Capacity Assurance Report

Submitted by:

City of Millbrae

In Response to:

Consent Decree between San Francisco Baykeeper and City of Millbrae

Effective November 15, 2010

June 2012

WEST YOST ASSOCIATES

Consulting Engineers

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SEAL OF THE CITY OF MILLBRAE
CALIFORNIA

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Executive Summary

Effective November 15, 2010, the City of Millbrae (City) entered into a Consent Decree with San Francisco Baykeeper (Baykeeper). Section IX of the Consent Decree includes a requirement for the City to submit a Hydraulic Model Workplan by August 1, 2011 and to complete a Capacity Assurance Report (CAR) for the City's sanitary sewer collection system by June 30, 2012. The City submitted the Hydraulic Model Workplan as required by the Consent Decree. This CAR was developed by the City and West Yost Associates (West Yost) in compliance with Section IX of the Consent Decree, using the hydraulic modeling approach that is described in the Hydraulic Model Workplan.

ES.1 BACKGROUND AND INTRODUCTION

The primary purpose of the CAR is to identify all necessary capacity improvements to convey peak wet weather flows from a specific design storm to the Millbrae Water Pollution Control Plant (WPCP) without sanitary sewer overflows (SSOs). The CAR also includes a schedule for construction of the proposed capacity improvements. The schedule completes construction of the improvements within four (4) years from the date of the final Capacity Assurance Report or the termination date of the Consent Decree, whichever is sooner.

The City will continue to review the recommendations in the CAR and study the feasibility of the recommendations in the CAR. There will be an iterative process of refining the CAR as the feasibility analysis continues. The City reserves the right to adopt alternative capital improvements to address system capacity.

This CAR is comprised of the following eight chapters:

- Chapter 1 – Introduction
- Chapter 2 – Existing Wastewater System
- Chapter 3 – System Flows
- Chapter 4 – Hydraulic Model Development
- Chapter 5 – Planning Criteria
- Chapter 6 – Capacity Analysis
- Chapter 7 – Pipeline Rehabilitation and Replacement Program
- Chapter 8 – Capital Improvement Program

The CAR was developed with the following components:

- Development of a fully dynamic hydraulic model that is calibrated to dry and wet weather conditions;
- Use of the hydraulic model for capacity analysis of the sewer collection system using a 10-year, 24-hour design storm as defined by the Consent Decree;

- Development of a pipeline rehabilitation and replacement program that targets pipelines with significant structural defects as determined through the City's closed circuit television inspection program, and identifies additional priority inspection needs based on pipe age and maintenance issues; and
- Development of a prioritized capital improvement program that provides sufficient collection system capacity to convey peak flows from the design storm, and addresses severe collection system defects that are likely to be high contributors to rainfall dependent inflow and infiltration.

ES.2 EXISTING WASTEWATER SYSTEM

The City's existing wastewater system is described in Chapter 2. The existing wastewater collection system service area includes all areas within the City's limits, as well as Capuchino High School in the City of San Bruno. The service area includes approximately 6,500 service connections and 21,500 residents, as of the 2010 Census. Elevations within the service area range from 620 feet above Mean Sea Level on the City's western boundary to 5 feet below Mean Sea Level near San Francisco Bay. The City's service area is shown on Figure ES-1.

The City's wastewater flows generally east/northeast out of the hills and into the flat regions of the City. When flows reach El Camino Real, they are collected and transported to the WPCP in the southeast corner of the City.

The City owns 55 miles of gravity sewer pipe and also owns and maintains 55 miles of publicly owned sewer laterals. The City's gravity mains range in diameter from 6-inch to 36-inch diameter, with 6-inch diameter pipeline predominating. The City owns and operates three pump stations and associated force mains, ranging in firm capacity (*i.e.,* with the largest pump out of service) from 300 to 1,425 gallons per minute.

The City recently completed the WPCP Renovations Project, the goal of which was to increase the reliability of the plant. This project included construction of 1.2 million gallons of flow equalization storage, among other improvements. The dry weather capacity of the WPCP is 3.0 million gallons per day (mgd), and the wet weather capacity is limited by contract to 9.0 mgd.

ES.3 ESTIMATION OF SYSTEM FLOWS

The methodology used to estimate the initial dry weather flow 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 4. The City's estimated existing base wastewater flow generated by sewer system users is 1.56 mgd.

Estimation of system flows was accomplished using wastewater flow monitoring data, City of Millbrae water billing information, and information from the City's 2010 Urban Water Management Plan.

ES.3.1 Wastewater Flow Monitoring Data

A flow monitoring program was conducted by V&A Consulting Engineers (V&A) in February 2011. The program included ten (10) gravity meters and two (2) rain gauges. The ten meters delineated the collection system into eight (8) basins that capture the majority of the City's wastewater flow. Figure ES-2 presents the flow meter locations and associated basins within the collection system. Two rain gauges were installed within the City's service area during the flow monitoring effort. Rain Gauge No. 1 was installed near the shoreline at the intersection of Monterey Street and Bay Street, capturing the rain in the flat areas of the City near the San Francisco Bay coast. Rain Gauge No. 2 was installed at the intersection of Richmond Drive and Geraldine Drive, capturing rainfall in the City's hills.

V&A calculated average dry weather flows from data collected during the flow monitoring program, and West Yost refined these estimates further during development of the CAR. Average dry weather flows include base wastewater flow generated from residential, commercial, and public users, and groundwater infiltration which includes other inflow that is not the result of precipitation.

The largest rainfall event during the flow monitoring period occurred from February 15-20, 2011. A smaller storm occurred between February 24-26, 2011. Rainfall events are classified based on recurrence interval and duration. The National Oceanic and Atmospheric Administration has developed atlas maps, based on long-term historical rainfall data, that provide classifications for various recurrence interval and duration rainfall events. As calculated by V&A and confirmed through review of the atlas maps, each of these storm events had a classification of less than a 2-year recurrence interval, 24-hour duration storm event.

ES.3.2 City of Millbrae Water Billing Information

The City operates its own water distribution system within the same area served by the sanitary sewer system. The City provided water billing records for the months from September 2010 to August 2011. These records contain the amount of water consumed at each address in the City, with no customer information provided.

The monthly water consumption data was matched with parcel Geographic Information System (GIS) data in order to spatially locate the water consumption. Large water consumption parcels were identified in order to determine if these parcels would generate large sewer flows as well, or if the water consumption was used for irrigation or industrial processes that would not be returned to the sanitary sewer. Existing sewer flows were then generated based on Return-to-Sewer ratios that were applied to the various land uses within the City.

ES.3.3 City of Millbrae 2010 Urban Water Management Plan

The City of Millbrae's 2010 Urban Water Management Plan, dated June 2011, projected wastewater flows for the City to the year 2035. These projections are based upon proposed redevelopment within the City, and accounting for the recent reductions in per capita water demand and wastewater generation identified in the plan. These values were used to confirm sewer flow estimates that were used for the buildout analysis in the hydraulic model.

ES.3.4 Dry Weather Flow Estimation

The initial average dry weather flow component was calculated using the following steps:

1. Calculate the winter water consumption for each parcel in the City based upon water billing records.

2. Calculate water consumption per dwelling unit for Single Family Residential parcels, and per acre for Multi-Family Residential parcels across the City.

3. Develop Return-to-Sewer ratios for each zoning in order to calculate the amount of wastewater returned to the sanitary sewer system for each unit of water consumed.

4. Use the ratios to calculate the estimated wastewater flow for each parcel in the City. Wastewater flows for residential parcels are calculated based upon the residential factors calculated above. Flow for non-residential parcels is calculated based upon the actual measured consumption for such parcels.
5. 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.

Build-out flows were calculated by assigning unit flow factors to build-out zonings, according to the following steps:

- Populate all vacant residential parcels,
- Add flow from the projects identified by City staff, and
- Increase flows generally across the City to account for general densification. Total build-outflows match flows predicted for 2035 in 2010 Urban Water Management Plan.

ES.4 HYDRAULIC MODEL DEVELOPMENT

The fully-dynamic computer-based hydraulic model of the City's wastewater collection system serves as a tool for assessing the flows and capacities of the City's major sewers, and for identifying solutions to identified capacity issues. The hydraulic model includes the City's main trunk sewers and associated facilities, and is a simplified representation of the City's total sewer system in its configuration and operation. The City's model also includes some smaller diameter sewers to assess anticipated potential capacity needs in the neighborhood collector sewers. This section summarizes the components of model development that are discussed further in Chapter 4 of this report.

The City's hydraulic model consists of approximately 13 miles of sewer pipeline ranging in diameter from 6-inches to 36-inches. The model includes all 8-inch diameter and larger trunk lines, and associated manholes and pump stations. Six-inch diameter pipelines were then added to the model as requested by City staff and as needed to provide network connectivity. The 13 miles of pipeline represent approximately 20 percent of the City's sanitary sewer system. In addition to the gravity mains described above, the hydraulic model contains approximately one mile of forcemains ranging from 6-inches to 14-inches.

The City's three collection system pump stations are included in the hydraulic model: Madrone, Hacienda, and Plaza Bay Pump Stations. Because the WPCP has limited hydraulic capacity that affects the operation of the collection system, the WPCP, including the new flow equalization facility is actively modeled in the hydraulic model.

ES.4.1 Dry Weather Flow Generation

Flow factors derived from the various sources described above were applied to land use to generate average base wastewater flows. The land use and unit flow factors are also described further in Chapter 2. Additional factors were then used to distribute average flows in a way that best represents the daily variation in sewer flows, or diurnal variation, for every land use. This process generated a peak dry weather flow for every basin.

West Yost adjusted or calibrated these calculated flows to closely match measured dry weather flow peaks and volumes from the City's 2010/11 flow monitoring program.

ES.4.2 Wet Weather Flow Generation

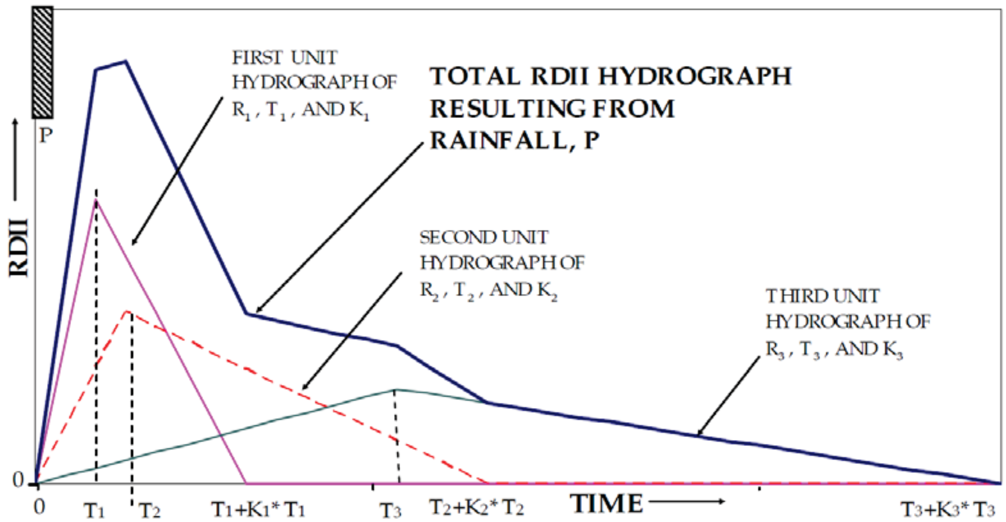
Wet weather flows components were then calculated and applied to the City's hydraulic model to replicate measured wet weather flow data. The key elements of wet weather flow generation in the model include Rainfall Dependent Infiltration and Inflow and Peak Wet Weather Flow (PWWF).

The City's model uses the "RTK" method to calculate wet weather inputs to the hydraulic model. The RTK method generates a series of three triangular hydrographs that represent short-term, medium-term, and long-term RDII response. The RTK parameters represent the percentage of rainfall that enters the sewer collection system (R), the time from the onset of the rainfall to the peak I&I (T), and a ratio that calculates the time of recession of I&I (K).

RTK parameters were adjusted by basin, beginning with the upstream basins, until measured flows during the calibration storm closely matched model-generated flows in peak, distribution and volume.

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

Figure ES-3. Components of RTK Hydrograph



ES.5 PLANNING CRITERIA

The planning criteria used to evaluate system capacity and size new replacement facilities are discussed further in Chapter 5. The criteria include generally accepted industry standard criteria, as reviewed and confirmed by the City. Planning criteria address items such as collection system capacity, gravity sewer slopes, and maximum depth of flow.

The design storm specified for the City is rainfall event with a 10-year recurrence interval and 24-hour duration (10-year, 24-hour storm). This design storm is defined in the Consent Decree as having a total depth of 3.14 inches as measured at the San Francisco International Airport.

As a guideline, existing pipelines were considered to require capacity improvements by 2016 if flow through the pipe causes a predicted SSO in the hydraulic model. Existing pump stations were considered capacity deficient if the station was unable to convey peak flows with the largest pump out of service.

New or replacement pipelines were sized to address the capacity deficiency. Also, under peak dry weather flow conditions, velocity above 2 feet per second was maintained where possible to facilitate self-cleaning. Under all conditions, maximum allowable velocity was limited to 10 feet per second.

ES.6 HYDRAULIC CAPACITY ANALYSIS RESULTS

The City's modeled collection system network was evaluated for its capacity to convey flows that are predicted to occur during the design storm event. The analysis is summarized in Chapter 6. The hydraulic model predicted peak hourly flow from the design storm of 17.0 mgd.

Analyses were conducted as follows:

- The system was evaluated for its ability to maintain water level below the manhole rim or ground elevation, as described in Chapter 5. Pump stations and pipelines with a capacity deficiency that triggered exceeded criteria were flagged.
- Projects were developed to address the capacity issues. Projects included upsizing pipes to meet the required planning criteria and/or introducing relief sewers to convey the excess capacity as needed to eliminate the predicted SSOs.

The resulting capacity improvement project recommendations are show in Table ES-1. All improvements are planned for completion by 2016. The recommendations captured in Table ES-1 and discussed in the CAR describe a combination of pipeline, pump station, and storage improvements to address SSOs that are predicted to result from the design storm event. The proposed combination of projects presents a solution that appears viable and practical, based on the information that was known as of the date of the CAR. These proposed projects are proposed alternatives that are subject to change and revision as the City moves forward with the implementation of the CAR. Additional information that is gained through preliminary design activities (permitting, easement acquisition, environmental documentation, etc.) and additional evaluation of the capacity of the City's system is expected to lead to changes in the final project descriptions, costs, and the implementation timeline, and may also result in changes to the types of projects implemented. The proposed projects have not been subject to the CEQA process. Also, the City's concurrent, ongoing efforts to reduce I&I will result in a reduced need for the planned capacity improvements. Therefore, the proposed capital improvement program is an evolving planning tool that will be refined throughout the term of the Consent Decree. Any changes to the proposed projects and program will continue to uphold the City's commitment to meet the SSO reduction requirements of the Consent Decree, and the City will update Baykeeper with any changes to the CIP that occur throughout the duration of the Consent Decree.

Table ES-1. Capacity Improvement Projects		
Project Name	**Description**	**Conceptual Cost, $M**
Wet Weather Storage at Corporate Yard	Construction of 0.9 MG of wet weather storage, with associated entry piping and exit pumping.	2.76
Madrone Pump Station Replacement and Upstream Conveyance System Improvements	Relocation of the Madrone Pump Station and associated force main.	7.26
Pipeline Replacements Near Capuchino High School	Upsizing of approximately 3,000 of 8-inch and 10-inch pipeline to 12-inch and 18-inch.	0.85
Pipeline Replacement at Aviador Avenue and East Millbrae Avenue	Replacement of 1,250 feet of 12-inch pipeline with 18-inch pipeline.	0.77
Pipeline Replacement in Murchison Drive	Replacement of 1,600 feet of 10-inch diameter pipeline with 15-inch diameter pipeline.	0.50
Pipeline Replacement Along Highline Canal Right-of-Way	Replacement of the parallel 18-inch and 12-inch pipelines along the canal with a single 36-inch diameter pipeline.	2.04
Pipeline Replacement in Anita Drive and Richmond Drive Near El Camino Real	Replacement of approximately 3,500 feet of 8-inch diameter and 10-inch diameter pipeline with 12-inch diameter pipeline.	0.89
Pipeline Replacement in El Camino Real	Replacement of approximately 3,500 feet of 10-inch diameter to 15-inch diameter pipelines with 18-inch and 24-inch pipelines.	3.13
	Total	**22.05**

ES.7 PIPELINE REHABILITATION AND REPLACEMENT

Chapter 7 supplements the capacity assessment with a proposed approach for near-term pipeline rehabilitation and replacement planning. Using this approach, the City has developed an initial list of rehabilitation projects for implementation in conjunction with capacity improvements.

The initial rehabilitation program includes proposed repair and CCTV inspection projects to be implemented in FY 2012/2013. This project list relied on available CCTV inspection results as a primary indicator of likelihood of failure. Where CCTV data were not available, the assessment was based on age, O&M history and geologic setting. As the City completes additional CCTV inspection of the system, the information gained is likely to require adjustments to the likelihood of failure assessment and associated project priorities. The initial list of projects is presented in Table ES-2.

Project Name	Description	Conceptual Cost
Priority Line Segment Replacements	• Six line segments • 6-inch diameter • Combined length of 1,425 lf • Structural PACP score of 53XX • Some pipes have chronic O&M issues	$265,000
CCTV 11 Priority Pipe Segments	• Combined length of 1,807 lf • High potential for defects based on O&M needs and age • Many segments are also associated with needed capacity improvements	$3,650
CCTV Hillcrest and Hawthorne Neighborhood Pipelines	• Approximate length of 20,000 lf • These neighborhoods experienced a high number of lower lateral SSOs in 2011	$40,000
Hillcrest and Hawthorne Area and Other Pipeline Replacements	• This project is included as a placeholder to address immediate pipeline rehabilitation and replacement needs that are identified through the planned CCTV inspections	$360,000

Table ES-2. Pipeline Rehabilitation and Replacement and CCTV Inspection Projects Planned for FY2012-2013

The primary objective of evaluating gravity pipeline risk was to identify pipe segments that have the highest potential to cause an SSO, regardless of consequence. The City utilized a risk-based prioritization tool, named the Risk Management Model (RMM), to complete this analysis. The RMM uses a numerical algorithm to evaluate initial risk in the context of Likelihood of Failure. The RMM also has the capability to refine priorities using parameters related to Consequence of Failure.

ES.8 CAPITAL IMPROVEMENT PROGRAM

The CIP was developed to primarily address the City's need to reduce SSOs caused by capacity restrictions during the design storm, and also to address immediate collection system condition needs. The following criteria were used to prioritize the various projects and develop a timeline for implementation.

1. Projects to Eliminate SSOs. The CIP prioritizes and schedules completion of projects that eliminate capacity-related SSOs from the 10-year, 24-hour design storm. Projects were ordered such that downstream capacity improvements are completed first.
2. Projects to Address Known Maintenance Issues. The CIP prioritizes and schedules completion of pipeline replacements to address pipe segments with substantial structural defects, as determined through the CCTV inspection program. The CIP also prioritizes CCTV inspection of areas with anticipated issues as determined through average pipe age and known maintenance issues.

3. Distribution of Capital Costs. The City has established an implementation schedule for the recommended projects that meets the requirements of the Consent Decree between the City and San Francisco Baykeeper. The Consent Decree requires all projects needed to address capacity issues from the design storm event to be completed by 2016.

Table ES-3 presents the proposed CIP, which begins implementation in Fiscal Year 2012/13 and extends into Fiscal Year 2016/17. The CIP has been developed to address all capacity requirements of the 10-year, 24-hour design storm with conservative assumption that there will be no short-term reduction in I&I. Concurrently, the City will implement rehabilitation and replacement projects that address known structural defects, and should therefore contribute to a reduction in I&I. This CIP is intended to be an evolving document that is adjusted as needed to address future conditions that are identified as more data is collected through CCTV inspection and maintenance activities.

Additional information on the individual projects, including detailed cost estimates, can be found in Chapters 6 and 7 of this report, including associated appendices. The total estimated CIP cost is $21.4 million, to be implemented by 2016.

Table ES-3. City of Millbrae Capacity Assurance Report Capital Improvement Project Implementation Plan (Note 1)

R&R Project #	Project Name	Estimated Cost	2012/2013	2013/14	2014/15	2015/16	2016/17
1	Rehabilitate Six Pipe Segments	$265,000	265,000				
2	Conduct CCTV Inspection of Eleven Priority Pipe Segments	$3,650	3,650				
3	Conduct CCTV Inspection of Hawthorne and Hillcrest Neighborhoods	$40,000	40,000				
4	Additional Pipeline Rehabilitation Projects (Based on CCTV Results)	$360,000	360,000				
5	Future Pipeline Rehabilitation and Replacements	$2,525,000	0	575,000	650,000	650,000	650,000
	Subtotal R&R	$3,193,650	668,650	575,000	650,000	650,000	650,000
Capacity Project #	Project Name	Estimated Cost	2012/2013	2013/14	2014/15	2015/16	2016/17
1	Wet Weather Storage at Corporate Yard	$2,760,000	690,000	2,070,000	0		
2	Madrone Pump Station Replacement	$7,256,000			3,628,000	3,628,000	
3	Pipeline Replacements Near Capuchino High School	$850,000				425,000	425,000
4	Pipeline Replacements at Aviador Avenue and E. Millbrae Avenue	$772,000				386,000	386,000
5	Pipeline Replacements in Murchison Drive	$501,000				501,000	
6	Pipeline Replacements Along Highline Canal ROW	$2,046,000		511,500	1,534,500		
7	Pipeline Replacements in Anita Drive and Richmond Drive at El Camino Real	$890,000				445,000	445,000
8	Pipeline Replacements in El Camino Real	$3,129,000	782,250	2,346,750			
	Subtotal Capacity	$18,204,000	1,472,250	4,928,250	5,162,500	5,385,000	1,256,000
	Total CIP	**$21,397,650**	**2,140,900**	**5,503,250**	**5,812,500**	**6,035,000**	**1,906,000**

Note 1: Implementation schedule beginning in 2013/14 and beyond will be revised routinely based on new system information, and as needed to accommodate unexpected infrastructure repair projects.

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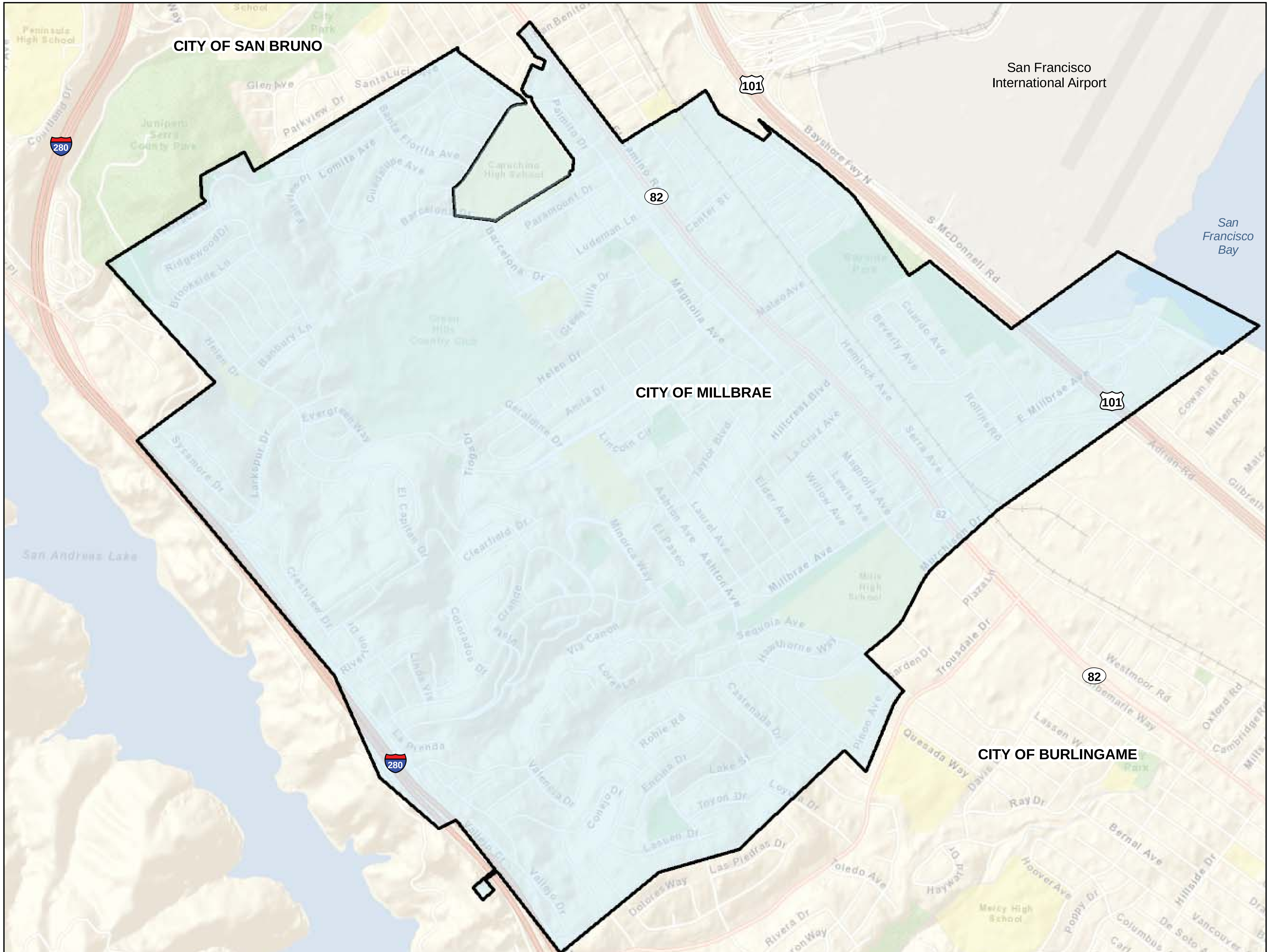

FIGURE ES-1

City of Millbrae
Capacity Assurance Report

WASTEWATER SERVICE AREA

N

0 625 1,250

Scale in Feet

1 inch = 1,250 feet

LEGEND

City of Millbrae Boundary

Wastewater Service Area

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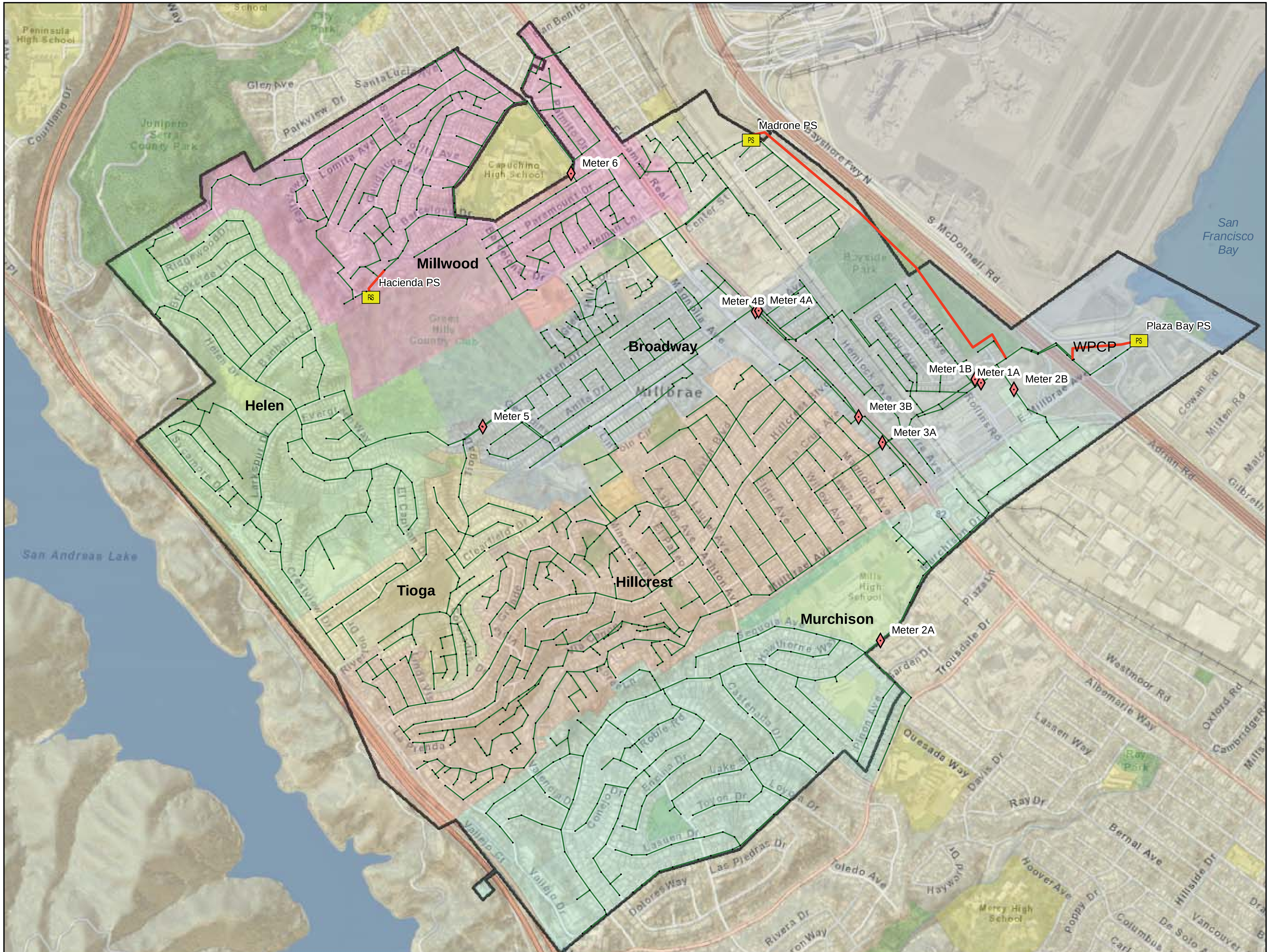
FIGURE ES-2

City of Millbrae
Capacity Assurance Report

FLOW MONITORING LOCATIONS AND SEWER BASINS



N

0 625 1,250
Scale in Feet

1 inch = 1,250 feet

LEGEND

- Manhole
- PS Pump Station
- 2010/11 Flow Meter Location Used in 2011/12
- Force Main
- Gravity Main

Flow Meter Basins

- Broadway
- Helen
- Hillcrest
- Millwood
- Murchison
- Tioga
- City of Millbrae Boundary

SEAL OF THE CITY OF MILLBRAE CALIFORNIA

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 1
Introduction

This introductory chapter provides background information on the scope and objectives of the City of Millbrae Capacity Assurance Report (CAR).

