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Executive Summary

In May 2010, West Bay Sanitary District (District) retained West Yost Associates (West Yost) to update its wastewater collection system master plan. The 2011 Master Plan assesses the conveyance capacity of the District's sewer collection system pipes and pump stations, evaluates facilities that may require rehabilitation or replacement, develops a prioritized capital improvement plan (CIP), and establishes a funding plan for the proposed CIP.

ES.1 BACKGROUND AND INTRODUCTION

The primary purpose of this update to the sewer collection system master plan (Master Plan) is to evaluate the District's gravity sewer collection system under a specific design storm, using a computerized hydraulic model. The purpose of the hydraulic model is to determine whether the system can convey flows without sanitary sewer overflows (SSOs). Where SSOs are predicted by the hydraulic model, this Master Plan provides recommended solutions. The Master Plan also recommends a schedule for maintenance-related replacements of the pipeline and pump station facilities.

From December 2010 through March 2011, the District completed a flow monitoring study. This study provided the flow data that was used as a basis for development and calibration of the District's hydraulic model, which is a component of this Master Plan. The 2010/11 flow monitoring study was successful in capturing flow data throughout the system during several high intensity storm events.

This Master Plan is comprised of the following eleven chapters:

- Chapter 1 – Introduction
- Chapter 2 – Existing Wastewater System
- Chapter 3 – System Flows
- Chapter 4 – Infiltration and Inflow Analysis
- Chapter 5 – Hydraulic Model Development
- Chapter 6 – Planning Criteria
- Chapter 7 – Capacity Analysis
- Chapter 8 – Pipeline Condition Assessment and Capital Improvement Projects
- Chapter 9 – Pump Station Rehabilitation Program
- Chapter 10 – Capital Improvement Program
- Chapter 11 – Financial Plan

The Master Plan was developed to meet the following objectives:

- Evaluate system-wide flow characteristics to identify areas that may contribute to high inflow and infiltration (I&I);
- Assess the existing hydraulic capacity of the collection system, including pipelines and pump stations;
- Evaluate pipeline and pump station condition and potential replacement needs; and
- Develop a prioritized capital improvement program (CIP) and funding approach to provide an affordable and sustainable level of service to the District's ratepayers.

The recommendations that are presented in this Master Plan must be considered in conjunction with proposed improvements at the District's Menlo Park Pump Station (MPPS). Master planning activities for the MPPS are in progress by South Bayside Systems Authority (SBSA).

ES.2 EXISTING WASTEWATER SYSTEM

The District's existing wastewater system is described in Chapter 2. The District currently provides wastewater collection service to all or portions of the communities of Menlo Park, Atherton, Portola Valley, East Palo Alto, and Woodside, located in both Santa Clara and San Mateo Counties in the San Francisco Bay Area. The District's service area ranges from hilly, tree-covered residential areas to relatively low-topography communities, and includes narrow residential roadways, heavily traveled transportation corridors, and several commercial districts.

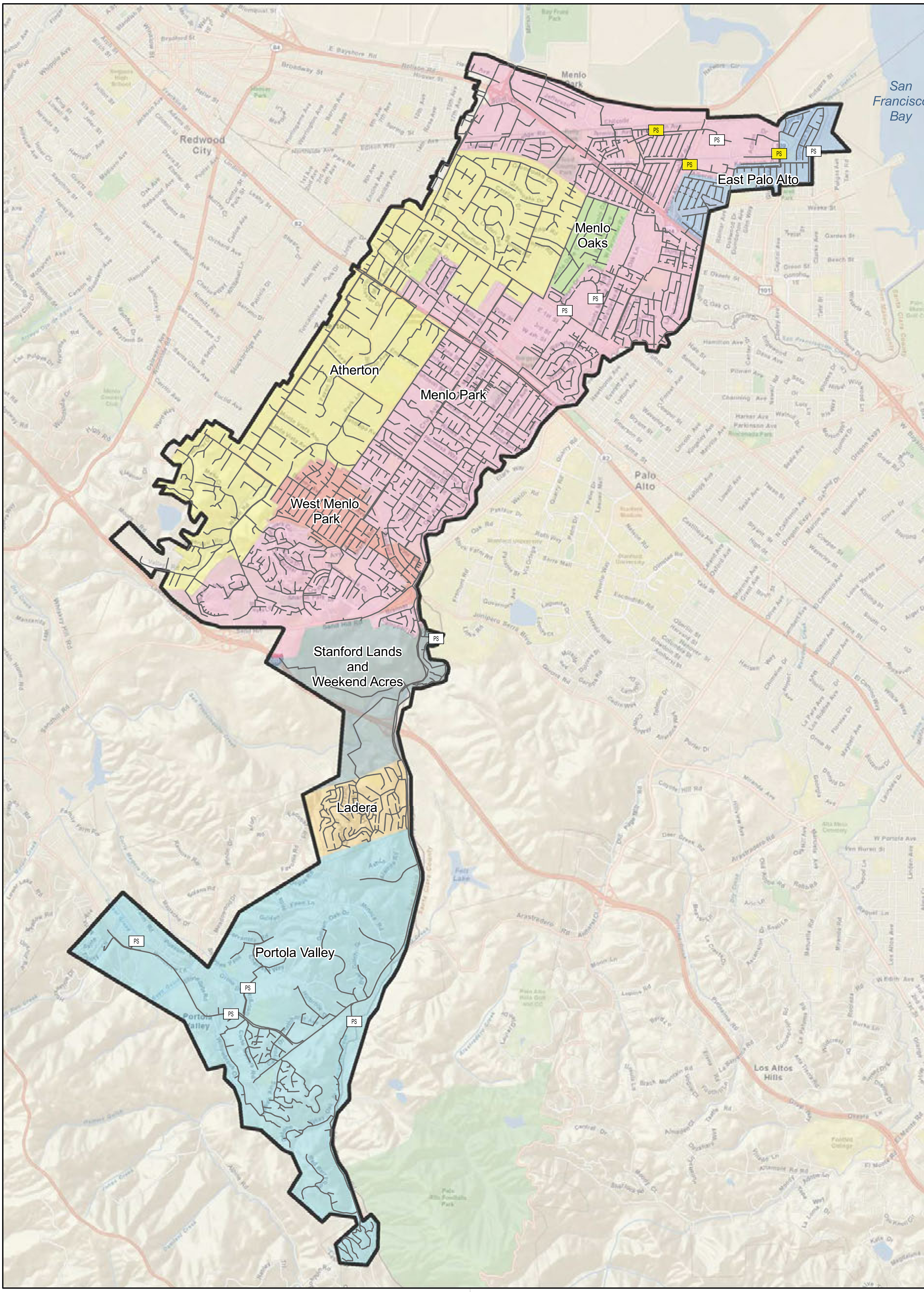
The District' service area, shown on Figure ES-1, encompasses nearly 8,325 acres, or 13 square miles, and includes approximately 20,000 service connections to serve a population of 52,900. The District serves no independent satellite systems.

The most southerly portion of the system, in the Town of Portola Valley, will experience the greatest change in sewer flows in the future. Wastewater flows for most of this area are currently addressed through private septic systems. As the existing septic systems reach the end of their useful lives and require replacement, these systems will, over time, be required to connect to the public sewage collection system.

The District operates and maintains the collection system in accordance with the requirements of the State Water Resources Control Board, as administered through the Statewide SSO Waste Discharge Requirements and RWQCB Sewer System Management Plan guidelines.

ES.3 ESTIMATION OF SYSTEM FLOWS

The methodology used to estimate the initial dry weather or base wastewater flow (BWF) component of the collection system hydraulic model is described in Chapter 3. These initial flows were further refined through the hydraulic model calibration process that is discussed in Chapter 5, Hydraulic Model Development. The District's BWF, as measured during the 2009/10 flow monitoring program that is discussed below, is 4.6 million gallons per day (mgd). This BWF translates to approximately 87 gallons per capita per day (gpcpd), which is within industry standard and closely matches the District's design criteria of 85 gpcpd.



LEGEND

PS Modeled Pump Stations

PS Pump Station

Sewer Pipes

Parcels

WBSD Service Area

N

0 0.5 1

Miles

FIGURE ES-1

West Bay Sanitary District

Collection System Master Plan

WBSD Service Area

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

The initial BWF component was calculated using the following steps:

- Standardization of the City of Menlo Park land use designations, and those of the surrounding unincorporated communities;
- Identification of flows from large industrial or commercial parcels;
- Application of unit flow factors to the assigned land use categories, and then separately, assignment of large commercial flows, to estimate dry weather sanitary sewer flows per sewer basin;
- Adjustment of unit flow factors so that estimated dry weather sanitary sewer flows across all basins closely agreed with the metered flows measured during the 2009/10 flow monitoring program that is discussed below; and
- A separate calculation was used for Portola Valley, where all parcels adjacent to a constructed sewer were assumed to be connected to the sewer. This count was compared to and is consistent with connection information provided by the District.[1]

The same methodology was used to calculate buildout flows. Buildout flows were calculated by assigning unit flow factors to buildout land uses, according to the following steps:

- Populate all vacant residential parcels,
- Add flow from all new development projected in the individual General Plans, and
- Add flow from the Town of Portola Valley assuming that all parcels currently on septic will be connected to the sewer system at buildout in order to estimate total system flows, and that parcels will connect at a rate of 10 parcels per year to assess localized surcharge conditions.

ES.4 INFLOW & INFILTRATION ANALYSIS

The 2009/10 flow monitoring program, conducted by V&A Consultants (V&A), captured rainfall events from December 2009 to March 2010. During this flow monitoring period, the District experienced several high intensity and relatively short duration storm events, which are ideal for evaluating inflow and infiltration (I&I) and for calibrating the hydraulic model. Using collected data, V&A completed an I&I evaluation to quantify the extent of I&I entering the collection system by basin during this period. Results from this program and the subsequent analysis of flows are discussed in Chapter 4.

[1] This calculation of existing flows is conservative, as some parcels in the Town of Portola Valley adjacent to the existing sewer system will continue to use a septic system until the parcel owner elects or is required by the Town to connect to the sewer system.

ES.4.1 Data Collection

The flow monitoring program included twelve (12) gravity meters and two (2) rain gauges. The twelve meters were located in manholes that delineated the collection system into ten basins, comprising two major drainage zones and two minor zones. Figure ES-2 presents the flow meter locations and associated flow monitoring basins within the collection system.

Depth and velocity readings were collected at each flow meter in 15-minute increments. This data was compiled into hourly flows for use in the I&I analysis and hydraulic model calibration.

The two rain gauges were located in Portola Valley at the Village Square Pump Station, and at Menlo Park Pump Station in Menlo Park. In addition, V&A utilized data from an existing public weather station that was located in the central portion of the District's service area, on Cedar Avenue south of Barney Avenue in Menlo Park.

ES.4.2 Description of Flows

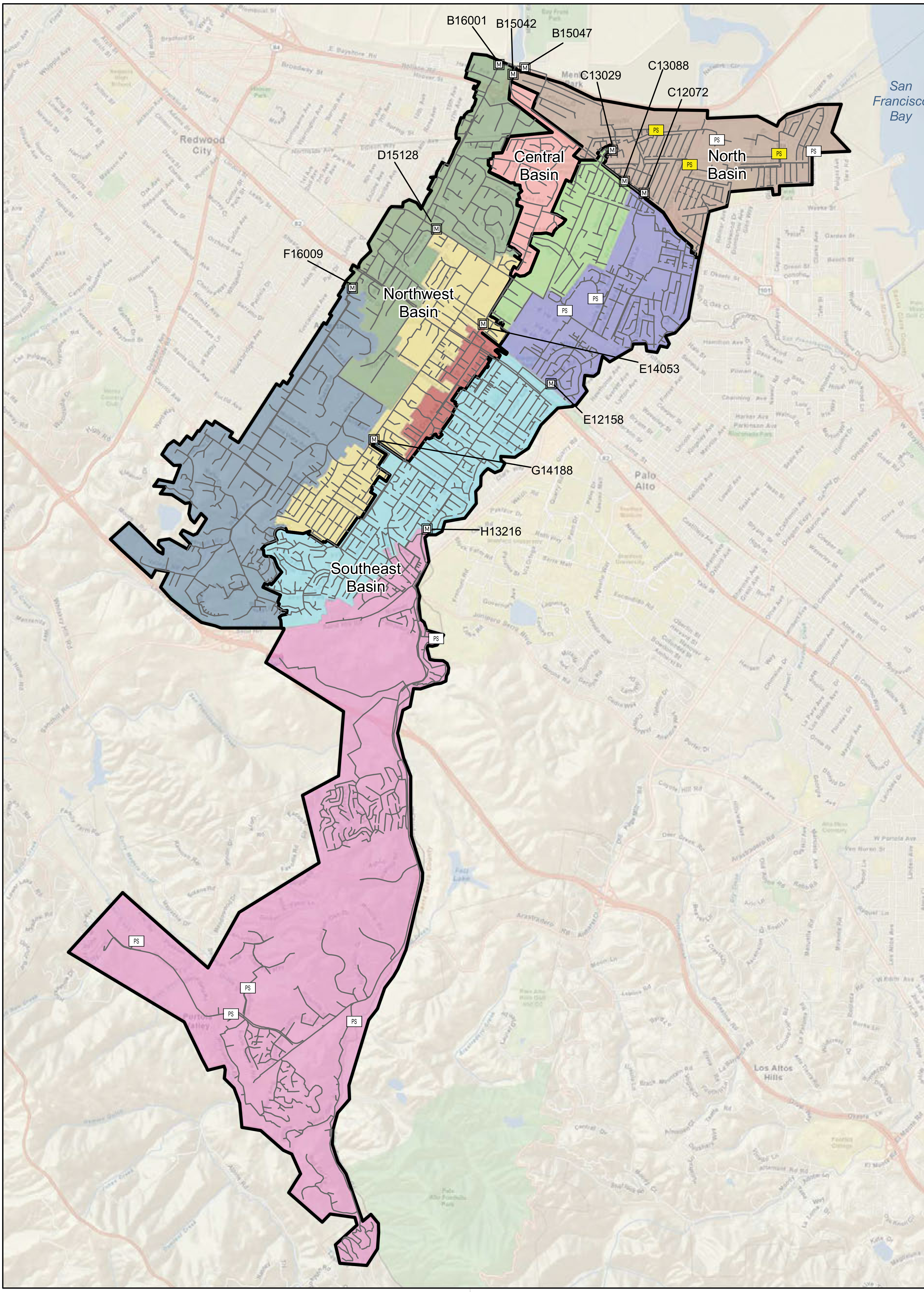
The flow monitoring program measured dry and wet weather flows through the District. The District's BWF, measured across weekday and weekend periods, was 4.6 mgd. BWF includes the wastewater generated from residential, commercial, and public users. The total measured peak hourly wet weather flow, estimated by adding measured flows from the three meters directly upstream of the MPPS, was 19.8 mgd on January 20, 2010. This calculation is conservatively high because it assumes that flows at the three meters peaked simultaneously. Actual flows showed some attenuation, reaching their peak values a slightly offset times. However, the offsets were not sufficient to significantly change hourly peak flow values.

The largest rainfall event during the flow monitoring period occurred from January 18-23, 2010. Several smaller storms occurred from December 26, 2009 through March 12, 2010. The total rainfall volume recorded by each of the three rain gauges during the January 18-23 storm event is presented in Table ES-1.

Table ES-1. Summary of January 18-23, 2010 Rainfall Data

Rain Gauge	Elevation of Rain Gauge, feet	Daily Rainfall Volume, inches						Total 6-Day Storm Volume, inches
		18th	19th	20th	21st	22nd	23rd	
1	432	0.72	1.46	3.01	0.80	0.34	0.45	6.78
2	8	0.56	.082	1.25	0.59	.032	0.28	3.82
3	111	0.77	1.33	2.27	0.81	0.35	0.24	5.77

As calculated by V&A and confirmed through review of the NOAA atlas maps, the maximum 24-hour rainfall total for the January 18-23, 2010 storm event had a classification of less than a 2-year, 24-hour storm event.



LEGEND

- Basin 1A
- Basin 1B
- Basin 1C
- Basin 1D
- Basin 1E
- Basin 2A
- Basin 2B
- Basin 2C
- Basin 3
- Basin 4

- M Flow Meter / Manhole ID
- PS Modeled Pump Stations
- PS Pump Station
- Sewer Pipes
- District Basin Boundaries

Basin Key	
Central	3
North	1A
Northwest	1C, 2B, 2C , 4
Southeast	1B, 1C, 1D, 1E

N

0 0.5 1

Miles

FIGURE ES-2

West Bay Sanitary District

Collection System Master Plan

Flow Monitoring Basins

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

O:\Clients\453 West Bay Sanitary Dist\02-10-03 Collection System Model MP & Fee Estimate\GIS\Figure ES-2.mxd 5/16/2011

In 2010 and 2011, the District conducted supplemental wet weather flow monitoring, with the objective of capturing additional flow data from the basins that exhibited the highest I&I in 2009/10. The flow data provide information that will be useful in the definition of future capital project alternatives for the District (and have been used by the District to refine options for pipe replacements in Atherton Avenue). However, because the 2010/11 wet weather events did not exceed the rainfall depths and intensities that were measured during the 2009/10 program, the information gathered was not instrumental in refining model results.

ES.4.3 Inflow and Infiltration Analysis

Based on the data collected during the 2009/10 flow monitoring study, the largest contributors to wet weather inflow in the City's system appear to be the communities that are tributary to the trunk sewer located in Atherton Avenue, upstream of Mercedes Avenue. Inflow was evaluated based on a comparison of peak I&I to average dry weather flow, and also by calculating and comparing peak I&I per acre. High inflow was also measured by the meter that was located on Middlefield Road northwest of Watkins.

Rainfall dependent infiltration (RDII) was analyzed using used data from January 17 through 24, 2010. V&A compared total I&I per acre of drainage area, and total I&I as a percent of ADWF during this period. The basin monitored by Meter 2, comprised of the area north of Highway 101 and adjacent to the San Francisco Bay, consistently displayed the highest RDII within the District's service area. This basin also showed the presence of excess groundwater infiltration (GWI), with an estimated volume of 300 gallons per day.

ES.5 HYDRAULIC MODEL DEVELOPMENT

West Yost developed a computer-based hydraulic model of the District's wastewater collection system, developed using Innovyze® InfoWorks™ CS software, in order to assess system capacity. The hydraulic model includes the District's main trunk sewers (12-inch diameter and larger) and associated facilities, and is a skeletonized representation of the sewer system in its configuration and operation. This section summarizes the components of model development that are discussed further in Chapter 5 of this report.

The hydraulic model consists of approximately 37 miles of sewer gravity pipes and forcemains ranging in diameter from 6 inches to 54 inches. The model includes all 12-inch diameter and larger trunk lines, and associated manholes and lift stations. Many 10-inch diameter pipelines have been included, as well as 6-inch and 8-inch diameter lines in selected areas as needed to provide connectivity. The 37 miles of pipeline represent over 18 percent of the District's system.

Three of the District's twelve pump stations are included in the hydraulic model: Hamilton Henderson, Willow Avenue, and University Avenue. The terminus of the District's system is represented by an open outfall, since flows conveyed to the MPPS would either be pumped to SBSA or bypassed for storage in the District's FEF. There should be no flow limitations at the MPPS that would require the District to store wastewater within its collection system.

ES.5.1 Dry Weather Flow Generation

The District's unit flow factors were applied to land use to generate BWF. The land use and unit flow factors are described further in Chapter 2, Existing Wastewater System. The key elements of dry weather flow generation in the hydraulic model include:

- Average dry weather flow (Q_a)
- Peak dry weather flow (Q_{pdwf})

Residential Q_a calculation was based input by equivalent dwelling unit, based on the land use data provided by the City of Menlo Park, as described in Chapter 2. The non-residential land use Q_a was assigned by acreage, and large dischargers were represented by point loads applied to the manhole(s) where the discharger connects to the system. A diurnal usage pattern was applied to the flows and refined as a part of the dry weather calibration to establish Q_{pdwf}. Buildout flows are included as a separate input to the model.

West Yost refined these unit flow factors by comparing calculated average daily flows per basin to the dry weather flow results from the flow monitoring program. Unit flow factors for the various land use categories were then adjusted within normal range until predicted BWF closely matched measured the collected flow data.

ES.5.2 Wet Weather Flow Generation

Wet weather flows were then calculated and input to the District's hydraulic model to replicate measured flow data. The key elements of wet weather flow generation in the model include:

- Rainfall Dependent Infiltration and Inflow (RDII or I&I)
- Peak Wet Weather Flow (Q_{pwwf})

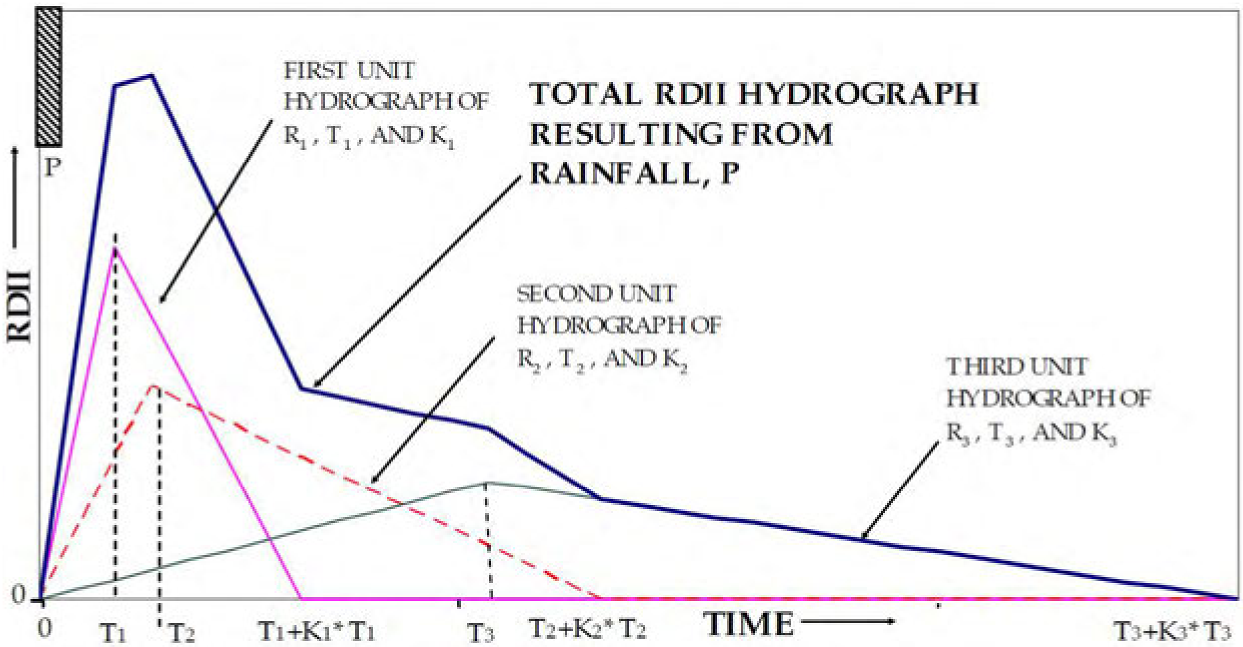
The District's model uses the RTK method to calculate wet weather inputs to the hydraulic model. The RTK method generates hydrographs from each subcatchment that represent flows during and immediately after rainfall events caused by seepage of water into the collection system. The RTK method generates a series of three triangular hydrographs that represent short-term, medium-term, and long-term rainfall response. The RTK parameters include:

1. R is the area of the graph representing the portion of rainfall falling on a subcatchment that enters the sewer collection system.
2. T is the time from the onset of rainfall to the peak of the triangle.
3. K is the ratio of the "time to recession" to the "time to peak" of the hydrograph.

RTK parameters were adjusted by basin, beginning with the upstream basins, until measured flows during the calibration storm closely matched model-generated flows in temporal distribution and volume. Because the District was able to collect high quality flow data during the 2009/10 flow monitoring period, and during the peak rainfall events the system was able to convey all flows without SSOs, wet weather calibration was achieved for flows at all meter locations.

The components of the RTK method are presented in Figure ES-3.

Figure ES-3. Components of RTK Hydrograph



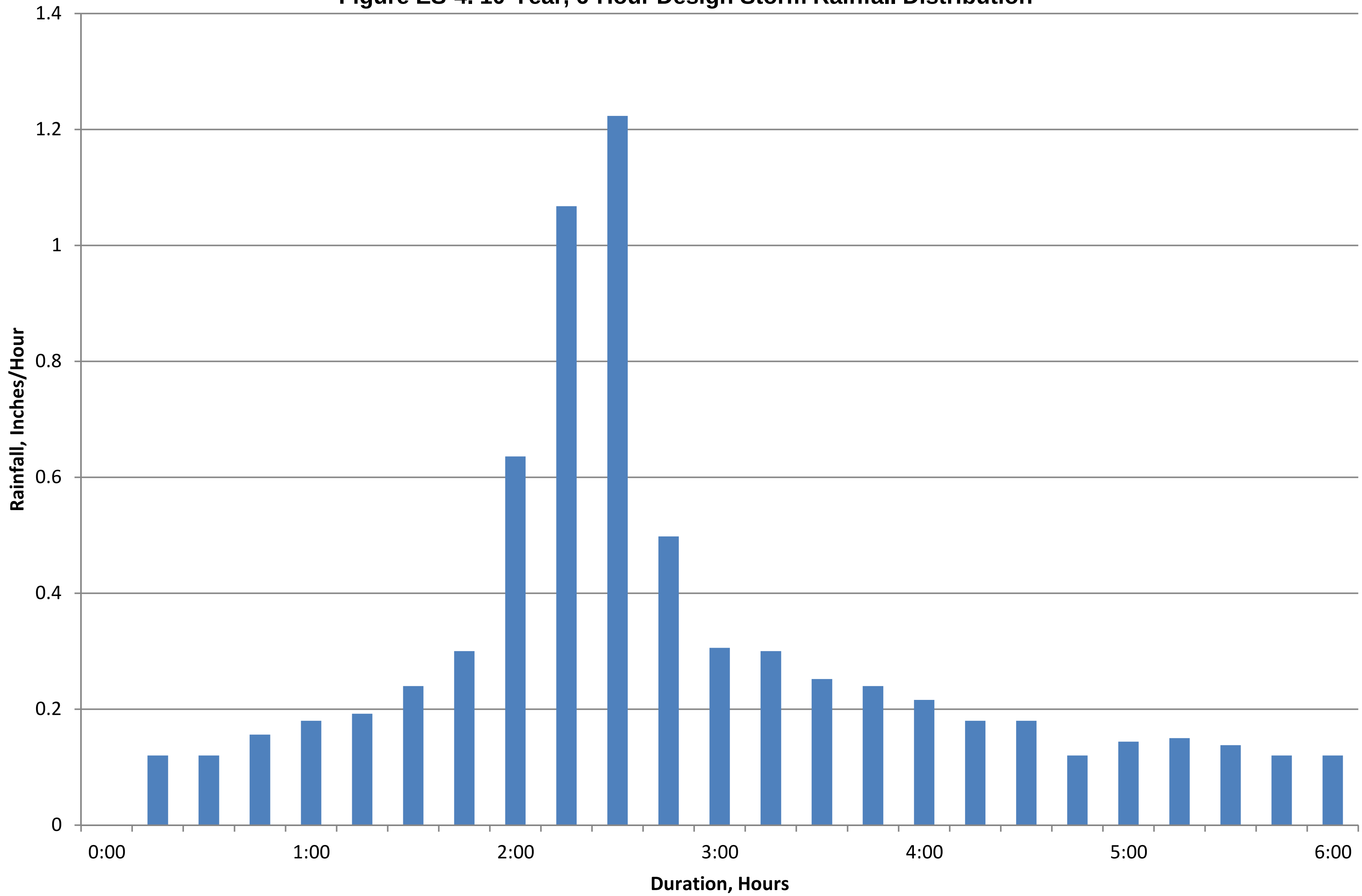
ES.6 PLANNING CRITERIA

The planning criteria used to evaluate system capacity and size new replacement facilities are discussed further in Chapter 6. The criteria include generally accepted industry standard criteria, as reviewed and confirmed by the District. Planning criteria address items such as collection system capacity, gravity sewer slopes, and maximum depth of flow.

Although there are no regulatory requirements for sewer sizing, the District selected a design storm with a 10-year recurrence interval and 6-hour duration (10-year, 6-hour storm). This design storm was selected to match criteria adopted by the City of South San Francisco, under a recent settlement agreement signed by a federal court. This design storm has a total depth ranging from 1.8 inches at the MPPS to 3.2 inches in Portola Valley. The rainfall was distributed using the U.S. Soil Conservation Service (SCS, now Natural Resource Conservation Service) Type I rainfall distribution curve.[2] Figure ES-4 presents the design storm rainfall distribution for predicted MPPS rainfall.

[2] Rainfall depth is published in the Precipitation-Frequency Atlas of the Western United Stations, Volume XI-California, published by the National Oceanic and Atmospheric Administration.

Figure ES-4. 10-Year, 6-Hour Design Storm Rainfall Distribution



As a guideline, existing pipelines could be considered capacity deficient if, under a surcharge condition, the surcharged manhole had freeboard, measured from the water surface to ground elevation, of less than five feet. For pipes with less than five feet of cover, the pipe was considered capacity deficient when flows exceeded full pipe capacity. Existing pump stations were considered capacity deficient if the station was unable to convey peak flows with the largest pump out of service. Exceptions to these criteria were applied on a case by case basis, depending on actual flow conditions and pipeline configuration.

New or replacement pipelines were sized to meet the following criteria, when possible:

- Under PDWF conditions, velocity above 2 feet per second to facilitate self-cleaning; and
- Under PWWF conditions, maximum flow depth (d) as compared to pipe inside diameter (D) d/D of 0.67 for pipes 10-inches in diameter and smaller, and 0.80 for pipes 12-inches in diameter and above.

Under all conditions, maximum allowable velocity was limited to 10 feet per second.

ES.7 HYDRAULIC CAPACITY ANALYSIS RESULTS

The District's modeled collection system network was evaluated for its capacity to convey flows that are predicted to occur during the selected design storm event. The analysis is summarized in Chapter 7. The hydraulic model predicted peak hourly flow from the design storm of 26.5 mgd.

Analyses were conducted as follows:

- The system was evaluated for its ability to meet established surcharge criteria. Pipe diameter increases that may be required to convey peak flows and meet surcharge criteria were determined. These projects, as further refined through Step 2, form the District's long-term capacity improvement needs.
- The system was evaluated for its ability to convey flows without SSOs. Pipe diameter increases, as developed in Step 1, that may be required to address overflow issues were moved up in priority to form the District's priority capacity improvement needs.
- The projected peak wet weather flow rate at the MPPS was documented and conveyed to SBSA. SBSA is currently completing a separate conveyance system master plan that addresses wet weather capacity requirements for the MPPS and downstream facilities.
- Proposed improvements were then reviewed based on the relatively minor increases that are projected for buildout flow and adjusted where required.

The resulting priority and long-term capacity improvement project recommendations are show in Table ES-2.

Table ES-2. Capacity Improvement Project Estimated Costs		
Project Name	Estimated Construction Cost	Estimated Total Cost
James Avenue Diversion	$4.5M	$5.4M
Lower Ringwood Avenue	$811k	$957k
Lower Valparaiso Avenue	$347k	$444k
Lower Santa Cruz-Avy	$548k	$667k
Upper Willow Pump Station Gravity Main	$176k	$226k
Total Estimate for Priority Projects	$6.4M	$7.7M
Upper Ringwood Avenue	$183k	$234k
Middlefield at Fair Oaks	$395k	$502k
Atherton Avenue East	$3.6M	$4.2M
Upper Valparaiso Avenue	$142k	$182k
Upper Santa Cruz-Avy	$411k	$519k
Lower Willow PS Gravity Main	$839k	$991k
Total Estimate for Long-Term Projects	$5.6M	$6.6M

ES.8 PIPELINE REHABILITATION AND REPLACEMENT

The Master Plan assesses the District's ongoing rehabilitation and replacement needs, to be completed in conjunction with capacity improvements. This assessment and associated recommendations are presented in Chapter 8. These recommendations are guidelines that are intended to be revisited as system needs are known, and reprioritized as needed to best meet the overall objective of reducing SSOs and increasing sewer system reliability.

The rehabilitation and replacement (R&R) CIP was developed through an assessment of the District's operations, maintenance, and spill records, including the following:

- Collection system pipe and facility asset information;
- District SSOs, using the State Water Resources Control Board, CIWQS and District's records;
- District cleaning schedules and comparison of high frequency cleaning locations with historical SSOs; and
- District closed circuit television (CCTV) condition ratings, provided in hardcopy format.

Analysis of this information yielded 22 sewer pipeline projects that are recommended for implementation as part of the long-term, R&R CIP. These projects, listed in order of planned implementation in Table ES-3 were prioritized based on their potential impact to the District's ability to provide a desired level of service to its constituents.

Table ES-3. Summary of Planned Projects

R&R CIP Priority	Project Name	Priority Basis
1	Atherton Avenue	To be completed before road overlay project
2	Ladera Outfall	Previous SSOs reported along sewer
3	Fletcher	Previous SSO within private residence
4	Willow Road	Past SSOs
5	North Palo Alto Concrete	Potentially defective sewer close to creek
6	Menalto Easements	Past SSOs due to grease
7	Roble Avenue	Sewers with known potential structural defects
8	Encinal Avenue A	Sewers with known potential structural defects
9	Oak Grove Avenue	Sewers with known potential structural defects
10	Encinal Avenue B	Maintenance issues
11	Bayfront Expressway	Corrugated metal pipe in poor condition
12	Berkeley Avenue	Sewers with known potential structural defects
13	Santa Cruz Avenue	Sewers with known potential structural defects
14	College Park North	Sewers with known potential structural defects
15	Stevenson Lane	Sewers with known potential structural defects
16	Elena Avenue	Sewers with known potential structural defects
17	Fair Oaks Lane	Sewers with known potential structural defects
18	Frederick	Maintenance issues
19	Suburban Park (formerly Flood Park)	Sewers with known potential structural defects
20	Oak Knoll Area	Sewers with known potential structural defects
21	Haven	Maintenance issues
22	Carlton-Madera Easements	Maintenance issues

The highest priority sewer rehabilitation and replacement projects have been implemented by the District since the initial development of this plan. These projects include Atherton Avenue, Ladera Outfall, Fletcher pipeline improvement, and Willow Avenue pipeline replacement.

During the development of this Master Plan, the District advertised for bid and received unexpectedly competitive bids for these projects. As a result, the District is using the remaining available budget to accelerate and complete design of the Encinal Avenue A and Oak Grove pipeline rehabilitation projects (Nos. 9 and 10). These projects will be relined using cured-in-place pipe, in order to defer the planned replacement projects that were originally anticipated in the latter years of the CIP.

In addition, the District has designed and will implement a capacity upgrade for the Corte Madera forcemain. These five projects will complete construction in fiscal year 2011/12. In fiscal year 2012/13, the District will begin design of the Bayfront Expressway pipeline replacement project.

Chapter 8 provides background information on available pipeline materials and installation methods, and the associated ranges of unit costs that were used to develop planning-level cost estimates for each project. The District prefers to use open trench construction methods for replacement pipelines, and cured-in-place-pipe for in-situ pipe rehabilitation. However, pipeline materials and construction needs will be determined on a project-by-project basis.

ES.9 PUMP STATION REHABILITATION

An analysis was conducted to assess the District's modeled and unmodeled pump stations, and to recommend improvements that may be needed over time to maximize the reliability and useful life of the conveyance system. This evaluation and associated recommendations are presented in Chapter 9.

The District owns and maintains 12 pump stations in the collection system, and also owns the flow equalization facility (FEF) transfer pump station and the Menlo Park Pump Station (MPPS). SBSA operates the FEF transfer station and operates and maintains the MPPS. The 12 pump stations plus the FEF transfer pump station are listed in Table ES-4.

Table ES-4. District Owned and Maintained Pump Stations

Pump Station	Location
Hamilton Henderson	Menlo Park, east of 101
Willow Road	Menlo Park, east of 101
Menlo Industrial	Menlo Park, east of 101
University	East Palo Alto
Illinois	East Palo Alto
Vintage Oaks #1	Menlo Park
Vintage Oaks #2	Menlo Park
Stowe Lane	Unincorporated San Mateo County
Los Trancos	Santa Clara County
Sausal Vista	Portola Valley
Corte Madera	Portola Valley
Village Square	Portola Valley
Flow Equalization Facility	Menlo Park, east of 101

ES.9.1 Dry Weather and Wet Weather Conveyance Capacity

ES.9.1.1 Wet Well Time to Overflow

The pump station capacity analysis included an evaluation of the time to overflow from each pump station wet well in the event of pump failure or some other disruption, during dry and wet weather flow. Of the District's 12 pump stations, the Willow and University Pump Stations have less than one hour of wet well storage under average dry weather flow conditions. Under peak wet weather flow conditions, all pump station wet wells except for Vintage Oaks #1 and Village Square pump stations have less than one hour of storage capacity in the event of pump failure.

ES.9.1.2 Wet Weather Firm Pumping Capacity

The hydraulic model results confirmed that the three modeled pump stations, Hamilton Henderson, Willow, and University pump stations, have sufficient firm capacity to convey design storm wet weather flows. Firm capacity is defined as pumping capacity with the largest pump out of service.

Using a wet weather peaking factor of 5, the remaining pump stations were assessed and determined to have sufficient firm capacity with regard to conveyance of peak flows. It should be noted, however, that District staff observed the Corte Madera pump station as nearing capacity, with both pumps in service, during the January 20, 2010 rainfall event. This event, which was classified as having a lower rainfall intensity than the design storm, should not have resulted in this capacity level using a wet weather peaking factor of 5.

It is believed that the gravity sewers upstream and tributary to the Corte Madera pump station receive direct stormwater inflow from one or more developments. This stormwater inflow raises the wet weather peaking factor significantly, which increases wet weather flow through the Corte Madera pump station. The District is completing the design of a project to increase the capacity of the Corte Madera force main. It is recommended that the District conduct localized wet weather flow monitoring upstream of the Corte Madera pump station, in order to quantify I&I and establish wet weather design criteria for the pump station and force main.

ES.9.2 Condition Assessment

In addition to evaluating pump station capacity, the Master Plan includes recommendations for ongoing pump station improvements that are needed to maximize facility useful life and reliability. Improvements are recommended for five pump stations: Willow Road, Hamilton Henderson, Sausal Vista, and Stowe Lane. These improvements include MCC replacement, conversion of the Sausal Vista pump station to 3-phase power, replacement of corroded materials, generator replacement, and replacement and possible reconfiguration of the Stowe Lane pump station.

ES.10 CAPITAL IMPROVEMENT PROGRAM

The recommendations from the pipeline capacity assessment, pipeline R&R evaluation, and pump station evaluation are consolidated and prioritized in a CIP that is presented in Chapter 10. The CIP establishes initial project priorities and a funding timeline for implementation. The CIP strives to maintain capital improvements in the first four years of the CIP at approximately

$3.5 million per year, increasing to $3.8 million in years five through seven, and increasing again to $4.0 million in years eight through ten, all in current dollars.

Table ES-5 presents the proposed updated CIP, which begins implementation in Fiscal Year 2011/12 and extends into Fiscal Year 2021/22. The CIP is intended to be an evolving document that is adjusted as needed to address future issues and priorities that may arise after completion of the Master Plan. The most critical components of the CIP include four priority pipeline rehabilitation and replacement projects, one priority pump station forcemain expansion project, and five priority capacity improvement projects. Additional information on the individual projects, including detailed cost estimates, can be found in Chapters 7 through 9 of this report.

The remainder of the CIP includes 18 additional pipeline rehabilitation or replacement projects, three pump station improvement projects, and six capacity improvement projects that are not considered priority projects but could be completed during the proposed CIP timeframe.

The total estimated CIP cost for planning purposes is $46.3 million, to be implemented during and after the ten-year CIP timeframe.

ES.11 FINANCIAL PLAN

The District's operating costs, proposed capital improvement program, and anticipated SBSA expenses were reviewed, and a schedule of rates and connection fees developed, as discussed in Chapter 11. The proposed rates and fees are sufficient to pay the cost of service and to also enable the District to maintain adequate reserves.

Operating costs were derived from the District's FY2011/12 budget planning documents. Capital improvement program costs reflect the projects and priorities that are described in Chapter 10 of this report. Anticipated SBSA expenses were determined by calculating the District's current percentage, 23.7 percent, of costs provided to the District by SBSA.

ES.11.1 Revenue Required for Projected Capital and Operating Expenses

Figure ES-5 shows the District and SBSA revenue requirements. Historically, the District's sewer service charges have increased incrementally over time to accommodate inflationary increases to annual operations and maintenance costs. In the past year and now looking to the future, the District has and will continue to be faced with significant increases in SBSA capital improvement costs, as well as additional capital improvements to renew and replace District infrastructure.

Table ES-5. West Bay Sanitary District Example Capital Improvement Project Implementation Plan (Notes 1 through 3)

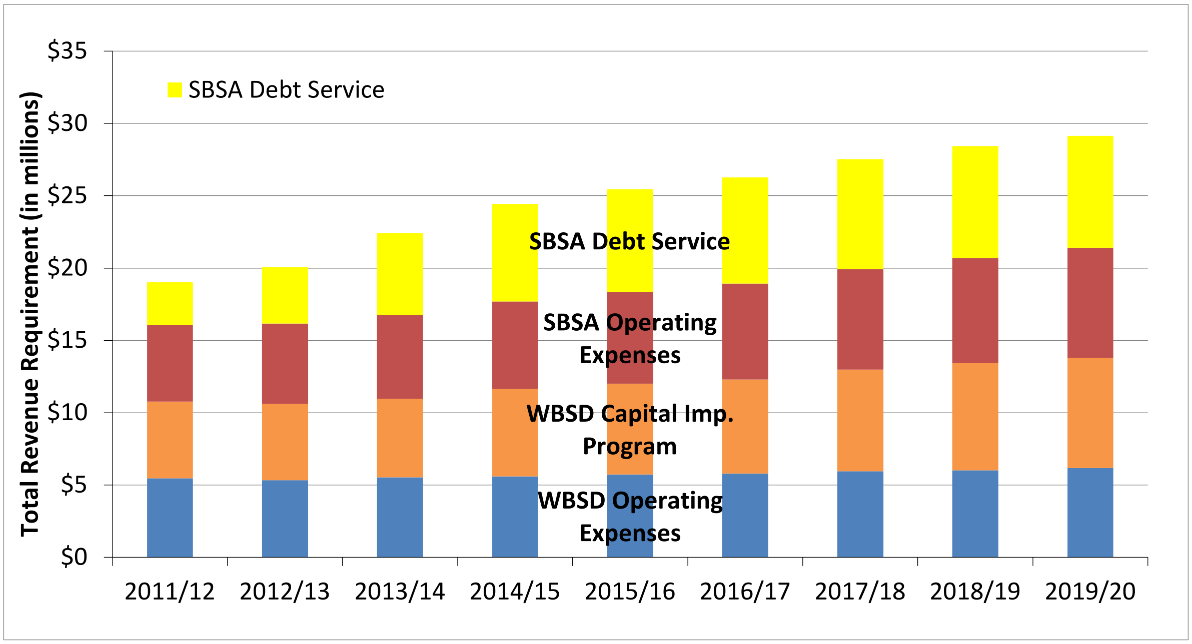
R&R Project #	Project Name	Estimated Cost	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	Future (listed projects only)	Check
1	Fletcher (Replaced by Haven)													
2	Atherton													
3	Ladera Outfall	$2,100,000	$2,100,000											2,100,000
4	Willow													
5	Corte Madera Forcemain													
6	Sausal Vista PS and Forcemain	$1,298,000			1,298,000									1,298,000
7	North Palo Alto Concrete	$1,487,000			1,487,000									1,487,000
8	Fair Oaks	$3,025,000					302,500	2,722,500						3,025,000
9	Santa Cruz	$1,004,000							1,004,000					1,004,000
10	Roble	$2,630,000							2,630,000					2,630,000
11	Stevenson	$1,155,000								1,155,000				1,155,000
12	Elena	$1,802,000									901,000	901,000		1,802,000
13	Carlton-Madera Easements	$2,504,000									1,252,000	1,252,000		2,504,000
14	College Park North	$2,213,000											2,213,000	2,213,000
15	Oak Grove	$1,009,000	243,000								766,000			1,009,000
16	Encinal A	$1,174,800	763,800								411,000			1,174,800
16a	Miscellaneous Pipe Segment Repair	$590,200	590,200											590,200
17	Oak Knoll	$769,000								769,000				769,000
18	Encinal B	$2,170,000											2,170,000	2,170,000
19	Bayfront Expressway	$3,000,000		3,000,000										3,000,000
20	Menalto Easements	$788,000										788,000		788,000
21	Berkeley	$1,213,000								1,213,000				1,213,000
22	Frederick	$209,000		209,000										209,000
23	Haven	$205,000						205,000						205,000
24	Suburban Park	$250,000		250,000										250,000
25	Willow Road Pump Station	$256,000						256,000						256,000
26	Hamilton Henderson Pump Station	$154,000					154,000							154,000
27	Stowe Lane Pump Station	$1,003,000			100,300	902,700								1,003,000
	Subtotal R&R	$32,009,000	3,697,000	3,459,000	2,885,300	902,700	456,500	3,183,500	3,634,000	3,137,000	3,330,000	2,941,000	4,383,000	32,009,000
Capacity Project #	Project Name	Estimated Cost	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	Future (listed projects only)	Check
1	James Avenue Diversion	$5,365,000			536,500	2,414,250	2,414,250							5,365,000
2	Lower Ringwood	$957,000					957,000							957,000
3	Lower Valparaiso Avenue	$444,000						444,000						444,000
4	Lower Santa Cruz-Avy	$667,000								667,000				667,000
5	Upper Willow PS Gravity Main	$226,000							226,000					226,000
6	Upper Ringwood	$234,000								234,000				234,000
7	Upper Valparaiso Avenue	$182,000						182,000						182,000
8	Upper Santa Cruz-Avy	$519,000									519,000			519,000
9	Middlefield at Fair Oaks	$502,000									251,000	251,000		502,000
10	Atherton Avenue East	$4,209,000										209,000	4,000,000	4,209,000
11	Lower Willow PS Gravity Main	$991,000										495,500	495,500	991,000
	Subtotal Capacity	$14,296,000	0	0	536,500	2,414,250	3,371,250	626,000	226,000	901,000	770,000	955,500	4,495,500	14,296,000
	Total CIP	**$46,305,000**	**3,697,000**	**3,459,000**	**3,421,800**	**3,316,950**	**3,827,750**	**3,809,500**	**3,860,000**	**4,038,000**	**4,100,000**	**3,896,500**	**8,878,500**	**46,305,000**

Note 1: Implementation schedule beginning in 2012/13 and beyond will be revised routinely based on new system information, and as needed to accommodate unexpected infrastructure repair projects.

Note 2: The District has replaced the Fletcher Project described in Chapter 8 with a repair on Haven Avenue, completed design, and received bids for Projects 1 through 5. Bids received were unexpectedly low. Based on bids received, the District has estimated a total cost for these projects, including design, construction contingency and engineering support during construction, of $2.1M.

Note 3: The District has scheduled a portion of Projects 15 and 16 for completion in FY11/12. Pipes will be rehabilitated using CIPP lining in lieu of pipe replacement. Based on bids received, the District has estimated a total cost for the accelerated projects, including design, construction contingency and engineering support during construction, of $1M. In addition, the District has budgeted $590k (including all contingencies) for miscellaneous pipeline rehabilitation projects that are not in the Master Plan. These projects are included in this Table as No. 16a.

Figure ES-5. District and SBSA Revenue Requirements



ES.11.2 District Reserves

In addition to the revenue requirement shown in the previous figure, the District is building up three reserve funds as follows:

- The Operations Reserve, which is currently in place at the District, provides working capital for monthly O&M expenses. The District receives sewer service charges from the County tax assessor, and is paid in arrears. This reserve covers the time period between the District's cash outlay and receipt of the service charge payments from the County. Maintaining the minimum balance for the Operations Reserve is the highest priority for the District's three reserves.
- The Emergency Reserve is a new fund that will be increased to a $5 million level over time to provide for unforeseen contingencies. Should an emergency strike, the District cannot suddenly raise rates to generate additional funds. Maintaining the target balance for the Emergency Reserve is the second highest priority after meeting the minimum balance for the Operations Reserve.
- The Capital Reserve is a new fund that provides liquidity to fund construction for projects that are funded on a pay-as-you-go basis (as opposed to those that are funded from debt). With adequate capital reserves, the District is able to pay contractors without encroaching on the Operations Reserve, which is the current practice. The target balance for the Capital Reserve depends on the level of anticipated construction. A minimum balance equal to the average annual construction costs (approximately $3.8 million) was used for determining an appropriate target balance.

Maintaining the target balance for the Capital Reserve would be a lower priority after meeting the minimum balance for the Operations and Emergency Reserves.

To provide additional protection until all reserves are fully funded, the cost of a line of credit is included in the revenue goals in the spreadsheet rate model.

ES.11.3 Existing and Needed Future Sewer Rates

The District approved, through a prior rate-setting process, a six percent (6%) increase in residential sewer rates for fiscal year 2011/12. The District's rate study shows that this increase will be sufficient to sustain the planned level of funding for this fiscal year.

A spreadsheet model was developed to derive revenue and rate requirements for FY 2012/13 through FY 2019/20 (however, the recommended planning timeframe for rate setting is no greater than 5 years). The model used various escalation factors to project operating and capital costs over time.

