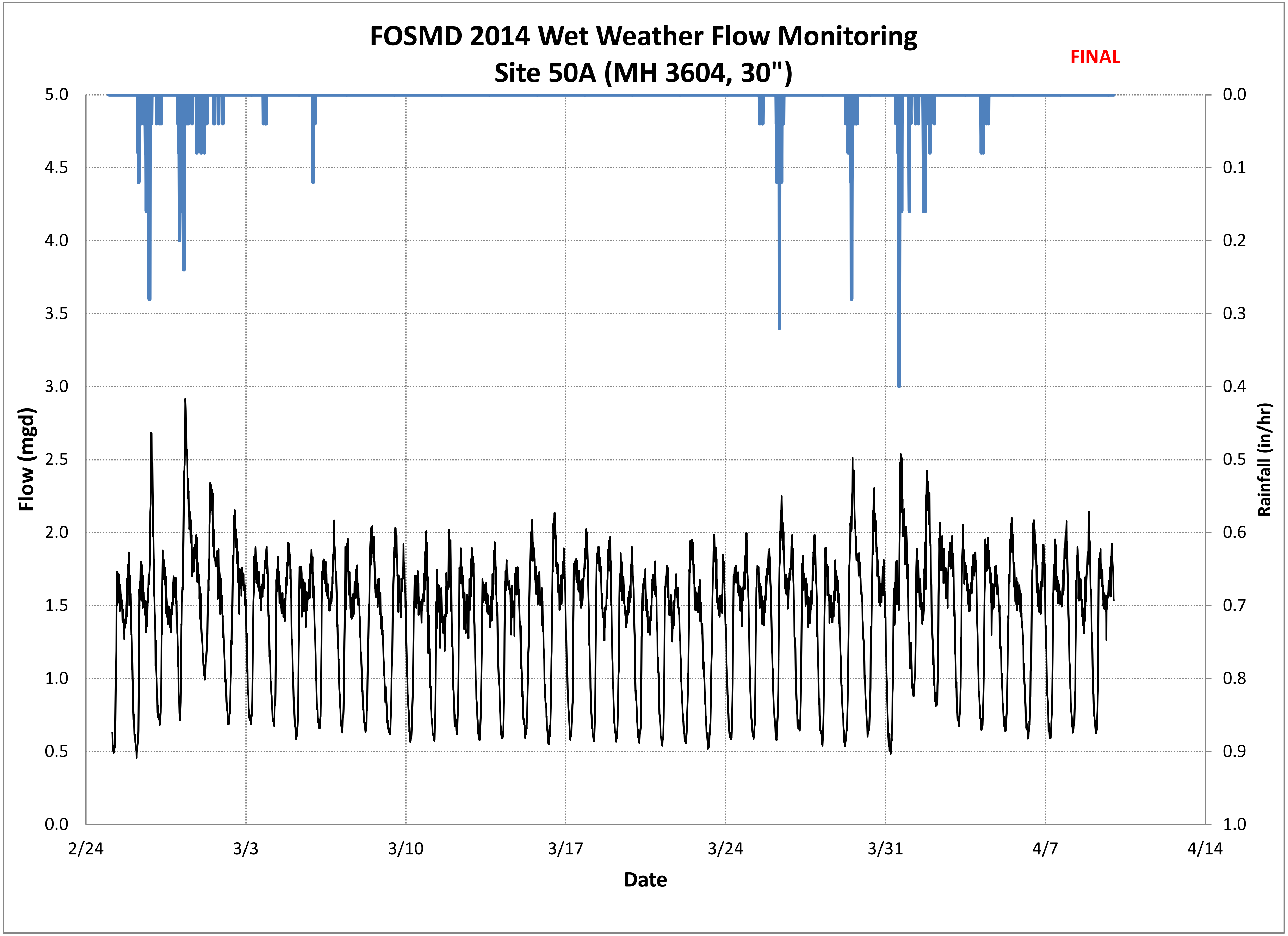
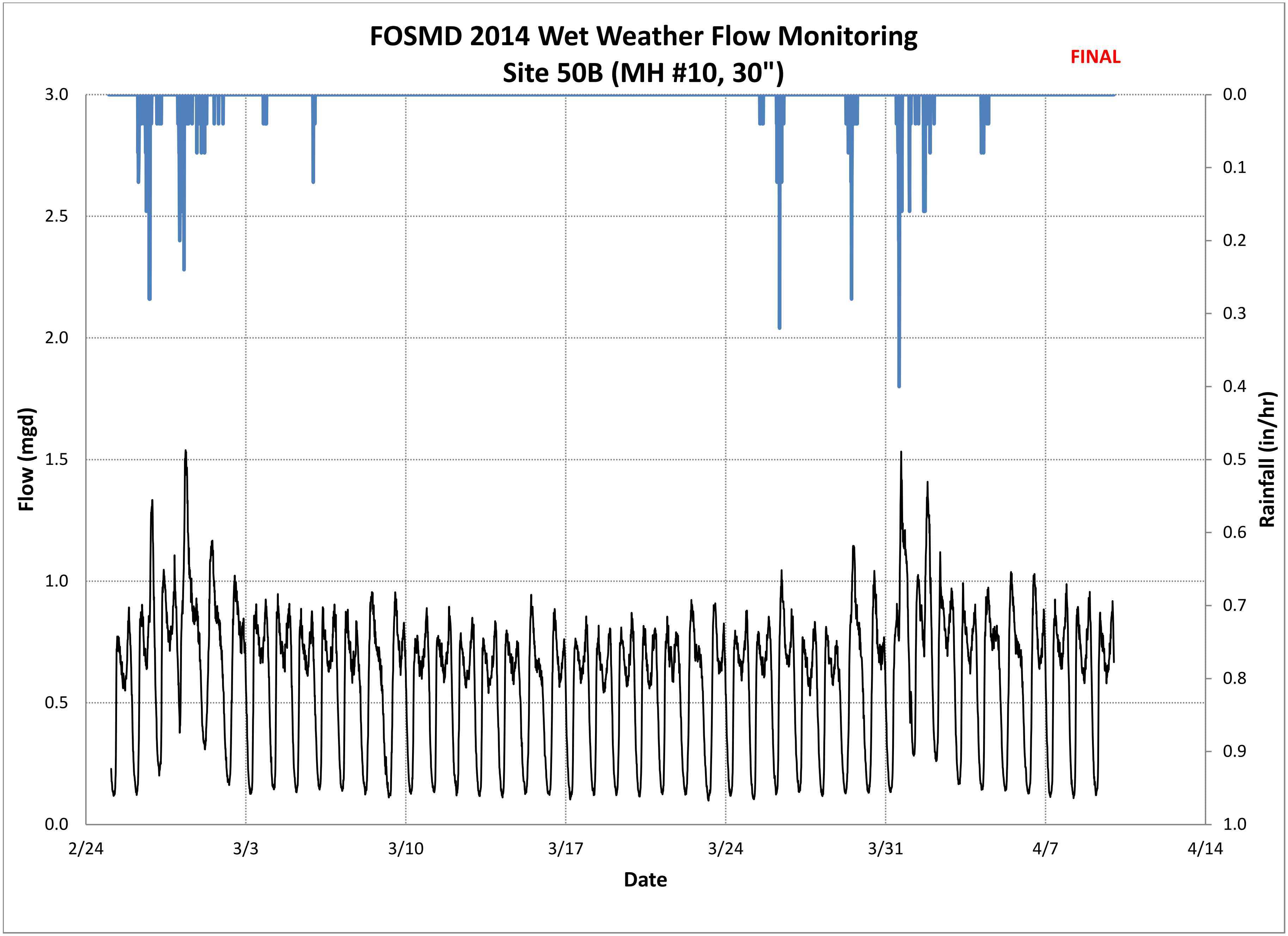
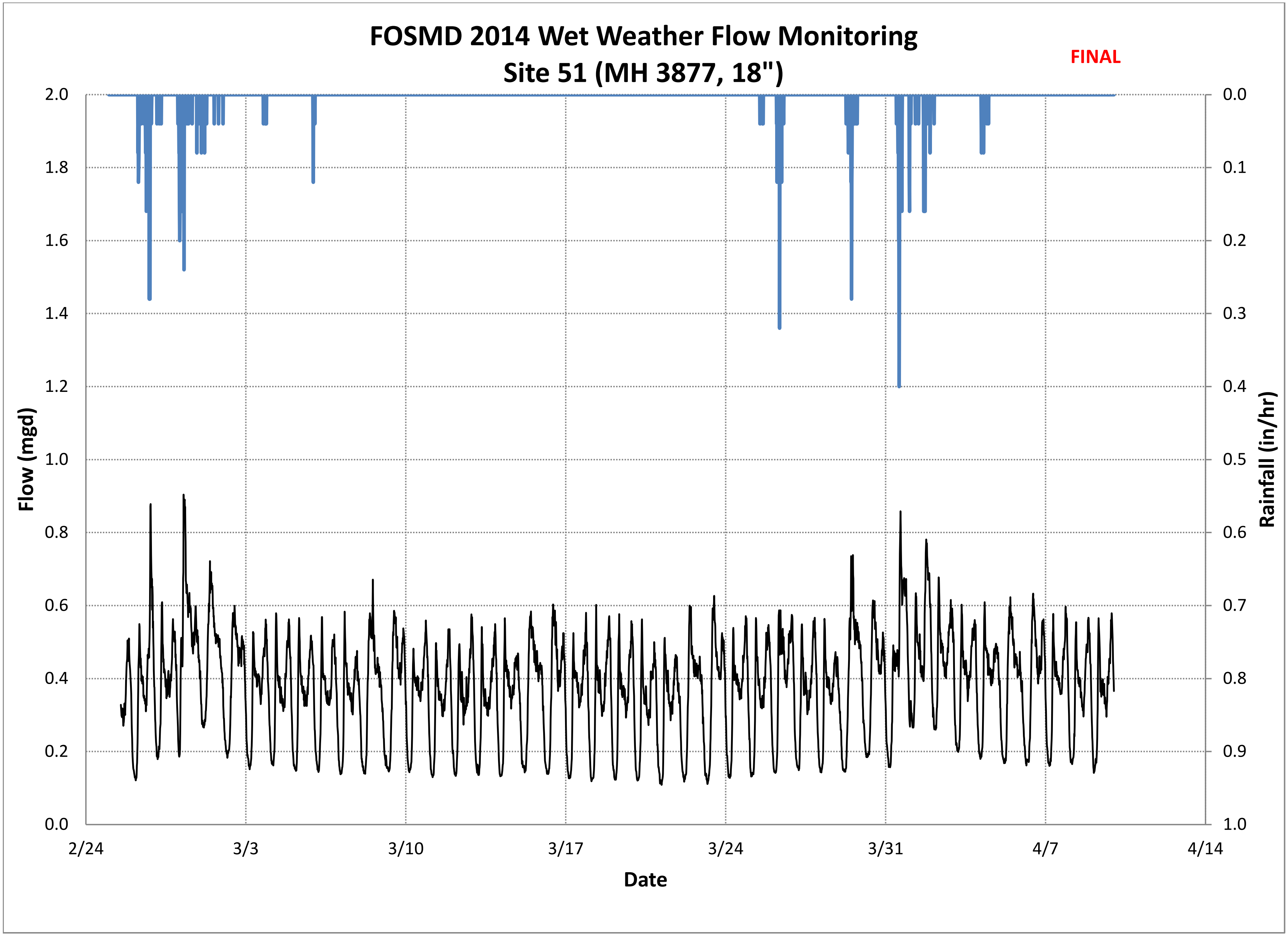
Appendices

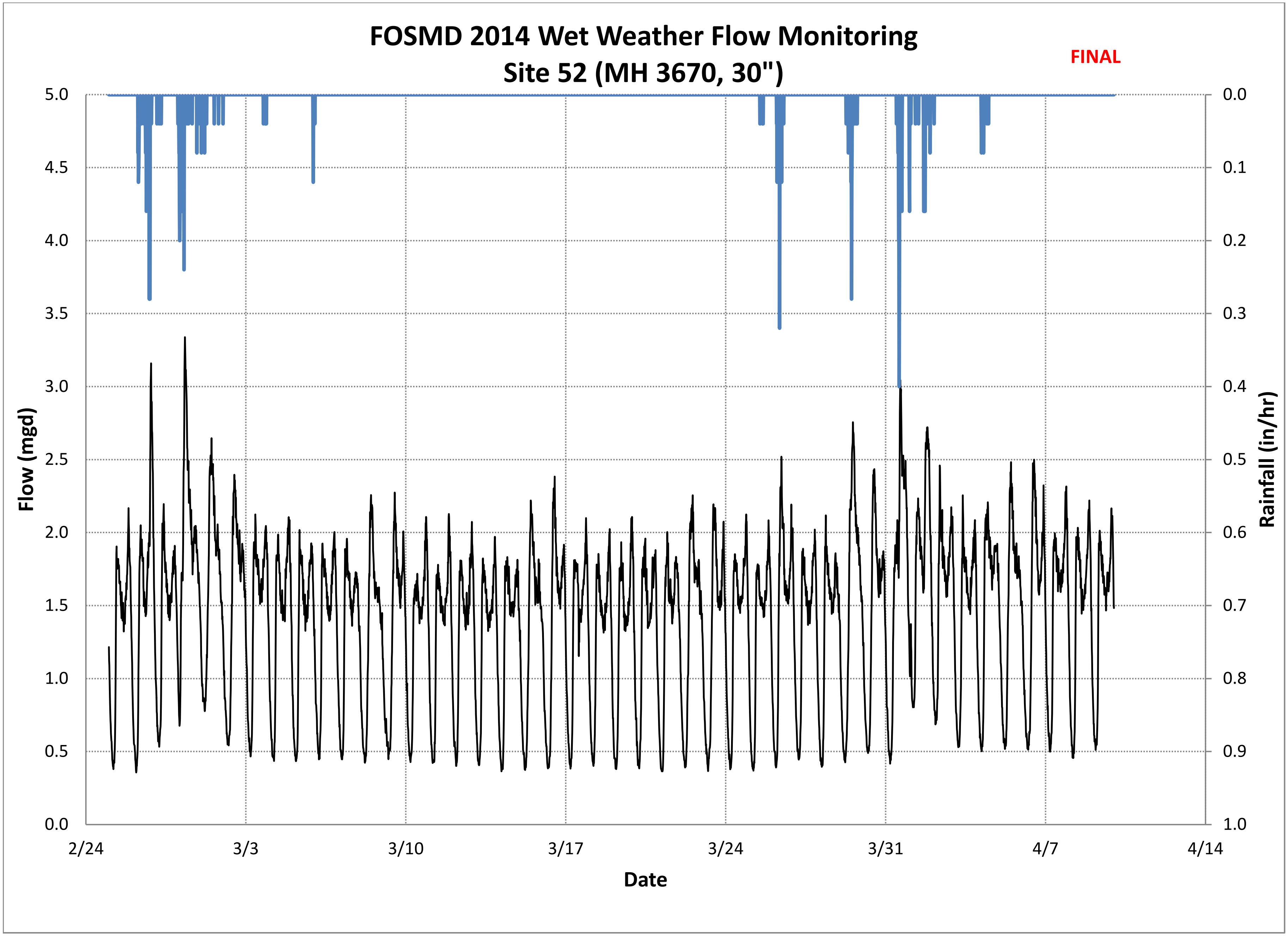
Appendix A - Plots of Flow Monitoring Data

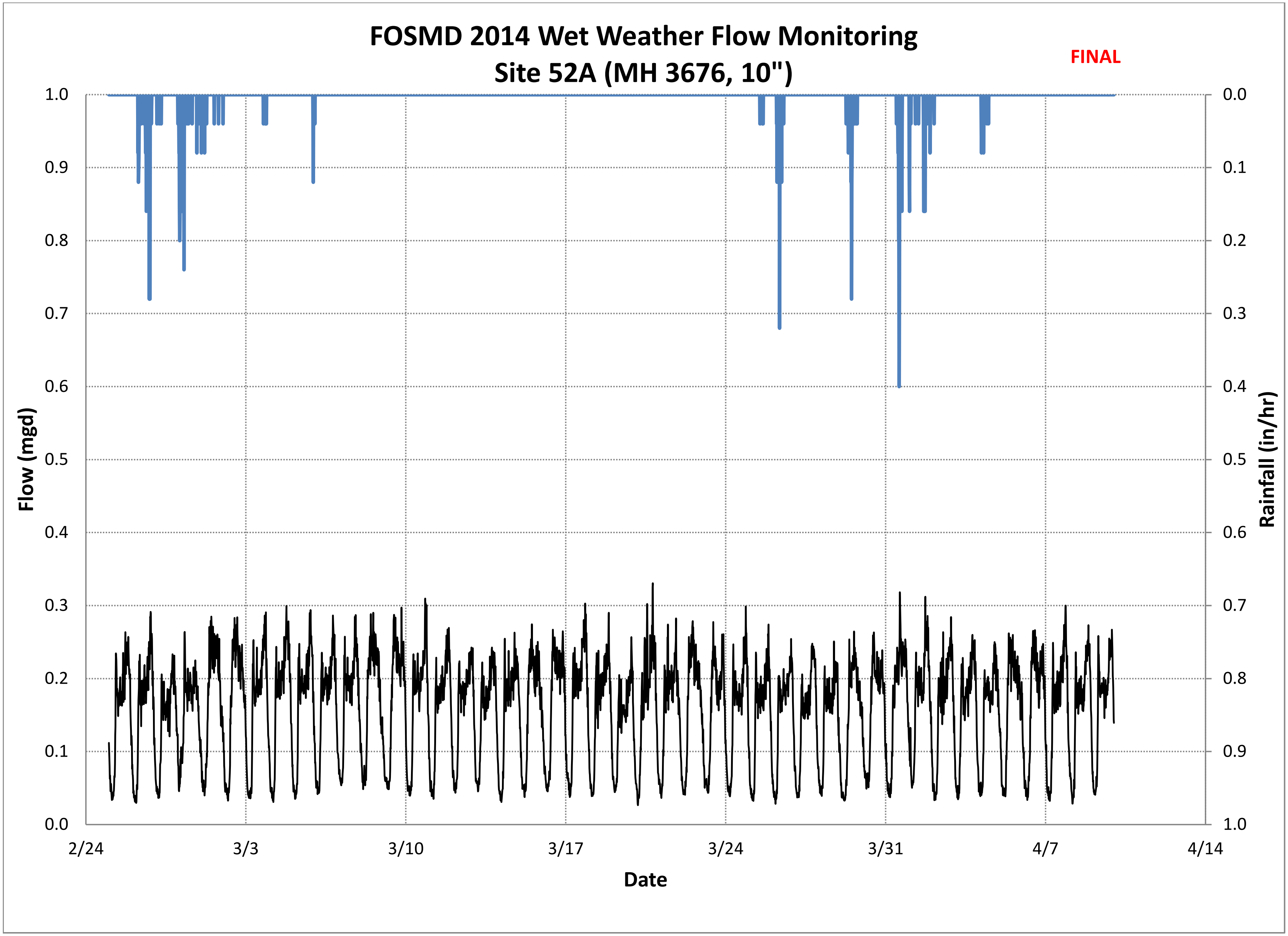

FOSMD 2014 Wet Weather Flow Monitoring
Site 50A (MH 3604, 30")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