1.1 BACKGROUND AND PROJECT OBJECTIVES

Effective November 15, 2011, the City of Millbrae (City) entered into a Consent Decree with San Francisco Baykeeper (Baykeeper). One component of the Consent Decree specified that the City would complete a CAR for the City's sanitary sewer collection system by June 30, 2012. Through its counsel of Hanson Bridgett, LLP, the City retained West Yost Associates (West Yost) in August 2011 to complete the CAR. The CAR has the following broad components:

- Development of a computerized hydraulic model that is calibrated to dry and wet weather conditions for capacity analysis of a 10-year, 24-hour design storm in the collection system;
- Evaluation of pipeline and pump station capacity;
- Development of a pipeline rehabilitation and replacement (R&R) program that reduces inflow and infiltration (I&I) into the collection system;
- Development of a prioritized capital improvement program (CIP) that strategically replaces and repairs assets in order to provide sufficient collection system capacity, to improve collection system condition, and ultimately to reduce the number of SSOs experienced in the City's collection system.

This chapter is the introductory chapter for the CAR and serves as a roadmap to the document. The City will continue to review the recommendations in the CAR and study the feasibility of the recommendations in the CAR. There will be an iterative process of refining the CAR as the feasibility analysis continues. The City reserves the right to adopt alternative capital improvements to address system capacity.

1.2 REPORT ORGANIZATION

The CAR 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 8 chapters and appendices.

1.2.1 Executive Summary

The Executive Summary provides a comprehensive overview of the CAR contents and results while summarizing the key aspects of each chapter.

1.2.2 Chapter 1 – Introduction

This introductory chapter provides background information on the scope and objectives of the CAR, and also presents its contents and organization.

1.2.3 Chapter 2 – Existing System

This chapter describes the City's existing service area, presents land use and zoning within the City, and describes existing facilities including pump stations, pipelines, treatment, storage, and disposal.

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. The chapter also presents West Yost's analysis of contributions to system-wide inflow and infiltration that followed completion of the City's 2010/11 Flow Monitoring Program, completed by V&A Consulting Engineers (V&A).

1.2.5 Chapter 4 – Hydraulic Model Development

This chapter documents the tasks required to build and calibrate the Innovyze® InfoSWMM hydraulic model. The hydraulic model is the primary analytical tool used for determining the flows and capacities of the City's collection system, and to identify required capacity improvements, including pipeline, pump station, and storage requirements.

1.2.6 Chapter 5 – Planning Criteria

This chapter documents the planning criteria used to calculate existing and future flows, and to assess hydraulic capacity in the collection system. These criteria are based on industry standards in conjunction with criteria created specifically to satisfy the Consent Decree.

1.2.7 Chapter 6 – Capacity Analysis

This chapter presents the results of the existing and buildout system hydraulic capacity analyses of the City'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.

1.2.8 Chapter 7 – Pipeline Repair and Replacement Program

This chapter presents the City's potential gravity sewer system repair, renewal, and replacement needs based on results from closed circuit television (CCTV) inspections, and system knowledge provided by City staff. Similar to the capacity discussion in Chapter 6, the chapter includes conceptual cost estimates.

1.2.9 Chapter 8 – Recommended Capital Improvement Projects

This chapter consolidates recommendations presented in Chapters 6 and 7, and compiles the projects into a prioritized CIP that addresses downstream improvements first, considers rehabilitation and replacement in conjunction with capacity improvements, and distributes cost across the CIP timeframe, which extends into calendar year 2016. This chapter includes descriptive maps that also summarize findings and planning information for each project.

1.2.10 Appendices

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

- Appendix A – Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study (August 2011)
- Appendix B – Sample Flow Hydrographs
- Appendix C - Flow Meter Diurnal Curves
- Appendix D - Dry Weather Flow Calibration Hydrographs
- Appendix E - Wet Weather Flow Calibration Hydrographs
- Appendix F – Hydraulic Profiles of Capacity Limitations
- Appendix G – Capacity Project Cost Estimates
- Appendix H – Detailed Risk Management Model Reports

1.3 ACRONYMS AND ABBREVIATIONS

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

2010 UWMP	City of Millbrae 2010 Urban Water Management Plan
ABAG	Association of Bay Area Governments
ADWF	Average Dry Weather Flow
AIMS	Asset Information Management System
BI	Base Infiltration
BWF	Base Wastewater Flow
CalTrans	California Department of Transportation
CAR	Capacity Assurance Report
CCTV	Closed Circuit Television
CIP	Capital Improvement Program
CIPP	Cured in Place Pipe
City	City of Millbrae
CIWQS	California Integrated Water Quality System
County	County of San Mateo
DIP	Ductile Iron Pipe
DWF	Dry Weather Flow
ENR CCI	Engineering News Record Construction Cost Index
FAA	Federal Aviation Administration

fps	Feet Per Second
General Plan	City of Millbrae General Plan 1998-2015
GIS	Geographical Information System
gpd	Gallons Per Day
gpm	Gallons Per Minute
GWI	Groundwater Infiltration
HDD	Horizontal Direction Drilling
HDPE	High-Density Polyethelyne
HGL	Hydraulic Grade Line
I&I	Inflow and Infiltration (or Rainfall-Dependent Inflow and Infiltration)
ID	Identification Numbers
JPA	Joint Powers Agreement
JUFM	Joint Use Force Main
LOS goals	Level of Service Goals
MGD	Million Gallons Per Day
MS Access	Microsoft® Access
MSL	Mean Sea Level
NASSCO	National Association of Sewer Service Companies
NOAA	National Oceanic and Atmospheric Administration
PACP	Pipeline Assessment and Certification Program
PDWF	Peak Dry Weather Flow
PWWF	Peak Wet Weather Flow
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
RDII	Rainfall-Dependent Inflow and Infiltration
Report	Capacity Assurance Report
RMM	Risk Management Model
RTS	Return-to-Sewer
SCS	Soil Conservation Service (now Natural Resource Conservation Service)
SFPUC	San Francisco Public Utilities Commission
SSO	Sanitary Sewer Overflow
SSOs	Sewer System Overflows
SUH	Synthetic Unit Hydrograph

SWRCB	State Water Resources Control Board
V&A	V&A Consulting Engineers
VCP	Vitrified Clay Pipe
WERF	Water Environment Research Foundation
West Yost	West Yost Associates
WPCP	Water Pollution Control Plant
WWF	Wet Weather Flow

CHAPTER 2
Existing Wastewater System

The purpose of this chapter is to describe the City's existing 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 City's existing wastewater collection system:

- Existing Service Area,
- Population Served and Land Use Characteristics, and
- Existing Collection System Facilities.

2.1 EXISTING SERVICE AREA

The City of Millbrae encompasses 2,100 acres, or approximately 3.2 square miles, on the San Francisco Peninsula. The City is situated in San Mateo County (County), approximately 15 miles south of downtown San Francisco. It is bounded to the north by the City of San Bruno, to the west by the California Fish and Game Refuge that includes San Andreas Lake and Reservoir, to the south by the City of Burlingame, and to the east by San Francisco Bay and San Francisco International Airport.

The existing wastewater collection system service area includes all areas within the City's limits, as well as Capuchino High School in the City of San Bruno. The service area includes approximately 6,550 service connections and 21,500 residents, as of the 2010 Census. Elevations within the service area range from 620 feet above Mean Sea Level (MSL) on the City's western boundary to 5 feet below MSL near San Francisco Bay. The City's service area is shown on Figure 2-1.

The City's wastewater flows generally east/northeast out of the hills and into the flat regions of the City. When flows reach El Camino Real, they are collected and transported to the Water Pollution Control Plant (WPCP) in the southeast corner of the City. The WPCP provides primary-secondary treatment and disinfection to the flow, which is then dechlorinated and pumped into the joint use force main (JUFM), which is described further below, for disposal through a deep water outfall to San Francisco Bay.

2.2 POPULATION SERVED AND LAND USE CHARACTERISTICS

This section describes the current and build-out population projections, and associated land use as outlined in the General Plan for the City's service area.

2.2.1 Existing Population and Land Use

The City's population is 21,500, as reported in the 2010 Census. This population resides within 7,994 households throughout the City, for an average of 2.69 persons per household.

Land use within the City is identified and controlled by the *City of Millbrae General Plan, 1998-2015* (General Plan). The overriding goal of the Land Use Element of the General Plan is to maintain the quality of life in Millbrae, achieving "build-out" in a manner that addresses long-term community needs while preserving the community's character and dignity.

Zoning designations describe how a particular parcel within the City is currently utilized. The City tracks zoning in a Geographical Information System (GIS) database. The City's acreage is summarized by zoning designation in Table 2-1. City zoning designations are shown on Figure 2-2. Although Capuchino High School is not within the City limits and is not covered by the General Plan, it is treated as are the other high schools for the purposes of this study. Planning level wastewater flow generation rates will be developed for each of these zoning designations as part of the hydraulic modeling process.

Table 2-1. City of Millbrae Zoning Summary

Zoning Code	Zoning Description	Area, acre	Area, %
R1LD	Residential 1 Low Density	145	6.88%
R1	Residential 1 Low Density	991	47.17%
R-1-O	Residential 1 Office	0.1	0.00%
R2	Residential 2	68	3.22%
R3	Residential 3	60	2.87%
RG	Residential Growth	15	0.72%
RM	Residential Senior Center	2	0.11%
C1	Commercial 1	39	1.86%
C1H	Commercial 1 High Density	10	0.50%
C2	Commercial 2	1	0.05%
DIA	Downtown Improvement Area	8	0.39%
PD	Planned Development	43	2.02%
I	Industrial	69	3.27%
O	Open Space	171	8.14%
U	Utility	8	0.36%
N/A	Undesignated	24	1.16%
CHS	Capuchino High School (City of San Bruno)	32	1.52%
ROW	Right of Way	415	19.76%
Total		2,100	100.00%

2.2.2 Build-out Population and Land Use

Whereas zoning designation describes the current use of a parcel of land, the land use designation describes the ultimate use and extent to which a parcel may be developed. As the City matures and essentially built-out, there will be relatively little change between existing use of land and ultimate use. Population and land use projections indicate that there is predicted to be a small amount of specific development and general infill between now and the City's planning horizon of 2035. The *City of Millbrae 2010 Urban Water Management Plan* (2010 UWMP) shows population projections for the City through 2035. These projections are based upon Association of Bay Area Governments (ABAG) 2009 Projections. These population projections, shown in Table 2-2, are used for the Capacity Assurance Report (Report) as well. As shown, population within the City is projected to increase from 21,532 to 26,700 by 2035, an increase of 24 percent.

Table 2-2. Current and Projected Population in the City of Millbrae[a]

2010	2015	2020	2025	2030	2035
21,532	22,600	23,600	24,700	25,700	26,700

(a) Taken from City of Millbrae 2010 Urban Water Management Plan.

Built-out land use is described by the General Plan Land Use designation, which the City tracks in a GIS database. General Plan Land Use is summarized by acreage in Table 2-3, and is shown on Figure 2-3. Parcels within the Millbrae Station Area Planned Development land use have development that is controlled by specific plan documents rather than by the General Plan.

Table 2-3. City of Millbrae Land Use Summary

Land Use Code	Land Use Description	Area, acre	Area, %
VLDR	Very Low Density Residential	37	2.20%
LDR	Low Density Residential	1,051	62.35%
MDR	Medium Density Residential	67	3.99%
HDR	High Density Residential	67	3.97%
GC	General Commercial	93	5.55%
MSASP	Millbrae Station Area Specific Plan	85	5.05%
PF	Public Facility	69	4.11%
P & O	Park and Open Space	159	9.46%
N/A	Undesignated	24	1.45%
CHS	Capuchino High School (City of San Bruno)	32	1.90%
Total		1,685	100.00%

The following individual developments were identified by the City as recently completed:

- The Belamor Millbrae Paradise development, and residential/commercial complex located at 151 El Camino Real
- The Park Broadway development, a residential/commercial complex located at 1388 Broadway
- A luxury condominium complex located at 88 S. Broadway
- A new retail site at Wilson Plaza on Adrian Road between Highway 101 and Millbrae Avenue

The development at 151 El Camino Real is approximately 60 percent occupied, and thus retains some potential for increased wastewater flow as the remaining space is filled. The other developments listed above are considered complete, with all wastewater flow accounted for in existing conditions.

Two individual developments have been identified that will be adding density and future wastewater flow to the City. The first of these is a 50-unit development that will take place at 120 El Camino Real. The second is the redevelopment of the Safeway grocery store at 525 El Camino Real. The floor space, water usage, and wastewater generation will be increased at the Safeway location. Wastewater flow will be increased under future conditions in the hydraulic model in order to account for this known redevelopment.

Finally, a large parcel within the Millbrae Station Area Planned Development land use, currently a parking lot, retains the potential for redevelopment. There is currently no project planned for this location that would generate wastewater flow. Other than the individual developments and potential developments discussed in this Chapter, changes in population, density, and wastewater flow within the City will be assumed to be general infill.

2.3 EXISTING COLLECTION SYSTEM FACILITIES

This section describes facilities that are owned, operated, and maintained by the City. Existing facility information was derived from the City's GIS database, which is maintained as part of the City's Asset Information Management System (AIMS). Figure 2-4 shows the City's wastewater collection system 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 8-inches and larger, with some pipelines 6-inches in diameter and smaller as needed to complete model connectivity, and associated pump stations.

2.3.1 Pipeline Characteristics

The City owns 55 miles of gravity sewer pipe as reported in the State Water Resources Control Board (SWRCB) California Integrated Water Quality System (CIWQS) database. The City also owns and maintains 55 miles of publicly owned sewer laterals. The City's gravity mains range in diameter from 6 to 36 inches in diameter, as shown on Figure 2-5. The percentage of each diameter throughout the system is summarized in Table 2-4. As the table shows, 6-inch diameter gravity mains predominate in the City's collection system.

Table 2-4. Percentage of Gravity Mains by Diameter	
Gravity Main Diameter	Percent of System
5” [a]	<1%
6”	84%
8”	6%
10”	4%
12”	2%
14”	<1%
15”	1%
16”	1%
18”	1%
24”	<1%
33”	<1%
36”	<1%
Unknown	<1%

(a) For the purposes of this Master Plan, 5-inch pipe diameter describes 6-inch pipe that has been repaired through lining. The Master Plan references these pipes as having 5-inch diameter to be consistent with the City’s database.

Currently, the majority of gravity mains in the City’s collection system are vitrified clay (VCP). Polyvinyl chloride (PVC) and high-density polyethylene (HDPE) together comprise less than 10 percent of the collection system. However, because these two materials are used for recently installed and replaced gravity mains, their percentage of the system will increase with time. Table 2-5 summarizes the City’s system by gravity main material. The location of these materials in the collection system can be seen on Figure 2-6.

Table 2-5. Percentage of Gravity Mains by Material Type	
Gravity Main Material	Percent of System
Vitrified Clay Pipe	84%
Unknown Material	9%
High-Density Polyethylene	4%
Polyvinyl Chloride	2%
Cast Iron Pipe	<1%
Reinforced Concrete Pipe	<1%
Ductile Iron Pipe (DIP)	<1%
Plastic Liner	<1%
Other Liner	<1%
Transite	<1%

2.3.2 Pump Station Characteristics

The City owns and operates three pump stations and associated force mains, ranging in firm capacity (*i.e.*, with the largest pump out of service) from 300 to 1,425 gallons per minute (gpm). Table 2-6 lists the City's pump stations and summarizes operating characteristics.

Table 2-6. Wastewater Collection System Pump Station Summary

Pump Station Name	Number of Pumps	Pump Type	Year Built	Firm Capacity, gpm
Madrone Pump Station	3	Dry Pit	1980	1,425[a]
Hacienda Lift Station	1	Dry Pit	1970	300[b]
Plaza Bay Pump Station	2	Submersible	1970	1,000

(a) This value is taken from pump tests as reported by City staff.
(b) This value is the reported capacity of the single pump, and as such is not a true firm capacity.

2.3.3 City of Millbrae Water Pollution Control Plant

The WPCP is located on the northeast corner of US Highway 101 and Millbrae Avenue. The main units of the plant were constructed in 1952, with improvements for secondary treatment added in 1967. The City recently completed the WPCP Renovations Project, the goal of which was to increase the reliability of the plant. This project included construction of 1.2 million gallons of flow equalization, construction of a new headworks and influent pump station, and modification of the primary sedimentation, aeration system, solids handling, yard piping, and stand-by power systems. Additionally, a new Operations Center was constructed as part of the project. The dry weather capacity of the WPCP is 3.0 million gallons per day (mgd), and the wet weather capacity is 9.0 mgd.

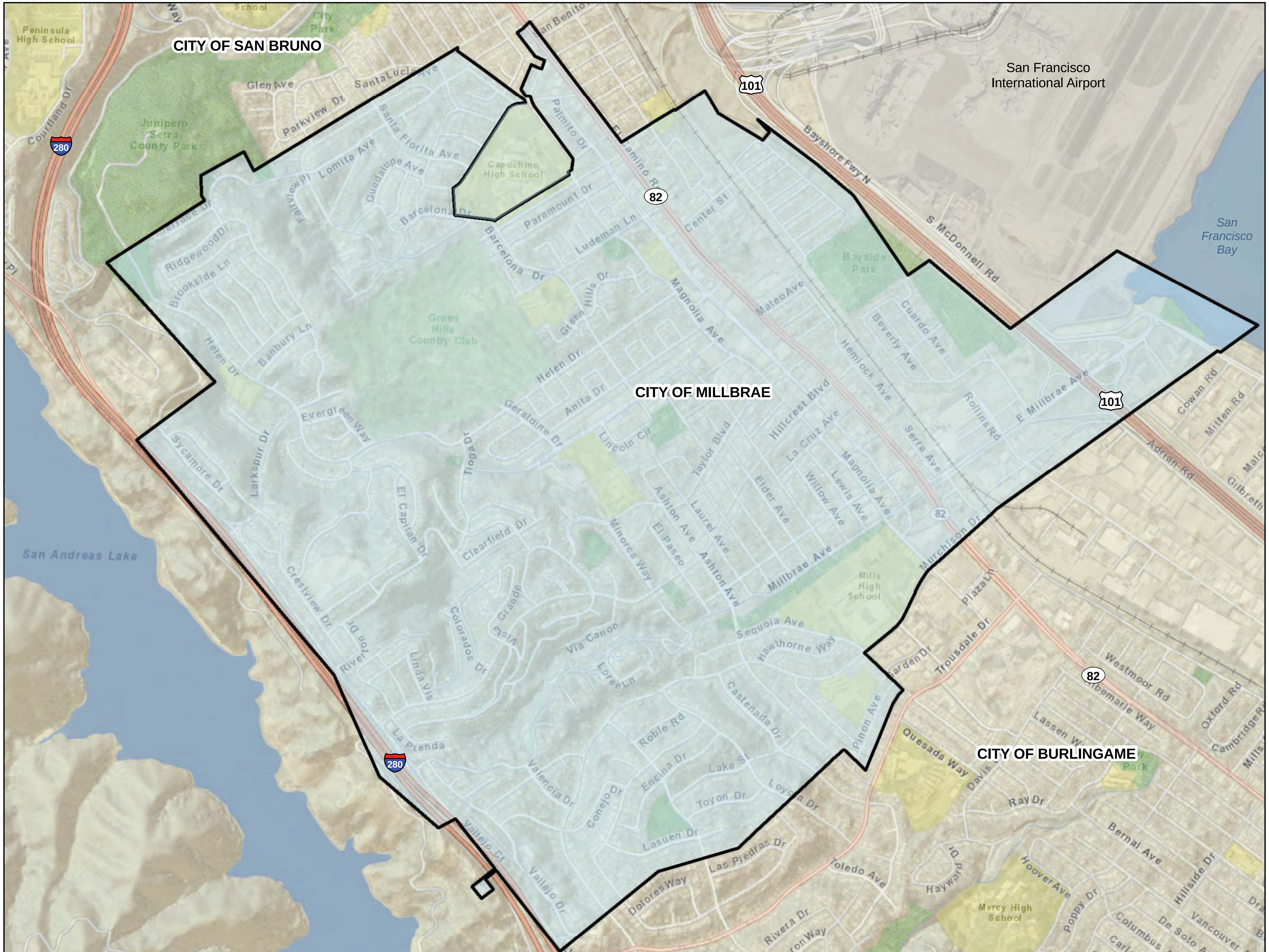
Effluent from the WPCP is discharged into the JUFM, through which the effluent is pumped to the South San Francisco Wastewater Treatment Plan for ultimate disposal through the deep water outfall at Oyster Point in San Francisco Bay. The JUFM is administered under a Joint Powers Agreement (JPA) entered in 1973 by the City of Burlingame, the City of San Bruno, the City of South San Francisco, the City and County of San Francisco (San Francisco International Airport), and the City. Under the JPA, the City has capacity rights to 9.0 mgd in the JUFM and outfall.

W:\Clients\465 Hanson Bridgett\06-11-02 Millbrae Capacity Assurance Report\GIS\Figures\Ch2\Fig 2-1_Service_Area.mxd 11/28/2011

FIGURE 2-1

City of Millbrae
Capacity Assurance Report

WASTEWATER SERVICE AREA



N

0 625 1,250
Scale in Feet

LEGEND

- City of Millbrae Boundary
- Wastewater Service Area

W:\Clients\465 Hanson Bridgett\06-11-02 Millbrae Capacity Assurance Report\GIS\Figures\Ch2\Fig 2-2_Zoning.mxd 11/28/2011

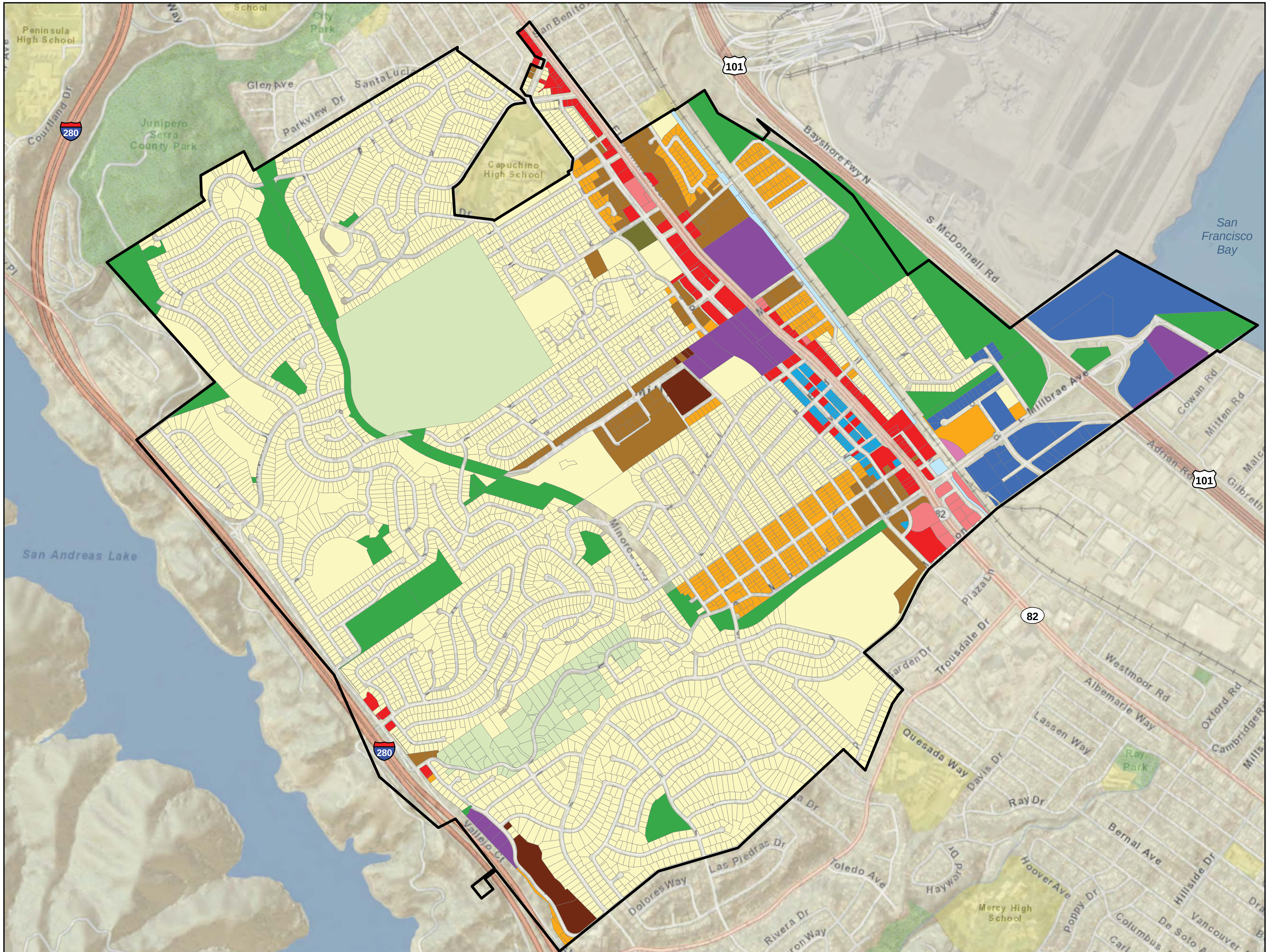


FIGURE 2-2

City of Millbrae Capacity Assurance Report

CITY OF MILLBRAE ZONING

N

0 625 1,250

Scale in Feet

LEGEND

City of Millbrae Boundary

Zoning Designation

- Residential 1 Low Density
- Residential 1
- Residential 1 / Office
- Residential 2
- Residential 3
- Residential Growth
- Residential Senior Center
- Commercial 1
- Commercial 1 Hig Density
- Commercial 2
- Downtown Improvement Area
- Planned Development
- Industrial
- Open Space
- Utility

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Consulting Engineers

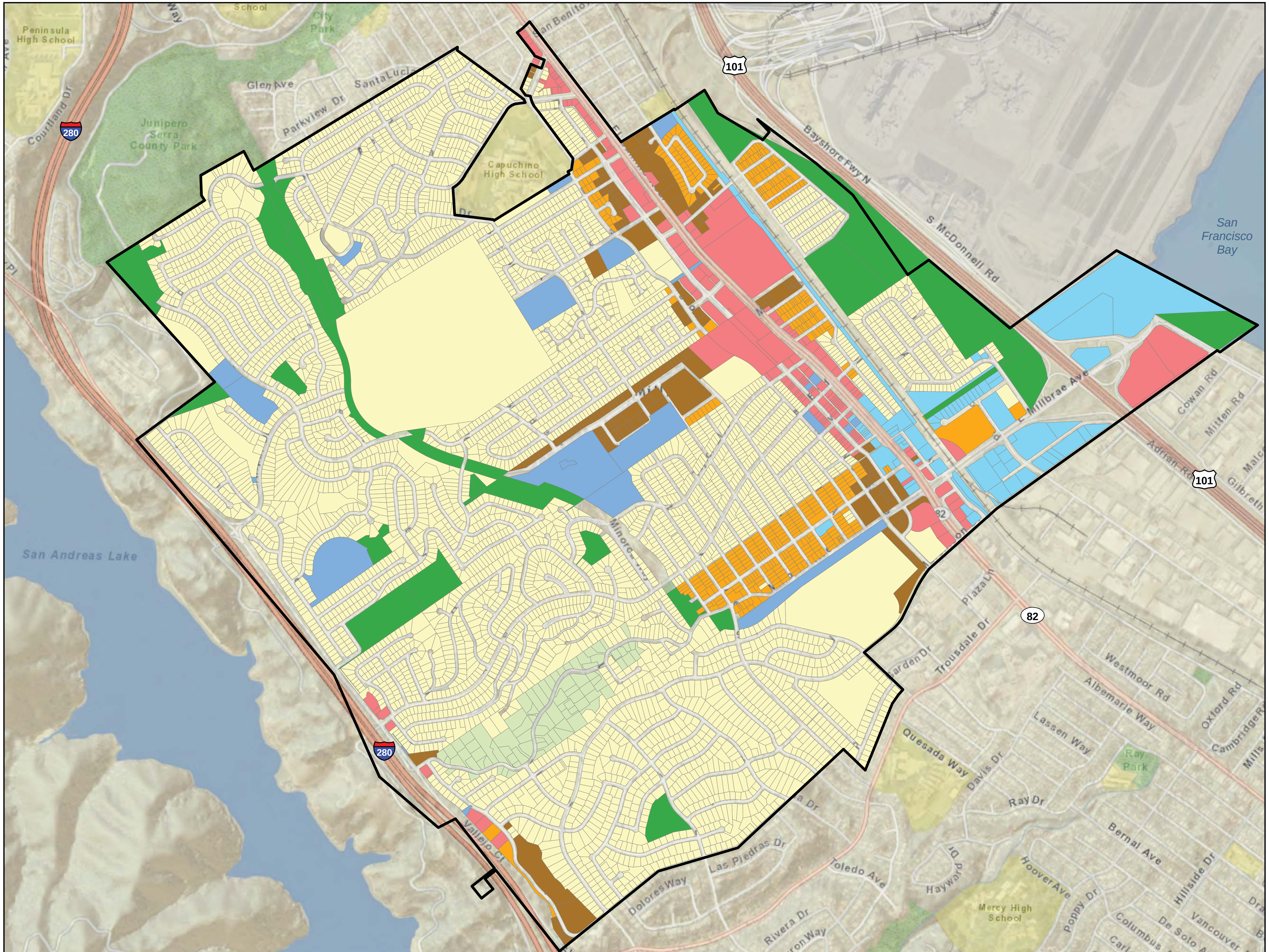

FIGURE 2-3

City of Millbrae Capacity Assurance Report

CITY OF MILLBRAE GENERAL PLAN LAND USE

N

0 625 1,250

Scale in Feet

LEGEND

City of Millbrae Boundary

General Plan Land Use

- Very Low Density Residential
- Low Density Residential
- Medium Density Residential
- High Density Residential
- General Commercial
- Millbrae Station Area Specific Plan
- Public Facility
- Park and Open Space