Based on this model, and assuming no modifications to current projections, in future years the FY2010/11 annual sewer service charge of $650 will need to be adjusted in future years as follows:

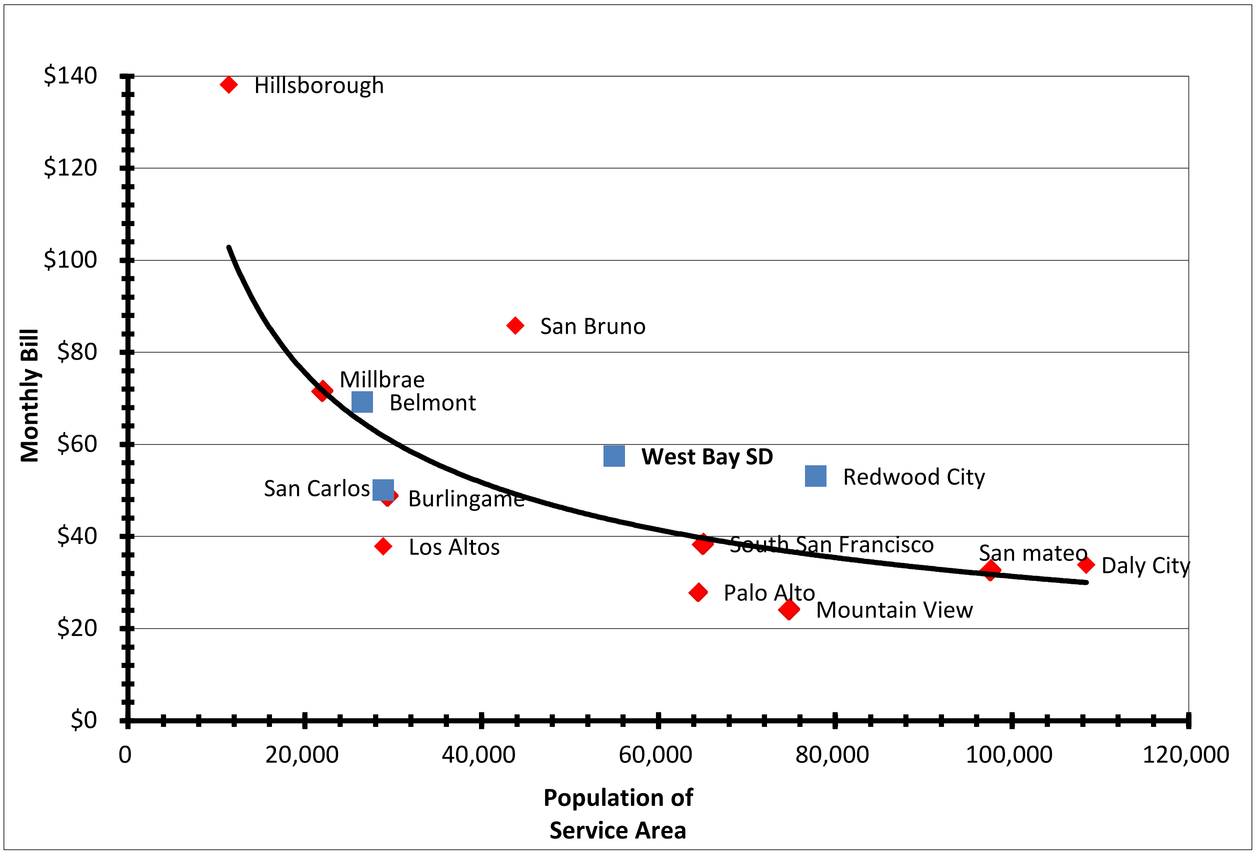
Fiscal Year Ending	2012	2013	2014	2015	2016	2017	2018	2109	2020
Proposed Annual Charge	$690	$828	$969	$1,085	$1,128	$1,162	$1,197	$1,233	$1,270
Percentage Increase	6%	20%	17%	12%	4%	3%	3%	3%	3%

The proposed sewer service charges improve the District's revenue stream over several years, until the revenue reaches the District's target annual revenue goals, including planned reserves, in FY 2015/16.

Based on available sources, Figure ES-6 shows the recent charges[3] for sewer service among various San Mateo and Santa Clara County agencies. The data indicates that the District's current sewer rates track the trend line along with the other SBSA member agencies (identified with blue squares in Figure ES-6). It should be noted that the other SBSA member agencies are faced with similar additional costs as the District. It is expected that these agencies will be also required to increase their rates substantially to cover their share of SBSA costs. Even with the projected rate increases, we would not expect the District's position among its neighbors to change significantly.

[3] In most cases, the proposed increases in sewer service charges are already adopted. In some cases, the final charge is pending adoption at the respective agency's public hearing.

Figure ES-6. Comparison of Monthly Residential Sewer Rates (FY2010/11)



ES.11.4 Connection Fee Evaluation

In addition to evaluating the District's rates, this Master Plan provides a preliminary review of the District's connection fee. The determination of the connection fee includes an evaluation of the value of the District's current proportional responsibility for SBSA assets. However, SBSA issues its own connection fee to assess the cost additional treatment capacity. In order to eliminate overlap between the District's connection fee assessment and the SBSA assessment, additional information is needed from SBSA regarding the calculation of its connection fee. This information was not available at the time the Master Plan was completed. Therefore, the connection fee information that is included in this Master Plan will be adjusted in the future to reflect new information as it becomes available.

The connection fee evaluation estimates the replacement cost of the District's linear assets (pipes), facilities (pump stations), and other assets (fleet, buildings, Flow Equalization Facility, cash equity, and land). The evaluation used as a baseline the values that were provided in the District's 2005 Sanitary Sewer Master Plan and Connection Fee Update (Freyer & Laureta, 2006). These values were then updated to include any known additions since June 30, 2006, as well as deletions related to retirements and replacements.

The connection fee evaluation added the new linear assets and facilities that are planned under the District's Capital Improvement Program, and spread the cost of these facilities to the total number of equivalent dwelling units (EDU) that are served by the District. The resulting connection fee establishes a proposed "buy-in" amount to assess to those that are adding connections or EDUs to the sewer system.

The current District residential connection fee is $2,549 per EDU. The current SBSA residential connection fee is $1,740 per EDU, for a total connection fee of $4,289 per EDU. Based on the updated connection fee evaluation, as presented in Chapter 11, the District's connection fee may increase to between $5,000 and $7,000 per EDU. This increase does not include any potential parallel increases to the SBSA connection fee. This value will be refined further after additional information is obtained from SBSA regarding the components of the SBSA fee.

CHAPTER 1
Introduction

This introductory chapter provides background information on the scope and objectives of the West Bay Sanitary District Wastewater Collection System Master Plan (Master Plan).

1.1 BACKGROUND AND PROJECT OBJECTIVES

In April 2010, the West Bay Sanitary District (District) retained West Yost Associates (West Yost) to complete a comprehensive revision to the District's wastewater collection system master plan and rate study. The study had multiple objectives, as follows:

- Evaluate system-wide flow characteristics to identify areas that may contribute to high inflow and infiltration (I&I);
- Assess the existing and buildout hydraulic capacity of the collection system;
- Evaluate pump station condition and potential replacement needs;
- Evaluate pipeline condition and replacement needs;
- Develop a prioritized capital improvement program (CIP) that strategically replaces or repairs facilities over time to provide sufficient capacity, maximize useful life, and reduce risk to the District; and
- Evaluate rates and connection fees needed to support this long-term program.

In addition to these objectives, the 2011 Master Plan provided updated information regarding flows and storage needs that was incorporated into a separate conveyance system planning effort underway by South Bayside System Authority (SBSA).

The West Yost team included Freyer & Laureta, Inc. (F&L) as a subconsultant to gather relevant data, assist with the pipeline project prioritization, and complete the pump station assessment and rehabilitation program. F&L has served as the District's Engineer for many years, and is very familiar with District staff and facilities. The team also included HF&H Consultants as a subconsultant to conduct the rate and connection fee analysis.

This chapter is the introductory chapter for the Collection System Master Plan Report (Report) that serves as a roadmap for both near-term and long-term sewer system infrastructure management.

1.2 REPORT ORGANIZATION

The Report comprises the following chapters. The sequence of chapters generally conforms to the tasks outlined in the scope of work for the project. This section describes the contents of each of the nine chapters and appendices.

1.2.1 Executive Summary

The Executive Summary provides a comprehensive overview of the Report contents and summarizes key aspects of each chapter.

1.2.2 Chapter 1 – Introduction

This introductory chapter provides background information on the scope and objectives of the Master Plan, and presents the contents and organization of the Report.

1.2.3 Chapter 2 – Existing Wastewater System

This chapter describes the District's existing service area, presents land uses for the various communities within the service area, describes existing facilities including pump stations, pipelines, and wet weather storage facilities, and discusses the relationship between the District and SBSA and its members. SBSA provides wastewater treatment for the District's flows, and for flows generated by the Cities of Redwood City, Belmont, and San Carlos.

1.2.4 Chapter 3 – System Flows

This chapter presents the methodology used to determine existing and future dry weather and wet weather wastewater flows for the purposes of collection system capacity modeling. This chapter also presents West Yost's analysis of contributions to system-wide inflow and infiltration that followed completion of the District's 2009/10 flow monitoring program.

1.2.5 Chapter 4 – Flow Monitoring and Inflow/Infiltration Analysis

This chapter summarizes contributions to system-wide inflow and infiltration based on results from the District's 2009/2010 flow monitoring program that was conducted by V&A Consultants (V&A).

1.2.6 Chapter 5 – Hydraulic Model Development

This chapter documents the tasks required to build and calibrate the Innovyze® InfoWorks™ CS hydraulic model. The hydraulic model is the primary analytical tool that was used to determine the flows and capacities of the District's major sewers, and to identify any potentially needed capacity improvements, including wet weather storage requirements.

1.2.7 Chapter 6 – Planning Criteria

This chapter documents the planning criteria used to calculate existing and future flows, and to assess whether any hydraulic deficiencies may occur in the collection system. These criteria are based on standard design criteria in use by the District, and modeled criteria that resulted from hydraulic model calibration as discussed in Chapter 4.

1.2.8 Chapter 7 – Capacity Analysis

This chapter presents the results of the existing and buildout system hydraulic capacity analyses of the District's wastewater collection system. The chapter presents the results of both analyses, identifies existing pipelines requiring capacity relief, and describes proposed capital improvement projects, including conceptual cost estimates and a discussion of initial project criticality.

1.2.9 Chapter 8 – Pipeline Condition Assessment and Capital Improvement Project

This chapter presents the District's near-term and long-term potential gravity sewer system repair, renewal, and replacement needs based on results from closed circuit television (CCTV) inspections, and system knowledge provided by District staff. Similar to the capacity discussion in Chapter 6, the chapter includes conceptual cost estimates and a discussion of initial project criticality.

1.2.10 Chapter 9 – Pump Station Rehabilitation Program

This chapter presents the District's near-term and long-term pump station and lift station repair, renewal, and replacement needs based on results from site investigations conducted by F&L and system knowledge provided by District staff. The chapter includes conceptual cost estimates and a discussion of initial project criticality.

1.2.11 Chapter 10 – Capital Improvement Program

This chapter consolidates recommendations presented in Chapters 6 through 8 and compiles the projects into a prioritized CIP that addresses the most potentially critical projects first, balances facility replacements to enable completion of a variety of projects in each year, and distributes cost to ensure an attainable and fundable long-term program. This chapter includes descriptive maps that also summarize findings and planning information for each project.

1.2.12 Chapter 11 – Financing Plan

This chapter presents evaluations completed by HF&H consultants to establish a funding plan to support the near-term and long-term capital improvement program that is outlined in Chapter 10. The study includes background and analyses completed to establish sewer service charges and connection fees needed to fund the cost of service to the District's ratepayers.

1.2.13 Appendices

The following appendices to this Wastewater Collection System Master Plan contain additional technical information and assumptions:

- Appendix A – V&A Flow Monitoring Report
- Appendix B – Flow Hydrograph
- Appendix C - Flow Meter Diurnal Curves
- Appendix D - Dry Weather Flow Calibration Hydrographs
- Appendix E - Wet Weather Flow Calibration Hydrographs
- Appendix F - Detailed Cost Estimates for Capacity Improvement Projects
- Appendix G - Rehabilitiation and Replacement Project Figures
- Appendix H – Detailed Cost Estimateas for Rehabilitation Projects
- Appendix I – CCTV Inspection Report

- Appendix J – Pump Stations Technical Memorandum, Freyer & Laureta, Inc., May 2011
- Appendix K – Rate Model & Connection Fee Model

1.3 ACRONYMS AND ABBREVIATIONS

The following acronyms and abbreviations have been used throughout this Report to improve document clarity and readability.

ADWF	Average Dry Weather Flow
BWF	Base Wastewater Flow
CCTV	Closed Circuit Television
CIP	Capital Improvement Program
CIPP	Cured in Place Pipe
CIWQS	California Integrated Water Quality System
CMP	Corrugated Metal Pipe
County	County of San Mateo
District	West Bay Sanitary District
DU	Dwelling Unit
DWF	Dry Weather Flow
EDU	Equivalent Dwelling Unit
ENR CCI	Engineering News Record Construction Cost Index
F&L	Freyer & Laureta, Inc.
FEF	Flow Equalization Facility
fps	Feet Per Second
GBA	GBA Master Series
GIS	Geographical Information System
GPAD	Gallons Per Acre Per Day
gpcpd	Gallons Per Capita Per Day
gpd	Gallons Per Day
gpd-idm	Gallons Per Day Per Inch-Diameter-Mile
gpm	Gallons Per Minute
GWI	Groundwater Infiltration
HDD	Horizontal Direction Drilling
HDPE	High Density Polyethelyne
HGL	Hydraulic Grade Line
I&I	Inflow and Infiltration
ID	Identification Numbers

Master Plan	West Bay Sanitary District Wastewater Collection System 2011 Master Plan
MCC	Motor Control Center
Menlo Park	City of Menlo Park
mgd	Million Gallons Per Day
MPPS	Menlo Park Pump Station
NASSCO	National Association of Sewer Service Companies
NOAA	National Oceanic and Atmospheric Administration
NPA	North Palo Alto
PACP	Pipeline Assessment and Certification Program
PAYGo	Pay-as-you-go
PVC	Polyvinyl Chloride
Q_A	Average Daily Dry Weather Flow
Q_{PDWF}	Peak Hourly Dry Weather Flow
Q_{PWWF}	Peak Wet Weather Flow
R&R	Rehabilitation and Replacement
Report	Collection System Master Plan Report
RDII	Rainfall-Dependent Inflow and Infiltration
SBSA	South Bayside System Authority
SCS	Soil Conservation Service (now Natural Resource Conservation Service)
SSO	Sanitary Sewer Overflow
SUH	Synthetic Unit Hydrograph
SWRCB	State Water Resources Control Board
TCE	Temporary Construction Easement
V&A	V&A Consulting Engineers
VA	Veteran's Affairs
VCP	Vitrified Clay Pipe
West Yost	West Yost Associates
WWF	Wet Weather Flow

CHAPTER 10
Capital Improvement Program

Chapter 10 presents the recommended CIP for the District's sewer collection system. The project recommendations, configurations, and conceptual costs that are presented in this chapter were described previously in Chapters 7 (capacity improvements), 8 (pipeline improvements), and 9 (pump station improvements). This chapter summarizes and presents a consolidated list of projects by proposed priority and implementation schedule. The recommended CIP identifies the improvements at a master planning level, and does not constitute conceptual or preliminary design of these improvements. Subsequent alignment studies and preliminary designs are recommended to finalize pipeline configuration, pump station needs, and to determine the final sizes, locations, and details of the proposed improvements.

The recommendations from Chapters 7 through 10, including project descriptions, project data sheets, costs, and the proposed schedule for implementation have been summarized and consolidated in a single CIP summary document that is published separately from this Master Plan.

This chapter is organized as follows:

- Basis for Capital Improvement Program Development, and
- Proposed CIP.

10.1 BASIS FOR CAPITAL IMPROVEMENT PROGRAM DEVELOPMENT

The CIP was developed to create a fundable program that addresses the District's competing needs for capacity improvement and reliability. The following criteria were used to prioritize the various projects and develop a timeline for implementation.

1. Projects to Eliminate or Reduce SSOs. The CIP prioritizes and schedules completion of projects that reduce or eliminate capacity-related SSOs from the 10-year, 6-hour design storm.
2. Projects to Reduce Maintenance-Related SSOs. The District has an ongoing, effective program to rehabilitate portions of the pipeline network with likely structural defects or maintenance issues . This program is intended to prolong system useful life and reduce, over time, the volume of wet weather inflow and infiltration that enters the system through structural defects.
3. Projects to Maintain Pump Station Reliability. Similar to the pipeline replacement program, the District conducts ongoing maintenance of its pumping facilities in order to maximize facility useful life and reliability. The CIP includes the significant planned upgrades that are recommended to be implemented over time in addition to routine maintenance.

4. Distribution of Capital Costs. In addition to meeting the above requirements, the District established an implementation schedule for the recommended projects that spreads costs equitably across the CIP timeframe. During the first four implementation years, total cost is approximately $3.5 million per year. For each of the next three years, assuming rates can be raised sufficiently, total cost is approximately $3.8 million per year. For each of the final three years of the ten-year program, again assuming rates can be raised sufficiently, total cost is approximately $4 million per year. By maintaining this consistent and fundable implementation schedule, the District will maximize the success of completing the planned improvements in the proposed timeline.

10.2 PROPOSED CIP

Table 10-1 presents the proposed CIP, which begins implementation in Fiscal Year 2011/12 and extends into Fiscal Year 2020/21. The CIP is intended to be an evolving document that is adjusted as needed to address future issues and priorities that may arise after completion of the Master Plan. The most critical components of the CIP are summarized below. Additional information on the individual projects, including detailed cost estimates, can be found in Chapters 7 through 9 of this report. The total estimated CIP cost is $46.3 million, to be implemented over 10+ years.

10.2.1 Priority Capacity Improvement Projects

The CIP includes five recommended Capacity Improvement Projects that address potential capacity-related SSOs from a 10-year, 6-hour design storm. These projects are planned for implementation starting in Fiscal Year 2011/12 through Fiscal Year 2018/19. The projects would add 1.6 miles of diversion pipe to convey peak wet weather flows through the northwest quadrant of the service area. The projects also would include replacement of additional gravity sewer pipe with larger diameter pipe to relieve bottlenecks that are associated with model-predicted SSOs that may result from the design storm.

The total estimated combined cost of the priority capacity improvement projects would be $7.7 million.

10.2.2 Priority Rehabilitation and Replacement Projects

The following projects are considered a priority for the District, and are scheduled for completion by the end of FY 2012 due to their assigned importance. During the development of this Master Plan, the District initiated the design of these projects. The actual configuration of these projects may have changed from the description provided in this Master Plan.

- Fletcher Drive Sewer Replacement[1]. This project replaces a sewer pipeline that has heavy root intrusion and structural defects. Replacement of this pipe with a new PVC pipeline is a high priority project for the District.
- Atherton Avenue Sewer Replacement. This project includes a combination of point repairs and pipe segment replacements to address structural defects found in the 8- and 10-inch diameter sewers within the lower reach of Atherton Avenue. This project was accelerated in the CIP in order to complete pipeline construction prior to implementation of planned pavement replacement on this roadway by the City of Menlo Park.
- Willow Avenue Sewer Rehabilitation Project. This project includes a combination of point repairs and pipe segment replacements for the 6- and 8-inch pipelines within Willow Avenue. This project was prioritized by the District to address historic maintenance-related SSOs from these pipelines.
- Ladera Outfall Sewer Rehabilitation Project. This project includes a combination of point repairs and pipe segment replacements to address structural defects. In addition, root intrusion within the upper section of the Ladera Outfall has made maintenance of portions of the pipeline is difficult due to limited accessibility. The District's design includes cured-in-place pipe liner in the upper portion of the pipe to abate root intrusion, and a spot repair in the lower reach of the alignment. This project was elevated in priority by the District in order to reduce historic maintenance-related SSOs.

  During the completion of the 2011 Master Plan, the District completed designs for and bid the four rehabilitation projects described above, and also the Corte Madera Forcemain project that is described below. Bids received were substantially lower than planned costs. As a result, the District provided an updated total cost estimate of $2.1 million for the four rehabilitation projects and single forcemain project. The District's costs include design, construction, and anticipated additional costs that could be incurred during construction. This updated cost was included in the CIP in lieu of previously-estimated planning level costs.

10.2.3 Priority Pump Station and Forcemain Projects

The following projects are needed to address both capacity and reliability issues at two pump stations that are located in Portola Valley. The Corte Madera forcemain project is under design by the District, and is scheduled for completion by the end of FY 2012. The Sausal Vista PS project is scheduled for completion in FY 2013/14. These pump stations operate in series, and the Corte Madera pump station is reported to have nearly exceeded capacity during the 2009/2010 wet weather period.

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[1] During completion of the 2011 Master Plan, the District removed the Fletcher Drive project from the CIP because it is already in progress and added a project to rehabilitate a pipeline on Haven Avenue. Although this Haven Avenue project is not described in this chapter, the CIP cost reflects this adjustment.

- Corte Madera Forcemain project. This station is located adjacent to a waterway, and within a residential neighborhood. This pump station likely receives a high level of I&I, and the existing forcemain may not be sufficiently sized. As a result, the existing pumps may not be able to convey maximum design flows. In order to increase pumping capacity, this project would replace the existing forcemain with a larger diameter pipeline. It is recommended that the District confirm peak wet weather flow rates before installing the replacement pipeline.

- Sausal Vista Pump Station project. This station is operates in series with the Corte Madera pump station, and is the District's second oldest station. The pump station assessment that is discussed further in Chapter 9 discusses two options for rehabilitation or replacement of the Sausal Vista pump station. The selected alternative may impact planned improvements for the Corte Madera pump station.

The total estimated cost of the Corte Madera Forcemain project is included with the priority pipeline cost estimates discussed above. The estimated cost for the Sausal Vista Pump Station project is $1.3 million.

10.2.4 Ongoing CIP Projects

The remainder of the CIP includes 19 additional pipeline rehabilitation or replacement projects, three pump station improvement projects, and six additional capacity improvement projects that are needed over time to reduce likely surcharge conditions during the design storm. The total cost of these projects is approximately $36.6 million. The projects are listed in the CIP generally in priority order, based on information known as of the date of this Master Plan. However, the sequence of implementation is flexible, and it is expected that the project list and implementation schedule will change in order to address new system information or priorities.

Table 10-1. West Bay Sanitary District Example Capital Improvement Project Implementation Plan (Notes 1 through 3)

R&R Project #	Project Name	Estimated Cost	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	Future (listed projects only)	Check
1	Fletcher (Replaced by Haven)													
2	Atherton													
3	Ladera Outfall	$2,100,000	$2,100,000											2,100,000
4	Willow													
5	Corte Madera Forcemain													
6	Sausal Vista PS and Forcemain	$1,298,000			1,298,000									1,298,000
7	North Palo Alto Concrete	$1,487,000			1,487,000									1,487,000
8	Fair Oaks	$3,025,000					302,500	2,722,500						3,025,000
9	Santa Cruz	$1,004,000							1,004,000					1,004,000
10	Roble	$2,630,000							2,630,000					2,630,000
11	Stevenson	$1,155,000								1,155,000				1,155,000
12	Elena	$1,802,000									901,000	901,000		1,802,000
13	Carlton-Madera Easements	$2,504,000									1,252,000	1,252,000		2,504,000
14	College Park North	$2,213,000											2,213,000	2,213,000
15	Oak Grove	$1,009,000	243,000								766,000			1,009,000
16	Encinal A	$1,174,800	763,800								411,000			1,174,800
16a	Miscellaneous Pipe Segment Repair	$590,200	590,200											590,200
17	Oak Knoll	$769,000								769,000				769,000
18	Encinal B	$2,170,000											2,170,000	2,170,000
19	Bayfront Expressway	$3,000,000		3,000,000										3,000,000
20	Menalto Easements	$788,000										788,000		788,000
21	Berkeley	$1,213,000								1,213,000				1,213,000
22	Frederick	$209,000		209,000										209,000
23	Haven	$205,000						205,000						205,000
24	Suburban Park	$250,000		250,000										250,000
25	Willow Road Pump Station	$256,000						256,000						256,000
26	Hamilton Henderson Pump Station	$154,000					154,000							154,000
27	Stowe Lane Pump Station	$1,003,000			100,300	902,700								1,003,000
	Subtotal R&R	$32,009,000	3,697,000	3,459,000	2,885,300	902,700	456,500	3,183,500	3,634,000	3,137,000	3,330,000	2,941,000	4,383,000	32,009,000
Capacity Project #	**Project Name**	**Estimated Cost**	**2011/12**	**2012/13**	**2013/14**	**2014/15**	**2015/16**	**2016/17**	**2017/18**	**2018/19**	**2019/20**	**2020/21**	**Future (listed projects only)**	**Check**
1	James Avenue Diversion	$5,365,000			536,500	2,414,250	2,414,250							5,365,000
2	Lower Ringwood	$957,000					957,000							957,000
3	Lower Valparaiso Avenue	$444,000						444,000						444,000
4	Lower Santa Cruz-Avy	$667,000								667,000				667,000
5	Upper Willow PS Gravity Main	$226,000							226,000					226,000
6	Upper Ringwood	$234,000								234,000				234,000
7	Upper Valparaiso Avenue	$182,000						182,000						182,000
8	Upper Santa Cruz-Avy	$519,000									519,000			519,000
9	Middlefield at Fair Oaks	$502,000									251,000	251,000		502,000
10	Atherton Avenue East	$4,209,000										209,000	4,000,000	4,209,000
11	Lower Willow PS Gravity Main	$991,000										495,500	495,500	991,000
	Subtotal Capacity	$14,296,000	0	0	536,500	2,414,250	3,371,250	626,000	226,000	901,000	770,000	955,500	4,495,500	14,296,000
	Total CIP	**$46,305,000**	**3,697,000**	**3,459,000**	**3,421,800**	**3,316,950**	**3,827,750**	**3,809,500**	**3,860,000**	**4,038,000**	**4,100,000**	**3,896,500**	**8,878,500**	**46,305,000**

Note 1: Implementation schedule beginning in 2012/13 and beyond will be revised routinely based on new system information, and as needed to accommodate unexpected infrastructure repair projects.

Note 2: The District has replaced the Fletcher Project described in Chapter 8 with a repair on Haven Avenue, completed design, and received bids for Projects 1 through 5. Bids received were unexpectedly low. Based on bids received, the District has estimated a total cost for these projects, including design, construction contingency and engineering support during construction, of $2.1M.

Note 3: The District has scheduled a portion of Projects 15 and 16 for completion in FY11/12. Pipes will be rehabilitated using CIPP lining in lieu of pipe replacement. Based on bids received, the District has estimated a total cost for the accelerated projects, including design, construction contingency and engineering support during construction, of $1M. In addition, the District has budgeted $590k (including all contingencies) for miscellaneous pipeline rehabilitation projects that are not in the Master Plan. These projects are included in this Table as No. 16a.

CHAPTER 11
Financial Plan

11.1 TASK BACKGROUND AND SUMMARY

This chapter presents a financial plan for the District that incorporates the capital improvements identified in this Master Plan. The financial plan was developed by HF&H Consultants, as a subconsultant to West Yost Associates. The financial plan comprises projected operating and capital expenses for the District, including its share of SBSA costs, projected revenues from the District's sewer service charges, and projected District reserves for the period from FY 2010/11 to FY 2019/20. The results of the financial plan indicate the annual increases in sewer service rates and charges that are projected to adequately fund the District's expenses and maintain adequate reserves. The financial plan also introduces work in progress to establish the District's connection fee. Detailed spreadsheets comprising the rate model and connection fee model are included in Appendix K.

11.1.1 Regional Context

The financial plan is based on the following information received from the District and SBSA regarding EDUs. The District provides wastewater collection and conveyance services to approximately 32,000 residential and non-residential EDUs through a system of pipelines and pump stations that transport their wastewater to the SBSA for treatment and discharge into San Francisco Bay. SBSA is a Joint Powers Authority that provides wastewater treatment services to the Cities of Redwood City, San Carlos, and Belmont. Table 11-1 shows the distribution of contracted capacity to each member agency.

Table 11-1. SBSA Member Agencies

Agency	Percent Allocation
West Bay Sanitary District	23.70%
City of Redwood City	53.70%
City of San Carlos	13.70%
City of Belmont	8.80%
Total	100.00%

Source: SBSA Financial Plan Update 2/24/2011

SBSA's operating and capital costs that are common to all the member agencies are allocated based on their proportionate shares of the total EDUs. Facilities that are not shared by all member agencies (*e.g.*, pump stations and mains) are allocated based on each agency's use of the facilities.

11.1.2 Existing Sewer Rates

The District charges sewer customers annually on the tax rolls, which is a common practice for billing for sewer service. Billing on the tax rolls is less expensive than it would be if the District issued its own bills and it allows the District to easily levy liens for nonpayment. Even though the District bills through the tax rolls, its sewer service charges are not a tax or assessment. Unlike taxes or assessments, which are based on land-related characteristics such as assessed value or parcel size, the District's sewer charges are a form of service fee or charge that is proportionate to the cost of providing sewer service.

Historically, the District's sewer service charges have increased primarily in response to increases in SBSA's treatment charges, but also to maintain the level of service required to continue to safely and reliably meet the sewer service needs of the District ratepayers. More recently, the District has also been faced with additional capital improvements to renew and replace aging District infrastructure, in addition to significant increases in SBSA capital improvement needs.

11.1.3 Summary

The results of the financial plan indicate the need for the following adjustments to future rates:

Year	Rate Increase
FY 2011/12	6%
FY 2012/13	20%
FY 2013/14	17%
FY 2014/15	12%
FY 2015/16	4%
FY 2016/17	3%
FY 2017/18	3%
FY 2018/19	3%
FY 2019/20	3%

The 6% increase for FY 2011/12 was approved in the District's prior rate-setting process, is scheduled for adoption, and is integrated with the financial plan. The subsequent increases are predicated on the current assumptions and available information. The substantial increases for FY 2012/13 through FY 2014/15 are recommended primarily because of the need to fund the District's share of SBSA's debt service as well as to build the District's reserves. Because the SBSA debt service projection continues to change over time, prior to adopting future rate increases, it is recommended that the District update these assumptions to reflect the most current information available from SBSA.

11.2 REVENUE REQUIREMENT PROJECTIONS

A spreadsheet model was developed to derive revenue requirements for FY 2012/13 through FY 2019/20. However, the actual recommended planning timeframe for rate setting is no greater than 5 years. The District's O&M budget for FY 2011/12 served as the starting point for projecting the District's expenses and revenues. The escalation factors summarized in Table 11-2 were incorporated in the model for projecting expense and revenues.

Table 11-2. Key Modeling Assumptions

	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	Source
General inflation	Per Budget	Per Budget	4.00%	4.00%	4.00%	4.00%	4.00%	4.00%	4.00%	4.00%	WBSD Budget
Utilities	Per Budget	Per Budget	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	Estimate
Salaries & Benefits	Per Budget	Per Budget	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	Estimate
SBSA O&M Increase	Per Budget	Per Budget	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	SBSA Budget
Interest on Earnings	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	WBSD Budget
Non-rate Revenues	Per Budget	Per Budget	1.00%	1.00%	1.00%	1.00%	1.00%	1.00%	1.00%	1.00%	Estimate
% Growth in Accounts & Demand	Per Budget	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	0.50%	Estimate
Construction Cost Inflation	Per Budget	Per Budget	3.64%	3.64%	3.64%	3.64%	3.64%	3.64%	3.64%	3.64%	ENR CCI; 5 YR Avg

11.2.1 District O&M Expenses

The District's O&M expenses are projected to increase by a few percent per year from approximately $5.6 million to $5.7 million annually. Annual increases are no greater than the estimated rate of inflation for most recurring expenses.

11.2.2 District Capital Expenses

The District's capital expenses are summarized by category in Table 11-3. On average, the District expects to spend approximately $5.5 million annually on these projects, the majority of which (approximately $3.8 million per year) concerns Master Plan projects that are described further in Chapter 10 of this Master Plan. The Master Plan costs have been escalated to estimate inflation over time. The remaining capital expenses comprise various ongoing administrative and other capital expenditures.

Table 11-3. CIP Summary

	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20
Administration	$465,000	$165,000	$169,950	$175,049	$180,300	$185,709	$191,280	$197,019	$202,929	$209,017
Collection Facilities	$387,800	$327,500	$343,875	$361,069	$379,122	$398,078	$417,982	$438,881	$460,825	$483,867
Subsurface Lines										
Proposed (Master Plan)	$0	$3,630,000	$3,584,908	$3,668,459	$4,218,782	$4,406,727	$4,571,915	$5,038,854	$5,355,844	$5,512,194
Other	$4,090,000	$654,000	$677,806	$702,478	$728,048	$754,549	$782,014	$810,480	$839,981	$870,557
Environmental Review	$10,000	$10,000	$10,364	$10,741	$11,132	$11,537	$11,957	$12,393	$12,844	$13,311
Manhole Raising	$50,000	$100,000	$103,640	$107,412	$111,322	$115,374	$119,574	$123,927	$128,437	$133,113
Allow. For Unanticipated Cap Ex	$50,000	$75,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000

The District plans to fund all of these capital improvements on a pay-as-you-go (PAYGo) basis without issuing debt, which continues the District's historical practice. The funding for the District's capital expenses will take the form of contributions of rate revenue to the Capital reserve.

11.2.3 Contributions to District Reserves

In addition to covering annual expenses, sewer service charges need to generate revenue to maintain adequate operation and capital reserves. To determine what constitutes adequate reserve amounts, the reserve balance was subdivided into Operations, Capital, and Emergency Reserves. In this way, it is possible to set recommended target balances for each purpose.

11.2.3.1 Operations Reserve Minimum Balance

The Operations Reserve provides working capital for monthly O&M expenses. Because of the nine-month lag between sewer service charge payments from the County tax assessor, the minimum Operations Reserve balance is reasonably set equal to five months of O&M expenses. If this minimum balance is maintained, the District should be able to fund its monthly operations cash flow over this extended period without relying on the Capital Reserve for a short-term loan.

Maintaining the minimum balance for the Operations Reserve is recommended as the highest priority for the District's three reserves.

11.2.3.2 Emergency Reserve Target Balance

The target balances for the Operations and Capital Reserves are sufficient to provide working capital on an ongoing basis, but do not provide for unforeseen contingencies such as emergencies. Should an emergency strike, the District cannot suddenly raise rates to generate additional funds due to state law requirements for such rate increases (e.g., Proposition 218). Therefore, an Emergency Reserve of $5.0 million is recommended for emergencies. With such a reserve, the District would have funds on hand to take immediate remedial steps without waiting to procure a loan or issue bonds.

Maintaining the target balance for the Emergency Reserve is recommended as the second highest priority after meeting the minimum balance for the Operations Reserve. The Emergency Reserve can be used for funding capital projects until the Capital Reserve is fully funded.

11.2.3.3 Capital Reserve Target Balance

The Capital Reserve provides liquidity to fund construction for projects that are funded on a PAYGo basis (as opposed to those that are funded from debt). With adequate capital reserves, the District is able to pay contractors without encroaching on the Operations Reserve. The target balance for the Capital Reserve depends on the level of construction. A minimum balance equal to the average annual construction costs (approximately $3.8 million) was used for determining an appropriate and reasonable target balance.

Maintaining the target balance for the Capital Reserve is recommended to be a lower priority after meeting the minimum balances for the Operations and Emergency Reserves.

To provide additional protection until all reserves are fully funded, the cost of a line of credit is included in the revenue recommendations.

11.2.4 SBSA Expenses

SBSA's treatment charge currently is 52% of the District's total revenue requirement, and is the District's single largest expense. The District's charge is allocated in proportion to the number of its EDUs compared with the other SBSA member agencies. SBSA's cost has recently increased significantly to fund the debt service on the series of bonds that have been issued to fund the rehabilitation of its interceptors, pump stations, and wastewater treatment plant. By the projected completion of the project, SBSA's debt service allocation to the District will equal $7.7 million, an increase of $6.8 million over the current debt service.

11.2.5 Total Revenue Requirements

The foregoing modeling assumptions lead to the projected revenue requirements shown in Figure 11-1 and Table 11-4. Figure 11-1 shows that:

- There will be very little increase projected in the District's own O&M expenses.
- The District's funding for capital improvements will gradually increase.
- The projected SBSA O&M expenses increase gradually; although current estimates may not reflect future O&M after SBSA completes its capital improvement program.
- SBSA's capital costs increase significantly as SBSA issues bonds to construct its capital improvement program.

Unlike the District's local costs, SBSA costs are largely beyond the District's control.

Table 11-1 contains the same data as Figure 11-1 in tabular form.

Figure 11-1. Projected Revenue Requirements

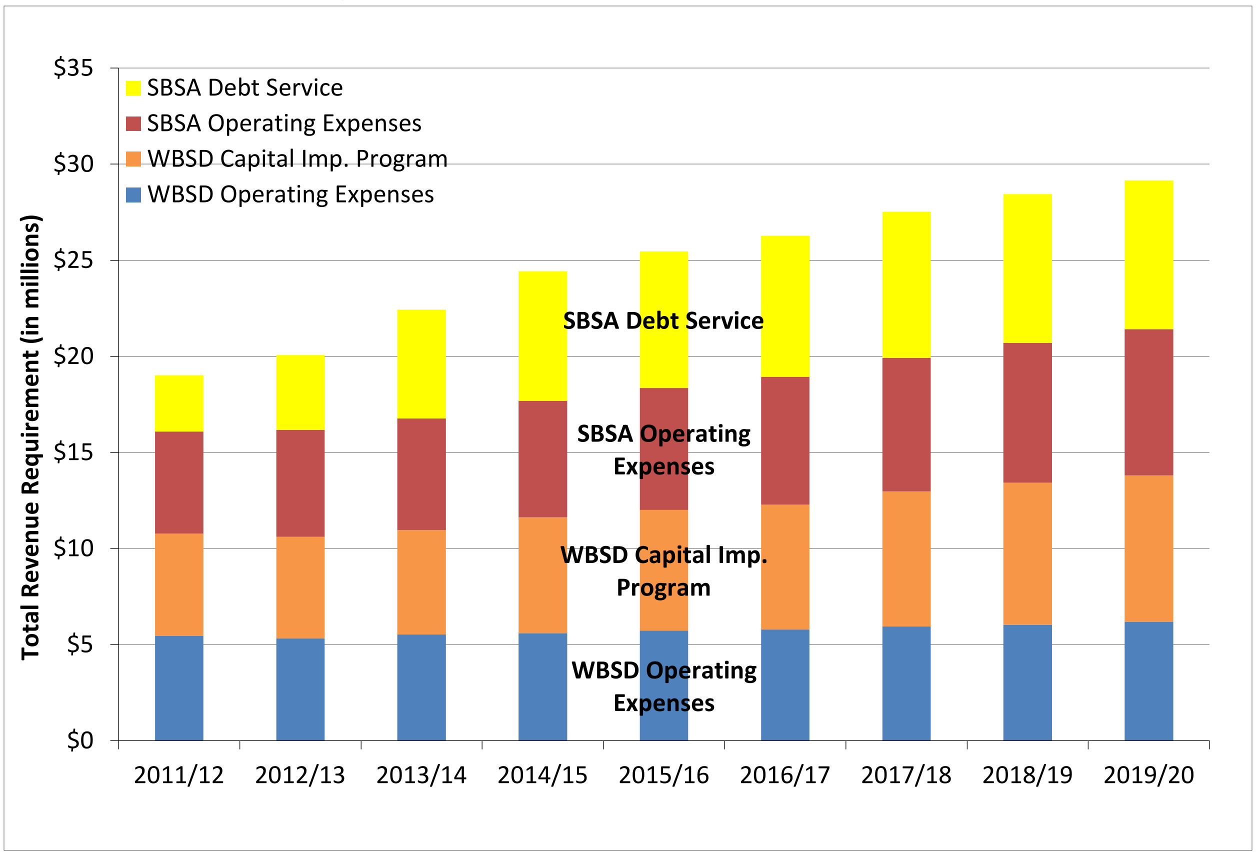


Table 11-4. Projected Revenue Requirements

	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20
WBSD Operating Expenses	$5,294,029	$5,472,142	$5,326,470	$5,538,462	$5,597,394	$5,739,078	$5,804,305	$5,953,295	$6,026,153	$6,182,985
SBSA Operating Expenses	$5,445,270	$5,306,322	$5,547,356	$5,800,443	$6,066,183	$6,345,211	$6,638,190	$6,945,818	$7,268,828	$7,607,988
SBSA Debt Service	$1,141,228	$2,933,054	$3,895,212	$5,664,392	$6,736,640	$7,105,990	$7,334,390	$7,601,724	$7,752,402	$7,740,896
WBSD Capital Imp. Program	$6,552,800	$5,311,500	$5,290,542	$5,425,208	$6,028,706	$6,271,975	$6,494,723	$7,021,553	$7,400,861	$7,622,058

Figure 11-2 shows the annual increases and Figure 11-3 shows the anticipated cumulative increases in revenue requirements attributable to the District and SBSA. SBSA's share of the increases would be greatest in the first four years because of the issuance of bonds for its capital improvement program. The District's share of the revenue requirement increases is also greatest in the next four years as the District builds up its reserves.

Figure 11-2. Annual Increases in Revenue Requirements

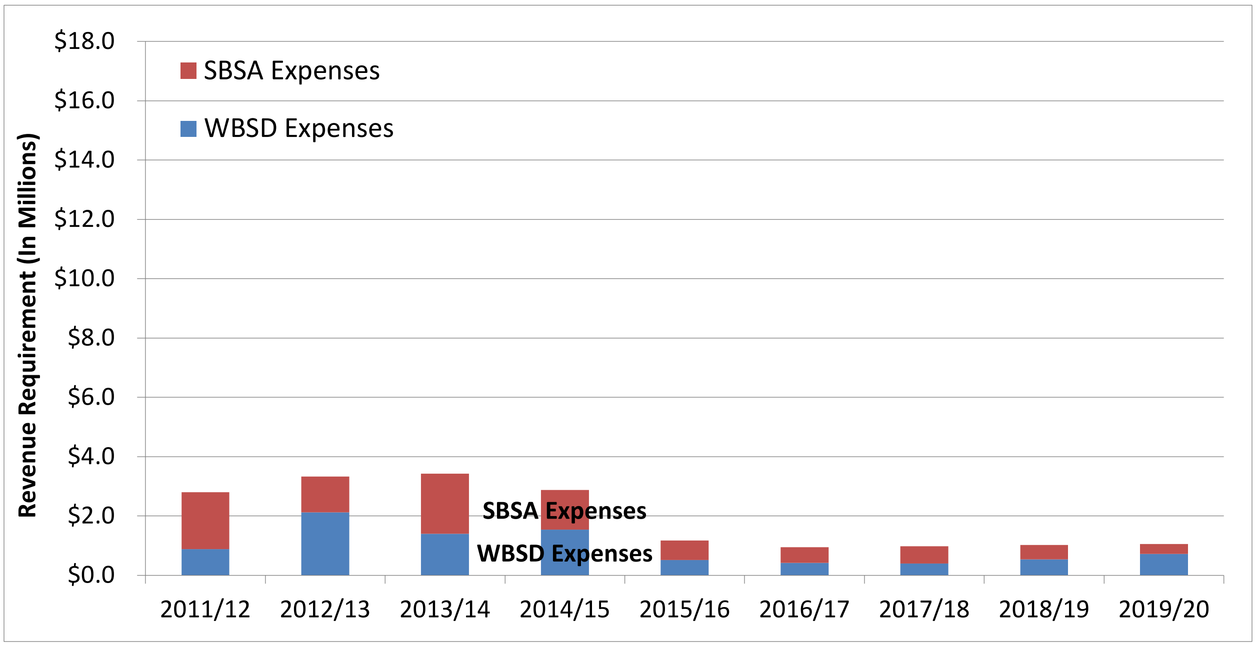
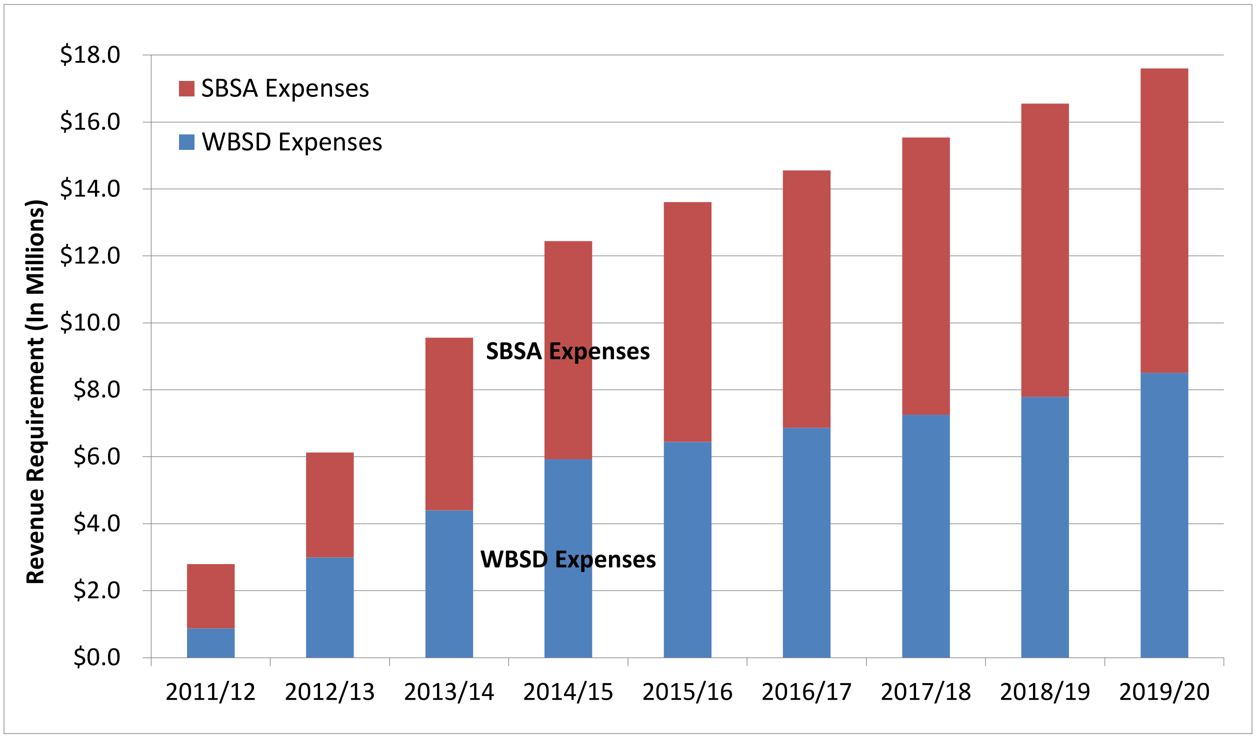


Figure 11-3. Cumulative Increases in Revenue Requirements



11.3 PROJECTED RATE INCREASES

11.3.1 Revenue from Existing Rates

Revenue from the District's rates included the 6.0% increase that was previously approved for adoption in FY 2011/12. Subsequent revenue from rates assumed 0.5% annual growth. This revenue projection serves as the baseline for comparison with the projected revenue requirements to determine future revenue increases that are recommended to cover the increased revenue requirements.

11.3.2 Revenue and Rate Increases

Current rates cannot support the projected revenue requirements shown in Figure 11-1. The revenue increases and corresponding sewer service charges that are recommended are summarized in Table 11-5. The revenue increase represents how much more revenue is needed compared to existing rates.

Table 11-5. Projected Revenue and Rate Increases

	CURRENT	ADOPTED								
	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20
Sewer Service Charge per EDU	$650	$690	$828	$969	$1,085	$1,128	$1,162	$1,197	$1,233	$1,270
Annual Increase in Charge		$40	$138	$141	$116	$43	$34	$35	$36	$37
Annual Increase		6%	20%	17%	12%	4%	3%	3%	3%	3%
Cumulative Increase			27%	49%	67%	73%	79%	84%	89%	95%

11.3.3 Fund Balance

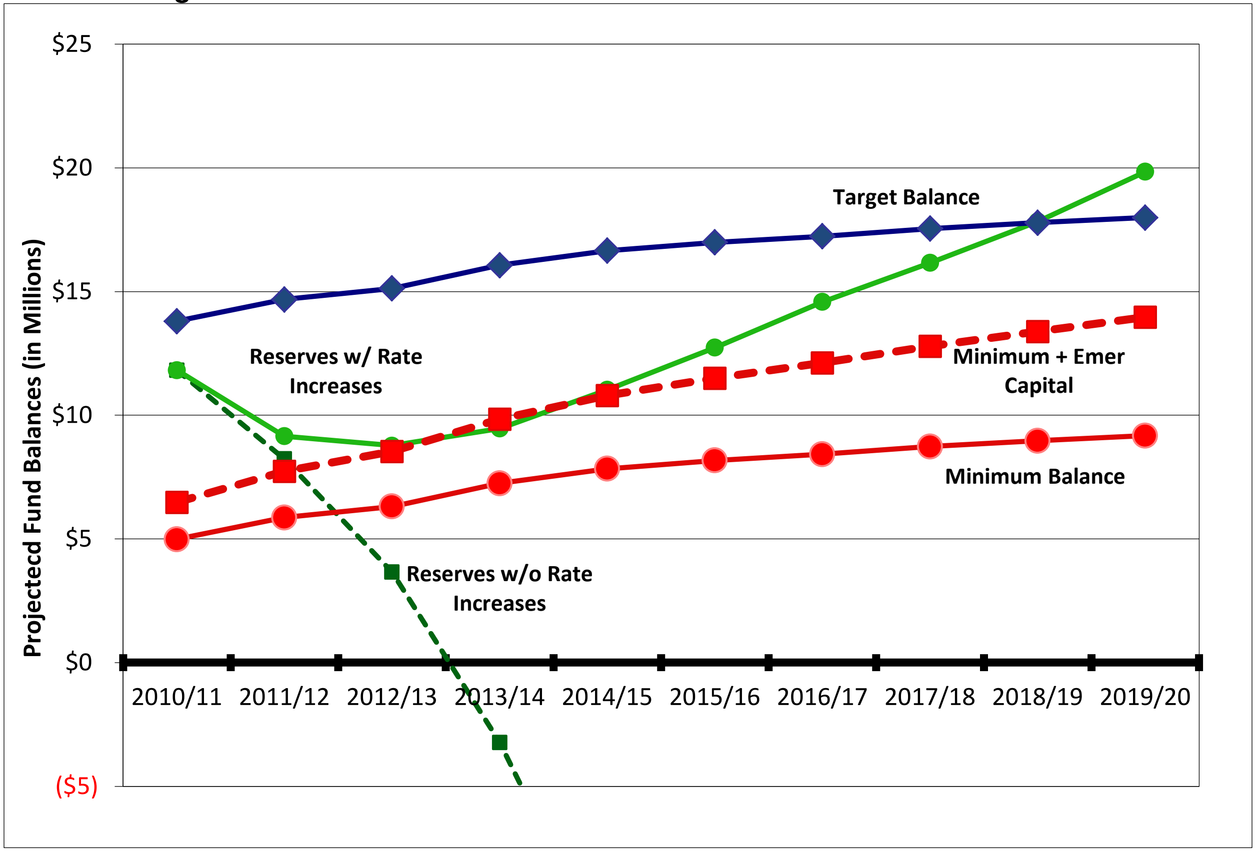
Figure 11-4 shows the projected fund balances with the rate increases (solid green line) and without the rate increases (dashed green line). Although the projections show straight lines between years, the fund balance will fluctuate down substantially during each year. In other words, the reserves are actively drawn on at all times but only periodically added to when payments are received from the County. The reserves are not simply accumulated without being used.