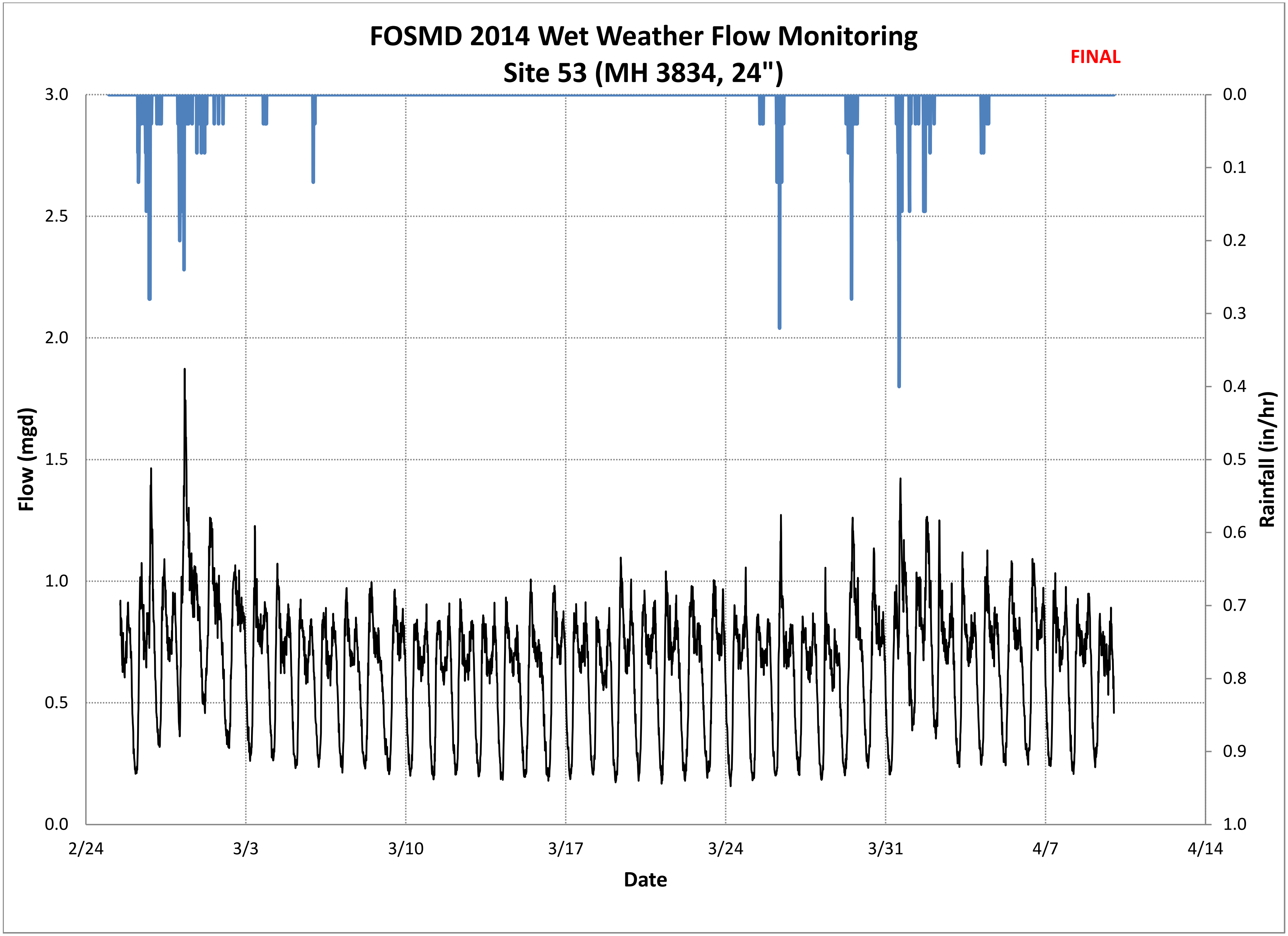

FOSMD 2014 Wet Weather Flow Monitoring
Site 50B (MH #10, 30")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

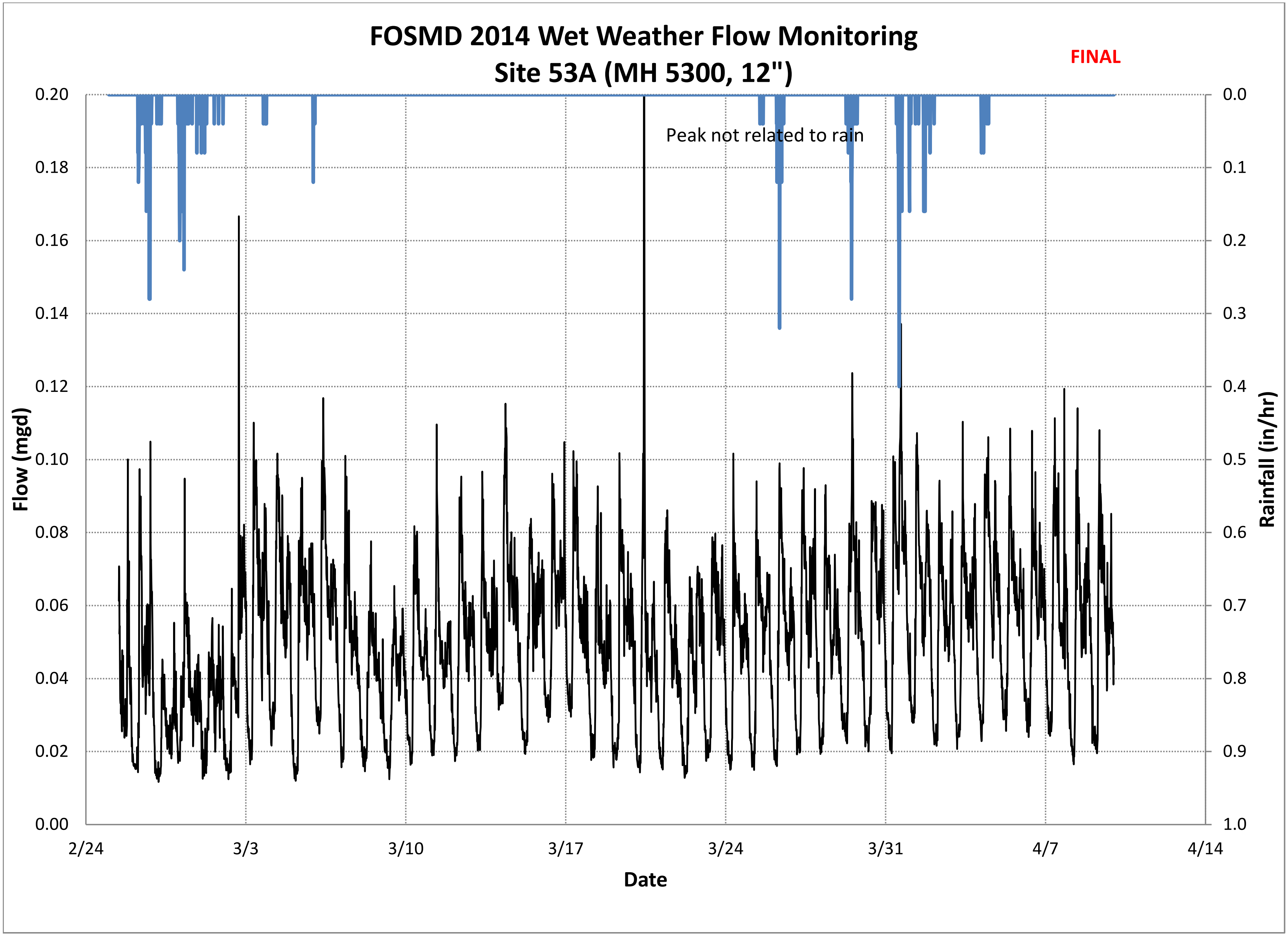


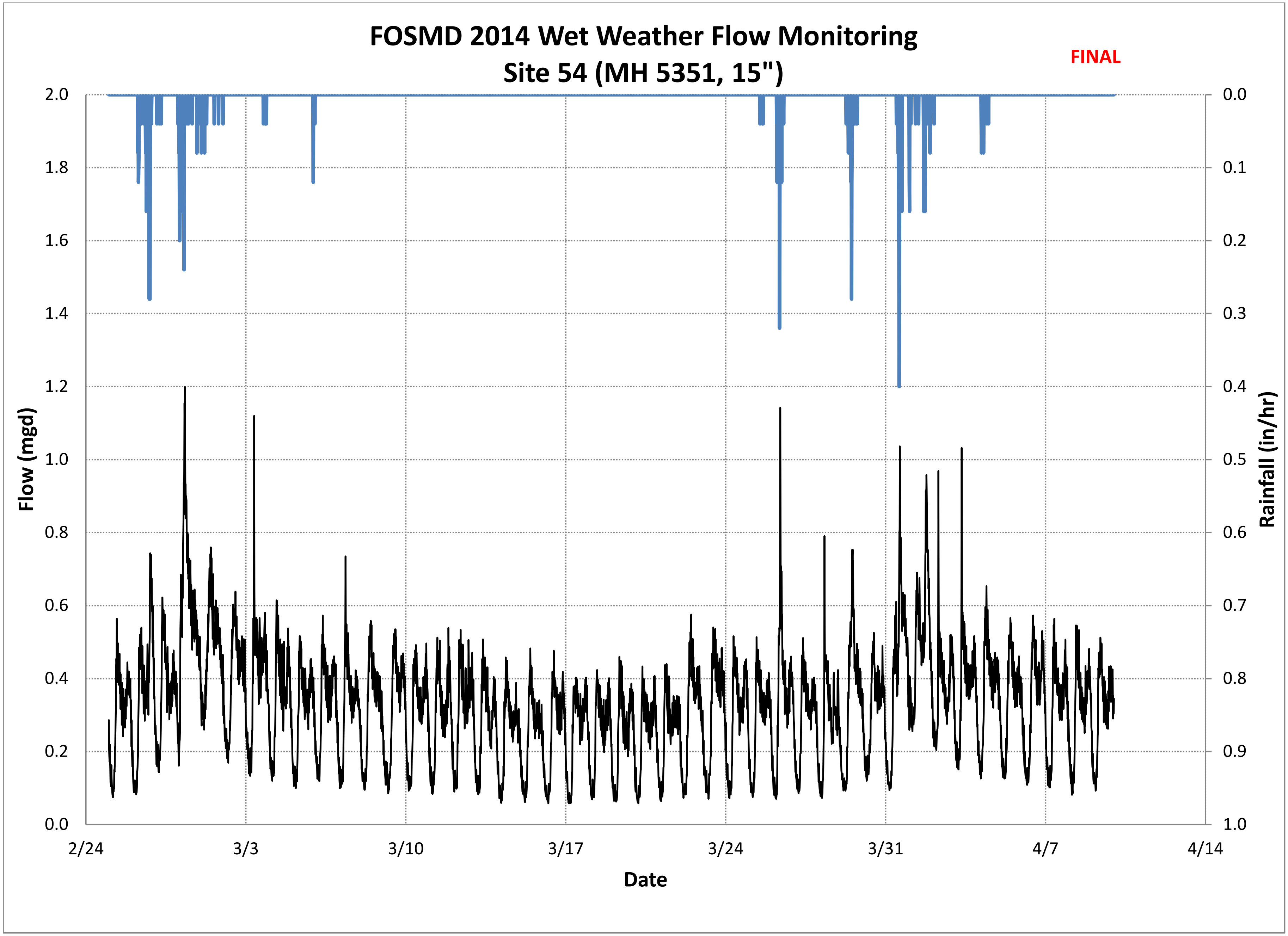
FOSMD 2014 Wet Weather Flow Monitoring
Site 51 (MH 3877, 18")
FINAL
Flow (mgd)
Rainfall (in/hr)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14
Date

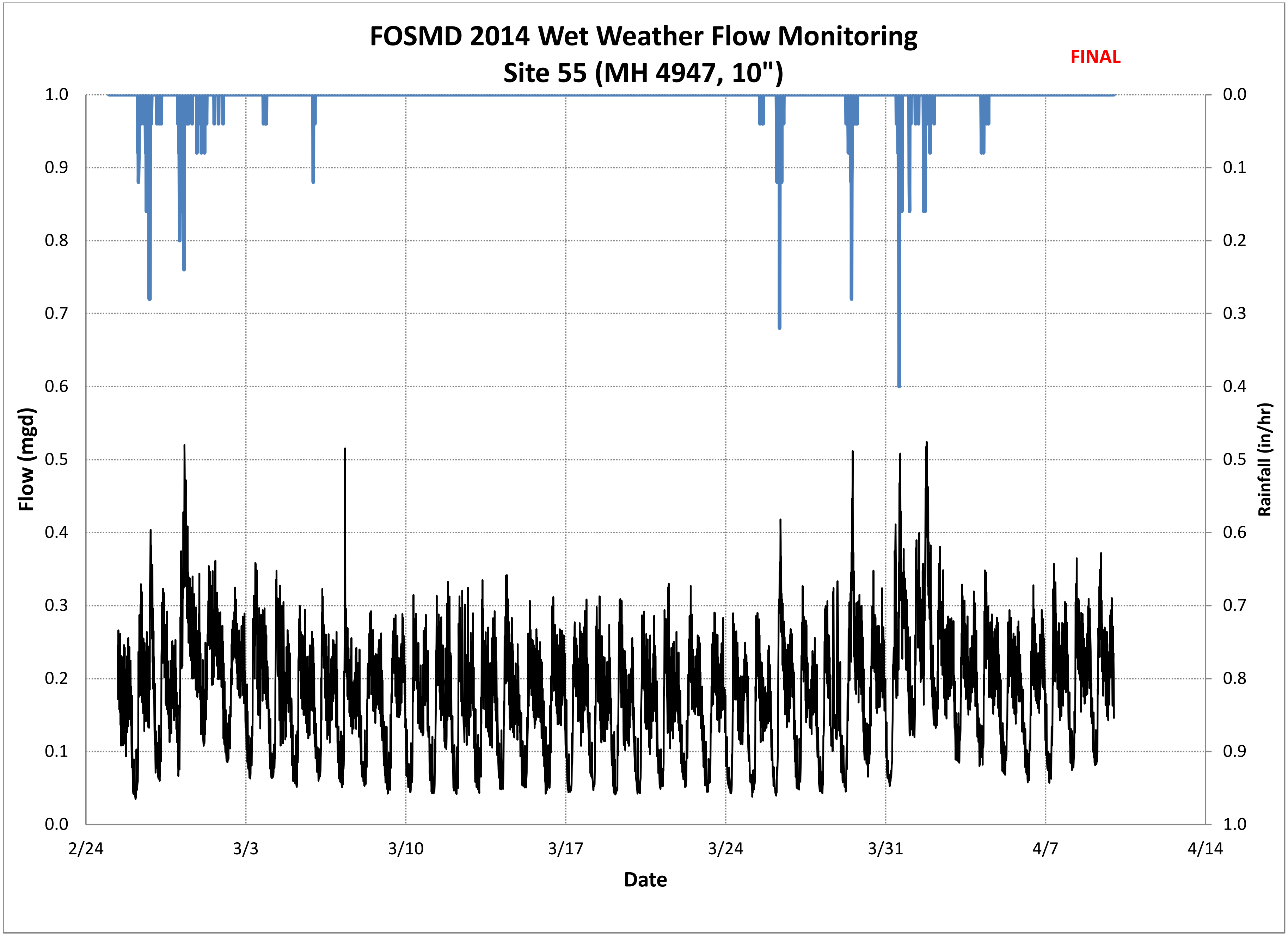

FOSMD 2014 Wet Weather Flow Monitoring
Site 52 (MH 3670, 30")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

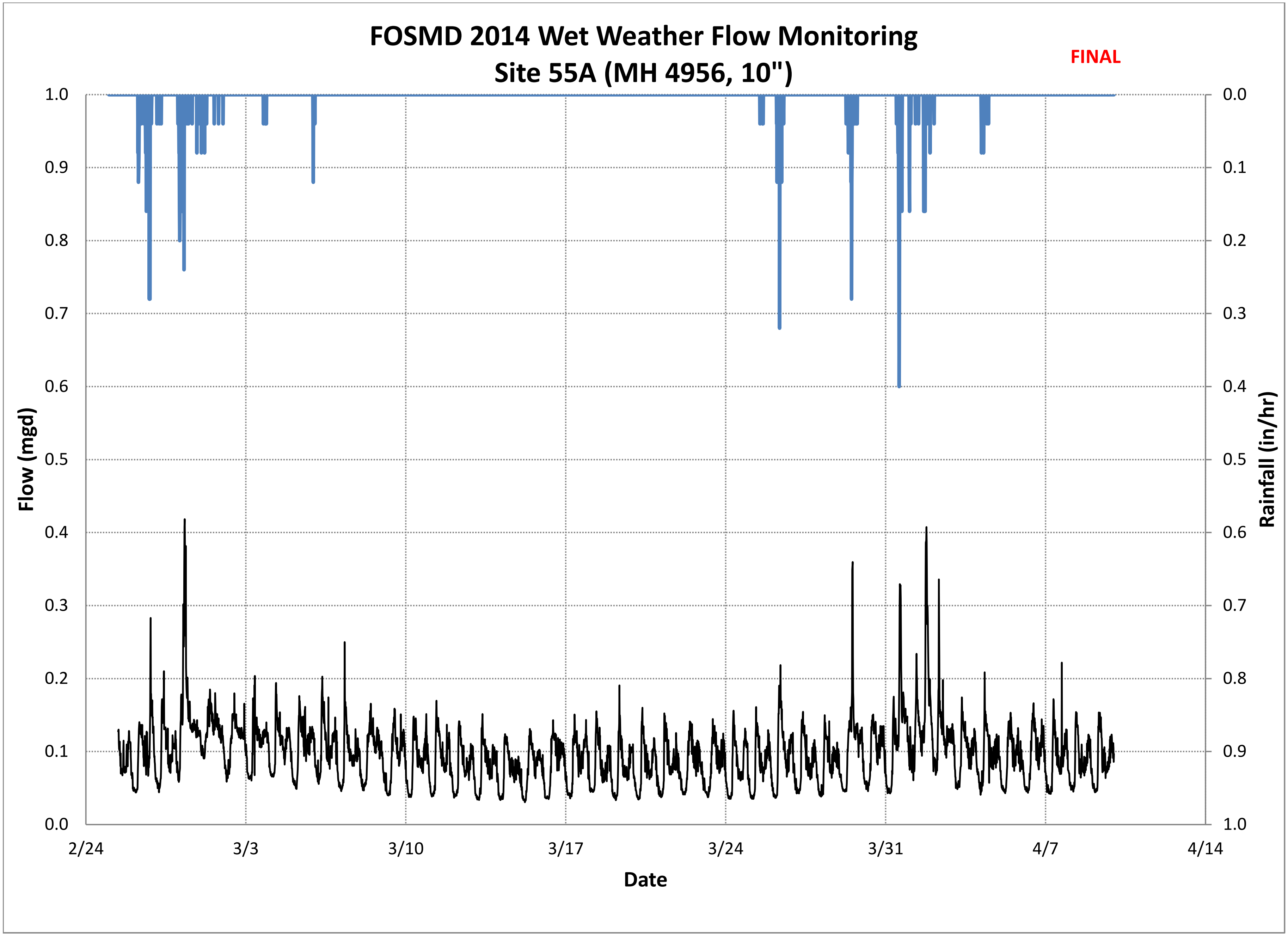

FOSMD 2014 Wet Weather Flow Monitoring
Site 52A (MH 3676, 10")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14


FOSMD 2014 Wet Weather Flow Monitoring
Site 53 (MH 3834, 24")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

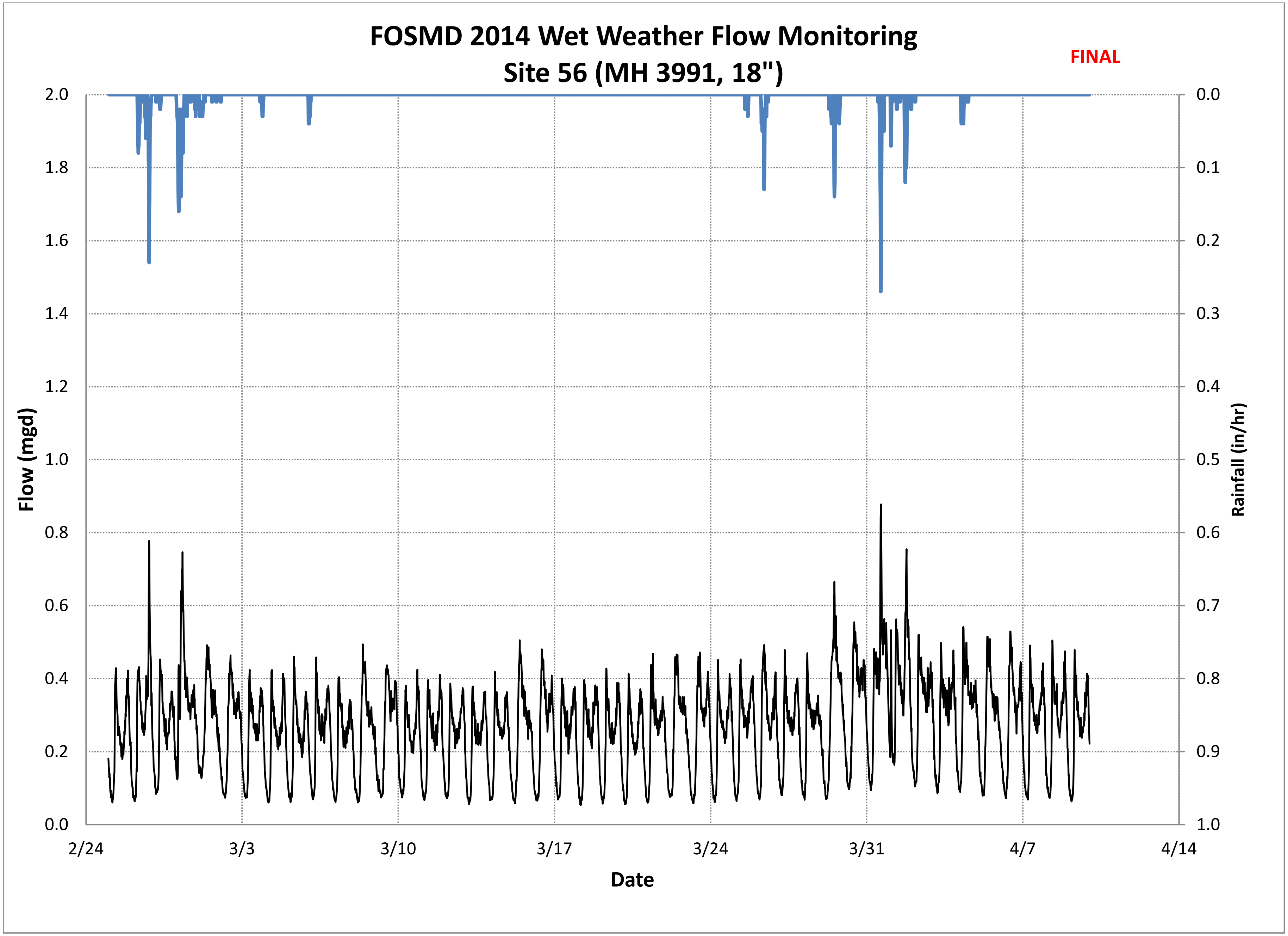

FOSMD 2014 Wet Weather Flow Monitoring
Site 53A (MH 5300, 12")
FINAL
Peak not related to rain
Flow (mgd)
Rainfall (in/hr)
Date
0.20
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0.00
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14