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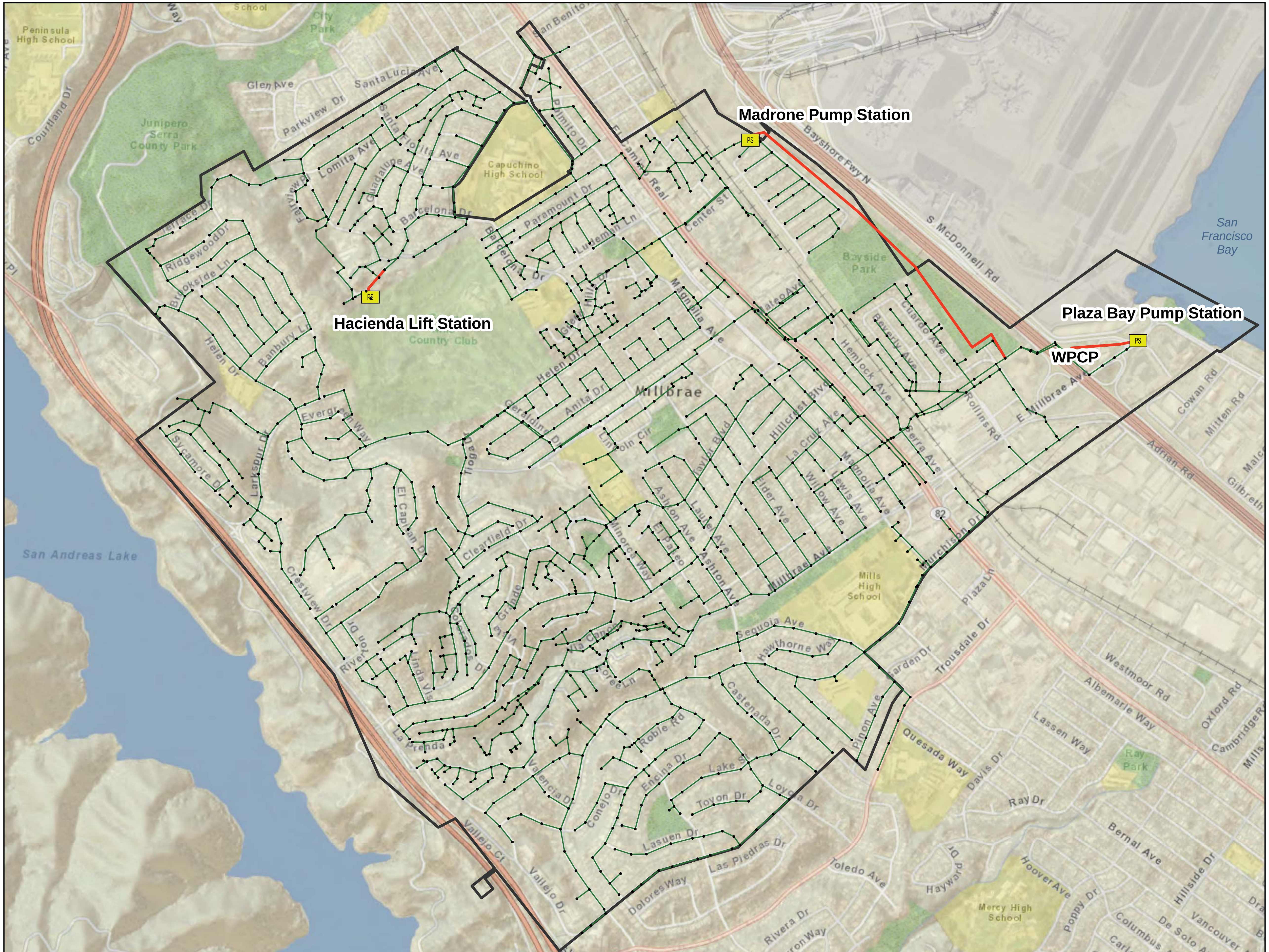

FIGURE 2-4

City of Millbrae Capacity Assurance Report

WASTEWATER COLLECTION SYSTEM FACILITIES

N

0 625 1,250

Scale in Feet

1 inch = 1,250 feet

LEGEND

- Manholes
- PS Pump Stations
- Force Mains
- Gravity Mains
- City of Millbrae Boundary

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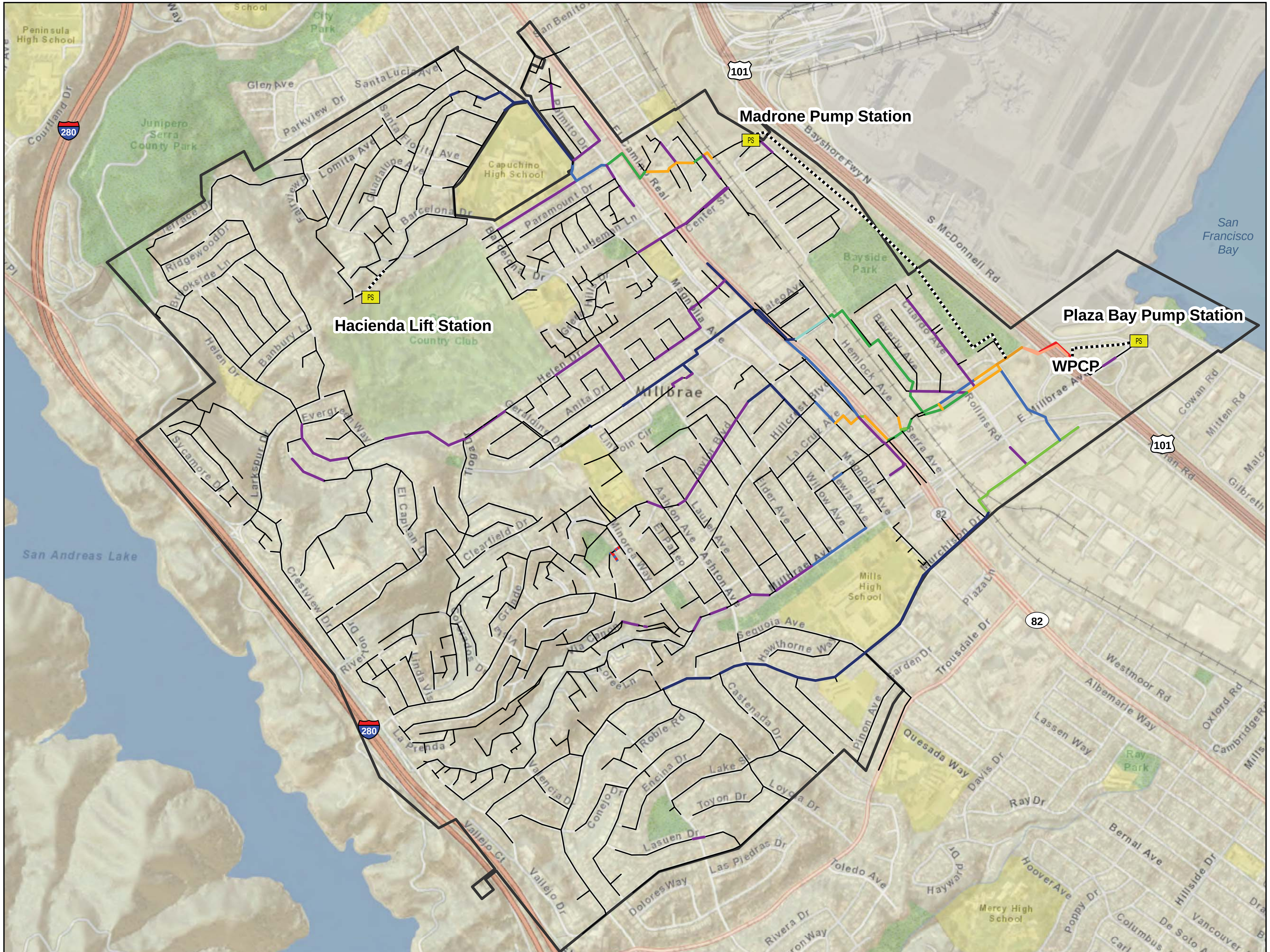

FIGURE 2-5

City of Millbrae Capacity Assurance Report

GRAVITY MAINS BY DIAMETER

N

0 625 1,250

Scale in Feet

LEGEND

- City of Millbrae Boundary
- PS Pump Station
- Force Main

Gravity Main Diameter

- 5"
- 6"
- 8"
- 10"
- 12"
- 14"
- 15"
- 16"
- 18"
- 24"
- 33"
- 36"
- Unknown

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W:\Clients\465 Hanson Bridgett\06-11-02 Millbrae Capacity Assurance Report\GIS\Figures\Ch2\Fig 2-6_Syst_by_Material.mxd 12/7/2011

FIGURE 2-6

City of Millbrae
Capacity Assurance Report

GRAVITY MAINS BY MATERIAL

N

0 625 1,250

Scale in Feet

LEGEND

- City of Millbrae Boundary
- PS Pump Station
- Force Main

Gravity Main Material

- Unknown
- Vitrified Clay Pipe (VCP)
- High Density Polyethylene (HDPE)
- Poly Vinyl Chloride (PVC)
- Reinforced Concrete Pipe (RCP)
- Ductile Iron Pipe (DIP)
- Cast Iron Pipe (CIP)
- Plastic Liner
- Other Liner
- Transite

WEST YOST ASSOCIATES

Consulting Engineers

CHAPTER 3
System Flows

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

- Sources of Wastewater Flow Data, and
- Calculation of Dry Weather Flows.

3.1 SOURCES OF WASTEWATER FLOW DATA

The main sources of data used to estimate wastewater flows for the City's hydraulic model were flow monitoring data, water billing data, and the 2010 UWMP.

3.1.1 Wastewater Flow Monitoring Data

In 2011, the City completed a system-wide flow monitoring program. The program was conducted by V&A Consulting Engineers (V&A) from February 2011 to March 2011. Data and results are presented in the V&A report titled, *Sanitary Sewer Flow Monitoring and Inflow / Infiltration Study (August 2011).* This report is referenced in this chapter as the Flow Monitoring Report, and is included in Appendix A.

3.1.1.1 Data Collection

The flow monitoring program included ten (10) gravity meters and two (2) rain gauges. The ten meters were located in manholes that delineated the collection system into eight (8) basins that capture the majority of the City's wastewater flow. Figure 3-1 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 inflow and infiltration (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 3-2 presents a schematic illustrating the direction of flow and interconnection between basins. Table 3-1 lists the flowmeter that captures flow exiting each sewer basin, and the Manhole ID defining the location of each meter.

Figure 3-2. Flow Monitoring Basin Schematic

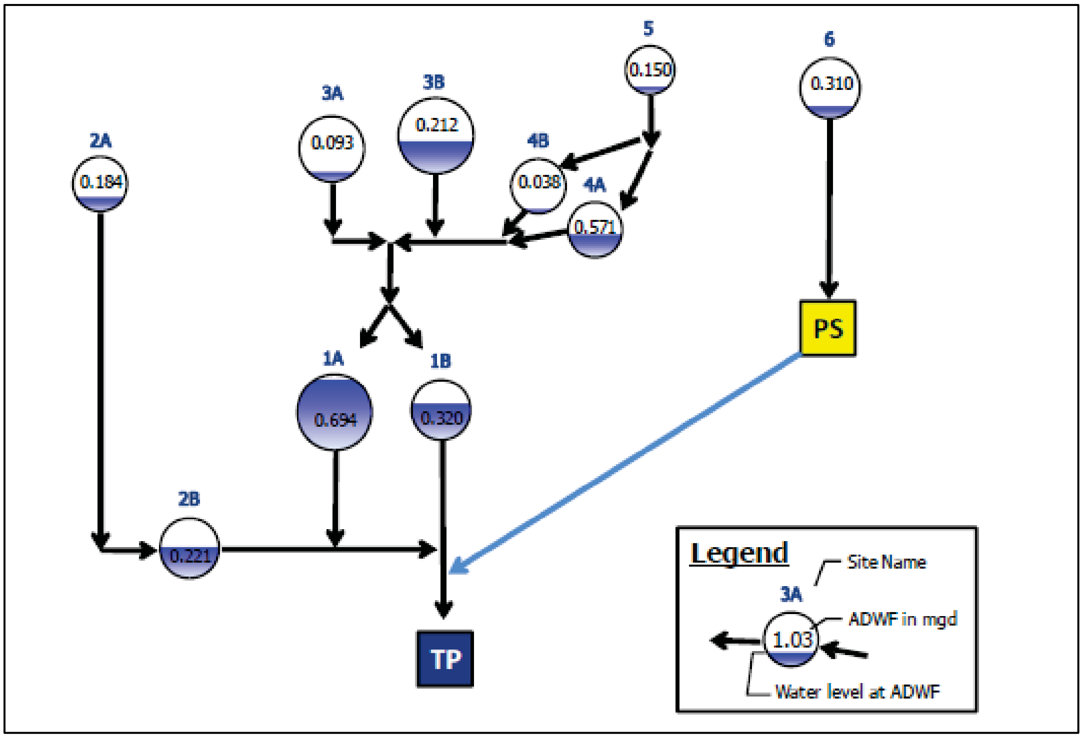


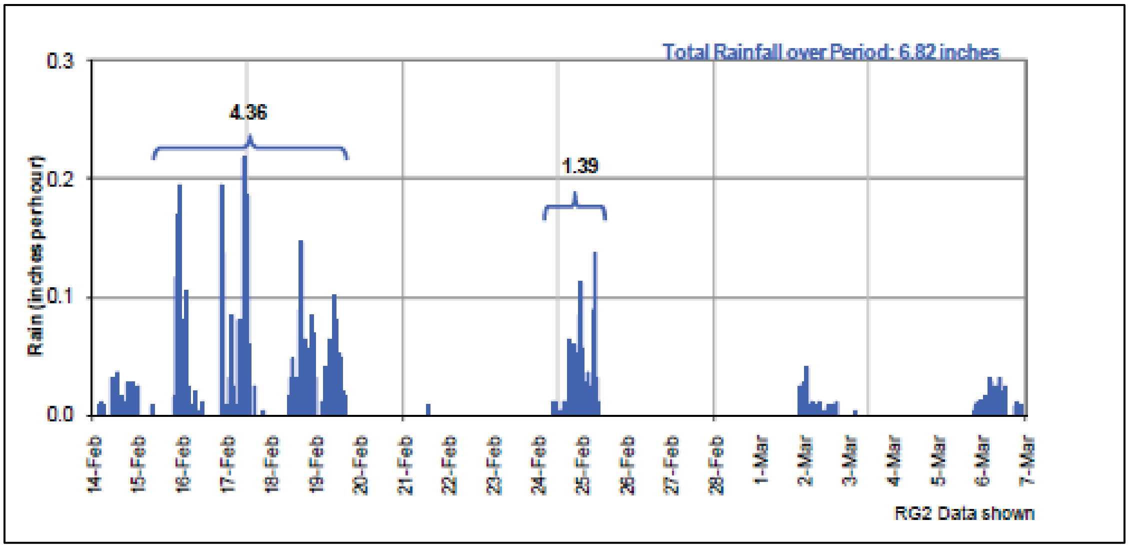
Table 3-1. Summary of Basins and Associated Flowmeters

Basin	Meter Number	Location of Flowmeter (Manhole)	Cross Street
1	1A and 1B	115046 and 115047	West of Aviador Avenue next to the Flood Control Channel
2A	2A	225049	Murchison Drive between Ogden Drive and Sequoia Avenue, in front of Mills High School
2B	2B	115064	Aviador Avenue behind Millbrae Avenue Chevron
3A	3A	118003	El Camino Real between Victoria Avenue and Chadbourne Avenue
3B	3B	114006	El Camino Real between La Cruz Avenue and Victoria Avenue
4	4A and 4B	410005 and 410006	El Camino Real between Meadow Glen Avenue and Silva Avenue
5	5	500908	Helen Drive south of Geraldine Drive
6	6	607053	Magnolia Avenue north of Millwood Drive, east side of Capuchino High School

3.1.1.2 Rainfall Data

Two rain gauges were installed by V&A within the City's service area during the 2010/11 flow monitoring effort. Rain Gauge No. 1 was installed near the shoreline at the intersection of Monterey Street and Bay Street, capturing the rain in the flat areas of the City near the coast. Rain Gauge No. 2 was installed at the intersection of Richmond Drive and Geraldine Drive, capturing rainfall in the City's hills. Figure 3-3, taken from Figure 11 in the flow monitoring report, presents the recorded rainfall during the 2010/11 wet weather season in a graphical format.

Figure 3-3. Rainfall Events Captured During Flow Monitoring Period



The largest rainfall event during the flow monitoring period occurred from February 15-20, 2011. A smaller storm occurred between February 24-26, 2011. The total rainfall volume recorded by each of the rain gauges during the flow monitoring period is presented in Table 3-2.

Table 3-2. Summary Flow Monitoring Period Rainfall Data

Rainfall Event	Rain Gauge No. 1, inches	Rain Gauge No. 2, inches
Event 1: February 15 – 20, 2011	4.20	4.36
Event 2: February 24 – 26, 2011	1.05	1.39
Total over Flow Monitoring Period	6.11	6.82

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

3.1.1.3 Dry Weather Flow Results

V&A calculated average dry weather flows (ADWF) from data collected during the system-wide flow monitoring period. Average dry weather flows include Base Wastewater Flow (BWF) and Groundwater Infiltration (GWI). BWF includes the wastewater generated from residential, commercial, and public users. GWI includes Base Infiltration (BI) and other inflow that is not dependent on precipitation. The ADWF values recorded at each flow monitoring site are shown in Table 3-3.

Table 3-3. Average Dry Weather Flow Data

Site No.	City Manhole ID	Weekday ADWF, mgd	Weekend ADWF, mgd	Overall ADWF, mgd	Weekend/ Weekday Ratio
1A	115046	0.684	0.719	0.694	1.05
1B	115047	0.310	0.344	0.320	1.11
2A	225049	0.182	0.189	0.184	1.04
2B	115064	0.220	0.223	0.221	1.01
3A	118003	0.092	0.094	0.093	1.02
3B	114006	0.213	0.209	0.212	0.98
4A	410005	0.569	0.576	0.571	1.01
4B	410006	0.037	0.041	0.038	1.11
5	500908	0.150	0.152	0.150	1.01
6	607053	0.121	0.127	0.122	1.05

3.1.1.4 Wet Weather Flow Results

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 rainfall-dependent inflow and infiltration (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 3-4, below. Table 3-4 includes a calculation of the wet weather peaking factor for each basin. It should be noted that when a basin has low dry weather flows, a small increase in wet weather flow measured by volume may constitute a large increase measured in percentage. Therefore, high peaking factors for basins with low flows are not necessarily indicative of a systematic I&I issue.

Table 3-4. Summary of Wet Weather Flow Monitoring Data

Meter Site No.	City Manhole ID	Pipe Diameter, inches	ADWF, mgd	Peak Flow, mgd	Wet Weather Peaking Factor	Surcharge Above Crown of Pipe, ft
1A	115046	18	0.694	3.46	5.0	8.5
1B	115047	12	0.320	1.87	5.8	8.2
2A	225049	10	0.184	1.21	6.6	n/a
2B	115064	12	0.221	0.93	4.2	7.6
3A	118003	13.5	0.093	2.18	23.6	1.2
3B	114006	18	0.212	1.56	7.4	1.4
4A	410005	10	0.571	1.39	2.4	1.1
4B	410006	10	0.038	1.25	32.7	0.8
5	500908	8	0.150	0.48	3.2	n/a
6	607053	12	0.122	1.32	10.8	n/a

Example flow hydrographs for each metering site are presented in Appendix B.

3.1.1.5 Infiltration and Inflow Analysis

V&A completed an I&I evaluation based on the 2010/11 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 City's Capacity Assurance Report. Separate evaluations were conducted for inflow and for RDII.

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. 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 3-5 below.

Table 3-5. Inflow Analysis			
Basin.	ADWF, mgd	Peak I&I Rate, mgd	Peak I&I to ADWF Ratio
1	1.014	4.18	4.12
2A	0.184	1.02	5.52
2B	0.221	0.70	3.15
3A	0.093	2.08	22.41
3B	0.212	1.05	5.93
4	.0610	1.81	2.96
5	0.150	0.37	2.72
6	0.122	1.17	9.56

The basin with the highest potential inflow was monitored by Meter 3A. Direct household storm drain connections may contribute to the measured stormwater inflow. One potential source could include private business stormwater connections, which would be unpermitted, and therefore unknown to the City. Although the City operates separate stormwater and sewer systems, another source could include historical connections between the two systems that are also unknown to the City.

V&A used data from February 11, 2011 to quantify RDII. Using this data, V&A evaluated RDII by comparing 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.

Overall basin rankings are presented in Table 9 of the Flow Monitoring Report, and also in Table 3-6, below. Based on the comparison of rankings, the basins monitored Meter No. 3B and No. 2A displayed the highest overall potential I&I within the City's service area.

Table 3-6. RDII Analysis			
Basin	ADWF, mgd	Peak RDII Rate, mgd	RDII to ADWF Ration
1	1.014	0.334	31.9%
2A	0.184	0.107	57.2%
2B	0.221	0.074	33.3%
3A	0.093	0.009	49.0%
3B	0.212	0.132	63.2%
4	.0610	0.059	9.7%
5	0.150	0.067	44.1%
6	0.122	0.063	50.0%

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. A more sophisticated analysis for BI and GWI was subsequently performed by West Yost. This subsequent analysis used the statistical methods contained in the Minimum Basin Method, the Wastewater Production Method, and the Stephens/Schutzbach Method in order to quantify BI over what might be considered normal in the various flow monitoring basins. For the GWI analysis, weekday ADWF was used to avoid the potential for large industrial and commercial weekend flow patterns to skew the results. The excess GWI that was estimated is shown in Table 3-7. The impact of GWI in the overall I&I analysis is discussed above.

Table 3-7. GWI Analysis

Basin	ADWF, mgd	Excess GWI, mgd
1	1.014	0.000
2A	0.184	0.041
2B	0.221	0.057
3A	0.093	0.016
3B	0.212	0.000
4	.0610	0.200
5	0.150	0.048
6	0.122	0.015

3.1.2 City of Millbrae Water Billing Information

The City of Millbrae operates its own water distribution system within the same area served by the sanitary sewer system. The City provided water billing records for the months from September 2010 to August 2011. These records contain the amount of water consumed at each address in the City, with no customer information provided. As the City reads water meters on a rotating, bi-monthly schedule, the water billing records were processed in order to provide monthly water consumption at each address in the City.

The monthly water consumption data was matched with parcel GIS data in order to spatially locate the water consumption. Large water consumption parcels were identified in order to determine if these parcels would generate large sewer flows as well, or if the water consumption was used for irrigation or industrial processes that would not be returned to the sanitary sewer. The City's large water users are shown on Figure 3-4. The large water users are identified by parcel in Table 3-8.

Table 3-8. Large Water Users			
Point Load	APN	Address	Type
1	021470030	608 End of Geraldine-Country Club/ Helen Drive	Golf Course
2	021324290	1100 El Camino Real	Hotel
3	021435280	509 Poplar Avenue	Apartment
4	021150010	1544 Magnolia Avenue	School
5	024344080	51 Millbrae Avenue	Restaurant
6	024154220	250 El Camino Real	Hotel
7	021311260	33 Mateo Avenue	Hospital
8	024337080	150 Serra Avenue	Hospital
9	021362310	979 Broadway	Commercial / Gym
10	021131220	1671 El Camino Real	Restaurant
11	024320070	400 Murchison Avenue at 75 Magnolia Avenue	School
12	024284010	240 Millbrae Avenue	School or Senior Community
13	100710010	1550 Frontera Way	Apartment
14	024344020	88 South Broadway	Condo
15	021290270	1201 Broadway	Senior Community
16	021281490	1380 El Camino Real	Apartment
17	024152010	279 El Camino Real	Restaurant
18	021352210	1388 Broadway	Condo
19	103670010	300 Murchison Avenue	Apartment
20	021281380	1280 El Camino Real	Apartment
21	024146020	349 Broadway	Restaurant
22	021324230	1180 El Camino Real	Restaurant
23	103500010	1396 El Camino Real	Apartment
24	021434230	401 Richmond Drive	Apartment
25	021363350	1065 El Camino Real	Laundromat (Commercial)
26	021311270	1001 Hemlock Avenue	Senior Community
27	021420140	525 El Camino Real	Senior Community
28	021281710	1300 El Camino Real	Apartment
29	100070010	360 Vallejo Way	Apartment/Condo

Table 3-8. Large Water Users			
Point Load	APN	Address	Type
30	024352020	110 South El Camino Real	Hotel
31	021281310	1260 El Camino Real	Hotel
32	021311280	1007 Hemlock Avenue	Apartment
33	021361310	1051 Broadway	Apartment
34	024146150	104 Hillcrest Blvd	Commercial / Residential
35	021278040	1375 El Camino Real	Hotel
36	024015190	1410 Millbrae Avenue	Apartment
37	024361020	370 Adrian Road	Industrial (Food Distribution Center)
38	024334140	34 Broadway	Apartment
39	021281590	6 Berni Court	Apartment
40	021131100	144 Park Blvd	Laundromat
41	021434240	421 Richmond Drive	Apartment
42	024115050	465 Broadway	Restaurant
43	024332060	100 Millbrae Avenue	Apartment
44	021434220	432 Lincoln Circle	Apartment
45	021352220	1017 Magnolia Avenue	Apartment
46	024147070	235 Broadway	Restaurant
47	024152270	245 El Camino Real	Restaurant
48	021292060	1101 El Camino Real	Restaurant
49	021041210	1225 Helen Drive	Swimming Club
50	021362110	950 Magnolia Avenue	Apartment

3.1.3 City of Millbrae 2010 Urban Water Management Plan

The 2010 UWMP, dated June 2011, satisfies the Urban Water Management Planning Act for the City. As part of this planning effort, wastewater flows for the City of Millbrae were projected to the year 2035. Table 3-9, taken from Table 4-1 in the 2010 UWMP, shows these wastewater flow projections. These projections are based upon proposed redevelopment within the City, account for the recent reductions in per capita water demand and wastewater generation identified in the 2010 UWMP.

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Table 3-9. 2010 Urban Water Management Plan Wastewater Collection Projections						
Description	Existing (2010)	2015	2020	2025	2030	2035
Total Flow, AFY	1,736	2,097	2,140	2,259	2,367	2.475
Total Flow, mgd	1.55	1.87	1.91	2.02	2.11	2.21

3.2 DEVELOPMENT OF FLOWS FOR CAPACITY ASSURANCE REPORT

This section summarizes the methodology used to develop the initial dry and wet weather flows for the collection system hydraulic model. These initial flows were further refined through the hydraulic model calibration process, as discussed in Chapter 4, Hydraulic Model Development.

3.2.1 Dry Weather Flows

Numerous methodologies are available to estimate ADWF that rely on population estimates, water usage, zoning designations, and other resources. Water usage and zoning designations were available and used for existing ADWF development in the City of Millbrae.

The initial ADWF component was calculated using the following steps:

1. Calculate the winter (November, December, January, and February) water consumption for each parcel in the City based upon water billing records.
2. Calculate water consumption per dwelling unit for Single Family Residential parcels across the City.
3. Calculate water consumption per acre for all Multi-family Residential parcels across the City.
4. Develop Return-to-Sewer (RTS) ratios for each zoning in order to calculate the amount of wastewater returned to the sanitary sewer system for each unit of water consumed. RTS ratios are based upon industry standards and calibrated specifically to the City's data.
5. Use the RTS ratios in order to calculate the estimated wastewater flow for each parcel in the City. Wastewater flows for residential parcels are calculated based upon the residential factors calculated above. Flow for non-residential parcels is calculated based upon the actual measured consumption for such parcels.
6. 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.

Build-out flows were calculated by assigning unit flow factors to build-out zonings, according to the following steps:

- Populate all vacant residential parcels,
- Add flow from the following projects identified by City staff
  - 1388 Broadway
  - 88 S. Broadway
  - 120 El Camino Real
  - Wilson Plaza
- Increase flows generally across the City to account for general densification. Total build-outflows match flows predicted for 2035 in 2010 UWMP.