By June 30, 2011, the projected fund balance was nearly zero. Clearly, the District would not be able to continue expenditures that would result in a negative fund balance. The dashed green line represents the level of funding that would be required from the General Fund to support the projected expenditures. More likely, the District would have to severely curtail expenditures if rates are not increased.

In subsequent years, the fund balances are projected to be below the minimum balance even with rate increases, which could mean that some mid-year funding from the General Fund will be needed until revenue is received from the County. The sewer service charges were recommended to be increased so that the resulting fund balance eventually exceeds the minimum required balance (red line) in subsequent years. This is essential for an enterprise fund like the Sewer Fund to be self-sufficient. At the point that the fund balance exceeds the minimum balance, the Sewer Fund should not need to rely on the General Fund.

The sewer service charges were also increased so that the fund balance is trending toward the target balance (blue line). However, the proposed rate increases may not be sufficient to achieve the target balance by FY 2015/16. Until the upper target balance is met, funding the CIP will call for careful cash flow management. Once the target balance is met by approximately FY 2017/18, the Sewer Fund will have sufficient liquidity to fund operating and capital needs, but should not be regarded as being amply endowed. Additional funding that can be accumulated above the target balance will provide the District with a contingency for emergencies or other unanticipated events.

Figure 11-4. Fund Balance With and Without Increased Rate Revenue

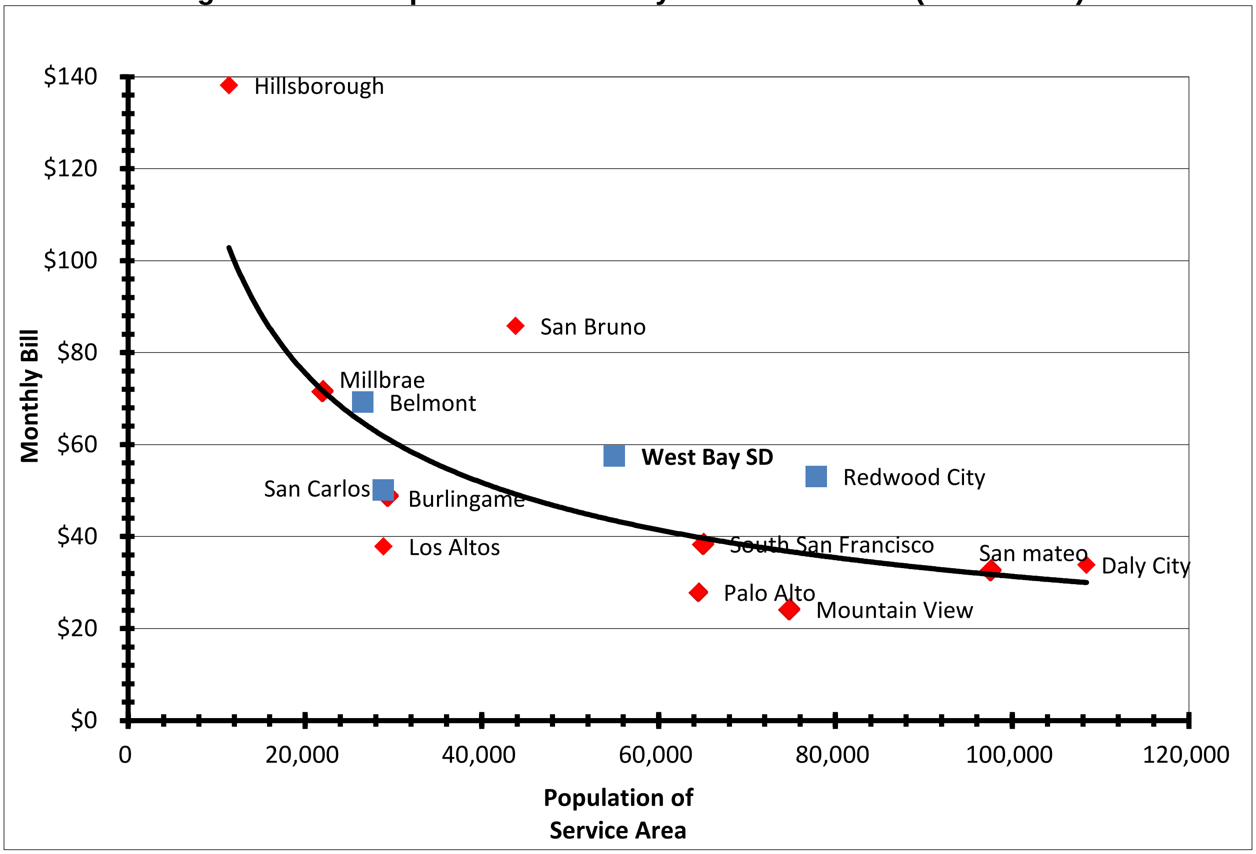


11.3.4 Comparison of Recent and Proposed Sewer Costs

Based on available sources, Figure 11-5 shows the recent charges[1] for sewer service among various San Mateo and Santa Clara County agencies. Figure 11-5 indicates that the District's current sewer rates track the trend line along with the other SBSA member agencies (identified with blue squares in Figure 11-5). It should be noted that the other SBSA member agencies are faced with similar additional costs as the District. It is expected that these agencies will also be required to increase their rates substantially to cover their share of SBSA costs. Even with the projected rate increases, we would not expect the District's position among its neighbors to change significantly.

[1] In most cases, the proposed increases in sewer service charges are already adopted. In some cases, the final charge is pending adoption at the respective agency's public hearing.

Figure 11-5. Comparison of Monthly Residential Bills (FY 2010/11)



11.4 NEXT STEPS – RATE STUDY

The District's current rates have been reasonably set to fund its historic expenses. Going forward, rate increases are recommended to provide continued funding. The District currently has a 6% increase planned for adoption in FY 2011/12. Rates for subsequent years have been projected in this financial plan that are based on a number of assumptions and information that will require review prior to adopting any future rate increases. For present purposes, the rate increases indicated in the financial plan provide a preview of the increases that may eventually be required.

For rate increases after FY 2011/12, the District is advised to update the financial planning model in conjunction with an update to the capital improvement program and associated O&M. A critical area for consideration is SBSA's capital costs, which are dependent on the rate with which it makes progress with its capital improvement program. Certain conditions should be considered in updating the financial plan to determine the appropriate rate increases:

- The number of years of future rates to adopt for rate increases may be affected by a new law (Government Code Section 53755 *et seq*.) that could require agencies to issue Proposition 218 notices every time rates are increased. Heretofore, agencies including the District have issued a single notice to rate payers at the beginning of multi-year rate increases.

- Potential modifications of the District's rate structure for multifamily and commercial customers. Single family rates, which are flat charges per account, will probably not be converted to flow-based rates because of the difficulty posed in acquiring water billing data from five or six water companies that provide water service within the District's service area.

In other respects, the District should expect to follow its past precedent in setting rates in compliance with the District's policy and the governing law.

11.5 FUTURE CONNECTION FEES

In addition to evaluating the District's rates, this Master Plan provides a preliminary review of the District's sewer connection fee. The determination of the connection fee included an evaluation of the value of the District's current proportional responsibility for SBSA assets. However, SBSA issues its own connection fee to charge for the cost of additional treatment capacity. In order to eliminate overlap between the District's connection fee and the SBSA connection fee, additional information is needed from SBSA regarding the calculation of its connection fee. This information was not available at the time the Master Plan was completed. Therefore, the connection fee information that is included in this Master Plan should be adjusted in the future to reflect new SBSA information as it becomes available.

The connection fee evaluation estimates the replacement cost of the District's linear assets (pipes), facilities (pump stations), and other assets (fleet, buildings, Flow Equalization Facility, cash equity, and land). The evaluation used as a baseline the values that were provided in the District's 2005 Sanitary Sewer Master Plan and Connection Fee Update (Freyer & Laureta, 2006). These values were then updated to include any known additions since June 30, 2006, as well as deletions related to retirements and replacements.

The connection fee evaluation added the new linear assets and facilities that are planned under the District's Capital Improvement Program, and spread the cost of these facilities to the total number of EDUs that are served by the District. The resulting connection fee establishes a proposed "buy-in" amount to recover from those that are adding connections or EDUs to the sewer system.

The current District residential connection fee is $2,549 per EDU. The current SBSA residential connection fee is $1,740 per EDU, for a total connection fee of $4,289 per EDU. Based on the updated connection fee evaluation, the District's connection fee may increase to between $5,000 and $7,000 per EDU. This increase does not include any potential parallel increases to the SBSA connection fee. This value should be refined further after additional information is obtained from SBSA regarding the components of the SBSA fee.

CHAPTER 2
Existing Wastewater System

The purpose of this chapter is to describe the District's existing (as of end of 2010) wastewater collection system. System information was obtained through the review of previous reports, maps, plans, operating records, general plans, and other available data. The following sections of this chapter describe the components of the District's existing wastewater collection system:

- Existing Service Area,
- Population Served and Land Use Characteristics,
- Existing Collection System Facilities, and
- Relationship with South Bayside System Authority and Member Agencies.

2.1 EXISTING SERVICE AREA

The District currently provides wastewater collection service to all or portions of the communities of Menlo Park, Atherton, Portola Valley, East Palo Alto, and Woodside, located in both Santa Clara and San Mateo Counties in the San Francisco Bay Area. The District's service area ranges from hilly, tree-covered residential areas to relatively low-topography communities, and includes narrow residential roadways, heavily traveled transportation corridors, and several commercial districts.

As shown on Figure 2-1, the District service area encompasses nearly 8,325 acres, or 13 square miles, and includes approximately 20,000 service connections to serve a population of 52,900. The District serves no independent satellite systems.

The most southerly portion of the system, in the Town of Portola Valley, will experience the greatest change in sewer flows in the future. Wastewater flows for most of this area are currently addressed through private septic systems. Newly constructed facilities are required to connect to the District's system. As the existing septic systems reach the end of their useful lives and require replacement, these systems will, over time, also connect to the public sewage collection system.

The District's system flows in a northwest direction and terminates at the Menlo Park Pump Station, which is owned by the District and operated by SBSA and located at the north end of Marsh Road near the San Francisco Bay. From this location, flow is pumped to the SBSA Wastewater Treatment Plant. The District is one of four members of the Joint Powers Agency comprising SBSA.

2.2 POPULATION SERVED & LAND USE CHARACTERISTICS

This section describes the current and build-out population projections, and associated land use as outlined in the General Plans for each community within the District's service area.

2.2.1 Existing Population and Land Use

The estimated population served by the District is 52,900 according to information compiled by the San Mateo County Local Agency Formation Commission. Land use in the District's service area is primarily residential, with localized business districts and commercial or industrial uses.

Land use information was derived from several sources collected for the communities served by the District, including:

- Land Use Database – Existing land use data in Geographical Information System (GIS) format was obtained from the City of Menlo Park (Menlo Park). Menlo Park GIS land use files included land use designations within city limits, supplemental detailed descriptions for these parcels, and additional descriptions for land use in area located outside of city limits and within the District's service area.
- General Plan Information – Additional land use data from Menlo Park and neighboring communities' General Plan updates were gathered and reviewed.
- Aerial Photographs – Aerial photographs of the service area were reviewed to identify vacant parcels and properties where actual land use clearly varied from the assigned land use designation.

Figure 2-2 shows the land use designations that were defined for the City of Menlo Park. These land uses consolidate the land use categories that were provided by the City in the Menlo Park GIS database, as shown in Table 2-1. The land uses were further defined to locate specific individual parcels as follows:

- Vacant parcels and Holy Cross Cemetery near Santa Cruz Avenue. These parcels, identified through a review of aerial photography for the service area, were assigned zero flow in the existing system analysis.
- Parcels that included large areas of open space. These parcels, which included the Menlo Circus Club and several private and public schools and colleges, were divided so that unit flow factors could be assigned only to the built-out portion of the total acreage.
- Commercial and industrial wastewater dischargers. These parcels, including Stanford Linear Accelerator Center, Diageo Global Supply, Veteran Affairs Hospital, the United States Geological Survey, SRI International, and Tyco Electronics, were assigned flows based actual user records in lieu of unit flow factors.

Figure 2-3 shows existing land use designations assigned by West Yost for the areas within the District's service area but outside of Menlo Park city limits. Land uses for these areas were derived from description information provided within the Menlo Park GIS database, and confirmed through review of individual General Plan documents.

Table 2-1. Summarized Land Use Categories

City of Menlo Park Land Use Description	Hydraulic Model Land Use Description
Very Low Density Residential	Single Family Residential
Low Density Residential	Single Family Residential
Medium Density Residential	Multi-Family Residential
High Density Residential	Multi-Family Residential
Professional and Administrative Offices	Office
Retail/Commercial	Commercial
Limited Industry	Industrial
Public Facilities	Public Facilities
Parks & Recreation	Not Modeled
Non-Urban	Not Modeled

2.2.2 Build-out Population and Land Use

The estimated build-out population to be served by the District is 54,458, as compiled by SBSA during development of the SBSA Conveyance System Master Plan (TM No. 2, Design Flow Rates and Design Criteria, Winzler & Kelly, 2009). Land use in the District's service area will continue to be primarily residential, with small expansions planned within the existing business districts and commercial or industrial areas. The build-out condition assumes that all vacant lots identified for the existing system have been built out according to their respective zoning designation or, if more conservative, as shown in the General Plan documents.

Figure 2-4 shows build-out land use within the District's service area, as determined through available General Plan data. Build-out expectations for sewered connections in the Portola Valley were not discussed in the General Plans. To be consistent with the assumptions made in the SBSA Conveyance System Master Plan (TM No. 2, Winzler & Kelly, 2009), all parcels are assumed to be connected to the sewer system in the calculation of total system flows at build-out. In order to develop a phased plan for accommodating increased flows from the Portola Valley area, a build-out rate of 10 units per year is considered for the duration of the CIP.

2.3 EXISTING COLLECTION SYSTEM FACILITIES

This section describes facilities that are owned and maintained by the District. Existing facility information was derived from the District's GIS database, which was updated to include details from recent pipeline improvement projects completed by the District. Figure 2-5 shows the District's pipeline and pump station facilities, as documented in GIS. Pipeline and manhole GIS layers will be used to develop the collection system network in the collection system hydraulic model, primarily for pipelines with a diameter of 12-inches and larger, with some pipelines 10-inches in diameter and smaller, and associated pump stations.

2.3.1 Pipeline Characteristics

The District is comprised of over 200 miles of gravity sewer pipe as reported in the State Water Resources Control Board (SWRCB) California Integrated Water Quality System (CIWQS) database. The District's pipes range in diameter from 4 to 54 inches. In addition, the District's customers own and maintain an additional 150 miles (approximate) of private service laterals. The predominant District pipeline materials are vitrified clay and asbestos cement pipe, with isolated occurrences of reinforced and unreinforced concrete and ductile iron pipe. Recently installed pipe is comprised of polyvinyl chloride (PVC). Table 2-2 summarizes the District's system by pipe material.

Table 2-2. Percentage of Pipe by Material Type

Pipe Material	Percent of System
Vitrified Clay Pipe	73%
Asbestos Cement Pipe	14%
Polyvinyl Chloride	10%
Ductile Iron	<1%
Unreinforced Concrete	<1%
Reinforced Concrete	<1%
Corrugated Metal Pipe	<1%

The District's service area includes two major sewer basins and two minor sewer basins. The major drainage basins divide the service area in approximately half along a southwest to northeast axis. Flows travel from south to north through the service area. The northwest major sewer basin collects flow along Atherton Road and Marsh Road. The southeast major sewer basin collects flow along Oak Avenue, Middle Avenue, Linfield Drive, Santa Monica Avenue, and Hollyburne Avenue, before traversing northwest along the Highway 101 corridor. The central minor basins are located in the center of the service area, generally between Middlefield Road and Highway 101. The north minor basin is located north of Highway 101. The District's major and minor sewer basins are presented in Figure 2-6.

2.3.2 Pump Station Characteristics

The District owns and operates twelve pump stations ranging in firm capacity from 110 to 2,500 gallons per minute (gpm). Additional information on the District's pump stations, including an assessment of pump station rehabilitation needs, is presented in Chapter 9, Pump Station Rehabilitation Program. Table 2-3 lists the District's pump stations and summarizes operating characteristics.

Table 2-3. District Pump Stations				
Pump Station Name	Number of Pumps	Pump type	Year Built	Firm Capacity, gpm
Hamilton Henderson	2	Submersible	1991	2500
Willow Road	2	Submersible	1981	1450
Menlo Industrial	2	Submersible	2003	300
University	3	Submersible	1984	300
Illinois	2	Submersible	2010	600
Vintage Oaks #1	2	Submersible	1995	295
Vintage Oaks #2	2	Submersible	1996	295
Stowe Lane	2	Dry Pit	1950	280
Los Trancos	2	Submersible	2000	200
Sausal Vista	2	Submersible	1978	100
Corte Madera	2	Submersible	2000	350
Village Square	2	Submersible	2004	110

2.3.3 Other Facilities

The District owns approximately 20 acres of land at the northern terminus of Marsh Road in Menlo Park that contains four storage basins. These basins are collectively named the Flow Equalization Facility (FEF). The two basins closest to the Menlo Park Pump Station are currently maintained and used to provide wet weather storage for the District. The estimated capacity of Pond 1, which is the District's primary wet weather storage facility, is under 10 million gallons. This land and these basins were part of the District's wastewater treatment facilities, prior to the forming of SBSA in 1980.

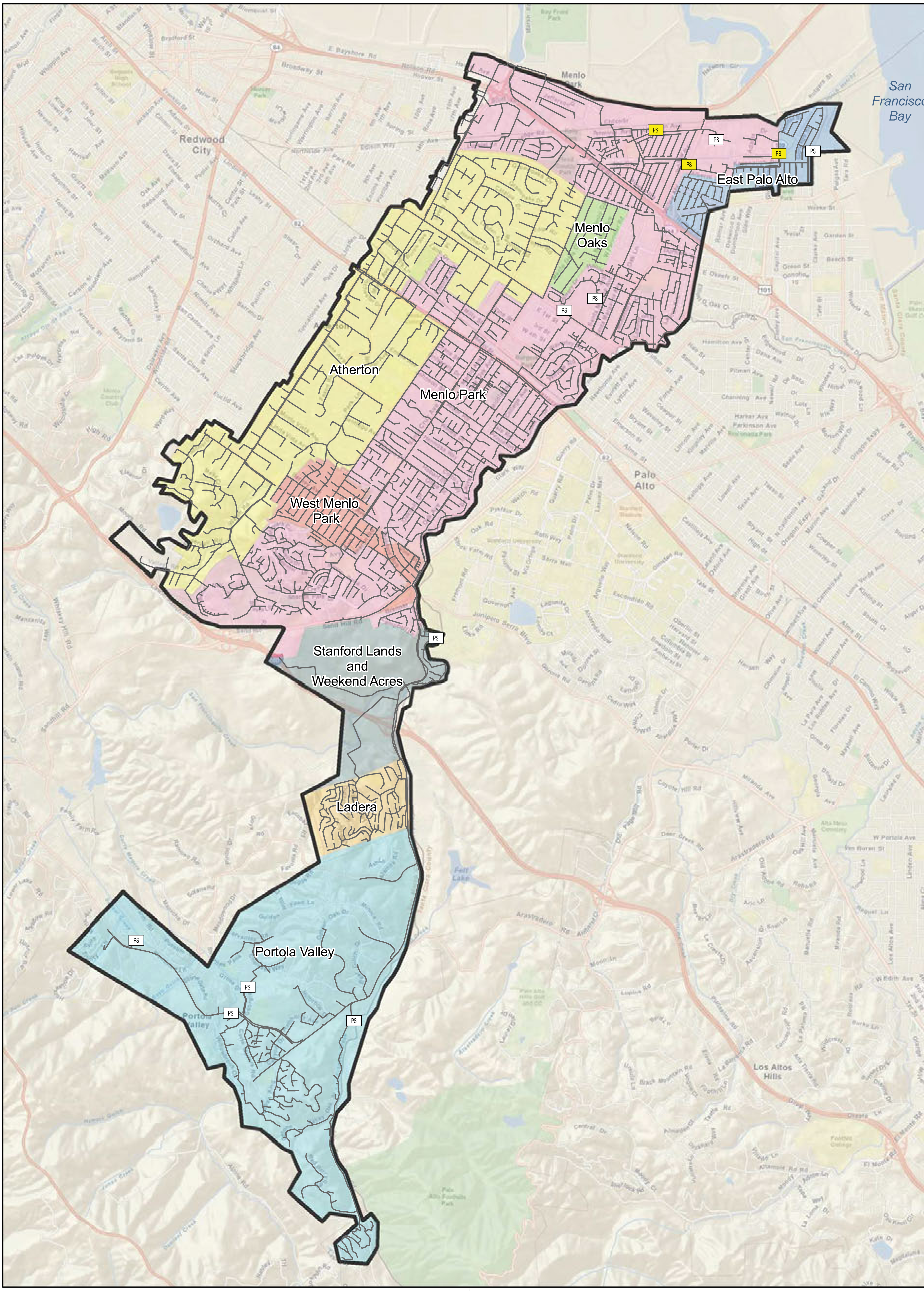
The District currently has the capability to bypass the Menlo Park Pump Station and flow directly to the FEF during extreme wet weather events. The District also owns a transfer pump station that returns stored flow back to the Menlo Park Pump Station after wet weather events. This transfer pump station has a firm capacity of 8,660 gpm, and is operated by SBSA. Figure 2-7 provides an aerial view of the District's FEF.

2.4 RELATIONSHIP WITH SOUTH BAYSIDE SYSTEMS AUTHORITY AND MEMBER AGENCIES

The District, along with the Cities of Belmont, San Carlos, and Redwood City comprise the Joint Powers Agency of SBSA. SBSA is currently undertaking an upgrade and expansion of the wastewater treatment facilities. When the expansion is completed, SBSA will have 80 million gallons per day (mgd) of wet weather capacity; the District is allocated 16.4 mgd of this capacity.

SBSA operates the pump stations that are located at the terminus of each member's collection system, including the District-owned Menlo Park Pump Station located at the northern end of Marsh Road, south of the District's FEF. The District and SBSA have entered into an agreement that allows use of the FEF by SBSA during wet weather events. SBSA reimburses the District for this use on an annual basis.

When needed during wet weather, SBSA submits a request to the District to bypass the Menlo Park Pump Station and flow directly to the FEF. By preventing District flows from entering the SBSA system, SBSA increases capacity for the other members, both within the SBSA conveyance system and the SBSA wastewater treatment plant. When flows through the SBSA system have decreased sufficiently after the wet weather event, the District pump flows from the FEF to the Menlo Park Pump Station for conveyance and treatment.



LEGEND

- PS Modeled Pump Stations
- PS Pump Station
- Sewer Pipes
- Parcels
- WBSD Service Area

N

0 0.5 1
Miles

FIGURE 2-1

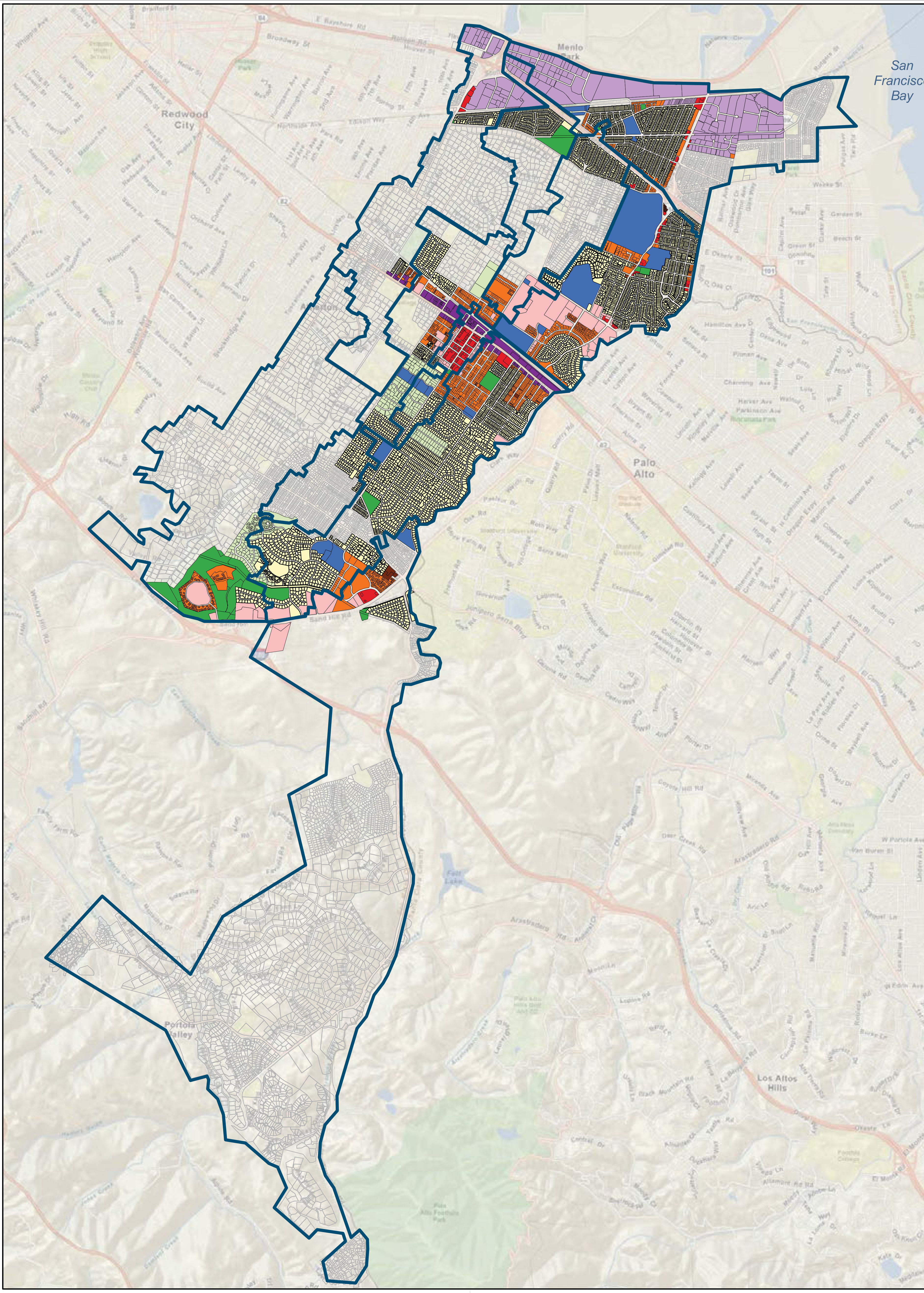
West Bay Sanitary District

Collection System Master Plan

WBSD Service Area

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

- Basin
- Very Low Density Residential
- Low Density Residential
- Medium Density Residential
- High Density Residential
- Professional and Administrative Offices
- Retail/Commercial
- El Camino Real Professional/Retail Commercial
- Limited Industry
- Public Facilities
- Parks and Recreation
- Other
- Parcels

N

0 2,000 4,000

Scale in Feet

FIGURE 2-2

West Bay Sanitary District

Collection System Master Plan

City of Menlo Park
Land Use

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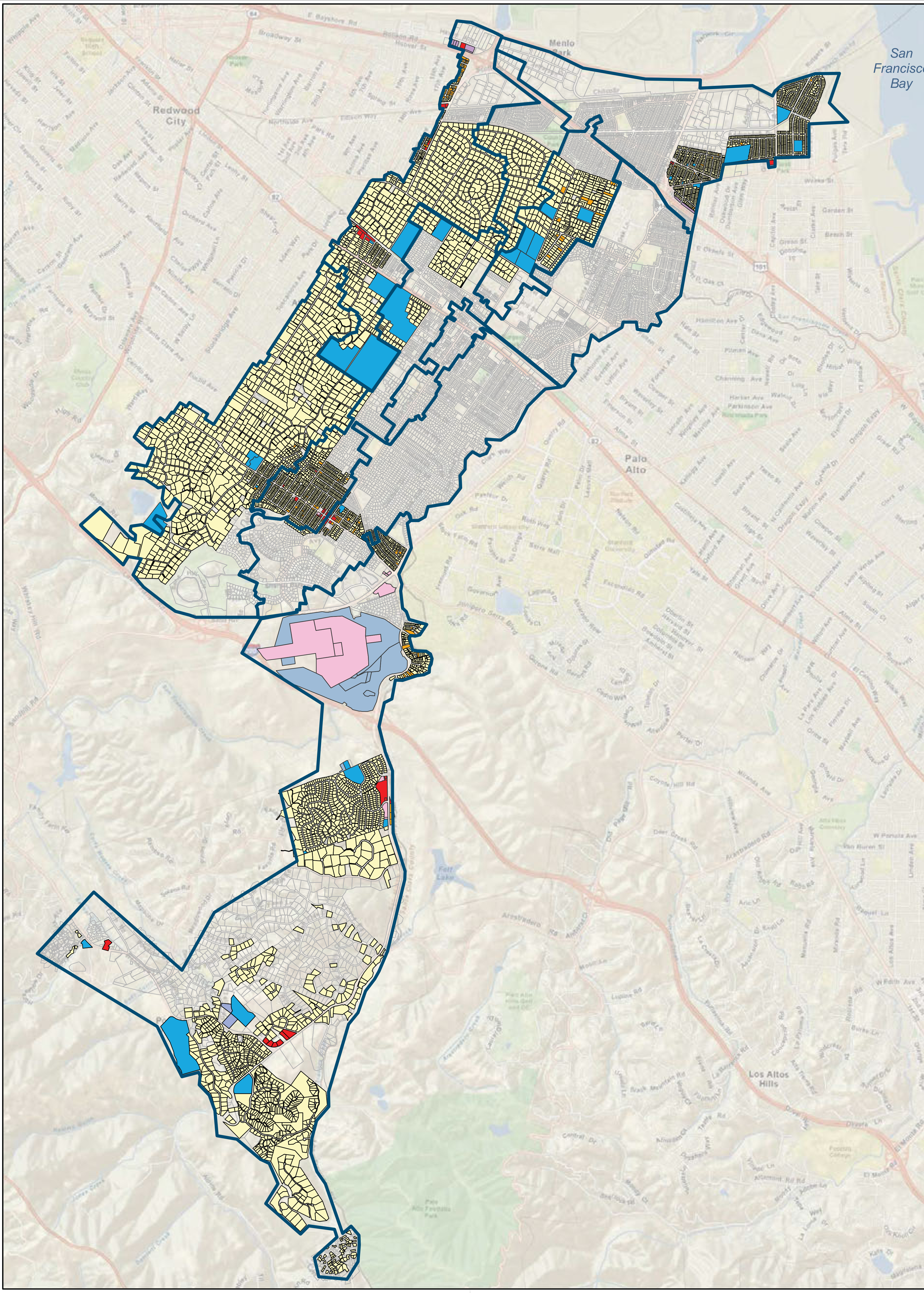


LEGEND

- Basin
- Very Low Density Residential
- Low Density Residential
- Medium Density Residential
- High Density Residential
- Professional and Administrative Offices
- Retail/Commercial
- El Camino Real Professional/Retail Commercial
- Limited Industry
- Public Facilities
- Parks and Recreation
- Other
- Parcels

N

0 2,000 4,000

Scale in Feet

FIGURE 2-3

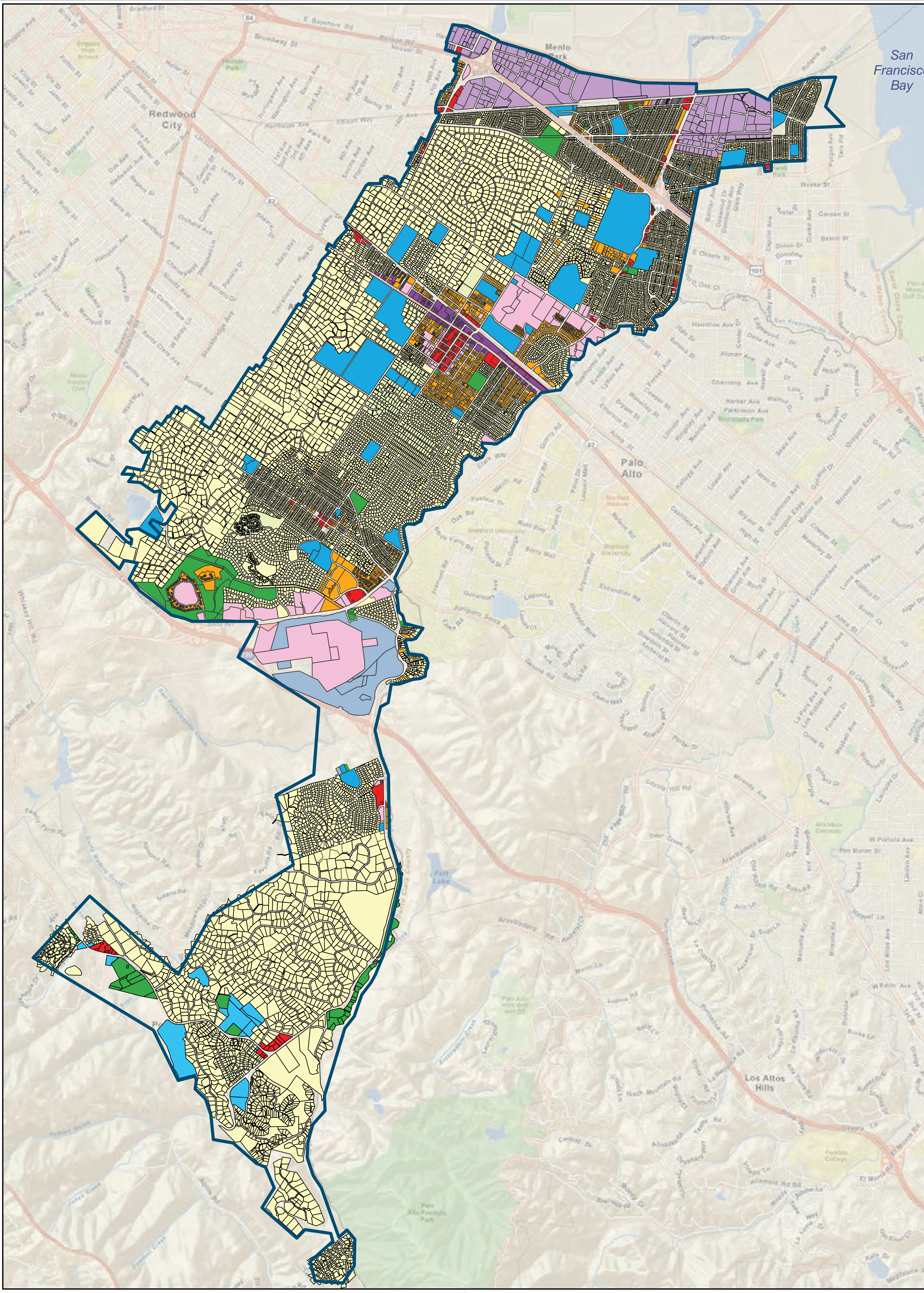
West Bay Sanitary District

Collection System Master Plan

Land Use Outside of the City of Menlo Park

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

- Very Low Density Residential
- Low Density Residential
- Medium Density Residential
- High Density Residential
- Professional and Administrative Offices
- Retail/Commercial
- El Camino Real Professional/Retail Commercial
- Limited Industry
- Public Facilities
- Parks and Recreation
- Other
- WBSD Service Boundary

N

0 2,000 4,000

Scale in Feet

FIGURE 2-4

West Bay Sanitary District

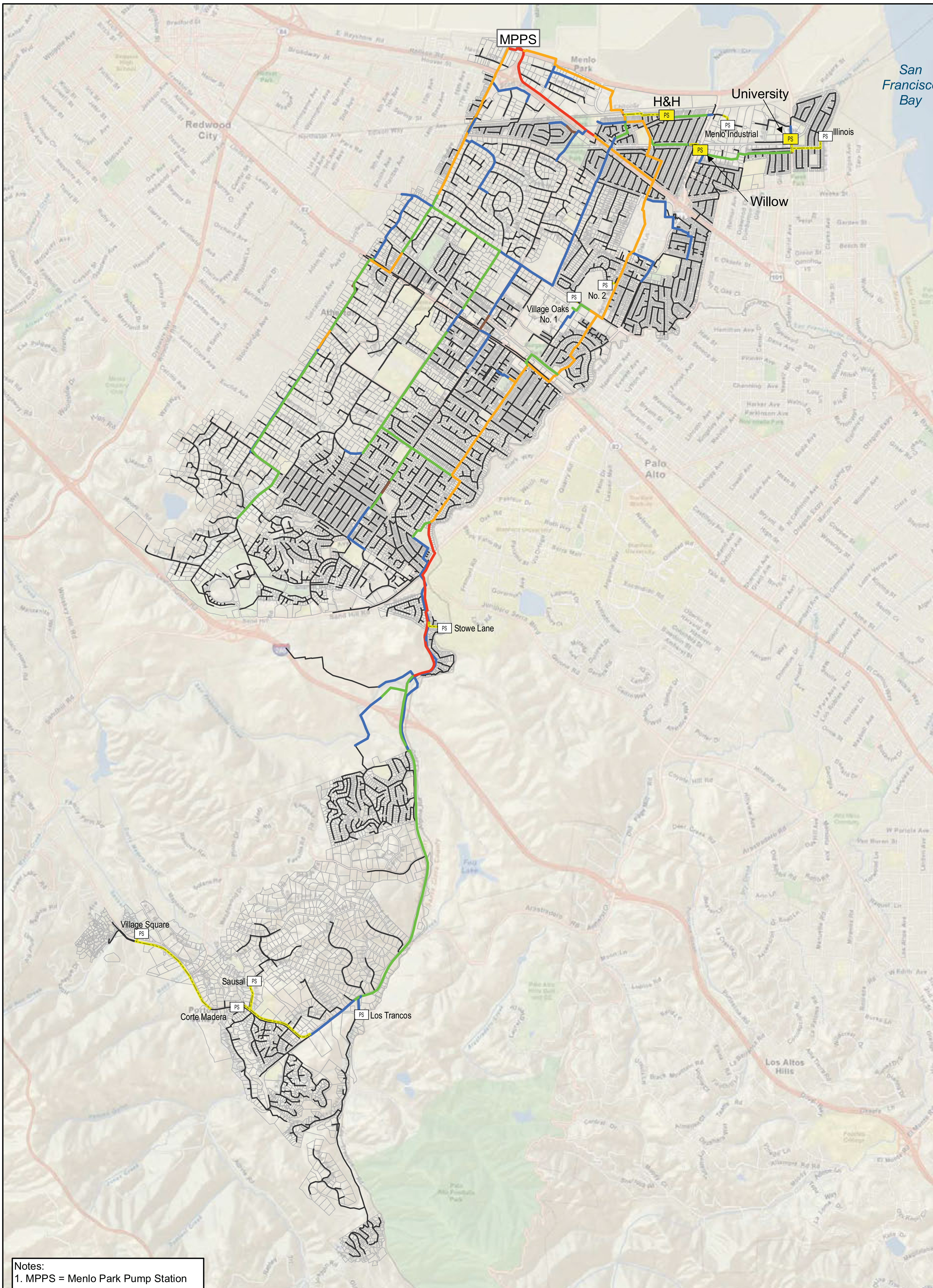
Collection System Master Plan

Buildout Land Use

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES

Consulting Engineers



LEGEND

Modeled Pipes

- 6 - 8"
- 10 - 12"
- 15 - 21"
- 24 - 30"
- >30"

- PS Modeled Pump Stations
- PS Pump Station
- Force Main
- Sewer Pipes
- Parcels

N

0 2,000 4,000

Scale in Feet

FIGURE 2-5

West Bay Sanitary District

Collection System Master Plan

District's Wastewater Collection System Facilities

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

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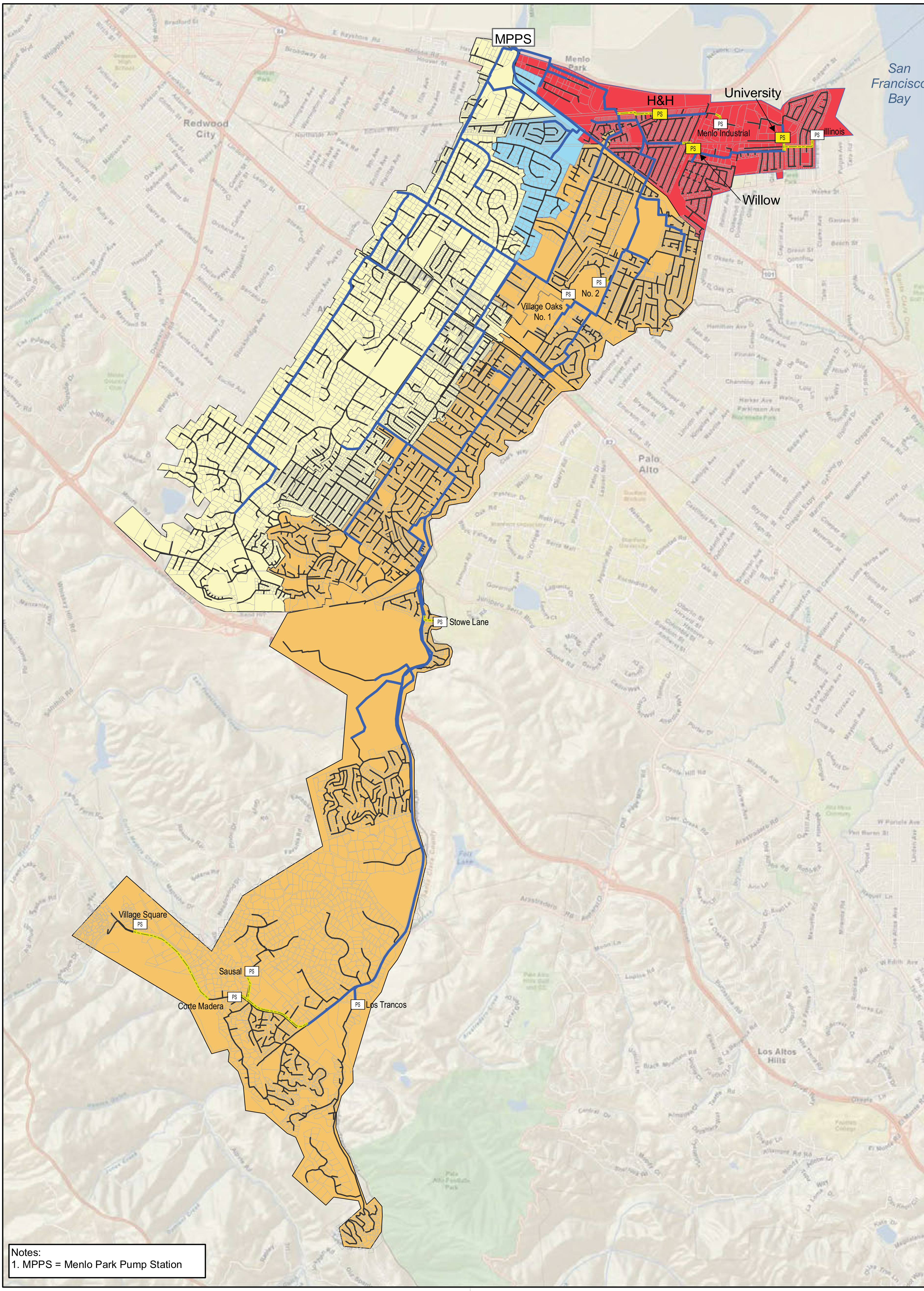

Notes:
1. MPPS = Menlo Park Pump Station

LEGEND

Sewer Basins
- Central
- North
- Northwest
- Southeast
- Parcels

- PS Modeled Pump Stations
- PS Pump Station
- Force Main
- Model Pipe
- Sewer Pipes

N

0 2,000 4,000
Scale in Feet

FIGURE 2-6

West Bay Sanitary District

Collection System Master Plan

Major and Minor Sewer Basins

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES
Consulting Engineers

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LEGEND

Flow Equalization Facility

N

0 250 500

Scale in Feet

FIGURE 2-7

West Bay Sanitary District

Collection System Master Plan

Flow Equalization Facility

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES
Consulting Engineers

This chapter presents the background and methodology used to determine existing and future dry weather wastewater flows for input to the District's collection system hydraulic model. This chapter is organized as follows:

- Sources of System Data, and
- Calculation of Dry Weather Flows.

3.1 SOURCES OF SYSTEM DATA

The main sources of data used to estimate wastewater flows for the District's hydraulic model include land use information, aerial photography, and District unit flow factors.

3.1.1 Land Use Information and Aerial Photography

The following resources were used to develop land use projections for the communities that are currently served by the District.

- City of Menlo Park. Menlo Park provided land use information in GIS format. The GIS database included Menlo Park general plan land use designations as well as parcel descriptions for the District's entire service area. Menlo Park also provided General Plan land use information in pdf format (2008), and Specific Plans adopted for various developments including the Menlo Gateway Project and the Downtown Specific Plan.
- Town of Atherton. The Town of Atherton provided its General Plan in pdf format. The General Plan, adopted in 2002, provides anticipated land uses through the buildout horizon of 2020.
- Town of Portola Valley. The Town of Portola Valley is a unique contributor to sanitary sewer flow as this community is transferring from septic to sewered systems over time. The Town provided its General Plan in pdf format. The General Plan, last updated in 2009, listed anticipated land uses through the buildout horizon of 2020. The General Plan also includes the unincorporated community of Ladera.
- County of San Mateo. The County of San Mateo (County) provided general plan maps in pdf format for the unincorporated communities of West Menlo Park and Menlo Oaks.
- City of East Palo Alto. The District serves small portion of the City of East Palo Alto. East Palo Alto provided its General Plan in pdf format. The General Plan, adopted in 1999, provides a description of planned land uses without reference to a buildout horizon.
- Arial Photography. In addition to land use information, West Yost reviewed aerial photography obtained for the District's service area to identify parcels that are currently vacant.

3.1.2 Industrial Discharge Data

The District provided West Yost with discharge data for specific commercial and industrial facilities, including Stanford Linear Accelerator Center, Veteran's Affairs (VA) Hospital, United States Geological Survey, SRI International, Diageo Global, and Tyco Electronics. This data was used to isolate large industrial dischargers and to represent their flows as point loads in the hydraulic model. Significant discharges cause inaccuracy in dry whether flow calculations if not considered independently from flows estimated using standard wastewater flow generation factors and assigned land use.

3.1.3 District Design Criteria

The District publishes standard design criteria for the sizing of new sewers, which includes unit flow factors to be applied to individual customer classes. The criteria are directed toward new developers, homeowners, or businesses that request to connect to the existing system, and represent a conservative representation of anticipated flows by land use designation. The District's criteria also includes peaking factors in order to estimate peak dry weather flows, and I&I allowances to calculate anticipated wet weather flows. Table 3-1 lists the District's standard design criteria related to wastewater flow generation.

Table 3-1. District Design Criteria for Sewer Flows

Customer Class Description	Criteria	
Commercial	90 gallons per day per 1,000 square feet or 2,500 gallons per acre per day	
Office	300 square feet per employee and 15 gallons per day per employee or 2,000 gallons per acre per day	
Restaurant	1 gallon per day per square foot	
Industrial	3,000 gallons per acre per day	
Infiltration	The larger of 600 gallons per acre per day or 500 gallons per inch/diameter/mile per day	
Average Dry Weather Flow Per Capita	85 gallons per day	
Average Dry Weather per Single Family Dwelling	220 gallons per day	
Ratio of Peak to Average Flow	Tributary Population	Peak to Average Flow
	1,000 and less	5.0
	2,000	4.4
	3,000	4.0
	4,000	3.7
	5,000	3.5
	10,000	3.2
	20,000	2.8
	50,000	2.4
	Commercial, Industrial, or Office	10,000 gallons per acre per day

3.2 CALCULATION OF DRY WEATHER FLOWS

This section summarizes the methodology used to develop the initial dry weather or base wastewater flow (BWF) component of the collection system hydraulic model. These initial flows were further refined through the hydraulic model calibration process, as discussed in Chapter 5, Hydraulic Model Development. The District's BWF, as measured during the 2009/10 flow monitoring program, is 4.58 mgd.