FOSMD 2014 Wet Weather Flow Monitoring
Site 54 (MH 5351, 15")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

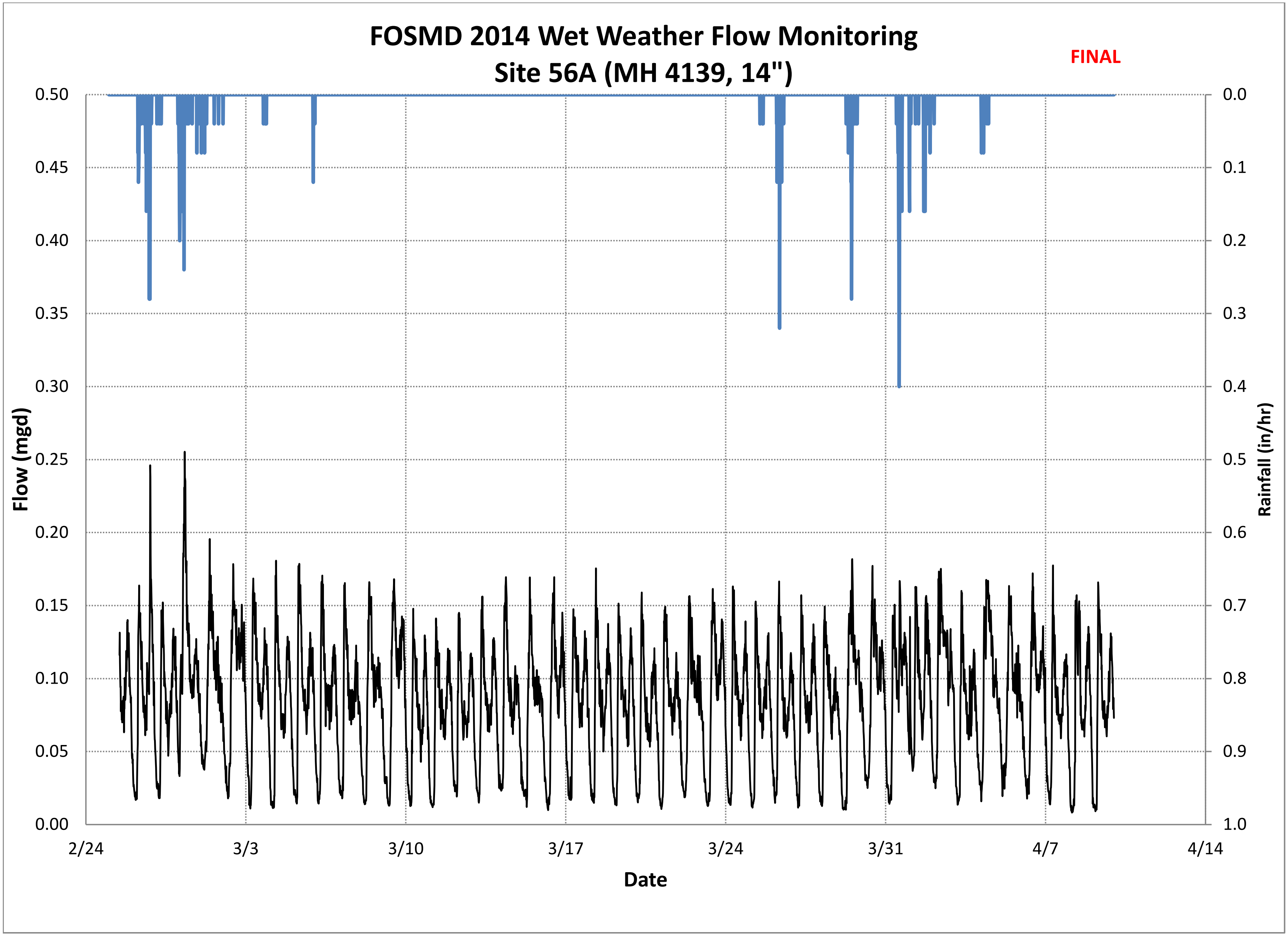

FOSMD 2014 Wet Weather Flow Monitoring
Site 55 (MH 4947, 10")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

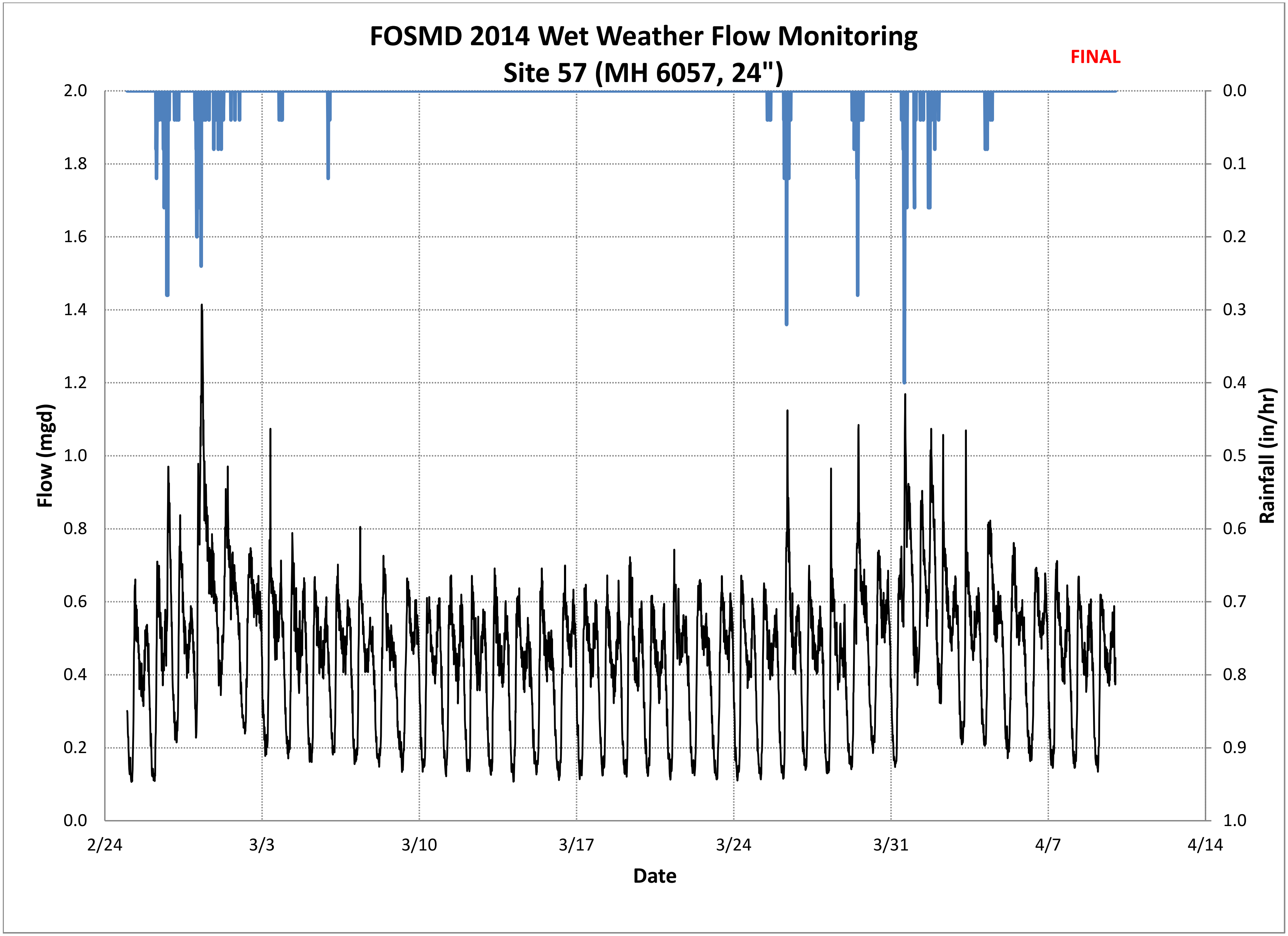


FOSMD 2014 Wet Weather Flow Monitoring
Site 55A (MH 4956, 10")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14

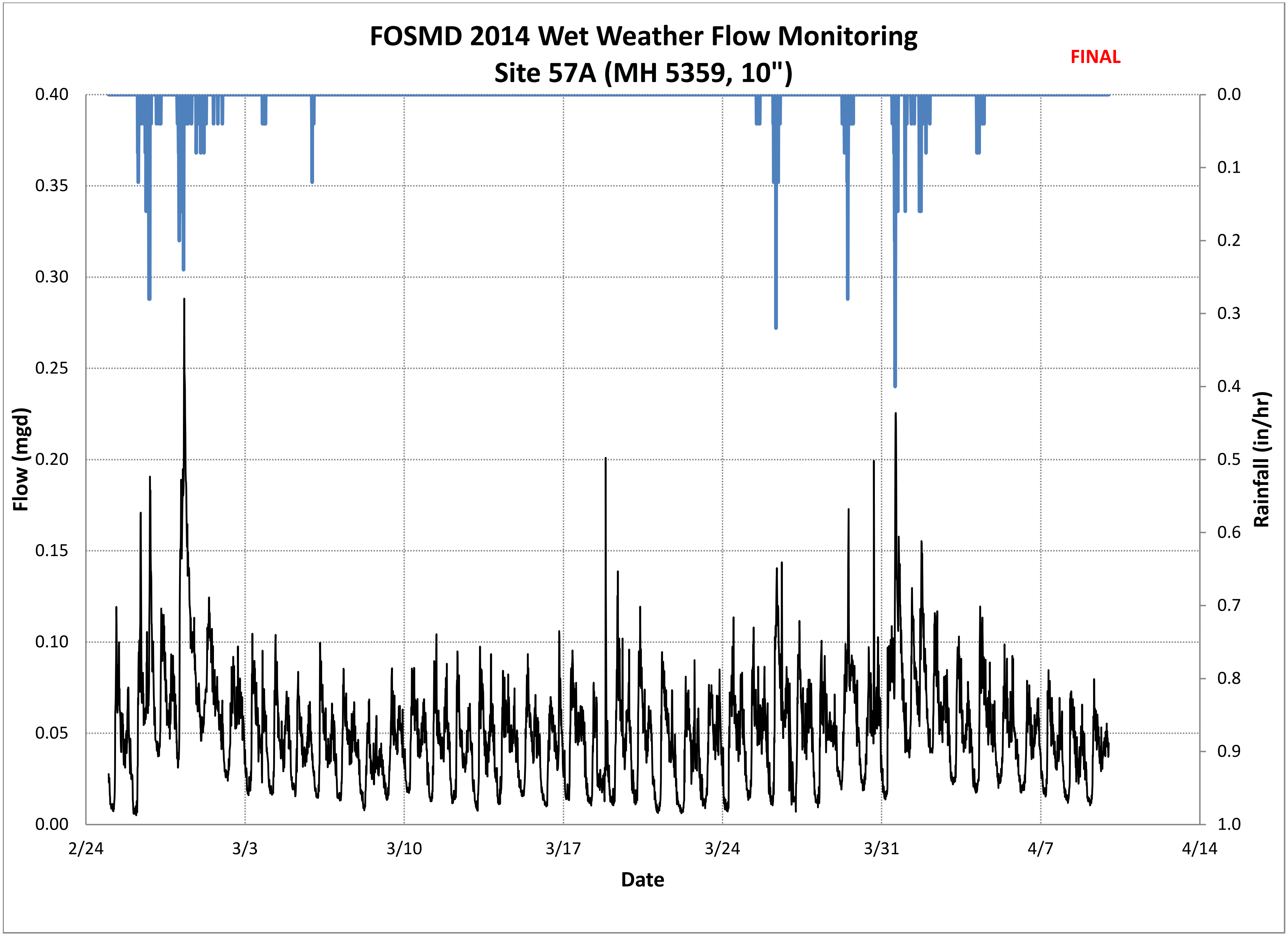


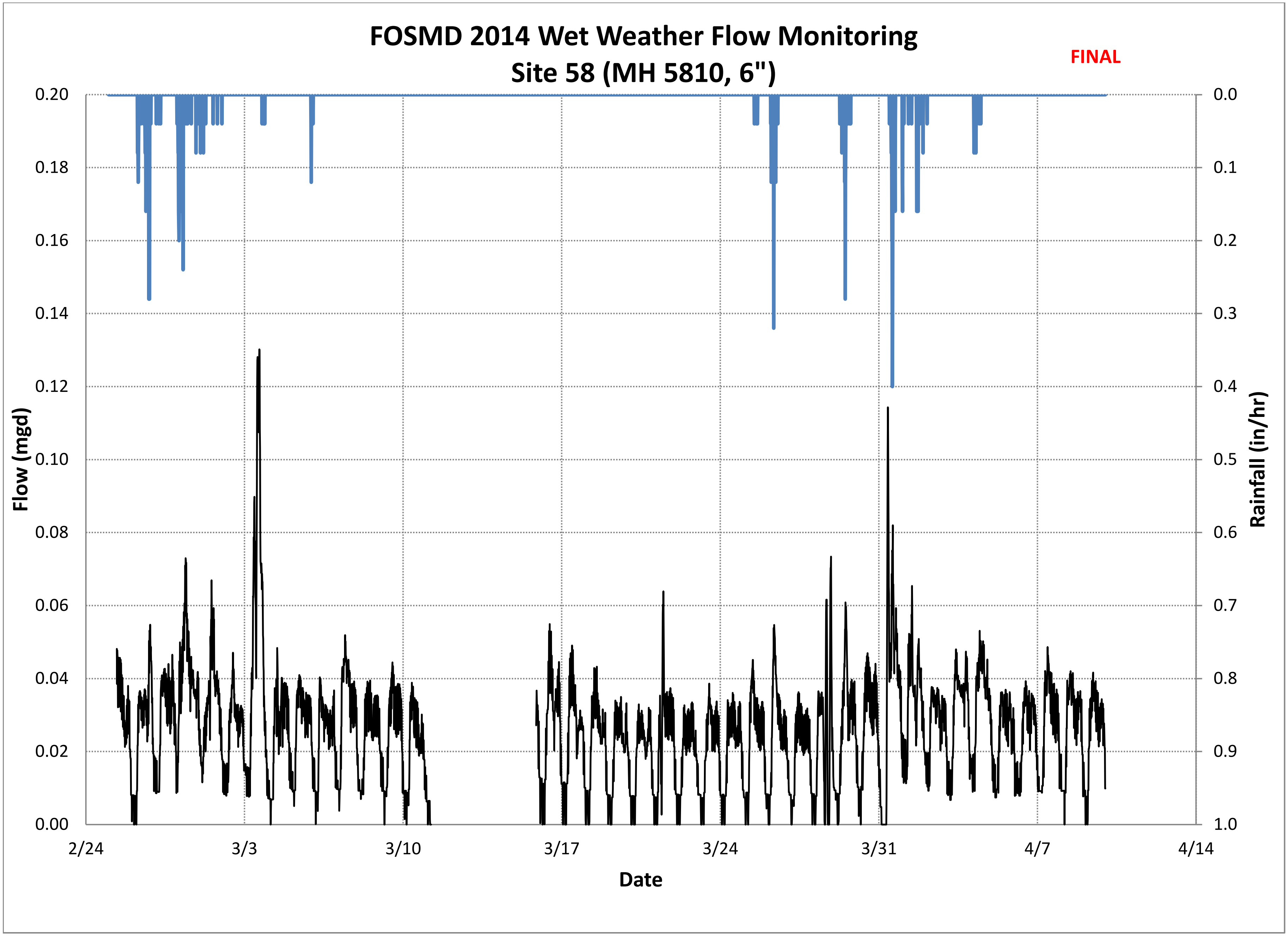
FOSMD 2014 Wet Weather Flow Monitoring
Site 56 (MH 3991, 18")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.1
0.3
0.5
0.7
0.9
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14


FOSMD 2014 Wet Weather Flow Monitoring
Site 56A (MH 4139, 14")
FINAL
Flow (mgd)
Rainfall (in/hr)
Date
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14



FOSMD 2014 Wet Weather Flow Monitoring
Site 57 (MH 6057, 24")
FINAL
Flow (mgd)
Rainfall (in/hr)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14
Date

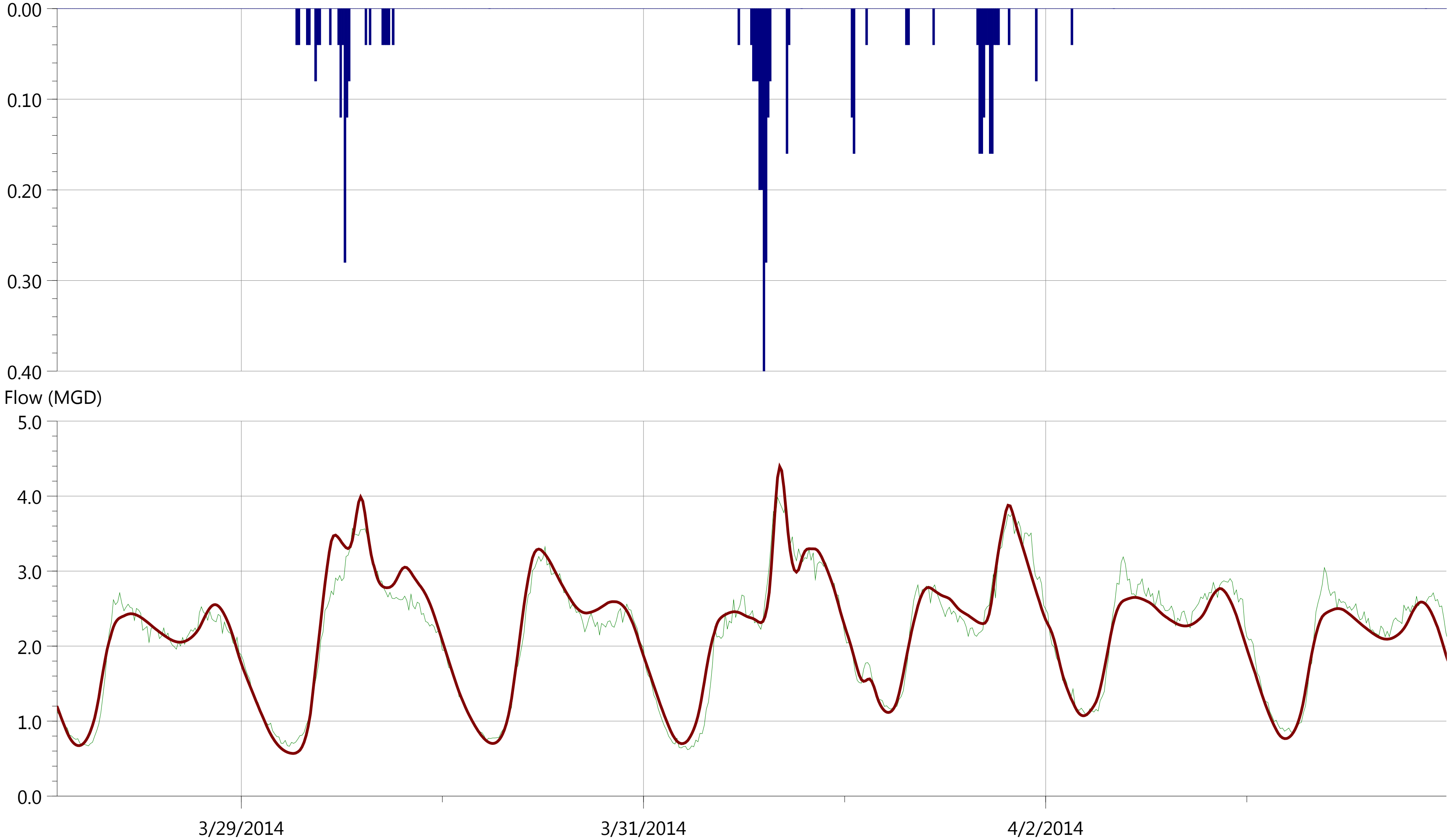

FOSMD 2014 Wet Weather Flow Monitoring
Site 57A (MH 5359, 10")
FINAL
Flow (mgd)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Rainfall (in/hr)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14
Date