The residential flow factors and RTS Ratios developed for all zonings are shown in Table 3-10.

Table 3-10. Zoning Flow Factors, Adjusted to Balance Flows with Measured Data

Zoning[a]	Wastewater Flow Factor gpd/parcel or gpd/acre	RTS Ratio
Residential 1 Low Density	160	
Residential 1	160	
Residential 1 Office	1,000	
Residential 2	1,800	
Residential 3	4,300	
Residential Growth	2,040	
Residential Senior Center	1,000	
Commercial 1	-	0.80
Commercial 1 High Density	-	0.80
Commercial 2	-	0.90
Downtown Improvement Area	-	0.95
Planned Development	-	0.80
Industrial	-	0.80
Open Space	-	0.00
Utility	-	0.00
Undesignated	-	0.00
Capuchino High School (City of San Bruno)	-	0.60
Right of Way	-	0.00

(a) Residential 1 Low Density and Residential 1 both have units of gpd/parcel. All other residential zonings have units of gpd/acre.

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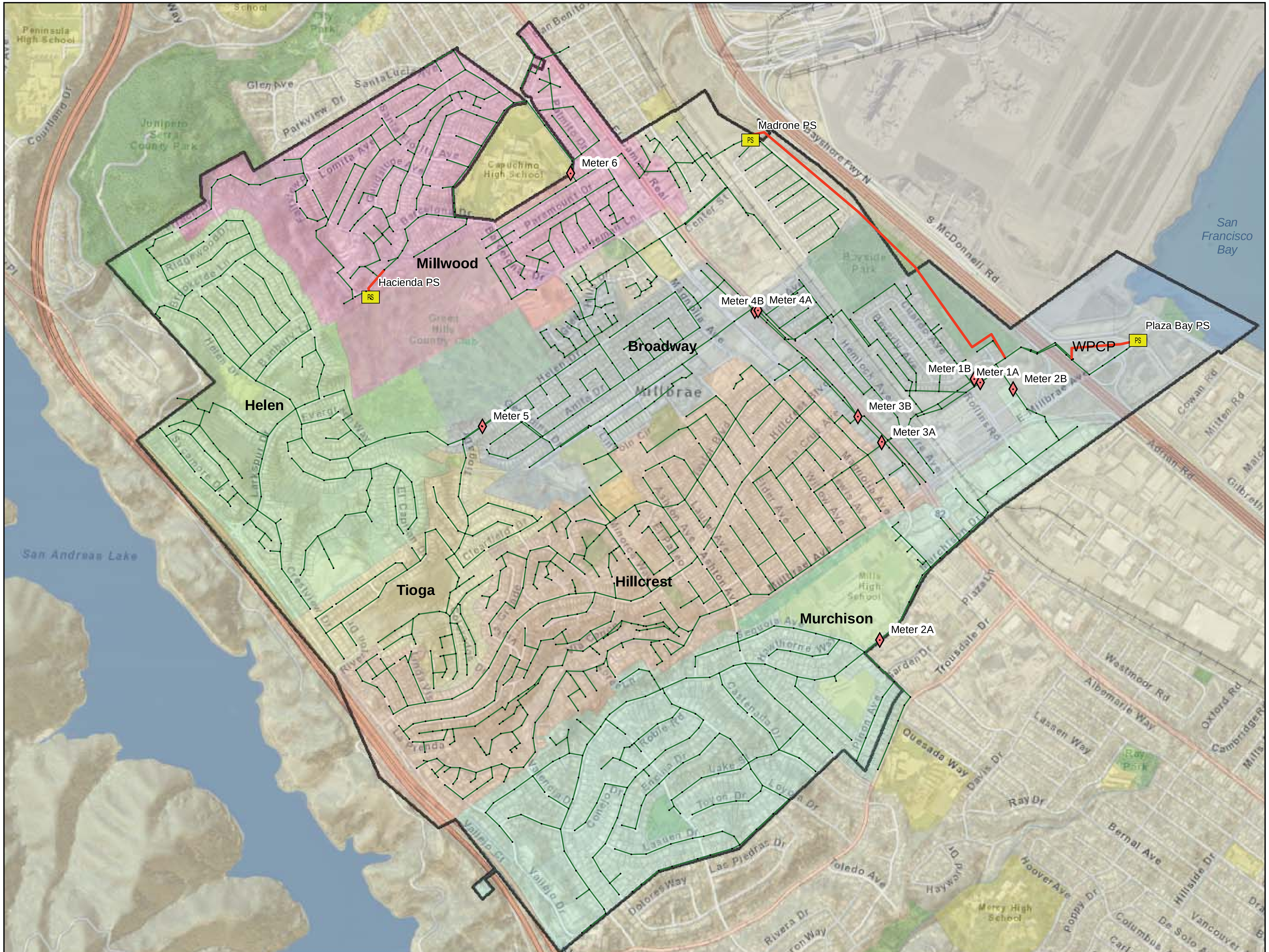

FIGURE 3-1

City of Millbrae
Capacity Assurance Report

FLOW MONITORING LOCATIONS AND SEWER BASINS

N

0 625 1,250

Scale in Feet

1 inch = 1,250 feet

LEGEND

- Manhole
- Pump Station
- 2010/11 Flow Meter Location Used in 2011/12
- Force Main
- Gravity Main

Flow Meter Basins

- Broadway
- Helen
- Hillcrest
- Millwood
- Murchison
- Tioga
- City of Millbrae Boundary

WEST YOST ASSOCIATES Consulting Engineers

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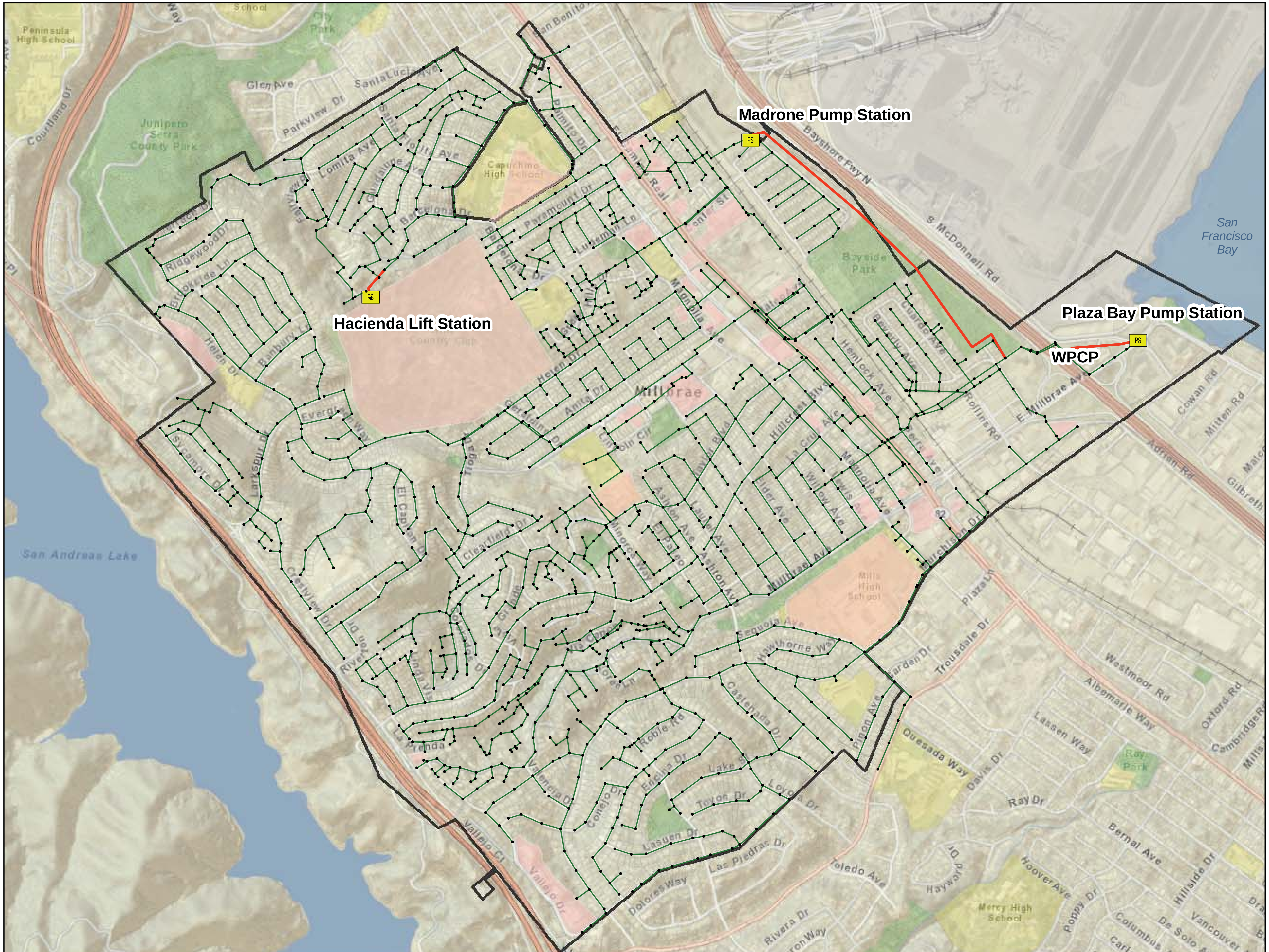


FIGURE 3-4

City of Millbrae
Capacity Assurance Report

CITY OF MILLBRAE LARGE WATER USERS

N

0 625 1,250

Scale in Feet

1 inch = 1,250 feet

LEGEND

- Manholes
- PS Pump Stations
- Force Mains
- Gravity Mains
- Large Water Users
- City of Millbrae Boundary

SEAL OF THE CITY OF MILLBRAE CALIFORNIA

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 4
Hydraulic Model Development

The computer-based hydraulic sewer model of the City's wastewater collection system, developed using fully dynamic Innovyze® InfoSWMM™ software, serves as a tool for assessing the flows and capacities of the City's major sewers, and for identifying solutions to potential capacity issues. The hydraulic model can also be used as 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 City's main trunk sewers and associated facilities, and is a simplified representation of the City's total sewer system in its configuration and operation. The City'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 primary sections of this chapter include:

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

4.1 MODEL DEVELOPMENT

The City'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 City'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, orifices, 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.

4.1.1 Model Description

The City's sanitary sewer infrastructure data is stored in the Asset Information Management System (AIMS), a web-based computerized maintenance management system. Asset information was exported from AIMS into a GIS database. The hydraulic model system configuration was developed using the GIS pipe, manhole, and lift station layer data. All spatial locations were imported into the hydraulic model from the GIS. In addition, attribute data including pipe diameter and pipe length was imported into the model from the GIS. The City's AIMS database did not contain any elevation information for the sanitary sewer system. Invert elevations for pipes in the model were calculated by first establishing rim elevations for manholes. Rim elevations were interpolated from ground surface elevations taken from topographic lines provided by the City. Pipe invert elevations were calculated by subtracting manhole depths from these rim elevations. Manhole depths were collected by City Operations and Maintenance staff.

The City's hydraulic model consists of approximately 13 miles of sewer pipeline ranging in diameter from 6-inches to 36-inches. The model includes all 8-inch diameter and larger trunk lines, and associated manholes and lift stations. Six-inch diameter pipelines were then added to the model as requested by City staff and as needed to provide network connectivity. The 13 miles of pipeline represent approximately 20 percent of the City's sanitary sewer system. A summary of the modeled gravity mains is provided in Table 4-1. The City's hydraulic model contains more 6-inch diameter pipelines than are typically included. However, the inclusion of these lines is appropriate given the size of the City's collection system, the total amount of 6-inch pipeline in the system, and the hydraulic significance of many of the 6-inch pipelines. In addition to the gravity mains described above, the hydraulic model contains approximately a mile of forcemains ranging from 6-inches to 14-inches.

Table 4-1 Hydraulic Model Gravity Main Summary

Diameter, inches	Length, feet	Length, miles	Percentage
6	16,676	3.16	25.06%
8	17,626	3.34	26.49%
10	13,364	2.53	20.09%
12	5,905	1.12	8.87%
14	1,349	0.26	2.03%
15	1,616	0.31	2.43%
16	4,656	0.88	7.00%
18	3,368	0.64	5.06%
24	832	0.16	1.25%
36	1,145	0.22	1.72%
Total	66,537	12.6	100.00%

The City's three collection system pump stations are included in the hydraulic model: Madrone, Hacienda, and Plaza Bay Pump Stations. The pump station parameters in the model are summarized in Table 4-2. The modeled collection system facilities are presented in Figure 4-1.

Table 4-2. Collection System Pump Station Model Parameters

	Wet Well		Pumps				
Lift Station	Size	Base Elevation, ft	No. of Pumps	Speed	Lead/Lag Pump On Elevation, ft	Lead/Lag Pump Off Elevation, ft	Pump Discharge Rate, gpm
Madrone	5 ft x 22 ft by 11 ft deep	-14.0	3	Variable	-6.4/-5.4	-13.0/-13.0	1,425
Hacienda	3.5 ft diameter by 2 ft deep	160.0	1	Fixed	163.0	160.0	150
Plaza Bay	8 ft diameter by 6.0 ft deep	-6.0	2	Fixed	-5.5	-1.0	1,000

The City's collection system flows terminate at the City's Water Pollution Control Plant (WPCP). Because the WPCP has limited hydraulic capacity that affects the operation of the collection system, the WPCP is actively modeled in the hydraulic model. The influent wet well is modeled as a storage device, and the WPCP itself is modeled as a pump that discharges to the hydraulic model system outfall. Additionally, the WPCP contains flow equalization storage. Flow equalization is modeled as a storage device with a pump for supply and an orifice to return flow to the influent wet well. The WPCP parameters are summarized in Table 4-3. The operation of the influent wet well and flow equalization storage was modified between the time of model calibration and current operation, as will be discussed further in this document. These changes are reflected in the hydraulic modeling.

Table 4-3. Water Pollution Control Plan Model Parameters

	Wet Well		Pumps				
Facility	Size	Base Elevation, ft	No. of Pumps	Speed	Pump On	Pump Off	Pump Discharge Rate, mgd
WPCP Process	800 square ft by 18 ft deep	-21.0	1	Variable	n/a	n/a	9.0
Flow Equalization	7,900 square feet by 22 ft deep	-18.25	1	Fixed	Manual Flowrate Control	When Flow Equalization Storage is Full	3.0

4.2 DATA VALIDATION

After development of the model network as described above, 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; and
- Populate global parameters such as standard manhole diameters and Manning's "n" coefficient, which is entered as 0.013 for sewer pipelines.

4.3 FIELD INVESTIGATIONS

Following data validation, West Yost reviewed the City'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. Three manholes in two 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 three most critical manholes and related observations.

- Site 1 – Manhole 318002 on Broadway Avenue & Alley Between Victoria Avenue and Chadbourne Avenue. An inspection was performed in order to verify that flow moves toward Manhole 318002A to the northeast, and not down Broadway Avenue to the northwest. Visual inspection of the configuration through the open manhole confirmed this flow. Also confirmed was the fact that there is a steep slope in the pipe exiting the manhole to the northeast, and flow moves rapidly.
- Site 2 – Manhole No. 118003 on El Camino Real & Alley Between Victoria Avenue and Chadbourne Avenue. An investigation was performed to verify the configuration of the manhole and to confirm that flow does not northwest along El Camino Real. It was confirmed that flow moves toward Manhole 114003 to the northeast. The investigation confirmed that Flow Monitoring Basin No. 3A is hydraulically isolated from Flow Monitoring Basin No. 3B at this location.
- Site 3 – Manhole No. 707018 on Landing Lane. This manhole was investigated in order to confirm preliminary modeling results that showed very flat pipes and very slow flows. Discussion with City staff confirmed that the area consistently has high and slow flows. Visual inspection of the manhole revealed that the slow velocity and direction were almost indiscernible. The investigation included visual review of the easements between the manhole being investigated and Madrone Pump Station. These easements include back yards and an open parcel that has Federal Aviation Administration (FAA) access restrictions.

Field investigation sites 1 through 3 are shown in Figures 4-2 through 4-4, respectively.

4.4 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 were divided into three categories. All of these flows are discussed further in this section, and are also described in Chapter 3, System Flows:

- Base Wastewater Flow (BWF) includes the sanitary flow contribution from permitted connections to the collection system;
- Groundwater Infiltration (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
- Rainfall-Dependent Inflow and Infiltration (RDII) results 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 264 sewersheds to facilitate the assignment of sewer flows in the hydraulic model. Each sewershed defines a group of parcels where baseflow generated in the parcels is assigned to a specific node (or manhole) in the model. Each sewershed encompasses a particular subdivision or grouping of parcels that flows to a single point in the collection system. Figure 4-5 shows the sewersheds that were included in the hydraulic model. Each sewershed is identified by model manhole through which it contributes flows into the model.

4.4.1 Dry Weather Flow Generation

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

4.4.1.1 Base Wastewater Flows

BWF can be calculated based one or more factors, including population, population density, water consumption, and land uses. As discussed further in Chapter 3, System Flows, the City's hydraulic model is loaded with BWF that is based upon water consumption and zoning. For residential zoning, BWF was calculated per dwelling unit and per acre. For non-residential zoning, actual winter water use was multiplied by a Return-to-Sewer (RTS) ratio in order to determine BWF. The zoning groupings within the City 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 (ADWF)
- Peak Dry Weather Flow (PDWF)

The residential ADWF calculation is based on the individual parcel wastewater flows as described above and in Chapter 3, System flows. Individual parcel residential flows were summed by sewershed. For parcels containing Single Family Residential dwelling units, the methodology described yielded a value of 160 gallons per day (gpd) per dwelling unit, which was assigned to each parcel. For parcels containing Multi-Family Residential dwelling units, a value of flow per acre was multiplied by parcel acreage, based upon zoning designation.

For non-residential zoning, ADWF was again calculated per sewershed by summing individual non-residential parcel flows within the sewershed. Individual parcel flows were created by multiplying winter water usage by RTS ratios in order to arrive at estimated wastewater generation per parcel. Non-residential land use flow included flow contributions from all land use categories with a non-residential designation, including schools and public facilities.

West Yost refined these unit flow factors by calculating the overall flow generated from the City's service area for each of the basins monitored during the City's 2010/11 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.

4.4.1.2 Diurnal (24-Hour) Flows

BWF typically varies throughout the day, with the peak flow generally occurring in the morning and evening periods. V&A generated 24-hour weekday and weekend diurnal patterns for each monitored basin within the City's service area. Data was derived from flows collected in 15 minute increments, 24 hours per day, for the flow monitoring period.

A sample weekend diurnal curve is presented in Figure 4-6 for Basin 2A. 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 ADWF, within each monitored basin to distribute the ADWF over a 24-hour period. Weekend diurnal patterns were used for the dry weather flow calibration because the weekend patterns generally had a slightly higher peak ratio, and are therefore more conservative.

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 GWI values that were estimated for each basin were discussed in Chapter 3, System Flows.

4.4.2 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 City's hydraulic model to replicate measured flow data. The key elements of wet weather flow generation in the model include:

- Infiltration and Inflow (RDII or I&I)
- Peak Wet Weather Flow (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.

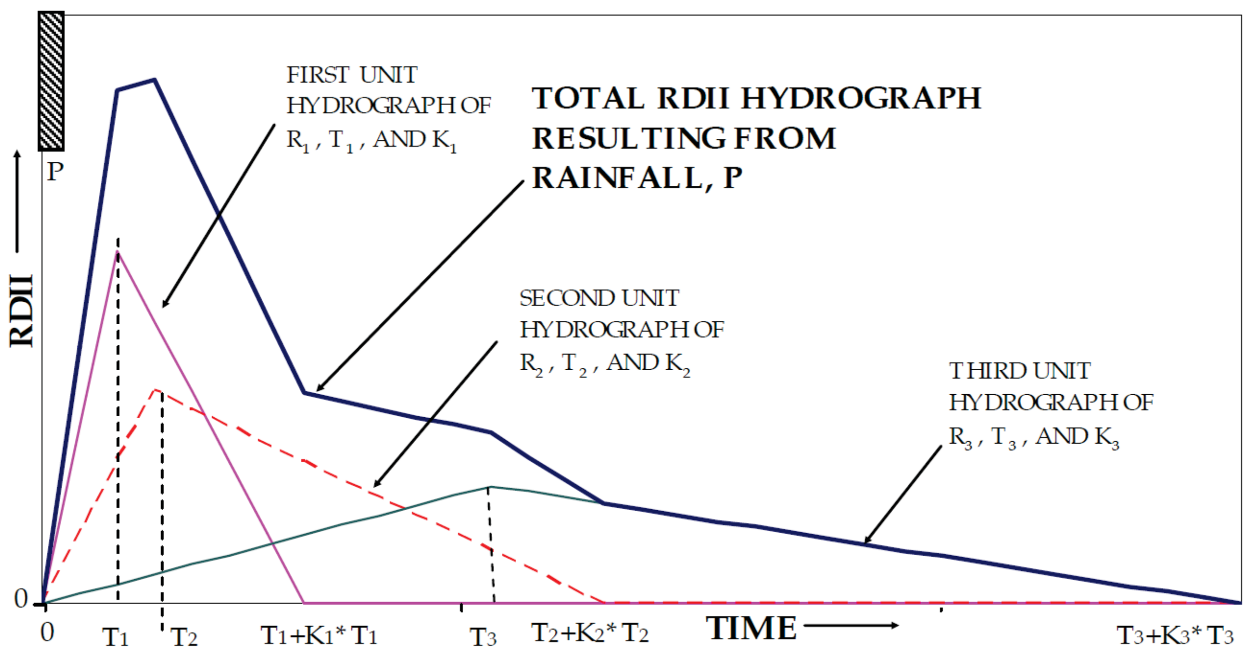
Studies conducted by the Water Environment Research Foundation (WERF) have concluded that the SUH and rainfall/flow regression methods are the two most accurate methods for predicting peak flows and event volumes for storm events. The RTK method is the most widely used SUH prediction methodology for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

West Yost used the RTK method to calculate RDII inputs to the City'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 4-7.

Figure 4-7. 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. In San Francisco Bay Area communities, typically, the peak wet weather flow will occur shortly after the hourly peak intensity of the rainfall event.

4.5 DRY WEATHER FLOW CALIBRATION

The City'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:

1. Determine the average dry weather 24-hour flow (Q_a) for the entire service area for the calibration period.
2. 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.
3. Compare the modeled Q_a values with the measured Q_a values for the entire service area and at each flow metering site.
4. Adjust the model flow factors to maximize agreement between the modeled and metered Q_a 24-hour values.
5. Adjust the diurnal curve for each basin to maximize agreement between the modeled and weekday metered Q_a hourly values.

4.5.1 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 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.

A sample dry weather flow calibration hydrograph for Basin 2A is provided in Figure 4-8. The remaining DWF calibration hydrographs are presented in Appendix D. The weekday dry weather flow calibration results for each meter are presented in Table 4-4. The metering locations and basin delineations were presented in Chapter 3, in Figure 3-3.

Table 4-4. Weekend 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
1A	0.24	1.16	0.69	0.40	1.12	0.73	62.45%	-3.30%	4.49%
1B	0.12	0.55	0.32	0.14	0.52	0.32	18.17%	-4.86%	1.24%
2A	0.06	0.35	0.18	0.08	0.34	0.18	32.71%	-2.20%	-0.21%
2B	0.08	0.37	0.22	0.09	0.39	0.22	10.70%	4.05%	-0.11%
3A	0.08	0.15	0.09	0.08	0.15	0.09	-1.71%	1.30%	0.44%
3B	0.08	0.33	0.21	0.07	0.34	0.21	-13.20%	1.70%	-0.43%
4A	0.30	0.89	0.57	0.35	0.87	0.59	18.84%	-1.83%	2.82%
4B	0.01	0.07	0.04	0.01	0.07	0.04	74.77%	-1.10%	-2.70%
5	0.06	0.21	0.15	0.08	0.21	0.15	23.80%	-0.88%	3.02%
6	0.03	0.21	0.12	0.02	0.22	0.12	-16.41%	3.64%	0.72%

4.6 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 City'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 ten percent for the calibration event, and five percent where possible. Match peak flows first, and also consider total volume and the temporal distribution of flows.
- After the modeled flows closely match metered flows, compare flows from a second or extended rainfall period to validate the accuracy of the calibration.

The largest storm event that was captured during the 2010/11 flow monitoring season occurred from February 15-20, 2011. The wet weather model calibration included the time period from February 16 to February 20, 2011. Normally a shorter time period of one to two days would be chosen for calibration as discussed above. However, given the low intensity but long distribution of the storm captured during flow monitoring, a longer calibration period was necessary to capture a significant amount of rainfall. The longer calibration process increased the difficulty of calibration but also increased confidence in the ability of the model to predict wet weather response.

Wet weather flow calibration results are provided in Table 4-5. Figure 4-9 presents a graphical sample of successful wet weather flow calibration. The remaining calibration graphs are presented in Appendix E. As noted in Table 4-5, several basins have comingled flow. As a result, calibration could not be achieved for the individual basins to the desired level of consistency. However, when connected basins were considered together, Calibration was achieved to a level that is within the accepted range for hydraulic model development.

Table 4-5. Wet Weather Flow Calibration Results for February 16-20, 2011

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, %
1A[(a)]	6.79	3.45	6.90	3.94	1.64%	14.20%
1B[(a)]	3.15	1.87	2.86	1.60	-9.22%	-14.14%
2A	1.97	1.21	1.97	1.24	-0.09%	1.94%
2B	1.90	0.96	2.00	1.01	5.38%	4.94%
3A[(b)]	0.84	2.18	0.88	0.68	4.34%	-68.80%
3B	2.15	1.34	2.16	1.40	0.52%	4.79%
4A[(c)]	3.36	1.28	3.46	1.57	3.01%	22.78%
4B[(c)]	0.52	1.20	0.42	0.79	-19.80%	-34.18%
5	1.04	0.43	1.05	0.42	0.18%	-1.46%
6	1.70	1.32	1.70	1.35	0.28%	2.22%

(a) Flow monitors No. 1A and No. 1B are not hydraulically isolated, but share flows from the basins that drain into them. The flow volumes and peak flows of the two monitors together were checked for overall calibration.

(b) The peak flow of 2.18 mgd was a one-time occurrence that stood apart from flow values that came before and after. The peak could not be replicated during calibration and is assumed to be an anomaly.

(c) Flow monitors No. 4A and No. 4B are not hydraulically isolated, but share flows from the basins that drain into them. The flow volumes and peak flows of the two monitors together were checked for overall calibration.

4.6.1 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.

