

CITY OF SAN MATEO
STORM DRAIN MASTER PLAN

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Job # CSMT.58

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PREPARED FOR:

City of San Mateo
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San Mateo, CA 94403

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City of San Mateo
Storm Drain Master Plan

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List of Abbreviations

Abbreviation	Description
1D	one-dimensional
2D	two-dimensional
ACP	asbestos cement pipe
AF	acre-feet
AMC	antecedent moisture condition
ATS	automatic transfer switch
BFE	base flood elevation
BHP	brake horsepower
CAD	computer-aided design
C-CAP	Coastal Change Analysis Program
CCI	Construction Cost Index
CCR	California Code of Regulations
CCTV	closed-circuit television
CEQA	California Environmental Quality Act
cfs	cubic feet per second
CIP	Capital Improvement Program
CIPP	cured-in-place pipe
CMP	corrugated metal pipe
CN	curve number
CSM	City of San Mateo Datum
D/S	downstream
DEM	digital elevation model
DFS2	DHI gridded data format
DHI	Danish Hydraulic Institute
DIP	ductile iron pipe
DSS	data storage system
EIR	Environmental Impact Report
ENR	Engineering News-Record
EPA	United States Environmental Protection Agency
EPA-CAT	EPA Climate Assessment Tool
ESM2	Canadian Earth System Model 2
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRO	forecast-informed reservoir operation
GE	General Electric
GIS	geographic information system
gpm	gallons per minute
HDPE	high-density polyethylene
HEC	Hydrologic Engineering Center
HEC-22	Hydraulic Engineering Center Circular No. 22

Abbreviation	Description
HEC-HMS	Hydrologic Engineering Center's Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center’s River Analysis System
HEC-SSP	Hydrologic Engineering Center’s Statistical Software Package
HGL	hydraulic grade line
HHW	Higher High Water
HMS	Hydrologic Modeling System
hp	horsepower
HSG	hydrologic soil group
HW	High Water
LF	linear feet
LiDAR	light detection and ranging
LLW	Lower Low Water
LW	Low Water
MCC	master control center
MHHW	Mean Higher High Water
MIKE+	DHI MIKE+ stormwater modeling software
N/A	not applicable
NASSCO	National Association of Sewer Service Companies
NAVD88	North American Vertical Datum of 1988
ND	Negative Declaration
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRCS	National Resources Conservation Service
NYE	New Year's Eve
O&M	operations and maintenance
OPC	California Ocean Protection Council
PCP	precast concrete pipe
PG&E	Pacific Gas and Electric Company
PTO	power take-off
PVC	polyvinyl chloride
R&R	repair and replacement
RAS	River Analysis System
RCP	reinforced concrete pipe
RDII	rainfall-dependent infiltration and inflow
ROW	right-of-way
RPM	revolutions per minute
RR	railroad
SCADA	Supervisory Control and Data Acquisition
SCS	Soil Conservation Service
SDMP	Storm Drain Master Plan
SFHA	special flood hazard area
SFPUC	San Francisco Public Utilities Commission

Abbreviation	Description
SLR	sea level rise
SMCDM	San Mateo County Drainage Manual
SSURGO	Soil Survey Graphic Database
SWMM-CAT	Storm Water Management Model Climate Adjustment Tool
TDH	total dynamic head
TP40	NOAA Technical Paper 40
TWL	Total Water Level
U/S	upstream
UC	University of California
UHM	Unit Hydrograph Method
UPRR	Union Pacific Railroad
UPS	uninterruptible power supply
US	United States
USDA	United States Department of Agriculture
USGS	United States Geological Survey
V	volt
VCP	vitrified clay pipe
VHS	vertical hollow shaft
WSEL	water surface elevation
Xing	crossing
yr	year

1. Executive Summary

San Mateo's storm drainage system is one of the City's least visible but most important public safety systems. In a typical storm, rainwater flows from streets and neighborhoods into pipes, creeks, channels, pump stations, lagoons, and ultimately, San Francisco Bay. In larger storms, however, the system can be overwhelmed. Historic events, including the 1955 flood and the New Year's Eve 2022 storm, have shown the consequences: water in streets and neighborhoods, unsafe underpasses, and heavy pressure on Marina Lagoon, San Mateo Creek, Laurel Creek, and the City's pump stations. This Storm Drain Master Plan (SDMP) was prepared to turn those experiences, the experiences of residents and City staff, and research into a practical, prioritized plan for reducing flood risk.

The City and Schaaf & Wheeler have developed this plan by combining engineering analysis with local knowledge. The team reviewed previous plans, mapping data, record drawings, pump station information, field inspections, pipe inspection videos, tide data, creek and channel surveys, and reports of known flooding. Because the City's drainage records were incomplete in some places, the study also improved the City's storm drain mapping and used reasonable engineering assumptions where full field verification was not practical. Public input was central to the effort; presentations to the Sustainability and Infrastructure Commission and City Council, community meetings, an interactive public flood mapping tool, and public review of the draft SDMP report helped identify problem areas, test the results against lived experience, and shape project priorities.

Computer modeling was used as a way to rehearse storms before they happen. The citywide one-dimensional (1D) modeling and supplemental two-dimensional (2D) modeling in the low-lying urban areas of the City estimate where rainfall will go, how much water will enter the pipes and creeks, where the system is likely to back up or overflow, and which improvements would most reduce flooding. These results matter because they help the City move from responding to drainage complaints one location at a time to understanding how the whole drainage system works together. The SDMP uses this information to identify projects that are intended to provide a minimum 10-year level of service for the storm drainage system and a 100-year level of service for terminal pump stations and major channels while recognizing that unusually large storms can still cause flooding even after improvements are built.

The SDMP also looks ahead. For future planning, the SDMP evaluates stronger storms, higher Bay water levels, and sea level rise, including 2050 and 2100 precipitation scenarios that increase modeled extreme rainfall by 8% to 30%, depending on the scenario. These projections show that some investments are needed to solve today's recurring problems while also remaining adaptable to future conditions. The result is a prioritized Capital Improvement Program (CIP) that includes pipe and creek capacity projects, pump station rehabilitation, repair or replacement of aging infrastructure, and targeted maintenance, with the highest-priority work recommended for implementation over the next decade and the full program refined over time as projects are designed, funded, and updated with new information.

Specific projects have been identified that are intended to provide a 10-year minimum level of service throughout the City, with a 100-year level of service for terminal pump stations and major open channels. Projects are prioritized as low, medium, high, and very high priority based on the magnitude of existing capacity deficiency and the potential or known extent of flooding impact surrounding the project. Recommendations are provided for completing a subset of the recommended improvements over a period of 10 years, with a 25-year maximum time horizon.

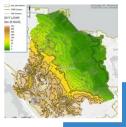
1.1 SDMP Purpose and Products

This SDMP is intended to function as a multipurpose storm drain system resource guide for the City's staff and residents. This report provides sufficient background information and data to City engineers responsible for storm drainage CIP implementation and/or modification, as well as an evaluation of the condition and function of all of the City's existing storm drain pump stations. It also provides a summary of system deficiencies and potential solutions to the community.

Appendix H of this report includes planning-level cost estimates for the alternatives presented in the report. Appendix O includes preliminary conceptual designs for select, very high priority projects to aid the City in preparing grant applications. Products of the study also include a geographic information system (GIS)-enabled, Citywide hydrologic and hydraulic model and updates to the City's storm drain system.

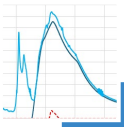
1.2 Study Objective

As part of this SDMP, Schaaf & Wheeler conducted a study of the City's storm drain systems. The basic objective of this study was to provide an examination of Citywide flood risks and recommend actions necessary to accomplish an appropriate level-of-service for the City's storm drain systems. Several activities were completed as part of this study, as indicated in the graphic below. Schaaf & Wheler:



Reviewed

- Prior master plan
- GIS data
- CAD files
- As-built drawings
- Pump system data and specifications



Analyzed

- Field investigations
- CCTV recordings
- Hydrologic & hydraulic models
- Tide conditions
- Potential pipe system capacity improvements
- Open Channel capacity deficiencies
- Climate change



Recommended

- Condition improvements
- Projects to increase conveyance capacity
- Pump station rehabilitation
- Prioritized Capital Improvement Program (CIP)

The adoption of this document is exempt from the requirements of the California Environmental Quality Act (CEQA). This is a statutory exemption under CCR Title 14, Section 15262 (Feasibility and Planning Studies). However, CEQA must be satisfied for any capital improvement project described in this report that may be implemented by the City in the future, through the preparation of an appropriate Environmental Impact Report (EIR) or Negative Declaration (ND), or the capital improvement project must be determined to be categorically excluded.

1.3 Background

The City's storm drain system consists of a mixture of open channel conveyance (ditches), inlet structures, storm drain pipes, pump stations, and storage facilities (ponds or lagoons). The system relies on both gravity outfalls and pump stations to convey stormwater to San Francisco Bay. An overview of the City's drainage system is shown in Figure 1-1. Small segments of Belmont City, Burlingame City, and San Mateo County systems are included in the maps and models to show the drainage from these other jurisdictions that is handled by City systems.

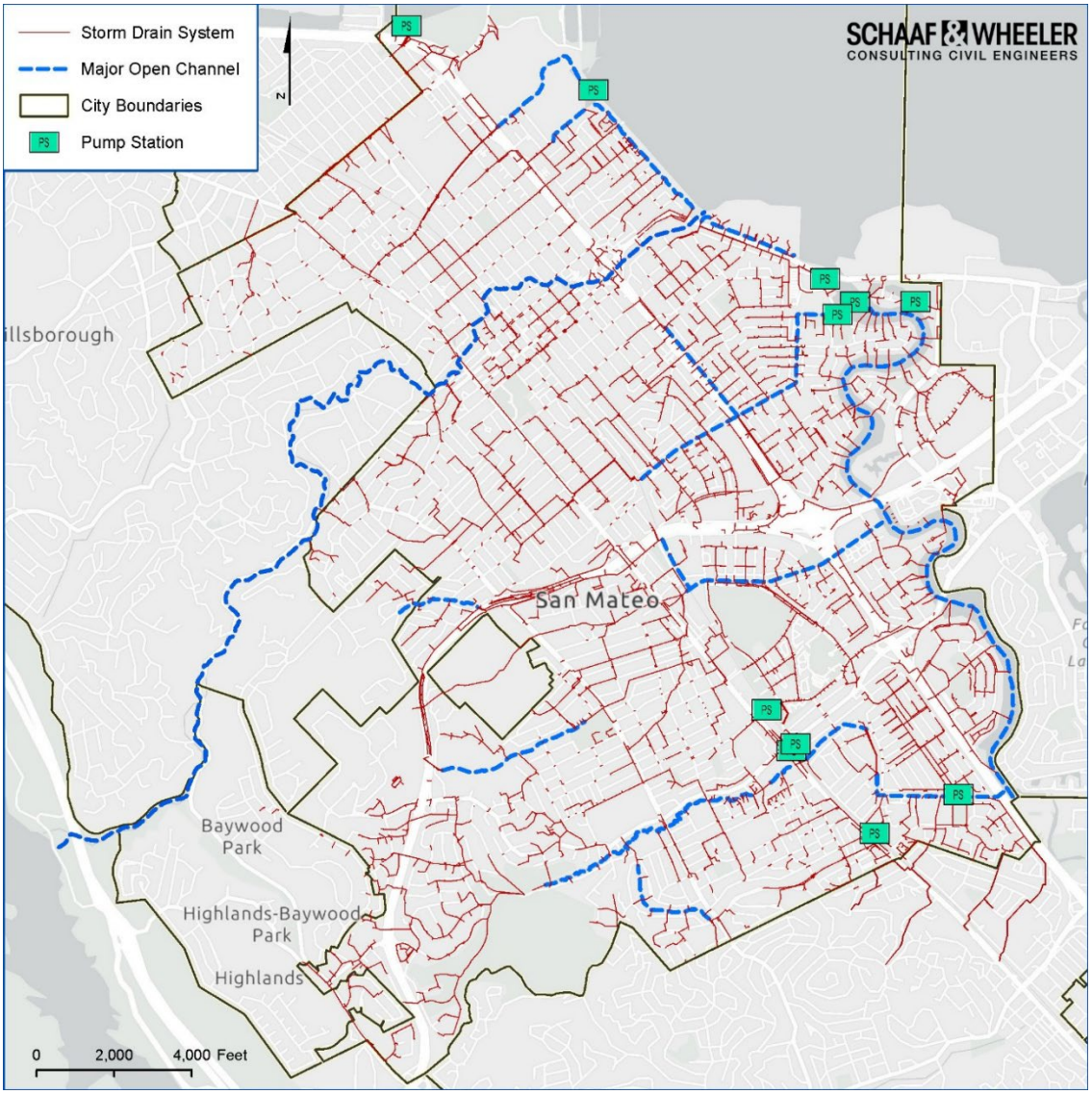


Figure 1-1: City of San Mateo Drainage System Overview

1.4 Known Drainage Issues

City staff and residents identified a wide range of known or suspected drainage issues across San Mateo, as shown in Figure 2-7 and summarized in Table 2-1. These locations include recurring street ponding, undersized or inefficient drainage systems, bubble-up systems that leave water in gutters, railroad (RR) and roadway underpasses that can flood during larger

storms, creek and channel capacity constraints, aging pump station facilities, sediment or blockage concerns, and hillside runoff that affects downstream properties. The reported issues are spread throughout the City and include areas such as B Street, Sunnybrae Blvd and Grant St, Poplar Ave, 2nd Ave and Delaware Street, the 16th Avenue Channel area, Hillsdale underpass, O'Neill Drive near Casanova Pump Station, the East Laurel Creek culvert at Laurelwood Dr, and the 42nd Ave underpass.

These staff and resident reports together with hydraulic modeling, have been utilized to better understand why flooding occurs and where similar problems may occur during design storm events. In this context, “ponding” refers to stormwater collecting in low areas, such as sag points in streets or gutters, where water may not drain effectively because an inlet is missing, blocked, undersized, or affected by high water downstream. The issues shown in Figure 2-7 and summarized in Table 2-1 are not intended to represent every drainage concern in the City. Rather, they provide a practical record of recurring and suspected problem areas that helps test the model against real-world experience and supports prioritization of improvements based on the frequency and severity of past flooding.

1.5 System Evaluation Methods

Schaaf & Wheeler developed a Danish Hydraulic Institute (DHI) MIKE+ rainfall-runoff model for the City that contains the storm drainage pipe system, open channels, ponds, lagoons, and pump stations that provide essential conveyance and storage capacity for stormwater runoff. The 1D system model was supplemented in low-lying areas, generally east of Alameda de Las Pulgas and El Camino Real. The 2D surface model shows a more refined evaluation of storm drain system function and interaction between otherwise disconnected systems.

The models utilize a rainfall pattern based on the 72-hour December 1955 storm. For the purposes of this SDMP, the storm was rebalanced to match rainfall statistics extracted from the MetStat precipitation-frequency estimates for 15-minute, 30-minute, 1-hour, 2-hour, 3-hour, 6-hour, 12-hour, 24-hour, and 72-hour durations, for 2-, 10-, 25-, and 100-year storm events. The model uses these storms to estimate runoff to conveyance systems.

A detailed model calibration was completed using approximately 70 years of gage data from nearby San Francisquito Creek and a Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) hydrology model. The calibration included an evaluation of soil moisture conditions and baseflow parameters to replicate peak flow statistics from the gage and to verify the performance of the hydrology model for a real storm event (New Year's Eve 2022).

The physical condition of the drainage system was widely evaluated using topside, pole-mounted camera observations, except where surcharged pipes prevented condition assessment. Essential reaches of the system and metal pipes susceptible to corrosion were identified for additional closed-circuit television (CCTV) inspection to further evaluate repair, replacement, or maintenance needs. CCTV was completed on about 28,000 linear feet of closed conduit, or about 4% of citywide system. Most of the observed system is in good condition. However, some aging metal pipes are beginning to corrode and fail and need replacement.

Condition-related projects are identified in this report for reaches with known condition deficiency. In the low-lying areas of the City, some reaches of pipe are affected by heavy debris and sediment accumulation. In Chapter 7, maintenance activities are also identified and prioritized for inspected systems.

Detailed review, field investigations, analysis, and modeling of the area's storm drainage system led to several conclusions. Improvements to the system are recommended to reduce flood risk within the City. The recommended improvements are preliminary in nature and based on currently available information. Detailed project designs will ultimately require more data, including utility locations, which were not part of the scope of this SDMP and have yet to be obtained.

1.6 Climate Change

Climate change, including sea level rise (SLR) and changes in precipitation characteristics, is addressed to a limited extent within this SDMP to ensure that proposed projects remain resilient and adaptable. For planning purposes, modeled 10- and 100-year precipitation events are scaled up to expected 2050 and 2100 extreme event depths by 8% to 30%, depending on the scenario. Climate adaptation and resiliency should be further incorporated into each project as each is funded and designed.

This document does not fully consider coastal protection needs (e.g., erosion protection, armoring, flood walls, or levees). However, the City falls within the San Mateo County Flood and Sea Level Rise Resiliency District (OneShoreline), established in 2020 with the mission of protecting residents and businesses against the impacts of SLR. OneShoreline is currently considering projects that provide resilient mitigation measures against SLR. The City has also recently constructed a number of projects providing Federal Emergency Management Agency (FEMA)-accredited coastal protections, including San Mateo Creek flood walls, Bayfront Levee, and the Coyote and Poplar Pump Station improvements.

1.7 Outreach and Community Engagement

As part of the SDMP efforts, City staff prepared Council and local commission presentations, hosted community meetings, and published reporting tools to encourage engagement and gather input on stormwater and drainage concerns from a spectrum of San Mateans.

Council and Commission Presentations

City staff prepared presentations to City Council and the Sustainability and Infrastructure Commission to obtain feedback on community engagement, capital improvement projects, prioritization, and the overall structure of the plan and recommendations. Council and commission presentations included:

- January 14, 2026: Initial presentation to the Sustainability & Infrastructure Commission
- January 26, 2026: Informational Update at a City Council Special Meeting
- June 10, 2026: Progress update to the Sustainability & Infrastructure Commission

Community Meetings

Based on feedback from City Council and the Sustainability and Infrastructure Commission, staff hosted community meetings, which included one in-person meeting and two virtual meetings to maximize accessibility and encourage broader community involvement:

- April 15, 2026: In-person community meeting at the MLK Center
- April 16, 2026: Virtual community meeting
- April 22, 2026: Virtual community meeting

Public Comment and Mapping Tool

City staff simultaneously collected public input through an interactive online mapping tool alongside the first full draft release of the master plan document. This allowed members of the community to review the plan and an initial set of identified drainage issues and proposed projects, then to provide feedback based on real word experiences with flooding.

Feedback received during outreach events and from the mapping tool helped to identify community priorities, local drainage concerns, and areas of interest. This input was used to refine and verify the results of the citywide stormwater system hydraulic model, supporting holistic SDMP development by integrating engineering analysis with community experience. This combined approach helped identify and prioritize improvements that address both modeled system performance and observed on-the-ground conditions, resulting in a more comprehensive prioritized CIP.

1.8 Capital Improvement Program Recommendations

Schaaf & Wheeler has developed a CIP based on modeling and condition assessment to ensure that the system meets the level-of-service standards discussed in this report. About $224.5 million in system-wide capital improvements are identified that address capacity and condition deficiencies. The actual costs of the improvements depend on the combination of projects and alternatives selected for design, permitting, and construction.

Schaaf & Wheeler recommends that the Very High priority projects identified in this document be completed over the next ten years, at a total estimated cost of $148.3M (in 2026 dollars). It is assumed that the City will re-analyze and re-prioritize remaining projects in the next SDMP update. The cost of system capacity improvements, repairs, and pump station rehabilitation investment requirements explored in this report are summarized broadly in Table 1-1. The majority of identified projects are within public right-of-way (ROW) or easements. The City may need to consider whether easements should be obtained in the future on a case-by-case basis.

Table 1-1: Approximate Cost of Citywide Capital Improvement Projects by Priority

Improvement Priority	Description	Estimated Cost (in Millions)
Very High	Projects that address risk of urgent flooding issues with the highest risk of erosion or property damage; Repairs or replacement of existing system where complete failure is imminent or has occurred	$148.3
High	Risk of high magnitude, high impact flooding with potential for heavy erosion and property damage; Repairs or replacement of existing system in extremely poor condition with no immediate risk of failure	$25.3
Medium	Risk of moderate magnitude flooding with relatively extensive impact on regional and local systems, posing some risk of property damage or erosion; Repair or replacement of existing system in poor condition	$26.6
Low	Risk of low magnitude flooding with relatively low impact on regional and local systems and little risk of property damage or erosion; Low priority repairs	$24.3
	Total:	**$224.5**

Baseline CIP costs do not include approximately $50 million in additional projects that may be required to mitigate climate change impacts on City pipe systems, pump stations, and open conveyance channels.

1.9 Conclusion

This storm drain system analysis provides a tool for the City to use in their efforts to reduce both localized flooding and the likelihood of more serious storm water related hazards to private and/or public property in the community. The study, report, and capital improvement alternatives herein comprise a conceptual foundation to begin improvements.

We anticipate that each project will require detailed studies and alternatives analyses to identify the most affordable and effective capacity and condition improvement projects, with information gathered as part of the design process. This may include more detailed topography, utility conflicts, available easements and ROWs, construction impacts, permitting needs, and long-term operation and maintenance. This report considers these factors in developing an alternatives analysis for various improvement strategies, but further detailed information will provide better accuracy to inform the best decisions.

2. Introduction

2.1 Overview

This document includes a capacity analysis and condition assessment of the City's existing storm drain collection systems, a condition assessment of the City's stormwater pump stations, identification of level-of-service standards desired by the City, and recommended improvement projects to reduce the risk of flooding, including estimated project costs. The primary focus is on City-owned drainage facilities, although private systems, portions of Caltrans highways, San Mateo County land, and neighboring cities' land contribute stormwater runoff to City systems.

This analysis should be used to guide the City in planning, financing, engineering, and maintaining its own infrastructure. Each chapter of this report is intended to help the City identify problems, manage resources, and provide cost-effective and comprehensive solutions.

This chapter provides a general discussion of drainage and flood management systems and issues currently affecting the community. It also describes the objectives of this analysis, explains the criteria used to evaluate storm drain system performance, and presents a summary of the data collected to support this effort.

2.2 Setting

The City is an incorporated community situated in San Mateo County, resting between the Pacific Coast Mountain range to the west and the San Francisco Bay to the east. Neighboring incorporated jurisdictions include the City of Burlingame and Town of Hillsborough to the west, the City of Belmont to the south, and Foster City to the east. The City also shares jurisdictional boundaries with unincorporated parts of San Mateo County to the west. The City is situated approximately 20 miles south of San Francisco.

The hills on the west side of the City are partially forested, with rural residential development and paved roads, and range in elevation from 30 feet to about 1,000 feet on the 1988 North American Vertical Datum (NAVD88). Low-lying areas on the eastern side of the City are more heavily developed, including a mix of single-family residential properties and commercial developments surrounding Highway 101.

The low-lying portion of the City is mostly protected from coastal flooding sources by levees, with some Zone AE and AH Special Flood Hazard Area (SFHA), where average depths of greater than one foot of flooding occur, concentrated at Marina Lagoon, San Mateo Creek, and the Coyote Point area. The SFHA in the vicinity of the City is shown in Figure 2-1. Other major open channel conveyances that are not mapped include 16th Ave Channel (Leslie Creek), 19th Ave Channel (Borel Creek), and Laurel Creek.

The study area, defined by the drainage area to the existing stormwater conveyance system in the City, covers an area of approximately 14.7 square miles, as shown in Figure 2-2.

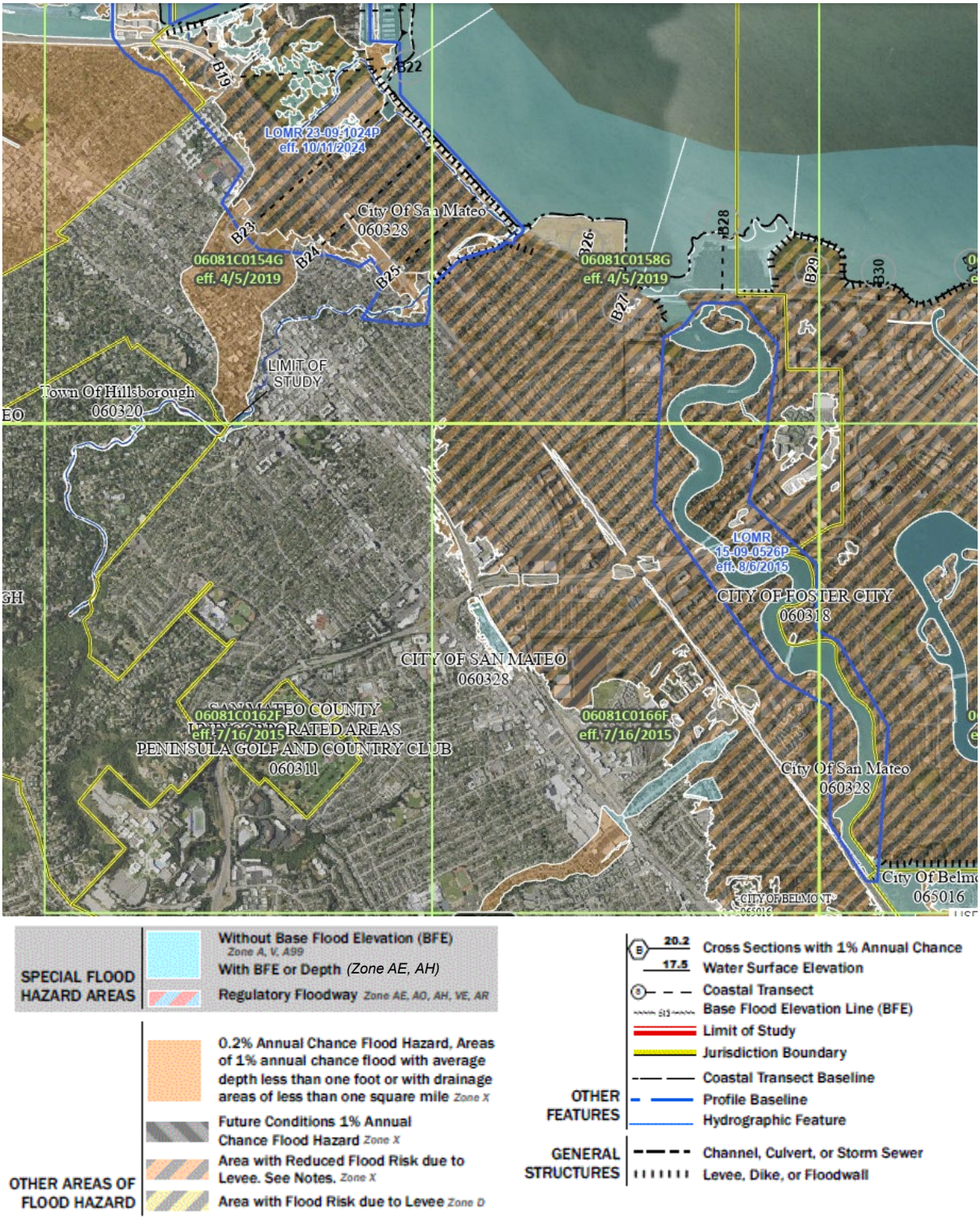


Figure 2-1: FEMA Special Flood Hazard Areas in the Vicinity of the City of San Mateo

2.3 Study Area

Runoff generated by precipitation within the City is drained through various conveyance systems. A map of the area is shown in Figure 2-2.

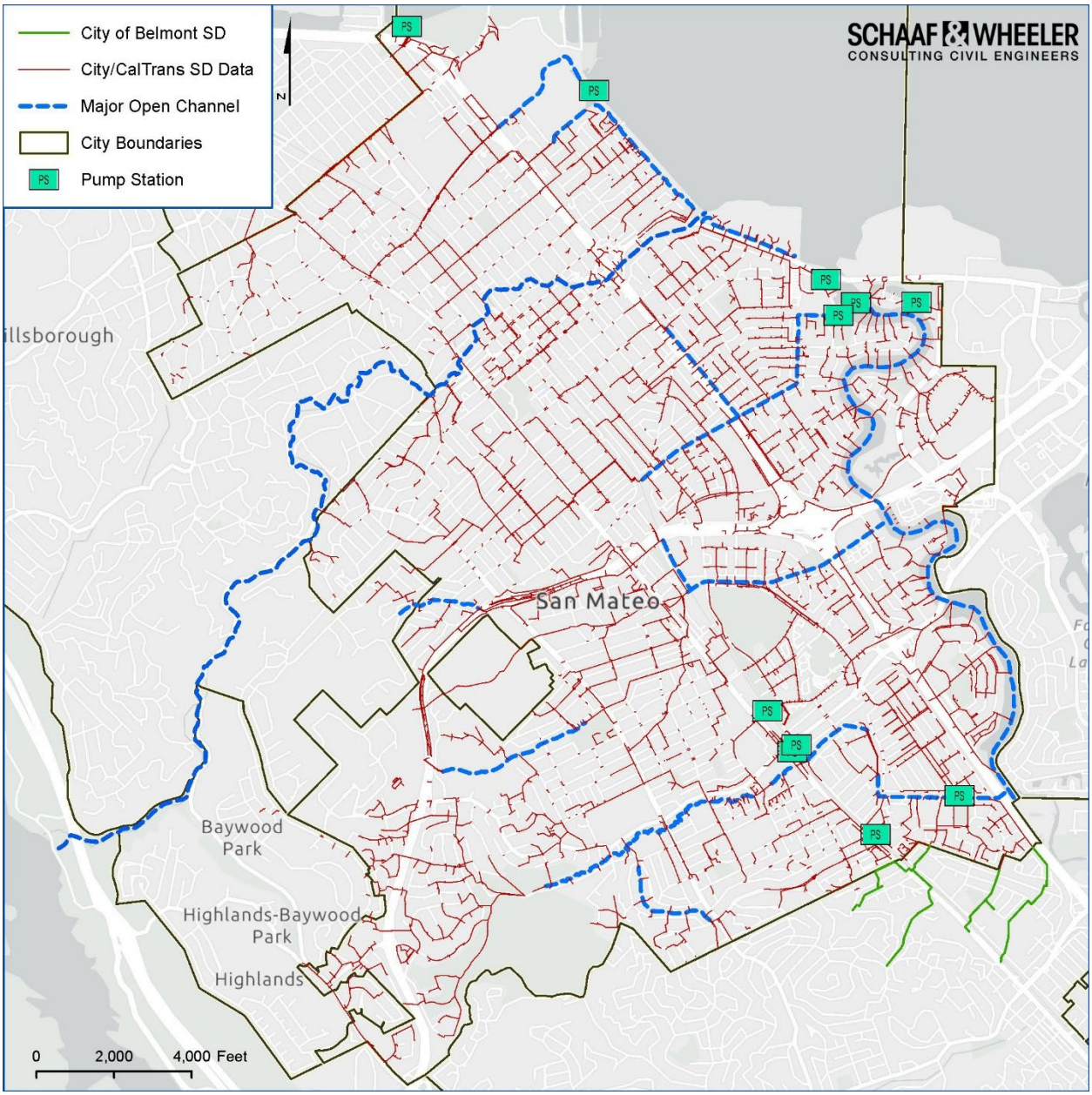


Figure 2-2: Approximate Study Area and Existing Storm Drainage System

Four major Creeks carry concentrated discharge from City pipe systems and areas outside of the City's jurisdictional boundary. A map of major watersheds is shown in Figure 2-3. This watershed map excludes Crystal Springs Reservoir, which has a low level outlet and a spillway discharging to San Mateo Creek. The reservoir is operated as a water supply facility and does not always discharge to San Mateo Creek, except for very limited continuous environmental flows of 3 to 17 cfs.

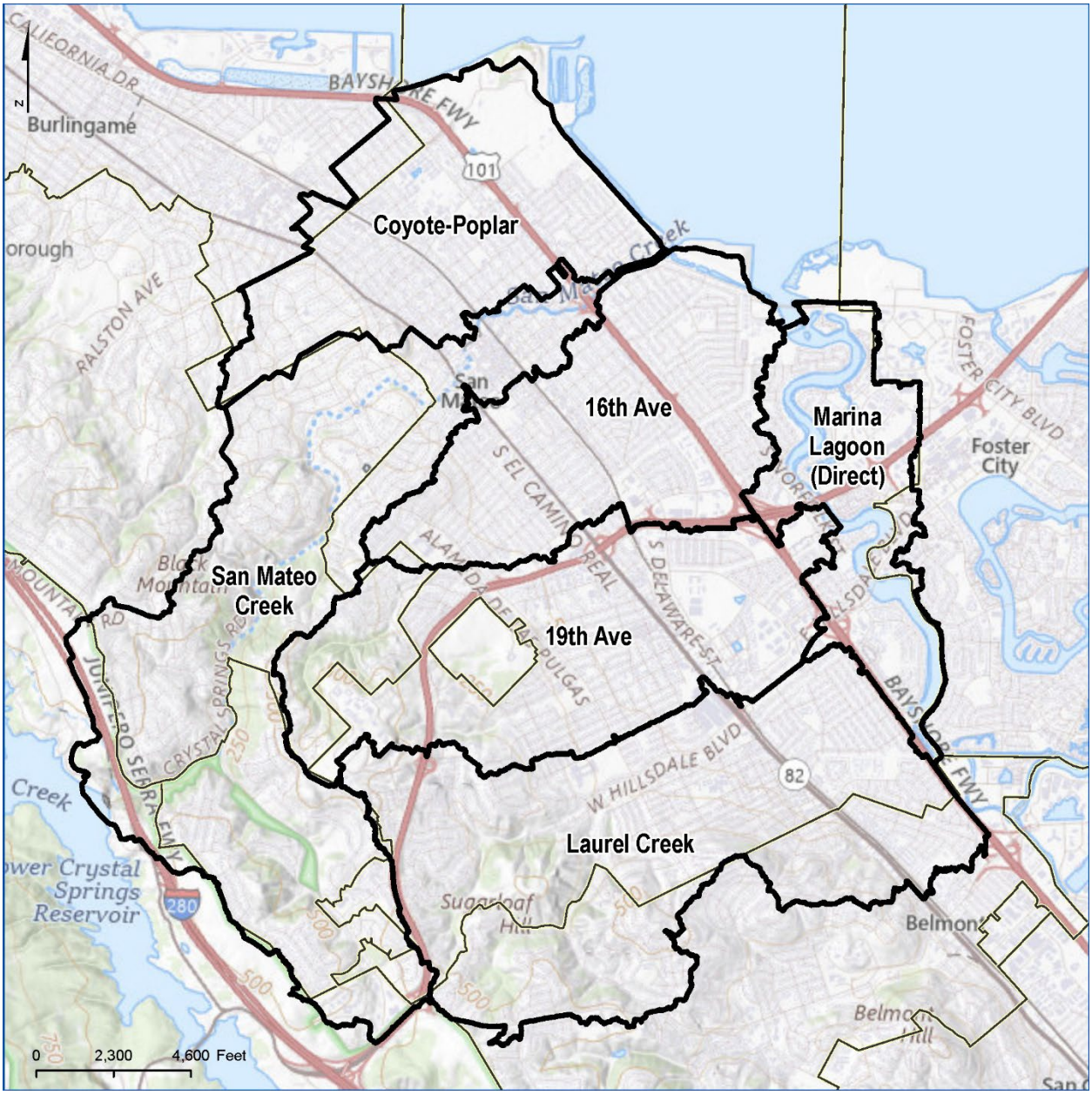


Figure 2-3: City of San Mateo Major Local Watershed Map

The northern portion of the City drains through closed conduit storm drain systems to Poplar Ave Pump Station and Coyote Point Pump Station, which discharge to San Francisco Bay. A small area of the City drains directly to Polhemus Creek and San Mateo Creek. Most of the City (11.26 square miles) drains to Marina Lagoon through a mixture of closed conduit and open channel conveyances. Two dams exist at the upstream end of Laurel Creek capturing runoff from 1.75 square miles.

Runoff from more heavily developed, low-lying areas is more dominantly managed by closed conduit storm drain systems that discharge to the four major pump stations (Marina Lagoon, Detroit Ave, Poplar, and Coyote Point) responsible for discharging stormwater to San Francisco Bay.

2.4 Climate

The study area has a climate consisting of long, dry summers with a short, cold, and wet winter. The average annual high temperature is 73°F, and the average annual low temperature is 42°F. While mean annual precipitation varies throughout the study area, the City-wide average is about 34 inches per year. Most of the rainfall occurs during fall and winter months (November through March).

2.5 History of Drainage Issues and Flooding

Historical floods are well documented in San Mateo County, with 1955 and New Year's Eve 2022 being the most prominent. San Mateo Creek, which is understood to lack capacity for large storm events at El Camino Real, overtopped its banks in 1955 and 1958.

The US Army Corps of Engineers report on the 1955 floods described San Mateo Creek overflowing its banks at Highway 101, inundating the first floor of a nearby hospital, the approach to the San Mateo-Hayward bridge, and flooding in the low-lying North Shoreview neighborhood, where the pump station could not handle the volume of flood water from the storm. The San Mateo Creek levees also saturated and began to seep water through, inundating areas south of the creek, with up to three feet of water. The event represented the damage potential of a large storm during an elevated tide condition.

The City has undertaken numerous projects recently to improve protection against both coastal and interior flooding that could resemble the 1955 event, including elevating the Bayfront Levee between Coyote Point and San Mateo Creek, adding floodwalls at the downstream end of San Mateo Creek, and elevating the levees around Coyote Point Pump Station. In tandem with these efforts, the City recently completed improvements that increased the capacity of Coyote Point Pump Station and Poplar Pump Station to reduce interior flooding. County projects improving the Coyote Point Promenade have also been completed, providing protection for some areas of the City.

In the 1970s and 1980s, the two dams at the upstream end of Laurel Creek were overtopped, damaging homes downstream.

During the New Year's Eve 2022 event, more than 4.5 inches of precipitation fell in 12 hours (measured at City of San Mateo precipitation gage AU981), representing about a 200-year, 12-hour rainfall depth. That storm lacked the short-duration, high intensity rainfall primarily responsible for overwhelming conveyance systems with smaller local drainage areas. However, it still caused flooding in areas with known drainage issues, and the volume of runoff during the storm affected Marina Lagoon and its pumps (Figure 2-4). Other areas known to be susceptible to flooding also experienced flooding during the storm, including Sunnybrae and San Mateo Creek (Figure 2-5), as well as the 42nd Ave railroad underpass (Figure 2-6).



Source: Provided to the City of San Mateo by Resident (Kanda)

Figure 2-4: Flooding over a Trail Adjacent to Marina Lagoon During the New Year's Eve 2022 Storm Event



Figure 2-5: Flooding During the New Year's Eve 2022 Storm Event on Arroyo Court (top) and in the Sunnybrae Neighborhood (Bottom)

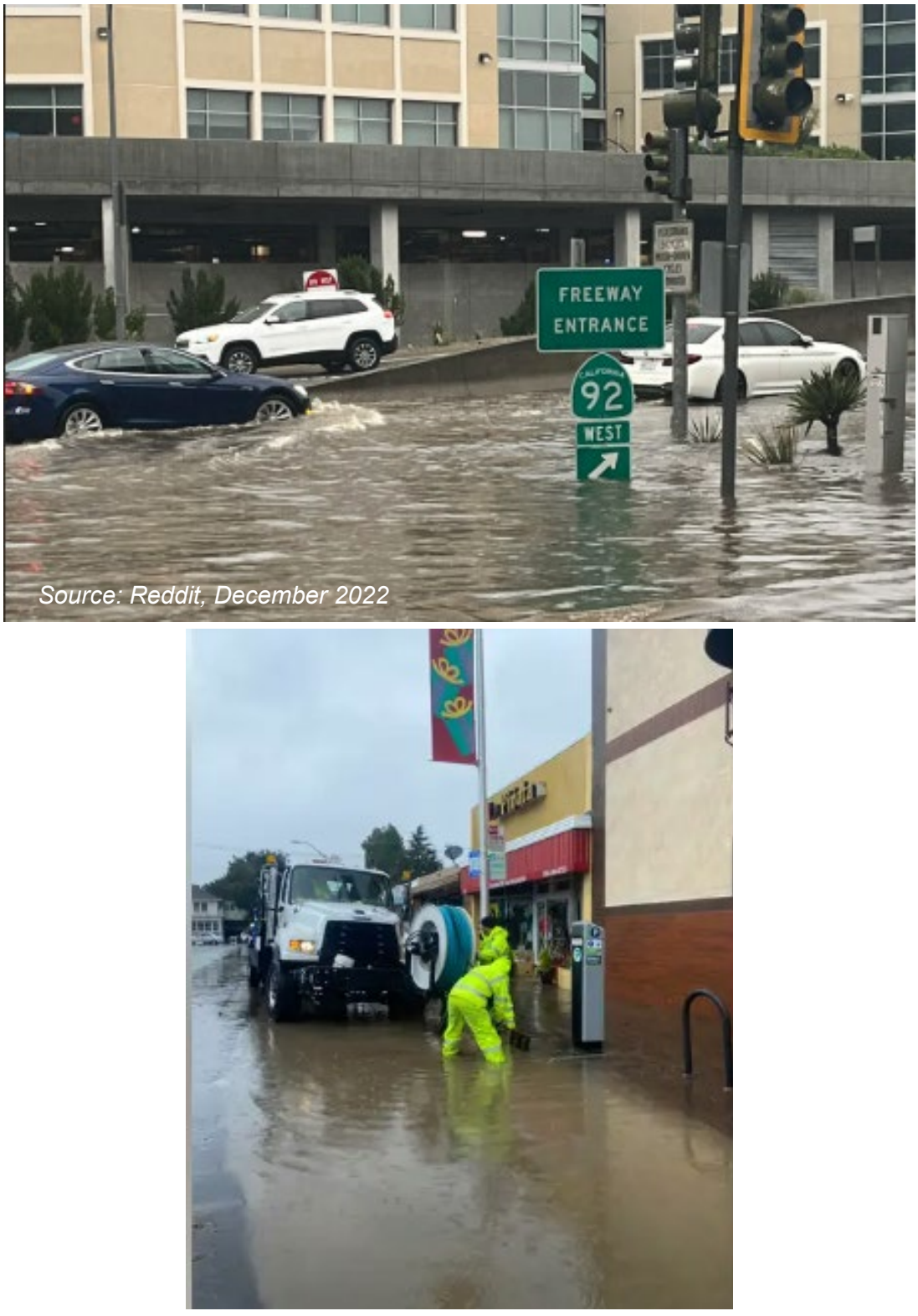


Figure 2-6: Flooding During the New Year's Eve 2022 Storm Event at Concar Dr at the Highway 92 West On-Ramp (Top Left) and B St (Bottom)

City staff identified some known, recurring problems or deficiencies that have either been observed directly or reported by residents. The models have been used to confirm the reason for those issues and to identify other areas likely to experience significant flooding during a design storm event. These are shown in Figure 2-7 and summarized in Table 2-1. For the purposes of this master plan, "ponding" refers to the collection of stormwater in depressed areas, such as low-lying sag points in gutters. This may occur where no inlet exists, or where an inlet is present but not removing stormwater from the streets effectively due to several potential factors (e.g. blockage, undersized pipe, or a high water condition at an outfall).

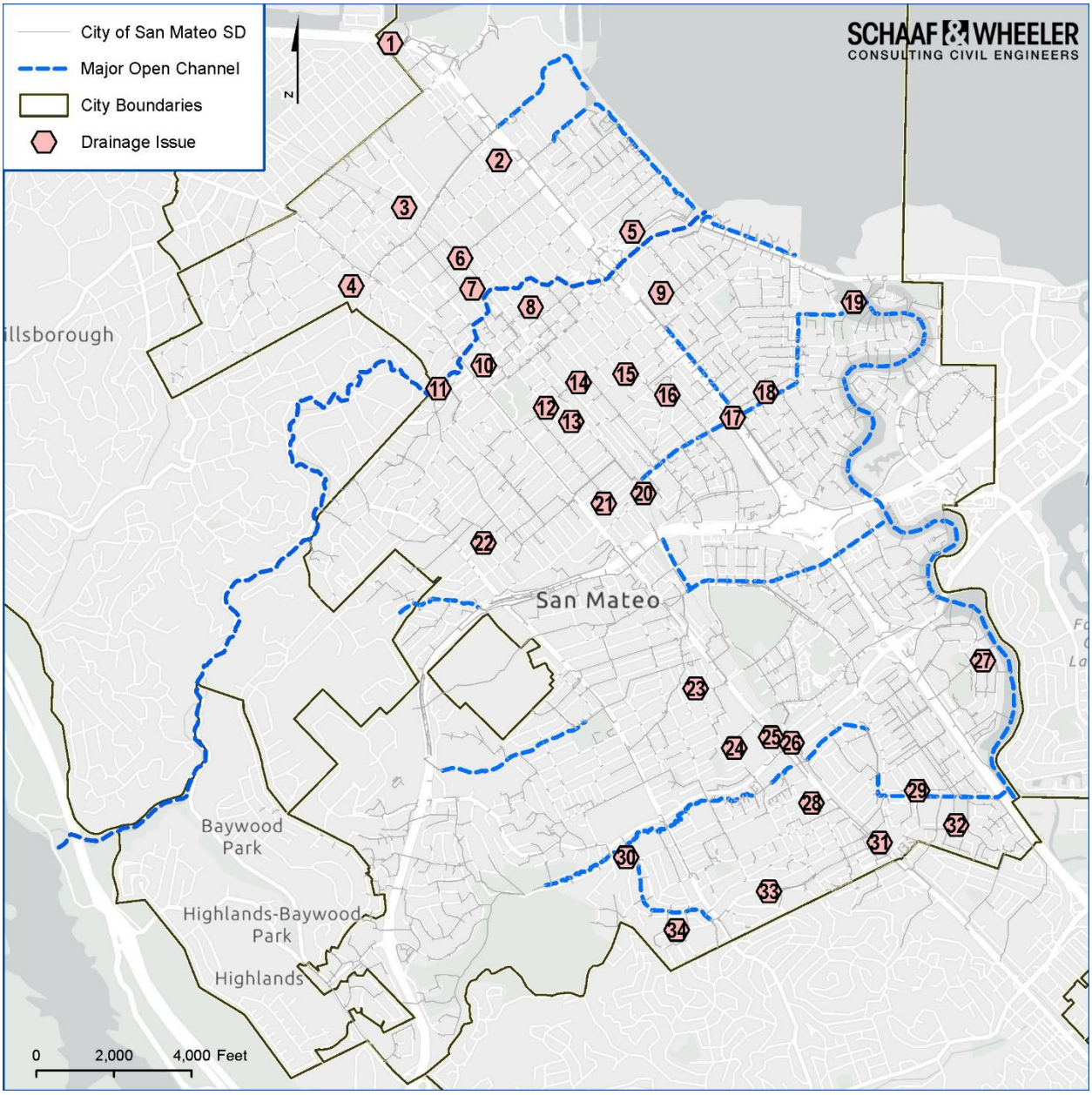


Figure 2-7: Known or Suspected City of San Mateo Drainage System Issues

Table 2-1: City of San Mateo Storm Drainage Deficiencies and Recurring Issues

Issue	Description	Category
1	Surcharge from neighboring Burlingame system flows along Amphlett to City of San Mateo	Known; Recurring
2	Models show potential flooding >1-ft. deep during 100-yr storm at Indian Ave and N Humboldt St	Modeled (100-yr)
3	City Staff note drainage issues at E Bellevue RR crossing	Known; Recurring
4	City Staff note drainage issues at W Poplar Ave, where bubble-up systems manage drainage	Known; Recurring
5	Alleyway near 228 S Norfolk St experiences ponding during storms on a frequent basis and is unable to drain for long periods of time	Known; Recurring
6	Models show potential flooding of >1-ft deep during 10-yr storm at railroad underpasses at Tilton Ave, Monte Diablo Ave, and E Santa Inez Ave	Modeled (10-yr)
7	Flooding was observed on S B St during the New Year's Eve 2022 storm event. Modeling indicates the existing pipe system lacks capacity to capture and convey peak runoff from large storms	Observed (NYE); Modeled (10-yr)
8	Bubble-up systems at 2nd and Delaware cause excessive runoff accumulation in street gutters. The City is actively working on connecting the bubble-up at the intersection to a nearby pipe system	Known; Recurring
9	Models show flat pipe systems around Patricia Ave that are causing flooding of >0.5 ft during 10-yr storm	Modeled (10-yr)
10	City Staff note drainage issues on 3rd Ave near Draper University	Known; Recurring
11	Flooding from San Mateo Creek was observed at Arroyo Court during the New Year's Eve 2022 storm event. San Mateo Creek also spills upstream of El Camino Real during 100-year events (indicated by modeling spills from Crystal Springs Dam), with overflows accumulating at Highway 101	Observed - NYE
12	City Staff note drainage issues and/or need to enlarge inlets on 9th Ave at Laurel Ave (bubble-up systems)	Known; Recurring
13	Rosewood Dr near 10th Ave: residents report that the existing bubble-up system does not adequately drain runoff from the street	Known; Recurring
14	City Staff note drainage issues at 9th Ave and S Claremont St	Known; Recurring
15	Bubble-up systems at 10th Ave and Grant St cause excessive runup accumulation in gutters in the neighborhood	Known; Recurring
16	Systems surrounding Sunnybrae Blvd and Grant St are undersized, with flooding occurring during small storm events	Known; Recurring
17	Highway 101 culvert (5'x10' box) at 16th Ave Channel is undersized and causes backwater into adjacent neighborhoods	Known; Recurring
18	Models show potential flooding >1-ft deep along Lodi St near 16th Ave Channel during 100-yr storm event	Modeled (100-yr)
19	Marina Lagoon Pump Station is aged and equipment requires replacement. New equipment should align better with the needs of the City in operating the lagoon as both a flood control and recreational facility	Known; Maintenance
20	City Staff note drainage issues at the end of 17th Ave on the west side of the railroad	Known; Recurring
21	An obstructed gutter near 40 and 44 16th Ave that is causing ponding in the street was reported by residents	Known; Recurring

Issue	Description	Category
22	City Staff note drainage issues along Hobart from Alameda de las Pulgas to Quince St. Excess gutter flows at the Alameda/Quince intersection travel to inlets at low points, which do not have sufficient capacity to drain accumulated flow/volume	Known; Recurring
23	City Staff note drainage issues on Flores St at 27th Ave and 28th Ave	Known; Recurring
24	City Staff note issues with flooding here at Arrowood Ln and Edison St	Known; Recurring
25	Models show potential significant flooding along the Hillsdale Shopping Center frontage during 100-year storm event that could contribute to and worsen underpass flooding	Modeled (100-yr)
26	Flooding occurs at the Hillsdale underpass at S El Camino and the railroad	Known; Recurring
27	City Staff note ponding issues at Llano St and Bahia St	Known
28	Bubble-up systems around King Lane cause excessive runoff accumulation in street gutters	Known; Recurring
29	Pipe systems surcharge at O'Neill Dr near the Casanova Pump Station, even during frequent storm events	Known; Recurring
30	The 48"-diameter East Laurel Creek culvert inlet at Laurelwood Dr surcharges frequently	Known; Recurring
31	The 42nd Ave underpass floods during large storm events. Vehicles have become incapacitated in the flooded underpass as recently as the New Year's Eve 2022 storm	Known; Recurring
32	Models show potential flooding >1-ft deep along Laurie Meadows Dr during a 10-year storm	Modeled (10-yr)
33	Residents report hillside drainage impact to properties along 42nd Ave, west of Camden Ave	Known; Recurring
34	City Staff noted issues along Kingridge Dr and 42nd Ave but did not locate a catch basin at this location	Known; Recurring

This map and list of known flooding issues is not exhaustive; it represents a current list of issues identified by City staff and public outreach efforts. In many cases, documented issues provide a litmus test for model performance and aid in prioritizing projects based on the frequency and severity of past flooding.