FOSMD 2014 Wet Weather Flow Monitoring
Site 58 (MH 5810, 6")
FINAL
Flow (mgd)
0.20
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0.00
Rainfall (in/hr)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2/24
3/3
3/10
3/17
3/24
3/31
4/7
4/14
Date

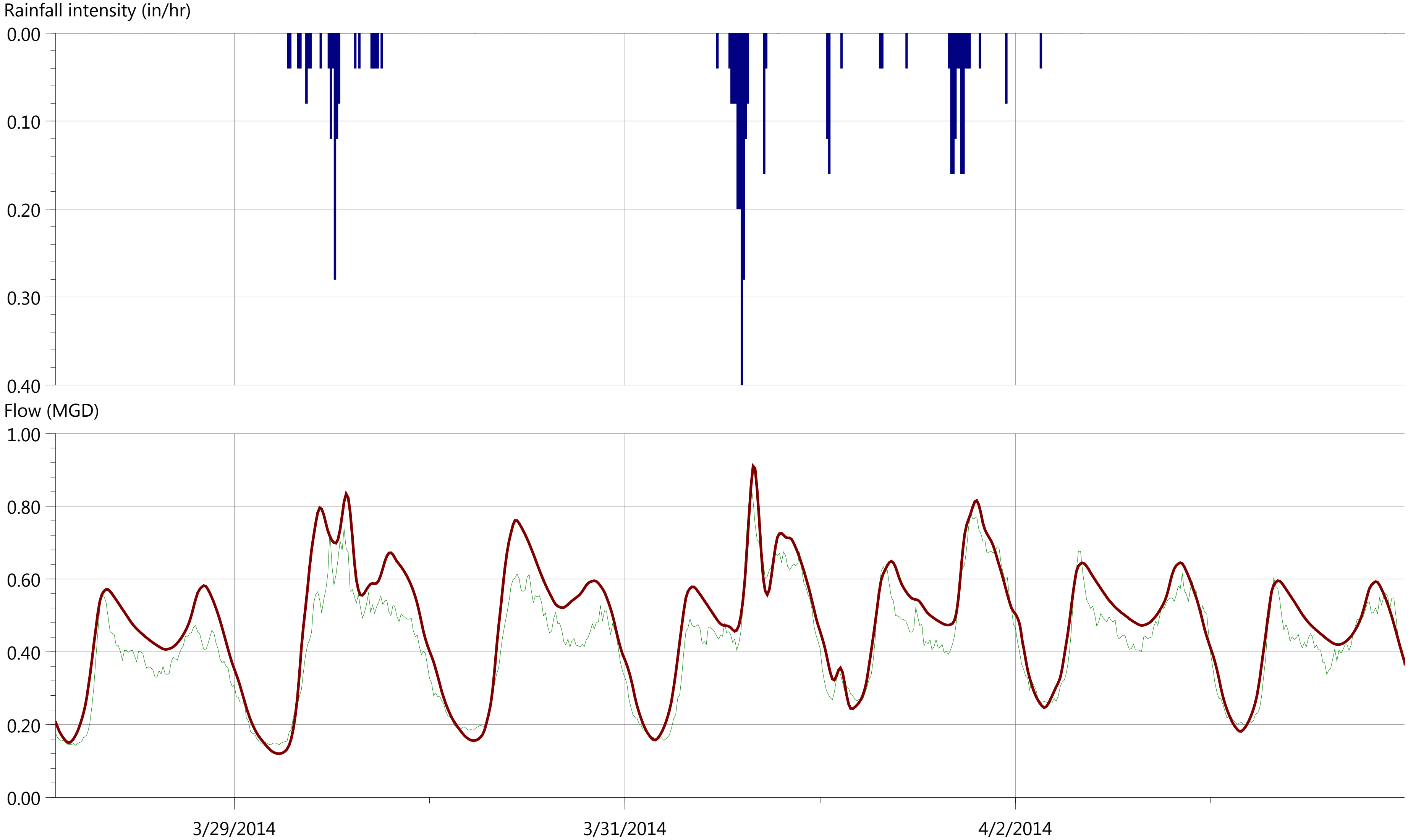
Appendix B - Model Calibration Graphs

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 2 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 50A/B, Model Location (Pred.) D/S DS_DU1.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.619	3.984	14.827
...04>2014 Calibration				0.569	4.393	14.882

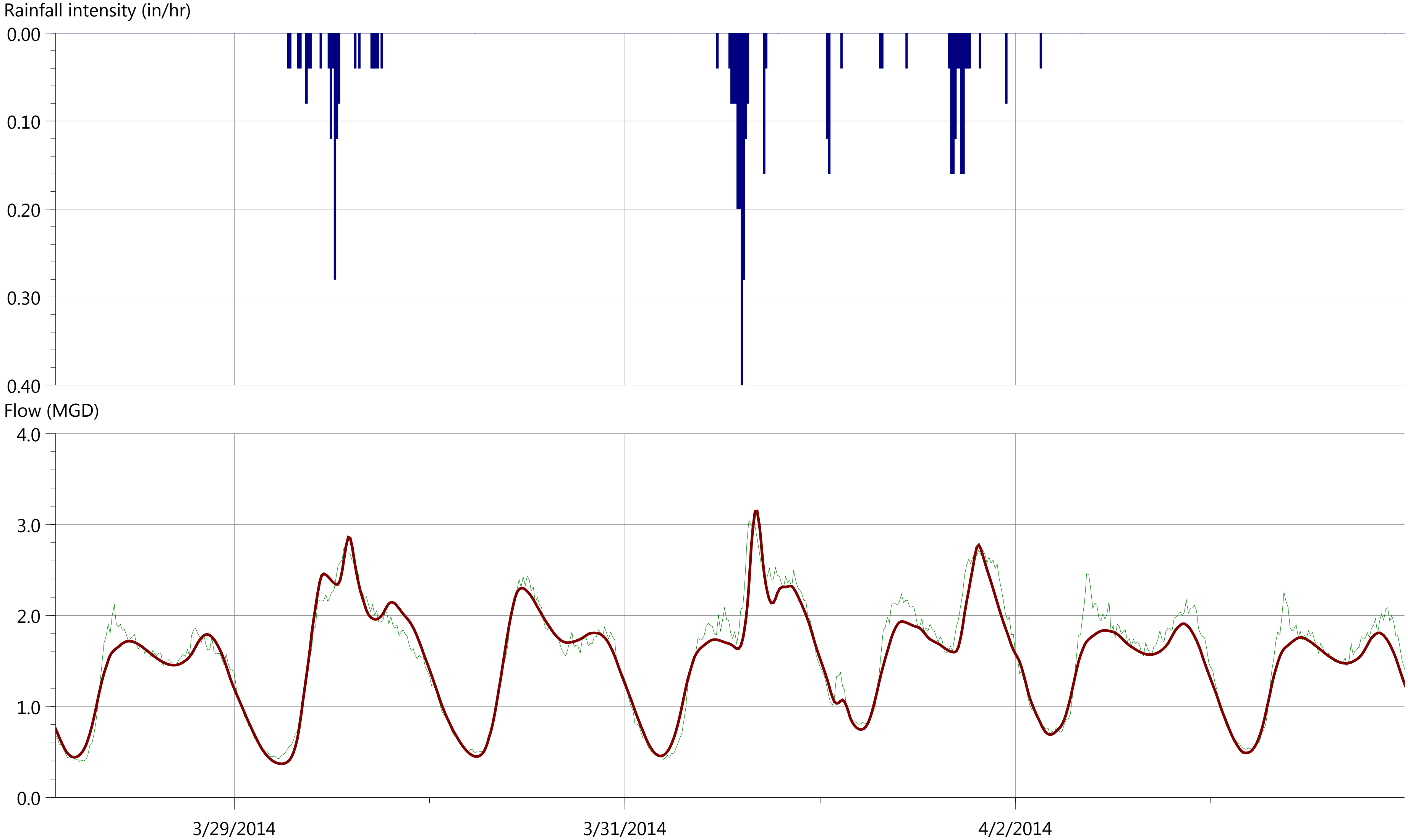
Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 4 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 51, Model Location (Pred.) D/S 3877.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.143	0.858	2.915
...04>2014 Calibration				0.120	0.910	3.281

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 5 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)

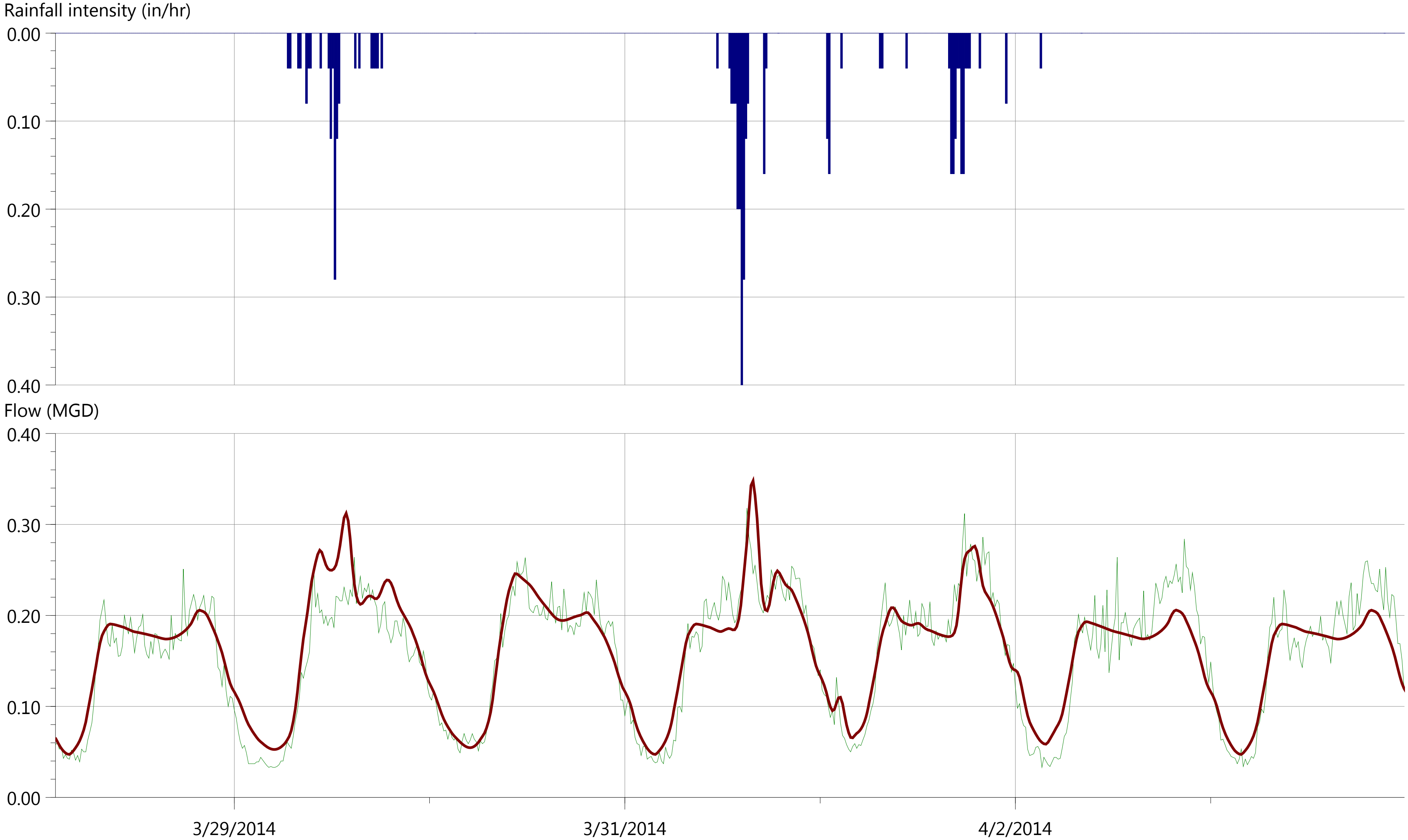
Flow Survey Location (Obs.) FM 52, Model Location (Pred.) D/S 3670.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.395	3.045	10.727
...04>2014 Calibration				0.370	3.148	10.338

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 6 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)

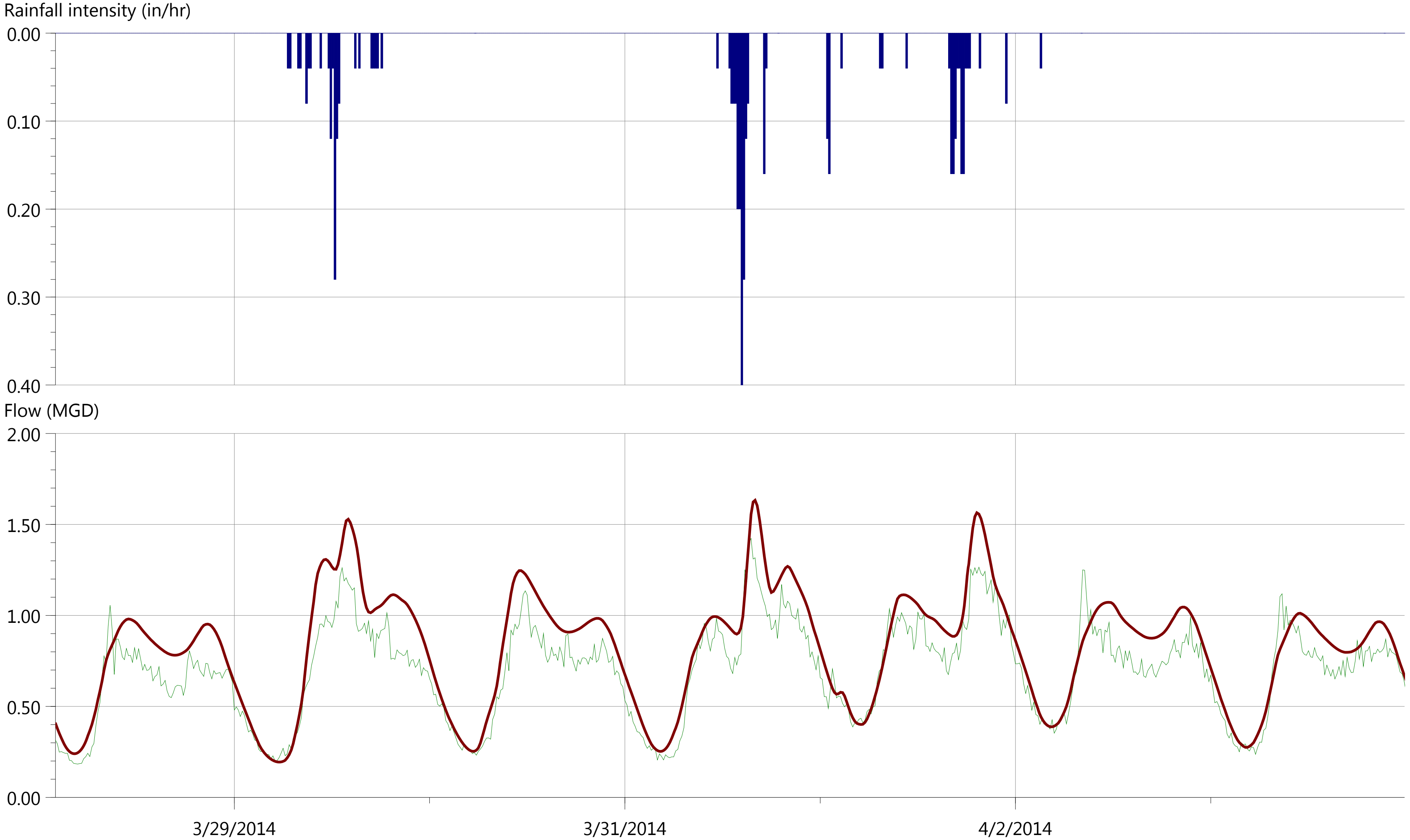
Flow Survey Location (Obs.) FM 52A, Model Location (Pred.) D/S 3676.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.033	0.318	1.095
...04>2014 Calibration				0.047	0.348	1.116

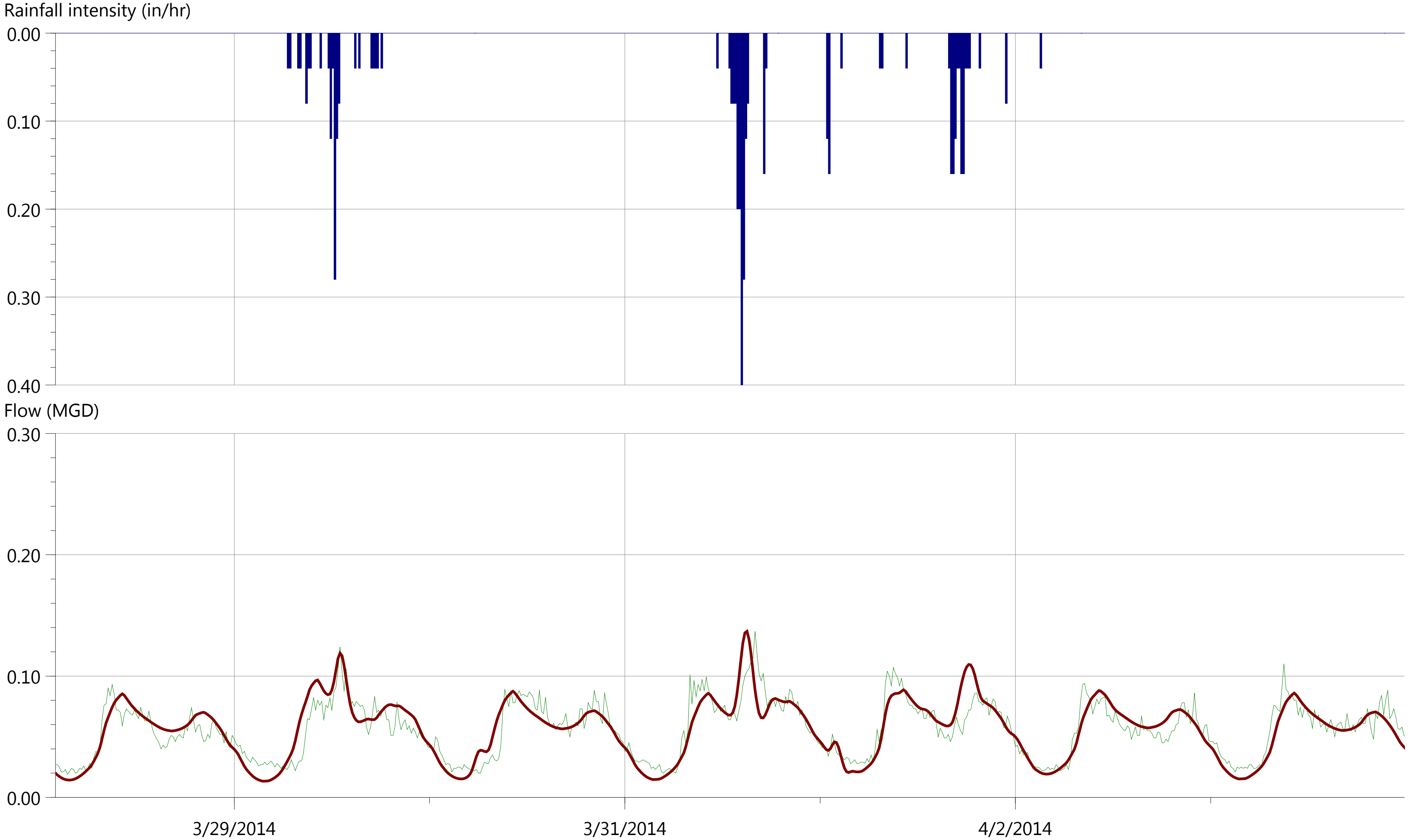
Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 7 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)

Flow Survey Location (Obs.) FM 53, Model Location (Pred.) D/S 3834.1, Rainfall Profile: 1