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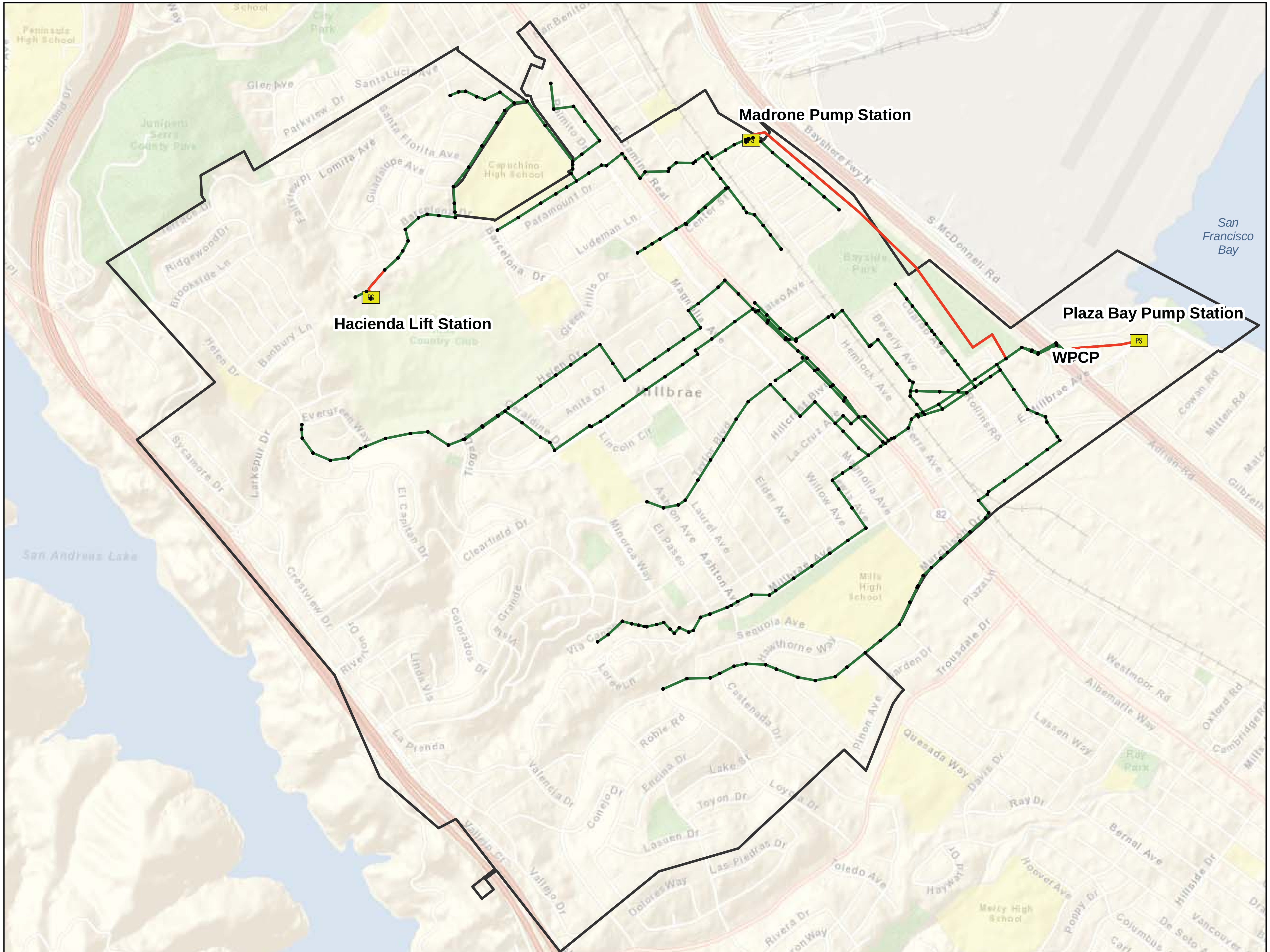

FIGURE 4-1

City of Millbrae
Capacity Assurance Report

CITY OF MILLBRAE MODELED COLLECTION SYSTEM FACILITIES

N

0 625 1,250

Scale in Feet

1 inch = 1,250 feet

LEGEND

- Modeled Manholes
- Modeled Pump Stations
- Modeled Force Mains
- Modeled Gravity Mains
- City of Millbrae Boundary

WEST YOST ASSOCIATES
Consulting Engineers

FIGURE 4-2

City of Millbrae
Capacity Assurance Report

SITE 1 FIELD INVESTIGATION

N

0 20 40
Scale in Feet

1 inch = 40 feet

LEGEND

- Modeled Manholes
- Modeled Force Mains
- Modeled Gravity Mains
- City of Millbrae Boundary

SEAL OF THE CITY OF MILLBRAE CALIFORNIA

WEST YOST ASSOCIATES
Consulting Engineers

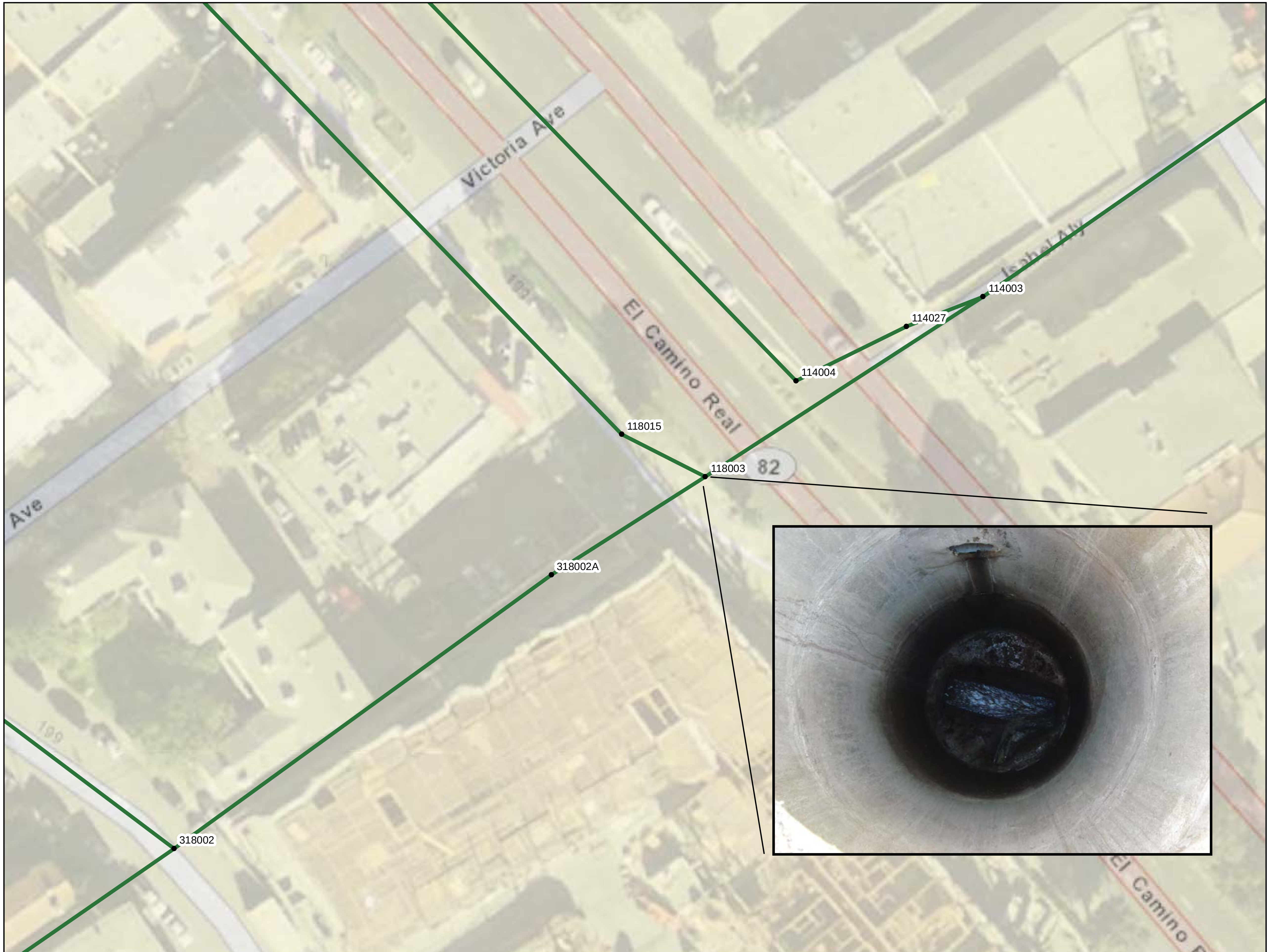

FIGURE 4-3

City of Millbrae
Capacity Assurance Report

SITE 2 FIELD INVESTIGATION

N

0 20 40
Scale in Feet

1 inch = 40 feet

LEGEND

- Modeled Manholes
- Modeled Force Mains
- Modeled Gravity Mains
- City of Millbrae Boundary

WEST YOST ASSOCIATES
Consulting Engineers

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FIGURE 4-4

City of Millbrae
Capacity Assurance Report

SITE 3 FIELD INVESTIGATION

N

0 20 40

Scale in Feet

1 inch = 30 feet

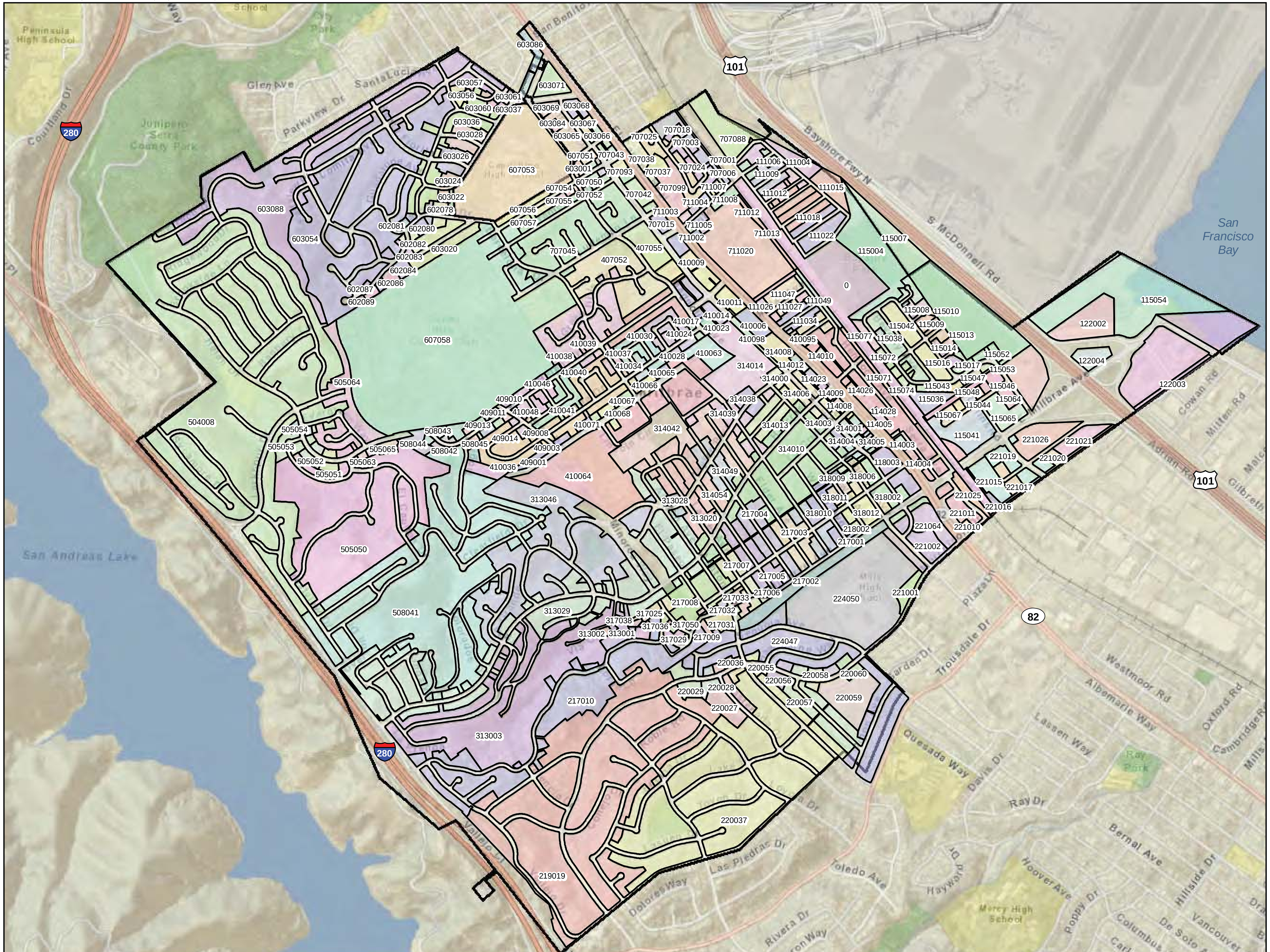
LEGEND

- Modeled Manholes
- Modeled Force Mains
- Modeled Gravity Mains
- City of Millbrae Boundary

FIGURE 4-5

City of Millbrae
Capacity Assurance Report

CITY OF MILLBRAE HYDRAULIC MODEL SEWERSHEDS



0 625 1,250
Scale in Feet

LEGEND

City of Millbrae Boundary

ID (Various) Sewershed

WEST YOST ASSOCIATES
Consulting Engineers

Figure 4-6. Diurnal Curve for Flow Monitor No. 2A

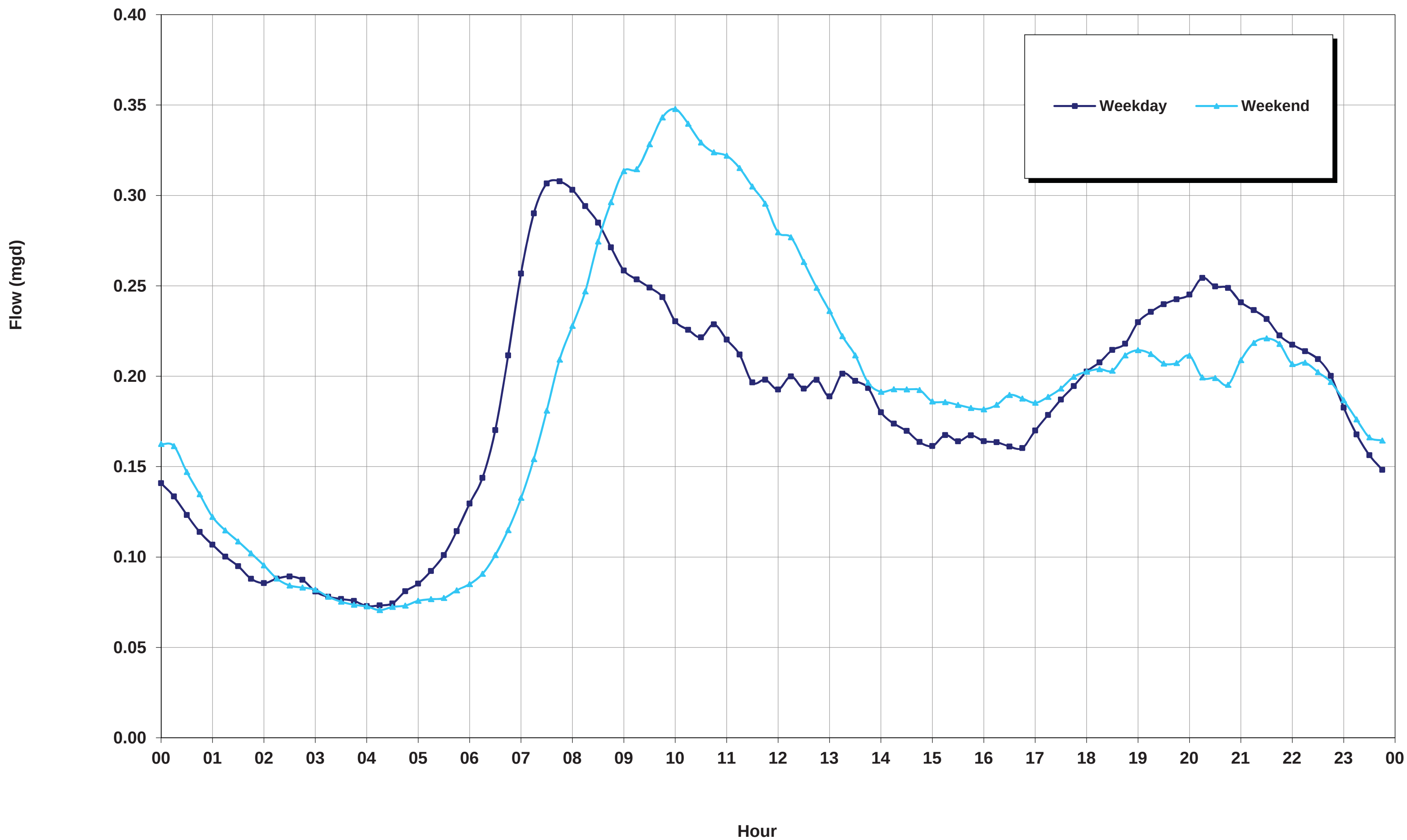


Figure 4-8. Sample Dry Weather Calibration Hydrograph for Flow Monitor No. 2A

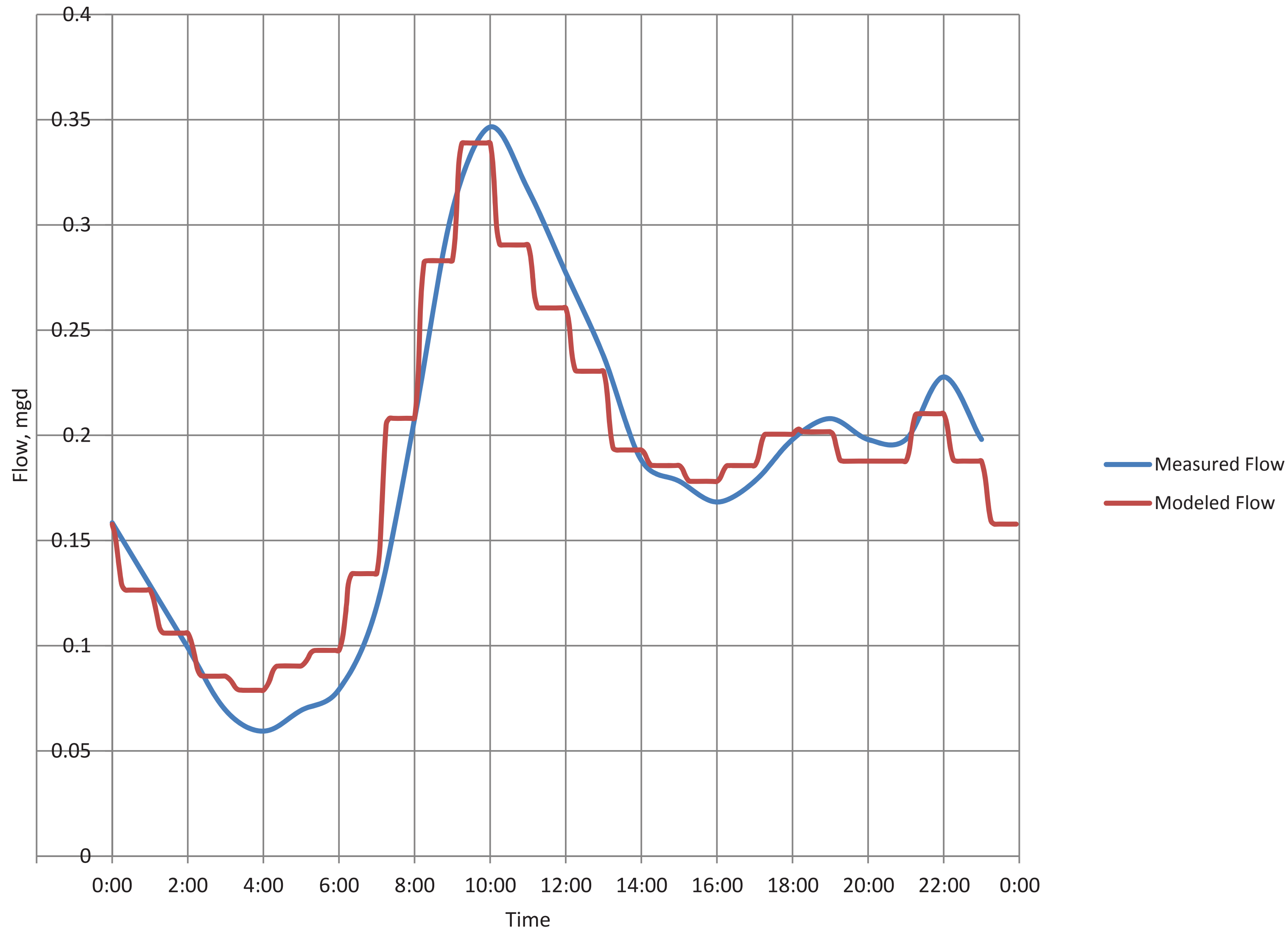
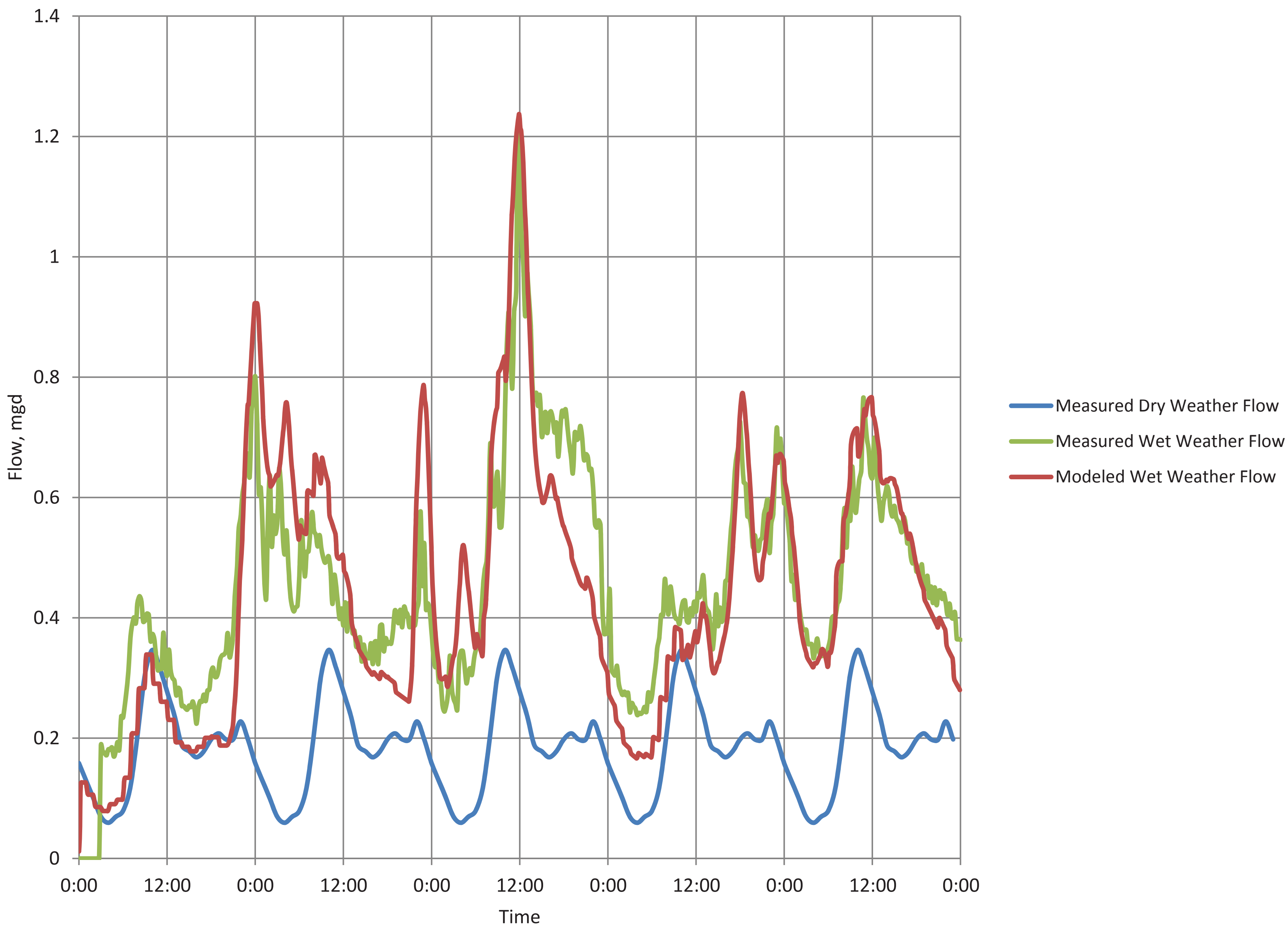


Figure 4-9. Sample Wet Weather Flow Calibration Hydrograph for Flow Monitor No. 2A



CHAPTER 5
Planning Criteria

The purpose of this chapter is to present planning criteria that can be used to evaluate system capacity and guide the development of proposed new replacement facilities. The criteria include generally accepted industry standard criteria, as reviewed and confirmed by the City. 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.

5.1 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 City is required to reduce sewer system overflows (SSOs) related to the designated design storm to specific limits over time to a maximum of three SSOs per hundred miles of pipe by 2016. This goal allows some storage within the existing manhole structures throughout the system, provided that adequate freeboard in the manhole is available.

The design storm specified for the City is rainfall event with a 10-year recurrence interval and 24-hour duration (10-year, 24-hour storm) This design storm is defined in the Consent Decree between the City and San Francisco Baykeeper as having a total depth of 3.14 inches as measured at the San Francisco International Airport. The rainfall is distributed using the U.S. Soil Conservation Service (SCS, now Natural Resource Conservation Service) Type IA rainfall distribution curve. Figure 5-1 presents the design storm rainfall distribution.

5.2 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. Exceptions to these criteria may be made on a case-by-case basis, depending on specific flow conditions and facility configuration. Capacity improvement projects have been proposed for all capacity deficient pipelines as discussed in Chapters 6, Capacity Analysis, and 8, Capital Improvement Program.

- Gravity Mains: A gravity main shall be considered to require capacity improvements by 2016 if flow through that gravity main results in a Hydraulic Grade Line (HGL) that exceeds the ground level, *i.e.*, if the flow results in a predicted Sanitary Sewer Overflow (SSO) in the hydraulic model.
- Force Mains: Force mains and their associated pump stations will be considered to require capacity improvements by 2016 if the associated firm capacity is not sufficient to convey the design storm. In addition, for future planning purposes, a force main shall be reviewed for potential capacity improvements if maximum velocity exceeds 8 feet per second (fps) during peak hourly flows.

5.3 NEW OR REPLACEMENT PIPELINE DESIGN CRITERIA

New (parallel relief) or replacement pipelines were designed to meet the following criteria. These criteria do not necessarily apply to the rehabilitation and replacement of isolated 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.

5.4 PUMP STATION DESIGN AND OPERATING CRITERIA

Pump Stations were sized to convey model-generated flows from the 10-year, 24-hour design storm event, with the largest pump out of service.

5.5 USE OF THE FLOW EQUALIZATION FACILITY

The City’s existing flow equalization facility is designed to equalize diurnal variation in dry weather flows in order to optimize the treatment process at the WPCP. However, the facility can be used in order to store wet weather flow as well, when flows into the plant exceed the 9 mgd rating of the WPCP treatment process. For the purposes of evaluation, it was assumed that 1.3 million gallons of storage space are available in the flow equalization facility.

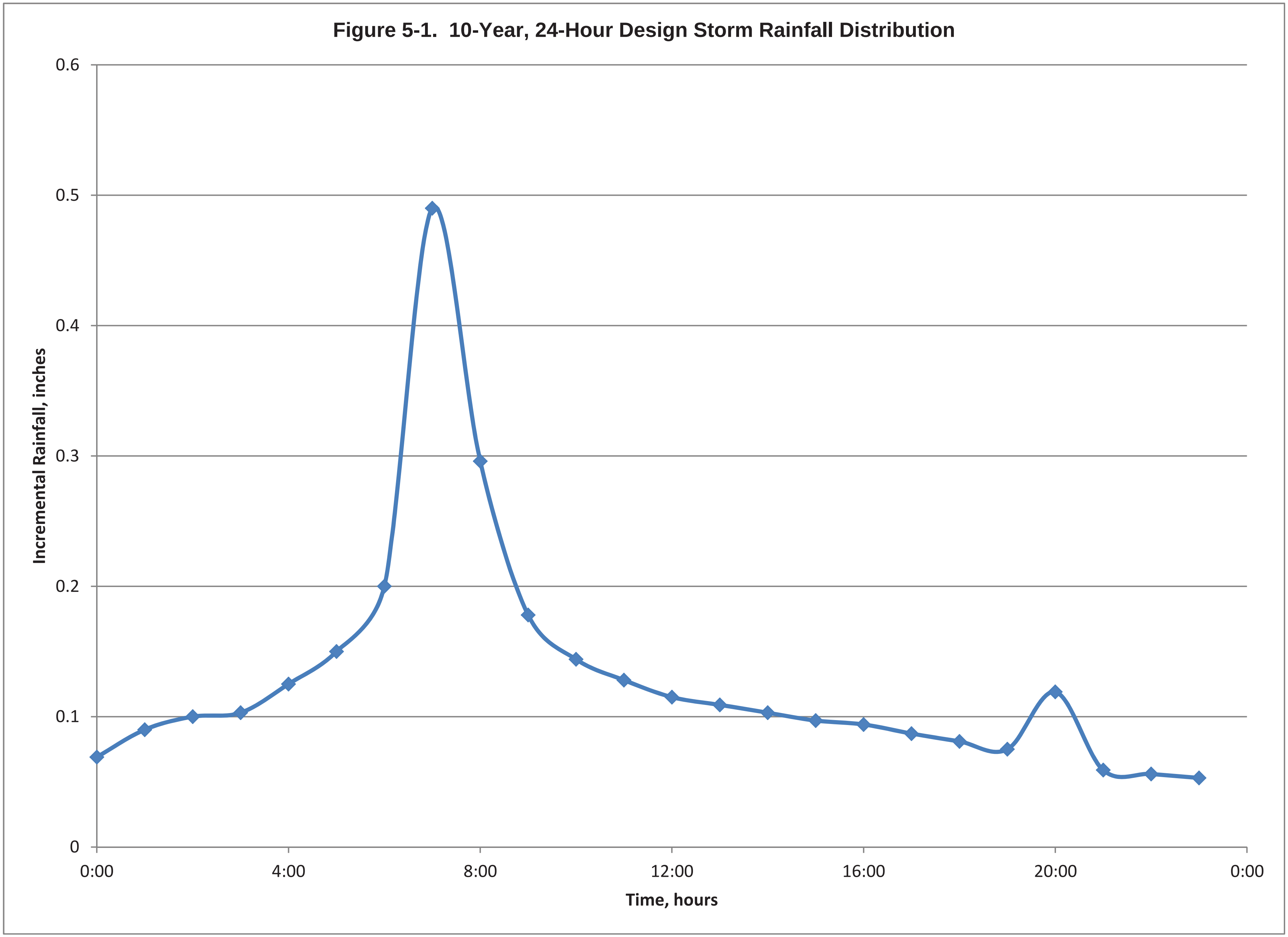


Figure 5-1. 10-Year, 24-Hour Design Storm Rainfall Distribution
0.6
0.5
0.4
0.3
0.2
0.1
0
Incremental Rainfall, inches
0:00
4:00
8:00
12:00
16:00
20:00
0:00
Time, hours

CHAPTER 6
Capacity Analysis

Chapter 6 presents the results of the analysis of available hydraulic capacity within the City's collection system, under wet weather conditions. The analysis reviews the ability of the system to convey flows without sanitary sewer overflows (SSOs) related to capacity under a designated design storm rainfall scenario.

6.1 HYDRAULIC CAPACITY ANALYSIS RESULTS

The City's modeled collection system network was evaluated for its capacity to convey flows that are predicted to occur during a design storm event. The analysis was completed using a design storm with a published recurrence interval of 10 years and duration of 24 hours (10-year, 24-hour design storm). The design storm and other hydraulic evaluation criteria are discussed further in Chapter 5, Planning Criteria.

Analyses were conducted as follows:

1. The system was evaluated for its ability to meet the Hydraulic Grade Line (HGL) criteria (*i.e.*, water level relative to the manhole rim or ground elevation) described in Chapter 5. Pump stations and pipelines with a capacity deficiency that triggered the HGL criteria were flagged.
2. Projects were developed to address the capacity issues within one pump station and several pipelines. Projects included upsizing pipes to meet the required planning criteria and/or introducing relief sewers to convey the excess capacity need to eliminate the predicted SSOs.

