Ross Valley Sanitary District

Final Report

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

August 2006

MWH

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BWF	base wastewater flow
CCI	construction cost index
CIP	Capital Improvement Program
CMSA	Central Marin Sanitation Agency
d/D	depth to diameter ratio
District	Ross Valley Sanitary District
DWF	dry weather flow
ENR	Engineering News-Record
FM	flow meter
gpcd	gallons per capita day
gpd	gallons per day
GWI	groundwater infiltration
I/I	infiltration and inflow
IDF	intensity-duration-frequency
MGD	million gallons per day
NOAA	National Oceanic and Atmospheric Administration
NRCS	National Resources Conservation Service
RDI/I	rainfall dependent infiltration and inflow
RVSD	Ross Valley Sanitary District
RWQCB	San Francisco Bay Regional Water Quality Control Board
SCS	U.S. Soil Conservation Service
SHECAP	Sanitary Sewer System Hydraulic Evaluation and Capacity Assurance Plan
SSMP	Sewer System Management Plan
SSO	Sanitary Sewer Overflow
WWTP	wastewater treatment plant

Executive Summary

This executive summary presents the results of the Sanitary Sewer System Hydraulic Evaluation and Capacity Assurance Plan (SHECAP) study for Sanitary District No. 1 of Marin County, also known as the Ross Valley Sanitary District (RVSD or District). This study was prepared by MWH under an agreement with the District dated September 17, 2004.

The purpose of this SHECAP study is to evaluate the hydraulic performance of the existing sewer system, identify capacity deficiencies in the system, and recommend potential sewer improvement projects to correct the identified problems. The recommended projects will be incorporated into the District's 5-year and long-term Capital Improvement Plans. The study will also serve to fulfill the requirements of the Capacity Management element of the District's Sewer System Management Plan (SSMP), which is required by the San Francisco Bay Regional Water Quality Control Board (RWQCB) pursuant to Section 13267 of the California Water Code. A similar requirement will be included in future statewide Waste Discharge Requirements expected to be issued in 2006 to all collection system agencies in California by the State Water Resources Control Board. The objective of the Regional and State SSMP requirements is to reduce existing and potential sanitary sewer overflows (SSOs) from wastewater collection systems.

SERVICE AREA AND SEWER SYSTEM

The District serves the towns of Fairfax, San Anselmo, and Ross; the City of Larkspur; and the unincorporated areas known as Sleepy Hollow, Kentfield, Kent Woodlands, Oak Manor, and Greenbrae (see **Figure ES-1**). The District owns, operates and maintains approximately 186 miles of collection sewer lines, 7 miles of force mains, and 20 pumping stations which collect and transport an average of approximately 5 million gallons per day (MGD) of wastewater to Central Marin Sanitation Agency (CMSA) for treatment and disposal. **Figure ES-1** shows the District's collection system, including the major (trunk) sewers and pump stations that were included in the collection system hydraulic evaluation.

MODEL DEVELOPMENT

To analyze the capacity requirements of the collection system, a computer model was developed of the trunk sewer network. The modeled sewer network for the SHECAP study consists of the gravity trunk system (including primarily all 10-inch and greater diameter pipes) plus the seven major pump stations and their respective force mains that convey flow to the CMSA wastewater treatment plant (WWTP).

Data for the model were developed based on the District's AutoCAD sewer mapping and a field survey which determined manhole coordinates, rim elevations, and pipe invert elevations at approximately 500 manholes in the system. Wastewater flows were estimated based on population data obtained from U.S. Census information, customer and water use data from the District's sewer billing records, and flows measured at 20 flow meters during a temporary flow monitoring program conducted for this study during the 2004-05 winter season. The flows were used to develop and calibrate the hydraulic model.

CAPACITY EVALUATION

Based on historical experience, and as confirmed by the flow monitoring data, it is clear that rainfall greatly increases peak flows in the District's sewer collection system due to infiltration and inflow (I/I) of stormwater and groundwater into the system. Therefore, the capacity of the system was evaluated with respect to a design storm event. Based on discussions with District staff and consistent with criteria used by other Bay Area agencies, a 5-year frequency rainfall event was selected as the basis for evaluation and sizing of trunk system facilities for this study.

The hydraulic model was used to evaluate collection system performance. Evaluation criteria, defined based on the level of surcharge (i.e., flow depth above the top of the pipe) that would be considered acceptable, were developed and used to identify problem areas. The trunk system was evaluated under both dry weather and design storm scenarios. Based on the model runs, the peak flow condition was determined for each pipe, and locations with deficiencies were identified.

No capacity deficiencies were identified for dry weather conditions, but significant portions of the system were found to have inadequate capacity for wet weather flows. **Figure ES-2** shows the existing capacity deficiencies and potential overflow locations under the design storm event. In the figures, the "Surcharge due to Capacity" deficiency includes pipes that were surcharged in the model due to insufficient pipe capacity. The "Surcharge due to Backup" deficiency indicates pipes surcharged with backwater due to downstream capacity deficiencies. Only pipes with a surcharged water level within 10 feet of the manhole rim are shown as a deficiency.

The model results indicate that the system would surcharge significantly under a design wet weather event and potentially result in overflows in many locations in the District if the sewer system remains in its current condition. The model results were reviewed with District staff, who confirmed that many of the locations indicated in the model as being potential overflow points have, in fact, experienced overflows or surcharge during wet weather.

Pump Station and Force Main Capacity

The capacities of the pump stations and force mains were also analyzed under the design storm scenario. All of the pump stations except for Bon Air have sufficient capacity to handle the predicted 5-year design storm peak wet weather flows under normal pump operation. However, three of the pump stations (Bon Air, Larkspur Main, and Kentfield) may not have sufficient capacity to handle peak storm flow with the largest pump out of service. (The deficiency at the Kentfield Pump Station could be addressed by force main improvements, as discussed below.) The need for improvements at the pump stations is being evaluated as part of the Pump Station Replacement Master Plan in the District's ongoing Sewer System Assessment and Capital Project Planning program.

The only identified capacity deficiency in the force main system was in the 36-in pipeline between the Kentfield Pump Station and the end of South Eliseo Drive near Lower Via Casitas. The size of the 36-inch force main limits the ability of the Kentfield Pump Station to pump to its design capacity. The entire force main would need to be replaced with a 39-inch diameter pipe to provide adequate capacity. Alternately, only a portion of the force main, e.g., the section located along Eliseo Drive, could be replaced with a 42-inch pipe to provide the needed capacity, and the existing 36-inch upstream portion could remain (or be rehabilitated by lining). The replacement of this force main is being evaluated as part of the Force Main Replacement Master Plan in the District's ongoing Sewer System Assessment and Capital Project Planning program.

CAPACITY IMPROVEMENTS

Potential solutions to the predicted wet weather overflow problems fall into two basic categories: providing additional hydraulic capacity in the system (“relief solutions”), or repairing sewers to reduce I/I flows (“rehabilitation solutions”). Relief solutions involve replacement of existing sewers with larger pipes and/or construction of parallel relief sewers. Rehabilitation solutions involve identifying and eliminating direct inflow connections (if they exist), replacing or lining existing sewer lines, repairing manholes, and replacing or repairing service laterals (possibly including the portions located on private property).

Sewer rehabilitation is the ideal solution to high I/I flows because it corrects the actual cause of the problem. However, rehabilitation is generally only cost effective if relief solutions are prohibitively expensive and rehabilitation can be implemented at a reasonable cost but still achieve sufficient reductions in I/I to eliminate the need for relief facilities. For the District, the cost of rehabilitating the existing sewer system to provide capacity relief within a reasonable period of time would be very high. A gross-level analysis to assess the potential cost-effectiveness of sewer system rehabilitation to reduce I/I in areas upstream of identified capacity deficiencies determined that those costs would far exceed the costs for providing trunk sewer capacity improvements.

Therefore, to address the hydraulic problems in the system, relief solutions were developed. A model of the potential relief solutions was developed to size the relief facilities and verify that they would alleviate all of the predicted overflows and most of the surcharge in the system. Based on the results of the solutions modeling, specific capacity relief projects were developed.

For each project, a visual field assessment was made to identify feasible construction methods, viable pipeline alignments, and constructability issues. Likely construction methods were identified for each project for purposes of estimating construction costs. Methods included pipe bursting, open cut construction of new or replacement sewers, and microtunneling. For all sewers to be replaced with a larger diameter pipe, it was also assumed that the lower laterals connected to the existing sewer would also be replaced to the property line and a property line cleanout installed, as is the District’s current practice for sewer rehabilitation and replacement projects.

RECOMMENDED CAPACITY IMPROVEMENT PROGRAM

Twenty one capacity relief projects were developed to address the identified capacity deficiencies in the trunk sewer system. The projects include approximately 45,500 feet of relief and replacement sewers ranging in size from 8 to 24 inches in diameter. Capital costs were estimated for the construction of the facilities in each project. The costs presented are intended to be conservative, and are considered planning level estimates with an estimated accuracy of -30 to +50 percent. The estimated capital cost of the Capacity Improvement Program (CIP) is approximately $22 million.

The proposed capacity improvement projects were prioritized based on the relative severity of identified capacity deficiencies in the existing trunk sewer system (as evidenced by the extent of model-predicted overflows or surcharge, or actual historical overflows or surcharging during large storm events). The improvement projects were grouped into 5- and 10-year CIPs. The Capital Improvement Programs, with projects listed in priority order, are summarized in **Table ES-1**. **Figure ES-3** shows the locations of the projects. The project priorities should be modified as needed based on other system needs such as sewer rehabilitation to address structural deficiencies or maintenance issues. The SHECAP priorities and CIP will be incorporated into the District's overall CIP as part of its Sewer System Assessment and Capital Project Planning program.

Table ES-1. Trunk Sewer Capacity Capital Improvement Program

Proposed CIP Implementation Schedule	Priority	SHECAP Project No.	Project Name	Estimated Capital Cost ($)
5-Year	1	16	Kentfield Relief Sewer	$ 999,000
	2	12	Upper Shady Lane Trunk Sewer	$ 913,000
	3	10	Sir Francis Drake / Winship	$ 975,000
	4	9	Miracle Mile	$ 1,743,000
	5	14	Goodhill	$ 767,000
	6	15	Woodland / College	$ 1,306,000
	7	19	William / Holcomb / Meadowood	$ 1,303,000
	8	2	Spruce / Park / Merwin / Broadway	$ 1,750,000
	9	4	Creek / Bolinas	$ 1,675,000
Subtotal 5-Year				*$ 11,431,000*
10-Year	10	5	Upper Butterfield	$ 1,582,000
	11	3	Cascade	$ 572,000
	12	17	Laurel Grove / McAllister	$ 949,000
	13	6	Lower Butterfield / Meadowcroft / Broadmoor / SFD	$ 1,980,000
	14	8	Sonoma / Nokomis	$ 1,785,000
	15	20	Magnolia	$ 836,000
	16	7	The Alameda / Brookmead	$ 764,000
	17	11	Bolinas / Fernhill	$ 1,074,000
	18	21	Eliseo	$ 66,000
	19	1	Westbrae / Hawthorne	$ 424,000
	20	13	Sir Francis Drake / Berry	$ 471,000
	21	18	Manor Easement	$ 338,000
Subtotal 10-Year				*$ 10,841,000*
Total				**$ 22,272,000**

CONCLUSIONS AND RECOMMENDATIONS

The results of flow monitoring and hydraulic modeling of the District's sewer collection system indicate that the system has adequate capacity to convey peak dry weather flows, but has substantial capacity deficiencies that could potentially result in sanitary sewer overflows during large storm events. This study identifies trunk sewer improvement projects which, if constructed, would increase system capacity and thereby reduce the risks of wet weather overflows. Although the focus of this study has been on expanding trunk conveyance capacity, sewer rehabilitation, as part of an overall program to upgrade the District's wastewater collection system, should help reduce I/I in the system over the long-term and reduce the risk of SSOs. I/I should be used as a factor, along with maintenance history, age, pipe size, material, and other considerations, to prioritize sewer condition assessment and rehabilitation efforts. The need for lower priority capacity improvement projects should be re-evaluated at the time of proposed implementation to determine if sewer rehabilitation has or could eliminate the need for some of these projects.

The following recommendations are made based on the results and conclusions of this study:

- The District should budget for construction of the highest priority projects as soon as possible and initiate predesign studies to confirm the viability of the proposed alignments and the estimated construction costs.

- The alignments and sizes of all recommended projects should be verified with detailed predesign analyses, including topographic surveys, geotechnical investigations, utility research, and constructability reviews. Viable alternative alignments should also be considered during predesign.

- The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.

- The District should advise CMSA of the updated flows projections for the RVSD service area based on the relief improvements identified in this study so that CMSA can assess the potential impact of those flows on downstream treatment and disposal facilities.

- The District should address I/I on an area-specific basis. Targeted I/I source detection and correction should be considered in areas with particularly high I/I flows.

- The District should implement an on-going program of closed-circuit television inspection and a long-term renewal and replacement program to preserve the District's sewer pipeline assets. Specific recommendations for a long-term sewer condition assessment and rehabilitation and replacement program are being developed as part of the Sewer System Management Plan and Sewer Replacement Master Plan in the District's ongoing Sewer System Assessment and Capital Project Planning program.

- The District should develop a comprehensive SSMP following the guidelines developed by the San Francisco Bay RWQCB. The work conducted for this study provides a foundation for preparation of a Capacity Management Plan, one of the key components of the SSMP.

Section 1 - Introduction

This report presents the results of the Sanitary Sewer System Hydraulic Evaluation and Capacity Assurance Plan (SHECAP) study for Sanitary District No. 1 of Marin County, also known as the Ross Valley Sanitary District (RVSD or District). This report was prepared by MWH under an agreement with the District dated September 17, 2004.

BACKGROUND AND PURPOSE OF STUDY

The purpose of this SHECAP study is to evaluate the hydraulic performance of the existing sewer system, identify capacity deficiencies in the system, and recommend potential sewer improvement projects to correct the identified problems. The recommended projects will be incorporated into the District's 5-year and long-term Capital Improvement Plans. The study will also serve to fulfill the requirements of the Capacity Management element of the District's Sewer System Management Plan (SSMP), which is required by the San Francisco Bay Regional Water Quality Control Board (RWQCB) pursuant to Section 13267 of the California Water Code. A similar requirement will be included in future statewide Waste Discharge Requirements expected to be issued in 2006 to all collection system agencies in California by the State Water Resources Control Board. The objective of the Regional and State SSMP requirements is to reduce existing and potential sanitary sewer overflows (SSOs) from wastewater collection systems.

DISTRICT SERVICE AREA

The District serves the towns of Fairfax, San Anselmo, and Ross; the City of Larkspur (including Bon Air); and the unincorporated areas known as Sleepy Hollow, Kentfield, Kent Woodlands, Oak Manor, and Greenbrae (see **Figure 1-1**). Under contract, the District also serves the wastewater collection system in Murray Park and conveyance system (pump station and force main) for San Quentin Prison. The District is bordered by San Rafael on the east, Woodacre and Marin County to the north and west, and Mill Valley and Corte Madera to the south. U.S. Highway 101 cuts through the southern corner of the District. The study area largely consists of single and multi-family residential development and commercial areas, primarily located along major roadways such as Sir Francis Drake Boulevard. Major features include the College of Marin, Marin General Hospital, the Larkspur Landing ferry terminal, and the San Quentin State Prison, which is located just east of the District's service area.

The terrain in the District generally slopes from northwest to southeast towards San Francisco Bay. San Anselmo Creek drains the northwest portion of the District and the Tamalpais and Larkspur Creeks drain the southern portion of the District. All three creeks drain into Corte Madera Creek and then into San Francisco Bay.

WASTEWATER COLLECTION SYSTEM DESCRIPTION

The District owns, operates and maintains approximately 186 miles of collection sewer lines, 7 miles of force mains, and 20 pumping stations which collect and transport an average of approximately 5 million gallons per day (MGD) of wastewater to Central Marin Sanitation Agency (CMSA) for treatment and disposal. Approximately 95 percent of the District's collection system was installed prior to 1955. **Figure 1-1** shows the District's collection system, including the major (trunk) sewers, and pump stations that were included in the collection system hydraulic evaluation. The remainder of this report describes the hydraulic evaluation, including the development of the model used in the evaluation, the results of the hydraulic assessment, and the recommended capacity improvements.

Section 2 - Computer Model Development

To analyze the capacity requirements of the collection system, a computer model was developed of the trunk sewer network. The computer model was developed using InfoWorks™ by Wallingford Software. InfoWorks is a fully dynamic hydraulic model that can be used to simulate the performance of both gravity and pumped systems and provide an accurate representation of the flow rates, velocities, and water levels in the system under a variety of flow conditions. Major components of the model include the sewer network (pipes and manholes), pump stations, and sewer subbasins and associated data used to generate flows in the model. Each of these model components is discussed below.

MODELED SEWER NETWORK

The modeled sewer network for the SHECAP study consists of the gravity trunk system plus the major pump stations and force mains that convey flow to the CMSA wastewater treatment plant (WWTP). The gravity trunk system was initially defined to include all 10-inch diameter and larger pipes plus selected smaller pipes that effectively serve as trunk sewers. The initial network was refined to include additional sewers that were identified by District staff as potential capacity problems. Some lines were deleted from the model because they served relatively small tributary areas and/or could not be located or accessed for surveying. The final modeled network includes 526 manholes and 122,121 feet of gravity sewers ranging from 6 to 42 inches in diameter. This represents about 12 percent of the total length of gravity sewers in the District's system. The modeled network also includes seven of the District's pump stations and 31,518 feet of force mains ranging from 8 to 54 inches in diameter. .

DEVELOPMENT OF MODEL NETWORK DATA

The basis for the development of model network data was the District's AutoCAD sewer system maps. Each manhole or other system structure has a unique identifier. The maps indicate pipe diameters and limited information on pipe material, but no other attribute data are available. Therefore, as part of this study, approximately 500 manholes in the trunk sewer system (most of the modeled manholes) were surveyed and inspected to determine horizontal coordinates, rim elevations, and depth to pipe inverts. The work was done by two subconsultant firms. Oberkamper & Associates surveyed manhole coordinates and rim elevations. E2 Consulting Engineers performed manhole inspections, which included photographs and a sketch of the configuration of each manhole, a visual estimate of inlet and outlet pipe diameters, and measured depths to each pipe invert. These data were used in conjunction with available record drawings provided by the District to develop the necessary manhole and pipe attribute data for the model.

For the force main system, a previous model of the pump and force main system developed by Nolte Engineers for CMSA was used as the initial source of information for the force main diameters and numbering system for force main junctions. The horizontal locations (X-Y coordinates) of key junctions were taken from this model. The force main pipe alignments were then digitized using District maps and record drawings, and elevation information and confirmation of pipe diameters were obtained from the record drawings. Additional nodes were added for air relief valves and other structures, as well as some low and high points along the force mains as needed to create a realistic model.

PUMP STATIONS

There are 20 pump stations in the District's wastewater collection system, of which seven are included in the hydraulic model. The seven modeled pump stations are those that discharge directly into the force main system that conveys all of the District's wastewater flow to the CMSA WWTP (see **Figure 1-1**). The remaining 13 pump stations serve small local areas in the un-modeled portion of the collection system. Information on the modeled pump stations (pump discharge rates, pump on/off levels, and wet well dimensions) and associated valves, gates, and force mains were obtained from District staff and available as-built drawings. The pump stations were modeled using the pump curves associated with each pump. The modeled pump stations are listed in **Table 2-1**. The standby pumps only run when another pump fails. Therefore, the standby pumps were not used in the modeling.

Table 2-1. Modeled Pump Stations

Pump Station	Pump Number (Pump Operation)	Each Pump Design Discharge (MGD)
PS10 (Landing B)	1 (Duty), 2 (Standby)	0.72
PS11 (San Quentin)	1 (Duty), 2 (Assist), 3 (Standby)	2.02
PS12 (Bon Air)	1 (Duty)	0.43
	2 (Assist)	0.72
PS13 (Greenbrae)	1 (Duty), 2 (Standby)	2.02
	3 (Assist), 4 (Standby)	5.76
	5 (Standby)	6.48
PS14 (Larkspur Main)	1 (Duty), 2 (Assist)	1.73
	3 (Assist)	1.73
PS15 (Kentfield)	1 (Dry Duty), 5 (Dry Assist)	5.76
	2 (Wet Duty), 3 (Wet Assist), 4 (Wet Assist)	23.04
PS24	1 (Duty), 2 (Standby)	0.72
PS25	1 (Duty), 2 (Assist), 3 (Standby)	0.72

SEWER SUBBASINS

To develop tributary flows to the modeled network, 100 sewer subbasins were delineated based on general flow directions and connection points of local sewers to the trunk lines, as shown in **Figure 2-1**. The subbasins averaged approximately 125 acres in size. Typically, the local sewers within each subbasin discharge to the trunk sewer system at a single point or at several locations in close proximity, called a model load manhole. The subbasins were assigned ID numbers based on the flow meter area in which they were located (see discussion of flow monitoring in next section of the report). The basis for calculation of wastewater flows is discussed in subsequent sections of this report.

Section 3 - Basis of Flow Estimates

Wastewater flows were estimated for the District based on population data obtained from U.S. Census information, customer and water use data from the District's sewer billing records, and flows measured during a temporary flow monitoring program conducted for this study during the 2004-05 winter season. The flows were used to develop and calibrate the hydraulic model. This section describes the flow monitoring and the basis for calculating base wastewater flows and infiltration/inflow.

FLOW MONITORING

The main purpose of monitoring sewer flow and rainfall was to measure existing flow conditions in the system in order to calibrate the hydraulic model. E2 Consulting Engineers collected flow monitoring and rainfall data in the study area from December 2004 through March 2005. Twenty flow meters were installed throughout the District as shown in **Figure 3-1**. The figure also shows the areas that contribute flow to each meter.

Concurrent with the wet weather flow monitoring, three rain gauges were placed throughout the District. The rain gauges were placed at the Fairfax City Hall (Gauge 1), Ross Town Hall (Gauge 2), and at the Larkspur Main Pump Station (Gauge 3). The rain gauge locations are also shown in **Figure 3-1**.

Flow information used for model calibration came from dry (non-rainfall periods) and wet weather data flow data gathered during the flow monitoring period. **Table 3-1** lists the flow meters and information about the installation of each meter.

MODEL FLOWS

Wastewater flows have three basic components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration and inflow (RDI/I). A typical representation of these components is shown in **Figure 3-2**. The basis for quantifying each of these flow components is described below.

Table 3-1. Flow Monitors Used for Model Calibration

Meter No	Date Installed	Date Removed	Manhole	Diameter (inches)	Location
1	12/6/04	2/21/05	F003.01	15	Easement between Dominga Ave. and Bolinas Rd.
2	12/6/04	2/25/05	F002.10	18	Dominga Ave. at Court Ln.
3	12/7/04	2/21/05	S900.22	18	The Alameda north of Arroyo Ave.
4	12/7/04	2/21/05	S800.01	24	Saunders Ave. near Berlin Ave.
5	12/7/04	2/21/05	S000.53	36	San Anselmo Ave. at Hazel Ave.
6	12/7/04	2/21/05	S600.01	12	Madrone Ave. near Nokomis Ave.
7	12/7/04	2/21/05	S001.00	18	San Anselmo Ave. at Ross Ave.
8	12/7/04	3/16/05	R000.30	36	Shady Lane south of Fern Hill Ave.
9	12/10/04	2/22/05	K000.04	39	Back of 913 Sir Francis Drake Blvd. adjacent to Corte Madera Creek
10	12/13/04	2/22/05	K100.23	27	Stadium Way east of College Ave.
11	12/10/04	2/21/05	W514.08	10	Woodland Road between Evergreen Dr. and Rancheria Rd.
12	12/13/04	2/22/05	K200.01	12	McAllister Ave. at Berens Dr.
13	12/14/04	3/16/05	G300.01	12	Easement south of Manor Dr.
14	12/28/04	3/16/05	G000.10	30	Parking lot at 575 Sir Francis Drake Blvd.
15 [1]	2/4/04	3/16/05	G000.02	10	Upstream of Greenbrae PS
16 [2]	1/28/05	3/16/05	L150.03	21	Doherty Dr. east of Larkspur Main PS
17 [2]	1/28/05	3/16/05	L152.03	21	Doherty Dr. west of Larkspur Main PS
18	12/28/04	3/16/05	Pump Station		At Bon Air PS
19	12/16/04	3/7/05	L000.04	10	Larkspur Landing Circle off of Sir Francis Drake Blvd.
20	12/14/04	3/16/05	K000.02	42	Bike path upstream of Kentfield PS

1. This meter was initially installed in the wrong inlet pipe at the metering manhole; it was relocated to the correct pipe at the beginning of February.
2. Due to pump station construction, these meters were initially installed further upstream but were still impacted by significant surcharge due to the construction activities. At the end of January, after the new pumping facilities became operational, the meters were relocated to their intended locations on the two pump station influent sewers.

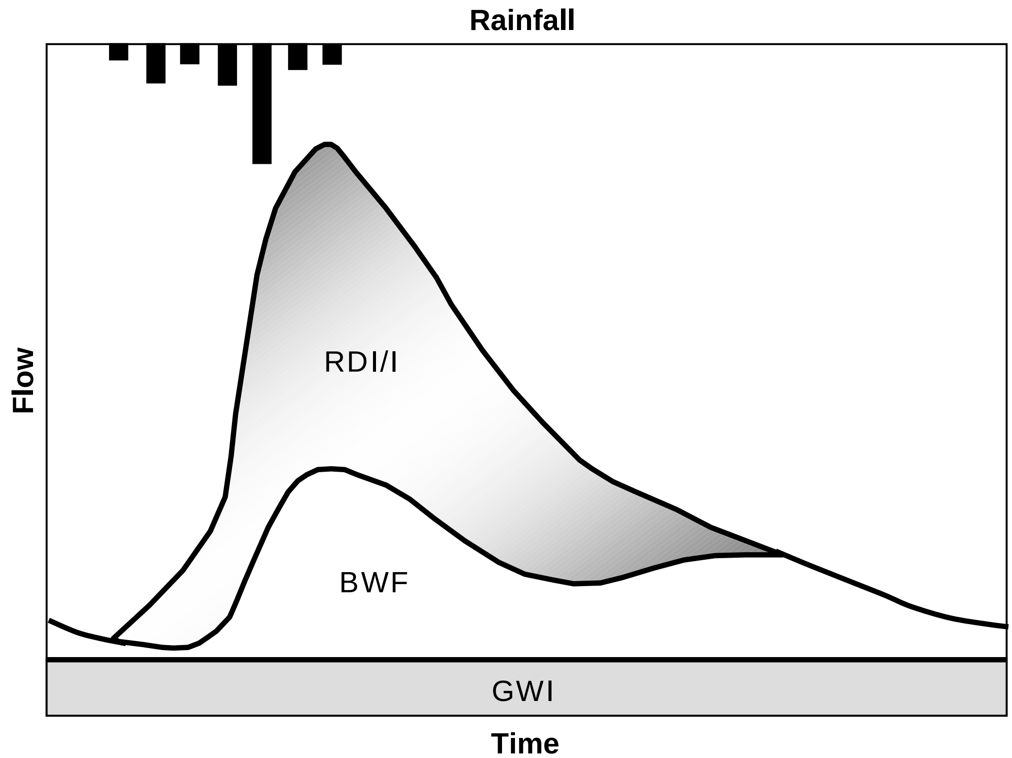

Figure 3-2. Wastewater Hydrograph Components

Base Wastewater Flow Estimates

Base wastewater flow is flow that is discharged from residential, commercial, industrial, and institutional (public and quasi-public) users of the sewer system. Residential BWF was calculated based on 2000 U.S. Census population data. **Figure 3-3** shows the distribution of residential population based on the Census data. The total population for the District's service area was found to be approximately 43,500. The total population was determined for each sewer subbasin by a graphical intersection of census blocks with the sewer subbasins. The model calculated the residential BWF by multiplying the population by a unit flow rate of 60 gallons per capita day (gpcd). This unit flow rate was found during model calibration to best match measured flows.

Non-residential BWF was calculated from municipal water billing data as the average winter water use for the winters of 2004 and 2005. Because very little irrigation or lawn sprinkling is done during the winter months, most of the water used from municipal water systems discharges to the sewer system. The water use was distributed by locating each customer on county parcel maps. The average winter water use for the non-residential users was summed for each subbasin and entered into the model. There were 629 non-residential customers that were identified. Those customers represent 24 percent of the total combined residential and non-residential BWF.

It should be noted that the District's service area is almost fully built-out and very little new development is anticipated. Therefore, increases in BWF in the future are expected to be minor and insignificant in comparison to peak wet weather flows. For this reason, with the District's concurrence, no future growth scenarios were considered in this study.

Diurnal Flow Variations

In domestic wastewater systems, BWF varies throughout the day. In residential areas, BWF generally peaks in the morning and again in the evening hours. BWF patterns in non-residential areas are determined by the type of establishments, but are typically characterized by more gradual elevated flows over an extended period of time during the middle of the day. For both residential and non-residential areas, there are also differences between typical weekday and weekend diurnal flow patterns.

Weekday and weekend diurnal wastewater flow curves were developed from the flow monitoring data. The curves represent the hourly variation in base wastewater flow for a typical subbasin and were applied to the subbasin BWF estimates to determine the BWF loads to the modeled system. **Figure 3-4** shows the residential diurnal patterns developed during model calibration. The curves show typical residential flow patterns, with the peak flow occurring during the morning hours and a lower peak during the evening. Three sets of weekday and weekend patterns were developed for different areas of the system based on the flow data from those areas. The primary difference between the sets of patterns is that flows peaked higher in some areas than in others. A separate diurnal pattern (not shown in the figure) was developed for the San Quentin State Prison based on actual recorded flows. **Figure 3-5** shows the non-residential patterns. The same non-residential patterns were used for weekdays and weekends. Diurnal 1 is the primary curve applied to non-residential BWF. Diurnals 2 and 3 were developed for specific locations to better match metered flows, as discussed in the calibration section.

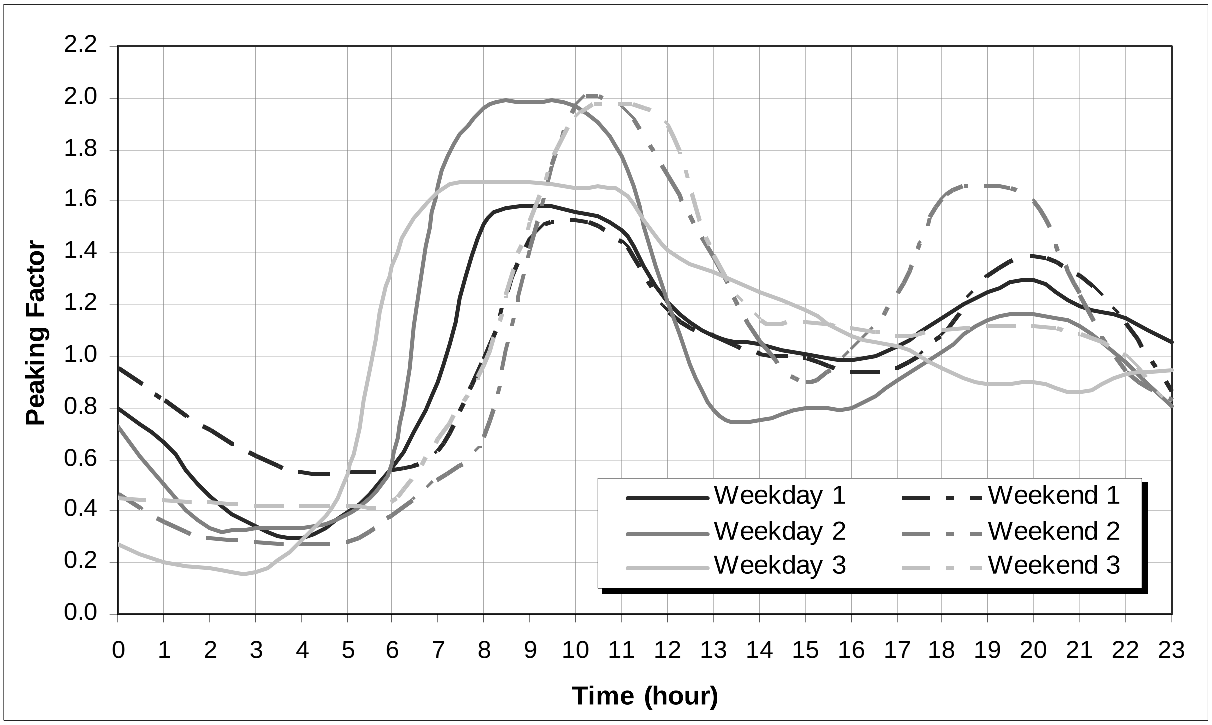

Figure 3-4. Residential Diurnal Patterns

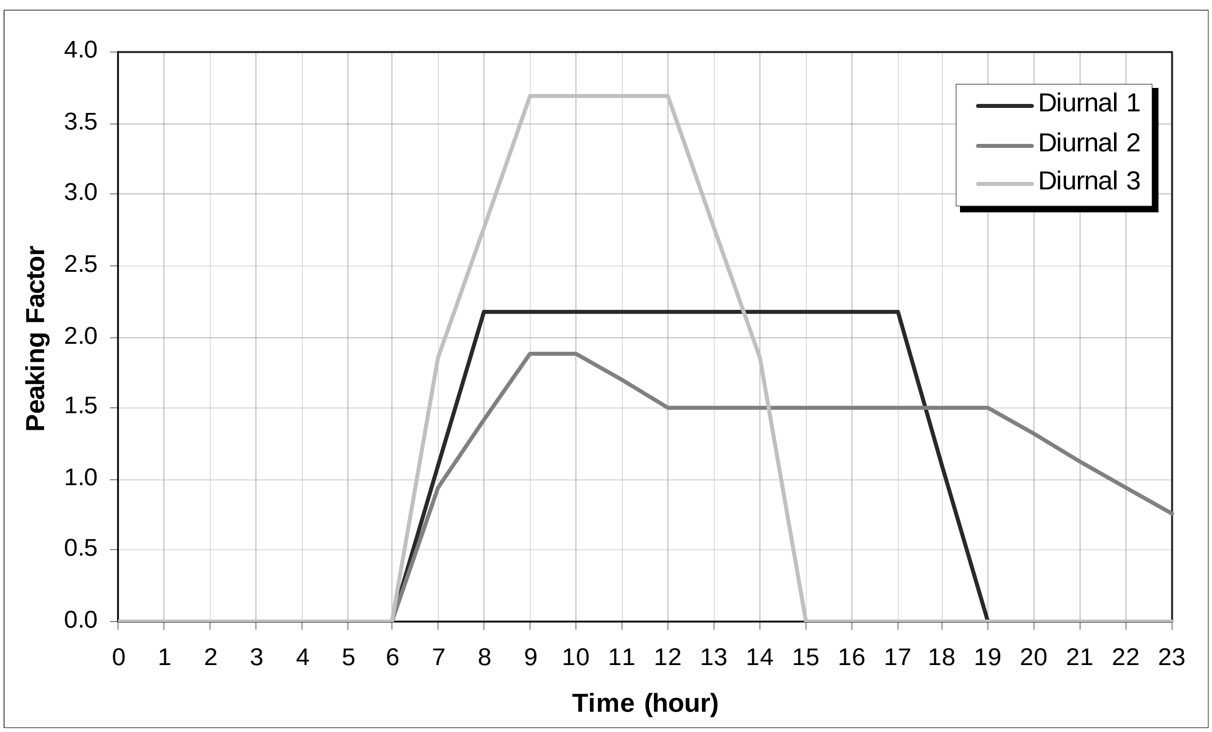

Figure 3-5. Non-Residential Diurnal Patterns

Groundwater Infiltration

GWI enters the sewer system through cracks and defects in sewer pipes, manholes, and service laterals. GWI varies depending on the location and condition of the sewers and is typically greater in late winter and spring following the rainy season when groundwater levels are higher. GWI may vary from a very minor component of flows in the sewer system to a significant percentage of the flow during non-rainfall periods. The magnitude of GWI can only be determined based on actual flow monitoring data.

GWI was quantified by comparing flow monitoring data for non-rainfall periods at each monitor site to modeled dry weather flows calculated based on the BWF methodology described above. The difference between monitored non-rainfall flow and modeled BWF was assumed to be due to GWI where the magnitude of the difference did not vary throughout the day. Based on the comparison of modeled to metered flows, estimated GWI rates in gallons per day (gpd) per acre were computed for each flow meter contributing area (each flow meter contributing area generally consists of several model subbasins). Each subbasin was assigned a GWI rate based on the flow meter that it contributes to directly. **Figure 3-6** and **Table 3-2** (in next subsection) show the GWI rate calculated for each metered area.

Rainfall-Dependent Infiltration and Inflow

RDI/I consists of storm water entering the collection system either as direct inflow of storm water runoff or rainfall-induced infiltration. Inflow occurs when storm water flows directly into the collection system through connected catch basins, manhole covers, area drains, or downspouts. Inflow usually occurs very rapidly during rain events and can become more severe if surface flooding occurs and manholes are submerged or used to drain low lying areas. Rainfall-induced infiltration is caused by storm water percolating through the ground and entering the sewer pipes, manholes, and service laterals through cracks and defective joints.

The magnitude of RDI/I is related to the intensity and duration of the rainfall, the relative soil moisture at the time of the rainfall event (typically a function of the amount of antecedent rainfall prior to the event), the condition of the sewers, and other factors such as soil type and topography. In most areas, peak flows during rainfall events are the highest flow rates that occur in the sewer system, although in communities where the sewers are relatively new and inflow and infiltration is minimal, peak wet weather flows may not be appreciably higher than peak dry weather flows.

RDI/I flows are typically quantified based on the volume of runoff entering the sewer system, usually expressed as a percentage of the rainfall volume and the shape of the resulting RDI/I hydrograph. The shape of the hydrograph is defined by separating the total RDI/I hydrograph into three components, representing fast, medium, and slow flow response to rainfall, as shown in **Figure 3-7.** The three components are defined based on their respective percentages of the total RDI/I volume and other parameters that define the time duration from rainfall to peak flow response and the time of flow recession. In some cases, there may also be a fourth very slow component representing a prolonged elevated flow response after the rainfall.

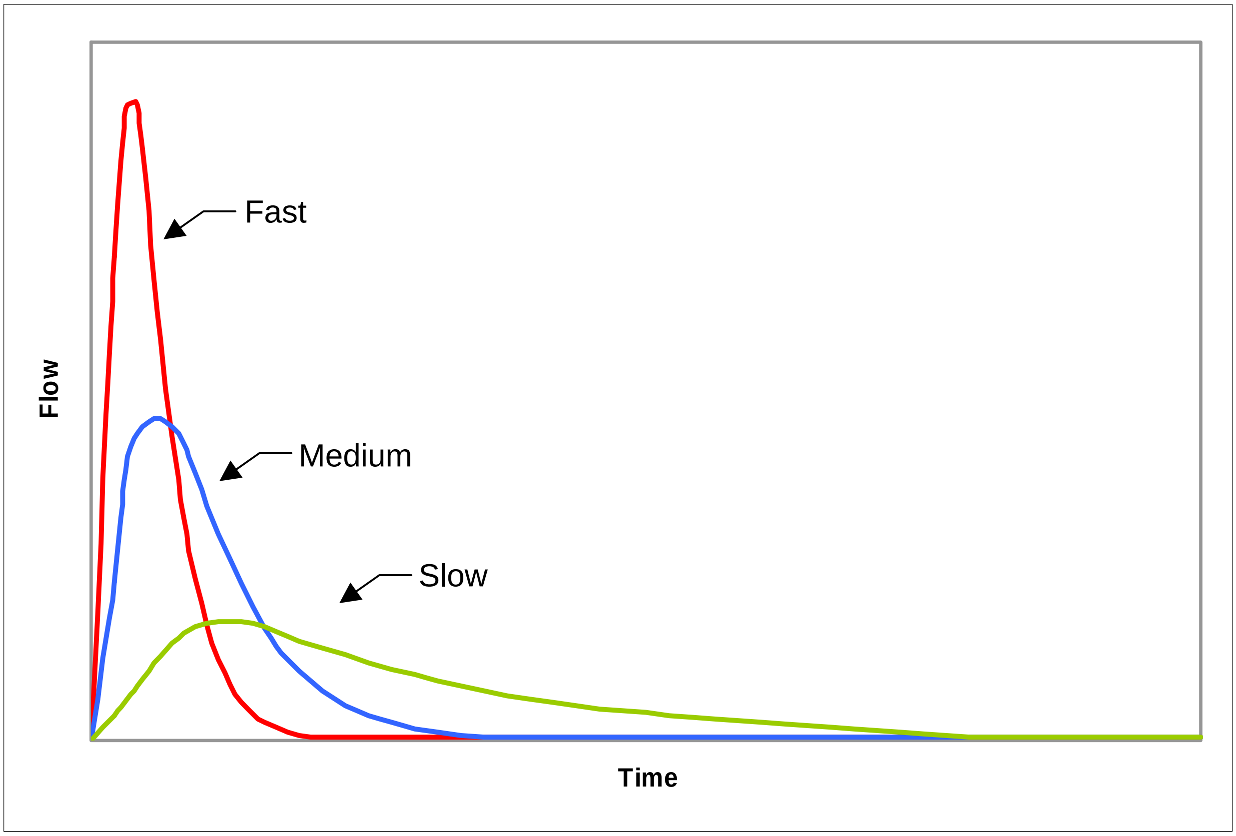

Figure 3-7. RDI/I Hydrograph Components

As part of the model calibration process, MWH analyzed the flow monitoring data to develop estimates of RDI/I during storm events. The flow monitoring data displayed examples of fast, medium, and slow rainfall response. **Figure 3-8** shows rainfall response in Ross Valley at two flow meters during a storm from February 15 through February 17, 2005. The top graph shows the storm rainfall intensity. The middle and bottom graphs show the flow response during and immediately following the rainfall event at two meters. The usual dry weather flow pattern is also shown on the graphs for comparison. The middle graph shows fast and medium response at FM 14 on Sir Francis Drake Blvd. The high peak flow indicates possible inflow sources and/or rapid infiltration into shallow sewers and service laterals. The bottom graph shows the rainfall response at FM 11 on Woodland Road. This meter exhibits a similar high fast/medium response to the rainfall, but the flow after the storm had not receded to normal dry weather flow after more than 24 hours. This slow response is possibly due to elevated groundwater, highly saturated soils, and the influence of creeks.

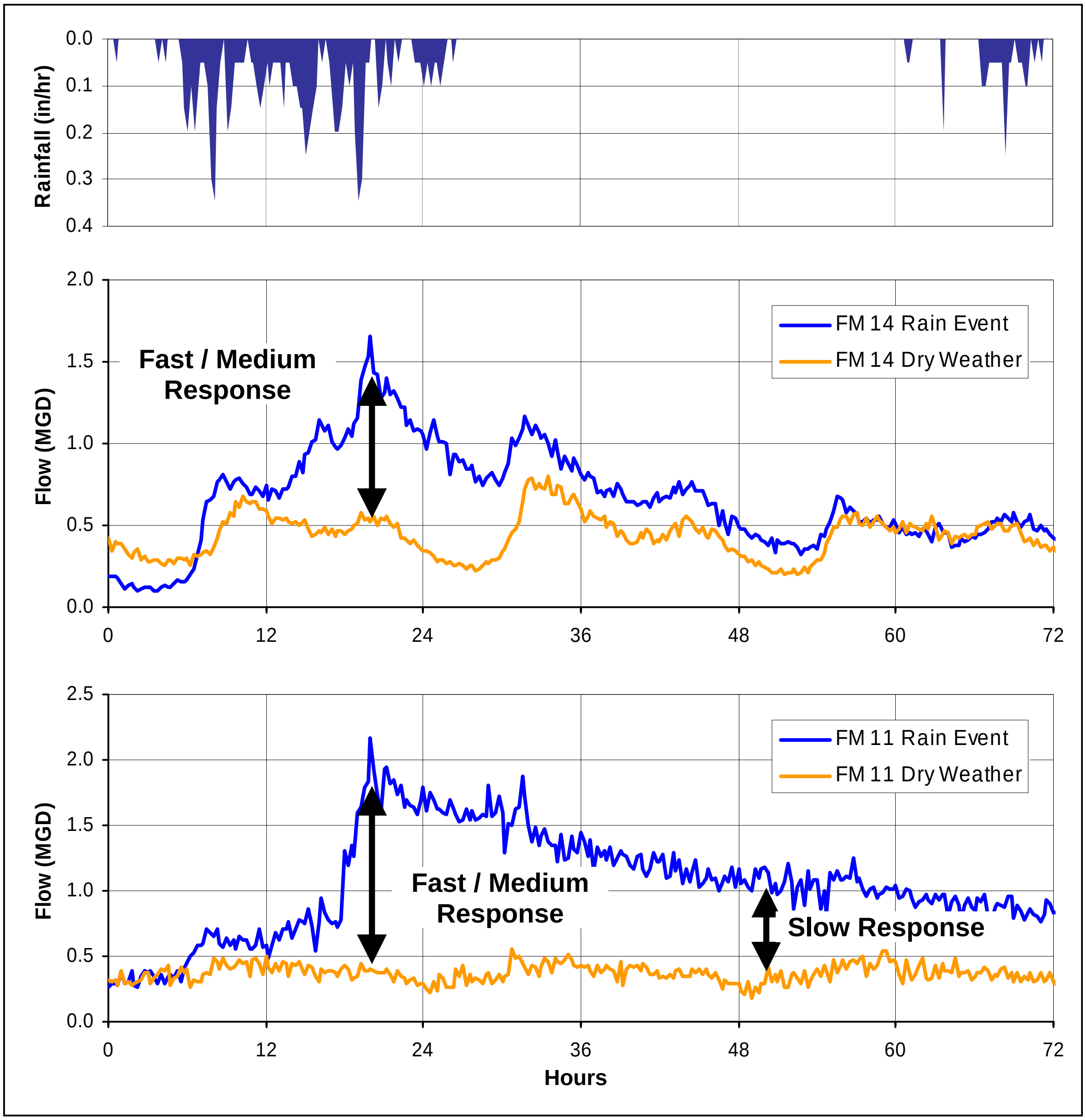

Figure 3-8. Examples of Rainfall Response

The RDI/I for each meter was estimated as a volume percentage of the rainfall that fell on the meter tributary area. For the storm events, the total RDI/I volume percentages ranged from approximately 2 to 14 percent. **Table 3-2** lists the RDI/I volume percentages by flow meter and the distribution into fast, medium, and slow response components. **Figure 3-9** shows the total RDI/I percentage for each subbasin. **Figure 3-10** shows the fast plus the medium RDI/I percentages for the subbasins. The subbasins with large fast and medium percentages will tend to have a high peak flow response because the volume of fast and medium response has the most impact on peak RDI/I flows.

Table 3-2. Model GWI and RDI/I Parameters

Meter	Contributing Area (acres) [1]	GWI (gpd/acre)	RDI/I (Percent of Rainfall Volume)				
			Fast	Medium	Slow	Very Slow	Total
1	622	234	-	1	4	-	5
2	394	1,054	-	-	5	5	10
3	839	393	-	1	4.5	-	5.5
4	286	113	2.5	1.5	4	-	8
5	505	106	0.5	-	2.5	-	3
6	184	1,053	2	2	4	-	8
7	366	163	0.5	1	4	-	5.5
8	437	357	1.5	1.5	3	-	6
9	308	105	1	1	2.5	-	4.5
10	461	520	0.5	2	2	-	4.5
11	453	691	0.5	0.5	2.5	1	4.5
12	253	512	-	1	2	-	3
13	159	284	2	3	6	-	11
14	258	508	1	0.5	2.5	-	4
15	208	218	0.5	0.5	3	2	6
16	408	47	1.5	2.5	7	2.5	13.5
17	263	71	2	-	6	-	8
18	265	732	1	-	1	-	2
19	144	709	0.5	-	4.5	-	5
20 [2]	-	-	-	-	-	-	-

1. The contributing area represents the area assumed to contribute infiltration and inflow, which does not include undeveloped parcels or open space. The area is the incremental, not the total, tributary area to the meter. For example, meter 4 is downstream of meter 3 so the contributing area for meter 4 includes the region upstream of meter 4 but downstream of meter 3.
2. Meter 20 measured total flow to the Kentfield Pump Station; its tributary area is included in upstream meters.

Other Flows

Flows from San Quentin State Prison and from Corte Madera (Sanitary District 2) enter the District's system upstream of the CMSA WWTP. The flows have little impact on the District's collection system because they discharge directly to downstream force mains. Flows for both Corte Madera and San Quentin were estimated based on flow data obtained from CMSA for the San Quentin Pump Station and the Ross Valley force main (combined Sanitary Districts 1 and 2 flows).

Model calibration involves comparing modeled and monitored flows and adjusting model parameters to achieve the best fit to measured data. InfoWorks provides graphing capabilities that allow the visual comparison of measured (observed) and modeled (predicted) flows, as well as statistical comparison information (minimum, maximum, and total/average values). Graphing includes comparisons of flow depth, velocity, and flow rate. For this study, flow rate was the primary value considered for calibration. Additional calibration to depth and velocity data is more difficult, as actual flow depths and velocities are impacted by sediment deposition in the sewers as well as differences in actual pipe slopes from designed slopes, e.g., due to sewer sags. An example calibration plot is shown in **Figure 4-1**.

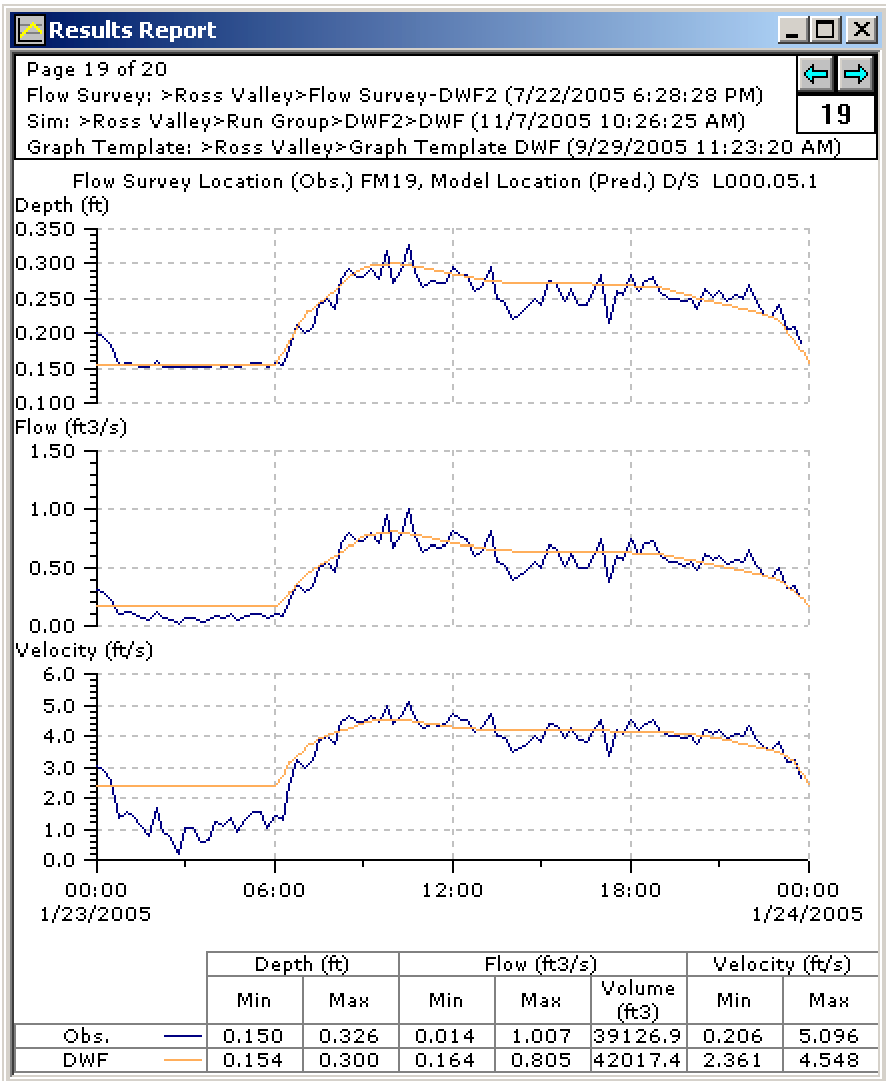

Figure 4-1. Sample InfoWorks Calibration Plot

Calibration is an iterative process during which model flow parameters are refined in order to achieve the best overall fit between modeled and metered data. The model is calibrated for dry weather conditions first to develop and confirm BWF and GWI rates and then for wet weather conditions to develop RDI/I parameters.

DRY WEATHER CALIBRATION

Two dry weather days were selected from the flow monitoring period for the dry weather calibration. The two days were selected by viewing flows for the entire flow monitoring period and selecting days that were not influenced by storm events and had dry weather flows that were typical for the flow monitoring period. **Table 4-1** lists the one weekend and one weekday that were selected.

Table 4-1. Dry Weather Calibration Events

Event	Date
DWF1, Dry Weather Flow Day 1	December 22, 2004 (Wednesday)
DWF2, Dry Weather Flow Day 2	January 23, 2005 (Sunday)

The main components of the model that were adjusted to improve dry weather calibration are described below.

- **Diurnal Profiles.** In addition to adjusting average modeled flow to reflect average measured flow, it was also important to have a reasonable match of peak flow rates and time of peak flows. Most decisions relating to pipe capacity are made based on peak flow, so it was important that the model accurately depict a peak flow situation. Initial diurnal profiles were created based on flow monitoring data. Those profiles were modified during the calibration to better match observed flows.

- **Flow Adjustments.** During the calibration, it was found that the dry weather flows measured at flow meters 6, 13, and 19 were significantly higher than the initial model-predicted flows. After careful review of the flow monitoring data and the customer billing database, no obvious explanation could be found for these discrepancies. To account for these flows, additional discharges of 0.10 MGD 0.04 MGD, and 0.17 MGD, respectively, were input to the model at the subbasin just upstream of the locations of these three meters. Non-residential diurnal pattern 2 was applied to the additional flow at meters 6 and 19 and diurnal pattern 3 was applied to the additional flow at meter 13.

- **Groundwater Infiltration.** GWI was estimated as the additional constant flow that needed to be added to the BWF to match the observed flows. GWI was computed for each drainage basin (which corresponded to a flow meter tributary area) and distributed on a gpd per acre basis according to the sewered (contributing) area of the subbasins within the basin. All subbasins within a particular drainage basin were considered to have similar GWI characteristics. Model adjustments for GWI were made only after all other land use or network-related issues had been resolved. The GWI appeared to be higher for several flow meters in the January 2005 data as compared to the December 2004 flow data due to rainfall in late December. To be more conservative, the model was adjusted to match the higher GWI rate.

WET WEATHER CALIBRATION

Two wet weather events were selected from the flow monitoring period for the wet weather calibration. **Table 4-2** lists the selected events. Rainfall was applied to the subbasins in the model by applying the total rainfall from the nearest rain gauge to each subbasin.

Table 4-2. Wet Weather Calibration Events

Event	Date	Description	24 Hour Rainfall Total (in) at Rain Gauges		
			Gauge 1 (Fairfax)	Gauge 2 (Ross)	Gauge 3 (Larkspur Main PS)
Storm A	December 26-28, 2004 (Sun-Tues)	Approx 2-10 year, 24-Hour storm (rainfall varied throughout District)	6.7	6.1	4.2
Storm B	February 15-17, 2005 (Tues-Thurs)	< 2-Year, 24-Hour storm	1.6	Not Available	1.4

The main component of the model that was adjusted to improve wet weather calibration is described below. Storm A, the larger storm, was primarily used for the calibration because there were several reported overflows during the storm which were replicated in the model. Storm B was used in cases where the recorded flows were poor at a meter during Storm A (e.g., for the meters upstream of the Larkspur Main Pump Station, which were impacted by the pump station construction during December 2004). Storm B was also used to verify that the model predicted accurate flows for another storm event.

- **Rainfall Dependent I/I.** RDI/I parameters including fast, medium, slow, and very slow response were adjusted. The calibration process consisted of setting initial parameters and then adjusting the parameters to achieve the best fit between observed and modeled flows. This was an iterative process in which the upstream basins were calibrated first followed by the downstream basins.

CALIBRATION RESULTS

The model calibrated fairly well to each of the calibration events. **Appendix B** contains plots of the calibration for the 2 dry weather events and the 2 storm events at every meter. An issue that was noted during the calibration was that the measured flows at FM11 (on Woodland Road between Evergreen Drive and Rancheria Road) did not decrease as would be expected after the rainfall ended after Storm A (see **Appendix B** plots). After the storm, the flows actually increased and remained high. District staff believe that this elevated flow may be due to the combination of unstable, wet hillside soils and leaky O-ring rubber gasket joints on the VCP pipelines in the area. Further investigation as to the possible causes and sources of infiltration in this area should be undertaken (see Section 7 for general recommendations on I/I investigation and prevention).

Section 5 - Hydraulic Evaluation

This section presents the design flow and hydraulic evaluation criteria used for the capacity analysis of the trunk sewer system and the results of the system hydraulic evaluation.

DESIGN FLOW AND RAINFALL CONDITIONS

Wastewater collection systems are typically sized for a specific design condition, often represented by a wet weather event in combination with designated system performance criteria. This section describes the design flow conditions to be used in evaluating system capacity and sizing improvements.

Design Flows

As discussed earlier, wastewater flows have three basic components: BWF, GWI, and RDI/I. As noted earlier, because the District's service area is almost fully built out, and very little new development is anticipated, no future growth scenarios are being considered in this study. Therefore, design flow criteria were based on existing BWF, GWI, and RDI/I parameters as described above. Note that these criteria also assume that I/I will not increase in the future due to system deterioration, i.e., that the District will continue an on-going sewer rehabilitation program (possibly including private laterals) that will maintain a "cap" on I/I. (The potential cost-effectiveness of reducing I/I to minimize required capacity upgrade capital improvements is discussed in the next section of the report.)

Design Storm

From the flow monitoring data, it is clear that rainfall greatly increases peak flows in the District's wastewater collection system. Therefore, the wastewater collection system needs to be evaluated with respect to the peak flows that occur during storm events and designed to accommodate those storm events. Typically, this is done by defining a design storm that represents the peak condition for which system capacity is evaluated and new or upgraded facilities are sized. A design storm may be an actual historical rainfall event, or a synthetic storm created for a specific storm frequency, storm duration, total rainfall depth, and temporal rainfall distribution. Another factor in defining the design storm is the time of day at which the rainfall is assumed to occur, particularly with respect to the normal diurnal base wastewater flow pattern.