Numerous methodologies are available to estimate BWF that rely on population estimates, water usage, land use designations, and other resources. When a service area has complex flow characteristics caused by factors such as extensive mixed land use or low or variable occupancy rates, a more detailed methodology is required to accurately predict BWF. The District's service area has minimal mixed land use and is nearly built out. Although commercial occupancy rates have declined in recent years due to economic factors, the District's occupancy rate is generally high. These attributes justify a straightforward calculation of BWF using flow factors and land use designations.

The initial BWF component was calculated using the following steps:

- Review each city's general plan land use designations. For general plans with differing land use designations, consolidate to conform to the City of Menlo Park's land use designations as shown in Section 2.
- Designate each parcel within the service area according to the consolidated land use designations.
- Review aerial photography for confirmation and identification of currently vacant parcels. Sewer loads were not assigned to vacant parcels.
- Assign specific flows to industrial or commercial parcels for which more customized large wastewater flows have been provided by the District as shown in Table 3-2.
- Beginning with the District's unit flow factors, apply unit flow factors to each land use category other than vacant or defined commercial/industrial parcels to estimate dry weather sanitary sewer flows.
- Adjust the unit flow factors so that estimated dry weather sanitary sewer flows across all basins closely agree with the metered flows provided in flow monitoring reports. The adjusted flow factors (after initial adjustment but before model calibration) are presented in Table 3-3.

- Apply a separate calculation for Portola Valley, where all parcels adjacent to a constructed sewer are assumed to be connected to the sewer. This count was compared to and is consistent with connection information provided by the District.[1]

The same methodology was used to calculate build-out flows. Build-out flows were calculated by assigning unit flow factors to build-out land uses, according to the following steps:

- Populate all vacant residential parcels,
- Add flow from all new development projected in the individual General Plans, and
- Add flow from the Town of Portola Valley assuming that all parcels currently on septic will be connected to the sewer system at build-out in order to estimate total system flows, and that parcels will connect at a rate of 10 parcels per year to plan phased improvements.

Table 3-2. Flows from Large Commercial and Industrial Dischargers

Dischargers	Flows, gpd
SRI International	65,812
USGS	3,403
Stanford Linear Accelerator Center	35,947
VA Hospital	50,156
Tyco Manufacturing	167,633
Diageo	8,023

[1] This calculation of existing flows is conservative, as some parcels in the Town of Portola Valley adjacent to the existing sewer system will continue to use a septic system until the parcel owner elects or is required by the Town to connect to the sewer system.

Table 3-3. Land Use Flow Factors, Adjusted to Balance Flows with Measured Data

Land Use[b]	Basins[a]		
	1A, 2C, 3, 4, 2A, 1E, 1D, gpd/parcel or gpd/acre	1B and 2B, gpd/parcel or gpd/acre	1C, gpd/parcel or gpd/acre
Very Low Density Residential	170		
Low Density Residential	170	250	80
Medium Density Residential	1,900	2,700	900
High Density Residential	4,100	6,400	
Professional and Administrative Offices	900	1,200	
Retail/Commercial	900	1,000	
El Camino Real Professional/Retail Commercial	1,200	1,800	
Limited Industry	900	1,200	
Public Facilities	600	900	400
Parks and Recreation	50	50	50

(a) Basin numbering and locations are described further in Chapter 4.
(b) Very Low Density Residential and Low Density Residential both have units of gpd/parcel. All other land uses have units of gpd/acre.

CHAPTER 4
Infiltration and Inflow Analysis

During the 2009/2010 wet weather period, the District completed a system-wide flow monitoring program. Although planned previous to that time, the program was conducted by V&A from December 2009 to March 2010. Data and results are presented in the V&A report titled, *Sanitary Sewer Flow Monitoring and Inflow / Infiltration Study (May 2010)*. The V&A Report is referenced in this chapter as the Flow Monitoring Report, and is included in Appendix A.

This chapter highlights the findings from this report that are key factors in the development of a strategic I&I reduction program. This chapter is organized as follows:

- Findings and Recommendations,
- General Sources of Inflow and Infiltration,
- System-Wide Flow Monitoring Program,
- Description of Flows, and
- Inflow and Infiltration Analysis.

4.1 FINDINGS AND RECOMMENDATIONS

The following findings and recommendations were developed to address, over time, the potential issues within the system that are the most significant contributors to I&I. Through the control of I&I, the District will also likely reduce the potential for wet weather related sewer system overflows (SSOs).

4.1.1 Findings

The 2009/2010 flow monitoring program captured rainfall events with a statistical recurrence interval of equal to or less than 2 years (*i.e.*, 50 percent probability of occurring in any given year). Peak flows generated by these rainfall events may have resulted in surcharge conditions, with the flow level exceeding the crown of the pipe, at five of the metered locations.

The recommendations presented in this chapter prioritize areas for I&I reduction, in order to help reduce the potential for future system surcharges. Although addressing the highest impact contributors to I&I will likely reduce system flows, I&I reduction is a long-term strategy for flow management. Near-term solutions to capacity related issues should also consider capacity improvements, such as pipe size increases, flow diversions, and relief sewers.

4.1.2 Recommendations

The primary contributors to wet weather I&I in the City's system appear to be direct stormwater inflow from the communities that are tributary to the trunk sewer located in Atherton Avenue, and groundwater infiltration from the portion of the service area located north of Highway 101, adjacent to the San Francisco Bay. Because stormwater management is the responsibility of the respective city or county, and not the District, and groundwater infiltration in low-lying areas is often ubiquitous, and both of these sources of I&I will be difficult to evaluate and address in a cost effective manner. However, over time, as the District continues to implement system rehabilitation and replacement projects in these areas, attention could be given to locating and disconnecting illegal connections and improving system performance through the use of water-tight replacement methods and materials.

4.2 GENERAL SOURCES OF INFLOW AND INFILTRATION

I&I defines extraneous flows that enter the sanitary sewer system. I&I can potentially adversely impact the capacity of wastewater collection systems by increasing both peak flows and total flow volume. Rainfall-dependent inflow and infiltration (RDII), groundwater infiltration (GWI), and inflow from illegal connections can all be contributors of I&I.

I&I can enter the collection system through different mechanisms. Inflow is water that enters the collection system through a direct unpermitted connection. Inflow enters the sewer pipe independent of groundwater level and can be seen in the collection system immediately following a rainfall event. Infiltration is water that enters the collection system by percolating through the soil and then into the collection system through defects in pipelines, manholes, and joints. Infiltration occurs over a longer period of time, and depending on soil conditions, can occur for days, weeks, or seasonally.

Figure 4-1 provides examples of common I&I sources. Examples of inflow sources include direct connections such as roof leaders, foundation or yard drains, open cleanouts in low-lying areas, storm drain cross-connections, and leaky manhole covers. Examples of RDII and GWI sources are cracked or broken pipes and laterals, misaligned joints, pipe offsets and sags, deteriorated manholes, and root intrusion.

RDII generally occurs after a rainfall event, and can enter the collection system on the same day that the rainfall event begins, and may last for several days after the rain event has ceased. GWI varies seasonally as the groundwater table fluctuates, and can typically be seen in flow data as a mounding effect into the collection system over the entire wet weather season. GWI patterns may reflect movement of the groundwater table, which generally rises gradually during the wet weather season and falls during the dry weather season. On a day-to-day basis, GWI may occur relatively steadily throughout the day and even over several weeks. Figure 4-2 shows typical RDII response in the District flow monitoring data.

Within the low lying areas surrounding the San Francisco Bay, tidal infiltration can be another source of infiltration into a collection system. Tidal infiltration rates vary daily and fluctuate with the tide table. The extent of tidal infiltration depends on the proximity to the San Francisco Bay, and the depth of these pipelines and manholes relative to the tide elevation.

4.3 SYSTEM-WIDE FLOW MONITORING PROGRAM

This section describes the 2009/10 flow monitoring program, including meter and rain gauge locations, and basin delineations.

4.3.1 Data Collection

The flow monitoring program included twelve (12) gravity meters and two (2) rain gauges. The twelve meters were located in manholes that delineated the collection system into ten basins, comprising two major zones and two minor zones. Figure 4-3 presents the flow meter locations and associated flow monitoring basins within the collection system. The two major zones are designated on Figure 4-3 as Northwest Basin (comprised of Basins 2A, 2B, 2C, and 4) and Southeast Basin (comprised of Basins 1B, 1C, 1D, and 1E). The two minor zones are designated as North Basin and Central Basin.

Depth and velocity readings were collected at each flow meter in 15-minute increments. This data was compiled into hourly flows for use in the I&I analysis. Some basins are defined by a combination of flow meters – one meter measures flow into the basin, and the second meter measures flow leaving the same basin. In order to measure basin-specific flows, for these specific basins, net flow was calculated by subtracting incoming flow values from outgoing flows.

Figure 4-4 presents a schematic illustrating the direction of flow and interconnection between basins. Two meters (Manholes G14162 and C12072) were installed to capture flows diverted from one basin to another.

Table 4-1 lists the flowmeter that captures flow exiting each sewer basin, and the Manhole ID defining the location of each meter.

Table 4-1. Summary of Basins and Associated Flowmeters

Basin	Meter Number	Location of Flowmeter	Cross Street
1-A	2	B15047	Adjacent to Hwy 84 near Marsh Road
1-B	4 and 5 (overflow)	C13088 and C12072	Pierce Road @ Berkeley Avenue (M4) and Hollyburne Avenue (M5)
1-C	3	C13029	Hamilton and Hill Avenues
1-D	6	E12158	Easement NE of Willow Road and Alma Street
1-E	7	H13216	Bay Laurel Drive and Oak Knoll Lane
2-A	9 and 8 (overflow)	D15128 and G14188	Valparaiso Ave of Politzer Drive (M8) and Middlefield Road West of Watkins Avenue (M9)
2-B	1	B16001	North End of Haven Avenue
2-C	10	F16009	Atherton Avenue North of Mercedes Lane
3	11	B15042	Independence Drive South of Constitution Drive
4	12	E14053	Oak Grove Avenue South of Laurel Street

4.3.2 Rainfall Data

Two rain gauges were installed by V&A within the District's service area during the 2009/10 flow monitoring effort. Rain Gauge No. 1 was installed in Portola Valley at the Village Square Pump Station, and captured the rainfall in the higher elevations of the District's service area. Rain Gauge No. 2 was installed at Menlo Park Pump Station in Menlo Park, and captured the lower elevations of the District's service area. In addition, V&A utilized data from an existing public weather station (Rain Gauge No. 3) that was located in the central portion of the District's service area, on Cedar Avenue and south of Barney Avenue, in Menlo Park. Figure 4-5 presents the recorded rainfall during the 2009/10 wet weather season in a graphical format and demonstrates the disparity of rain volumes in the District.

The largest rainfall event during the flow monitoring period occurred from January 18-23, 2010. Several smaller storms occurred from December 26, 2009 through March 12, 2010. The total rainfall volume recorded by each of the three rain gauges during the January 18-23 storm event is presented in Table 4-2.

Table 4-2. Summary of January 18-23, 2010 Rainfall Data

Rain Gauge	Elevation of Rain Gauge, feet	Daily Rainfall Volume, inches						Total 6-Day Storm Volume, inches
		18th	19th	20th	21st	22nd	23rd	
1	432	0.72	1.46	3.01	0.80	0.34	0.45	6.78
2	8	0.56	.082	1.25	0.59	.032	0.28	3.82
3	111	0.77	1.33	2.27	0.81	0.35	0.24	5.77

The highest 24-hour rainfall volume occurred between January 19-20, and measured 3.21, 1.38, and 2.36 inches for Rain Gauge No. 1, 2, and 3, respectively. The highest 1-hour rainfall volume measured during this period was 0.73, 0.28 and 0.46 inches for Rain Gauge No. 1, 2, and 3, respectively.

Rainfall events are classified based on recurrence interval and duration. The National Oceanic and Atmospheric Administration (NOAA) has developed atlas maps, based on long-term historical rainfall data, that provide classifications for 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year storm events with 6-hour and 24-hour durations. The NOAA classification for a 2-year 24-hour storm event at the location of Rain Gauges No. 1, No. 2 and No. 3 are 4.0, 2.5 and 3.0 inches, respectively. As calculated by V&A and confirmed through review of the NOAA atlas maps, the maximum 24-hour rainfall total for the January 18-23, 2010 storm event had a classification of less than a 2-year, 24-hour storm event.

4.4 DESCRIPTION OF FLOWS

This section summarizes the V&A analysis of dry weather and wet weather flows collected during the system-wide flow monitoring program.

4.4.1 Dry Weather Flow

V&A calculated average dry weather flows (ADWF) or BWF from data collected during the system-wide flow monitoring period. BWF includes the wastewater generated from residential, commercial, and public users. V&A reported these flows in Tables 7 and 10 of the Flow Monitoring Report. V&A Table 7 lists ADWF with the groundwater infiltration component included, and Table 10 removes the excessive groundwater infiltration component from ADWF. Table 10 assumes that it is reasonable to expect a typical amount of groundwater in the ADWF. The ADWF, across weekday and weekend periods, as measured by V&A was 4.58 mgd.

Example weekday flows from V&A Table 10 are reported in Table 4-3.

Table 4-3. Average Dry Weather Flow Data

Site No.	District Manhole ID	Basin	Contributing Basins	Weekday with GWI, mgd	Excess GWI, mgd
1	B16001	2-B	2-A, 2-C, 4	1.43	
2	B15047	1-A	1-C	1.25	
3	C13029	1-C	n/a	0.10	0.30
4	C13088	1-B	1-D	1.87	
5	C12072	1-B (Overflow)	n/a	n/a	
6	E12158	1-D	1-E	1.13	
7	H13216	1-E	n/a	0.57	
8	G14188	2-A (Overflow)	n/a	n/a	
9	D15128	2-A	n/a	0.58	
10	F16009	2-C	n/a	0.31	
11	B15042	3	1-B	1.90	
12	E14053	4	n/a	0.12	

The flow meter installed in Manhole E14053 may have been affected by a potential pipe blockage upstream that may have affected the monitoring data from January 12-29, 2010. Prior to and after this time frame, average dry weather flow at this location was approximately 0.12 mgd.

The flow meter installed in Manhole C12072 did not capture BWF. This meter was installed on the downstream side of a weir structure to capture overflows in the 24-inch line along Hollyburne Avenue, north of Pierce Road. Flows are only captured at this meter location when high flow occurs in the upstream 24-inch diameter sewer. The V&A Report describes the location and provides a photograph of the Site 5 weir structure.

A flow meter was installed in Manhole G14188 to capture flows between Basin 2-A and 1-D. Prior to installation, the assumption was that flows could transfer out of Basin 2-A and into Basin 1-D through a 16-inch diameter relief pipeline along the property right-of-way (extending east from Valparaiso Avenue to Olive Street/ Santa Cruz Avenue). Fluctuating flow monitoring data at this location (between positive and negative values) initially indicated that flow direction alternated throughout each day. However, District staff have since confirmed that Manhole G14162, which is located directly west of the meter, has an invert elevation that is higher than the meter location. Therefore, the fluctuating values represent flow that is entering and then exiting the pipe segment between G14188 and G14162 as dry weather flows vary throughout the day. No dry weather flow leaves Basin 2-A during dry weather. However, this pipe does serve as a wet weather relief pipeline.

4.4.2 Wet Weather Flow

Wet weather flows occur in a collection system during a rainfall event, and can continue for several days after the rainfall event ceases, depending on soil conditions and associated drainage characteristics. Wet weather flows captured during the flow monitoring event provide a means to quantitatively estimate the peak and volume of RDII entering the system.

Peak wet weather hourly flows and depths of surcharge recorded at each monitoring site during the flow monitoring period are presented in Table 12 of the Flow Monitoring Report, and also Table 4-4, below. Table 4-4 includes a calculation of the wet weather peaking factor for each basin. It should be noted that high peaking factors for basins with low flows are not necessarily indicative of a systematic I&I issue. The total peak hourly flow rate, measured by Meters 1, 2, and 11, was 19.8 mgd on January 20, 2010.

It is difficult to compare the District's wet weather peaking factors with other Bay Area agencies, since I&I is influenced by many factors, including system age, geological setting, topographic variation, pipe materials and installation practices, historical maintenance programs, etc. However, it appears that the District's peaking factors tend to be lower than peaking factors of many other Peninsula cities of similar age and setting, potentially indicating less I&I.

Table 4-4. Summary of Wet Weather Flow Monitoring Data

Meter Site No.	District Manhole ID	Pipe Diameter, inches	ADWF, mgd	Peak Flow, mgd	Wet Weather Peaking Factor	Surcharge Above Crown of Pipe, ft
1	B16001	30	1.43	8.64	6.0	3.2
2	B15047	30	1.25	4.84	3.9	4.3
3	C13029	15	0.10	0.81	8.1	n/a
4	C13088	24	1.87	5.75	3.0	n/a
5	C12072	24	n/a	1.91	n/a	n/a
6	E12158	24	1.13	7.04	6.2	n/a
7	H13216	24	0.57	2.96	5.2	n/a
8	G14188	16	n/a	0.22	n/a	n/a
9	D15128	21	0.58	2.98	5.1	1.2
10	F16009	18	0.31	3.24	10.4	13.7
11	B15042	54	1.90	6.36	3.3	n/a
12	E14053	12	0.12	0.64	5.3	30.4[a]

[a] Site No. 12 was surcharged due to a pipe blockage and not as a result of wet weather flows from the storm event.

Site No. 12 showed significant surcharging during the January 19-20, 2010 storm event. The surcharging may have been caused by a temporary pipe obstruction, most likely in the vicinity of the railroad undercrossing. Evidence of an obstruction was indicated by the pulsing pattern in measured depths and velocities found in the flow monitoring data at Site 12. This pattern was present from January 13-19, 2010, and then ceased on January 20th. After January 20, 2010, the meter resumed a more conventional flow pattern.

Records show that SBSA conducted maintenance on the Menlo Park Pump Station (MPPS) on January 20, 2010. District staff expressed concern that this work may have caused the surcharging that was captured on January 20, 2010 at Site Nos. 1 and 2. After review of the MPPS wet well configuration, it appears that the influent pipeline operates in a surcharge condition during normal operations. Also, the District's FEF was utilized during the January 19 and 20 rainfall event, indicating that peak influent flows may have exceeded MPPS pumping capacity. Therefore, the surcharge condition measured during January 19 and 20 may have occurred even without concurrent maintenance activities.

West Yost and V&A reviewed flow data collected from Meters 1 and 2 on January 19 and 20, 2010. The data do not indicate the occurrence of backwater effects during this period. If backwater effects are not observed, then even in a surcharge condition, the meters will provide accurate flow data. Therefore, although the maintenance activities may have impacted the surcharge condition within pipelines and manholes upstream of the MPPS on January 19 and 20, 2010, the District was still successful in collecting accurate flow data from Meters 1 and 2 during this period.

Example flow hydrographs for each metering site, from January 18–23, 2010, are presented in Appendix B.

4.4.3 Supplemental Flow Monitoring Program

The District conducted supplemental wet weather flow monitoring subsequent to the 2009/10 flow monitoring program, and during the development timeframe of this Master Plan, with the objective of capturing additional flow data from the basins that likely exhibited the highest I&I in 2009/10. The flow data provide information that will be useful in the definition of future capital project alternatives for the District (and have been used by the District to refine options for pipe replacements along Atherton Avenue). However, because the 2010/11 wet weather events did not exceed the rainfall depths and intensities that were measured during the 2009/10 program, the information gathered was not instrumental in refining model results.

Figure 4-6 shows the locations and durations of the District's subsequent flow monitoring activities.

4.5 INFILTRATION AND INFLOW ANALYSIS

V&A completed an I&I evaluation based on the 2009/2010 flow monitoring data to quantify the potential extent of I&I entering the collection system by basin, during this period. This section summarizes these results as related to their relevance to the District's collection system master planning needs.

This section includes the following discussions:

- Inflow Analysis,
- Rainfall Dependent Infiltration Analysis, and
- Groundwater Infiltration Analysis.

4.5.1 Inflow Analysis

Inflow comprises water that is discharged directly into the sewer system from direct connections, such as downspouts and yard drains, as well as public and private storm drain systems. The effects of inflow can be seen in a collection system immediately following rainfall. Because of this quick response, inflow is typically quantified using peaking factors. A peaking factor is defined as the peak hourly wet weather flow divided by the average dry weather flow.

Inflow was evaluated based on a comparison of peak I&I to average dry weather flow, and also by calculating and comparing peak I&I per acre. Based on these comparisons, the basins were ranked from 1 to 10, with a ranking of 1 signifying the highest potential inflow. Overall basin ranking is included in Table 8 of the Flow Monitoring Report, and also in Table 4-5 below.

Table 4-5. Inflow Analysis

Meter Site No.	Manhole ID	Overall Inflow Ranking[(a)]	Comments
1	B16001	3 (tie)	
2	B15047	5 (tie)	
3	C13029	3 (tie)	
4 & 5	C13088	9	
6	E12158	5 (tie)	
7	H13216	8	
9	D15128	2	Middlefield Road along northwest service boundary. Inflow may be from direct household storm drain connections.
10	F16009	1	Atherton Avenue. Inflow may be from direct household storm drain connections.
11	B15042	10	
12	E14053	5 (tie)	

(a) Inflow rank does not necessarily translate to capacity issues within the sewers that flow tributary to the meters.

The basin with the highest potential inflow was monitored by Meter 10. This meter collected flow from Atherton Avenue, south of El Camino Real. The basin with the next highest potential inflow was monitored by Meter 9, located on Middlefield Road northwest of Watkins. In both of these locations, direct household storm drain connections may contribute to the measured stormwater inflow. Other potential sources include private business and City stormwater connections. These connections would be unpermitted, and therefore often unknown to the District.

4.5.2 Rainfall Dependent Infiltration Analysis

V&A used data from January 17 through 24, 2010 to quantify rainfall dependent infiltration. Using this data, V&A evaluated rainfall dependent infiltration by comparing infiltration per acre of drainage area, and infiltration as a percent of ADWF. The volume of infiltration is defined as the total flow volume minus the baseflow (or average dry weather flow) volume.

Basins were ranked from 1 to 10, with a ranking of 1 signifying the basin with the highest infiltration. Overall basin rankings are presented in Table 9 of the Flow Monitoring Report, and also in Table 4-6, below. Based on the comparison of rankings, the basin monitored by Meter 2, comprised of the area north of Highway 101 and adjacent to the San Francisco Bay, consistently displayed the highest overall potential I&I within the District's service area. The District should consider pipelines from this basin in its ongoing, long-term rehabilitation and replacement plan to systematically reduce RDII.

Table 4-6. Rainfall Dependent Infiltration (RDI) Analysis

Meter Site No.	Manhole ID	RDI Ranking, gallons per acre per day[(a)]	RDI Ranking (% of ADWF)[(a)]	Comments
1	B16001	3	5	
2	B15047	1	1	North of Highway 101, adjacent to San Francisco Bay
3	C13029	9 (tie)	9	
4 & 5	C13088	8	8	
6	E12158	5	2	
7	H13216	7	3	
9	D15128	2	7	Middlefield at Watkins also had high inflow.
10	F16009	6	4	
11	B15042	4	6	
12	E14053	9 (tie)	9	

(a) RDI rank does not necessarily translate to capacity issues within the sewers that flow tributary to the meters.

4.5.3 Groundwater Infiltration Analysis

GWI is typically considered a part of wastewater baseflow when it occurs in relatively small amounts compared to the flow generated from residential, commercial and public users. Small quantities of GWI are common in a collection system and are not usually considered problematic unless the volume of GWI flow becomes excessive.

Typically, wastewater flow generated in the system follows a predictable diurnal pattern. A diurnal pattern will peak in the morning between 8 and 11 am, and in the evening between 6 and 9 pm, and recede slightly in the afternoon and substantially in the middle of the night. When a basin has a large quantity of GWI occurring, the basin diurnal pattern has distinctly flatter peaks, with a noticeable quantity of flow occurring in the middle of the night.

V&A evaluated groundwater infiltration based on rate per acre and rate as a percentage of ADWF. For the GWI analysis, weekday ADWF was used to avoid the potential for large industrial and commercial weekend flow patterns to skew the results. Table 10 of the Flow Monitoring Report identifies one basin, monitored by Meter No. 2, as having approximately 0.3 mgd in excess GWI. The impact of GWI in the overall I&I analysis is discussed above.

Figure 4-1. Inflow and Infiltration Sources

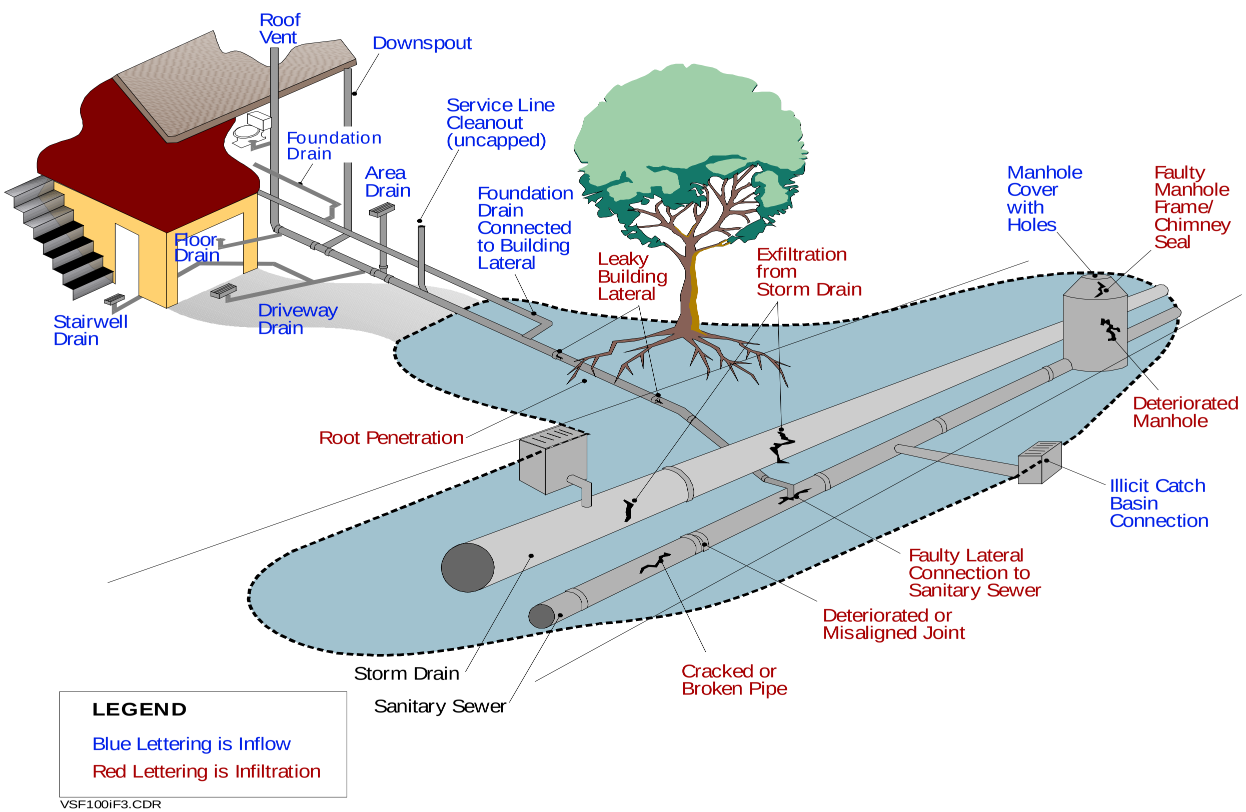
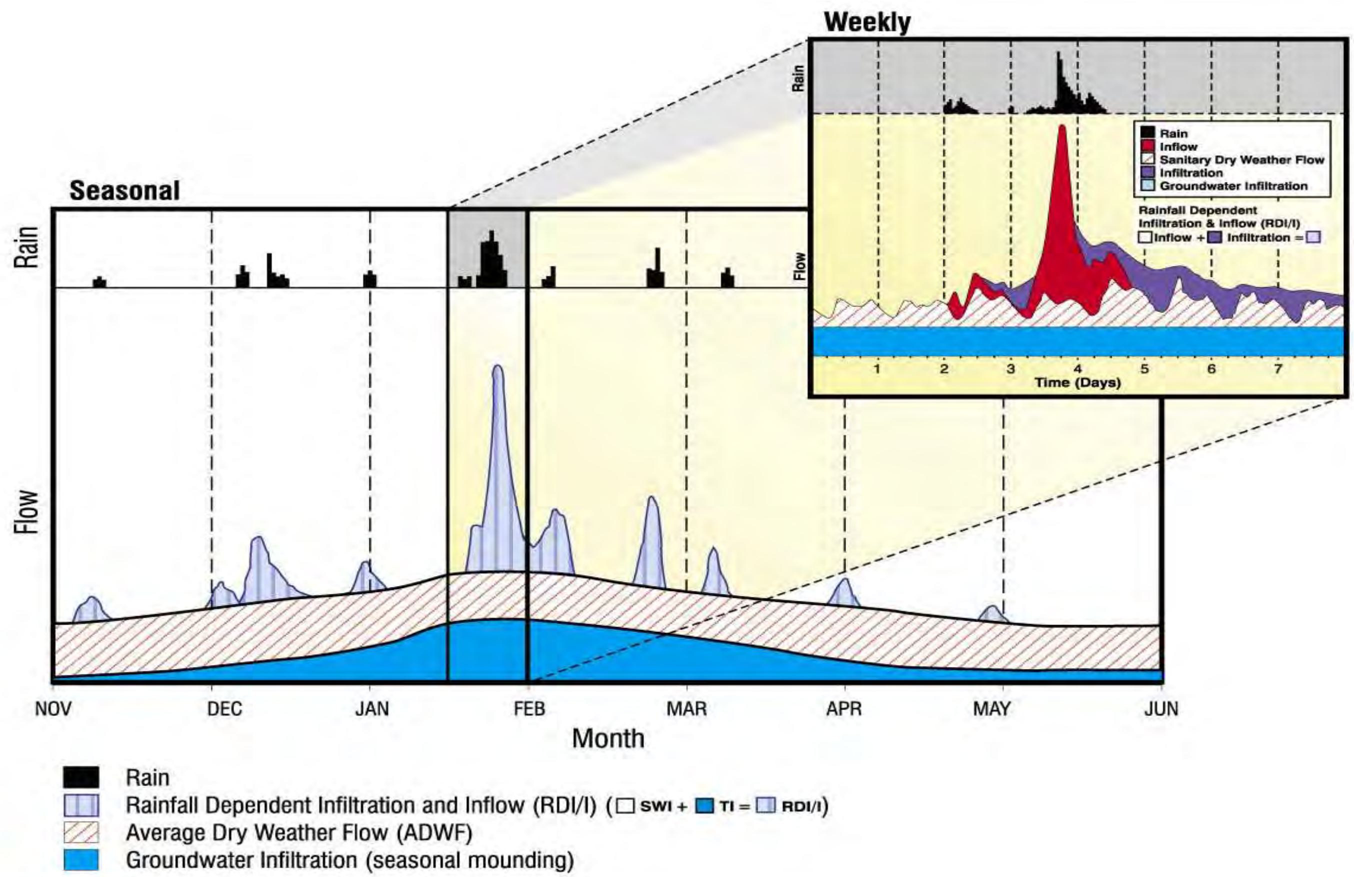
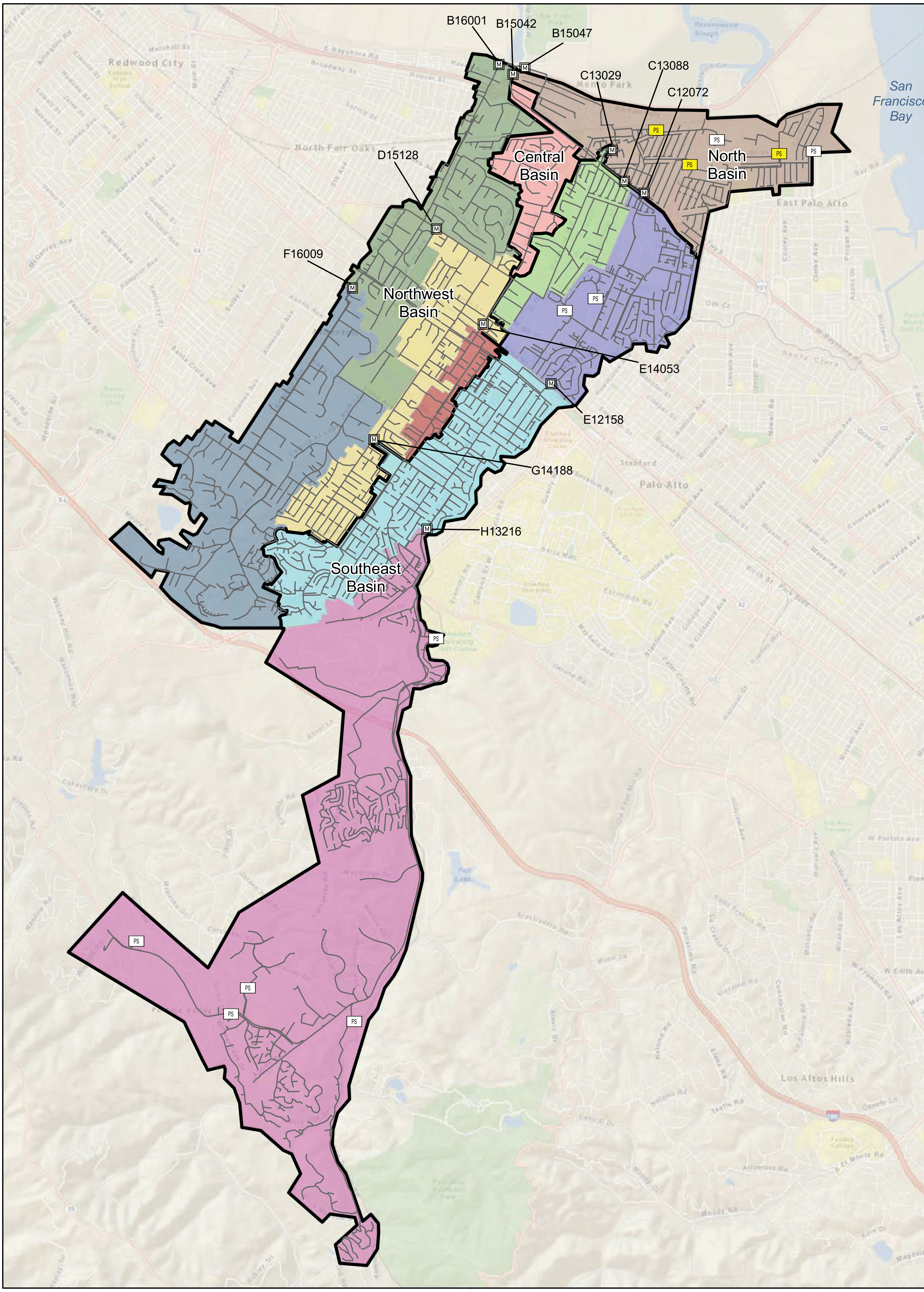


Figure 4-2. RDII Response





LEGEND

- Basin 1A
- Basin 1B
- Basin 1C
- Basin 1D
- Basin 1E
- Basin 2A
- Basin 2B
- Basin 2C
- Basin 3
- Basin 4

- Flow Meter / Manhole ID
- Modeled Pump Stations
- Pump Station
- Sewer Pipes
- District Basin Boundaries

Basin Key	
Central	3
North	1A
Northwest	2A, 2B, 2C, 4
Southeast	1B, 1C, 1D, 1E

N

0 0.5 1

Miles

FIGURE 4-3

West Bay Sanitary District

Collection System Master Plan

Flow Monitoring Basins

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES

Consulting Engineers

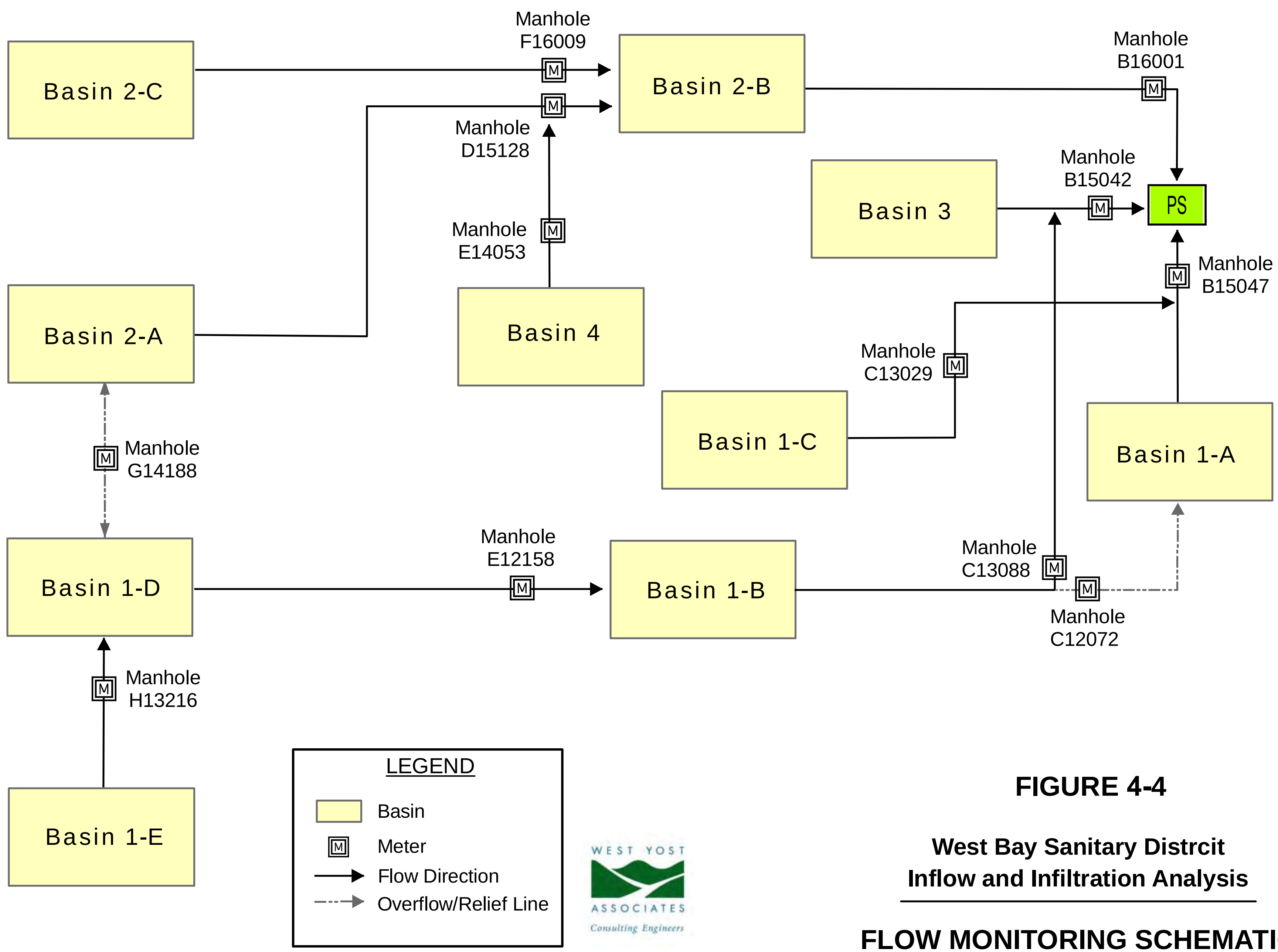
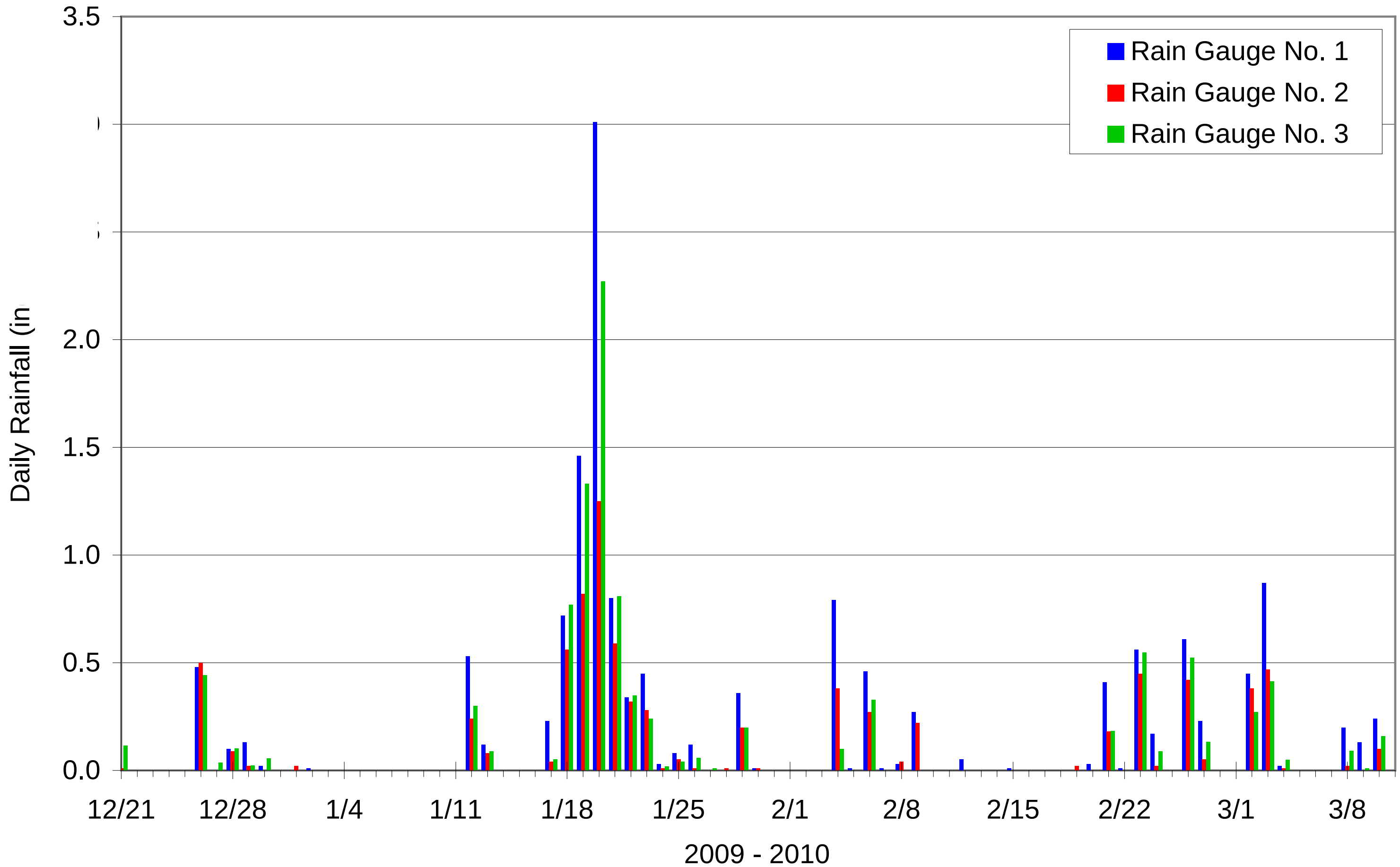


FIGURE 4-4

West Bay Sanitary Distrcit
Inflow and Infiltration Analysis

FLOW MONITORING SCHEMATIC

Figure 4-5. Rainfall Data from Flow Monitoring Period



O:\Clients\453 West Bay Sanitary Dist\02-10-03 Collection System Model MP & Fee Estimate\GIS\Figures\Figure 4-6 2010-11 Supplemental Wet Weather Flow Monitoring.mxd 6/27/2011

LEGEND

Modeled Pipes
- 6 - 8"
- 10 - 12"
- 15 - 21"
- 24 - 30"
- >30"
- Force Main
- Sewer Pipes
- Parcels
- Flow Meter
- Mahole

N

0 1,000 2,000

Scale in Feet

FIGURE 4-6

West Bay Sanitary District

Collection System Master Plan

2010/11 Supplemental Wet Weather Flow Monitoring

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

Hydraulic Model Development

The computer-based hydraulic sewer model of the District's wastewater collection system, developed using Innovyze® InfoWorks™ CS software, serves as a tool for assessing the flows and capacities of the District's major sewers, and for identifying solutions to identified potential capacity issues. The hydraulic model is also a tool for performing "what if" scenarios to assess the impacts of future developments, land use changes, and system configuration changes. The hydraulic model includes the District's main trunk sewers and associated facilities, and is a simplified representation of the District's total sewer system in its configuration and operation. Hydraulic model development typically focuses on trunk sewers only. The District's model also includes some smaller diameter sewers to assess anticipated potential capacity needs in the neighborhood collector sewers.

This chapter presents a summary of hydraulic model development and calibration. The major sections of this chapter include:

- Model Development,
- Data Validation,
- Field Investigations,
- Load Allocation,
- Dry Weather Flow Calibration, and
- Wet Weather Flow Calibration.

5.1 MODEL DEVELOPMENT

The District's hydraulic model transforms information about the physical and operational characteristics of the sewer system into a mathematical model. The model solves a series of differential equations for continuity and momentum (Saint-Venant equations) to simulate various flow conditions for specified sets of flow loads. The modeling results provide information on flows, flow depth, velocity, surcharging, and backwater conditions that are used to analyze system performance and identify possible system deficiencies. The model is also used to verify the adequacy of recommended or proposed system improvements.

The hydraulic model comprises a skeletonized network of nodes (*e.g.*, manholes) and links (*e.g.*, pipelines). Several types of nodes and links are used for defining the physical entities within a collection system. The following descriptions provide additional information on elements used in the development of the District's model.

<u>Node</u>: Nodes represent manholes, split manholes, diversion structures (with no other physical component such as a weir), storage facilities, and outfalls in a collection system. Storage facilities include lift station wet wells and off-line storage (*i.e.*, equalization basins). All flows loaded into the model are attached to a node structure. The data required for node structures include elevation data (pipe invert and manhole rim) and manhole diameter.

Conduit: Conduits represent facilities that convey wastewater from one point in the system to another. Conduits include gravity pipes, force mains, pumps, and weirs. Several different types of pumps and weir structures are available as standard elements. The physical data for gravity pipes and force mains include invert elevation data, size, length, and friction factor. The physical data for pumps include type of pump, elevation data, head-discharge relationship, and operational parameters such as on/off elevations and sequencing.

Sewersheds or subcatchments: Subcatchments represent an area that is tributary to an individual node in the model. Subcatchments usually represent a particular subdivision or grouping of parcels that connect into one location along a major trunk sewer. The subcatchment layer serves several purposes, including defining land use, diurnal curves, and dry and wet weather flow inputs. The data required for subcatchments are node connection, land use, flow factors, total and contributing area, diurnal curve profile, rainfall profile, inflow & infiltration parameters, and groundwater parameters.

5.1.1 Model Description

The hydraulic model system configuration was developed using the District's GIS pipe, manhole, and lift station layers as well as other information obtained from the District, such as pipeline invert and manhole rim elevations, pipeline diameter and pipeline length data.

The District's hydraulic model consists of approximately 37 miles of sewer pipeline ranging in diameter from 6-inches to 54-inches. The model includes all 12-inch diameter and larger trunk lines, and associated manholes and lift stations. Many 10-inch diameter pipelines have been included, as requested by District staff, as well as 6-inch and 8-inch diameter lines in selected areas as needed to provide connectivity. The 37 miles of pipeline represent approximately 18 percent of the District's 200-mile system.

Three of the District's twelve pump stations are included in the hydraulic model: Hamilton and Henderson, Willow, and University Avenue Pump Stations. The pump station parameters in the model are summarized in Table 5-1. The modeled collection system facilities are presented in Figure 5-1.

The District's collection system flows terminate at the MPPS, which is operated by SBSA. The MPPS was not included as an element in the hydraulic model. In place of this pump station, an outfall node was placed at the location of MPPS to quantify the District's overall flows entering either the regional collection and treatment system or the District's wet weather storage or FEF.

Early analysis results revealed that total system flows were higher than anticipated, and higher than the planned capacity of the MPPS (as determined by SBSA). Based on this finding, the District should consider revising the hydraulic model to include the MPPS as an operating pump station. By including the MPPS, the District would be able to more readily assess options for handling the anticipated system flows.

Table 5-1. Pump Station Model Parameters

Lift Station	Wet Well		Pumps					Force Main	
	Size	Base Elevation, ft	No. of Pumps	Speed	Lead/Lag Pump On Elevation, ft	Lead/Lag Pump Off Elevation, ft	Pump Discharge Rate, gpm	Diameter, inches	Length, ft
Hamilton & Henderson	11 ft diameter by 20 ft deep	82.5	2	Fixed	94/95.25	84.5/84.5	2,500	12	1,850
Willow	12 ft x 12 ft by 20 ft deep	77.4	2	Fixed	86.4/87.4	81.5/81.5	1,450	10	735
University Avenue	8 ft diameter by 27.5 ft deep	81.0	2	Fixed	93.8/84.7	88.2/88.2	300	8	616

5.2 DATA VALIDATION

After the model network was constructed, West Yost conducted data validation to confirm that the model comprised a fully-connected network. Data validation included the following steps:

- Ensure each pipe and manhole has a unique identifier;
- Check the modeled network for connectivity, and add smaller pipes as needed to ensure no missing links or manholes in the network;
- Check for missing or inconsistent data such as missing manhole rim or pipe invert elevations, negative pipe slopes, or abrupt elevation changes;
- Identify manholes with more than one outlet pipe, constituting a potential flow split, that require further investigation in the field;
- Populate global parameters such as standard manhole diameters and Manning's "n" coefficient, which is entered as 0.013 for sewer pipelines; and
- Use system flags provided in InfoWorks™ CS to document identified issues and any changes made to the model.

Table 5-2 lists the system flags used during model verification.