3. Data

Schaaf & Wheeler reviewed and used readily available land use, topographical, geological, geographical, and storm drain system data within the study area. The available data had some missing or incorrect information. This SDMP included efforts to improve and supplement City GIS data to create a more complete understanding of the City storm drain and creek system capacity for modeling purposes. This chapter summarizes the findings and data acquired as part of this storm drainage study. It also summarizes data limitations, assumptions, and impacts.

3.1 Hydrology and Storm Drain Model Data

While some field measurements of the storm drain systems was performed, surveying the entire Citywide system would have been an enormous undertaking and prohibitively expensive. Instead, assumptions and engineering judgment are applied to fill data gaps within reason.

3.1.1 Citywide Topography and Aerial Imagery

All project data and results are in vertical datum NAVD88 (feet) and the State Plane California Zone III coordinate system.

Schaaf & Wheeler created an integrated, Citywide digital elevation model from available 2017 San Mateo County light detection and ranging (LiDAR) data, 2022 LiDAR data near Coyote Pump Station, and 2024 channel and Marina Lagoon bathymetry (Figure 3-1) to aid in developing the hydrologic and hydraulic models for the system analysis. Aerial imagery available in ArcGIS was used to obtain related data, such as road networks, land use, and water bodies. Survey performed as part of this effort focused on channels and Marina Lagoon and is discussed further in Section 3.2.

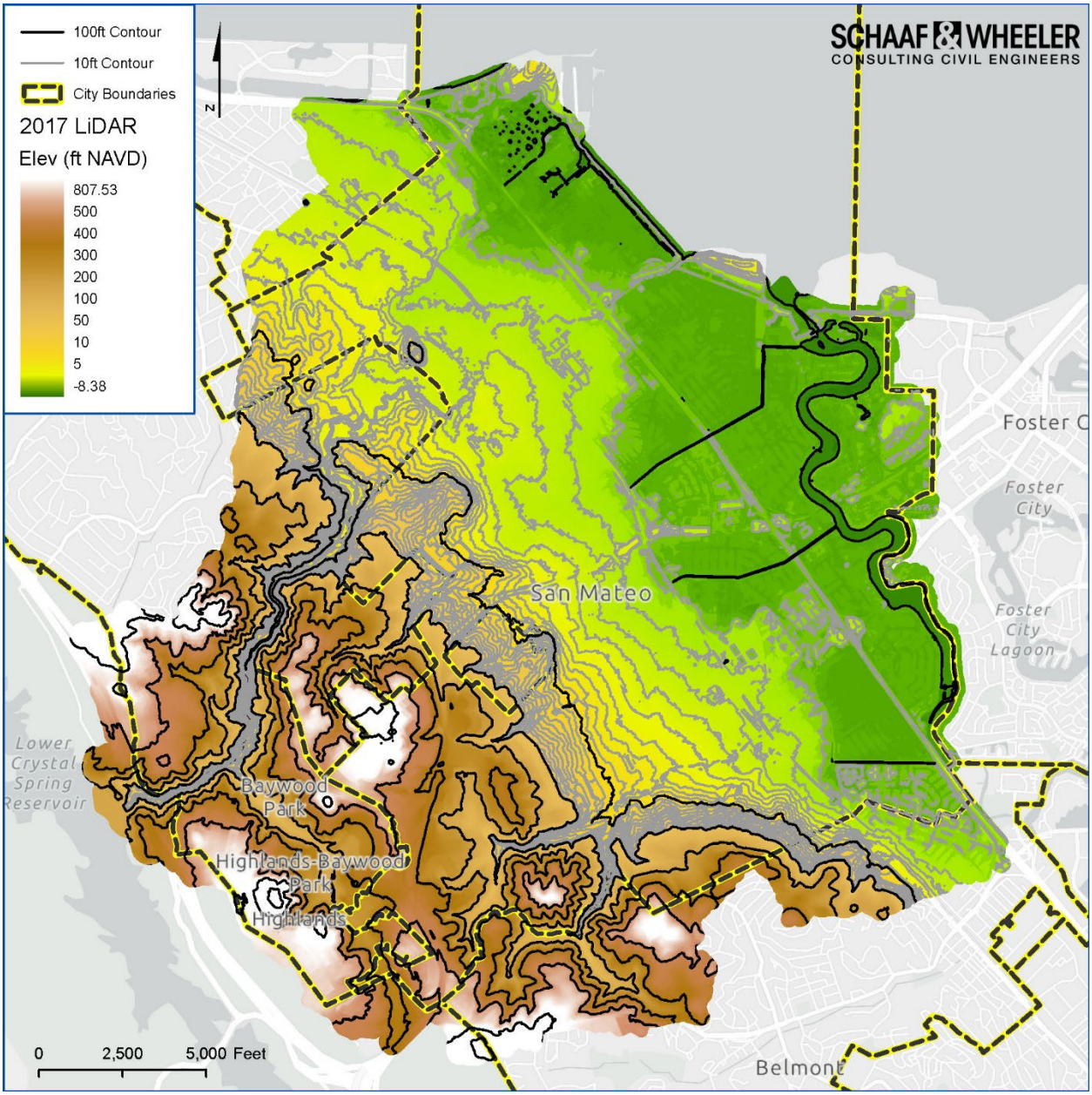


Figure 3-1: San Mateo County 2019 LiDAR Topographic Data

3.1.2 Storage Facility Topography

The topography of storage facilities, including Marina Lagoon, the two dams at the upstream end of Laurel Creek, the regional Bay Meadows detention pond, and the Poplar Pump Station forebay are included in the model to evaluate the function of connected pump stations and the attenuation of runoff from their respective tributary areas. Marina Lagoon is modeled as a linear river element with cross sections, providing a thorough hydraulic analysis of the facility, which could have some water surface gradient during a storm. This also creates a more useful model, should the City decide to use it to evaluate tide gate operations at O'Neill Slough. Where those dynamics are less important, storage elements are modeled as nodes with simpler elevation-storage curves or are included in the 2D domain (at Bay Meadows).

Elevation-storage curves were developed from a combination of 2017 LiDAR topography, recent aerial topographic survey, and prior work on the Poplar Pump Station that included information regarding the invert elevation of the forebay. The reservoirs in the Laurel Creek drainage area appear to be empty during the 2017 LiDAR flights.

The locations of lagoons and basins included in the model are shown in Figure 3-2. A general summary of basin characteristics (including depth, total volume, approximate active flood storage capacity, and surface area) are provided in Table 3-1.

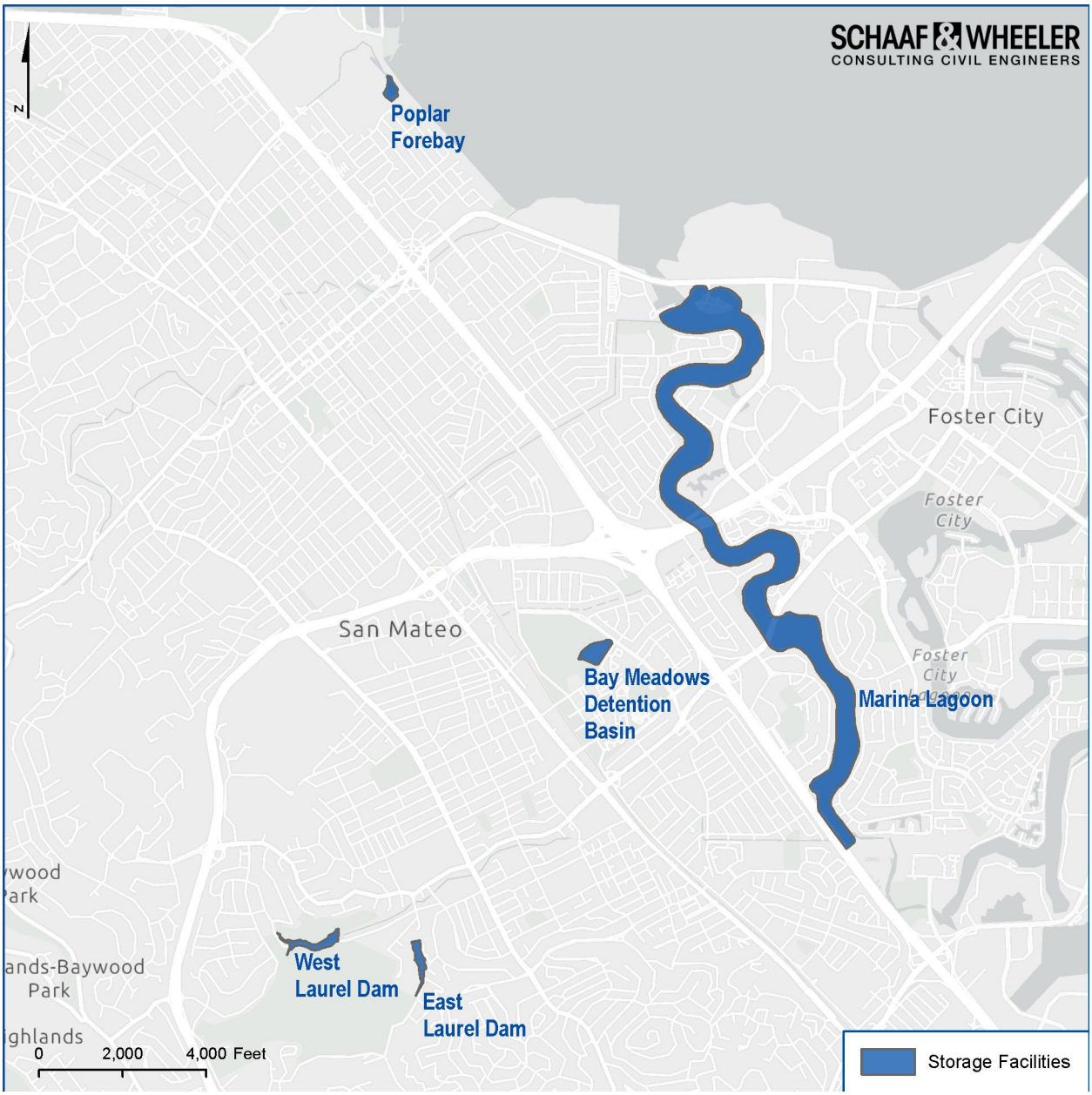


Figure 3-2: Modeled Storage Basin Locations

Table 3-1: Approximate Characteristics of Modeled Storage Basins

Basin	Approx. Area (Acres)	Depth (ft)*	Max. Water Surface (ft NAVD)*	Total Volume (AF)*	Notes
Marina Lagoon	240	8.3	2.5	927 (480 Active)	Active flood control elevation 2.5 ft NAVD (100-yr)
Bay Meadows	6.1	3.5	5.75	7.3 (5.4 Active)	1.75 ft of dead pool storage below outlet pipe
West Laurel	5.1	43	176.25	125	Assumes reservoir is full to dam crest
East Laurel	3.7	24	136.9	58	Assumes reservoir is full to dam crest
Poplar Forebay	2.7	5.5	2	17.5	Creeks act as additional storage during a storm

*Depth, Max Water Surface Elevation (WSEL), and total volume (in acre-feet) determined based on the maximum containment level without causing flooding in the immediate basin vicinity

Detailed elevation-storage curves and elevation maps are provided in Appendix A.

3.1.3 Storm Drain System Data

City staff provided available GIS and CAD data representing storm drain nodes (e.g. inlets and outfalls) and storm drain pipes and/or open channels to Schaaf & Wheeler in shapefile and CAD formats. Initial datasets contained:

- City of San Mateo storm drain system GIS
- City of San Mateo 2003 SDMP model system GIS
- Creek and major open channel centerlines
- San Mateo County storm drain system GIS (including modeled pipe systems in Belmont)

These datasets included information regarding:

- Pipe locations and lengths
- Approximate open channel centerline locations, including Marina Lagoon
- Node types (Catch Basin, Manhole, Junction, or Outfall/River Junction)
- Invert elevation or depth for 659 nodes (about 8% of modeled nodes)
- Shape and size for approximately 90% of modeled closed conduit elements

A map of the entire City storm drain system GIS is shown in Figure 3-3.

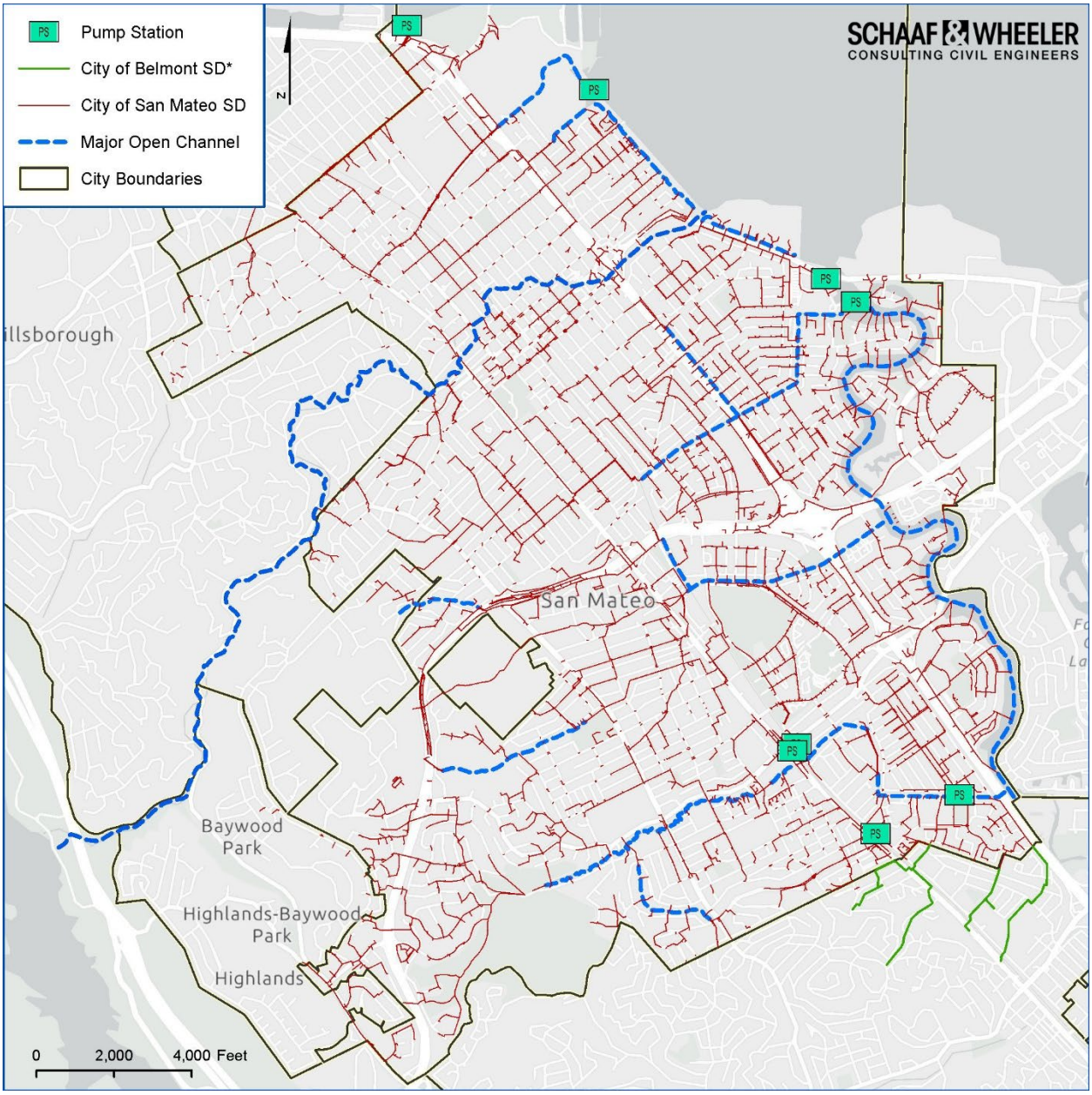


Figure 3-3: City Storm Drainage Network Map

The modeled existing system consists of approximately 8,500 nodes; more than 148 miles of closed conduit (7,470 elements); and 11.2 miles of open channels (creeks, ditches, and Marina Lagoon).

Schaaf & Wheeler identified missing data as well as items in need of verification. Information needed to create the model for analysis included:

- Pipe and culverts that are missing from City GIS (including some Caltrans systems)
- City of Belmont systems that drain into City systems
- Verified pipe diameters
- Node depth and rim elevations

Nodes were assigned ground elevations based on as-builts and LiDAR topography. Node inverts were assigned based on depths in the GIS set for the 155 nodes where that information was field-collected. As-builts were used to fill in additional invert elevations. Because datums varied and/or were not listed in many of the as-built drawings, depths were calculated for various structures and used to assign the invert in the model based on the LiDAR ground elevation. This ensured that a uniform datum was applied to the entire model.

It is not practical or necessary to obtain measurements for every bit of remaining missing data for storm system modeling purposes. For information still missing after exhausting available data sources, system ends were initially assigned a depth to achieve a minimum of two feet of cover over the largest connected pipe, unless deviation was required to work around various constraints (approximately 1,770 nodes). The DHI software model (discussed in detail in Section 4) was then used to interpolate about 17,730 missing invert elevations throughout the entire model. Interpolated values were constantly reviewed and manually adjusted as the model was developed. Conditions that warranted adjustment included ensuring that system profiles were reasonable (i.e. they achieve a positive slope, unless measurements or as-builts indicate otherwise), and open channel elements had inverts assigned based on other available data.

Modeling requires certain information for each element in order to route runoff and evaluate system capacity. This is discussed in greater detail in Section 4. Where practical, missing pipe diameters were estimated based on the size of surrounding pipes, using the smallest diameter of connecting pipe at the middle of a continuous run or using the diameter of the next pipe downstream following a junction of multiple systems upstream.

Upon examination of system profiles in the model, it became apparent that some areas of the system are likely shallower than the two-foot cover assumption. This was confirmed in some areas with as-built drawings. In these locations, inverts were corrected and re-interpolated, as necessary, to create sensible profiles to complete the hydraulic model. There were also notable irregularities in the existing GIS data, including pipes that appeared to be missing or assigned incorrect diameters, and these were investigated in the field and updated. Field investigations are further discussed in Section 3.2.

Schaaf & Wheeler conducted a number of field visits in 2024 to perform targeted data collection and verification. During the field visits, invert depths and pipe diameters were noted and photos were taken to help improve the gravity system model's accuracy. Photos of creek crossings and channel conditions were collected where necessary to improve open channel model inputs.

3.1.4 Land Cover and Soil Characteristics

The hydrologic parameters discussed in Section 4 are required to calculate the amount of rainfall that is expected to enter stormwater systems as runoff during modeled storm events. Estimating the hydrologic parameters of the City system's drainage areas requires a characterization of land cover and consideration of soil characteristics. Soil information was acquired from the United States Department of Agriculture (USDA) Soil Survey Geographic Database (SSURGO).

A map of Hydrologic Soil Groups assigned to the soils in the SSURGO database is provided in Figure 3-4.

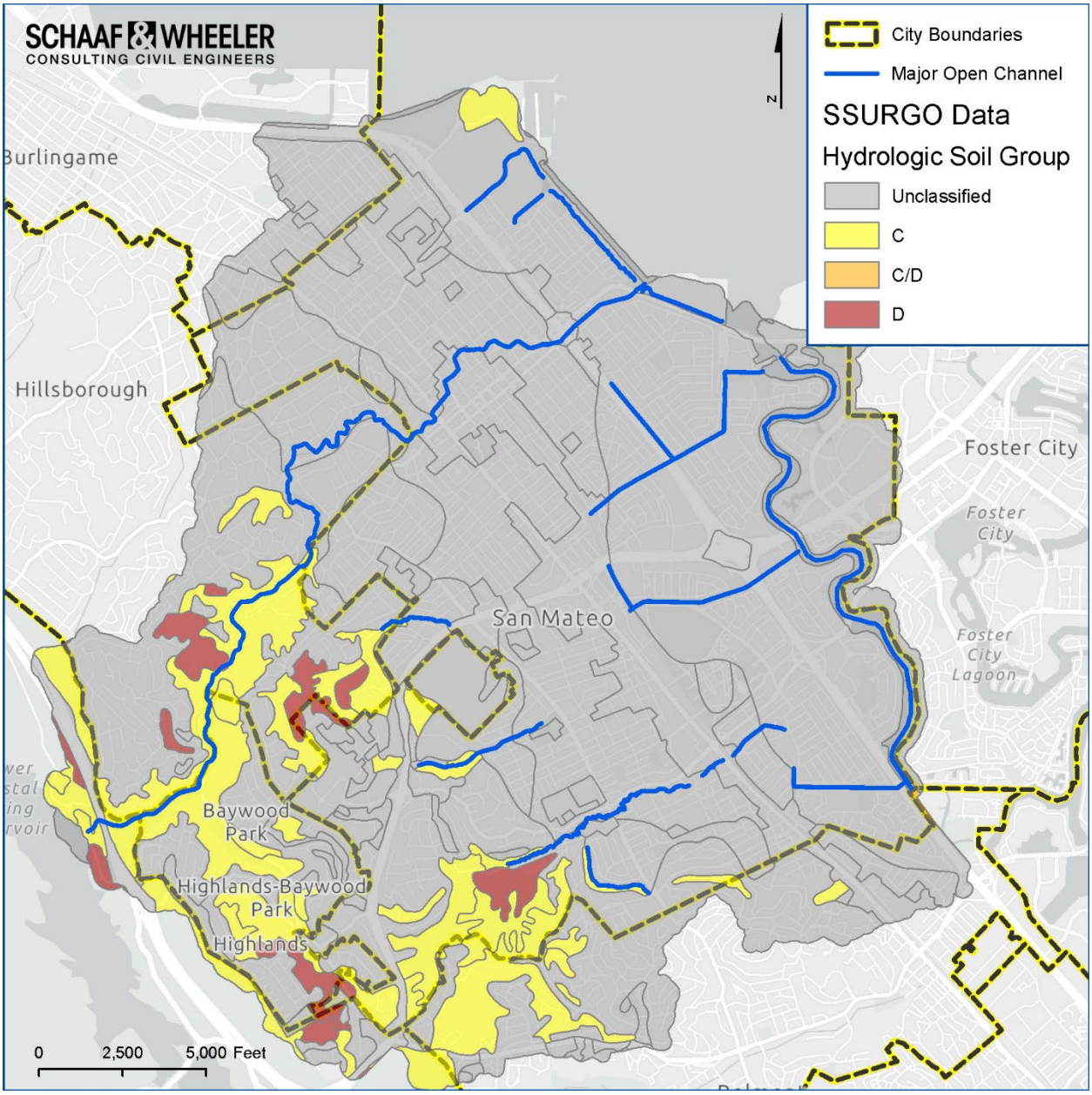


Figure 3-4: Hydrologic Soil Group Classifications Used for Hydrologic Modeling

Land use data was defined based on a combination of aerial imagery and local jurisdiction parcel information to reflect conditions affecting hydrology as accurately as possible. The City's storm drain system captures drainage from the City of Burlingame, City of Hillsborough, City of Belmont, and small areas of Foster City. Because drainage from other surrounding cities and towns enters City systems, land cover data had to be reviewed and composited to create a continuous dataset covering all drainage areas in the SDMP.

This effort required some reclassification to bring varied or inconsistent land cover classifications into unified categories. For example, each City has its own set of single-family residential classifications, which were first generalized as a "Single-Family" category, then

assigned areas based on the GIS-calculated parcel areas (e.g. less than 5,000 square feet, 5,000-7,500 square feet, etc). Generalized zoning for parcels in the area is shown in Figure 3-5.

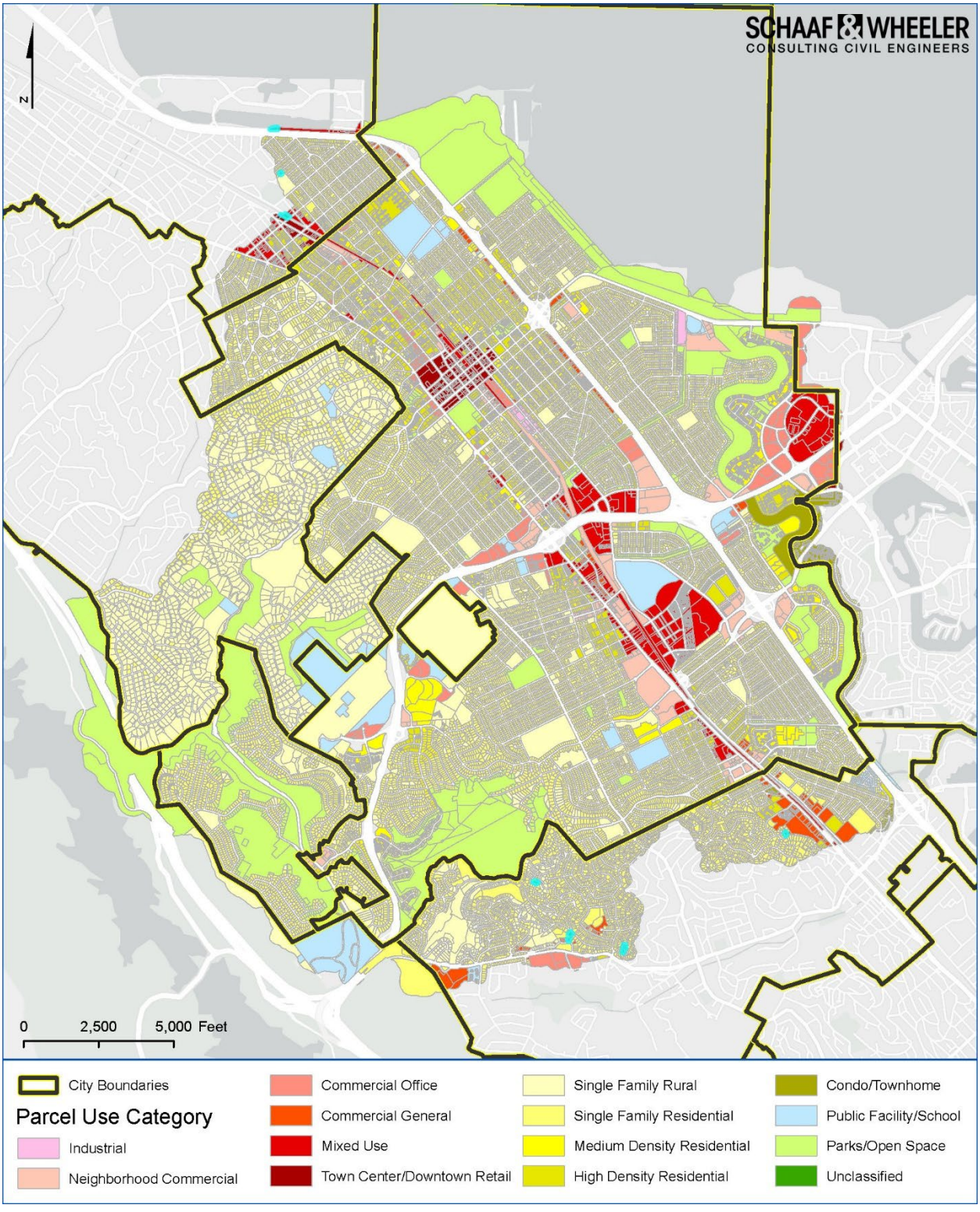


Figure 3-5: Land Use Information Used in the Development of the Hydrologic Model

Table 3-2: City of San Mateo SDMP Model Land Use and Zoning Summary (Acres)

	Hydrologic Soil Group				Total Area
Description	**C**	**C/D**	**D**	**N/A***	
Commercial General	2	–	–	87	89
Commercial Office	2	–	–	196	199
Community Commercial	–	–	–	203	203
Condo/Townhome	–	–	–	112	112
High Density Residential	<1	–	–	2,251	2,251
Industrial	–	–	–	18	18
Medium Density Residential	21	–	<1	1,478	1,499
Mixed Use	–	< 1	–	1,255	1,256
Parks/Open Space	703	2	135	941	1,781
Public Facilities	52	–	52	315	419
Right-of-Way	106	<1	9	1,226	1,341
Rural Residential	505	–	58	1,419	1,983
Service Commercial	7	–	–	59	66
Single-Family Residential	142	–	5	3,418	3,565
Town Center/Downtown Retail	–	–	–	68	68
Unclassified	<1	–	–	2	2
Grand Total	**1,540**	**2**	**259**	**13,048**	**14,851**

*Hydrologic Soil Group N/A represents uncategorized soil groups, primarily consisting of urban complexes

3.1.5 Impervious Surface

Imperviousness is a crucial factor in estimating runoff from catchment surfaces. Highly impervious urban areas shed far more rainfall than pervious surfaces. For this SDMP, impervious percentages have been estimated for each land cover type based on available data. It is assumed that the average impervious percentage estimated for each land cover type will produce a reasonable estimate of runoff when applied to all parcels in the SDMP drainage areas.

The National Oceanic and Atmospheric Administration (NOAA) Coastal Change Analysis Program (C-CAP) produces gridded remote sensing data that includes impervious surface percentages within each grid cell. This data is used as a foundation for estimating an average percent impervious for each land cover type in the composite land cover dataset using GIS spatial analysis tools.

In order to account for variability in development standards across jurisdictions, land cover classes have been analyzed for impervious surface percentage separately for each City or Town within the drainage area. This analysis has been performed on a 500-foot buffer around the SDMP drainage area.

Table 3-3: City of Belmont Land Use Classes, Area, and Average C-CAP Impervious Percentages

Description	Area Evaluated (Acres)	C-CAP Percent Impervious
Commercial	33.0	90%
Commercial, Special Setback	2.5	35%
Duplex 3,000 sf	8.0	48%
Executive Office & Warehouse	17.9	51%
Hillside Res & Open Space, Subdivided	75.2	8%
Hillside Res & Open Space, Unsubdivided	58.8	25%
Multi-Family Residential 1,450 sf	2.5	88%
Multi-Family Residential 1,950 sf	24.1	54%
Neighborhood Commercial	1.4	36%
Planned Unit Development	21.0	54%
Service Commercial	32.4	43%
Single-Family <5,000 sf	97.8	58%
Single-Family 5,000–6,000 sf	495.9	45%
Single-Family 6,000–9,600 sf	15.8	44%
Single-Family 9,600 sf–1 acre	0.65	1%

Table 3-4: City of Burlingame Land Use Classes, Area, and Average C-CAP Impervious Percentages

Description	Area Evaluated (Acres)	C-CAP Percent Impervious
Anza Area	3.53	0%
Anza Point South	3.08	86%
Bayswater Mixed	9.13	84%
Bayswater Mixed, High Density Multifamily Residential	0.82	92%
Burlingame Ave Commercial	0.54	92%
California Auto Row	10.14	92%
High Density Multifamily Residential	17.91	85%
Howard Mixed Use	7.19	96%
Medium Density Multifamily Residential	20.21	78%
Myrtle Mixed Use	6.00	83%
Single-Family < 5000 sf	3.38	56%
Single-Family 5,000–7,500 sf	44.17	59%
Single-Family 7,500 sf–1/4 acre	98.05	56%
Single-Family 1/4–1/2 acre	19.29	61%
Single-Family Dwellings, Medium Density Multifamily	2.82	73%
Single-Family Rural (>1/2 acre)	7.64	35%
Unclassified	0.54	62%
Unclassified, Anza Area, Dept of Transportation	0.38	0%

Table 3-5: City of Foster City Land Use Classes, Area, and Average C-CAP Impervious Percentages

Description	Area Evaluated (Acres)	C-CAP Percent Impervious
Condo	28.2	87%
Open Space	17.4	9%
ROP	28.0	59%
Single-Family <5000 sf	0.3	62%
Single-Family 5,000–7,500 sf	3.7	63%
Single-Family 7,500 sf–1/4 acre	5.0	58%
Single-Family 1/4–1/2 acre	9.5	53%
Single-Family Rural (>1/2 acre)	1.9	49%
Town Center	4.2	69%
Townhouse	19.0	70%
Water	3.3	30%

Table 3-6: Town of Hillsborough Land Use Classes, Area, and Average C-CAP Impervious Percentages

Description	Area Evaluated (Acres)	C-CAP Percent Impervious
Parks/Open Space	71.6	0%
Public Facilities	16.1	43%
School	10.2	38%
Single-Family < 5000 sf	1.4	20%
Single-Family 5,000 – 7,500 sf	1.2	29%
Single-Family 7,500 sf - 1/4 acre	4.1	46%
Single-Family 1/4 - 1/2 acre	151.4	44%
Single-Family Rural (> 1/2 acre)	1,024	34%

Table 3-7: City of San Mateo and Unincorporated Land Use Classes, Area, and Average C-CAP Impervious Percentages

Description	Area Evaluated (Acres)	C-CAP Percent Impervious
Downtown Retail Core	40.0	95%
Downtown Retail Core Support	13.8	87%
Executive Office	137.1	85%
Executive Office/High Density Multi-Family	30.9	90%
High Density Multi-Family	371.4	85%
Low Density Multi-Family	383.9	72%
Major Institution/Special Facility	200.1	39%
Manufacturing	17.7	77%
Medium Density Multi-Family	260.2	81%
Neighborhood Commercial	54.8	90%
Neighborhood Commercial/ High Density Multi-Family	25.6	88%
Parks/Open Space	1,782	45%
Public Facility	150.3	56%
Regional/Community Commercial	204.7	85%
Regional/Community Commercial/ High Density Multi-Family	111.4	87%
Service Commercial	34.0	97%
Single-Family <5000 sf	147.4	61%
Single-Family 5,000–7500 sf	1,453	58%
Single-Family 7,500 sf–1/4 acre	637.1	50%
Single-Family 1/4–1/2 acre	399.9	39%
Single-Family Rural (>1/2 acre)	816.9	39%
Transit-Oriented Development	148.4	88%
Transportation Corridor	63.5	82%
Utilities	22.1	58%

The greatest share of analyzed area is within City limit, followed by Foster City, Hillsborough, Belmont, and Burlingame.

Table 3-8: Imperviousness by Jurisdiction Within 500-foot Buffer Around SDMP Area (Excluding Right-of-Way)

Jurisdiction	Area Evaluated (Acres)	Area Within SDMP Catchments (Acres)	C-CAP Percent Impervious
Belmont	895	727	49%
Burlingame	255	75	66%
Foster City	120.4	<10	60%
Hillsborough	1,280	239	36%
San Mateo	7,506	6,320	67%

A map of impervious percentages by land use type in each parcel classification, with adjustments where C-CAP data shows significant variation are shown in Figure 3-6. Use of raw impervious percentages in hydrologic models can lead to overestimation of runoff volume and rates, as certain impervious surfaces may not shed runoff directly into storm drain systems. Single-family residences, for example, may shed some roof runoff into backyards, onto adjacent pervious surfaces where some runoff may infiltrate. Alameda County Hydrology Manual considers this effect by separating "directly connected" from "non-directly connected" impervious surfaces. The modeling efforts in this SDMP assume that similar portions of each property type (e.g. commercial, residential, etc.) are directly connected in San Mateo.

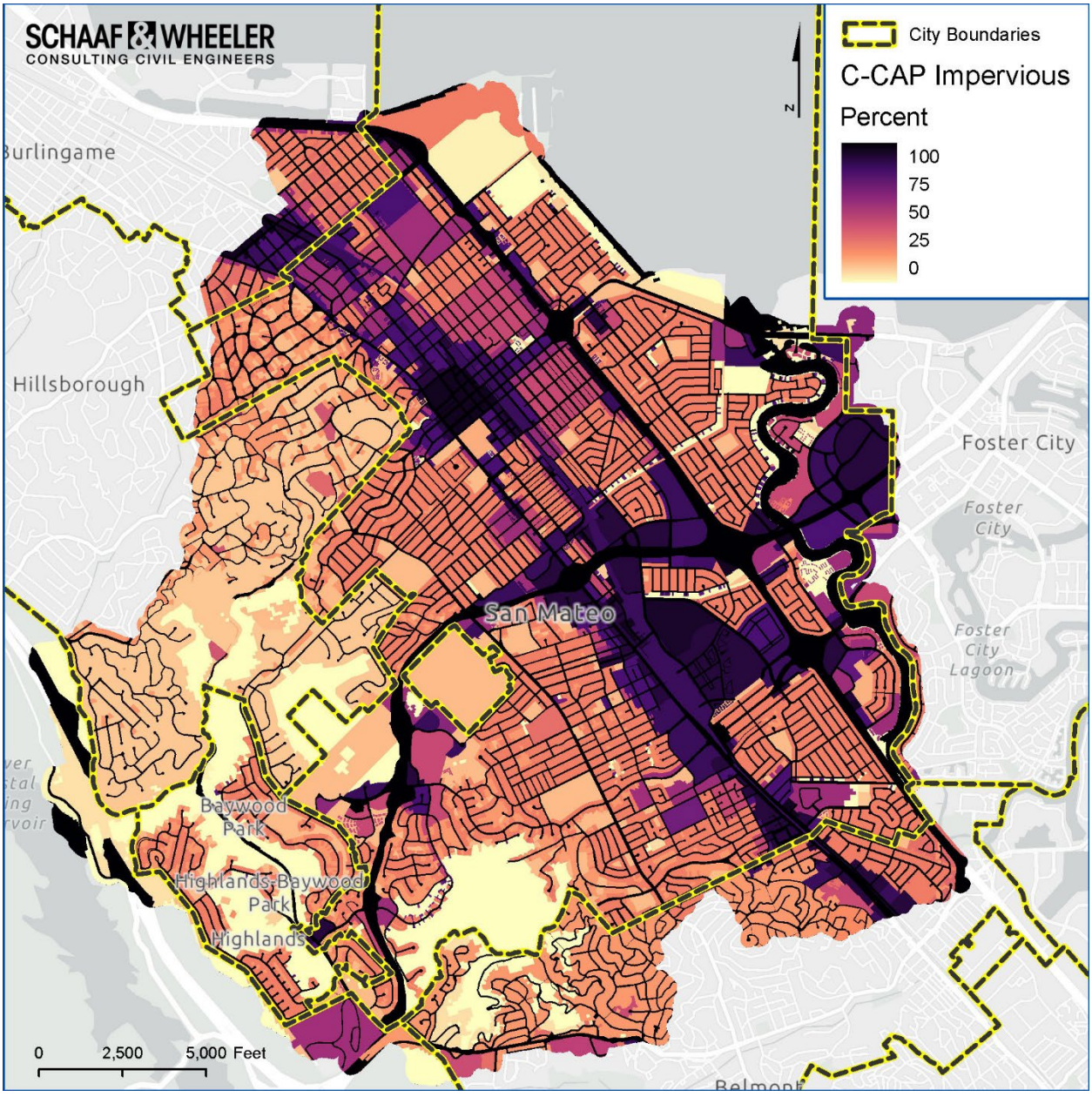


Figure 3-6: C-CAP Impervious Percentage Data Within the Evaluated Area (by Land Cover)

Curve Numbers (CN)

Natural Resources Conservation Service (NRCS) Curve Number (CN) methodology is applied to aid in estimating runoff from catchments based on soil and land cover characteristics and a parameter known as antecedent moisture condition (AMC). AMC is defined as a metric of a soil's moisture content prior to the start of a defined precipitation event. AMC is characterized by the NRCS as:

AMC I Soils are Dry

AMC II Soil Moisture is Average

AMC III Soils are Saturated after Heavy Rainfall

CNs vary from 0 to 100, with 0 representing no runoff, and 100 meaning that all precipitation will run off.

A summary of select CNs adapted from the National Engineering Handbook Part 630, Chapter 9, "Hydrologic Soil-Cover Complexes" (USDA, 2004) is provided in Table 3-9. These values have been assigned to GIS polygons based on land cover classifications and converted to a gridded raster dataset shown in Figure 3-7.

Table 3-9: Pervious AMC II CN Summary by Land Use & Soil Type

	Hydrologic Soil Group			
Description	**A**	**B**	**C**	**D**
Developed Covers (Commercial, Single-Family, etc)	44	58	71	74
Open Space	35	51	65	72
Evergreen Forest	45	57	69	80
Mixed Forest	40	52	63	72
Emergent Herbaceous Wetlands	98	98	98	98
Open Water	98	98	98	98
Right-of-Way	44	58	71	74
Shrub/Scrub	35	51	65	72
Grassland	48	60	74	80

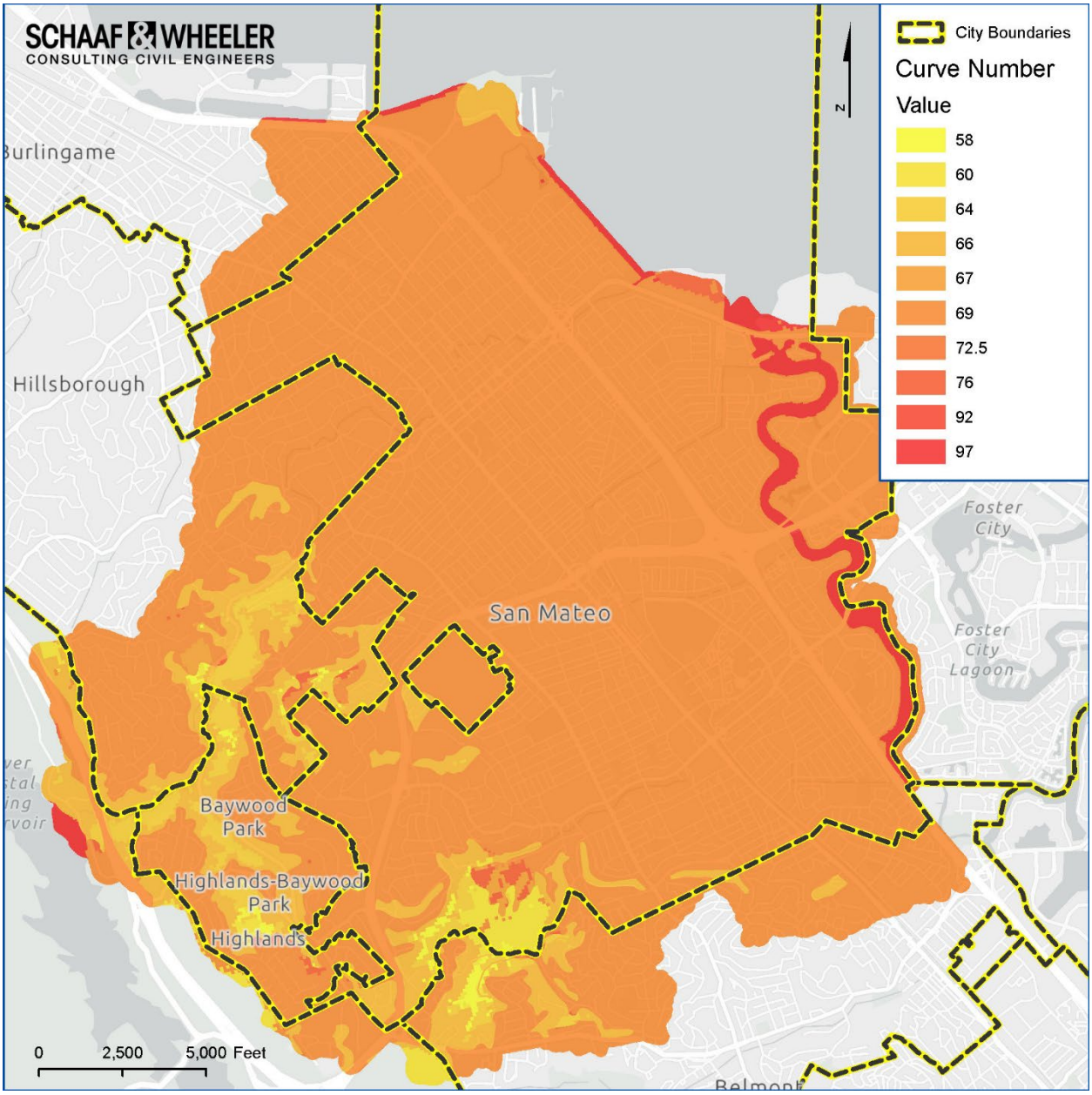


Figure 3-7: Pervious Area AMC 1.75 CN Map in the San Mateo SDMP Model Vicinity

3.1.7 Pump Stations

There are ten pump stations in the City's hydraulic model. Four of the stations are responsible for discharging stormwater from substantial areas of the City directly into San Francisco Bay (Coyote Point, Poplar, 3rd Ave/Detroit, and Marina Lagoon). The other stations discharge stormwater from smaller areas into Laurel Creek and Marina Lagoon (31st Ave, El Camino Real Hillsdale Underpass, Hillsdale, Casanova, and 4000 Pacific).

The Fathom Dr Station is primarily meant to drain a small wetland between Fathom Dr and 3rd Ave. This station has minimal drainage area, no backup power, and is not considered in the SDMP model.

The 16th Ave Channel Station near Marina Lagoon is only active in the summer months. This station drains low flows from the 16th Ave Channel, and the nearby tide gate prevents backflow from Marina Lagoon into the Channel. It is not relevant during winter design storm events and is, therefore, not included in the SDMP models.

Pump station data, including number of pumps, type of pump, approximate capacity, and age, are summarized in Table 3-10.

Table 3-10: San Mateo Pump Stations Summary

Pump Station	Originally Constructed	Last Updated	Number of Pumps	Approximate Capacity (cfs)	Backup Power
3rd & Detroit	1983	--	2	89	Generator
16th Ave	1983	--	1	10	N/A
31st Ave	2021	--	2	3.7	N/A
4000 Pacific	1988	--	2 + Sump	12	Manual
Casanova	1968	2004	4 + Sump	100	Manual
Coyote	1973	2021	3	300	Generator
El Camino Real*	--	2006	2	5	Manual
Fathom Dr	1966	2016	2	7	N/A
Hillsdale	1964	--	2	15	N/A
Marina Lagoon	1983	--	5	1,670	Generator
Poplar Ave	1973	2021	4	300	Generator

*El Camino Real is a Caltrans pump station. It is modeled, but maintenance and operation is not a City responsibility. Original construction date is not known.

The Coyote and Poplar Pump Stations were completely reconstructed in 2021, with emergency generators and electrical equipment elevated above the 100-year base flood elevations. This update was part of the North Shoreview flood protection project, which included improvements to levee systems to achieve accreditation.

Drainage areas to the pump stations are shown in Figure 3-8.

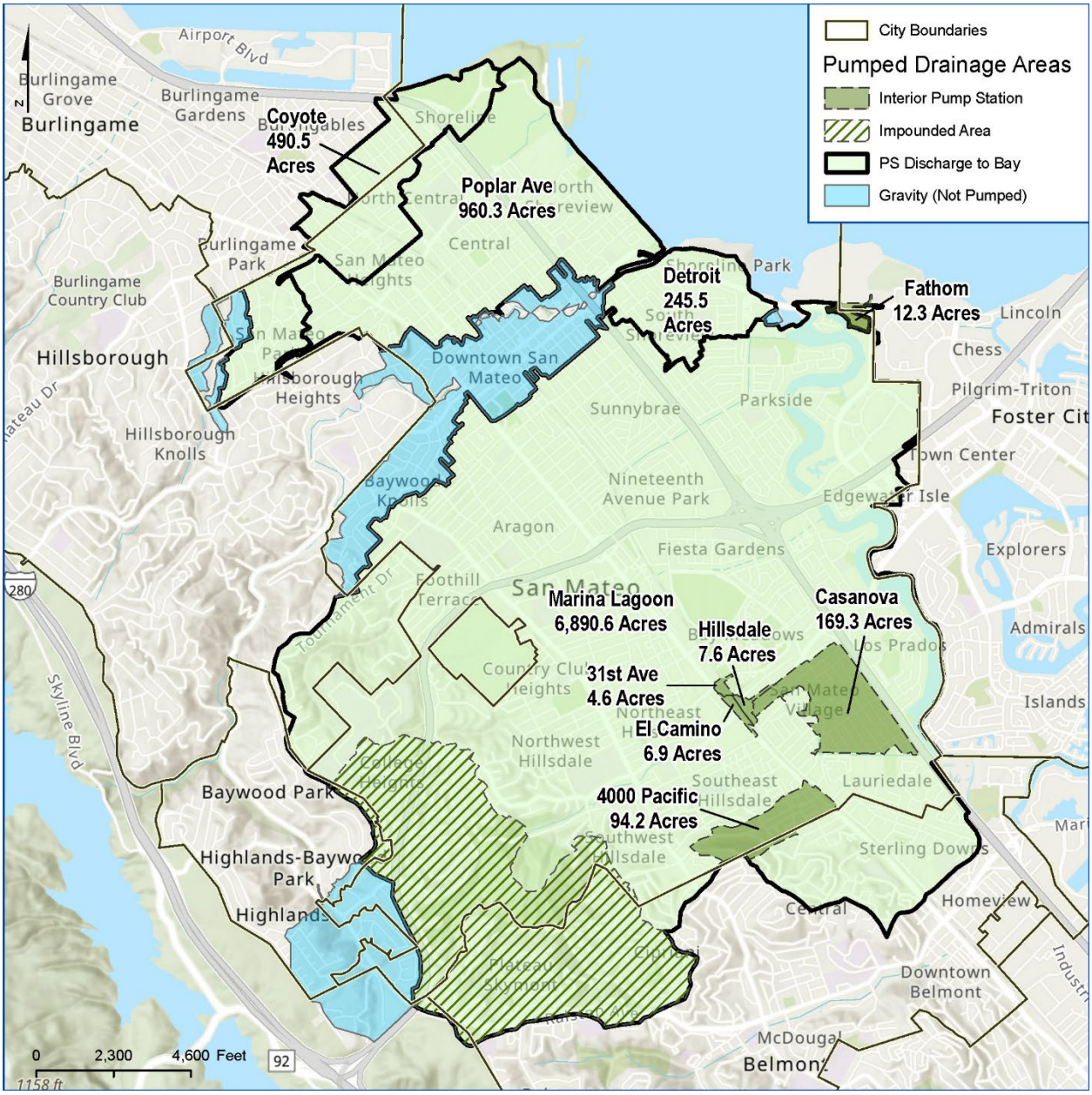


Figure 3-8: Pump Station Drainage Areas

Schaaf & Wheeler performed a full assessment of the condition and function of these pump stations and associated tide gate structures as part of this SDMP effort. More details on the City's pump stations can be found in Chapter 5. System curves collected as part of the pump station assessment effort were used as a basis for developing model pump curve inputs. However, not all elements of the pump systems were included in the storm drain models. Instead, system curves were "de-rated," modifying the pump curves based on calculated losses at each flow rate. The de-rated pump curves and calculations are provided in Appendix B.