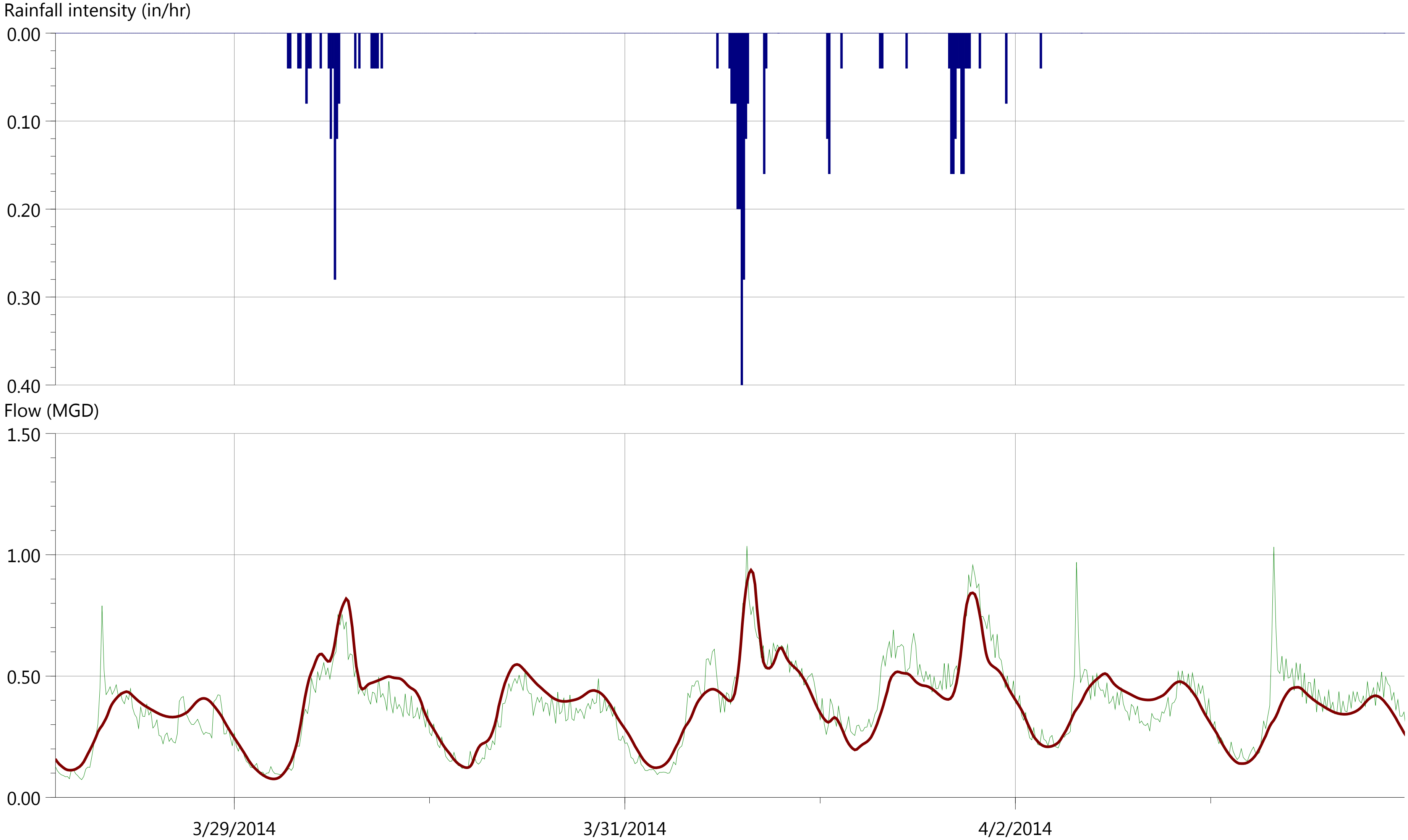
	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.183	1.423	4.814
...04>2014 Calibration				0.194	1.633	5.633

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 8 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 53A, Model Location (Pred.) D/S 5300.1, Rainfall Profile: 1



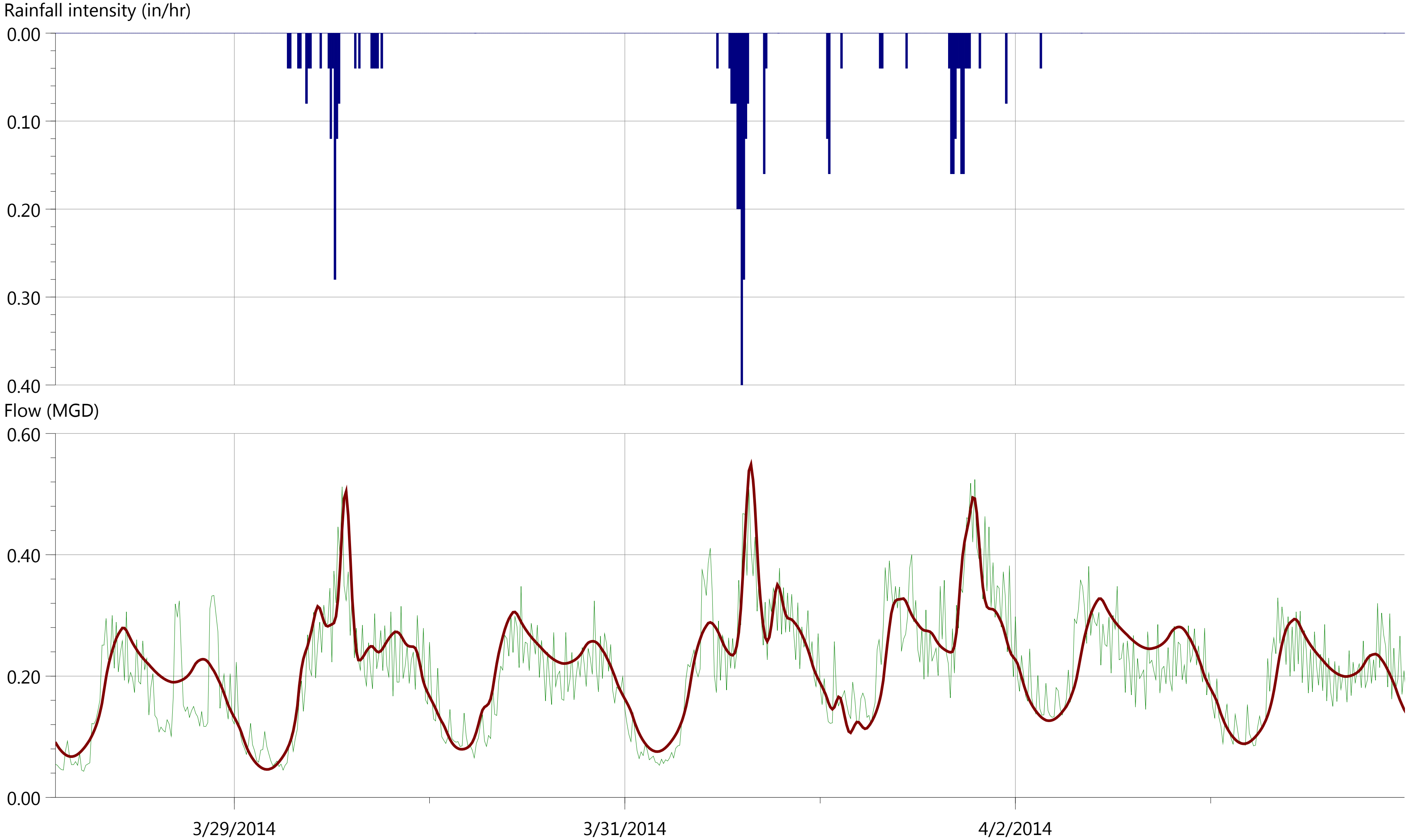
	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.019	0.137	0.390
...04>2014 Calibration				0.013	0.137	0.389

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 9 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 54, Model Location (Pred.) D/S 5351.1, Rainfall Profile: 1



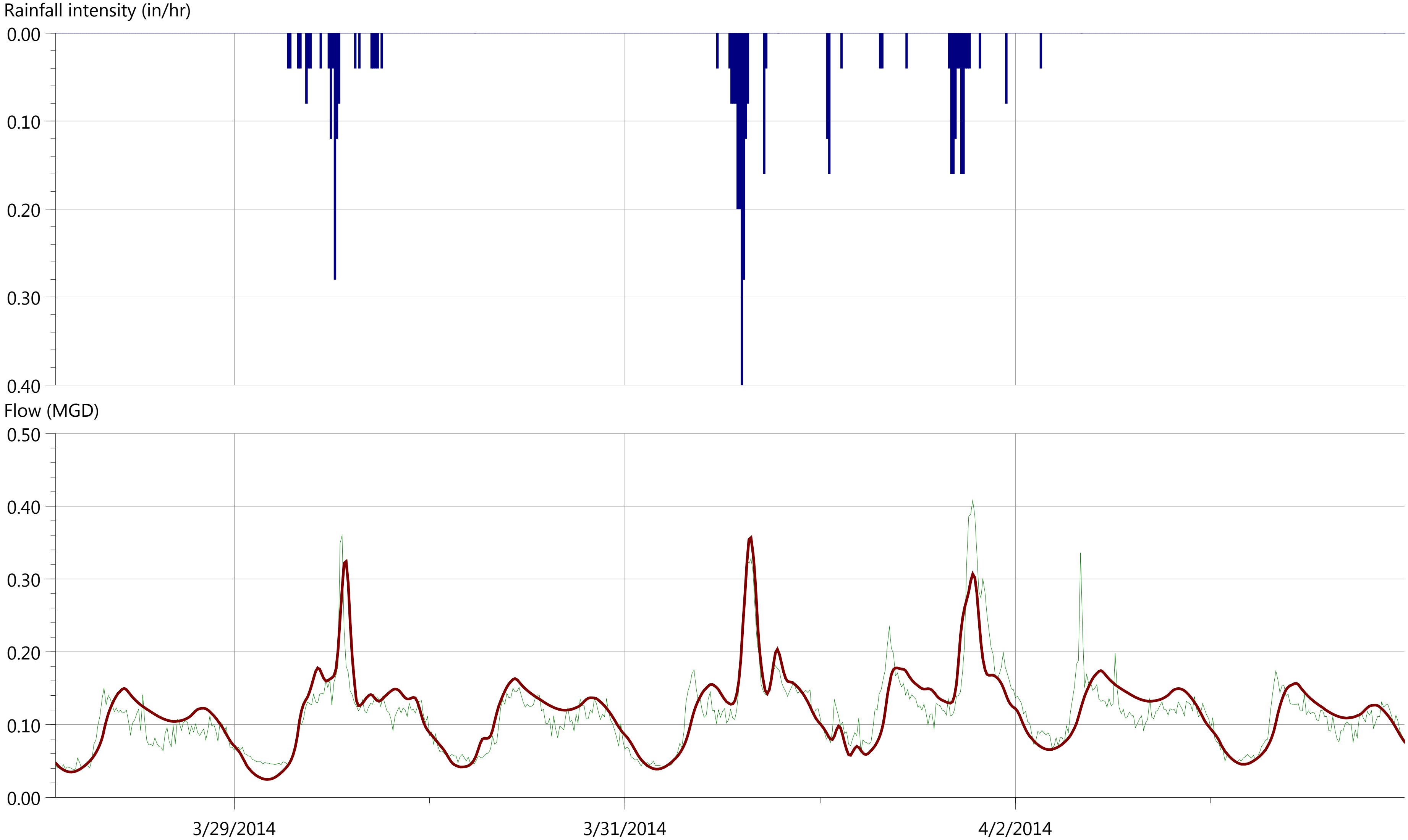
	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.073	1.036	2.603
...04>2014 Calibration				0.077	0.937	2.599

Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 55, Model Location (Pred.) D/S 4947.1, Rainfall Profile: 1



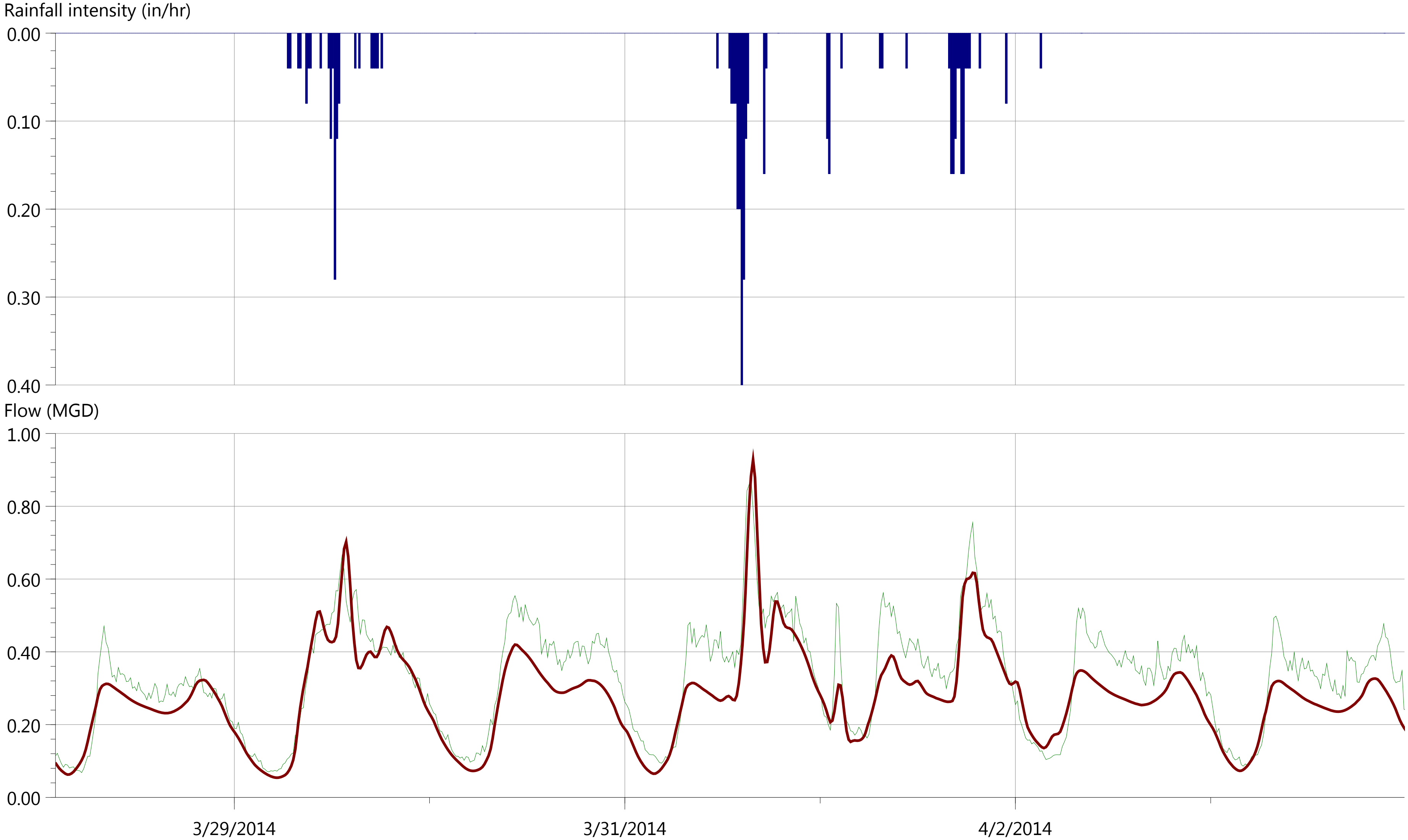
	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.043	0.524	1.444
...04>2014 Calibration				0.046	0.549	1.483

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 11 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 55A, Model Location (Pred.) D/S 4956.1, Rainfall Profile: 1



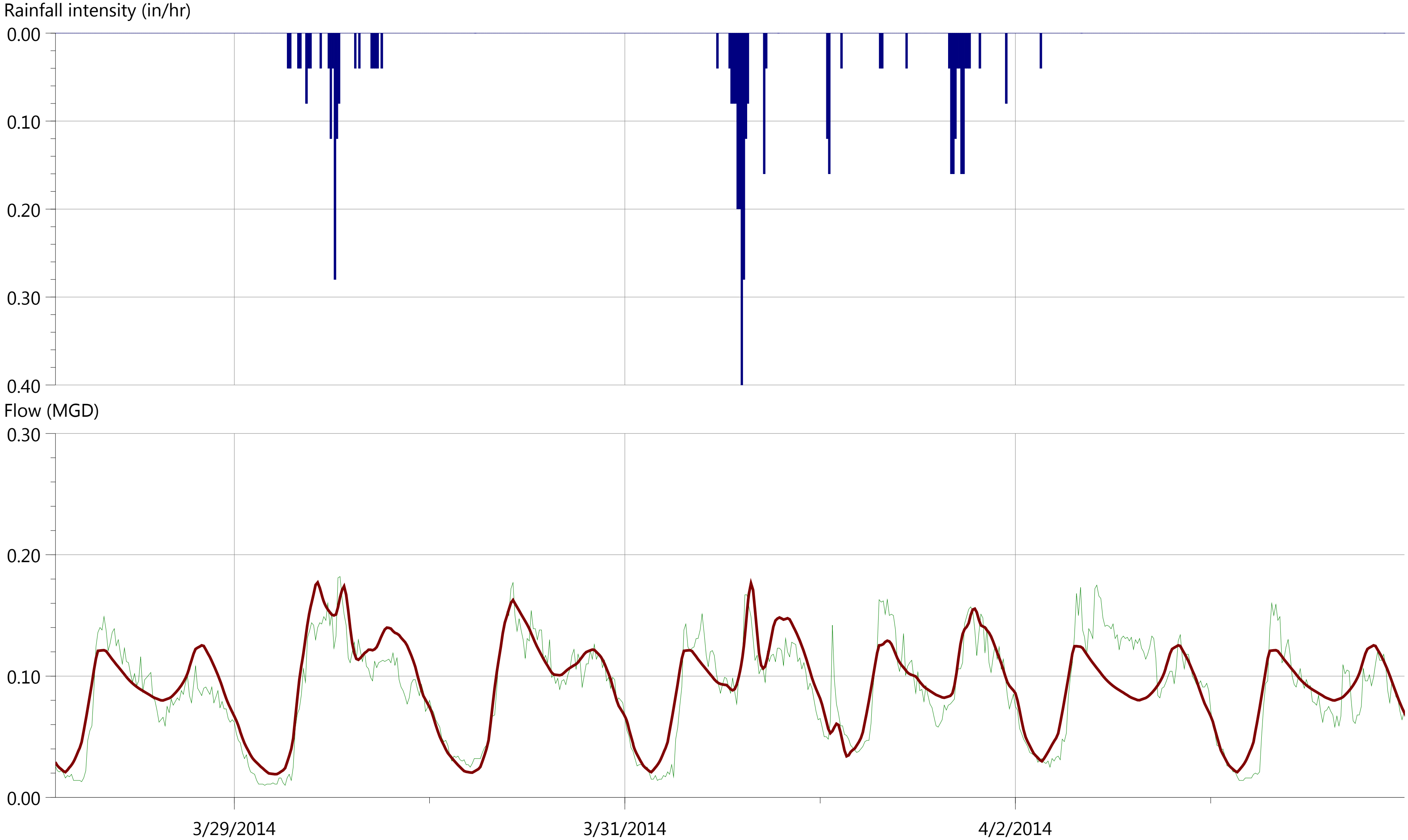
	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.038	0.408	0.785
...04>2014 Calibration				0.025	0.357	0.811

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 12 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 56, Model Location (Pred.) D/S 3991.1, Rainfall Profile: 1



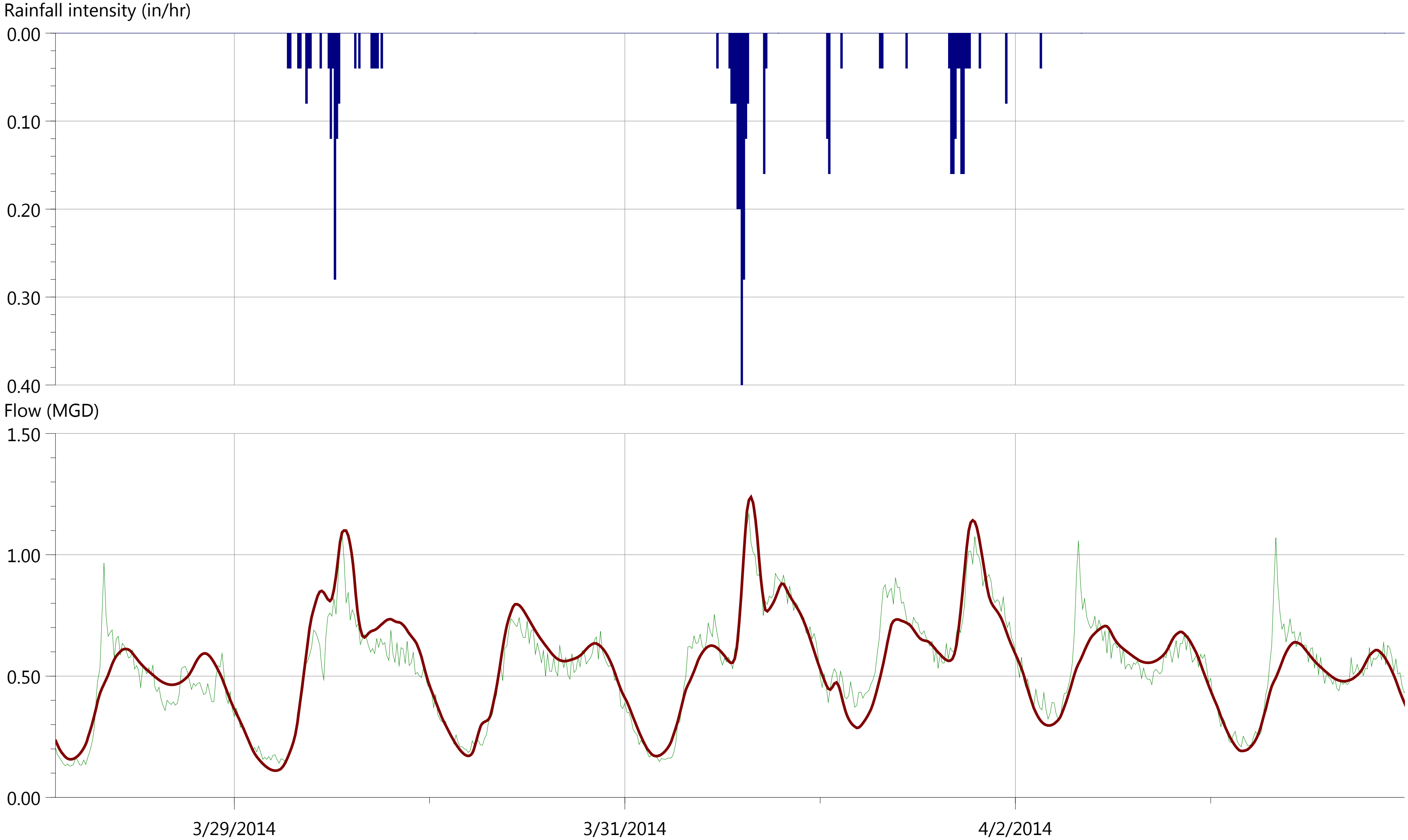
	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.068	0.877	2.276
...04>2014 Calibration				0.054	0.928	1.914

Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 13 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 56A, Model Location (Pred.) D/S 4142.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.010	0.182	0.612
...04>2014 Calibration				0.019	0.177	0.636

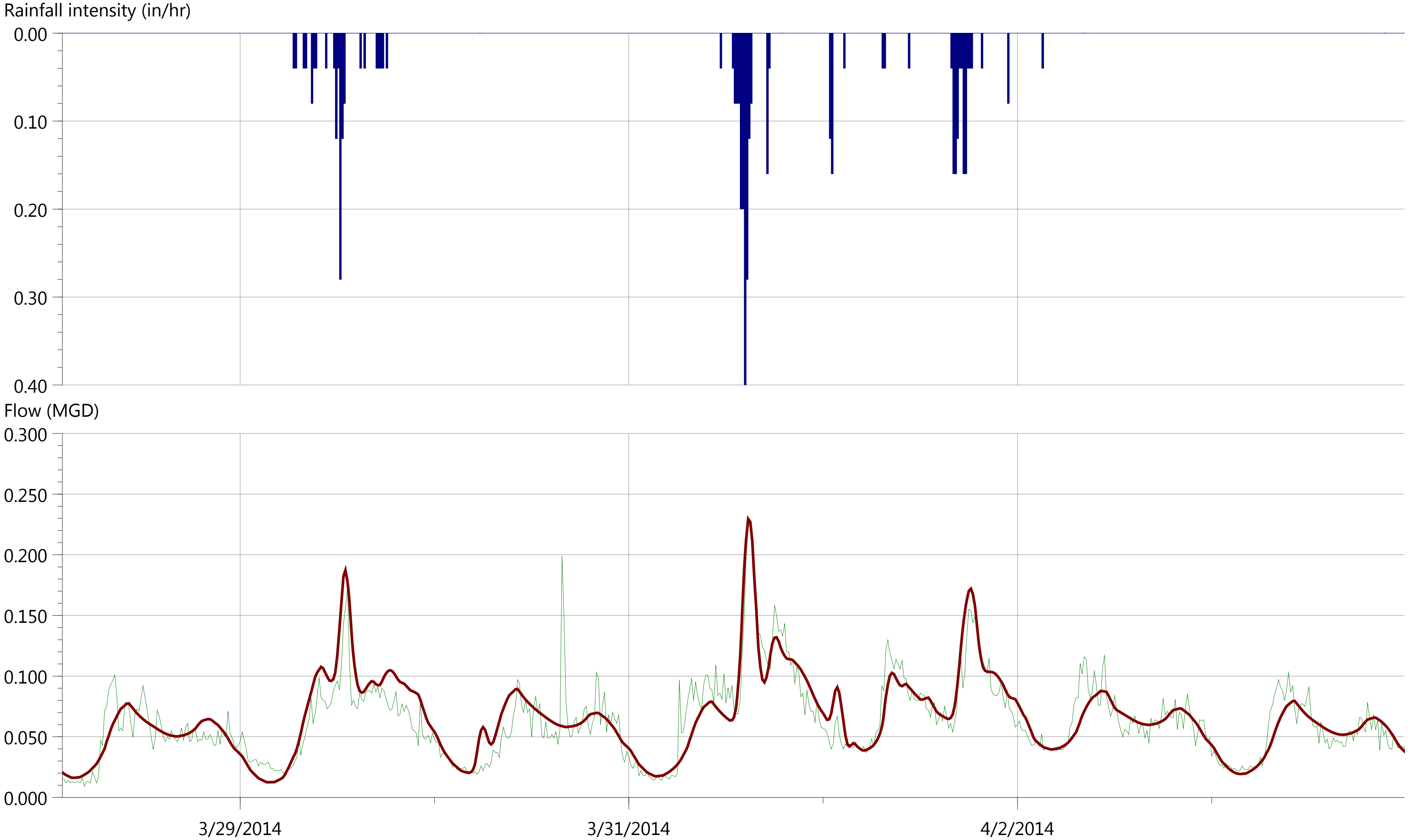
Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 14 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 57, Model Location (Pred.) D/S 6057.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.130	1.169	3.687
...04>2014 Calibration				0.111	1.238	3.704

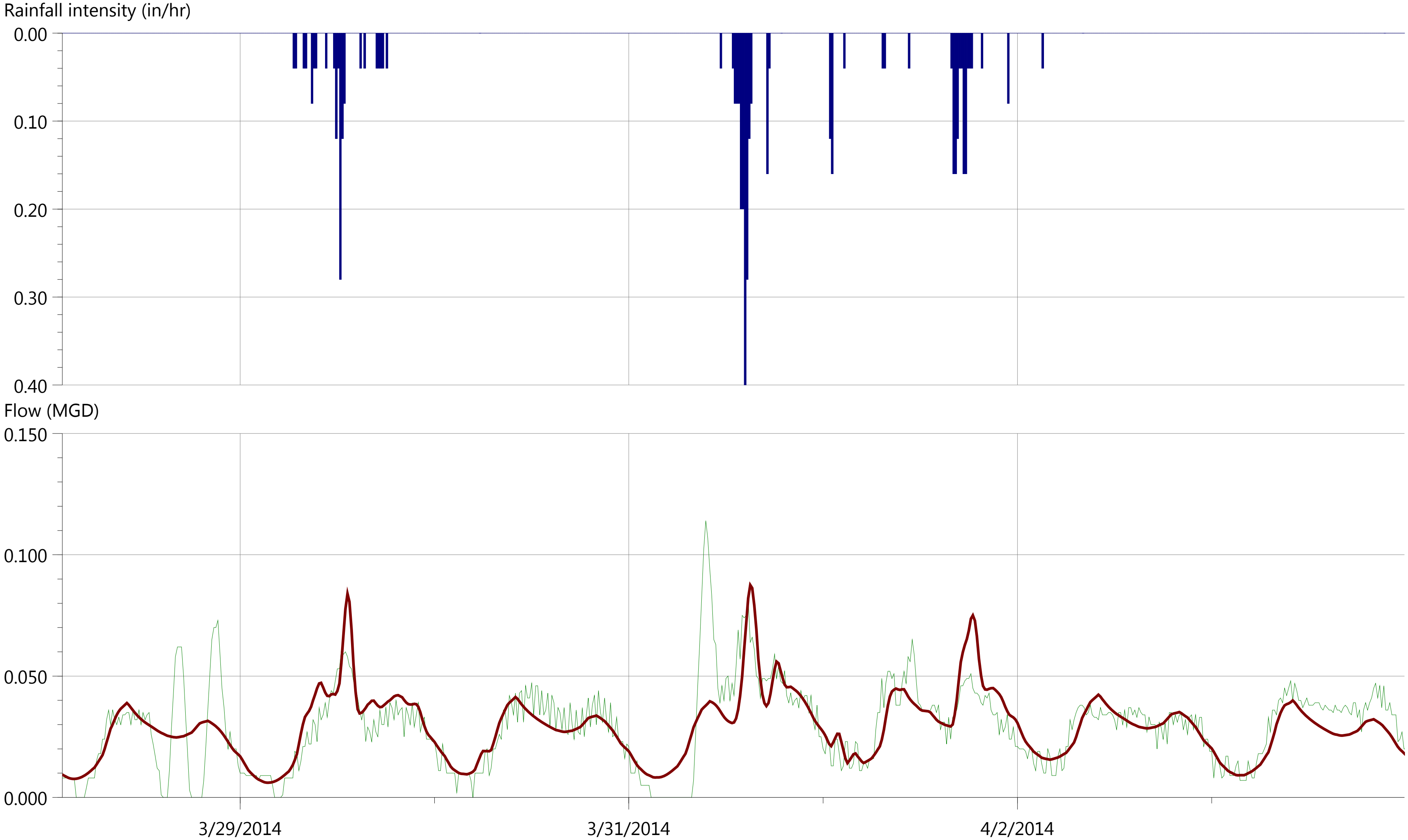
Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 15 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)

Flow Survey Location (Obs.) FM 57A, Model Location (Pred.) D/S 5359.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.009	0.226	0.438
...04>2014 Calibration				0.013	0.230	0.451

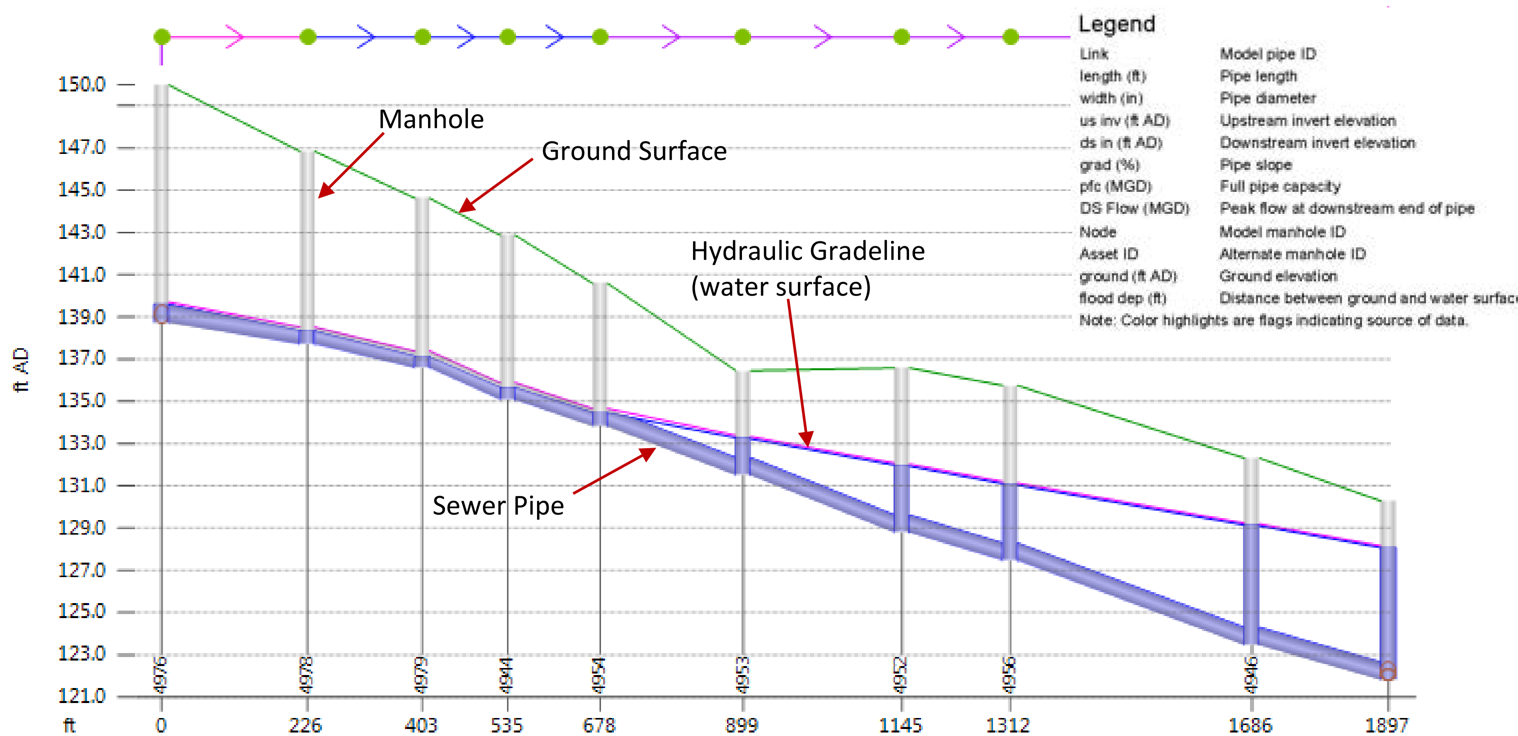
Observed / Predicted Report Produced by cvanlienden (5/22/2015 2:10:15 PM) Page 16 of 16
Flow survey: >Flow Survey Group>Flow survey (8/19/2014 3:48:51 PM)
Sim: >Run Group>WWF Calibration 3/25-4/04>2014 Calibration (5/22/2015 1:37:02 PM)
Flow Survey Location (Obs.) FM 58, Model Location (Pred.) D/S 5810.1, Rainfall Profile: 1



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	1.180	0.400	0.007			
Observed				0.000	0.114	0.202
...04>2014 Calibration				0.006	0.088	0.201

Appendix C - Model Hydraulic Profiles

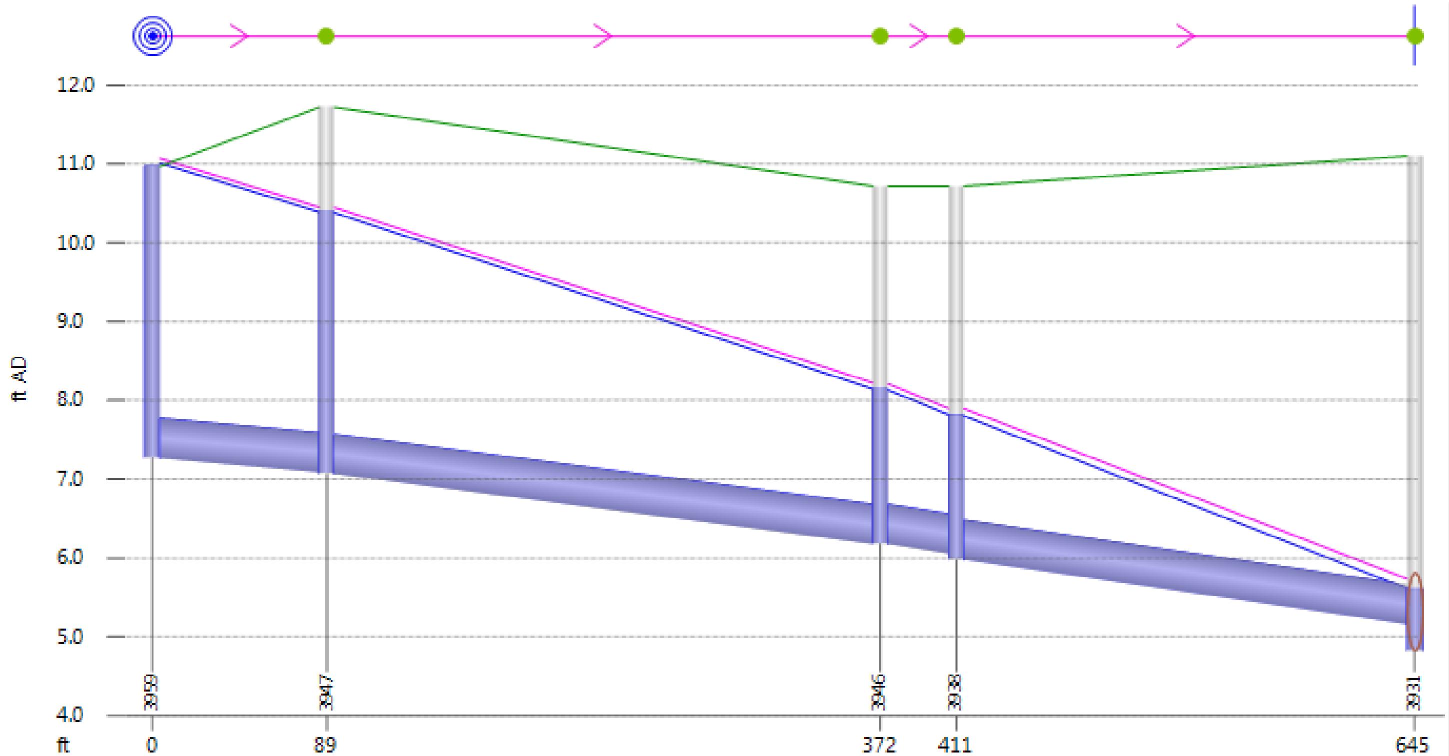
Profile Legend



Link	4976.1	4978.1	4979.1	4944.1	4954.1	4953.1	4952.1	4956.1	4946.1
length (ft)	225.6	177.7	132.2	143.0	221.0	245.5	167.1	373.9	210.6
width (in)	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
us inv (ft AD)	138.730	137.690	136.570	135.060	133.830	131.500	128.780	127.450	123.460
ds inv (ft AD)	137.690	136.570	135.060	133.830	131.500	128.780	127.450	123.460	121.820
grad (%)	0.461	0.630	1.142	0.860	1.054	1.108	0.796	1.067	0.779
pfc (MGD)	0.96	1.12	1.51	1.31	1.45	1.49	1.26	1.46	1.25
DS flow (MGD)	1.0068	1.0067	1.0066	1.0469	1.0444	1.0334	1.0202	0.9789	0.9751

Node	4976	4978	4979	4944	4954	4953	4952	4956	4946	4949
Asset ID	2138	2138	2137		2136	2138	2134	2133	2132	1986
ground (ft AD)	-	146.840	144.620	-	140.580	136.450	136.580	135.750	132.310	-
flood dep (ft)	-	-8.509	-7.518	-7.215	-6.127	-3.175	-4.609	-4.652	-3.168	-

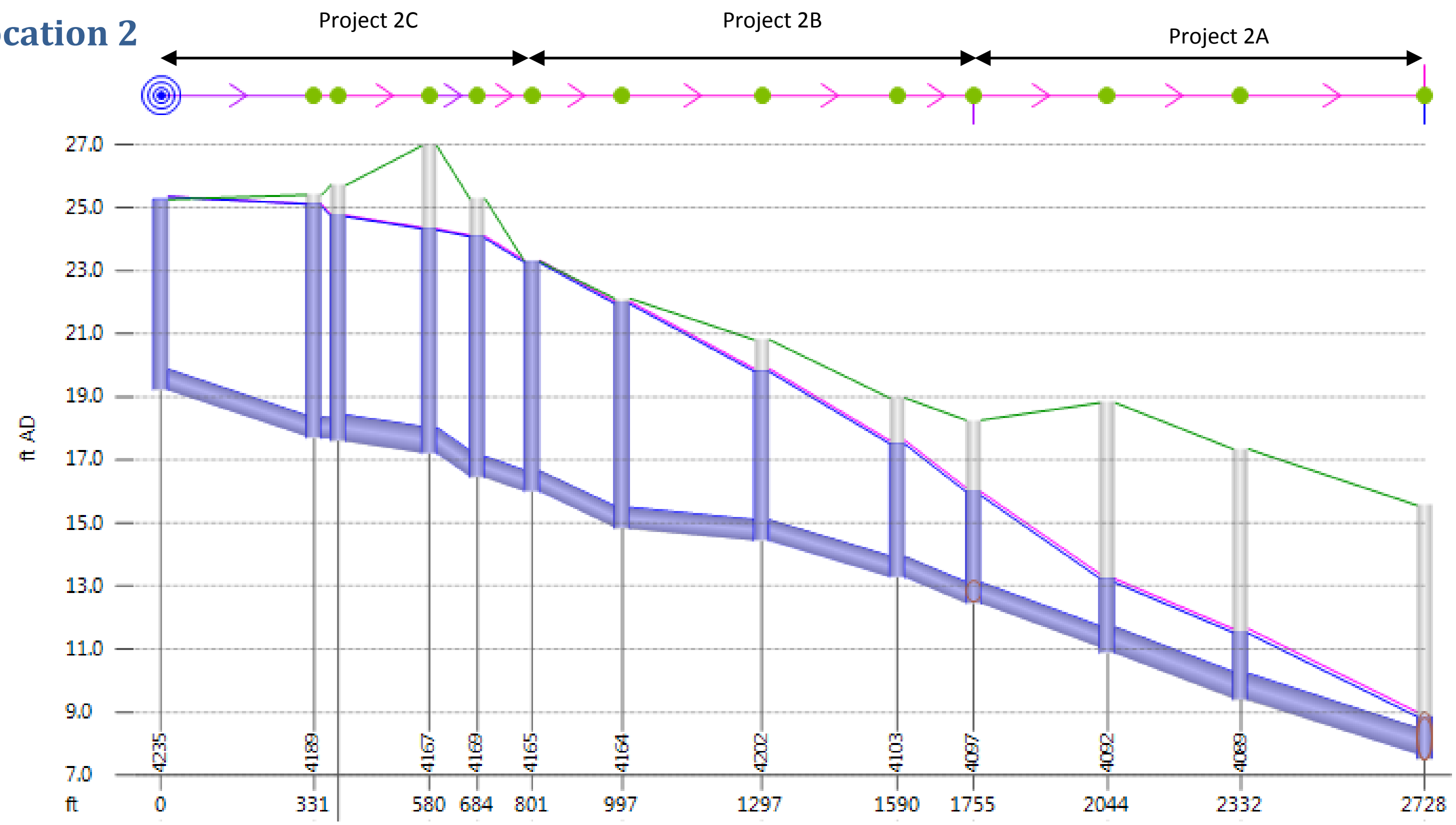
Location 1



Link	3959.1	3947.1	3946.1	3938.1
length (ft)	89.2	282.7	38.8	234.0
width (in)	6.0	6.0	6.0	6.0
us inv (ft AD)	7.270	7.070	6.182	5.980
ds inv (ft AD)	7.090	6.182	6.060	5.160
grad (%)	0.202	0.314	0.314	0.350
pfc (MGD)	0.16	0.20	0.20	0.21
DS flow (MGD)	0.3123	0.3194	0.3209	0.3432