Table 5-2 System Flags

Flag	Source
#CD	CAD data
#D	Model Default Values
#FD	Field Investigations
#FL	Freyer & Laureta Staff
#GS	GIS Data
#WB	WBSD Staff
#WY	West Yost Staff

5.3 FIELD INVESTIGATIONS

Following data validation, West Yost reviewed the District's GIS pipeline, manhole, and pump station files further to confirm the locations of diversion structures, validate network connectivity, identify inconsistent elevation data, and locate system anomalies. Eleven manholes in four general areas required field investigations in order to better understand system configuration and hydraulics, or validate GIS data. The following is a summary of the field investigation work.

- Site 1 – Manhole No. B12047 on Ivy Drive & Sevier Avenue. An inspection was performed in order to verify an offset at the upstream pipe connection, which was indicated by pipe elevation data from the District's "linedata" database. The field investigation revealed no offset in invert elevation for the 15-inch diameter upstream pipe. The observed flow through the manhole was steady and slow.
- Site 2 – Manhole No. E13068 on El Camino Real & Santa Cruz Avenue. An investigation was performed to verify the configuration and flow split at Manhole No. E13068. The inspection revealed no flow split. Most, if not all of the flow, continues along Santa Cruz Avenue in the 12-inch diameter sewer. Very minimal to no flow travels southeast toward the 8-inch pipeline along El Camino Real. For hydraulic modeling purposes, it was assumed that no flow would be diverted to the 8-inch diameter sewer.
- Site 3 – Manholes Nos. C16040, C16041, and D16005 along Marsh Road. Three manholes were inspected along Marsh Road. Manhole Nos. C16040 and C16041 were inspected to determine if they were hydraulically connected to divert flows. The inspection determined that the two manholes are not connected. There was no evidence of a pipe between Manhole Nos. C16041 and C16040, and Manhole No. C16040 appeared to be the beginning of a new sewer line. There was very slow, minor flow in Manhole No. C16040.

  Manhole No. D16005 was investigated to determine if the 12-inch sewer from the southeast was offset as it entered the main 24-inch diameter trunk sewer along March Road. It was very difficult to see into this manhole due to the depth of the sewer to determine if there was an offset. The invert elevation data, supplied by the District, was used that matched inverts at this location.
- Site 4 – Manhole Nos. E14049 and E14034 on Laurel Street & Oak Grove Avenue. The purpose of this investigation was to verify the configuration and possible flow split at Manhole E14049. There was no evidence of a flow split. There was no connection northeast of Manhole E14049 into a parallel sewer along Oak Grove Avenue. Manhole E14034 was also inspected to determine if there was a sewer pipe coming into the manhole from the southwest. Neither Manhole No. E14049 nor E14034 included a pipe connecting the two manholes. In addition, flow from the sewer along Pine Street appeared to be the only flow entering the Manhole No. E14034.
- Site 5 – Manhole No. C12073 on Pierce Road & Hollyburne Avenue. The purpose of this inspection was to verify the hydraulics through Manhole No. C12073. The inspection determined that flow monitoring data gathered at Manhole No. C12072 define the hydraulics through the collection system model at Manholes Nos. C12072 and C12073.

The flow entering Manhole No. C12073 was fast and cascaded into the manhole approximately one to two feet. The flow through the manhole was very turbulent, circling and then exiting the manhole northwest along Pierce Road. It does not appear that much flow continues northwest along Hollyburne Ave. The existing pipe from Manhole No. C12073 along Hollyburne Avenue was also offset. However, the extent of the offset is unknown due to the volume of flow and turbulence in the manhole at the time of inspection.

A flow monitor was installed, downstream of Manhole No. C12072 to capture any flow diverted to the 24-inch diameter Hollyburne sewer. During the installation of the flow meter, V&A located a weir structure in the pipe between Manhole Nos. C12073 and C12072. The height of the weir structure was 8 to 12-inches high. Photographs of the weir structure were presented in the V&A report titled, *Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, 2010* (included in Appendix A) and are shown in Figure 5-2.

- <u>Site 6 – Manhole Nos. H12074 and H12024 on Leland Avenue & Sand Hill Road</u>. The purpose of this investigation was to confirm whether a pipeline exists between Manhole Nos. H12024 and H12074, and if flow could be routed toward Leland and Perry Avenue. The field work validated that there is no pipe connecting these two manholes. Manhole No. H12024 is the beginning of a new line along Leland Avenue.
- <u>The Site 7 – Manhole Nos. D14106 and D14111, on Oak Grove Avenue & Middlefield Road</u>. The purpose of the inspection was to verify the configuration and connectivity between Manhole No. D14106 and Manhole No. D14111. Manhole No. D14106 could not be found and, according to District maintenance staff, was thought to be paved over. Upon inspecting Manhole No. D14111, there was no pipe connecting the two manholes. Flow toward Manhole No. D14111 was fast and turned northwest, while flow traveling southeast through Manhole No. E13002 was minimal and slow, and representative of flows generated from several lateral connections along Middlefield Road south of Oak Grove Avenue.

Field investigation sites 1 through 7 are shown in Figures 5-3 through 5-9, respectively.

5.4 FLOW ALLOCATION

This section summarizes how sewer flows were calculated and input into the computerized hydraulic model. Wastewater flows for analysis and design of sanitary sewers can be divided into three categories. All of these flows are discussed further in this section:

- BWF includes the sanitary flow contribution from permitted connections to the collection system;
- GWI is generally caused when flows from a high groundwater table infiltrate the system through defects in the system, during dry weather and wet weather periods; and
- RDII may result when flows from wet weather events infiltrate the system, either through defects in existing facilities, or unpermitted connections that convey stormwater into the sewer system.

Wastewater flows were estimated by sewershed or subcatchment, and assigned to the node at the downstream end of the subcatchment. West Yost digitized 211 sewersheds to facilitate the assignment of sewer flows in the hydraulic model. Each sewershed defines a geographic area where baseflow generated in the area is assigned to a specific node (or manhole) in the model. Each sewershed encompasses a particular subdivision or grouping of lots that flows to a single point in the collection system. Figure 5-10A and Figure 5-10B show the sewersheds that were included in the hydraulic model.

5.4.1 Dry Weather Flow Generation

This section describes the tasks completed to calculate dry weather flows.

5.4.1.1 Base Wastewater Flows

BWF can be calculated based one or more factors, including population, population density, water consumption, and land uses. For the District's hydraulic model, the District's unit flow factors, as documented in design standards, were applied to land use to generate BWF. The land use and unit flow factors are described further in Chapter 2, Existing Wastewater System. The key elements of dry weather flow generation in the hydraulic model include:

- Average dry weather flow (Q_a)
- Peak dry weather flow (Q_{pdwf})

The residential Q_a calculation is based on the land use data provided by Menlo Park, as described in Chapter 2. The number of residential dwelling units (DU) was input into each subcatchment in the "Population" field of the hydraulic model. District design flow factors were then input into the "Per capita flow (US gallons per day)" field in the hydraulic model. The population and per capita flow values were multiplied to generate Residential Q_a, per subcatchment. Although the hydraulic model database fields are titled "population" and "flow per capita", the values for these fields are "dwelling units" and "flow per dwelling unit", respectively.

The non-residential land use Q_a was into the "Trade Flow" field in hydraulic model, per subcatchment. The flow units for "trade flow" are million gallons per day. Non-residential land use flow included flow contributions from all land use categories with a non-residential designation.

The hydraulic model provides separate inputs representing build-out for the majority of the district service area. Vacant parcels were assigned flows after DWF calibration was completed, and the total flow for each subcatchment was provided in the "additional foul flow" field within the model. The "additional foul flow" field provides a constant flow without a diurnal pattern, but allows build-out flows to remain separate from existing flows. Since build-out flows were generally very small for most of the service area, with the exception of Portola Valley, this approach was found acceptable. For the City of Portola Valley, build-out flows were very large, requiring a diurnal pattern for accurate results. Since a separate diurnal pattern could not be applied to a discrete build-out field, build-out equivalent dwelling units (EDUs) for the City of Portola Valley are included in the Population field for this area. Two user defined fields were incorporated into the model database to track existing and buildout EDUs, named "Existing Pop" and "Additional Build-out Pop" respectively. The diurnal patterns developed during calibration apply to these combined existing and build-out flows.

West Yost refined these unit flow factors by calculating the overall flow generated from the District's service area for each of the basins monitored during the District's 2009/10 flow monitoring program. Average daily flows per basin were then compared with the metered flow data and adjusted per land use category and per monitored basin, until predicted BWF generally matched measured data throughout the entire service area. The total calculated BWF of 4.4 mgd or approximately 83 gallons per capita per day (gpcpd) is within industry standard and closely matches the District's design criteria of 85 gpcpd.

5.4.1.2 Diurnal (24-Hour) Flows

BWF typically varies throughout the day, with the peak flow generally occurring in the morning and evening periods. V&A Consulting generate 24-hour weekday and weekend diurnal patterns for each monitored basin within the District's service area. Data was derived from flows collected in 15 minute increments, 24 hours per day, for a 3-month period that included dry weather.

A sample weekday diurnal curve is presented in Figure 5-11 for Basin 2-B. A complete set of diurnal curves from all flow monitors is included in Appendix C. Diurnal flow characteristics were applied to the individual land use Q_a, within each monitored basin to distribute the Q_a over a 24-hour period. Weekday diurnal patterns were used for the dry weather flow calibration.

In order to reliably compare calculated-to-measured flow values, contributions to GWI and other sources of infiltration were considered and added to individual basin flows on a case by case basis. The highest levels of GWI in the District's system are located in Basin 1-A, as further discussed in Chapter 4.

5.4.2 Wet Weather Flow Generation

Extraneous water may enter the sewer system during wet weather periods through cracks and open joints in sewer mains, manholes, and building laterals, as well as through direct connections between storm drains and the sanitary sewer, or from illegal drainage connections on private property. These extraneous flows may cause significant increases in peak flows in the system. Wet weather flows were calculated and input to the District's hydraulic model to replicate measured flow data. The key elements of wet weather flow generation in the model include:

- Rainfall Dependent Infiltration and Inflow (RDII or I&I)
- Peak Wet Weather Flow (Q_{pwwf})

Several broad categories of RDII quantification are used in wastewater master planning, including the following:

- The constant unit rate method calculates RDII as a fixed constant (*e.g.*, gal/acre·in rainfall) multiplied by measurements of tributary sewershed characteristics (*e.g.*, area, land use, population, pipe diameter, pipe length, and pipe age);
- The R-Value method calculates RDII as a fixed percentage of rainfall;
- Synthetic unit hydrograph (SUH) method calculates the RDII hydrograph from a specified "unit" hydrograph shape that relates RDII to unit precipitation volume and duration;
- Probabilistic method calculates RDII of a given recurrence interval from long-term sewer flow records using probability theory. The method estimates the relationship of peak RDII flow to recurrence interval; and
- Rainfall/sewer flow regression method estimates peak RDII flows from rainfall data through a relationship between rainfall and RDII flows. This regression, expressed as an equation, is derived from rainfall and flow monitoring data in sewers using multiple linear regression methods and considering dry and wet antecedent conditions.

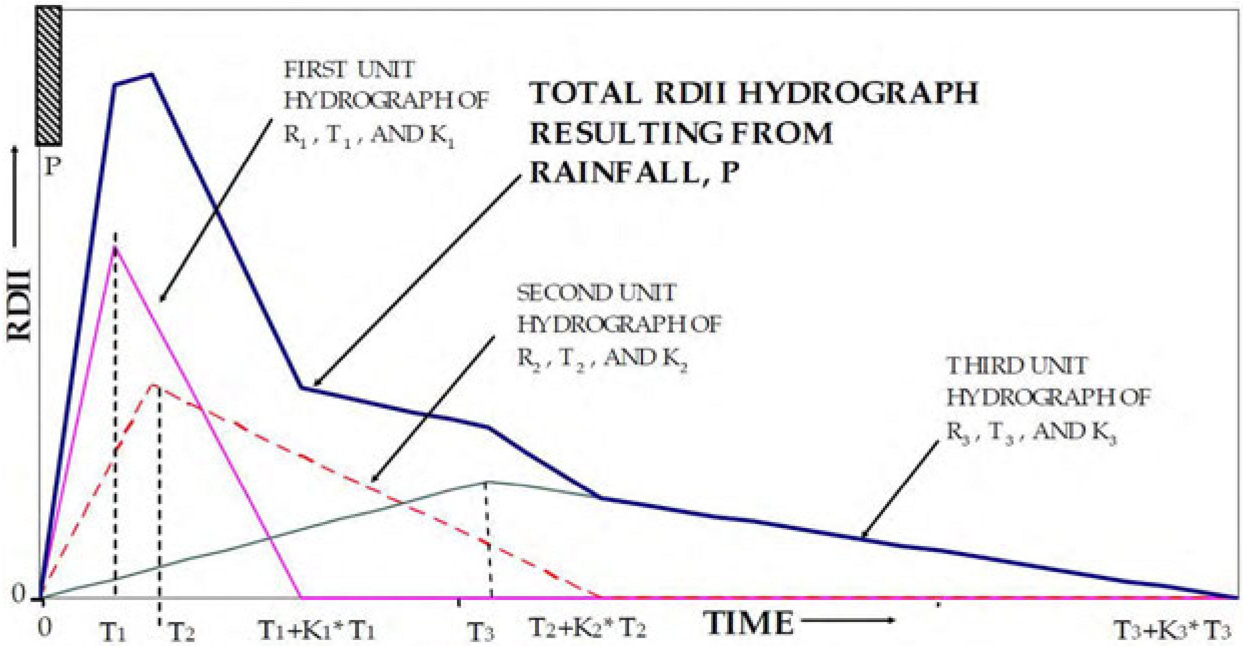
Studies conducted by the Water Environment Research Foundation (WERF) have concluded that the SUH and rainfall/flow regression methods are the two most accurate methods for predicting peak flows and event volumes for storm events. The RTK method is the most widely used SUH prediction methodology for collection system model development.

West Yost used the RTK method to calculate RDII inputs to the District's hydraulic model. The RTK method generates hydrographs from each subcatchment that represent estimated flows during and immediately after rainfall events caused by potential seepage of water into the collection system. The RTK method generates a series of three triangular hydrographs that represent short-term, medium-term, and long-term rainfall response. The RTK parameters include:

1. R is the area of the graph representing the portion of rainfall falling on a subcatchment that enters the sewer collection system.
2. T is the time from the onset of rainfall to the peak of the triangle.
3. K is the ratio of the "time to recession" to the "time to peak" of the hydrograph.

Components of the RTK hydrograph are provided courtesy of the EPA Office of Research and Development, and are presented in Figure 5-12.

Figure 5-12. Components of RTK Hydrograph



When a wet weather flow simulation is run in the model, the RTK parameters are applied to represent a specific rainfall event. These parameters generate a wet weather flow hydrograph for each sewershed.

Hourly peak wet weather flows (Q_{pwwf}) are generated in the model by combining the dry weather flow with flows from the I&I hydrographs, by sewershed. Typically, the peak wet weather flow will occur shortly after the hourly peak intensity of the rainfall event.

5.5 DRY WEATHER FLOW CALIBRATION

The District's hydraulic model was calibrated to confirm that the computer simulation will accurately estimate the operation of the collection system under dry weather flow conditions. The major steps in the dry weather flow calibration included the following:

4. Determine the average dry weather 24-hour flow (Q_a) for the entire service area for the calibration period.
5. Determine Q_a at each flow metering site. For each metering site, establish which sewersheds correspond to the tributary area upstream of the flow meter.
6. Compare the modeled Q_a values with the measured Q_a values for the entire service area and at each flow metering site.
7. Adjust the model flow factors to maximize agreement between the modeled and metered Q_a 24-hour values.
8. Adjust the diurnal curve for each basin to maximize agreement between the modeled and weekday metered Q_a hourly values.

5.5.1 Calibration Results

The calibration steps listed above were conducted using the base dry weather flow hydrographs developed by V&A as the baseline for flow. Calibration was considered completed when minimum, maximum, and average modeled flows as well as the temporal distribution of flow over a 24-hour period were within five percent of measured flows. Exceptions were made for basins that have very low dry weather flows, and for which any minor increment in flow constitutes a large percentage change.

A sample dry weather flow calibration hydrograph for Basin 2-A is provided in Figure 5-13. The remaining DWF calibration hydrographs are presented in Appendix D. The weekday dry weather flow calibration results for each meter are presented in Table 5-3. The metering locations and basin delineations were presented in Chapter 4, in Figure 4-3.

Table 5-3. Weekday Dry Weather Flow Calibration Results

Flow Meter	Meter			Model			Calibration Difference		
	Minimum Flow, mgd	Maximum, Flow, mgd	Average Flow, mgd	Minimum Flow, mgd	Maximum, Flow, mgd	Average Flow, mgd	Minimum Flow, mgd	Maximum, Flow, mgd	Average Flow, mgd
Site 1: B16001	0.57	2.12	1.43	0.51013	2.12	1.41	10.5%	0.2%	1.4%
Site 2: B15047	0.77	1.58	1.26	0.64294	1.84	1.28	16.5%	16.3%	1.4%
Site 3: C13029	0.04	0.17	0.11	0.03898	0.17	0.10	2.6%	0.9%	6.8%
Site 4: C13088	0.69	2.82	1.87	0.70967	2.83	1.88	2.9%	0.3%	0.7%
Site 5: C12072[a]	0	0	0	-0.00001	0.00	0.00	n/a	n/a	n/a
Site 6: E12158[b]	0.35	1.81	1.13	0.34418	1.86	1.14	1.7%	2.5%	0.5%
Site 7: H13216	0.23	0.93	0.58	0.20682	0.92	0.57	10.1%	1.3%	1.9%
Site 8: G14188[c]	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Site 9: D15128	0.15	0.93	0.58	0.14007	0.97	0.58	6.6%	3.8%	0.8%
Site 10: F16009	0.11	0.5	0.31	0.1116	0.49	0.31	1.5%	1.1%	0.0%
Site 11: B15042	0.96	3.18	2.07	0.75721	2.97	1.99	21.1%	6.5%	4.1%
Site 12: E14053	0.026	0.18	0.11	0.02748	0.20	0.12	5.7%	9.4%	9.7%

(a) Site 5 monitored flows downstream of a diversion structure with an approximate 1-foot weir. The depth of flow was below the height of the weir during dry weather periods.

(b) Basin 1-B and 1-D were combined (unit flow factor calculation, not the diurnal curves) for the dry weather flow calibration. It's suspected that Meter E12158 was recording low during the monitoring program.

(c) Site 8 monitored flow along a very flat cross-connection between the Valparaiso Avenue and Santa Cruz Avenue trunk sewers. Minimal flows occurred during dry weather periods and alternated direction of flow at different times during the monitoring period. See Villalobos & Associates *Flow Monitoring and Inflow/Infiltration Study, May 2010* for additional information regarding this metering site.

Weekend and weekday flows were similar in most basins. Overall, flows tended to decrease slightly on the weekend compared with the weekday flows. A comparison of average weekend and weekday flow for each meter is presented in Table 5-4.

Table 5-4. Weekday and Weekend Dry Weather Flow Comparison

Flow Meter	Meter	
	Weekday Average Flow, mgd	Weekday Average Flow, mgd
Site 1: B16001	1.43	1.35
Site 2: B15047	1.26	1.15
Site 3: C13029	0.11	0.10
Site 4: C13088	1.87	1.79
Site 5: C12072[(a)]	0	0
Site 6: E12158	1.13	1.09
Site 7: H13216	0.58	0.56
Site 8: G14188[(b)]	n/a	n/a
Site 9: D15128	0.58	0.59
Site 10: F16009	0.31	0.28
Site 11: B15042	2.07	1.96
Site 12: E14053	0.11	0.12

(a) Site 5 monitored flows downstream of a diversion structure with an approximate 1-foot weir. The depth of flow was below the height of the weir during dry weather periods.

(b) Site 8 monitored flow along a very flat cross-connection between the Valparaiso Avenue and Santa Cruz Avenue trunk sewer. Minimal flows occurred during dry weather periods and alternated direction of flow at different times during the monitoring period. See V&A Report for additional information regarding this metering site.

5.6 WET WEATHER FLOW CALIBRATION

Following completion of dry weather calibration, West Yost calibrated the model for wet weather flow (WWF) conditions. A model that is sufficiently calibrated to wet weather flow is then expected to simulate inflow and infiltration entering the sewer collection system during a rainfall event. WWF calibration consisted of the following steps:

- Identify a representative wet weather calibration event from the flow monitoring data. The event should represent a time period with significant rainfall, and without extensive flow anomalies that would impact the accuracy of calibration results.
- Establish the appropriate methodology for potential I&I generation. The District's model uses the RTK method.
- Estimate the contribution of wet weather flow that may enter the system using I&I parameters per monitored basin based on the selected methodology.

- Generate system flows using the selected rainfall data. Compare metered data with model simulation results, and adjust the estimated I&I calculation parameters if necessary, to maximize agreement to within five to ten percent for the calibration event. Match peak flows first, and also consider total volume and the temporal distribution of flows.
- After the modeled flows closely match metered flows, select a second storm to validate the accuracy of the calibration.

The largest storm event that was captured during the 2009/10 flow monitoring season occurred from January 18-23, 2010. The wet weather model calibration included the time period from January 19 to 20, 2010. After calibrating the model to flows from January 19-20, 2010, the analysis was extended several days to confirm that the modeled flows continued to track metered flows in subsequent rainfall events.

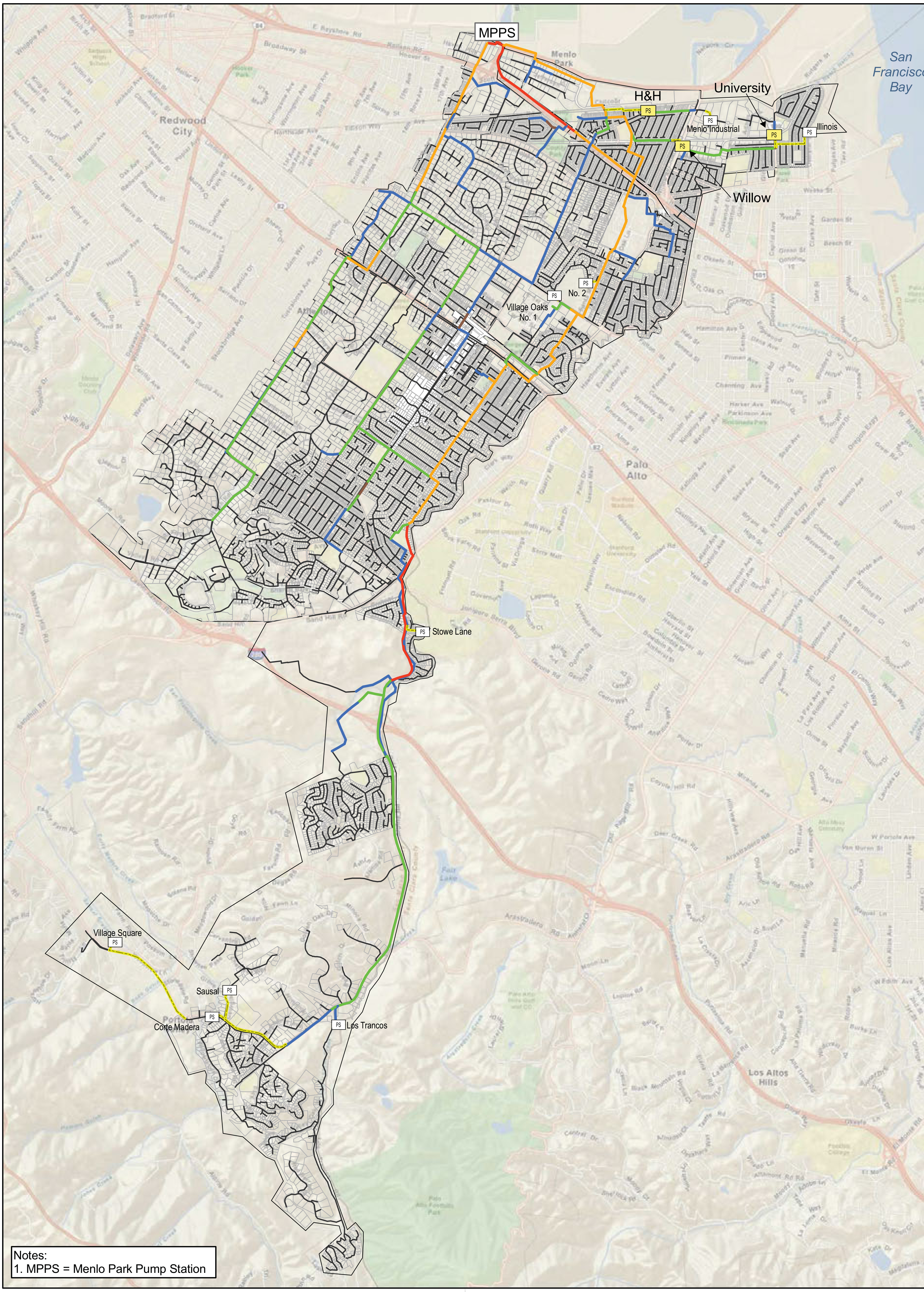
Wet weather flow calibration results are provided in Table 5-5. Figure 5-14 presents a graphical sample of successful wet weather flow calibration. The remaining calibration graphs are presented in Appendix E.

Table 5-5. Wet Weather Flow Calibration Results for January 19-20, 2010

Flow Meter	Meter Flow Volume, MG	Meter Peak 15-min Flow, mgd	Model Flow Volume, MG	Model Peak 15-min Flow, mgd	Percent Difference in Flow Volume, %	Percent Difference in Peak Flow, %
Site 1: B16001	3.74	8.64	3.66	8.56	2.1%	0.88%
Site 2: B15047	2.46	5.00	2.62	5.03	6.3%	0.51%
Site 3: C13029	0.23	0.83	0.25	0.85	5.4%	2.3%
Site 4: C13088	3.38	7.38	3.58	7.46	5.9%	1.1%
Site 5: C12072	Same as Site 4					
Site 6: E12158	2.52	6.61	2.60	7.04	3.4%	6.4%
Site 7: H13216	1.38	2.96	1.33	2.92	3.4%	1.5%
Site 9: D15128	1.21	2.88	1.26	2.97	3.5%	3.4%
Site 10: F16009	1.33	3.34	1.32	3.29	1.0%	1.6%
Site 11: B15042	3.61	6.59	3.60	6.54	0.14%	0.67%
Site 12: E14053	0.22	0.59	0.21	0.57	3.4%	2.3%

5.6.1 Hydraulic Model Calibration Findings and Conclusions

In summary, the results from dry and wet weather calibration are within allowable calibration parameters and indicate that the model is well calibrated to existing flow conditions.



LEGEND

Modeled Pipes

- 6 - 8"
- 10 - 12"
- 15 - 21"
- 24 - 30"
- >30"

- Modeled Pump Stations
- Pump Station
- Force Main
- Sewer Pipes
- Parcels

N

0 2,000 4,000

Scale in Feet

FIGURE 5-1

West Bay Sanitary District

Collection System Master Plan

Modeled Collection System

Figure 5-2.
Site Information Report

Monitoring Site:
Site 05

Upstream Overflow Weir

Upstream Overflow Weir

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LEGEND

- Inspected Manhole
- Manhole
- PS Pump Station
- Modeled Sewer Main
- Other Sewer Main
- Parcels

N

0 100 200

Scale in Feet

FIGURE 5-3

West Bay Sanitary District

Wastewater Collection System Master Plan

Site 1 - Manhole No. B12047

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

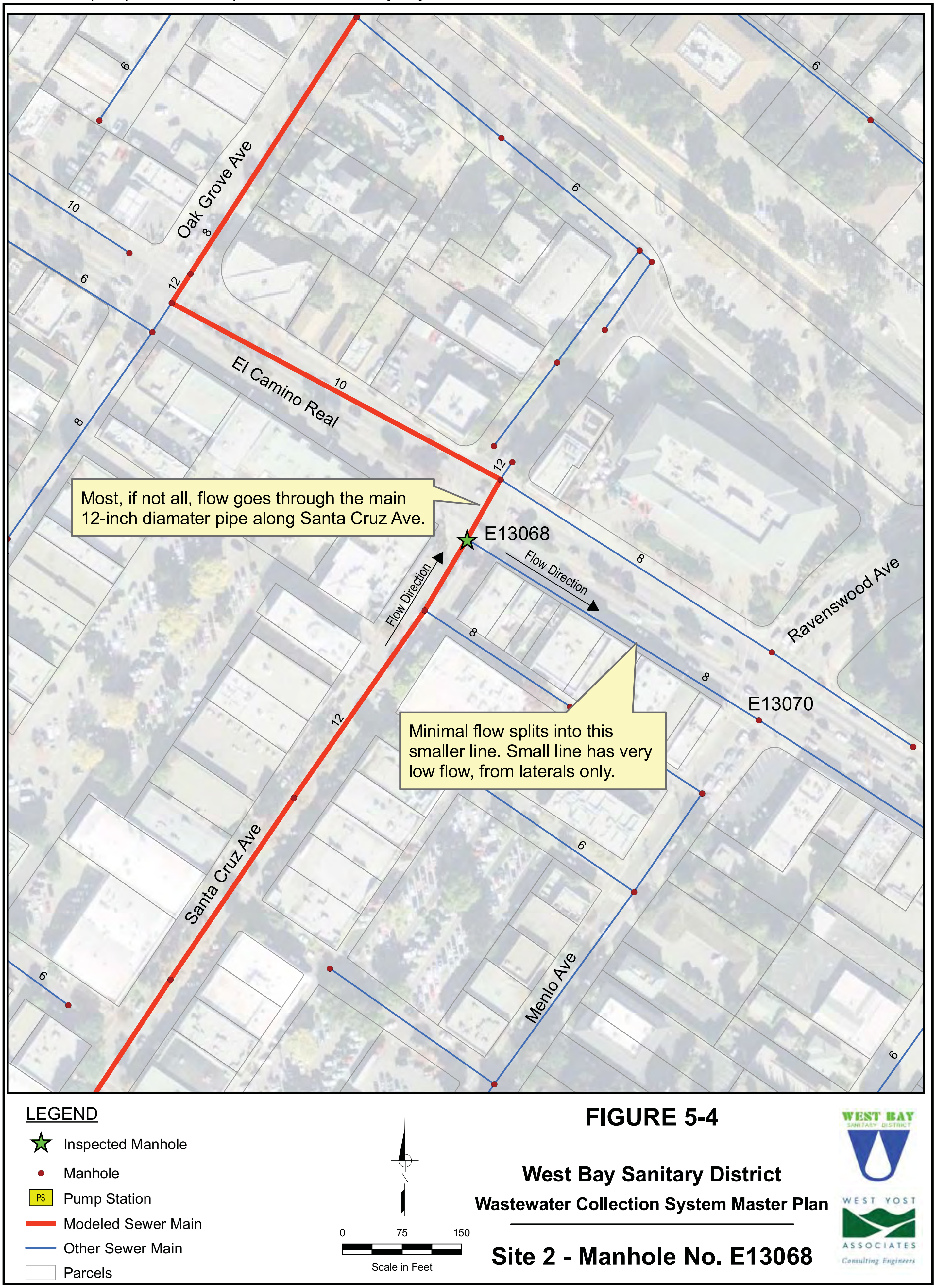
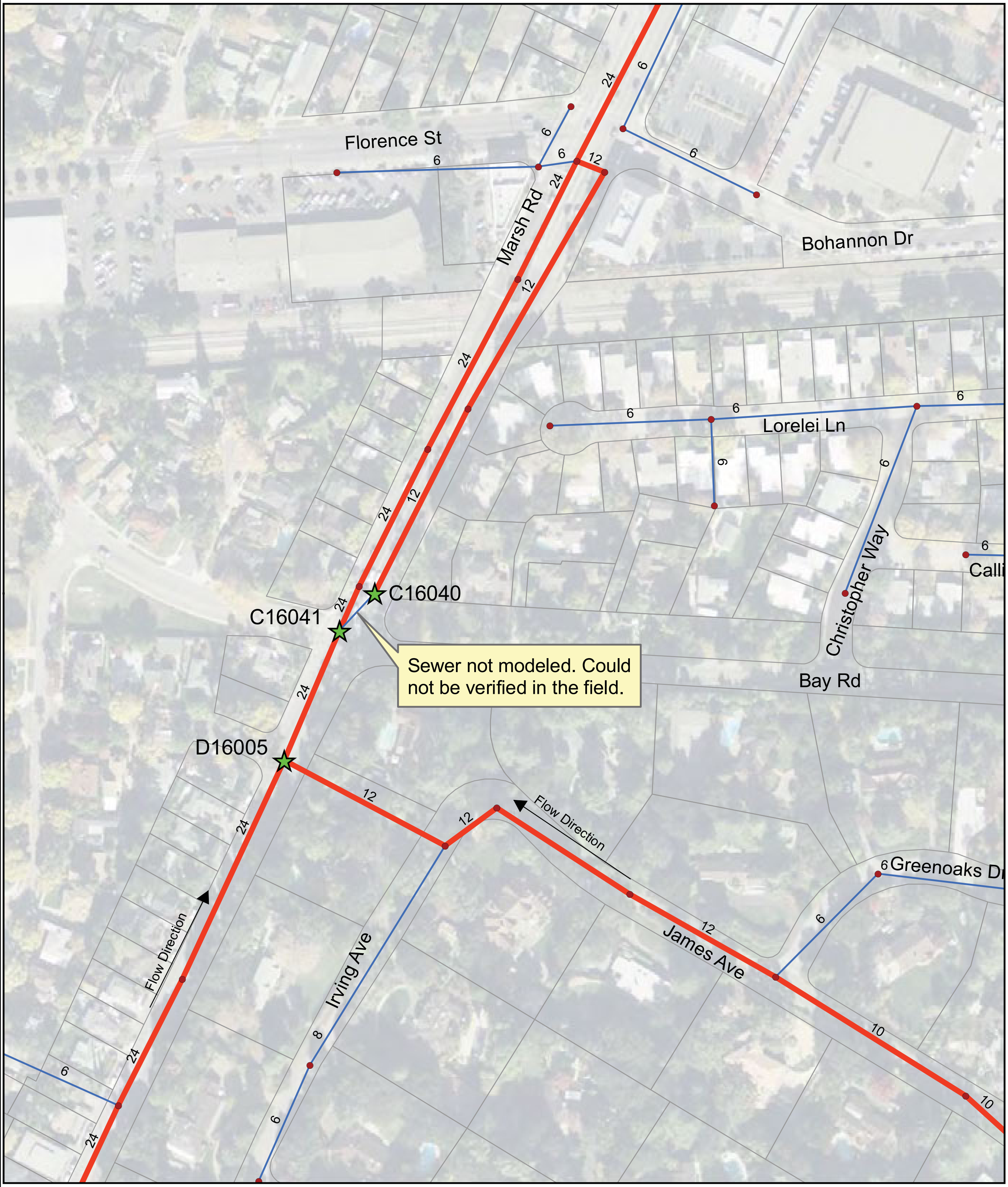


FIGURE 5-4

West Bay Sanitary District

Wastewater Collection System Master Plan

Site 2 - Manhole No. E13068



LEGEND

- Inspected Manhole
- Manhole
- PS Pump Station
- Modeled Sewer Main
- Other Sewer Main
- Parcels

N

0 75 150
Scale in Feet

FIGURE 5-5

West Bay Sanitary District
Wastewater Collection System Master Plan

Site 3 - Manhole No. C16040, C16041, D16005

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

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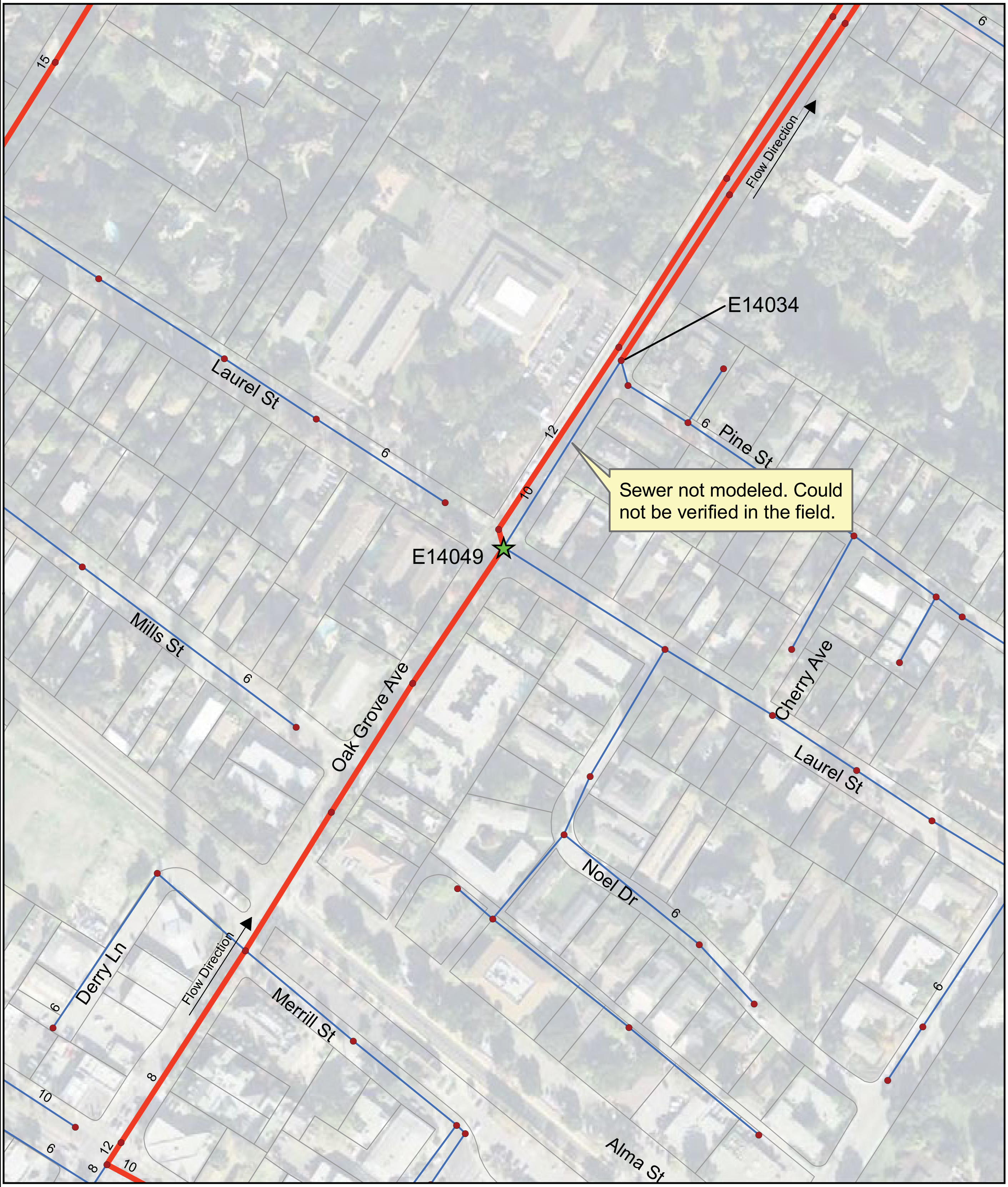


LEGEND

- Inspected Manhole
- Manhole
- PS Pump Station
- Modeled Sewer Main
- Other Sewer Main
- Parcels

N

0 75 150

Scale in Feet

FIGURE 5-6

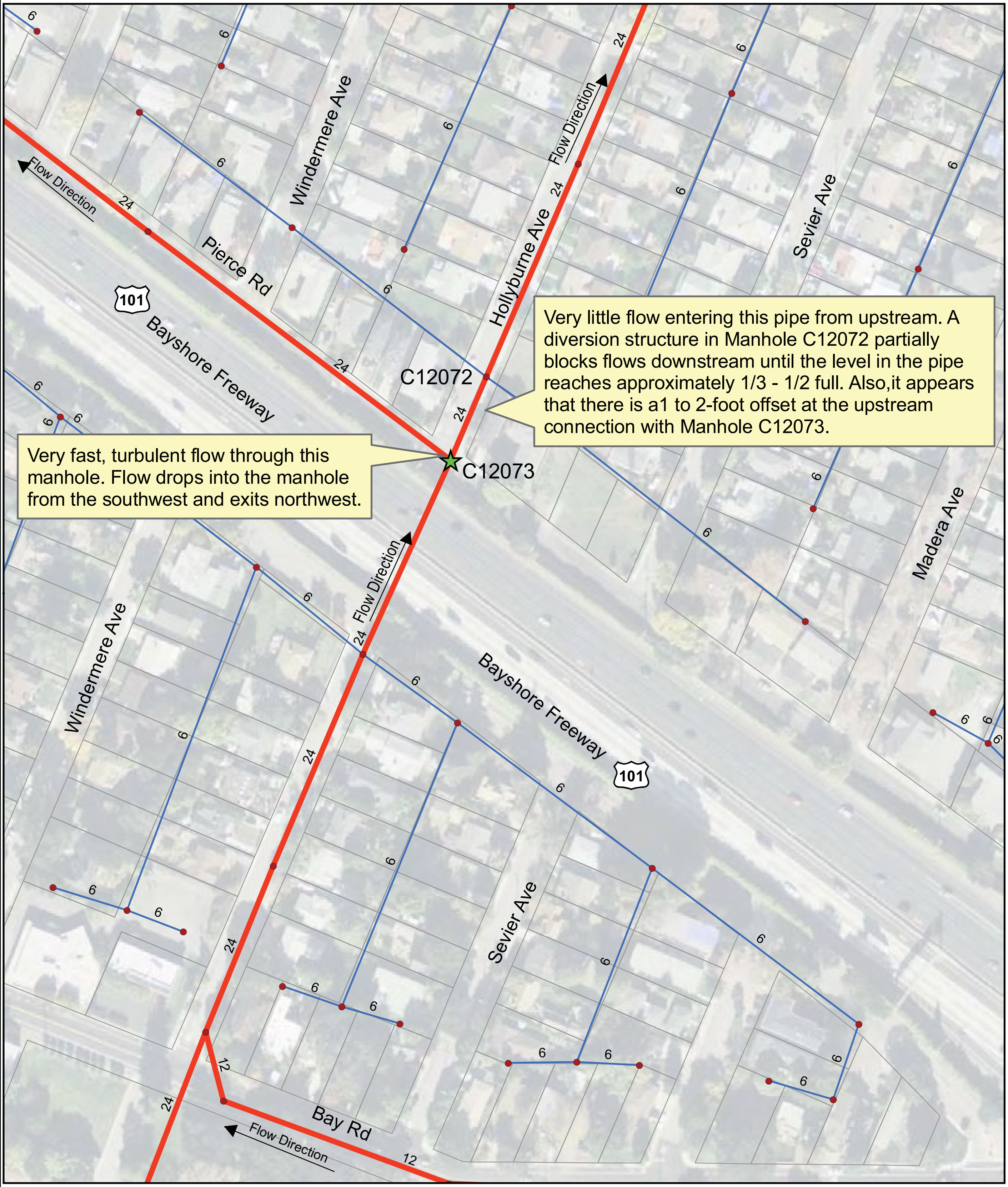
West Bay Sanitary District

Wastewater Collection System Master Plan

Site 4 - Manhole No. E14049 and E14034

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

- Inspected Manhole
- Manhole
- PS Pump Station
- Modeled Sewer Main
- Other Sewer Main
- Parcels

N

0 75 150

Scale in Feet

FIGURE 5-7

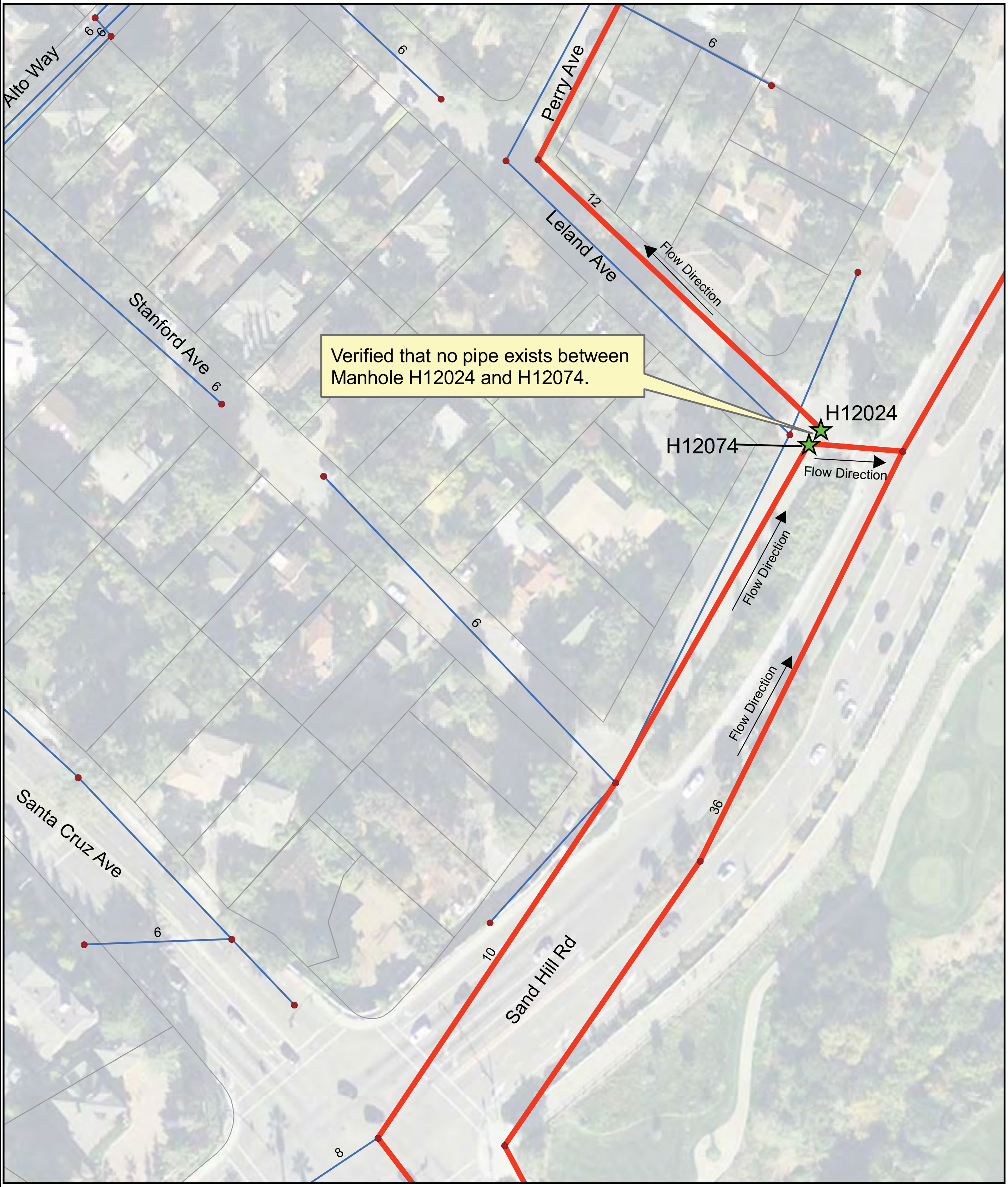
West Bay Sanitary District

Wastewater Collection System Master Plan

Site 5 - Manhole No. C12073

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

- Inspected Manhole
- Manhole
- PS Pump Station
- Modeled Sewer Main
- Other Sewer Main
- Parcels

N

0 75 150

Scale in Feet

FIGURE 5-8

West Bay Sanitary District

Wastewater Collection System Master Plan

Site 6 - Manhole No. H12024 and H12074

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

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LEGEND

- Inspected Manhole
- Manhole
- PS Pump Station
- Modeled Sewer Main
- Other Sewer Main
- Parcels

N

0 75 150

Scale in Feet

FIGURE 5-9

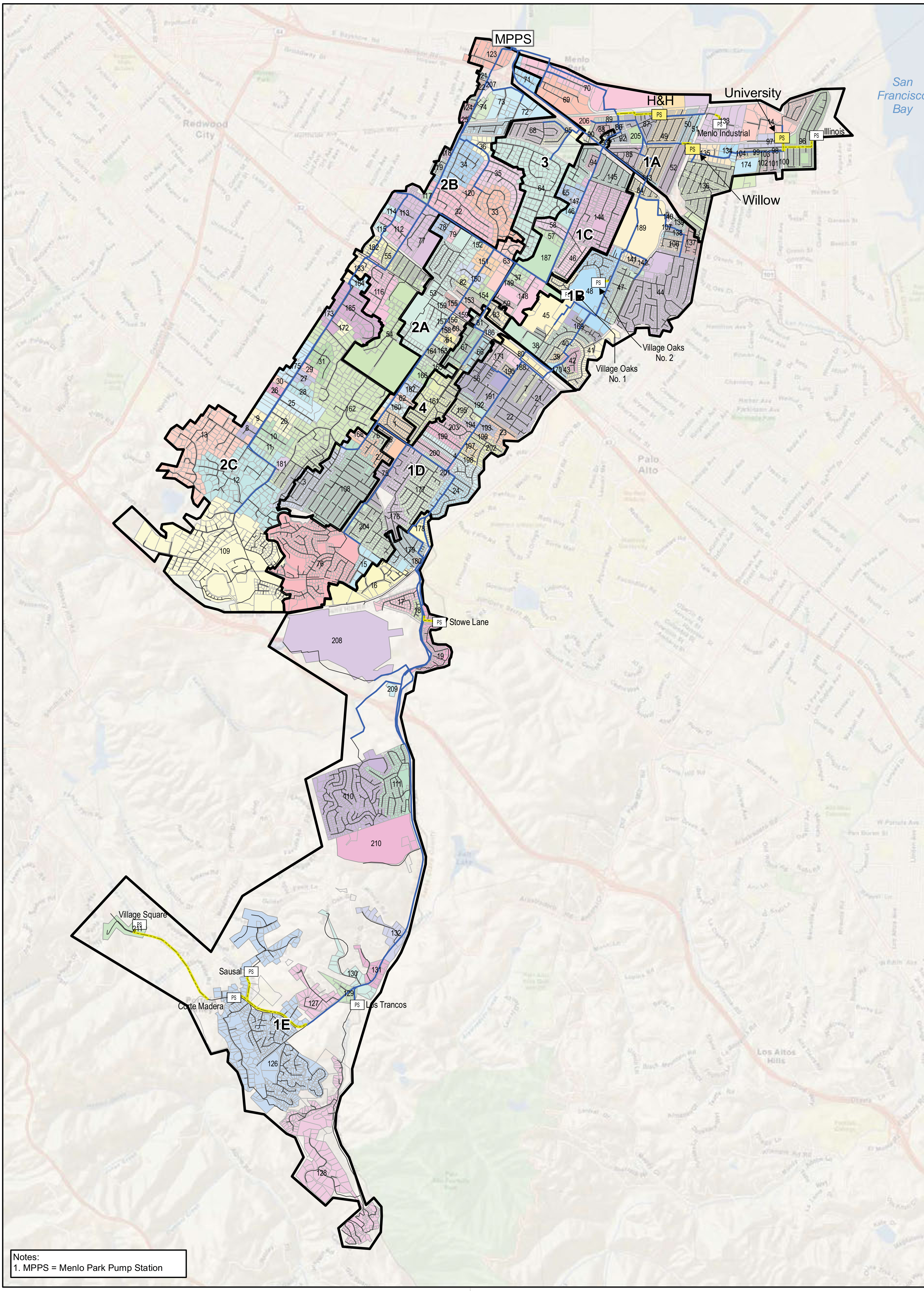
West Bay Sanitary District

Wastewater Collection System Master Plan

Site 7 - Manhole No. D14106 and D14111

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

Various Colors
- Sewersheds
- Model Pipe
- Sewer Pipes
- Force Main
- PS Modeled Pump Stations
- PS Pump Station
- WYA Basins
- Parcels