3.1.8 Precipitation Statistics

Storm drain master planning efforts rely on the evaluation of design precipitation events. These model storms are "statistically balanced," idealized storm events of a particular return period. For example, a 10-year, 24-hour balanced storm time series includes 10-year depths for all durations within the storm, starting from the smallest time increment (e.g. 15-minute, 30-minute, 60-minute, 2-hour, 3-hour, 6-hour, 12-hour, and 24-hour periods).

To develop statistically balanced rainfall patterns requires an understanding of precipitation depths on various return periods and time intervals in San Mateo. In 2016, MetStat, Inc. completed a precipitation frequency analysis covering Santa Clara, Alameda, and San Mateo Counties. The product of that analysis was gridded precipitation frequency data at about 4,000-feet resolution. These grids capture spatial and temporal variability in rainfall characteristics in San Mateo and provide the necessary information to develop statistically balanced storm inputs for a variety of return periods (10-, 25-, and 100-year events).

3.1.9 Tidal Data

Stormwater runoff from large portions of the City is managed by pipe systems and channels with outfalls that are either tidally influenced or completely exposed to tidal conditions.

Gravity outfalls to the Bay and other tidally influenced areas must be assigned boundary conditions. In order to evaluate the impact of the tide on these systems, Schaaf & Wheeler extracted information from the NOAA Tides and Currents system and from the FEMA 2015 San Francisco Bay Tidal Datums and Extreme Tides Study to develop a harmonic tide cycle used as a basis for model boundary conditions.

The tide cycle was developed based on tidal datums: Higher High Water (HHW), High Water (HW), Low Water (LW), and Lower Low Water (LLW). The SF Bay Tidal Datums GIS point dataset was used to establish the tidal datum points near Coyote Point. The approximately 24-hour harmonic tide cycles are shown in Figure 3-9. This cycle is repeated and shifted as necessary to cover the duration of the 72-hour storm, and to place the HHW peak at a point where it is as close to coincident as possible with peak runoff for each model scenario.

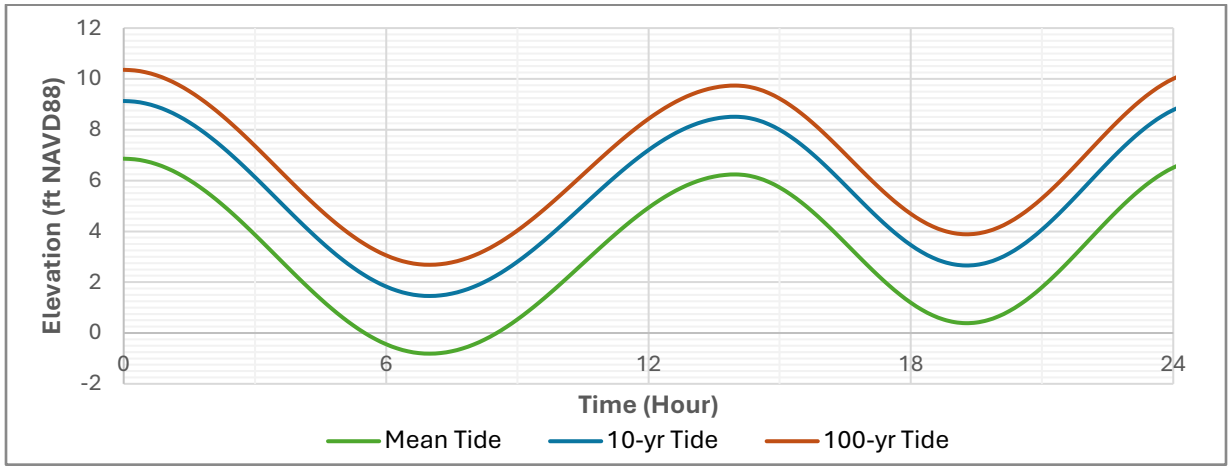


Figure 3-9: Tide Cycle Boundary Conditions (Based on the 2015 FEMA Tide Study)

Tidal datums and extreme tide elevation for each cycle (mean, 10-year, and 100-year) are summarized in Table 3-11.

Table 3-11: Existing Tidal Datum Elevations

Harmonic Tide Point	Mean Tide	10-year	100-year
HHW	6.86	9.13	10.36
LLW	-0.81	1.46	2.69
HW	6.24	8.51	9.74
LW	0.39	2.66	3.89

Statistical tidal benchmark points are applied in this study, yet in reality, diurnal tide varies throughout the year. The highest tides of the year are known colloquially as "king tide" events, occurring when the sun and moon's gravitational pull on the Earth is as its strongest.

Terminal pump stations, including Detroit Ave, Coyote, Poplar, and Marina Lagoon, discharge to the Bay, with outfalls that can be partially or fully submerged by high tide conditions. On the southeast end of Marina Lagoon, the O'Neill Slough Tide Gate is used to regulate levels in the Lagoon in conjunction with Marina Lagoon Pump Station. The gate can be opened to allow full exposure to tidal fluctuations and maintain water quality conditions, or it can be closed and the level drawn down to maintain flood control storage in the winter months. The gate was reconstructed in 2006. A photo of the structure taken shortly after the reconstruction is shown in Figure 3-10.



Figure 3-10: O'Neill Slough Tide Gate, Shortly After Reconstruction in 2006

3.1.10 Surface Roughness

Surface roughness, both in-channel and in surrounding creeks, is an essential input to hydraulic models. Roughness is represented by Manning's n values. In-channel roughness values were largely already defined by previous hydraulic modeling efforts. Roughness value ranges for reaches with similar characteristics are summarized in Table 3-12.

Table 3-12: Summary of In-Channel Manning's n Values in Existing 1D Models

Channel Description	Percent Impervious
Low gradient, straight, concrete-lined (urban areas)	0.015 – 0.02
Low gradient, straight, natural (urban-bay interface)	0.03 – 0.06
Higher gradient, sinuous, natural (urban-rural interface)	0.05 – 0.1

These values are generally in line with those suggested in *Open Channel Hydraulics* (Chow, 1959). For steeper channels upstream of urban areas, values were checked with the following equation from *Determination of Roughness Coefficients for Streams in Colorado* (Jarrett, 1985):

$$n = 0.39\left(S_f^{0.38}\right)\left(R^{-0.16}\right)$$

Where: S_f = Energy grade line slope

R = Channel section hydraulic radius (flow area divided by wetted perimeter)

The study found that, for slopes between 0.002 and 0.04 and hydraulic radii of 0.5 to 7 feet, n-values could deviate greatly from those suggested by Chow based on physical characteristics alone. This is because steeper slopes result in lesser depths, critical flows, and greater influence on flow by roughness elements (e.g., vegetation, boulders, etc.). This equation was applied to spot check and verify Manning's n values applied to upper watershed streams (primarily above Highway 280).

Roughness values must also be applied to 2D surface areas in order to model varied flow resistance across creek overbanks and urban areas. Some surfaces are smoother and less resistant to shallow flows (e.g., streets and parking lots) and are represented by a lesser n value while other surfaces are highly resistant to flow (e.g., structures, fences, heavily vegetated areas). Roughness outside of the 1D channels is represented by assigning roughness values to composite land cover GIS data, consisting of parcels and the National Land Cover Database (NLCD) 2021 dataset.

Cover types aside from "Developed" areas are assigned roughness values based on the Hydrologic Engineering Center's River Analysis System (HEC-RAS) 2D manual. Main channel areas are defined based on satellite and ground elevation imagery to cover areas with more frequent, concentrated flows and apparently less dense vegetation than surrounding herbaceous wetland areas.

Buildings, fences, dense landscaping, and other obstructions common to urban residential areas can block the conveyance of flow but are not impervious to water. That is, when a house floods, it gets wet inside, but the walls of the house do not allow for the free flow of floodwater through the structure. In essence, these obstructions to flow tend to increase the roughness within their ground elements, and Manning's roughness coefficients are adjusted using the Hejl

Method to reflect those obstructions. The Hejl Method provides an estimate of Manning's roughness coefficient in urban areas, characterized by an open street network with blockages created by buildings.

Figure 3-11 shows the application of this method, which uses the following equation to adjust the "raw" Manning's coefficient. This is the coefficient absent urban blockage, which is set to 0.030 within open residential lots as an average value that reflects a combination of driveways and other hardscape (n=0.025) and lawn/landscaping (n=0.035).

The urban roughness coefficient adjustment is as follows:

$$n_u = n_o\left[\left(\frac{3}{2}\right)\left(\frac{W_T}{\sum W_o}\right)+\left(1-\left(\frac{W_T}{\sum W_o}\right)\right)\left(\frac{\sum L_o}{L_T}\right)-\left(\frac{1}{2}\right)\right]$$

Where: n_u = Adjusted Manning's Roughness Value

n_o = Original Manning's Roughness Value

W and L are defined in Figure 3-11

Manning's roughness values used in the 2D model domain are summarized in Table 3-13. Other land cover classifications are present in the model and represent a mixture of commercial and residential (e.g. Transit-Oriented Development and Transportation Corridor). These land covers are assigned n values by aggregating and weighting roughness values for each land use type present in the parcel(s). A map of Manning's n values used as an input to the 2D surface model is shown in Figure 3-12.

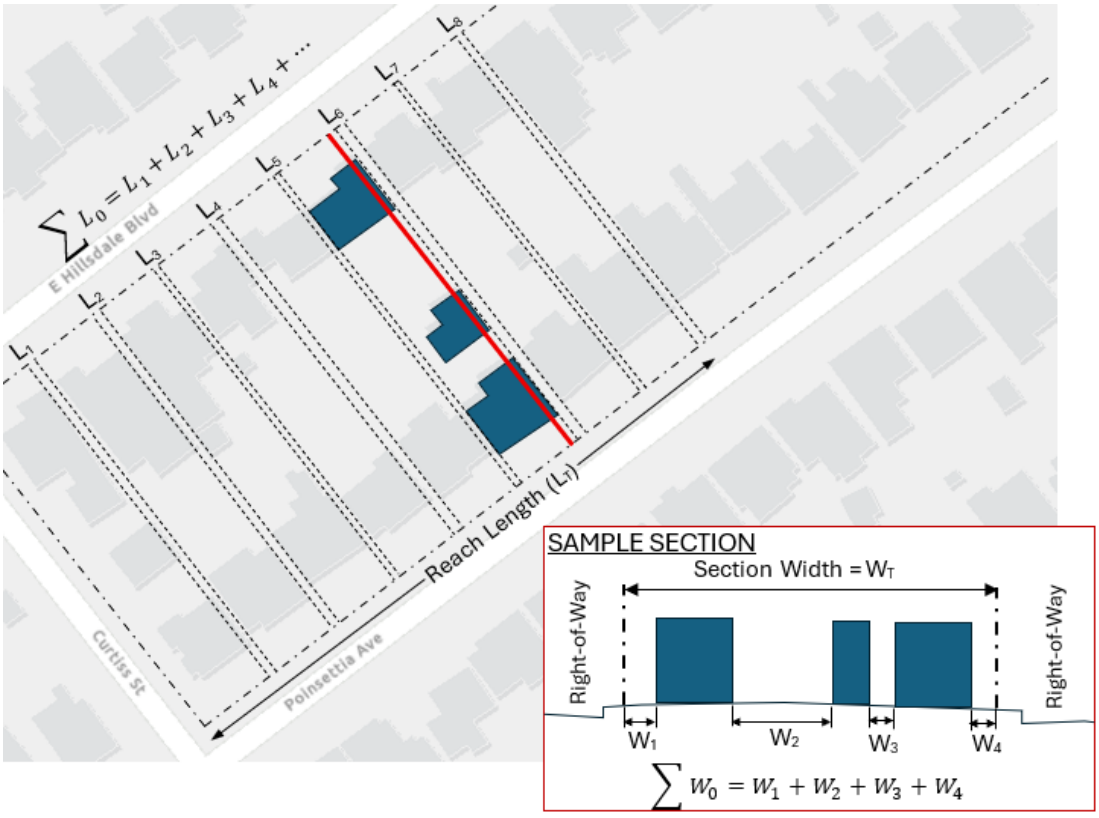


Figure 3-11: Example Application of Hejl Method for Floodplain Roughness in a Residential Area

Table 3-13: Summary of 2D Area Manning's n Values

Surface Classification	Range of n Values	Adopted n Value	Notes
Right-of-Way	0.012–0.03*	0.015	
Single-Family Rural	0.06–0.2**	0.05	
Single-Family Residential (<1/2 acre)		0.11	
Medium Density Residential		0.14	
High Density Residential		0.21	
Townhome/Condo		0.15	
Public Facility		0.07	
Executive Office		0.05	Characterized by large parking lots
Downtown Commercial		3	City blocks effectively fully blocked
Other Commercial/Industrial		0.1-0.3	Varies by subclassification
Open Water	0.025–0.05	0.035	
Barren Land (Rock/Sand/Clay)	0.023–0.03	0.03	
Evergreen Forest	0.08–0.16	0.15	
Mixed Forest	0.08–0.2	0.12	
Shrub/Scrub	0.07–0.16	0.08	
Grassland/Herbaceous	0.025–0.05	0.04	Assume higher than developed pervious areas
Pasture/Hay	0.025–0.05	0.04	
Cultivated Crops	0.02–0.05	0.05	
Woody Wetlands	0.045–0.15	0.10	
Emergent Wetlands	0.05–0.085	0.07	

*Ranges from USGS "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains (1989) suggested by HEC-RAS 2D Reference Manual

**Range is for developed classifications from NLCD layer. Roughness is varied based on zoning/land use type and estimated using the Hejl method.

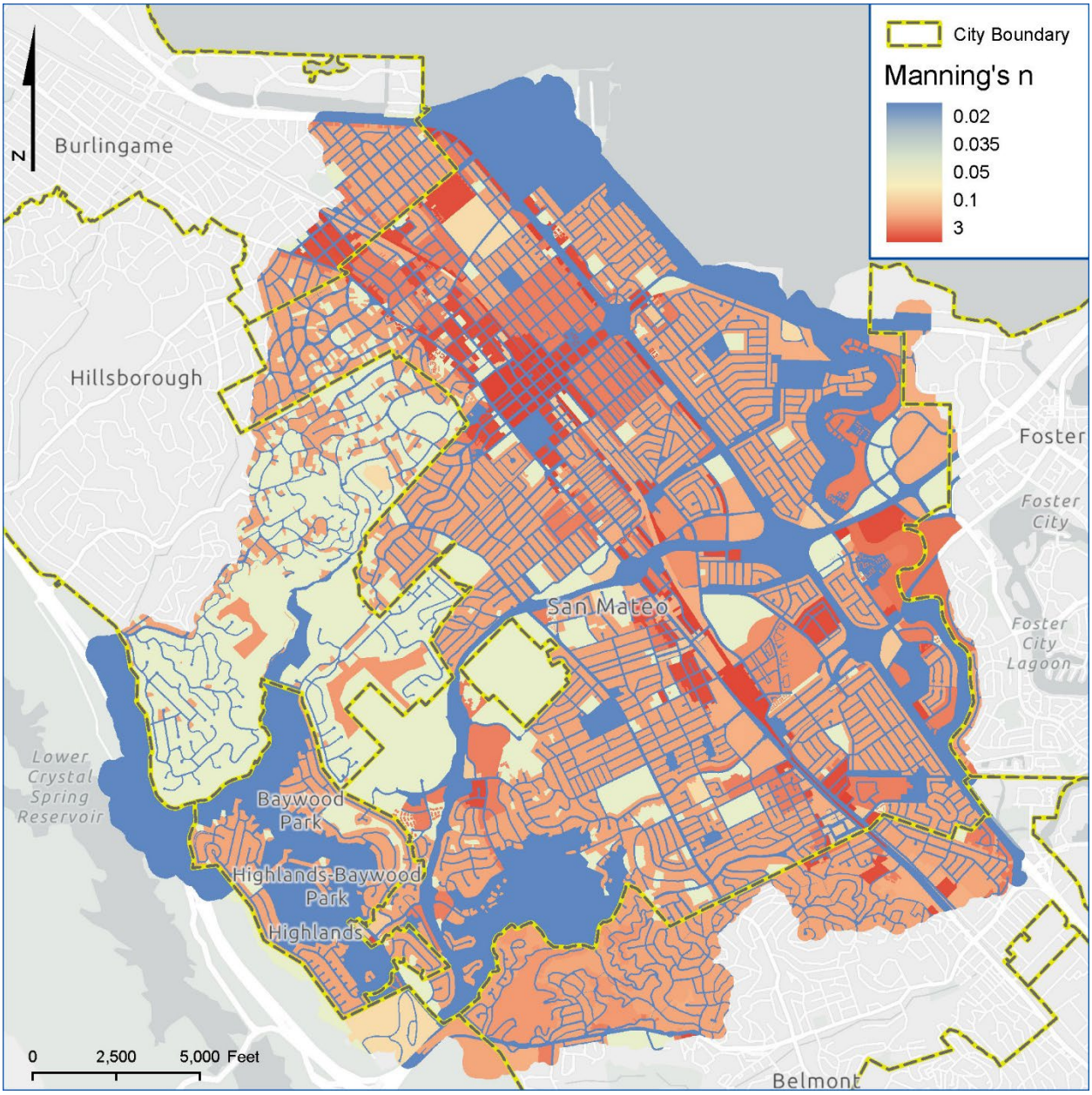


Figure 3-12: Manning n Grid for 2D Surface Model

3.2 Open Channel and Lagoon Model Data

Open channel modeling utilizes some of the same foundational datasets as the citywide storm drain system model, including topography, land cover, soil characteristics, and other data used to generate hydrologic parameters for drainage areas. However, the open channel models require greater attention to cross-sectional characteristics (roughness, topography, bridge and culvert properties, etc.).

LiDAR generally provides sufficient quality surface data to evaluate overland flow paths outside of heavily vegetated channels. However, thick vegetation reduces the density of ground returns in LiDAR point clouds. This means that cross sections cut from LiDAR may not fully capture the shape of the channel or its conveyance capacity. Because runoff is highly concentrated in the

Laurel Creek Pump Station, 16th Ave Channel, and 19th Ave Channel, HEC-RAS models for these major stormwater conveyances must capture conveyance capacity as accurately as possible.

Topographic survey was completed by Kier & Wright in 2025 to supplement citywide topographic data discussed in Section 3.1.1 and improve channel modeling in HEC-RAS and MIKE+. Within major conveyance channels, Kier & Wright performed cross-section survey, collecting point survey data defining the geometry of transects perpendicular to the direction of stream flow. Points were generally captured that defined the top of bank, flood control features (e.g. flood walls), headwalls, the stream thalweg, and other points that define the shape and conveyance capacity of the channels. Cross sections were collected on either side of bridge/culvert crossings (within about 10-15 feet), and a maximum of about 500 feet between crossings to support the development of hydraulic models. An example is shown in Figure 3-13, and an overview of surveyed points and the aerial topographic survey is shown in Figure 3-14. Additional measurements were collected for bridge and culvert crossings at locations shown in Figure 3-15. Approximate properties of bridges and culverts included in the models are summarized in Table 3-14.

All survey was tied to benchmark/control points within the City. Data was provided on the California State Plane 3 horizontal coordinate system (in US feet) and on the NAVD88 vertical datum (in feet).

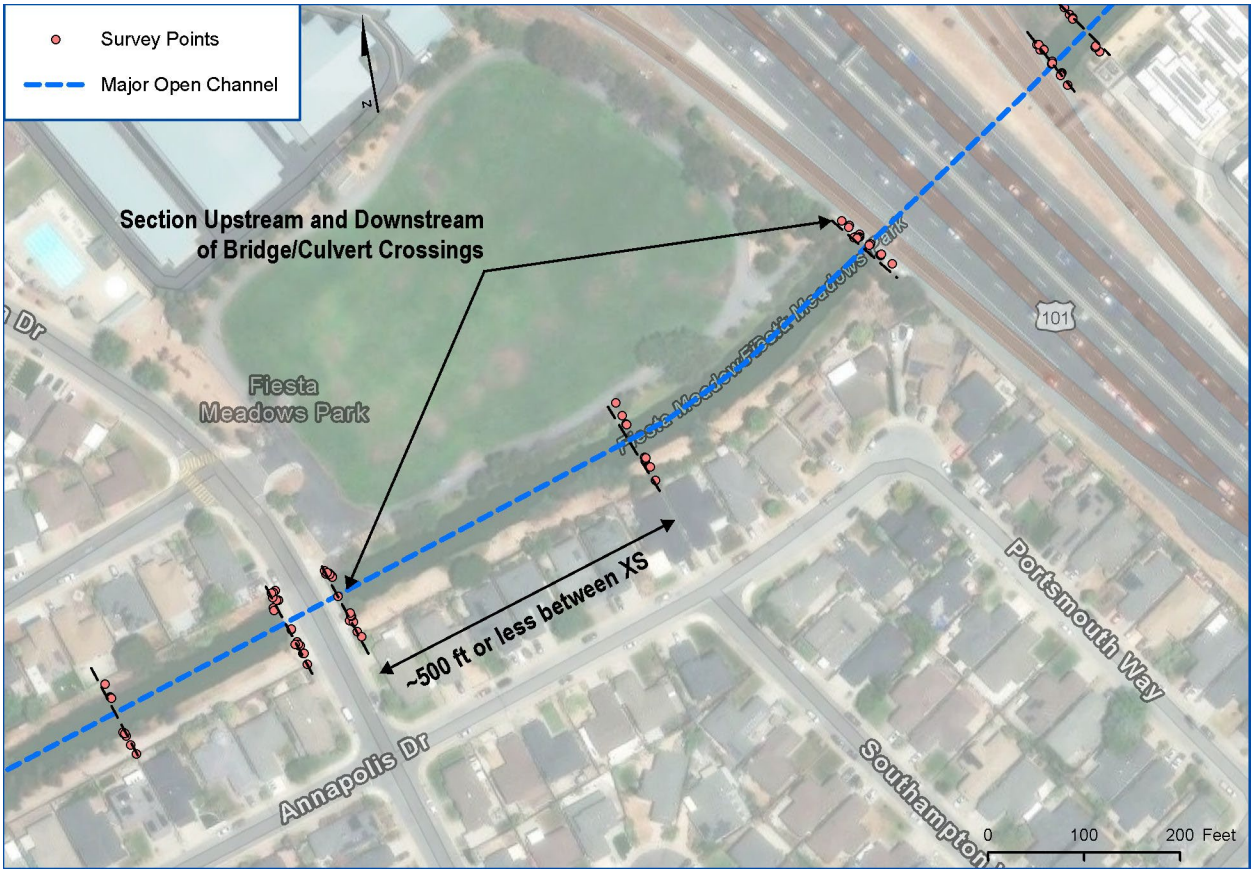


Figure 3-13: Cross Section Point Survey Sample at 19th Ave Channel near Highway 101

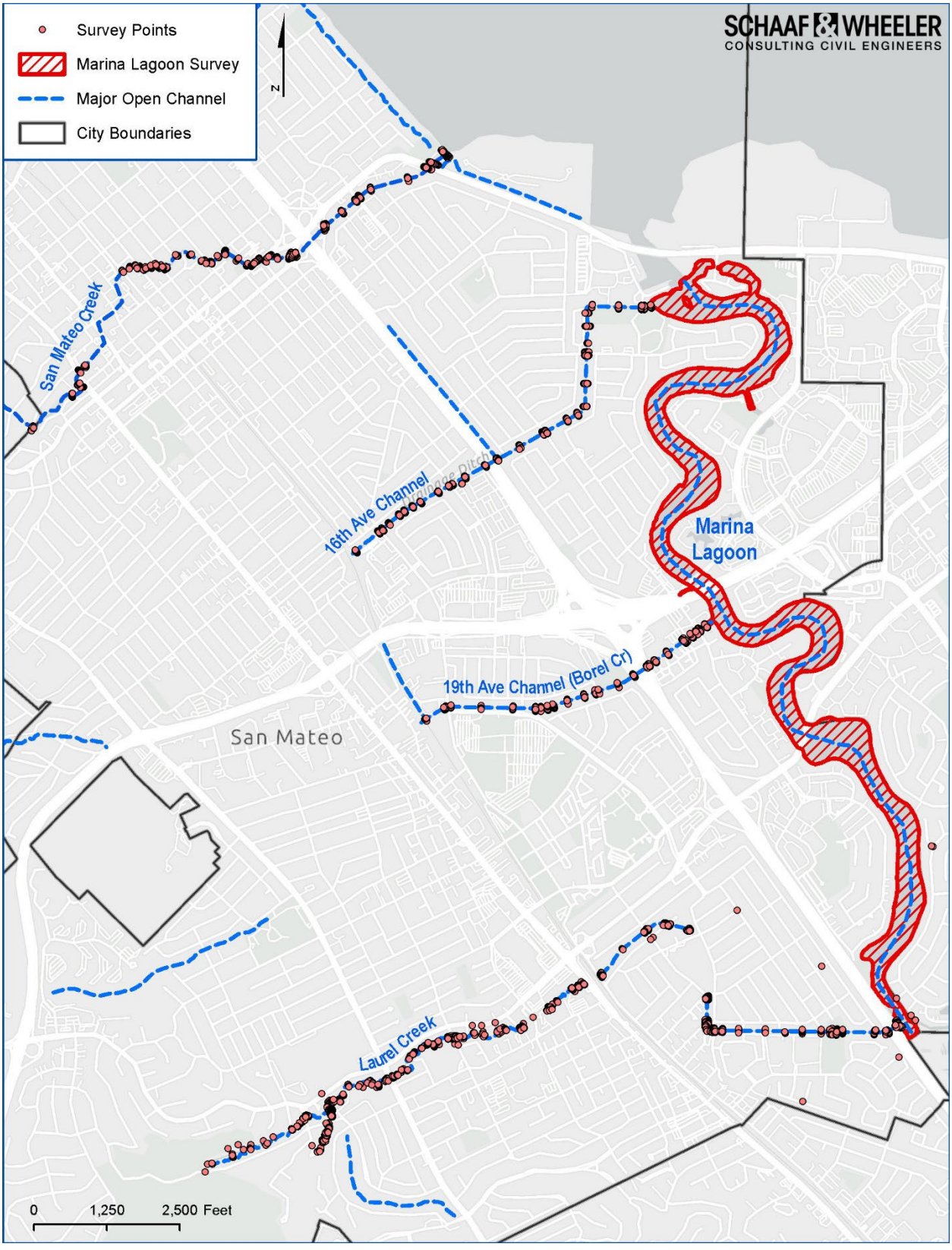


Figure 3-14: Open Channel and Lagoon Topographic Survey Overview

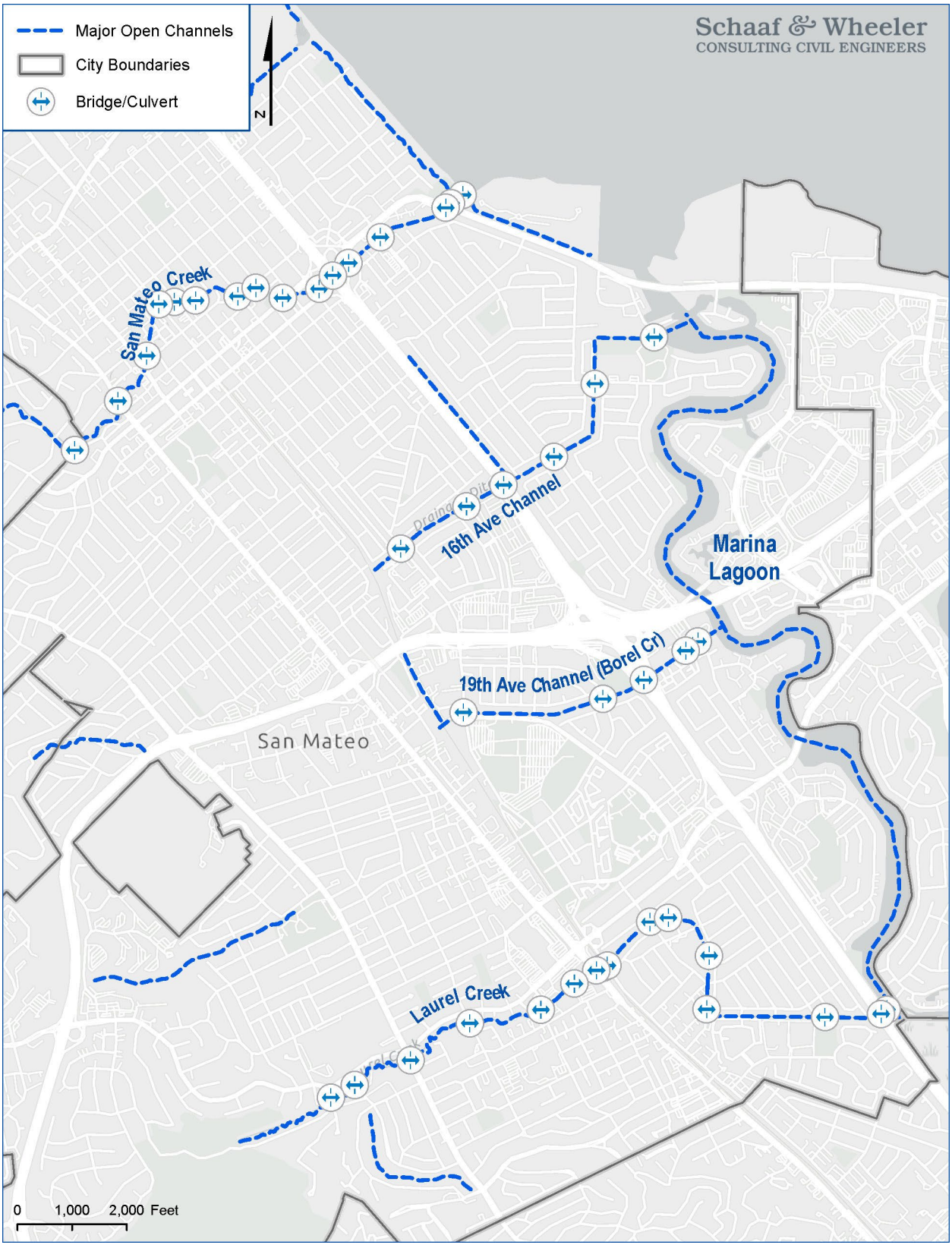


Figure 3-15: Bridge/Culvert Crossing Measurement Locations

Table 3-14: Bridges and Culverts Included in San Mateo SDMP HEC-RAS Models

Creek	Location/Street	RAS River Station	Description/Approx. Size
San Mateo	Crystal Springs	9960	Arch Culvert (Approx 12'H x 20'W)
San Mateo	El Camino	8425	Arch Culvert (Approx 13'H x 40'W)
San Mateo	Mills Health Center	7000	Culvert (2- 10'H x 10'W Boxes)
San Mateo	Railroad Ave/UPRR	6580	Bridge (7' High Free Span)
San Mateo	Claremont	6190	Bridge (11' High With 2 Piers)
San Mateo	Delaware	5830	Arch Culvert (Approx 10'H x 30'W)
San Mateo	Fremont	4850	Bridge (10' High With 1 Pier)
San Mateo	Ped Bridge nr 2nd & Grant	4510	Bridge (13' High Free Span)
San Mateo	E 3rd Ave	3885	Bridge (9' High With 2 Piers)
San Mateo	Highway 101 SB Ramps	3289	Bridge (12' High With 2 Piers)
San Mateo	Highway 101	2914	Bridge (8.5' High With 1 Pier)
San Mateo	Highway 101 NB On-Ramp	2575	Bridge (8' High With 2 Piers)
San Mateo	Highway 101 NB Off-Ramp	2518	Bridge (7.5' High With 2 Piers)
San Mateo	Norfolk	1799	Bridge (7' High Free Span)
San Mateo	J Hart Clinton	400	Culvert (4- 8.7'H x 14'W Boxes)
16th Ave (Leslie)	S Delaware St	7000	Culvert (6'H x 13'W Box)
16th Ave (Leslie)	S Grant St	5550	Bridge (7' High Free Span)
16th Ave (Leslie)	Highway 101	4700	Culvert (5'H x 10'W Box)
16th Ave (Leslie)	S Norfolk St	3700	Bridge (4.5' High With 1 Pier)
16th Ave (Leslie)	Kehoe Ave	1950	Bridge (7.5' High With 1 Pier)
16th Ave (Leslie)	Joinville Park Ped bridge	50	Bridge (11.5' High Free Span)
19th Ave (Borel)	S Delaware St	4975	Culvert (2- 5'H x 13'W Boxes)
19th Ave (Borel)	Bermuda Dr	2400	Bridge (5.5' High With 3 Piers)
19th Ave (Borel)	Highway 101	1500	Bridge (9' High Free Span)
19th Ave (Borel)	Bridge St	630	Bridge (8' High With 2 Piers)
19th Ave (Borel)	S Norfolk St	350	Culvert (2- 5'H x 13'W Boxes)
Laurel	East Laurel Creek Dr	14250	Culvert (5.5' Diam)
Laurel	Fernwood St	13500	Culvert (2- 6'H x 6'W Boxes)
Laurel	Alameda de las Pulgas	12000	Culvert (2- 4'H x 8'W Boxes)
Laurel	Hacienda St	10175	Culvert (6.5'H x 11'W Box)
Laurel	Apartments at Edison St	8500	Culvert (6.5'H x 10'W Box)
Laurel	El Camino Real	7400	Culvert (2- 4'H x 11'W Boxes)
Laurel	Pacific Blvd	7200	Culvert (5'H x 10'W Box)
Laurel	Curtiss St	6045	Culvert (5'H x 10'W Box)
Laurel	Otay Ave	5654	Culvert (2- 5'H x 8'W Boxes)
Laurel	George Hall Elem.	4700	Culvert (2- 5.5' Diam; 5.6'H x 14'W Arch)
Laurel	E 40th	3575	Culvert (3- 7'H x 12'W Boxes)
Laurel	Casanova Park Ped Bridge	1330	Bridge (8' High Free Span)
Laurel	Highway 101	270	Bridge (11' High With 2 Piers)

3.3 System Condition

Topside investigations and CCTV inspections were conducted to collect data on the system and analyze the condition of the pipes. All fieldwork data collected and condition assessment results are placed into the GIS database and will be entered into the City's asset management software. Field investigations were based on a review of the data and were conducted to assess the variety of materials and sizes of pipes throughout the study area. The CCTV inspection locations were conducted in the specific areas that were challenging to collect during field investigations and in locations with known issues.

3.3.1 Field Investigations

Fifteen days were spent in the field collecting data from roughly 30% of the City's system. A pole-mounted GoPro camera was used to collect photos that were reviewed and assigned a structural and condition rating. Field investigation map and ratings are contained in Appendix C. This preceded CCTV inspections, providing a more rapid, surface-level assessment of condition that guided the selection of pipe reaches included in the CCTV effort discussed in Section 3.3.2.

3.3.2 CCTV Inspections

After the initial topside investigations were complete, field notes, photos, and videos were reviewed to identify potential problem areas where CCTV investigation would be beneficial. Additional reaches of corrugated metal pipe (CMP) were identified for CCTV inspection, where greater susceptibility to corrosion, crushing, or failure made portions of the existing system ineffective. A total of 14 days were spent in the field collecting over 28,000 linear feet of pipe video inspection.

CCTV inspections of portions of the system resulted in two separate ratings for each inspected pipe reach, based on the National Association of Sewer Service Companies (NASSCO) rating system, as defined below:

1. Structural: Influenced by factors such as failing or corroding pipes, cracks, failed joints, holes, or collapse which are likely to permanently affect the capacity of the pipe unless it is repaired or replaced.
2. Operational/Maintenance: Influenced by factors such as settled deposits, debris accumulation, sag, and root mass intrusion which are likely to temporarily impact the capacity of the pipe unless action is taken to clean or remove obstructions.

A map of CCTV inspection locations is shown in Figure 3-16.

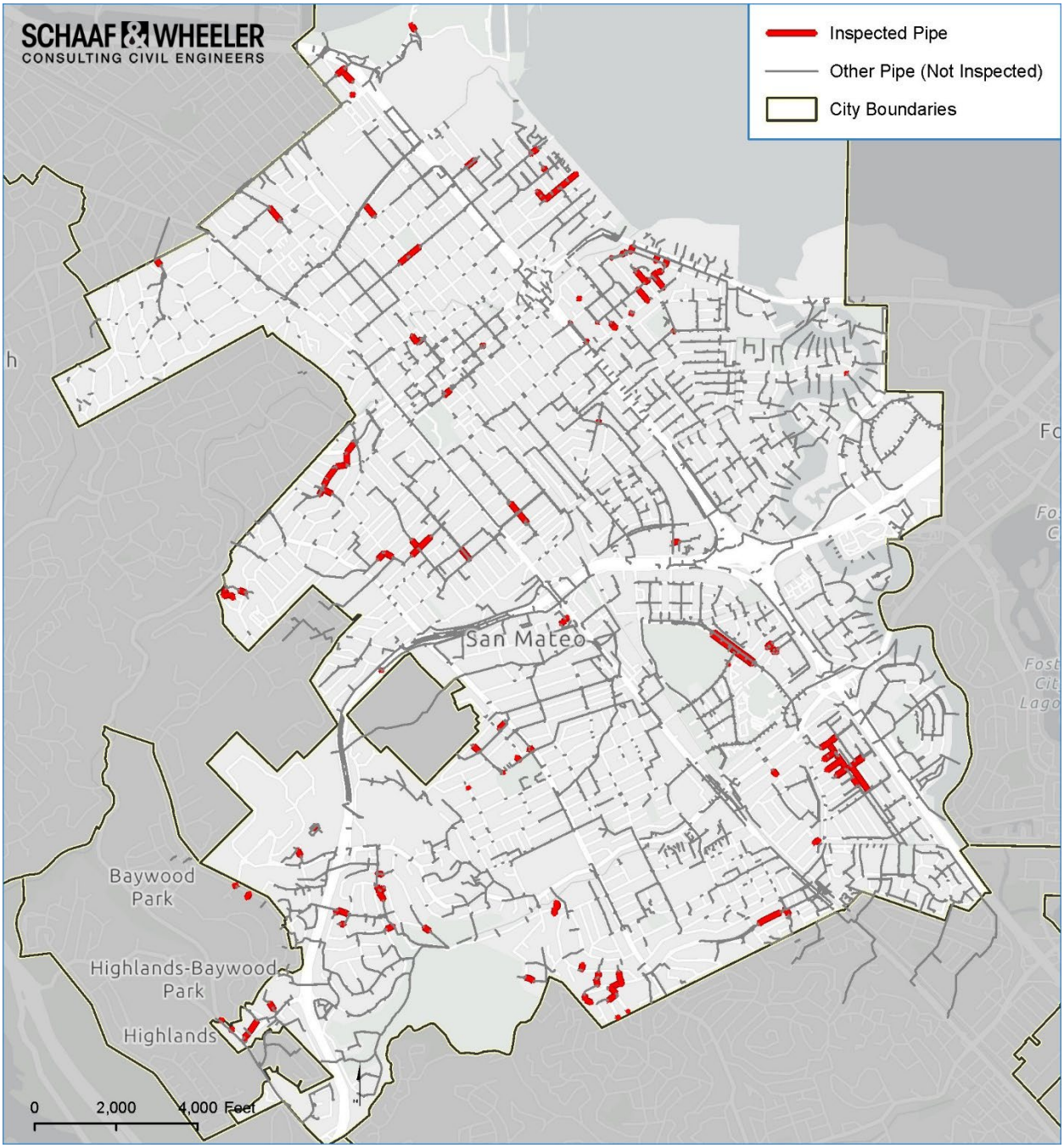


Figure 3-16: CCTV Inspections Overview Map

Based on available GIS data, about 61% of the City's drainage system material is not identified. The majority of the known material in the system is reinforced concrete pipe (RCP), followed by open channels (primarily highway ditches that are likely owned by Caltrans), and metal pipes. The system is broken down by material in Table 3-15, not including elements identified in the GIS as abandoned (about 920 linear feet). CCTV inspections focused most heavily on corroding CMP and concrete with obvious cracking, spalling, or other issues identified during topside assessment.

Table 3-15: City Drainage System Material Classifications

Material Class	Length (ft)	Share of Total
Unknown	537,766	60.7%
RCP	236,714	26.7%
Open Channel	32,861	3.7%
CMP/Other Metal	21,074	2.4%
Elliptical (Unknown Material)	16,234	1.8%
Other Plastic (PCP)	12,261	1.4%
Polyethylene (incl. HDPE)	8,910	1.0%
PVC	7,174	0.8%
ACP	4,688	0.5%
Concrete Box	4,600	0.5%
Perforated Drain	2,617	0.3%
VCP	646	0.1%
Ductile Iron	120	0.0%
Total:	**885,666**	**100%**

ACP, asbestos cement pipe; CMP, corrugated metal; HDPE, high-density polyethylene; PCP, precast concrete pipe; PVC, polyvinyl chloride; RCP, reinforced concrete pipe; VCP, vitrified clay pipe

Table 3-16: Inspected Drainage System Element Material Classifications

Material Class	Length (ft)	Share of Total
Asbestos Cement	284	0.9%
Corrugated Metal	8,650	28.9%
Polyethylene	5,279	17.7%
PVC	1,232	4.1%
Reinforced Concrete	14,438	48.3%
Vitrified Clay	25	0.1%
Total:	**29,908**	**100%**

PVC, polyvinyl chloride

4. Methodology

4.1 Overview

The criteria used to evaluate storm drain system performance must be technically sound and simple to understand and implement. Ideally, the same methodology used to analyze system performance for this report will continue to be used for future infrastructure design.

Schaaf & Wheeler applied CN methodology to estimate stormwater runoff based on land cover, percent impervious, and soil types present in the study area. The land cover CNs presented in Table 3-9 were used and percent impervious was included to develop a weighted CN.

Additionally, MIKE+ storm drain modeling software by DHI was used to evaluate system performance, identify deficiencies, and evaluate and recommend necessary improvements. Physical parameters used in the analysis are based on information detailed in Chapter 3.

Schaaf & Wheeler discussed and agreed upon evaluation criteria with City staff, as described in the following section.

4.2 Evaluation Criteria

Schaaf & Wheeler created hydrologic analysis and 1D/2D hydraulic models for 10-, 25-, and 100- year events. The 10-year storm event was used as the design event for the storm drain system evaluation, which is consistent with neighboring SDMP studies. The 10-year storm event has a 10% chance of occurring each year.

Pump stations, lagoons, and open channels are generally more critical facilities whose operation impacts greater areas. These systems and the connecting pipe systems within 100 feet of a pump station wet well have been evaluated with a 100-year level-of-service standard, which is a storm with a 1% chance of occurring each year.

The hydraulic model applies harmonic tides (see Section 3.1.9) as boundary conditions at outfall points and to San Mateo Creek. With pump curves input for each pump, the model considers the impacts of tidal levels on pump hydraulics by evaluating total dynamic head on each pump instantaneously, with each time step. Extreme precipitation and tide events are usually considered to be independent of one another—in other words, a 100-year storm does not necessarily occur coincident with a 100-year tide. Coincident frequency analysis has not been performed for this effort. Instead, the model assumes that a 10-year storm occurs coincident with a 10-year extreme tide, and a 100-year storm occurs coincident with a mean tide, keeping the focus primarily on the performance of interior storm drain systems. Aside from a handful of small pipe systems exposed to the tides at the downstream end of San Mateo Creek, most of the interface between interior drainage systems and the Bay occurs at terminal pump stations, where hydraulics are considered in detail outside of the models.

The model assumes that 10- and 100-year design storms occur during winter months, when the flood control pool level is maintained in Marina Lagoon preceding the storm. The O'Neill Slough Tide Gate is considered completely closed for all modeled events, placing the burden of flood control wholly on Marina Lagoon Pump Station.

This document recommends improvements to reduce the 10-year hydraulic grade to no higher than the rim elevation in the pipe network where feasible and to mitigate 100-year flooding at

major flood control facilities and terminal pump stations. There are a few low lying areas where water may be ponded on the streets for a short amount of time until tide elevations recede.

4.3 Modeling Software

Schaaf & Wheeler selected DHI MIKE+ software to model the City's storm drain system. MIKE+ is a package of software programs designed for the analysis, design, and management of urban drainage systems, including storm water sewers and sanitary sewers.

The model can simulate runoff, open channel flow, pipe flow, water quality, sediment transport, and two-dimensional surface flow.

The City's modeling package consists of four interrelated products, as described below.

1. **MIKE Collection Systems** is a group of hydrologic, hydraulic, water quality, and sediment transport modeling modules that can be used together or independently. The modules used in the City storm drain model include the Catchments (Surface Runoff) Module, which computes surface runoff using one of five computational methods, and the Hydrodynamic Pipe Flow Module, which calculates an implicit finite-difference numerical solution of St. Venant flow equations for the modeled pipe network.
2. **MIKE River Networks** is a package of 1D tools for modeling open channel hydraulics and water quality, defined by a river centerline, cross sections with elevations and roughness defined, and other features, including culverts, bridges, and pump stations. Inflows to rivers can be either directly from MIKE catchments or from Collection Systems link elements. Rivers can also discharge into Collection Systems elements.
3. **MIKE 2-D Overland** (MIKE 21) is a package of 2D flow analysis tools capable of simulating flows, waves, and sediments across a surface mesh. It computes 2D flows over an input surface using an implicit finite difference solver method with elevation and Manning's roughness values defined on either a regular rectangular or irregular mesh. The module couples with 1D pipe flow and River Networks modules to model storm drain system surcharge/overflow with greater accuracy than the 1D model alone.
4. **MIKE+** is a GIS-based program that combines the above three products into one interface to develop urban drainage models. MIKE+ provides a graphical user interface for data input and editing. It serves as a bridge between GIS data inputs and the hydrology and the 1D pipe flow modules.

Capabilities of this modeling software include import and export of model data, network editing and gap-filling, catchment delineation, and network simplification. MIKE+ can also be used to present results, including plan, longitudinal, and cross-section views; animation of results; presentation of flooding, including water depth and pressure; and overlay of results on background graphics, such as maps or aerial photos.

HEC-RAS models have also been developed for each of the four major creeks. HEC-RAS model software is developed by the US Army Corps of Engineers and free to use. It allows for more rapid, focused modeling of open channel systems in the City, including overland routing of flow overtopping creek banks and spilling onto adjacent land.

4.4 Model Operation

MIKE+ performs multiple separate calculations for the model. First, a runoff calculation (hydrologic analysis) estimates the amount of water entering the storm drain system during a design rainfall event. Second, network and river flow calculations (hydraulic modeling) replicates how the storm drain and channel system will convey flows to pumps and outlet locations.

Flows resulting from the runoff calculation are used as inflows for the subsequent network flow calculation. Finally, where the storm drain and river systems surcharge or overflow, a 2D overland hydraulic calculation is performed. 2D calculations are performed simultaneously with 1D network and river flow calculations.

The MIKE+ runoff model offers a choice of infiltration methods. The City's storm drain models use the Soil Conservation Service (SCS) Unit Hydrograph Method (UHM) to estimate surface runoff from the delineated catchment areas. A simulation can be started at any point during the chosen design storm to assess surface runoff for any period of the design storm, with computations made based on a user-specified time step.

4.4.1 MIKE+ Storm Drain Model

Surface runoff calculations require two types of input data: boundary data and catchment data. Pipe network calculations require definition of drainage system characteristics and runoff inputs. Boundary data for the run-off computation consists of an input rainfall time series representing the design storm event for the model.

Pipe network data required by the model include pipe shapes, sizes, lengths, and roughness characteristics, as well as node type, diameter, and ground and invert elevation. Pipe network "link" objects are also included to represent pumps, with head-discharge curves used to model pump discharge during storm events.

Parts of the drainage network are also input as Rivers, which each require an alignment and cross section data (x-y points and Manning's n roughness values). Cross section data has been developed in HEC-RAS from the LiDAR digital elevation model (DEM) and refined with cross section survey points. MIKE+ does not have an existing tool to import this data, so custom tools have been developed to import HEC-RAS cross section data into the MIKE+ interface.

Catchment data required by the model includes relevant physical and hydrologic parameters, such as surface area, basin lag time, and CNs.

2D model domains are defined by a series of "mesh arcs," which are polylines that enclose an area to be discretized and define interior features to be represented by the mesh (e.g. road crests, ditch or channel thalwegs, dikes, or dams). An overview of the MIKE Model GIS inputs is shown in Figure 4-1. Only the outer boundary arcs are shown for the 2D mesh for clarity. Internal arcs and mesh elements are omitted from the figure for clarity.