Long-term historical rainfall data are not readily available for the Ross Valley. A synthesized long-term rainfall record was developed by a consultant to CMSA for its WWTP capacity study. The synthesized record was developed by correlating 2 years of recent rainfall data at the CMSA WWTP to historical data from various other gauges in Marin County and vicinity. The rainfall at the CMSA WWTP probably has similar storm patterns, but is not necessarily representative of the *magnitude* of rainfall in the Ross Valley. Therefore, for this study, the CMSA long-term record was used to review storm characteristics, but a synthetic design storm was created specifically for the Ross Valley for use in this study.

The following sections describe each aspect of a design storm. **Table 5-1** summarizes the design storm elements.

Table 5-1. Design Storm Summary

Design Storm Element	Value
Frequency	5 years
Duration	24 hours
Depth	Varies across District, obtained from NOAA intensity-duration-frequency (IDF) data
Rainfall Distribution	SCS Type IA
Storm Timing	Storm applied at average BWF

Storm Frequency

The design storm frequency is the expected recurrence interval of a storm of a given magnitude. For example, a 5-year storm has a magnitude that would be expected to occur approximately once every five years. Wastewater regulatory agencies have generally not specified a standard or required design storm frequency for wastewater collection systems. However, in the San Francisco Bay Region, most agencies utilize 5- or 10-year recurrence frequency design storms. Based on discussions with District staff, a 5-year event was selected as the basis for evaluation and sizing of the District's system for this study. A 5-year event is also consistent with criteria being used by CMSA for evaluation and design of WWTP wet weather facilities.

Storm Duration

The storm duration is the time from the start to the end of the rainfall for a single storm. A design storm is often selected based on the estimated travel time or "time of concentration" of flows in the sewer basin. Alternately, a storm duration based on typical historical storm periods can be selected. MWH analyzed the synthesized long-term rainfall data for the CMSA WWTP and found that most large historical storms in this area had durations of approximately 24 hours. Therefore, a 24-hour duration storm was selected for the design storm to be used in this study.

Storm Depth

The total rainfall depth for a design storm is the total amount of rainfall that falls during the entire storm, generally expressed in inches. The National Oceanic and Atmospheric Administration (NOAA) publishes intensity-duration-frequency (IDF) data for California in NOAA Atlas 2 (Reference 1). MWH extracted NOAA storm depths for a 24-hour storm for a storm frequency of 5 years. Because the amount of rainfall varies across the District's service area during a single storm, the storm depth was found at the location of each of the three rain gauges used during the flow monitoring program. The NOAA data showed that Rain Gauge 3 area (Larkspur) had a total storm depth close to the storm depth for the Rain Gauge 2 area (Ross) and much higher than the Rain Gauge 1 area (Fairfax) total. From analysis of the rain data collected during the flow monitoring and through discussions with District staff, it was determined that the Larkspur area would most likely have the lowest storm depth. Because the storm depth contours on the NOAA data were difficult to read with accuracy, the storm depth for

Rain Gauge 3 area was calculated based on the ratio of rainfall totals during the flow monitoring period. The storm depths listed in **Table 5-2** for each rain gauge area were applied in the design storm model run.

Table 5-2. 5-Year, 24-Hour Design Storm

Area To Apply Design Storm Depths	Rainfall Depth (inches)
Rain Gauge 1 (Fairfax City Hall)	5.0
Rain Gauge 2 (Ross Town Hall)	7.2
Rain Gauge 3 (Larkspur Main Pump Station)	4.0

As a comparison, the storm return frequency was calculated for the larger storm used for model calibration, on December 26-28, 2004. Because the total rainfall for the storm varied throughout the District's service area, the storm had a higher return frequency in some areas. The total rainfall at rain gauge 1 would classify the storm as greater than a 10-year storm. The storm was less than a 5-year storm at rain gauge 2 and between a 5- and 10-year storm at rain gauge 3.

Rainfall Distribution

Rainfall distribution refers to the change in rainfall intensity during a storm. The U.S. Soil Conservation Service (SCS), now known as the Natural Resources Conservation Service (NRCS), has published typical rainfall distributions for different geographic areas in the United States. The synthesized long-term rainfall record for the CMSA WWTP was also analyzed to find common rainfall distributions. The recorded 24-hour duration storms with approximately a 5-year return frequency matched the SCS Type IA rainfall distribution very well. The District service area also lies within the SCS Type IA rainfall distribution boundary (Reference 3). Therefore, the SCS Type IA rainfall distribution (shown in **Figure 5-1**) was used for the rainfall distribution for the design storm. The resultant 5-year design rainfall event for the Ross Valley is shown in **Figure 5-2**.

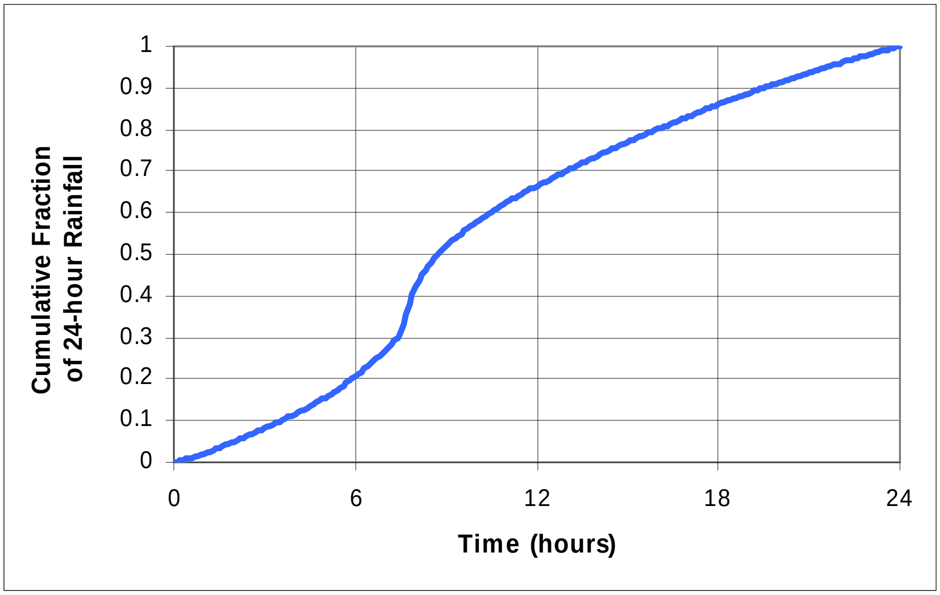

Figure 5-1. SCS Type IA Rainfall Distribution

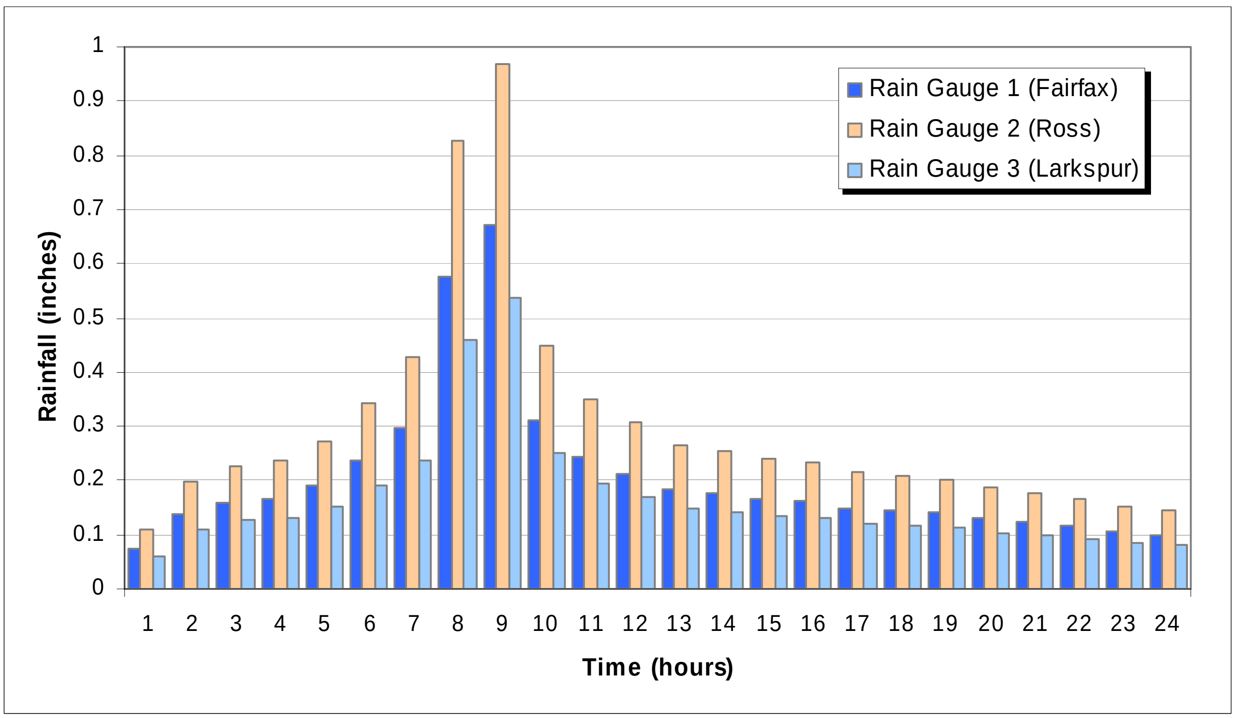

Figure 5-2. 5-Year Design Rainfall

Storm Timing

The timing of the storm can influence the magnitude of peak flow. For example, if the peak RDI/I (which typically occurs within an hour or so after the peak rainfall intensity) occurs at the same time as normal peak dry weather flows, the resultant peak storm flows will be higher than if the rainfall had occurred at a different time of day. If the peaks do coincide, the actual flow recurrence frequency could be greater (less frequent) than the design storm frequency. To avoid artificially increasing the flow recurrence frequency, average BWF flows were used throughout the duration of the design storm in the model runs. Average BWF flows were achieved by using flat residential and non-residential diurnal profiles.

EVALUATION CRITERIA

The hydraulic model was used to evaluate collection system performance using the design flow conditions detailed above. **Table 5-3** lists the criteria to be used when evaluating the collection system model under dry and wet weather conditions.

Table 5-3. Evaluation Criteria

Condition	Criterion
Peak Flow during Design Storm	If pipe is surcharged[1], distance between the manhole rim and the top of water should be greater than 10 feet
Peak Dry Weather Flow	No greater than full pipe

1. Surcharge refers to situations when the flow depth is higher than the top of the pipe.

The criteria listed in **Table 5-3** were used to initially identify problem areas. The criteria were developed to avoid extended periods of surcharging but to allow for brief periods of surcharging that would not be high enough to cause a significant risk of overflows. Each identified problem area was then analyzed in detail. Engineering judgment was used in analyzing each problem area to avoid recommending potentially large expenditures for capital improvements to correct marginally deficient situations. Recommendations for improvements were evaluated and discussed with District staff on a case-by-case basis.

SYSTEM EVALUATION

The trunk system was evaluated under existing dry weather and design storm scenarios. Based on the model runs, the peak flow condition was determined for each pipe, and locations with deficiencies were identified. No capacity deficiencies were identified for dry weather conditions, but significant portions of the system were found to have inadequate capacity for wet weather flows. **Figure 5-3** shows an overall view of existing capacity deficiencies and potential overflow locations under the 5-year design storm event. The “Surcharge due to Capacity” deficiency includes pipes that were surcharged in the model due to insufficient pipe capacity. The “Surcharge due to Backup” deficiency indicates pipes surcharged with backwater due to downstream capacity deficiencies. Only pipes with a surcharged water level within 10 feet of the manhole rim are shown as a deficiency. **Figures 5-4** through **5-6** show the deficiencies in more detail.

The model results indicate that the system would surcharge significantly under a design wet weather event and potentially result in overflows in many locations in the District if the sewer system remains in its current condition. The model results were reviewed with District staff, who confirmed that many of the locations indicated in the model as being potential overflow points have, in fact, experienced overflows or surcharge during wet weather. There are no capacity deficiencies or predicted surcharge or overflows under dry weather conditions.

Note that the model may over-predict overflows because additional storage volume could be available in the upstream, un-modeled sewers in the collection system, which would reduce the potential volume of overflows. To account for the storage available in upstream non-modeled manholes and pipes, the storage volume of those pipes and manholes was summed and added to upstream manholes. Note also that the locations of potential overflows are not necessarily the areas with the most critical capacity deficiencies, but rather the locations where backwater from downstream deficiencies reaches a high enough level to exceed the elevations of the manhole rims. The model does not impose any physical limitations on how much can overflow from any particular manhole. Therefore, actual overflows might occur at additional or different manholes in the vicinity of the manholes predicted to overflow in the model.

In situations where the flow is throttled or surcharged, as occurs in many parts of the existing system under peak wet weather flows, the actual peak flow in the sewers would be lower than the flow that could be conveyed if the constrictions were relieved. Therefore, the design flow for the sewer reaches cannot be determined until relief facilities are incorporated into the model.

Pump Station and Force Main Capacity

The capacities of the pump stations and force mains were also analyzed under the design storm scenario. **Table 5-4** lists the capacity of each pump station as compared to the peak 5-year design storm flow. It should be noted that the peak flows reflect the effects of relief sewers upstream of the pump stations (as developed in the next section of the report) that would be needed to convey the peak flows to the pump stations. As indicated in the table, all of the pump stations except for Bon Air have sufficient capacity to handle the predicted 5-year design storm peak wet weather flows under normal pump station operation. However, three of the pump stations, (Bon Air, Larkspur Main, and Kentfield) may not have sufficient firm capacity (with the largest pump out of service) to handle the design storm peak flow. (The deficiency at the Kentfield Pump Station could be addressed by force main improvements, as discussed below.) The need for improvements at the pump stations is being evaluated as part of the Pump Station Replacement Master Plan in the District's ongoing Sewer System Assessment and Capital Project Planning program.

Table 5-4. Pump Station Capacity Results

Pump Station	Capacity Under Normal Operation [1] (MGD)	Firm Capacity (Largest Pump Out of Service) (MGD)	5-Year Design Storm Peak Flow (MGD)
PS10 (Landing B)	1.37	1.37	1.12
PS11 (San Quentin)	2.88	2.88	1.67
PS12 (Bon Air)	1.15	0.43	1.86
PS13 (Greenbrae)	9.96	9.96	5.51
PS14 (Larkspur Main)	8.41	5.88	8.56
PS15 (Kentfield)	41.9	36.9	39.0
PS24	1.52	1.52	0.45
PS25	1.41	1.41	0.70

1. Pump station capacity using normal operational settings (standby pumps are off). This capacity equals the capacity in the next column in the cases where the standby pump is the same size as the largest pump.

The only identified capacity deficiency in the force main system was in the 36-in pipeline between the Kentfield Pump Station and the end of South Eliseo Drive near Lower Via Casitas. At that point, the force main connects to a larger 42-inch force main. The size of the 36-inch force main limits the ability of the Kentfield Pump Station to pump to its design capacity, which is adequate to handle the projected design storm peak wet weather flow. The entire force main would need to be replaced with a 39-inch diameter pipe to provide adequate capacity. Alternately, only a portion of the force main, e.g., the section located along Eliseo Drive, could be replaced with a 42-inch pipe to provide the needed capacity, and the existing 36-inch upstream portion could remain (or be rehabilitated by lining). The replacement of this force main is being evaluated as part of the Force Main Replacement Master Plan in the District's ongoing Sewer System Assessment and Capital Project Planning program.

The next section of the report discusses proposed solutions to the capacity deficiencies identified through the hydraulic modeling, and presents recommended capital improvement projects to provide the required hydraulic capacity in the system.

Section 6 - Capacity Assurance Plan

This section presents the capacity assurance plan which includes recommended improvements and costs. Each recommended improvement is described in detail. The criteria used for the development of improvements are also included.

HYDRAULIC DESIGN CRITERIA

Hydraulic design criteria needed for master planning include pipe friction factors, allowable flow depth to diameter ratio (d/D), and minimum and maximum pipe slopes and velocities. **Table 6-1** presents the hydraulic design criteria used in the master planning design of sewer improvements or future facilities.

Table 6-1. Design Criteria

Criterion	Description	Value
Pipe Capacity	Pipe capacity at peak flow during design storm	75% d/D for pipes smaller than 12 inches Full pipe for 12-inch and larger pipes
Velocity	Minimum (pipe half full)	2 feet/second
	Maximum	10 feet/second
Pipe slope	Minimum	Sufficient to obtain minimum velocity
	Maximum	Less than slope that would produce maximum velocity

RECOMMENDED CAPACITY IMPROVEMENTS

Potential solutions to the predicted wet weather overflow problems fall into two basic categories: providing additional hydraulic capacity in the system (“relief solutions”), or repairing sewers to reduce I/I flows (“rehabilitation solutions”). Relief solutions involve replacement of existing sewers with larger pipes and/or construction of parallel relief sewers. Rehabilitation solutions involve identifying and eliminating direct inflow connections (if they exist), replacing or lining existing sewer lines, repairing manholes, and replacing or repairing service laterals (possibly including the portions located on private property).

Sewer rehabilitation is the ideal solution to high I/I flows because it corrects the actual cause of the problem. However, rehabilitation is generally only cost effective if relief solutions are prohibitively expensive and rehabilitation can be implemented at a reasonable cost but still achieve sufficient reductions in I/I to eliminate the need for relief facilities. For the District, the cost of rehabilitating the existing sewer system would be very high, for a number of reasons:

- Previous field investigations (e.g., smoke testing and sewer inspections) conducted to locate suspected sources of I/I in the District’s system indicated that defects in sewer pipes and laterals are the primary sources of I/I rather than direct inflow connections. This situation has been found to be typical of most of the older sewer systems in the San Francisco Bay Area.

- Many sewers are located in backyard or sideyard easements or narrow roads, making access for construction very difficult.
- The sewers are predominantly 6-inch diameter, which limits the potential rehabilitation methods that can be used (for example, lining methods that further reduce the pipe diameter would not be acceptable).
- Some sewers may have been subject to ground subsidence because of their location, resulting in areas with sags and adverse slopes. Most sewer rehabilitation methods cannot correct these types of problems.
- Achieving a substantial enough reduction in I/I to reduce the flows in the system to a level that would not substantially surcharge the existing system during a 5-year design storm event would require rehabilitation of the service laterals as well as the sewer mains.

Realistically, the only way to achieve substantial reductions in I/I through sewer rehabilitation would be through a comprehensive program of sewer line and service lateral replacement. However, this type of rehabilitation effort could be prohibitively expensive (see discussion of Cost-Effectiveness of I/I Reduction later in this section). This is not to say that the District should not implement a long-term program to rehabilitate its sewer system in order to minimize SSOs, preserve the structural integrity of the system, prevent I/I flows from increasing further, and reduce operation and maintenance costs. However, the most expedient way to address the critical capacity problems in the system within a reasonable length of time would probably be to provide additional capacity through the construction of new trunk sewers.

Therefore, to address the hydraulic problems in the system, relief solutions were developed. These solutions include potential relief sewers to divert flows from existing deficient sewers to other sewers with available capacity or to new parallel pipes or pipes in other alignments, as well as upsizing (by pipe bursting or remove-and-replace) existing deficient sewers.

A model of the potential relief solutions was developed to size the relief facilities and verify that they would alleviate all of the predicted overflows and most of the surcharge in the system. Based on the results of the solutions modeling, specific capacity relief projects were developed as described in the following subsection.

Project Descriptions

Twenty-one capacity relief projects were developed to address the identified capacity deficiencies in the trunk sewer system. The proposed relief projects are described below and shown in **Figure 6-1**. **Figures 6-2** through **6-5** show the improvement projects in more detail. A visual field assessment was made of each proposed project to identify feasible construction methods, viable pipeline alignments, and constructability issues. Likely construction methods were identified for each project for purposes of estimating construction costs. Methods included pipe bursting, open cut construction of new or replacement sewers, and microtunneling. For all sewers to be replaced with a larger diameter pipe, it was also assumed that the lower laterals connected to the existing sewer would also be replaced to the property line and a property line cleanout installed, as is the District's current practice for sewer rehabilitation and replacement projects. **Appendix C** contains photographs of proposed sewer alignments.

Project 1 – Westbrae/Hawthorne. This project would involve upsizing 1,278 feet of existing 8-inch sewer to 10-inch diameter pipe in an alignment located parallel to and southwest of Sir Francis Drake Boulevard. The existing sewer is located in an easement through an apartment frontage and across a creek and then along Westbrae Drive, terminating at Hawthorne Court. The sewer upsizing could be accomplished using pipe bursting construction methods.

Project 2 – Spruce/Park/Merwin/Broadway. This project would include upsizing 405 feet of existing 15-inch sewer to 18-inch diameter pipe on Spruce Road and Park Road from Arroyo Road to Merwin Avenue; and installing approximately 2,000 feet of 18-inch diversion sewer on Merwin Avenue from Park Road to Broadway and then eastward along Broadway to Pacheco Avenue. The diversion sewer would provide the required capacity relief while avoiding the need for difficult construction in easements and busy and narrow streets in downtown Fairfax. The diversion sewer would connect into the existing 30-inch trunk sewer at Pacheco Avenue at Broadway.

The sewer upsizing in Spruce and Park could be accomplished by pipe bursting. A double-barrel siphon would be needed to cross the creek on Merwin. There is also a hill on Broadway between Merwin and Bank Street where the new sewer would be very deep. In these portions of the alignment, the sewer could be constructed using microtunneling. The remaining portions of the new diversion sewer could be constructed using traditional open-cut construction methods.

Project 3 – Cascade. This project would involve upsizing 1,727 feet of existing 6-inch sewer located on Cascade Drive south of Bolinas Road to an 8-inch diameter pipe. The sewer upsizing could be accomplished by pipe bursting.

Project 4 – Creek/Bolinas. This project would involve upsizing 4,079 feet of existing 10-inch sewer located on Bolinas Road, Porteous Avenue, and Creek Road, and in an easement in a ravine parallel to and northwest of Bolinas Road. The pipe diameter would need to be increased to 12 inches in the ravine area and along Bolinas Road, and to 15 inches on Porteous Avenue and Creek Road. Pipe bursting would be the likely construction method. The section of the existing sewer mounted on the underside of the Creek Road Bridge would also need to be replaced. Note that the upstream portion of this project is already under design by the District as part of its sewer rehabilitation program.

Project 5 – Upper Butterfield. This project would involve upsizing 3,836 feet of existing 10- and 12-inch sewer located on Butterfield Road from Van Tassel Court to Fawn Drive. The pipe diameter would need to be increased to 12 inches upstream of Legend Road and to 15 inches downstream. The upsizing could be accomplished by pipe bursting.

Project 6 – Lower Butterfield/Meadowcroft/Broadmoor/Sir Francis Drake. This project would include upsizing, by pipe bursting, 493 feet of existing 6-inch sewer to 8-inch diameter pipe in Butterfield Road south of Carlson Avenue; and constructing approximately 3,000 feet of new diversion and parallel sewers in Butterfield Road from south of Rosemont Avenue to Meadowcroft Drive, and then along Meadowcroft Drive, Broadmoor Avenue, and Sir Francis Drake Boulevard to Mountain View Avenue. The diversion sewer in Butterfield Road and Meadowcroft Drive would provide the required capacity relief without the need for difficult construction in existing easements between Butterfield and Brookside Drive.

The existing 6-inch sewer on Butterfield Road south of Rosemont Avenue would be replaced with a 12-inch diameter pipe using open cut construction methods. A new 10-inch relief sewer would be installed on Butterfield Road and Meadowcroft Drive, likely using microtunneling construction methods. Microtunneling would be required due to the deep sewer depths (~27 feet). The 10-inch relief sewer would continue on Meadowcroft to Broadmoor, then south to Sir Francis Drake Boulevard. A new 15-inch relief sewer would be constructed on Sir Francis Drake from Broadmoor to Mountain View. The pipeline on the lower portion of Meadowcroft and along Broadmoor and Sir Francis Drake could be constructed using open cut construction methods.

Project 7 – The Alameda/Brookmead. This project would include upsizing 132 feet of existing 18-inch sewer located on The Alameda south of Arroyo Avenue to 21-inch diameter pipe; constructing approximately 1,000 feet of new 15-inch diversion sewer in The Alameda and Berkeley Avenue; and upsizing 535 feet of existing 18-inch sewer in the schoolyard north of Brookmead Place and along Brookmead to Brookside Drive to 21-inch diameter pipe. The diversion sewer would provide required capacity relief while avoiding the need for construction in the existing easement from The Alameda to the school property. Upsizing of the existing 18-inch sewers could be accomplished using pipe bursting construction methods. The new 15-inch relief sewer in The Alameda and Berkeley Avenue could be installed by open cut construction.

Project 8 – Sonoma/Nokomis. This project would include replacing 965 feet of existing 10- and 12-inch sewers with 12- and 15-inch pipe on Alderney Road from the park southeast of San Francisco Boulevard to Sonoma Avenue and on Sonoma Avenue to Sir Francis Drake Boulevard; and installing approximately 1,800 feet of new 15-inch diversion sewer from Sonoma Avenue across Sir Francis Drake along Sais, Nokomis, and Madrone Avenues to San Anselmo Avenue. The diversion sewer would provide required capacity relief and avoid difficult construction in Sir Francis Drake Boulevard and in easements between Bella Vista Avenue and San Anselmo Avenue. The new sewer would connect into the existing 36-inch trunk sewer in San Anselmo Avenue.

The pipe upsizing along Alderney and Sonoma would likely be done by remove and replace methods because the existing sewers are PVC pipe and could not be pipe burst. The new diversion sewer could be installed by open cut methods, except the Sir Francis Drake crossing which would probably be microtunneled, and would involve removal and replacement of existing smaller diameter lines in the alignment. A new double-barrel siphon would also be required on Nokomis Avenue at the creek crossing.

Project 9 – Miracle Mile. The project would include upsizing approximately 2,000 feet of existing 6-, 8-, and 10-inch sewers to 12-inch diameter pipe on Greenfield Avenue from Hilldale Drive to Sir Francis Drake Boulevard; and installing approximately 1,250 feet of new 12-inch diameter diversion sewer along Sir Francis Drake Boulevard south to Tunstead Avenue. The new sewer would connect into the existing 36-inch trunk sewer at Tunstead.

The pipe upsizing in Greenfield Avenue could be accomplished by using pipe bursting and remove and replace construction methods. The 12-inch relief sewer segments in Sir Francis Drake would likely be constructed using microtunneling. A new double-barrel siphon under San Anselmo Creek would also be required.

Project 10 - Sir Francis Drake/Winship. This project would include upsizing 1,123 feet of existing 8-inch sewer and 149 feet of 6-inch sewer on Sir Francis Drake Boulevard north of Winship Avenue to 12-inch and 10-inch diameter pipe, respectively; replacing 190 feet of 6-inch sewer across the Winship Avenue bridge with 8-inch pipe; and installing 450 feet of parallel 12-inch relief sewer in Bolinas Avenue from Sir Francis Drake to Shady Lane. The pipe upsizing in Sir Francis Drake Boulevard could be accomplished using pipe bursting construction methods. The existing pipe on the Winship Avenue bridge is built into the bridge superstructure; the new pipe would likely need to be attached to the side of the bridge. The new 12-inch relief sewer across Sir Francis Drake and in Bolinas Avenue would likely be constructed by microtunneling. This pipe would connect into the new Upper Shady Lane trunk relief sewer (Project 12).

Project 11 – Bolinas/Fernhill. This project would involve upsizing 1,461 feet of existing 10-inch sewer on Bolinas Avenue west of Shady Lane to 12- and 15-inch diameter pipe; and upsizing 851 feet of existing 10-inch sewer on Fernhill Avenue west of Shady Lane to 15-inch pipe. The pipe upsizing could be accomplished using pipe bursting construction methods.

Project 12 – Upper Shady Lane Trunk Sewer. This project would include installing 800 feet of 24-inch parallel relief sewer on Shady Lane from Bolinas Avenue south to Locust Avenue; and replacing 650 feet of the existing 21-inch trunk sewer in Shady Lane from Locust Avenue to Norwood Avenue with a new, deeper 21-inch pipe. The deeper sewer would allow more flow to be diverted from the existing 36-inch Shady Lane trunk sewer to reduce the predicted surcharge further downstream in the main trunk leading to the Kentfield Pump Station. The parallel and replacement sewers would likely be constructed by a combination of open cut and microtunneling construction methods.

Project 13 – Sir Francis Drake/Berry. This project would involve upsizing 998 feet of existing 10-inch sewer to 15-inch diameter pipe in Sir Francis Drake Boulevard from Laurel Grove Avenue to south of Berry Lane; and upsizing 105 feet of 12-inch sewer to 15-inch pipe in an easement from Sir Francis Drake to the 42-inch trunk sewer easement between Sir Francis Drake and Kent Avenue. The pipe upsizing could be accomplished by pipe bursting.

Project 14 – Goodhill. This project would involve upsizing 1,691 feet of existing 6-inch sewer to 10-inch pipe on Goodhill Road from Live Oak Way to Vineyard Way; upsizing 513 feet of 10-inch sewer to 12-inch pipe in the easement from Goodhill to Kent Avenue; and replacing 189 feet of 8-inch sewer with 15-inch pipe in Kent Avenue north of Stadium Way. The pipe upsizing in Goodhill and in the easement could be accomplished using pipe bursting construction methods. The section in Kent Avenue would need to be removed and replaced due to the diameter difference.

Project 15 – Woodland/College. This project would include replacing 1,600 feet of existing 10- and 12-inch sewer on Woodland Road from Evergreen Drive to College Avenue with 18- and 21-inch diameter pipe; and installing 650 feet of new 12-inch parallel relief sewer on

College Avenue from Woodland Road to Stadium Way. The relief sewer would connect into the proposed new Kentfield Relief Sewer (Project 16) at Stadium Way. The parallel relief sewer could be constructed by open cut methods, except for a new siphon, which would need to be installed under the storm drain culvert that crosses College Avenue just south of Woodland Road. The siphon would be installed adjacent to the existing double-barrel siphon.

Project 16 – Kentfield Relief Sewer. This project would involve installing 900 feet of a new 21-inch parallel relief sewer through the College of Marin campus from College Avenue and Stadium Way to the west side of Corte Madera Creek. A new 21-inch inverted siphon under Corte Madera Creek would also be required to connect to the existing Kentfield trunk sewer upstream of the Kentfield Pump Station. The new relief sewer could be constructed using a combination of microtunneling and open cut construction methods.

Project 17 – Laurel Grove/McAllister. This project would involve upsizing 2,256 feet of existing 8-, 10-, and 12-inch sewers to 10-, 12-, and 15-inch piping on Laurel Grove Avenue, Sir Francis Drake Boulevard, and McAllister Avenue from Cypress Avenue to Berens Drive. The pipe upsizing could be accomplished using pipe bursting.

Project 18 – Manor Easement. This project would involve upsizing 864 feet of existing 12-inch sewer to 15-inch diameter pipe in an easement located south of the intersection of Sir Francis Drake Boulevard and Manor Drive. The pipe upsizing could be accomplished by pipe bursting.

Project 19 – William/Holcomb/Meadowood. This project would include upsizing existing sewers and constructing new diversion sewers in William Avenue, Holcomb Ave, Meadowood Drive, and adjacent easements, including:

- Upsizing 516 feet existing 10-inch sewer to 15-inch pipe on William Avenue from Magnolia Avenue to Monte Vista Avenue.
- Installing 334 feet of new 12-inch sewer on William Avenue from Monte Vista Avenue to Holcomb Avenue.
- Upsizing 643 feet of existing 10-inch sewer to 12-inch diameter pipe on Holcomb Avenue from William Avenue to Cane Street.
- Installing 153 feet of new 12-inch sewer on the Holcomb Avenue and along the railroad easement north of Cane Street.
- Upsizing 300 feet of existing 10-inch sewer to 15-inch pipe in the Holcomb Avenue and the railroad easement to Meadowood Drive.
- Replacing 1,080 feet of existing 15-inch sewer on Meadowood Drive and in the easement east of Meadowood with 21-inch pipe.

Pipe upsizing could be accomplished by pipe bursting, and new pipe and pipe replacement by open cut construction methods.

Project 20 – Magnolia. This project would involve upsizing two sections of existing sewers located on Magnolia Avenue. The 290 feet of existing 6-inch sewer between Frances Avenue and Murray Avenue would be upsized to 10-inch diameter pipe, and 1,981 feet of existing 12-inch sewer from north of Bon Air Road to Cross Creek Lane would be upsized to 15-inch pipe. Pipe upsizing could be accomplished by pipe bursting.

Project 21 – Eliseo. This project would involve upsizing 218 feet of existing 6-inch sewer to 8-inch pipe on Eliseo Drive, north of Corte Cayuga. Pipe upsizing could be accomplished by pipe bursting.

RECOMMENDED CAPACITY IMPROVEMENT PROGRAM

Capital construction costs were estimated for the construction of the facilities in each project. The costs presented are intended to be conservative, and are considered planning level estimates with an estimated accuracy of -30 to +50 percent. The estimated costs were calculated using the following equation:

	Baseline Pipe Construction Cost
+	5% Mobilization/Demobilization
=	***Estimated Construction Cost Subtotal***
+	30% Contingencies for Unknown Conditions
=	***Estimated Construction Cost Total***
+	25% Engineering, Administration, and Legal Costs
=	**Estimated Capital Improvement Cost Total**

Unit costs for pipe construction were derived from data developed by MWH for similar projects and bid results on past Ross Valley sewer rehabilitation and replacement projects. **Appendix D** includes itemized estimates of probable construction and capital improvement costs for each of the proposed relief projects. Note that the costs presented in this report represent typical construction costs for the San Francisco Bay Area in 2006, as represented by an approximate Engineering News-Record (ENR) Construction Cost Index (CCI) value of 8444 for San Francisco in March 2006.

The proposed capacity improvement projects were prioritized based on the relative severity of identified capacity deficiencies in the existing trunk sewer system (as evidenced by the extent of model-predicted overflows or surcharge, or actual historical overflows or surcharging during large storm events). The recommended project priorities are also listed in **Table 6-2**.

Table 6-2. Proposed Capacity Improvement Project Priorities and Estimated Costs

SHECAP Project No.	Project Name	Estimated Capital Cost ($)	Priority	Comments
1	Westbrae / Hawthorne	$ 424,000	19	Predicted surcharge near surface
2	Spruce / Park / Merwin / Broadway	$ 1,750,000	8	Predicted overflow, new relief line fixes overflow on existing line
3	Cascade	$ 572,000	11	Predicted overflow
4[1]	Creek / Bolinas	$ 1,675,000	9	Predicted overflow
5	Upper Butterfield	$ 1,582,000	10	Predicted overflow
6	Lower Butterfield / Meadowcroft / Broadmoor / SFD	$ 1,980,000	13	Predicted overflow, new relief line fixes overflow on existing line
7	The Alameda / Brookmead	$ 764,000	16	Predicted surcharge within 3 ft of surface
8	Sonoma / Nokomis	$ 1,785,000	14	Predicted overflow, new relief line fixes overflow on existing line near siphon
9	Miracle Mile	$ 1,743,000	4	Predicted overflow
10	Sir Francis Drake / Winship	$ 975,000	3	Predicted overflow at upstream end of one line
11	Bolinas / Fernhill	$ 1,074,000	17	Predicted surcharge near surface for one leg, within 2 ft of surface for other leg
12[2]	Upper Shady Lane Trunk Sewer	$ 913,000	2	Predicted surcharge within 7 ft of surface - Main trunk line should be fixed along with project 10 for reported overflow.
13	Sir Francis Drake / Berry	$ 471,000	20	Predicted surcharge near surface
14	Goodhill	$ 767,000	5	Predicted overflow
15	Woodland / College	$ 1,306,000	6	Predicted overflow
16	Kentfield Relief Sewer	$ 999,000	1	This project should be done before projects 14 and 15, otherwise the additional flow that would be conveyed from the upstream improvements could flood the downstream siphon.
17	Laurel Grove / McAllister	$ 949,000	12	Predicted overflow
18	Manor Easement	$ 338,000	21	Predicted surcharge within 1 ft of surface
19	William / Holcomb / Meadowood	$ 1,303,000	7	Predicted overflow
20	Magnolia	$ 836,000	15	Predicted overflow at very upstream segment, otherwise surcharge within 3 ft of surface
21	Eliseo	$ 66,000	18	Predicted surcharge within 5 ft of surface

1. A major portion of this project is currently in design under the sewer rehabilitation program.
2. This trunk sewer may need structural rehabilitation due to age and condition.

The improvement projects were grouped into 5- and 10-year Capital Improvement Programs (CIPs). The Capital Improvement Programs, with projects listed in priority order, are summarized in **Table 6-3**. **Figure 6-6** shows the location of the 5- and 10-year projects. The project priorities should be modified as needed based on other system needs such as sewer rehabilitation to address structural deficiencies or maintenance issues. The SHECAP priorities and CIP will be incorporated into the District's overall CIP as part of its Sewer System Assessment and Capital Project Planning program.

Table 6-3. Trunk Sewer Capacity Capital Improvement Program

Proposed CIP Implementation Schedule	Priority	SHECAP Project No.	Project Name	Estimated Capital Cost ($)
5-Year	1	16	Kentfield Relief Sewer	$ 999,000
	2	12	Upper Shady Lane Trunk Sewer	$ 913,000
	3	10	Sir Francis Drake / Winship	$ 975,000
	4	9	Miracle Mile	$ 1,743,000
	5	14	Goodhill	$ 767,000
	6	15	Woodland / College	$ 1,306,000
	7	19	William / Holcomb / Meadowood	$ 1,303,000
	8	2	Spruce / Park / Merwin / Broadway	$ 1,750,000
	9	4	Creek / Bolinas	$ 1,675,000
Subtotal 5-Year				*$ 11,431,000*
10-Year	10	5	Upper Butterfield	$ 1,582,000
	11	3	Cascade	$ 572,000
	12	17	Laurel Grove / McAllister	$ 949,000
	13	6	Lower Butterfield / Meadowcroft / Broadmoor / SFD	$ 1,980,000
	14	8	Sonoma / Nokomis	$ 1,785,000
	15	20	Magnolia	$ 836,000
	16	7	The Alameda / Brookmead	$ 764,000
	17	11	Bolinas / Fernhill	$ 1,074,000
	18	21	Eliseo	$ 66,000
	19	1	Westbrae / Hawthorne	$ 424,000
	20	13	Sir Francis Drake / Berry	$ 471,000
	21	18	Manor Easement	$ 338,000
Subtotal 10-Year				*$ 10,841,000*
Total				**$ 22,272,000**

COST-EFFECTIVENESS OF I/I REDUCTION

A preliminary, gross-level analysis was performed to assess the potential cost-effectiveness of sewer system rehabilitation to reduce I/I in sewer subbasins upstream of identified capacity deficiencies. Sewer system rehabilitation of non-trunk sewer mains and lower laterals were assumed to yield an estimated 30 percent reduction in I/I. This assumption is based on experience from sewer rehabilitation projects throughout the country, which show a very wide range of I/I reductions, but which overall have demonstrated that significant reductions (e.g., 50 percent or higher) can only be achieved if private service laterals are also included (Reference: "Reducing Peak Rainfall-Derived Infiltration/Inflow Rates – Case Studies and Protocol," Water

Environment Research Foundation, 2003). These results are derived from systems such as RVSD, where direct inflow has not been found to be a significant source of I/I.

The analysis was performed by reducing RDI/I by 30 percent over the entire service area. (I/I during peak flows consists primarily of RDI/I. In the District, I/I due to GWI comprises less than 5 percent of the total I/I during the design storm event. Therefore, only RDI/I was reduced for this analysis.)

The model was run with the reduced I/I under design storm conditions. Each improvement project was reviewed and the projects that would not be necessary under the 30 percent I/I reduction scenario were identified. **Table 6-4** summarizes the results of the analysis.

Table 6-4. Improvement Project Summary after System-wide 30 Percent I/I Reduction

SHECAP Project No.	Project Name
Project Not Necessary after I/I Reduction	
1	Westbrae / Hawthorne
7	The Alameda / Brookmead
8	Sonoma/Nokomis
12	Upper Shady Lane Trunk Sewer
13	Sir Francis Drake / Berry
16	Kentfield Relief Sewer
20	Magnolia
21	Eliseo
Most of Project Not Necessary after I/I Reduction	
9	Miracle Mile
17	Laurel Grove / McAllister
Project Necessary after I/I Reduction	
2	Spruce / Park / Merwin / Broadway
3	Cascade
4	Creek/Bolinas
5	Upper Butterfield
6	Lower Butterfield / Meadowcroft / Broadmoor / SFD
10	Sir Francis Drake / Winship
11	Bolinas / Fernhill
14	Goodhill
15	Woodland/College
18	Manor Easement
19	William / Holcomb / Meadowood

Table 6-4 shows that of the 21 improvement projects, 8 of the projects would not be necessary if sewer rehabilitation achieved a 30 percent I/I reduction. For two other projects, most of the pipeline improvements would not be necessary. For the remaining 11 projects, the improvements would still be necessary to fix capacity deficiencies, but perhaps with smaller diameter piping.

The projects that would not be needed or only partially needed were analyzed in more detail in order to compare the costs of the capacity improvement projects versus the I/I reduction (sewer rehabilitation) costs. For each capacity relief project that could potentially be eliminated or substantially reduced through upstream I/I reduction, the subbasins tributary to the project were identified, and the costs for rehabilitation of the non-trunk sewer mains and lower laterals in those subbasins were calculated. The costs were based on the assumption that pipe bursting would be the rehabilitation method used. (Note that Projects 12 and 16 were not analyzed because they would serve a very large part of the District service area.) **Table 6-5** compares the capital cost of the improvement projects that could be eliminated (relief project cost savings) with the estimated cost of sewer rehabilitation to reduce I/I.

Table 6-5. Cost Effectiveness of 30 Percent I/I Reduction

SHECAP Project No.	Project Name	Tributary Subbasins	Total Length of Sewer Mains (ft)	Estimated Relief Project Capital Cost Savings	Estimated I/I Rehabilitation Cost
[0]1	Westbrae / Hawthorne	01A,F	20,724	$ 424,000	$ 5,103,881
7	The Alameda / Brookmead	03A-G, 04A,C	101,403	$ 764,000	$ 24,426,102
8	Sonoma / Nokomis	06A,B,C	21,188	$ 1,785,000	$ 5,808,744
9	Miracle Mile	07A,B,C,D	51,001	$ 1,743,000	$ 12,985,758
13	Sir Francis Drake / Berry	09A	18,432	$ 471,000	$ 4,012,105
17	Laurel Grove / McAllister	12A, 12B	28,858	$ 949,000	$ 7,127,663
20	Magnolia	17A,B,C,D	26,455	$ 836,000	$ 6,281,044
21	Eliseo	18A	17,123	$ 66,000	$ 4,275,334
Total			*285,185*	*$ 7,038,000*	*$ 70,020,633*

As **Table 6-5** shows, the costs for sewer rehabilitation to achieve 30 percent I/I reduction is about 10 times more than the potential cost savings in the corresponding capacity improvement projects. Reduction in I/I flows could also result in some potential cost savings for treatment and disposal of the wastewater at CMSA facilities, but it is unlikely that these additional savings would be significant in comparison to the costs for the sewer rehabilitation that would be needed to achieve the reduction in I/I.

CONCLUSIONS AND RECOMMENDATIONS

The results of flow monitoring and hydraulic modeling of the District's wastewater collection system indicate that the system has adequate capacity to convey peak dry weather flows, but has substantial capacity deficiencies that could potentially result in sanitary sewer overflows during large storm events. This study identifies trunk sewer improvement projects which, if constructed, would increase system capacity and thereby reduce the risks of wet weather overflows. Although the focus of this study has been on expanding trunk conveyance capacity, sewer rehabilitation, as part of an overall program to upgrade the District's wastewater collection system, should help reduce I/I in the system over the long-term and reduce the risk of SSOs. I/I should be used as a factor, along with maintenance history, age, pipe size, material, and other considerations, to prioritize sewer condition assessment and rehabilitation efforts. The need for lower priority capacity improvement projects should be re-evaluated at the time of proposed implementation to determine if sewer rehabilitation has or could eliminate the need for some of these projects.

The following recommendations are made based on the results and conclusions of this study:

- The District should budget for construction of the highest priority projects as soon as possible and initiate predesign studies to confirm the viability of the proposed alignments and the estimated construction costs.

- The alignments and sizes of all recommended projects should be verified with detailed predesign analyses, including topographic surveys, geotechnical investigations, utility research, and constructability reviews. Viable alternative alignments should also be considered during predesign.

- The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.

- The District should advise CMSA of the updated flows projections for the RVSD service area based on the relief improvements identified in this study so that CMSA can assess the potential impact of those flows on downstream treatment and disposal facilities.

- While I/I reduction does not appear to be cost effective solely from the standpoint of reducing the need for capacity improvement projects, I/I may increase in the future if the condition of the sewer system continues to deteriorate. Therefore, an on-going I/I prevention and sewer system condition assessment and rehabilitation program is recommended, as discussed in the following section of the report.

Section 7 – I/I Prevention Plan

Infiltration and inflow of stormwater and groundwater into the District's wastewater collection system is the major cause of the hydraulic capacity deficiencies in the system. While sewer rehabilitation to reduce I/I may not be cost effective solely for the purposes of reducing the extent of required hydraulic capacity improvements in the system, I/I may increase in the future if the system continues to deteriorate. Therefore, a long-term program to control I/I as part of an on-going sewer condition assessment and rehabilitation program is recommended. This section describes methods used to detect and control I/I, and provides general recommendations for long-term approaches for controlling I/I in the RVSD system.

SUBBASINS WITH HIGH I/I

The magnitude of I/I in the RVSD collection system was discussed previously in Section 3. I/I flows were quantified based on flow monitoring data from the year 2004-05 wet weather season. The flow monitoring data and resultant modeling showed that peak flow rates during large storms range from approximately 5 to 15 times average dry weather flows, and that many sewers in the system surcharge under wet weather flow conditions.

The high I/I flows in the RVSD collection system reflect both the high rainfall that falls in the local area, which is one of the wettest portions of the San Francisco Bay Area, as well as the hydrogeology of the District and the condition of the sewer system. The condition of the system is comparable to other Bay Area systems of similar size, age, and location, which also experience high I/I.

Based on the results of the analyses conducted for this study, some areas of the District appear to have higher rates of GWI and/or RDI/I than others. Based on the information presented in **Table 3-2** and in **Figures 3-6**, **3-8**, and **3-9**, the following areas (designated based on flow meter basin) appear to have the highest I/I rates (greater than 600 gpd/acre GWI and/or greater than or equal to 8% rainfall runoff RDI/I):

- Meter 02 (GWI, RDI/I) – Along Bolinas Road, south of Park Road, in Fairfax
- Meter 04 (RDI/I) – Along Butterfield Road, north of Sir Francis Drake, in San Anselmo
- Meter 06 (GWI, RDI/I) – Along San Francisco Boulevard, north of Sir Francis Drake, in San Anselmo
- Meter 11 (GWI, this area did not have a high peak RDI/I response, but the I/I continued much longer after the rainfall than at any other meter) – Along Woodland Road, in Kent Woodlands
- Meter 13 (RDI/I) – East of Wolfe Grade, north of Sir Francis Drake, along border of Kentfield and Greenbrae.
- Meter 16 (RDI/I) – Along Magnolia Avenue, south of Doherty Drive, in Larkspur
- Meter 17 (RDI/I) – Along Magnolia Avenue, north of Doherty Drive, in Larkspur
- Meter 18 (GWI) – Just west of US 101, north and south of Sir Francis Drake, in Larkspur
- Meter 19 (GWI) – Just east of US101, north of Sir Francis Drake, in Larkspur

These areas should be considered priority areas for potential sewer rehabilitation efforts to control I/I in the District's system.

Sources of I/I

A necessary step in identifying potential I/I control measures is a realistic assessment of the actual sources of I/I in the collection system. Based on the pattern and magnitude of flows in the District's collection system, the likely sources of both GWI and RDI/I flows are defects in pipes, manholes, and sewer laterals, and possibly some direct connections (e.g., illegally connected roof and area drains, direct connections from the storm drain system, etc.) Previous closed-circuit television (CCTV) inspection of sewers in the system has shown typical sewer defects in older sewers (offset joints, cracks, root intrusion, defective lateral connections, etc.) that are contributors to I/I.

I/I SOURCE DETECTION AND CONTROL METHODS

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 below.

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 primary 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 (in terms of cost per foot of sewer 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, system-wide program, which requires public notification measures and access onto private property to document the smoke returns. Over the past ten years, the District has conducted smoke testing programs in areas of suspected inflow, but has found relatively few direct connections. However, additional smoke testing targeted at specific areas with high peak RDI/I rates may be warranted if there is reason to suspect that direct inflow sources may exist.

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.

Manholes subject to ponding or located in drainage courses are also considered to 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 CCTV inspection work, or as a separate effort.

Some agencies have used manhole inserts to reduce inflow into manholes. Inserts are typically plastic “dishes” placed under the manhole cover to collect water that enters through the cover or around the frame. The inserts are generally inexpensive; however, they have been found to be maintenance headaches (often debris and other material collect in the dish, the insert may become dislodged and fall into the manhole, and it requires removal for accessing the manhole for cleaning or inspection). For manholes subject to significant inflow, more permanent corrective measures, as noted above, are preferred.

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 is a significant, often prohibitive, institutional problem (see discussion below on correction of private property I/I sources).

Infiltration Sources in Sewer Mains and Manholes

Infiltration sources are primarily 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 are detected by inspection: visual inspection in the case of manholes, CCTV inspection for sewer mains. Inspection during wet or high groundwater periods is desirable to increase the probability of observing actual infiltration.

Infiltration correction methods generally 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 generally 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 effect 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 volume 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.

Isolation of areas of infiltration can be done by flow monitoring or other flow measurement techniques. A traditional technique used to isolate areas or pipe segments with high GWI is called nighttime flow isolation, in which instantaneous measurements of flow are taken manually in sewer manholes using hand-held weirs, typically during the nighttime or early morning hours when sanitary flows are at a minimum. Flow isolation, as well as flow monitoring, are both labor-intensive activities which do not always yield conclusive results (due to the errors inherent in accurately measuring flows in sewers). They are most effective where very localized problems are suspected (e.g., a sewer located adjacent to a creek, an area of the sewer system installed by a specific contractor known to have used substandard materials or installation practices, etc.). If problems are more widespread and/or not particularly severe, then it may be more practical to include inspection and correction as part of a long-term cyclic inspection and renewal/replacement program rather than as a one-time rehabilitation effort.

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 using a special type of camera, or tested for leaks using air or water pressure tests. These types of inspections and tests require that the lateral have cleanout access, ideally at both the connection to the building plumbing and at or near the property line. Lateral inspection and testing is labor-intensive and requires an intensive public relations effort. In general, such programs are only conducted where I/I problems are severe and specific actions are required to meet administrative orders or regulatory requirements.

Correction of private property defects can be expensive due to the necessity of manual excavation of trenches and measures to protect and/or restore improvements such as driveways, fences, patios, and landscaping. Work on private property also involves issues relating to who pays, as it is often politically or legally impossible to utilize public funds for improvements to private property, and difficult to get property owners to make sewer improvements without a legal basis for requiring them.

One method that has been implemented by a number of sewerage agencies is an ordinance requiring testing or inspection of the sewer lateral at the sale of the property. If the lateral fails to pass the inspection or test, then appropriate repairs must be made before the sale can close. The cost of the repairs can be added to the sale price or closing costs. In some areas, this approach has met with resistance from the real estate community. In other areas, where the problems caused by I/I and the need for sewer and lateral rehabilitation was effectively communicated to the community, a lateral ordinance was considered to be the most effective way to implement a private property rehabilitation program with the least financial impact.

RECOMMENDATIONS

- The District should address I/I on an area-specific basis. Targeted I/I source detection and correction should be considered in areas with particularly high I/I flows. In any areas with suspected direct inflow connections to the sewer system (directly connected roof or area drains or storm drain cross-connections), smoke testing could be conducted to identify potential sources.

- The District should implement an on-going program of closed-circuit television inspection of the sewer system to identify sewers with structural defects and potential sources of I/I, and implement a long-term renewal and replacement program to preserve the District's sewer pipeline assets and address identified problems before they become more severe.

- The District should develop a comprehensive SSMP following the guidelines developed by the San Francisco Bay Regional Water Quality Control Board and Bay Area Clean Water Agencies (BACWA). The SSMP includes a number of components to ensure proper and effective management, operation, and maintenance of wastewater collection systems. The work conducted for this study provides a foundation for preparation of a Capacity Management Plan, one of the key components of the SSMP.

Specific recommendations for a long-term sewer condition assessment and rehabilitation and replacement program are being developed as part of the Sewer System Management Plan and Sewer Replacement Master Plan in the District's ongoing Sewer System Assessment and Capital Project Planning program.

Appendix A

REFERENCES

1. National Oceanic and Atmospheric Administration, *NOAA Atlas 2, Volume 1*, 1973, http://www.nws.noaa.gov/ohd/hdsc/noaaatlas2.htm.
2. Nolte, *Interceptor Network Hydraulic Model Final Report*, September 2004.
3. U.S. Department of Agriculture, *Urban Hydrology for Small Watersheds (TR-55)*, 1986, ftp://ftp.wcc.nrcs.usda.gov/downloads/hydrology_hydraulics/tr55/tr55.pdf.

Appendix B

CALIBRATION PLOTS

This appendix contains plots of the calibration for the 2 dry weather events and the 2 storm events at each of the 20 flow meters. The plots come from the InfoWorks model.

PHOTOGRAPHS OF PROPOSED SEWER ALIGNMENTS

This appendix contains photographs of the alignments proposed for the improvements in the Capital Improvements Plan.

Appendix D

ITEMIZED COST ESTIMATES

This appendix contains the detailed data used to develop the improvement project cost estimates.