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

6.1.1 Capacity Analysis Results – Total System Flows at the WPCP

The model predicted locations within the collection system where the HGL exceeded the ground surface. The elevated HGL was caused by one of two factors: 1) pipeline capacity deficiency at the location of the elevated HGL or 2) lack of capacity downstream of the area in question, causing the water surface to become elevated further upstream. For example, the contractual capacity of the WPCP is lower than influent flows during some wet weather events. In this case, water is first stored at the WPCP facility, and when this storage capacity is utilized, is stored within the collection system upstream of the WPCP.

The City's hydraulic model predicts peak hourly flow during the design storm event of 17.0 million gallons per day at the WPCP, which has a contractual capacity of 9.0 mgd. Due to the sharply defined rainfall distribution curve that is required for the design storm, this peak flow is predicted to occur briefly, and then quickly recede. However, because of the flat topography near the WPCP, even this brief period of excess flow results in surcharging in the upstream gravity mains. The flow hydrograph resulting from the design storm at the WPCP can be seen on Figure 6-1. The hydrograph utilizing the storage available in the flow equalization basin is also shown on the figure. As can be seen, the existing flow equalization reduces the peak flow to below 14 mgd. However, because this peak exceeds the contractual limit of 9.0 mgd, further equalization storage is required for design storm conditions. The location and size of the required storage is described later in this chapter. Presuming that the City will implement the recommended storage in conjunction with the proposed pipeline capacity improvements, a new hydraulic analysis was conducted for the collection system upstream of the WPCP assuming that all predicted peak flows exceeding 9.0 mgd will be handled through existing and future storage and that the HGL at the WPCP will not create an elevated HGL upstream of the plant.

6.1.2 Capacity Analysis Results - Pump Stations

The City's three collection system pump stations were included in the hydraulic model. The model predicts that the Madrone Pump Station exceeds its current firm capacity (*i.e.*, pumping capacity with the largest pump out of service) during the design storm. The remaining pump stations have sufficient firm capacity to handle peak flows from the design storm.

The strategy for improving the performance at the Madrone Pump Station under design storm conditions is discussed later in this chapter. Presuming that Madrone Pump Station capacity improvements will be completed in conjunction with upstream gravity pipeline capacity improvements, a new hydraulic model analysis for the gravity mains upstream of the Madrone Pump Station was conducted assuming that the PWWF under design storm conditions will not be impacted by an elevated HGL at the pump station.

6.1.3 Capacity Analysis Results - Pipelines

The hydraulic model identified six areas where the projected HGL may exceed the ground surface elevation at manhole locations during the design storm event due to lack of pipeline capacity at or downstream of the location of the predicted SSO. These areas, described as system bottlenecks, are shown on Figure 6-2 and discussed below.

6.1.3.1 Conveyance to Madrone Pump Station

In addition to the Madrone Pump Station being under capacity for the design storm, as discussed above, the pipelines in the basin contributing flow to Madrone Pump Station are under capacity in several areas. These under-capacity pipelines include the pipes that cross beneath the BART tracks and sound wall upstream of the pump station, as well as pipelines farther upstream in the area of Capuchino High School.

6.1.3.2 Murchison Drive and Aviador Avenue

Another bottleneck area is located in the area of Murchison Drive and Aviador Avenue. Capacity constraints on the flat lines of Aviador Avenue lead to upstream areas where the HGL exceeds the ground level. Additionally, some pipes farther up Murchison Avenue are predicted to be undersized for the design storm and will require capacity increases.

6.1.3.3 Highline Canal Right of Way

The majority of the City's wastewater flow is conveyed to the 18-inch diameter and 12-inch diameter pipelines that are parallel to the Highline Canal, before consolidating and carrying flow under Highway 101 and into the WPCP. There is insufficient capacity in these parallel lines to carry the design storm, and the resulting bottleneck leads to predicted HGL values above ground level not only in these pipelines, but also in upstream pipelines in Aviador Avenue, in the Victoria Avenue easement that extends back to the Hillcrest area, and in some adjacent local pipelines.

6.1.3.4 Broadway Avenue and Magnolia Avenue

Pipelines in the Broadway Avenue/Magnolia Avenue/Taylor Boulevard area are predicted to have insufficient capacity during the design storm, especially where the pipelines in this area enter El Camino Real. A combination of capacity increases in this area and in El Camino Real will be required in order to maintain predicted HGL values below ground level.

6.1.3.5 Helen Drive and Richmond Drive

Pipelines in Helen Drive and Richmond Drive serve a large residential area that flows to El Camino Real. The hydraulic model predicts HGL values above ground level for stretches of pipeline in both Helen Drive and Richmond Drive, as well as pipelines in El Camino Real where these two streets enter.

6.1.3.6 El Camino Real

The gravity main pipelines that run along the length of El Camino Real are under capacity during design storm conditions. The lack of capacity extends from the north where the Helen Drive pipelines enter El Camino Real to the south where the Victoria Ally easement pipelines enter El Camino Real. In addition to hydraulic limitations, several areas of flow splits and flow convergences lead to complex local flow conditions are may increase the potential for SSOs. In addition to increasing the capacity of the pipelines, configuration modifications will help to alleviate local turbulence and backwater conditions in order to maintain HGL values below ground surface levels in this area.

6.2 RECOMMENDED PROJECTS

Eight projects are recommended to address storage, pump station capacity needs, and the areas in the sanitary sewer collection system that are predicted to have SSOs during the design storm. The City will continue to study the feasibility of these projects after completion of the CAR. There will be an iterative process of refining the CAR as the feasibility analysis continues. The City reserves the right to adopt alternative capital improvements to address system capacity.

6.2.1 Wet Weather Storage at Corporate Yard

As described above, the PWWF that is seen at the WPCP during design storm conditions approaches 17.0 mgd, which exceeds the 9.0 mgd contractual capacity of the WPCP plant. This peak flow exceeds 9.0 mgd even with the full 1.3 million gallons of existing flow equalization storage in use. The recommended solution is for the City to construct additional wet weather storage to hold the remaining peak flows (after initial storage is utilized) and release them over time after the peak has passed. Included with this storage project would be the associated piping and pumping required to convey wet weather flows to and from the storage facility.

Space for storage is constrained at the WPCP. Therefore, alternative storage locations were considered for this CAR. The City owns a large open triangular-shaped lot between Aviador Avenue, the Highline Canal and BART tracks that is currently being used as a storage yard for municipal equipment. This yard is approximately 1.5 acres in size, and has the advantage of containing sufficient area to construct wet weather storage, of being near trunk lines in the collection system, and of being sufficiently close to the WPCP that enough flow can be removed from the system in order to reduce peak flow at the WPCP below 9.0 mgd. The hydraulic model predicts that 0.43 million gallons of storage are required for a single design storm. In order to accommodate consecutive storms, or to accommodate rainfall patterns that are distributed over a longer period than the design storm, it is recommended that the City build 0.9 million gallons of storage at the City-owned lot. The location of the recommended project is shown in Figure 6-3.

Construction of storage should precede completion of the capacity improvements further upstream in the system. The project will require preliminary design as needed to obtain environmental clearance. Therefore, planning activities for this project should be initiated as soon as possible after the adoption of this Capacity Assurance Report.

6.2.2 Madrone Pump Station Replacement and Upstream Conveyance System Improvements

The hydraulic model predicts that the Madrone Pump Station is hydraulically insufficient for the design storm conditions. The firm capacity of the pump station is approximately 2.5 mgd, and the peak flow predicted at Madrone Pump Station is approximately 6.0 mgd. The gravity mains entering the pump station also have insufficient capacity for the design storm, and rank highly in relative risk of failure in the Risk Management Model that is discussed in Chapter 7, Pipeline Rehabilitation and Replacement Program. The existing pipelines are located in private easements and run through parcels through which the Federal Aviation Administration (FAA) restricts access. Additionally, the force main that extends nearly 5,000 lineal feet from the Madrone Pump Station to the collection system near the WPCP is very difficult to access for regular maintenance and condition assessment.

The recommended solution to resolve capacity issues at the Madrone Pump Station was developed in conjunction with City staff to address both the hydraulic capacity deficiency of the pump station and the upstream gravity mains while improving facility operations and maintenance. Although upsizing the pump station, gravity mains, and force main in place would solve the capacity needs, this solution would not improve facility access. The recommended solution will relocate the Madrone Pump Station to a City-owned right-of-way on Oak Street north of Center Street. Gravity flow that is currently conveyed to the Madrone Pump Station will be intercepted east of Landing Lane, before crossing under the BART tracks, and conveyed via gravity flow to the new pump station. The forcemain from the new pump station will be located within an extension of an existing easement through San Francisco Public Utilities Commission (SFPUC) property between Oak Street and Hermosa Avenue. At Hermosa Avenue, the forcemain will enter the public right of way and turn southwest down Hermosa Avenue before tying into the gravity main on El Camino Real.

With the flow intercepted before Madrone Pump Station as described above, the only wastewater flow reaching Madrone Pump Station will be from the small neighborhood located between Santa Paula Avenue and Madrone Street, and Bay Street and Monterey Street. Madrone Pump Station would be downsized to serve as a neighborhood pump station. Flow would be re-routed to the southeast through a new, 6-inch diameter forcemain that is slipped into the existing gravity main that serves Madrone Pump Station. The re-routed forcemain would connect with the interception point for the new pump station and the existing forcemain would be abandoned. The location and components of this recommended project are shown on Figure 6-4.

6.2.3 Pipeline Replacements Near Capuchino High School

This project addresses the capacity issues that are predicted for the pipelines to the south and to the east of Capuchino High School. The recommended project consists of increasing the diameter of approximately 3,000 feet of pipeline to 12-inch and 18-inch diameter. The recommended project location is shown on Figure 6-5.

6.2.4 Pipeline Replacement at Aviador Avenue and East Millbrae Avenue

This project addresses the bottleneck on Aviador Avenue that creates capacity issues further upstream, to Murchison Drive. The recommended project consists of removing and replacing approximately 1,250 feet of 12-inch diameter pipeline on Aviador Avenue with 18-inch diameter pipeline. The replacement will run from Adrian Road under East Millbrae Avenue. The recommended project location is shown on Figure 6-6.

6.2.5 Pipeline Replacement in Murchison Drive

This project consists of removing and replacing approximately 1,600 feet of 10-inch diameter pipeline in Murchison Drive with 15-inch diameter pipeline. The replacement will run from west of Magnolia Avenue to California Drive adjacent to the BART tracks. The recommended project location is shown on Figure 6-7.

6.2.6 Pipeline Replacement Along Highline Canal Right-of-Way

This project addresses the capacity constraints that are predicted in the 18-inch and 12-inch pipelines that are parallel to the Highline Canal. The recommended project will abandon the 18-inch pipeline on the southern side of the canal and increase the diameter of the 12-inch diameter pipeline on the northern side of the canal to 36-inches in diameter. The project extends west of El Camino real into the Victoria Alley, with the improvement of 6-inch diameter pipelines to 12-inch diameter. This project will simplify flow and maintenance conditions while providing needed capacity. Wastewater flow entering the parallel lines from Aviador Avenue, as well as from local neighborhood lines, will be reconfigured to flow into the single 36-inch diameter line. The recommended project is shown on Figure 6-8.

6.2.7 Pipeline Replacement in Anita Drive and Richmond Drive Near El Camino Real

This project is designed allow the gravity mains in Anita Drive and Richmond Drive to carry the design storm flows away from the Helen area and to El Camino Real. The recommended project consists of upsizing approximately 3,500 feet of 8-inch to 10-inch diameter pipeline to 12-inch diameter pipeline. The location and extent of the project is shown on Figure 6-9.

6.2.8 Pipeline Replacement in El Camino Real

This project is designed to allow the design storm flows to be conveyed down El Camino Real before they turn east parallel to Highline Canal and then to the WPCP. In addition to increasing the capacity in El Camino Real, the project is intended to remove the parallel line that runs between Richmond Drive and Hermosa Avenue, and decrease the amount of flow that splits off of El Camino Real at Hermosa Avenue. Through this improvement, the project will reduce the amount of flow in the Aviador Avenue relief sewers, and will streamline the flow transitions coming from the west into El Camino Real in order to remove local flow disruptions. The recommended project consists of upsizing approximately 3,500 feet of 10-inch to 15-inch pipelines to 18-inch and 24-inch pipelines, and reconfiguring manholes and flow splits during the replacement. The recommended project extent and location are shown on Figure 6-10.

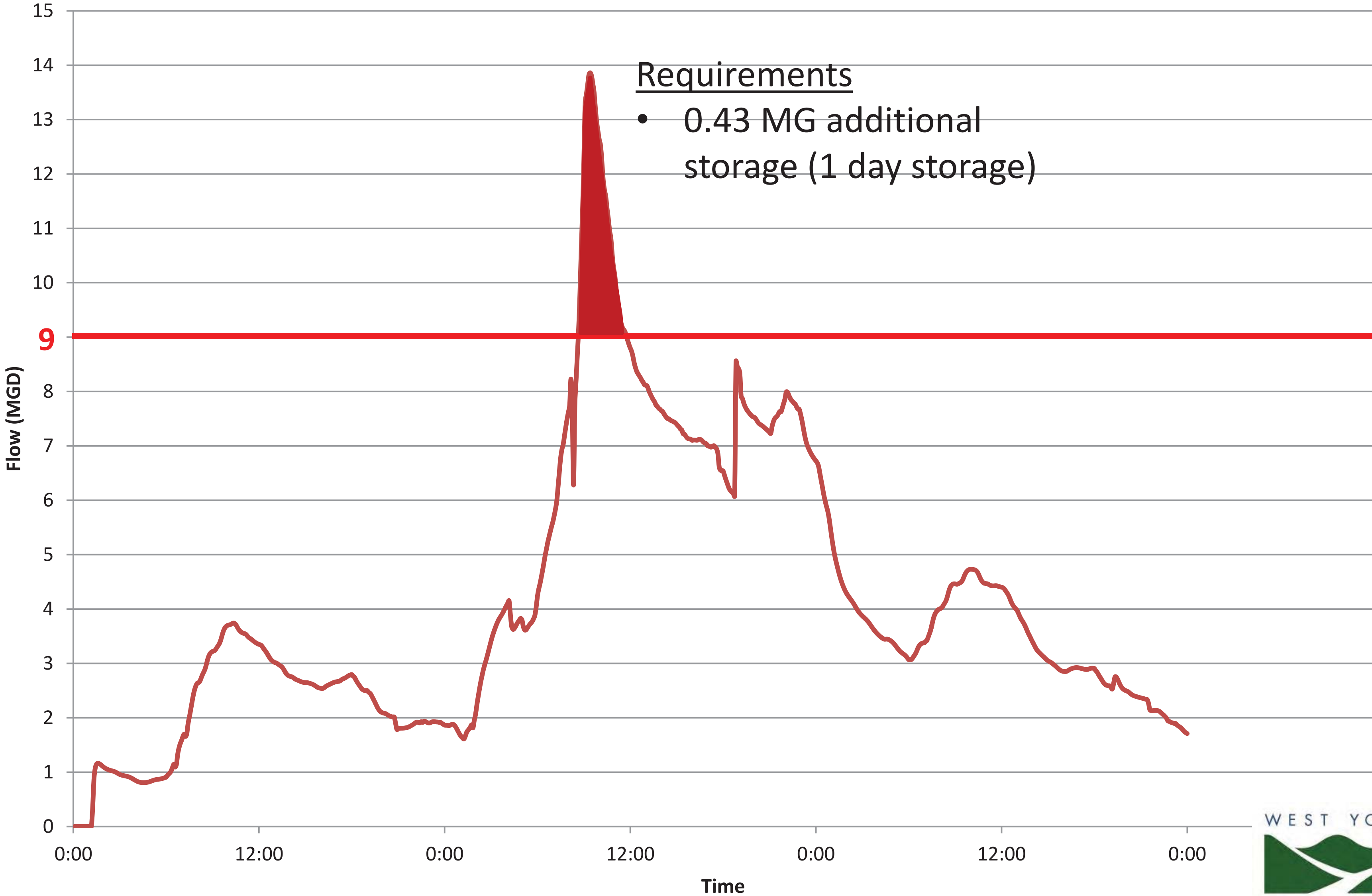
6.3 PROJECT COSTS AND IMPLEMENTATION SCHEDULE

Conceptual costs for the proposed projects are discussed in further detail in Chapter 8. Project costs and their anticipated timeline for implementation are summarized in Table 6-1. Detailed project cost estimates can be seen in Appendix G.

Table 6-1. Capacity Improvement Projects

Project Name	Description	Conceptual Cost, $M	Implementation Timeline
Wet Weather Storage at Corporate Yard	Construction of 0.9 MG of wet weather storage at corporate yard, with associated entry piping and exit pumping.	2.76	2014
Madrone Pump Station Replacement and Upstream Conveyance System Improvements	Relocation of the Madrone Pump Station to a City-owned right-of-way on Oak Street north of Center Street.	7.26	2016
Pipeline Replacements Near Capuchino High School	Upsizing of approximately 3,000 of 8-inch and 10-inch pipeline to 12-inch and 18-inch.	0.85	2016
Pipeline Replacement at Aviador Avenue and East Millbrae Avenue	Replacement of 1,250 feet of 12-inch pipeline with 18-inch pipeline.	0.77	2016
Pipeline Replacement in Murchison Drive	Replacement of 1,600 feet of 10-inch diameter pipeline with 15-inch diameter pipeline.	0.50	2015
Pipeline Replacement Along Highline Canal Right-of-Way	Replacement of the parallel 18-inch and 12-inch pipelines along the canal with a single 36-inch diameter pipeline.	2.04	2015
Pipeline Replacement in Anita Drive and Richmond Drive Near El Camino Real	Replacement of approximately 3,500 feet of 8-inch diameter and 10-inch diameter pipeline with 12-inch diameter pipeline	0.89	2016
Pipeline Replacement in El Camino Real	Replacement of approximately 3,500 feet of 10-inch diameter to 15-inch diameter pipelines with 18-inch and 24-inch pipelines	3.13	2014

Figure 6-1. City of Millbrae Wet Weather Storage Needs Assuming Existing Storage Capacity is Available



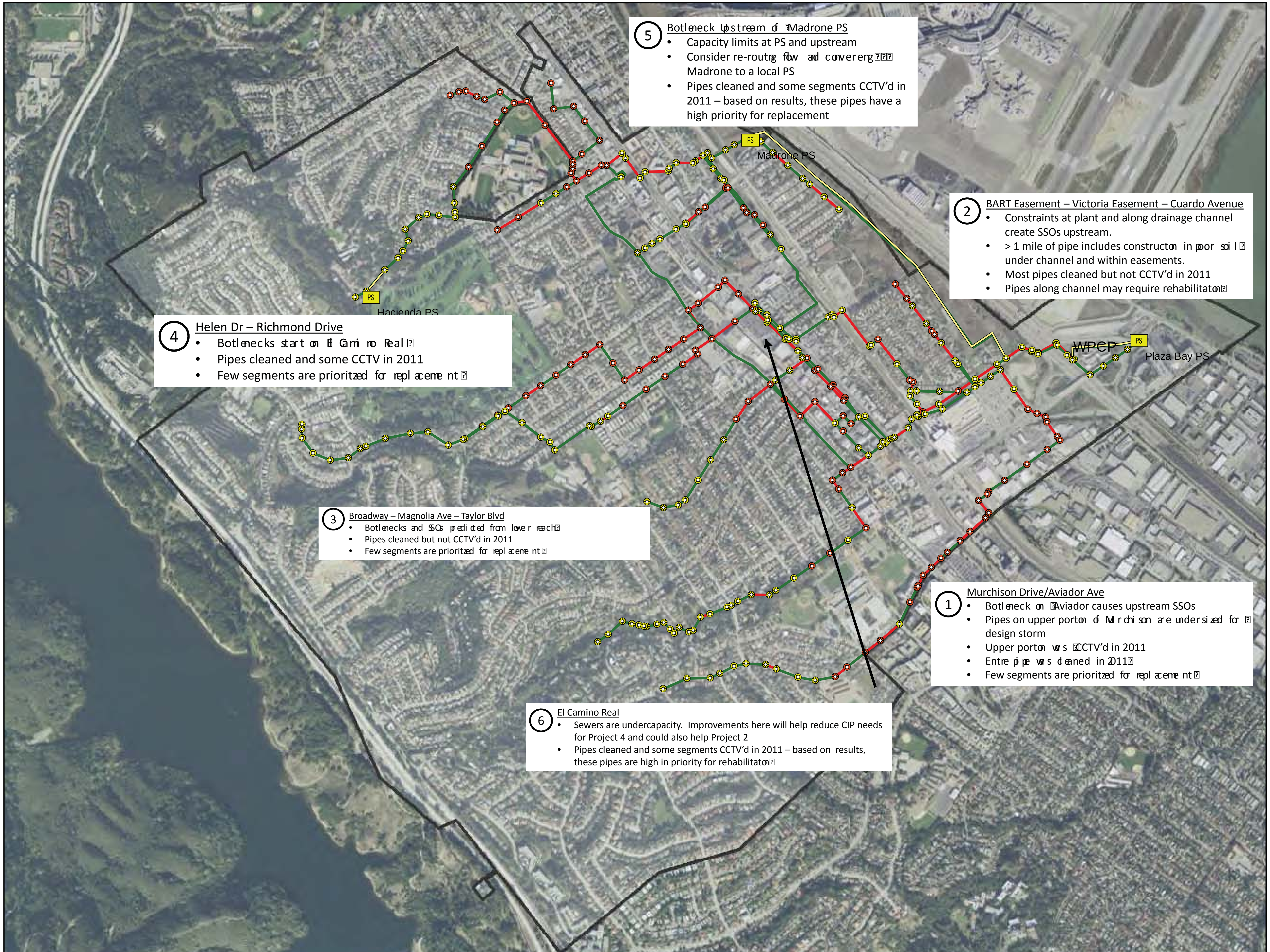


FIGURE 6-2

City of Millbrae
Capacity Assurance Report

COLLECTION SYSTEM BOTTLENECKS

N

0 625 1,250

Scale in Feet

1 inch = 1,250 feet

LEGEND

- Predicted HGL Below Rim
- Predicted Flooding
- Force Main
- Gravity Main with Sufficient Capacity
- Gravity Main with Insufficient Capacity
- City Boundary

SEAL OF THE CITY OF MILLBRAE CALIFORNIA

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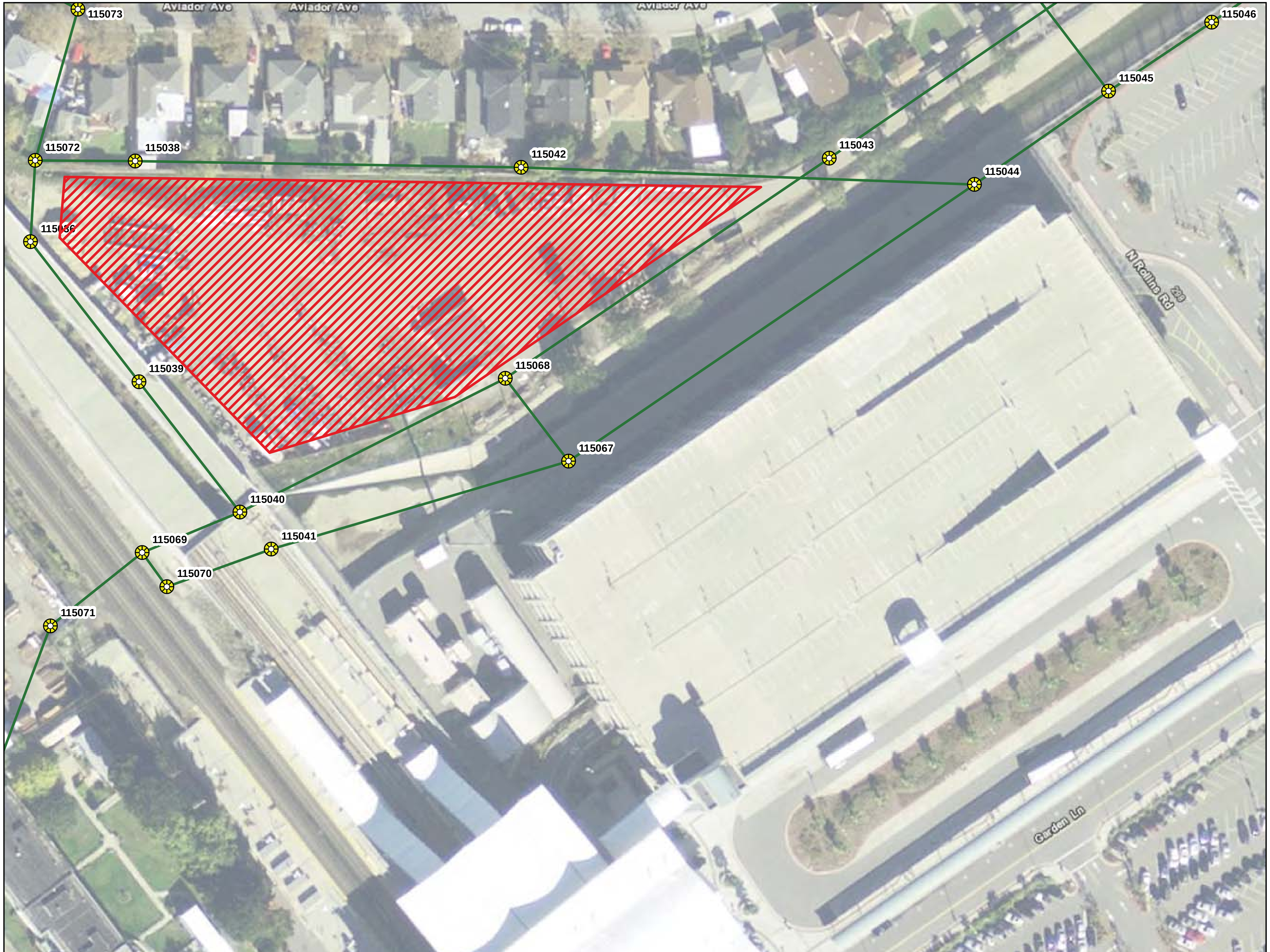
FIGURE 6-3

City of Millbrae
Capacity Assurance Report

RECOMMENDED WET WEATHER STORAGE LOCATION



N

0 37.5 75

Scale in Feet

1 inch = 75 feet

LEGEND

- Manhole
- Pump Station
- Gravity Main
- Force Main
- Potential Project Location
- City Boundary

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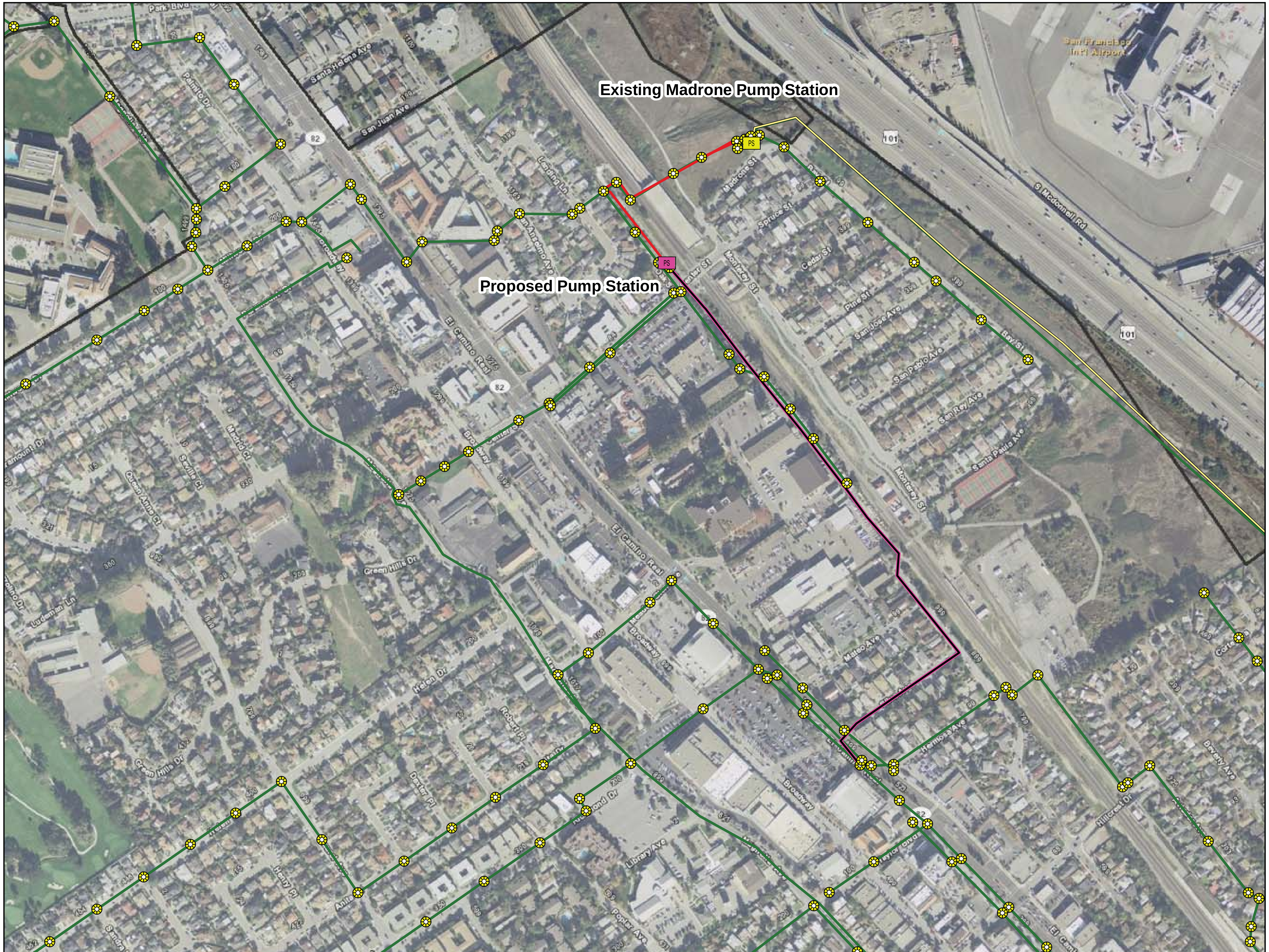