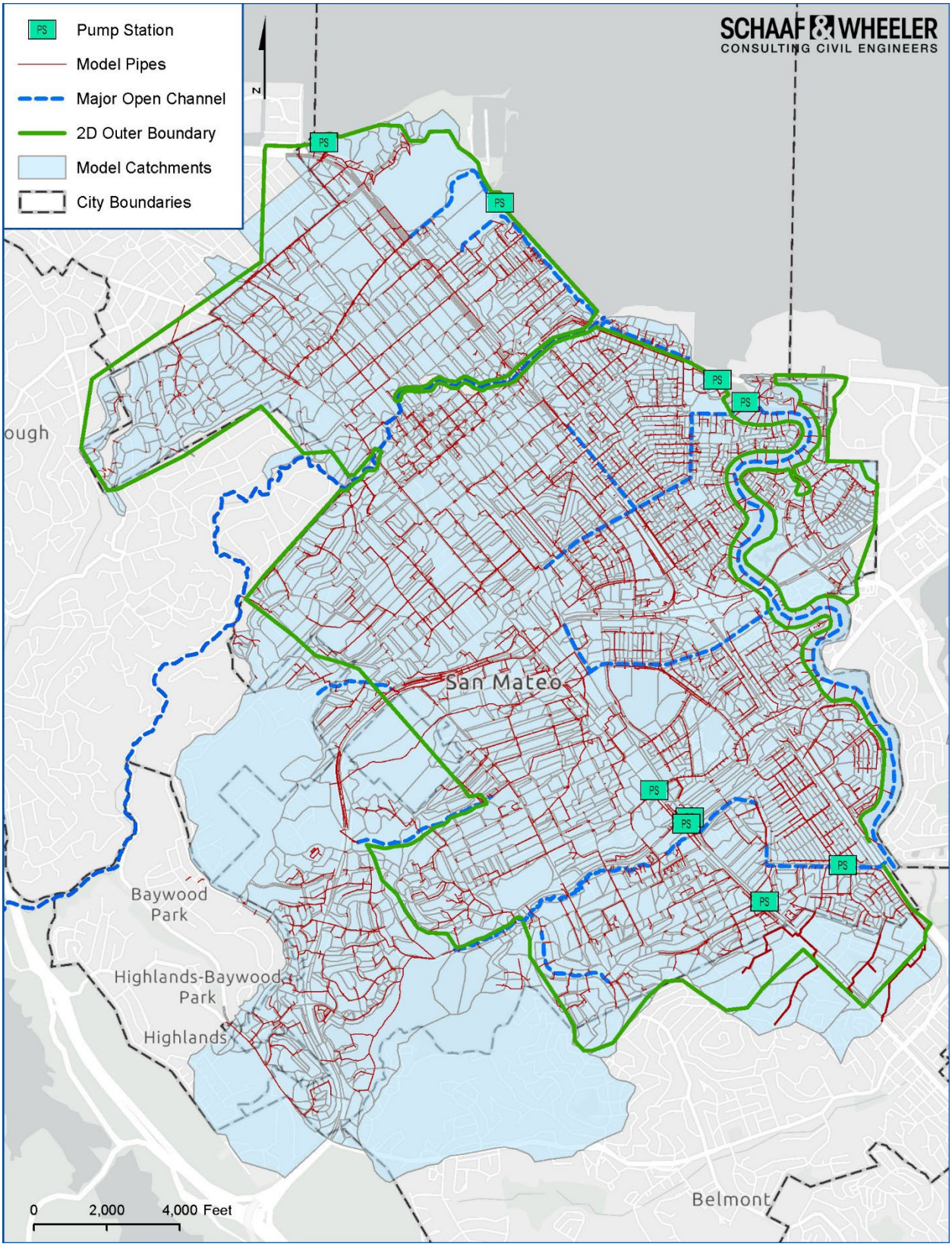


Figure 4-1: City of San Mateo Storm Drain System Catchments and Modeled System Elements

Table 4-1: Summary of MIKE+ Storm Drain Model Input and Output

Model	Inputs	Outputs
Runoff	*Boundary Data* ▪ Rainfall time series *Urban Catchment Data* ▪ Drainage catchments ▪ Lag time ▪ CN ▪ Distributed precipitation gage weights (automatically calculated by MIKE+)	▪ Runoff hydrographs for each individual catchment
Pipe Flow	*Storm Drain Network* ▪ Nodes (catch basins, manholes, outlets, etc.) ▪ Links (pipes, culverts, open channels, pumps) ▪ Operational data (pump curves, basin elevation-volume curves, etc) ▪ Catchment connections ▪ Junction losses ▪ Boundary data (e.g. water surfaces at outfalls) ▪ Catchment runoff hydrographs ▪ Water surface elevation time series	▪ Water level at each node ▪ Water level in network links ▪ Velocity in network links ▪ Water volume in the system ▪ Discharges at each link or structure (pump, weir, orifice) ▪ Discharge to/from 2D Domain
Rivers	*1D Open Channels* ▪ River centerlines ▪ Cross sections ▪ Culverts/Bridges (size, shape, invert elevations)	▪ Water surface profiles ▪ Water levels at cross sections and connected pipe elements ▪ Discharge to/from 2D Domain
2D Overland	*2D Domain* ▪ Mesh type (irregular, rectangular) ▪ Mesh areas and subdivisions ("mesh arcs") ▪ Cell sizes ▪ Elevation grid data (DEM) ▪ Surface roughness (Gridded DFS2 layer) *Boundary Conditions* ▪ Closed, water levels, or inflows	▪ Depth of water (interpolated continuously where grid is inundated) ▪ Flux of flow across surface

4.4.2 HEC-HMS and HEC-RAS Creek Models

The HEC-HMS creek hydrology model includes two Basin Model elements—one representing San Mateo Creek drainage area, and the other representing drainage areas and creeks that drain into Marina Lagoon (16th Channel, 19th Channel, and Laurel Creek). The models include large impoundments at the upstream end of the creek systems (Crystal Springs Dam, West Laurel Dam, and East Laurel Dam) and Marina Lagoon and its pump station.

The San Mateo Creek model includes subbasins and reaches delineated downstream of Crystal Springs Dam, supplemented with subbasins and reaches from existing models for the Crystal Springs/San Andreas Dam system.

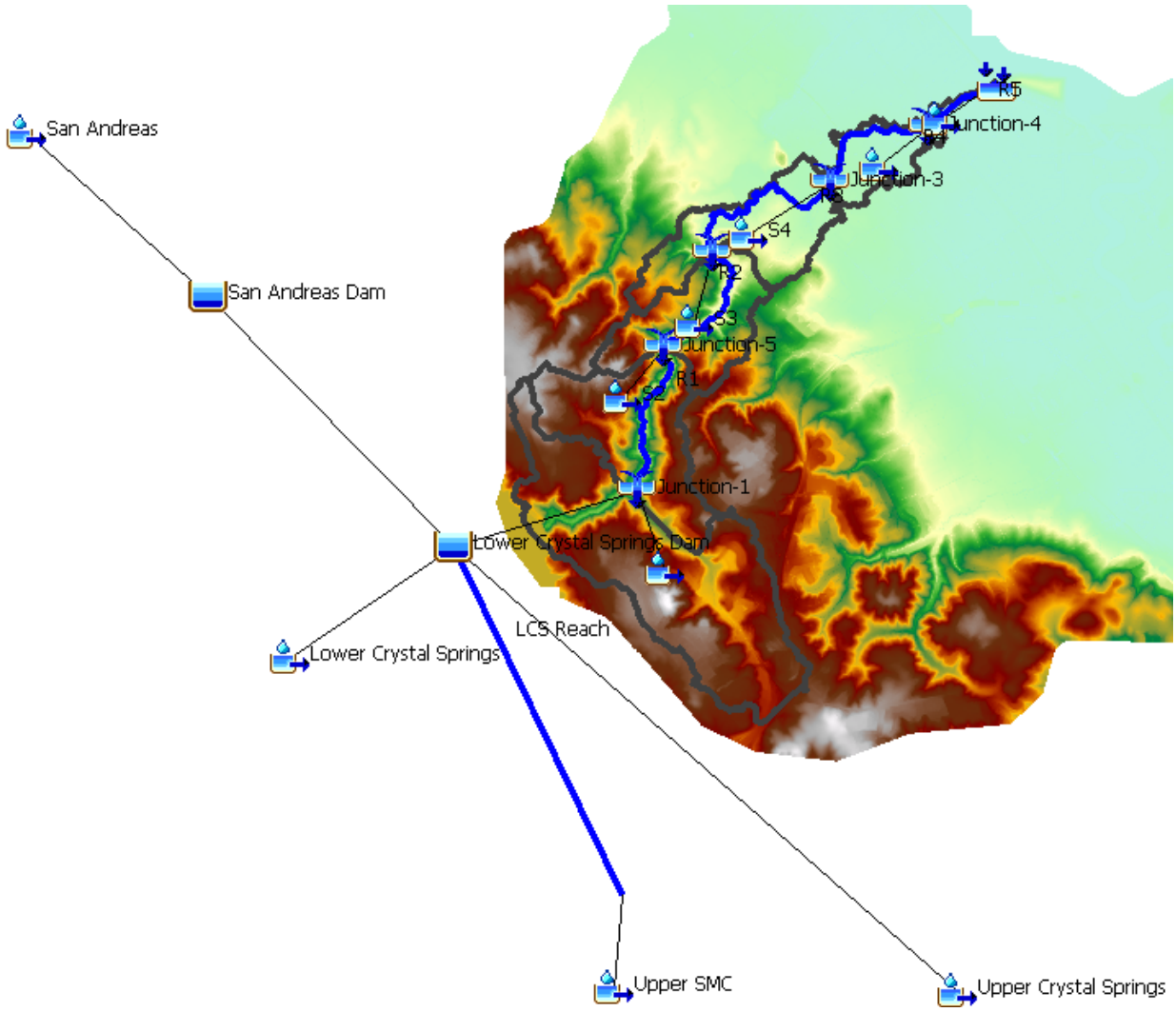


Figure 4-2: San Mateo Creek HEC-HMS Basins Model

The Marina Lagoon basins model (shown in Figure 4-3) covers 11.3 square miles, including flat, urbanized areas of San Mateo, where closed conduit storm drainage systems can store and attenuate runoff. In steeper subbasins, storm drain system surcharge may travel overland and ultimately end up in the creeks with little to no attenuation. When local system capacity is exceeded in flatter, low-lying subbasins, stormwater tends to pond in street gutters, parking lots, and other depressions, and it drains as capacity becomes available in pipe systems and pump stations. This process results in less peak discharge in local streams than might be expected without attenuation.

The MIKE+ model results have been examined at the creeks to develop storage/discharge relationships for certain subbasins in the HMS model that represent the actual capacity of storm drain systems and surface storage in each area. These storage/discharge relationships are used with the Modified Puls routing method to modify runoff hydrographs before discharging to creek reaches and Marina Lagoon for five subbasins (S-16th-3, S-16th-4, S-19th-3, S-Laurel5, and S-Laurel6).

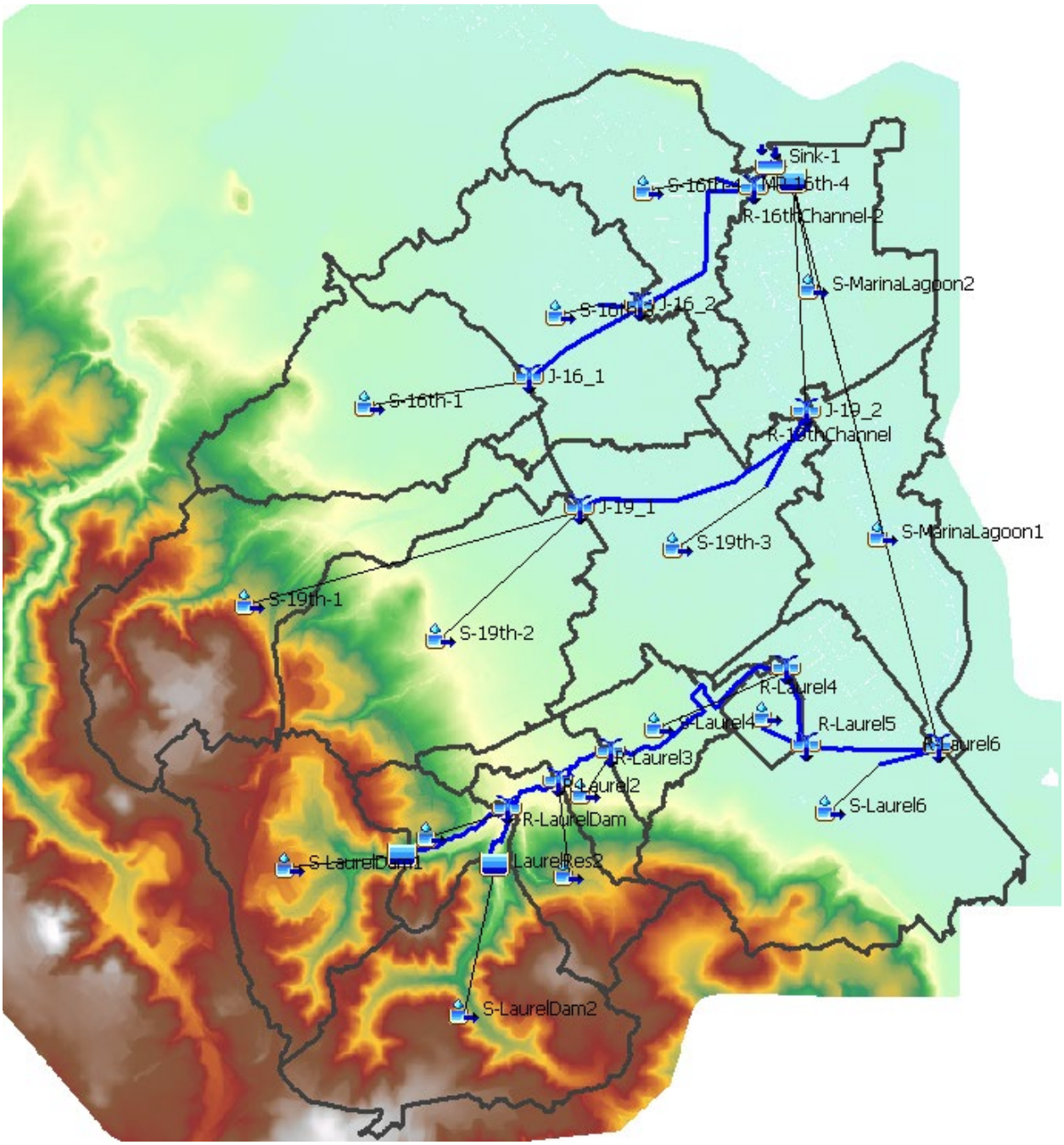


Figure 4-3: Marina Lagoon Drainage Area HEC-HMS Basins Model

Unitized stage-storage curves (on a per square mile basis) are shown in Figure 4-4. The curves are scaled by subbasin area to establish final stage-storage curves input into HMS.

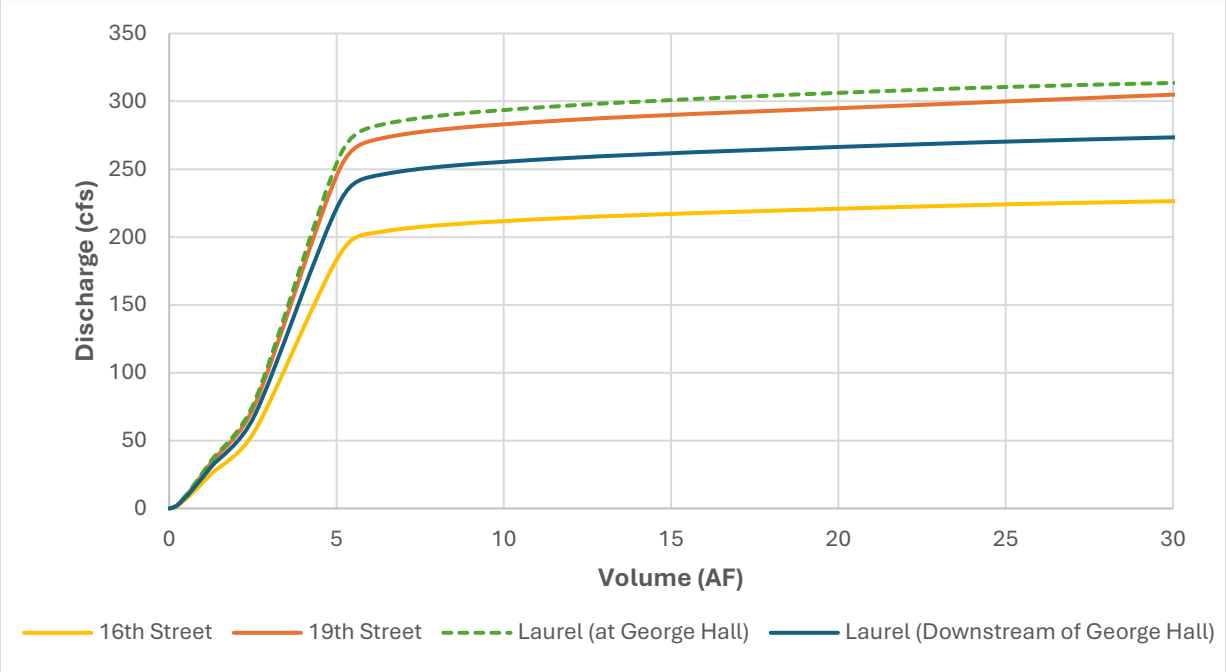


Figure 4-4: Modified Puls Routing Unitized Storage-Discharge Curves for One Square Mile of the Study Area

Separate HEC-RAS models have been developed for each of the four creeks. The models include 1D channels linked to 2D meshes by lateral structure elements to route spills from the channel into overbanks. The models also include a limited set of pipe elements, using the HEC-RAS 6.6+ pipe systems feature. While the pipe networks feature remains in beta as of this master plan effort, the capacity of those local pipe systems can have a significant influence on the extent and depth of flooding.

HEC-RAS model geometries developed for the master plan are shown in Figure 4-5 through Figure 4-9, with critical features for routing spills or surcharge from the channels annotated.

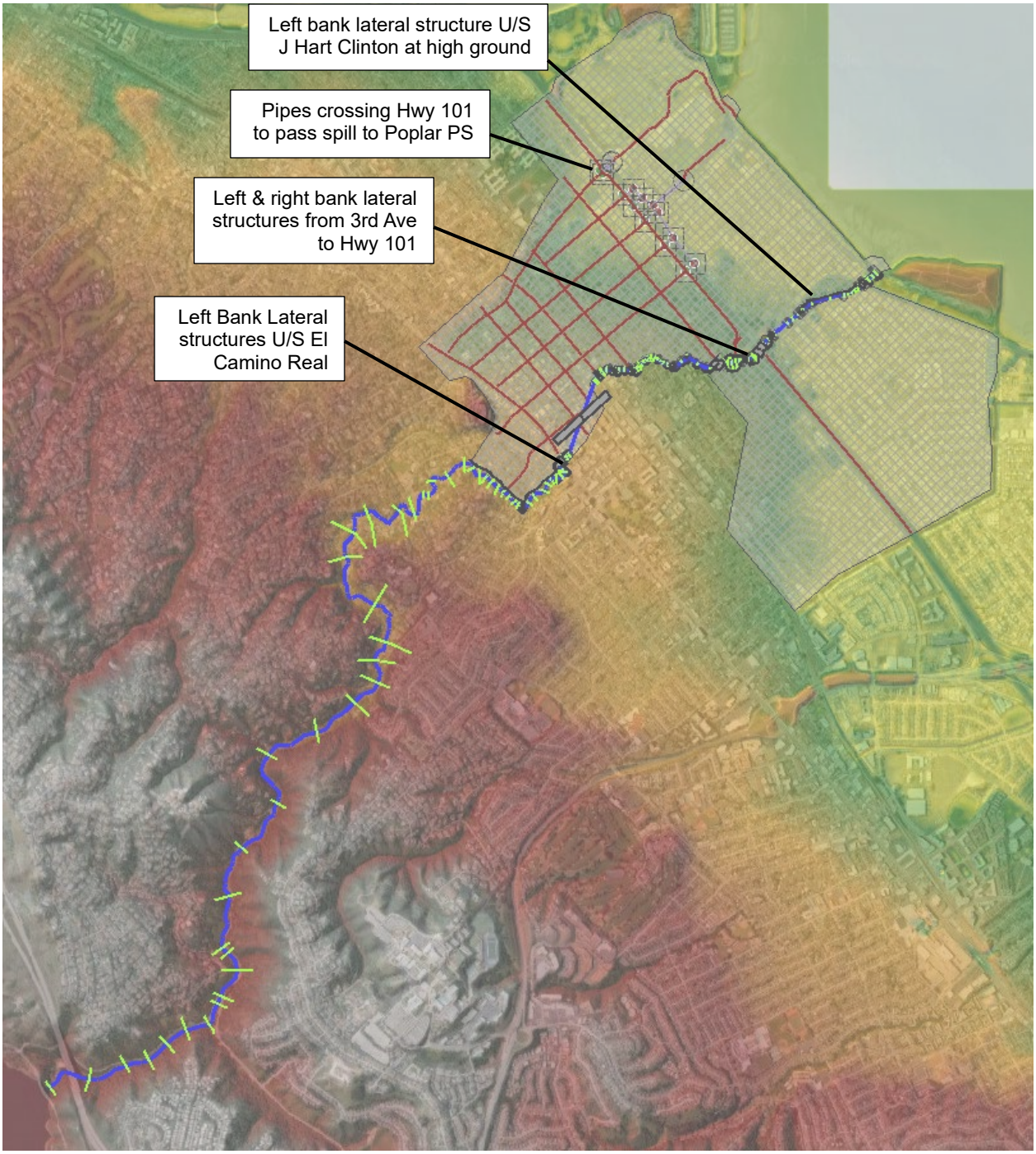


Figure 4-5: Annotated San Mateo Creek HEC-RAS Model Geometry Overview

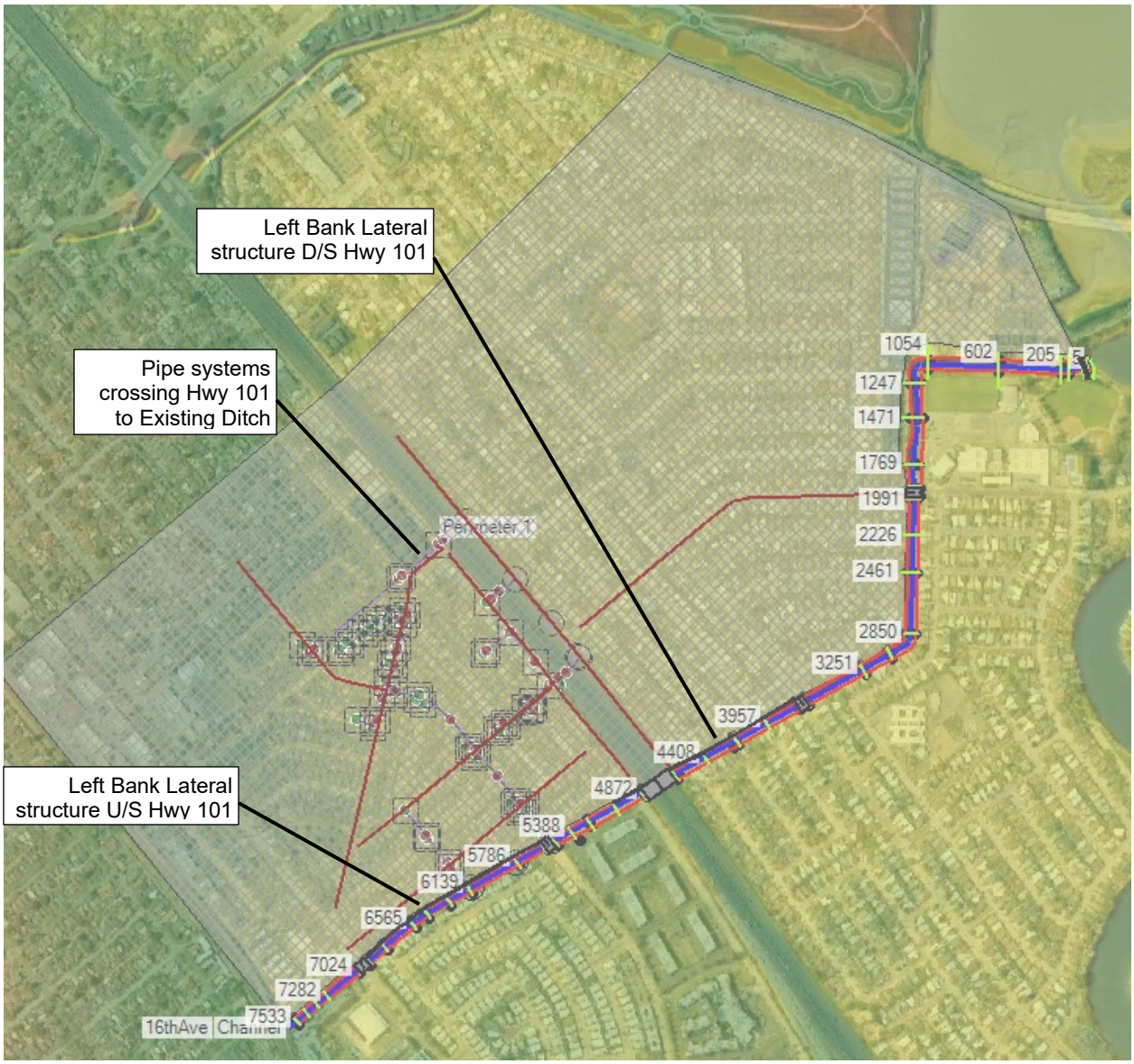


Figure 4-6: Annotated 16th Ave Channel HEC-RAS Model Geometry

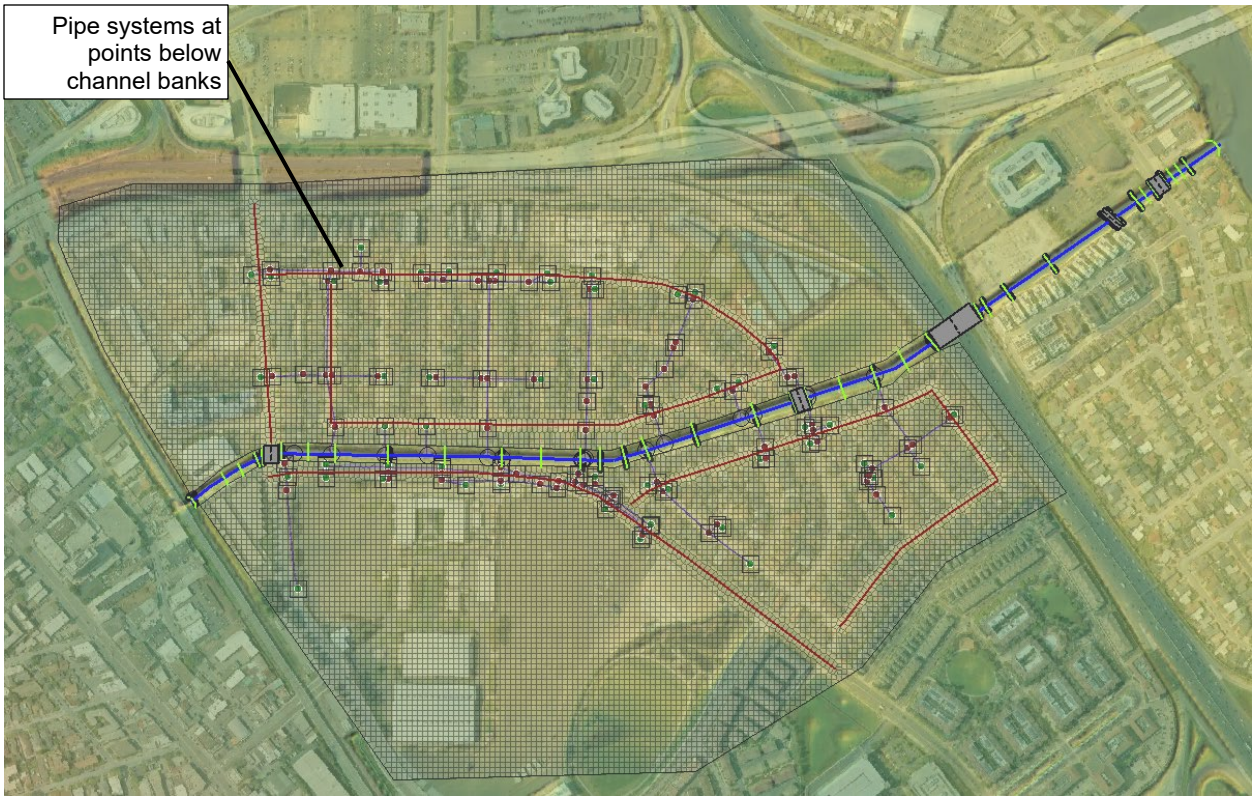


Figure 4-7: Annotated 19th Ave Channel HEC-RAS Model Geometry

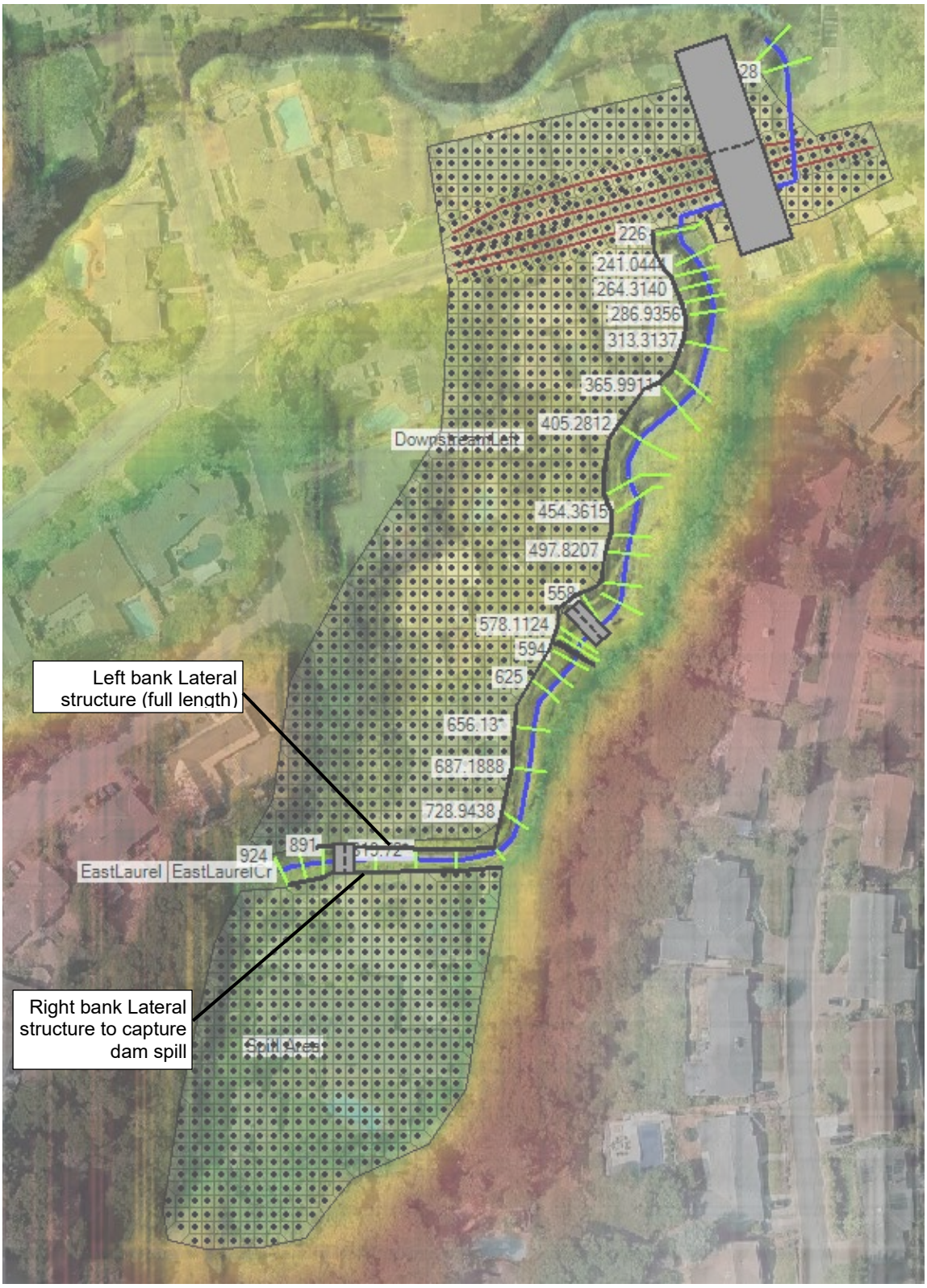


Figure 4-8: Annotated East Laurel Creek HEC-RAS Model Geometry

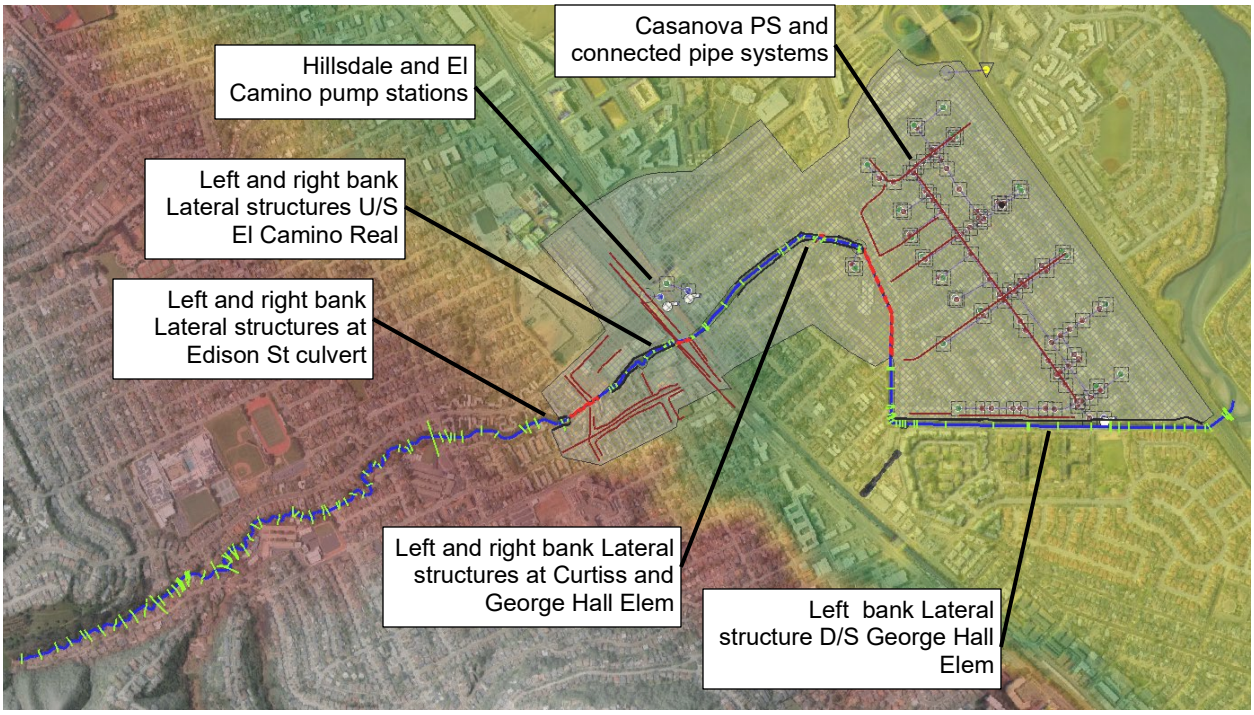


Figure 4-9: Annotated Laurel Creek HEC-RAS Model Geometry

The input/output data and parameters for the HEC-HMS and HEC-RAS models are summarized in Table 4-2.

Table 4-2: Summary of HEC-HMS and HEC-RAS Model Input and Output

Model	Inputs	Outputs
HEC-HMS	*Boundary Data* ▪ Rainfall time series *Subbasin Data* ▪ Drainage area boundaries/area ▪ Lag time ▪ CN ▪ Subbasin impervious percentage ▪ Baseflow parameters *Paired Data* ▪ Storage-Discharge (Dams, Modified Puls) ▪ Elevation-Storage and/or Elevation-Area ▪ Elevation-Discharge (Pump Curves) *Grid/Terrain Data* ▪ Gridded CN, % impervious ▪ Gridded rainfall timeseries	▪ Runoff hydrographs for each individual catchment ▪ Storage and Elevation for reservoir elements
HEC-RAS	*1D River Model* ▪ River centerlines ▪ Cross sections ▪ Culverts/Bridges (size, shape, invert elevations) ▪ Lateral Structures (Connected to 2D) *2D Domain* ▪ Mesh Perimeters and Breaklines ▪ Cell sizes ▪ Elevation grid data (Terrain) ▪ Surface roughness *Boundary Conditions* ▪ Water levels, inflows *Storm Drain Network (Version 6.6+)* ▪ Nodes (catch basins, manholes, outlets, etc.) ▪ Pipes ▪ Pumps and Operational data (pump curves, set levels)	▪ Water surface profiles ▪ Water levels at cross sections and connected pipe elements ▪ Discharge to/from 2D Domain ▪ Depth of water (interpolated continuously where grid is inundated) ▪ Flux of flow across surface

4.5 Rainfall Depth and Pattern

Given the size of the model area, variability in precipitation characteristics has been considered by developing gridded design storm patterns. These timeseries grids were created using Matlab scripts, which automatically re-balance the baseline 1955 storm pattern for each grid cell in the MetStat precipitation-frequency grids. The grids were then stored in NetCDF format for further use in GIS and for import to various model formats (e.g., DSS, DFS2, etc.). The 1955 storm and its damaging floods have been heavily documented and are considered the cause of one of the costliest flood events ever recorded in the state of California. These have been and continue to be used as the basis of calibrated hydrologic modeling regionally by many jurisdictions.

Two hydrology models have been developed for this SDMP. The first applies a fully gridded modeling approach, with grid format precipitation as a boundary input. The second is catchment-based, routing runoff from catchment areas with aggregated runoff parameters into the City's pipe systems. The second method uses precipitation timeseries extracted at 10 points, with the model assigning weighting factors to precipitation points within an input distance of each catchment area.

The 10-year storm patterns for the San Mateo model are shown in Figure 4-10, with 72-hour precipitation totals between 4.5 inches (at Point 8) and 5.9 inches (at Point 9). A map of the points overlain on the MetStat balanced 10-year storm grid is shown in Figure 4-11.

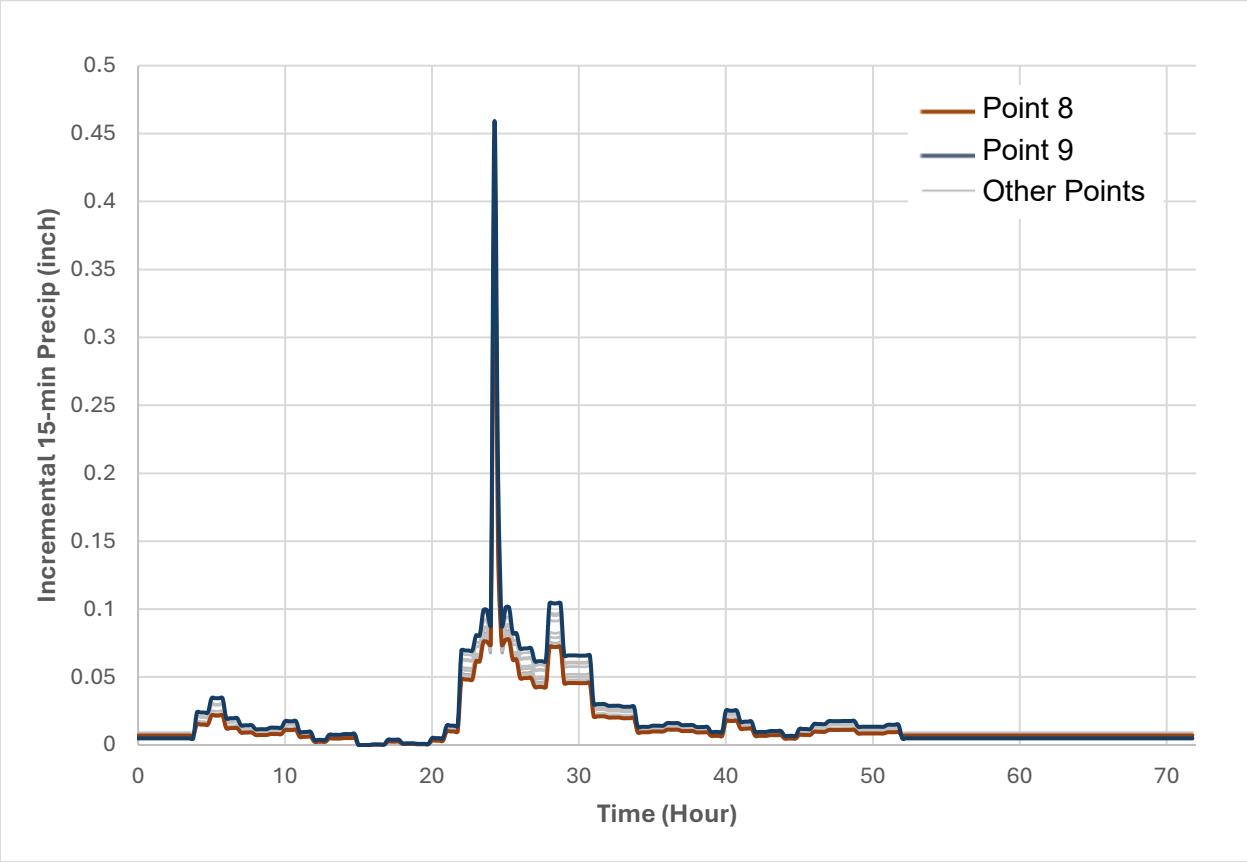


Figure 4-10: 10-year, 72-hour Storm Patterns for the City of San Mateo

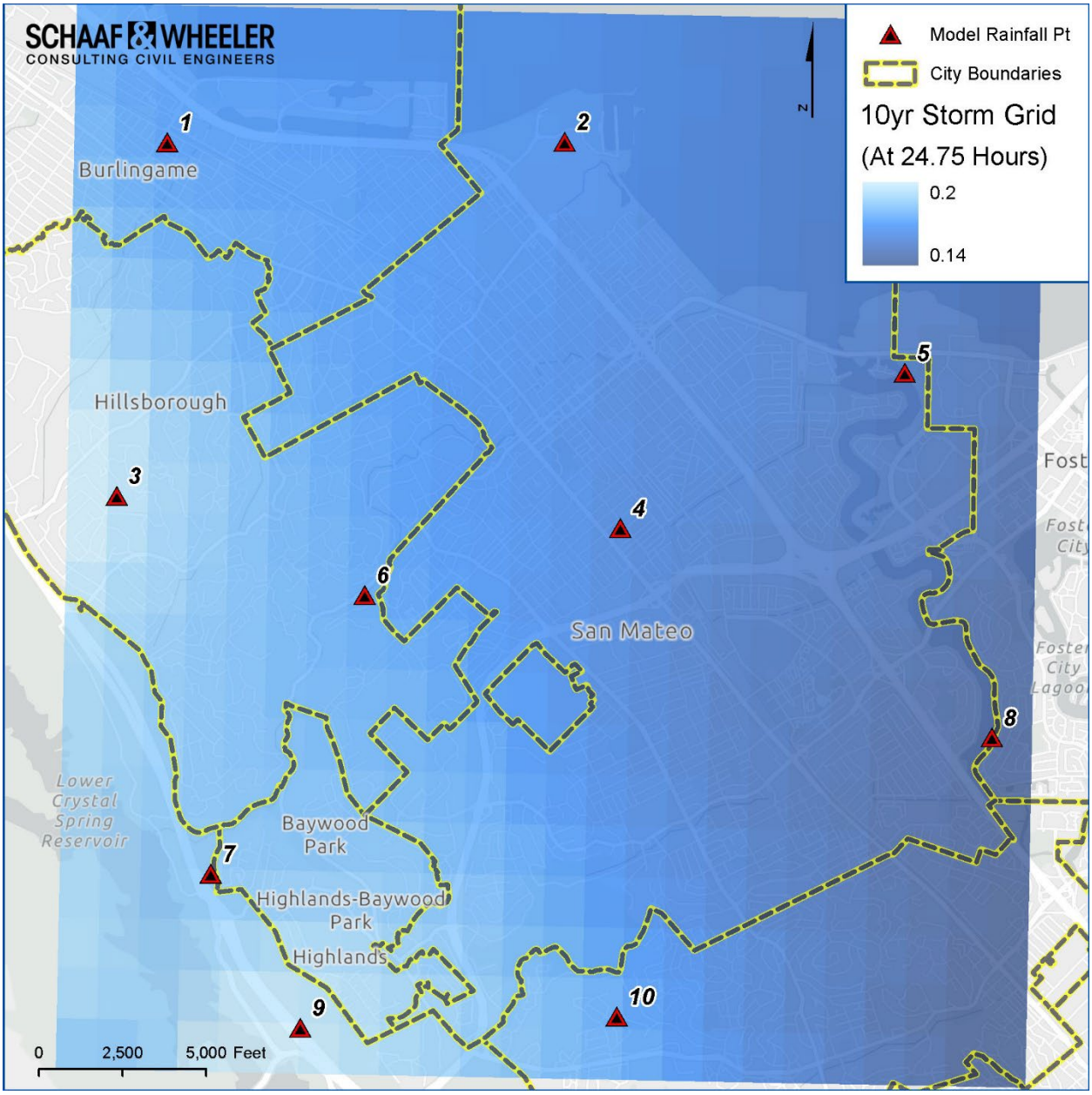


Figure 4-11: Catchment Model Rainfall Points for the City of San Mateo MIKE+ Model

4.6 Catchment Data

4.6.1 Curve Numbers (CNs)

The CN method applies a unit hydrograph as a numerical representation of the time response of catchment runoff caused by rainfall, applied uniformly over a unit of time. Land use and soil information were overlain in GIS (presented in Chapter 3), and CNs were assigned to the resulting polygons. These were converted to a spatially varied CN raster grid. GIS tools were then used to estimate pervious CNs within each individual catchment area. Weighted CNs (mapped in Figure 4-12) represent both pervious and impervious surfaces in each catchment area. The MIKE+ model uses separate pervious and impervious area CNs as inputs, ensuring

that, if lesser storm events are modeled, the impacts of direct runoff from pavement to storm drain systems are better preserved.

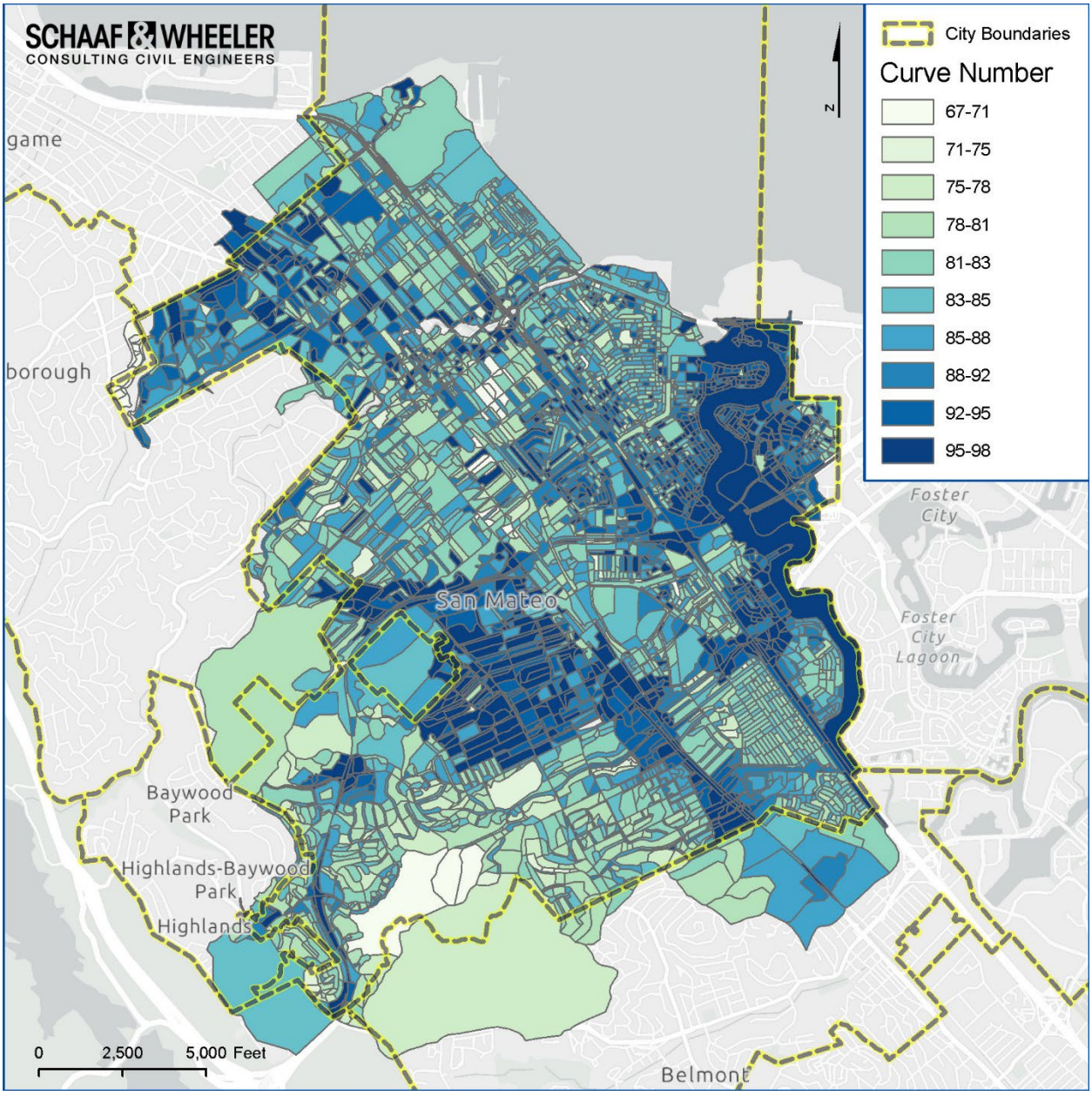


Figure 4-12: Map of CNs (Weighted; AMC 1.75) Assigned to San Mateo Model Catchments

4.6.2 Lag Time

The NRCS Method compares the curve numbers that dictate how much rainfall is abstracted into soil vs discharged as runoff. The SCS Dimensionless Transform method refers to the transformation of that excess rainfall from the CN method into a runoff hydrograph over time.

The SCS Dimensionless Transform method was used to derive a synthetic unit hydrograph. This method is widely accepted, well documented in the USDA National Engineering Handbook,

and built into the MIKE+ model. In addition to surface area calculated as a geometric property of the catchment, the UHM requires the definition of lag time for the catchments.

Many methods are available for calculating lag time. This study uses the Basin N lag method, originally developed by Snyder and later modified by the US Army Corps of Engineers and US Bureau of Reclamation. This method calculates lag as a function of the longest flow path, measured from the catchment outlet to the most remote point in the catchment, the centroidal flow path, a basin surface roughness characteristic, and the average slope along the principal flow path.

$$t_l = K * N * \left(\frac{L * L_c}{\sqrt{S_0}}\right)^{0.38}$$

Where: $K = \begin{cases} 24 & \text{for } L > 1.7\ mi \\ 15.22 + 2.1464 * L + \frac{8.6981}{L} & \text{for } L \leq 1.7 \text{ miles} \end{cases}$

$L =$ Longest flow path (mi)

$L_c =$ The centroidal flow path length (distance from the outlet to a point orthogonal to the catchment centroid; mi)

$S_0 =$ The average slope of the longest flow path (ft/mi)

$N =$ Basin roughness parameter, estimated in GIS based on land cover and values presented in Table 4-3.

Table 4-3: Surface Roughness (N) Values Associated with Each Zoning/Land Cover Type

Symbol	Description	Surface Roughness (N)
C-1	Neighborhood Business	0.04
C-2	General Commercial	0.04
COSC	Community Open Space	0.06
M-1	Light Industrial	0.04
M-2	Heavy Industrial	0.04
M-B	Industrial Buffer	0.04
O	Office	0.04
OR	Office/Residential	0.04
PUD	Planned Unit Development	0.04
R-1	Single-Family Residential	0.05
R-2	Two-Family Residential	0.05
RD	Road	0.04
RM	Resource Management	0.10
R-M	Multi-Family Residential	0.06

Flow paths were estimated using HEC-HMS GIS tools based on LiDAR topographic data. HEC-HMS is capable of providing tabulated longest flow paths, centroid flow paths, and flow path

slopes with built-in tools. These were extracted from the HEC-HMS interface and adjusted as necessary where automated tools did not correctly capture expected flow paths.

In order to account for the relatively small size and mild slopes in some of these basins, as well as the dominance of residential land cover and the density of tree canopy, a minimum lag time of 5 minutes is used, even if the calculated lags are less.

4.7 Hydrologic Model Calibration

Calibration is an important step in ensuring that the SDMP model represents design storm runoff characteristics as accurately as possible. CNs and lag times are derived from measurable physical characteristics of drainage areas. However, CN values initially assume average soil moisture (AMC II). For this SDMP, calibration is focused on determining whether that condition is appropriate using an analog drainage area with stream gage data that is readily available.

The nearest gaged water course is situated southeast of the City at San Francisquito Creek. The Creek has a gage near Stanford University, at the Stanford Golf Course (United States Geological Survey [USGS] gage 11164500), with a drainage area of approximately 32 square miles. Gage data spanning the period between 1951 and 2021 were evaluated with Bulletin 17B methodology in Hydrologic Engineering Center Statistical Software Package (HEC-SSP; Figure 4-13). Bulletin 17C was also evaluated, but the 17C method identified an excessive number of non-zero low outliers for the San Francisquito Creek gage (41 low outliers with a threshold of 1,560 cfs), which could improperly skew statistical analysis.