Node	3959	3947	3946	3938	3931
Asset ID	53	52	51	51	50
ground (ft AD)	10.970	11.720	10.720	10.720	11.100
flood dep (ft)	0.000	-1.330	-2.574	-2.901	-5.492

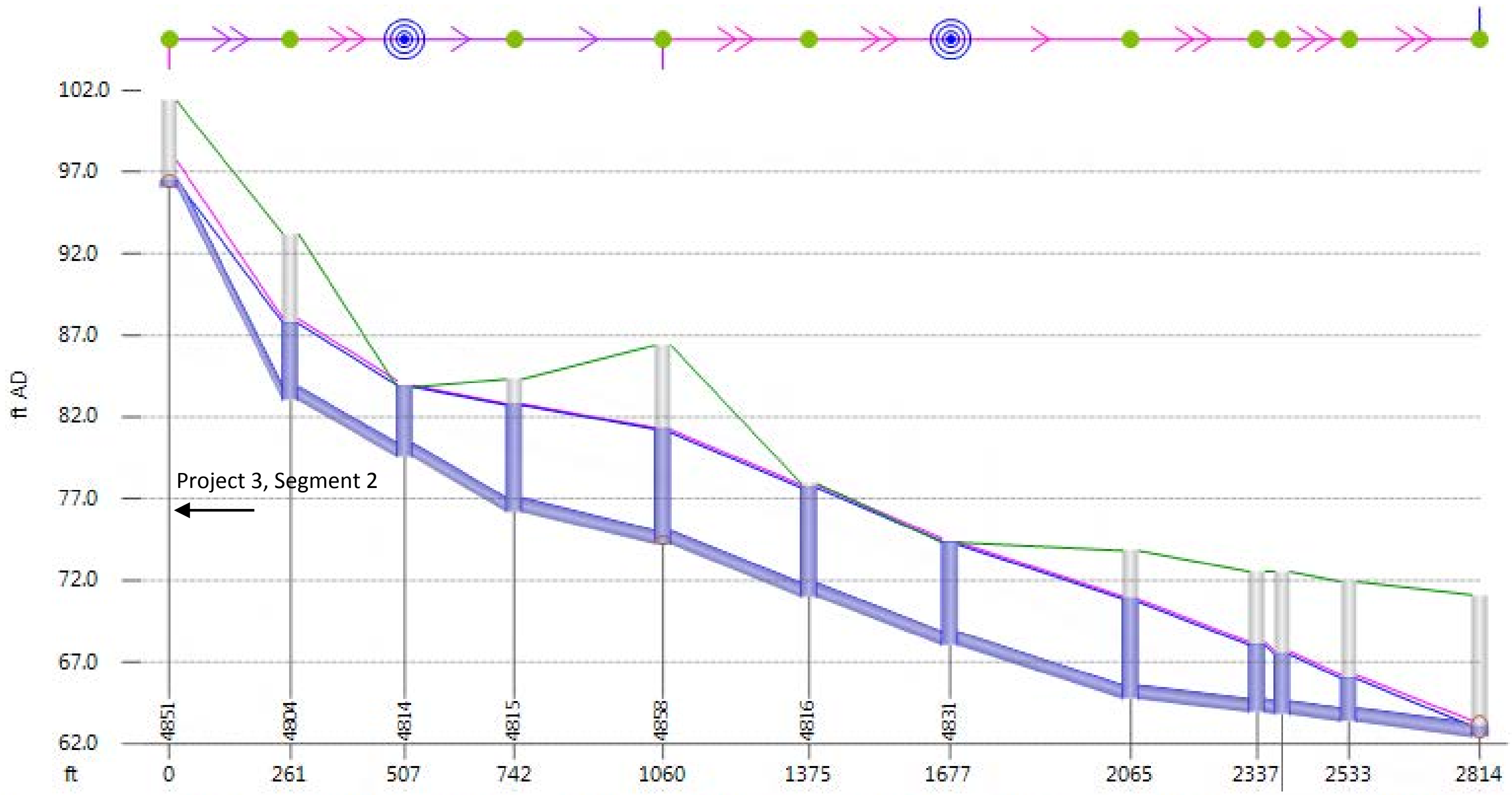
Location 2



Link	4235.1	-	4187.1	-	-	4165.1	4164.1	4202.1	4103.1	4097.1	4092.1	4089.1
length (ft)	331.2	-	196.4	-	-	195.7	300.2	293.0	164.7	289.5	287.4	396.5
width (in)	8.0	-	10.0	10.0	8.0	8.0	8.0	8.0	8.0	8.0	10.0	10.0
us inv (ft AD)	19.200	-	17.590	-	-	15.980	14.820	14.420	13.260	12.440	10.860	9.390
ds inv (ft AD)	17.740	-	17.220	-	-	14.860	14.440	13.280	12.460	11.060	9.440	7.660
grad (%)	0.441	-	0.188	-	-	0.572	0.127	0.389	0.486	0.477	0.494	0.436
pfc (MGD)	0.52	-	0.61	1.19	0.48	0.59	0.28	0.49	0.54	0.54	1.00	0.94
DS flow (MGD)	0.5004	-	0.7472	-	-	0.6647	0.6625	0.6786	0.7346	0.7818	1.0510	1.1720

Node	4235	4189	-	4167	-	4165	4164	4202	4103	4097	4092	4089	4067
Asset ID	1324	1323	-	1321	-	1319	1317	1316	1312	1300	1294	1298	1160
ground (ft AD)	-	25.400	-	-	-	-	22.110	20.810	18.960	18.260	18.810	17.340	15.560
flood dep (ft)	-	-0.290	-	-	-	-	-0.158	-1.001	-1.448	-2.281	-5.587	-5.820	-6.724

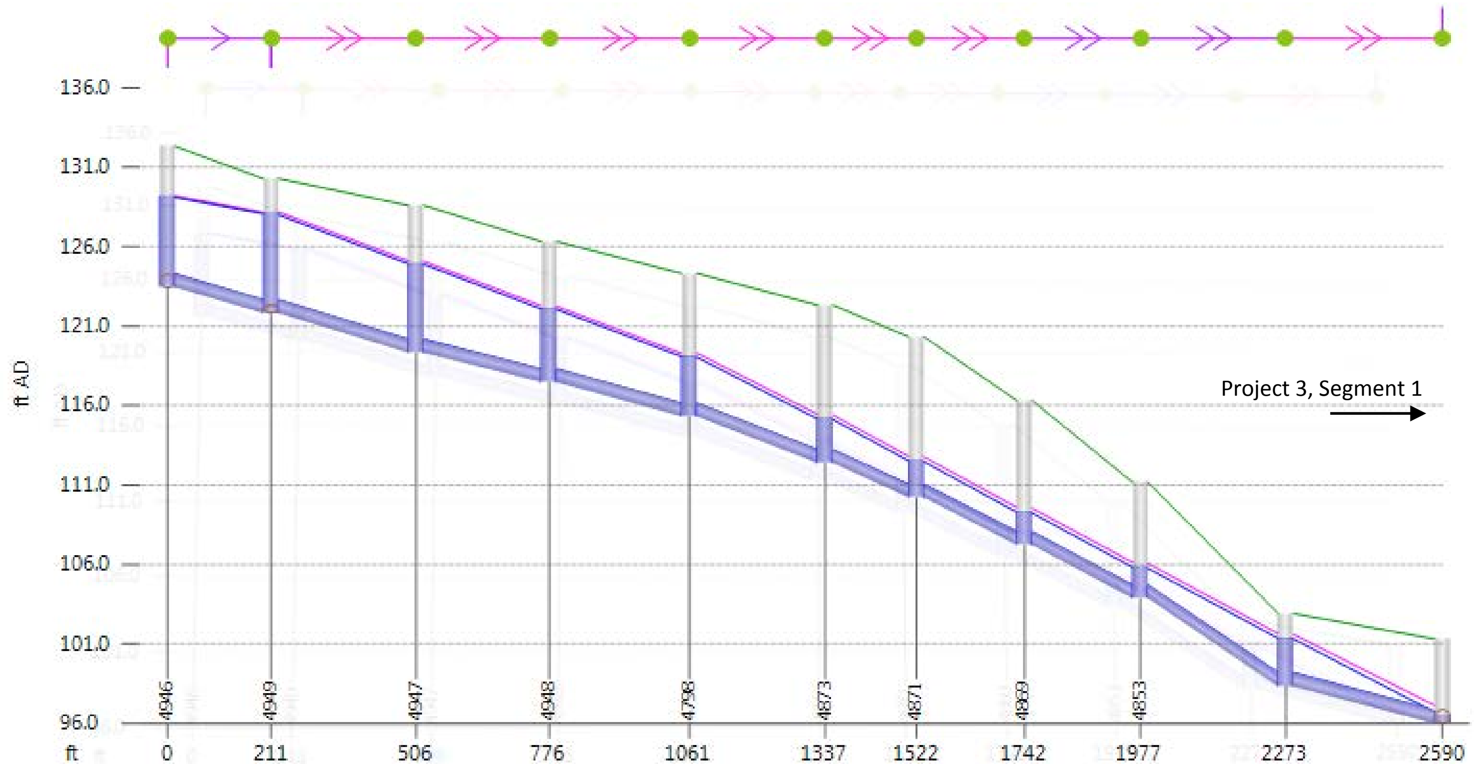
Location 3, Segment 1



Link	4851.1	4804.1	4814.1	4815.1	4858.1	4816.1	4831.1	4838.1	-	-	4834.1
length (ft)	261.2	245.8	234.6	318.9	314.4	302.0	388.2	272.1	-	143.3	280.8
width (in)	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	-	10.0	10.0
us inv (ft AD)	96.000	83.010	79.560	76.200	74.210	71.000	68.010	64.740	-	-	63.350
ds inv (ft AD)	83.010	79.560	76.200	74.210	71.000	68.010	64.740	63.910	-	-	62.420
grad (%)	4.973	1.403	1.432	0.624	1.021	0.990	0.842	0.305	-	0.300	0.331
pfc (MGD)	3.16	1.68	1.70	1.12	1.43	1.41	1.30	0.78	-	0.78	0.82
DS flow (MGD)	1.7213	1.7213	1.0854	1.0942	1.4336	1.4538	1.3253	1.3770	-	-	1.4172

Node	-	4804	4814	4815	4858	4816	4831	4838	4836	-	4834	-
Asset ID	-	1973	1972	1971	1932	1931	1930	1925	1924	-	1920	-
ground (ft AD)	-	93.210	83.810	84.320	86.380	77.920	74.320	73.820	-	-	71.920	-
flood dep (ft)	-	-5.451	0.000	-1.586	-5.139	-0.243	0.000	-2.953	-	-	-5.906	-

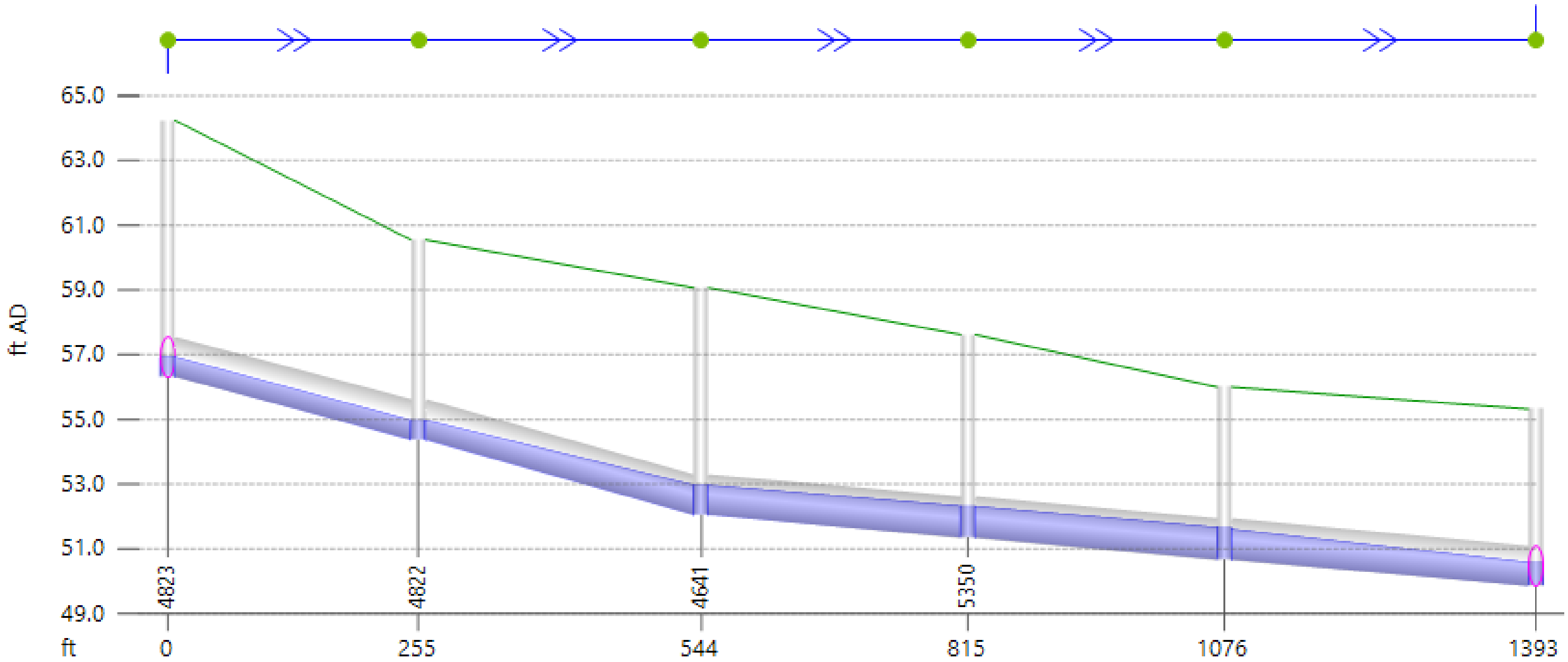
Location 3, Segment 2



Link	4946.1	4949.1	4947.1	4948.1	4798.1	4873.1	4871.1	4869.1	4853.1	4852.1
length (ft)	210.6	295.0	269.9	285.0	276.3	185.1	219.8	235.0	295.9	317.5
width (in)	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
us inv (ft AD)	123.460	121.820	119.340	117.470	115.320	112.390	110.210	107.240	103.880	98.370
ds inv (ft AD)	121.820	119.340	117.470	115.320	112.390	110.210	107.240	103.880	98.370	96.000
grad (%)	0.779	0.841	0.693	0.754	1.061	1.178	1.351	1.430	1.862	0.746
pfc (MGD)	1.25	1.30	1.18	1.23	1.46	1.54	1.65	1.69	1.93	1.22
DS flow (MGD)	0.9751	1.4308	1.4296	1.4310	1.6483	1.6468	1.6688	1.6683	1.6878	1.6877

Node	-	4949	4947	4948	4798	4873	4871	4869	4853	4852	4851
Asset ID	-	1986	1985	1984	1983	1982	1981	1980	1978	1977	1974
ground (ft AD)	-	130.280	128.590	126.320	124.320	122.320	120.320	116.320	111.180	102.920	-
flood dep (ft)	-	-2.208	-3.674	-4.242	-5.240	-7.159	-7.805	-7.032	-5.353	-1.566	-

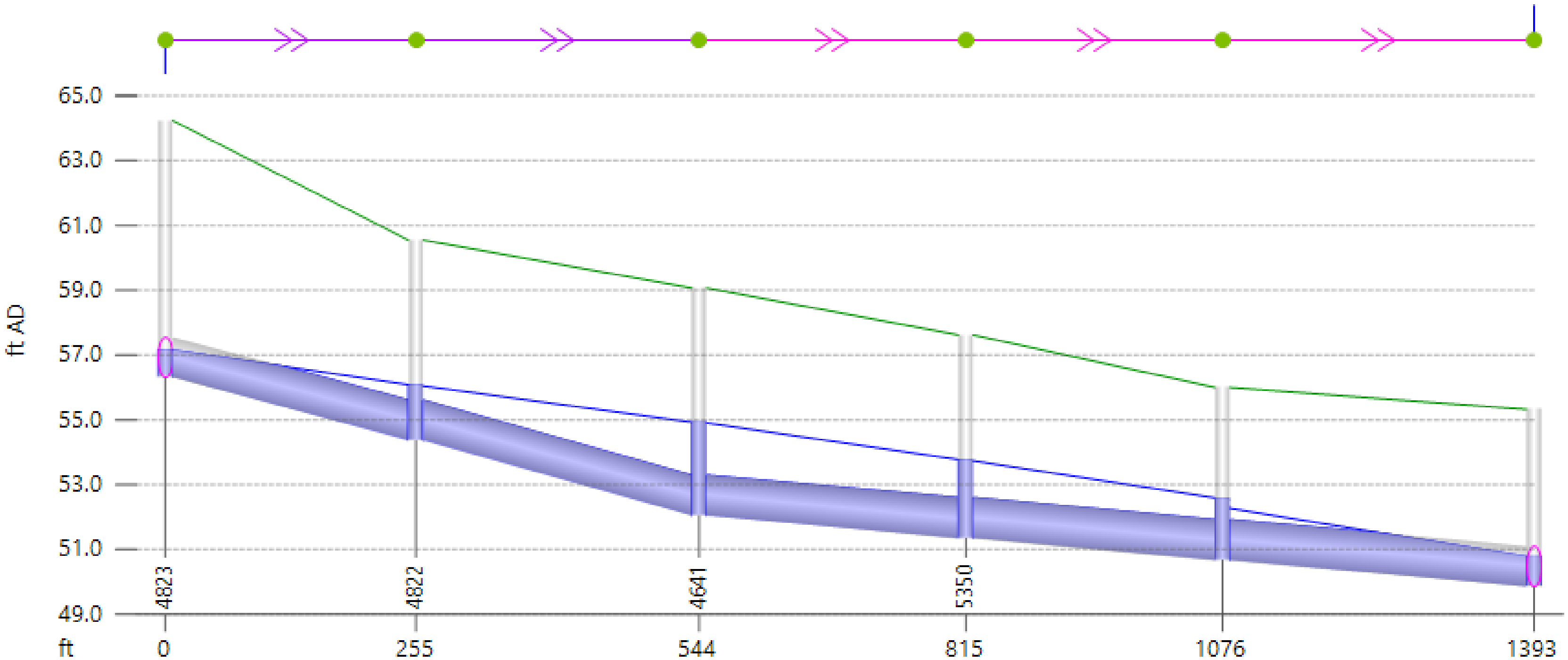
Location 4, before Improvement Project 3 Implementation



Link	4823.1	4822.1	4641.1	5350.1	5348.1
length (ft)	255.1	288.5	271.5	261.2	317.0
width (in)	15.0	15.0	15.0	15.0	15.0
us inv (ft AD)	56.320	54.360	52.040	51.350	50.670
ds inv (ft AD)	54.360	52.040	51.350	50.670	49.860
grad (%)	0.768	0.804	0.254	0.260	0.256
pfc (MGD)	3.66	3.74	2.11	2.13	2.11
DS flow (MGD)	1.6500	1.7206	1.8410	1.9208	1.9280