FIGURE 6-4

City of Millbrae
Capacity Assurance Report

RECOMMENDED IMPROVEMENTS TO MADRONE PUMP STATION

N

0 200 400
Scale in Feet

1 inch = 400 feet

LEGEND

- Manhole
- Existing Pump Station
- Proposed Pump Station
- Existing Force Main
- Existing Gravity Main
- Proposed Forcemain
- Proposed Downsized Forcemain
- City Boundary

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FIGURE 6-5

City of Millbrae
Capacity Assurance Report

RECOMMENDED PIPELINE REPLACEMENTS NEAR CAPUCHINO HIGH SCHOOL

N

0 100 200
Scale in Feet

1 inch = 200 feet

LEGEND
- Manhole
- PS Pump Station
- Proposed Gravity Main Upsize
- Gravity Main
- Force Main
- City Boundary

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FIGURE 6-6

City of Millbrae
Capacity Assurance Report

RECOMMENDED PIPELINE REPLACEMENTS AT AVIADOR AVENUE AND EAST MILLBRAE AVENUE

0 50 100
Scale in Feet

1 inch = 100 feet

LEGEND

- Manhole
- Pump Station
- Proposed Gravity Main Upsize
- Gravity Main
- Force Main
- City Boundary

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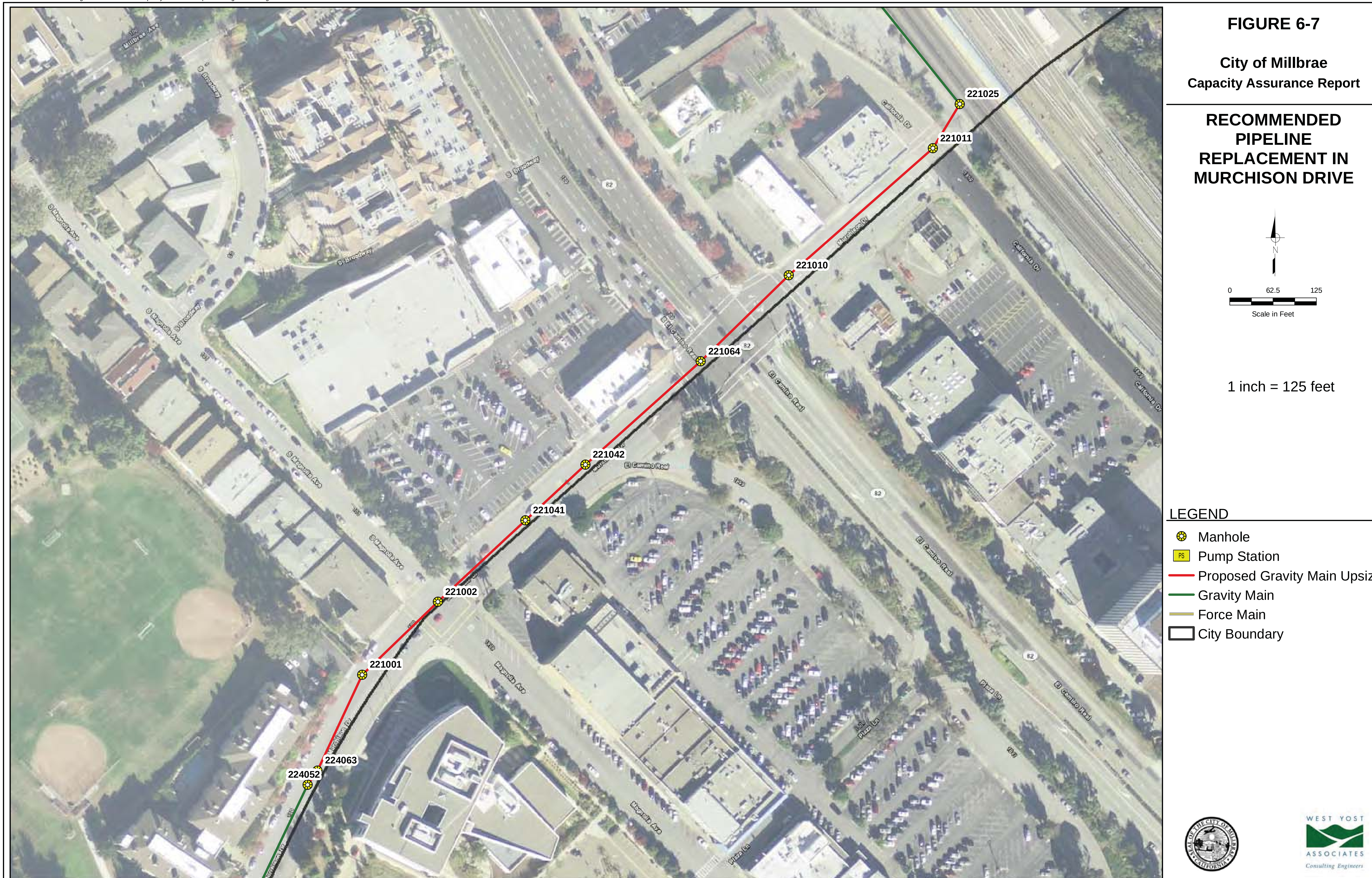

FIGURE 6-7

City of Millbrae
Capacity Assurance Report

RECOMMENDED PIPELINE REPLACEMENT IN MURCHISON DRIVE

1 inch = 125 feet

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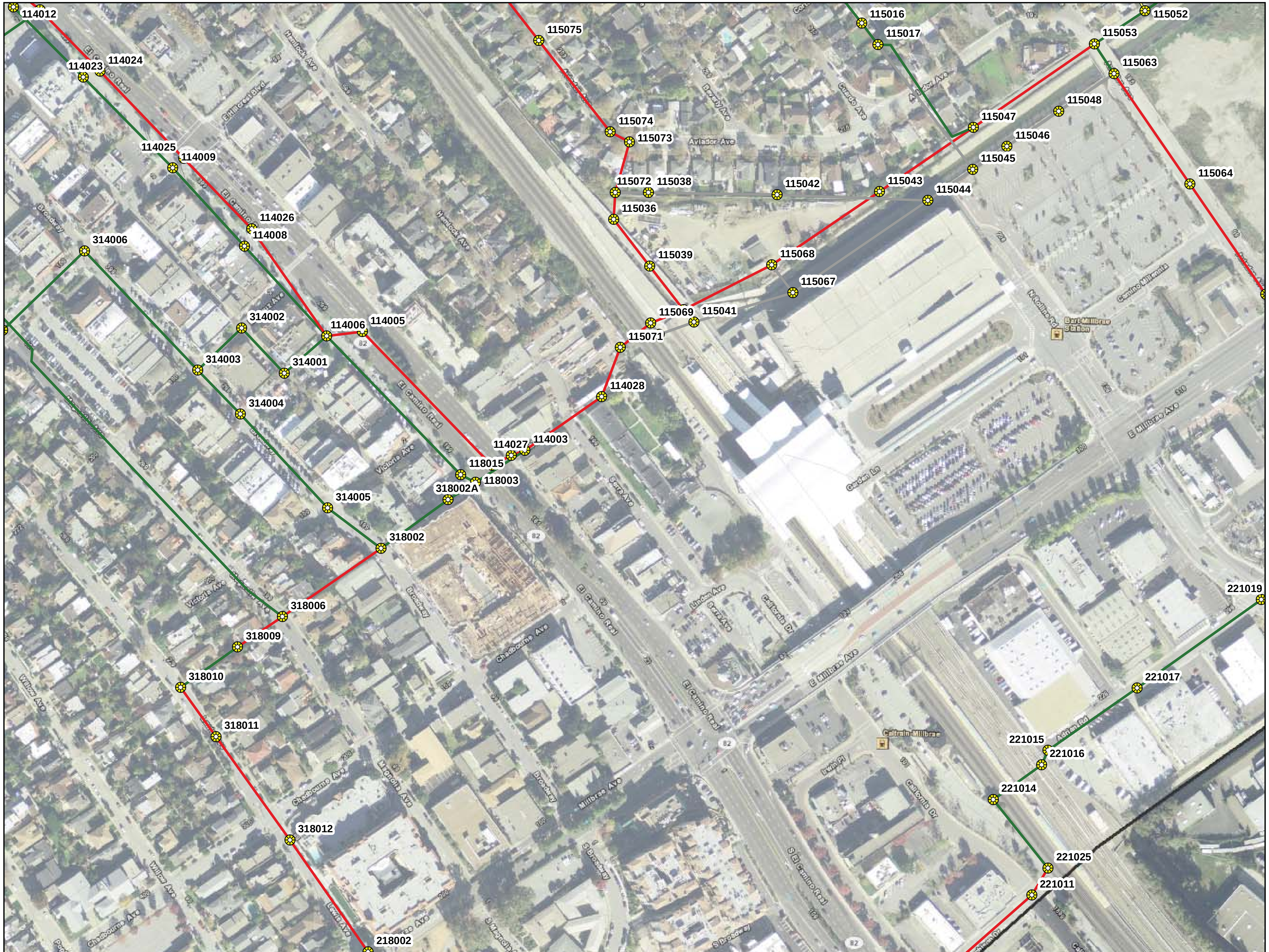

FIGURE 6-8

City of Millbrae
Capacity Assurance Report

RECOMMENDED PIPELINE REPLACEMENT ALONG HIGHLINE CANAL RIGHT-OF-WAY

N

0 117.5 235

Scale in Feet

1 inch = 225 feet

LEGEND

- Manhole
- PS Pump Station
- Force Main
- Proposed Abandoned Gravity Main
- Proposed Gravity Main Upsize
- Gravity Main
- City Boundary

SEAL OF THE CITY OF MILLBRAE CALIFORNIA

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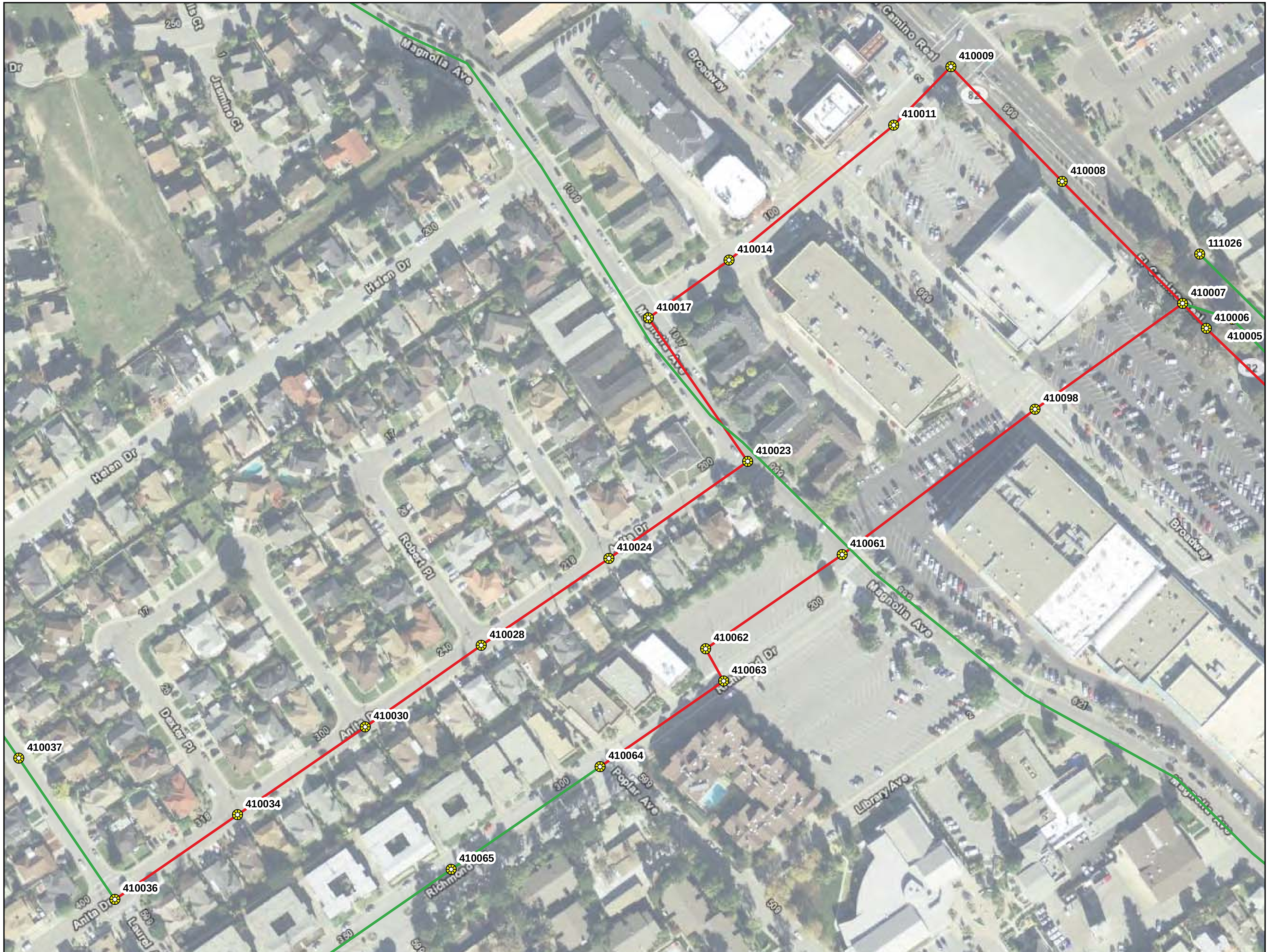
FIGURE 6-9

City of Millbrae
Capacity Assurance Report

RECOMMENDED PIPELINE REPLACEMENT IN ANITA DRIVE AND RICHMOND DRIVE NEAR EL CAMINO REAL



0 75 150
Scale in Feet

1 inch = 150 feet

LEGEND

- Manhole
- Pump Station
- Force Main
- Proposed Gravity Main Upsize
- Gravity Main
- City Boundary

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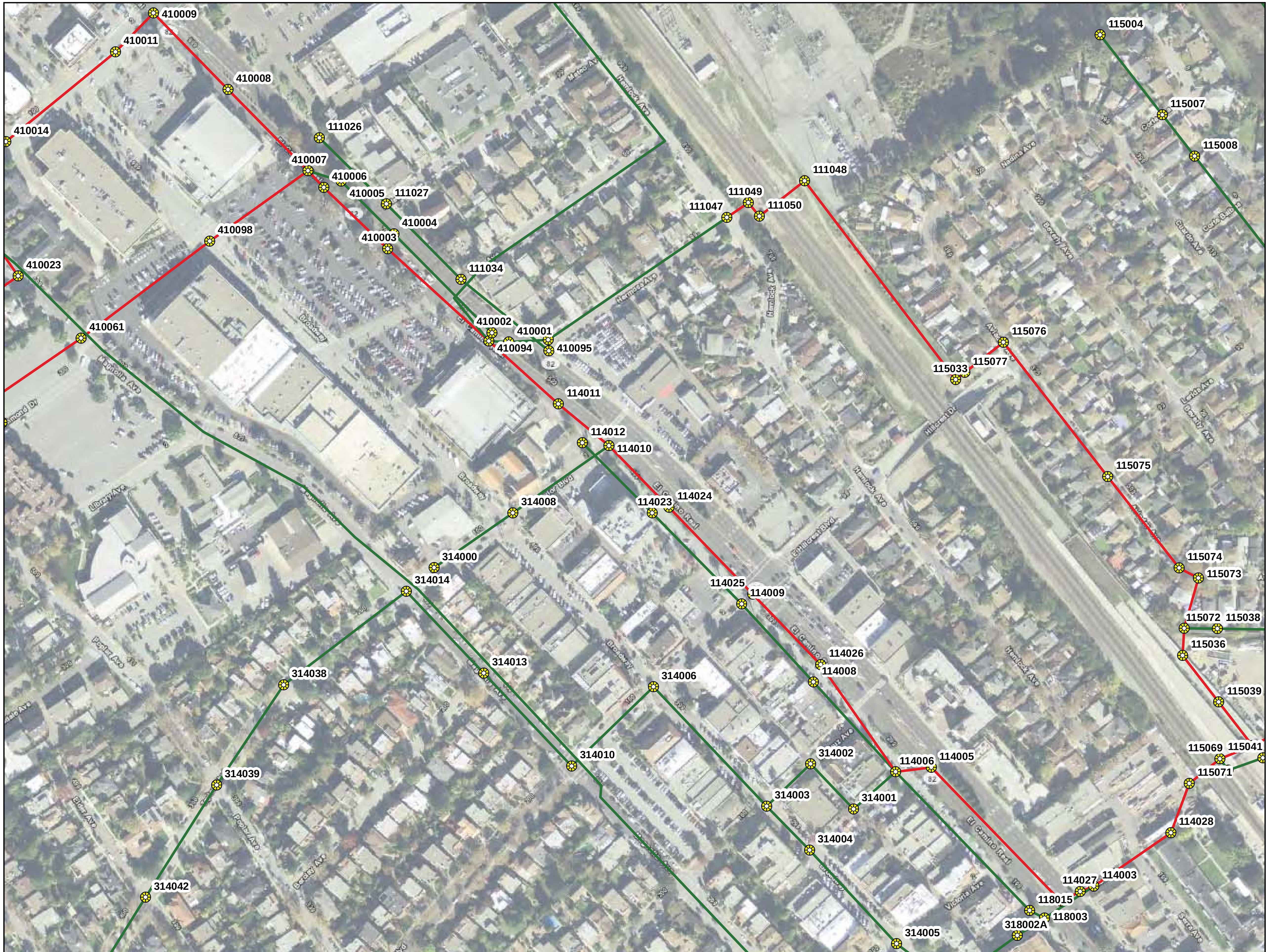

FIGURE 6-10

City of Millbrae

Capacity Assurance Report

RECOMMENDED PIPELINE REPLACEMENT IN EL CAMINO REAL

N

0 112.5 225

Scale in Feet

1 inch = 225 feet

LEGEND

- Manhole
- PS Pump Station
- Proposed Gravity Main Upsize
- Gravity Main
- Force Main
- City Boundary

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Pipeline Rehabilitation and Replacement Program

7.1 INTRODUCTION

The primary purpose of the City's Capacity Assurance Report is to identify system bottlenecks that result from a design storm event and propose recommendations for needed capacity improvements. This chapter supplements the capacity assessment with a proposed approach for near-term pipeline rehabilitation and replacement (R&R) planning. Using this approach, the City has developed an initial list of R&R projects for implementation in conjunction with capacity improvements.

This chapter is organized as follows:

- Background and Summary,
- Risk Management Model (RMM),
- RMM Results, and
- Recommended Projects , Costs and Timelines.

7.2 BACKGROUND AND SUMMARY

Until 2016, as required by the Consent Decree between the City and San Francisco Baykeeper, the City must focus on reducing the number of sanitary sewer overflows (SSOs) that occur in the defined design storm event. The design storm is discussed further in Chapter 5, Planning Criteria. In response to this requirement, the City has developed an initial, prioritized list of R&R projects for completion in parallel with or as part of planned capacity assurance projects.

The initial R&R program includes proposed repair and CCTV inspection projects to be implemented before 2016. This project list relied on available CCTV inspection results as a primary indicator of likelihood of failure. Where CCTV data were not available, the assessment was based on age, O&M history and geologic setting. As the City completes additional CCTV inspection of the system, the information gained is likely to require adjustments to the likelihood of failure assessment and associated project priorities.

The initial project list was developed with the primary objective of minimizing the number of SSOs, as required by the Consent Decree. The City plans to continue its R&R program after the Consent Decree compliance period concludes. This project list does not include immediate pipe segment point repair needs which are being tracked and implemented on an ongoing basis by City O&M staff.

Analysis of this information yielded six sewer pipe segments that are recommended for replacement in FY2012/2013. In addition, eleven pipe segments are identified as being high priority for CCTV inspection in FY2012/13. CCTV inspection results will determine whether these segments should be added to the prioritized replacement list. Because some of these pipe segments are also identified as having capacity constraints, the CCTV inspection results will help facilitate project planning for the associated capacity improvement projects.

The remaining pipe segments were listed in ranked order based on various likelihood of failure factors, with emphasis on the estimated average age of original homes within the vicinity of the pipe segment. This list was developed to assist with ongoing CCTV inspection planning. The prioritized list includes the Hillcrest Avenue and Hawthorne Boulevard neighborhoods, which are scheduled for inspection in 2012. This inspection program and a placeholder for potential projects that are identified as a result of this program are also included in in Table 7-1. Project locations are shown in Figure 7-1.

The City has sufficient budget to complete the projects that are proposed for FY2012/13, and will complete the line segment replacements and priority pipeline inspections by June 2013. The Hillcrest Avenue and Hawthorne Boulevard CCTV inspection program will be conducted in summer 2012.

Table 7-1. Pipeline Rehabilitation and Replacement and CCTV Inspection Projects Planned for FY2012-2013

Project Name	Description	Conceptual Cost
Priority Line Segment Replacements	• Six line segments • 6-inch diameter • Combined length of 1,425 lf • Structural PACP score of 53XX • Some pipes have chronic O&M issues	$265,000
CCTV 11 priority pipe segments	• Combined length of 1,807 lf • High potential for defects based on O&M needs and age • Many segments are also associated with needed capacity improvements	$3,650
CCTV Hillcrest and Hawthorne neighborhood pipelines	• Approximate length of 20,000 lf • These neighborhoods experienced a high number of lower lateral SSOs in 2011	$40,000
Hillcrest and Hawthorne area and other pipeline replacements	• This project is included as a placeholder to address immediate pipeline rehabilitation and replacement needs that are identified through the planned CCTV inspections	$360,000

In addition to the projects shown, the CIP includes a placeholder in each subsequent year through FY2017 for rehabilitation and replacement projects that will be identified over time through the City's CCTV inspection program. This placeholder includes $575,000 in FY2013-14, and $650,000 in each of the subsequent fiscal years. It should be noted that two of the projects identified in this process, the projects by Taylor School and in Castaneda Ave. have recently been repaired by the City as part of ongoing rehabilitation and repair.

7.3 RISK MANAGEMENT MODEL (RMM)

The primary objective of evaluating gravity pipeline risk was to identify pipe segments that have the highest potential to cause an SSO, as required by the Consent Decree. The City utilized a risk-based prioritization tool, named the RMM, to complete this analysis. The RMM, developed by West Yost using Microsoft® (MS) Access, uses a numerical algorithm to evaluate initial risk in the context of Likelihood of Failure. The RMM also has the capability to refine priorities using parameters related to Consequence of Failure. These additional parameters were not put into effect here because the focus of the Consent Decree is to identify pipe segments that have the highest potential to cause an SSO.

The RMM utilizes the tables, forms and formulas that are provided within the MS Access user interface. Through this process, the contents, use and functionality of the RMM are easily understood by a user who is proficient in MS Access, and use of the RMM requires a general understanding of Microsoft® Office tools without specific knowledge of MS Access. Also, viewing and updating the RMM components can be achieved without specialized programming expertise.

7.3.1 Sewer Collection System Asset Data

The City maintains a record of sewer system assets and maintenance activities in the Asset Information Management System (AIMS) database. The AIMS database is built on an MS Access platform, and includes physical asset information, CCTV inspection condition ratings, maintenance results, and SSO data. West Yost exported AIMS asset data to a GIS database, where the data was augmented for use in determining risk.

Each asset comprises a single pipe segment spanning from manhole to manhole. The City's 1,647 gravity pipeline assets were supplemented with publicly available information on average home age (gained through recent home sale information) and geotechnical conditions (obtained from U.S. Geologic Survey liquefaction maps). Pipe segment crossings that were located within ten and 100 feet of a waterway, commercially zoned area, and major transportation corridor were also considered.

Asset information from AIMS and other sources that were used for the City's risk assessment is shown in Table 7-2.

Table 7-2. Asset Data Used for Pipeline Risk Analysis

Asset Description	RMM Field Name	Source
Upstream Node or Manhole Identification Numbers(ID)	us_node_id	AIMS Database
Downstream Node or Manhole ID	ds_node_id	AIMS Database
Asset Number (Upstream ID – Downstream ID)	assetno	Derived for RMM
Asset Length	Length	AIMS Database
Pipe Material	Material	AIMS Database
Year Installed[a]	YIS	Public Data
Geology[b]	Geo	Public Data
Proximity to Waterway[c]	Water	Public Data
Proximity to Commercial Districts[d]	Commerce	Land Use Maps
Proximity to Transportation Corridors[c]	Tran	Public Data
Operations and Maintenance[e]	OandMIssues	AIMS Database

(a) Year installed derived from publicly available home sales information.
(b) Geology based on U.S. Geologic Survey liquefaction maps in GIS.
(c) Waterways and transportation corridors identified through aerial photographs.
(d) Commercial Districts identified through land use planning designations in GIS.
(e) Operations and maintenance based on hot spot cleaning work plan schedule.

The RMM includes a separate table that supplements the information described in Table 7-2 with theoretical service life predictions that are shown in Table 7-3. Pipes that were not assigned a pipe material in the City's AIMS database were assumed to have a theoretical service life of 50 years.

Table 7-3. Average Theoretical Service Life Estimations for Pipe

Material	Average Useful Life
Cast Iron (CIP)	50 to 70 Years
Ductile Iron (DIP)	50 to 70 Years
High Density Polyethylene (HDPE)	50 to 70 Years
Poly Liner (PL)	50 Years
Polyvinyl Chloride (PVC)	70 to 90 Years
Reinforced Concrete Pipe (RCP)	50 Years
Vitrified Clay (VCP)	70 to 90 Years
Unknown	50 Years

The RMM also utilizes CCTV inspection results that are stored as separate MS Access tables in the AIMS database. These tables provide condition rating information in the form of National Association of Sewer Service Companies (NASSCO) Pipeline Assessment Condition Program (PACP) Quick Ratings. The Quick Rating system provides a code that summarizes the number of the two highest (worst) condition ratings found in any pipe segment (defined as spanning from manhole to manhole). The codes are differentiated by structural defects and operational & maintenance defects, and provide information for both the defect rating and number of occurrences. For example, a Structural Quick Rating of 5343 conveys that the segment has three Grade 5 structural defects and three Grade 4 structural defects. The defects and their individual numerical coding are assigned through the NASSCO PACP program, and are assigned by NASSCO-certified technicians using NASSCO-compliant inspection equipment.

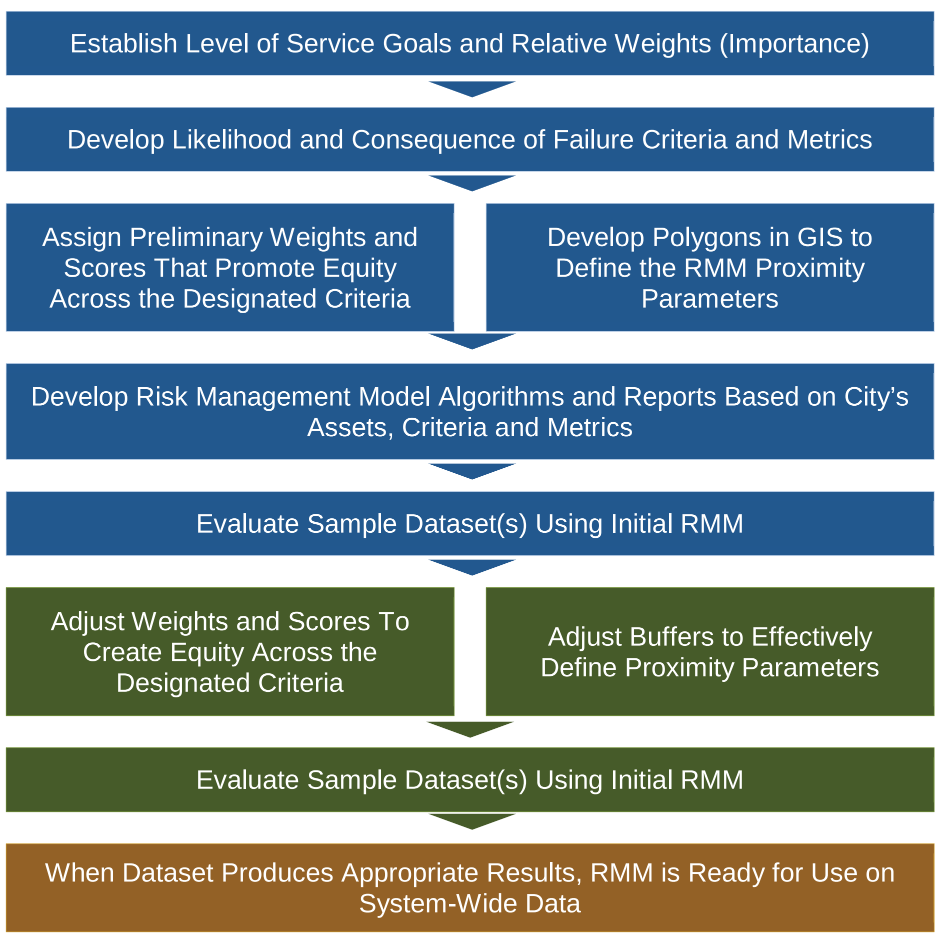
7.3.2 Risk Management Model Components

The RMM includes the following MS Access tables and queries:

- Level of Service Goals;
- Likelihood and Consequence of Failure Metrics, Importance Factors, Weights and Scoring; and
- Component Scores and Total Risk Score.