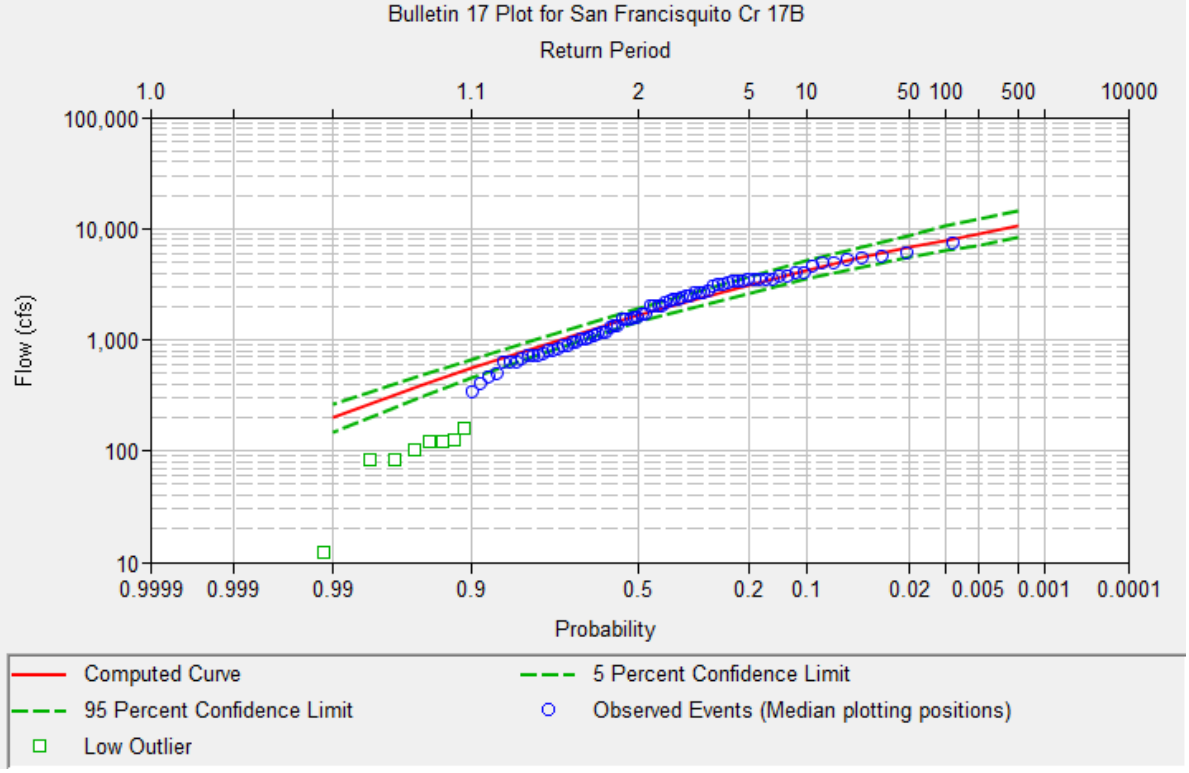


Figure 4-13: HEC-SSP Duration-Frequency Curves for San Francisquito Creek

Hydrologic properties were estimated for the 32-square-mile drainage area to the USGS gage (the AMC II CN and the lag time). Four subbasins in the model are shown in Figure 4-14.

Estimated hydrologic parameters are summarized in Table 4-4. Similar to the City drainage system catchments, this evaluation used the SSURGO soils database. However, instead of zoning polygons, the 2023 National Land Cover Database (NLCD) was used to estimate the basin's composite CN and percent impervious. Over a large, less developed area, this satellite-derived dataset is better suited to characterizing vegetated cover.

Based on the NLCD dataset, the drainage area is approximately 58% forested lands (a mix of evergreen and deciduous), 22% open space, 11% grassland/shrubs, 7% developed (roads and rural properties), and 2% other cover types. The soils in the basin are classified as 9% Type A and B hydrologic soil groups (HSGs), 73% Type C HSG, 4% Type D HSG, and 15% unclassified (considered Type D for hydrologic purposes).

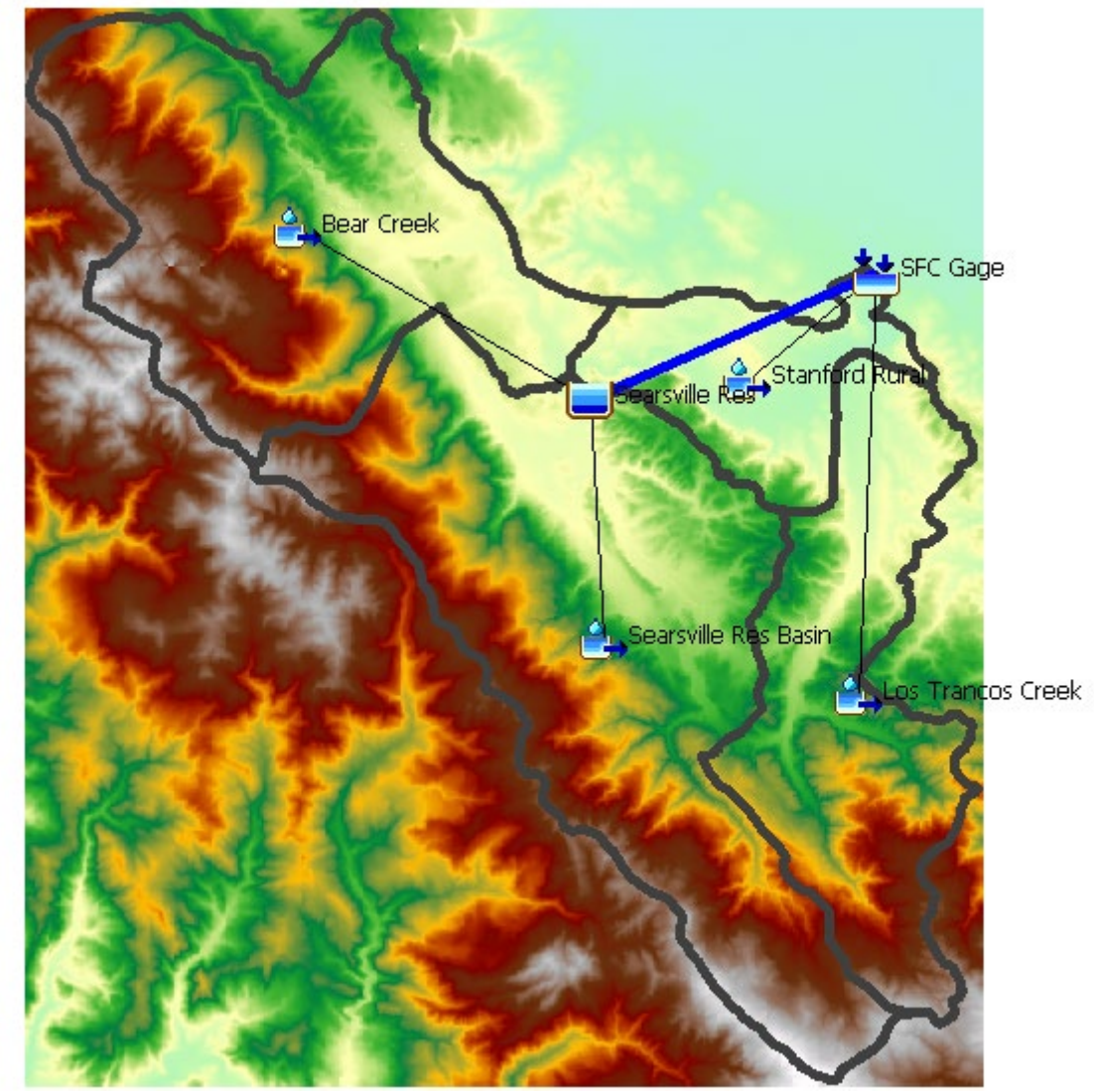


Figure 4-14: HEC-HMS Model Basin Geometry for San Francisquito Creek Gage Hydrology Calibration

Table 4-4: San Francisquito Creek at Stanford University Drainage Area Hydrologic Parameters

Subbasin	Area (sq mi)	Percent Impervious	Lag Time (min)	100-yr Rainfall Depth (in)	AMC II Pervious CN
Bear Creek	11.7	3%	136	12.29	72.2
Los Trancos Creek	7.5	4%	128	9.88	72.3
Searsville Reservoir	14.7	3%	151	10.68	69.4
Stanford Rural	3.5	13%	127	8.66	76.2

Four events were evaluated in HEC-HMS with the estimated AMC II CN (the calculated CN value from GIS analysis of the drainage area). All of the peak flow results produced by the model initially exceeded their respective HEC-SSP gage statistics, and the peaks of three out of four events exceeded the upper confidence limits produced by HEC-SSP.

Multiple factors were considered as adjustments to the model to calibrate hydrologic model inputs, including the following:

1. **Soil moisture:** Adjusting the CN values in the model due to AMC that deviates from average conditions nationally can affect peak magnitude and volume
2. **Baseflow:** Depending on the characteristics of the storm, baseflow can have some influence on the peak flow produced in even small storm events
3. **Depth-Area Reduction:** Large areas are not necessarily subject to the same precipitation return period across their entire surface. Variability in precipitation intensity during a storm can be accounted for by applying depth-area reduction factors

Schaaf & Wheeler has performed extensive modeling on the New Year's Eve 2022 event, which provides a basis for evaluating calculated lag times and CNs, as well as estimating appropriate baseflow model inputs.

Baseflow and area reduction factors are generally expected to have lesser impact on the modeled peak flows than modification to the CN, so adjustments were first made to the AMC based on prior modeling efforts. The results of that analysis are summarized in Table 4-5.

Table 4-5: San Francisquito Creek USGS Gage Peak Flow (cfs) & AMC Evaluation

Return Period (Exceedance Prob.)	HEC-SSP Predicted	HEC-SSP Confidence Limits	HEC-HMS (AMC II)	HEC-HMS (AMC I-3/4)
100-year (1%)	7,890	6,280-10,460	9,600	8,100
25-year (4%)	5,600	4,590-7,140	7,670	6,750
10-year (10%)	4,150	3,680-5,100	5,900	5,020
2-year (50%)	1,640	1,420-1,890	2,840	2,215

The New Year's Eve 2022 Event functions as a recent event with an approximate 30-year peak flow rate of 5,880 cfs to use in establishing reasonable baseflow parameters. A HEC-HMS model has been developed of the San Francisquito Creek drainage area that uses multiple precipitation gages plus a radar-derived bias grid to improve the representation of spatial variability during the storm. Linear Reservoir is the selected baseflow method, which preserves volume loss from the surface and routes it through subsurface layers based on a time coefficient and a fraction of groundwater storage occurring in each defined layer.

Additional gages upstream of the San Francisquito Creek gage were used to evaluate reasonable baseflow inputs for each subbasin in the model. Conditions across the drainage area were highly variable during this particular storm, which was centered north of the gage, nearer to San Carlos and San Mateo. The calibration model accounts for that variability as closely as possible by varying CNs to better reproduce peaks at each gaged point. Model results are shown in Figure 4-15.

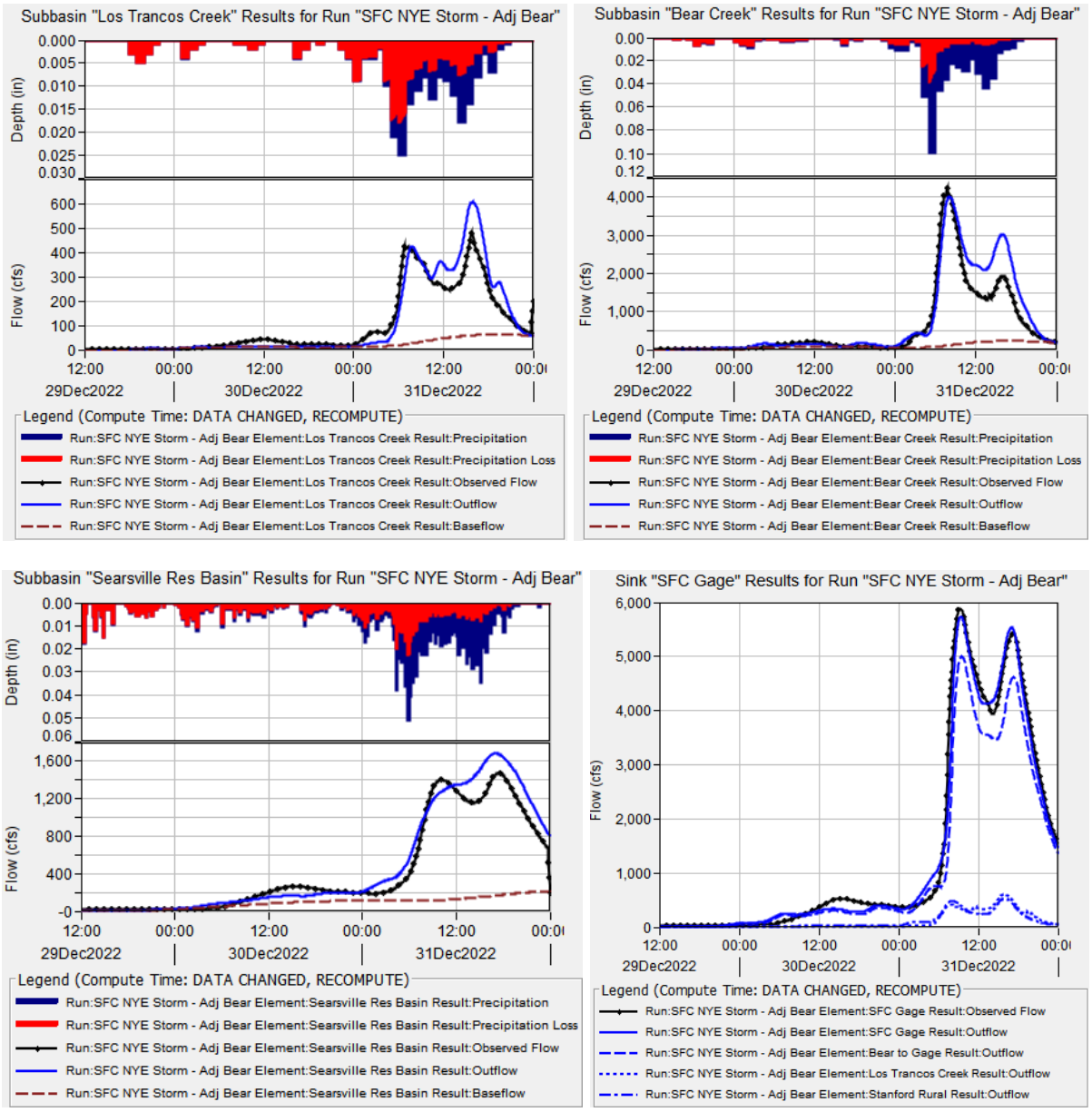


Figure 4-15: Baseflow Calibration Model Results at Gaged Points

A single baseflow reservoir layer was used to represent the more highly responsive soil layers that contribute most to increasing peak flow. Deeper groundwater layers that respond to rainfall loss on a longer timescale were ignored. Input parameters for the New Year's Eve event linear reservoirs are shown in Figure 4-16.

Linear Reservoir [SanFrancisquito - NYE Bias]

Filter: --None--

Subbasin	Number of Layers	Initial Type	GW 1 Flow Type	GW 1 Initial (CFS)	GW 1 Initial (CFS/MI2)	GW 1 Fraction	GW 1 Coefficient (HR)	GW 1 Reservoirs
Searsville Res Basin	1	Discharge Per Area	Baseflow		0	0.5	3	5
Bear Creek	1	Discharge Per Area	Baseflow		0	0.4	4	3
Los Trancos Creek	1	Discharge Per Area	Baseflow		0	0.1	4	3
Stanford Rural	1	Discharge Per Area	Baseflow		0	0.5	4	5

Figure 4-16: Linear Reservoir Inputs for the New Year's Eve 2022 Event

Spatial variability in the New Year's Eve model is accounted for by using different precipitation gages and radar rainfall data. That event is an excellent example of the impacts of storms that center on areas in excess of 30 square miles, as the Bear Creek subbasin saw significantly greater precipitation depths than Los Trancos to the south.

For the design event models, the input precipitation grids represent statistically balanced rainfall occurring across the entire surface of the drainage area. This is idealized and may overestimate design event runoff at the gage point. In order to account for the size of the calibration basin, HEC-HMS built-in area reduction was applied based on NOAA Technical Paper 40 (TP40). Reduction factors for the subbasins estimated by HEC-HMS are small, at roughly 97.5%. However, precipitation reduction and baseflow bring the HEC-HMS modeled peak flows for 10-, 25-, and 100-year events very close to the HEC-SSP predicted peaks. Results are summarized in Table 4-6.

Table 4-6: San Francisquito Creek Calibrated Hydrology Model Results

Return Period (Exceedance Prob.)	HEC-SSP Predicted	HEC-SSP Confidence Limits	HEC-HMS (AMC I-3/4) w/Baseflow and Depth-Area Reduction	% Difference from Predicted
100-year (1%)	7,890	6,280-10,460	8,380	+6.2%
25-year (4%)	5,600	4,590-7,140	5,600	0.0%
10-year (10%)	4,150	3,680-5,100	4,170	+0.5%
2-year (50%)	1,640	1,420-1,890	2,220	+35%

Ultimately, AMC I-3/4 was deemed an appropriate condition for modeling the design storm events beyond a 2-year event. If models of lesser return periods are of interest, the AMC should be decreased to AMC I-1/4, which produces a peak 2-year flow in the calibrated hydrology model of 1,875 cfs (within one standard deviation of the predicted peak).

Given the proximity of the City system's drainage area to the San Francisquito Creek drainage area, and the similar regional climate patterns, it is assumed that the selection of AMC I-3/4 is also appropriate for the City system's catchments. Baseflow methodology in the MIKE+ software differs from baseflow methods in HEC-HMS. RDII baseflow parameters were previously estimated for modeling of Marina Lagoon and have been applied to the large pervious surface areas draining to the two Laurel Creek dams. In urban subbasin areas with significant impervious surface area, baseflow will not have an impact on storm drain systems that only handle surface runoff.

4.8 Climate Change Considerations

Various tools exist to evaluate the impacts of climate change. Climate change and global SLR can be extremely impactful factors in planning a resilient surface water management system. Sea levels have risen in San Francisco area by about eight inches over the past century, and the rate of rise is predicted to increase.

4.8.1 Sea Level Rise (SLR)

Coastal protection measures have evolved significantly in the City of San Mateo since the last SDMP update. The San Mateo Creek flood walls were improved in 2001-2002, followed by improvements to the Detroit Dr Floodwall, Seal Slough Levee, East End Levee, and Bayfront Levee in 2011. Improvements to Coyote Point, Poplar Pump Station (including an extension of the Bayfront Levee protection to the west), and the Eastern and Western Promenade were completed between 2013 and 2020. Protection and recent projects in the Coyote Point-North Shoreview area are shown in Figure 4-17.

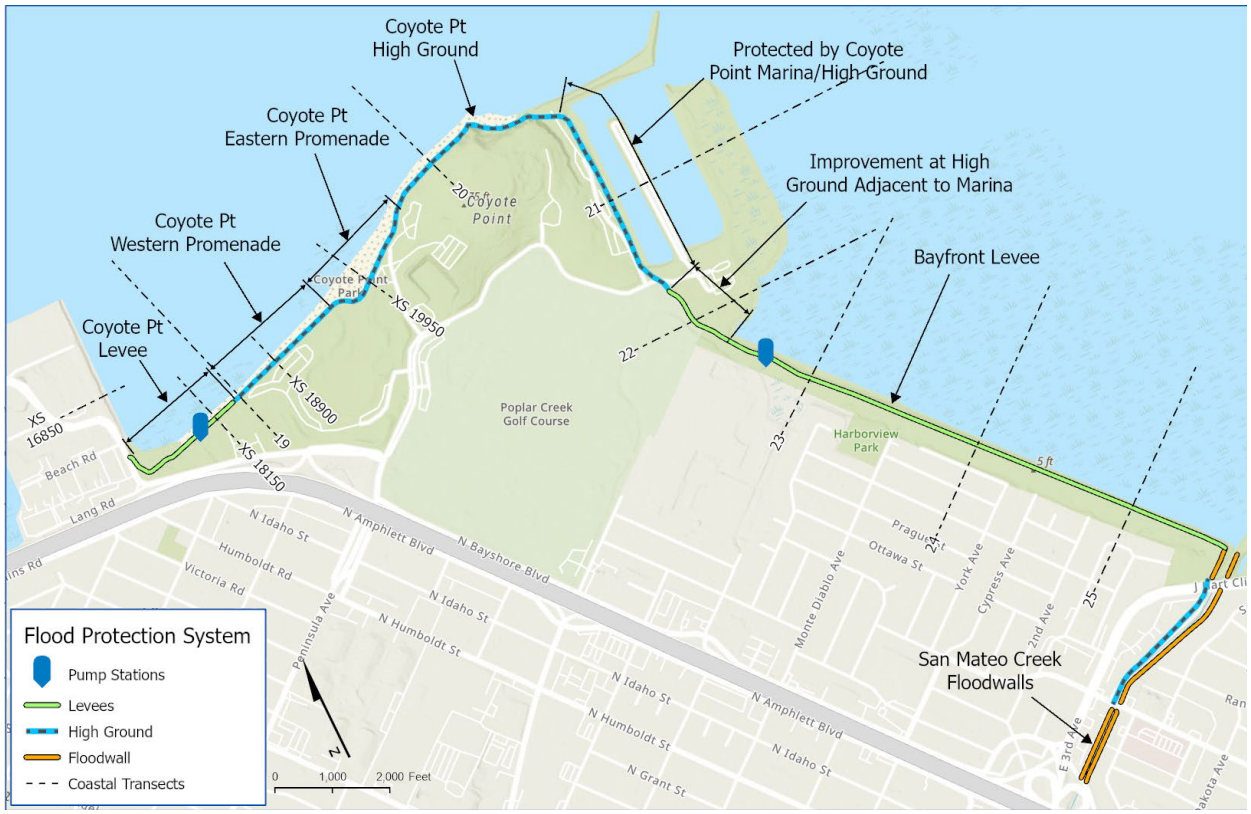


Figure 4-17: Recently Improved Shoreline Protection at Coyote Point, North Shoreview, and San Mateo Creek

While recent projects have resulted in FEMA granting accreditation to the City's levee systems, parts of the levee systems still remain vulnerable to the impacts of SLR.

SLR predictions are based on complex climate models with several variables and uncertainties. However, the California Ocean Protection Council (OPC) provides probabilistic projections of SLR intended to guide policy decisions and local project design efforts. The agency's 2024 "State of California Sea-Level Rise Guidance" report cites "Intermediate" to "High" predicted rise of 0.8 feet to 1.2 feet of rise by 2050 and 3.1 feel to 6.6 feet by 2100. These projections should

guide a number of activities in the City, including adaptation planning, coastal resilience projects, and development ordinances.

The full scope of those efforts are beyond the purpose of this SDMP. However, in the context of this effort, the impacts of SLR must be considered in the development of a prioritized CIP. In addition to evaluating stormwater facilities needed to meet drainage standards under current conditions, this SDMP should consider the impact of increased tailwater conditions during high tide events on pump station operation and on gravity outfalls to San Mateo Creek.

SLR has been evaluated with the models as an adjustment to tidal boundary conditions. Tidal boundary conditions have been shifted upward by 0.8 feet and 3.1 feet to represent 2050 and 2100 intermediate estimates from the latest OPC document, respectively. Impacts of this adjustment are evaluated for gravity systems and pump stations to aid in identifying the most effective solutions to known flooding problems and prioritizing recommended projects.

An approximate, high-level evaluation of the longevity of existing coastal protection has also been performed based on the approximate elevations of San Mateo shoreline features. This simple GIS exercise estimates when each segment of shoreline will become susceptible to overtopping by the 1% Total Water Level (TWL). Wave runup does not change linearly with sea levels, and exact changes in TWL with SLR have not been calculated. Instead, it is assumed that wave setup and runup increase by approximately 5%–10% based on prior coastal protection design and modeling work. These figures are simply meant to highlight the most susceptible reaches of shoreline as sea levels rise.

Shoreline susceptibility is shown in Figure 4-18 for the case of OPC intermediate SLR, and in Figure 4-19 for the case of OPC high SLR. The windsurfing ramps at the Western Promenade are already overtopped by 1% TWL conditions, and a short reach of the left bank of San Mateo Creek where no floodwall exists is overtopped by the 1% tidal stillwater condition. Overtopping has been mapped and acknowledged by FEMA at these locations. However, as sea levels rise, interior flooding caused by overtopping at these locations could worsen. The western promenade and high ground on the left bank of San Mateo Creek are potentially the most at-risk portions of San Mateo's shoreline, followed by high ground south of J Hart Clinton Dr near Marina Lagoon.

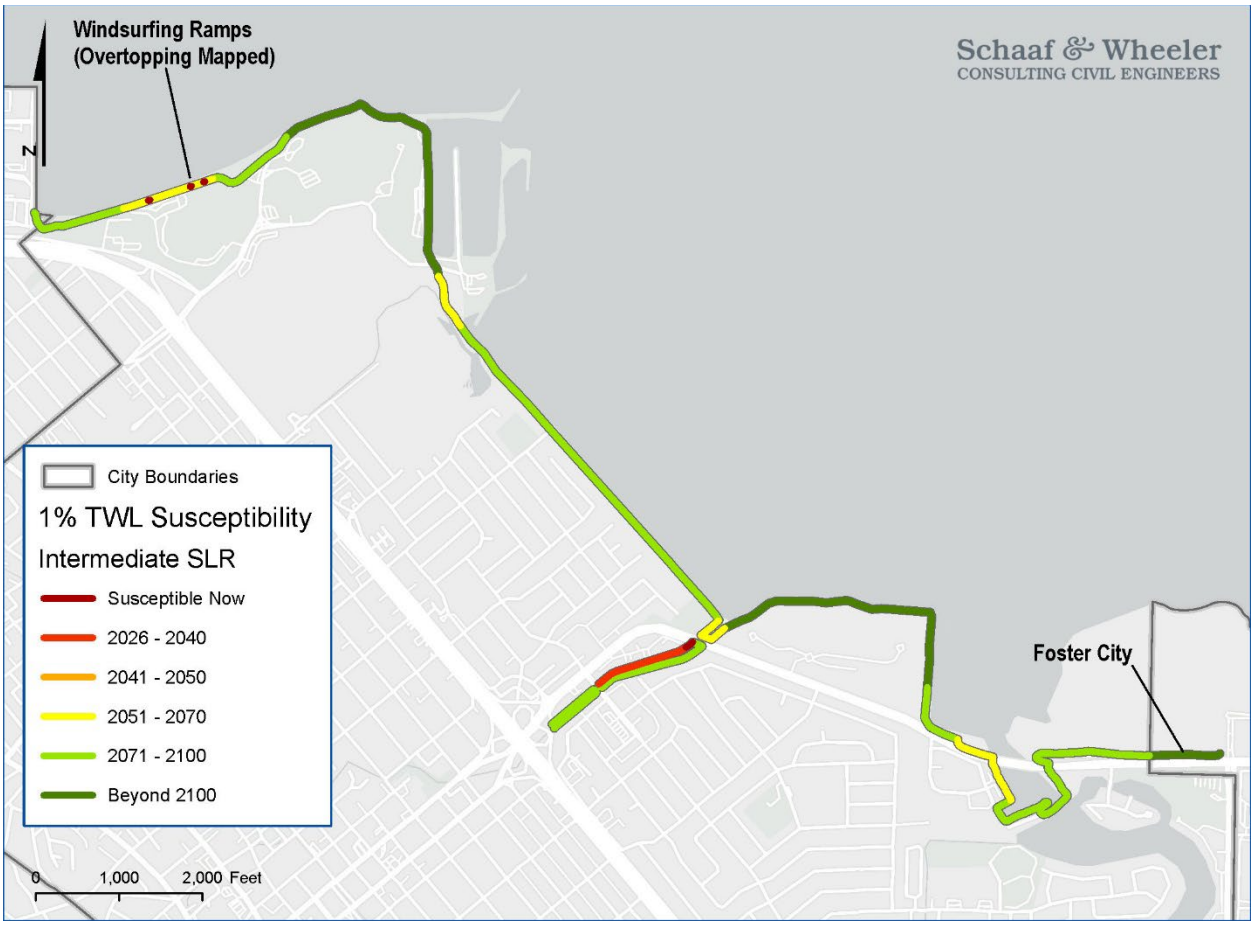


Figure 4-18: Shoreline Feature Susceptibility to SLR Based on Intermediate OPC Estimates

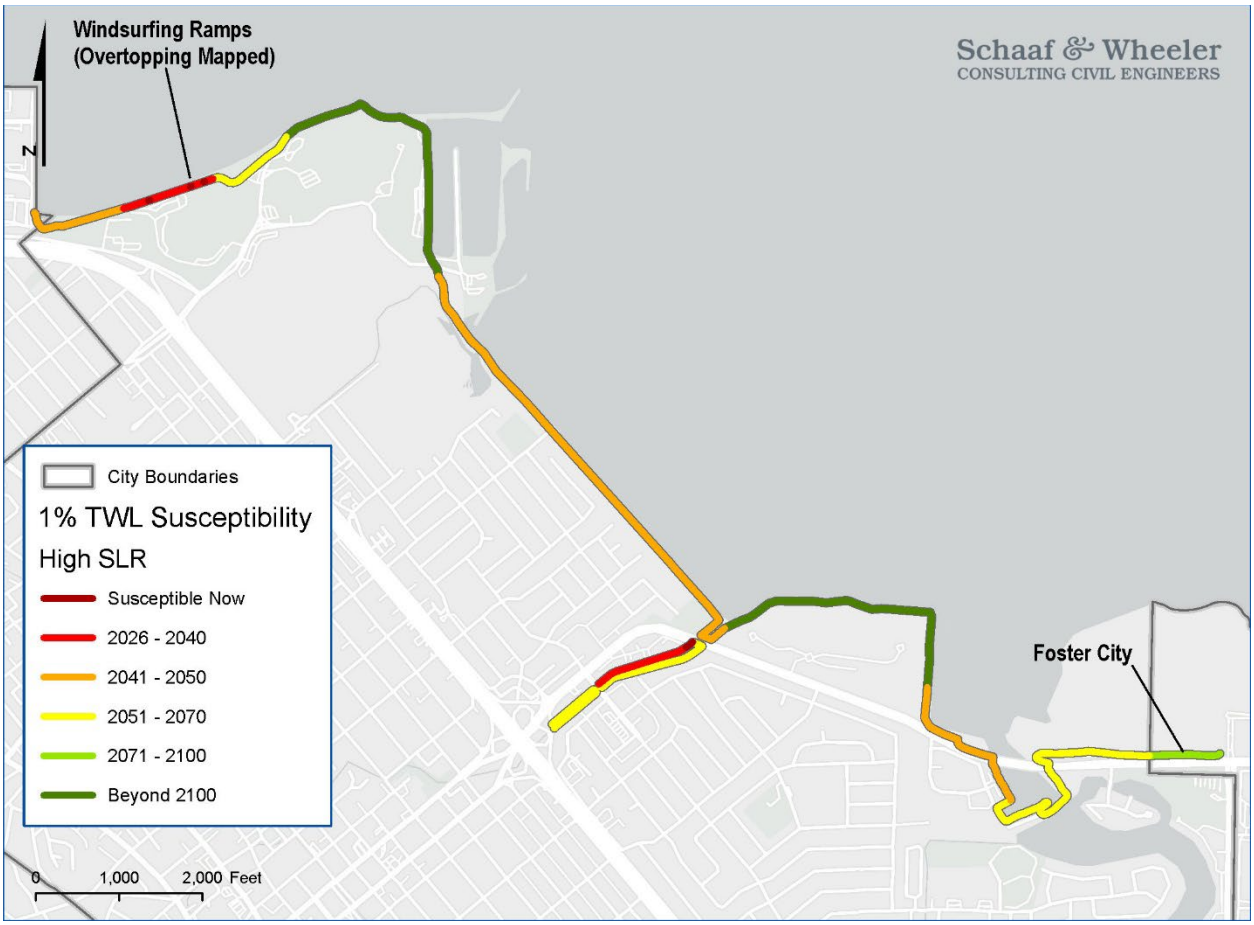
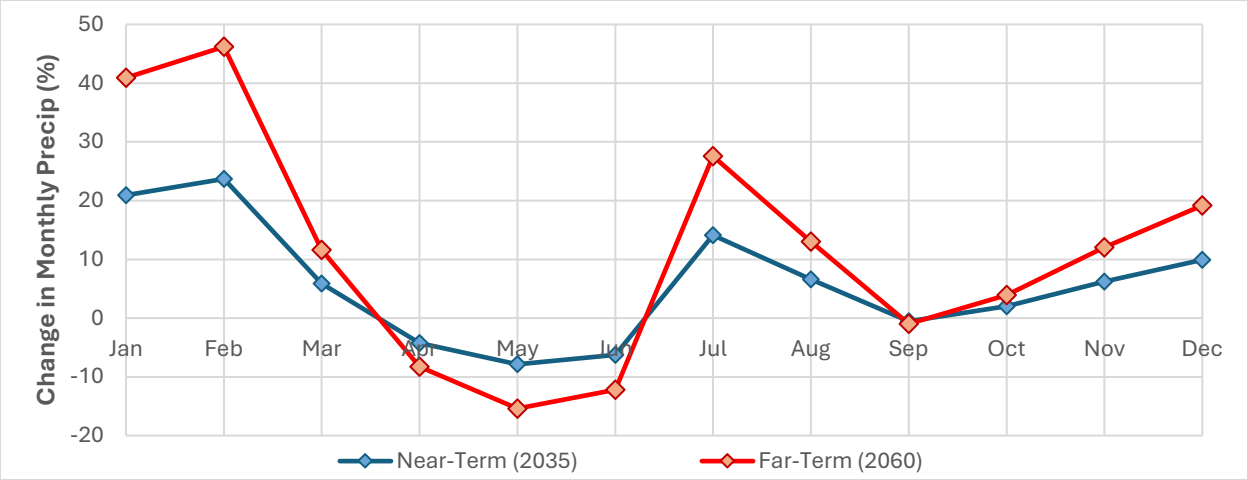


Figure 4-19: Shoreline Feature Susceptibility to SLR Based on High OPC Estimates

4.8.2 Precipitation

SDMP models also consider the potential impacts of climate change. This is accomplished by using the US Environmental Protection Agency (EPA) Storm Water Management Model Climate Adjustment Tool (SWMM-CAT). SWMM-CAT provides location-specific adjustment factors for near-term (2035) and far-term (2060) projects derived from global climate change models developed by the World Climate Research Program. The tool generates adjustment factors for several climate parameters. However, this analysis is primarily concerned with future changes in precipitation. Predictions generated by the tool are shown in Figure 4-20 for near- and far-term cases, respectively.



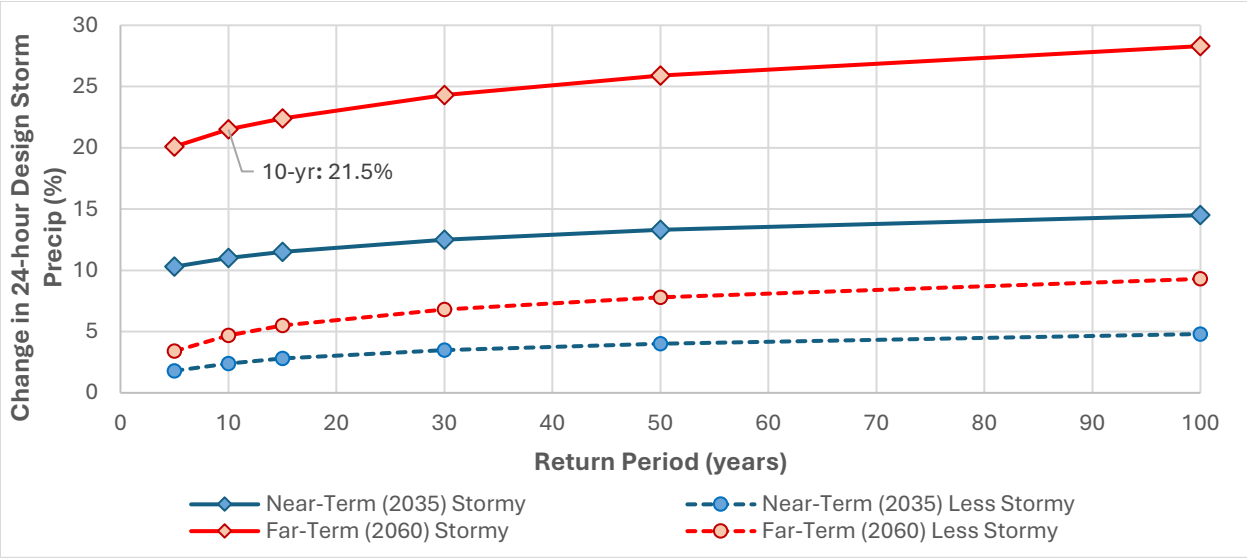


Figure 4-20: SWMM-CAT Near-Term Monthly Rainfall and 24-Hour Design Storm Change Predictions for San Mateo

The City stormwater system model uses single storms to evaluate system capacity and identify projects. While the predictions of SWMM-CAT indicate monthly and seasonal increases in precipitation depths up to nearly 50%, single 24-hour design storm changes are estimated on the order of 20%-25% for 10- to 30-year storm return period events.

A resource developed by Eagle Rock Analytics, UC Berkeley, the Lawrence Berkeley National Lab, and The California Energy Commission, Cal-Adapt provides another basis for evaluating the impacts of future climate change for California. The Cal-Adapt website includes a tool for estimating increases in extreme precipitation events, with a baseline dataset that covers the period from 1960 to 1990. Model estimates are provided for two climate scenarios (medium emissions and high emissions) and for two time periods (mid-century, from 2035-2064, and end-century, from 2065 to 2099). These are provided for a 24-hour precipitation event in San Mateo in Figure 4-21.

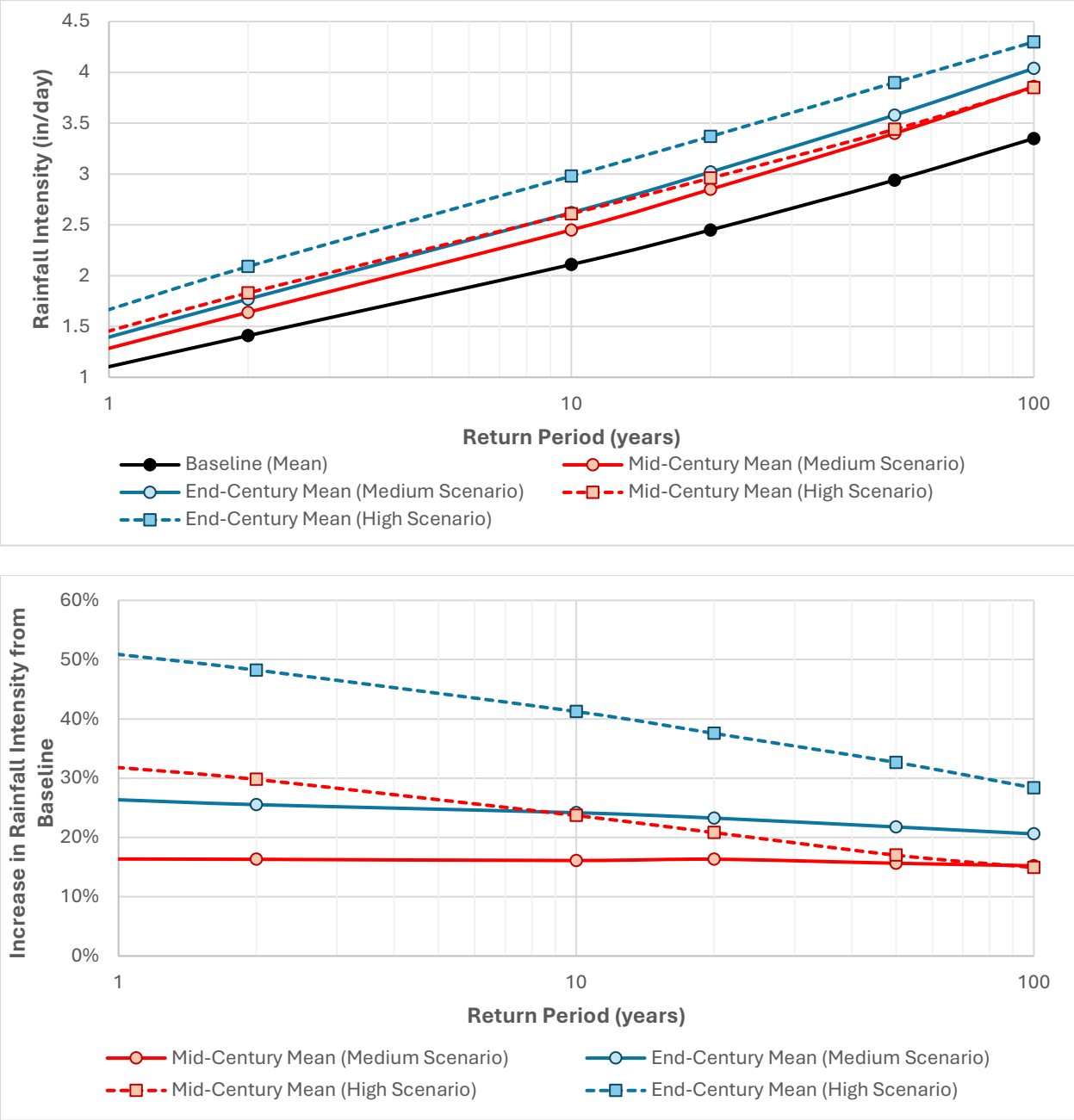


Figure 4-21: Cal-Adapt Extreme Precipitation Event Predictions (Top) and Percent Increase by Return Period (Bottom) for San Mateo.

Cal-Adapt notes that the baseline period of 1960 to 1990 covers the construction of California's most critical infrastructure and the start of significant anthropogenic climate change impacts.

The climate change model was not used to increase the size of drainage network elements for recommended projects. Instead, this climate change model is used to highlight where the City's CIP may require additional climate change consideration during design and to identify where additional projects may require consideration in the future, when the CIP is updated. There are several locations where additional, significant flooding appears in the climate change model.

Most correspond with projects that will likely be done after a future update to the SDMP. However, certain projects identified as Very High priority should be adjusted, as needed, during design to provide resilience against climate change.

In choosing an approach to modeling climate change, adjustments must be made to EPA Climate Assessment Tool (EPA-CAT) and Cal-Adapt values to account for a shift in baseline. For example, the precipitation-frequency dataset used to generate the design storm inputs to the models is from a MetStat analysis that uses measured precipitation data through 2015. For Cal-Adapt, trendline fits indicate that approximately 5%-10% of predicted change in extreme precipitation intensity compared with the baseline had already occurred by 2015. For SWMM-CAT, extreme precipitation events used as a baseline were taken from NOAA datasets covering the period from 1981 to 2010.

Change in extreme precipitation from both sources is summarized in Table 4-7 and Table 4-8. The ranges of predictions are shown graphically in Figure 4-10.

Table 4-7: Extreme Precipitation Event Change Predictions Based on Cal-Adapt (Can-ESM2 Model)

Return Period	Change Up to 2015 (%)	Change from 2015 to 2050 (%)	Change from 2015 to 2100 (%)
2	5–9	7–13	20–37
10	5–6	7–11	18–34
100	~5	6–8	15–22

Note: Ranges represent change for Medium and High emissions scenarios

Table 4-8: Extreme Precipitation Event Change Predictions Based on SWMM-CAT

Return Period	Change Up to 2015 (%)	Change from 2015 to 2050 (%)	Change from 2015 to 2100 (%)
2	0.1–0.9	1–10	3–32
10	0.3–1.3	3–12	8–35
100	0.6–1.7	5–15	15–46

Note: Ranges represent change for Less Stormy and Stormy conditions

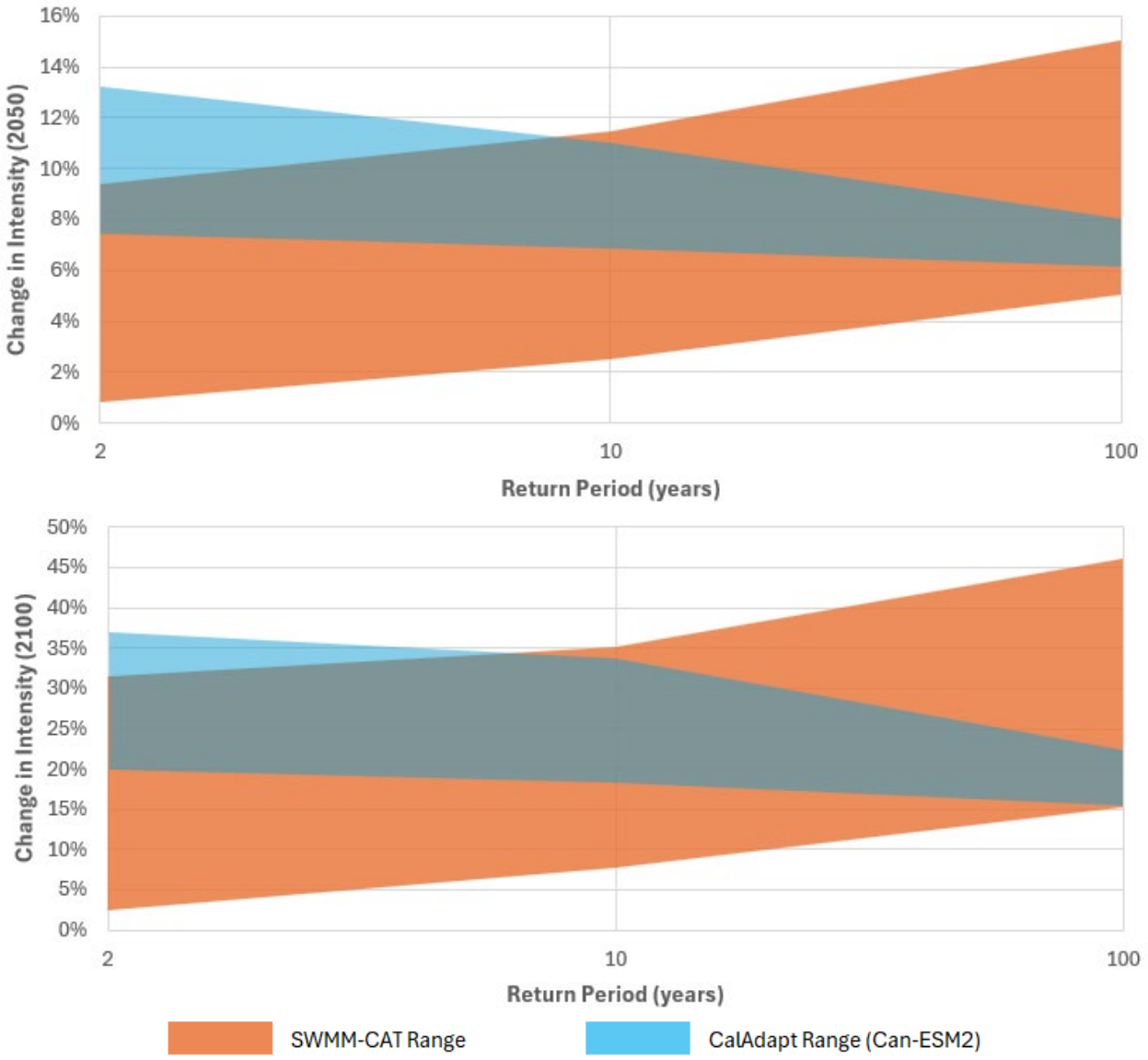


Figure 4-9: Composite Plots of Cal-Adapt and SWMM-CAT Extreme Precipitation Predictions

SWMM-CAT produces a wider range of conditions, as the Less Stormy condition represents ensemble models with less change in precipitation per degree of warming.

Climate change model scenarios uniformly increase design storm depths by percentages summarized in Table 4-10. This would likely cover the design lifetime of capital improvement projects taken forward through planning and construction as a result of this plan and falls within the range of predictions from both SWMM-CAT and Cal-Adapt for wet conditions.

Table 4-10: Climate Change Scenario Precipitation Increase Factors for Model Inputs

Scenario	10-yr Storm	100-yr Storm
Near-Term (2050)	10%	8%
Far-Term (2100)	30%	20%

5. Pump Station Condition Assessment

This section of the SDMP details a condition assessment for the City's storm drain pump stations. The section also summarizes the assessment, recommended improvements, and estimates of probable construction costs for the recommended improvements. The following pump stations are included in this assessment, and their locations are shown in Figure 5-1. The El Camion Real Pump Station near Hillsdale is not included in this assessment, as it is not maintained by the City

1. 4000 Pacific
2. Hillsdale
3. Casanova
4. Poplar Ave
5. Coyote Point
6. Fathom Dr
7. Marina Lagoon
8. 3rd & Detroit
9. 16th Ave
10. 31st Ave

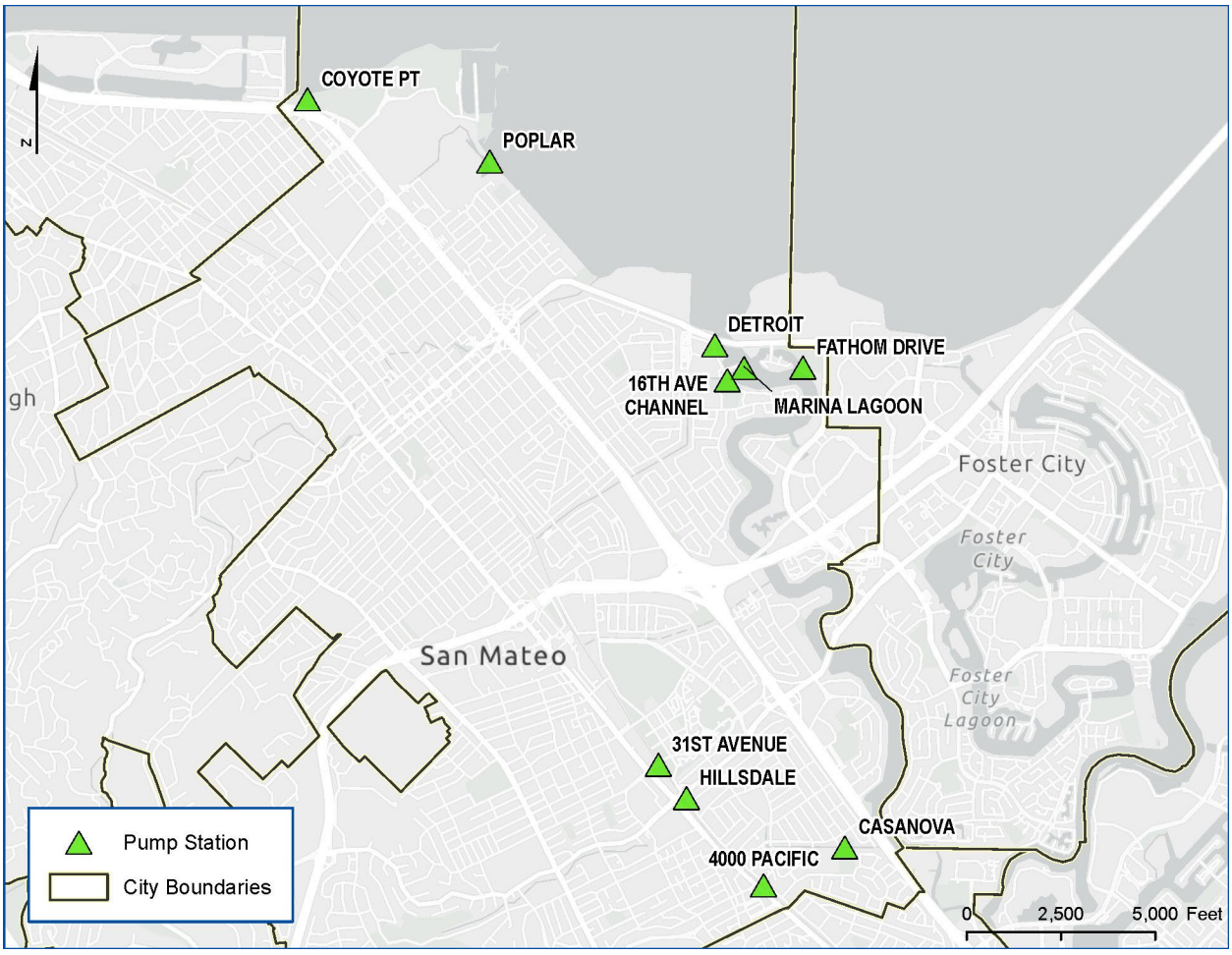


Figure 5-1: Pump Station Locations Map

Assessment of the pump stations is limited to visual observations, discussions with City staff, and information provided by the City. The pump station assessments were performed on April 22, April 30, and May 14, 2025, and on January 14, 2026. Information regarding operating conditions and known issues were obtained from City staff during the site visits and follow-up correspondence. Site visit forms with existing pump station equipment information can be found in Appendix E. Responses to operational questions from Operations and Maintenance (O&M) staff can be found in Appendix F.