Node	4823	4822	4641	5350	5348	5351
Asset ID	1911	1908	1905	1902	1901	1900
ground (ft AD)	64.240	60.550	59.070	57.620	56.000	55.320
flood dep (ft)	-7.323	-5.586	-6.112	-5.312	-4.352	-4.751

Location 4, after Improvement Project 3 Implementation



Link	4823.1	4822.1	4641.1	5350.1	5348.1
length (ft)	255.1	288.5	271.5	261.2	317.0
width (in)	15.0	15.0	15.0	15.0	15.0
us inv (ft AD)	56.320	54.360	52.040	51.350	50.670
ds inv (ft AD)	54.360	52.040	51.350	50.670	49.860
grad (%)	0.768	0.804	0.254	0.260	0.256
pfc (MGD)	3.66	3.74	2.11	2.13	2.11
DS flow (MGD)	2.5220	2.5807	2.6834	2.7512	2.7572

Node	4823	4822	4641	5350	5348	5351
Asset ID	1911	1908	1905	1902	1901	1900
ground (ft AD)	64.240	60.550	59.070	57.620	56.000	55.320
flood dep (ft)	-7.074	-4.470	-4.143	-3.852	-3.441	-4.545

Appendix D - Capital Improvement Project Details

Capacity Project 1: Hoover Street Easement

Date Saved: 5/13/2015

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Fair Oaks Sewer Maintenance District

Sewer System Capacity Analysis

Project 1: Hoover Street Easement

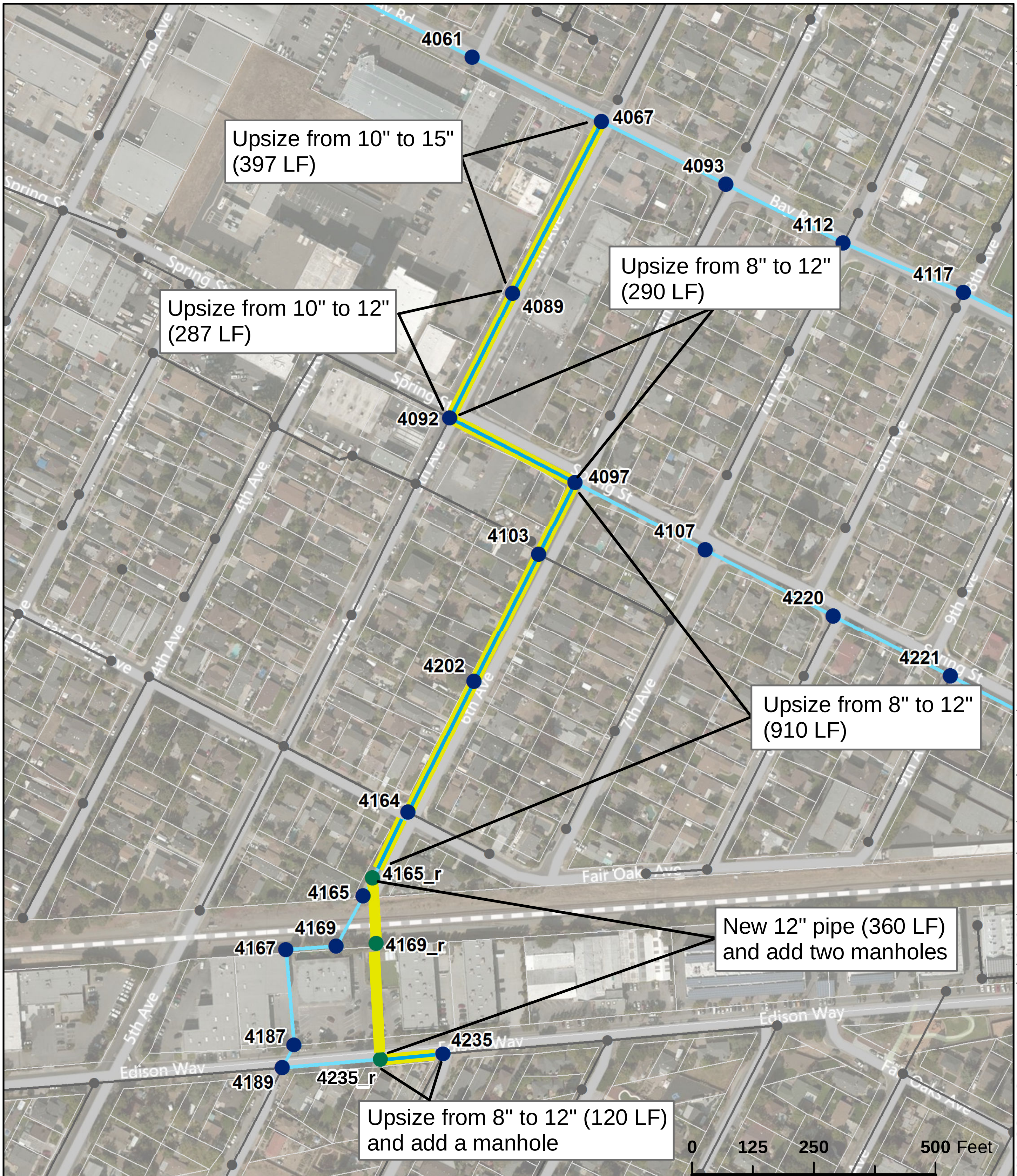
PROJECT DESCRIPTION	
Project ID	1
Project Name	Hoover Street Easement
Project Location	Easement between Hoover St and Rolison Rd east of Haven Ave
Description	Replace approximately 600 feet of 6-in pipe with 10-in pipe
Estimated Capital Improvement Cost	$274,000
Comments	(i) Pipes are listed in order from upstream to downstream
Assumptions	(i) Cost assumes pipe will be upsized using open cut, however pipe bursting (with necessary pavement repair) should be evaluated during the design phase (ii) Cost estimates are based on February 2015 ENR CCI of 11178
Alternatives	(i) Install parallel pipe

PROJECT COST DETAIL

U/S MH ID	D/S MH ID	Existing Diameter (inches)	New Diameter (inches)	Length (feet)	Slope (%)	Pipe Depth (feet BGL)	Construction Method	Unit Cost ($/LF)	Total Cost ($)
3959	3947	6	10	89	0.20	4	Open Cut	$196	$ 17,483
3947	3946	6	10	283	0.31	6	Open Cut	$196	$ 55,409
3946	3938	6	10	39	0.31	6	Open Cut	$196	$ 7,605
3938	3931	6	10	234	0.35	5	Open Cut	$196	$ 45,864

Total Baseline Pipe Construction Cost	$ 126,361
Lateral Reconnection, Total of 20	$ 10,000
Baseline Construction Cost:	**$ 136,361**
Bypass Pumping (Based on pipe length)	$ 10,500
Remove & Replace Factor (5% of pipe construction cost)	$ 6,318
Subtotal:	**$ 153,179**
Mobilization/Demobilization (10% of subtotal, $10k min.)	$ 15,318
Estimated Construction Cost Subtotal:	**$ 168,497**
Contingencies (30% of construction subtotal)	$ 50,549
Total Estimated Construction Cost:	**$ 219,046**
Engineering, Administration, Legal (25% of construction cost)	$ 54,762
Estimated Capital Improvement Cost:	**$ 274,000**

Capacity Project 2: Edison Way to Bay Road



Date Saved: 5/7/2015

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Fair Oaks Sewer Maintenance District
Sewer System Capacity Analysis

Service Layer Credits: Image courtesy of USGS © 2015

Project 2: Edison Way to Bay Road

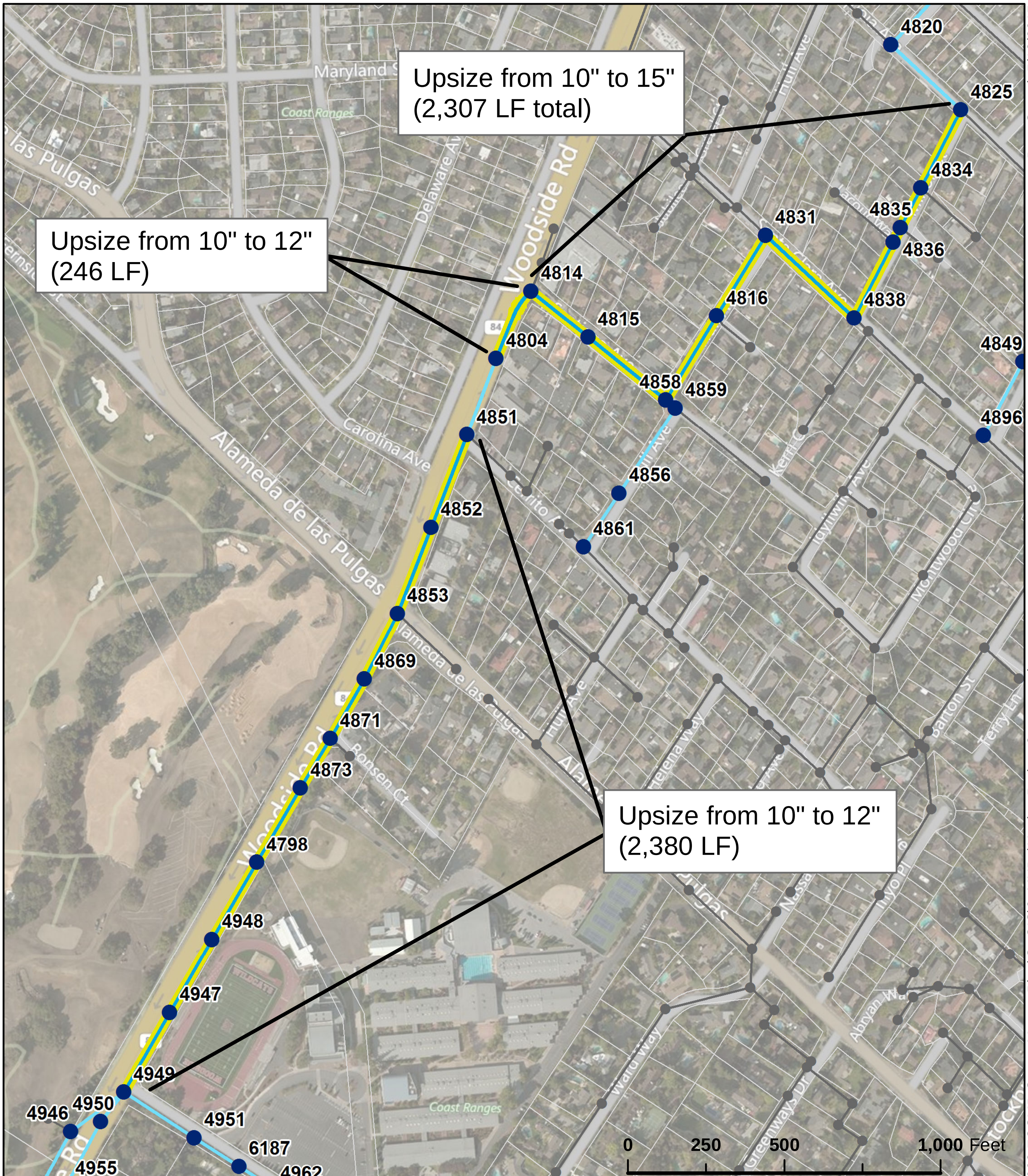
PROJECT DESCRIPTION	
Project ID	2
Project Name	Edison Way to Bay Road
Project Location	Bay Rd and 5th Ave to Edison Way and 7th Ave along 6th Ave
Description	Replace approximately 2,000 ft of 8" and 10" pipe with 12" and 15" pipe and install about 360 ft of 12" pipe, including a new pipeline under a railroad.
Estimated Capital Improvement Cost	$1,270,000
Comments	(i) Pipes are listed in order from upstream to downstream
	(ii) Project includes a new 12-inch pipe crossing railroad ROW
Assumptions	(i) Cost assumes most pipe will be upsized using pipe burst
	(ii) Cost estimates are based on February 2015 ENR CCI of 11178
Alternatives	(i) Install parallel pipe
	(ii) Could upsize pipeline from MH 4235 to MH 4169 (8" to 12") instead of installing new pipe from MH 4235_r to MH 4169_r
	(iii) Could installed a new pipeline from MH 4097 to MH 4093 instead of upsizing pipeline from MH 4097 to MH 4069

PROJECT COST DETAIL

U/S MH ID	D/S MH ID	Existing Diameter (inches)	New Diameter (inches)	Length (feet)	Slope (%)	Pipe Depth (feet BGL)	Construction Method	Unit Cost ($/LF)	Total Cost ($)
4235	4235_r	8	12	120	0.44	7	Pipe Burst	$116	$ 13,944
4235_r	4169_r	NEW	12	235			Open Cut	$210	$ 49,350
4169_r	4165_r	NEW	12	100			Microtunnel	$1,050	$ 105,000
4165_r	4164	8	12	152	0.57	7	Pipe Burst	$116	$ 17,662
4164	4202	8	12	300	0.13	7	Pipe Burst	$116	$ 34,883
4202	4103	8	12	293	0.39	6	Pipe Burst	$116	$ 34,047
4103	4097	8	12	165	0.49	6	Pipe Burst	$116	$ 19,138
4097	4092	8	12	290	0.48	7	Pipe Burst	$116	$ 33,640
4092	4089	10	12	287	0.49	8	Pipe Burst	$116	$ 33,396
4089	4067	10	15	397	0.44	8	Pipe Burst	$139	$ 54,955

Total Baseline Pipe Construction Cost	$	396,015
Jacking Pit	$	80,000
Receiving Pit	$	50,000
Insertion Trenches, Total of 5	$	25,000
Lateral Reconnection, Total of 44	$	22,000
Manhole Rehabilitation, Total of 7	$	17,500
New Manholes, Total of 3	$	36,000
Baseline Construction Cost:	**$**	**626,515**
Bypass Pumping (Based on pipe length)	$	25,800
Remove & Replace Factor (5% of pipe construction cost)	$	-
Traffic Control (10% of pipe construction cost for basic control plus additional 10% for complex)	$	26,667
Subtotal:	**$**	**678,982**
Mobilization/Demobilization (10% of subtotal)	$	67,898
Estimated Construction Cost Subtotal:	**$**	**746,880**
Contingencies (30% of construction subtotal)	$	224,064
Total Estimated Construction Cost:	**$**	**970,944**
Permanent ROW/Easement Acquisition ($8/SF)	$	56,400
Engineering, Administration, Legal (25% of construction cost)	$	242,736
Estimated Capital Improvement Cost:	**$**	**1,270,000**

Capacity Project 3: Woodside Road to Sequoia Avenue



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Fair Oaks Sewer Maintenance District
Sewer System Capacity Analysis

COUNTY OF SAN MATEO FOUNDED 1856

RMC water and environment

Service Layer Credits: Image courtesy of USGS © 2015

Project 3: Woodside Road to Sequoia Avenue

PROJECT DESCRIPTION

Project ID	3
Project Name	Woodside Road to Sequoia Avenue
Project Location	Woodside Rd near Churchill Ave to Sequoia Ave and Milton St, along Hull Ave, Santa Clara Ave and Milton St
Description	Replace approximately 4,900 feet of 10-in pipe with 12-in to15-in pipe
Scenario	Base
Estimated Capital Improvement Cost	$1,891,000
Comments	(i) Pipes are listed in order from upstream to downstream (ii) Additional 10% (total of 20%) cost factor added to traffic control cost due to high traffic on Woodside Rd
Assumptions	(i) Cost assumes pipe will be upsized using pipe burst except from Churchill to and including SFPUC crossing. (ii) Cost estimates are based on February 2015 ENR CCI of 11178
Alternatives	(i) Install parallel pipe

PROJECT COST DETAIL

U/S MH ID	D/S MH ID	Existing Diameter (inches)	New Diameter (inches)	Length (feet)	Slope (%)	Pipe Depth (feet BGL)	Construction Method	Unit Cost ($/LF)	Total Cost ($)
4949	4947	10	12	295	0.84	9	Open Cut	$210	$ 61,950
4947	4948	10	12	270	0.69	9	Open Cut	$210	$ 56,679
4948	4798	10	12	285	0.75	9	Open Cut	$210	$ 59,850
4798	4873	10	12	276	1.06	9	Open Cut	$210	$ 58,023
4873	4871	10	12	185	1.18	10	Pipe Burst	$116	$ 21,509
4871	4869	10	12	220	1.35	10	Pipe Burst	$116	$ 25,541
4869	4853	10	12	235	1.43	8	Pipe Burst	$116	$ 27,307
4853	4852	10	12	296	1.86	6	Pipe Burst	$116	$ 34,384
4852	4851	10	12	318	0.75	5	Pipe Burst	$116	$ 36,894
4804	4814	10	12	246	1.40	7	Pipe Burst	$116	$ 28,562
4814	4815	10	15	235	1.43	6	Pipe Burst	$139	$ 32,516
4815	4858	10	15	319	0.62	10	Pipe Burst	$139	$ 44,200
4858	4816	10	15	314	1.02	10	Pipe Burst	$139	$ 43,576
4816	4831	10	15	302	0.99	7	Pipe Burst	$139	$ 41,857
4831	4838	10	15	388	0.84	8	Pipe Burst	$139	$ 53,805
4838	4836	10	15	272	0.31	9	Pipe Burst	$139	$ 37,713
4836	4835	10	15	53	0.25	9	Pipe Burst	$139	$ 7,318
4835	4834	10	15	143	0.30	9	Pipe Burst	$139	$ 19,861
4834	4825	10	15	281	0.33	9	Pipe Burst	$139	$ 38,919

Total Baseline Pipe Construction Cost	$	730,461
Insertion Trenches, Total of 9	$	45,000
Lateral Reconnection, Total of 72	$	36,000
Manole Rehabilitation, Total of 16	$	40,000
Baseline Construction Cost:	**$**	**851,461**
Bypass Pumping (Based on pipe length)	$	73,700
Remove & Replace Factor (5% of pipe construction cost)	$	11,825
Traffic Control (10% of pipe construction cost for basic control plus additional 10% for complex)	$	120,866
Subtotal:	**$**	**1,057,852**
Mobilization/Demobilization (10% of subtotal)	$	105,785
Estimated Construction Cost Subtotal:	**$**	**1,163,638**
Contingencies (30% of construction subtotal)	$	349,091
Total Estimated Construction Cost:	**$**	**1,512,729**
Engineering, Administration, Legal (25% of construction cost)	$	378,182
Estimated Capital Improvement Cost:	**$**	**1,891,000**

Capacity Project 4: Himmel Avenue

Fair Oaks Sewer Maintenance District

Sewer System Capacity Analysis

Date Saved: 9/10/2015

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Project 4: Himmel Avenue

PROJECT DESCRIPTION	
Project ID	4
Project Name	Himmel Avenue
Project Location	Himmel Ave from Rutherford Ave to Nimitz Ave, along Nimitz Ave to Selby Ln.
Description	Replace approximately 1,200 feet of 15-in pipe with 18-in pipe
Scenario	Base
Estimated Capital Improvement Cost	$626,000
Comments	(i) Pipes are listed in order from upstream to downstream
	(ii) Project 4 is needed after the implementation of Project 3
	(iii) The pipe segment between MH 5351 and MH 5353 does not need replacement for capacity reasons. However, since the pipe downstream of MH 5353 was replaced as part of project 1/4622, it is recommended that the pipe between MH 5351 and MH 5353 be replaced for continuity purposes.
Assumptions	(i) Cost assumes pipe will be upsized using open cut
	(ii) Cost estimates are based on February 2015 ENR CCI of 11178
Alternatives	(i) Install parallel pipe

PROJECT COST DETAIL

U/S MH ID	D/S MH ID	Existing Diameter (inches)	New Diameter (inches)	Length (feet)	Slope (%)	Pipe Depth (feet BGL)	Construction Method	Unit Cost ($/LF)	Total Cost ($)
4641	5350	15	18	272	0.25	7	Open Cut	$243	$ 65,975
5350	5348	15	18	261	0.26	6	Open Cut	$243	$ 63,472
5348	5351	15	18	317	0.26	5	Open Cut	$243	$ 77,031
5351	5353	15	18	307	0.60	8	Open Cut	$243	$ 74,625

Total Baseline Pipe Construction Cost	$	281,102
Lateral Reconnection, Total of 26	$	13,000
Baseline Construction Cost:	**$**	**294,102**
Bypass Pumping (Based on pipe length)	$	14,000
Remove & Replace Factor (5% of pipe construction cost)	$	14,055
Traffic Control (10% of pipe construction cost for basic control plus additional 10% for complex)	$	28,110
Subtotal:	**$**	**350,268**
Mobilization/Demobilization (10% of subtotal)	$	35,027
Estimated Construction Cost Subtotal:	**$**	**385,295**
Contingencies (30% of construction subtotal)	$	115,588
Total Estimated Construction Cost:	**$**	**500,883**
Engineering, Administration, Legal (25% of construction cost)	$	125,221
Estimated Capital Improvement Cost:	**$**	**626,000**