N

0 2,000 4,000

Scale in Feet

FIGURE 5-10 A

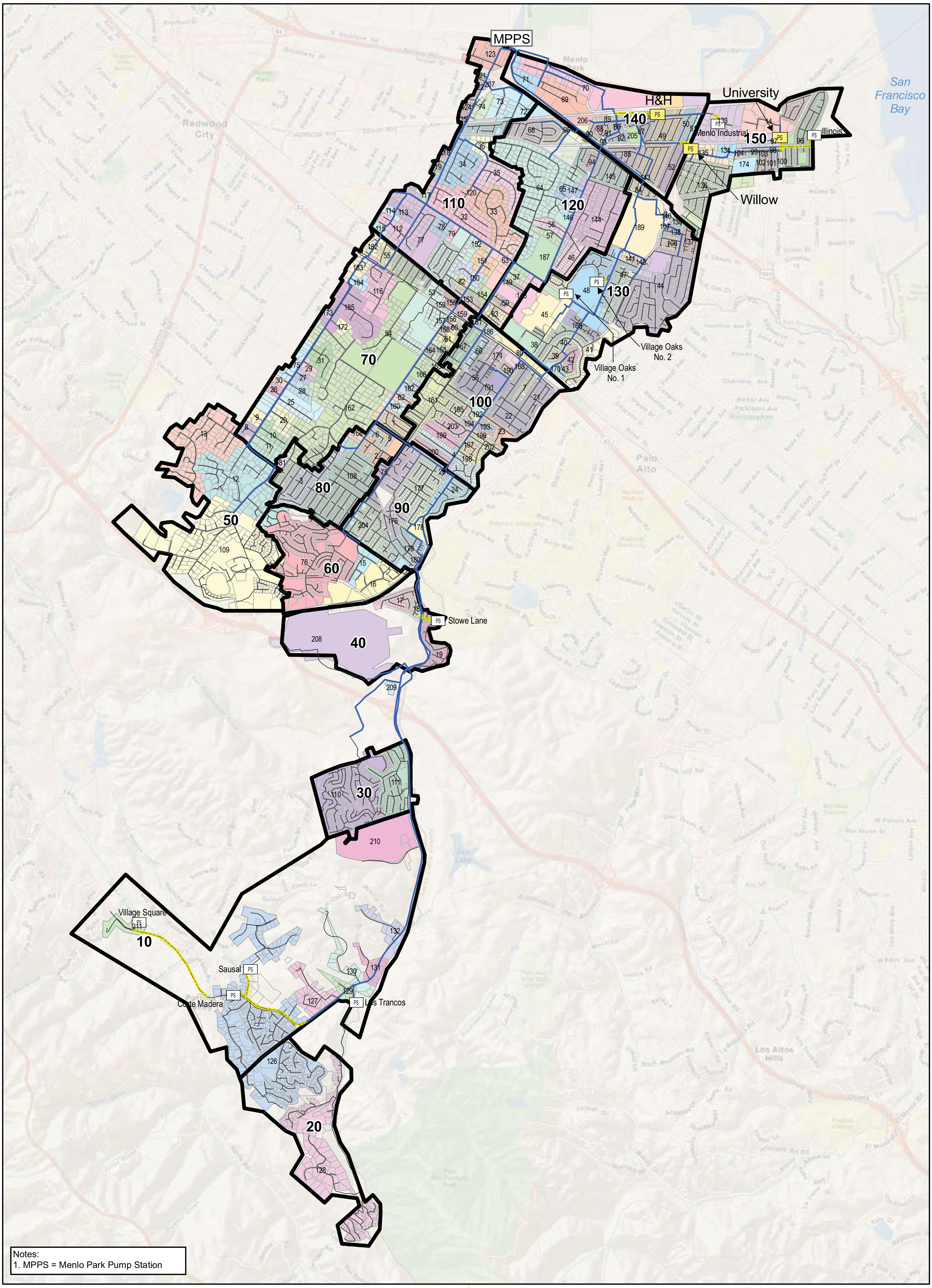
West Bay Sanitary District

Collection System Master Plan

Sewershed Map With Hydraulic Model Basins

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

- Sewersheds (Various Colors)
- Model Pipe
- Sewer Pipes
- Force Main
- Modeled Pump Stations
- Pump Station
- District Basins
- Parcels

0 2,000 4,000

Scale in Feet

FIGURE 5-10 B

West Bay Sanitary District

Collection System Master Plan

Sewershed Map With District Basins

Figure 5-11. Basin 2-B (Manhole B16001) Diurnal Curve

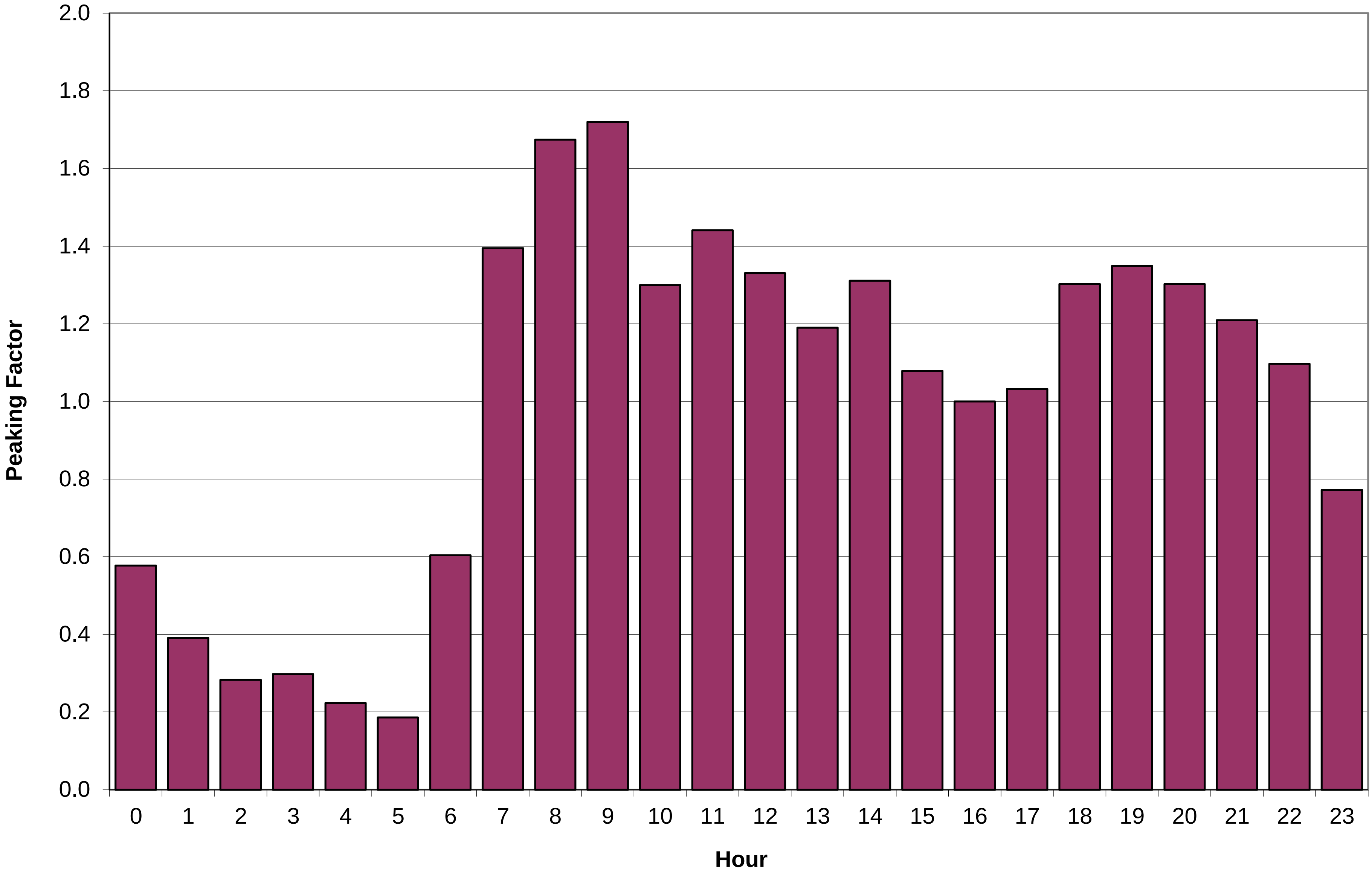


Figure 5-13. Site 9 - Manhole D15128
Dry Weather Flow Calibration

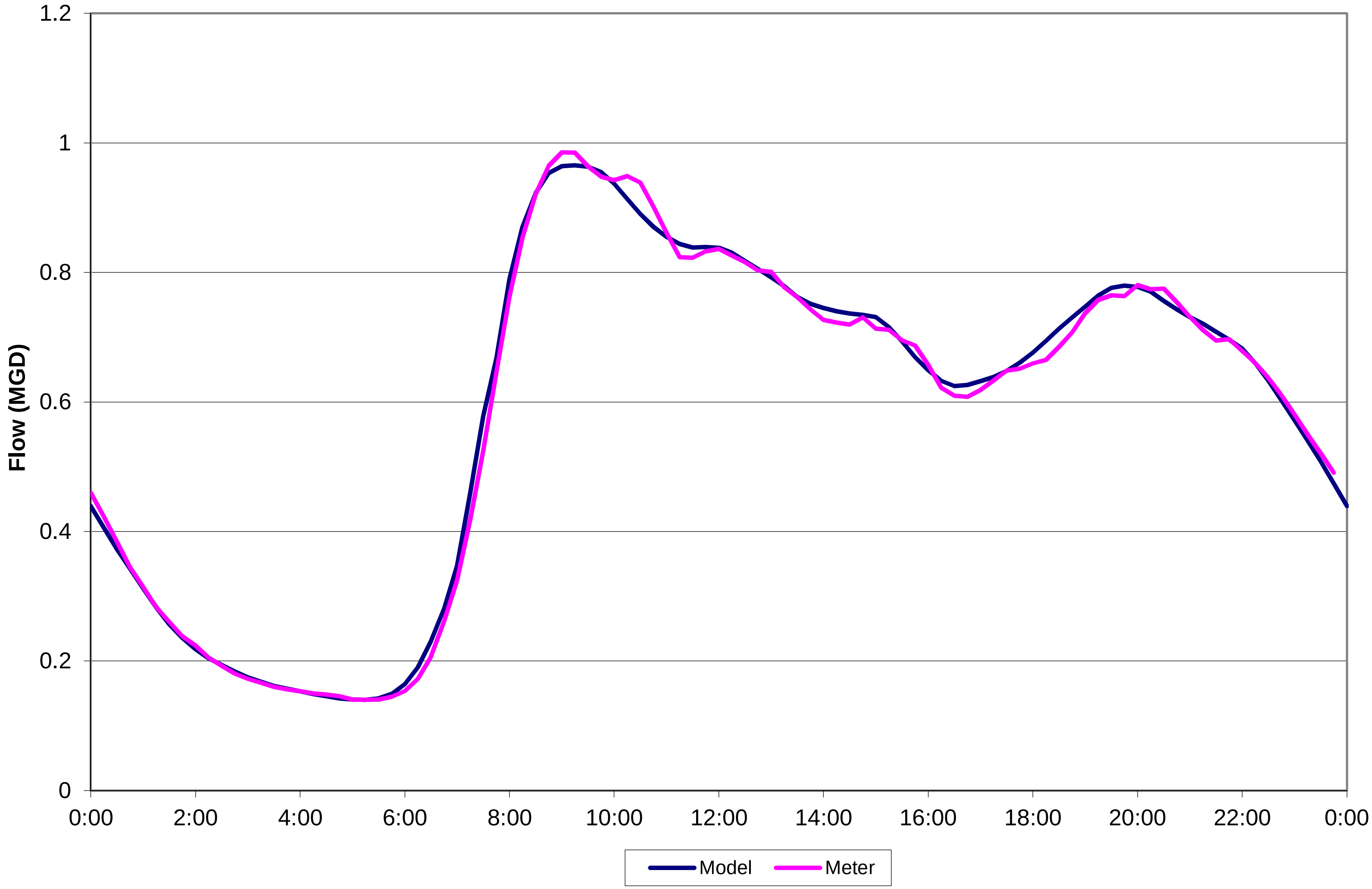
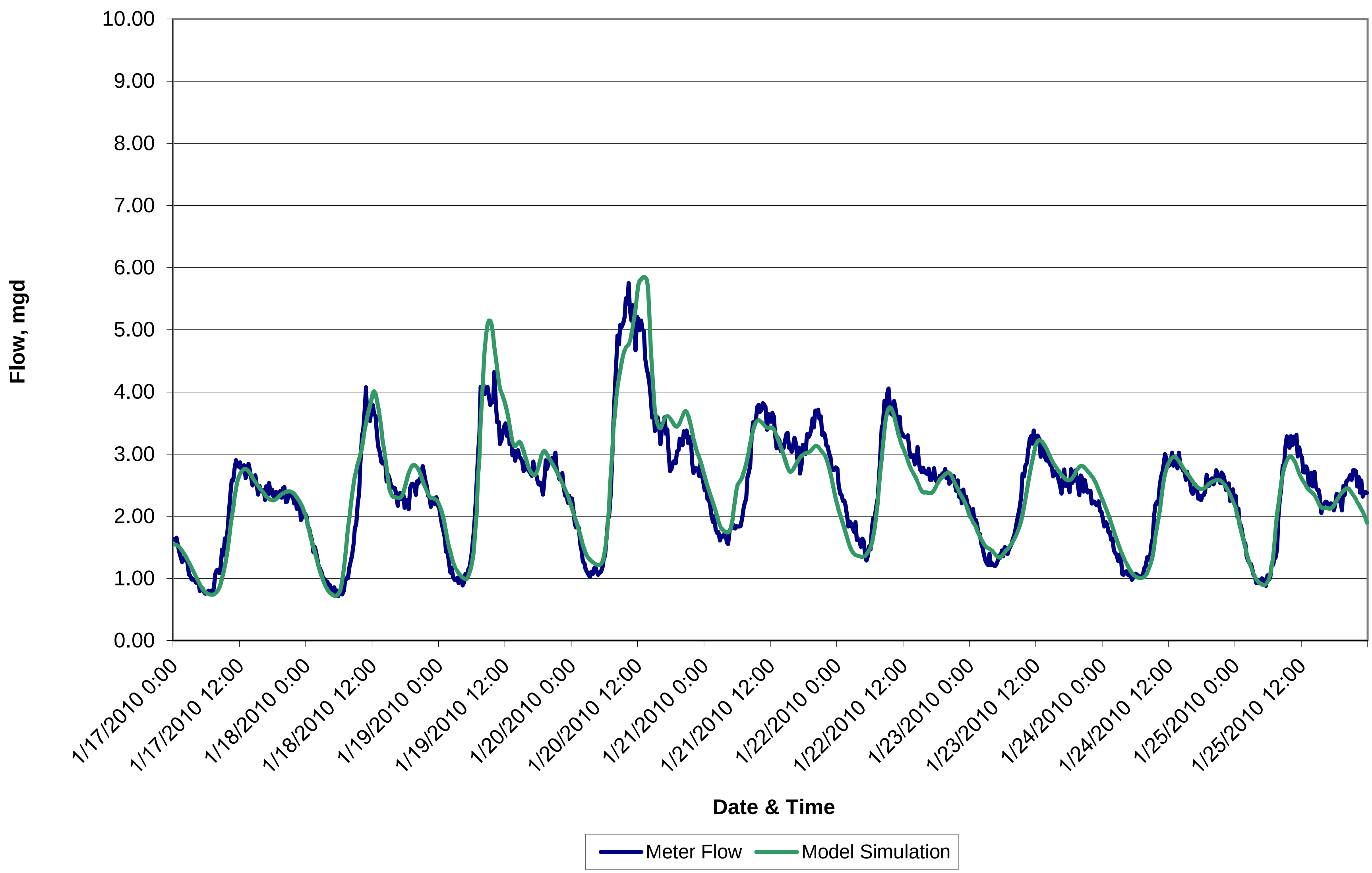


Figure 5-14. Site #4 (Manhole C13088) Wet Weather Flow Calibration
January 17-25, 2010



CHAPTER 6
Planning Criteria

The purpose of this chapter is to present planning criteria that can be used to evaluate system capacity and size any proposed new replacement facilities. The criteria include generally accepted industry standard criteria, as reviewed and confirmed by the District. Planning criteria address items such as collection system capacity, gravity sewer slopes, and maximum depth of flow. The major elements of this chapter include:

- Design Storm,
- Hydraulic Deficiency Criteria, and
- New Pipeline Design Criteria.

6.1 DESIGN STORM CRITERIA

Design storms are synthetic rainfall events used to evaluate collection system capacity under wet weather flow conditions. A design storm has a specific recurrence interval and rainfall duration. The District has been working on plans to eliminate, over time, all sewer overflows related to the designated design storm. This goal allows some storage within the existing manhole structures throughout the system, provided that adequate freeboard in the manhole is available.

In addition to eliminating overflows, this master plan evaluates the ability of the system to convey flows without surcharging under the selected design storm scenario. Although there are no regulatory requirements for sewer sizing, the District has selected as its design storm a rainfall event with a 10-year recurrence interval and 6-hour duration (10-year, 6-hour storm), as defined by the NOAA rainfall atlas[1] measured at the MPPS. This design storm has a total depth of 1.8 inches. The rainfall is distributed using the U.S. Soil Conservation Service (SCS, now Natural Resource Conservation Service) Type I rainfall distribution curve. This design storm criterion was selected to be consistent with the design storm that is required by a settlement agreement signed by a federal court applicable to the City of South San Francisco. Figure 6-1 presents the design storm rainfall distribution.

6.2 EXISTING PIPELINE HYDRAULIC CAPACITY CRITERIA

Hydraulic capacity or deficiency criteria are presented for gravity mains, force mains and lift stations. These criteria are intended to be used as planning tools to determine when flows are considered to have exceeded surcharge capacity during a specific storm event. Under these criteria, a facility may exceed surcharge capacity, yet not overflow. For existing pipelines, the pipe is considered to have a capacity deficiency (surcharge) when, under peak wet weather flow conditions for the design storm, the water level or hydraulic grade line (HGL) is located as identified in the following bullets. Exceptions to these criteria may be made on a case-by-case basis, depending on specific flow conditions and facility configuration. All capacity deficient pipelines should be considered for replacement over time, as discussed in Chapter 10, Capital Improvement Program.

[1] Rainfall depth is published in the Precipitation-Frequency Atlas of the Western United Stations, Volume XI-California, published by the National Oceanic and Atmospheric Administration.

- For pipes 15-inches in diameter and smaller, the water level or HGL is greater than the crown of the pipe.
- For pipes greater than 15-inches in diameter, the HGL is within five feet of ground surface. In some cases, the HGL may exceed the crown of the pipe.
- Force Mains: A force main shall be considered capacity deficient if maximum velocity exceeds 8 feet per second (fps) during peak hourly flows.

6.3 NEW OR REPLACEMENT PIPELINE DESIGN CRITERIA

New or replacement (parallel relief) pipelines were designed to meet the following criteria. These criteria do not apply to the replacement of sections of pipelines within existing alignments:

- Under Peak Dry Weather Flow (PDWF) conditions, velocity shall remain above 2 feet per second to facilitate self-cleaning.
- Under Peak Wet Weather Flow (PWWF) conditions, maximum flow depth (d) as compared to pipe inside diameter (D) d/D shall be as follows:
  - 10-inch diameter and smaller: Max d/D = 0.67
  - 12-inch diameter and above: Max d/D = 0.80
- Under all conditions, maximum allowable velocity is 10 feet per second.

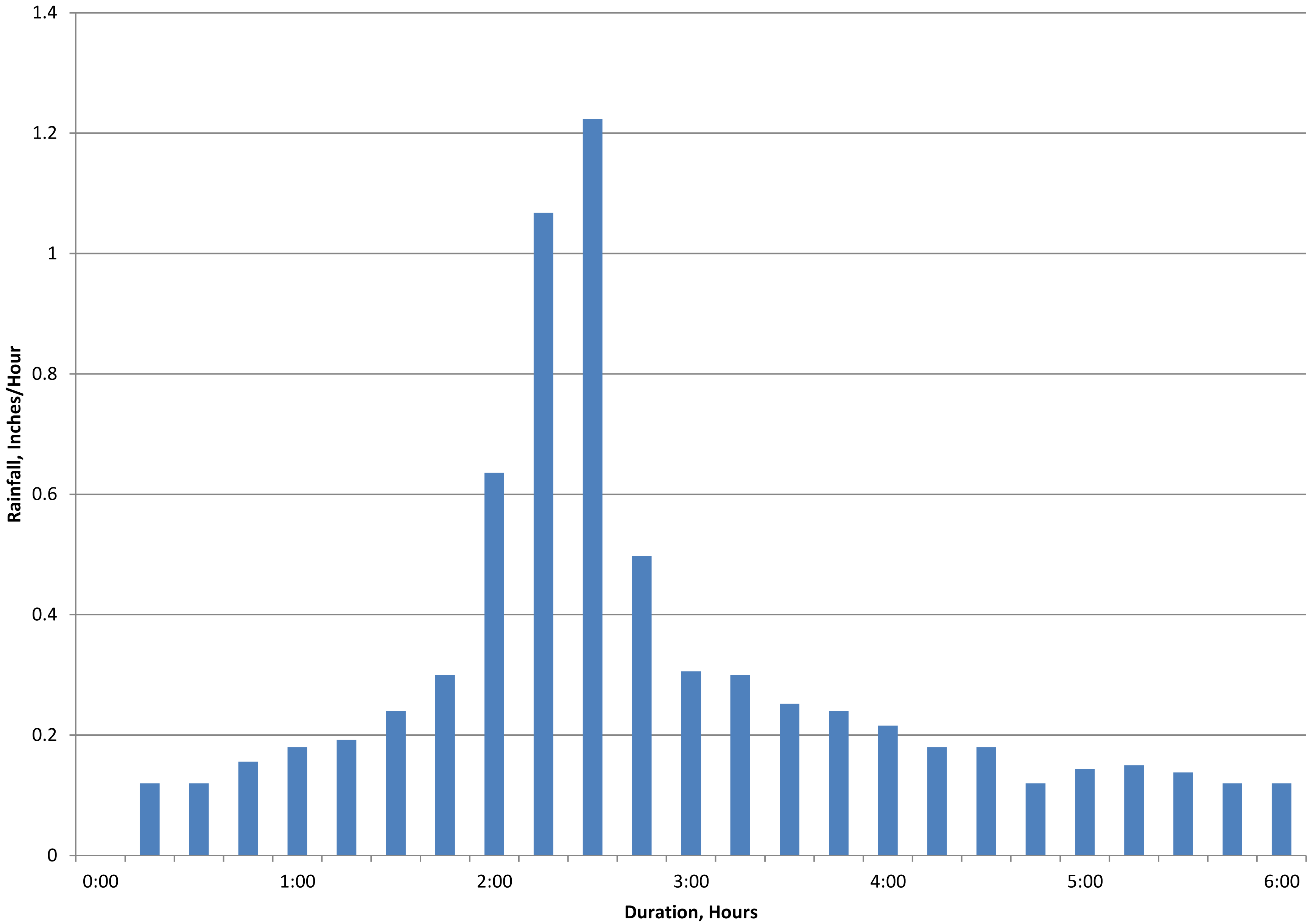
6.4 PUMP STATION DESIGN AND OPERATING CRITERIA

Pump Stations were sized to convey model-generated flows from the 10-year, 6-hour design storm event, with the largest pump out of service. All of the District's pump stations use fixed speed pumps. Each of the pump stations has one duty and one stand-by pump, with the exception of the University Pump Station, which has two duty and one stand-by pump. The pumps operate on level controls within the pump station wet wells. Additional information on pump station characteristics and capacity needs is provided in Chapter 9, Pump Station Condition Assessment.

6.5 USE OF THE FLOW EQUALIZATION FACILITY

The District's FEF is intended to store peak flows from the design storm for the District. Further, during wet weather periods when downstream SBSA member flows exceed their allocated SBSA capacity, SBSA desires that the District divert its flows to the FEF, in order to make the District's SBSA capacity available for the downstream agencies. This diversion is under separate review by SBSA. For the purposes of calculated needed WBSD storage only (not the additional storage needed to accommodate the downstream agencies), the Master Plan assumes that diversion to the FEF will occur 24 hours prior to the time that WBSD flows exceed the District's future SBSA capacity of 16.4 mgd. Flow will be stored in the FEF at least 24 hours beyond the peak flow period, and then will be returned at the maximum available pumping rate of the MPPS.

Figure 6-1. 10-Year, 6-Hour Design Storm Rainfall Distribution



CHAPTER 7
Capacity Analysis

Chapter 7 presents the results of the analysis of available hydraulic capacity within the District's collection system, under wet weather conditions. The analysis reviews the ability of the system to convey flows without surcharge or SSOs related to capacity under a designated design storm rainfall scenario. This chapter also describes proposed capital improvements and planning level cost estimates for recommended capacity improvements.

This chapter is organized as follows:

- Hydraulic Capacity Analysis Results,
- Recommended Projects, and
- Conceptual Costs.

7.1 HYDRAULIC CAPACITY ANALYSIS RESULTS

The District's modeled collection system network was evaluated for its capacity to convey flows that are predicted to occur during a design storm event. Although there is no standard design storm, the analysis was completed using a design storm with a published recurrence interval of 10 years and duration of 6 hours (10-year, 6-hour design storm). The design storm and other hydraulic evaluation criteria are discussed further in Chapter 6, Planning Criteria.

The projected rainfall depth associated with the design storm ranges from 1.8 inches, measured at the MPPS, to 3.2 inches in Portola Valley. Rainfall depth was estimated using the published curves from the NOAA Precipitation-Frequency Atlas that is also referenced in Chapter 6. The hydraulic analysis was conducted under the assumption that SBSA does not constrain the District's flow at the MPPS, which reflects the current, contractually agreed operating practice.

Analyses were conducted as follows:

1. The system was evaluated for its ability to meet the surcharge criteria (*i.e.*, water level relative to the crown of the pipe) described in Chapter 6. Pipe diameter increases that are required to convey peak flows and meet surcharge criteria were determined. These projects, as further refined through Step 2, form the District's long-term capacity improvement recommendations.
2. The system was evaluated for its ability to meet overflow criteria (*i.e.*, hydraulic gradeline or water level relative to ground surface elevation) described in Chapter 6. Pipe diameter increases, as developed in Step 1, that are more critical to address potential overflow issues were moved up in priority. These projects form the District's priority capacity improvement recommendations.
3. The projected peak wet weather flow rate at the MPPS was documented and conveyed to SBSA. SBSA is currently completing a separate conveyance system master plan that addresses wet weather capacity requirements for the MPPS and downstream facilities.
4. Proposed improvements were then reviewed based on the relatively minor increases that are projected for build-out flow and adjusted where necessary.

Hydraulic profiles that were generated using the hydraulic modeling software for the areas with potential capacity issues are included along with associated project costs in Appendix F.

7.1.1 Capacity Analysis Results - Pipelines

The hydraulic model identified five areas where the projected HGL may exceed the ground surface elevation at manhole locations during design storm events. Overall, the design storm has not and is not expected to cause widespread overflow issues within the District's system.

The most capacity limited trunk sewer in the system is located within Marsh Road. This sewer conveys flows from the entire western half of the service area. Surcharge conditions within this pipeline impact flows as far upstream as the central reaches of Atherton Avenue and Valparaiso Avenue. After the Marsh Road flows are addressed, the remaining areas with potential capacity-related overflows are on Ringwood Avenue, Valparaiso Avenue, Santa Cruz Avenue, Fair Oaks Boulevard, and in the gravity sewer directly downstream of the Willow Pump Station force main.

Potential capacity issues within the Marsh Road trunk sewer could be alleviated through the addition of a new, parallel trunk sewer that extends the entire length of Marsh Road, from Middlefield Avenue to the MPPS. Construction of a large diameter pipeline within Marsh Road could be challenging due to nature of the road as the primary traveled way from Highway 92 to the Menlo Park and Atherton area.

After discussions with District staff and review of alternative alignments, the recommended project to address possible Marsh Road trunk sewer capacity issues involves the installation of a new trunk sewer that begins at Middlefield Road and James Avenue, and proceeds north to the District's existing 54-inch interceptor located north of Highway 101. This new sewer will redirect approximately half of the peak wet weather flows that are projected to be conveyed through the Marsh Road trunk sewer. This alignment was selected because it is far enough to the west to capture a sufficient amount of flow to avoid the need for any capacity increases on Marsh Road. If an alternative alignment is selected for this diversion, it is highly recommended that the District utilize the hydraulic model to review the impact of this alternative alignment, if any, on predicted wet weather flows in the Marsh Road sewer.

7.1.2 Capacity Analysis Results - Pump Stations

Three pump stations were included in the hydraulic model: Hamilton Henderson, University, and Willow. The model predicts that the Willow Pump Station may exceed its current firm capacity (*i.e.*, pumping capacity with the largest pump out of service) during the selected design storm. However, after reviewing available pump curves and discussing current pump station operations with the District's pump station superintendent; it appears that the station is currently pumping at a higher rate than its published firm capacity. Therefore, the Willow Pump Station is currently configured to convey projected peak weather flows without retrofit.

Although the Willow Pump Station is sufficiently sized to convey peak wet weather flows, the downstream gravity sewer may not be able to convey design storm peak flows for the projected storm duration. The capacity improvement project that will address this potential pipeline deficiency is described below.

The District's smaller localized pump stations were also evaluated for their ability to convey projected design flows with one pump out of service. For these stations, a peaking factor of 5 was assumed. The peaking factor was selected to be marginally higher than the peaking factor of the associated sewer basin, since basin-wide peaking factors usually reflect attenuation of more localized peak flows.

Table 7-1 compares available firm capacity for each of the District's nine unmodeled pump stations to model-generated peak wet weather flows. Only one station, Los Trancos Pump Station, shows the potential to be under capacity during the design storm. The capacity of this pump station was developed based on build-out of the tributary area. However, currently, only five homes are connected to the District's sewer system. Therefore, this pump station currently has more than sufficient capacity, and should be reviewed closely with each new connection to confirm that sufficient wet weather capacity continues to be available to serve future homes.

Using a peaking factor of 5, the Corte Madera Pump Station appears to have sufficient firm capacity to convey peak wet weather design flows. However, during heavy storm events, the standby pump must be used to convey actual pump station flows. Therefore, the wet weather peaking factor at this pump station may exceed five times average dry weather flow. It is recommended that the District conduct flow monitoring within the sewersheds that are tributary to the Corte Madera Pump Station, to more accurately estimate projected wet weather flows and also identify potential options to reduce these flows.

The District is planning to upsize the pump station forcemain to the maximum size possible, as a temporary measure to convey future wet weather flows. This system may require additional retrofit after design storm flows are confirmed. Recommended improvements for the Corte Madera Pump Station are discussed further in Chapter 9 of this report.

Table 7-1. Local (Unmodeled) Pump Station Capacity Under Design Flow Conditions

Pump Station	Available Firm Capacity, gpm	Model-Generated PWWF, gpm
Menlo Industrial	310	305
Illinois	570	402
Vintage Oaks #1	280	56
Vintage Oaks #2	280	117
Stowe Lane	300	109
Los Trancos	80	109
Sausal Vista	95	32
Corte Madera	350	162
Village Square	130	4

7.1.3 Total System Flows

The District's hydraulic model predicts peak hourly flow during the design storm event of 26.5 million gallons per day. This peak flow may occur for several hours, and then quickly recede. The District and SBSA are currently discussing this projected flow rate in the context of MPPS rehabilitation needs as well as future storage needs at the District's flow equalization facility. The objective of this Master Plan is to identify improvements needed to convey these flows to the MPPS without overflows from the upstream collection system.

7.2 RECOMMENDED PROJECTS

Eleven projects are recommended to address: 1) sewer system overflows on a priority basis; and 2) surcharge conditions as part of the long-term capital improvement program.

7.2.1 Priority Projects to Address Potential Capacity-Related Sewer System Overflows

Based on the hydraulic analyses and results, five priority pipeline capacity improvement projects could be considered to eliminate potential capacity-related SSOs during the design storm. These projects are summarized in Table 7-2, and are also described below and presented graphically in Figures 7-1 through 7-5. The highest priority project would be the James Avenue Diversion. After this project is completed, the remaining projects can be completed in any order, since they each convey flow from separate areas of the District. The projects are recommended for completion within a ten-year timeframe.

Table 7-2. Priority Capacity Improvement Projects

Project Name	Approximate Length	Project Description
James Avenue Diversion	1.6 miles	24-inch flow diversion from Middlefield Road to James Avenue, Lilac Avenue, Greenoaks Drive, Flood Park, and across Highway 101.
Lower Ringwood Avenue Capacity Improvements	0.23 miles	15-inch diameter replacement pipeline from easement between Ringwood Ave and Sonoma Avenue, across Highway 101.[a]
Lower Valparaiso Avenue Capacity Improvements	0.16 miles	15-inch diameter replacement pipeline from Hoover Street to San Antonio Street.[a]
Lower Santa Cruz-Avy Capacity Improvements	0.22 miles	18-inch diameter replacement pipeline between Orange Avenue and Atkinson Lane.[a]
Upper Willow Pump Station Gravity Main Capacity Improvements	0.06 miles	21-inch diameter replacement pipeline from Carlton Avenue to the first downstream manhole, downstream of the Willow Pump Station force main.[a]

(a) The District should consider upsizing additional pipe adjacent to this alignment to meet surcharge criteria. See below for related long-term projects that can be combined with these priority projects.

Priority Project - James Avenue Diversion

This project would divert flow from Middlefield Road at James Avenue in order to reduce total flow volumes in the Marsh Road trunk sewer. The total project length is approximately 1.6 miles. This diversion is anticipated to alleviate predicted surcharging and overflows in the Marsh Road trunk sewer continuing upstream to Atherton Road, and in the Middlefield Road trunk sewer.

The flow diversion would require the installation of a 24-inch diameter pipeline that is parallel to the existing sewer in James Avenue.

The alignment would begin at Manhole D15131, continue north on James Avenue to Lilac Avenue, and turn east onto Greenoaks Drive. At Manhole C14085 on Greenoaks Drive, the pipe would turn north and replace or run parallel to the existing 10-inch diameter line in an easement through Flood Park (including a crossing of the SFPUC Bay Division pipelines), by Flood School, and under Highway 101 to its termination at Manhole C14122.

Due to the anticipated project cost, project construction may be completed in segments. It is recommended that the District complete this project before constructing the Lower Valparaiso Avenue project, which will likely increase peak wet weather flows in the Middlefield Road trunk sewer.

Lower Ringwood Avenue Capacity Improvements

This project would include upsizing approximately 0.23 miles of existing pipes beginning at Manhole C13115 in an easement west of Ringwood Avenue and north of Oakwood Place. The project would cross Highway 101, and then continue to Hamilton Avenue. The project is anticipated to eliminate potential capacity related overflows near Bay Road.

The existing 10-inch diameter pipeline would be increased to 15-inches in diameter from Manhole C131115 to C13045. This project would include installation of pipe through backyard

easements and require a trenchless crossing of Highway 101. Although the project is anticipated to eliminate potential capacity related overflows, the replacement of one additional 435 linear foot section of pipe upstream of this location would be needed in the future to meet the surcharge criteria that are discussed in Chapter 6. The District should consider combining the two projects if budget is available.

Lower Valparaiso Avenue Street Capacity Improvements

This project would include upsizing approximately 0.16 miles of existing pipes within Valparaiso Avenue between Hoover Street and San Antonio Avenue and would reduce the potential for a capacity related overflow at Chateau Drive.

The existing 12-inch diameter pipeline would be increased to a 15-inch diameter pipe from Manhole F14005 to E14150. Pipeline installation methods would need to consider the heavy traffic patterns due to the proximity to El Camino Real, Academy of the Sacred Heart, and Menlo School and College. The pipeline would also require a crossing of El Camino Real.

Lower Santa Cruz-Avy Capacity Improvements

This project would replace approximately 0.22 miles of existing 15-inch pipe from approximately Orange Avenue (Manhole H14170) to Atkinson Lane (Manhole G14165) with an 18-inch diameter pipe to assist in reducing potential capacity related SSOs from the parallel line within Santa Cruz Avenue. Replacement of an additional 851 linear feet of pipe upstream of this project would be required in the future to eliminate predicted surcharge conditions in the Santa Cruz pipelines. The District should consider combining the priority and future Santa Cruz Avenue projects if budget is available.

Upper Willow Pump Station Gravity Main Capacity Improvements

This project would include upsizing the first 300 linear foot section of gravity pipe downstream of the Willow Pump Station force main, from Carlton Avenue (Manhole B12029) to the first downstream manhole (Manhole B12041). This project would eliminate a brief period of predicted capacity deficiency that may occur only after sustained pumping during peak flow periods. The existing 15-inch pipeline would be increased to a 21-inch diameter pipeline.

Additional Capacity Projects at Fair Oaks Drive and on Atherton East (Burns Easement)

In addition to the projects described, the hydraulic model predicted a potential for a capacity related sanitary sewer overflow on Fair Oaks Drive at Middlefield Road. The District's GIS data shows the distance between manhole rim and pipeline invert in this location to be 5.4 feet, and the hydraulic model predicts an HGL that may fall several inches above ground level. A field measurement conducted by the District confirmed a corrected depth-to-invert measurement of over 6 feet. Therefore, the predicted surcharge at this location may remain below the ground surface elevation, but improvements should be completed to address the potential for capacity related overflow, and to address the surcharge condition as discussed in the following subsection.

Also, the hydraulic model predicted a potential for a capacity related sanitary sewer overflow on El Camino Real (Manhole E16082) that would be remedied through pipe upsizing through the Burns easement. The ground surface elevation that is presented in the District’s GIS database shows an 8-foot drop in ground surface elevation in the location of this manhole. Because the roadway is level at the intersection of Atherton Avenue and El Camino Real, it is suspected that the depicted drop in ground surface elevation is incorrect, and that the predicted potential; for overflow is not accurate. Therefore, the needed improvements are presented below, as required to address surcharge (but not overflow) conditions.

7.2.2 Long-Term Projects to Address Potential Surcharge Conditions

In addition to the projects described in Section 7.1.1, six long-term capacity improvement projects are recommended to meet the District's surcharge criteria under the design storm. These projects are presented in Table 7-3, and are also described below and presented graphically in Figures 7-2 through 7-7. These projects have equal priority, and should be distributed throughout the long-term CIP timeframe as funds become available.

Table 7-3. Long-Term Capacity Improvement Projects

Project Name	Approximate Length	Project Description
Upper Ringwood Avenue Capacity Improvements	0.08 miles	Install 15-inch diameter replacement from Manhole C13187 to C13115, in easement at the intersection of Oakwood Place and Ringwood Avenue.
Middlefield at Fair Oaks Capacity Improvements	0.19 miles	Install 15-inch diameter replacement pipe on Middlefield Road from Fair Oaks Lane to Marsh Road.
Atherton Avenue East Capacity Improvements (Burns Easement)	0.8 miles	Install 24-inch diameter replacement pipe on Atherton Avenue from Mercedes to El Camino Real, and from the SPRR right of way at Maple Avenue to Middlefield Road.
Upper Santa Cruz-Avy East Capacity Improvements	0.16 miles	Install 18-inch replacement pipe from Cloud Avenue to Orange Avenue.
Upper Valparaiso Avenue Capacity Improvements	0.06 miles	Install 15-inch replacement pipe from Chateau Drive to Hoover Street.
Lower Willow Pump Station Gravity Main	0.28 miles	Install 21-inch replacement pipe from Manhole B12041 to B13044 on Chilco Street.

Upper Ringwood Avenue Capacity Improvements

This project would upsize approximately one segment of existing pipe directly upstream of the priority Ringwood Avenue project described above. The project is anticipated to reduce surcharging during the design storm event. The existing 10-inch diameter pipeline would be increased to 15-inches in diameter.

Middlefield at Fair Oaks Capacity Improvements

This project would upsize approximately 0.19 miles of existing pipes within Middlefield Road from Fair Oaks Lane to Marsh Road. The project is anticipated to reduce potential surcharging upstream within Fair Oaks Lane. The existing 10-inch diameter pipeline would be increased to 15-inches in diameter from Manhole D16026 to D15104.

The project will likely encounter heavy traffic as the entire length of pipe is located within Middlefield Road, and the project terminates at the heavily traveled intersection with Marsh Road. It is recommended that the District combine this project with the lower portion of the project described below as Atherton Avenue East Capacity Improvements.

Atherton Avenue East Capacity Improvements

This project would upsize the existing 18-inch pipeline in Atherton Avenue from Mercedes Lane to El Camino Real, and then from the intersection of Maple Avenue and the SPRR right of way through the Burns easement to Middlefield Road.

The existing pipe would be replaced with a 24-inch diameter pipeline. The existing pipes on El Camino Real and Maple Avenue have a diameter of 24-inches and currently do not require replacement. The project is recommended to relieve surcharge issues within Atherton Avenue.

The proposed Atherton Avenue project may be completed at one time or in sections, as follows:

- Replace approximately 1,050 linear feet of pipe from Mercedes Lane (Manhole F16011) to El Camino Real (Manhole E16062). This segment is located on a heavily traveled roadway and is a priority project if the District wishes to coordinate with a planned City of Atherton pavement rehabilitation project in 2011 or 2012..
- Replace approximately 0.7 miles of pipe from Maple Avenue and the SPRR right of way (Manhole E15070) through the Burns Easement to Middlefield Road (Manhole D15089). This segment crosses a railroad right of way and then continues through backyard easements, ending on the heavily traveled upper portion of Marsh Road. This project intercepts the terminus of the Middlefield at Fair Oaks Capacity Improvements project described above.

Upper Santa Cruz-Avy East Capacity Improvements

This project would replace approximately 0.16 miles of existing 15-inch pipe from Cloud Avenue (Manhole H14173) to Orange Avenue (Manhole H14170) with an 18-inch diameter pipe to relieve surcharge issues. This project would also relieve potential surcharges in the parallel Santa Cruz-Avy West pipeline.

Upper Valparaiso Avenue Capacity Improvements

This project would include upsizing approximately 0.06 miles of existing pipe (two pipe segments) upstream of the Lower Valparaiso Avenue priority project, described above. The project is anticipated to eliminate the predicted surcharge on Valparaiso Avenue. The existing 12-inch diameter pipeline would be increased to a 15-inch diameter pipe.

Lower Willow Pump Station Gravity Main Capacity Improvements

This project would include upsizing the gravity pipe segments from Manhole B12041 to B13044 on Chilco Avenue. The project is anticipated to eliminate surcharging that occurs during the design storm. The existing 15-inch pipeline would be increased to a 21-inch diameter pipeline.

7.3 CONCEPTUAL COSTS

This section discusses the basis for cost estimates prepared for the five priority capacity improvement projects and six future long-term projects.

7.3.1 Cost Estimating Basis

For the purposes of estimating cost for the capacity improvement projects, open cut or open trench construction was assumed, with the exception of specific crossings at the SPRR right of way, SFPUC pipelines, and Highway 101. Open cut construction has historically been the most widely used approach for sewer pipe replacements, and is described in further detail in Chapter 8.

The unit cost of open cut construction varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, open cut pipeline installation costs range from $10 to $14 per inch diameter per foot of pipe installed. These base pipeline installation costs include excavation, shoring, pipe installation, backfill, and compaction.

The proposed projects anticipate normal to difficult construction conditions, depending on the specific project. The difficulty of construction is described as part of the individual project cost estimates that are included in Appendix F.

For most pipelines, the base pipeline installation cost was doubled to account for the additional costs of mobilization, manhole replacements, and other construction needs. For the James Avenue diversion, the base pipeline installation cost was increased by fifty percent for these same items, because this pipe will not require lateral connections or manhole retrofits. The James Avenue diversion estimate included a new manhole every 300 feet along the alignment.

A 30 percent contingency for construction unknowns was added, and then additional allowances were included to account for design and project administration costs. Costs were based on the December 2010 Engineering News Record Construction Cost Index (ENR CCI), San Francisco, 10,120.29. Detailed cost estimates are presented in Appendix F.

7.3.2 Estimated Project Costs

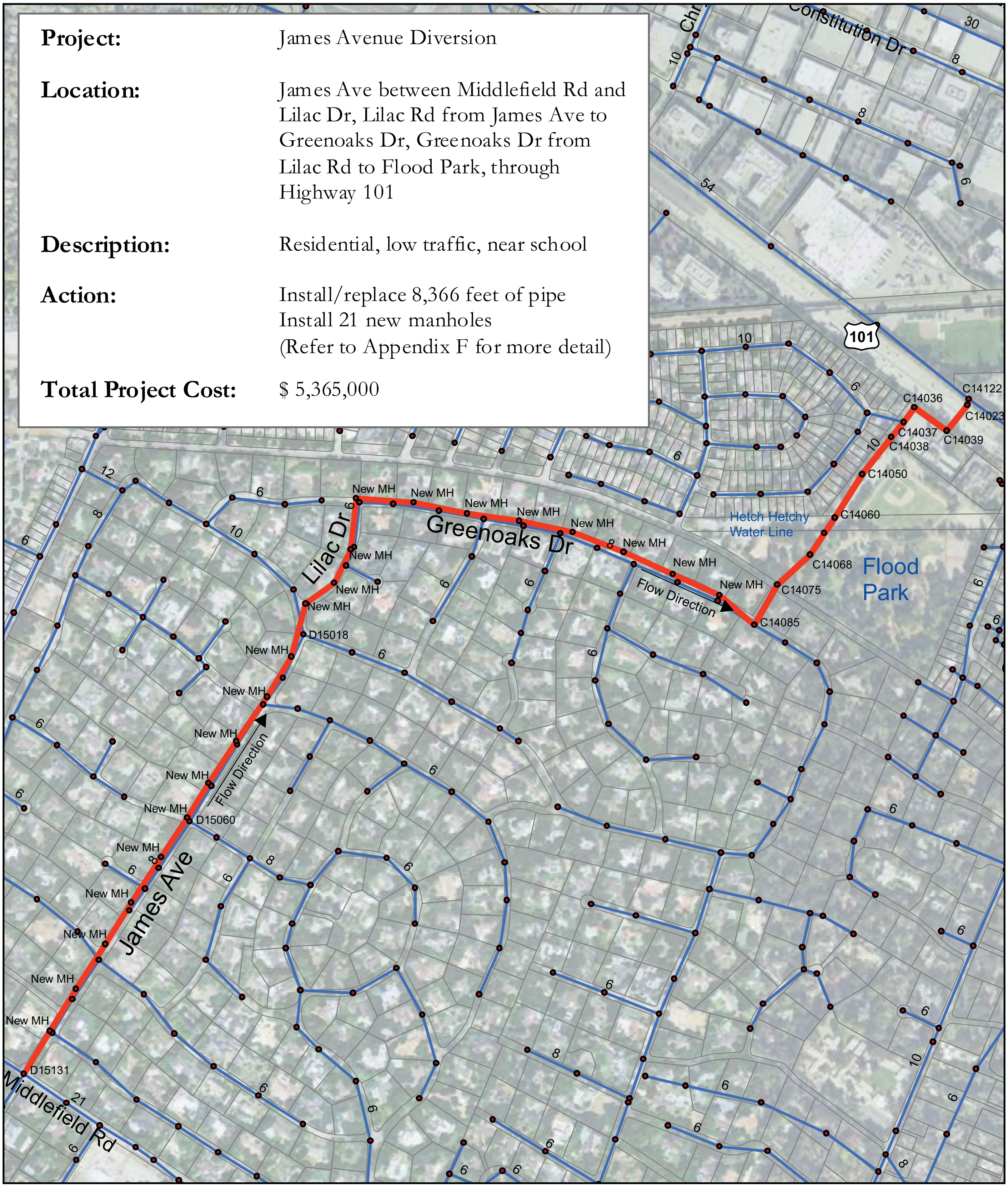
Table 7-4 summarizes the priority and long-term capacity improvement projects and associated costs. Construction costs include the total construction cost estimate including contingency. Total costs add project design and construction administration to the total construction cost.