The upgrades recommended herein are based on typical service life of pump station equipment and structures. Anticipated service life of pump station equipment varies greatly based on several factors. It may be necessary to replace some equipment earlier than noted, and some equipment may last longer than expected. The primary objective of this assessment is to identify capital projects necessary to improve the reliability and safety of the pump stations and to identify capacity constraints and operational issues.

5.1 Pump Station Evaluation Framework

This section describes the framework used to evaluate the City's ten storm drain pump stations. The evaluation combines hydraulic capacity analysis, performance metrics for different station types, condition and operations criteria, and a prioritization system that, together, provide a consistent basis for identifying and ranking improvement needs.

5.1.1 Overview of Pump Station Evaluation Approach

The pump station evaluation criteria provide a consistent framework for comparing condition and improvement needs across all of the City's storm drain pump stations. Each criterion focuses on a specific aspect of performance: capacity; station age and remaining life; physical condition; operations and maintenance burden; life safety; emergency capability; and operational performance. By scoring each station against the same set of criteria, the Master Plan can prioritize short-term and long-term rehabilitation projects in a transparent way.

These criteria are applied in conjunction with the hydraulic capacity evaluation and the prioritization metrics described in this section to develop station-specific assessments and recommendations.

5.1.2 Pump Station Hydraulic Capacity

The existing pump station capacities are estimated based on pump product data, pump performance curves, and system operating conditions. System curves are developed by calculating the total dynamic head (static head plus friction and minor losses) over a range of flow rates and plotting head versus flow. Total static head is estimated from the operating wet well level elevation to the controlling discharge pipe elevation (either outfall pipe or tailwater elevation) using available as-built plans and observations made during site visits. For each pump station, pipe lengths, diameters, fittings, and outfall or tide conditions are used to compute the head loss at each trial flow. The resulting head–flow relationship represents the system curve when combined with the static lift. The system curve and the pump performance curve are plotted together on the same graphical scale. The intersection of the system curve and the pump curve represent the operating capacity of the system for one pump. Depending on how many pumps there are at the pump station, this capacity is multiplied by the total number of pumps to determine the total station capacity.

Pump curves are obtained from City records or directly from pump manufacturers, where available. A pump curve is not available at 16th Ave Pump Station; therefore, the design

capacity cannot be identified. However, this station does not run during the winter rainy season and serves no flood control function, so its hydraulic capacity is not relevant for the purposes of the SDMP. In addition, Hillsdale Pump Station as-built drawings identify different motor speeds than those observed in the field, indicating that an equipment replacement has occurred. Pump nameplates are not accessible, so a hypothetical pump curve is used as a conceptual substitute, as discussed further in Section 5.3.2.

Typically, pump capacities are reduced over time due to component wear and additional hydraulic losses due to material deposition in the force mains, fittings, and valves. Therefore, the current operating capacity of the pump stations may be less than what has been estimated. Field verification of actual pump flow rates is impractical for this study due to the lack of flow meters at the stations and the lack of access to discharge piping to install temporary meters.

The resulting station capacities are compared to the performance criteria described below to determine whether each station meets its hydraulic requirements.

5.1.3 Pump Station Performance Evaluation Criteria

The pump hydraulic analysis evaluates how each pump station performs under existing and future hydraulic conditions so that capacity and reliability improvements can be prioritized. The storm drain system computer model is used to determine the required pump station capacity to meet system performance criteria, and that requirement is then compared to the hydraulic capacity estimated for each pump station. HEC-HMS software is used to approximate the 10- and 100-year flows to each pump station. Hydraulic modeling inputs and methodology are discussed in detail in Section 5.2. These flows are estimates, and it is recommended that the drainage basins be studied further as part of the pump station improvement design process.

For the purposes of this evaluation, a pump station's capacity is defined as the station's capacity during design storm and tailwater conditions, with all pumps operating, and with adequate backup power available. Electric motor-driven pumps that do not have either a portable generator connection or an on-site generator sized to run the pumps are not counted as usable capacity in this assessment. Several pump stations have portable generator hookups and share a single portable generator. In the tabulated capacities, it is assumed that these stations are operating on backup power when needed. However, based on discussions with O&M staff, this is not feasible systemwide during a large storm event, so crews prioritize certain stations over others when using the shared generator.

Pump stations are classified as "terminal," "interior," and/or "flood control," which determines the design metrics used for analysis. Terminal pump stations have outfalls that discharge directly to San Francisco Bay and are impacted by high tide events and SLR. All flood control pump stations are also terminal stations, but they provide flood protection and must meet FEMA standards. Interior pump stations do not discharge directly to the Bay, and therefore, are not tidally influenced; many interior stations receive flow from storm drain pipe systems and discharge to downstream storm drain pipe systems.

Of the City's storm drain pump stations, four are terminal: Poplar Ave, Coyote Point, Marina Lagoon, and 3rd Ave & Detroit Dr. For these pump stations, system curves are developed for the Mean Higher High Water (MHHW) and 100-year Stillwater Elevation (100-year tide) using San Francisco Bay tidal datums. Additionally, SLR elevation increases for years 2050 and 2100 are included in addition to the 100-year tide elevation. Intermediate increases of 0.8 feet for 2050 and 3.1 feet for 2100 are utilized from the State of California Sea Level Rise Guidance

(Ocean Protection Council, 2024). The remaining pump stations do not have tidally influenced outfalls and are therefore not considered terminal pump stations.

In determining whether a station has adequate capacity, interior pump stations are analyzed against the 10-year storm, and flood control pump stations are analyzed against the 100-year storm. Terminal pump stations are analyzed against the MHHW tailwater condition in alignment with the storm drain computer modeling. The 3rd Ave & Detroit Dr Pump Station is a terminal pump station, but not a flood control facility, so it is analyzed to the 10-year storm. Storm drain systems are not tributary to Fathom Dr and 16th Ave, so their hydraulics play no significant role in this SDMP.

Table 5-1 summarizes the capacity analysis metrics used for each station.

Table 5-1: Storm Drain Pump Station Analysis Metrics

Pump Station Name	Station Type	Capacity Metric
4000 Pacific	Interior	10-year
Hillsdale	Interior	10-year
Casanova	Interior	10-year
Poplar	Flood Control/Terminal	100-year/MHHW
Coyote Point	Flood Control/Terminal	100-year/MHHW
Fathom	Interior	Not hydraulically significant
Marina Lagoon	Flood Control/Terminal	100-year/MHHW
3rd and Detroit	Interior/Terminal	10-year/MHHW
16th Ave	Interior	Not hydraulically significant
31st Ave	Interior	10-year

Because terminal pump stations have tidally influenced outfalls and may be affected by SLR, it is also important to evaluate brake horsepower (BHP) requirements of the pumps, as these stations may experience higher tailwater conditions. Higher tailwater conditions require higher BHP to operate. The hydraulic analysis discusses the BHP required at each tailwater condition for the terminal pump stations to determine if current equipment is able to operate at extreme conditions.

5.1.4 Overall Pump Station Evaluation Criteria

The pump station evaluation criteria are used in combination with the hydraulic analysis to identify and compare deficiencies across facilities. These criteria address non-hydraulic aspects of performance, including age, condition, operations and maintenance, safety, emergency capability, and overall operational performance.

Capacity

The Capacity category evaluates whether the pump station can convey its required design storm flows under the governing tailwater condition with all usable pumps operating and adequate backup power available. Interior pump stations are checked against the 10-year storm, while flood control pump stations are checked against the 100-year storm. Terminal stations are also evaluated against MHHW and, where applicable, higher tailwater conditions associated with SLR.

Station Age and Life Remaining

Station Age and Life Remaining play an important role in determining long-term improvements. When a station reaches approximately 20 years of service, maintenance demand and reduced reliability associated with aging equipment begin to increase. After about 40 years of service, the pump station typically experiences significant maintenance demand, challenges finding replacement parts, considerable reliability deficiencies, and safety concerns that require full station rehabilitation, including an overhaul of mechanical and electrical systems and equipment. For stations older than 40 years, a detailed condition assessment to evaluate structure, force main piping, and outfall condition should be conducted.

Physical Condition

The Physical Condition category describes the general state of the pump station's structures and equipment. It considers signs of deterioration such as corrosion, cracking, leaks, or general wear. Physical condition deficiencies indicate that components may be reaching the end of their useful life, which can reduce reliability and increase maintenance needs.

O&M Burden

The O&M Burden category evaluates how the configuration and site characteristics of the station affect the effort required to operate and maintain it. Factors such as poor equipment accessibility, frequent required repairs, or obsolete components can increase the workload for staff. A high O&M burden typically results in greater operational costs and reduced efficiency in daily maintenance activities.

Life Safety

The Life Safety category considers the safety of personnel and staff during normal operations and maintenance activities. It identifies potential hazards such as electrical risks, confined space entry, fall hazards, or inadequate ventilation. Addressing life safety deficiencies ensures compliance with safety regulations and helps protect workers in routine and emergency activities.

Emergency Capability

The Emergency Capability category assesses the station's ability to remain functional during emergencies such as power outages, flooding, or equipment failures. Deficiencies may include the absence of reliable backup power, limited redundancy, or inadequate emergency backup controls. Strong emergency capability is essential for maintaining service and minimizing flood impacts during critical events.

Operational Performance

The Operational Performance category evaluates how the station's design, configuration, and operating conditions affect its ability to convey flows as intended under varying hydraulic conditions. A deficiency in this area that often arises is poor trash rack design that restricts flow to the pumps. Site-specific conditions, such as fluctuating lagoon or tidal levels, can also impact performance. These deficiencies indicate that, though equipment may be functional, the overall hydraulic performance of the station is limited by design and operating constraints, preventing it from consistently meeting necessary capacities.

Evaluations of the ten storm drain pump stations have been completed, and short- and long-term improvements have been identified based on noted deficiencies and general industry guidelines. The short-term improvements are intended to address condition and reliability issues. If the long-term improvements are not scheduled in the near future, then it is recommended to implement the short-term improvements. The long-term improvements include full pump station rehabilitations, with replacement of all major electrical and mechanical equipment and rehabilitation of the structures. The identified improvements do not include standard pump station O&M procedures and improvements.

5.1.5 Pump Station Prioritization Criteria

To systematically prioritize short- and long-term improvements for the City's ten storm drain pump stations, criteria have been developed based on physical deficiencies, capacity needs, and the impact of flooding on the community. Table 5-2 summarizes the prioritization criteria.

Based on prioritization criteria, points are assigned to short- and long-term projects for each pump station. The results confirm the importance of improving Marina Lagoon, which serves a significant portion of the City. For long-term projects that rank above short-term projects for the same pump station, it is recommended that the short-term projects be combined with the long-term projects to be completed together.

Table 5-2: Pump Station Prioritization Criteria

Metric	Description	Category/Method	Assigned Points
Existing Capacity	Increased priority if project resolves current system capacity deficiency	Yes	10
		No, project does not increase capacity	0
Climate Change Resiliency	Increased priority if project can increase capacity to 2100 SLR predictions	Yes	10
		No	0
Station Age and Life Remaining	Increased priority if project replaces mechanical and electrical equipment for stations over 40 years old	Yes	10
		No, station equipment is less than 40 years old	0
O&M Burden	Increased priority of projects that offset frequent maintenance	Offsets O&M burden	5
		No O&M improvement	0
Life Safety	Increased priority for projects that improve safety for O&M staff	Safety is improved for O&M staff	5
		Safety is not improved for O&M staff	0
Emergency Capability	Increased priority for projects that increase emergency capability	Adding/replacing on-site generator	5
		Upgrading SCADA to report wet well levels	5
		Adding backup controls	5
Equity Priority Area	Increased priority to projects serving Community Development Block Grant priority neighborhoods	Watershed within priority area	5
		Watershed not within priority area	0
Area of Impact	Total area of flooding impact during a design storm total pump failure	Scaled area range	0-15
Maximum Depth	Maximum depth during a design storm total pump failure	Scaled depth range	0-15
Response Time	O&M response time required before one foot of ponding	Scaled time range	0-15
Immediate Need	Increased priority to projects that serve an immediate need	Project serves an immediate need	20
		Project does not serve an immediate need	0

5.2 Systemwide Pump Station Hydraulic Station Flood-Risk Summary

This section summarizes how the City's storm drain pump stations perform collectively under design storm and failure conditions. Systemwide capacity, inundation, and response time results are presented to show how the stations function together, where failures would have the greatest community impacts, and how quickly operations staff must respond. These results provide context for the station-specific assessments and recommendations in Section 5.3.

5.2.1 Pump Station Hydraulic Model Validation and Capacity Summary

The hydraulic computer model is dynamic, simulating pump discharge over a range of wet well water levels and tailwater levels, which in turn produce varying discharge rates. In contrast, the pump station hydraulic analysis uses fixed system curves based on a single static lift condition

for each case, either the applicable tailwater elevation for terminal facilities or the top of discharge elevation for interior stations, together with the operating wet well level.

To align the two approaches, the static lift at maximum discharge from the model is compared to the maximum discharge at the same static lift on the system curves. The results are summarized in Table 5-3. For all stations, the modeled maximum capacities are within about 10% of the system-curve capacities, indicating that the model and system-curve analyses are in reasonable agreement.

Section 5.3 presents the detailed system-curve capacity evaluation for each pump station and describes how the available capacity compares to the required design flows. The station-by-station discussion provides the technical basis for the systemwide summary presented here.

Based on the computer model, upsizing the 4000 Pacific Pump Station to produce approximately 7,200 gpm is recommended. Although the 3rd & Detroit Pump Station is evaluated as having only the diesel pump available for design conditions, the modeling indicates that a second pump is not necessary to convey the 10-year storm flows.

Table 5-3: Pump Station Capacity Summary

Name	Capacity and Tailwater Metric	Maximum Modeled Discharge (gpm)	Static Lift at Maximum Modeled Discharge (ft)	Validation Discharge (gpm)
4000 Pacific	10-year	6,300	6.5	6,300
Hillsdale	10-year	2,900	11	2,700
Casanova	10-year	11,200	4	11,000
Poplar	100-year/MHHW	51,200	6.5	50,000
Coyote Point	100-year/MHHW	47,500	7.5	50,000
Fathom	Not hydraulically significant	–	–	–
Marina Lagoon	100-year/MHHW	149,900	2.7	160,000
3rd and Detroit	10-year/MHHW	20,200	12.5	21,300
16th Ave	Not hydraulically significant	–	–	–
31st Ave	10-year	2,200	16	2,100

5.2.2 Pump Station Hydraulic Failure Results

The extent of inundation area, maximum flood depth, and response time highlight the importance of the City's storm drain pump stations. Table 5-4 summarizes the inundation area with water depths greater than 0.5 feet under a complete pump station failure in public ROW and on private property, based on the computer model; depths less than 0.5 feet are assumed to be contained within public street curbs and gutters. Some pump stations are not included in the model, as discussed in Section 3.1.7.

Because Coyote Point and Poplar Pump Stations drain the same basin, they are analyzed together. The baseline inundation area represents flooding when all pumps are operating. The results show that Coyote Point, Poplar, and Marina Lagoon Pump Stations have particularly large areas of community impact. Plan-view figures of inundation area and depth are provided in Appendix G.

Table 5-4: Storm Drain Pump Station Failure Inundation and Depth

Pump Station	Capacity Metric	Baseline Inundation Area (Acres)	Pump Failure Inundation Area (Acres)	Baseline Inundation Depth (ft)	Pump Failure Maximum Depth (ft)
4000 Pacific	10-year	0.2	0.6	2.1	5.3
Hillsdale	10-year	--	0.5		6.1
Casanova	10-year	--	0.1	0.5	0.6
Coyote/Poplar	100-year	2.8	190	1.6	4.9
Marina Lagoon	100-year	42	598	2.9*	4.1*
3rd & Detroit	10-year	0.1	9.1	0.7	1.3
31st Ave	10-year	--	0.4	0.0	5.2

*Maximum depth location outside lagoon shoreline

Table 5-4 summarizes inundation depth under complete pump station failure in public ROW or private property. Baseline inundation depth represents the maximum water depth when all pumps are running. The results indicate that Coyote Point, Poplar, Marina Lagoon, 4000 Pacific, Hillsdale, And 31st Ave Pump Stations are especially critical with respect to flood depth. Maximum depth over two feet during a total station failure can create life-safety hazards for the surrounding community.

Table 5-5: Approximate Storm Drain Pump Station Failure Response Time

Pump Station	Capacity Metric	Response Time (hours)
4000 Pacific	10-yr	2.6
Hillsdale	10-yr	3.6
Casanova	10-yr	–
Poplar	100-yr	17.6
Coyote	100-yr	2.8
Marina Lagoon	100-yr	24.2
Detroit	10-yr	19.4
31st Ave	10-year	5

5.2.3 Key Systemwide Pump Station Findings

Taken together, the capacity and failure analyses show that the hydraulic model and system-curve analyses are in good overall agreement and most stations meet their assigned design storm criteria. Under a complete pump station failure, Coyote Point/Poplar and Marina Lagoon exhibit particularly large inundation areas, reflecting the extensive basins they serve, and Coyote Point/Poplar, Marina Lagoon, 4000 Pacific, and Hillsdale can experience modeled depths greater than two feet, creating potential life-safety concerns for the surrounding community.

Response time before flooding also varies significantly by station, with relatively short response windows at 4000 Pacific, Hillsdale, and Coyote Point, and longer, but still critical, response windows at Poplar, Marina Lagoon, and 3rd & Detroit. These findings establish how the pump stations perform at a systemwide level under both normal operation and failure conditions and,

together with the station-specific system-curve capacity analysis and condition assessments, inform the prioritization of near-term reliability improvements and long-term capital projects presented later in this report.

5.3 Pump Station Assessments and Recommendations

This section presents station-specific assessments and recommendations for each of the City's ten storm drain pump stations. Building on the evaluation framework in Section 5.1 and the systemwide hydraulic and flood-risk summary in Section 5.2, each individual station summary consolidates the key information needed to understand existing performance, identify deficiencies, and define appropriate improvements.

For each pump station, the assessment includes a brief description of existing facilities and operations, a summary of hydraulic performance relative to the applicable design storm and tailwater conditions, and an overview of condition and operational issues based on field observations and staff input. Recommended improvements are then presented in terms of near-term reliability and safety measures, followed by long-term rehabilitation or replacement projects. These station-level assessments provide the technical basis for CIP planning and prioritization presented in Chapter 7 of this SDMP.

5.3.1 4000 Pacific Pump Station Assessment

4000 Pacific Pump Station Existing Summary

Constructed in 1988 at 4000 Pacific Blvd, the 4000 Pacific Pump Station serves the 42nd Ave railroad undercrossing, pumping runoff collected in the depressed roadway to a box culvert on Pacific Blvd. See Figure 5-2 for site location. Pump station discharge ultimately drains to Laurel Creek. The tributary area of the pump station is 94 acres. Equipment consists of two Peerless 14-inch axial-flow stormwater pumps (3,300 gpm at 26 ft TDH, each pump) driven by 30-hp vertical electric motors located below grade in a dry well, and one submersible sump pump for wet well maintenance. Access to the dry well is through roof hatches. The motor control center electrical panels are above grade in a weather-tight enclosure on a fenced pad.

The pump station does not have a permanent generator or an automatic transfer switch at the site. Standby power is via a manual portable-generator connection. The pumps have the potential to clog easily because there is no trash rack present, only suction strainers on the pump intakes. O&M typically clears debris before and after storms but cannot clean out debris during storm events due to lack of safe access. If the intake clogs, O&M use the submersible pump to dewater the wet well after a storm event. High/low floats are present but only function as alarm switches. The overall condition of the pump station equipment is serviceable but beyond its 40 year life expectancy, with reliability limited by clogging risk and absence of a standby generator.

This station experienced a mechanical failure in December 2021, which caused significant flooding. Stormwater rose into the dry well, which required the eventual replacement of the two damaged electric pump motors. This is the most recent major rehabilitation noted at the station.

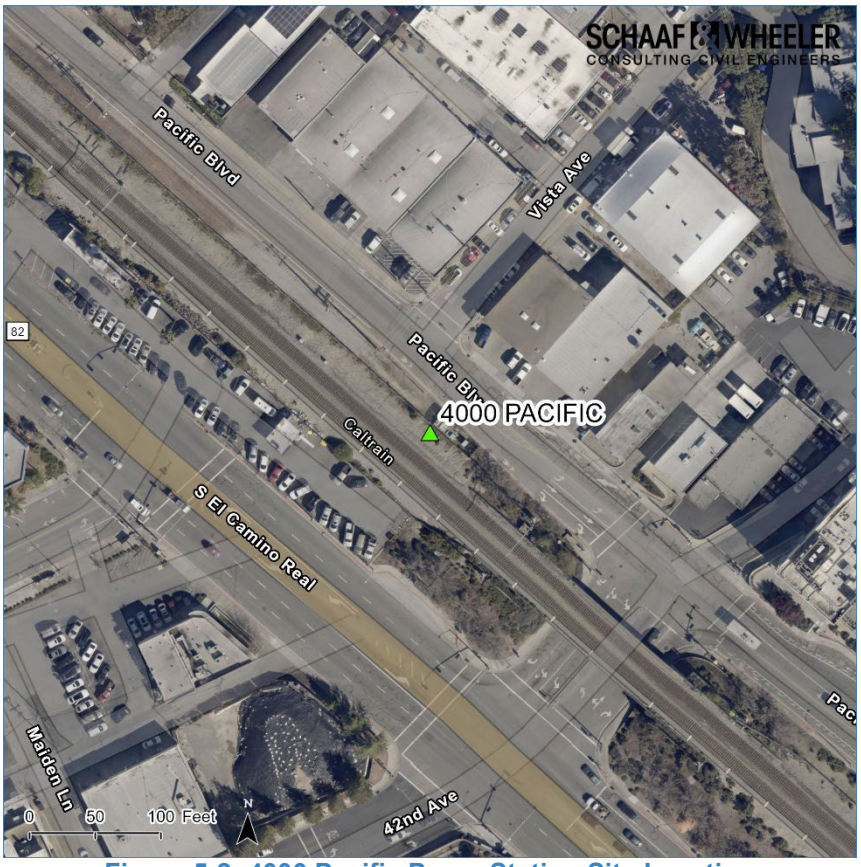


Figure 5-2: 4000 Pacific Pump Station Site Location



Figure 5-3: 4000 Pacific Pump Station Site



Figure 5-4: 4000 Pacific Pump Station Dry Well Through Access Hatches

4000 Pacific Pump Station Hydraulics

The existing pump performance and system curves for the 4000 Pacific Pump Station are shown in Figure 5-5. When both pumps operate, the system curve intersects near the edge of the pump performance curve, where the pumps run less efficiently, whereas the one-pump system curve intersects near the middle of the pump curve, where pump efficiency is higher.

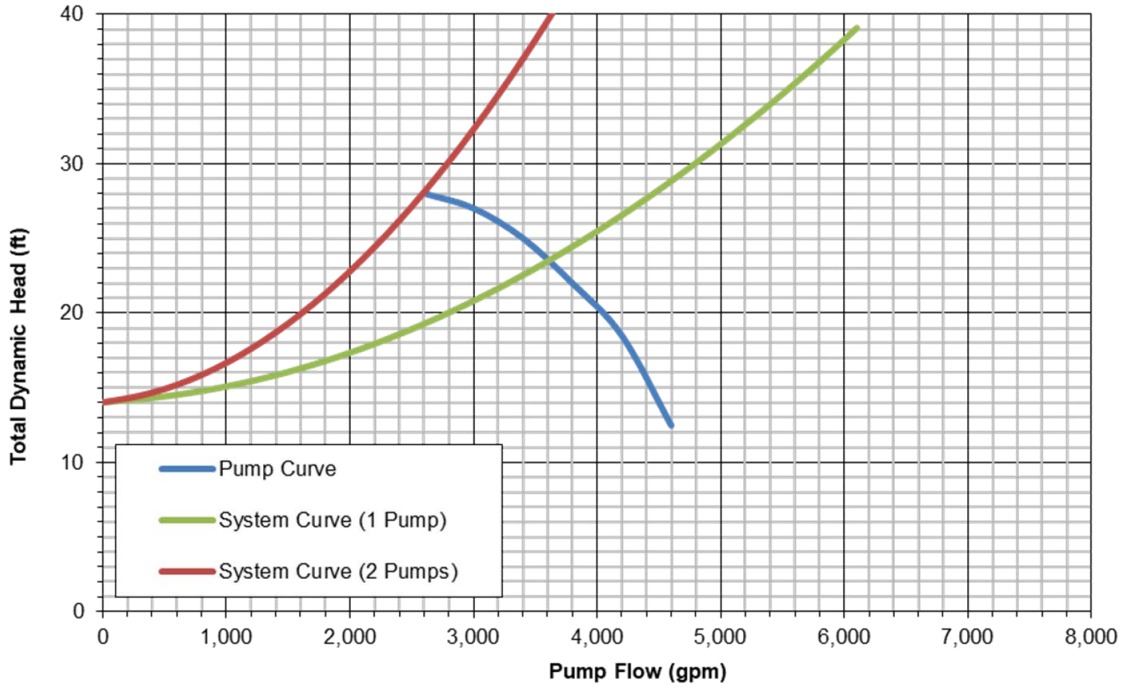


Figure 5-5: 4000 Pacific Existing Pump Station Hydraulics

The point where the blue pump curve intersects the one-pump system curve is the expected operating point when only one pump is running (approximately 3,600 gpm). The intersection with the two-pump system curve represents the operating point when both pumps run (approximately 5,400 gpm), which represents total station capacity. These flows define the usable hydraulic capacities summarized in Table 5-6 and are used to compare the station's performance against the required design flows from the hydrologic model.

Table 5-6: 4000 Pacific Pump Station Hydraulic Capacities

Hydraulic Condition	Station Capacity (gpm)
2 Pumps	5,400
1 Pump	3,600

4000 Pacific Pump Station Recommendations

Recommendations for the 4000 Pacific Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff (Table 5-7). The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-7: 4000 Pacific Pump Station Evaluation

Metrics	Evaluation
Capacity	Improvements recommended to address the design storm
Station Age	Built 1988 (37 years old)
Life Remaining before Rehabilitation	3 years
Physical Condition	Corrosion on electrical panels, Condition of ventilation system unknown
O&M Burden	Not able to clear clogged pumps during storm, History of parts being hard to source, Very short response time required, SCADA does not report wet well water level
Life Safety	Pumps are below grade and are not submersible rated, Access to mechanical equipment requires confined space entry, No direct access to wet well from ground surface, Ventilation system alarm not present
Emergency Capability	No on-site generator, No ATS, No room for trailer generator within PS site, No backup pump controls
Operational	Lacks permanent trash rack and adequate cleaning access, Pumps have intake strainer screens that can clog during operation

Recommended short-term improvements include:

- Upgrade Supervisory Control and Data Acquisition (SCADA) to report wet well water levels
- Install backup pump controls with mechanical floats
- Install a concrete pad adjacent to the station for an on-site generator, with fencing, for security and necessary electrical wiring as a semi-permanent installation

Depending on the timing of a full station rehabilitation (see long-term recommendations below), some of the above replacement items may be implemented as interim repairs rather than full component renewals. To provide greater resilience, upsizing the 4000 Pacific Pump Station for the design storm to produce a total station capacity of 7,200 gpm is recommended. Therefore, for long-term improvements, we recommend constructing a new pump station with the following components:

- New structure with a fixed trash rack and above-grade access for maintenance
- Code-compliant electrical system with ATS and backup generator
- Two new rail mounted submersible rated pumps both with an approximate single-pump operating point of 3,600 gpm at 40 TDH and 70 hp electric motors, such as Flygt pump NP 3306/605 3~ 870 with impeller diameter 400 mm
- New 24-inch discharge piping and force main with new outfall
- Above-grade electrical and control panels, located above flood level

5.3.2 Hillsdale Pump Station Assessment

Hillsdale Pump Station Existing Summary

Constructed in 1964 at 3397 Pacific Blvd, the Hillsdale Pump Station pumps stormwater runoff from the Hillsdale Blvd railroad undercrossing and discharges to the storm drain pipe network along Pacific Blvd.

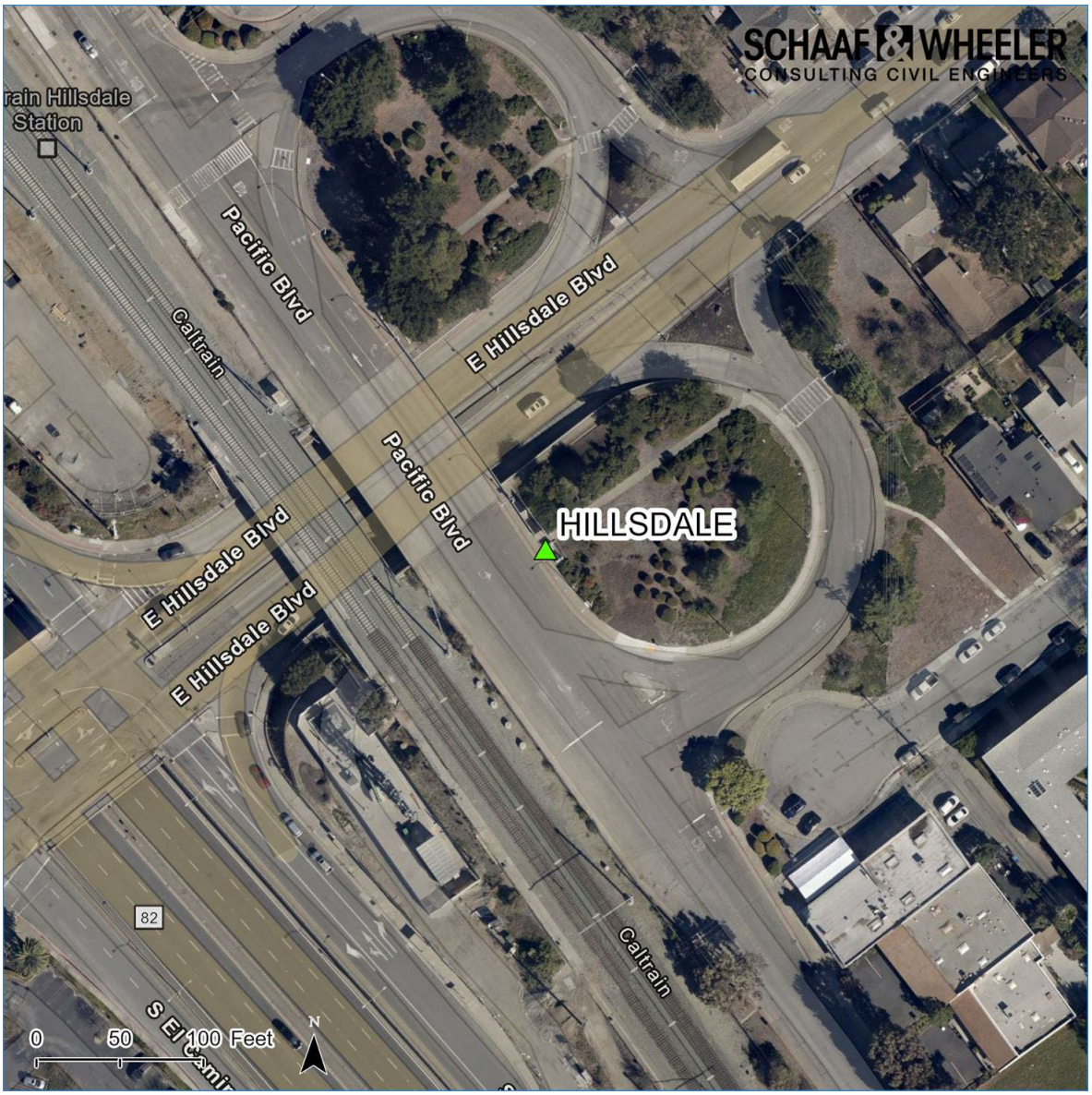


Figure 5-6: Hillsdale Pump Station Site Location

The tributary area of the pump station is 8 acres. As-built drawings indicate that the pump station has two Johnston 16-inch axial-flow stormwater pumps (2,600 GPM at 30 ft TDH, each pump) driven by 25-hp vertical electric motors. During the site visit, the pump nameplates were inaccessible, but the motors were observed to be 30-hp 1,200-RPM, suggesting the current pumps differ from the as-builts, though records are not available. Pumps and motors are located below grade in a confined underground structure with ladder access from a sidewalk manhole

and equipment access through roof hatches and steel plates. An electric service panel and controls are located above grade in a weather-tight enclosure.

The pump station does not have a permanent generator for backup power. Standby power is provided via a manual transfer switch for a portable generator. With motors below grade and the undercrossing prone to flooding, the station is operable but vulnerable during power outages and large storm events, and routine O&M requires confined-space entry. The pump station intake does not have a trash rack; however, the wet well has a weir wall that assists in collecting debris and there are suction strainers on each pump. O&M installed a temporary trash rack at the weir wall and report that trash rarely passes beyond the weir wall. Overall condition of the pump station is functional but obsolete and aging, with the primary known pump station upgrades being relocation of controls above grade and installation of the manual transfer switch following the 2004 SDMP.



Figure 5-7: Hillsdale Pump Station Site (Left) and Electrical Panel (Right)

Hillsdale Pump Station Hydraulics

The existing pump performance and system curves for the Hillsdale Pump Station are shown in Figure 5-8. When one pump operates, the system curve intersects near the edge of the pump performance curve, where the pump run less efficiently. When both pumps run, the system curve intersects near the middle of the pump curve, where pump efficiency is higher

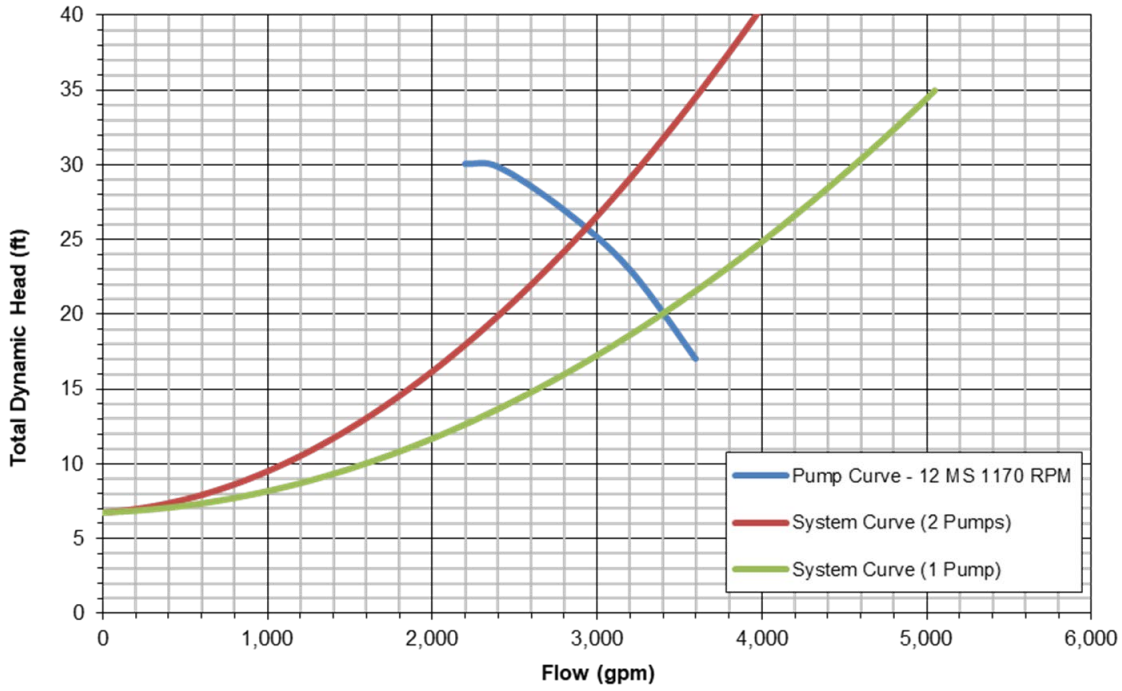


Figure 5-8: Hillsdale Existing Pump Station Hydraulics

The point where the blue pump curve intersects the one-pump system curve is the expected operating point when only one pump is running (approximately 3,400 gpm). The intersection with the two-pump system curve represents the operating point when both pumps run (approximately 5,400 gpm), which represents total station capacity. As-built drawings identify a Johnston Pump 10MS 1760 RPM, but during the site visit, the motor plate indicated 1200 RPM, suggesting that the original pump was replaced. Pump nameplates are not accessible, so a Johnston Pump 12 MS 1170 RPM, which has similar operating points to the original pump, is used as a conceptual substitute for this analysis.

Table 1-3: Hillsdale Pump Station Hydraulic Capacity

Hydraulic Condition	Station Capacity (gpm)
2 Pumps	5,800
1 Pump	3,400

Hillsdale Pump Station Evaluations & Recommendations

Recommendations for the Hillsdale Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-8: Hillsdale Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 10-year storm
Station Age	Built 1964 (61 years old)
Life Remaining before Rehabilitation	< 5 years
Physical Condition	Corrosion on electrical panels, Condition of ventilation system unknown
O&M Burden	No permanent trash rack, just weir and pump suction strainers, History of parts being hard to source, Very short response time required, SCADA does not report wet well water level
Life Safety	Pumps are below grade and are not submersible rated, Access to mechanical equipment requires confined space entry, No direct access to wet well from ground surface, Ventilation system alarm not present, Electrical switchboard panel is old and not code compliant, Ventilation of wet well is exhaust only
Emergency Capability	No on-site generator, No ATS, No room for trailer generator within PS site, No backup pump controls
Operational	Lacks permanent trash rack and adequate cleaning access, Pumps have intake strainer screen that can blind during operation

Recommended short-term improvements include:

- Upgrade SCADA to report wet well water levels
- Install backup pump controls with mechanical floats
- Install a concrete pad adjacent to station for an on-site generator and assess site security needs

The pump station has surpassed its 40-year life expectancy. The following long-term improvements are recommended for this station.

- Conduct a detailed condition assessment of all major structural components, including but not limited to:
  - Concrete structures (above and below grade)
  - Discharge pipe
- Replace existing pumps with two new rail mounted submersible pumps with a single pump operating point of 2,900 gpm at 26 TDH and 30 hp electric motors, such as NP 3171 LT 3~ 613 with impeller diameter 292 mm
- Replace all major electrical equipment including conduits, wires, switchboard panel, motor control center, etc.
- Install an automatic transfer switch (ATS) and backup generator
- Install accessible trash rack or trash capture devices that can be maintained during a storm event

5.3.3 Casanova Pump Station Assessment

Casanova Pump Station Existing Summary

Constructed in 1968 and replaced in 2004 (last major recorded improvement), the Casanova Pump Station serves a residential neighborhood, pumping runoff from the storm drain pipe network into Laurel Creek, which ultimately drains to Marina Lagoon. The tributary area of the pump station is 169 acres. Pumping equipment consists of four 20-inch Flygt submersible stormwater pumps (11,700 GPM at 15 ft TDH, each pump) in a below-grade wet well driven by 505 RPM motors, and one submersible sump pump for wet well drainage and maintenance. A trash rack is provided at the wet well intake, although it is relatively small for the station's rated capacity and requires routine monitoring for cleaning.

Electric service panels and controls are located above grade in a weather-tight enclosure. High/low floats are present but only function as alarm switches. The pump station does not have a permanent generator for backup power or an ATS. Standby power is provided through a manual transfer switch and connection for a portable generator, and there is no dedicated permanent generator located on site. Overall condition is good, and O&M staff report that the station is easy to operate and maintain. However, some corrosion was noted on the steel pump discharge piping and wet well vent. Corrosion can reduce the thickness of pipe walls or cause localized pitting. In severe cases, pressure piping can fail, reducing the capacity of the station or causing total loss of pumping capacity.

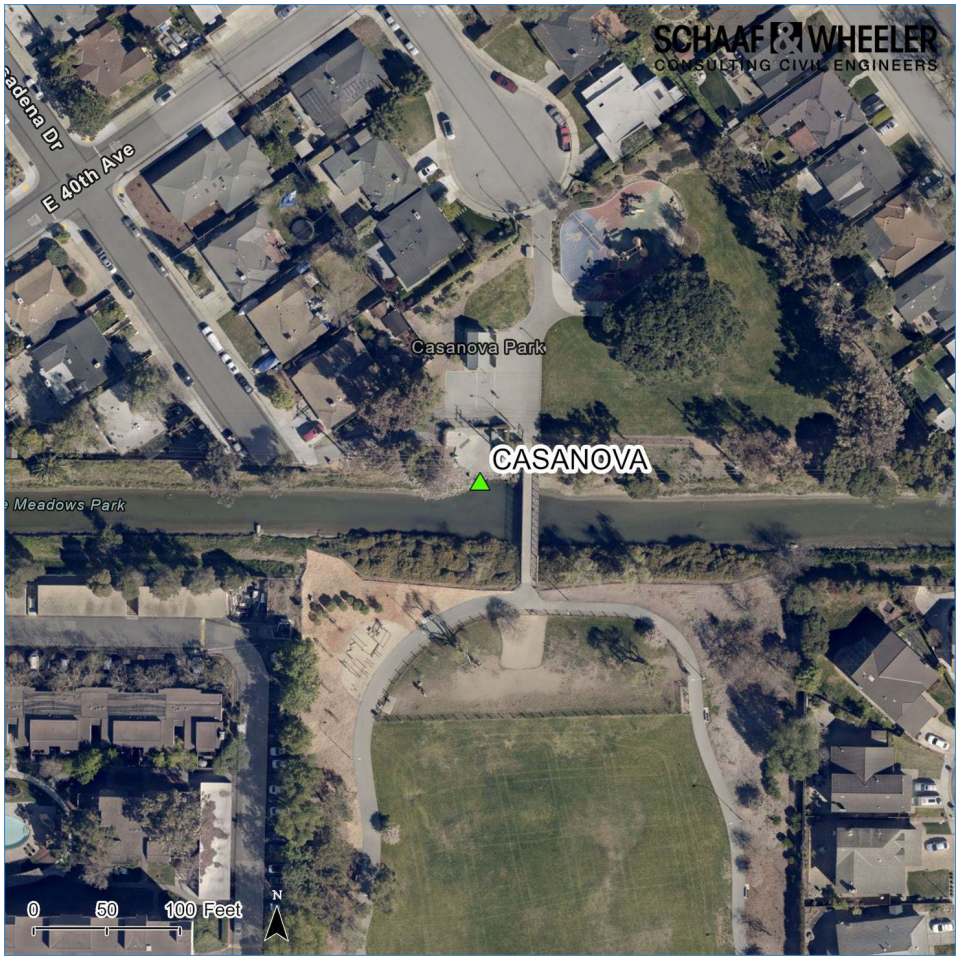


Figure 5-9: Casanova Pump Station Site Location



Figure 5-10: Casanova Pump Station Site As Seen From Casanova Park



Figure 5-11**: Casanova Pump Station Wet Well Viewed From Access Hatches**

Casanova Pump Station Hydraulics

The existing pump performance and system curves for the Casanova Pump Station are shown in Figure 5-12. Because each of the four pumps at this location has a separate discharge pipe, the system curve does not change when multiple pumps operate at the same time. Instead, total station capacity scales with the number of pumps in service.

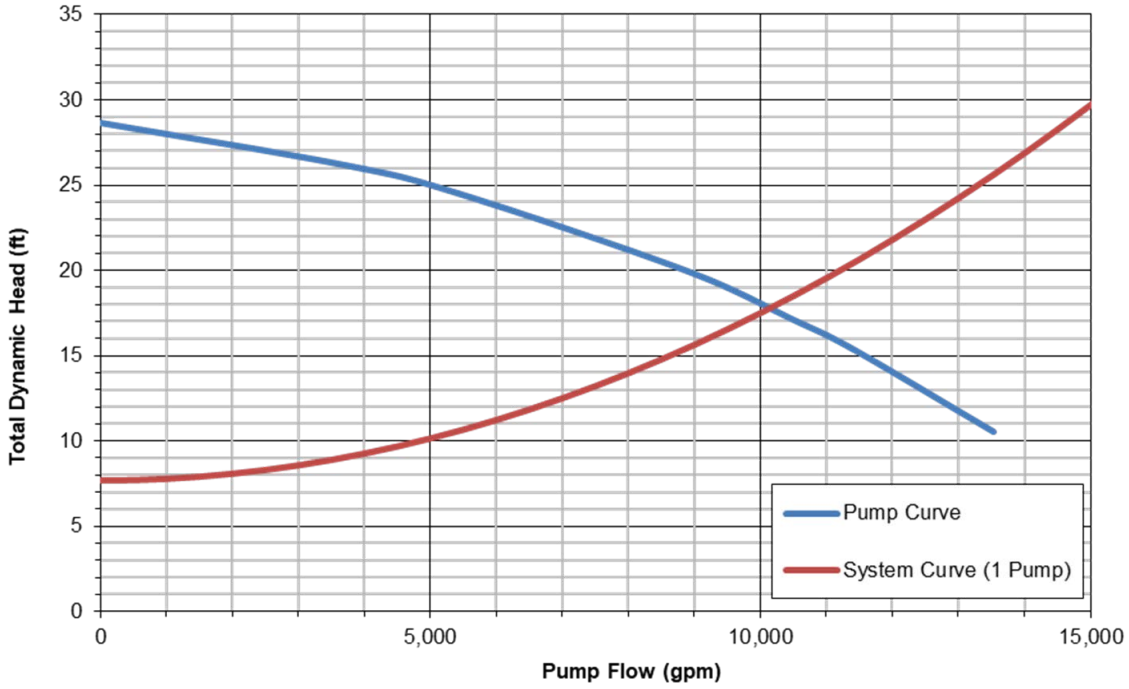


Figure 5-12: Casanova Existing Pump Station Hydraulics

The intersection of the pump curve and the system curve defines the operating point for a single pump (approximately 10,000 gpm), which occurs near the middle of the pump curve where efficiency is relatively high. With four pumps running, the total station capacity is, therefore, about 40,000 gpm, as summarized in Table 5-9.

Table 5-9: Casanova Pump Station Hydraulic Capacity

Hydraulic Condition	Station Capacity (gpm)
4 Pumps	40,000

Casanova Pump Station Evaluations & Recommendations

Recommendations for the Casanova Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-10: Casanova Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 10-year storm
Station Age	Built 2004 (22 years old)
Life Remaining before Rehabilitation	18 years
Physical Condition	Corrosion of pump discharge pipes, wet well vent pipe corrosion
O&M Burden	Trash rack may require more frequent cleaning due to size
Life Safety	Conduits between the wet well and electrical panel do not have duct seal
Emergency Capability	No on-site generator, no ATS, wet well water levels not transmitted to SCADA, no backup pump controls
Operational	Trash rack is small

Recommended short-term improvements include:

- Upgrade SCADA to report wet well water levels
- Install backup pump controls with mechanical floats
- Repair corroded pipes and fittings
- Repair wet well vent pipe

The pump station is over 20 years old, so the O&M burden will continue to increase as equipment shows more wear and replacement parts become harder to source. The City should consider a full rehabilitation within the next 20 years.

Recommended long-term rehabilitations include:

- Conduct a detailed condition assessment of all major structural, electrical, mechanical, and control systems, including but not limited to:
  - Concrete structures (above and below grade)
  - Pumps
  - Discharge pipe and valves
  - Pump control panels and components

5.3.4 Poplar Pump Station Assessment

Poplar Pump Station Existing Summary

Constructed in 1973 and reconstructed in 2020 (last major rehabilitation), the Poplar Ave Pump Station is a terminal facility situated against the Bayfront Levee. The station's intake is connected to a series of feeder channels that drain the golf course, areas to the west, and areas to the south through Shoreline Park, and the station discharges directly to San Francisco Bay. The tributary area of the pump station is 960 acres. Pumping equipment consists of four 48-inch Flygt axial flow submersible stormwater pumps rated at 40,400 GPM at 14 ft TDH driven by 505 RPM electric motors and a submersible Flygt sump pump.



Figure 5-13: Poplar Ave Pump Station Site Location

The station is equipped with an on-site 1,000-kW Cummins diesel generator and an ATS that provides standby power. The overall condition of the pump station is good; however, several reliability issues have been identified. The main breaker occasionally trips without generating an alarm, accompanied by popping sounds from the master control center (MCC) panel, and operators must manually reset the system by cycling the main breaker, control power, and uninterruptible power supply (UPS). When the MCC experiences problems, the ATS and generator do not always clearly indicate loss of power or start automatically. Pump cable disconnect panels are experiencing water intrusion and corrosion (note corrosion in the lower right pump cable panel in Figure 5-15), and Tideflex discharge check valves are sagging and not operating properly. Unreliable PG&E service (also noted in the 2004 SDMP), remains an important consideration when evaluating station reliability and the performance of the standby power system.