Verification Results - FM01 (F003.02.1)

Flow Monitoring　Model Simulation

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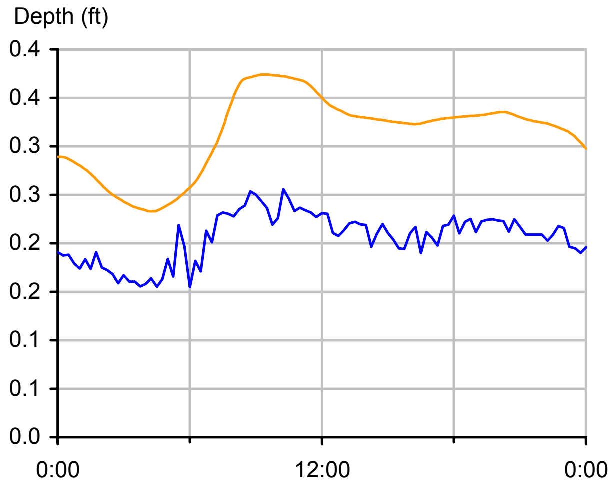

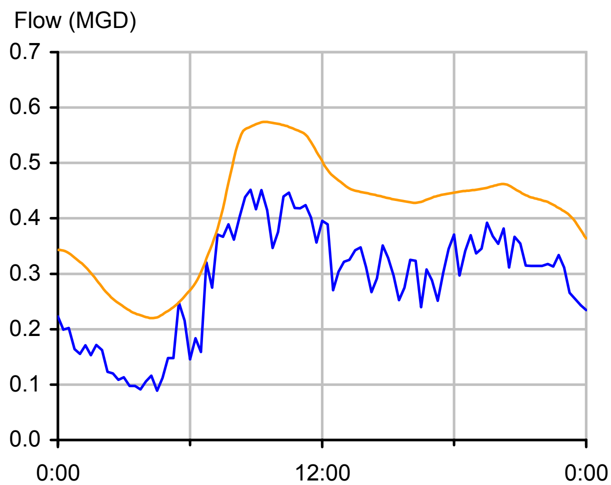

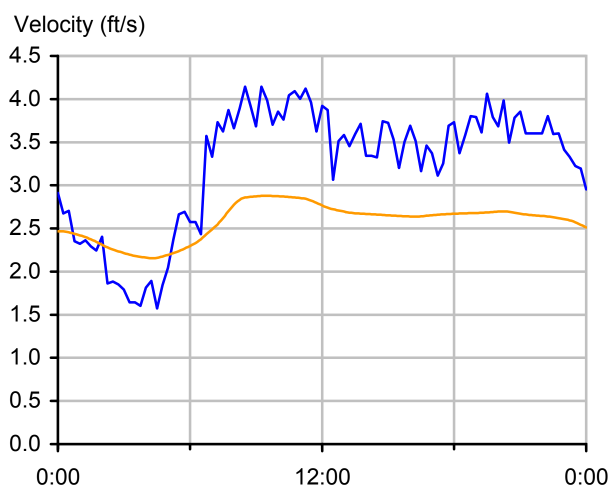

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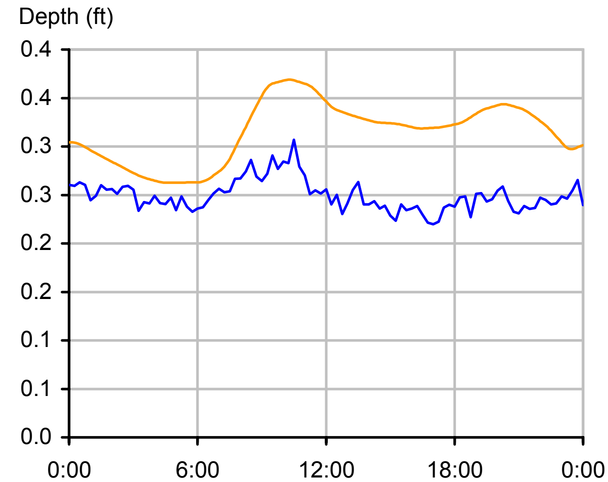

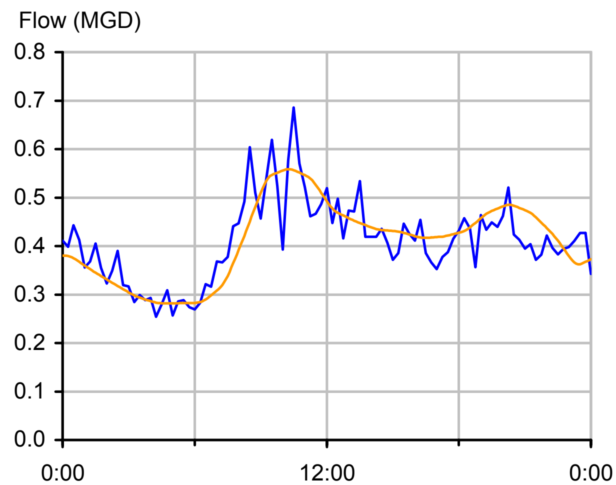

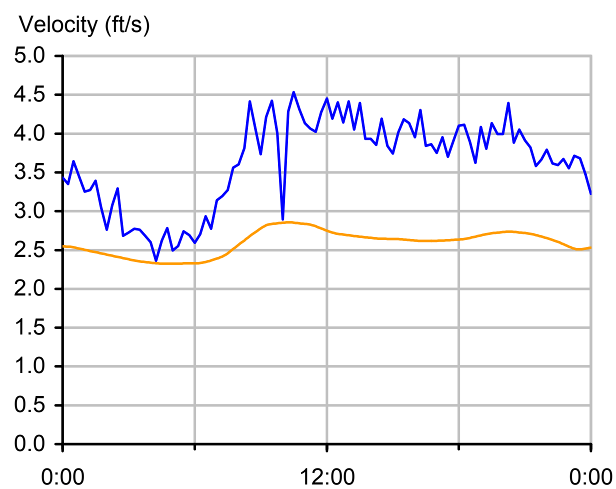

Verification Results - FM02 (F002.11.1)

Flow Monitoring — Model Simulation

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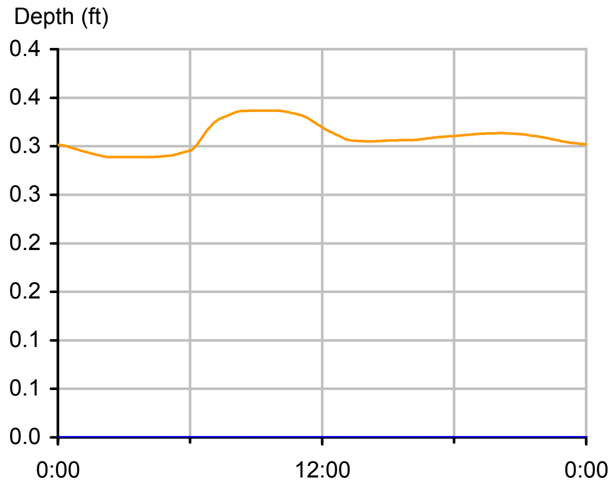

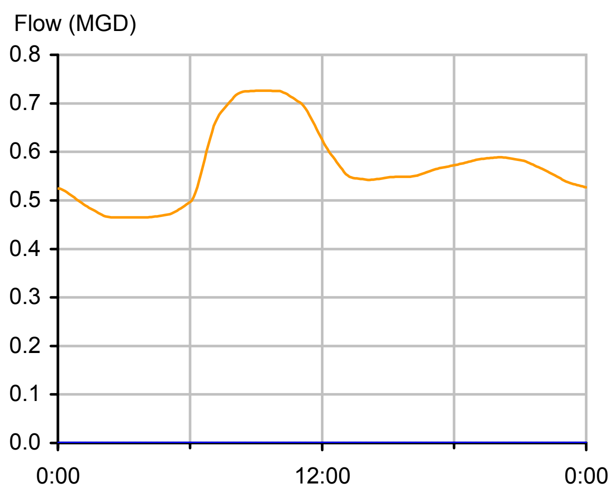

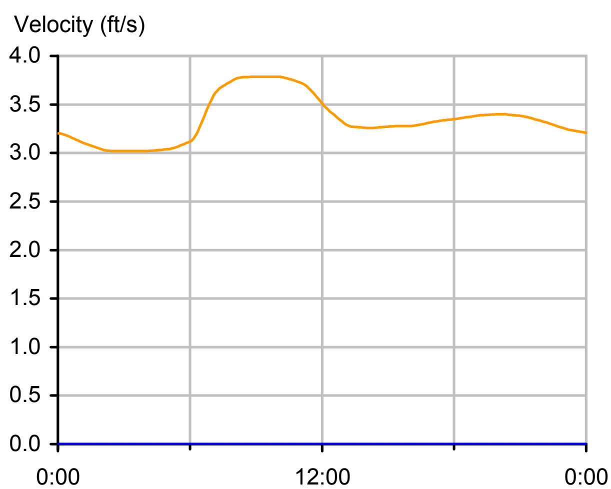

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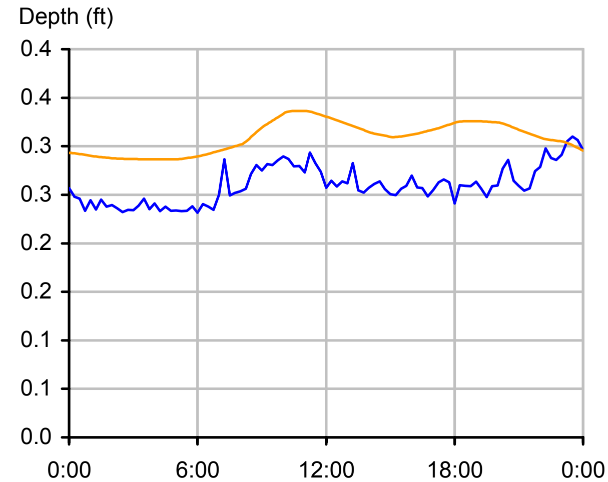

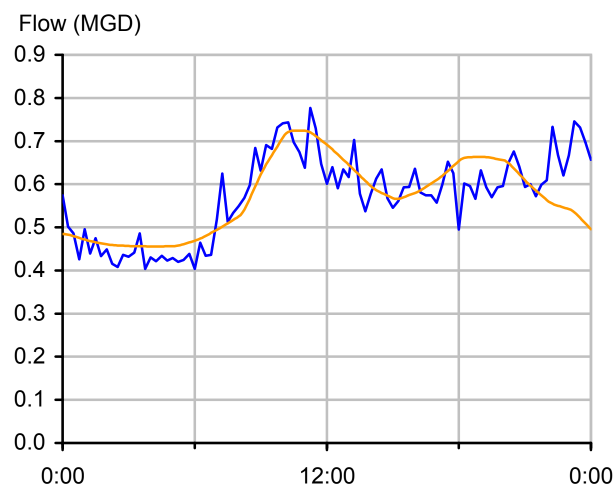

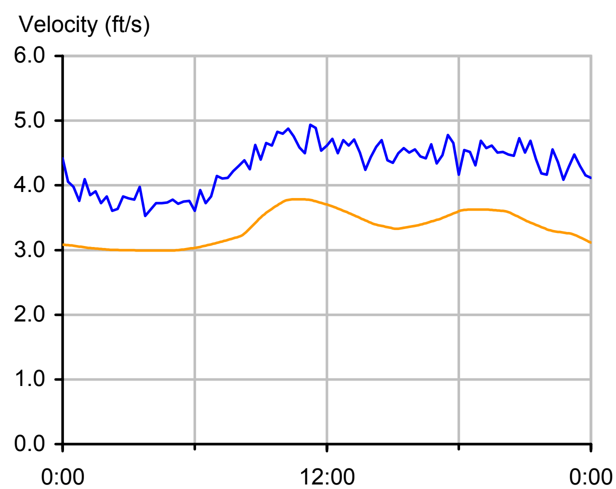

Verification Results - FM03 (S900.23.1)

Flow Monitoring   Model Simulation

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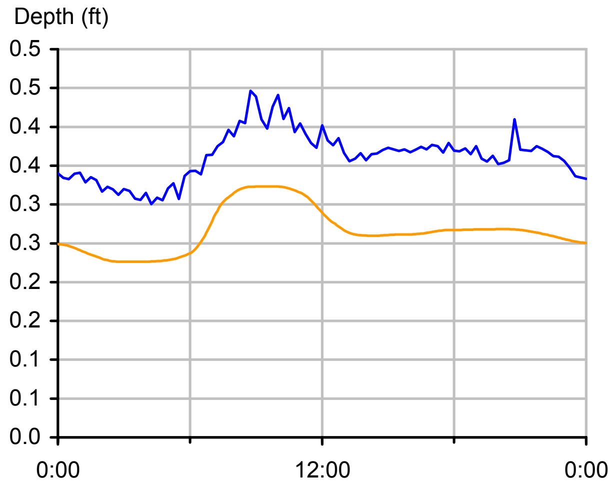

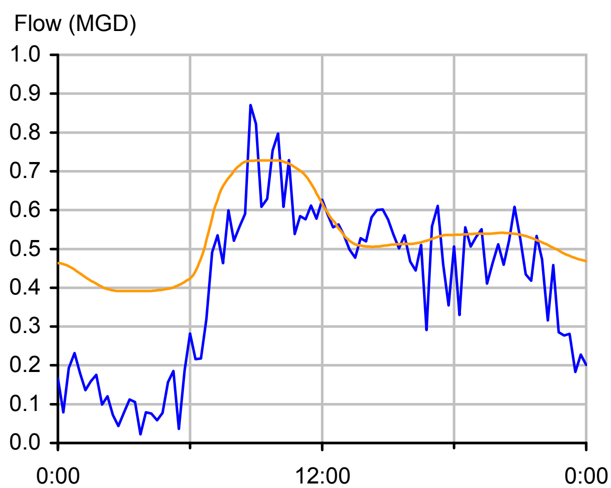

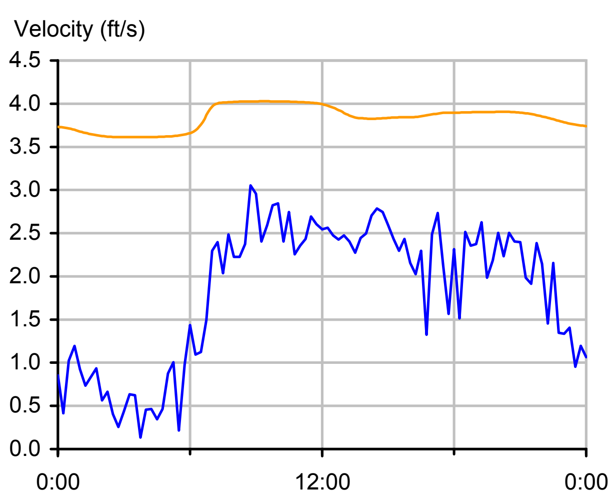

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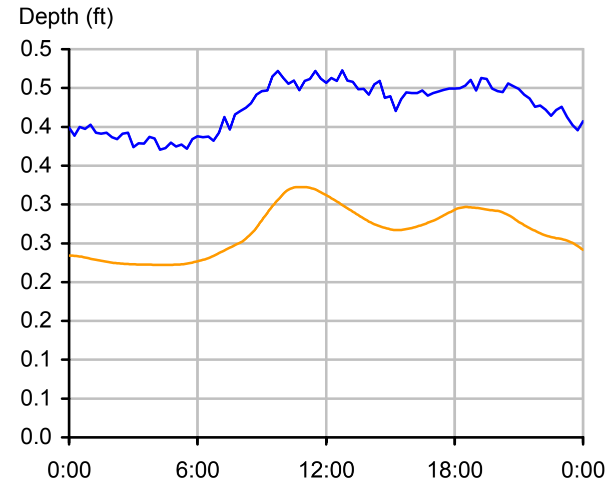

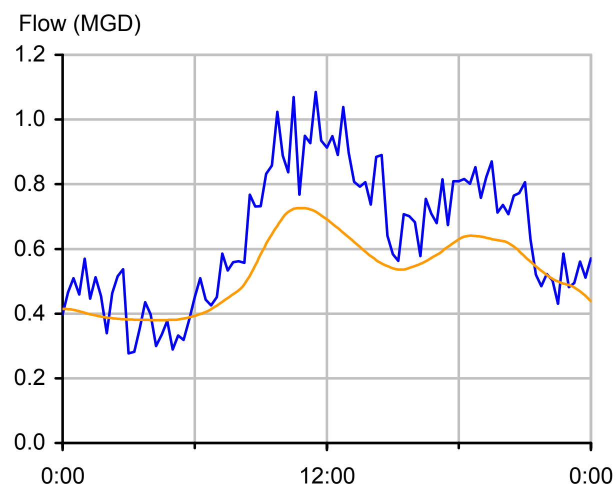

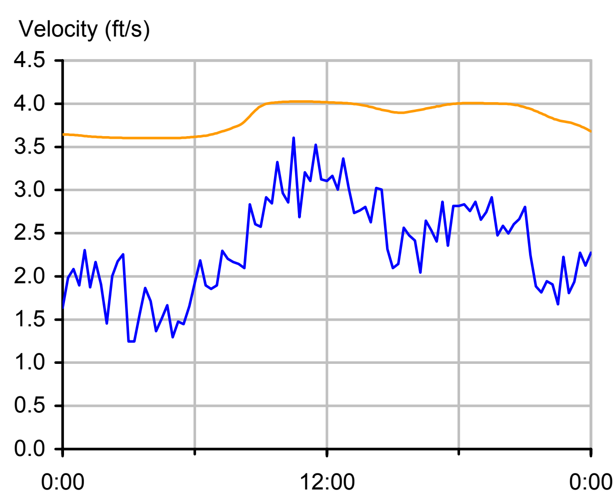

Verification Results - FM04 (S800.02.1)

Flow Monitoring — Model Simulation

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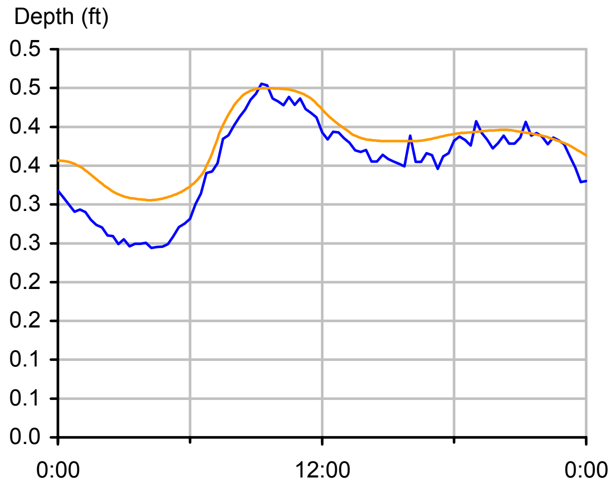

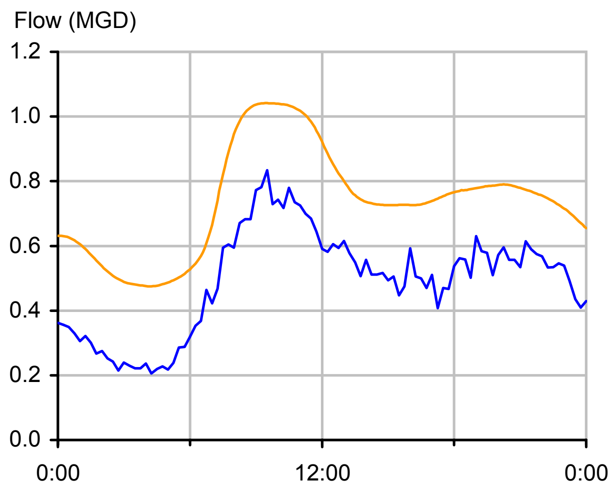

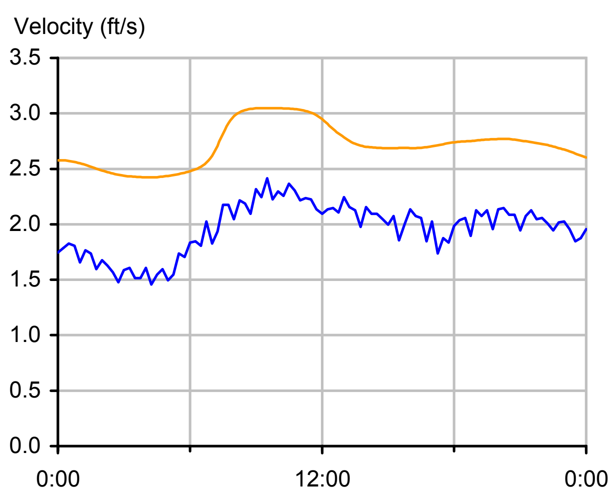

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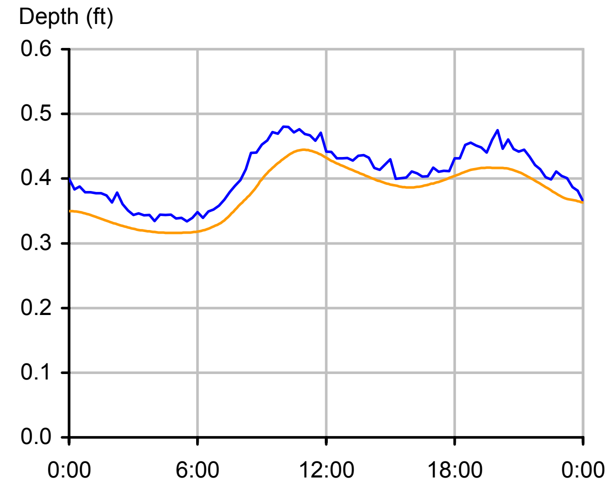

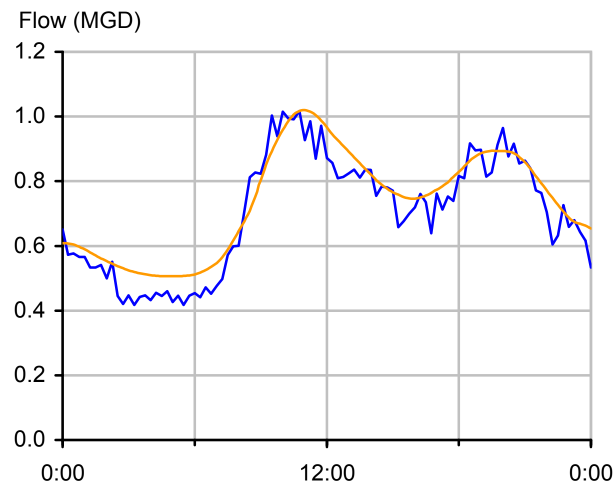

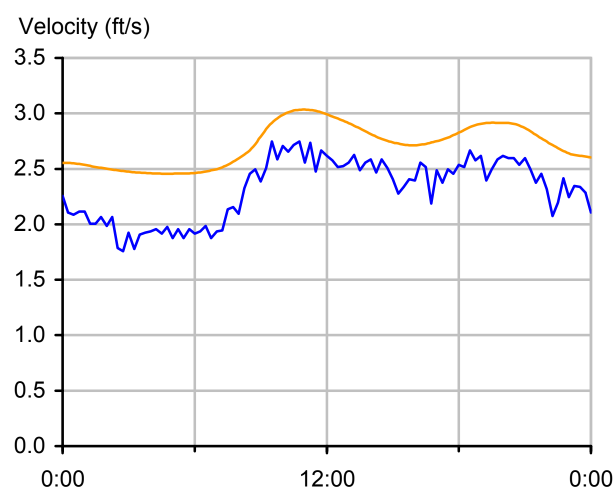

Verification Results - FM05 (S000.54.1)

Flow Monitoring   Model Simulation

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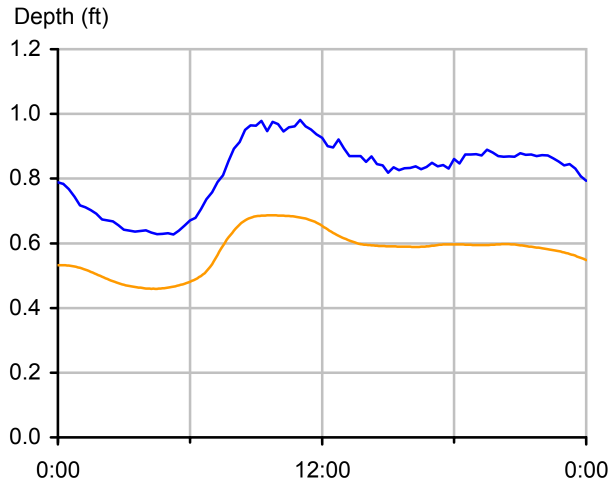

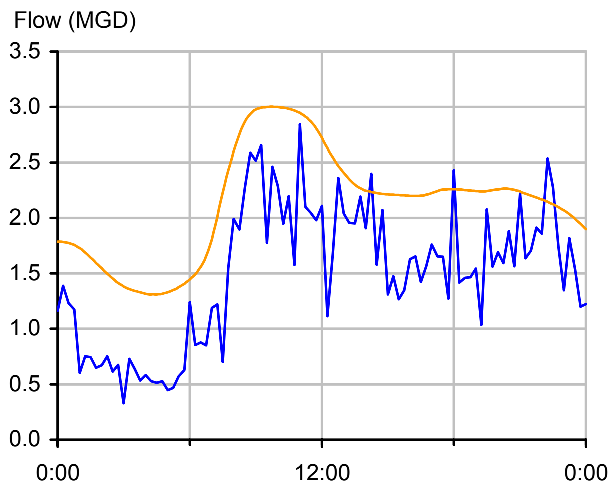

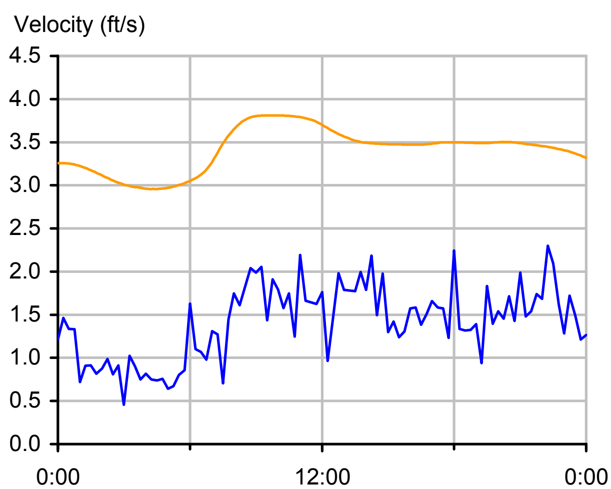

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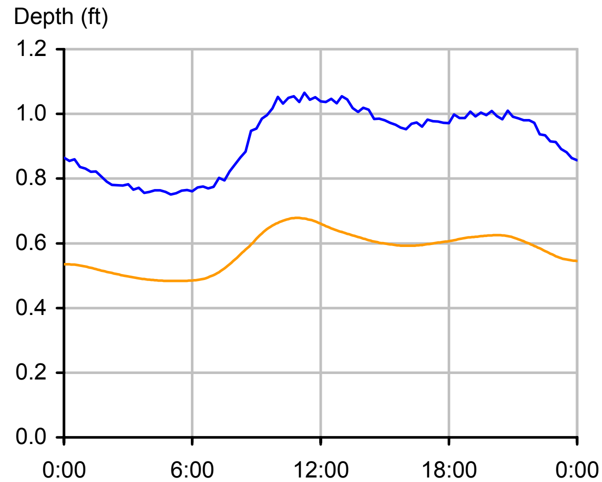

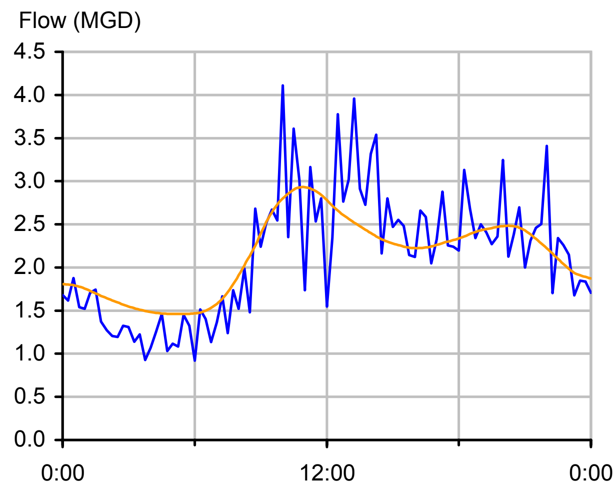

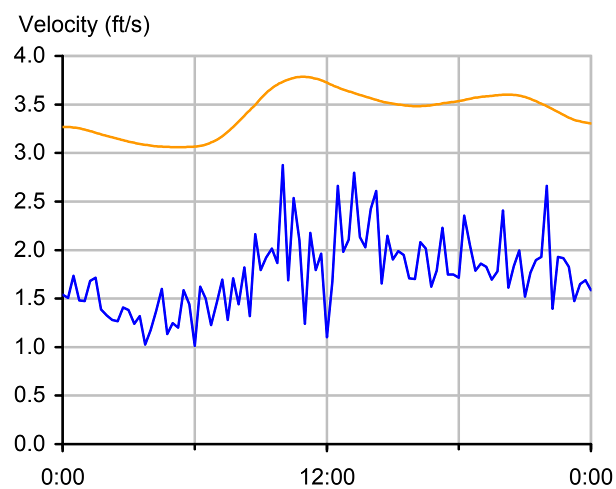

Verification Results - FM06 (S600.02.1)

Flow Monitoring   Model Simulation

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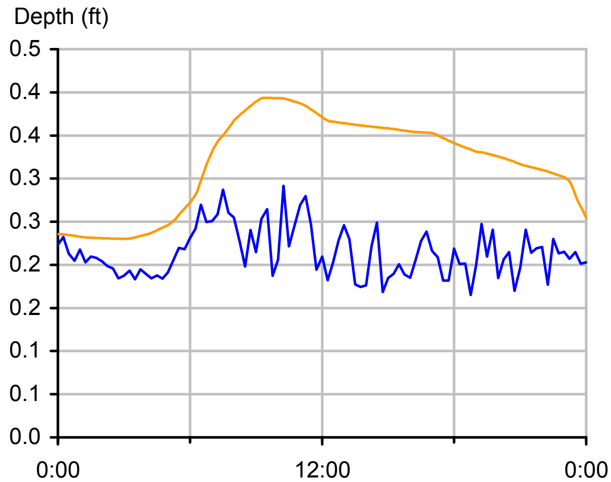

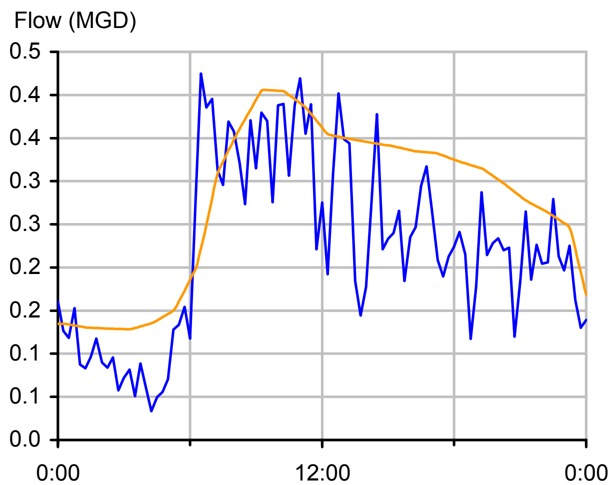

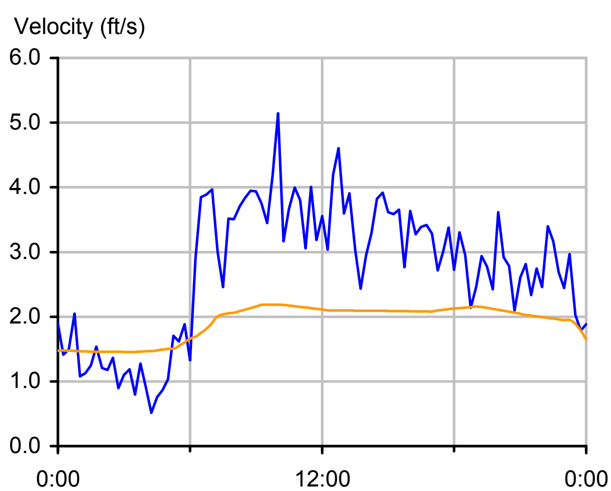

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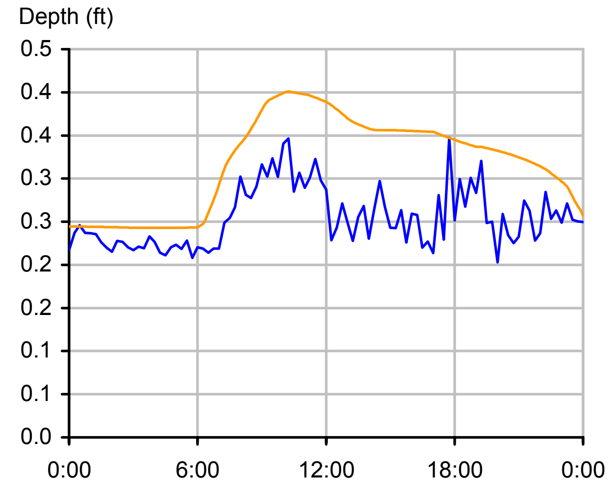

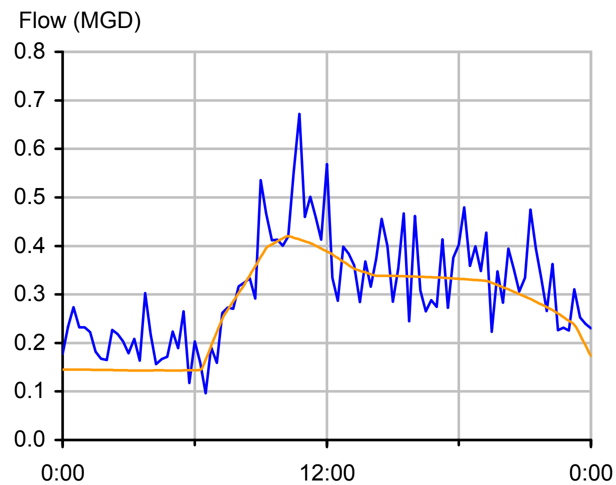

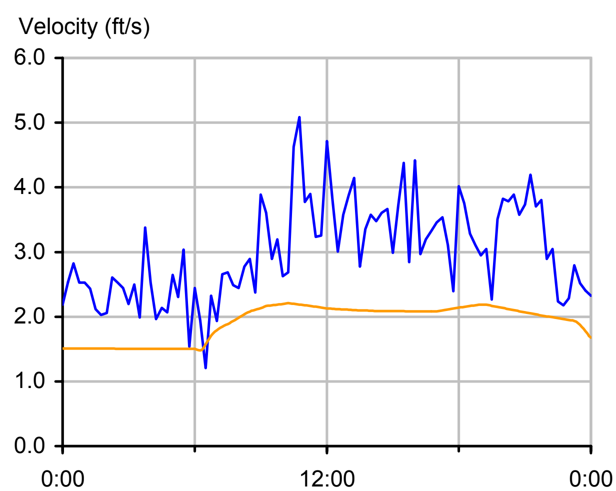

Verification Results - FM07 (S001.01.1)

Flow Monitoring — Model Simulation

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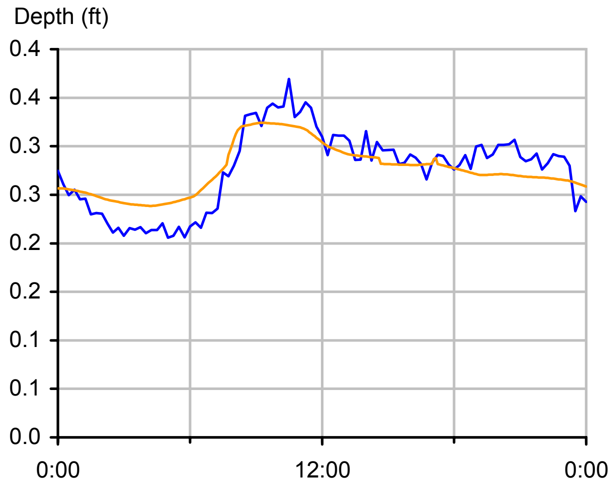

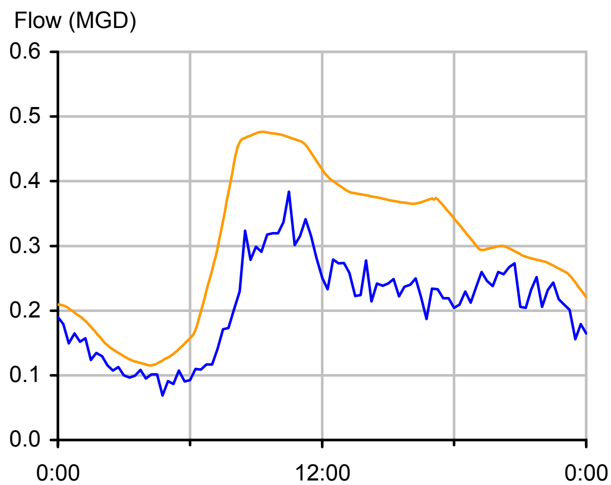

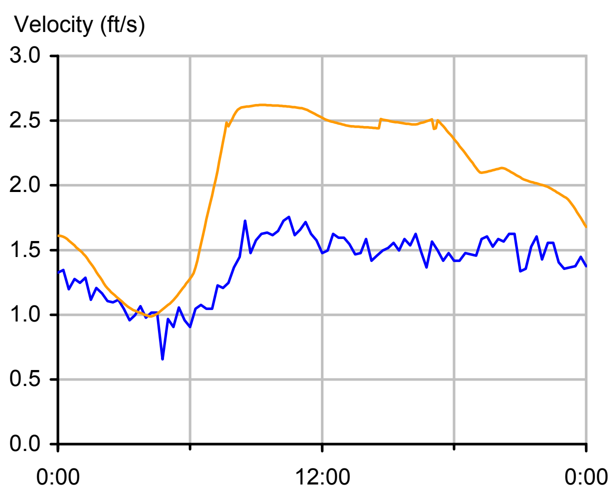

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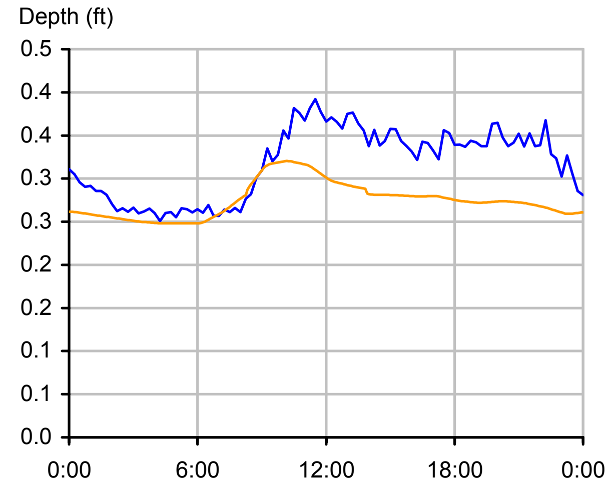

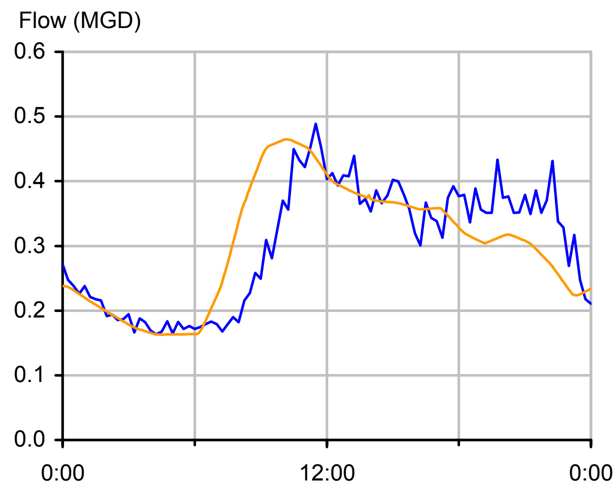

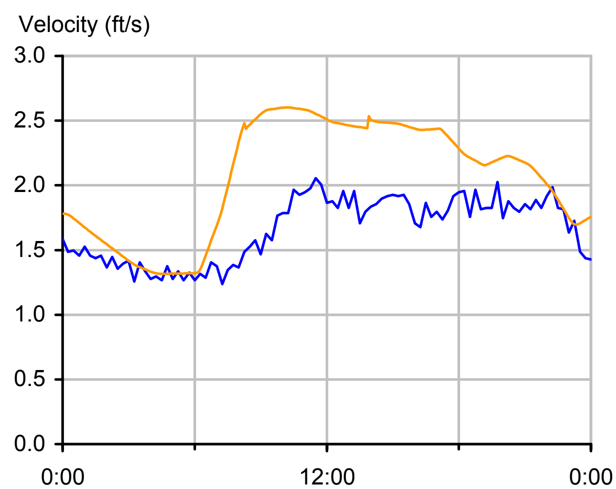

Verification Results - FM08 (R000.31.1)

Flow Monitoring   Model Simulation

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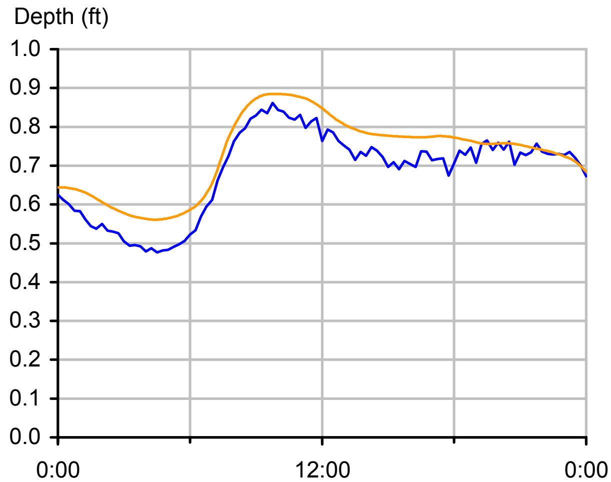

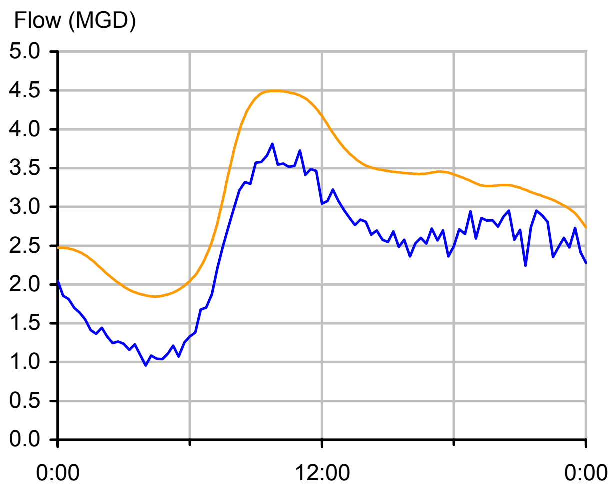

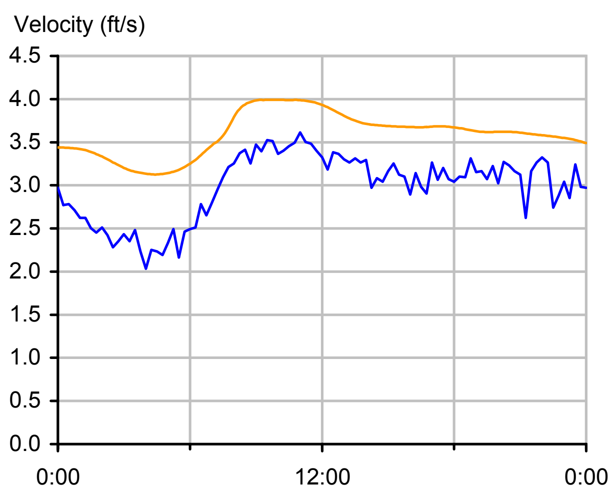

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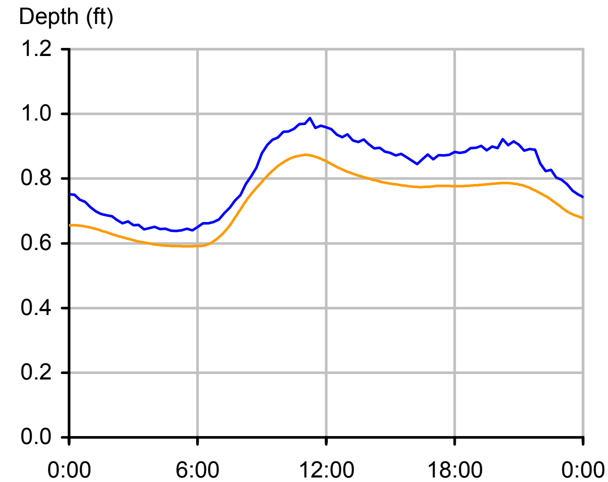

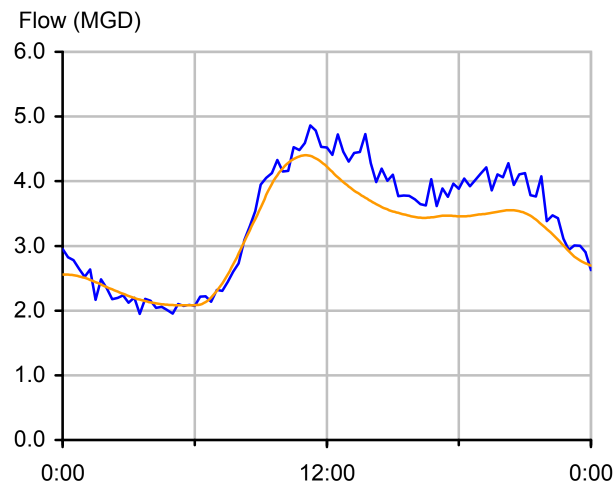

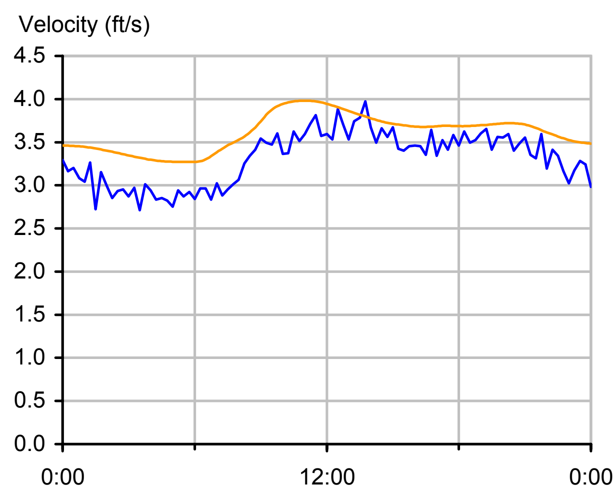

Verification Results - FM09 (K000.05.1)

Flow Monitoring — Model Simulation

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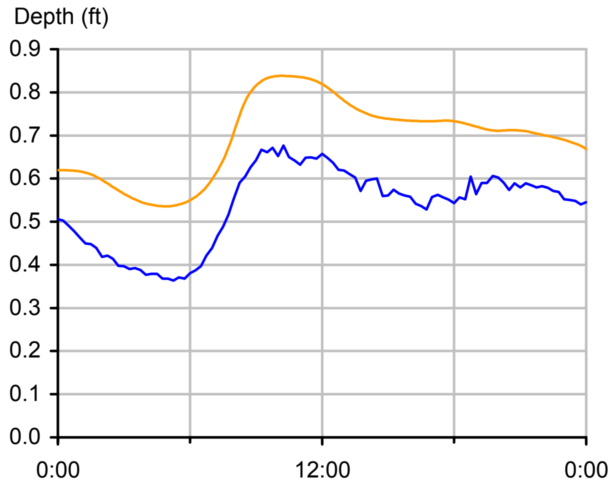

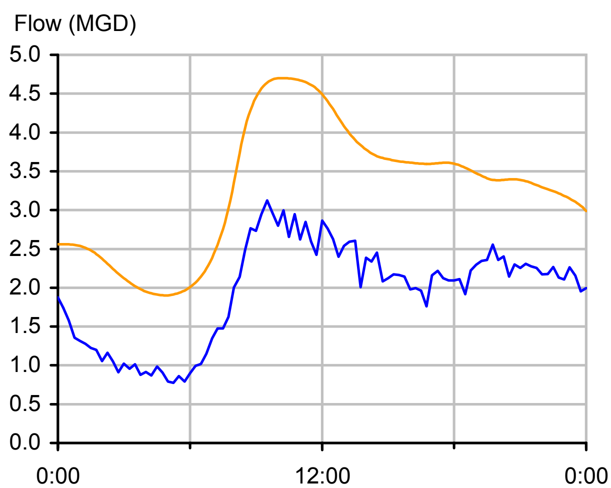

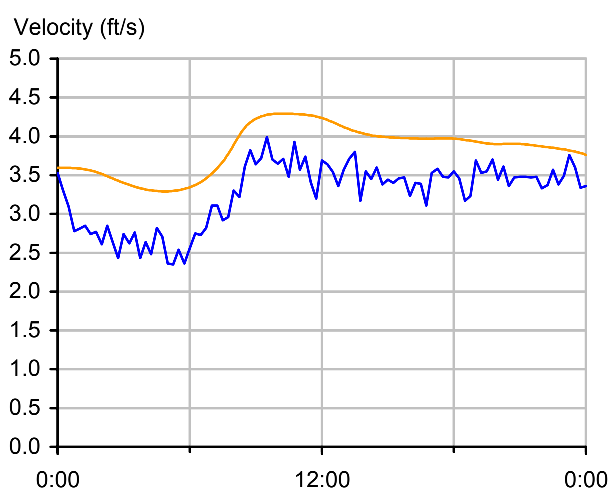

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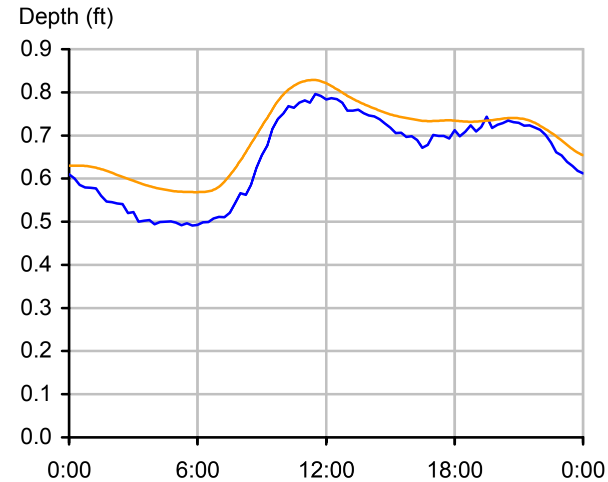

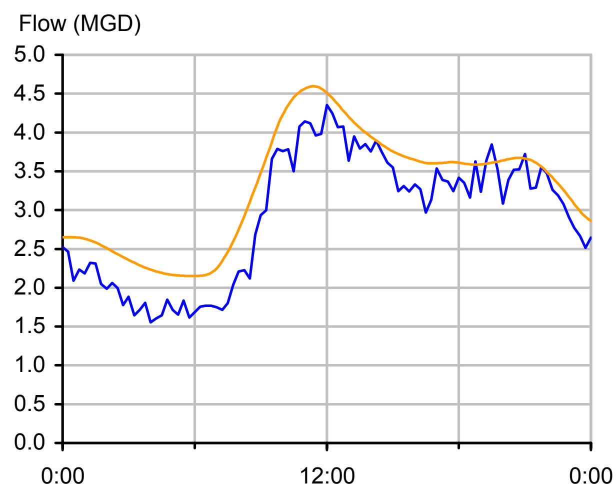

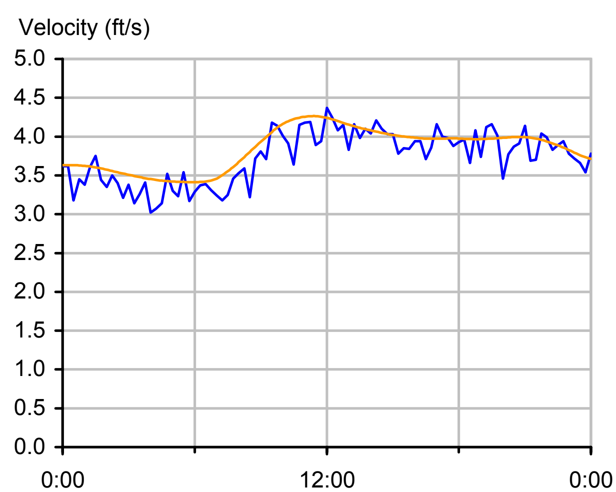

Verification Results - FM10 (K100.24.1)

Flow Monitoring   Model Simulation

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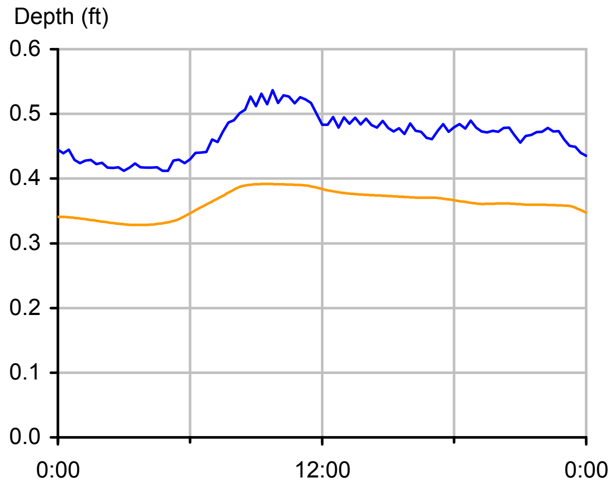
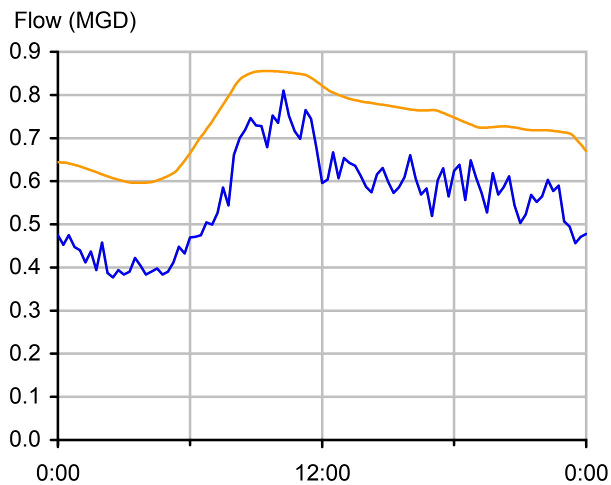
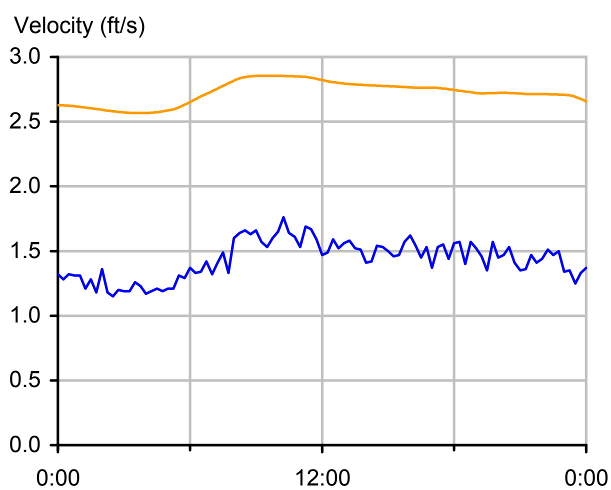
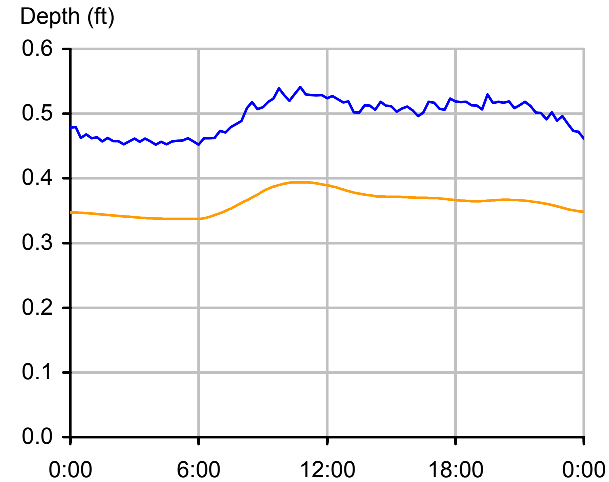
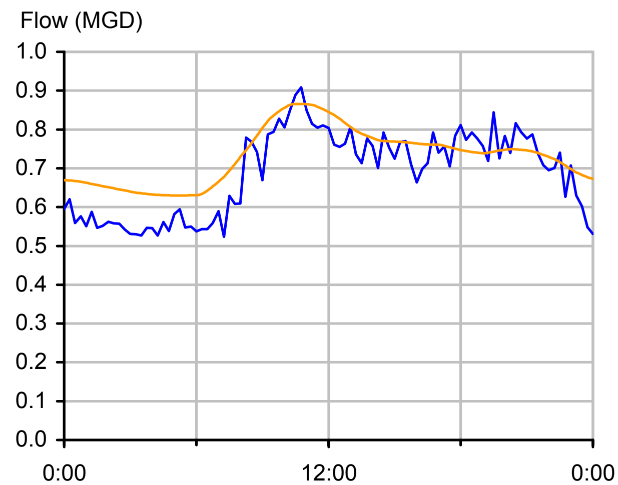


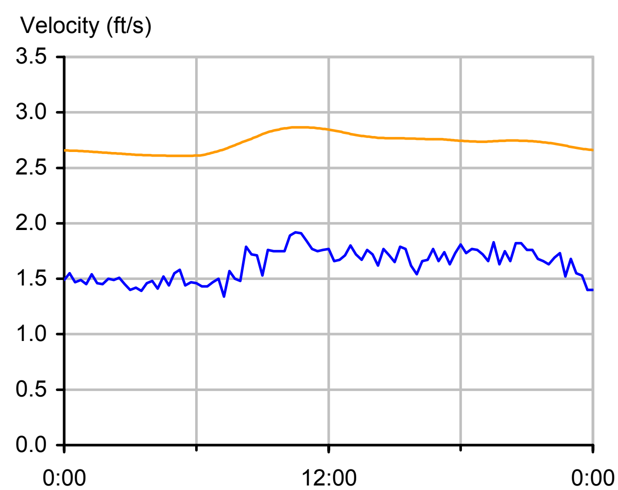




Dry Weather 2 (Sun Jan 23 2005)



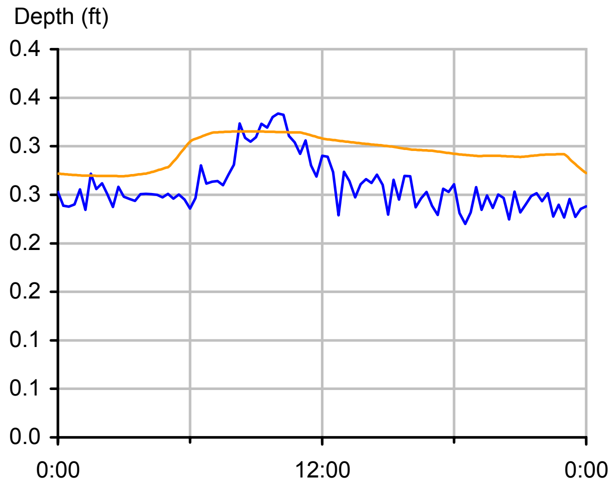


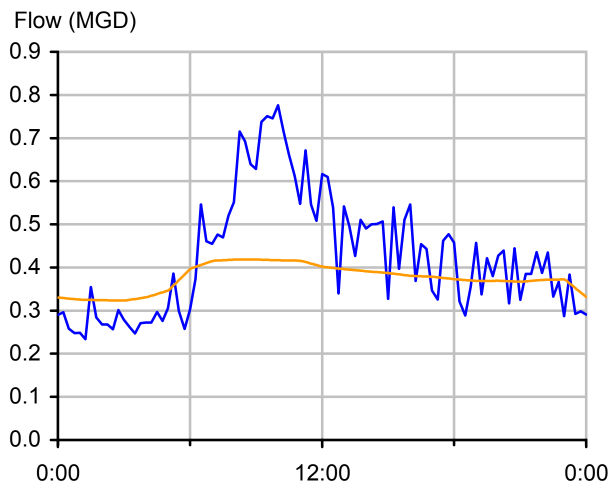


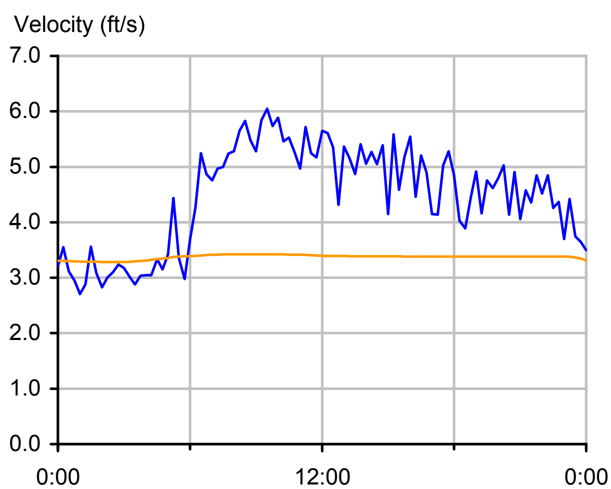
Verification Results - FM11 (W514.09.1)

Flow Monitoring   Model Simulation

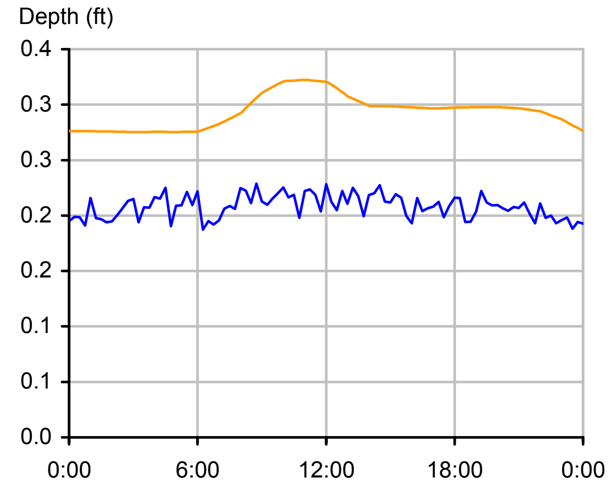
Dry Weather 1 (Wed Dec 22 2004)

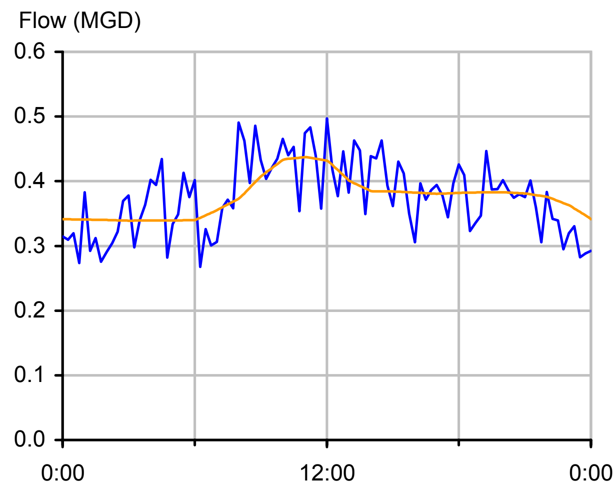


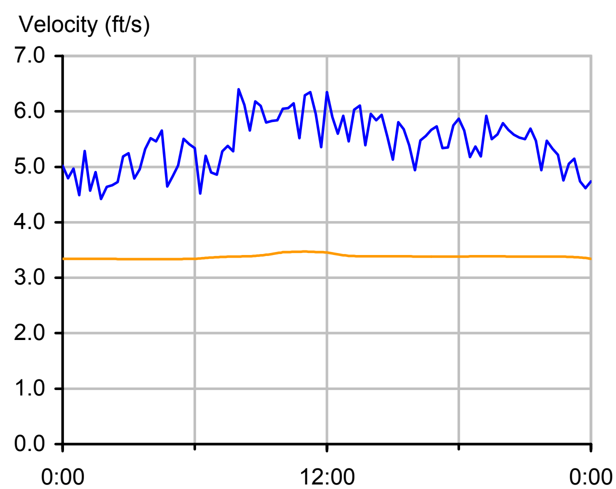




Dry Weather 2 (Sun Jan 23 2005)