Table 7-4. Capacity Improvement Project Estimated Costs

Project Name	Estimated Construction Cost	Estimated Total Cost
James Avenue Diversion	$4.5M	$5.4M
Lower Ringwood Avenue	$811k	$957k
Lower Valparaiso Avenue	$347k	$444k
Lower Santa Cruz-Avy	$548k	$667k
Upper Willow Pump Station Gravity Main	$176k	$226k
Total Estimate for Priority Projects	$6.4M	$7.7M
Upper Ringwood Avenue	$183k	$234k
Middlefield at Fair Oaks	$395k	$502k
Atherton Avenue East	$3.6M	$4.2M
Upper Valparaiso Avenue	$142k	$182k
Upper Santa Cruz-Avy	$411k	$519k
Lower Willow PS Gravity Main	$839k	$991k
Total Estimate for Long-Term Projects	$5.6M	$6.6M

O:\Clients\453 West Bay Sanitary Dist\02-10-03 Collection System Model MP & Fee Estimate\GIS\Figures\Figure 7-1 (James Avenue Diversion).mxd 6/24/2011

Project:	James Avenue Diversion
Location:	James Ave between Middlefield Rd and Lilac Dr, Lilac Rd from James Ave to Greenoaks Dr, Greenoaks Dr from Lilac Rd to Flood Park, through Highway 101
Description:	Residential, low traffic, near school
Action:	Install/replace 8,366 feet of pipe Install 21 new manholes (Refer to Appendix F for more detail)
Total Project Cost:	$ 5,365,000



LEGEND

- Manhole
- Pipe Segment
- Project Pipe Segment
- Parcels

N

0 300 600

Scale in Feet

FIGURE 7-1

West Bay Sanitary District

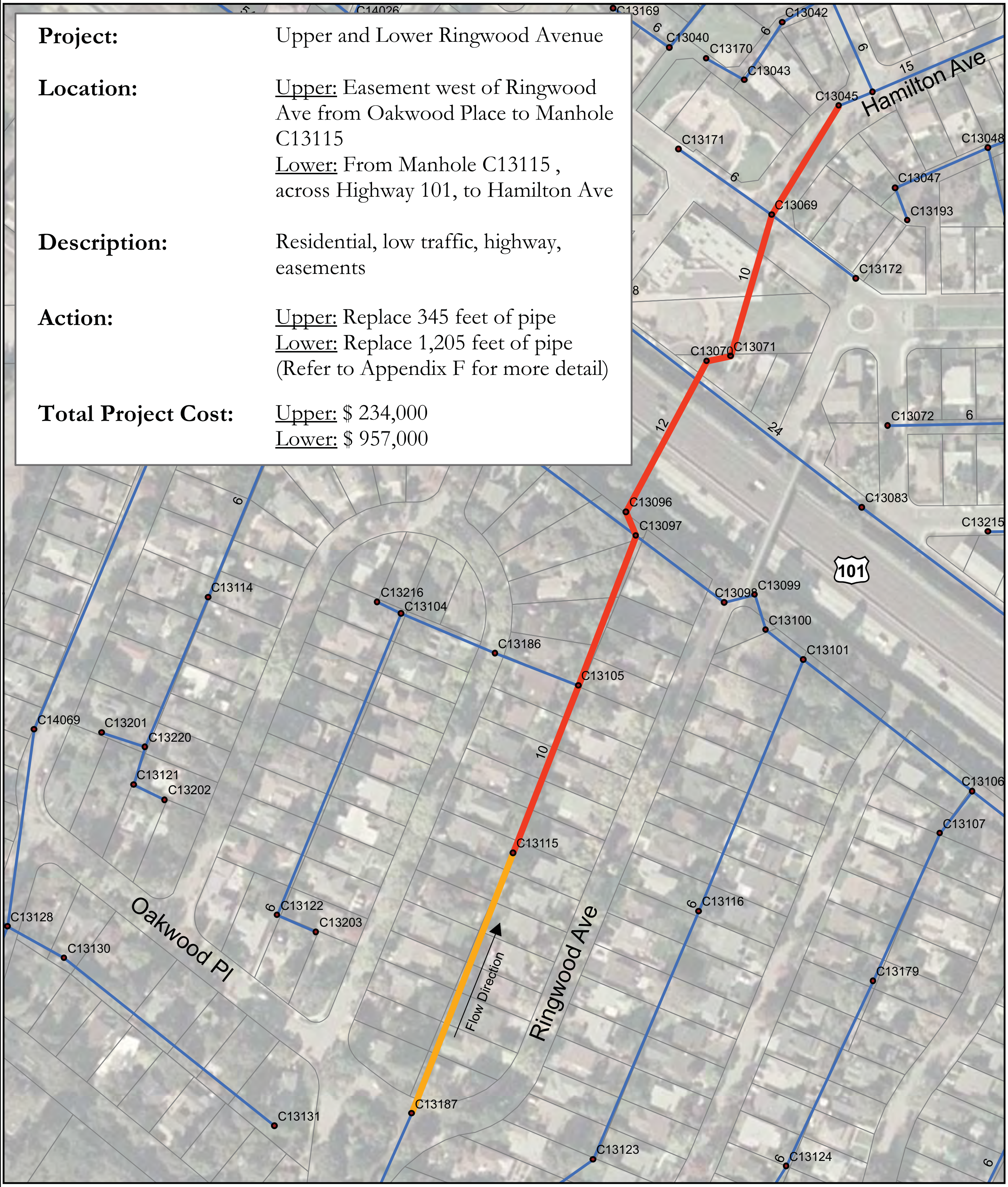
Wastewater Collection System Master Plan

James Avenue Diversion

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

Project:	Upper and Lower Ringwood Avenue
Location:	Upper: Easement west of Ringwood Ave from Oakwood Place to Manhole C13115 Lower: From Manhole C13115 , across Highway 101, to Hamilton Ave
Description:	Residential, low traffic, highway, easements
Action:	Upper: Replace 345 feet of pipe Lower: Replace 1,205 feet of pipe (Refer to Appendix F for more detail)
Total Project Cost:	Upper: $ 234,000 Lower: $ 957,000



LEGEND

- Manhole
- Upper Ringwood Ave Project Segment
- Lower Ringwood Ave Project Segment
- Pipe Segment
- Parcels

N

0 100 200

Scale in Feet

FIGURE 7-2

West Bay Sanitary District

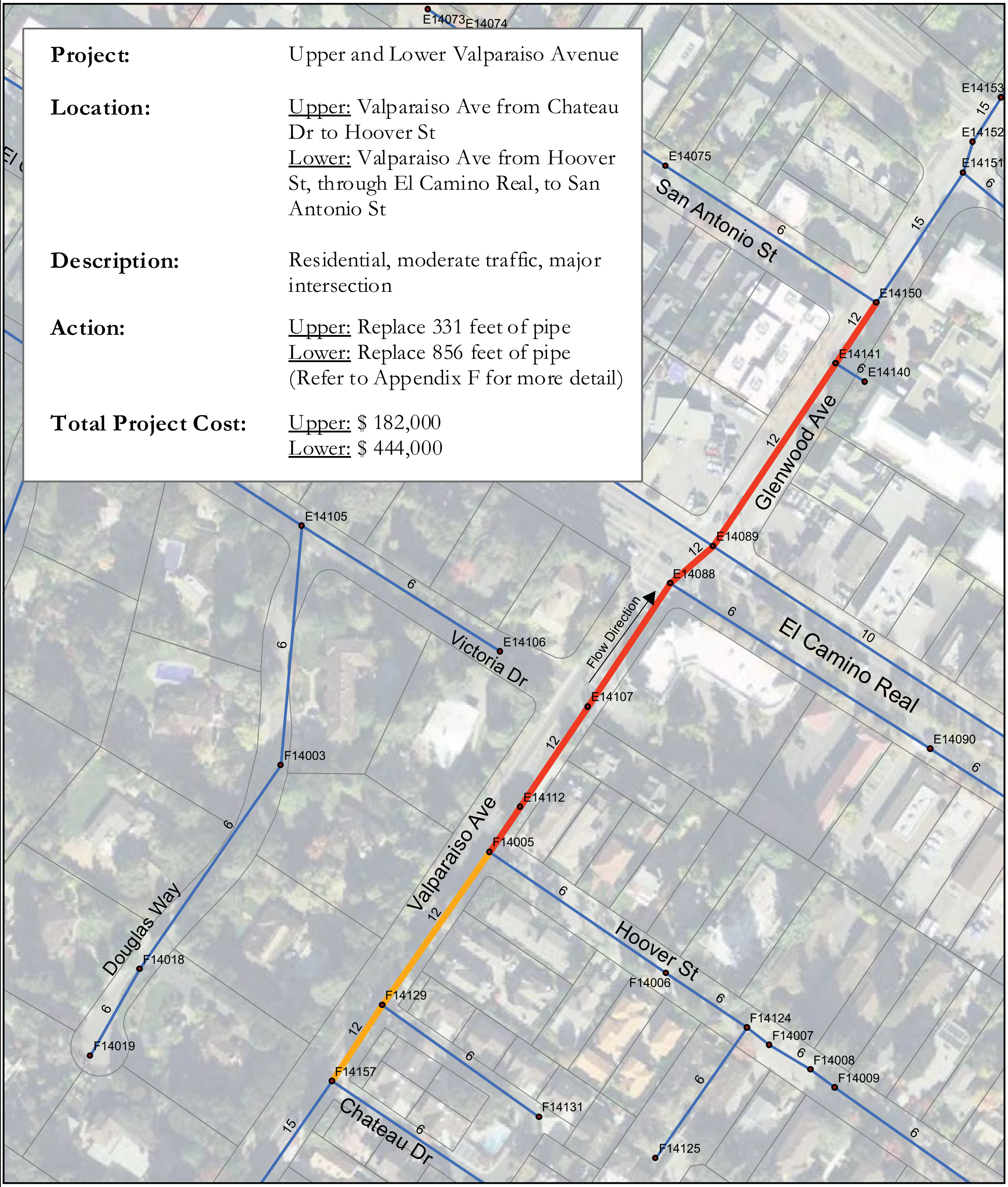
Wastewater Collection System Master Plan

Upper and Lower Ringwood Avenue

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

Project:	Upper and Lower Valparaiso Avenue
Location:	Upper: Valparaiso Ave from Chateau Dr to Hoover St Lower: Valparaiso Ave from Hoover St, through El Camino Real, to San Antonio St
Description:	Residential, moderate traffic, major intersection
Action:	Upper: Replace 331 feet of pipe Lower: Replace 856 feet of pipe (Refer to Appendix F for more detail)
Total Project Cost:	Upper: $ 182,000 Lower: $ 444,000



LEGEND

- Manhole
- Upper Valparaiso Ave Project Segment
- Lower Valparaiso Ave Project Segment
- Pipe Segment
- Parcels

N

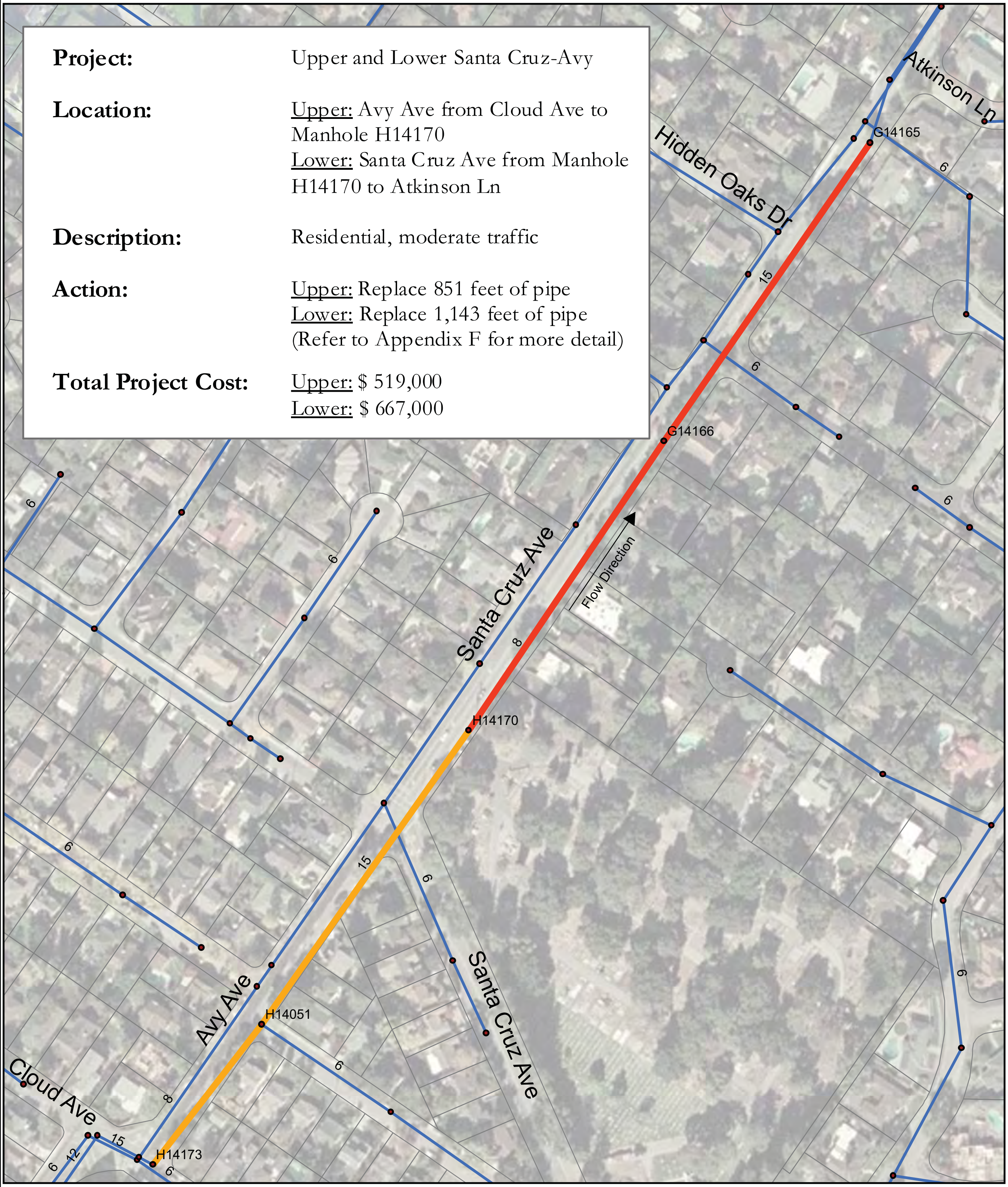
0 100 200
Scale in Feet

FIGURE 7-3
West Bay Sanitary District
Wastewater Collection System Master Plan

Upper and Lower Valparaiso Avenue

WEST BAY SANITARY DISTRICT
WEST YOST ASSOCIATES Consulting Engineers

Project:	Upper and Lower Santa Cruz-Avy
Location:	Upper: Avy Ave from Cloud Ave to Manhole H14170 Lower: Santa Cruz Ave from Manhole H14170 to Atkinson Ln
Description:	Residential, moderate traffic
Action:	Upper: Replace 851 feet of pipe Lower: Replace 1,143 feet of pipe (Refer to Appendix F for more detail)
Total Project Cost:	Upper: $ 519,000 Lower: $ 667,000



LEGEND

- Manhole
- Upper Santa Cruz-Avy Project Segment
- Lower Santa Cruz-Avy Project Segment
- Pipe Segment
- Parcels

N

0 100 200

Scale in Feet

FIGURE 7-4

West Bay Sanitary District

Wastewater Collection System Master Plan

Upper and Lower Santa Cruz-Avy

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

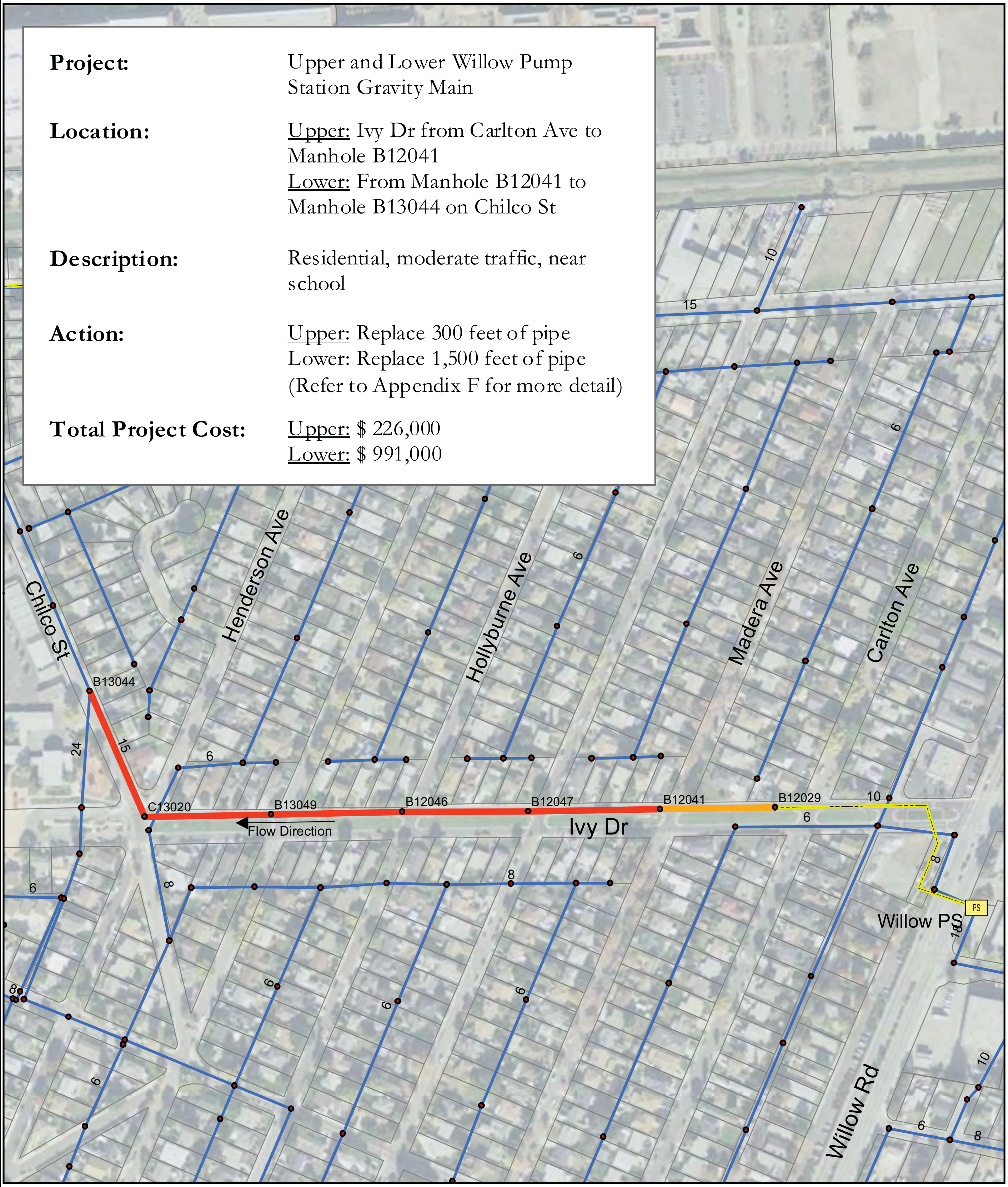
Project: Upper and Lower Willow Pump Station Gravity Main

Location: Upper: Ivy Dr from Carlton Ave to Manhole B12041
Lower: From Manhole B12041 to Manhole B13044 on Chilco St

Description: Residential, moderate traffic, near school

Action: Upper: Replace 300 feet of pipe
Lower: Replace 1,500 feet of pipe
(Refer to Appendix F for more detail)

Total Project Cost: Upper: $ 226,000
Lower: $ 991,000



LEGEND
- Manhole
- PS Modeled Pump Station
- PS Pump Station
- Upper Willow Pump Station Gravity Main Project Segment
- Lower Willow Pump Station Gravity Main Project Segment
- Pipe Segment
- Force Main
- Parcels

N

0 150 300
Scale in Feet

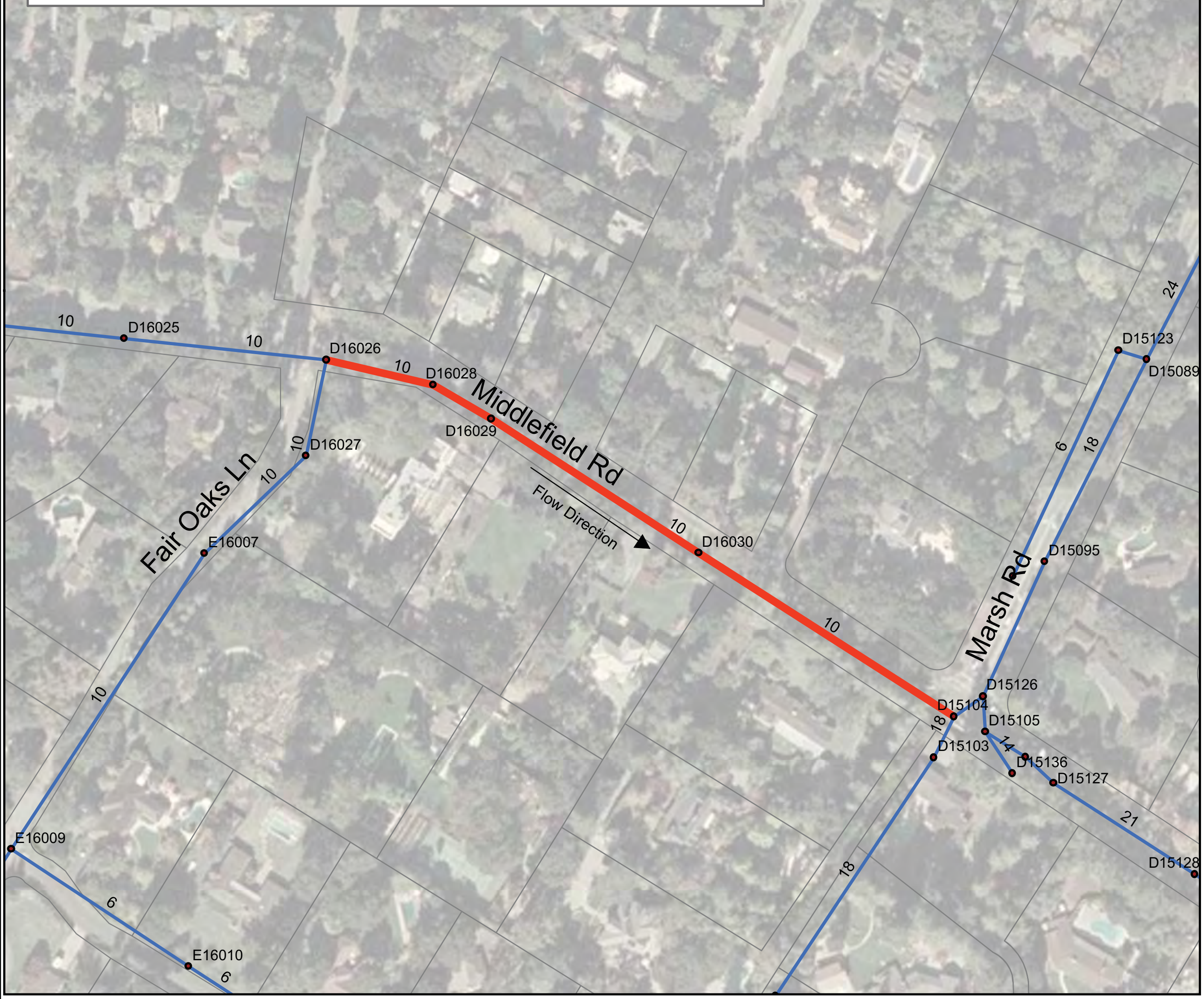
FIGURE 7-5
West Bay Sanitary District
Wastewater Collection System Master Plan

Upper and Lower Willow Pump Station Gravity Main

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

Project:	Middlefield at Fair Oaks
Location:	Middlefield Rd between Fair Oaks Ln and Marsh Rd
Description:	Residential, moderate traffic
Action:	Replace 980 feet of pipe (Refer to Appendix F for more detail)
Total Project Cost:	$ 502,000



LEGEND

- Manhole
- Pipe Segment
- Project Pipe Segment
- Parcels

N

0 75 150
Scale in Feet

FIGURE 7-6

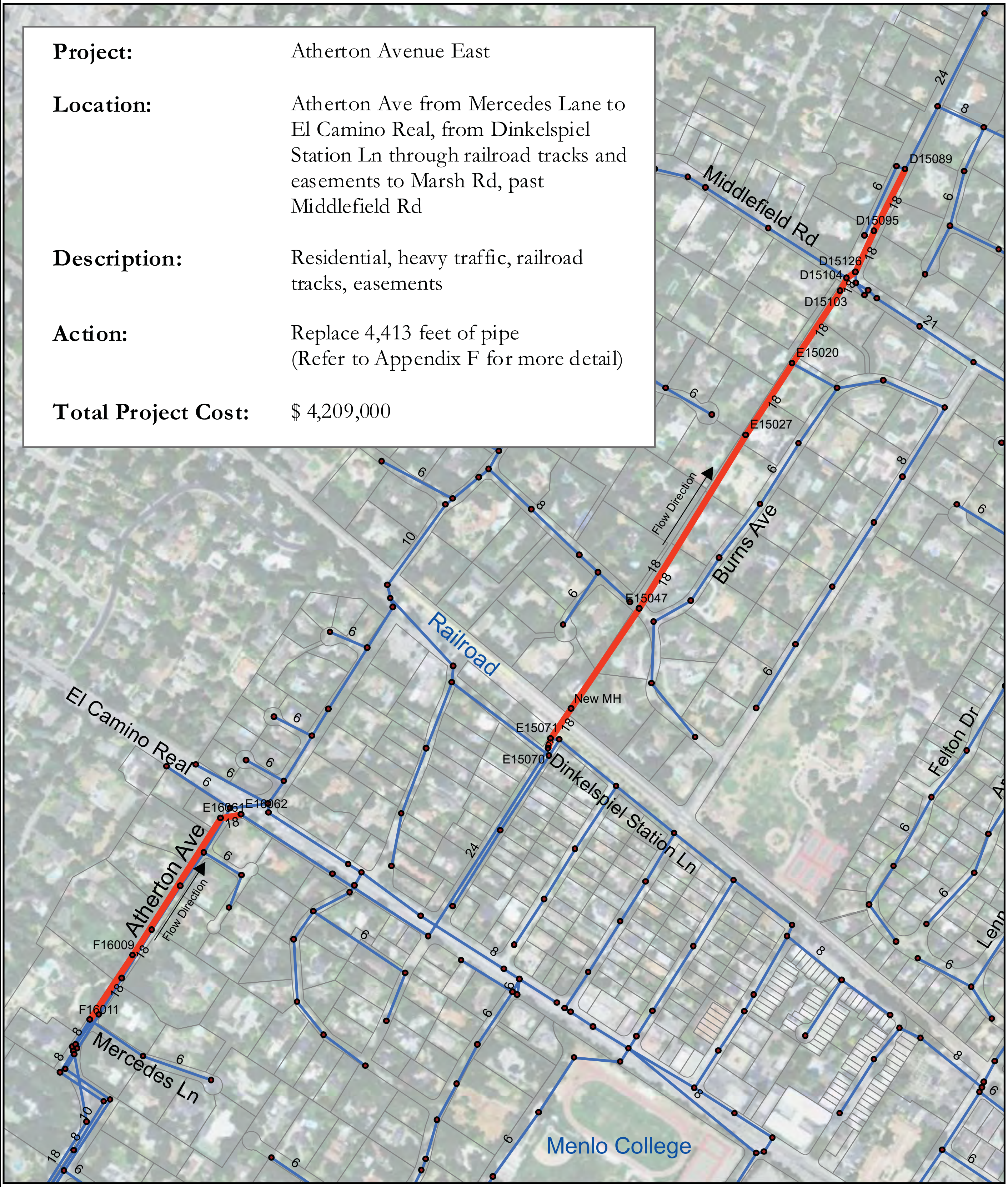
West Bay Sanitary District
Wastewater Collection System Master Plan

Middlefield at Fair Oaks

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES
Consulting Engineers

Project:	Atherton Avenue East
Location:	Atherton Ave from Mercedes Lane to El Camino Real, from Dinkelspiel Station Ln through railroad tracks and easements to Marsh Rd, past Middlefield Rd
Description:	Residential, heavy traffic, railroad tracks, easements
Action:	Replace 4,413 feet of pipe (Refer to Appendix F for more detail)
Total Project Cost:	$ 4,209,000



LEGEND

- Manhole
- Pipe Segment
- Project Pipe Segment
- Parcels

N

0 200 400

Scale in Feet

FIGURE 7-7

West Bay Sanitary District

Wastewater Collection System Master Plan

Atherton Avenue East

WEST BAY SANITARY DISTRICT

WEST YOST ASSOCIATES Consulting Engineers

CHAPTER 8
Pipeline Condition Assessment and Rehabilitation Projects

In June 2009, the District Board decided to update its CIP to include additional collection system replacement projects. In December 2009, West Yost developed a prioritized CIP to provide the District with projects that could be initiated in 2010. Following this early planning effort, the District designed three critical projects to replace pipelines that had known potential maintenance issues and were located adjacent to waterways. These projects have been designed and are currently under construction. Construction is expected to be completed in 2012.

This chapter comprises a list of pipeline replacement projects that expand upon the District's previous efforts to create a long-term pipeline rehabilitation program. The long-term program is a component of the 10-year CIP that is presented in Chapter 10. It should be noted that only the projects identified for FY 2011/12 are assured to be designed and implemented on their proposed schedule. Beyond this fiscal year, the rehabilitation program is intended to be an evolving program that is revisited as system needs are known, and reprioritized as needed to best meet the overall objective of reducing or eliminating SSOs and increasing system reliability over time.

The pipeline replacement program was developed through an assessment of pipeline rehabilitation needs. Implementation of the proposed pipeline replacement projects is intended to improve collection system performance as measured by a reduction in the number and volume of SSOs. To develop the project list, West Yost assessed the District's operations, maintenance, and spill records to identify recommended projects, and then completed an analysis process to prioritize these projects.

This chapter is organized as follows:

- Task Background and Summary,
- Description and Analysis, and
- Recommended Projects / Discussion of Materials and Methods.

8.1 TASK BACKGROUND AND SUMMARY

The District serves all or portions of the communities of Menlo Park, Atherton, Portola Valley, East Palo Alto, and Woodside, located in both Santa Clara and San Mateo Counties. The District's service area ranges from hilly, tree-covered residential areas to relatively low-topography communities, and includes narrow residential roadways, heavily traveled transportation corridors, and several commercial districts.

The District maintains a record of sewer system assets and maintenance activities in GIS, database, and paper formats. West Yost reviewed available data resources that have been compiled since at least 2005, and used the data to develop and prioritize a list of rehabilitation and replacement project needs. These resources, which are listed below, are not included as part of this report. However, they are available from the District as needed to confirm the recommendations presented herein, or to complete any ongoing planning activities related to the District's future pipeline replacement program.

To supplement information gained through review of the following resources, West Yost met in a workshop setting with District staff on July 15, 2010 to further refine rehabilitation and replacement recommendations.

- Collection system pipe and facility asset information;
- Past District SSOs, using the State Water Resources Control Board, CIWQS and District's records;
- District cleaning schedules and comparison of high frequency cleaning locations with historical SSOs; and
- District CCTV condition ratings.

Analysis of this information yielded 22 sewer pipeline projects that are recommended for implementation over time[1]. The actual schedule will be determined by actual project cost, available budget, and other competing priorities determined through future planning efforts. These projects include collection system pipelines that have known potential structural issues, as documented through CCTV inspections.

In addition to the pipeline rehabilitation projects, the Pump Station Technical Memorandum (Freyer & Laureta, Inc., May 2011) proposed five forcemain or lift station upgrade/replacement projects. These projects are discussed in Chapter 9 of this report. Also, projects that are recommended solely to provide additional capacity to convey wet weather flow are discussed in Chapter 7 of this report. Recommendations from Chapters 7, this Chapter 8, and Chapter 9 are summarized and prioritized in the District's long-term CIP, which is presented in detail in Chapter 10 of this Report.

The recommended pipeline rehabilitation and replacement (R&R) projects are presented in Table 8-1 in order of relative priority. The estimated costs and proposed implementation timeline for these projects are presented in Chapter 10.

[1] [1] During completion of the 2011 Master Plan, the District solicited bids for five proposed projects. The District received bids that were lower than anticipated. Therefore, subsequent to the adoption of the 2011 Master Plan, the District added an additional project for completion in FY 2011/12 that rehabilitates miscellaneous pipe segments using cured-in-place pipe lining. Although the added project is not discussed in this chapter, the project and associated budget have been included in the District's CIP that is presented in Chapter 10.

Table 8-1. Summary of Planned Projects

R&R CIP Priority	Project Name	Priority Basis
1	Atherton Avenue	To be completed before road overlay project
2	Ladera Outfall	Previous SSOs reported along sewer
3	Fletcher	Previous SSO within private residence
4	Willow Road	Past SSOs
5	North Palo Alto Concrete	Potentially defective sewer close to creek
6	Menalto Easements	Past SSOs due to grease
7	Roble Avenue	Sewers with known potential structural defects
8	Encinal Avenue A	Sewers with known potential structural defects
9	Oak Grove Avenue	Sewers with known potential structural defects
10	Encinal Avenue B	Maintenance issues
11	Bayfront Expressway	Corrugated metal pipe in poor condition
12	Berkeley Avenue	Sewers with known potential structural defects
13	Santa Cruz Avenue	Sewers with known potential structural defects
14	College Park North	Sewers with known potential structural defects
15	Stevenson Lane	Sewers with known potential structural defects
16	Elena Avenue	Sewers with known potential structural defects
17	Fair Oaks Lane	Sewers with known potential structural defects
18	Frederick	Maintenance issues
19	Suburban Park (formerly Flood Park)	Sewers with known potential structural defects
20	Oak Knoll Area	Sewers with known potential structural defects
21	Haven	Maintenance issues
22	Carlton-Madera Easements	Maintenance issues

8.2 DESCRIPTION AND ANALYSIS

This section summarizes the reviews completed by West Yost, and associated findings from each of these reviews, for each of the sources of data that was used to develop the rehabilitation and replacement project list.

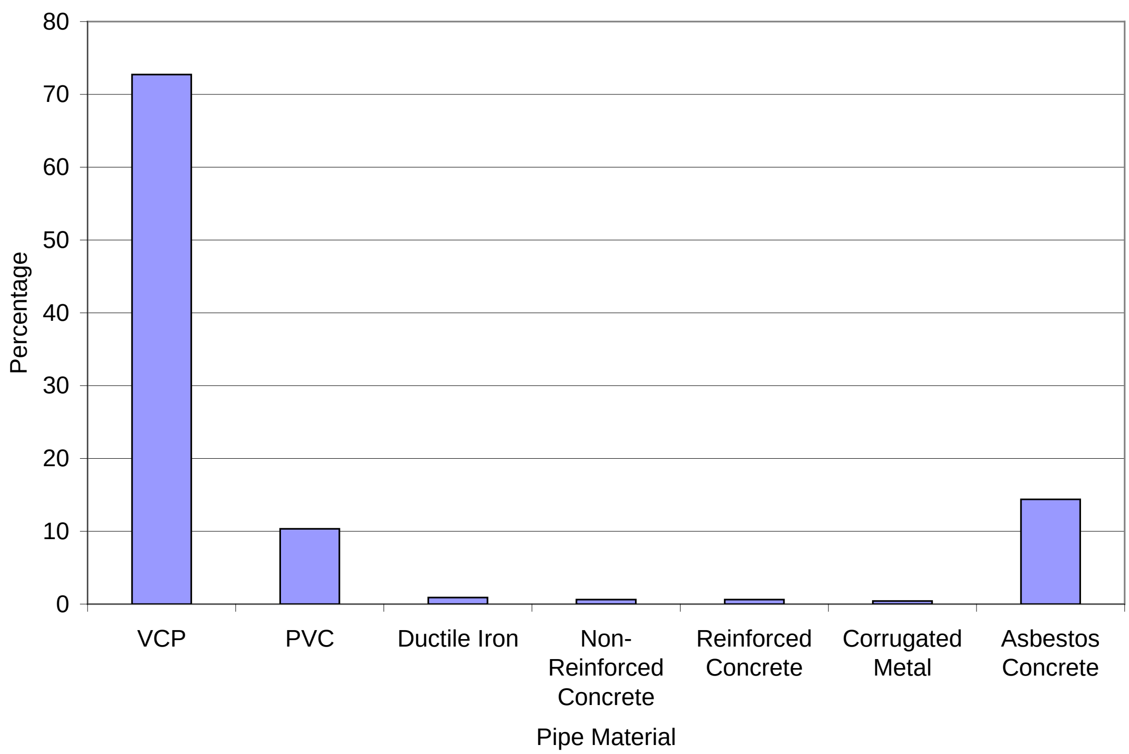
8.2.1 Collection System Asset Information

District staff provided collection system asset information for use in development of the rehabilitation program. Information reviewed included pipeline and manhole locations, diameters, and CCTV condition ratings following the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment and Certification Program (PACP) protocol. To supplement this information, the District has provided hard copy maintenance and CCTV inspection data for specific pipe segments, produced from the District's maintenance management software, GBA Master Series (GBA).

District staff has estimated the age of the sewer network based on known growth patterns within the District's service area, adjusted to reflect known system replacements completed by the District's rehabilitation crew (for point repairs) or outside contractors (for larger pipeline replacement projects). Age in itself is not an indicator of condition or need for replacement. However, the age distribution of pipes in the system is important to document, and it is recommended that the District watch aging pipes (in the District's case, nearing 100 years old) closely. Approximately six percent of the District's pipes were installed between 1900 and 1920.

The majority of the District's collection system is comprised of 6-inch vitrified clay pipe (VCP). However, pipe diameters range from 4-inches or smaller in the residential collection system to 54-inch trunk sewers. Figure 8-1 summarizes the District's system by pipe material.

Figure 8-1. Distribution of Pipe Material in WBSD System



8.2.2 Sewer System Overflow Data

West Yost reviewed SSO reports from 2005 to 2009, obtained from the CIWQS database. Reported SSOs were grouped by cause - roots, grease, paper/rags, foreign objects, mud, other, and unknown. Over two thirds of the historic SSOs were caused by root intrusion. Some overflows were also caused by residential grease buildup, which appeared to be concentrated in areas with 6-inch diameter pipes and relatively flat pipe slopes. Ninety percent of the pipes that experienced an SSO event, and 100 percent of the pipes showing multiple SSOs since 2005, had a diameter of 6-inches.

~~Two hundred twenty one (221) unique pipe sections, identified by upstream and downstream manhole identification numbers (IDs), were responsible for the 258 reported SSO events since 2005. The 27~~ Pipe sections that had two or more SSOs since 2005 were compared against the District's high frequency cleaning list, and, if not listed, were added to the list, as described in the following section.

8.2.3 Comparison of Cleaning Schedule to SSO Occurrence

West Yost reviewed the District's cleaning schedules, obtained through GBA. The District cleans its pipes on 1-month, quarterly, bi-annually, annually, and greater frequency depending on the asset's age, location, size, and maintenance needs. West Yost compared the pipe sections on a less-than-annual cleaning schedule, identified as the high frequency cleaning schedule, to SSO records to confirm that pipes with known SSOs were systematically designated as high frequency cleaning locations. Of the pipe segments that have had two or more SSOs since 2005, many were already listed on the high frequency cleaning schedule. The remaining segments not previously on the list were moved to the less than one year high frequency cleaning list in approximately January 2010.

Similarly, pipe segments that have experienced one SSO between 2006 and 2009 attributed to roots or grease were compared to the 1-year high frequency cleaning list, and if not present, placed on this list.

8.2.4 CCTV Condition Ratings

West Yost reviewed CCTV records and associated hard copy inspection and maintenance reports that were provided by District staff. The District uses Pearpoint inspection equipment and captures CCTV inspections and condition ratings in a Flexidata database, and stores the data in GBA, where it can be uploaded from an external hard drive. West Yost relied upon the District's condition rating scores to evaluate pipeline condition. For several groupings of pipe segments, hard copy inspection reports were also available and provided added information on pipeline condition. The District's condition ratings followed the NASSCO PACP rating system, which rates pipe defects from 1 to 5. Rating descriptions are detailed and descriptive, and can be summarized as follows:

- Grade 1 – Acceptable structural condition
- Grade 2 – Minimal collapse risk
- Grade 3 – Failure unlikely in the near future
- Grade 4 – Failure likely in foreseeable future
- Grade 5 – Failure probable within the next five years

According to the NASSCO recommendations, pipes with multiple structural grade 5 defects should be planned for replacement. The pipeline replacement program focused on pipes with structural Grade 5 defects, with particular emphasis on pipelines located in close proximity to one another or near a tributary to the San Francisco Bay.

The District is in the process of inspecting its entire collection system. Those sections of the system that have not been visually inspected using CCTV, yet were installed in approximately 1900 and are adjacent to a grouping of pipes with PACP ratings of 4 or 5, were assumed to be in poor condition for the purposes of developing the preliminary replacement program. However, this assumption is conservative, as these factors do not necessarily indicate pipeline defects. Therefore, this assumption should be verified during design activities, through focused CCTV inspections.

8.3 RECOMMENDED PROJECTS / DISCUSSION OF MATERIALS AND METHODS

The evaluation of available data resulted in the identification of projects that compose the recommended long term pipeline rehabilitation and replacement program. The proposed projects are intended to replace pipe sewer segments with structural defects having at least one PACP structural Grade 5. In order to condense the number of projects and create efficiencies in project implementation, sewer segments with at least one grade 5 structural defect were grouped into larger projects based on location.

West Yost completed field visits of the proposed sections of pipeline to be replaced. West Yost staff made general observations of the areas and noted the potential for ground movement, heavy loading, tree growth, impacts from adjacent utilities, and other factors that could generally impact sewer condition. Field observations are summarized in the individual project descriptions. Also noted on the field visits were potential impacts, such as private easements or existing structures that would increase the cost of construction.

Descriptions for the projects recommended for the pipeline rehabilitation and replacement CIP are provided below. Individual capital improvement project fact sheets are included in Appendix G. Because the replacements are needed to address condition (not capacity) needs, all pipes will be replaced in kind, except 6-inch diameter pipe, which will be replaced with 8-inch diameter pipe in accordance with the District's standards for construction. The various pipeline materials and construction methods included in the project descriptions are described in the sections following the project description.

These proposed projects share at least two of the three following issues:

- A large number of pipeline segments were defined as requiring heavy maintenance, or were identified through CCTV inspection records as having structural defects or high maintenance needs;
- Pipes were installed in approximately 1900; and/or
- Pipes are located adjacent to a tributary to the San Francisco Bay, or are located within a heavily traveled transit corridor.

The proposed projects are generally of the same size, with a total estimated construction cost of no greater than $3 million. These projects were arbitrarily named as shown below. The names reflect the streets that would be impacted by the project, and can be revised as desired by District staff. The chapter section where the project is discussed is shown in parentheses.

- Atherton Avenue (8.3.1.1)
- Ladera Outfall (8.3.1.2)
- Fletcher (8.3.1.3)
- Willow Road (8.3.1.4)
- North Palo Alto Concrete (8.3.1.5)
- Menalto Easements (8.3.1.6)
- Roble Avenue (8.3.1.7)
- Encinal Avenue A (8.3.1.8)
- Oak Grove Avenue (8.3.1.9)
- Encinal Avenue B (8.3.1.10)
- Bayfront Expressway (8.3.1.11)
- Berkeley Avenue (8.3.1.12)
- Santa Cruz Avenue (8.3.1.13)
- College Park North (8.3.1.14)
- Stevenson Lane (8.3.1.15)
- Elena Avenue (8.3.1.16)
- Fair Oaks Lane (8.3.1.17)
- Frederick (8.3.1.18)
- Suburban Park (formerly Flood Park) (8.3.1.19)
- Oak Knoll Area (8.3.1.20)
- Haven (8.3.1.21)
- Carlton-Madera Easements (8.3.1.22)

8.3.1 Project Descriptions

The higher priority project descriptions are presented first. These priorities are likely to change over time, as new system information is gained by the District. Fletcher, Atherton Avenue, Ladera Outfall, and Willow Road are proposed as the highest priority projects for the District. The remaining 17 projects are generally of equal priority, and should be distributed in the CIP that is described in Chapter 10 as needed to obtain a balanced cash flow over the CIP timeframe.

A number of sewers displayed structural defects in isolated locations, but the remaining sewer was structurally sound. Most of these structural defects can be corrected by implementing “spot repairs.” The spot repairs do not replace the entire manhole-to-manhole reach of sewer; they replace only the broken or defective pieces of pipe. However, once the spot repair is completed, the sewer is considered to be sufficiently repaired for its entire length.

For cost estimating purposes, this analysis assumed that a minimum of 20 feet of sewer would be replaced for each spot repair. If a sewer displayed poor structural condition for the entire reach, then complete pipe replacement was assumed. The rehabilitation and replacement cost estimates included in Appendix H clarify in the “notes” field whether a project was a spot repair. Copies of CCTV inspection reports (if provided by the District) for the projects are included in Appendix I. The CCTV reports provide the location of the structural defect (distance from the upstream or downstream manhole) and the PACP rating.

The District is considering installing cross connections between sewers that serve different tributaries. The purpose of the cross connections would be to provide relief and conveyance of peak wet weather flows. The District has identified a few preliminary locations for the cross connections, but the details of linking the sewers should be completed as part of the design phase.

8.3.1.1 Atherton Avenue (Figure G-1)

The Atherton Avenue project would replace 1,380 feet of sewers in Atherton Avenue between Elena Avenue and El Camino Real as shown in Figure G-1. This project is proposed as the District's second highest priority project because it must be designed before the Town of Atherton begins a planned overlay in 2011. This project comprises a combination of point repairs and manhole to manhole replacements. As a result, the total length of pipe to be replaced is less than the total length in service within the project alignment. The total length of pipe to be addressed by this project would cover 6,521 linear feet.

In Atherton Avenue, parallel sewers convey flow from Elena Avenue to El Camino Real. From Elena Avenue to Mercedes Lane, there are three separate 8, 10, and 18-inch diameter sewers. From Mercedes Lane to El Camino Real, only the 10 and 18-inch diameter sewers convey flow. The 18-inch diameter sewer is 3,635 feet long, the 10-inch diameter sewer is 2,839 feet long, and the 8-inch diameter sewer is 3,682 feet long. Most of the sewers were originally installed in the 1950s.

CCTV reports were provided for these reaches of sewers. The 18-inch diameter sewer is in good condition, *i.e*,. has NASSCO PACP ratings of 3 or lower, and does not require rehabilitation or replacement. However, based on the structural condition ratings provided for these pipelines, the 8- and 10-inch diameter sewers may be considered to be in poor condition since some areas display pipe collapses, holes, breaks, circumferential fractures, cracks, and root intrusions. This project would include spot repairs to repair isolated occurrences of structural condition ratings of 4 and 5, and manhole-to-manhole pipe replacement to replace pipe segments with three or more recorded defects. The detailed cost spreadsheet in Appendix H provides more information on the pipe segments recommended for replacement.

The project is located within residential neighborhoods. However, Atherton Avenue is a main road and experiences heavy traffic. The intersection with El Camino Real is a busy, signalized intersection, which will likely complicate pipeline installation and construction staging. Therefore, the project will likely involve pipeline replacement using open trench methods. However, due to high traffic volume, the intersection with El Camino Real may require trenchless installation. The District may consider consolidating the 8- and 10-inch diameter sewers into a single equivalent size diameter pipeline, depending on hydraulics, and sewer main and lateral tie-ins. Pipeline hydraulics for Atherton Avenue replacement options will be reviewed as part of the capacity analysis.

8.3.1.2 Ladera Outfall (Figure G-2)

The Ladera Outfall is comprised of 3,125 feet of pipe located between Andeta Way in Ladera (west of I-280) and the west side of I-280 as shown in Figure G-2, and an additional 2,670 feet of pipe east of I-280, for a total length of 5,795 feet. The pipeline alignment runs through open space, rural roads, creek banks, and agricultural land. This project is proposed as a high priority for the District because the sewer runs parallel to a creek and SSOs have historically been reported along these reaches of the sewer. The District has also documented, through maintenance knowledge and CCTV inspection reports, partial sewer collapse and failure along the alignment. The CCTV inspection results also revealed the presence of multiple root balls and root intrusion at pipe joints, which could inhibit flow and potentially cause an SSO.

The pipes that would be replaced or rehabilitated in this area are generally 8- and 10-inch diameter VCP according to the CCTV inspection results (although the GIS data indicated that there were two pipe segments immediately downstream of Andeta Way that may be polyethelene and PVC material). CCTV reports were available for this pipeline and it was determined that spot repairs or replacement of the entire sewer would be recommended to correct structural defects.

Although the project limits stopped on the west side of I-280, the CCTV inspections continued east of the highway. The inspections indicated that there may be a broken piece of 10-inch diameter sewer under I-280 that should be fixed. Since complete replacement of the sewer is unlikely and likely unnecessary, this analysis assumed a structural liner would be installed to span the entire length of the highway crossing. Permitting and coordinating with Caltrans will likely add to the design duration and costs for this segment of the project.

Since the CCTV inspections revealed a number of root balls and root intrusions at pipe joints, the District should complete a non-structural liner project for the majority of the outfall to prevent additional roots from clogging the sewer. The cost estimate assumes that the entire reach will be rehabilitated, except for two reaches of sewer that will be completely replaced due to structural defects.

Due to the terrain and proximity to the creek, replacing or rehabilitating this pipeline presents many construction challenges. The project starts at Andeta Way, near the Ladera Recreation District in Ladera and progresses north towards San Francisquito Creek Road along a tree-lined, open space alignment. The alignment continues a short distance adjacent to San Francisquito Creek Road before turning north, following the creek along Ansel Lane to I-280.