Figure 5-14: Poplar Ave Pump Station Site View Facing San Francisco Bay

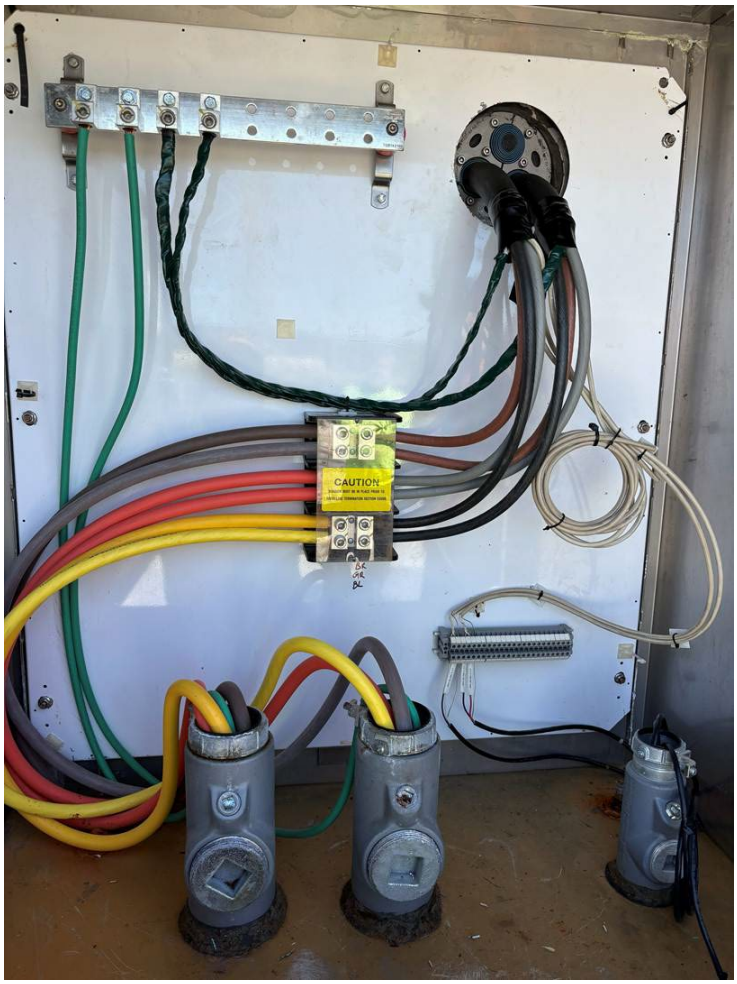


Figure 5-15: Poplar Ave Pump Station Pump Cable Panel

Poplar Ave Pump Station Hydraulics

The existing pump performance curve and system curves for the Poplar Ave Pump Station are shown in Figure 5-16. Because each pump has its separate discharge pipe, the system curve for one pump does not change when additional pumps operate. The intersection of the pump curve with each system curve defines the operating point for a single pump under that tailwater condition. Table 5-11 summarizes the capacities of Poplar Ave with all four pumps running. These operating points fall within the preferred region of the pump curve for all tailwater conditions.

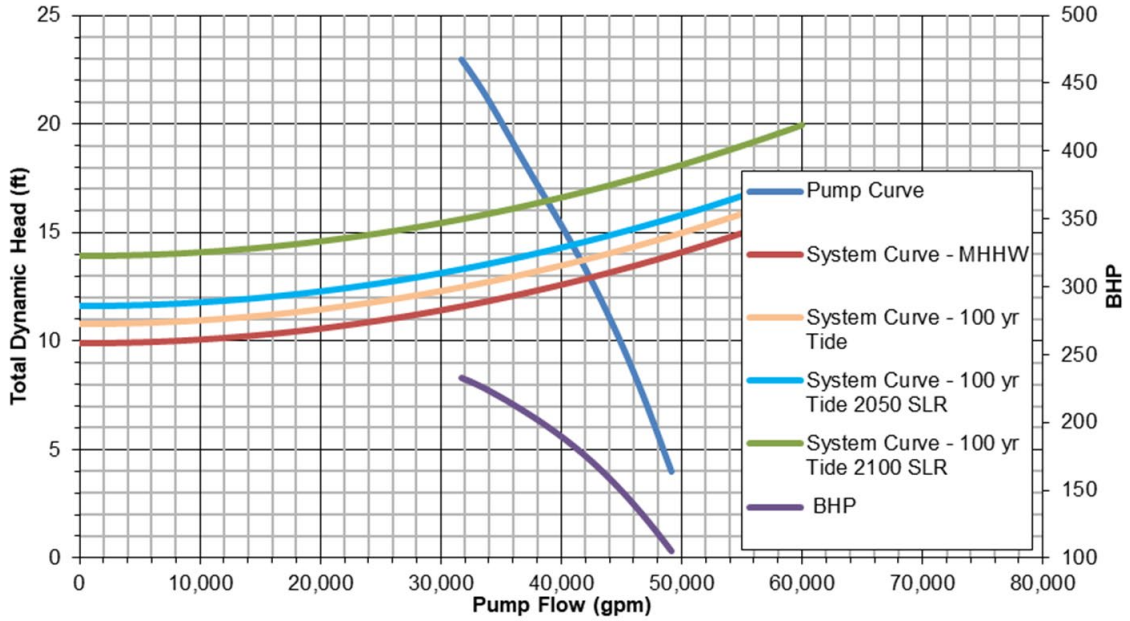


Figure 5-16: Poplar Existing Pump Station Hydraulics

Each pump is equipped with a 230-hp motor, and the station has an on-site generator rated at 1,000 kW (approximately 1,341 BHP). Figure 5-16 shows the BHP demand for a single pump over the range of flows. The resulting pump and pump motor BHP demands for current and future tailwater conditions are listed in Table 5-11. For all modeled conditions, the pump BHP demand remains below the 230-hp motor rating, indicating that both pump motors are adequately sized to operate the pumps under the modeled conditions.

Table 5-11: Poplar Pump Station Hydraulic Capacities

Hydraulic Condition	Station Capacity (gpm)	Pump BHP Demand	Pump Motor BHP
4 Pumps: MHHW	168,000	170	230
4 Pumps: 100 yr Tide	166,000	180	230
4 Pumps: 100-yr Tide 2050 SLR	164,000	190	230
4 Pumps: 100-yr Tide 2100 SLR	154,000	200	230

Poplar Ave Pump Station Evaluations & Recommendations

Recommendations for the Poplar Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-12: Poplar Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 100-year storm
Station Age	Built 2020 (6 years old)
Life Remaining before Rehabilitation	34 years
Physical Condition	Pump discharge Tideflex valves do not seal, corrosion and water intrusion on pump cable panels, corrosion on air vent piping
O&M Burden	No major deficiencies
Life Safety	No major deficiencies
Emergency Capability	Wet well water levels not transmitted to SCADA
Operational	MCC breaker often trips without generating a power fail alarm, electrical panel surge protection device (manufactured by ASCO) has failed

Recommended short-term improvements include:

- Upgrade SCADA to report wet well water levels
- Replace Tideflex pump discharge check valves
- Address water intrusion at pump cable panels to prevent corrosion
- Investigate electrical breakers tripping and potential power surges from the electrical utility

The pump station is newly built, and no long-term rehabilitations are recommended for this station. The operational deficiencies for Poplar Ave suggest that there is a power source issue at this station. Coyote Point is a very similar pump station and does not experience the same electrical issues. It is recommended that the City investigate the quality of the station's power source at this location and conduct a power systems inspection to determine if any on-site equipment is faulty.

5.3.5 Coyote Point Pump Station Assessment

Coyote Point Pump Station Existing Summary

Constructed in 1973 and reconstructed in 2020 (last major rehabilitation), the Coyote Point Pump Station is a terminal facility located at 20 Airport Blvd. It is situated against the Bayfront Levee at the western edge of Coyote Point Recreation Area near the Burlingame border, and discharges directly to San Francisco Bay.



Figure 5-17: Coyote Point Pump Station Site Location

The tributary area of the Coyote Point Pump Station is 490 acres. The facility is similar to the Poplar Pump Station, with three 48-inch Flygt axial-flow pumps rated at 51,000 gpm at 10 ft TDH (each pump), driven by 505-RPM motors, and one Flygt submersible sump pump.

The station is equipped with a permanent 750-kW Cummins diesel generator and an ATS, and it does not experience the power-supply or control-power reliability issues seen at Poplar. A trash capture device and trash rack are provided at the intake. However, the trash capture device clogs relatively quickly and must be cleaned with a vactor truck about twice per year. The trash capture weir is often relied upon during storms because the trash capture device clogs and is not cleaned until after the storm event. Pump discharge check valves (Tideflex) are sagging (as seen in Figure 5-19) and often remain open after pump shutdown, allowing backflow through the discharge piping. Sand accumulates in the discharge box and around the valves and can flow back toward the pumps if the valves remain stuck open. Overall, the pump station is in good condition with a layout that facilitates routine maintenance.



Figure 5-18: Coyote Point Pump Station Site Facing Bayfront Levee



Figure 5-19: Coyote Point Pump Station Tideflex Valves

Coyote Point Pump Station Hydraulics

The existing pump performance curve and system curves for the Coyote Point Pump Station are shown in Figure 5-20. Because each pump has a separate discharge pipe, the single-pump system curve does not change when multiple pumps run. Discharge outfall is at a higher elevation than MHHW and the 100-year tide, so the system curve for MHHW and 100-year tide tailwater condition is the same. The intersection of the pump curve with each system curve defines the operating point for a single pump under that tailwater condition. With all three pumps operating, Coyote Point has capacities of approximately 156,000 gpm at the 100-year tide, 153,000 gpm at the 2050 SLR, and 147,000 gpm at the 2100 SLR, as summarized in Table 5-13. These operating points remain within the preferred region of the pump curve for all tailwater conditions and do not exceed the motor horsepower ratings.

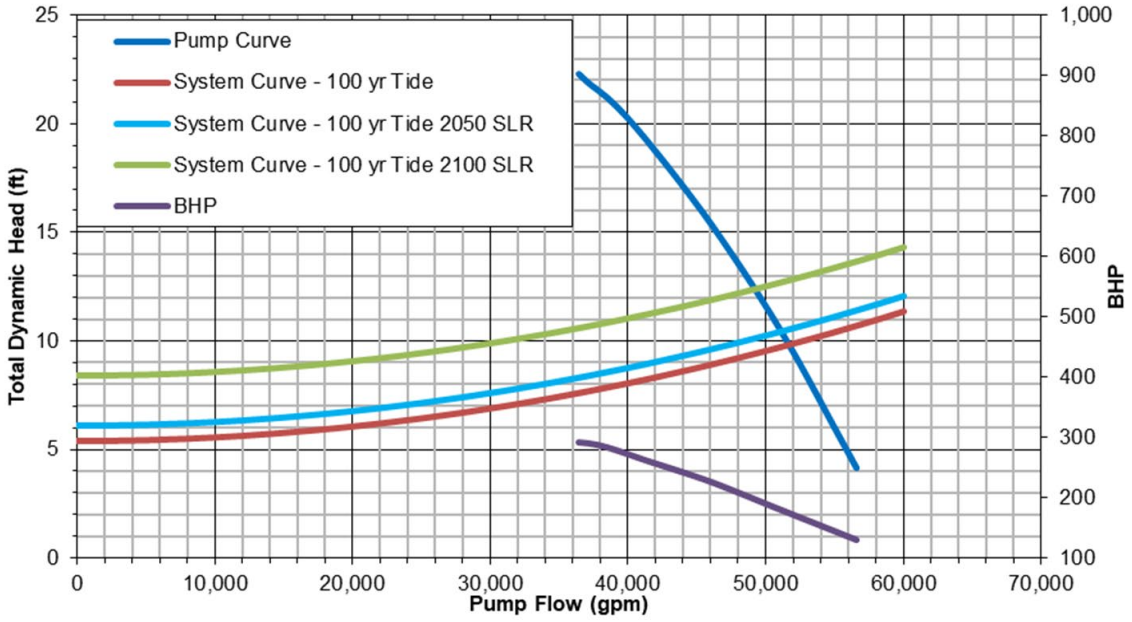


Figure 5-20: Coyote Point Existing Pump Station Hydraulics

Each pump is equipped with a 230-hp motor, and the station has an on-site generator rated at 750 kW (about 1,005 BHP). The curve on Figure 5-20 indicates the BHP demand for a single pump. The resulting pump and pump motor BHP demands for current and future tailwater conditions are listed in Table 5-13. For all modeled conditions, the pump BHP demand remains below the 230-hp motor rating, indicating that both the pump motors are adequately sized to operate the pumps under the modeled conditions.

Table 5-13: Coyote Point Pump Station Hydraulic Capacities

Hydraulic Condition	Station Capacity (gpm)	Pump BHP Demand	Pump Motor BHP
3 Pumps: 100 yr Tide	156,000	180	230
3 Pumps: 100 yr Tide 2050 SLR	153,000	200	230
3 Pumps: 100 yr Tide 2100 SLR	147,000	220	230

Coyote Point Pump Station Evaluations & Recommendations

Recommendations for the Coyote Point Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-14: Coyote Point Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 100-year storm
Station Age	Built 2020 (6 years old)
Life Remaining before Full Replacement	34 years
Physical Condition	Pump discharge Tideflex valves do not seal, corrosion and water intrusion on pump cable panels, corrosion on air vent piping
O&M Burden	Trash capture device clogs quickly, sediment in discharge pipes due to discharge pipes not sealing when pumps are off
Life Safety	No major deficiencies
Emergency Capability	Wet well water levels not transmitted to SCADA
Operational	No major deficiencies

Recommended short-term improvements include:

- Upgrade debris rack at trash capture device to ease cleaning
- Upgrade SCADA to report wet well water levels
- Replace Tideflex pump discharge check valves
- Address water intrusion at pump cable panels to prevent corrosion

The pump station is newly built, and no long-term rehabilitations are recommended for this station.

5.3.6 Fathom Dr Pump Station Assessment

Fathom Dr Pump Station Existing Summary

Constructed in 1966 at 401 Fathom Dr, the Fathom Dr Pump Station is a level-control station serving a small wetland area adjacent to J Hart Clinton Dr near the Foster City border. See Figure 5-21 for site location.



Figure 5-21: Fathom Dr Pump Station Site Location

The station was upgraded with new pumps and electrical panels in 2016. It does not receive urban storm drain inflow and is not a terminal facility. The tributary area is limited to the wetland's 12 acres. Stormwater is pumped from a channel and discharged to Marina Lagoon via storm drain piping in the street. The station has two 6-inch Flygt submersible pumps (800 gpm at 27 ft TDH, each pump) driven by 1,735-RPM motors. The wet well appears to have been

retrofitted from an earlier axial-flow configuration. A single 24-inch flap gate in a concrete structure provides backflow protection at the outfall.

Electrical service and motor controls are above grade in a weather-tight enclosure within the fenced pad, and there is no permanent generator. Backup power configuration includes a receptable for a portable generator and interlocked main breakers. Float switches are used for alarm functions only and do not directly control pump starts. Overall condition is functional with no reported O&M problems, and the station is considered reliable for its limited level-control role. However, the inlet trash rack is severely corroded and requires replacement.



Figure 5-22: Fathom Dr Pump Station Site



Figure 5-23: Fathom Dr Pump Station Trash Rack

Fathom Dr Pump Station Hydraulics

The existing pump performance and system curves for the Fathom Dr Pump Station are shown in Figure 5-24. The blue curve is the manufacturer's performance curve for a single pump, and the red curve is the system curve for one pump, representing the head the pump must overcome (static lift plus friction and minor losses in the discharge piping). Because each pump has a separate discharge pipe, operating multiple pumps at the same time does not affect the single-pump system curve.

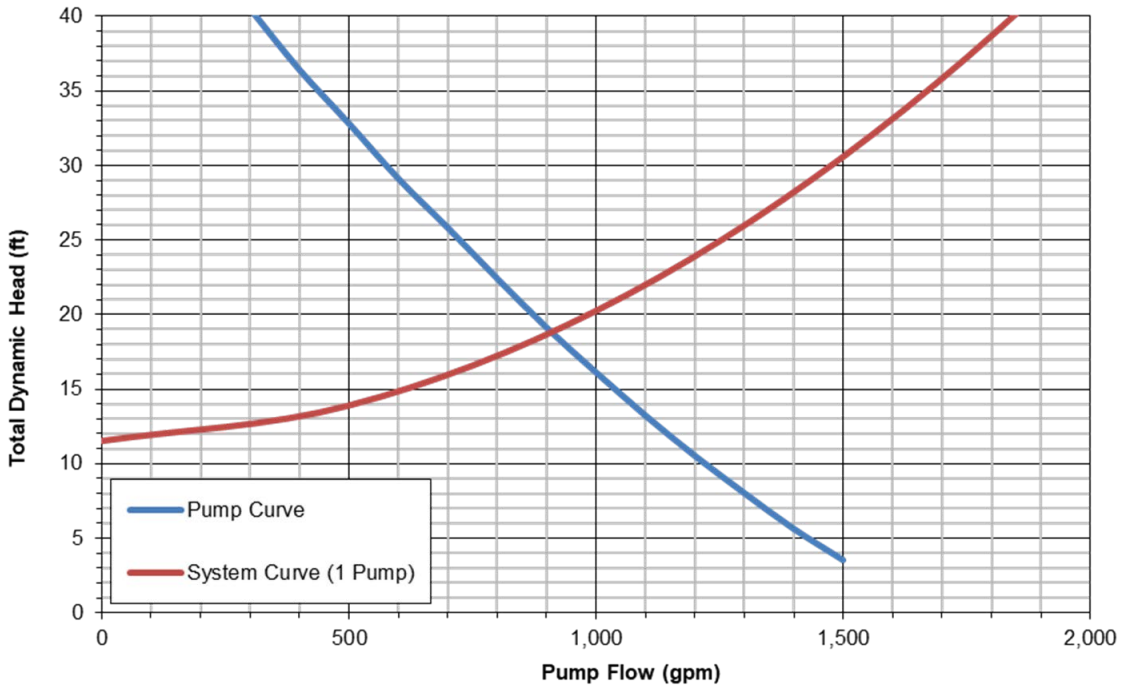


Figure 5-24: Fathom Dr Existing Pump Station Hydraulics

The intersection of the pump curve and the system curve defines the operating point for a single pump (approximately 910 gpm), which lies near the central portion of the pump curve where efficiency is relatively high. With both pumps running, the total station capacity is therefore about 1,820 gpm, as summarized in Table 5-15.

Table 5-15: Fathom Dr Pump Station Hydraulic Capacity

Hydraulic Condition	Station Capacity (gpm)
2 Pumps	1,820

Fathom Dr Pump Station Evaluations & Recommendations

Recommendations for the Fathom Dr Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, and operations and, where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-16: Fathom Dr Pump Station Evaluation

Metrics	Evaluation
Capacity	Not applicable because this is not a flood control facility
Station Age	Built 1966, Equipment Replaced 2016 (10 years old)
Life Remaining before Rehabilitation	Station: <5 years; replaced equipment: 10 years
Physical Condition	Corrosion on exposed rebar at pump station structure, trash rack severely corroded and missing invert, damage on chain link fence, site concrete cracking
O&M Burden	No major deficiencies
Life Safety	No site lighting for night operation, uneven concrete surfaces at pump station top slab (tripping hazard), wet well access hatches are loose metal panels (no fall protection), electrical panel located at wet well opening locations
Emergency Capability	No on-site generator, no ATS, wet well water levels not transmitted to SCADA, no backup pump controls, no room for trailer generator within PS site
Operational	Trash rack is ineffective due to corrosion

Recommended short-term improvements include:

- Upgrade SCADA to report wet well water level
- Install backup pump controls with mechanical floats
- Assess site security for portable back-up generator and install fencing as needed
- Replace trash rack
- Install site lighting
- Replace wet top slab and install hinged access hatches with fall protection.
- Relocate electrical panels to be compliant with building code and adequate separation from wet well

Though this station was originally built in 1966, the equipment was replaced in 2016. Therefore, the equipment is not in need of repair or replacement, but the structure is recommended to be inspected.

Recommended long-term rehabilitations include:

- Conduct a detailed condition assessment of all major structural components, including but not limited to:
  - Concrete structures (above and below grade)
  - Discharge pipes

5.3.7 Marina Lagoon Pump Station Assessment

Marina Lagoon Pump Station Existing Summary

Marina Lagoon Pump Station (2100 Detroit Dr; constructed 1983) is a terminal flood control facility with a total watershed area of 7,210 acres. See Figure 5-25 for site location.



Figure 5-25: Marina Lagoon Pump Station Site Location

Located at the downstream end of the 1,400 acre-ft Marina Lagoon storage basin, it pumps from the lagoon and discharges to San Francisco Bay. The indoor plant contains five Patterson 72×72 axial flow pumps (150,000 GPM at 10 ft TDH, each pump) driven through right angle gear drives by five 545 hp diesel engines, with a 2,000 gal underground fuel tank and 400 gal day tank. A smaller 48×48 electric axial flow pump (75,000 GPM) is provided for lagoon refill (reverse flow) and does not add flood capacity.

Pump drivers are diesel engines and independent of utility power. A 40 kW standby diesel generator supplies lighting/controls and auxiliary power at the station with an ATS, meeting FEMA automatic operation requirements. Pump starts are alternated and engines are regularly exercised. Overall condition is fair, but the facility remains aging and maintenance intensive. Several major components, including pumps, gear drives, trash racks, and the fuel system, have shown wear and corrosion and have required repair or replacement.

The City has completed several recent reliability and monitoring improvements. Rehabilitation of the cathodic protection system was completed in March 2025 and is currently active; the rectifier was replaced and missing anodes were installed. Floats have been installed for high alarms in the wet well and on the lagoon side. SCADA programming was completed in January 2025 to report wet well and lagoon water levels. In addition, three gearboxes are pending replacement; however, these gearboxes were inspected in 2023-2024 and found to be in good working condition, though replacement is still recommended as part of ongoing system upgrades. The City continues to inspect and rehabilitate equipment on an ongoing basis to address reliability concerns.



Figure 5-26: Marina Lagoon Pump Station Site



Figure 5-27: Marina Lagoon Pump Station Equipment Overview



Figure 5-28: Marina Lagoon Pump Station Pump Motor

Marina Lagoon Pump Station Hydraulics

The existing pump performance curve and system curves for the Marina Lagoon Pump Station are shown in Figure 5-29. The blue curve is the pump performance curve for a single unit, and the system curves represent one-pump operation under MHHW, 100-year tide, 100-year tide with 2050 SLR, and 100-year tide with 2100 SLR. Because each pump has a separate discharge pipe, the single-pump system curve does not change when additional pumps operate; total station capacity scales with the number of pumps in service. The system curve is based on the normal lagoon operating level, not the dock maintenance water level which is much lower. The intersection of the pump curve with each system curve defines the operating point for a single pump under that tailwater condition.

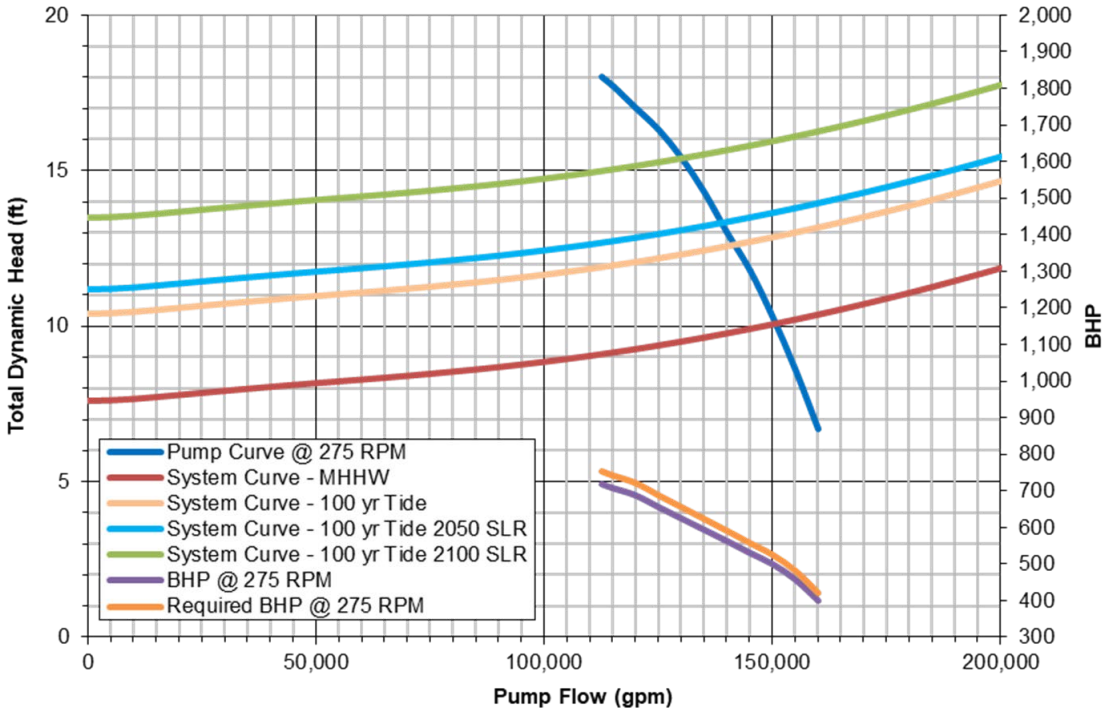


Figure 5-29: Marina Lagoon System Curve

The purple line on Figure 5-29 shows the BHP demand at the pump, and the orange line shows the BHP required at the diesel engine, assuming a right-angle gear box efficiency of 95 percent. The BHP curve indicates the brake horsepower demand for a single pump demand for each tailwater condition as listed in Table 5-17. Under the 100-year tide and both SLR scenarios, the required engine horsepower exceeds the available engine rating, indicating that pump output is limited by engine power at these higher tailwater conditions.

Table 5-17: Marina Lagoon Pump Station Hydraulic Capacity

Hydraulic Condition	Station Capacity (gpm)	Pump BHP Demand at Engine	Engine BHP Rating
5 Pumps: MHHW	755,000	520	545
5 Pumps:100 yr Tide	710,000	600	545
5 Pumps: 100 yr Tide 2050 SLR	695,000	620	545
5 Pumps: 100 yr Tide 2100 SLR	650,000	650	545

Marina Lagoon Pump Station Evaluations & Recommendations

Recommendations for the Marina Lagoon Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, and operations and, where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-18: Marina Lagoon Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 100-year storm
Station Age	Built 1983 (43 years old)
Life Remaining before Rehabilitation	<5 years
Physical Condition	Cathodic protection system not fully functional (pumps are not protected), diesel engine pump drivers have various wear and operational issues, fuel system has sludge, pumps have wear and corrosion problems and have been rebuilt, trash racks have corrosion, some gear reduction boxes have been replaced due to failure
O&M Burden	Replacement parts are difficult to obtain, multiple equipment systems require significant maintenance and there are common equipment failures, trash racks require frequent cleaning during storm events to maintain pump operation, scaling deposits on trash rack
Life Safety	No major deficiencies
Emergency Capability	Multiple mechanical systems do not have redundancy, wet well water levels not transmitted to SCADA, no backup pump controls
Operational	Pumps cannot safely operate at low lagoon levels, trash rack experience head loss during high pump flow rates or low lagoon levels, pump system power and equipment strength ratings exceeded at high tide discharge conditions or low lagoon levels

The recommended full rehabilitation of this pump station is extensive, particularly given the facility's critical role and the magnitude of its impact within the City's storm drain system. Because a complete rehabilitation will be both time- and budget-intensive, certain repairs and upgrades should be implemented in the interim to maintain reliability and reduce operational risk until the long-term improvements can be completed.

Recommended short-term improvements identified during the assessment include:

- Conduct a detailed condition assessment of all major structural, electrical, mechanical, and control systems, including but not limited to:
  - Concrete structures (above and below grade)
  - Cathodic protection system
  - Fuel storage and delivery system (tanks, piping, pumps, conditioners, etc.)
  - Diesel engine pump drivers (engine components, controls, PTO, etc.)
  - Pump gear-reduction boxes
  - Pumps and pump lubricators

    - Discharge pipe flap gates
    - Backup generator system
    - Pump control panels and components
- Complete a physical hydraulic model study (or equivalent detailed hydraulic analysis) to confirm pump operating conditions and evaluate potential hydraulic improvements, including operations at low lagoon levels and water levels used for dock maintenance period **[Completed July 2026]**
- Reconfigure the trash rack based on the findings of the hydraulic study, and consider installing an in-lagoon floating debris boom upstream of the rack to reduce clogging and head loss
- Rehabilitate the cathodic protection system **[Completed March 2025]**
- Replace or rehabilitate remaining original gear boxes with updated units and complete necessary maintenance/replacement of gear-reduction boxes already in poor condition
- Install backup pump level controls with mechanical floats **[Floats have been installed for high alarms in the wetwell and the Lagoon, as of early 2026]**
- Upgrade SCADA to report wet well water level **[Completed January 2025]**
- Complete structural/hydraulic improvements to the existing wetwell based on the results of the hydraulic study

Depending on the timing of full station rehabilitation (see long-term recommendations below), some of the above replacement items may be implemented as interim repairs rather than full component renewals. However, as discussed, the pump diver engines are not adequately sized to pump against high tide, low lagoon, or SLR conditions. Pump drivers will need to be replaced before the pump station starts experiencing the effects of SLR which may necessitate a full station rehabilitation before short-term improvements can be implemented.

The pump station has surpassed its 40-year life expectancy, and major equipment is beyond its useful service life. Recommended long-term rehabilitations include:

- Construct a new deep pump station located separate from the existing Marina Lagoon Pump Station, with at least the equivalent capacity of two existing pumps and capable of pumping at low lagoon levels as well as high SF Bay levels
- Once the new station is operational, fully reconstruct/rehabilitate the existing Marina Lagoon Pump Station to provide total pumping capacity comparable to the current facility, using modern mechanical, electrical, and control systems and incorporating the hydraulic and debris-handling improvements identified in the physical model study.

A detailed preliminary engineering (basis-of-design) study should define the required capacity of the new station, evaluate how to phase construction and rehabilitation to minimize the need for temporary bypass pumping, and determine which short-term replacement items can be deferred or incorporated into the full rehabilitation if it proceeds within the next 5–10 years.

5.3.8 3rd & Detroit Pump Station Assessment

3rd & Detroit Pump Station Existing Summary

3rd & Detroit Pump Station is a terminal facility at the mouth of Seal Slough on the Bay side of the Marina Lagoon plant. It drains 245 acres of mixed residential/commercial area bounded by San Mateo Creek, J. Hart Clinton Dr, 16th Ave Drain, and Hwy 101. See Figure 5-30 for site location.



Figure 5-30: 3rd & Detroit Pump Station Site Location

Flow from the storm drain pipe network enters the pump-forebay and is pumped to San Francisco Bay. The station houses two vertical axial-flow pumps (Peerless 36MF, 20,000 GPM at 19 ft TDH, 125 hp each): one is electric-motor driven pump (GE 125 hp VHS), and one is diesel-engine driven pump (Scania 125 hp via 3:1 right-angle gear). Equipment is located in an enclosed building with indoor access and the open intake structure has a

mechanical trash rack (bar screen) system. A spare bay is available for a future third pump of similar capacity.

A permanent 12-kW standby engine-generator serves only the 120-V lighting and control circuits. The generator does not have sufficient capacity to power the electric pump motor, so pumping during a utility outage is provided by the diesel-driven pump only. The mechanized bar screen is inoperable. The replacement parts required to repair the bar screen are no longer available, so O&M staff use the trash rack as a fixed trash rack and manually operate the rake. Overall condition of the pump station is serviceable but aging. There is no record of the latest major rehabilitation.



Figure 5-31: 3rd & Detroit Pump Station Site



Figure 5-32: 3rd & Detroit Pump Station Pump Motors

3rd & Detroit Pump Station Hydraulics

The existing pump performance curve and system curves for the 3rd & Detroit Pump Station are shown in Figure 5-33. The dark blue line is the pump performance curve for a single unit, and the system curves represent one-pump operation under the 100-year tide, 100-year tide with 2050 SLR, and 100-year tide with 2100 SLR. Because each pump has a separate discharge pipe, the single-pump system curve does not change when both pumps operate. The discharge outfall is at a higher elevation than MHHW and the 100-year tide, so the system curve for MHHW and 100-year tide tailwater condition is the same. The capacity analysis is considered for only the diesel pump. The on-site generator does not have sufficient capacity to power the electric pump, so the electric unit is not considered available during a power outage. The intersection of the pump curve with each system curve defines the operating point for the diesel pump under that tailwater condition.

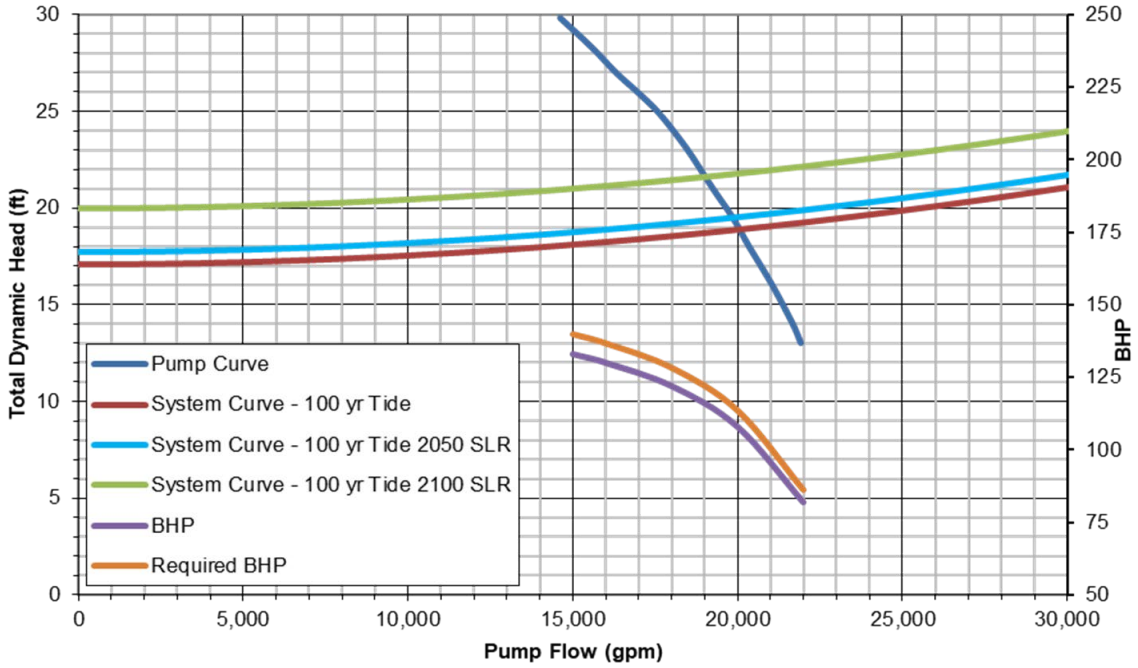


Figure 5-33: 3rd & Detroit System Curve

The purple line on Figure 5-33 shows the BHP demand at the pump, and the orange line shows the BHP required at the diesel engine, assuming a right-angle gear box efficiency of 95%. The diesel engine is rated at 150 BHP. Pump BHP demands for each tailwater condition (listed in Table 5-19) are below the diesel engine rating, indicating that the engine is adequately sized for the modeled conditions. Because the generator cannot power the electric pump, the station is assumed to operate with one diesel pump for the design capacity analysis.

Table 5-19: 3rd & Detroit Pump Station Hydraulic Capacity

Hydraulic Condition	Station Capacity (gpm)	Pump BHP Demand at Engine	Diesel Engine BHP Rating
1 Pump: 100-yr Tide	20,100	115	150
1 Pump: 100-yr Tide, 2050 SLR	19,900	115	150
1 Pump: 100-yr Tide, 2100 SLR	19,000	125	150

3rd & Detroit Pump Station Evaluations & Recommendations

Recommendations for the 3rd & Detroit Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-20: 3rd & Detroit Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 10-year storm
Station Age	Built 1983 (43 years old)
Life Remaining before Rehabilitation	<5 years
Physical Condition	Mechanized trash rack is inoperable, generator is in poor condition, some electrical panels are not in service, wet well structure settlement near trash rack, existing diesel engine pump driver unknown condition and rare, building roof and wood trim deficiencies, electrical panel corrosion, pump discharge pipe and outfall structure including flap gates not accessible and condition unknown
O&M Burden	History of parts being hard to source, access to clean trash is difficult
Life Safety	Building ventilation operation unknown
Emergency Capability	On-site generator does not power pumps (only controls and lighting) and there is no backup generator receptacle, no backup pump controls
Operational	Only one pump can be operated during utility power outage

Recommended short-term improvements identified during the assessment include:

- Replace mechanical trash rack system [completed February 2026]
- Upgrade SCADA to report wet well water levels
- Repair settlement at wet well structure cold joint near trash rack
- Install backup pump controls with mechanical floats
- Repair and/or replace roof

The pump station has surpassed its 40-year life expectancy, and the following long-term improvements are recommended for this station.

- Conduct a detailed condition assessment of all major structural components, including but not limited to:
  - Concrete structures (above and below grade)
  - Discharge pipes and outfall
- Replace existing pumps with two pumps of similar operating points and both with electric motors
- Replace all major electrical equipment including conduits, wires, switchboard panel, and motor control center, etc.
- Install an ATS and backup generator
- Install new trash rack and modify wet well as needed (unless already addressed during short-term improvements)

5.3.9 16th Ave Pump Station Assessment

16th Ave Pump Station Existing Summary

The 16th Ave Pump Station is a seasonal diversion facility at the gate structure on the 16th Ave Drain where it meets Marina Lagoon.



Figure 5-34: 16th Ave Pump Station Site Location

During the summer, the flood gates are typically closed to prevent lagoon backwater into the local storm drain system and to divert low flows from the 16th Ave Drain directly to San Francisco Bay via the pump station. During the winter, the gates remain fully open. The station does not provide flood control and is not a terminal drainage facility as it is only operated during the summer.

Equipment consists of one axial-flow pump with a 30-hp vertical electric motor in a small building adjacent to the flood gates. Pump equipment specifications were not accessible. There is no permanent generator and no transfer switch. The station condition is functional for its limited seasonal purpose but poor overall. Several elements, including electrical and mechanical equipment and the structure itself, do not meet current codes and pose safety hazards for staff.



Figure 5-35: 16th Ave Pump Station Site



Figure 5-36: 16th Ave Pump Station Pump Motor

16th Ave Pump Station Evaluations & Recommendations

Recommendations for the 16th Ave Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-21: 16th Ave Pump Station Evaluation

Metrics	Evaluation
Capacity	Not applicable since this is not flood control facility
Station Age	Built 1983 (43 years old)
Life Remaining before Rehabilitation	<5 years
Physical Condition	Overall, extremely poor condition: stairs need to be rebuilt, wooden piles are not structurally sound, seal at pump head is leaking, foundation is cracking, corrosion on exposed metal, pump discharge pipe duckbill not closing
O&M Burden	History of parts being hard to source
Life Safety	Overall life safety hazard for O&M due to poor physical condition, holes in floorboards covered by traffic signage, electrical equipment does not meet current code
Emergency Capability	Not applicable because this is not a flood control facility
Operational	Trash screens are not accessible for cleaning

Recommended short-term improvements include:

- Conduct an electrical system inspection and bring up to current code
- Replace damaged stairs to pump station entrance
- Reseal pump head
- Replace pump discharge check valve and outfall
- Replace corroded metal equipment
- Replace trash rack system
- Replace damaged and/or missing floorboards
- Assess foundation and make needed repairs

The pump station has surpassed its 40-year life expectancy. The replacement of all major electrical and mechanical equipment, rehabilitation of the structures, and a detailed condition assessment are recommended for long-term improvements of this station. Several structural foundation deficiencies that should be addressed in the short term to improve safety will likely be significant and may necessitate a full station reconstruction.

Recommended long-term rehabilitations include:

- Constructing a new pump station with a pump of similar operating point to the original and an electric motor, back up controls, SCADA water level reporting, and operable trash rack

5.3.10 31st Ave Pump Station Assessment

31st Ave Pump Station Existing Summary

Constructed in 2021 at 50 E 28th Ave, the 31st Ave Pump Station serves the Caltrain underpass along 31st Ave, pumping runoff collected in the depressed roadway to a storm drain manhole and the existing storm drain system along 31st Ave. The tributary area of the pump station is 4.6 acres. Equipment consists of two Flygt 6-inch submersible stormwater pumps (962 gpm at 28 ft TDH, each pump) driven by 10-hp motors and located below grade in a wet well. Entry is via a double-leaf hatch with manhole ladder rungs, but the ladder is not practically accessible due to its proximity to location of the opening of the roof hatch. Motor control center panels are located above grade, in a secure weather-tight enclosure. Construction plans identify the panels as 480 V, while field observations indicate that the panels are labeled 240 V.

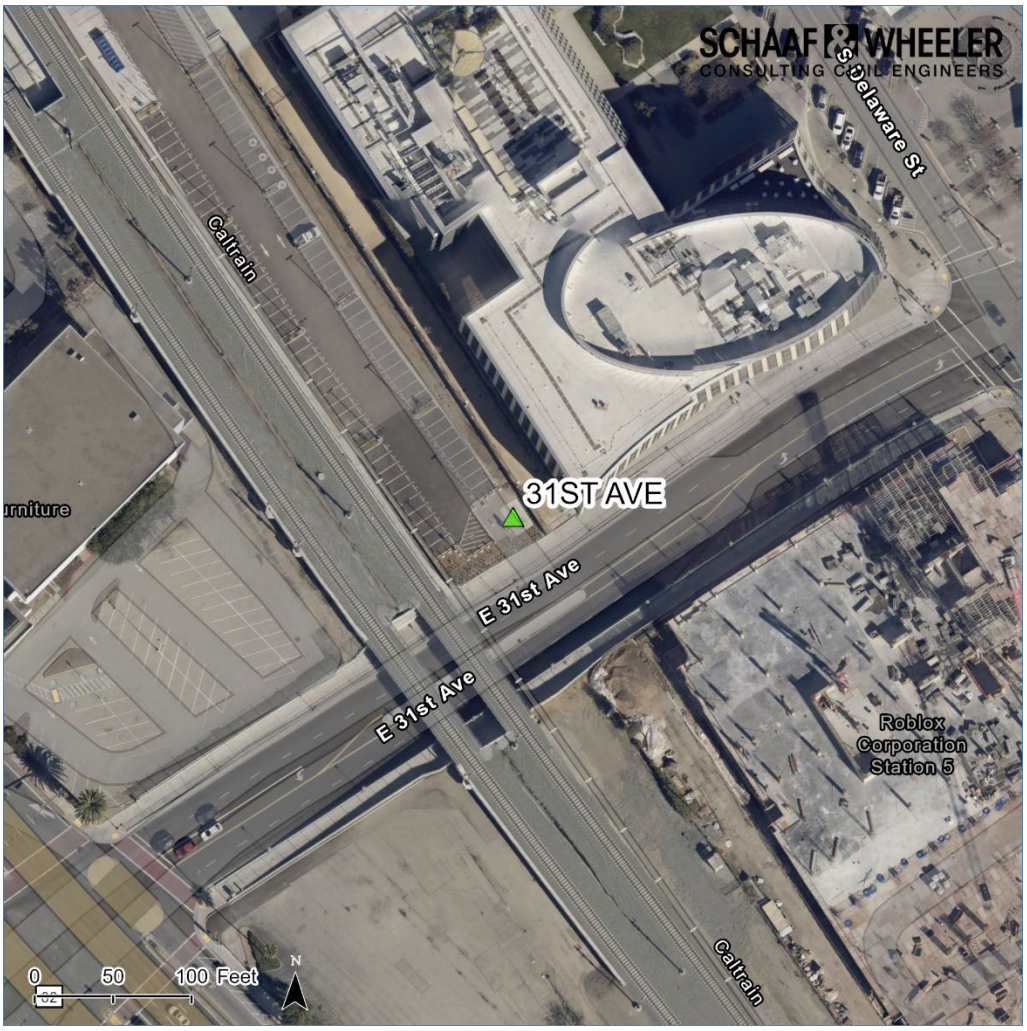


Figure 5-37: 31st Ave Pump Station Site Location

Standby power is provided through a portable-generator connection and manual transfer switch, which were added during construction but are not shown on the construction plans. The station does not have a permanent generator. There is no trash rack at the station, although O&M staff report no observed clogging of pumps from large debris. O&M also notes that the wet well cannot be fully pumped down for cleaning with the permanent pumps and that a vactor truck is required for station dewatering. While mechanical floats are shown on construction documents,

no mechanical floats were present during the site visit, and staff report that the station operates within an approximate one-foot wet well level range. The pumps do not auto-alternate lead duty. Instead, one pump serves as the lead pump and the other serves as the redundant pump, which is reflected in the individual pump runtime meters. The pump station has a separate valve vault structure that contained several inches of standing water during the site visit, and the pump check-valve counterweight lever arms had been removed. The pumps have not been serviced since installation, and no bypass capability is provided. Overall condition is good, as the station has been newly constructed, though several operational and maintenance limitations should be addressed as the facility ages.



Figure 5-38: 31st Ave Pump Station Site



Figure 5-39: 31st Ave Pump Station Valve Vault Structure

31st Ave Pump Station Hydraulics

The existing pump performance and system curves for the 31st Ave Pump Station are shown in Figure 5-40. When both pumps operate, the system curve intersects near the middle of the pump performance curve, where the pumps run more efficiently, whereas the one-pump system curve intersects near the right end of the pump curve, where pump efficiency is lower.

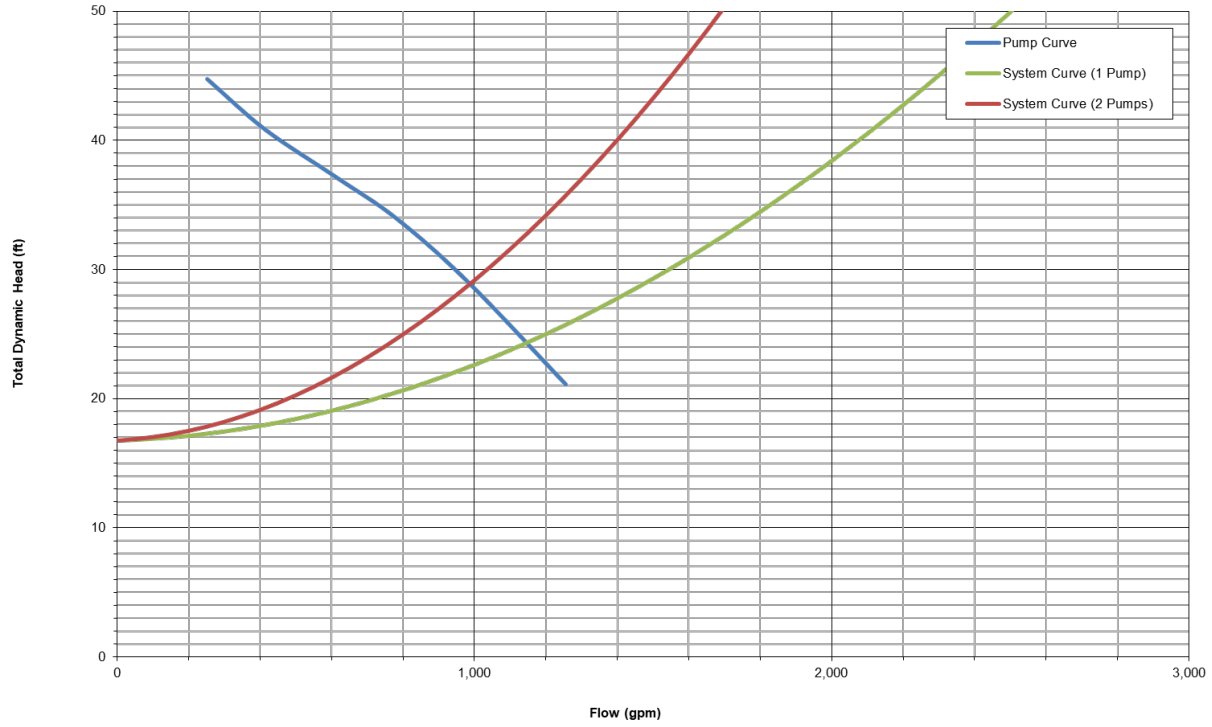


Figure 5-40: 31st Ave System Curve

The point where the blue pump curve intersects the one-pump system curve is the expected operating point when only one pump is running (approximately 1,100 gpm), while the intersection with the two-pump system curve represents the operating point when both pumps run (approximately 2,000 gpm) which represents total station capacity. These flows define the usable hydraulic capacities summarized in Table 5-22 and are used to compare the station's performance against the required design flows from the hydrologic model.

Table 5-22: 31st Ave Pump Station Hydraulic Capacities

Hydraulic Condition	Station Capacity (gpm)
2 Pumps	2,000
1 Pump	1,100

31st Ave Pump Station Evaluations & Recommendations

Recommendations for the 31st Ave Pump Station are based on the hydraulic evaluation, condition and operations assessments, and input from City staff. The proposed actions focus on improving reliability, safety, operations, and where appropriate, addressing long-term needs to support continued effective performance of the facility.

Table 5-23: 31st Ave Pump Station Evaluation

Metrics	Evaluation
Capacity	Meets capacity requirements for the 10-year storm
Station Age	Built 2021 (5 years old)
Life Remaining before Full Replacement	35 years
Physical Condition	Poor drainage in valve box, pump check valve counterweight lever arms are not installed, damage to valves could be present
O&M Burden	Wet well is difficult to clean due to depth and requires a vactor truck
Life Safety	No major deficiencies
Emergency Capability	Wet well water levels not transmitted to SCADA
Operational	Hatch location on wet well leaves man ladder inaccessible, pumps do not auto-alternate as lead pump, No backup pump controls

Recommended short-term improvements include:

- Upgrade SCADA to report wet well water levels
- Adjust controls so that pumps auto-alternate to extend life of station
- Install backup pump controls with mechanical floats (independent of existing pump controller)
- Address drainage in valve box to prevent ponding
- Install check valve counterweight lever arms, adjust weights for proper operation and inspect valves to confirm no existing damage

The pump station is newly built, and no long-term rehabilitations are recommended for this station.

5.4 Pump Station Capital Improvement Program

5.4.1 Pump Station Project Prioritization

Based on the prioritization criteria, points are assigned to short and long-term projects for each pump station. The results confirm the importance of improving the Marina Lagoon Pump Station, which serves a significant portion of the City, and the 42nd Ave Pump Station, which is under capacity for the 10-year storm.

For long-term projects that rise above short-term projects for the same pump station, it is recommended that the short-term projects be combined with the long-term projects to be completed together. See Table 5-24 for the full list of pump station project prioritizations.