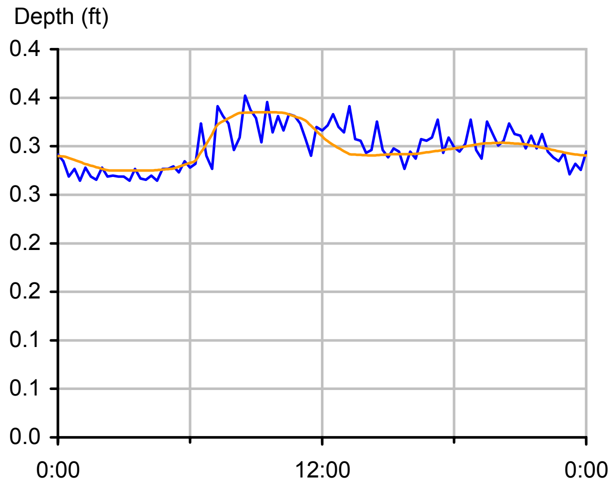


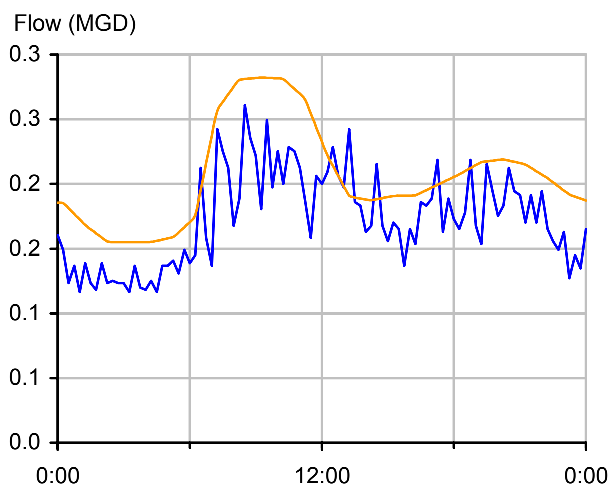


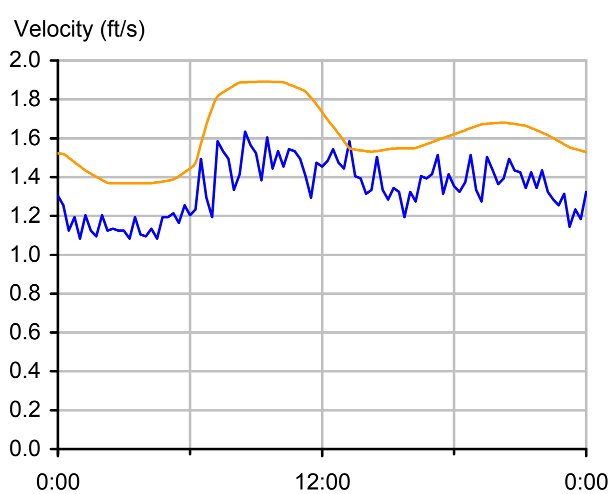
Verification Results - FM12 (K200.02.1)

Flow Monitoring — Model Simulation

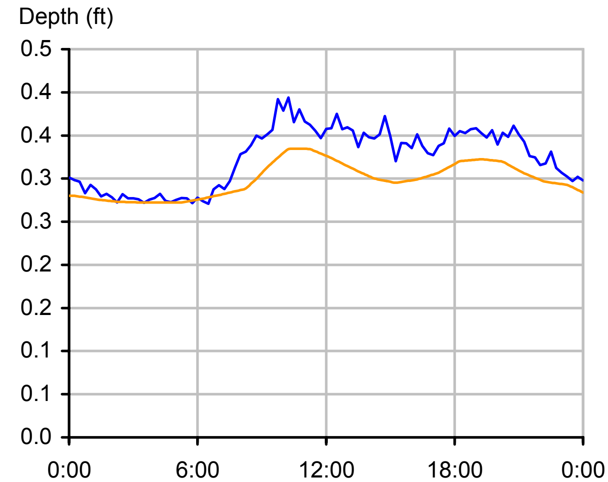
Dry Weather 1 (Wed Dec 22 2004)

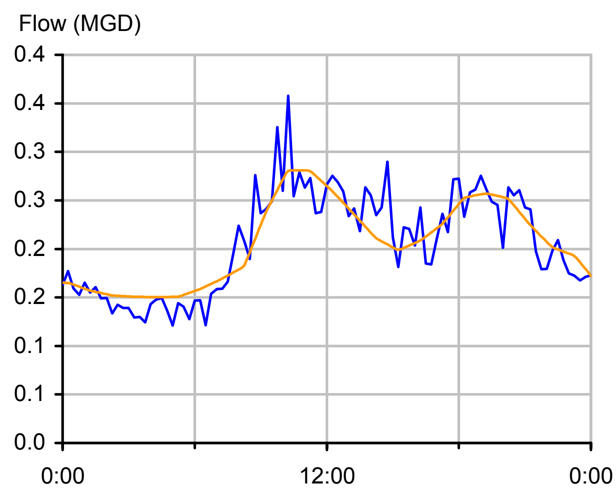


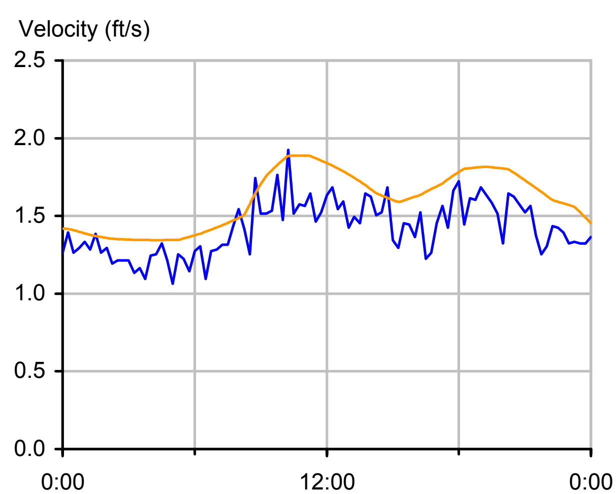




Dry Weather 2 (Sun Jan 23 2005)



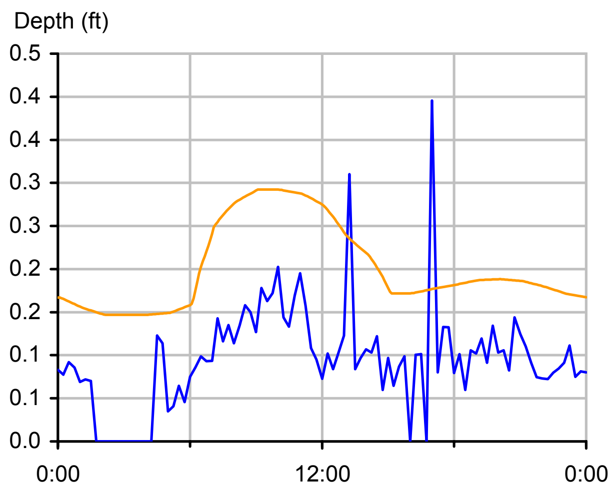


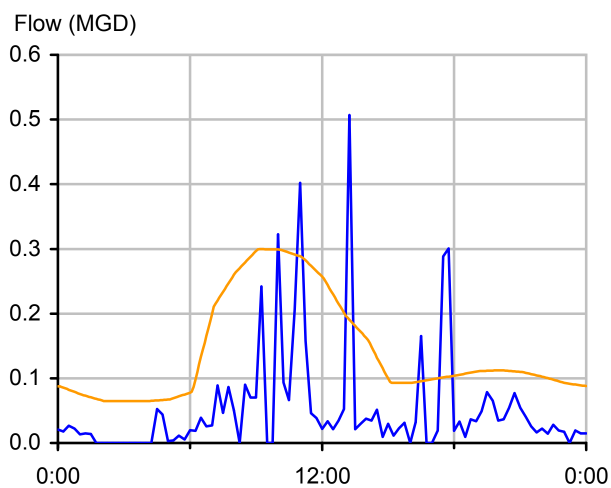


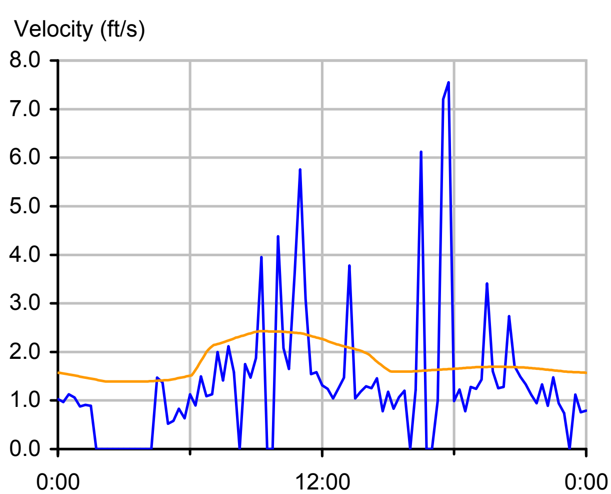
Verification Results - FM13 (G300.02.1)

Flow Monitoring   Model Simulation

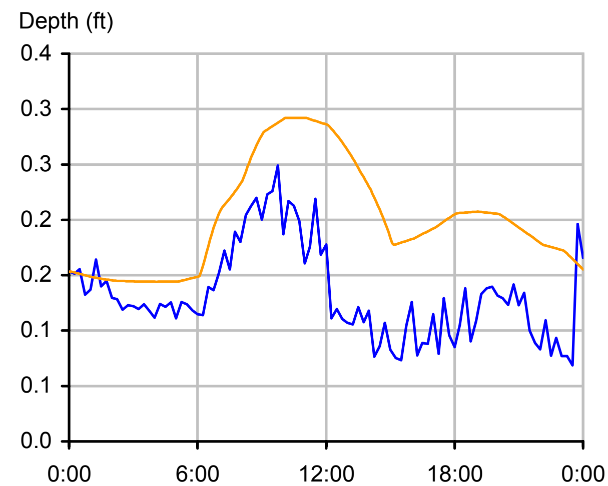
Dry Weather 1 (Wed Dec 22 2004)

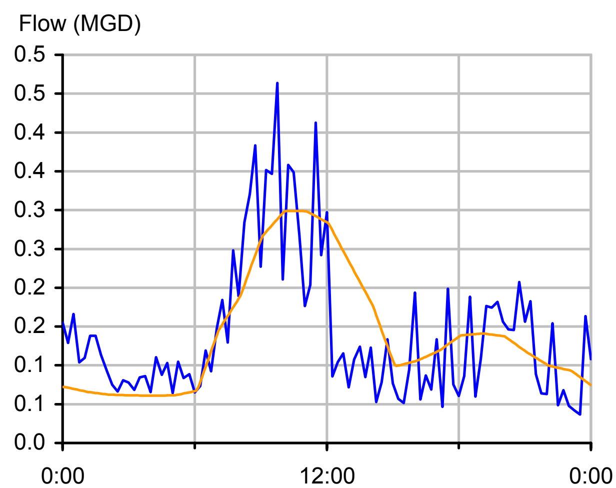


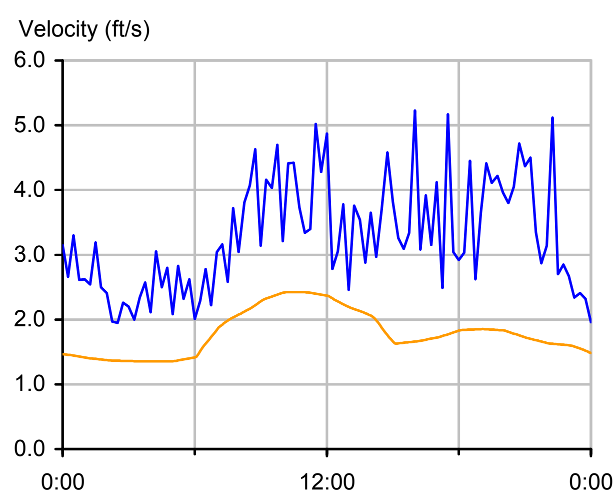




Dry Weather 2 (Sun Jan 23 2005)



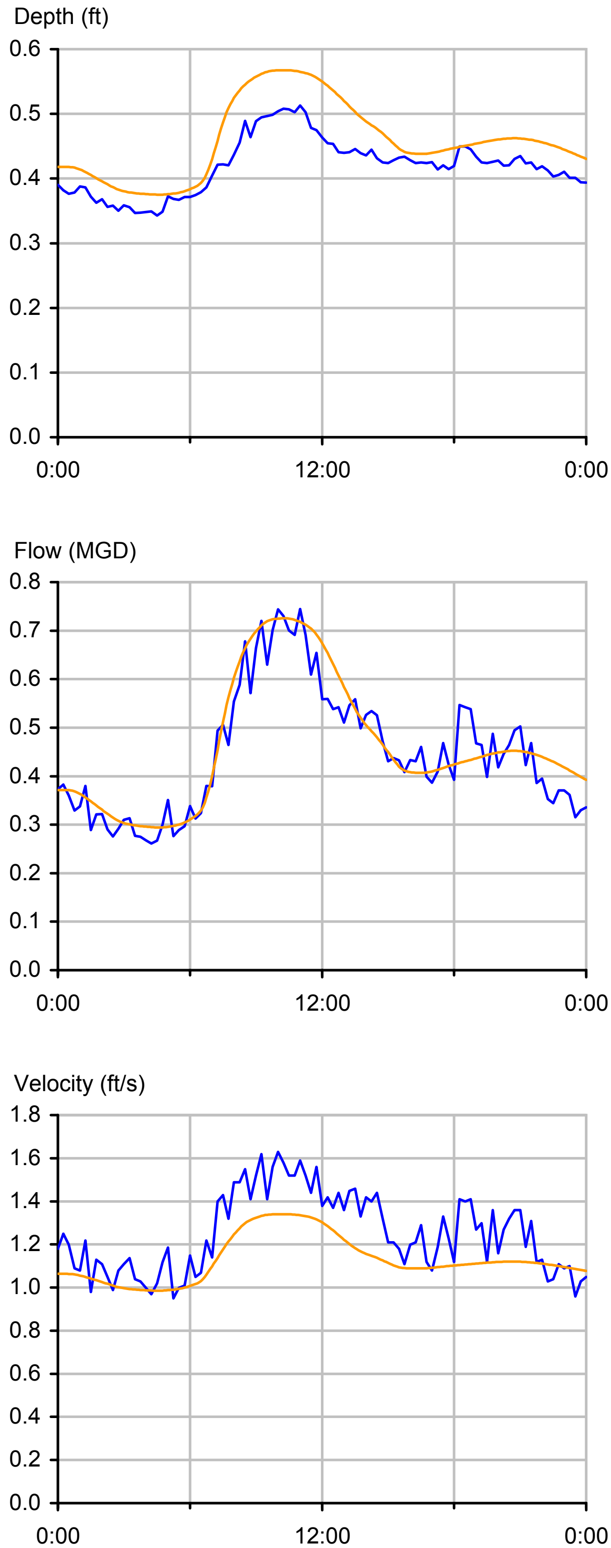




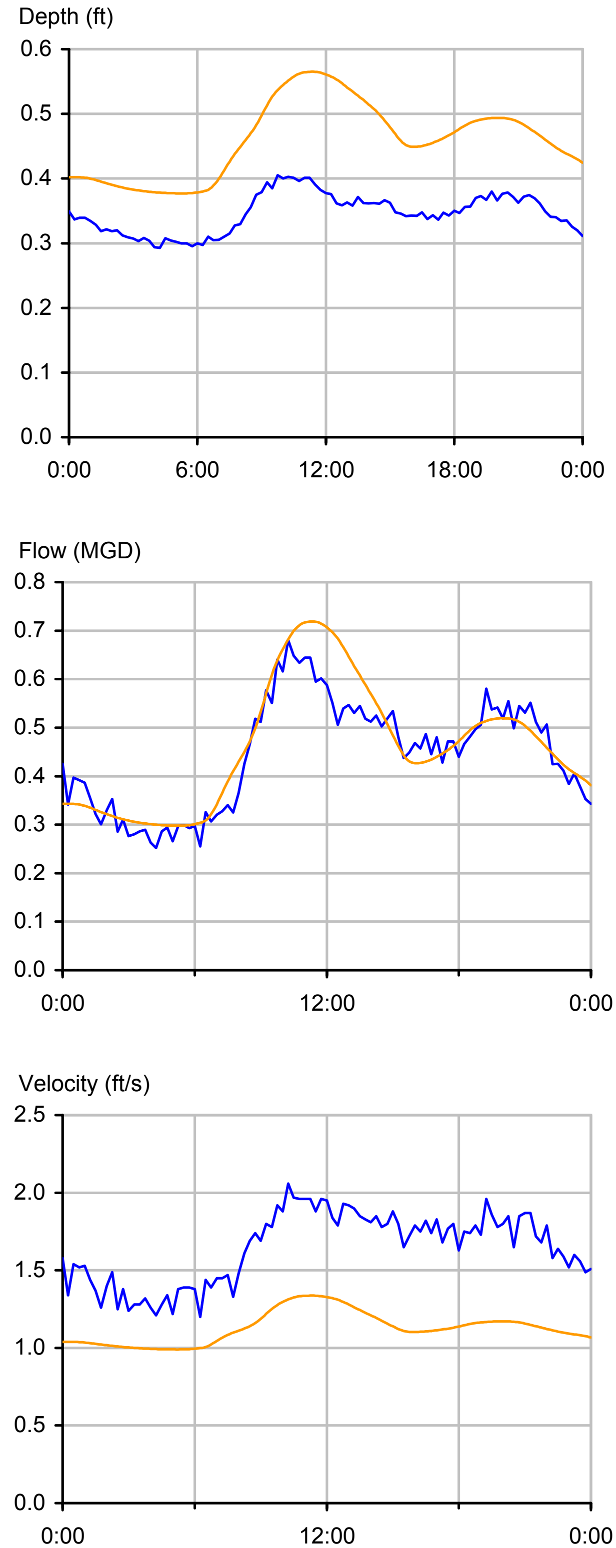
Verification Results - FM14 (G000.11.1)

Flow Monitoring   Model Simulation

Dry Weather 1 (Wed Dec 22 2004)



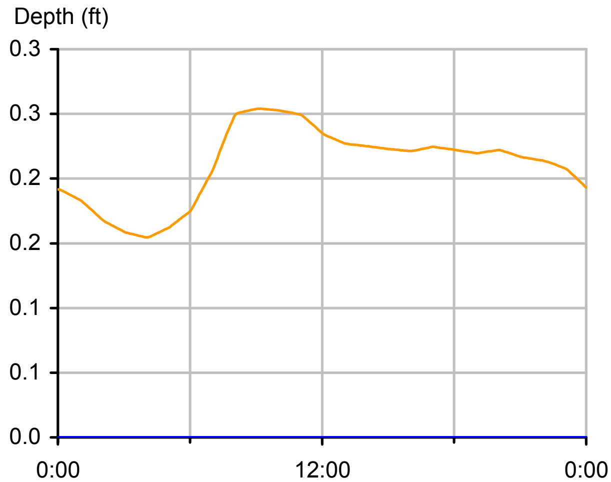
Dry Weather 2 (Sun Jan 23 2005)

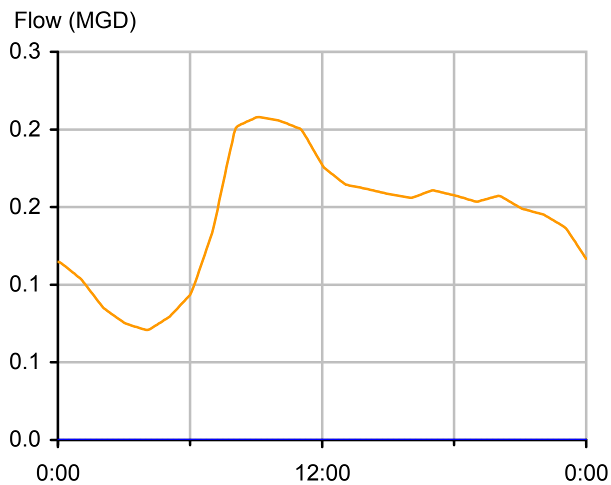


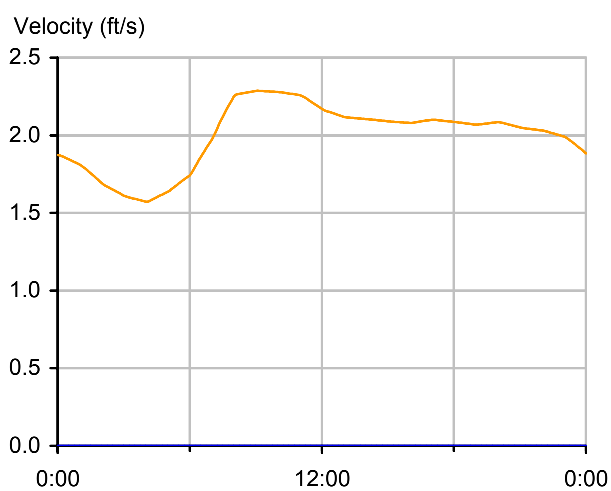
Verification Results - FM15 (G200.01.1)

Flow Monitoring — Model Simulation

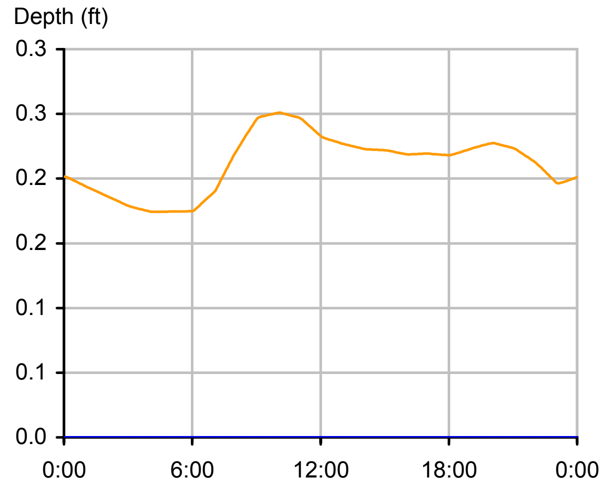
Dry Weather 1 (Wed Dec 22 2004)

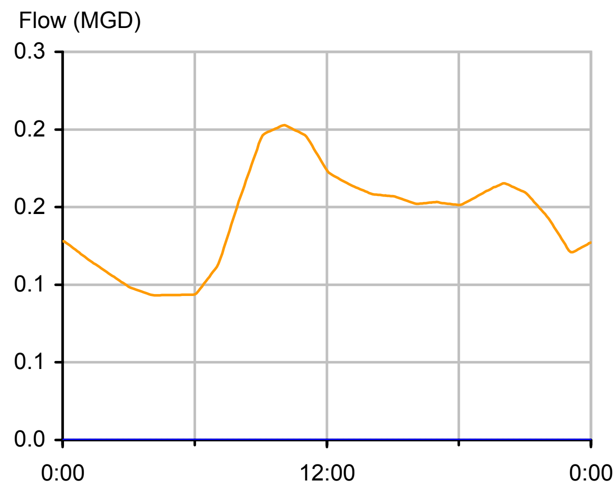


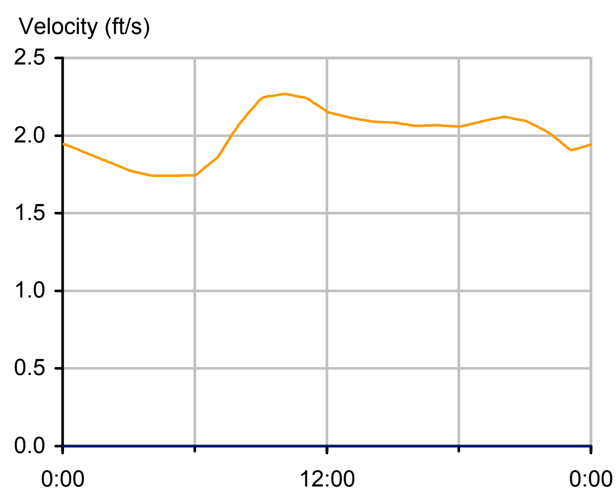




Dry Weather 2 (Sun Jan 23 2005)



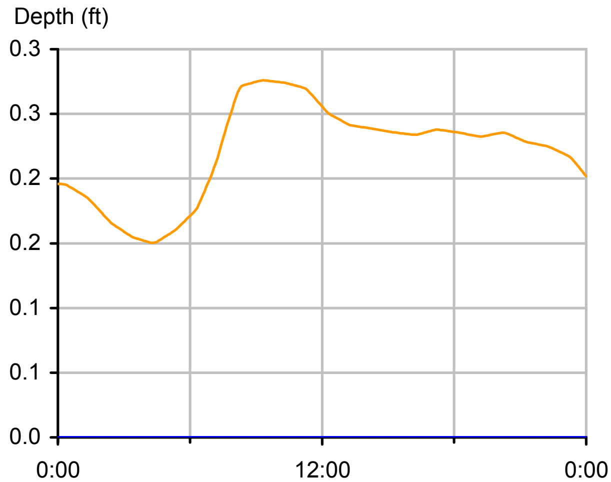


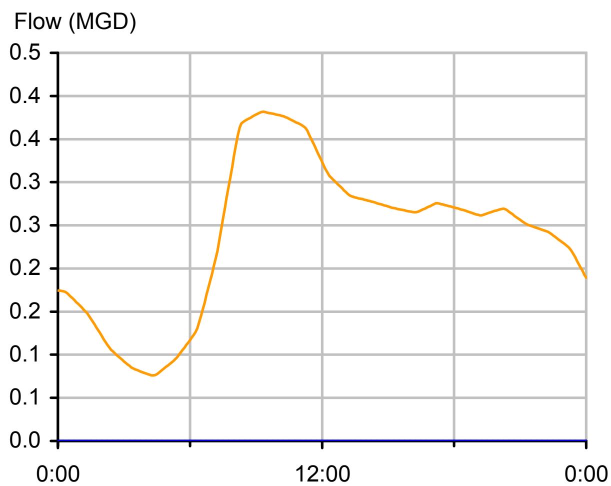


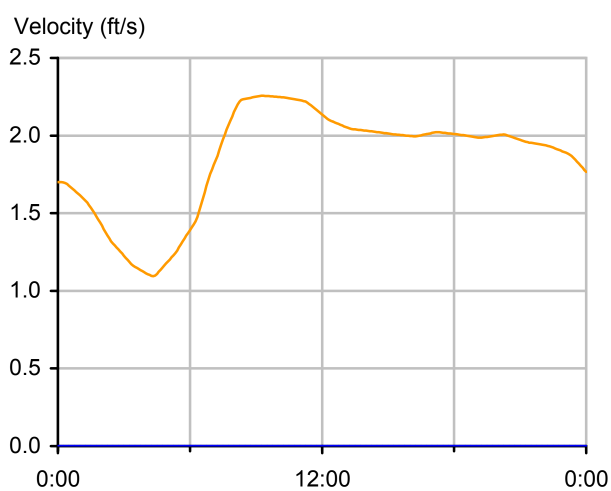
Verification Results - FM16 (L151.01.1)

Flow Monitoring   Model Simulation

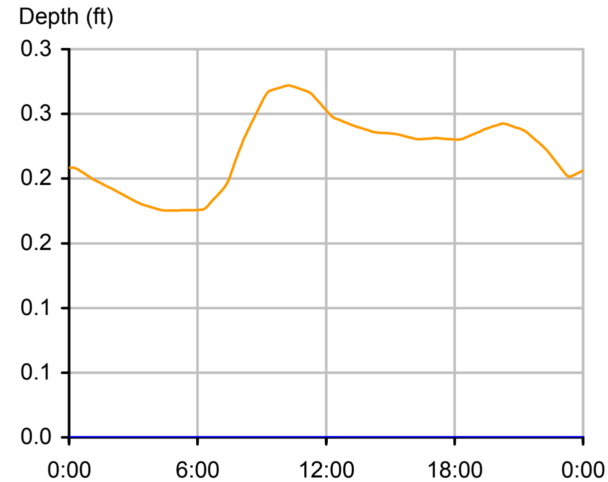
Dry Weather 1 (Wed Dec 22 2004)

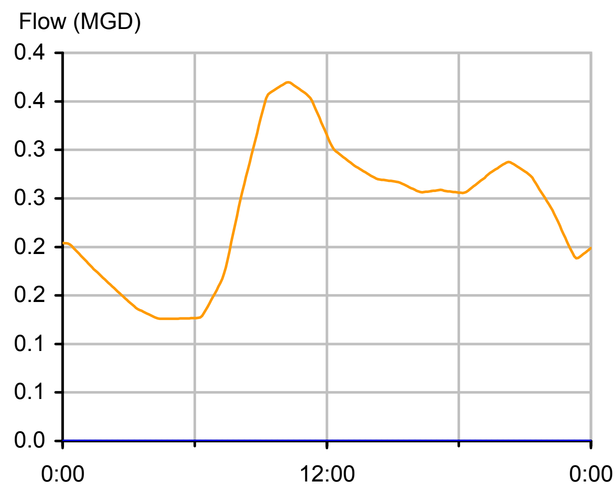


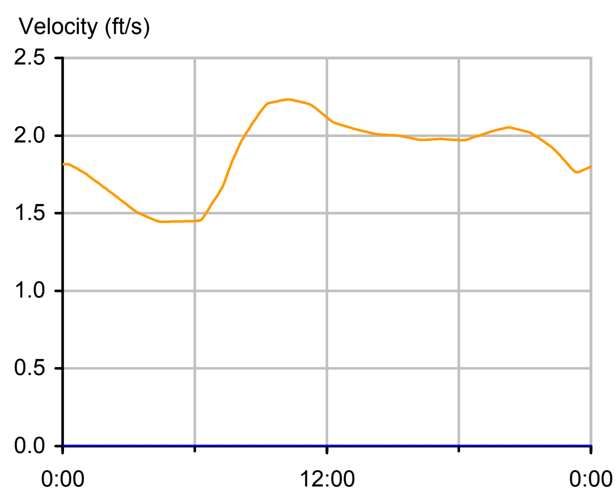




Dry Weather 2 (Sun Jan 23 2005)



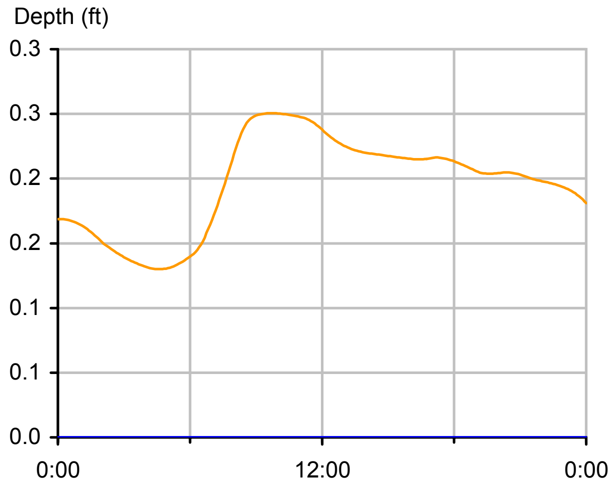


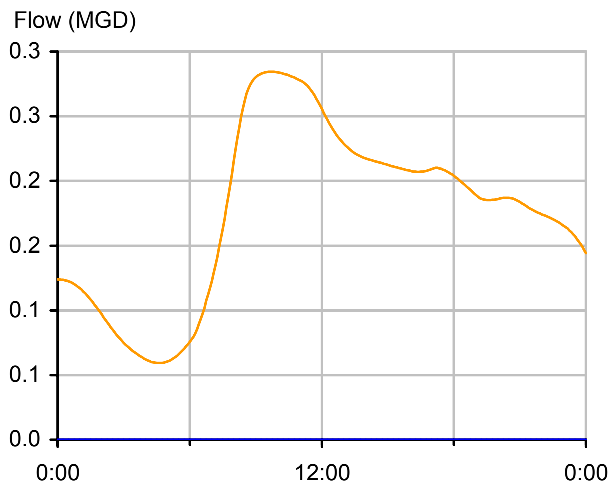


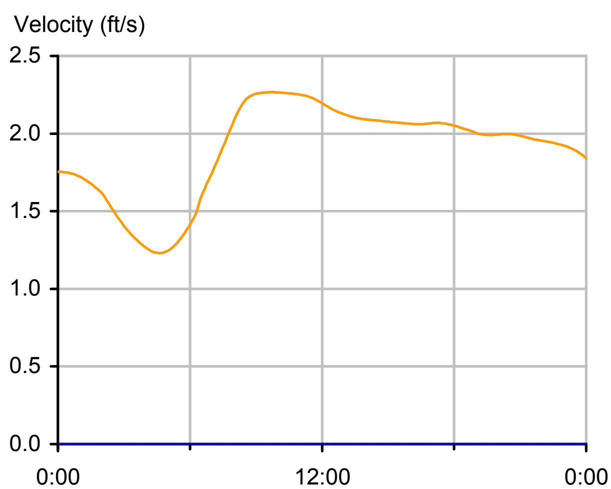
Verification Results - FM17 (L152.04.1)

Flow Monitoring   Model Simulation

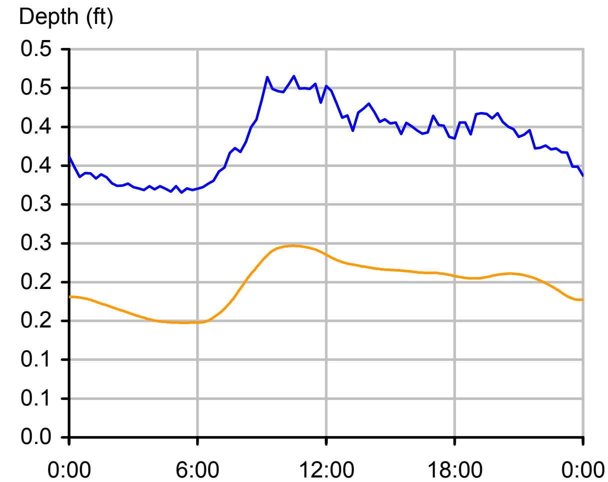
Dry Weather 1 (Wed Dec 22 2004)

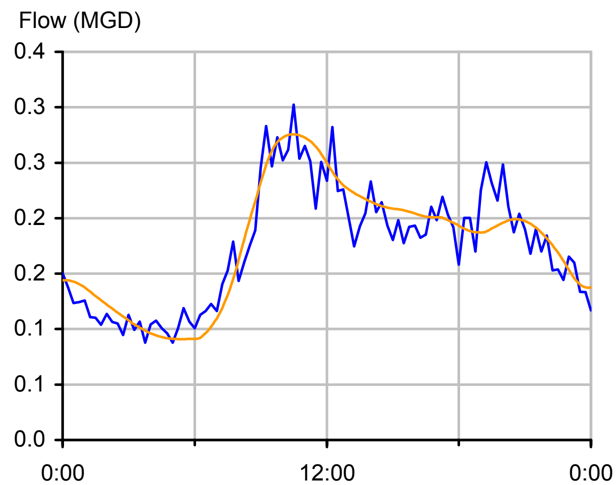


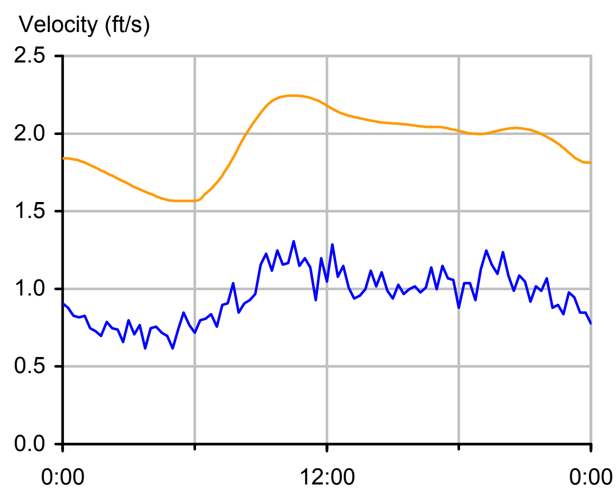




Dry Weather 2 (Sun Jan 23 2005)



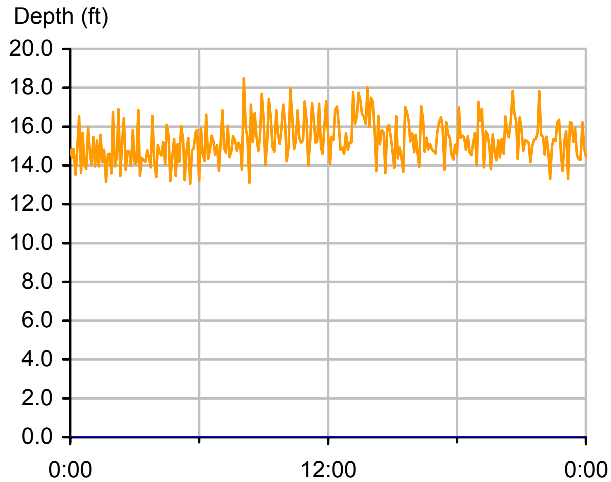


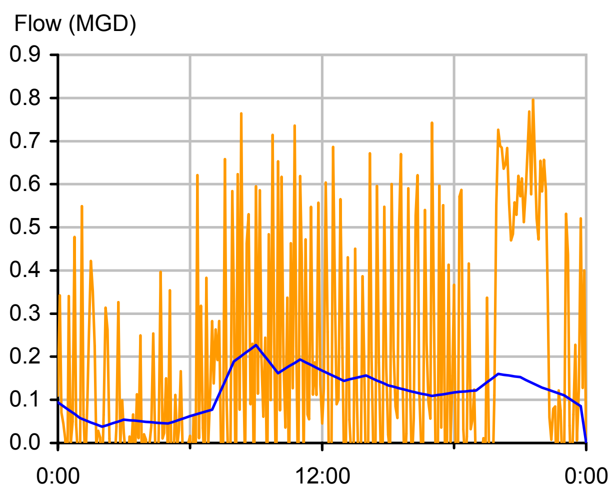


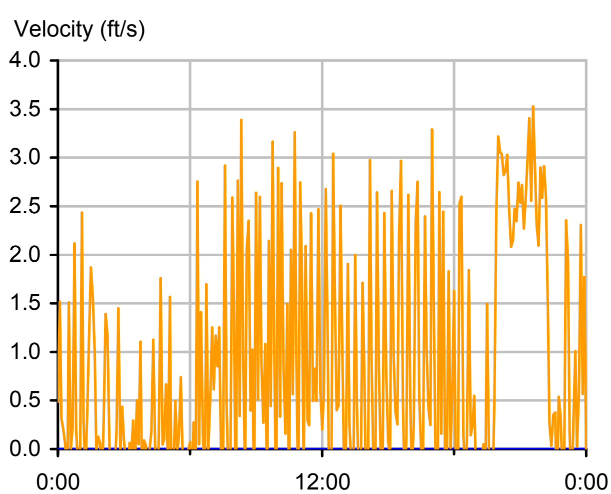
Verification Results - FM18 (PS12-FM.1)

Flow Monitoring — Model Simulation

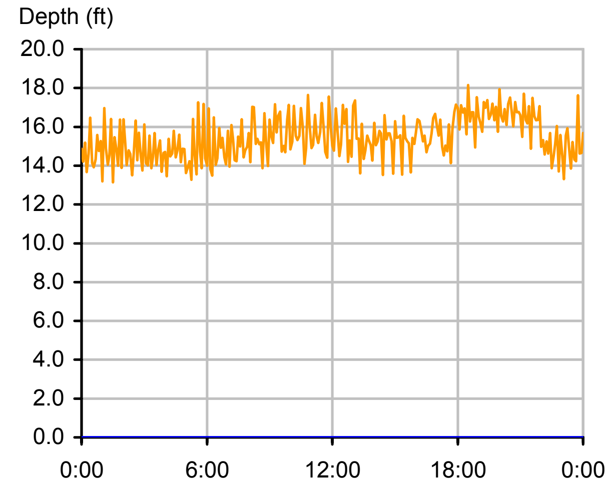
Dry Weather 1 (Wed Dec 22 2004)

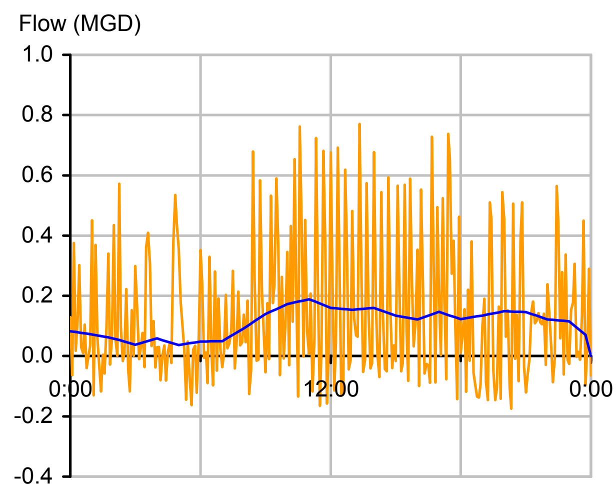


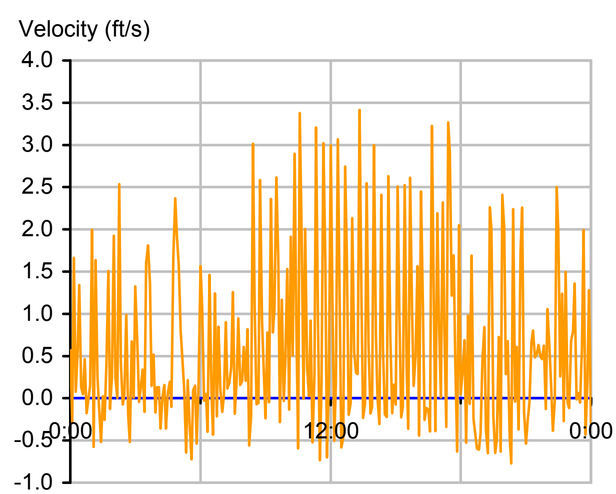




Dry Weather 2 (Sun Jan 23 2005)



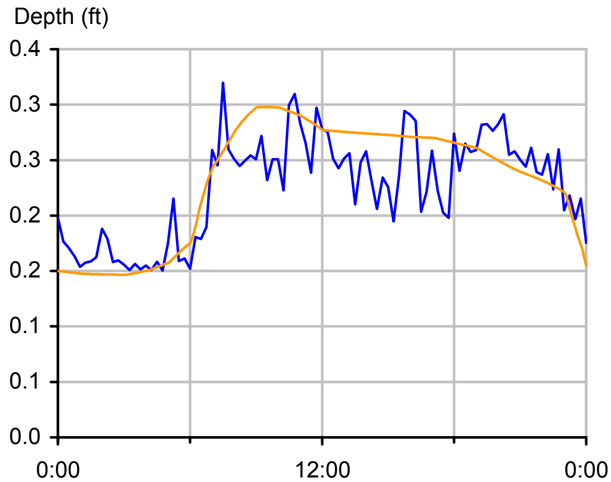


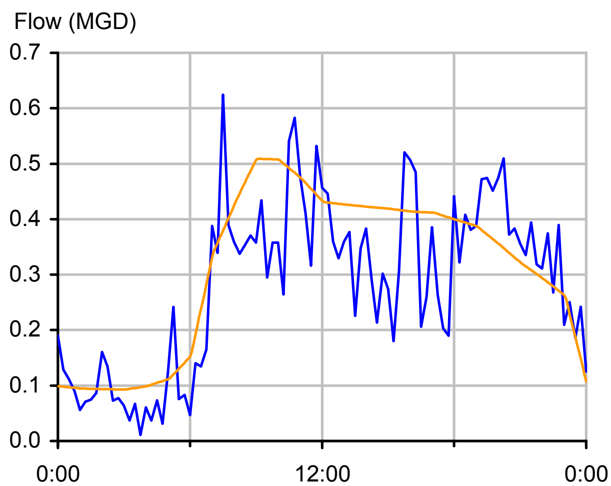


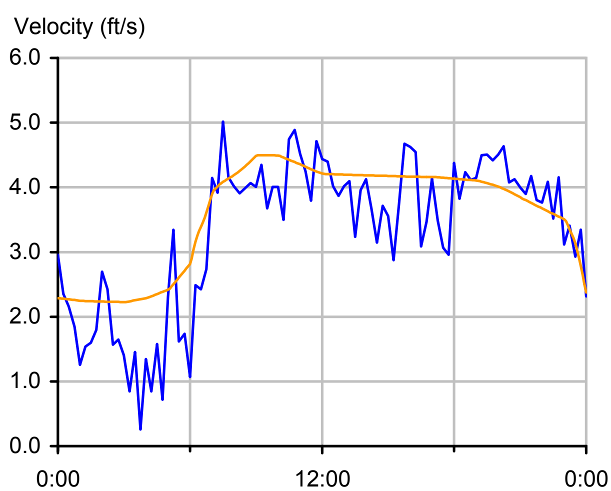
Verification Results - FM19 (L000.05.1)

Flow Monitoring   Model Simulation

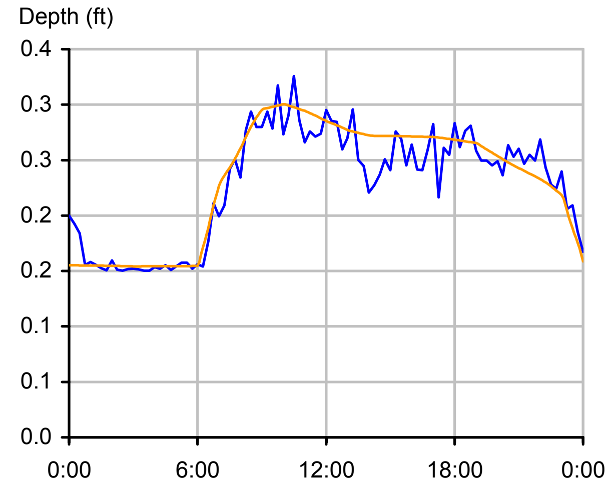
Dry Weather 1 (Wed Dec 22 2004)

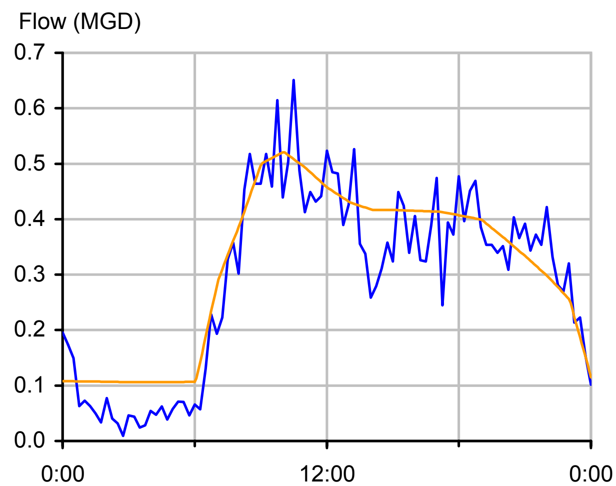


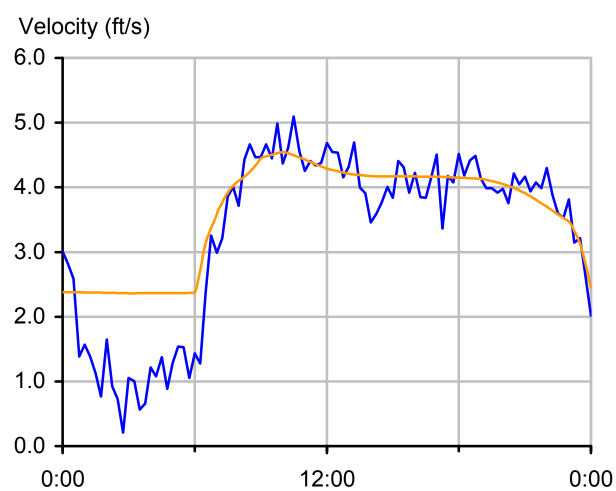




Dry Weather 2 (Sun Jan 23 2005)



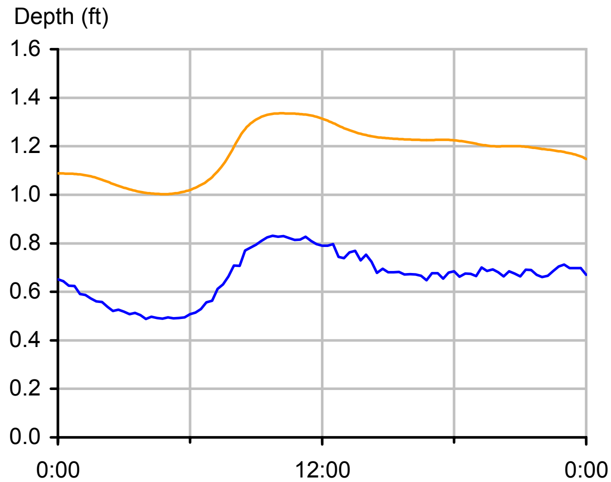


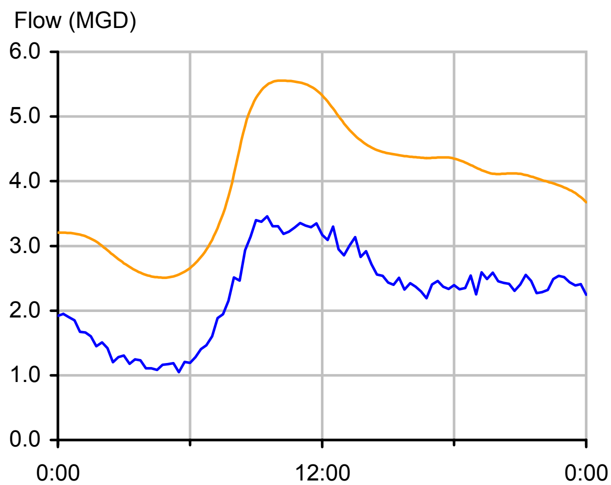


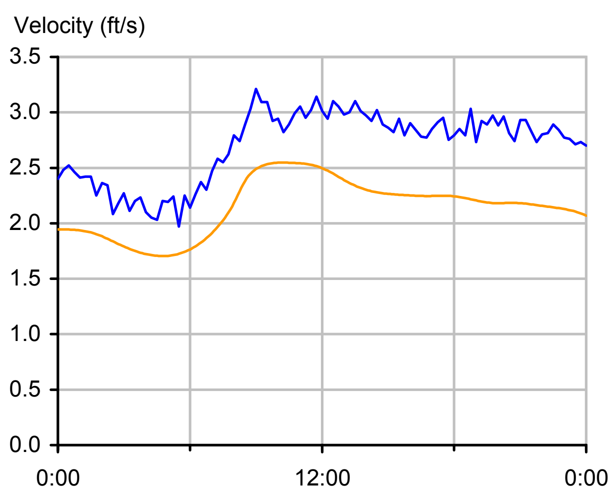
Verification Results - FM20 (K000.03.1)

Flow Monitoring   Model Simulation

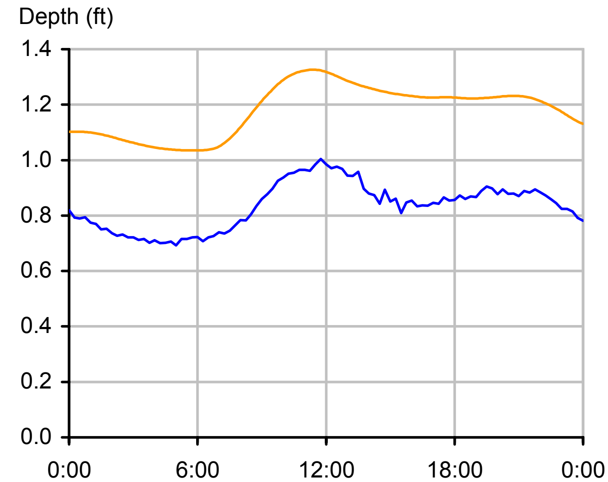
Dry Weather 1 (Wed Dec 22 2004)

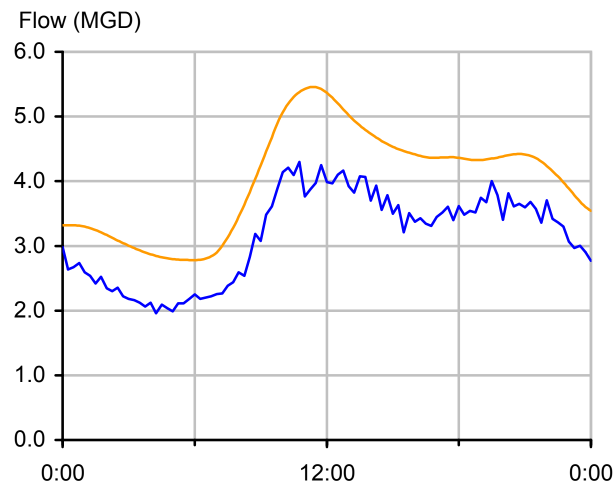


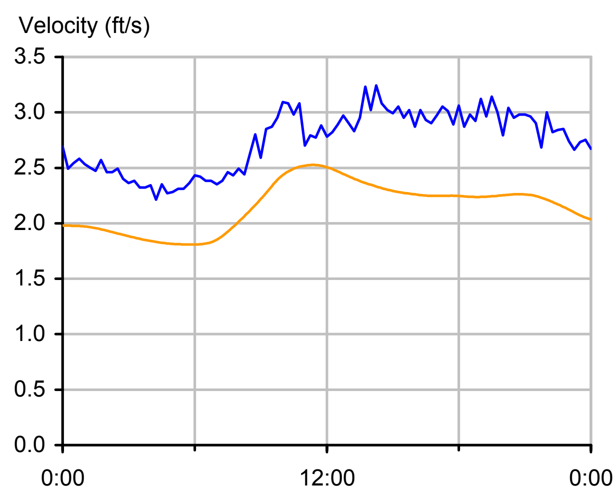




Dry Weather 2 (Sun Jan 23 2005)





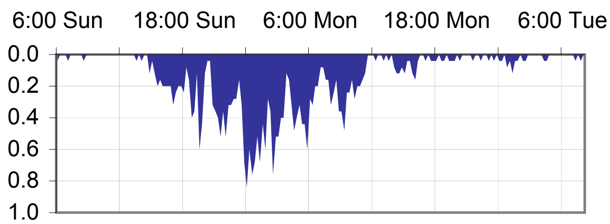


Verification Results - FM01 (F003.02.1)

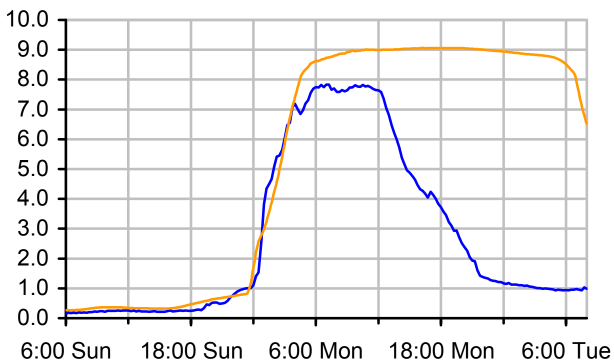
Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

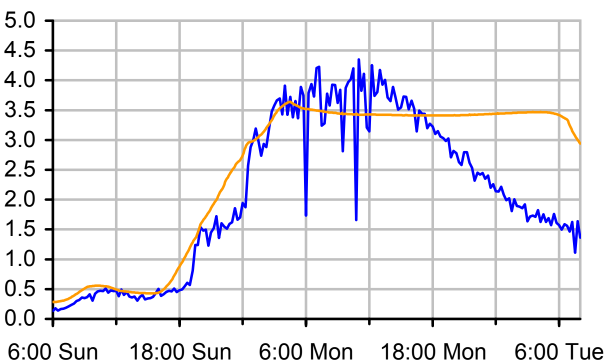
Rainfall (in/hr)



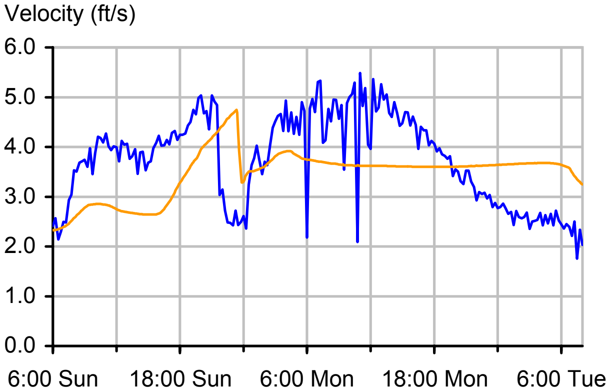
Depth (ft)



Flow (MGD)

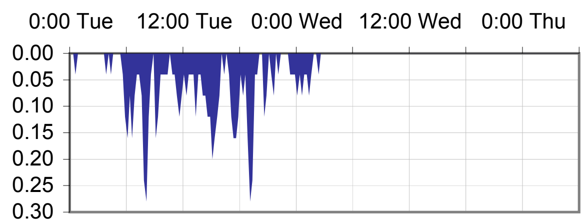


Velocity (ft/s)

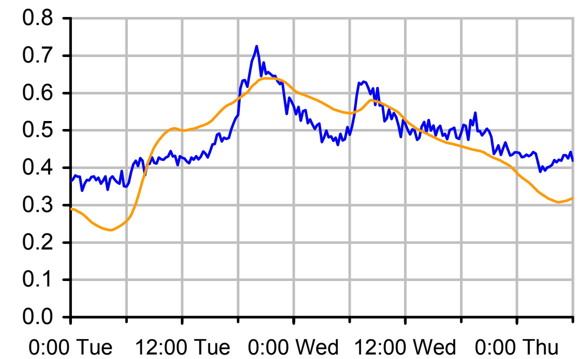


Storm B (Tue Feb 15 - Thu Feb 17 2005)

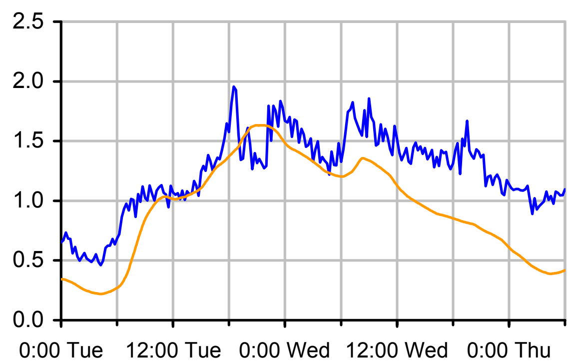
Rainfall (in/hr)