Several constraints will complicate construction and increase the costs to replace or rehabilitate this sewer. Alternative construction methodologies should be considered, such as pipe bursting or reaming, or pipeline relocation to avoid potential impacts. The presence of large trees and location of the existing sewer in close proximity to a creek bank will likely preclude open trench construction in many reaches. Open trench construction near or within a creek bank introduces potential lengthy and costly environmental permitting process with multiple agencies including, but not limited to, the U.S. Army Corps of Engineers, U.S. Environmental Protection Agency, U.S. Fish and Wildlife Service, and the California Department of Fish and Game. Due to the potential for material to enter the creek even if trenchless construction methods are employed, best management practices will likely be required to minimize the environmental impacts on the creek. A temporary construction easement (TCE) may be required in some reaches if the width of the existing sewer easement is insufficient to conduct all construction activities.

Due to the environmental resource permitting requirements, the potential construction impacts that will be encountered, and difficult terrain of this alignment, the construction contingency for this project was increased from 30 percent (used for all other projects) to 50 percent.

8.3.1.3 Fletcher (Figure G-3)[2]

The Fletcher project is located near the intersection of Fletcher Drive and Ridgeview Drive, within a 300 foot residential easement. In 2009, a SSO occurred within a private residence connected to this sewer. Fletcher is recommended to be the District's highest priority project because of the SSO threat in a private residence, due to liability from homeowner claims. The project would replace and upsize 400 feet of 6-inch diameter VCP with an 8-inch diameter pipeline. This sewer runs under large trees. In order to replace the sewer, existing trees on the private property will require removal. The District has had historical communications with the resident related to this need, and has met resistance. If the District is not able to secure permission or a permit to remove the trees impacting the sewer and private sewer lateral, then relocation of the sewer or lateral may be required. This section of pipeline has not been inspected, and the condition remains unknown. The project will likely involve pipeline replacement using both open trench and trench-less methods.

8.3.1.4 Willow Road (Figure G-4)

Willow Road would include parallel small diameter sewers along Willow Road between Middlefield Road and Coleman Avenue, as well as parts of McKendry Drive and Clover Lane. This proposed high priority project would replace 951 feet of sewer segments, each with a PACP structural rating of 4 or 5. The majority of sewer replacements require only spot repairs, and not manhole-to-manhole replacement of sewer reaches. Therefore, the length of repairs is less than the length of alignment that will be addressed by this project. The total length of alignment to be addressed would be 3,270 feet.

A CCTV inspection report was not provided for one 20 foot sewer segment recommended for replacement from manhole D12048 to D12049. This sewer should be inspected to confirm its structural condition and if there is a need for replacement, as noted in the detailed cost estimate in Appendix H.

This project is proposed as a priority for the District because Willow Road has experienced SSOs in the past three years, and CCTV inspection reports for the surrounding pipes have recorded likely structural defects. The Willow Road sewers are located close to the San Francisquito Creek and are recommended for replacement.

Willow Road is a major traffic arterial, leading to US Highway 101 and, therefore, traffic control is expected to add complexity to the construction activities. A four foot long spot repair was completed by the District in December 2010, which revealed an 18-inch thick layer of concrete beneath the asphalt. The construction contingency for this project was increased from 30 percent (common for other projects) to 50 percent to account for the 18-inch thick asphaltic concrete. The project will likely involve pipeline replacement using a combination of pipe bursting and open trench methods.

[2] Subsequent to adoption of the 2011 Master Plan, the District removed the Fletcher Project from the CIP, since this project is already underway. The District replaced this project with a pipeline to rehabilitate a short section of pipe on Haven Avenue in Menlo Park. Although the Haven Avenue project is not described in this chapter, the CIP reflects the updated cost for the Haven Avenue project.

8.3.1.5 North Palo Alto Concrete (Figure G-5)

North Palo Alto (NPA) Concrete would consist of 4,249 feet of 8 and 12-inch diameter VCP and un-reinforced concrete pipe on Walnut Street, Woodland Avenue, Pope Street, different easements and alleys near Chester Street and Laurel Avenue, and various other streets. A majority of the pipes that are included in this project have likely structural defects or are made of the same concrete material that is found in known defective pipelines, and is recommended to be replaced. Although all of the sewers are located in residential, low traffic areas, pipe replacement within several easements and constrained alleys will be challenging due to limited equipment access. In particular, the easements off Chester Street and Central Avenue pass under many large trees and fences.

The projects within public right of way would likely involve open trench construction methods. However, the sewers in easements would likely require trenchless replacement options. Some sewers in easements meet in a tee configuration, making trenchless rehabilitation options less practical. Also, some sewers on private property appear to cross below existing structures. It is highly recommended that the District relocate sewers that are not currently accessible, due to the presence of structures or other obstructions, to a location that is within a District easement on open property or to public right of way.

8.3.1.6 Menalto Avenue Easements (Figure G-6)

The Menalto Avenue Easements project would replace pipeline located within four easements. The project would be comprised of 2,245 feet of 8-inch diameter VCP located between Ralmar and Laurel Avenues to the west and east, and Alberni and Newbridge Streets on the north and south. This project has already been partially designed by the District. The original project was developed to replace pipes that have a history of grease-related SSOs. The project would also include additional pipelines that have been added to the designed project, which have likely structural defects. The project would likely involve pipeline replacement using both open trench and trench-less methods. The District should consider extending the sewer from Manhole C11007 to Menalto Avenue in order to improve maintenance access from a manhole within public right of way.

8.3.1.7 Roble Avenue (Figure G-7)

The Roble Avenue project would be comprised of 9,550 feet of pipe located along Menlo Avenue, Roble Avenue, and University Drive, including various easements. The sewer lines are comprised of either 6- or 8-inch in diameter VCP. These sewers were installed in 1915, and over half of the pipelines have likely structural defects. Replacement of pipe segments from manhole to manhole is recommended.

Along Menlo Avenue, the pipeline is located in a business district. Due to the commercial activity and narrow alleys along Menlo and Roble Avenues, construction will require traffic control and could impact local businesses. Also, the pavement along University Drive may be under moratorium, and would need to be replaced in its entirety after construction. The project will likely involve pipeline replacement using open trench methods.

8.3.1.8 Encinal Avenue A (Figure G-8)[3]

Encinal Avenue A is comprised of 8,718 feet of 8-inch diameter VCP located along Encinal Avenue, Felton Drive, Arden Drive, and Lennox Avenue. All the pipes are located on residential streets with little traffic. The pipes have likely structural defects and are recommended for replacement from manhole to manhole.

Along Felton Drive, Arden Avenue, and Lennox Avenue, the pavement is in very poor condition and may have to be entirely replaced after pipe excavation. Two oak trees, one on Arden and one on Lennox, stand in the middle of the road directly over the path of the sewer line and must be considered during design. The project also passes directly in front of Encinal School, which will impact traffic control and construction hours, and across Caltrain railroad tracks. The majority of the project will likely involve pipeline replacement using open trench methods. However, the section of pipe passing beneath the railroad tracks and existing oak trees may require the use of trenchless technology.

8.3.1.9 Oak Grove Avenue (Figure G-9)[4]

The Oak Grove Avenue project is comprised of 4,419 feet of pipeline on Oak Grove Avenue, between Laurel Street and Middlefield Road, including Laurel and Pine Streets to the west of Oak Grove. The pipes along Oak Grove Avenue and the streets directly south of Oak Grove Avenue have likely structural defects. Oak Grove includes two sewer pipelines, of which only the eastern pipeline requires replacement. The western pipeline was found to be structurally sound based on CCTV inspections. The sewer pipeline to be replaced would include 1,939 feet of 10-inch diameter VCP. The remaining project pipelines are currently either 6- or 8-inch diameter VCP.

The project approaches Menlo Atherton High School, and construction may impair school traffic. Further, Oak Grove is a major thoroughfare and the sewer line passes through three signalized intersections. A large oak tree at the corner of Pine Street and Cherry Avenue lies in the path of the sewer, and must be considered during the design. Despite these concerns, the project will likely involve pipeline replacement using open trench methods. The District might have the option to rehabilitate the 10-inch diameter sewer in Oak Grove Avenue with a structural or non-structural liner in lieu of replacement. The sewer should be inspected during design to determine whether rehabilitation is warranted.

[3] During development of the final draft of the 2011 Master Plan, the District scheduled the repair of approximately 7,300 lf of Encinal Avenue A pipelines for completion in FY2011/12. Pipe segments will be rehabilitated using cured-in-place pipe liner.

[4] During development of the final draft of the 2011 Master Plan, the District scheduled the repair of approximately 1,940 lf of Oak Grove Avenue pipelines for repair in FY2011/12. Pipe segments will be rehabilitated using cured-in-place pipe liner.

8.3.1.10 Encinal Avenue B (Figure G-10)

Encinal Avenue B is comprised of 7,000 feet of pipeline, including 550 feet of 10-inch diameter VCP along El Camino Real. The sewer extends along Encinal Avenue, through Menlo College, and ends at the corner of MacBain Avenue and Howard Way. Three lines extend east along San Antonio Avenue, Victoria Drive, and El Camino Real. Encinal B is a continuation of Encinal A, and involves replacing pipes from manhole to manhole, due to known maintenance issues. The project would include a 1,835-foot easement that runs under Menlo School and Menlo College that will be difficult to access. Further, the section of pipe along El Camino Real experiences high traffic and will present traffic control and other challenges during construction. The project will likely involve pipeline replacement using open trench methods, however trenchless technology may also be necessary for the section of sewer running under Menlo School and Menlo College.

8.3.1.11 Bayfront Expressway (Figure G-11)

The Bayfront Expressway project would include lining and spot repairs along 4,276 feet of 30-inch diameter corrugated metal pipe (CMP). District staff have reviewed CCTV inspection tapes for this pipeline and report the pipe to be in generally good condition, with only a few specific areas needing repair. The existing sewer alignment crosses the railroad tracks and Chilco Street near Modoc Avenue. The sewer runs within the Tyco property, through the parking lot, and eventually exits the property at Chilco Street, north of Constitution Drive. The sewer then continues within a private easement and runs adjacent to several industrial buildings along Bayfront Expressway.

Replacing this pipeline presents many construction challenges because of the railroad crossing, presence of overhead utilities, proximity to existing structures, and potential disruption of existing businesses. If pipeline replacement is required, a temporary construction easement may be required in some reaches if the width of the existing sewer easement is insufficient to conduct all construction activities. All these factors complicate and increase the cost of pipeline replacement. The District is considering installation of a structural liner, comprised of cured in place pipe (CIPP), in lieu of pipe replacement. In addition, specific areas would require point repairs using conventional trenching construction methods. Due to the cost of this project, the District could consider phasing the project over multiple years.

The cost estimate in Appendix H assumes that the entire reach would be rehabilitated with a CIPP structural liner. Use of a CIPP liner may alleviate many of the potential construction impacts that are discussed above.

8.3.1.12 Berkeley Avenue (Figure G-12)

The Berkeley Avenue project would replace 3,932 feet of 8-inch VCP along Berkeley Avenue, Bay Road, within an easement off Berkeley Avenue, Colby Avenue, Peninsula Way, and within an easement between Bay Road and Colby Avenue. The pipes are schedule for replacement due to anticipated condition issues, as indicated by PACP ratings of 4 or 5 for structural defects. CCTV inspection reports were not provided for a number of the sewer segments that are included in this alignment. These sewers should be inspected to confirm their structural condition and need for replacement, as noted in the detailed cost estimate in Appendix H.

The project is located next to Peninsula School, and managing school traffic must be considered during construction. A portion of the project is located within a 1,409-foot easement extending from Bay Road to Colby Avenue that crosses below numerous large trees, fences, structures, and utility poles. A second reach of the project crosses under a 574-foot easement extending north-west off Berkeley Avenue near Entrada Way that also crosses below similar structures. Replacing the pipe by open cut trenching does not seem feasible. Alternative construction methods, such as pipe bursting or reaming, should be considered for reaches of this project.

8.3.1.13 Santa Cruz Avenue (Figure G-13)

The Santa Cruz Avenue project would address 3,870-feet of 8-inch diameter or smaller VCP Pipe on Santa Cruz and Avy Avenues, and 80 feet of 15-inch diameter VCP on Cloud Avenue. Two parallel pipes are located within Santa Cruz and Avy Avenues; only the westerly pipe requires replacement in each street. The westerly pipelines within Santa Cruz Avenue have likely structural defects. On the eastern side, minimal or no structural defects were reported in the CCTV inspection logs.

Both Santa Cruz and Avy Avenues experience high traffic, and parts of Avy Avenue may be under moratorium. The project also passes by two schools, a Montessori preschool at the corner of Alameda de las Pulgas and Avy Avenue, and Hillview Middle School at Elder and Santa Cruz Avenues. The project will likely involve pipeline replacement using open trench methods. Design of this project must be coordinated with any capacity improvements that are planned on Santa Cruz and Avy Avenues, as described in Chapter 7.

8.3.1.14 College Park North (Figure G-14)

College Park North is comprised of 8,683 feet of 6-inch diameter VCP on Partridge, College, and Arbol Road, including portions of University Drive, Yale Road, and Princeton Road. The pipelines have likely structural defects, and are recommended for replacement from manhole to manhole. Although further removed from San Francisquito Creek than College Park South, these pipes are sufficiently close to the creek to consider replacement. All the pipes are located in residential areas and there are no apparent special construction considerations. The project would likely involve pipeline replacement using open trench methods.

8.3.1.15 Stevenson Lane (Figure G-15)

The Stevenson Lane project would run along Stevenson Lane from Atherton Avenue to Isabella Avenue, crossing several private residences with large backyards and swimming pools. Included in this project is a section of pipe along the western side of Elena Avenue and continuing south on Atherton Avenue. Also included would be a reach of sewer in Isabella Avenue, between Elena Avenue and the Stevenson Lane easement. The Elena Avenue sewer is 1,404 feet in length and comprised of 8-inch diameter VCP. The Stevenson line is comprised of 1,503 feet of 8-inch diameter VCP, and the Isabella sewer is 1,063 feet of 8-inch VCP, for a total project length of 3,970 feet. Both the Stevenson pipe and Elena pipe are anticipated to be in poor condition, and the project replaces entire pipe sections from manhole to manhole. The Stevenson Lane sewer pipeline and a small segment of Atherton Avenue sewer were inspected in May 2010 and the pipelines displayed a few breaks, circumferential fractures, cracks, and root intrusions in several locations.

The 838-foot easement connecting Stevenson Lane to Isabella Avenue would be particularly challenging as significant vegetation is located directly over the path of the sewer. In addition, two swimming pools may be located adjacent to the pipelines, and building foundations are located close to the alignment. Construction by traditional open-cut methods may not be feasible due to the proximity of existing structures and local restrictions for removing mature trees. Alternative methodologies should be considered, such as pipe bursting or reaming, or pipeline relocation.

8.3.1.16 Elena Avenue (Figure G-16)

The Elena Avenue project would be located directly south of the Stevenson Easement and would include the remaining length of sewer on Elena Avenue, the entire length of Park Lane, and the majority of Santiago Avenue. In total, the project will replace 6,980 feet of 8-inch diameter VCP, except for a 50-foot section of 10-inch entire length diameter sewer along Elena Avenue. Similar to the Stevenson Easement project, each pipeline from manhole to manhole is considered for replacement due to anticipated poor condition. A portion of the pipe is located within a 187-foot easement off Santiago Avenue that runs parallel to a driveway and passes under several large trees. The project would likely involve pipeline replacement using open trench methods.

8.3.1.17 Fair Oaks Lane (Figure G-17)

The Fair Oaks Lane project would be comprised of 7,410 feet of pipeline, primarily along Fair Oaks Lane, Middlefield Road, and Maple Avenue. The project includes 2,675 feet of 6-inch diameter VCP, 3,178 feet of 10-inch diameter VCP on Fair Oaks Lane, and a 1,557 foot, 18-inch diameter trunk line connecting Dinkelspiel Station Lane to Burns Avenue. The pipeline along Fair Oaks Lane is anticipated to be in poor condition, and the pipeline on Maple Avenue and the trunk line has likely structural defects.

One major concern regarding the project would be related to replacement of the 18-inch trunk line. It appears that this pipeline is located under dense vegetation and below or near one or more swimming pools. It is recommended that pipe bursting or pipe reaming, or if possible, relocation be considered as an acceptable alternative for conventional pipeline replacement methods.

This project crosses Caltrain railroad tracks at the corner of Dinkelspiel and Fair Oaks Lane, and passes City administration buildings, including the Atherton Police Department. Except for the replacement of the 18-inch trunk line, open trench methods are anticipated for this project. Design and construction of this project must also consider potential capacity improvement needs along this alignment, as discussed further in Chapter 7 of this report.

8.3.1.18 Frederick (Figure G-18)

The Frederick project would be located in a residential easement that would run from Frederick Avenue to the intersection of Bay Road and Del Norte Avenue. In total, the project will replace 510 feet of 6-inch diameter VCP. These pipes may be in poor condition. The alignment appears to run adjacent to existing pools, under large trees, and under other private structures. Due to the proximity of the existing pipeline to structures, it is recommended that replacement of the pipe include an alignment study to attempt to relocate the pipe away from these facilities. Without such a study, however, the project cost assumes replacement of the pipe in its current location by open trench methods.

8.3.1.19 Suburban Park (formerly called Flood Park, Figure G-19)

The Suburban Park project would encompass multiple streets and easements including, but not limited to, Greenwood Drive, Bay Road, Hedge Road, Dunsmuir Way, and Oakhurst Place. In total, the project would replace 140 feet of 6 and 8-inch diameter VCP. Any pipeline defects detected were isolated to specific segments and not the entire sewer reach. Therefore, the reported length includes only spot repairs in lieu of manhole to manhole replacement. The total length of pipe that would be addressed by these spot repairs is 2,240 feet.

All the sewers are located in two lane roads within quiet residential neighborhoods. The project will likely involve pipeline replacement using open trench methods.

8.3.1.20 Oak Knoll Area (Figure G-20)

The Oak Knoll Area project would encompass multiple streets and easements including but not limited to Campo Bello Lane, Santa Cruz Avenue, Alameda De Las Pulgas, and Stanford Avenue. In total, the project would replace 2,704 of 6-inch diameter VCP. Due to overall condition, in lieu of completing spot repairs, the pipes are recommended for replacement from manhole to manhole. Santa Cruz Avenue is a busy four lane road with traffic signals at the intersection of Alameda De Las Pulgas. Although a majority of the pipes can be replaced using open trench methods, specific locations such as on Santa Cruz Avenue may warrant trenchless construction methods.

8.3.1.21 Haven (Figure G-21)

The Haven project is located on Haven Avenue near Highway 101 and Highway 84, within an industrial area of Menlo Park. The project would replace 700 feet of 6-inch diameter pipe. The project would likely involve pipeline replacement using open trench methods.

8.3.1.22 Carlton-Madera Easements (Figure G-22)

The Carlton-Madera Easements project would replace pipelines located within two easements. The pipelines within these two easements are generally in good condition with minimal or no structural defects. The issues that have been described by District staff appear to be maintenance related, possibly caused by grease build-up and exacerbated by insufficient pipe slope in some reaches. This project would upsize existing pipes to facilitate pipeline maintenance in these areas. The project replaces 3,600 feet of 6-inch diameter VCP for the Carlton Avenue easement and 3,417 feet of 6-inch diameter VCP for the Madera Avenue easement, located between Pierce Road and Hamilton Avenue. Due to the existing location of the sewer in backyards and likely under structures and large trees, the project would likely involve pipeline replacement using trenchless methods, including pipe bursting or reaming.

8.3.2 Existing and Available Pipeline Materials

This section describes the pipe material that comprises the majority of the District's system (VCP), as well as common replacement pipeline materials. Although other materials are used in addition to those listed, these CIP projects would recommend that most pipeline replacements utilize PVC or high density polyethelyne (HDPE), as described in this section.

8.3.2.1 Vitrified Clay Pipe (VCP)

VCP is composed of crushed and blended clay and shale that is formed into pipes that are then dried and fired in a succession of temperatures. VCP is rigid and brittle, and is typically installed in short pipe lengths, which yields a large number of joints. The primary problem with VCP is cracking due to tension. Cracks typically occur in the crown (top) first, followed by the invert (bottom), and finally, at the springline (mid-line) of the pipe.

VCP is susceptible to structural and alignment defects resulting from construction or pipe bedding issues; these defects typically occur within the first two years after construction. Post-installation construction of adjacent utilities can also create localized defects. VCP manufactured before 1958 used rigid joints comprised of cement mortar; these joints are susceptible to failure due to soil movement, and to corrosive effects of certain soils. Root intrusion can exacerbate any of these defects and further degrade joint and pipe integrity.

The National Clay Pipe Institute estimates average useful life of VCP as 100 years. However, due to known joint weaknesses of VCP manufactured in the 1950s and prior, it is prudent to reduce the expected lifespan for older VCP (pre-1958) to a more conservative estimate of 70 to 80 years, although some can last longer. Following this guideline, the District's oldest pipes may be nearing the end of their useful lives.

8.3.2.2 Polyvinyl Chloride Pipe (PVC)

PVC is a light weight, high strength material with a low reactivity, making it well suited for water conveyance. Originally, PVC was most commonly used for water distribution systems, accounting for approximately 66 percent of water pipes in the US. However, more recently PVC has been gaining acceptance in the sewer industry as an ideal alternative to traditional materials, such as VCP or concrete pipes.

PVC can be manufactured with thicker walls and a more rigid structure than HDPE, making it more resistant to sagging and ovaling. PVC can be fused using plastic cement, or joined by a bell and gasket coupling. The life expectancy of PVC sewer pipes is generally 70 to 100 years.

8.3.2.3 High Density Polyethelyne Pipe (HDPE)

HDPE is similar to PVC, a type of plastic with relatively light weight and high strength. HDPE is joined by welding, creating a water-tight conveyance system for wastewater. This material is softer and more flexible than PVC, which makes it preferable in installations with tighter radius bends and limited access for installation. HDPE also exhibits a lower pressure rating at a similar thickness when compared to PVC. However, this pressure rating differential typically does not factor into the design of gravity sewers. The life expectancy of HDPE sewer pipes is generally 70 to 100 years.

8.3.3 Available Rehabilitation Methods

The following rehabilitation methods have been evaluated for the District's priority projects: open cut construction, pipe bursting, cured in place pipe, pipe reaming, and tunneling. Specific to the District's priority projects, factors that determine the most cost effective rehabilitation method include geological and physical setting, existing pipeline material and condition, and available construction access.

Obstructions, road age and condition, traffic loading, nearby vegetation, and proximity to public facilities and businesses were identified and recorded for each project, to assist in the identification of viable construction methods. This information was also used to assess the relative criticality of a particular project. Narrow roads, overhead obstructions, and dense vegetation in the path of the sewer lines make construction access difficult, thereby increasing costs. Critical facilities such as schools, hospitals, and businesses, as well as natural bodies of water represent high risk areas if pipe failure should occur.

Where trenchless rehabilitation methods are required, bid documents should allow a number of construction methods to encourage competition and reduce overall project costs.

8.3.3.1 Open Cut Construction

Description: Open cut or open trench construction, also known as cut and cover, has historically been the most widely used approach for sewer pipe replacements. A trench is excavated that is approximately 18 inches to two feet wider than the replacement pipe, and six to 12 inches deeper than the bottom of pipe. A new pipe is installed, backfill material placed and compacted, and pavement and surface facilities restored. Often, the new pipe is installed in a different location than the original pipe, and the original pipe abandoned in place. In this case, sewer flow continues through the original pipe, and a planned shutdown is scheduled during the "tie-in," when the new pipe is connected to the existing pipe. Alternatively, the existing pipe is removed to allow replacement of the new pipe in the same location. The existing flow is bypassed through a temporary pumped system during construction operations.

Advantages and Limitations: Historically, open cut construction has been more cost effective than trenchless technologies, and consequently, more widely used for pipe replacement. Open cut construction is appropriate in most soil conditions, and beneficial in locations where significant utility crossings are present. An open trench can be adjusted in the field to avoid existing underground obstructions, or to otherwise relocate the new pipe. This method enables installation of a larger diameter pipeline where capacity issues are present, or improved materials when available or needed.

One limitation to open cut construction is in shoring and dewatering. Shoring of the trench walls is required when a trench is greater than five-feet in depth. Excavation below the groundwater table, or in soils that permit infiltration of groundwater into the open trench necessitate aggressive dewatering methods. The added cost of these requirements can decrease the economic viability of open cut construction in specific situations. For pipeline installations in new alignments, a geotechnical investigation is recommended during the design phase to determine whether groundwater is anticipated during construction.

Open cut construction is also difficult where construction access is limited, or on steep hillsides. Open cut construction also impacts surface features and traffic, may introduce safety concerns in highly used or highly traveled locations, and creates temporary noise and dust impacts.

Probable Unit Costs: The unit cost of open cut construction varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, open cut pipeline installation costs range from $10 to $14 per inch diameter per foot of pipe installed.

These pipeline installation costs include excavation, shoring, pipe installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and for planning purposes, are considered equal to the cost of pipeline installation.

All costs in this chapter are based on an ENR CCI for San Francisco of 9909.67 (August 2010).

For the District’s projects, the following unit costs (rounded to the dollar) were applied:

Normal construction conditions:	$10 per inch diameter per foot of pipe
Difficult construction access:	$12
Construction with high groundwater:	$14

8.3.3.2 Pipe Bursting

Description: Pipe bursting is a trenchless construction method by which existing pipe is replaced with the same size or typically one size larger pipe in the same location. Pipe bursting is most effective in replacing pipes from 4- to 36-inches in diameter. This method is the most cost effective when there are few lateral connections, when the old pipe is structurally deteriorated, and when additional capacity is needed and trenchless methods are desired or required.

A conical pipe bursting head is conveyed through the pipe, exerting outward forces that fracture the existing pipe and displace fragments outward into the soil. The head is driven by pneumatic pressure, hydraulic expansion, or static pull; the head is connected to and pulls in the new pipe. The pipe bursting head is inserted and also retrieved through new access pits that are located at approximately 400 to 500 foot intervals.

The optimal pull length is dependent upon the size of the host pipe, the degree of upsize required, and the type of soil in the surrounding subsurface. Additional pits, typically two feet wide by two feet long, are required at each service lateral connection. Pipes suitable for pipe bursting are those made of brittle materials, such as vitrified clay. Typically the replacement pipe material will be HDPE or fused PVC.

Advantages and Limitations: Pipe bursting is quickly gaining popularity as a replacement methodology for small diameter sewers. A small pit is required for entry of the pipe bursting head, which can be extracted through an existing manhole. Pipe bursting replaces the existing pipe by up to 2 diameter sizes without significant open trenching, and therefore reduces surface impacts. The unit cost of pipe bursting is decreasing, and often comparable to open cut methods.

Existing conditions must be considered carefully when specifying pipe bursting. Flowing soils such as sand, highly incompressible soils such as rock, installations below the groundwater table, sensitive utilities located within two to three pipe diameters of the pipe to be burst, historical point repairs that are not conducive to bursting such as steel couplings, or significant sags or pipe collapses will limit the success of pipe bursting operations. Pipe bursting may also create ground vibrations and outward ground displacements adjacent to the pipe alignment; these displacements are exacerbated in shallow installations or when the pipe is significantly upsized. When the existing pipe is shallow, this ground displacement may be controlled by saw cutting pavement over the pipe in advance of the bursting operation. This approach localizes surface heave and provides for more simplified trench patch repair.

Pipe bursting is performed between pits spaced 400-500 feet apart. A manhole can be used in lieu of one pit. During the pipe bursting process, the rehabilitated pipe segment must be taken out of service by rerouting or bypassing sewer flows. Laterals are reconnected through external pits after the pipe bursting activities are completed.

The District has expressed its desire to use rehabilitation technologies other than pipe bursting as a result of past experience with ovaling of installed HDPE pipe. However, pipe bursting technology has continually improved over the years. With proper design and understanding of geotechnical and site limitations, pipe bursting is considered an acceptable alternative in numerous jurisdictions when open cut construction is not feasible, or where the cost of open cut construction exceeds the cost to pipe burst.

Probable Unit Costs: The unit cost of pipe bursting varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, pipe bursting costs range from $8 to $14 per inch diameter per foot of pipe installed. These pipeline installation costs include excavation and shoring of pits, pipe bursting and installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and considered equal to the cost of pipeline installation.

For the District's projects, the following unit costs (rounded to the dollar) were applied:

Normal construction conditions:	$8 per inch diameter per foot of pipe
Difficult construction access:	$14 per inch diameter per foot of pipe

8.3.3.3 Cured in Place Pipe (CIPP)

Description: CIPP is a trenchless repair method that installs a resin-saturated felt liner into the host pipe through existing manholes. The liner is made of interwoven polyester and may be fiber-reinforced for additional strength. Commonly manufactured resins include unsaturated polyester, vinyl ester, and epoxy, each having distinct chemical resistance to domestic wastewater. The CIPP liner is installed by inversion using water or pressurized air; after the liner is in place, the resin-impregnated tube is cured using hot water, steam, or high-intensity UV light, creating a seamless pipe that fits tightly against the host pipe wall. Laterals are then connected to the mainline pipe using a remote controlled cutting device.

Advantages and Limitations: CIPP is a viable rehabilitation technology in 6-inch or larger gravity sewers where the existing pipe has sufficient capacity. Because laterals are connected from inside the lined pipe, little or no trenching is required. Therefore, CIPP may be the preferred alternative in pipelines where trenching would be cost prohibitive. The CIPP method can be used to address structural problems such as cracks, offset joints, and structurally deficient segments as well as root intrusions because the liner forms itself generally to the shape of the host pipe, and can span gaps up to one inch in diameter. The flexibility of the resin tube allows installation through existing bends, further minimizing the need for excavation. The liner is resistant to chemical attack, eliminates groundwater from entering the sewer, and retards further corrosion and erosion of the pipeline.

The thickness of CIPP liner typically ranges from ½ inch to 1 inch and therefore, the final inside diameter is approximately 1 to 2 inches less than the inside diameter of the existing pipe.

CIPP installation requires bypass pumping, and installation length is generally limited to approximately 800 feet due to curing limitations. Therefore, if manholes are located further apart than 800 feet, intermediate trenched access locations are required. Another challenge associated with using CIPP is the procurement, treatment, and/or disposal of water used during the curing process; during the curing process of any resin system, volatile organic compounds are released and must be closely monitored.

CIPP is a viable alternative to pipeline replacement when pipeline replacement options are cost-prohibitive, and when existing pipe diameter can be reduced without compromising system performance. CIPP is not recommended when pipeline slopes or other constraints limit the use of jetting as a cleaning method.

Probable Unit Costs: The cost of CIPP varies significantly depending on site access, pipeline configuration, liner specifications, curing method, ease of disposal of curing water, and bidding climate. However, for conceptual estimating purposes, CIPP installation costs range from $8 to $12 per inch diameter per foot of liner installed in normal conditions. These costs do not include mobilization, trenching if needed, special disposal costs, lateral connections, or traffic control, which are estimated as a separate item, and considered equal to the cost of CIPP pipeline installation.

For the District's projects, the following unit costs (rounded to the dollar) were applied:

Normal construction conditions:	$8 per inch diameter per foot of pipe
Difficult construction access:	$12

8.3.3.4 Pipe Reaming

Description: Pipe reaming is very similar to pipe bursting in that an existing pipe is drilled out and a new pipe of equal or greater diameter inserted in its place. Because pipe reaming does not displace the broken pieces of the old pipe into the soil, this method is better suited to pipe rehabilitation where nearby pipes or utilities might be impacted by the displaced soil.

Pipe reaming employs a directional drill which pulverizes and grinds up the existing pipe while a new pipe is inserted behind it. The old pipe is accessed by an insertion trench, and the drill head is pulled through the pipe by a drill line which runs from an insertion trench where the pipe is accessed to the next manhole. The broken pipe is carried away through the old pipe by drill fluid and collected at the downstream manhole.

Pipe reaming can be used to remove brittle pipes such as those composed of vitrified clay, PVC, asbestos concrete, or ductile iron. Fused PVC or HDPE are typically used for the replacement pipe. Pipe reaming has been effective at replacing sections of sewer over 1000 feet in length or more with little soil disruption.

Advantages and Limitations: Like other trenchless technologies, pipe reaming is advantageous when trying to minimize the impact of construction on traffic and business. When using pipe reaming as a rehabilitation technology, adequate space must be available for the insertion pit and the heavy machinery necessary for directional drilling. Pipe reaming can become very expensive if there are a large number of laterals that must be reconnected to the replaced pipe.

Probable Unit Costs: Similar to pipe bursting, the unit cost of pipe reaming varies depending on site conditions and construction access limitations. However, in paved roadways underlain by generally cohesive soils above the groundwater table, and in areas without significant utility or traffic issues, pipe reaming costs range from $12 to $14 per inch diameter per foot of pipe installed. These pipeline installation costs include excavation and shoring of pits, pipe reaming and installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and considered equal to the cost of pipeline installation.

For the District's projects, the following unit costs (rounded to the dollar) were applied:

Normal construction conditions:	$12 per inch diameter per foot of pipe
Difficult construction access:	$14 per inch diameter per foot of pipe

8.3.3.5 Tunneling

Description: Where open cut construction is not feasible, practical, or cost effective, trenchless methods can be used to install the sewer pipe. A more commonly used trenchless method for small diameter sewer pipe is horizontal direction drilling (HDD). HDD involves drilling an initial pilot hole, enlarging the hole using a specialized cutting tool, and then pulling the new pipe, usually PVC or HDPE, through the drilled hole. Laterals are then connected to the mainline pipe through individual pits.

Advantages and Limitations: HDD presents similar advantages to pipe bursting related to minimized surface impacts when compared to open cut construction. Pipe size increase is not limited with HDD and longer lengths of pipe can be replaced through a single bore.

HDD requires precise location of existing utilities and is not applicable where the new pipeline must maintain a shallow slope or avoid numerous underground facilities. HDD requires experienced equipment operators who are skilled with the location and guidance of the drilling equipment.

Probable Unit Costs: The unit cost of HDD varies depending on site conditions and construction access limitations. However, in areas without significant utility or traffic issues, HDD costs range from $12 to $14 per inch diameter per foot of pipe installed, similar to pipe reaming. These pipeline installation costs include excavation and shoring of pits, drilling, pipe installation, backfill, and compaction. These costs do not include mobilization, paving, traffic control, or pipeline appurtenances, which are estimated as a separate item, and considered equal to the cost of pipeline installation.

For the District's projects, the following unit costs (rounded to the dollar) were applied:

Normal construction conditions:	$12 per inch diameter per foot of pipe
Difficult construction access:	$14 per inch diameter per foot of pipe

8.4 ESTIMATED PROJECT COSTS AND IMPLEMENTATION

Detailed cost estimates for the CIP projects are included in Appendix H and summarized in Chapter 10 of this report. Costs include pipeline installation based on unit costs; mobilization; lateral repair; other construction costs estimated as equal to pipeline installation cost; a 30 percent contingency for construction unknowns (unless otherwise noted); and design and project administration.

The highest priority sewer rehabilitation and replacement projects would be recommended to include the Haven Avenue repair (in lieu of Fletcher), Atherton, Ladera Outfall and Willow Road pipeline replacements, and the Corte Madera Forcemain replacement. These projects have been designed and will be constructed in FY 2011/12.

During development of the final draft of the 2011 Master Plan, the District completed design documents and received bids for these projects. The bids received were substantially less than the planned costs. As a result, in lieu of the planned costs that are shown in Appendix H for these projects, Chapter 10 includes an updated estimate of total cost. This updated estimate was provided by the District, and is based on actual bid and design costs, augmented by an allowance for additional anticipated costs that could be incurred during the construction phase.The currently favorable bid environment has provided an opportunity to potentially increase the number of projects that can be implemented by the District in FY2011/12 within the adopted budget. In order to utilize this available budget, the District has scheduled implementation of portions of the Encinal Avenue A and Oak Grove Projects in FY2011/12. Also, the District plans to implement additional pipe segment repairs likely using open trench construction or cured-in-place pipe, as budget is available. Because these adjustments were made at the same time the 2011 Master Plan was finalized, the associated cost adjustments are not reflected in Appendix H. However, the updated costs are included in the CIP that is discussed in Chapter 10.

8.5 NEXT STEPS

The projects in this report were selected to address the most critical condition issues that are present in the District's sewer system, based on existing operations and maintenance data. These proposed projects should be reviewed and revised if new information identifies a change in project need or priority.

The District is currently completing assessments that are likely to result in additional project proposals and an adjustment in project priorities, cash flows, and timelines. These assessments, which include completion of system-wide CCTV inspection, have started and will continue into FY2011 and later years.

CHAPTER 9
Pump Station Rehabilitation Program

Chapter 9 summarizes an evaluation of the District's modeled and unmodeled pump stations, and presents improvements that are recommended over time to maximize the reliability and useful life of the conveyance system. The recommendations from this analysis are also included in the CIP that is presented in Chapter 10 of this report. Chapter 7 of this report investigated the hydraulic capacity of the Hamilton Henderson, University, and Willow Road Pump Stations based on the hydraulic modeling results, and should be referenced for additional information regarding these three pump stations.

This chapter is organized as follows:

- Background Information
- Summary and Findings
    - Dry and Wet Weather Capacity
    - Condition Assessment
    - Summary of Recommendations
- Conceptual Costs

9.1 BACKGROUND INFORMATION

The District owns and maintains 12 pump stations in the collection system, and also owns the flow equalization facility (FEF) transfer pump station and the Menlo Park Pump Station (MPPS). SBSA operates the FEF transfer station and operates and maintains the MPPS. The 12 pump stations, plus the FEF transfer pump station, are listed in Table 9-1. An assessment was completed for the pump stations to determine their current condition, assess each facility's capacity to convey dry and wet weather flows, and identify recommended upgrades, including their cost and implementation timeline.

A technical memorandum that provides more detail on the capacity and condition assessments is included in Appendix J (Pump Stations Technical Memorandum, Freyer & Laureta, Inc, May 2011). Detailed descriptions for each pump station are included in Appendix J.

Table 9-1. District Owned and Maintained Pump Stations

Pump Station	Location
Hamilton Henderson	Menlo Park, east of 101
Willow Road	Menlo Park, east of 101
Menlo Industrial	Menlo Park, east of 101
University	East Palo Alto
Illinois	East Palo Alto
Vintage Oaks #1	Menlo Park
Vintage Oaks #2	Menlo Park
Stowe Lane	Unincorporated San Mateo County
Los Trancos	Santa Clara County
Sausal Vista	Portola Valley
Corte Madera	Portola Valley
Village Square	Portola Valley
Flow Equalization Facility	Menlo Park, east of 101

9.2 SUMMARY AND FINDINGS

9.2.1 Dry Weather and Wet Weather Conveyance Capacity

The pump station capacity analysis included an evaluation of the time to likely overflow from each pump station wet well in the event of pump failure or some other serious disruption. The evaluation considered both average dry weather and peak wet weather flow conditions. Average dry and peak wet weather flow rates for the Hamilton Henderson, Willow, and University pump stations were taken from the hydraulic model results. Peak wet weather flow rates for the remaining facilities used a wet weather peaking factor of 5 applied to each pump station's projected ADWF. ADWF was estimated by applying the appropriate unit flow factor to the contributing acreage upstream of each pump station.

As shown in Table 9-2, the Willow and University Pump Stations likely have less than one hour of wet well storage under average dry weather flow conditions. Under peak wet weather flow conditions, all pump station wet wells except for Vintage Oaks #1 and Village Square pump stations likely have less than one hour of storage capacity in the event of pump failure. Willow pump station has effectively no wet well storage capacity in the event of pump failure. These calculations were not completed for the FEF pump station.

Table 9-2. Pump Station Characteristics and Time to Overflow											
Pump Station	Number of Pumps	Pump Type	HP	Year Built	Wet Well Size (size , depth)	Force Main Size	Force Main Length	Flow Rate (firm capacity)[(a),] gpm	Flow Rate (all pumps on)[(b),] gpm	Time to Overflow Average Flow[(b)], min	Time to Overflow PWWF[(b)], min
Hamilton Henderson	2	Submersible	35	1991	12' dia x 21'	12"	2000'	2,100	2,650	74	14
Willow	2	Submersible	20	1981	10' x 10' x 21'	10"	700'	1,650	2,200	45	7
Menlo Industrial	2	Submersible	10	2003	8' dia x 15'	4"	600'	310	350	61	12
University	3	Submersible	5	1984	8' dia x 23'	8"	600'	320	700	54	19
Illinois	2	Submersible	20	2010	12' dia x 24'	6"	2100'	580	660	168	34
Vintage Oaks 1	2	Submersible	5	1995	8' dia x 24'	6"	150'	330	425	534	107
Vintage Oaks 2	2	Submersible	5	1996	8' dia x 24'	6"	800'	330	425	258	52
Stowe Lane	2	Dry Pit	7.5	1950	8' x 8' dry, 4' x 8' wet, 25'	6"	700'	340	460	182	37
Los Trancos	2	Submersible	5	2000	8' dia x 14'	4"	700'	100	140	160	32
Sausal Vista	2	Submersible	3.9	1978	6' dia x 13'	4"	1400'	120	135	284	57
Corte Madera	2	Submersible	23	2000	8'-5" x 9'-9.5" x 14'	6"	3650'	340	425	179	36
Village Square	2	Submersible	23	2004	10' dia x 17'	4"	5100'	160	175	5 days	1 day
Flow Equilization	3	Submersible	60	1990	16'-6" x 10'-9"	30"	-	-	-	-	-

(a) Derived from pump curves. Hazen Williams "C" factor of 130 was used in system curve calculations.
(b) This calculation uses ADWF and PWWF rates generated from the wastewater collection system hydraulic model.

The Hamilton Henderson, Willow, and University pump stations were included in the hydraulic model. The model results confirmed that these facilities each have sufficient firm capacity (*i.e.*, the largest pump out of service) to convey predicted peak flows from the 10-year, 6-hour design storm.

Using the peak wet weather flow estimates described above, the remaining pump stations were assessed for their firm capacity with regard to conveyance of peak flows. All of the stations are sufficiently sized to convey flows with a wet weather peaking factor of 5. It should be noted, however, that District staff observed the Corte Madera pump station as nearing capacity, with both pumps in service, during the January 20, 2010 rainfall event. This level of flow, which was classified as having a lower rainfall intensity than the design storm, should not have resulted using a wet weather peaking factor of 5.

It is believed that the gravity sewers upstream and tributary to the Corte Madera pump station receive direct stormwater inflow from one or more developments. This stormwater inflow may raise the wet weather peaking factor significantly, which would in turn increase wet weather flow through the Corte Madera pump station. The District is completing the design of a project to increase the capacity of the Corte Madera force main. It is recommended that the District conduct localized wet weather flow monitoring upstream of the Corte Madera pump station, in order to quantify I&I and establish wet weather design criteria for the pump station and force main.

9.2.2 Condition Assessment

Although a condition assessment was completed for each pump station, only five were recommended for upgrade. The discussion below presents the analysis and recommended improvement for each pump station upgrade.

9.2.2.1 Willow Road Pump Station

This pump station is located off Willow Road, between Ivy Drive and O'Brien Drive in east Menlo Park. The pump station, which is the second oldest pump station in the District, was constructed in 1981. In 2000, the backup emergency generator was replaced due to damage caused by a fire in the adjacent parking lot.

Replacement pumps for this station were purchased in 2006 and installed in 2007. The pumps are believed to be in good condition. However, the motor control center (MCC) is nearing the end of its useful life, and is recommended for replacement because if the components of the existing MCC fail, parts would potentially be difficult to locate due to the equipment age.

9.2.2.2 Corte Madera Pump Station

This pump station is located off Portola Road, directly adjacent to Corte Madera Creek, in Portola Valley. This pump station was originally designed and constructed in the 1960s with a wet well and dry well, which housed a duplex pump system. In 2000, the pump station was rehabilitated and the dry well was converted to a wet well with a duplex submersible pump station. This pump station collects flow from the Village Square Pump Station and the Sausal Vista Pump Station, in addition to the parcels tributary to the gravity lines (which include the

Sequoia's Retirement Center) that flow into the pump station. The generator at this pump station is recommended for replacement due to potential difficulty in finding replacement parts due to equipment age.

As discussed above, this pump station experiences very high wet weather flow, which the pump station has some difficulty conveying, even with both pumps running. The District could consider one of three options to address this problem:

- Replace the pump impellers and construct a new, larger diameter forcemain in Portola Road to increase pump station capacity. Initial calculations show that increasing the forcemain diameter to 10-inches and replacing the pump impellers could potentially increase the pump station's capacity to 660 gpm with one pump running. As noted above, peak flow and required capacity are not known, and should be confirmed through additional flow monitoring and flow evaluation.
- Conduct smoke tests or other I&I investigations for the area tributary to this pump station to determine areas of significant I&I. If significant I&I is located, an I&I reduction program could be implemented following the investigation to reduce peak flows conveyed to the pump station. Due to the difficulty in removing I&I, this option may result in only a partial I&I reduction, and may need to be conducted in conjunction with capacity improvements.
- Demolish this pump station and construct a new, larger pump station at the Sausal Vista Pump Station site. A gravity sewer would need to be extended from the Corte Madera Pump Station to the Sausal Vista Pump Station. Also, the existing forcemain in Portola Road would need to be upsized.

9.2.2.3 Sausal Vista Pump Station

This pump station is located off Georgia Lane in Portola Valley and was constructed in 1978 to serve a single family development off Cherokee Court. In 2008, a fuse in the MCC blew in the control transformer that caused the pump station to shut down, causing a minor overflow. A transformer is recommended for each pump and the controls due to the service being single phase. This configuration prompted the District to consider replacing the single phase power with three phase power. The District is currently working with PG&E to determine a means to obtain three phase power for this station.

This pump station is a good candidate for replacement, in conjunction with the Corte Madera Pump Station replacement option, as discussed above. The Corte Madera Pump Station would be demolished and the Sausal Vista Pump Station would be replaced with a larger capacity pump station to convey the increased flow from Corte Madera. A 2,000-foot, 8-inch diameter gravity sewer from the Corte Madera pump station to the Sausal Vista pump station would be constructed. A new 1,300-foot, 10-inch diameter forcemain would also be constructed in Grove Drive.

An alternative option for Sausal Vista is to upsize the pump station to convey greater peak wet weather flows and construct a new 1,300-foot, 6-inch diameter forcemain.

9.2.2.4 Hamilton Henderson Pump Station

This pump station is located near the intersection of Hamilton Avenue and Henderson Avenue in east Menlo Park. The pump station was constructed in 1991 and is 19 years old. During a 2008 inspection, two pumps were found to need repairs. As a result, wear rings and impellers were replaced. Currently, there are no known mechanical issues at this pump station.

The pump station has slightly corroded guide rails that are recommended for replacement. These bars would be replaced with stainless steel guide rail bars. Also, the emergency generator is recommended for replacement due to age and the potential difficulty in finding replacement parts.

9.2.2.5 Stowe Lane Pump Station

This pump station is located at the end of Stowe Lane in Unincorporated San Mateo County. The station is adjacent to the San Francisquito Creek. The Stowe Lane Pump Station is the District's oldest pump station, and was constructed in 1950. The station has both a dry well and wet well, and is the only District pump station that does not have submersible pumps in a wet well application. The Stowe Lane force main conveys flow to the gravity main in Alpine Road. This force main was replaced in 2003, at which time emergency bypass piping was installed.

This pump station is nearing the end of its design life. The District could convert the pump station to a submersible pump station by converting the dry well into a wet well, similar to the rehabilitation that was performed for the Corte Madera Pump Station.

9.2.3 Summary of Recommendations

Table 9-3 presents potential repair and/or replacement projects and the suggested priority for each project. The highest priority project would likely be the replacement of the Willow Road MCC, closely followed by the construction of a new 3,600-foot, 10-inch diameter forcemain in Portola Road and replacement of pump impellers to increase capacity at the Corte Madera Pump Station. As indicated earlier, Corte Madera Pump Station capacity would be increased by approximately 400 gpm through this project. Further investigation would be needed to confirm that this proposed project will provide sufficient wet weather capacity, and to determine the most feasible and economical option for replacing the Sausal Vista Pump Station.

Table 9-3. Pump Station Repair/Replacement Priority		
Pump Station	Proposed Project	Priority
Willow Road	Replace MCC.	1
Corte Madera	Replace generator, pump impellers and construct new 3,600 foot, 10-inch diameter forcemain.	2
Sausal Vista	Replace pump station and construct new 1,300-foot, 6-inch diameter forcemain.	3a
	Abandon Corte Madera, replace/upsize Sausal Vista, construct new 2,000-foot, 8-inch diameter gravity sewer from Corte Madera to Sausal Vista, construct new 1,300-foot, 10-inch diameter forcemain.	3b
Hamilton Henderson	Replace guide rails and generator.	4
Stowe Lane	Convert existing dry well to wet well and install submersible pumps. Replace 3-foot retaining wall.	5

9.3 CONCEPTUAL COSTS

Table 9-4 presents estimated costs for the proposed rehabilitation projects, as presented in the TM that is included in Appendix J.

Table 9-4. Pump Station Rehabilitation Estimated Project Costs				
Pump Station	Description	Priority	Estimated Construction Cost	Estimated Total Cost
Willow Road	Replace MCC	1	$200,000	$680,000
Corte Madera	Replace generator, impellers, construct 3,600-feet of 10-inch diameter forcemain	2	$900,000	$3,060,000
Sausal Vista	Replace pump station and construct 1,300-feet of 6-inch diameter forcemain	3a	$1,100,000	$3,740,000
Sausal Vista	Abandon Corte Madera. Replace Sausal Vista, construct 2,000-feet of 8-inch diameter gravity sewer, construct 1,300-feet of 10-inch diameter forcemain	3b	$2,220,000	$7,548,000
Hamilton Henderson	Replace guide rails and generator	4	$120,000	$408,000
Stowe Lane	Convert to wet well, install submersible pumps, and replace retaining wall	5	$850,000	$2,890,000