These tables and queries were refined through an iterative process that is shown in Figure 7-2.

Figure 7-2. Risk Management Model Development Process



7.3.2.1 Level of Service Goals

Level of Service Goals (LOS goals) define the RMM Consequence of Failure parameters and are not applicable to the current prioritization effort. The current effort was focused on meeting the overarching objective of meeting SSO performance requirements established in the Consent Decree.

The assessment of Risk and Criticality in a balanced risk assessment is benchmarked against whether and to what extent the failure of an asset will impact the City's ability to provide an acceptable level of service to its customers. In order for this assessment to be made, the parameters defining service must be defined and prioritized.

There are many ways to define Level of Service. For the purposes of this project and the RMM, Level of Service is defined from the customer's viewpoint. This perspective creates LOS goals that are broad and policy-based (external). The criteria and metrics that the City must meet in order to achieve the desired Level of Service frame the City's operational and maintenance priorities (internal). As an example of how Level of Service is defined, the customer may want the City to prevent SSOs on their street. However, they may not be concerned about the City's process for prioritizing CCTV inspection or cleaning. The City, on the other hand, is acutely aware that maintenance activities must be optimized in order to maximize the ability to anticipate and control SSOs. Therefore, preventing SSOs would form the LOS goal, and the various ways to achieve this goal, as well as specific parameters to measure success in implementing these activities, would form the associated criteria and metrics.

The following considerations were reviewed when developing the City LOS goals:

- Regulatory requirements, including requirements of the Consent Decree between the City and San Francisco Baykeeper,
- Expectations of the customer regarding the service that they feel they should receive as ratepayers,
- Physical capabilities (capacity, service life) of the City's linear assets, and
- Relationship of the City's system to other community needs (school safety, commercial districts, transportation corridors).

In addition to defining Level of Service, the RMM includes an importance factor that is assigned to each LOS goal. The importance factors indicate the relative weight of each LOS goal relative to the other assigned goals. It should be noted that a lower importance factor still indicates a high level of importance.

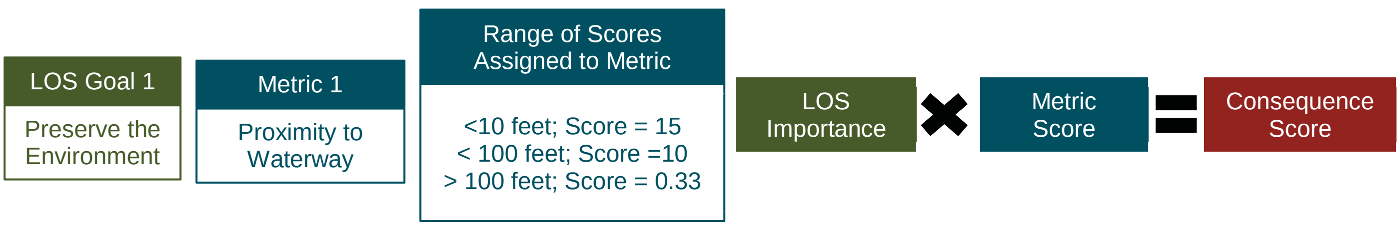
Level of Service goals and relative importance factors were developed with input from City staff. Table 7-4 presents the City's Level of Service goals and their associated importance factors, as related to risk assessment of the sewer collection system.

Table 7-4. Sewer Collection System Level of Service Goals and Relative Importance Factors

Level of Service Goal	Relative Importance Factor
Preserve the Environment	3
Provide Continuous Service	2
Promote Economic Development	1

Figure 7-3 uses one Level of Service Goal as an example to demonstrate how the Level of Service importance factor contributes to the Consequence of Failure weighted score.

Figure 7-3. Example of How LOS Goal Contributes to Consequence of Failure Weighted Score



7.3.2.2 Likelihood and Consequence of Failure Metrics

The RMM calculates the City's pipeline risk as the Likelihood of Failure, with consideration of specific Consequence of Failure metrics.

Likelihood of Failure or F_L metrics predict the theoretical likelihood of failure of a linear asset based on a combination of asset information, physical setting and maintenance history. F_L calculations first consider the CCTV inspection condition rating for a given pipeline asset. The City's RMM includes CCTV inspection results for approximately 12.6 miles of gravity pipelines. As discussed above, the CCTV inspection condition ratings follow NASSCO PACP protocol for structural and operations and maintenance defects.

For the remaining pipes, in the absence of a condition rating, age and geologic setting were used as a rough estimation of likelihood of failure. In the RMM, pipe age is considered to carry the same importance ranking as CCTV inspection ratings. However, age is only considered in the absence of CCTV inspection data. Also, age-based prioritization is only used to recommend CCTV inspection priorities, and not to recommend actual pipeline replacements.

The above data was then reviewed in conjunction with information from the City's Hot Spot Cleaning Workplan database.

In summary, F_L metrics comprise the following:

- NASSCO PACP Quick Ratings (Structural and O&M) for inspected pipe segments,
- OR Calculated remaining useful life and geologic setting (liquefaction potential),
- AND Maintenance needs based on schedule, if applicable, from Hot Spot Cleaning Work Plan.

F_L metrics and their associated importance factors and metric scores are presented in Table 7-5.

Table 7-5. Likelihood of Failure Metrics, Importance Factors and Metric Scores

Likelihood of Failure Metric	Relative Importance Factor	Metric Score	Total Possible Score
Structural PACP Quick Rating	6	3 or More Grade 5 Defects: 15	90
		1 or More Grade 5 and 3 or more Grade 4 Defects: 12	
		3 or more Grade 4 and 3 or more Grade 3 Defects: 8	
		Less than 4333 Quick Rating: .25	
		3 or more Grade 4 and 3 or more Grade 3 Defects: 8	
		Less than 4333 Quick Rating: .33	
Remaining Useful Life	5	Asset Exceeds Useful Life: 16	90
		90% of Service Life Expended: 12	
		80% of Service Life Expended: 5	
		Less than 80% of Service Life Expended: .20	
Geologic Setting	1	Very High Liquefaction Potential: 12	12
		Medium Liquefaction Potential:: 8	
		Low or No Liquefaction Potential: .33	
HSCWP Status	4	1 Month HSCWP Interval: 15	60
		3 Month HSCWP Interval: 12	
		6 Month HSCWP Interval: 8	
		Not on HSCWP: .25	

Consequence of Failure or F_C metrics provide information on the relative criticality of the asset in terms of the impact to the City and the community if that asset fails. The consequence score plays an important role in determining the overall risk presented by the asset, and is often integrated into a balanced risk assessment. The City's assessment utilizes several critical consequence factors (proximity to waterways, commercial districts and transportation corridors) to refine risk scores. However, integration of consequence components into the RMM has been deferred until a future update, because the City must first meet SSO reduction performance measures that are based on likelihood but not consequence of failure.

For the future analysis, the City will integrate F_C metrics for the following parameters into the RMM:

- Proximity to waterways, commercial districts and transportation corridors,
- Proximity to schools, hospitals and safety which comprise critical facilities, and
- Service impact, defined by pipe diameter.

7.3.2.3 Total Risk Score

The RMM relies upon the Likelihood of Failure score to prioritize asset risk. The relative risk for assets with similar scores was refined by comparing, for these assets, three consequence parameters (proximity to waterway, commercial district and transportation corridor). The following example shows how the Total Risk Score was calculated for gravity sewer Asset Number 5506 using the process described above. Results for this asset are presented in Table 7-6.

Risk Management Model
Example: Asset 5506 (313054-313053)
Results Shown in Table 7-6

• Material:	Vitrified Clay Pipe
• Length:	431 feet
• Approximate Year Installed:	1936
• Geologic Setting:	Low Liquefaction Potential
• HSCWP Frequency:	6-Month
• NASSCO PACP Structural Rating:	Three Structural Grade 5 Defects (5300)
• Distance from Waterway or Storm Drain:	>100 feet
• Distance from Commercial Zone:	>100 feet
• Distance from Transportation Corridor:	>100 feet

<table>
<tr><th colspan="7">Table 7-6. Risk Score for City Asset 5506</th></tr>
<tr><th>Component</th><th></th><th>Asset Data</th><th>Importance Factor</th><th>Metric Score</th><th>Weighted Score</th><th>Total Score</th></tr>
<tr><td rowspan="4">Likelihood of Failure</td><td>Structural PACP</td><td>5300</td><td>6</td><td>15</td><td>90</td><td rowspan="4">122</td></tr>
<tr><td>HSCWP</td><td>6 mos</td><td>4</td><td>8</td><td>32</td></tr>
<tr><td>Geology</td><td>Low</td><td colspan="3">CCTV data is available – not used</td></tr>
<tr><td>Useful Life</td><td>>100%</td><td colspan="3">CCTV data is available – not used</td></tr>
<tr><td rowspan="3">Consequence of Failure</td><td>Proximity to Waterway</td><td>>100’</td><td colspan="4" rowspan="3">Low consequence – However, F_L score determined the priority for this asset. If consequence were medium or high, then a pipeline with a lower F_L score may be elevated as a priority for the City.</td></tr>
<tr><td>Proximity to Transportation</td><td>>100’</td></tr>
<tr><td>Proximity to Commercial</td><td>>100’</td></tr>
</table>

7.4 RISK MANAGEMENT MODEL RESULTS

This section summarizes results from the City’s Risk Management Model. The full report showing RMM results for the City’s gravity pipeline assets is included in Appendix H.

As discussed previously and presented in Table 7-1, the RMM identified six pipe segments (1,425 lf) that are recommended for rehabilitation and 11 segments (1,807 lf) that are recommended for CCTV inspection in FY2012/13. The pipes that are recommended for CCTV inspection may requirement rehabilitation within the next two to five years. However, this need should be confirmed through inspection results. The remaining pipe segments were also ranked in order of decreasing priority to facilitate future CCTV inspection planning. The priority segments are shown in Table 7-7.

Table 7-7. Priority Repair and Replacement Assets

Asset Number	Length, ft	Risk Score	Risk Factors	Additional Considerations
313054-313053	431	122	Str PACP of 5644(32) / OM PACP of 5440 6 mo on HSCWP List	
224026-224027	121	104	Str PACP of 5232 / OM PACP of 4100 6 mo HSCWP List	Within 100' of waterway
407009-407001	113	122	Str PACP of 5340 6 mo HSCWP List	Within 100' of waterway
220022-224027	261	91	Str PACP of 5344	Within 100' of waterway
224022-224023	72	91	Str PACP of 5341(31)	Within 100' of waterway
114022-114021	427	91	Str PACP of 5331	
114022-114021	427	91	Str PACP of 5331	
114008-114006	303	114	>90% of useful life / Liquefaction moderate 1 mo HSCWP List	NEEDS CCTV Within 100' of commercial and transportation
711008-711007	183	114	>90% of useful life / Liquefaction high 3 mo HSCWP List	NEEDS CCTV Within 100' of waterway and transportation
707092-MadronePS	38	108	Exceeds useful life / Liquefaction moderate 3 mo HSCWP List	NEEDS CCTV
707001-707088	146	104	Exceeds useful life / Liquefaction high 6 mo HSCWP List	Needs CCTV Within 100' of waterway Within 10' of transport
111001-111003	253	104	Exceeds useful life / Liquefaction high 6 mo HSCWP List	Needs CCTV
707089-707090	121	104	Exceeds useful life / Liquefaction high 6 mo HSCWP List	Needs CCTV
707088-707089	174	104	Exceeds useful life / Liquefaction high 6 mo HSCWP List	Needs CCTV
711004-711008	328	98	>90% of useful life / Liquefaction high 6 mo HSCWP List	Needs CCTV Within 10' of waterway Within 100' of transport
111004-111002	110	92	>90% of useful life / Liquefaction moderate 6 mo HSCWP List	Needs CCTV
111003-111004	105	92	>90% of useful life / Liquefaction moderate 6 mo HSCWP List	Needs CCTV
707091-707092	50	92	>90% of useful life / Liquefaction moderate 6 mo HSCWP List	Needs CCTV

7.5 ESTIMATED PROJECT COSTS

Table 7-1 includes conceptual costs for the recommended projects. As also presented in Chapter 8, Capital Improvement Program, pipeline rehabilitation and replacement costs were developed as follows:

- Baseline pipeline cost: $10 per inch-diameter-foot of pipe
- Full pipe cost increases the baseline pipeline cost by 50 percent to allow for lateral reconnections, manholes, mobilization, demobilization, shoring, traffic control and other requirements
- 30 percent contingency was added to the full pipe cost as consistent with Association for the Advancement of Cost Engineering (AACE) guidelines for conceptual cost estimates
- 18 percent was added to the estimated project cost to allow for engineering, construction management, and project administration

In addition, costs for cleaning and CCTV inspection of pipelines were estimated using a conceptual unit cost of $2.00 per lineal foot of pipeline inspected.

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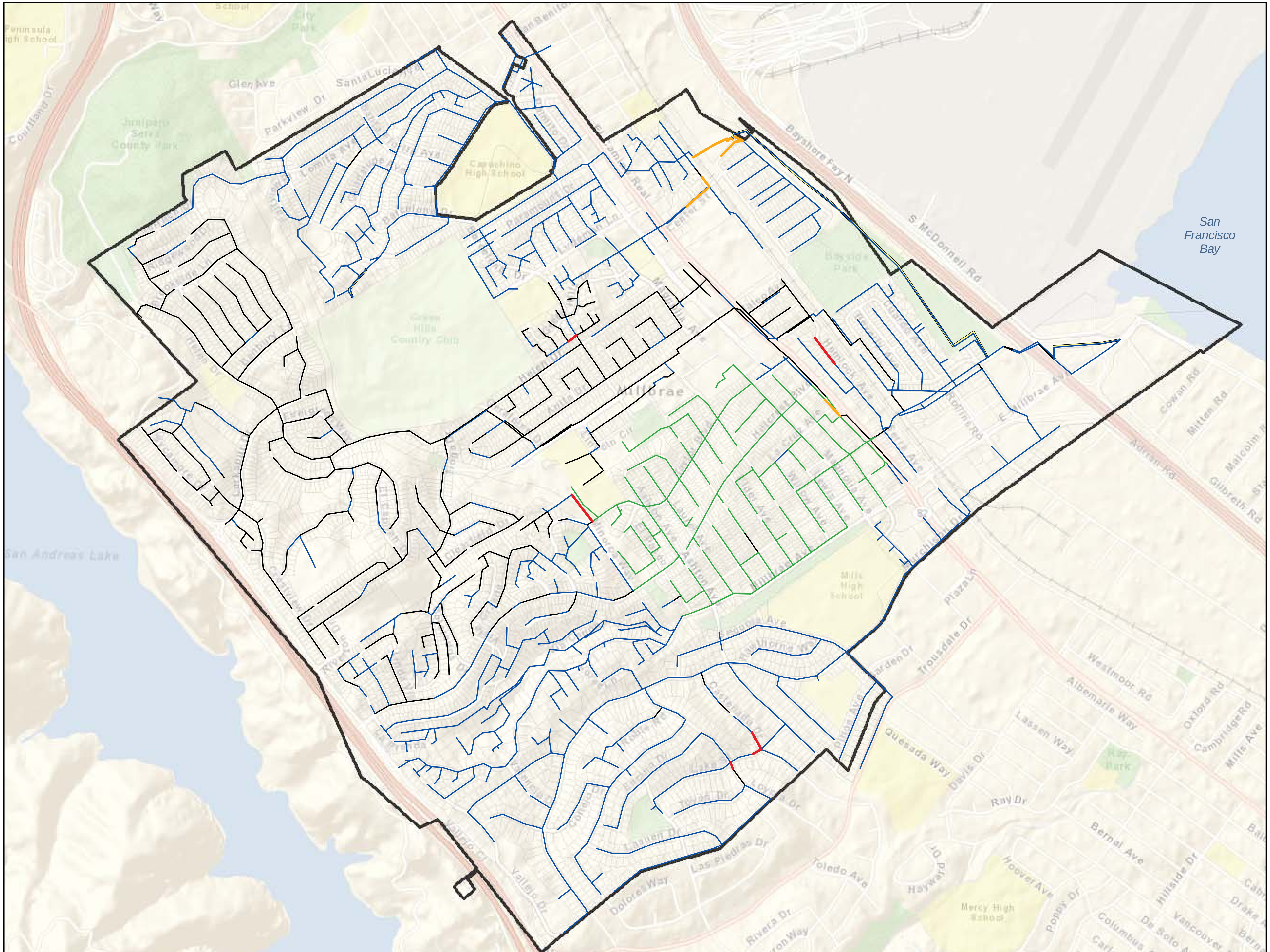

FIGURE 7-1

City of Millbrae
Capacity Assurance Report

PRIORITY PIPELINE REHABILITATION, REPLACEMENT AND CCTV RECOMMENDATIONS

N

0 625 1,250
Scale in Feet

1 inch = 1,250 feet

LEGEND

Preliminary Risk Results/Score

- Replace
- CCTV
- 80 - 140
- 40 - 80
- 0 - 40

OTHER

- City Boundary

SEAL OF THE CITY OF MILLBRAE CALIFORNIA

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 8
Capital Improvement Program

Chapter 8 presents the recommended capital improvement program (CIP) for the City's sewer collection system. The project recommendations, configurations, and conceptual costs that are presented in this chapter were summarized previously in Chapters 6, Capacity Assessment, and Chapter 7, Pipeline Rehabilitation and Replacement Program. 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 capital improvement program describes a combination of pipeline, pump station, and storage improvements to address sewer system overflows (SSOs) that are predicted to result from the design storm event. The proposed combination of projects presents a solution that appears viable and practical, based on the information that was known as of the date of the Capacity Assurance Report (CAR). These proposed projects are alternatives that are subject to change and revision as the City moves forward with the implementation of the CAR. Additional information that is gained through preliminary design activities (permitting, easement acquisition, environmental documentation, etc.) and additional evaluation of the capacity of the City's system is expected to lead to changes in the final project descriptions, costs, and the implementation timeline, and may also result in changes to the types of projects implemented.

The proposed projects have not been subject to the CEQA process. Also, the City's concurrent, ongoing efforts to reduce I&I will result in a reduced need for the planned capacity improvements. Therefore, the proposed capital improvement program is an evolving planning tool that will be refined throughout the term of the Consent Decree. Any changes to the proposed projects and program will continue to uphold the City's commitment to meet the SSO reduction requirements of the Consent Decree, and the City will update Baykeeper with any changes to the CIP that occur throughout the duration of the Consent Decree.

This chapter is organized as follows:

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

8.1 BASIS FOR CAPITAL IMPROVEMENT PROGRAM COSTS

The following sections describe the methods and associated costs evaluated for completing rehabilitation, repair, and replacement projects in the City's collection system for both capacity enhancement and condition repair.

8.1.1 Pipeline Rehabilitation, Repair, and Replacement Methods and Conceptual Costs

The following rehabilitation, repair, and replacement methods are potential options for the City's pipeline projects: open cut construction, pipe bursting, pipe reaming, and tunneling. For projects that require the installation of a new relief sewer to address wet weather flows, in-situ methods for the existing pipe, such as the use of cured-in-place pipe, may be considered in conjunction with construction of the new relief sewer pipeline. Specific to the City's projects, factors that determine the most cost effective rehabilitation method include geological and physical setting, existing pipeline material and condition, and available construction access.

8.1.1.1 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 could be beneficial in locations where significant utility crossings are present, depending on the depths of existing utilities. 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. Construction on El Camino Real falls under the jurisdiction of the California Department of Transportation (CalTrans). Historically, CalTrans has required trenchless construction methods to be used for the installation of new pipelines within this roadway.

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 fifty percent of the cost of pipeline installation.

For the City'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

8.1.1.2 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 that are less than 24-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. Construction using PVC requires longer pit lengths than with HDPE.

Advantages and Limitations: Pipe bursting is quickly gaining popularity as a replacement methodology for small diameter sewers. If HDPE pipe is used, a relatively small pit (as compared to open trench) 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 the receiving 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.

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 $12 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.

The City's projects generally require an increase in pipe diameter that is greater than recommended for pipe bursting. For the City's projects, the more conservative cost for open cut construction was used for all pipelines that are not anticipated to require installation using tunneling methods.

8.1.1.3 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 hydroflushing 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 $10 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 CAR, it is assumed that all of the City's projects will require the installation of new, larger pipe to address capacity constraints. However, during preliminary design, the opportunity to provide smaller, parallel relief sewers in conjunction with repair of the existing pipe using CIPP liner should be considered.

8.1.1.4 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. As discussed under pipe bursting, above, it was assumed that pipelines would be installed using open cut methods unless tunneling is required.

8.1.1.5 Tunneling

Description: Where open cut construction is not feasible, practical, or cost effective, trenchless methods can be used to install the sewer pipe. Commonly used trenchless methods include jack-and-bore and horizontal direction drilling (HDD). Both of these methods involve pre-drilling the pipeline alignment and then installing new pipe through the opening. . When installed below Caltrans or railroad right of ways, an additional casing may be required by the governing jurisdiction.

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

Tunneling requires precise location of existing utilities and is not always appropriate where the new pipeline must maintain a precise slope or avoid numerous underground facilities. Tunneling requires experienced equipment operators that are skilled with the location and guidance of the necessary equipment. For the CAR, tunneling is assumed to be required along and across Caltrans and railroad rights of ways, including El Camino Real.

Probable Unit Costs: The unit cost of tunneling varies depending on site conditions and construction access limitations. However, in areas without significant utility or traffic issues, tunneling costs are generally 1.5 to 2 times the cost of open cut construction, or from $14 to $20 per inch diameter per foot of pipeline installed. 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 as fifty percent of the cost of pipeline installation.

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

Jack and Bore:	$20 per inch diameter per foot of pipe
Microtunnel:	$20 per inch diameter per foot of pipe

8.1.2 Pump Station Expansion Methods and Conceptual Costs

The construction cost estimates for Madrone Pump Station improvements used as a basis pre-established West Yost costs curves for wastewater pump stations, and compared these cost curves with the costs curves presented in Shank's "Pumping Station Design." Although the West Yost curves do not differentiate between wet-pit/dry pit and submersible stations, the curves in "Pumping Station Design" provide separate curves for these configurations.

The pump station reliable capacity (the capacity of the station with the largest pump in reserve) is the key value to input to the curves. From the capacity value, a line is drawn to where capacity intersects the cost curve lines. Two lines are provided to reflect difficult construction conditions and comparatively easy construction conditions. For the CAR estimate, the estimate assuming difficult conditions was used to reflect the possibility of the pump station needing to be supported on piles to resist damage from liquefaction.

The cost curves return cost values linked to an Engineering News Report Construction Cost Index (ENRCCI) for the "20-Cities Average. This returned cost is then adjusted to better reflect the current value of money and the construction market in the San Francisco Bay Area. This adjustment is a ratio of the current ENRCCI to the ENRCCI used for the curve. Finally, a 30 percent contingency was added to define the range of the pump station costs based on this planning level of accuracy.

8.1.3 Wet Weather Storage Methods and Conceptual Costs

For the CAR, it is assumed that wet weather storage will consist of a covered, below-grade storage facility that includes pre-screening facilities. This flow will be pumped out of storage and returned to the collection system for conveyance to the WPCP over a duration of 24 hours following the wet weather event.

Storage cost estimates applied a unit cost of $2 per gallon of flow to account for storage, pumping, and conveyance facilities, and contingencies. A 30 percent contingency for construction unknowns was added to be consistent with pipeline and pump station cost estimates..

8.2 BASIS FOR CAPITAL IMPROVEMENT PROGRAM DEVELOPMENT

The CIP was developed to create a program that addresses the City's need to eliminate SSOs resulting from flows from a 10-year, 24-hour design storm within the required Consent Decree timeline. In addition, the CIP includes concurrent rehabilitation and replacement projects in order to continue the City's approach to conducting continuous, ongoing system maintenance. The following criteria were used to prioritize the various projects and develop a timeline for implementation.

1. Projects to Eliminate SSOs. The CIP prioritizes and schedules completion of projects that eliminate capacity-related SSOs from the 10-year, 24-hour design storm. Projects were ordered such that downstream capacity improvements are completed first.
2. Projects to Address Known Maintenance Issues. The CIP prioritizes and schedules completion of pipeline replacements to address pipe segments with substantial structural defects, as determined through the CCTV inspection program. The CIP also prioritizes CCTV inspection of areas with anticipated issues as determined through average pipe age and known maintenance issues.
3. Distribution of Capital Costs. The he City has established an implementation schedule for the recommended projects that meets the requirements of the Consent Decree between the City and San Francisco Baykeeper. The Consent Decree requires all projects needed to address capacity issues from the design storm event to be completed by 2016.

8.3 PROPOSED CIP

Table 8-1 presents the proposed CIP, which begins implementation in Fiscal Year 2012/13 and extends into Fiscal Year 2016/17. The CIP has been developed to address all capacity requirements of the 10-year, 24-hour design storm with conservative assumption that there will be no short-term reduction in rainfall-dependent inflow and infiltration (RDII). Concurrently, the City will implement rehabilitation and replacement projects that address known structural defects, and should therefore contribute to a reduction in RDII. This CIP is intended to be an evolving document that is adjusted as needed to address future conditions that are identified as more data is collected through CCTV inspection and maintenance activities.

The most critical components of the CIP are summarized below. Additional information on the individual projects is found in Chapters 6, Capacity Assessment, and 7, Pipeline Rehabilitation and Replacement Program. Detailed cost estimates can be found in Appendix G. The total estimated CIP cost is $21.4 million, to be implemented by 2016.

8.3.1 Priority Capacity Improvement Projects

The CIP includes eight recommended Capacity Improvement Projects that address potential capacity-related SSOs from a 10-year, 24-hour design storm. These projects are planned for implementation starting in Fiscal Year 2012/13 and completing in calendar year 2016. The projects provide wet weather storage, provide additional pumping capacity at Madrone Pump station, and include replacement of gravity sewer pipe with larger diameter pipe to relieve bottlenecks that are associated with model-predicted SSOs from the design storm.

The total estimated combined cost of the priority capacity improvement projects is $18.2 million.

8.3.2 Priority Rehabilitation and Replacement Projects

In addition to the identified capacity assurance projects, the CIP includes the replacement of six pipeline segments and CCTV inspection of an additional eleven segments in FY2012/13. CCTV inspection results will determine whether the additional segments should be added to the prioritized replacement list. The CIP also includes a budget for CCTV inspection of the Hillcrest Avenue and Hawthorne Boulevard neighborhoods in 2012, and a placeholder for pipe replacements that may result from these inspections or that may otherwise arise in FY2012/13. The total projected FY2012/13 budget for projects to supplement capacity improvement needs includes $625,000 in rehabilitation and replacement, $40,000 for CCTV inspection of the Hawthorne and Hillcrest neighborhoods, and $3,650 for additional priority CCTV inspections.

For the remainder of the CIP timeframe, the allocated budget for ongoing rehabilitation and replacement projects is $575k in FY2013/14, and $650k in future fiscal years.

Table 8-1. City of Millbrae Capacity Assurance Report Capital Improvement Project Implementation Plan (Note 1)							
R&R Project #	Project Name	Estimated Cost	2012/2013	2013/14	2014/15	2015/16	2016/17
1	Rehabilitate Six Pipe Segments	$265,000	265,000				
2	Conduct CCTV Inspection of Eleven Priority Pipe Segments	$3,650	3,650				
3	Conduct CCTV Inspection of Hawthorne and Hillcrest Neighborhoods	$40,000	40,000				
4	Additional Pipeline Rehabilitation Projects (Based on CCTV Results)	$360,000	360,000				
5	Future Pipeline Rehabilitation and Replacements	$2,525,000	0	575,000	650,000	650,000	650,000
	Subtotal R&R	$3,193,650	668,650	575,000	650,000	650,000	650,000
Capacity Project #	Project Name	Estimated Cost	2012/2013	2013/14	2014/15	2015/16	2016/17
1	Wet Weather Storage at Corporate Yard	$2,760,000	690,000	2,070,000	0		
2	Madrone Pump Station Replacement	$7,256,000			3,628,000	3,628,000	
3	Pipeline Replacements Near Capuchino High School	$850,000				425,000	425,000
4	Pipeline Replacements at Aviador Avenue and E. Millbrae Avenue	$772,000				386,000	386,000
5	Pipeline Replacements in Murchison Drive	$501,000				501,000	
6	Pipeline Replacements Along Highline Canal ROW	$2,046,000		511,500	1,534,500		
7	Pipeline Replacements in Anita Drive and Richmond Drive at El Camino Real	$890,000				445,000	445,000
8	Pipeline Replacements in El Camino Real	$3,129,000	782,250	2,346,750			
	Subtotal Capacity	$18,204,000	1,472,250	4,928,250	5,162,500	5,385,000	1,256,000
	Total CIP	**$21,397,650**	**2,140,900**	**5,503,250**	**5,812,500**	**6,035,000**	**1,906,000**

Note 1: Implementation schedule beginning in 2013/14 and beyond will be revised routinely based on new system information, and as needed to accommodate unexpected infrastructure repair projects.