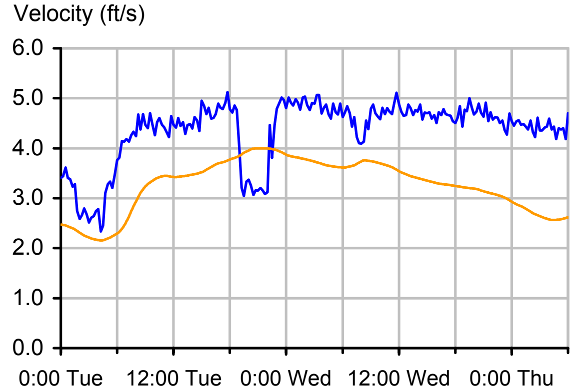
Depth (ft)



Flow (MGD)



Velocity (ft/s)

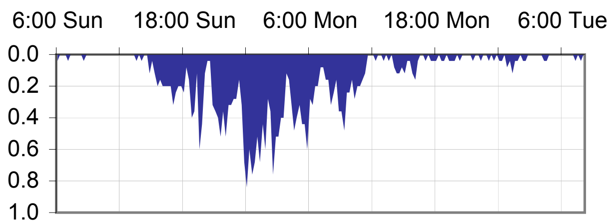


Verification Results - FM02 (F002.11.1)

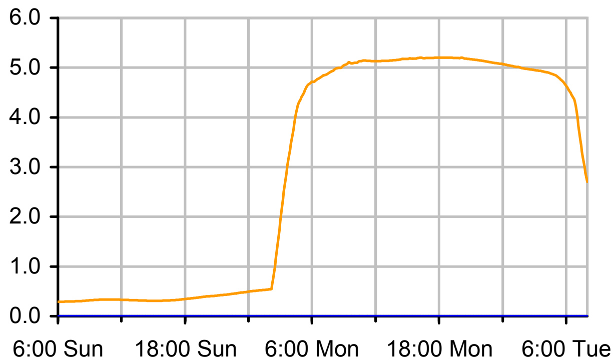
Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

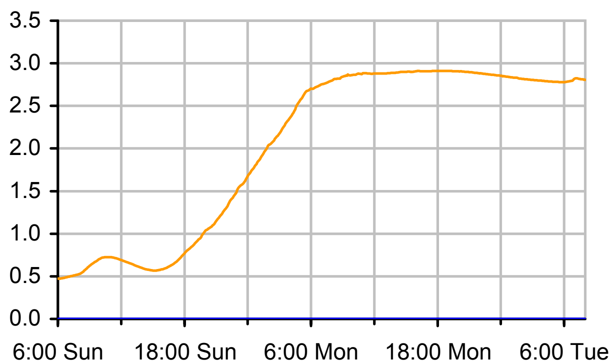
Rainfall (in/hr)



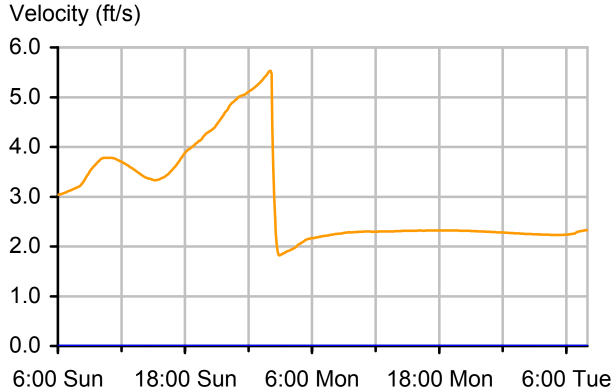
Depth (ft)



Flow (MGD)

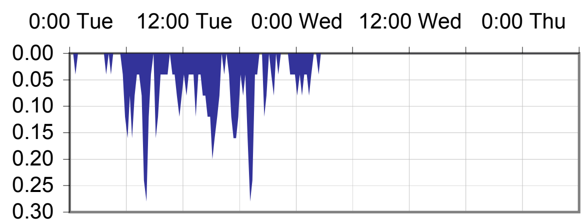


Velocity (ft/s)

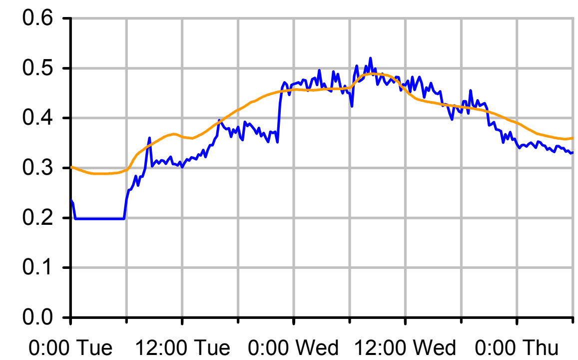


Storm B (Tue Feb 15 - Thu Feb 17 2005)

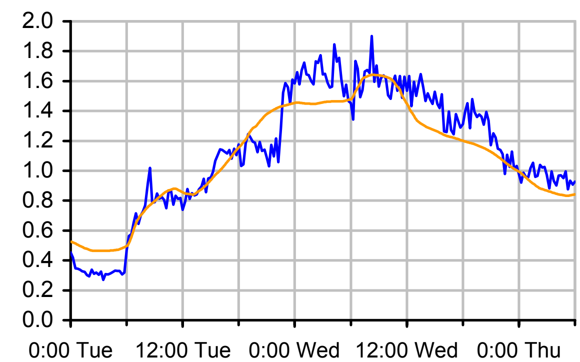
Rainfall (in/hr)



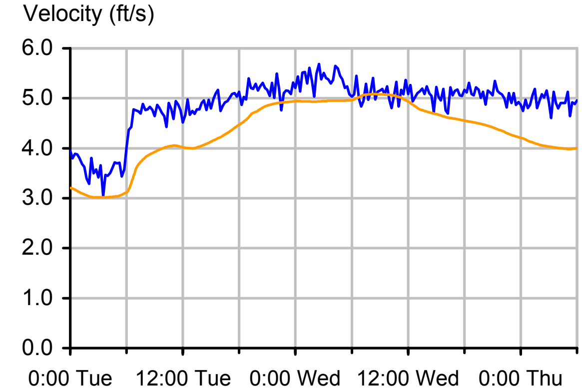
Depth (ft)



Flow (MGD)



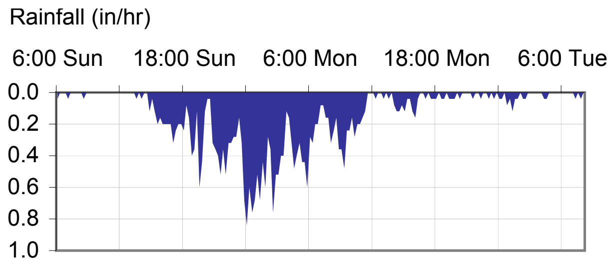
Velocity (ft/s)

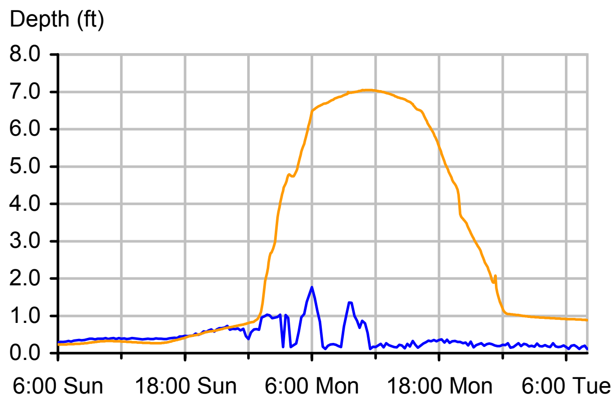


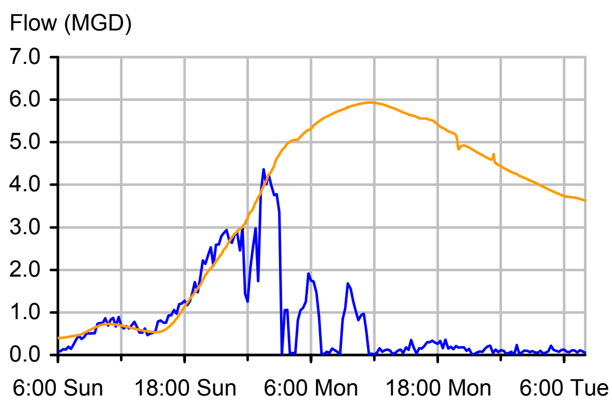
Verification Results - FM03 (S900.23.1)

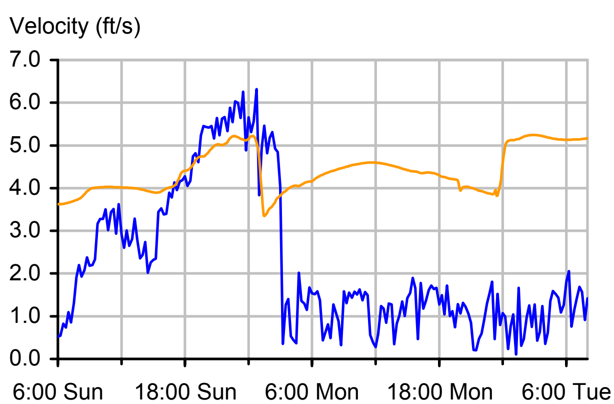
Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

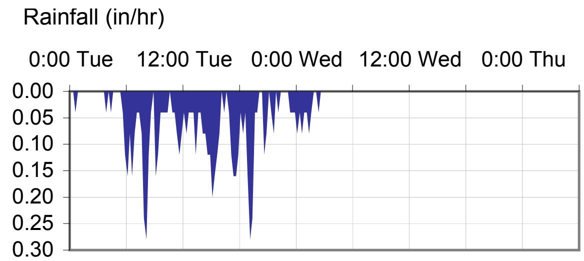


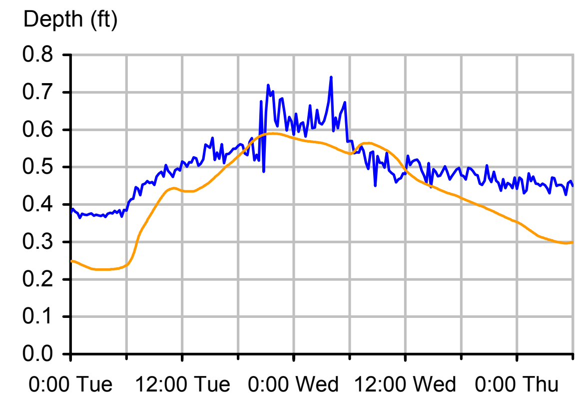


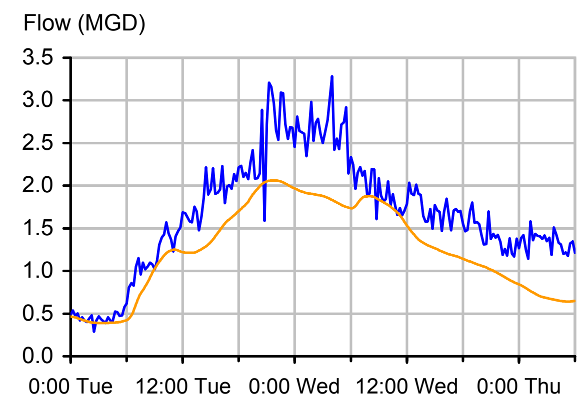


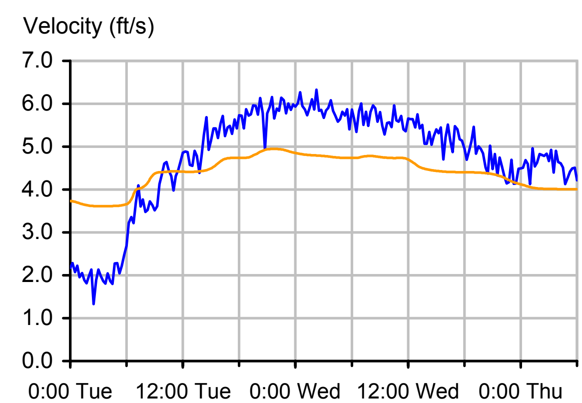


Storm B (Tue Feb 15 - Thu Feb 17 2005)





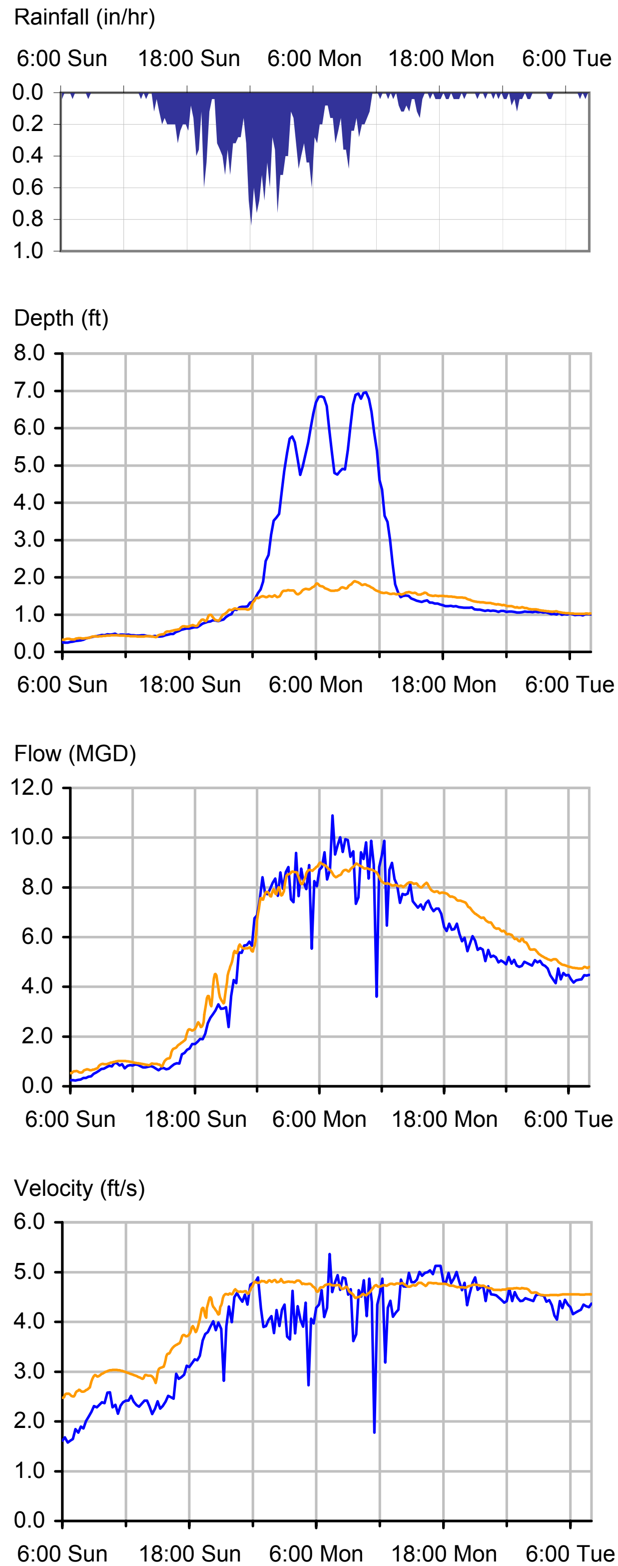




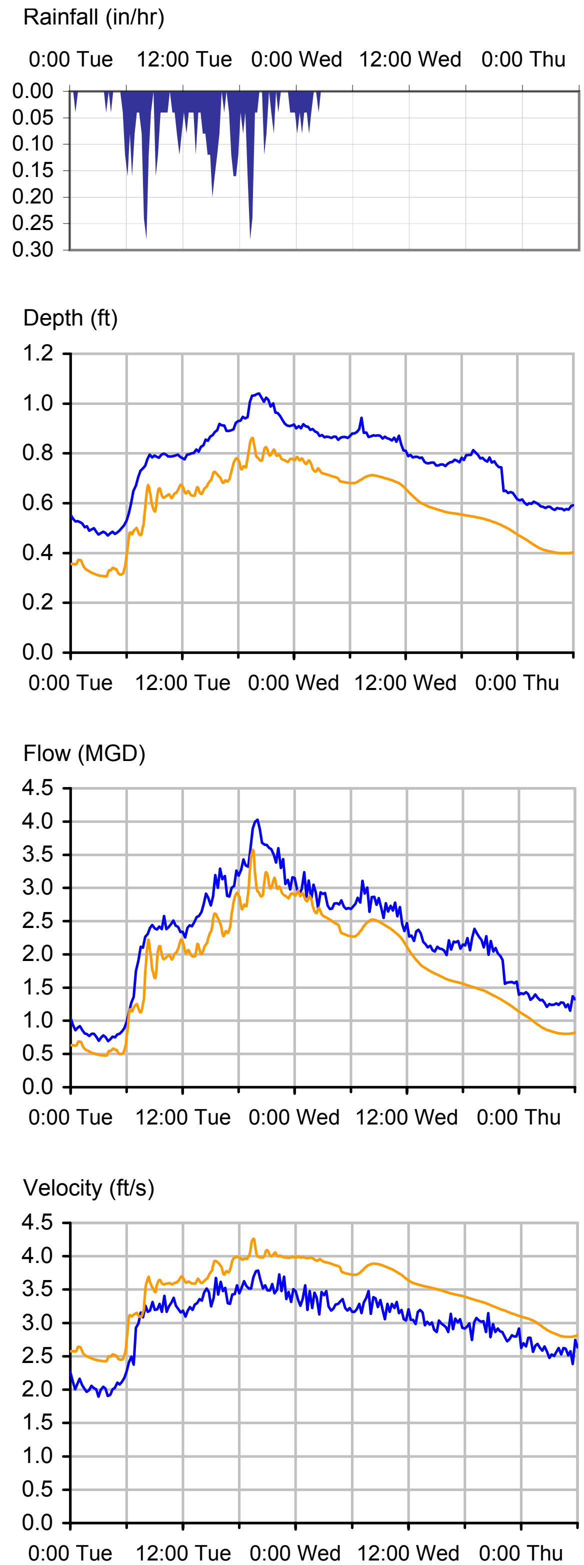
Verification Results - FM04 (S800.02.1)

Flow Monitoring — Model Simulation

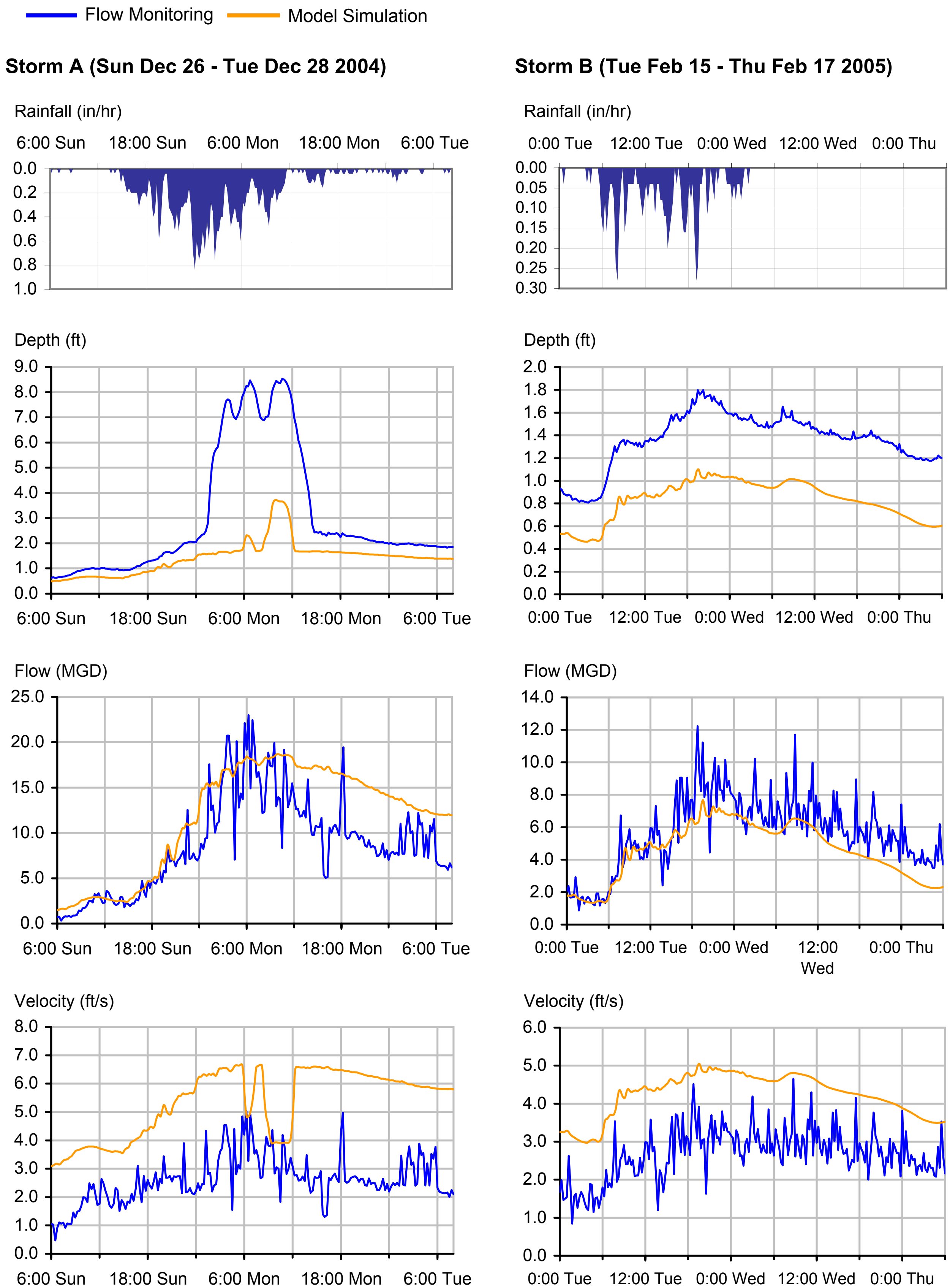
Storm A (Sun Dec 26 - Tue Dec 28 2004)



Storm B (Tue Feb 15 - Thu Feb 17 2005)



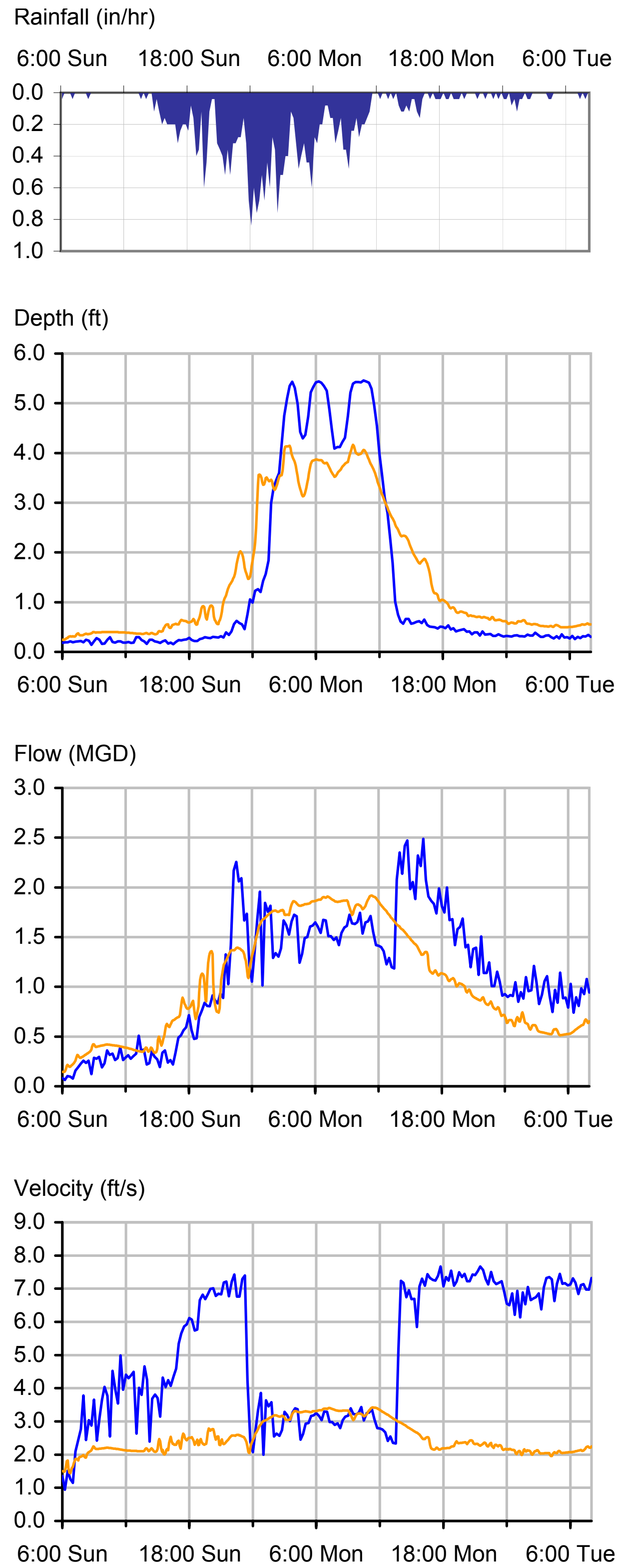
Verification Results - FM05 (S000.54.1)



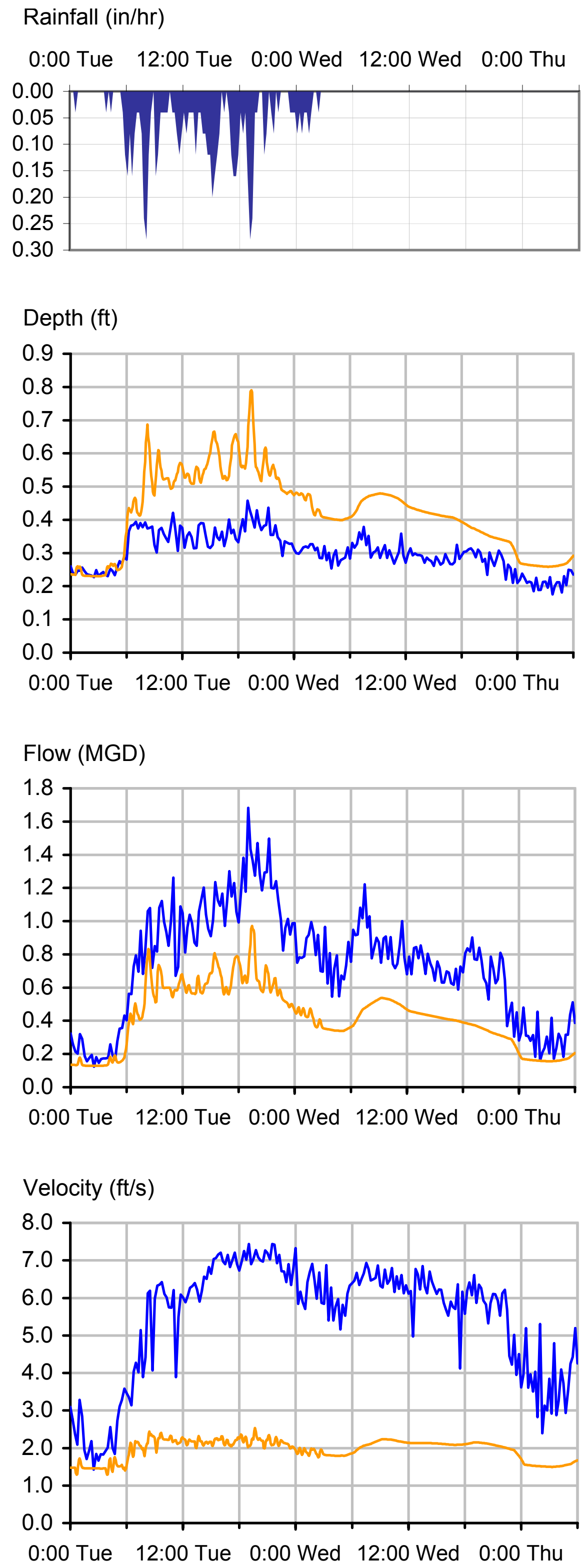
Verification Results - FM06 (S600.02.1)

Flow Monitoring   Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



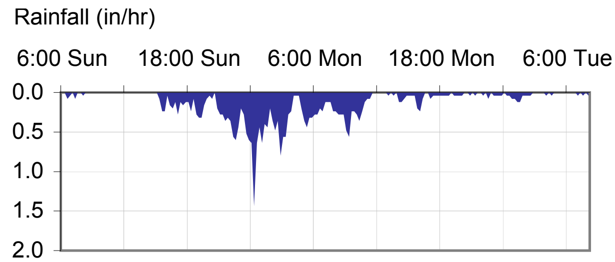
Storm B (Tue Feb 15 - Thu Feb 17 2005)

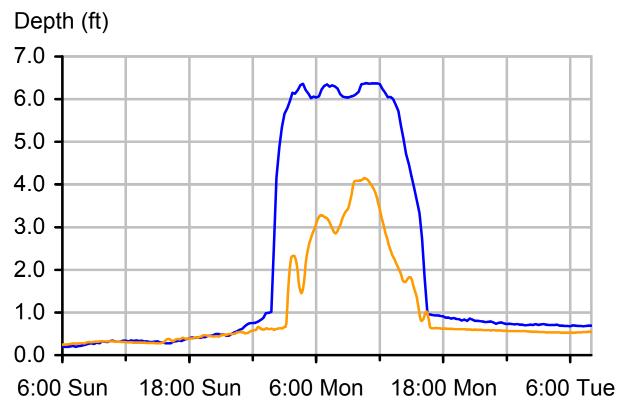


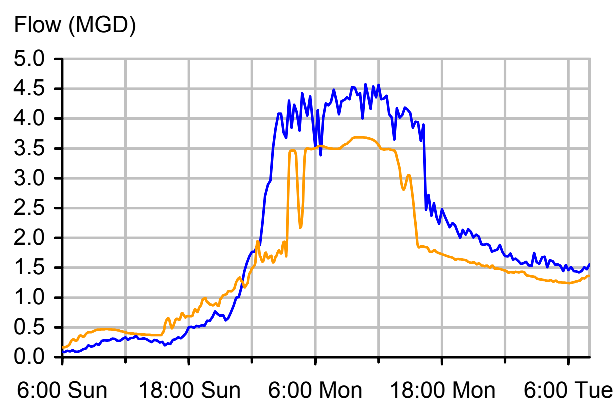
Verification Results - FM07 (S001.01.1)

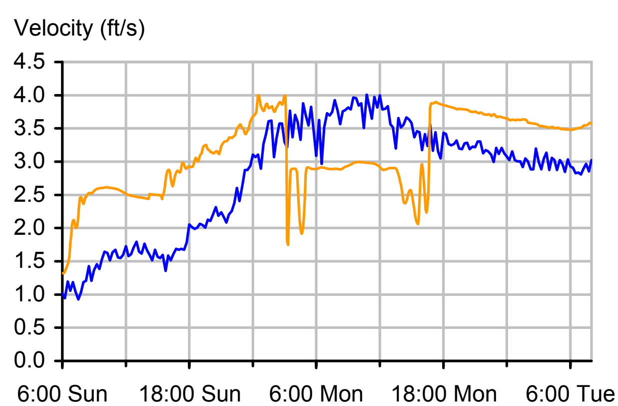
Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

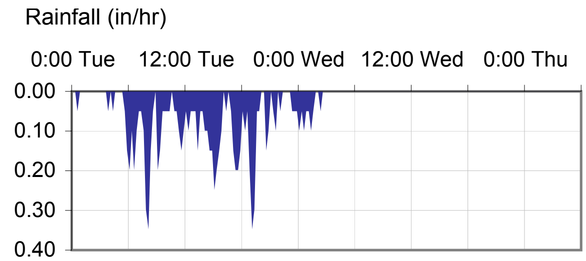


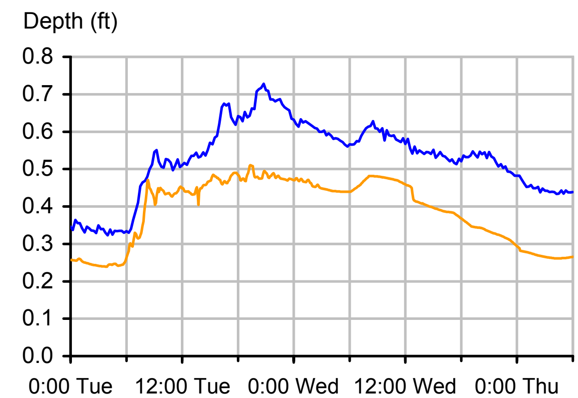


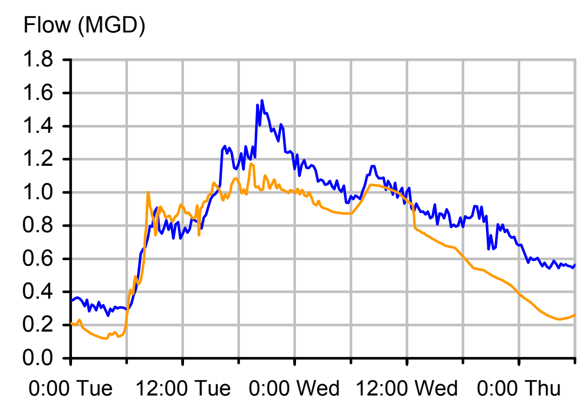


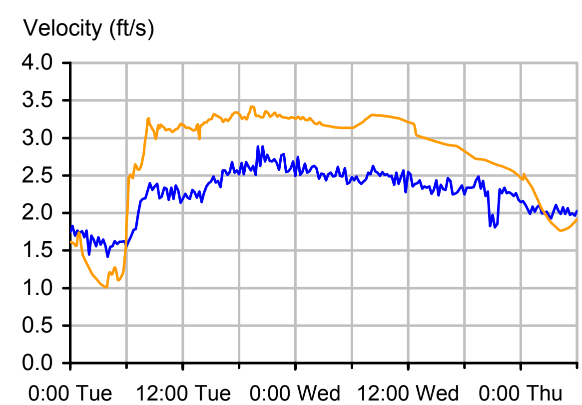


Storm B (Tue Feb 15 - Thu Feb 17 2005)





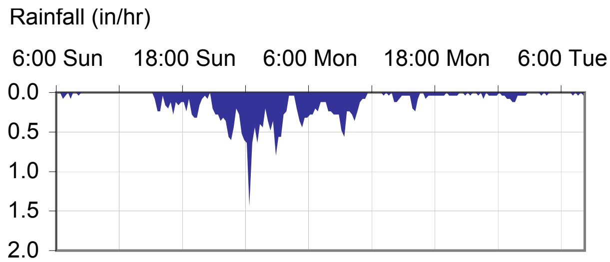


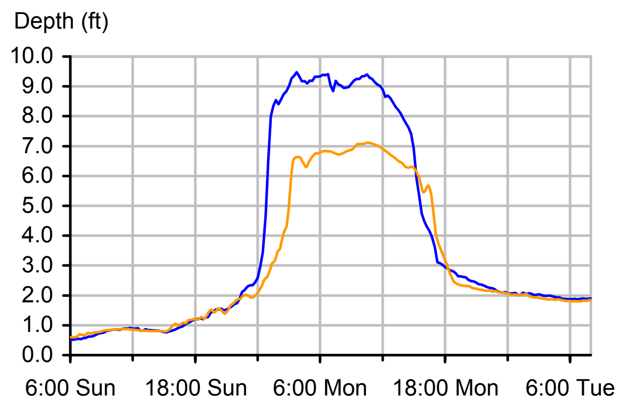


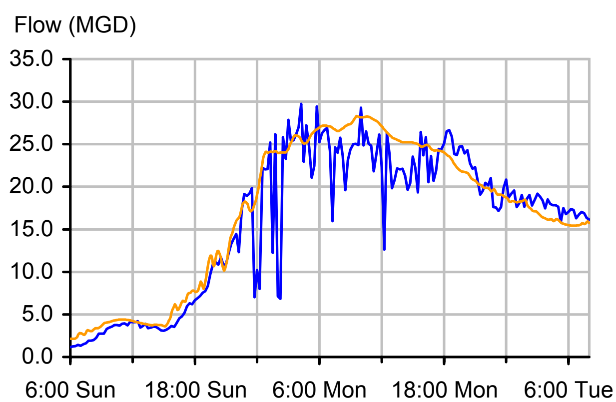
Verification Results - FM08 (R000.31.1)

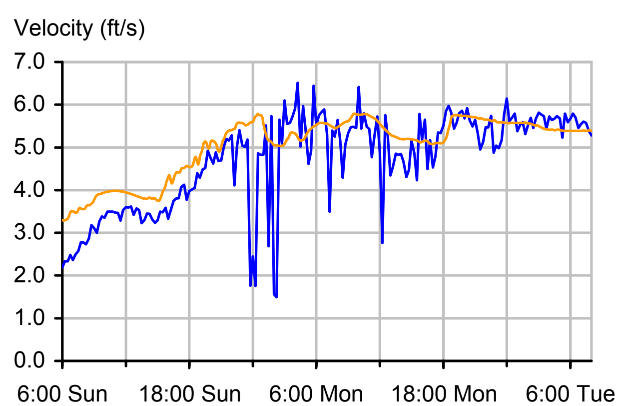
Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

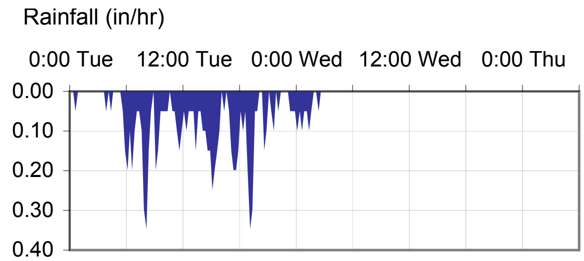


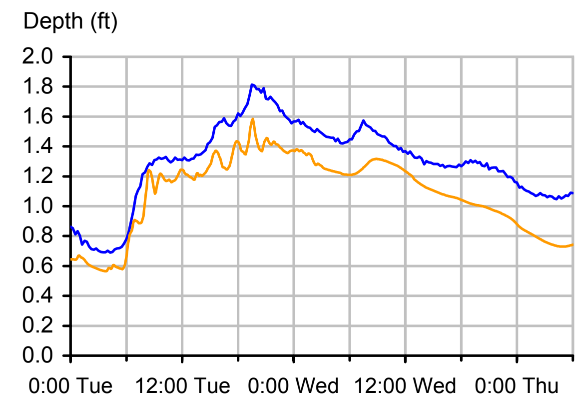


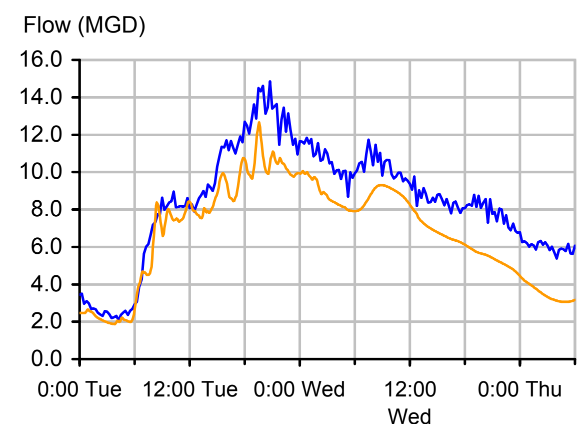


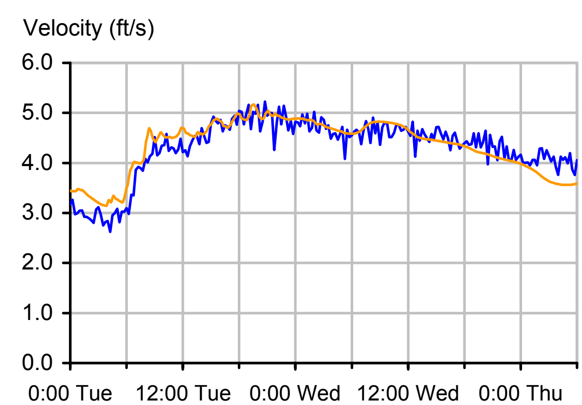


Storm B (Tue Feb 15 - Thu Feb 17 2005)





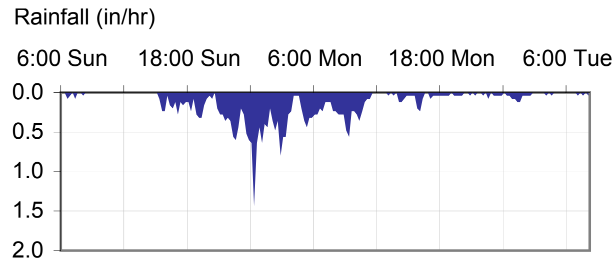


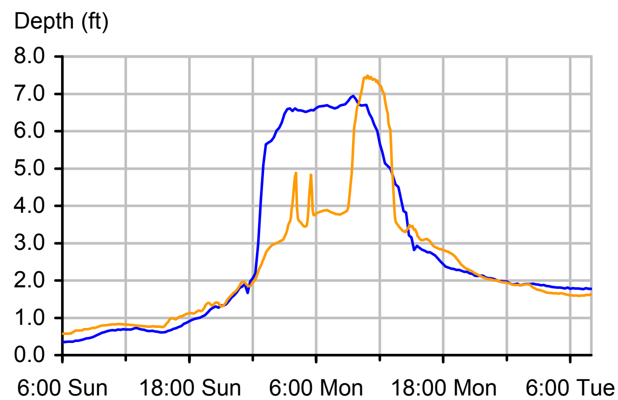


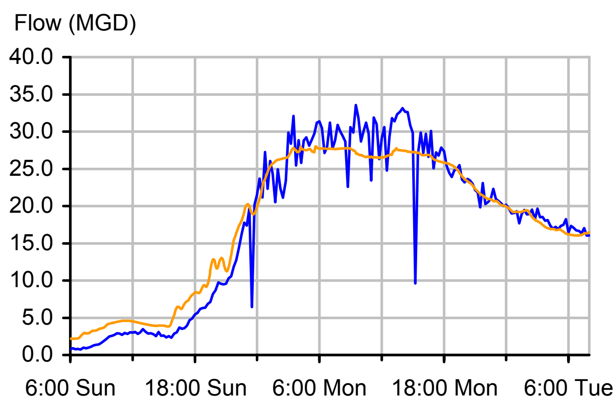
Verification Results - FM09 (K000.05.1)

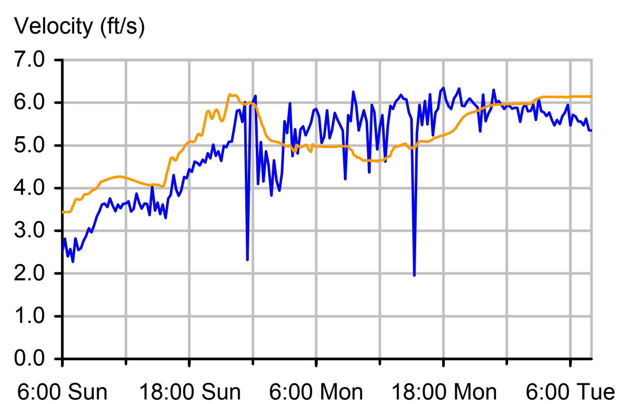
Flow Monitoring   Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

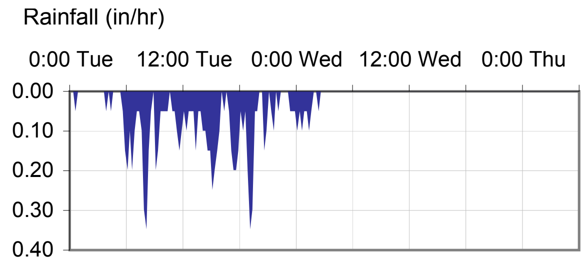


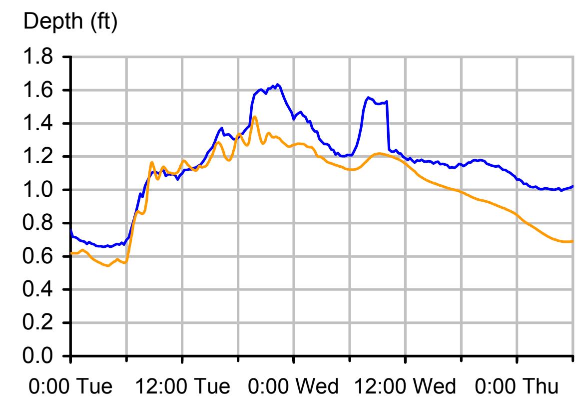


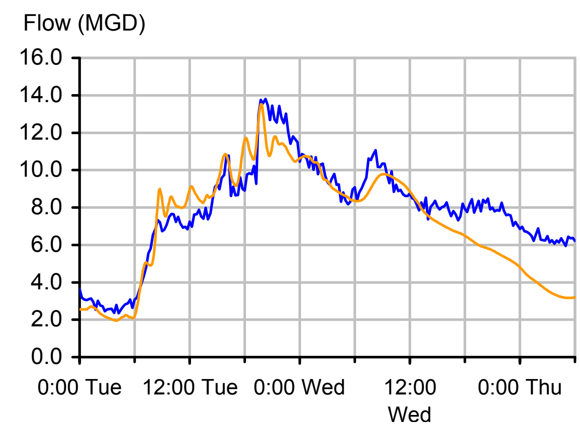


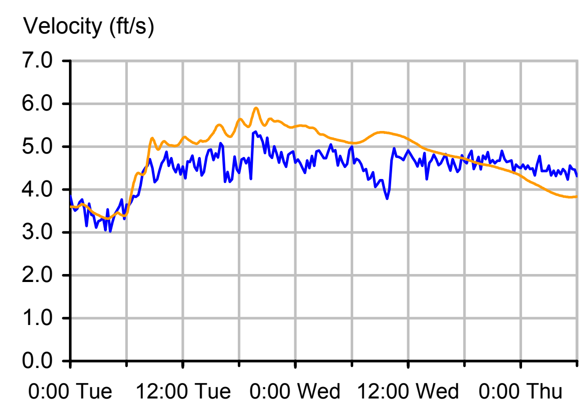


Storm B (Tue Feb 15 - Thu Feb 17 2005)





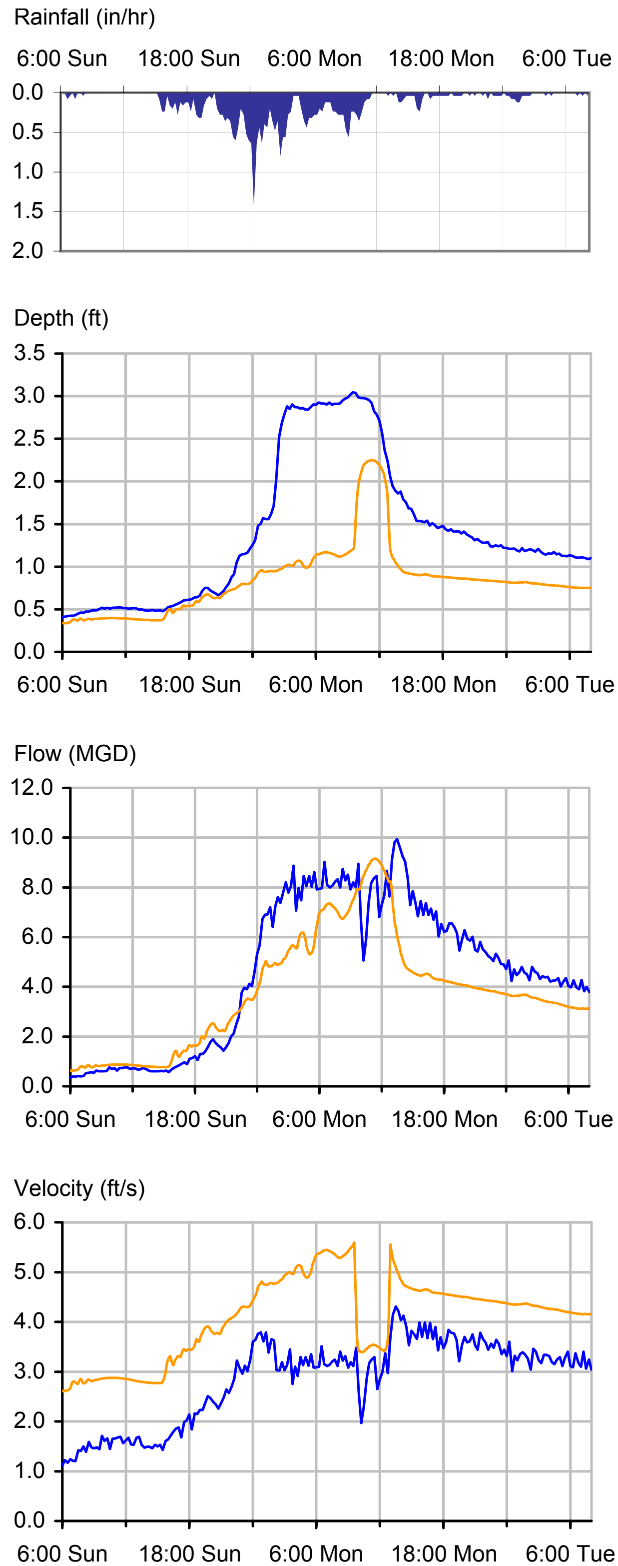




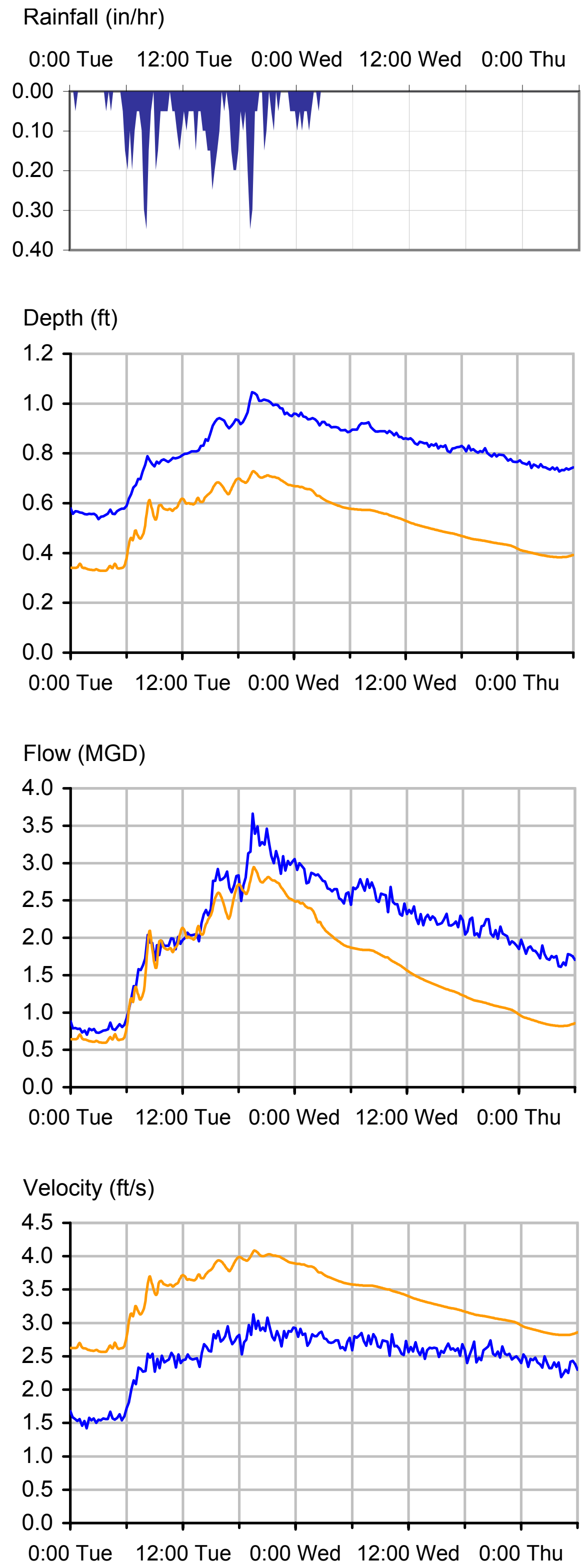
Verification Results - FM10 (K100.24.1)

Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



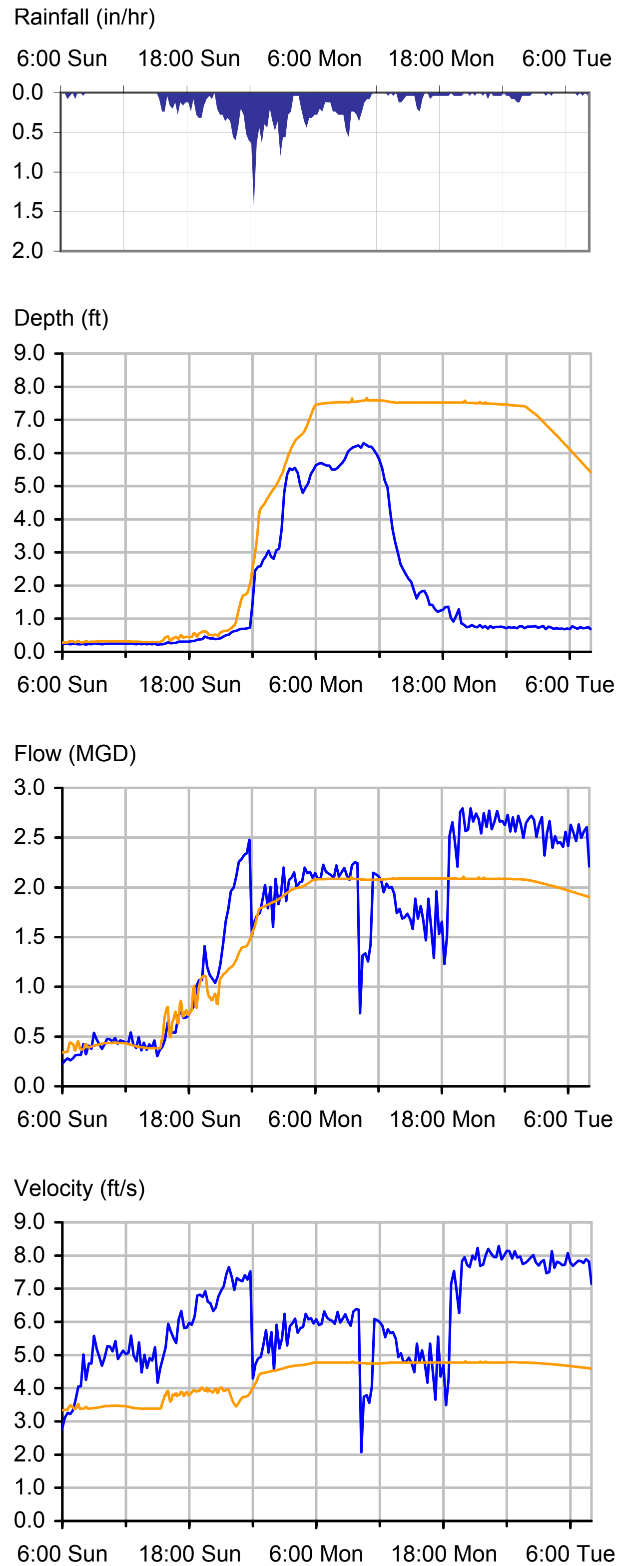
Storm B (Tue Feb 15 - Thu Feb 17 2005)



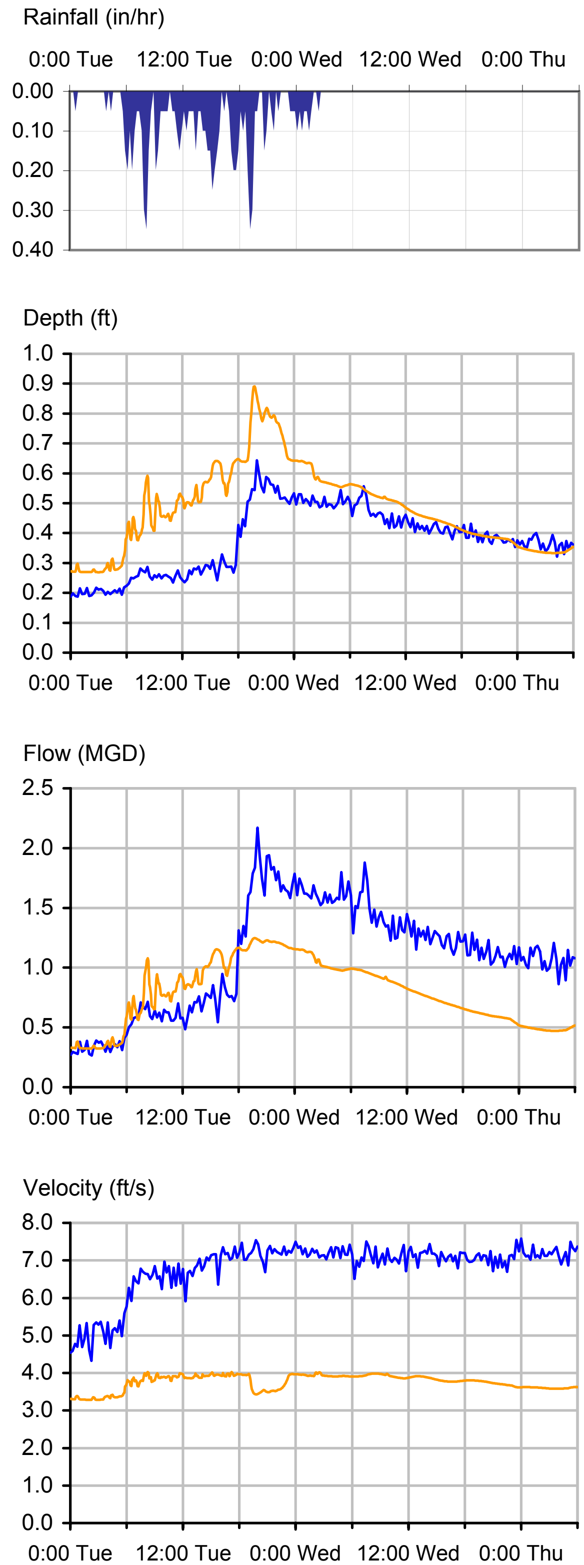
Verification Results - FM11 (W514.09.1)

Flow Monitoring   Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



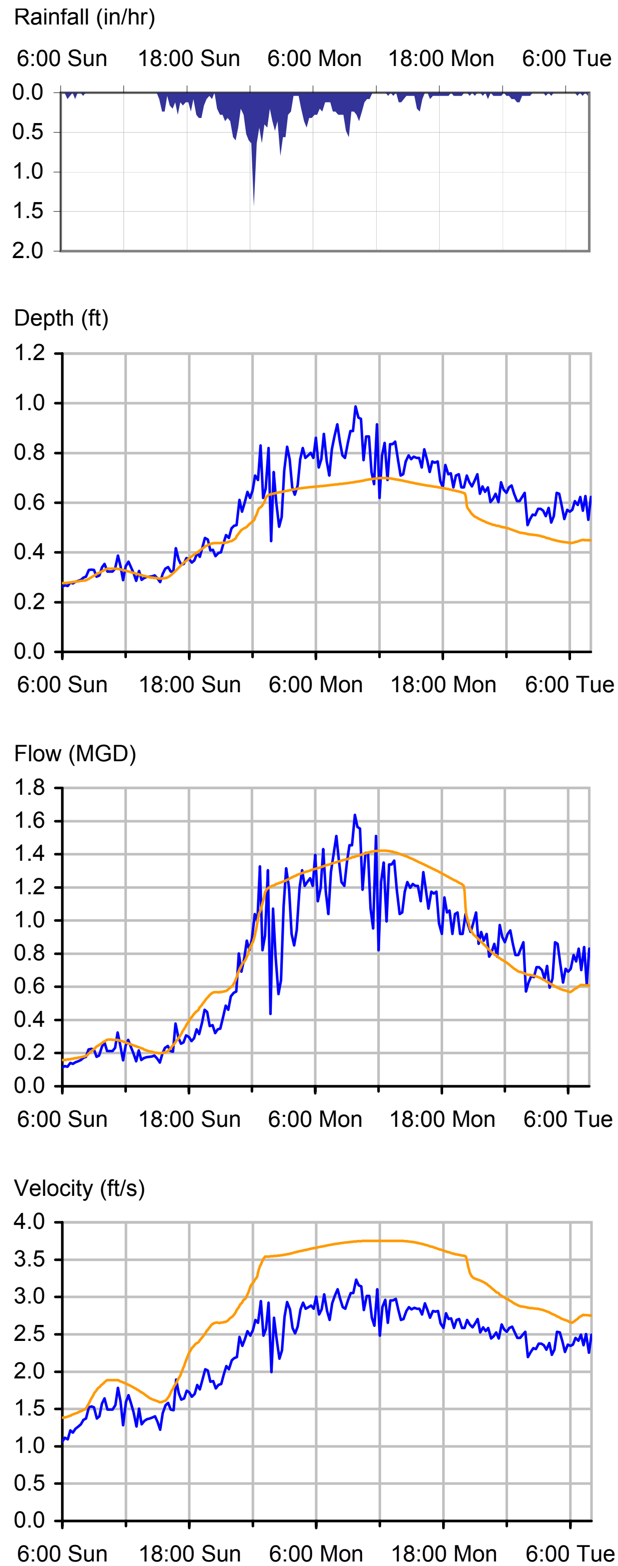
Storm B (Tue Feb 15 - Thu Feb 17 2005)



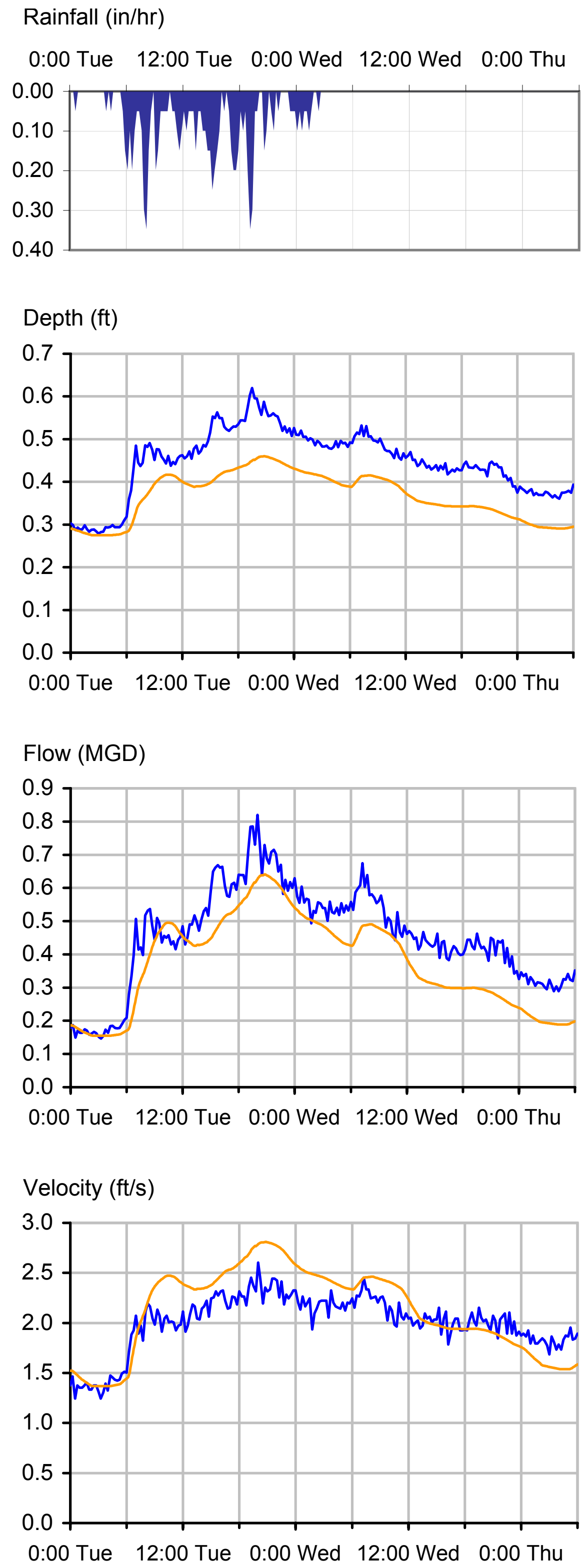
Verification Results - FM12 (K200.02.1)

Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



Storm B (Tue Feb 15 - Thu Feb 17 2005)

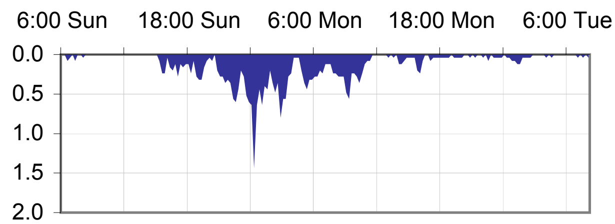


Verification Results - FM13 (G300.02.1)

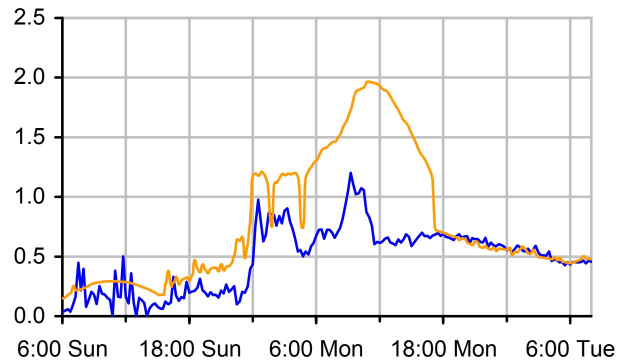
Flow Monitoring   Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)

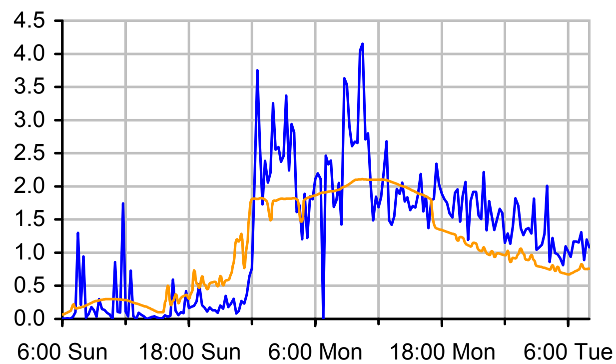
Rainfall (in/hr)



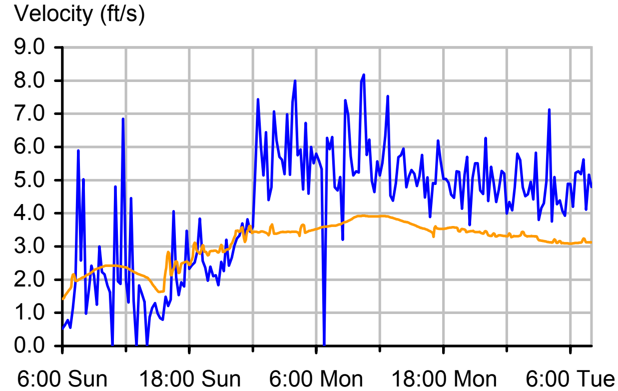
Depth (ft)



Flow (MGD)

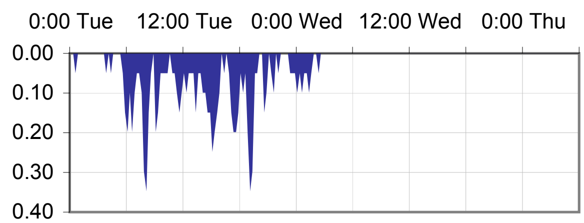


Velocity (ft/s)

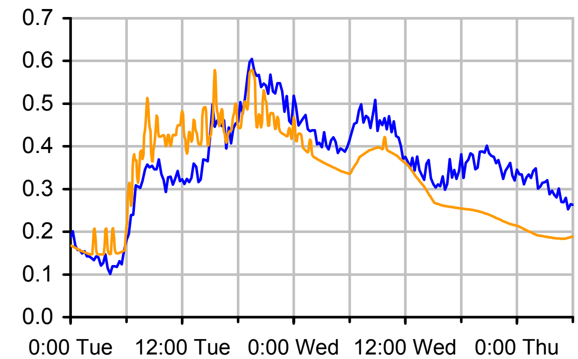


Storm B (Tue Feb 15 - Thu Feb 17 2005)

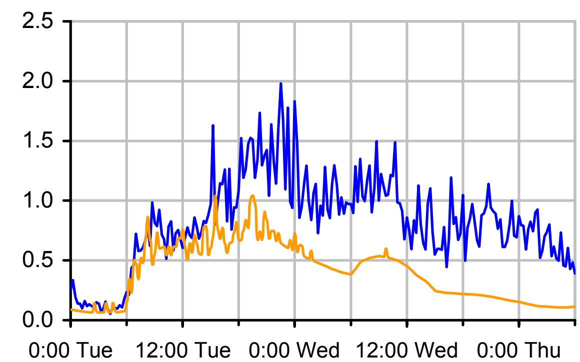
Rainfall (in/hr)



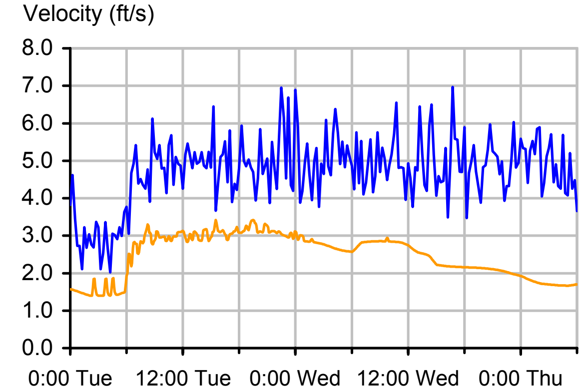
Depth (ft)



Flow (MGD)



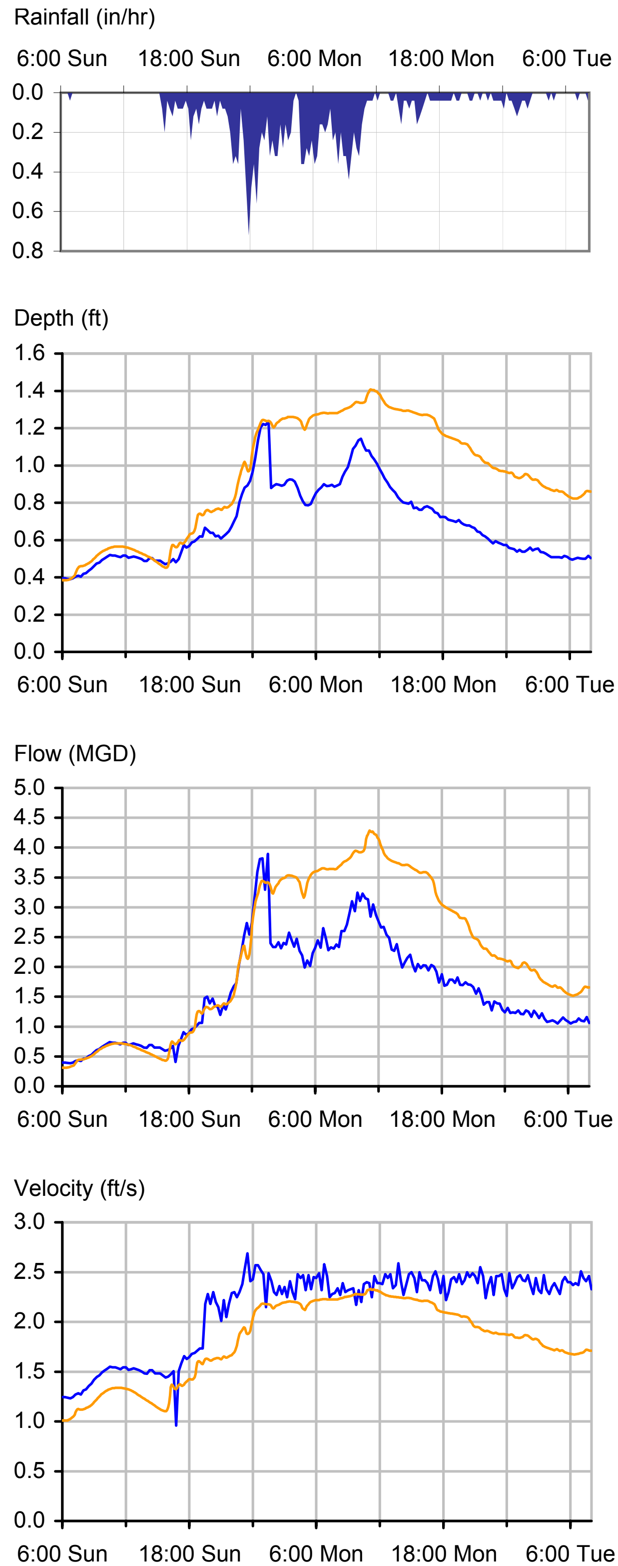
Velocity (ft/s)



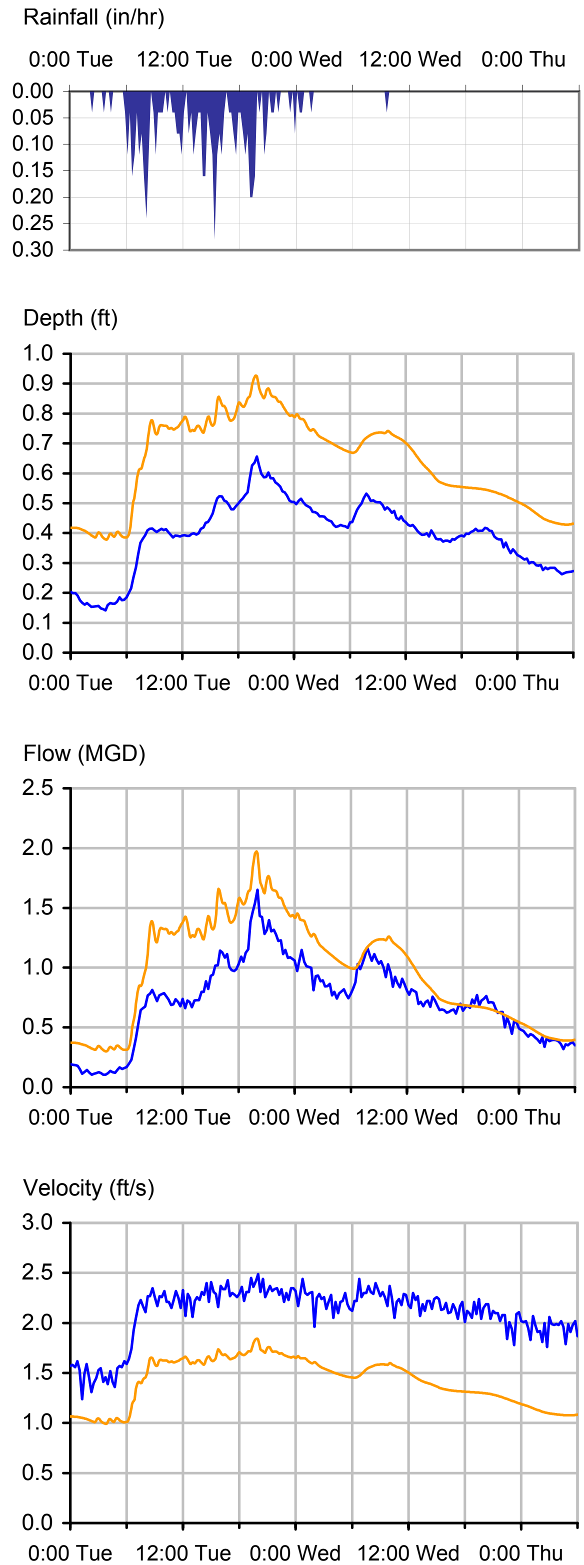
Verification Results - FM14 (G000.11.1)

Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



Storm B (Tue Feb 15 - Thu Feb 17 2005)

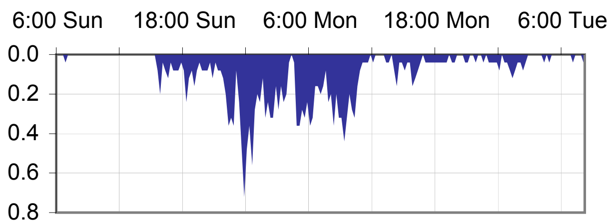


Verification Results - FM15 (G200.01.1)

Flow Monitoring — Model Simulation

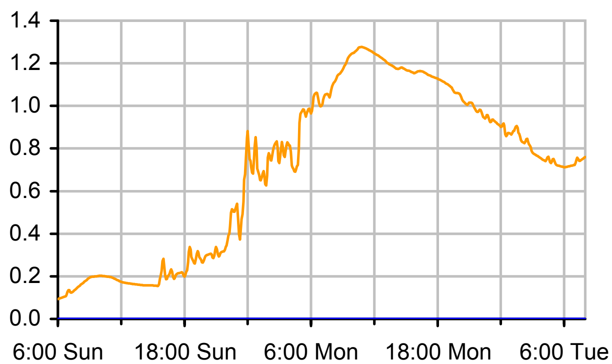
Storm A (Sun Dec 26 - Tue Dec 28 2004)

Rainfall (in/hr)

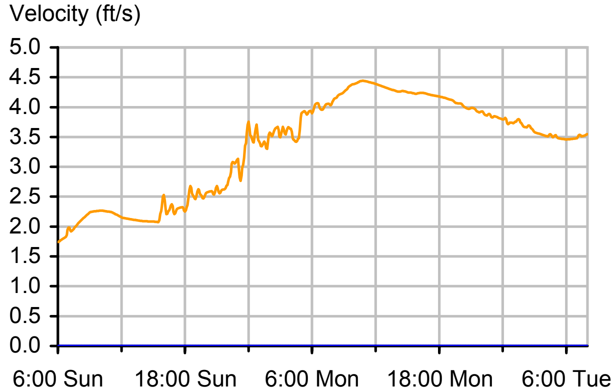


Depth (ft)

Flow (MGD)

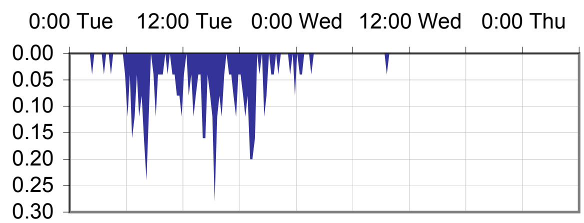


Velocity (ft/s)

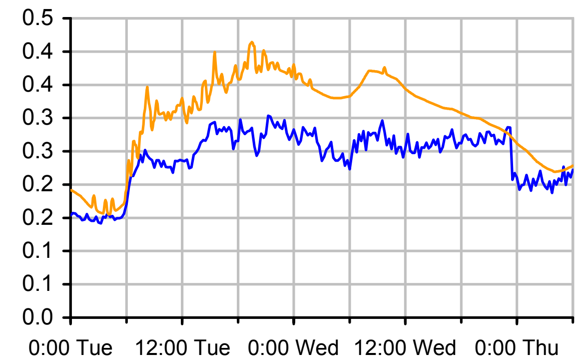


Storm B (Tue Feb 15 - Thu Feb 17 2005)

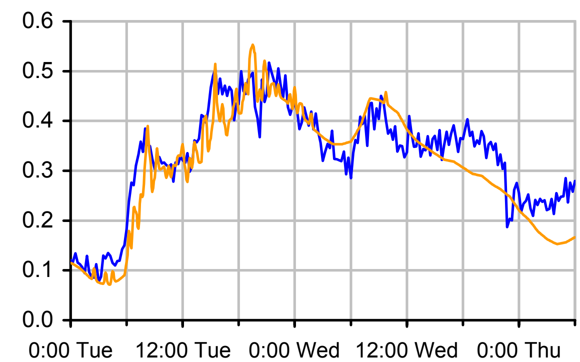
Rainfall (in/hr)



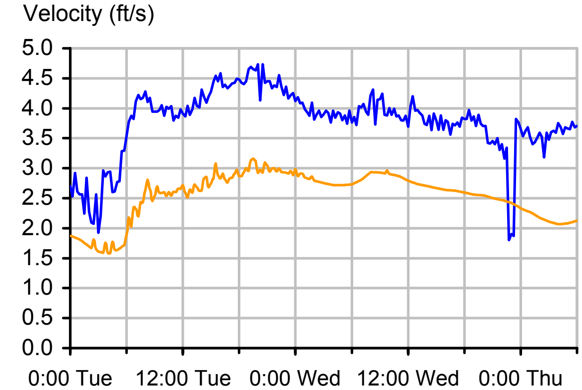
Depth (ft)



Flow (MGD)



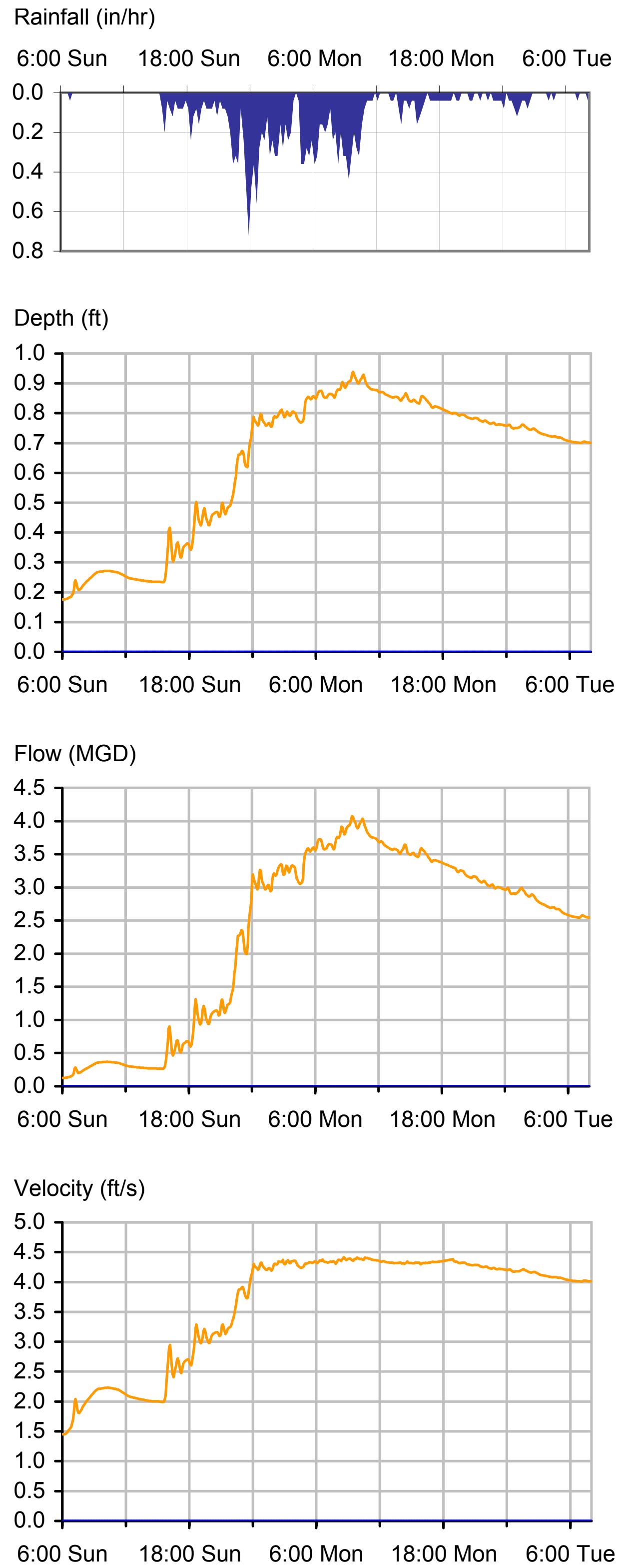
Velocity (ft/s)



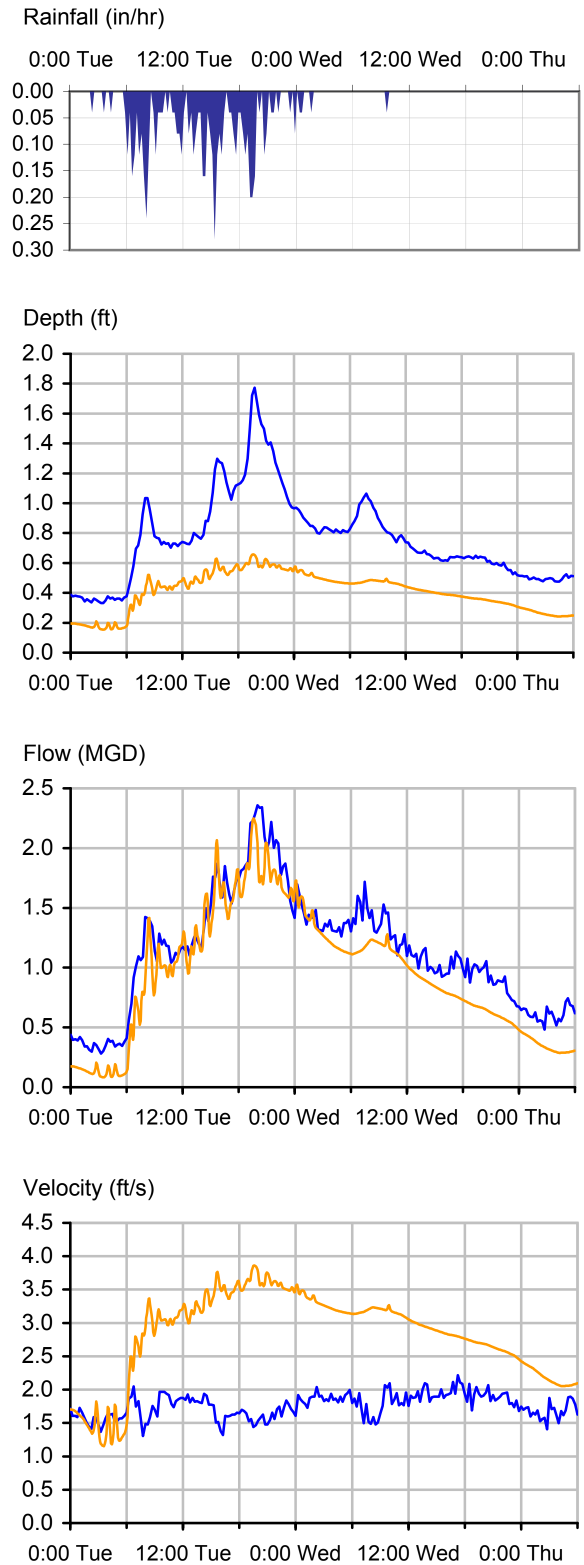
Verification Results - FM16 (L151.01.1)

Flow Monitoring   Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



Storm B (Tue Feb 15 - Thu Feb 17 2005)

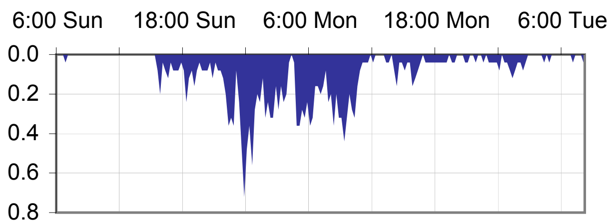


Verification Results - FM17 (L152.04.1)

Flow Monitoring   Model Simulation

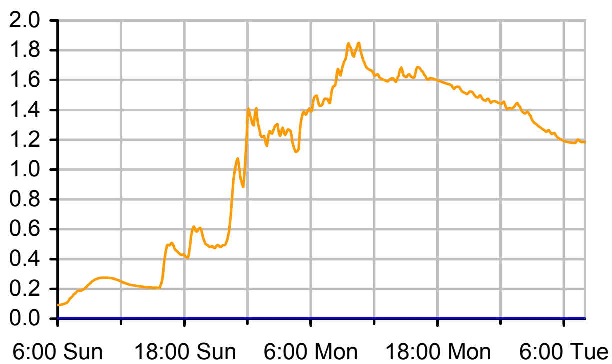
Storm A (Sun Dec 26 - Tue Dec 28 2004)

Rainfall (in/hr)

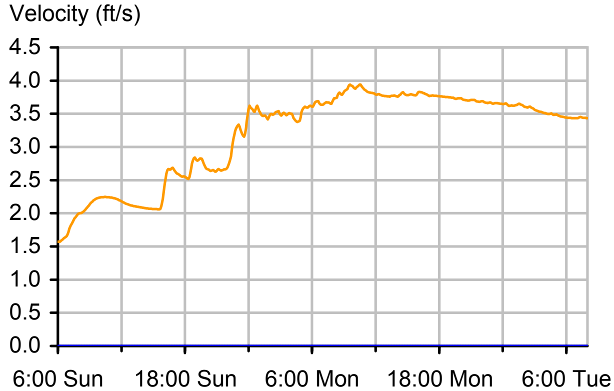


Depth (ft)

Flow (MGD)

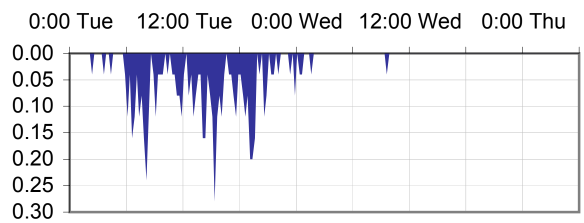


Velocity (ft/s)

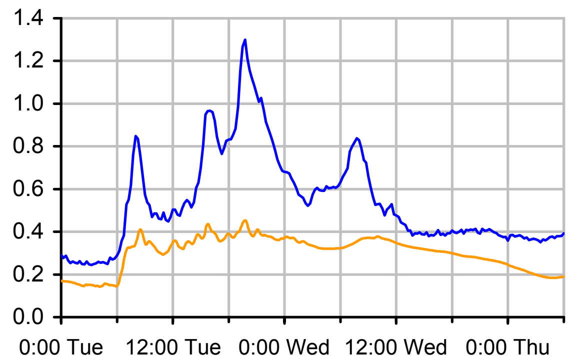


Storm B (Tue Feb 15 - Thu Feb 17 2005)

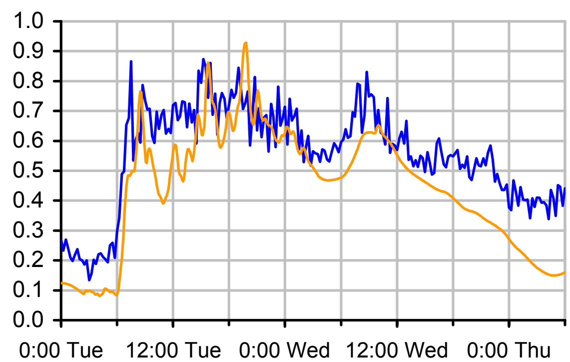
Rainfall (in/hr)



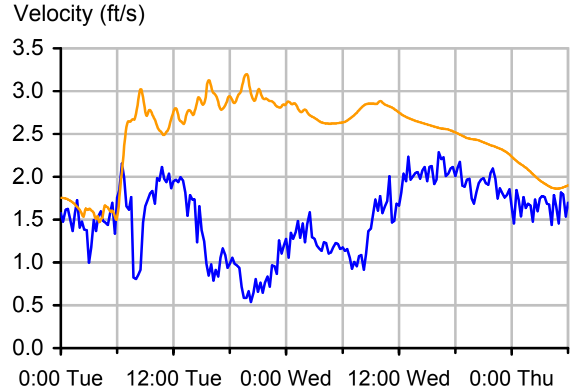
Depth (ft)



Flow (MGD)



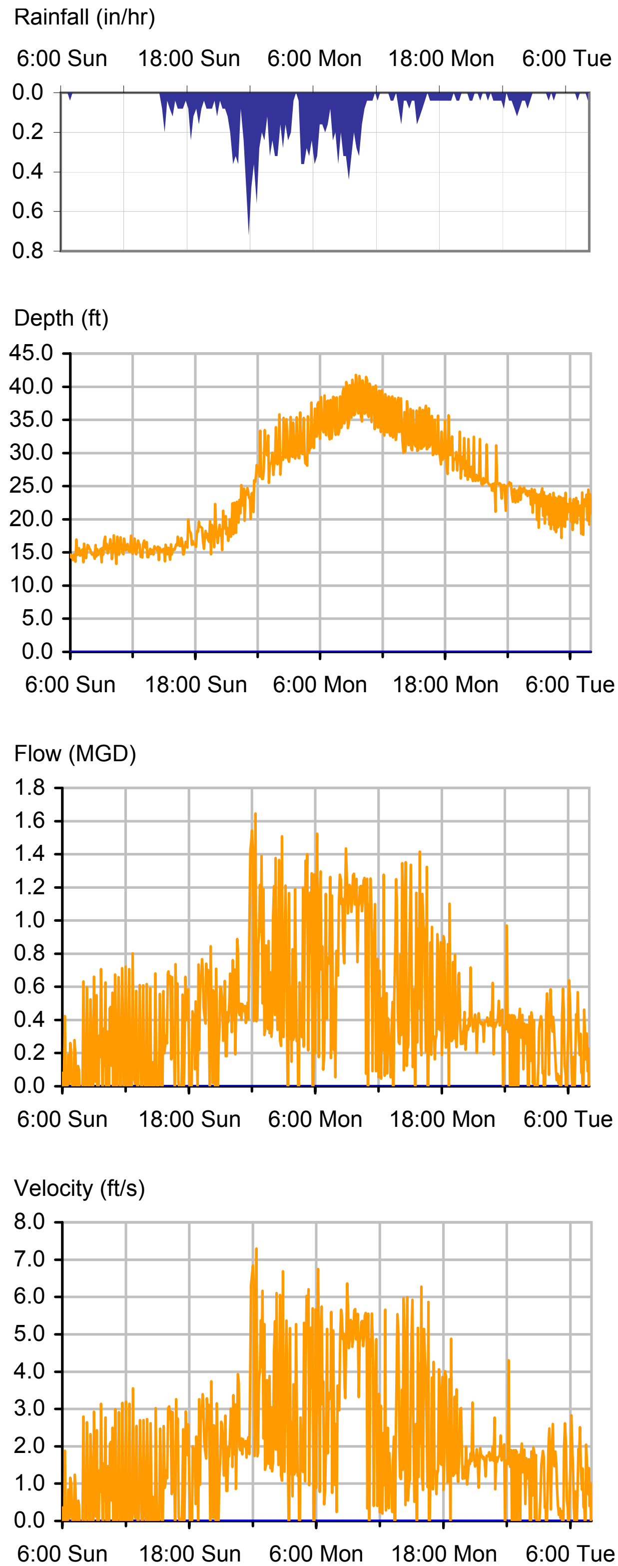
Velocity (ft/s)



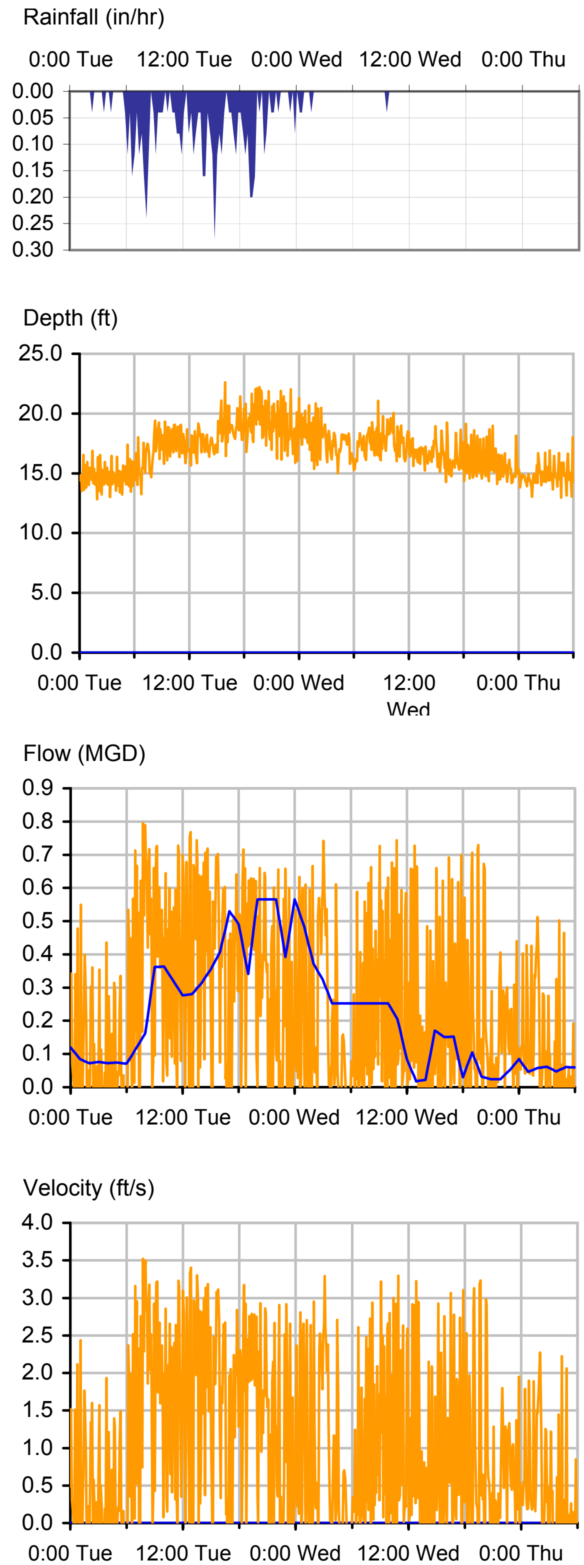
Verification Results - FM18 (PS12-FM.1)

Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



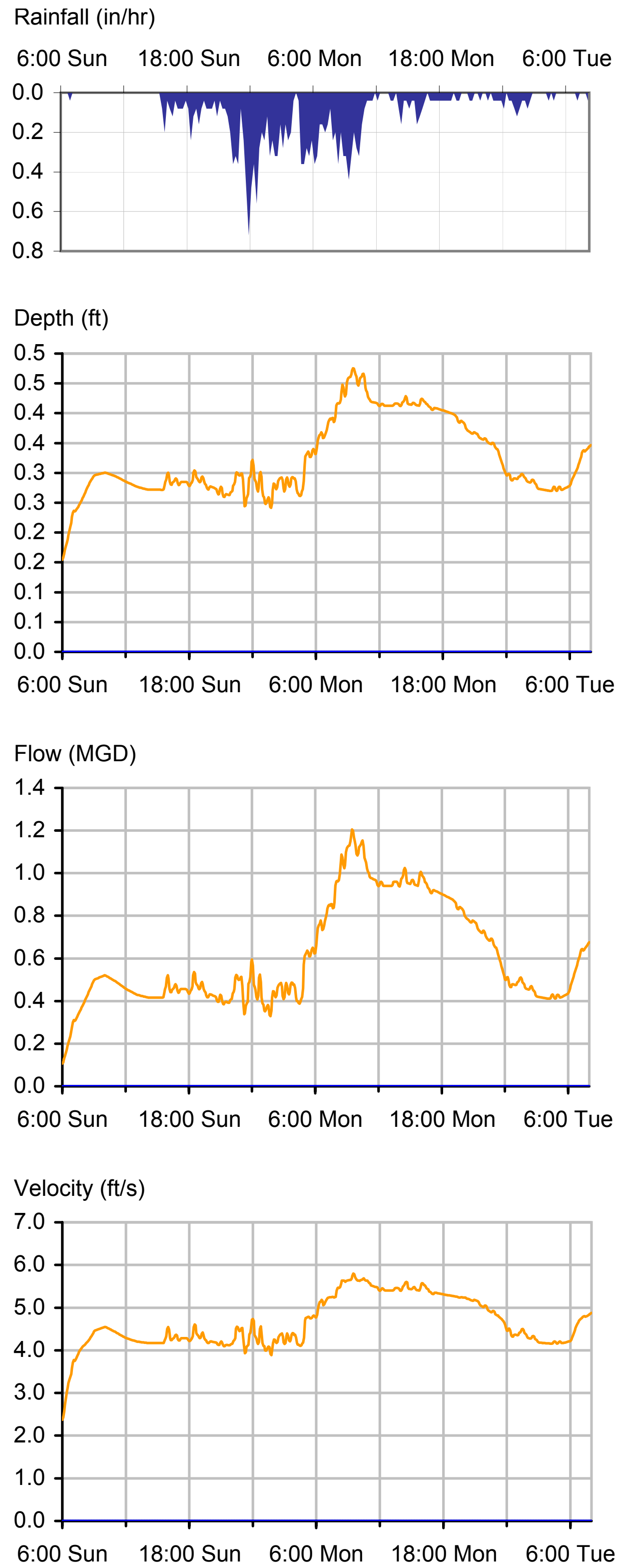
Storm B (Tue Feb 15 - Thu Feb 17 2005)



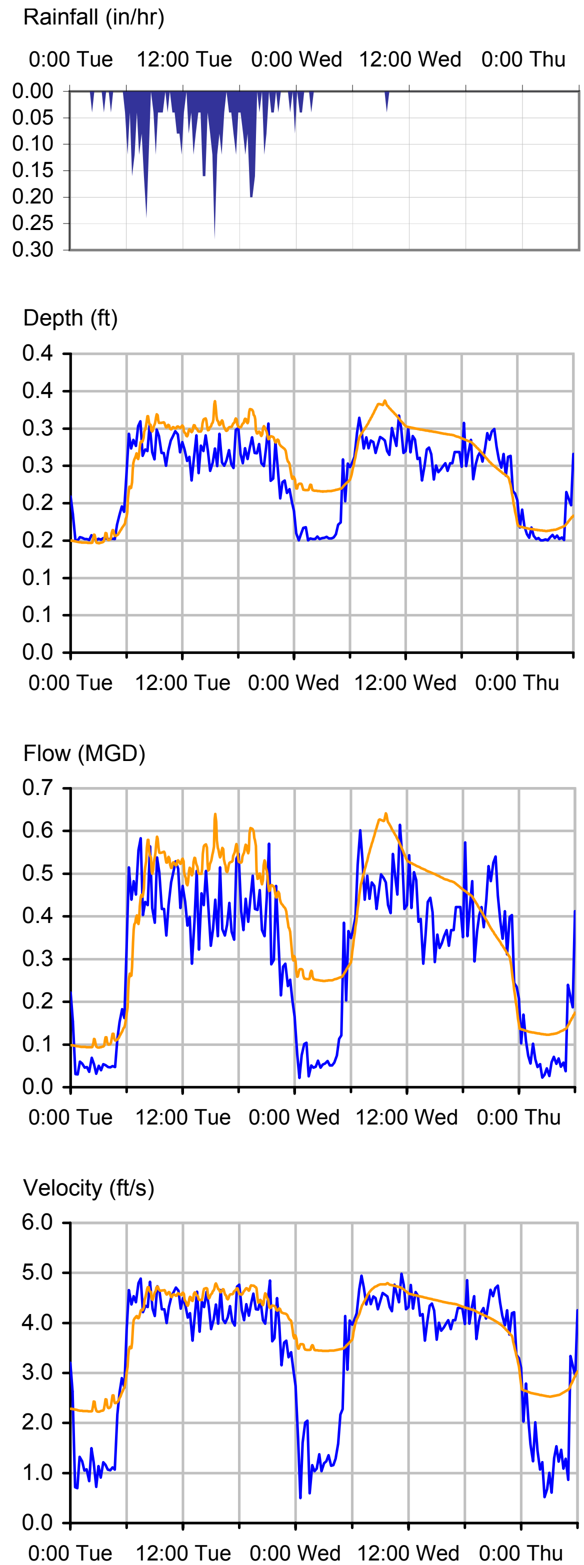
Verification Results - FM19 (L000.05.1)

Flow Monitoring — Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



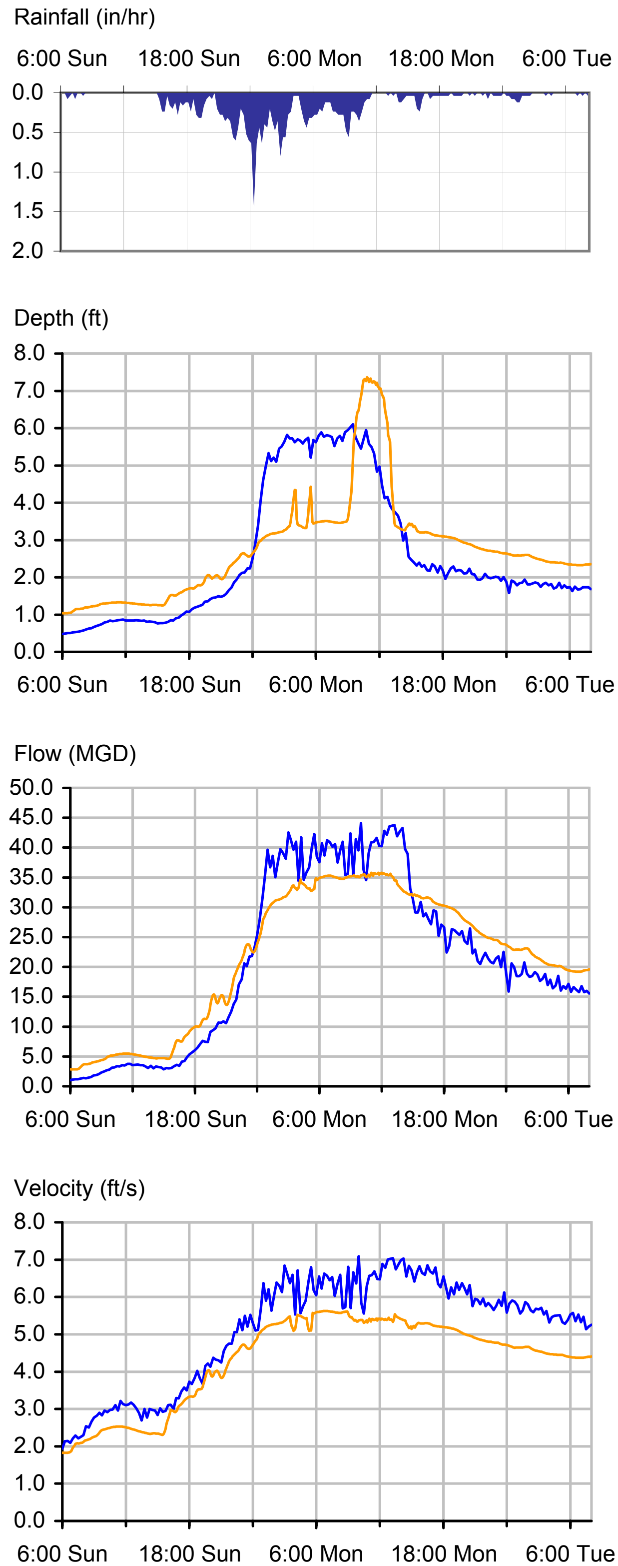
Storm B (Tue Feb 15 - Thu Feb 17 2005)



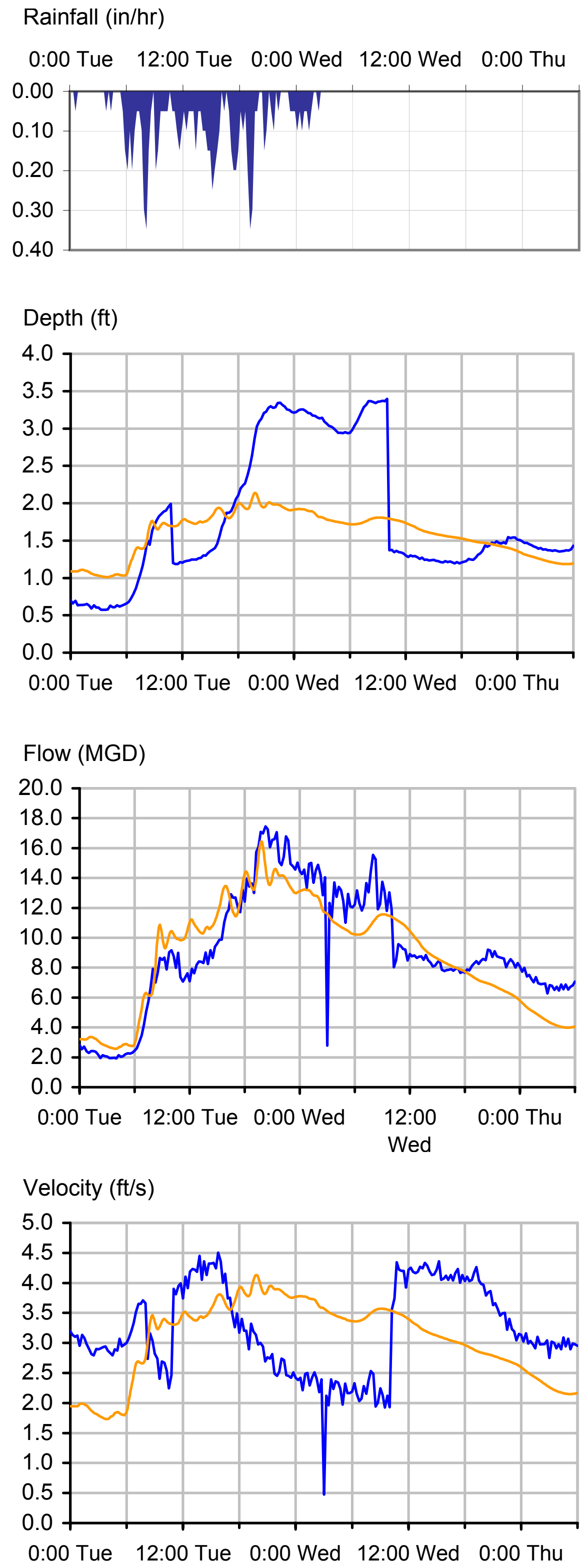
Verification Results - FM20 (K000.03.1)

Flow Monitoring   Model Simulation

Storm A (Sun Dec 26 - Tue Dec 28 2004)



Storm B (Tue Feb 15 - Thu Feb 17 2005)



Westbrae/Hawthorne

PROJECT ID……………………………………	**1**
LOCATION………………………………………	Parallel to and southwest of Sir Francis Drake along apartment frontage, crosses creek, along Westbrae Dr., terminates at Hawthorne Ct.
BRIEF PROJECT DESCRIPTION………………	Upsize existing 8-inch pipe to 10-inch using pipe bursting
PROJECT JUSTIFICATION……………………	Relieve predicted existing capacity deficiency (F700.06 to F003.26)
SPECIAL CONSIDERATIONS…………………	Creek crossing, sideyard easements.
ASSUMPTIONS…………………………………	Existing manholes assumed to be in good condition. Creek crossing not a siphon.
ALTERNATIVES…………………………………	

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 8" sewer from F700.06 to F003.26	10		1,278'	157 $/ft	$200,709
Lower lateral replacement			19	2,500 $/ea	$47,500
Subtotal					$248,209
Mobilization and Demobilization				5%	$12,410
Construction Cost Subtotal					**$260,619**
Contingencies for Unknown Conditions				30%	$78,186
Construction Cost Total					**$338,805**
Engineering, Administration, and Legal Costs				25%	$84,701
Capital Improvement Cost Total					**$423,506**
				rounded	**$424,000**

Spruce/Park/Merwin/Broadway

PROJECT ID……………………………………… **2**

LOCATION………………………………………… Spruce/Park/Merwin/Broadway from to Arroyo to Pacheco

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewer on Spruce and Park from Arroyo to Merwin, then install new diversion sewer on Merwin to Broadway, connect to existing trunk sewer at Pacheco.

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiency in sewers in Park, Dominga, Napa, and Pacheco (F003.12 to F002.09 to F000.77)

SPECIAL CONSIDERATIONS………………… Heavy traffic on Broadway & Bolinas, creek crossing on Merwin, hill on Broadway.

ASSUMPTIONS…………………………………

ALTERNATIVES………………………………… Horizontal Directional Drilling (HDD)

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipe bursting on Spruce & Park existing 15" from F003.12 to F003.08	18		405'	254 $/ft	$102,870
Lower lateral replacement			6	2,500 $/ea	$15,000
Open cut on Merwin from Park to siphon	18		200'	281 $/ft	$56,200
Double Barrell Siphon (10" and 15" pipe) under creek on Merwin NE of Park (microtunnel)	18		200'	389 $/ft	$77,800
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			1 pits	35,000 $/ea	$35,000
Open cut on Merwin from Park to Broadway	18		300'	281 $/ft	$84,300
Microtunnel hill on Broadway between Merwin and Bank Sts.	18		650'	518 $/ft	$336,700
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			1 pits	35,000 $/ea	$35,000
Open Cut on Broadway from Bank St. to Pacheco Ave.	18		650'	281 $/ft	$182,650
Subtotal					$1,025,520
Mobilization and Demobilization				5%	$51,276
Construction Cost Subtotal					**$1,076,796**
Contingencies for Unknown Conditions				30%	$323,039
Construction Cost Total					**$1,399,835**
Engineering, Administration, and Legal Costs				25%	$349,959
Capital Improvement Cost Total					**$1,749,794**
				rounded	**$1,750,000**

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Cascade

PROJECT ID……………………………………… 3

LOCATION………………………………………… On Cascade Road

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewer pipe

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies (F330.05 to F303.04)

SPECIAL CONSIDERATIONS…………………

ASSUMPTIONS………………………………… Pipebursting

ALTERNATIVES………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 6" on Cascade from F330.05 to F303.04	8		1,727'	132 $/ft	$227,990
Lower lateral replacement			43	2,500 $/ea	$107,500
Subtotal					$335,490
Mobilization and Demobilization				5%	$16,775
Construction Cost Subtotal					**$352,265**
Contingencies for Unknown Conditions				30%	$105,679
Construction Cost Total					**$457,944**
Engineering, Administration, and Legal Costs				25%	$114,486
Capital Improvement Cost Total					**$572,430**
				rounded	**$572,000**

Creek/Bolinas

PROJECT ID.. **4**

LOCATION.. On Bolinas and Creek Roads, and in easement in ravine parallel to and northwest of Bolinas

BRIEF PROJECT DESCRIPTION.................. Upsize existing sewer pipe

PROJECT JUSTIFICATION........................ Relieve predicted existing capacity deficiencies (F002.28 to F002.11)

SPECIAL CONSIDERATIONS...................... Heavy traffic on Bolinas, creek crossing on Creek, difficult access to ravine off Bolinas

ASSUMPTIONS.. Pipebursting

ALTERNATIVES.......................................

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 10" in ravine and on Bolinas from F002.28 to F002.17	12"		3,068'	181 $/ft	$555,272
Pipeburst existing 10" on Porteous and Creek Rd. from F002.17 to F002.12	15"		1,011'	218 $/ft	$220,420
Lower lateral replacement			62	2,500 $/ea	$155,000
Remove & Replace existing 12" Creek/Bridge crossing F002.12 to F002.11	15"		196'	260 $/ft	$50,830
Subtotal					$981,522
Mobilization and Demobilization				5%	$49,076
Construction Cost Subtotal					**$1,030,598**
Contingencies for Unknown Conditions				30%	$309,179
Construction Cost Total					**$1,339,777**
Engineering, Administration, and Legal Costs				25%	$334,944
Capital Improvement Cost Total					**$1,674,721**
				rounded	**$1,675,000**

Upper Butterfield Road

PROJECT ID…………………………………… **5**

LOCATION……………………………………… On Butterfield Rd. from Van Tassel Ct. to Fawn Dr.

BRIEF PROJECT DESCRIPTION……………… Upsize existing pipes using pipe bursting.

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies (H000.18 to H000.04)

SPECIAL CONSIDERATIONS………………… Busy residential road.

ASSUMPTIONS…………………………………

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 10" from H000.18 to H000.10	12"		2,142'	181 $/ft	$387,720
Pipeburst exisitng 10" from H000.10 to H000.08	15"		463'	218 $/ft	$100,912
Pipeburst existing 12" from H000.08 to H000.04	15"		1,231'	218 $/ft	$268,380
Lower lateral replacement			68	2,500 $/ea	$170,000
Subtotal					$927,012
Mobilization and Demobilization				5%	$46,351
Construction Cost Subtotal					**$973,363**
Contingencies for Unknown Conditions				30%	$292,009
Construction Cost Total					**$1,265,372**
Engineering, Administration, and Legal Costs				25%	$316,343
Capital Improvement Cost Total					**$1,581,714**
				rounded	**$1,582,000**

Lower Butterfield/Meadowcroft/Broadmoor/Sir Francis Drake

PROJECT ID……………………………………… 6

LOCATION………………………………………… Lower Butterfield/Meadowcroft/Broadmoor/Sir Francis Drake

BRIEF PROJECT DESCRIPTION……………… Pipebursting existing sewers and installing new relief sewers

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Butterfield Rd. from Carlson to Arroyo (S960.05 to S960.03); Butterfield, Kilgore Ct., and easements to Brookside (S942.09 to S900.11); and Broadmoor and SFD from Meadowcroft to Mountain View (S900.06 to S800.10).

SPECIAL CONSIDERATIONS…………………… Deep sewer at Butterfield/Meadowcroft area (27'), construction in SFD

ASSUMPTIONS……………………………………

ALTERNATIVES………………………………… Meadowcroft is steep enough to consider HDD (similar costs)

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 6" on Butterfield from MHs S960.05 to S960.03	8"		493'	132 $/ft	$65,023
Remove & Replace existing 6" on Butterfield from MHs S942.09 to S942.08	12"		150'	226 $/ft	$33,968
New relief sewer on Butterfield and Meadowcroft from Kilgore Ct. to Willow Walk (Microtunnel due to depth)	10"		1,200'	346 $/ft	$415,200
Jacking Pits			2 pits	50,000 $/ea	$100,000
Receiving Pits			3 pits	35,000 $/ea	$105,000
New relief sewer on Meadowcroft and Broadmoor to SFD, connect to MH S900.04 (Open cut)	10"		1,100'	190 $/ft	$209,000
New relief sewer in SFD from Broadmoor to Mountain View, connect to MH S800.10	15"		600'	241 $/ft	$144,600
Lower lateral replacement			35	2,500 $/ea	$87,500
Subtotal					$1,160,291
Mobilization and Demobilization				5%	$58,015
Construction Cost Subtotal					**$1,218,306**
Contingencies for Unknown Conditions				30%	$365,492
Construction Cost Total					**$1,583,797**
Engineering, Administration, and Legal Costs				25%	$395,949
Capital Improvement Cost Total					**$1,979,747**
				rounded	**$1,980,000**

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The Alameda/Brookmead

PROJECT ID…………………………………….. **7**

LOCATION………………………………………… The Alameda at Arroyo to Brookmead at Brookside

BRIEF PROJECT DESCRIPTION……………… Pipebursting existing sewers and installing new relief sewers

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies The Alameda and easements from Arroyo to Brookside (S900.20 to S900.12).

SPECIAL CONSIDERATIONS………………… Work in schoolyard.

ASSUMPTIONS…………………………………

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 18" on The Alameda from MHs S900.20 to S900.19	21"		132'	292 $/ft	$38,486
New relief sewer in The Alameda and Berkeley from S900.19 to S900.15	15"		976'	241 $/ft	$235,216
Pipeburst existing 18" in schoolyard and Brookmead from MHs S900.15 to S900.12	21"		535'	292 $/ft	$156,278
Lower lateral replacement			7	2,500 $/ea	$17,500
Subtotal					$447,480
Mobilization and Demobilization				5%	$22,374
Construction Cost Subtotal					**$469,854**
Contingencies for Unknown Conditions				30%	$140,956
Construction Cost Total					**$610,810**
Engineering, Administration, and Legal Costs				25%	$152,703
Capital Improvement Cost Total					**$763,513**
				rounded	**$764,000**

Sonoma/Nokomis

PROJECT ID…………………………………………… **8**

LOCATION…………………………………………… On Alderney, Sonoma, Sais, Nokomis, and Madrone

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewers on Alderney Rd. and Sonoma Ave from southeast of San Francisco to SFD., install new diversion sewer on Sais,Nokomis & Madrone Avenues from SFD to San Anselmo Ave.

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Alderney and Sonoma (S610.03 to S608.03); SFD, Bella Vista, and easements to Sycamore Ave. (S608.03 to S600.07 to S001.10)

SPECIAL CONSIDERATIONS…………………… SFDrake crossing, creek crossing on Nokomis

ASSUMPTIONS……………………………………

ALTERNATIVES………………………………… Pipeburst PVC

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Remove & replace existing 10" PVC sewer on Alderney from S610.03 to S610.01	12"		485'	226 $/ft	$109,610
Remove & Replace existing 12" PVC sewer on Sonoma from MHs S610.01 to S609.01	15"		480'	260 $/ft	$124,904
Microtunnel new sewer across SFD and along Sais to Nokomis	15"		400'	432 $/ft	$172,800
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			1 pits	35,000 $/ea	$35,000
Remove & replace ex. sewer on Nokomis from Sais to creek (Open cut)	15"		600'	260 $/ft	$156,000
New double barrel siphon under Creek (microtunnel)	10"		200'	346 $/ft	$69,200
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			1 pits	35,000 $/ea	$35,000
Remove & replace ex. sewer on Nokomis and Madrone from creek to San Anselmo, connect to MH S000.52 (Open cut)	15"		600'	260 $/ft	$156,000
Lower lateral replacement			35	2,500 $/ea	$87,500
Subtotal					$1,046,014
Mobilization and Demobilization				5%	$52,301
Construction Cost Subtotal					**$1,098,315**
Contingencies for Unknown Conditions				30%	$329,494
Construction Cost Total					**$1,427,809**
Engineering, Administration, and Legal Costs				25%	$356,952
Capital Improvement Cost Total					**$1,784,761**
				rounded	**$1,785,000**

Miracle Mile

PROJECT ID……………………………………. **9**

LOCATION………………………………………. Greenfield Ave. and Sir Francis Drake from Hilldale to Tunstead

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewer on Greenfield and construct new diversion sewer in SFD

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Hilldale, Greenfield, and Center from Red Hill to Bridge (S400.11 to S001.07)

SPECIAL CONSIDERATIONS………………… Heavy traffic area, downtown commercial district.

ASSUMPTIONS…………………………………. Pipeburst existing sewer, microtunnel new sewer. No laterals

ALTERNATIVES……………………………….

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 8" on Red Hill, MHs S400.11 to S400.10	12"		36'	181 $/ft	$6,588
Replace existing 6" on Greenfield from MHs S400.10 to S400.06	12"		1,258'	226 $/ft	$284,376
Pipeburst existing 10" on Greenfield from MHs S400.06 to S400.04	12"		709	181 $/ft	$128,293
Microtunnel across SFD/Red Hill Intersection, from S400.04 south on SFD to creek	12"		900	346 $/ft	$311,400
New double barrel siphon under Creek (microtunnel)	10"		200'	346 $/ft	$69,200
Microtunnel on SFD from siphon to ex. MH S000.40	12"		150	346 $/ft	$51,900
Jacking Pits			2 pits	50,000 $/ea	$100,000
Receiving Pits			2 pits	35,000 $/ea	$70,000
Subtotal					$1,021,757
Mobilization and Demobilization				5%	$51,088
Construction Cost Subtotal					**$1,072,845**
Contingencies for Unknown Conditions				30%	$321,853
Construction Cost Total					**$1,394,698**
Engineering, Administration, and Legal Costs				25%	$348,675
Capital Improvement Cost Total					**$1,743,373**
				rounded	**$1,743,000**

Sir Francis Drake/Winship

PROJECT ID……………………………………	**10**
LOCATION………………………………………	On Sir Francis Drake from Barber Ave. to Bolinas; Winship from Barber to SFD; and Bolinas from SFD to Shady Ln.
BRIEF PROJECT DESCRIPTION………………	Upsize existing sewers, install new relief sewer.
PROJECT JUSTIFICATION……………………	Relieve predicted existing capacity deficiencies in SFD (S200.04 to S200.01); Winship (S215.01 to S200.01); and Boinas (S200.01 to S000.34 to S00032)
SPECIAL CONSIDERATIONS…………………	Heavy traffic on SFD, old bridge on Winship
ASSUMPTIONS…………………………………	Pipebursting on SFD, new sewer on Winship bridge (off side), new sewer across SF Drake to Shady Lane Trunk.
ALTERNATIVES…………………………………	

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 8" on SFD, MHs S200.04 to S200.01	12"		1,123'	181 $/ft	$203,335
Pipeburst existing 6" on SFD, MHs S200.00 to S200.01	10"		149'	157 $/ft	$23,409
Install new pipe on Winship Bridge from S215.01 to S200.00	8"		190'	$179	$34,046
Microtunnel new sewer across SFD and along Bolinas from S200.01 to S000.32	12"		450'	$346	$155,700
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			2 pits	35,000 $/ea	$70,000
Lower lateral replacement			14	2,500 $/ea	$35,000
Subtotal					$571,490
Mobilization and Demobilization				5%	$28,574
Construction Cost Subtotal					**$600,064**
Contingencies for Unknown Conditions				30%	$180,019
Construction Cost Total					**$780,084**
Engineering, Administration, and Legal Costs				25%	$195,021
Capital Improvement Cost Total					**$975,105**
				rounded	**$975,000**

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Bolinas/Fernhill

PROJECT ID…………………………………… **11**

LOCATION………………………………………… Bolinas Ave. and Fernhill Ave. west of Shady Ln.

BRIEF PROJECT DESCRIPTION……………… Pipeburst ex. Sewers on Bolinas and Fernhill

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Bolinas (S101.06 to S000.32) and Fernhill (R564.04 to R000.31)

SPECIAL CONSIDERATIONS…………………

ASSUMPTIONS…………………………………

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 10" on Bolinas, MHs S101.06 to S101.05	12"		259'	181 $/ft	$46,843
Pipeburst existing 10" on Bolinas, MHs S101.05 to S000.32	15"		1,202'	218 $/ft	$261,971
Pipeburst existing 10" on Fernhill, MHs S564.04 to R000.31	15"		851'	218 $/ft	$185,583
Lower lateral replacement			54	2,500 $/ea	$135,000
Subtotal					$629,397
Mobilization and Demobilization				5%	$31,470
Construction Cost Subtotal					**$660,867**
Contingencies for Unknown Conditions				30%	$198,260
Construction Cost Total					**$859,127**
Engineering, Administration, and Legal Costs				25%	$214,782
Capital Improvement Cost Total					**$1,073,908**
				rounded	**$1,074,000**

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Upper Shady Lane Trunk Sewer

PROJECT ID……………………………………	**12**
LOCATION…………………………………………	Shady Lane from Bolinas to Norwood.
BRIEF PROJECT DESCRIPTION………………	New relief sewer
PROJECT JUSTIFICATION……………………	Relieve predicted existing capacity deficiencies in Ross Valley trunk sewer (S000.32 to R000.29 to K000.03) by diverting a portion of flow to old trunk sewer in Shady Ln.
SPECIAL CONSIDERATIONS…………………	Existing 21"/24" sewer in Shady Lane and Kent Ave. (old Ross Valley trunk sewer) needs rehabilitation; CIPP lining assumed from R500.16 to R500.01 but not included in this project.
ASSUMPTIONS……………………………………	Current creek crossing (R500.18 to R500.17) is at creek invert (visible). New sewer will be at lower elev., therefore siphon will not be necessary. Will need to microtunnel across creek however. Abandon existing sewer from R000.29 to R500.16. No laterals
ALTERNATIVES…………………………………	

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
New 24" sewer on Shady Lane, MHs S000.32 to R000.29 (open cut)	24"		800'	359 $/ft	$287,200
New relief sewer at lower elevation, MH R500.18 to R500.17 (microtunnel)	21"		150'	605 $/ft	$90,750
Remove and replace existing 21" sewer. Lower sewer to new elevation, MH R500.17 to R500.16. (open cut)	21"		500'	314 $/ft	$157,000
Subtotal					$534,950
Mobilization and Demobilization				5%	$26,748
Construction Cost Subtotal					**$561,698**
Contingencies for Unknown Conditions				30%	$168,509
Construction Cost Total					**$730,207**
Engineering, Administration, and Legal Costs				25%	$182,552
Capital Improvement Cost Total					**$912,758**
				rounded	**$913,000**

Sir Francis Drake/Berry

PROJECT ID……………………………………… 13

LOCATION………………………………………… Sir Francis Drake from Laurel Grove Ave. to Berry Ln.

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewer

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiency (R400.05 to R000.15)

SPECIAL CONSIDERATIONS………………… Heavy traffic on SFD

ASSUMPTIONS…………………………………… Pipebursting

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 10" sewer on SFD, MHs R400.05 to R400.01	15"		998'	218 $/ft	$217,477
Pipeburst existing 12" sewer on SFD, MHs R400.01 to R000.15	15"		105'	200 $/ft	$21,060
Lower lateral replacement			15	2,500 $/ea	$37,500
Subtotal					$276,037
Mobilization and Demobilization				5%	$13,802
Construction Cost Subtotal					**$289,839**
Contingencies for Unknown Conditions				30%	$86,952
Construction Cost Total					**$376,790**
Engineering, Administration, and Legal Costs				25%	$94,198
Capital Improvement Cost Total					**$470,988**
				rounded	**$471,000**

Goodhill

PROJECT ID…………………………………… **14**

LOCATION……………………………………… Goodhill Rd. from Live Oak to Vineyard and easement from Goodhill to Kent Ave.

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewer

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies (W101.10 to W100.28)

SPECIAL CONSIDERATIONS………………… Sideyard easement at W101.00 to W100.28

ASSUMPTIONS………………………………… pipebursting

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 6" sewer on Goodhill, MHs W101.10 to W101.00	10"		1,691'	157 $/ft	$265,503
Pipeburst existing 10" sideyard sewer, MHs W101.00 to W100.28	12"		513'	166 $/ft	$85,092
Replace existing 8" sewer on Kent, MHs W100.28 to R100.26T	15"		189'	260 $/ft	$49,140
Lower lateral replacement			20	2,500 $/ea	$50,000
Subtotal					$449,734
Mobilization and Demobilization				5%	$22,487
Construction Cost Subtotal					**$472,221**
Contingencies for Unknown Conditions				30%	$141,666
Construction Cost Total					**$613,887**
Engineering, Administration, and Legal Costs				25%	$153,472
Capital Improvement Cost Total					**$767,359**
				rounded	**$767,000**

4/24/2006

Woodland/College

PROJECT ID…………………………………… 15

LOCATION……………………………………… Woodland Rd. from Evergreen to Kent Ave., College Ave. from Woodland to Stadium Way

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewer on Woodland, new relief sewer on College, north to existing trunk

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Woodland and Kent Ave. (W514.10 to W514.07 to W100.32 to W100.27 to K100.25)

SPECIAL CONSIDERATIONS………………… New siphon across storm culvert (adjacent to ex. siphons)

ASSUMPTIONS………………………………… Remove and replace in Woodland due to diameter increse (10" to 18", 12" to 21")

ALTERNATIVES………………………………… Trenchless methods, pipebursting, reaming, etc.

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Remove & replace existing 10" sewer on Woodland, MHs W514.10 to W514.07	18"		1,000'	295 $/ft	$295,000
Remove & replace existing 12" sewer on Woodland, MHs W514.07 to W512.01 to K511.03	21"		600'	314 $/ft	$188,400
Siphon under strorm drain channel (microtunnel)	12"		90'	346 $/ft	$31,209
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			1 pits	35,000 $/ea	$35,000
New relief sewer from new siphon to K100.25 (open cut)	12"		650'	209 $/ft	$135,850
Lower lateral replacement			12	2,500 $/ea	$30,000
Subtotal					$765,459
Mobilization and Demobilization				5%	$38,273
Construction Cost Subtotal					**$803,732**
Contingencies for Unknown Conditions				30%	$241,120
Construction Cost Total					**$1,044,852**
Engineering, Administration, and Legal Costs				25%	$261,213
Capital Improvement Cost Total					**$1,306,065**
				rounded	**$1,306,000**

4/24/2006

Kentfield Relief Sewer

PROJECT ID……………………………………	**16**
LOCATION………………………………………	Stadium Way (crossing College of Marin) from College Ave. to east side of Corte Madera Creek
BRIEF PROJECT DESCRIPTION………………	New 21" relief sewer.
PROJECT JUSTIFICATION……………………	Relieve predicted existing capacity deficiencies (K100.25 to K000.03)
SPECIAL CONSIDERATIONS…………………	School courtyard crossing, creek crossing
ASSUMPTIONS…………………………………	Parallel existing trunk sewer from K100.25 to K000.03. 1/2 microtunnel (trough school area) 1/2 open cut (through ball field) New siphon across Corte Madera Creek
ALTERNATIVES………………………………	

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
New 21" relief sewer through school area (microtunnel)	21"		450'	605 $/ft	$272,250
New 21" relief sewer through ball field (open cut)	21"		450'	306 $/ft	$137,700
New single barrel siphon under Creek (microtunnel)	21		150'	605 $/ft	$90,750
Jacking Pits			1 pits	50,000 $/ea	$50,000
Receiving Pits			1 pits	35,000 $/ea	$35,000
Subtotal					$585,700
Mobilization and Demobilization				5%	$29,285
Construction Cost Subtotal					**$614,985**
Contingencies for Unknown Conditions				30%	$184,496
Construction Cost Total					**$799,481**
Engineering, Administration, and Legal Costs				25%	$199,870
Capital Improvement Cost Total					**$999,351**
				rounded	**$999,000**

Laurel Grove/McAllister

PROJECT ID……………………………………… 17

LOCATION………………………………………… Laurel Grove and McAllister from Cypress to Berens

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewers

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Laurel Grove and McAllister (K200.11 to K100.04)

SPECIAL CONSIDERATIONS………………… SFD

ASSUMPTIONS…………………………………… Pipe bursting

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 8" sewer on Laurel Grove, MHs K200.11 to K200.09	10"		377'	157 $/ft	$59,126
Pipeburst existing 10" sewer on Laurel Grove, MHs K200.09 to K200.06	12"		550'	181 $/ft	$99,550
Pipeburst existing 12" sewer on SFD, MHs K200.06 to K200.05	15"		146'	218 $/ft	$31,828
Pipeburst existing 12" sewer on McAllister, MHs K200.05 to K100.04	15"		1,183'	218 $/ft	$257,894
Lower lateral replacement			43	2,500 $/ea	$107,500
Subtotal					$555,898
Mobilization and Demobilization				5%	$27,795
Construction Cost Subtotal					**$583,693**
Contingencies for Unknown Conditions				30%	$175,108
Construction Cost Total					**$758,801**
Engineering, Administration, and Legal Costs				25%	$189,700
Capital Improvement Cost Total					**$948,501**
				rounded	**$949,000**

Manor Easement

PROJECT ID……………………………………… **18**

LOCATION………………………………………… Easement south of Manor and SFD.

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewers

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in Manor easement sewer (G300.03 to G000.15)

SPECIAL CONSIDERATIONS………………… SFD

ASSUMPTIONS…………………………………… Pipe bursting

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 12" sewer in easement, MHs G300.03 to G000.15	15"		864'	218 $/ft	$188,265
Lower lateral replacement			4	2,500 $/ea	$10,000
Subtotal					$198,265
Mobilization and Demobilization				5%	$9,913
Construction Cost Subtotal					**$208,178**
Contingencies for Unknown Conditions				30%	$62,453
Construction Cost Total					**$270,631**
Engineering, Administration, and Legal Costs				25%	$67,658
Capital Improvement Cost Total					**$338,289**
				rounded	**$338,000**

William/Holcomb/Meadowood

PROJECT ID…………………………………… **19**

LOCATION………………………………………… William Ave., Holcomb Ave. from Magnolia to Ward; Meadowood Dr. east of ward and easement east of Meadowood.

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewers, 2 new relief sewers.

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies in William, Monte Vista, Holcomb, and Meadowood (L220.02 to L165.03 to L164.06 to L163.03 to L151.04.

SPECIAL CONSIDERATIONS…………………… Holcomb is old RR easement

ASSUMPTIONS……………………………………

ALTERNATIVES…………………………………… Pipeburst existing 15" sewer on Meadowbrook and easement, MHs L164.01 to L151.04 - two diameters.

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 10" sewer on William, MHs L220.02 to L165.03	15"		516'	218 $/ft	$112,510
New 12" relief sewer on William from MHs L165.03 to L164.10	12"		334'	209 $/ft	$69,722
Pipeburst existing 10" sewer on Holcomb, MHs L164.10 to L164.07	12"		643'	181 $/ft	$116,293
New 12" relief sewer on Holcomb (easement) from MHs L164.07 to L164.03	12"		153'	209 $/ft	$31,935
Pipeburst existing 10" sewer in easement, MHs L164.03 to L164.01	15"		300'	218 $/ft	$65,291
Replace existing 15" sewer on Meadowbrook and easement, MHs L164.01 to L151.04	21"		1,079'	260 $/ft	$280,514
Lower lateral replacement			35	2,500 $/ea	$87,500
Subtotal					$763,765
Mobilization and Demobilization				5%	$38,188
Construction Cost Subtotal					**$801,953**
Contingencies for Unknown Conditions				30%	$240,586
Construction Cost Total					**$1,042,539**
Engineering, Administration, and Legal Costs				25%	$260,635
Capital Improvement Cost Total					**$1,303,174**
				rounded	**$1,303,000**

4/24/2006

Magnolia

PROJECT ID……………………………………… 20

LOCATION………………………………………… Magnolia from Francos to Murray, and from north Bon Air to Curst Creek Ln.

BRIEF PROJECT DESCRIPTION……………… Upsize existing sewers

PROJECT JUSTIFICATION…………………… Relieve predicted existing capacity deficiencies (L152.26 to L152.25, and L152.18 to L152.11)

SPECIAL CONSIDERATIONS………………… Heavy traffic on Magnolia

ASSUMPTIONS…………………………………

ALTERNATIVES…………………………………

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 6" sewer on Magnolia, MHs L152.26 to L152.25	10"		290'	157 $/ft	$45,499
Pipeburst existing 12" sewer on Magnolia, MHs L152.18 to L152.11	15"		1,981'	218 $/ft	$431,771
Lower lateral replacement			5	2,500 $/ea	$12,500
Subtotal					$489,769
Mobilization and Demobilization				5%	$24,488
Construction Cost Subtotal					**$514,258**
Contingencies for Unknown Conditions				30%	$154,277
Construction Cost Total					**$668,535**
Engineering, Administration, and Legal Costs				25%	$167,134
Capital Improvement Cost Total					**$835,669**
				rounded	**$836,000**

Eliseo

PROJECT ID.. **21**

LOCATION... Eliseo, north of Corte Cayuga

BRIEF PROJECT DESCRIPTION.................. Upsize existing sewer

PROJECT JUSTIFICATION....................... Relieve predicted existing capacity deficiency (B200.05 to B200.04)

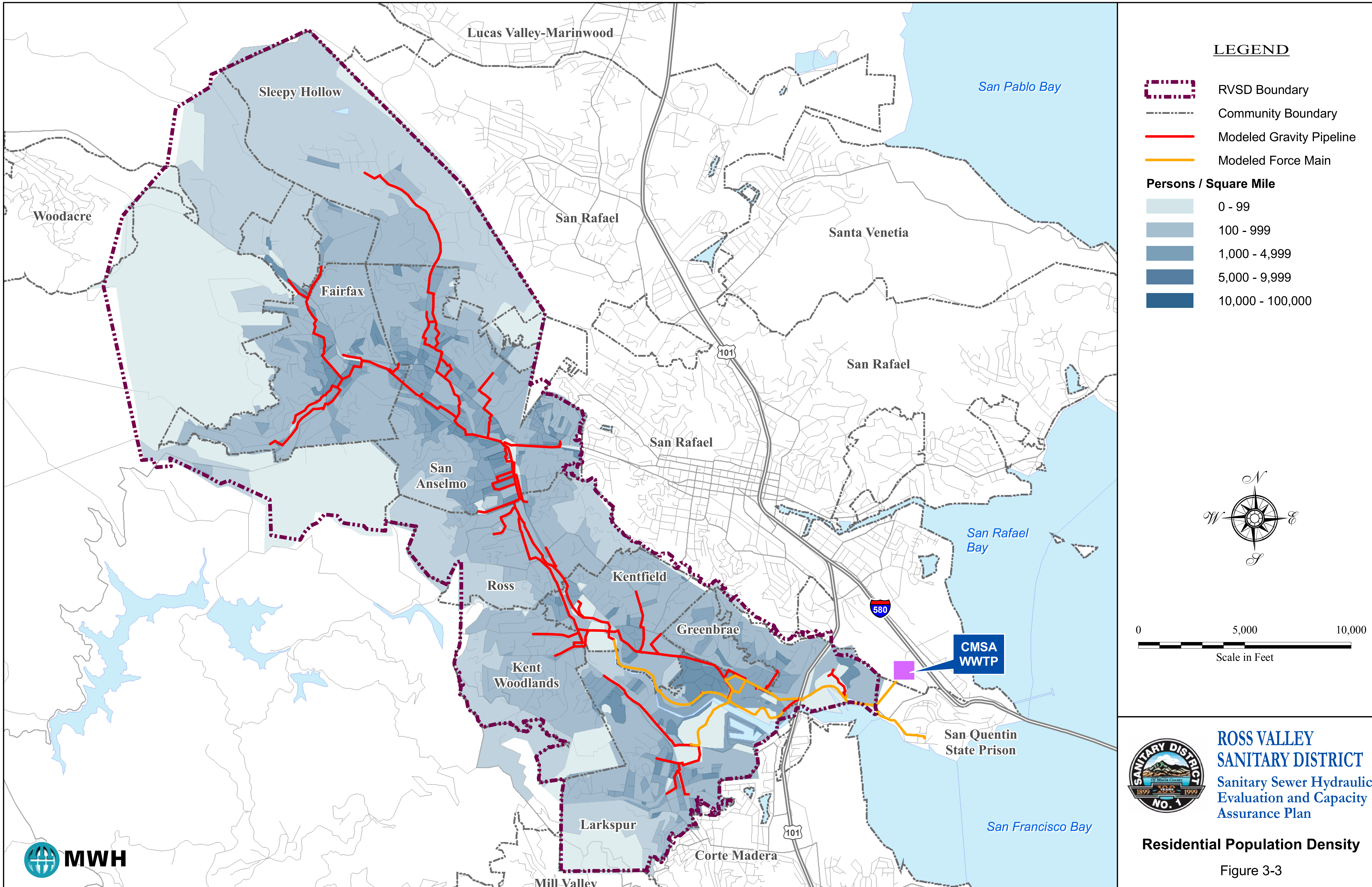
SPECIAL CONSIDERATIONS......................

ASSUMPTIONS.......................................

ALTERNATIVES.......................................

MAJOR ITEMS	DIA. (in.)	DEPTH (feet)	QUANTITY	UNIT COST	COST
Baseline Pipe Construction Cost					
Pipeburst existing 6" sewer on Eliseo, MHs B200.05 to B200.04	8"		218'	132 $/ft	$28,763
Lower lateral replacement			4	2,500 $/ea	$10,000
Subtotal					$38,763
Mobilization and Demobilization				5%	$1,938
Construction Cost Subtotal					**$40,701**
Contingencies for Unknown Conditions				30%	$12,210
Construction Cost Total					**$52,911**
Engineering, Administration, and Legal Costs				25%	$13,228
Capital Improvement Cost Total					**$66,139**
				rounded	**$66,000**

4/24/2006

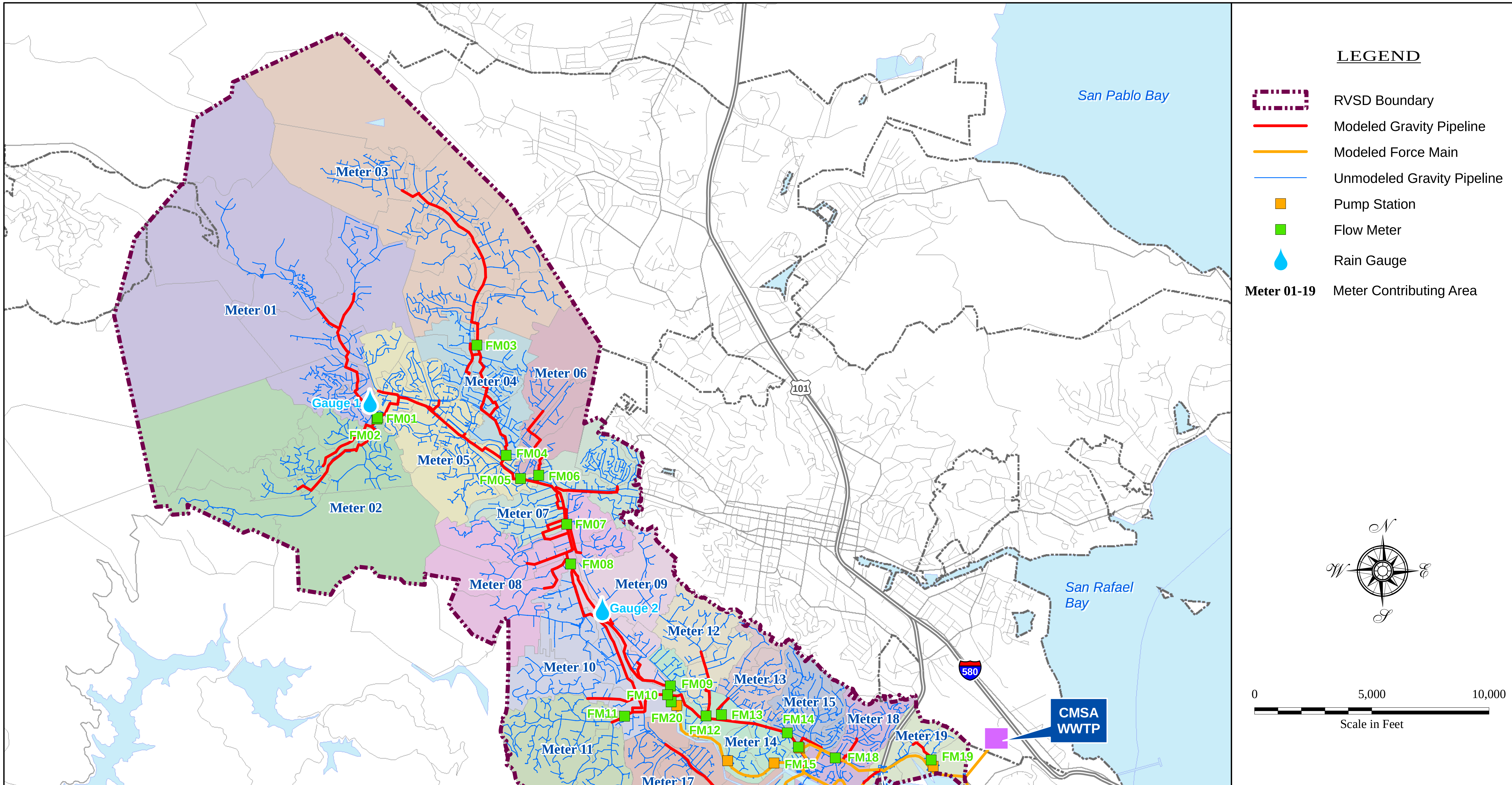


ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Residential Population Density

Figure 3-3

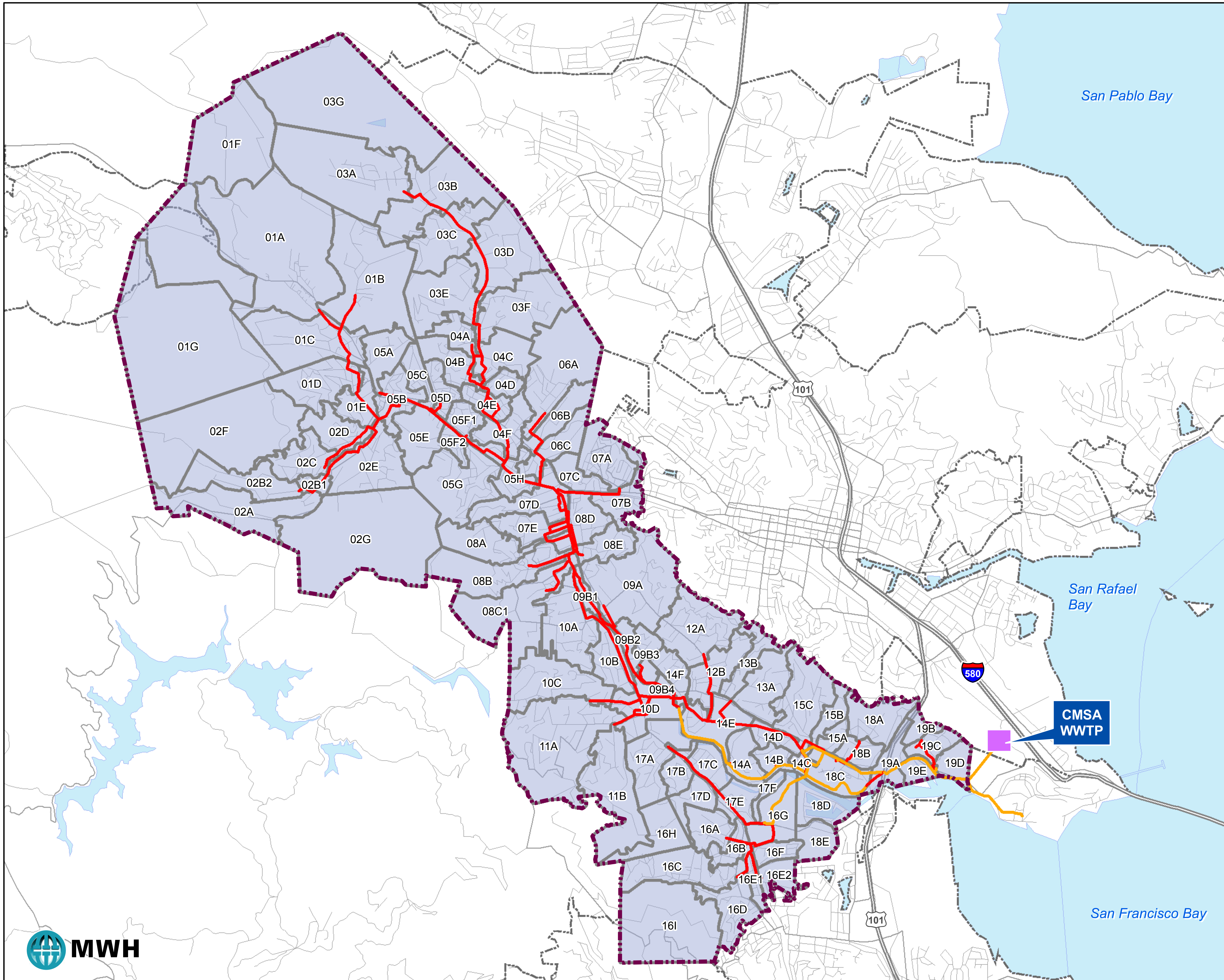


ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Flow Monitoring Locations and Meter Contributing Areas

Figure 3-1



LEGEND

- RVSD Boundary
- Modeled Gravity Pipeline
- Modeled Force Main
- Subbasin

0 5,000 10,000
Scale in Feet

N W E S

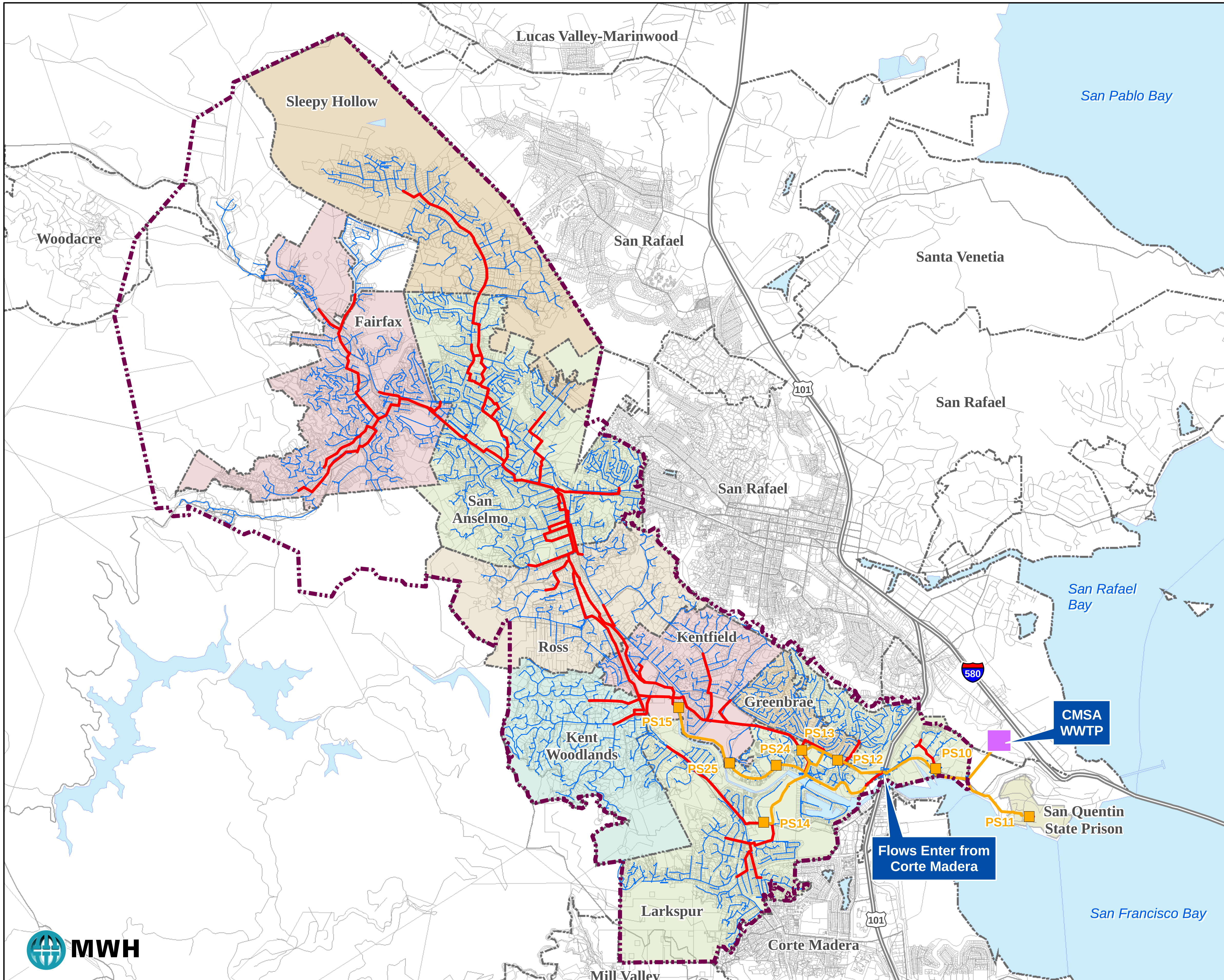
ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

SANITARY DISTRICT NO. 1 Of Marin County 1899 1999

Model Subbasins

Figure 2-1



LEGEND

- RVSD Boundary
- Community Boundary
- Modeled Gravity Pipeline
- Modeled Force Main
- Unmodeled Gravity Pipeline
- Pump Station

N W E S

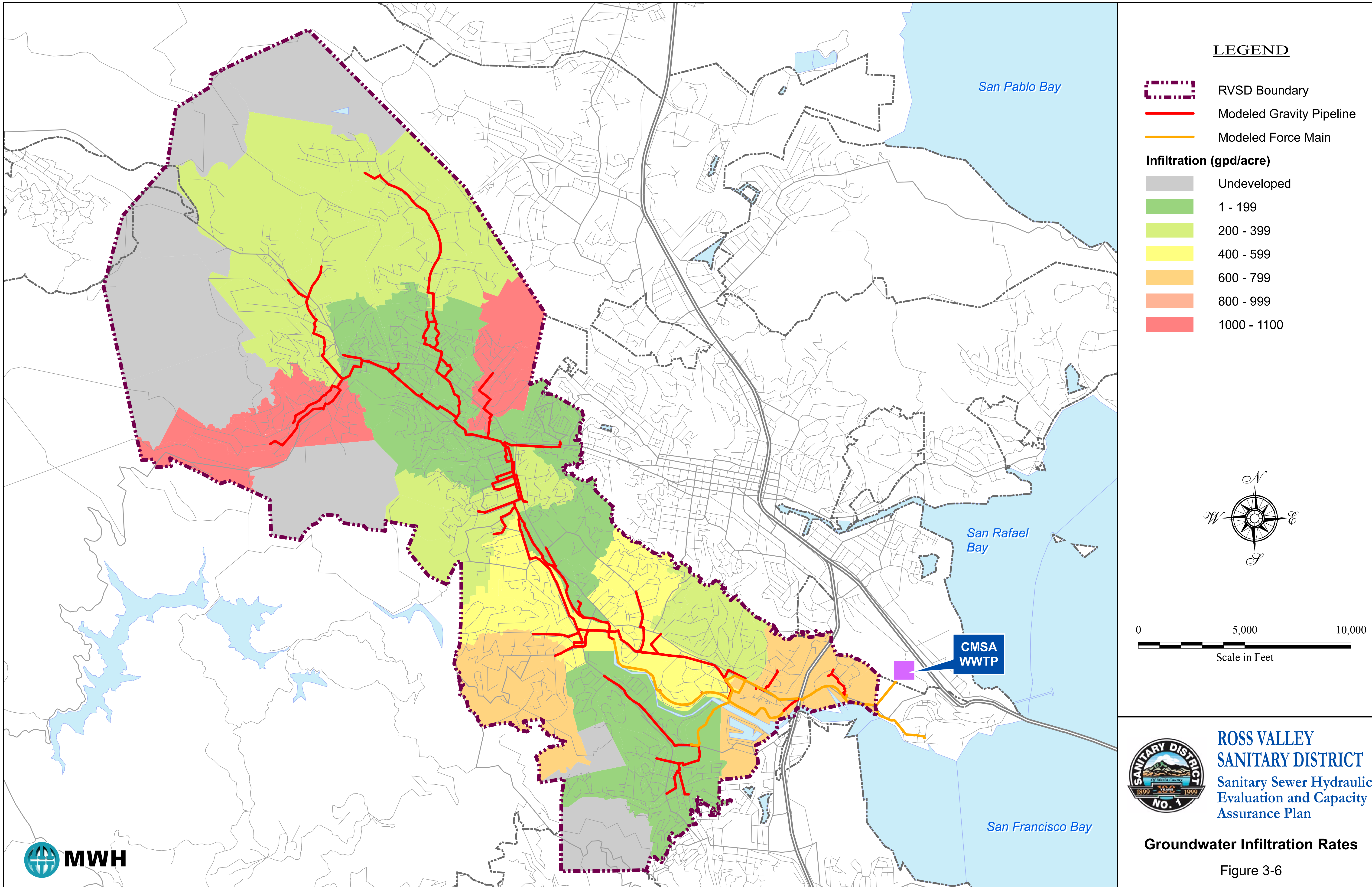
0 5,000 10,000
Scale in Feet

ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Existing Collection System

Figure 1-1

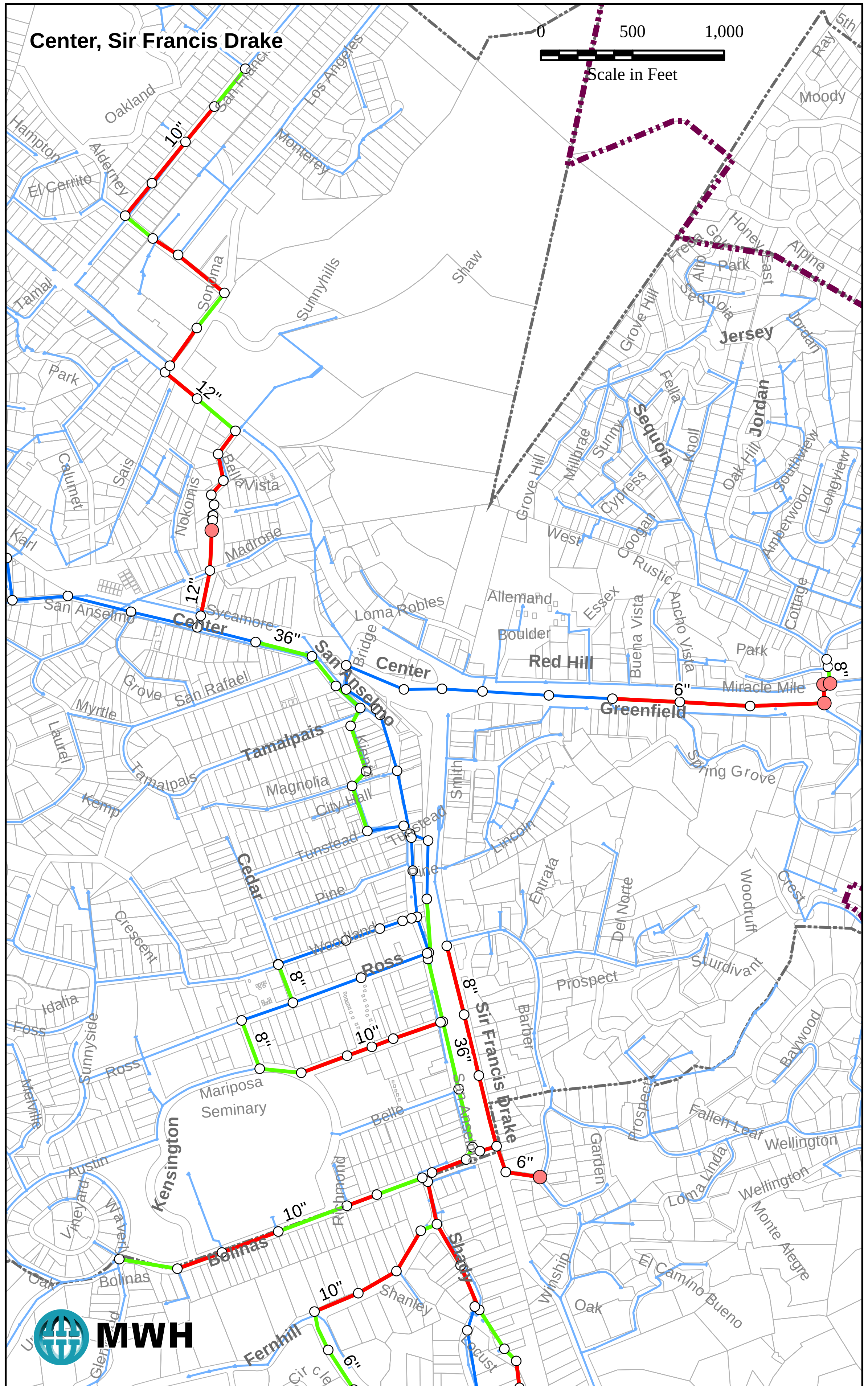


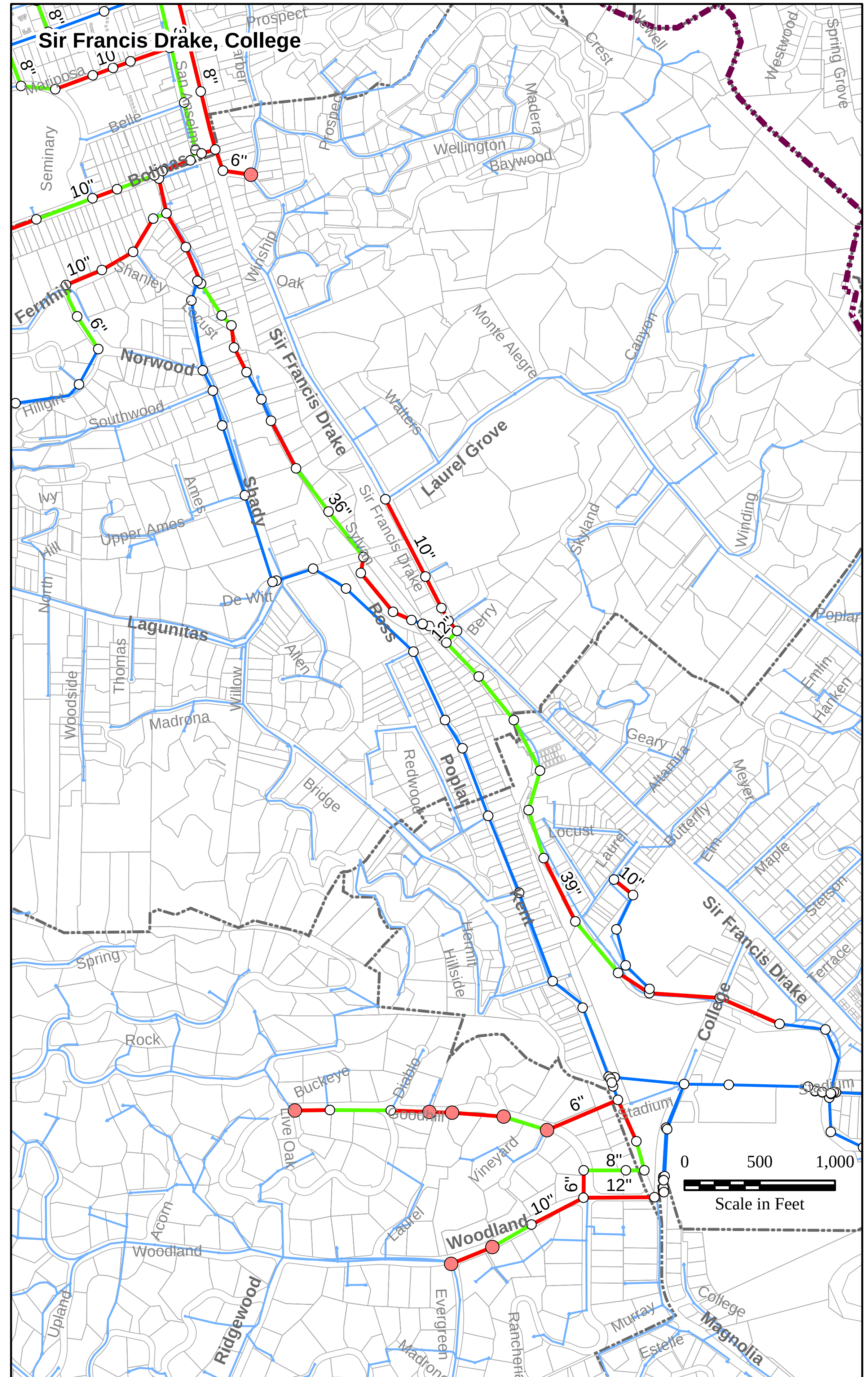
ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Groundwater Infiltration Rates

Figure 3-6

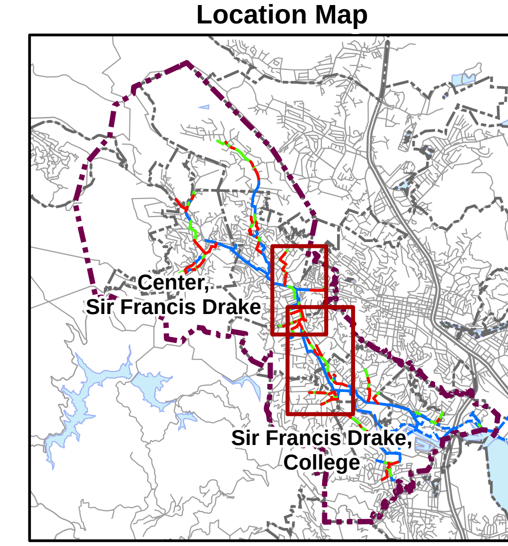




LEGEND

- Community Boundary
- RVSD Boundary
- Force Main
- Surcharge due to Capacity
- Surcharge due to Backup
- No Surcharge
- Unmodeled Gravity Pipeline
- Pump Station
- Manhole
- Approximate Location of Potential Manhole Overflow

Location Map



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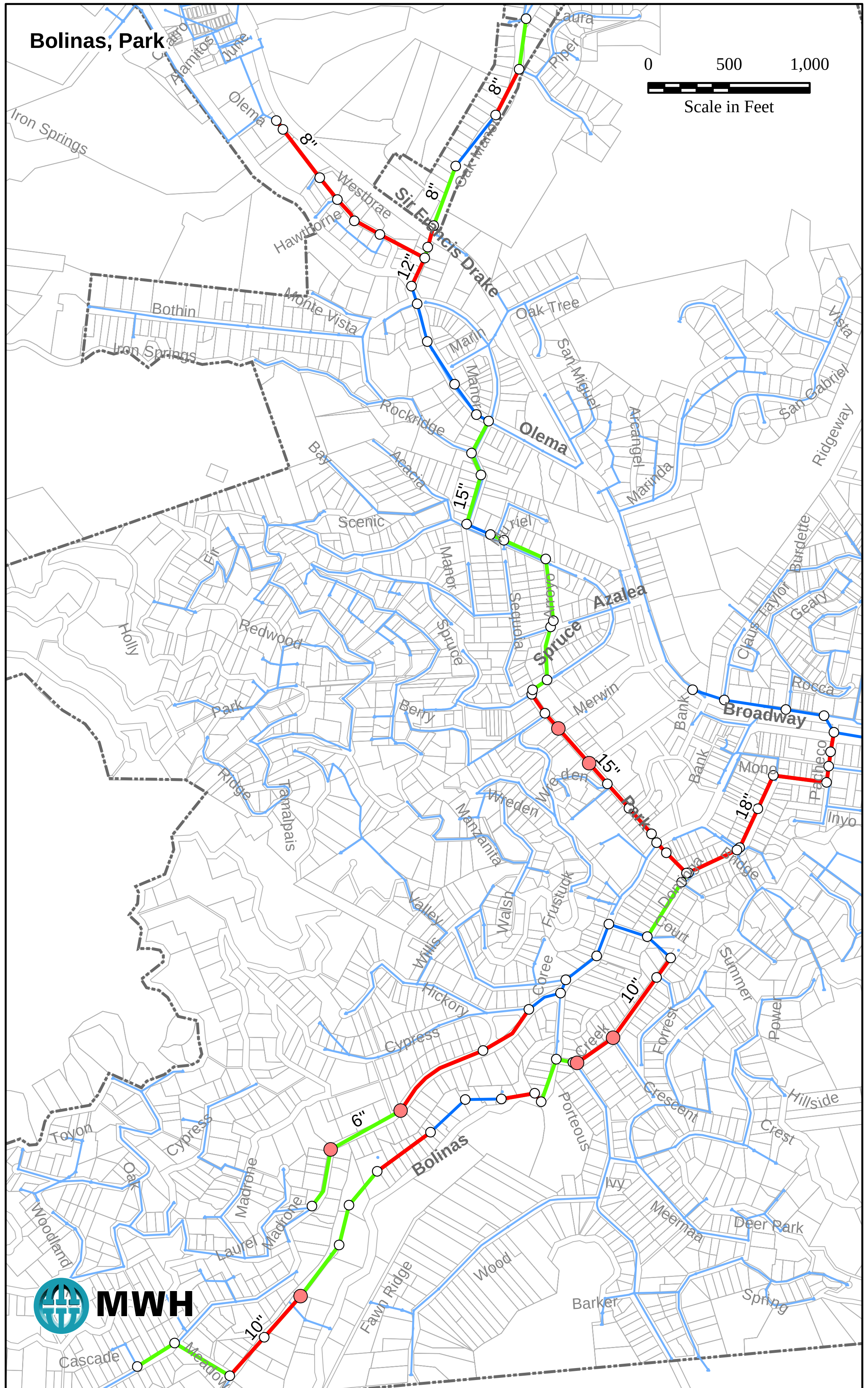
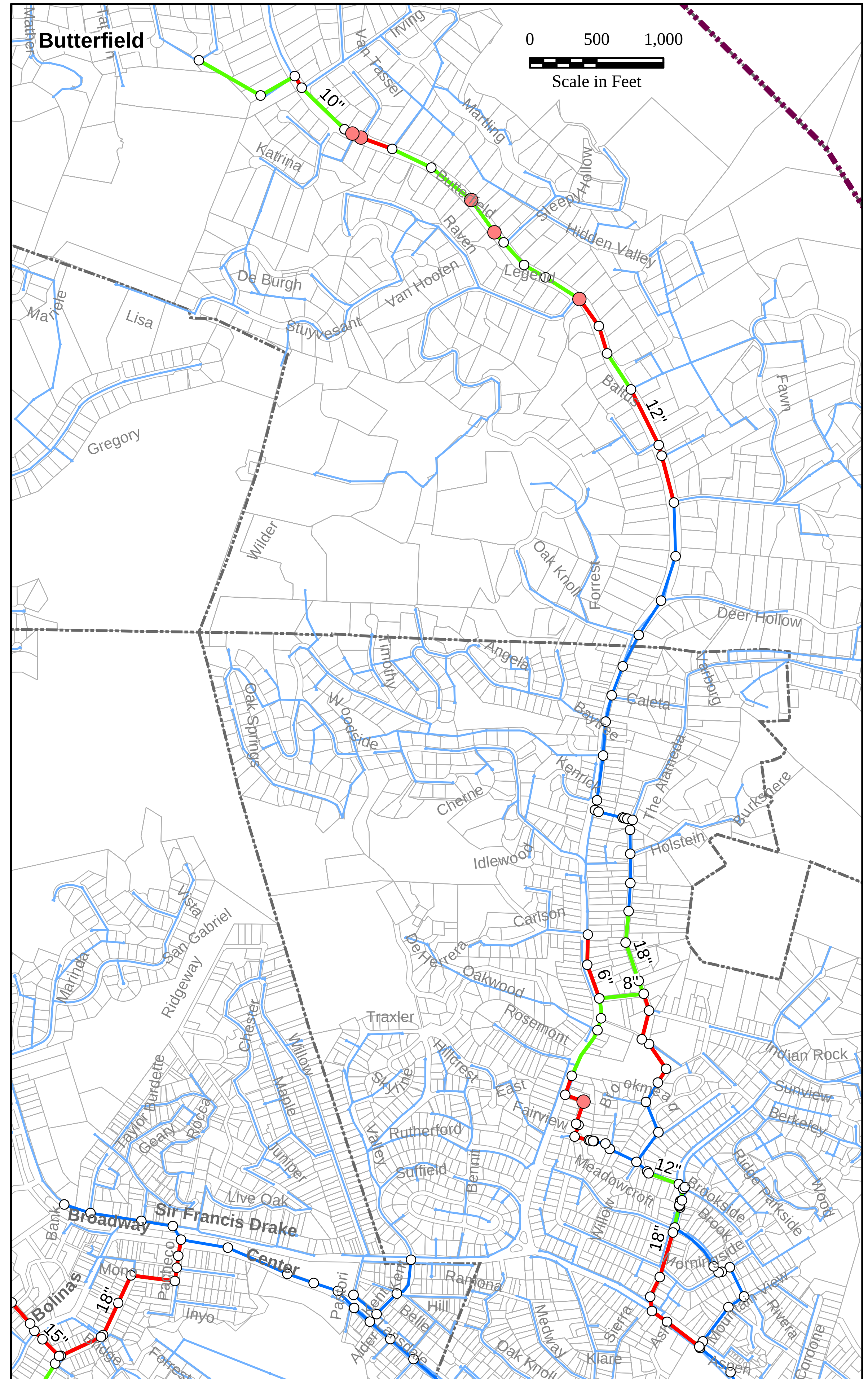
ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Existing Capacity Deficiencies Central Details

Figure 5-5

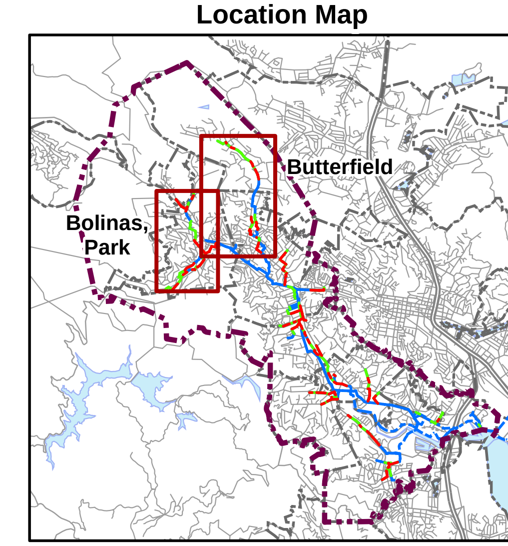




LEGEND

Community Boundary
RVSD Boundary
Force Main
Surcharge due to Capacity
Surcharge due to Backup
No Surcharge
Unmodeled Gravity Pipeline
Pump Station
Manhole
Approximate Location of Potential Manhole Overflow

Location Map



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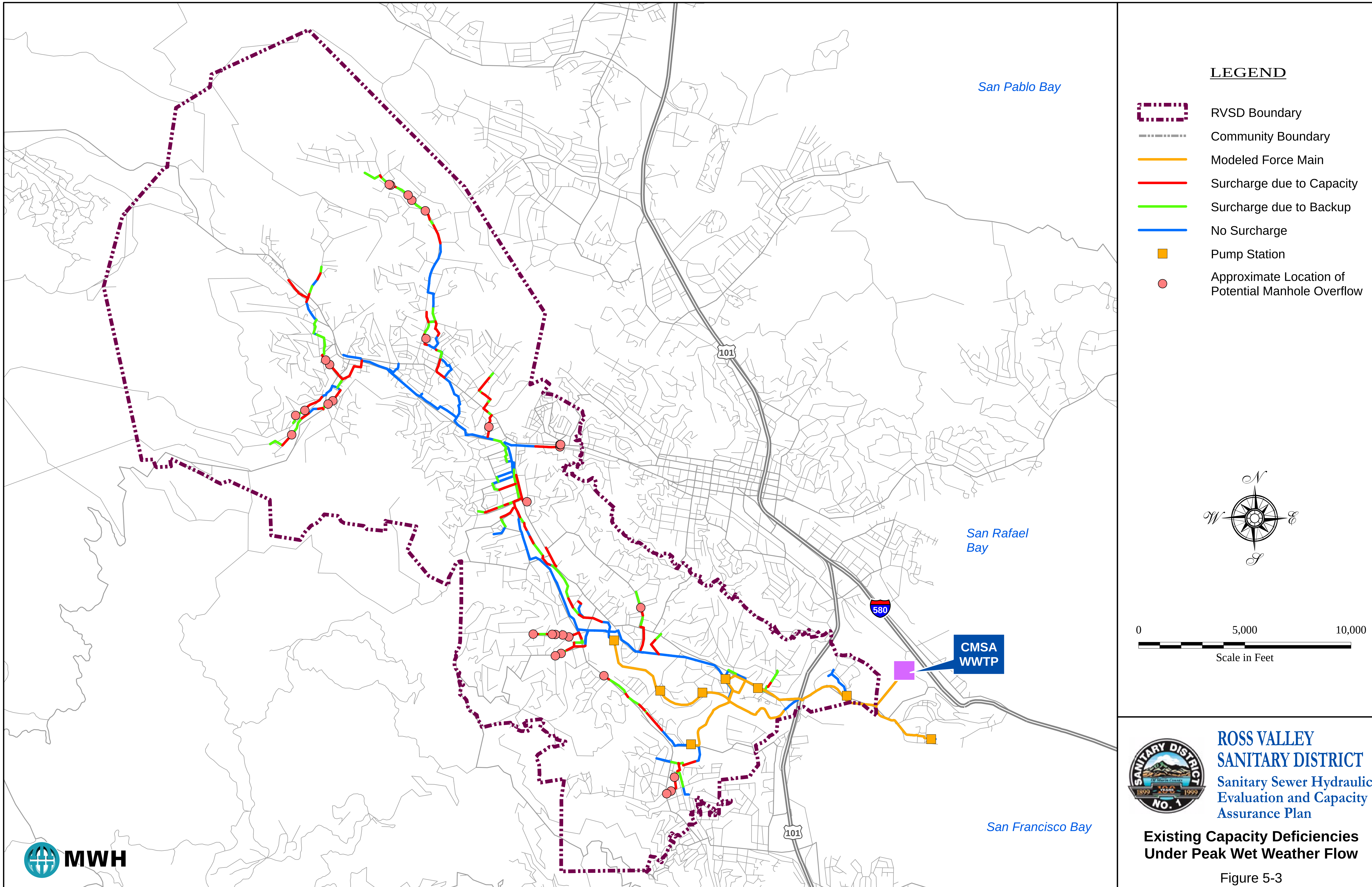
ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Existing Capacity Deficiencies North Details

Figure 5-4

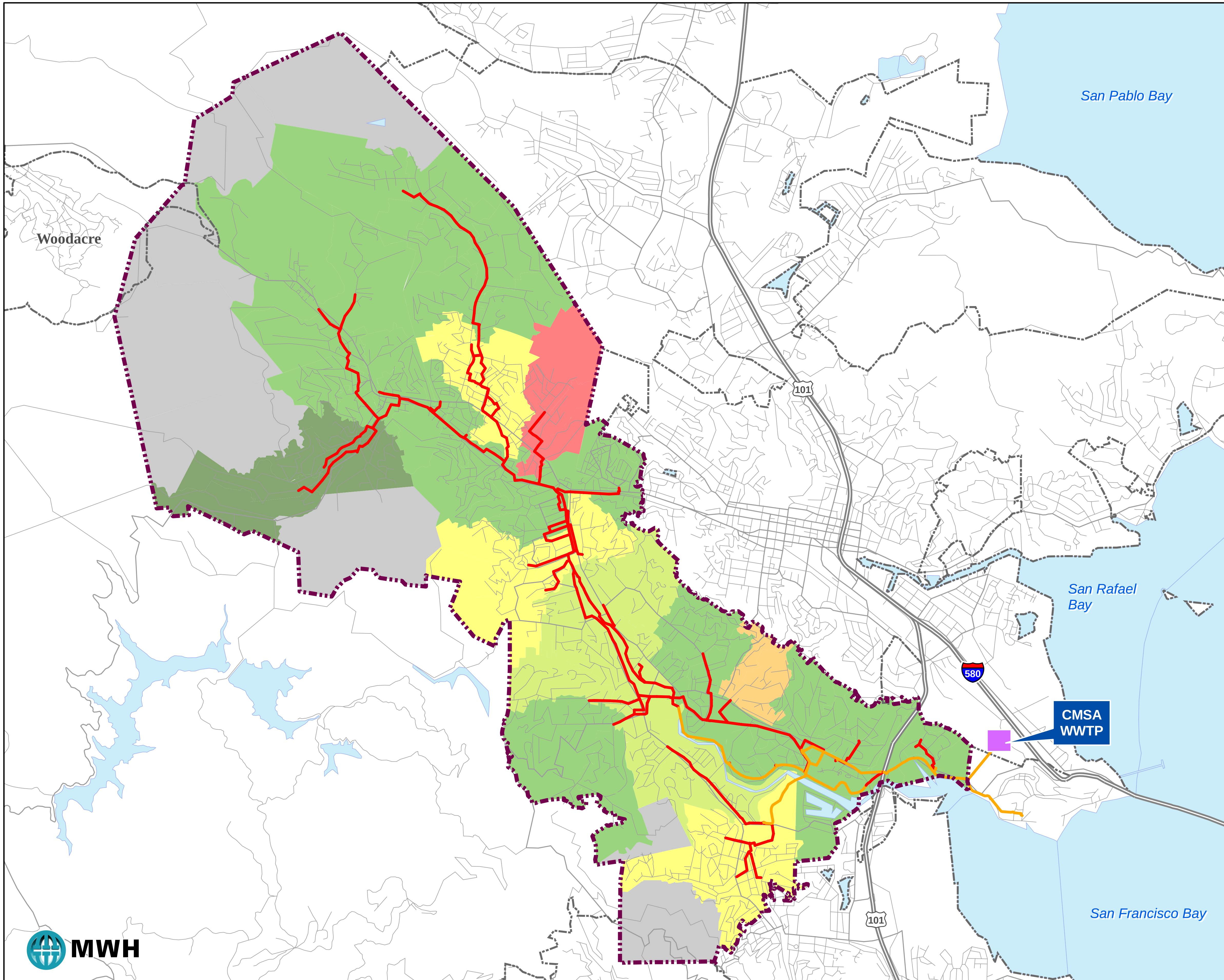


ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Existing Capacity Deficiencies Under Peak Wet Weather Flow

Figure 5-3

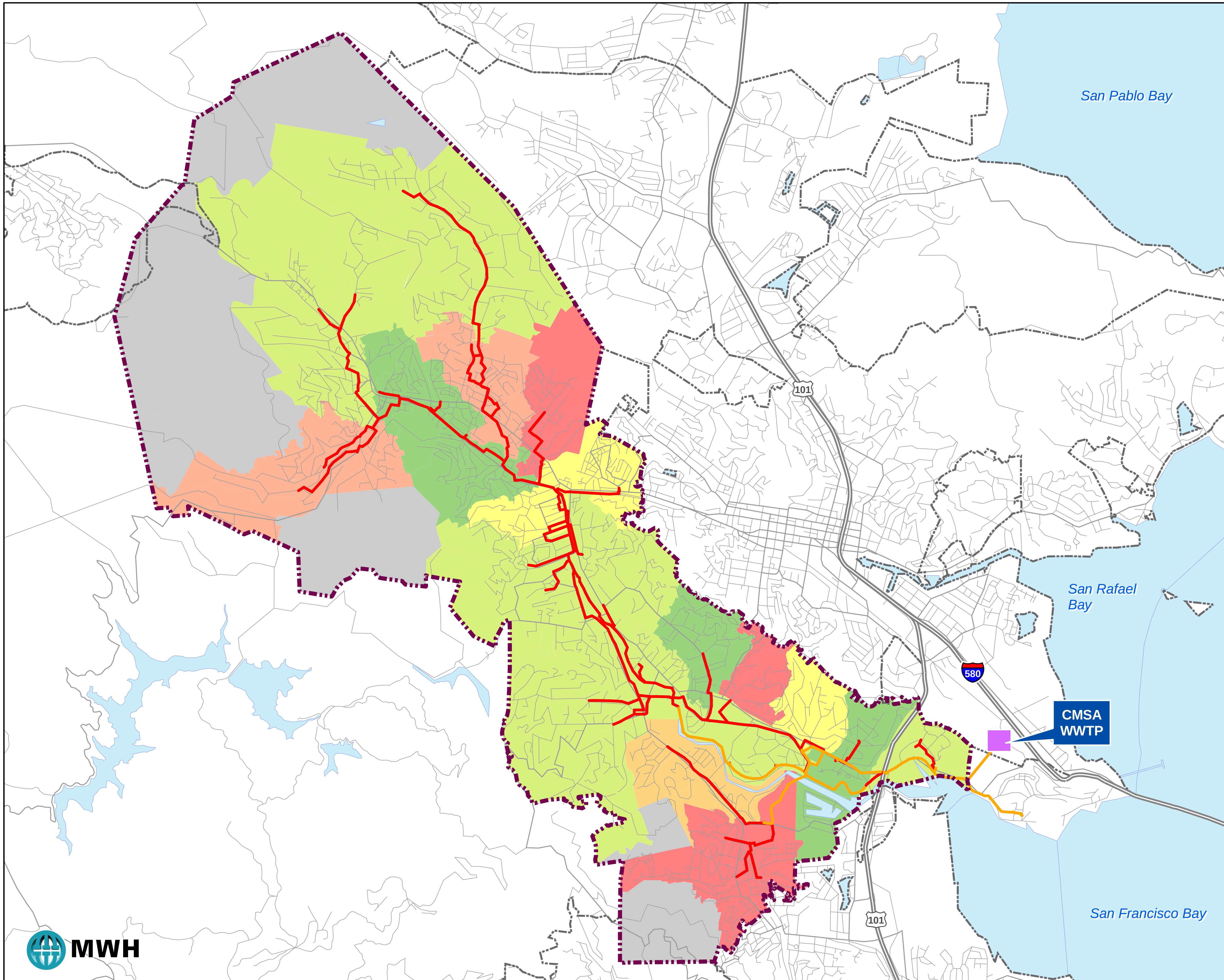


ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Fast and Medium RDI/I per Subbasin

Figure 3-10

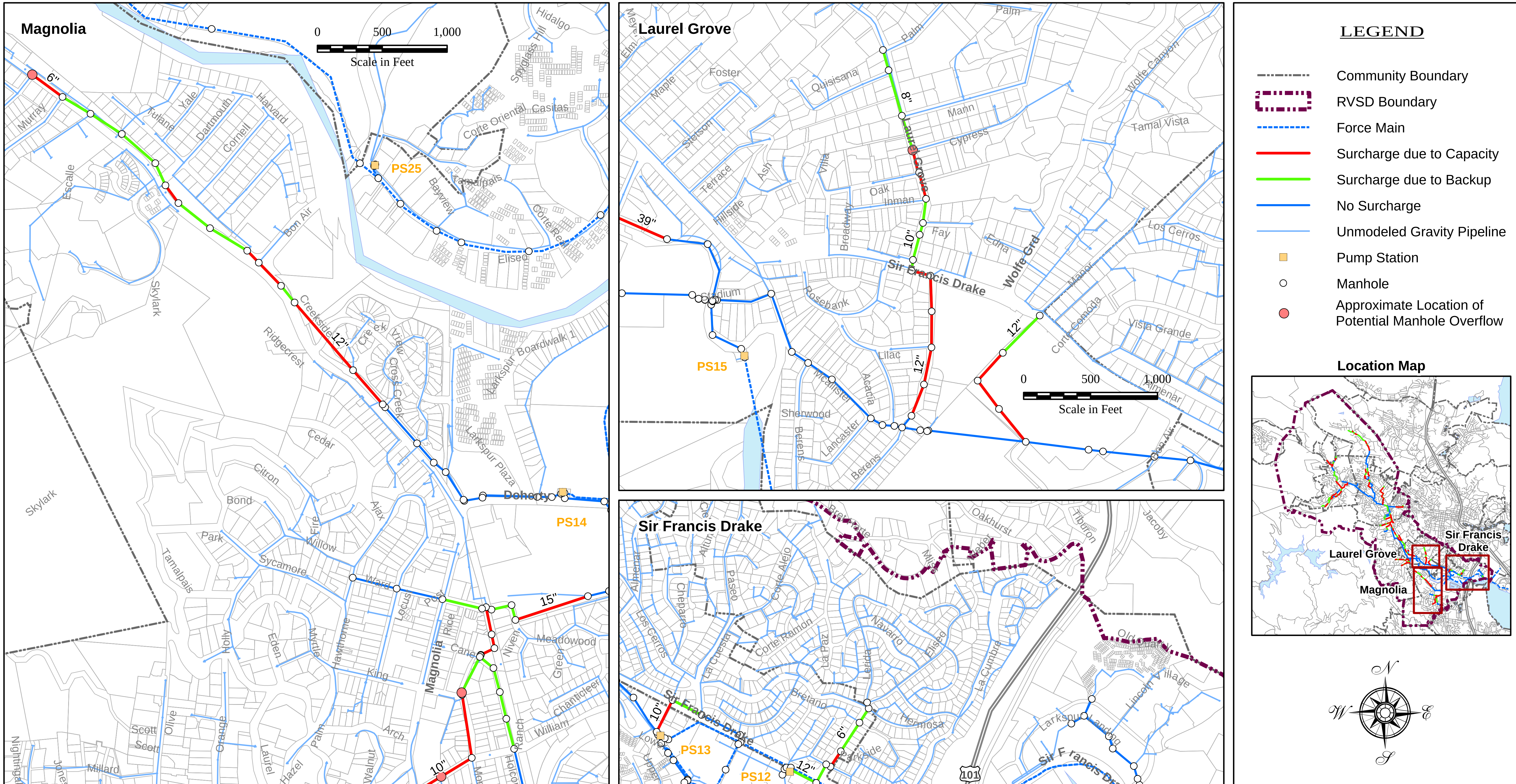


ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Total RDI/I per Subbasin

Figure 3-9

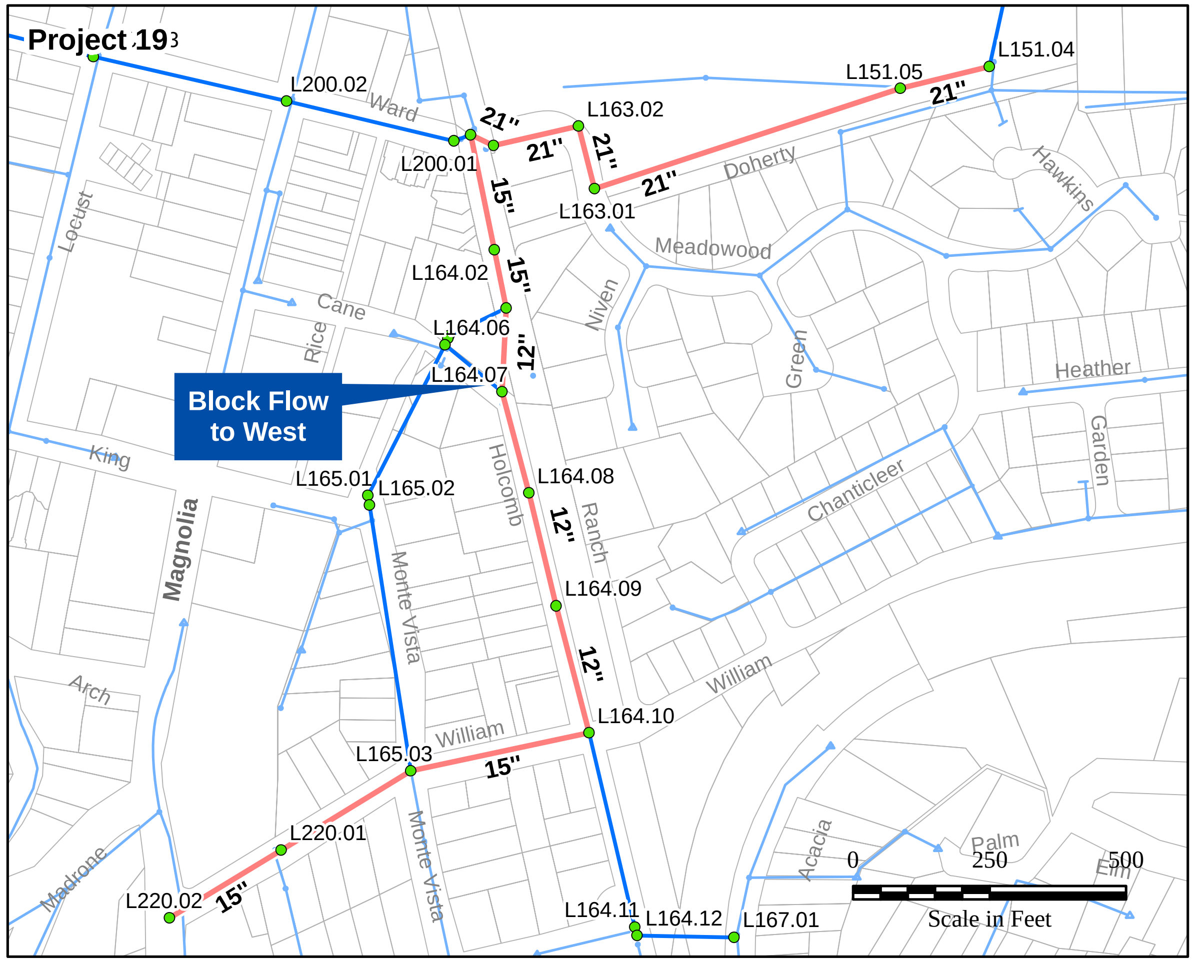


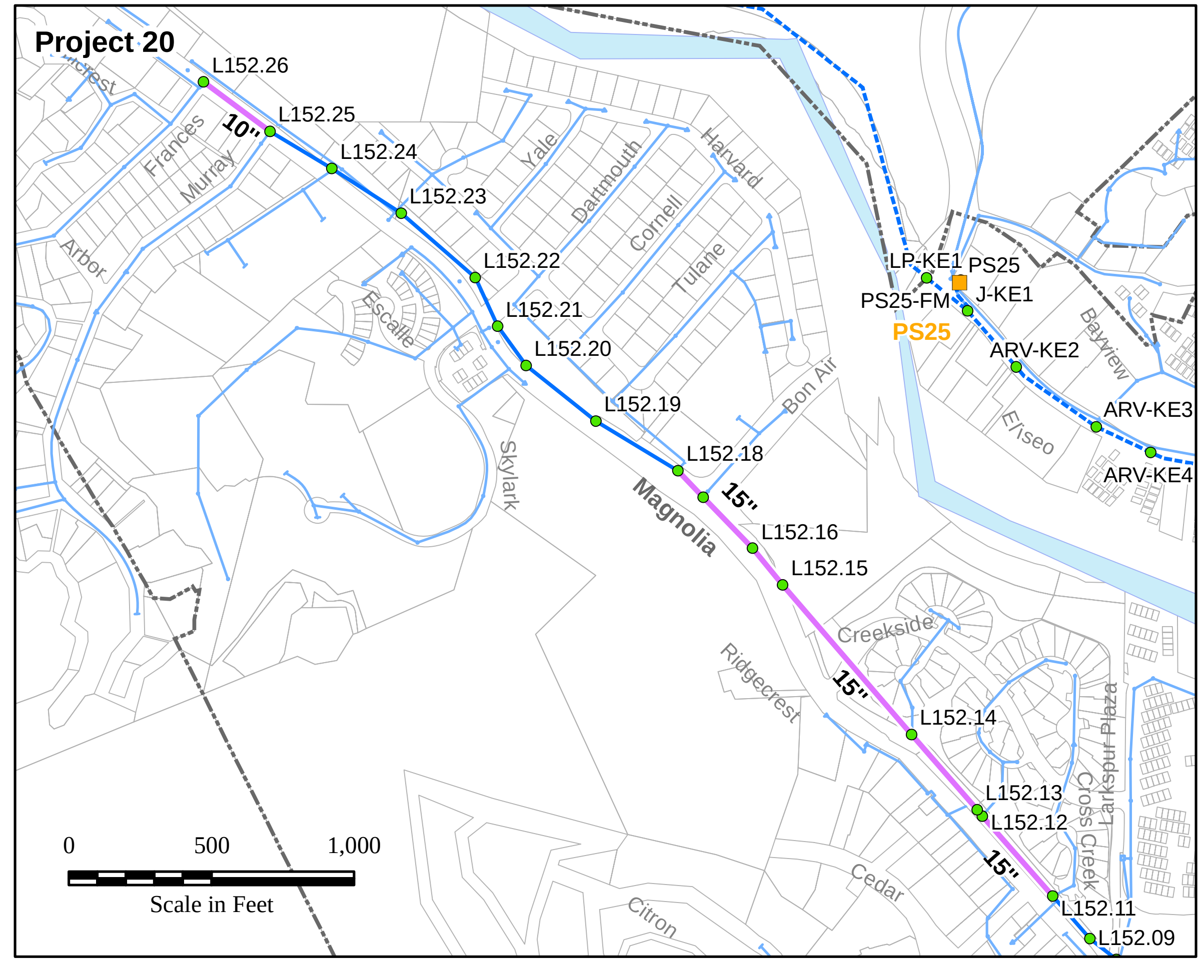
ROSS VALLEY SANITARY DISTRICT

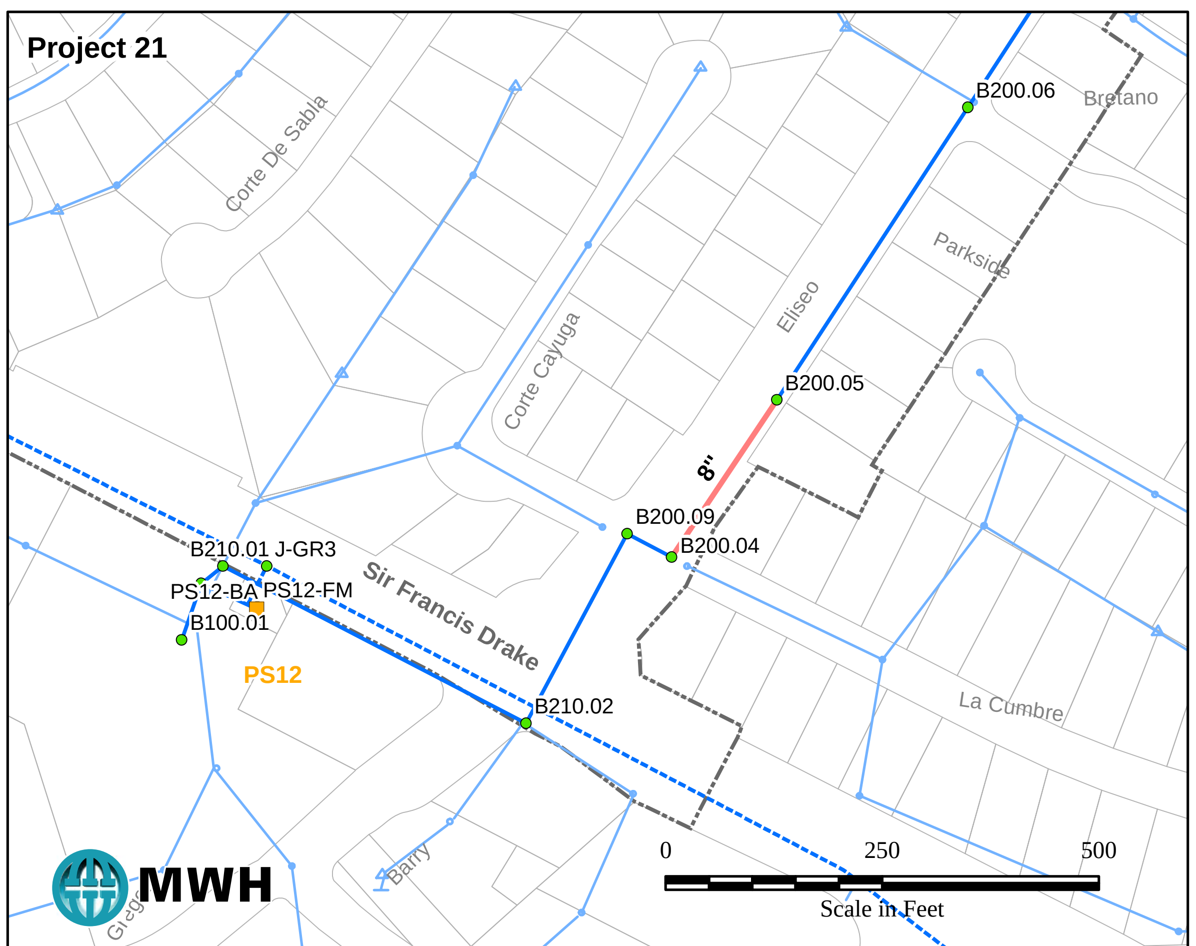
Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Existing Capacity Deficiencies South Details

Figure 5-6



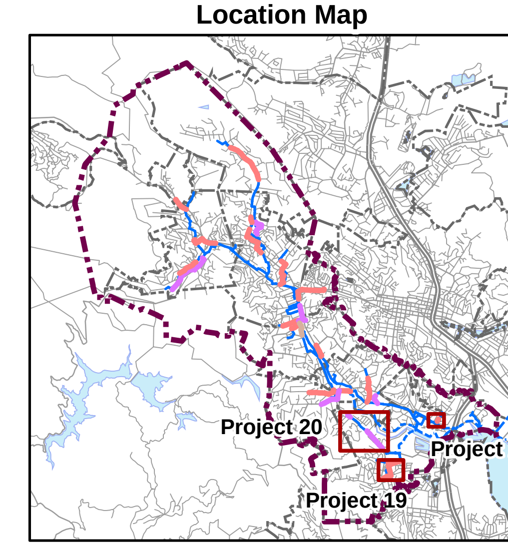




LEGEND

- RVSD Boundary
- Community Boundary
- Modeled Gravity Pipeline
- Modeled Force Main
- Manhole
- Pump Station
- Unmodeled Gravity Pipeline
- Improvement Project (Various Colors)

Location Map



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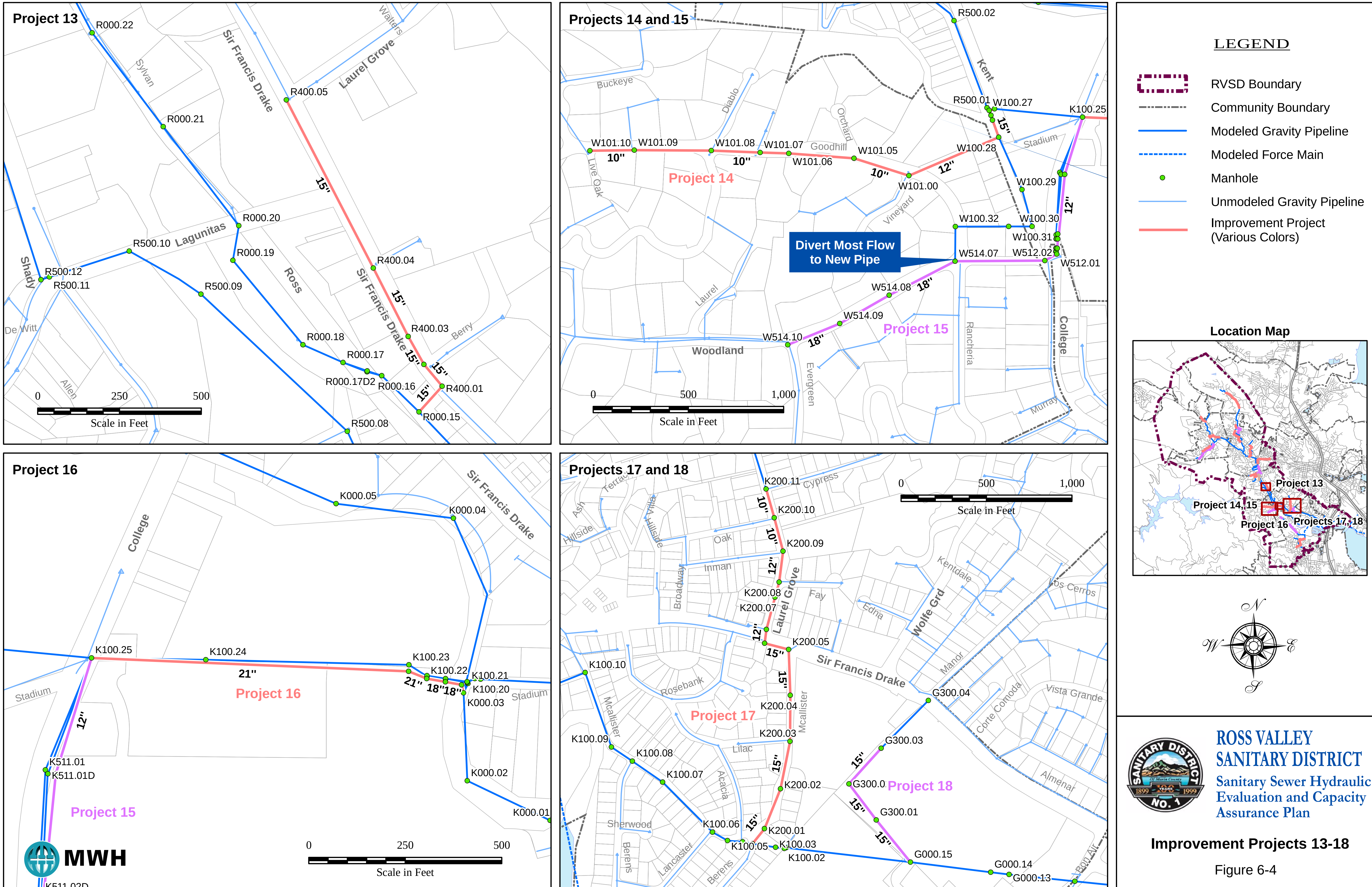
ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

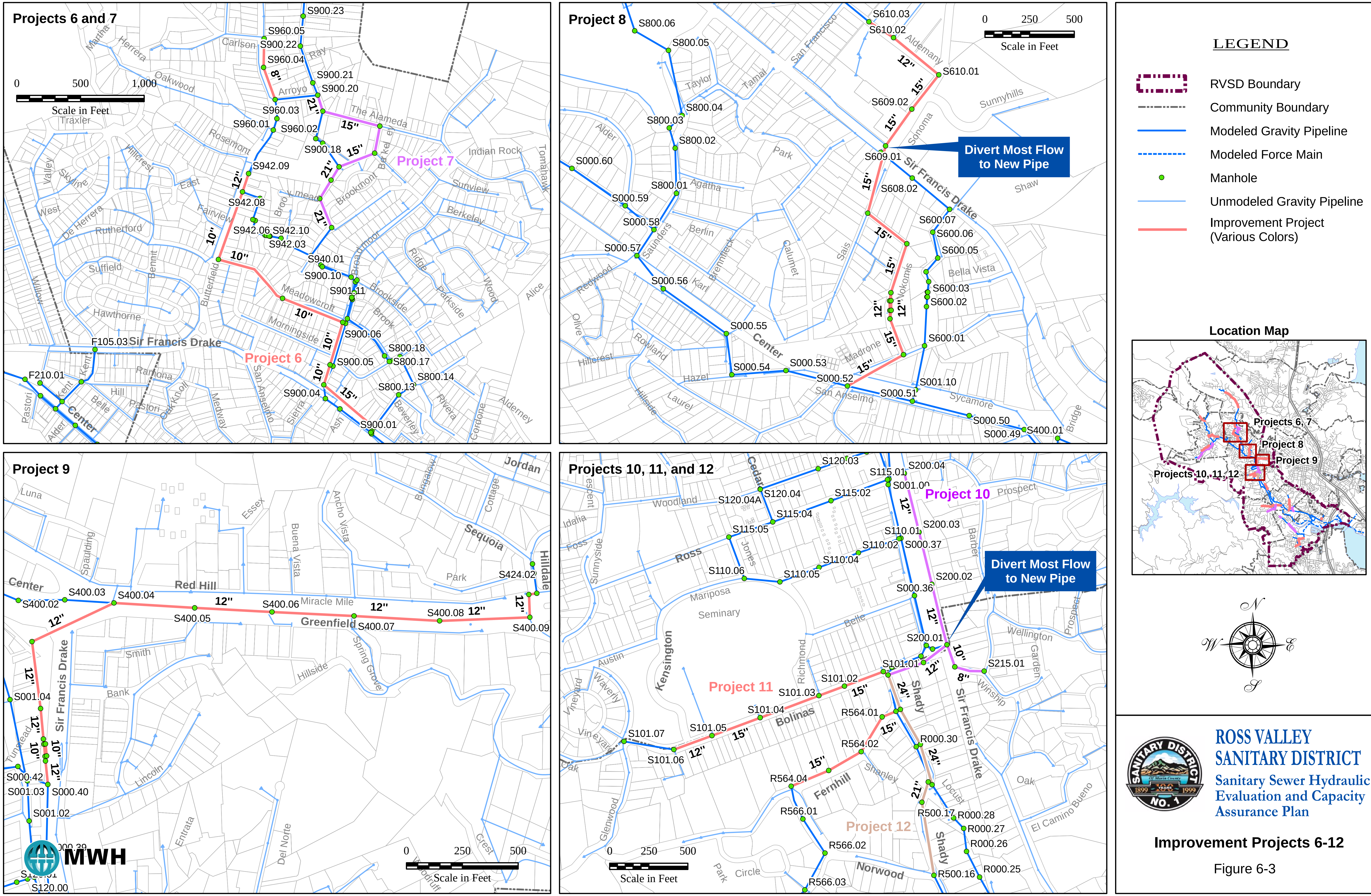
Improvement Projects 19-21

Figure 6-5

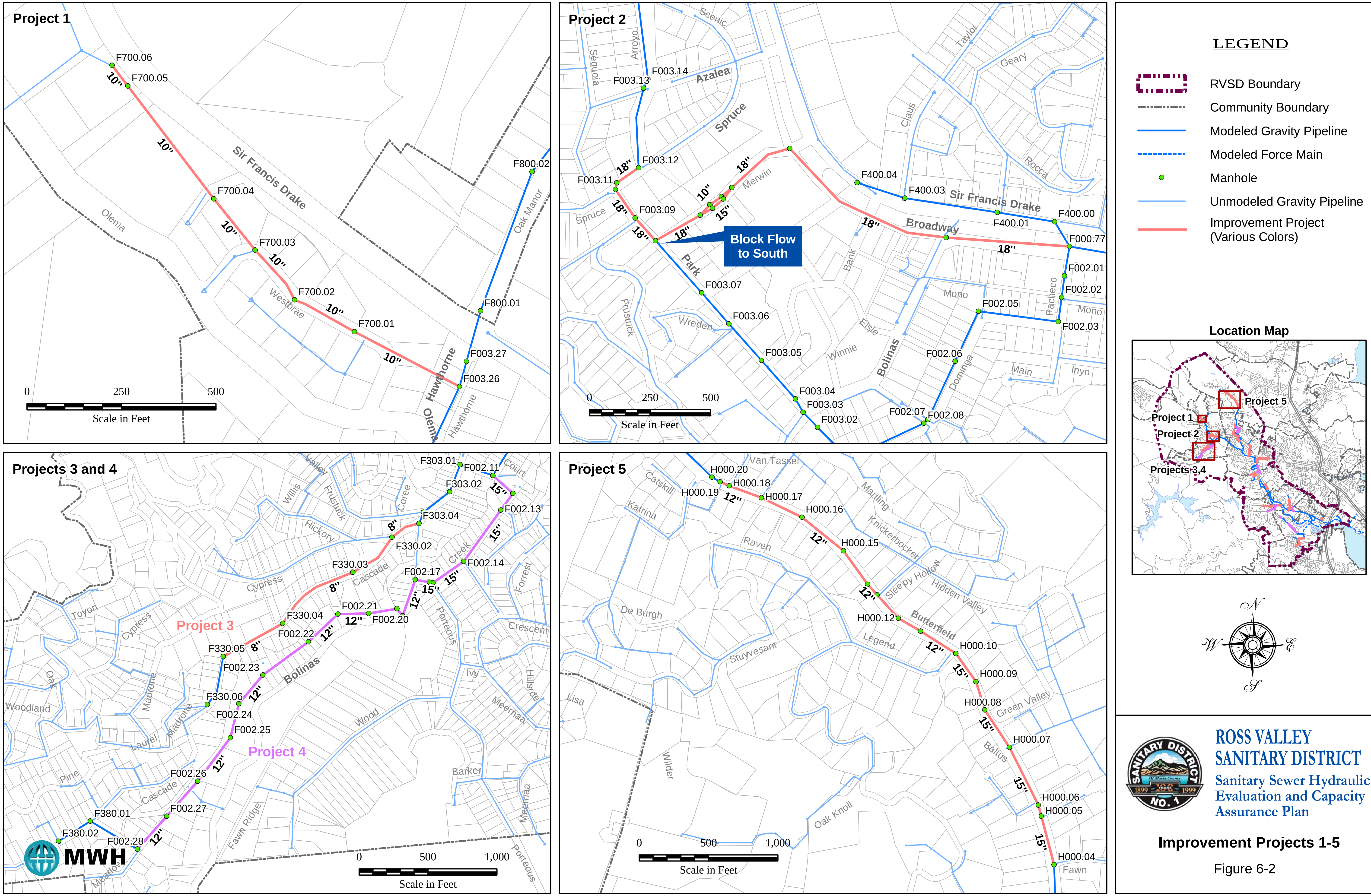


Improvement Projects 13-18

Figure 6-4

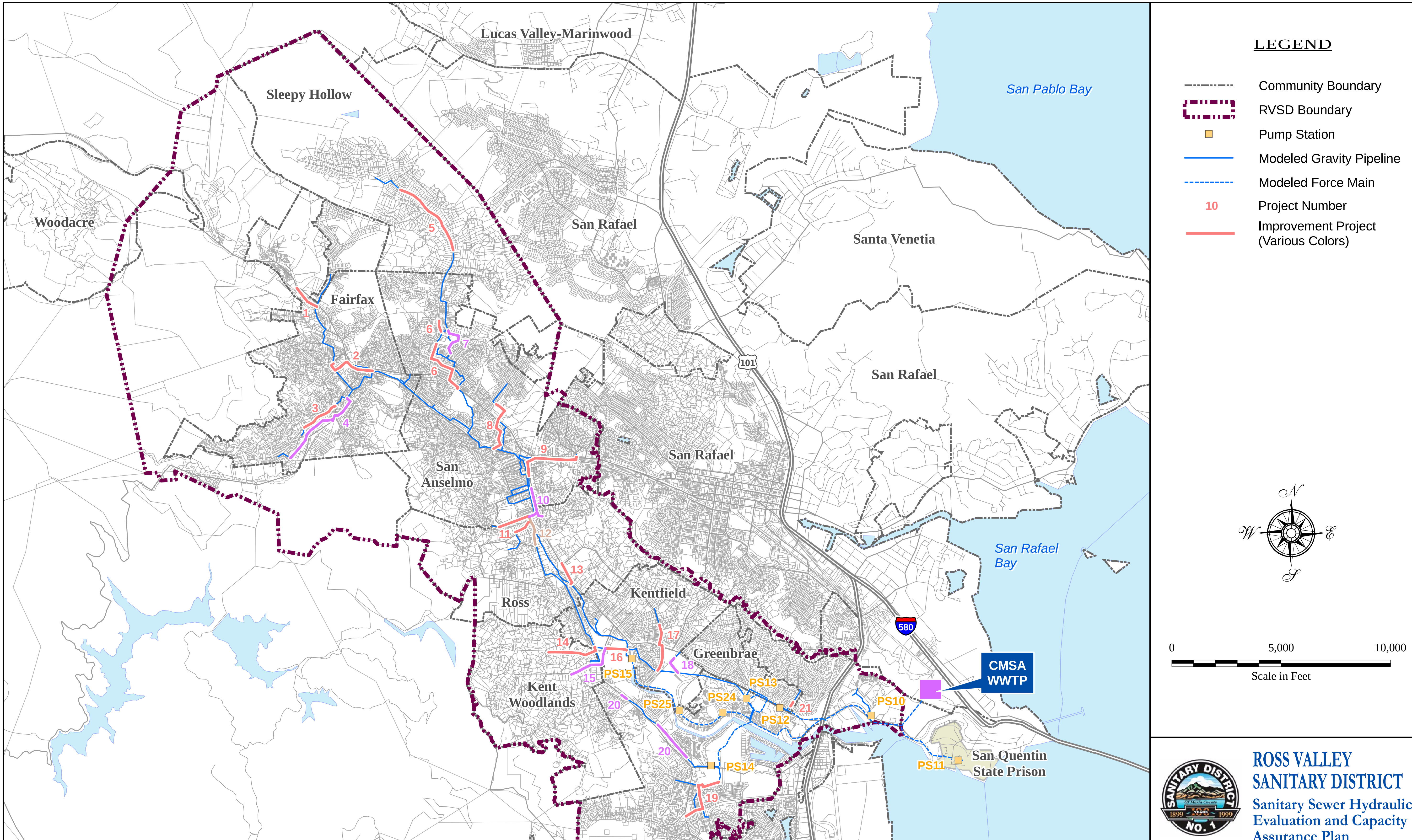


Projects 6 and 7
Project 6
Project 7
Scale in Feet
Project 8
Divert Most Flow to New Pipe
Project 9
Projects 10, 11, and 12
Project 10
Project 11
Project 12
Divert Most Flow to New Pipe
LEGEND
RVSD Boundary
Community Boundary
Modeled Gravity Pipeline
Modeled Force Main
Manhole
Unmodeled Gravity Pipeline
Improvement Project (Various Colors)
Location Map
Projects 6, 7
Project 8
Project 9
Projects 10, 11, 12
MWH
ROSS VALLEY SANITARY DISTRICT
Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan
Improvement Projects 6-12
Figure 6-3



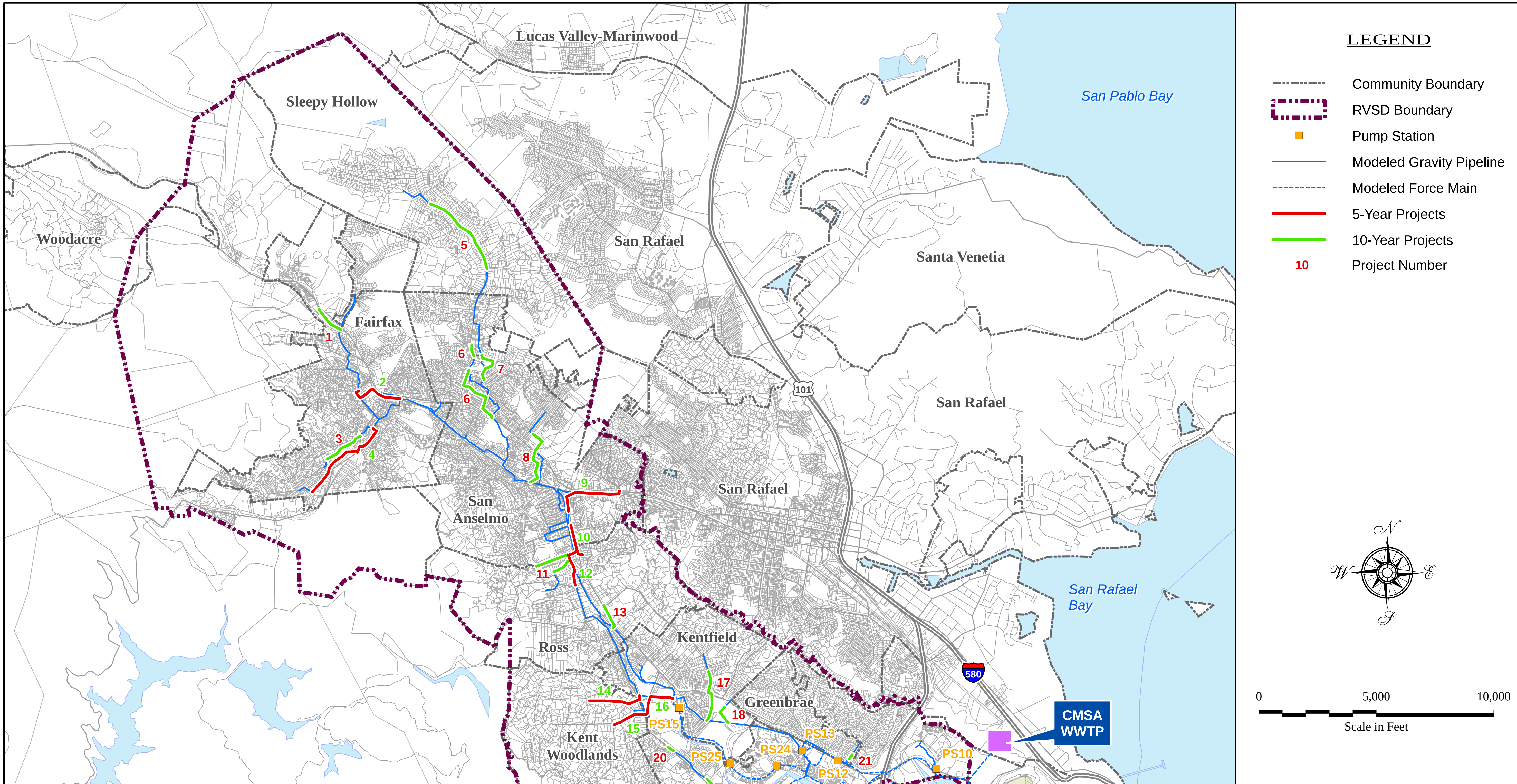
Improvement Projects 1-5

Figure 6-2



Gravity Sewer Capacity Improvement Projects

Figure 6-1

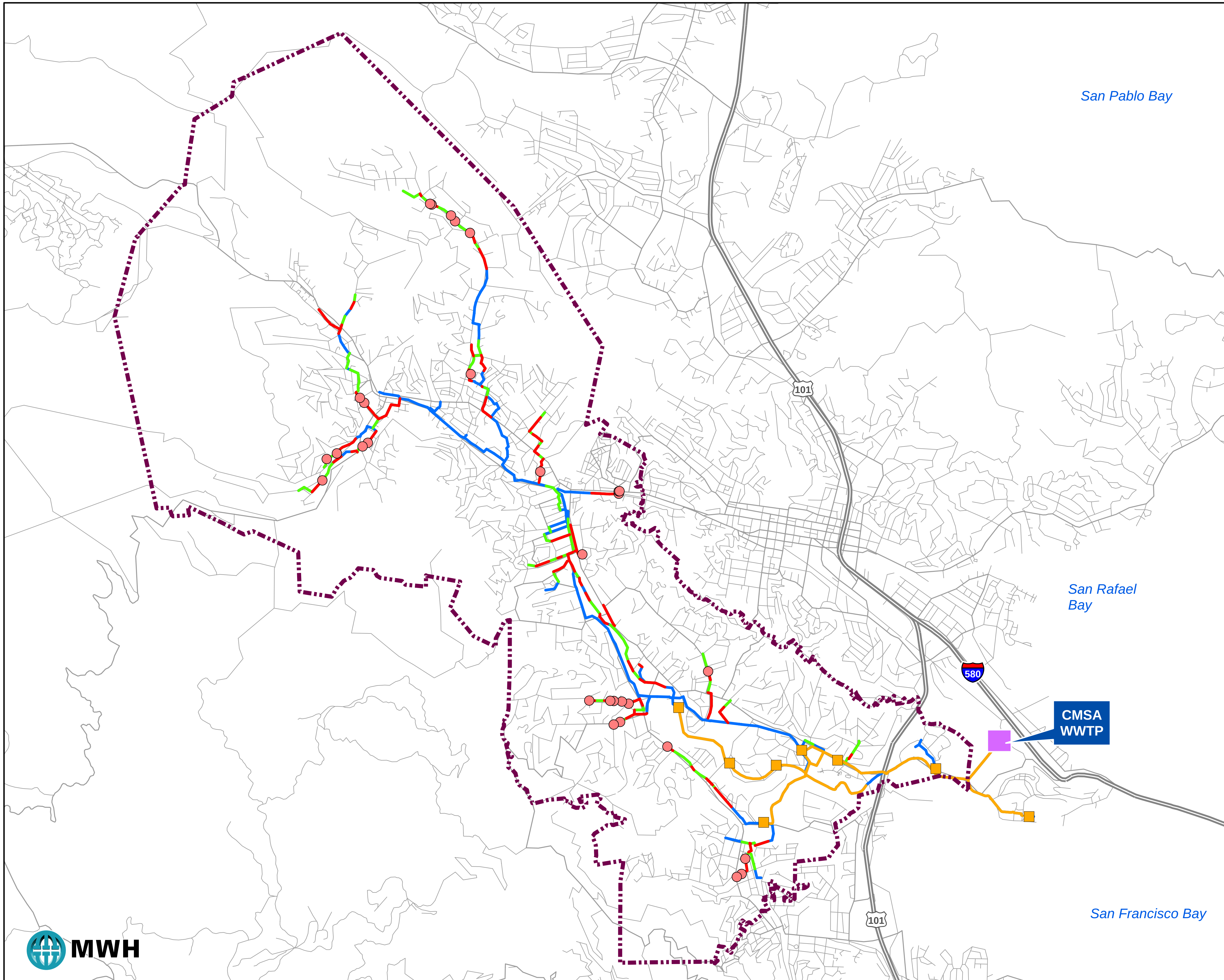


ROSS VALLEY SANITARY DISTRICT

Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

Proposed Capital Improvements Program

Figure 6-6



LEGEND

- RVSD Boundary
- Community Boundary
- Modeled Force Main
- Surcharge due to Capacity
- Surcharge due to Backup
- No Surcharge
- Pump Station
- Approximate Location of Potential Manhole Overflow

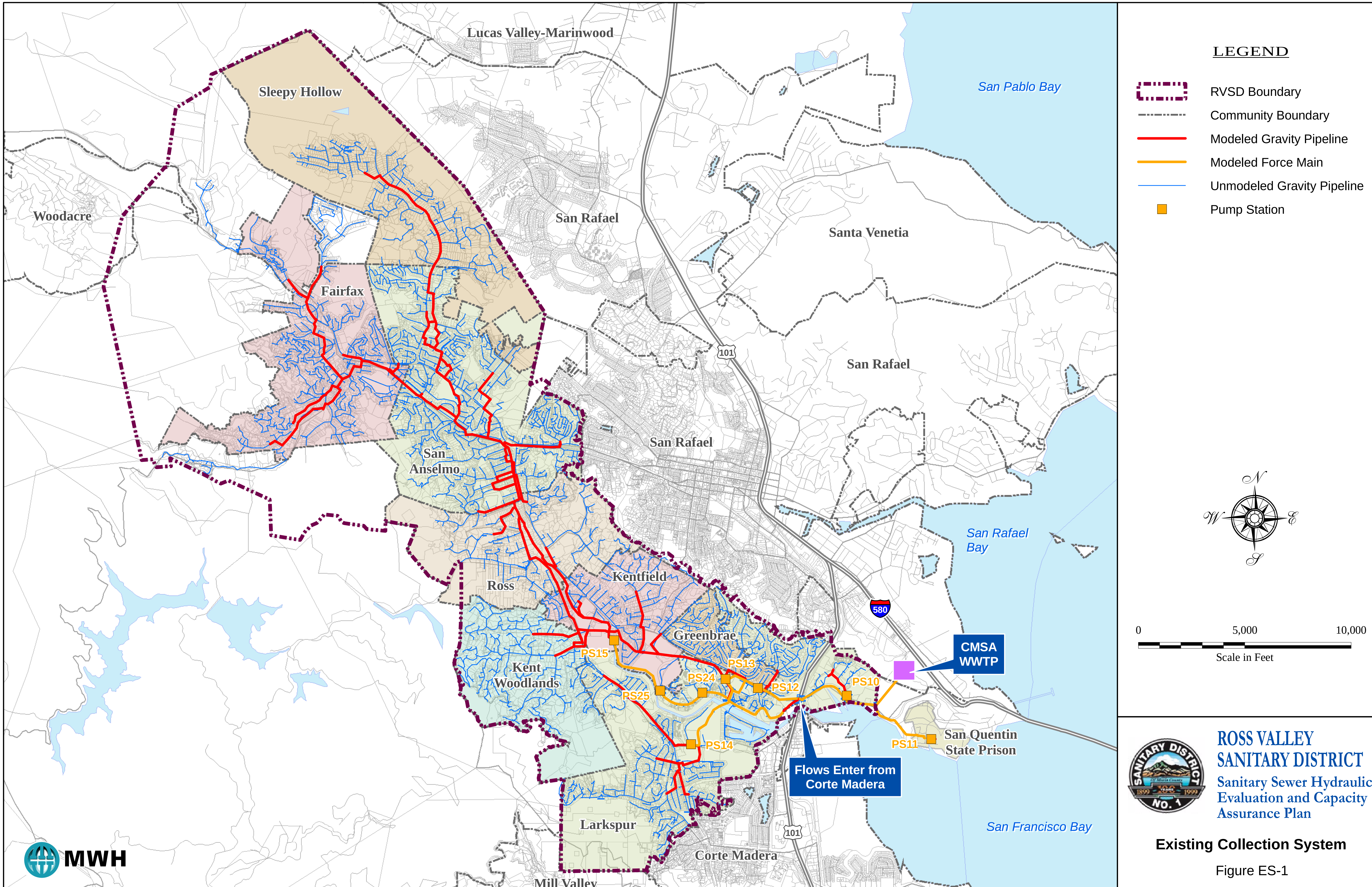
N W E S

0 5,000 10,000
Scale in Feet

ROSS VALLEY SANITARY DISTRICT
Sanitary Sewer Hydraulic Evaluation and Capacity Assurance Plan

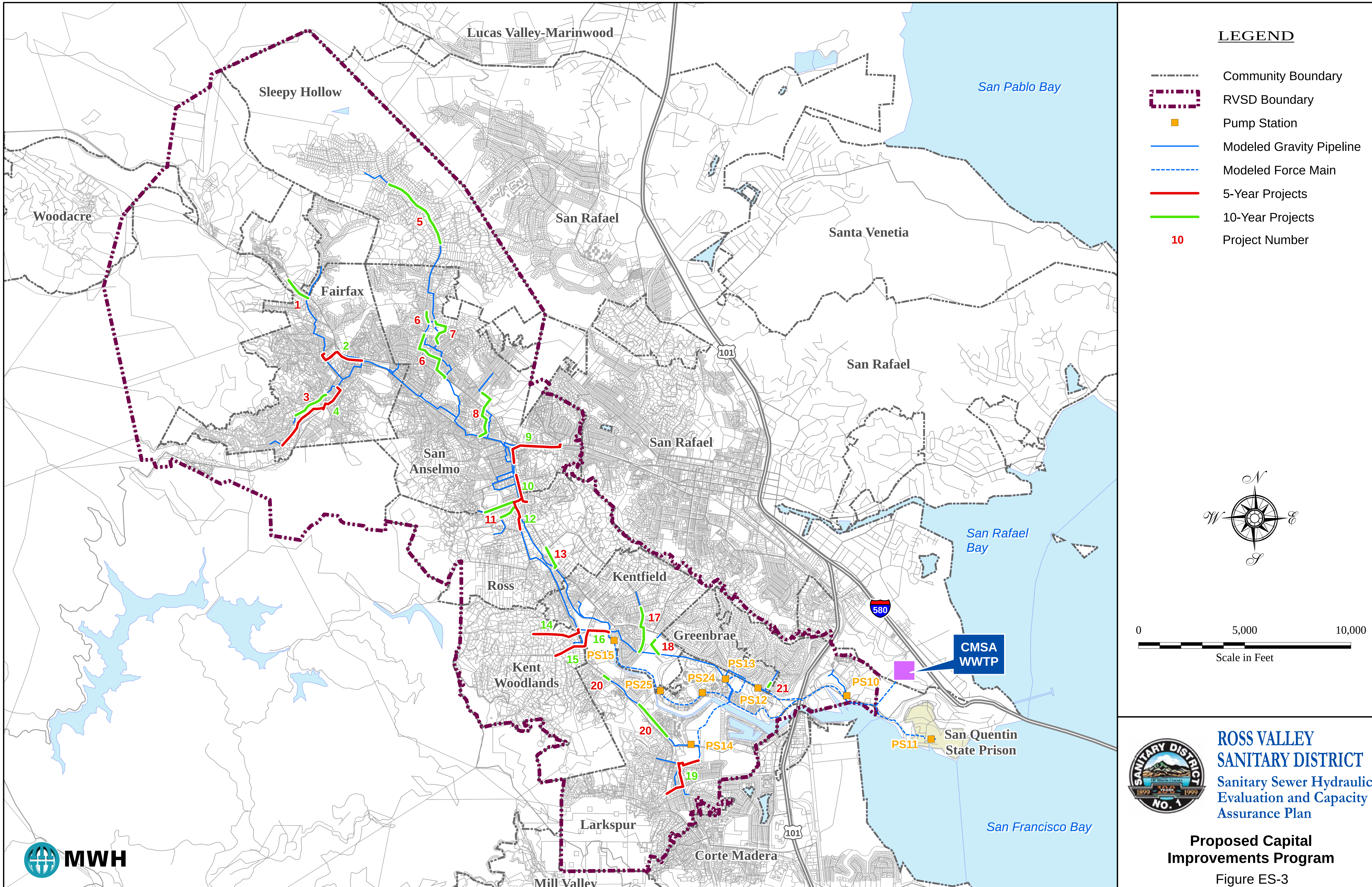
Existing Capacity Deficiencies Under Peak Wet Weather Flow

Figure ES-2



Existing Collection System

Figure ES-1



Proposed Capital Improvements Program

Figure ES-3