Table 5-24: Pump Station Project Scoring and Prioritization

Project Name	Existing Capacity	Climate Change Resiliency	Station Age & Life Remaining	O&M Burden	Life Safety	Emergency Capability	Equity Priority Area	Area of Impact	Maximum Depth	Response Time	Immediate Need	Total Points
Marina Lagoon Short-Term	0	0	10	10	0	10	5	15	13	4	20	**87**
Marina Lagoon Long-Term	0	10	10	10	0	15	5	15	13	4	0	**82**
4000 Pacific Long-Term	10	0	10	10	5	15	0	2	14	15	0	**81**
Hillsdale Short-Term	0	0	0	10	0	15	0	2	15	14	20	**76**
4000 Pacific Short-Term	0	0	0	10	0	15	0	2	14	15	20	**76**
Hillsdale Long-Term	0	0	10	10	5	15	0	2	15	14	0	**71**
3rd & Detroit Short-Term	0	0	0	10	0	15	5	4	5	5	20	**64**
Coyote Short-Term	0	0	0	0	0	5	0	7	14	15	20	**61**
Poplar Short-Term	0	0	0	0	0	5	5	7	14	2	20	**53**
31st Ave Short-term	0	0	0	10	0	10	0	2	14	14	0	**50**
Fathom Dr Short-Term	0	0	10	0	5	10	0	0	0	0	20	**45**
3rd & Detroit Long-Term	0	0	10	10	0	0	5	4	5	5	0	**39**
16th Ave Long-Term	0	0	10	10	5	10	0	0	0	0	0	**35**
16th Ave Short-Term	0	0	0	10	5	0	0	0	0	0	20	**35**
Casanova Short-Term	0	0	0	0	0	10	0	2	2	0	20	**34**
Casanova Long-Term	0	0	0	0	5	0	0	2	2	0	0	**9**
Fathom Dr Long-Term	0	0	0	0	0	0	0	0	0	0	0	**0**

5.4.2 Pump Station Overview Cost Estimates

This section presents planning-level cost estimates for the recommended pump station improvements. The estimates summarize total project costs by pump station and improvement type, including construction, contingencies, and allowances for engineering, permitting, and construction support. The intent is to provide an order-of-magnitude basis for budgeting, phasing, and comparing projects across the system. More detailed calculations supporting these estimates are provided in Appendix H.

Table 5-25: Pump Station Total Project Cost Estimate

Project Name	Total Project Cost
16th Ave Pump Station Long-Term	$860,000
3rd & Detroit Pump Station Long-Term	$7,800,000
4000 Pacific Pump Station Long-Term	$3,680,000
Casanova Pump Station Long-Term	$105,000
Fathom Dr Pump Station Long-Term	$70,000
Hillsdale Pump Station Long-Term	$2,260,000
Marina Lagoon Pump Station Long-Term	$100,000,000
16th Ave Pump Station Short-Term	$530,000
3rd & Detroit Pump Station Short-Term	$1,150,000
4000 Pacific Pump Station Short-Term	$960,000
Casanova Pump Station Short-Term	$180,000
Coyote Point Pump Station Short-Term	$700,000
Fathom Dr Pump Station Short-Term	$740,000
Hillsdale Pump Station Short-Term	$960,000
31st Ave Pump Station Short-Term	$120,000
Marina Lagoon Pump Station Short-Term	$7,250,000
Poplar Ave Pump Station Short-Term	$300,000
GRAND TOTAL	**$127,665,000**

6. Evaluation of Storm Drain Systems

6.1 Overview and Performance Criteria

A performance evaluation of the City storm drain system is the primary focus of this analysis. This chapter describes the City's storm drain facilities and discusses potential capacity deficiencies and other drainage issues in the system. Evaluation of the system is based on either 10- or 100-year level-of-service standards, as summarized in Table 6-1.

Table 6-1: Summary of Design Storm Events for Key Drainage Systems in San Mateo

Design Event Return Period	Applicable Systems
10-year	Gravity pipe systems Pump Stations: ▪ 31st Ave ▪ 4000 Pacific ▪ Casanova ▪ Detroit Ave ▪ Fathom ▪ Hillsdale
100-year	Marina Lagoon Creeks: ▪ San Mateo Creek ▪ Laurel Creek ▪ 16th Ave Channel ▪ 19th Ave Channel Pump Stations: ▪ Coyote ▪ Marina Lagoon ▪ Poplar

These standards are expected to provide a reasonable level of protection against frequent property damage and minimize areas in the special flood hazard areas in North Shoreview and around Marina Lagoon.

Input from City staff is invaluable in guiding development of the models to reflect known drainage issues and ensure that deficiencies are not predicted where none exist. This provides a means of verifying model results, field observations, County staff knowledge, and other calculations.

1D-model results are characterized by the model's Node Flood result. In the absence of a 2D mesh over a system node, the Node Flood depth is estimated based on storage of runoff that surcharges above the node's ground surface in an illusory conical storage area. The conical shape of the storage volume is meant to approximate the spread of flood waters over an adjacent surface and estimate the hydraulic grade line over the node without allowing surcharged waters to escape. In reality, those surcharged waters might continue down a gutter line to another inlet where capacity in the system is available, or they may collect and accumulate elsewhere.

The Node Flood result is therefore not considered a metric of actual flooding for purely 1D model elements. Instead, the Node Flood result is used as a guideline to identify reaches in the

system that do not meet the applicable standard and for developing capital improvement projects that will aid in meeting those standards.

A large portion of the model also includes a 2D domain because the interaction between existing stormwater drainage systems, creeks, and overland flow paths is complex. It is also important to understand how stormwater spreads or ponds across low-lying areas, where overflow from one continuous storm drain system may run down gradient, effectively increasing the volume of runoff that adjacent systems must handle beyond what is considered solely by 1D modeling of closed conduit systems.

6.2 Existing System Evaluation

6.2.1 Pipe System Capacity

Modeling of the City's stormwater system assumes that pipes are operating free and clear of blockage. Accumulation of sediment and debris within the closed conduit system (e.g. pipes and culverts) was evaluated for various reaches by CCTV inspection (discussed in Section 6.2.3). The pipe system model has generally been used to identify areas where overflow occurs due to a lack of available conveyance capacity. MIKE+ highlights system nodes where the hydraulic grade line rises above ground level, as well as the extent and depth of flooding on the surface. These results are shown in Figure 6-1 through Figure 6-6 for various modeled storm events. More detailed result maps are provided in Appendix I.

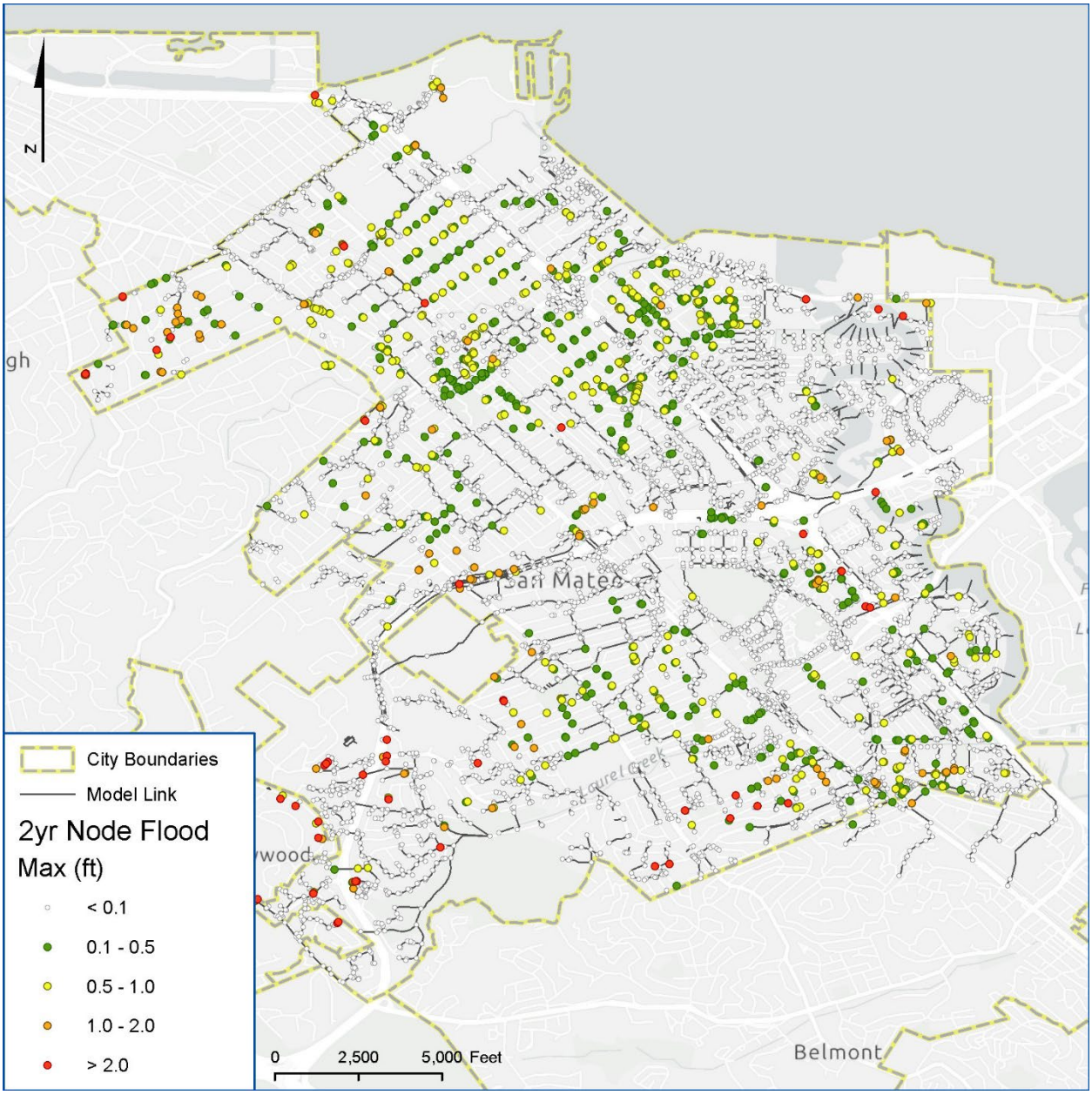


Figure 6-1: 2-year Storm, 1D Node Flood Result from the Existing Conditions MIKE+ Model

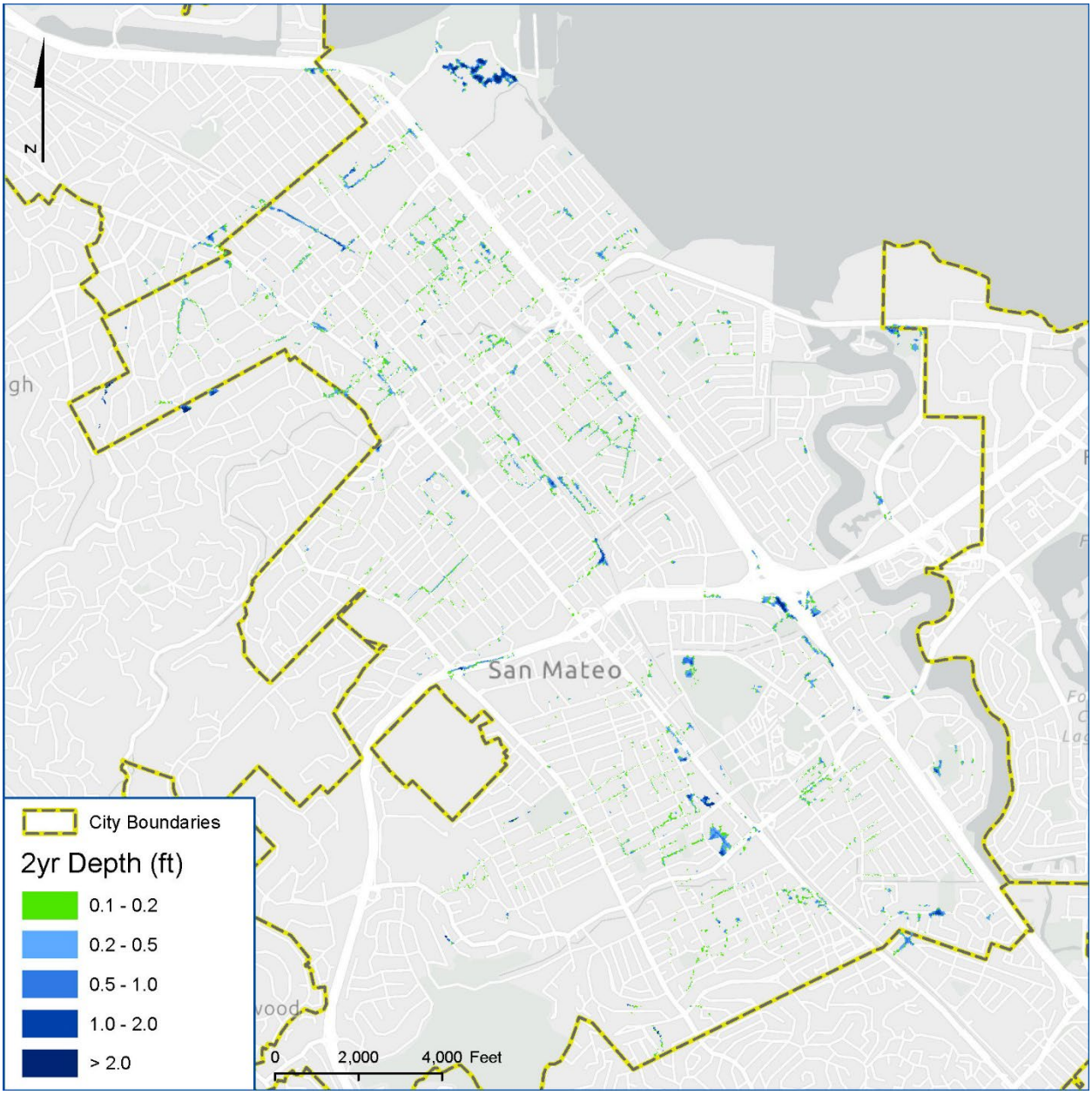


Figure 6-2: 2-year Storm, 2D Maximum Depth Result from the Existing Conditions MIKE+ Model

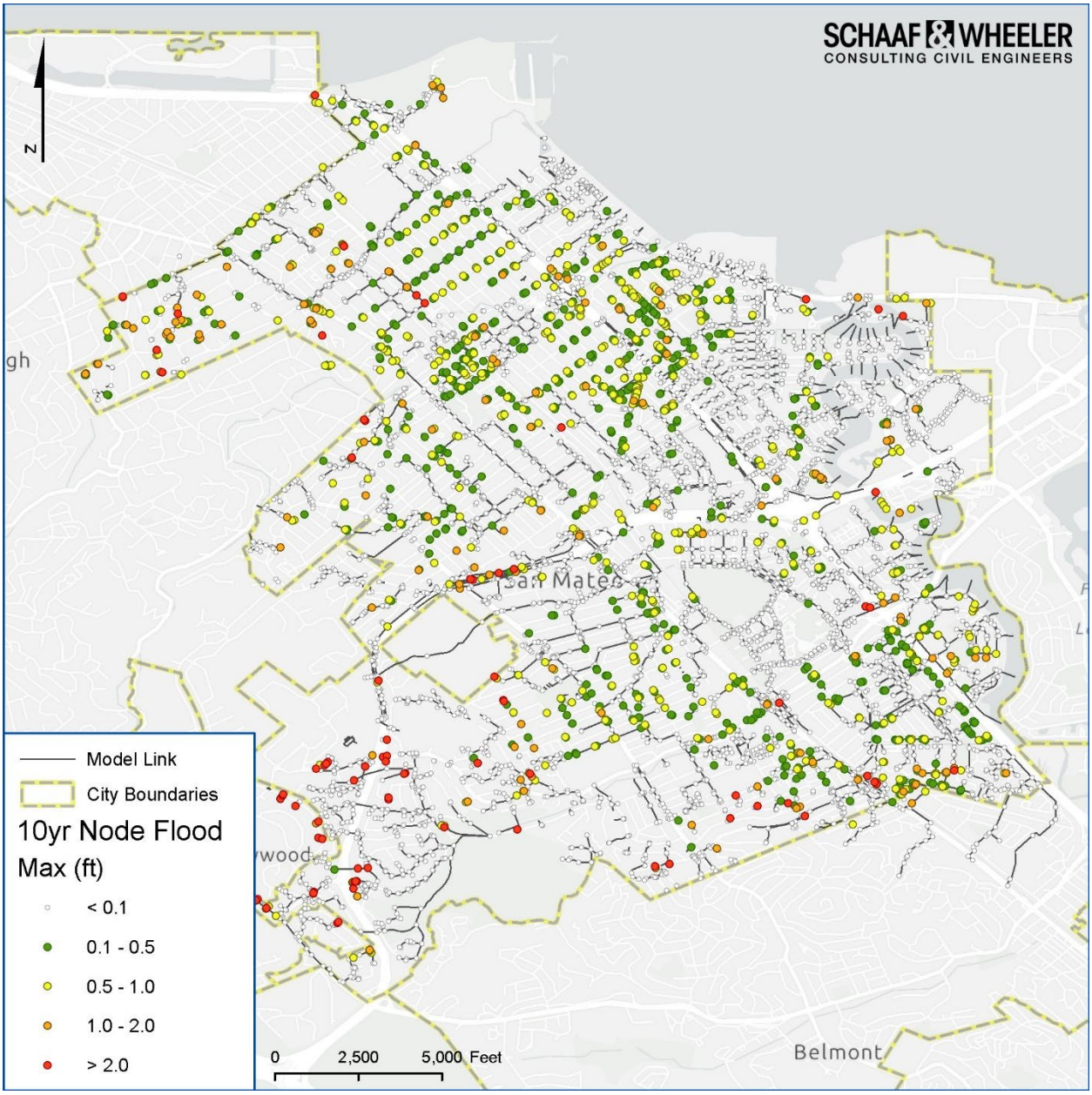


Figure 6-3: 10-year Storm, 1D Node Flood Result from the Existing Conditions MIKE+ Model

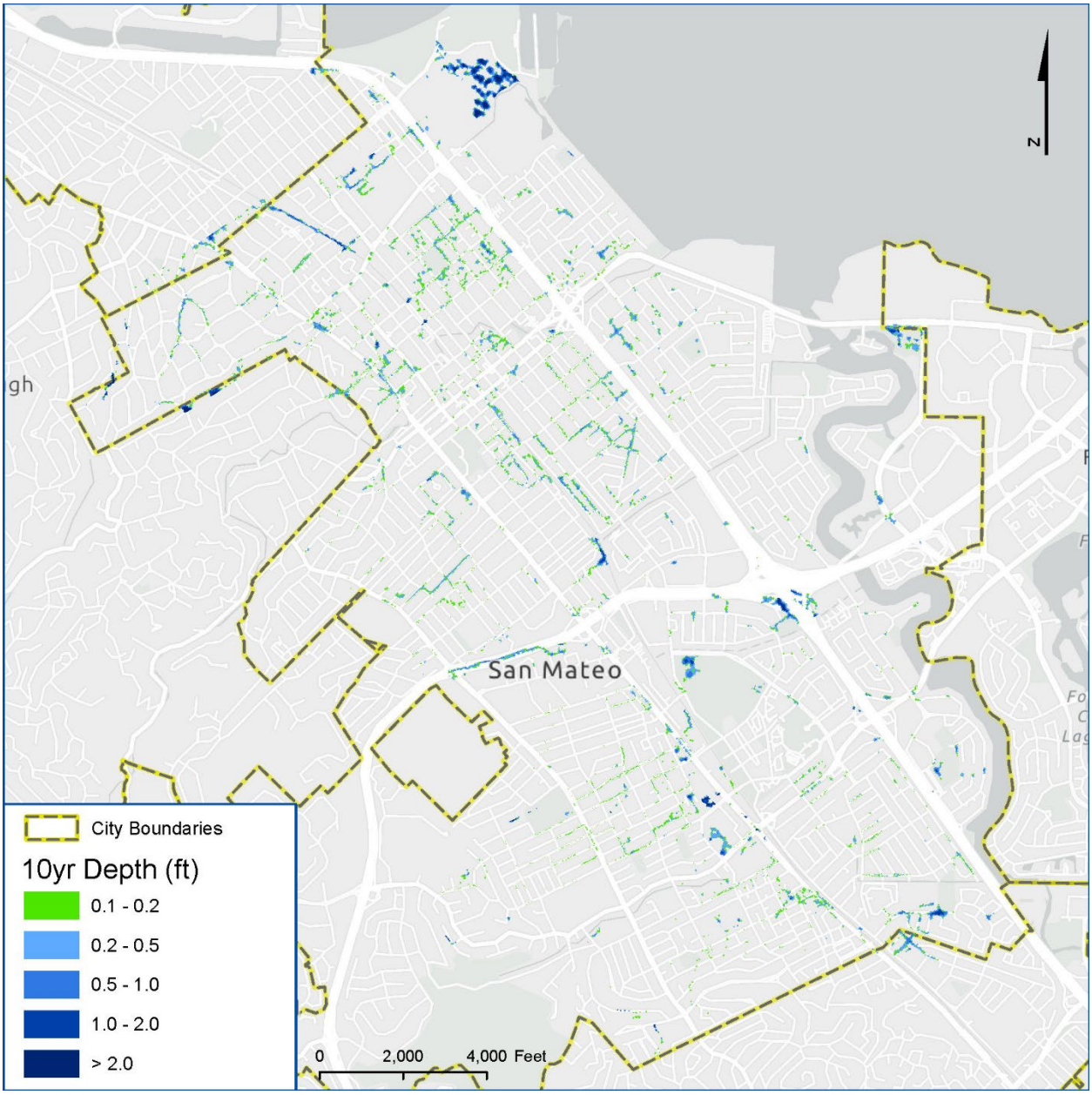


Figure 6-4: 10-year Storm, 2D Maximum Depth Result from the Existing Conditions MIKE+ Model

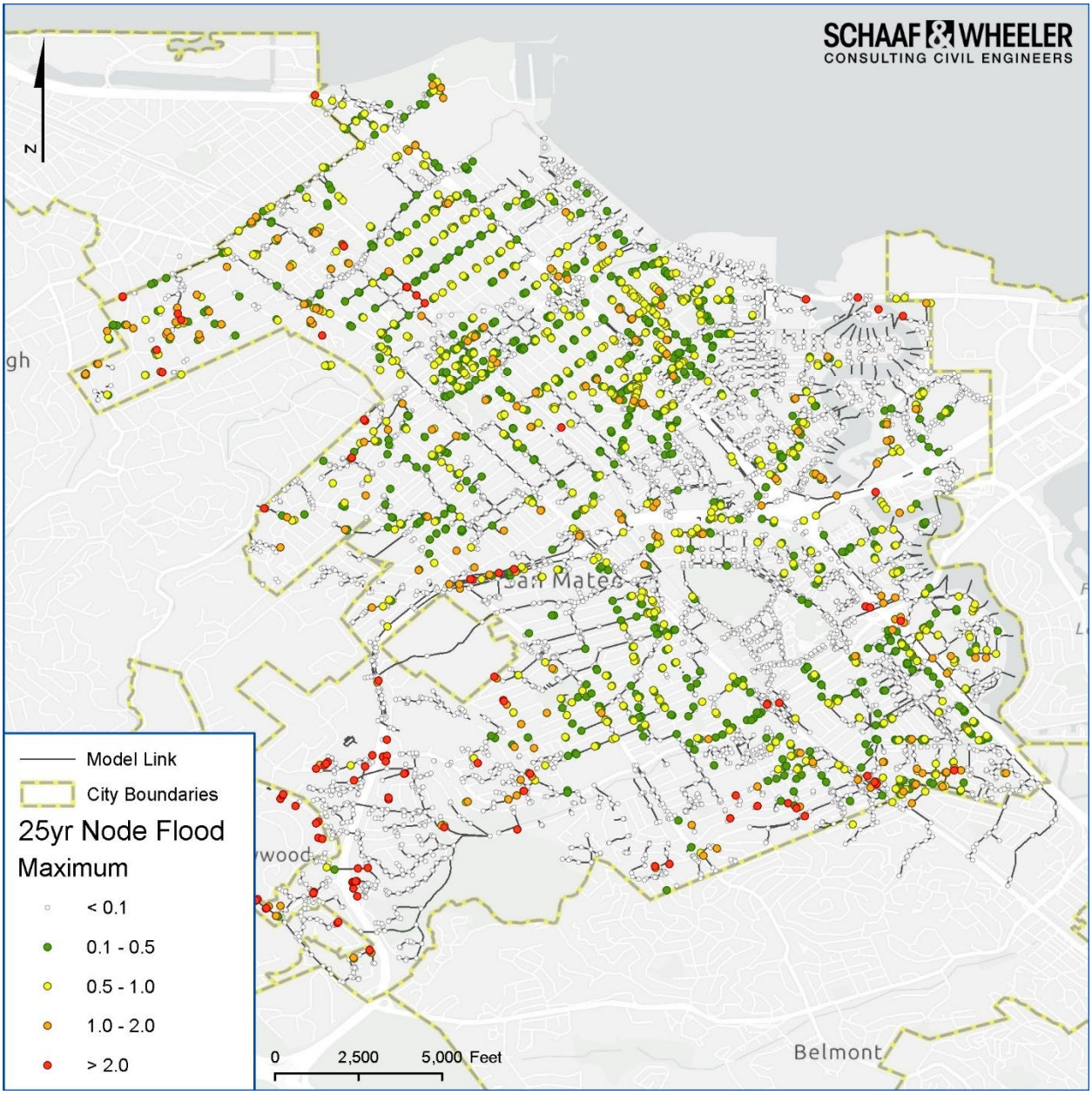


Figure 6-5: 25-year Storm, 1D Node Flood Result from the Existing Conditions MIKE+ Model

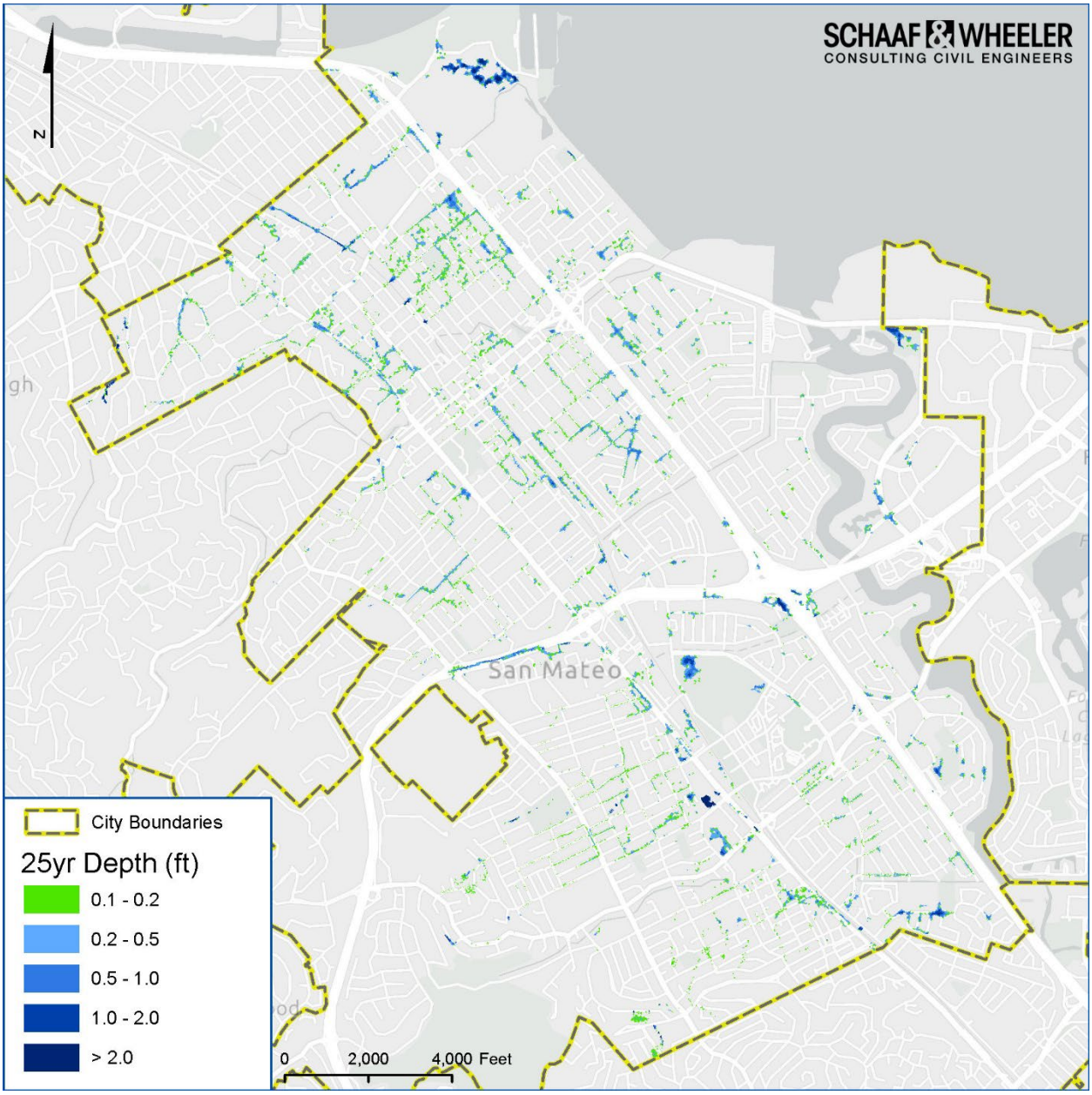


Figure 6-6: 25-year Storm, 2D Maximum Depth Result from the Existing Conditions MIKE+ Model

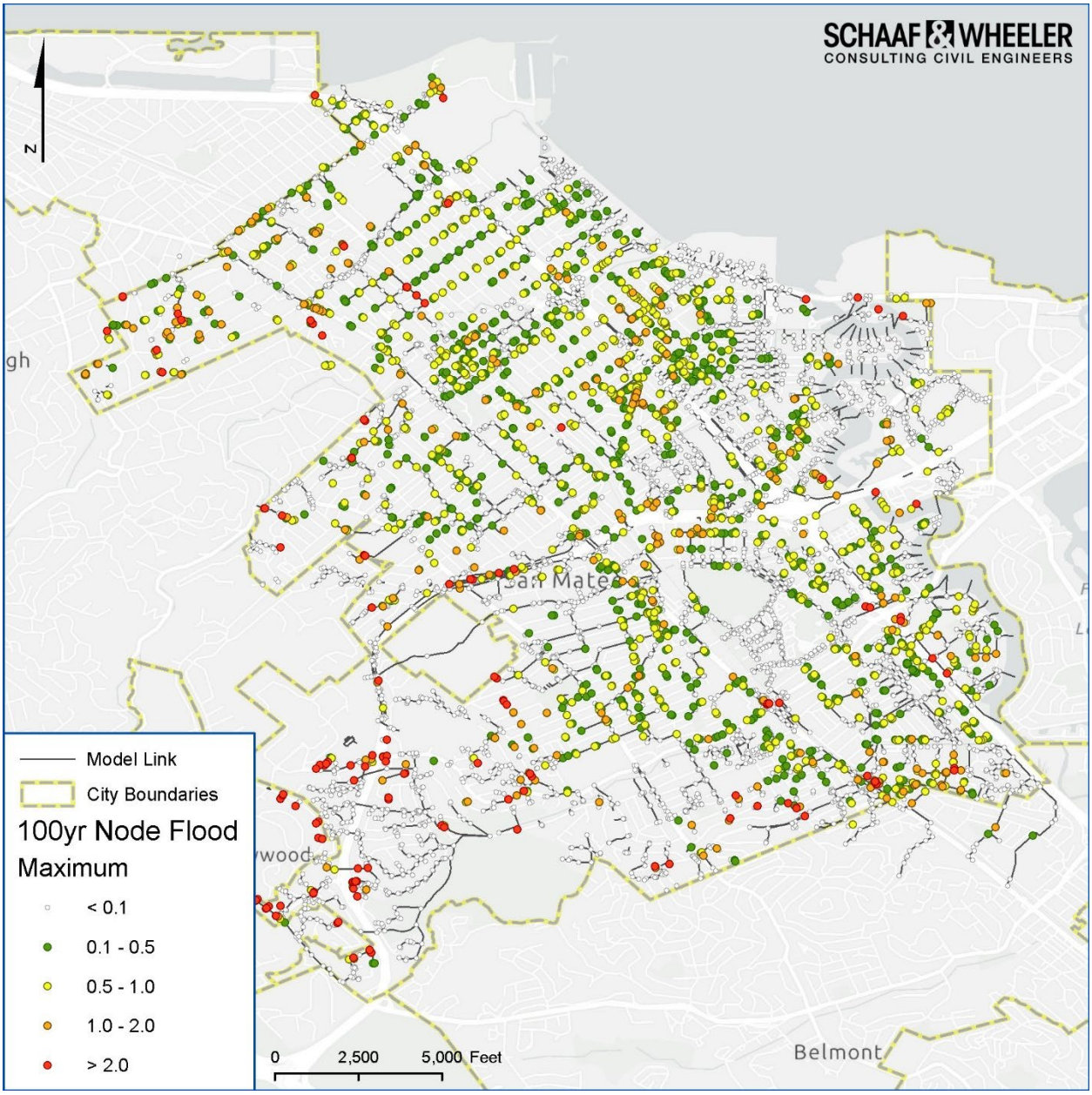


Figure 6-7: 100-year Storm, 1D Node Flood Result from the Existing Conditions MIKE+ Model

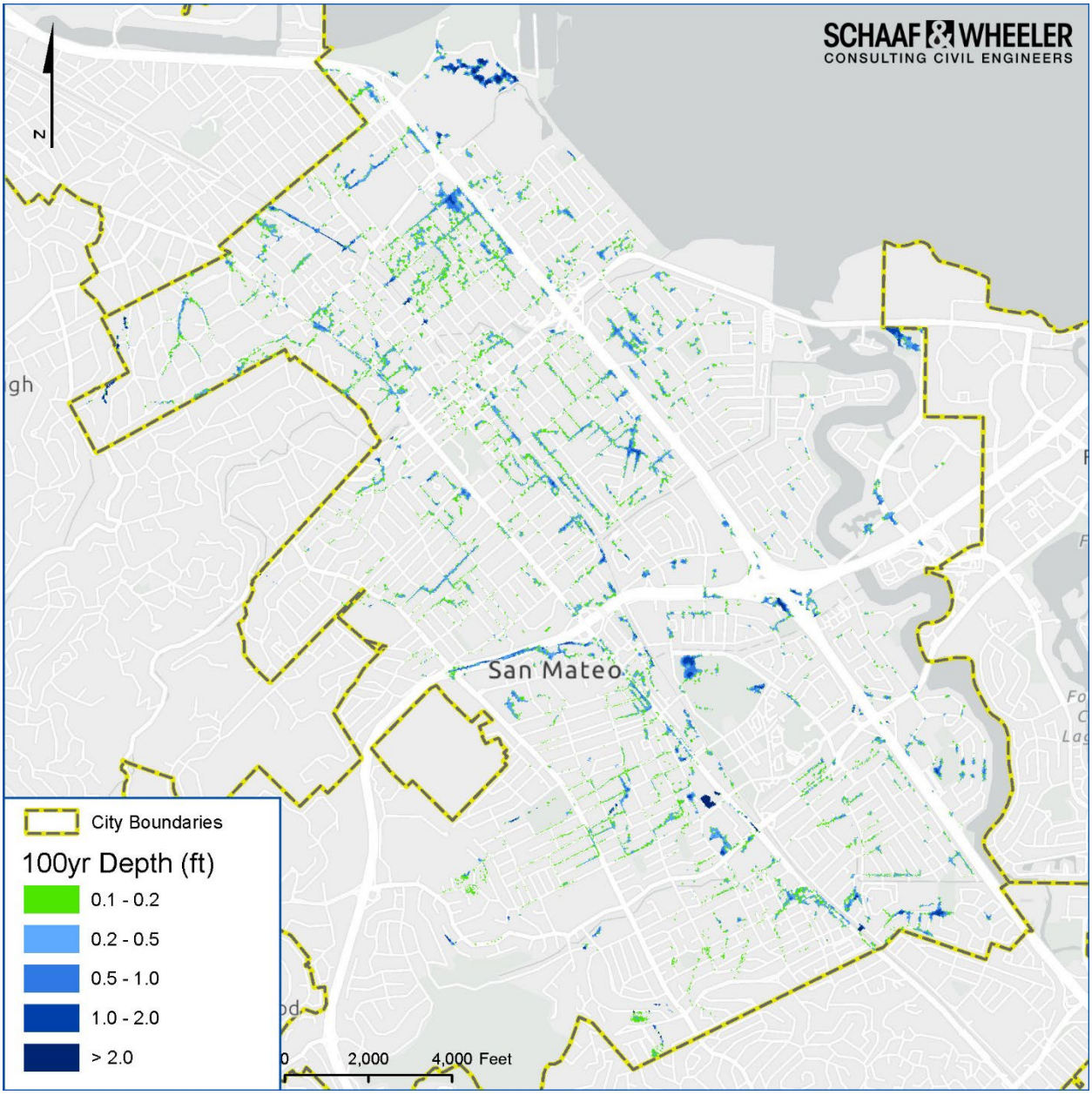


Figure 6-8: 100-year Storm, 2D Maximum Depth Result from the Existing Conditions MIKE+ Model

Model results have been verified by examining particular areas where flooding is known to occur, ensuring that the model replicates recurring issues that are suspected to be caused by undersized storm drain systems. Three examples of locations used to verify model results include the following:

- Sunnybrae Blvd and Grant St
- 42nd Ave Underpass
- 17th Ave at the railroad

The 1D and 2D model results for a 10-year storm confirm that flooding occurs as a result of a lack of pipe system capacity at Sunnybrae Blvd and Grant St and at 17th Ave. The 2D surface

mesh resolution does not fully capture the 42nd Ave underpass shape. However, a 1D model profile shows nearly two feet of flooding at the low point of the underpass, assuming that the pump station is fully functioning at its design capacity during the storm.



Figure 6-9: 10-year Storm, 2D Maximum Depth Result at Sunnybrae Blvd and Grant St

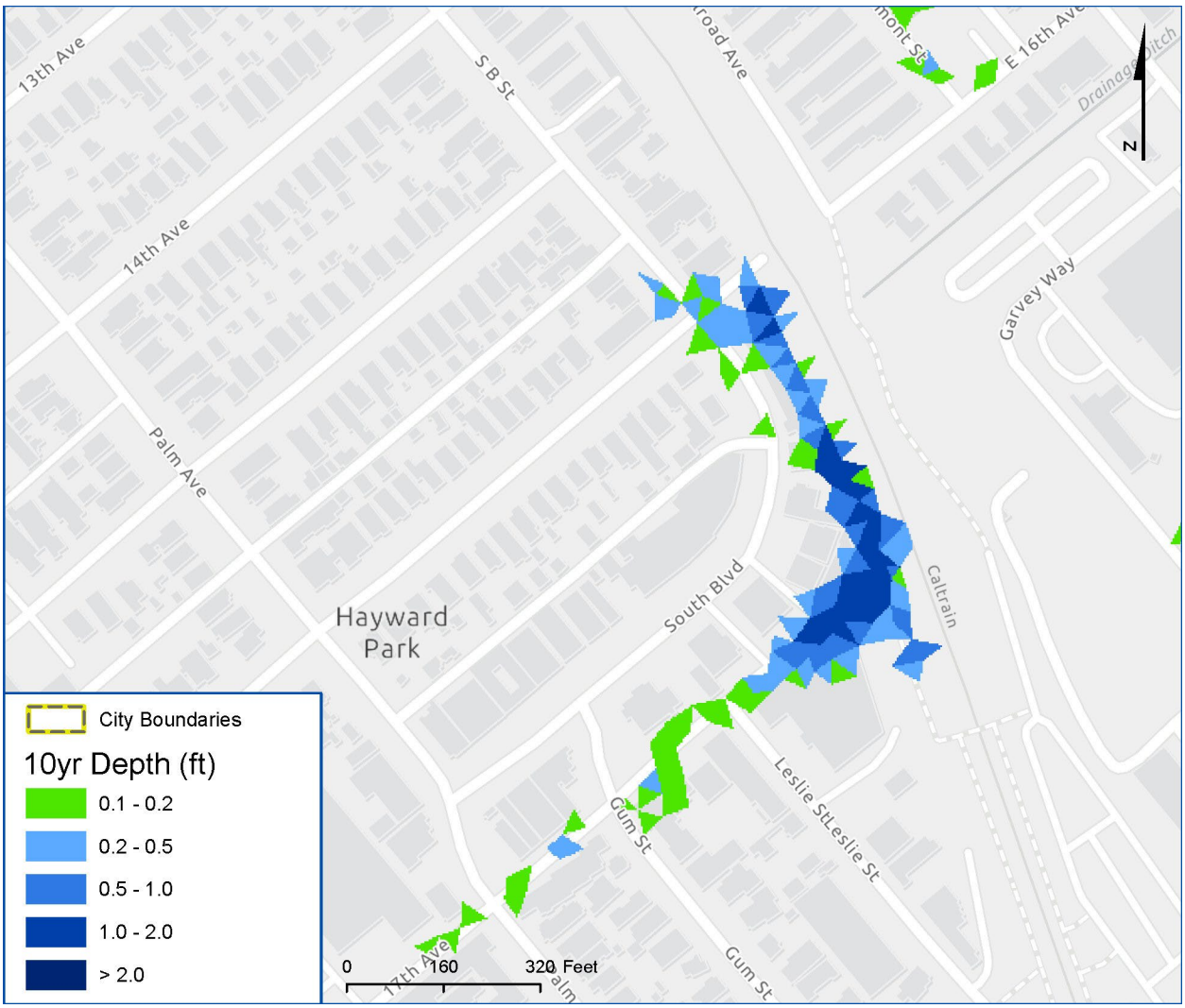


Figure 6-10: 10-year Storm, 2D Maximum Depth Result at 17th Ave near the Railroad

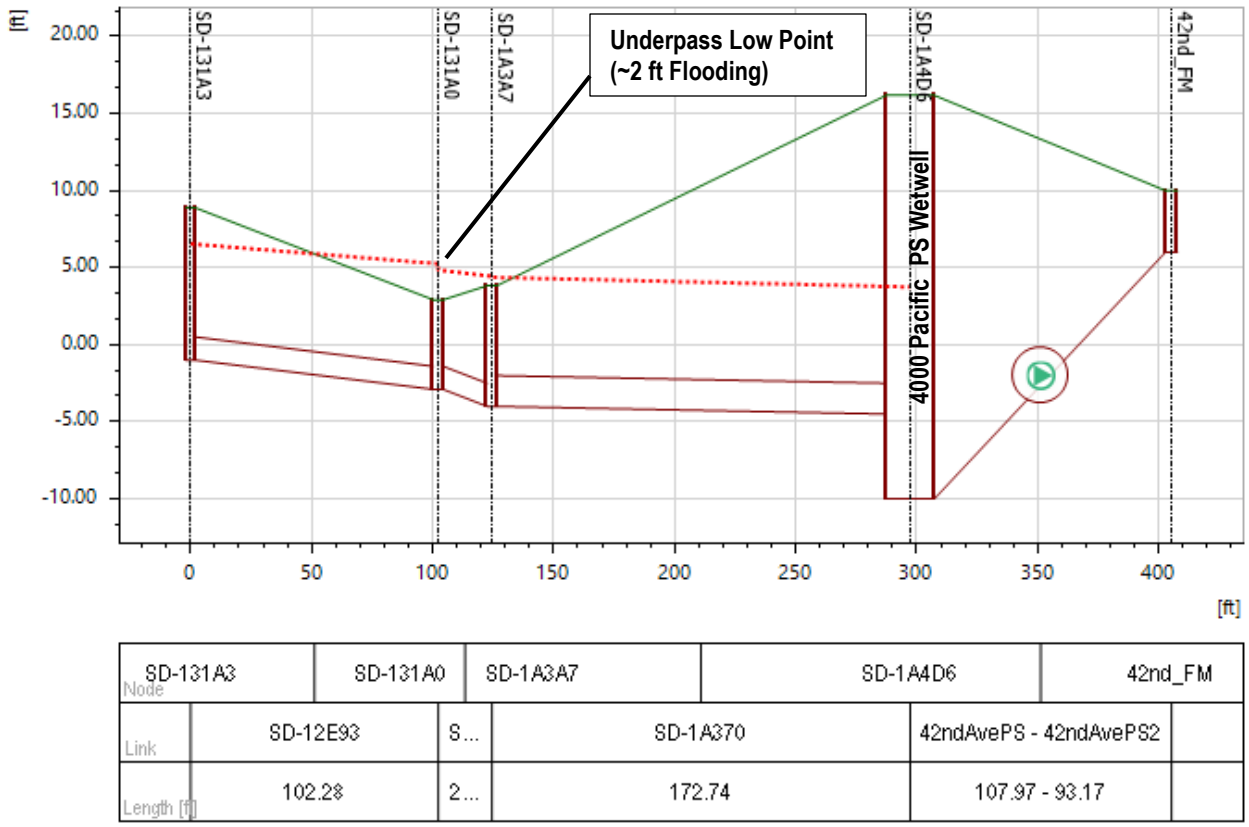


Figure 6-11: 10-year Storm, 1D Maximum Water Surface Profile at 42nd Ave Underpass and 4000 Pacific Pump Station

6.2.2 Pump Station Capacity

The MIKE+ system model includes pump stations, with existing capacities represented with de-rated system curves. 1D and 2D model results can be used to evaluate the adequacy of pump system capacity based on hydraulic grade lines and spill from the systems upstream of the pumps. The models are dynamic and allow all pumps to operate on any point along their system curves. Peak discharge rates from the pumps can, therefore, vary significantly throughout each modeled storm event and deviate from design operating points based on tailwater conditions and wet well levels at any given point in time. Maximum modeled discharge rates from each pump station are compared with expected design discharge rates in Table 6-2.

Table 6-2: Comparison of Modeled Pump Station Discharge with Design Capacity

Pump Station	Maximum Modeled Discharge (cfs)	Station Capacity (cfs)
4000 Pacific	13	12
Casanova	105	89
Coyote	318	348
Detroit	46	45
Hillsdale	14.5	10
Marina Lagoon	1,700	1,660
Poplar Ave	362	370

For the majority of pump stations, existing capacity is adequate to handle each station's respective design storm event. For many stations, the maximum discharge rate exceeds the expected design capacity instantly, usually at a point in the simulation when the TDH on the pump is above the set of parameters used to design the pump station and select pumps. This is expected behavior, and the model results generally indicate that actual conditions at each pump station are within anticipated operating ranges. The exception is the 4000 Pacific Pump Station. With CIPs complete upstream, increasing the capacity of this pump station to 7,200 gpm is recommended to address any potential flooding conditions during a design storm event. It is worth noting, however, that the pipe carrying runoff from the underpass to the wet well needs to be enlarged as well. Increasing the capacity of the station may not be beneficial without providing adequate gravity system capacity between the underpass and the wet well, and these two capital improvement projects should be designed and constructed together.

Deepening the nearby system on Vista Ave and providing a gravity overflow from the 4000 Pacific Pump Station may also be a viable strategy for mitigating flooding in the underpass in the event of pump failures. This is not identified as a CIP, but could be considered as an alternative if other constraints (e.g. utilities in the street) allow it.

6.2.3 Pipe System Condition

Initially, a topside condition assessment was performed with pole-mounted cameras at system access points (e.g. manholes, inlets, and outlets). This enabled a high-level evaluation of the condition of various portions of the system by visually inspecting for sediment and debris accumulation, cracks, and material degradation (corrosion, spalling, aggregate exposure, or deformation). Pipes were not inspected continuously throughout their length during the initial evaluation. Topside inspections served as the basis for identifying elements of the system to be further inspected with CCTV.

CCTV inspection focused on system assets where topside assessment noted potential for additional issues in the pipes that was not visible from the access points, additional CMP reaches, and on backbone pipes that the model indicated as lacking capacity.

Information collected from CCTV inspections is summarized in Appendix C. An overview of structural and operational/maintenance ratings are shown in Figure 6-12 and Figure 6-13.

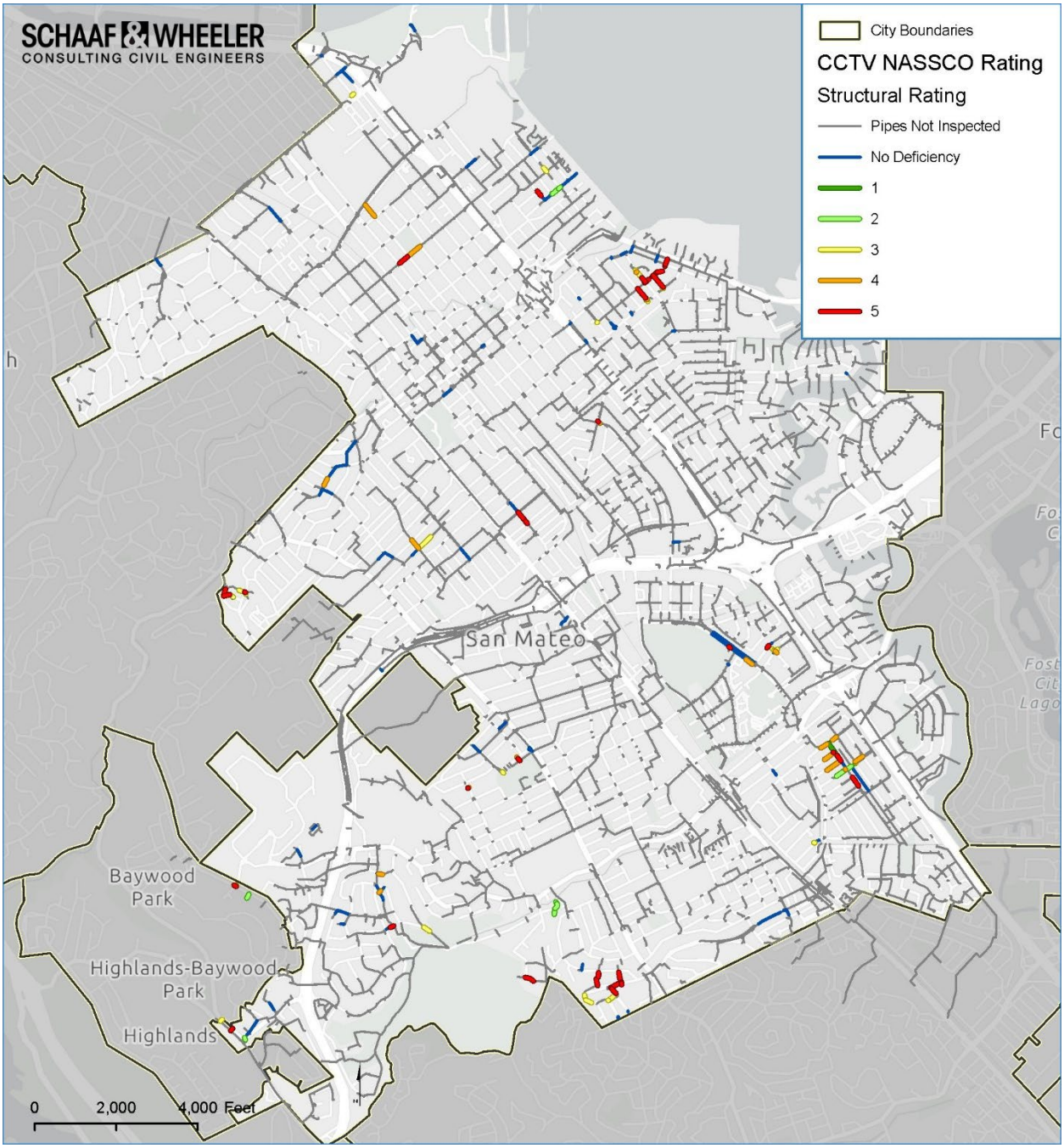


Figure 6-12: Structural Ratings Overview for CCTV Reaches

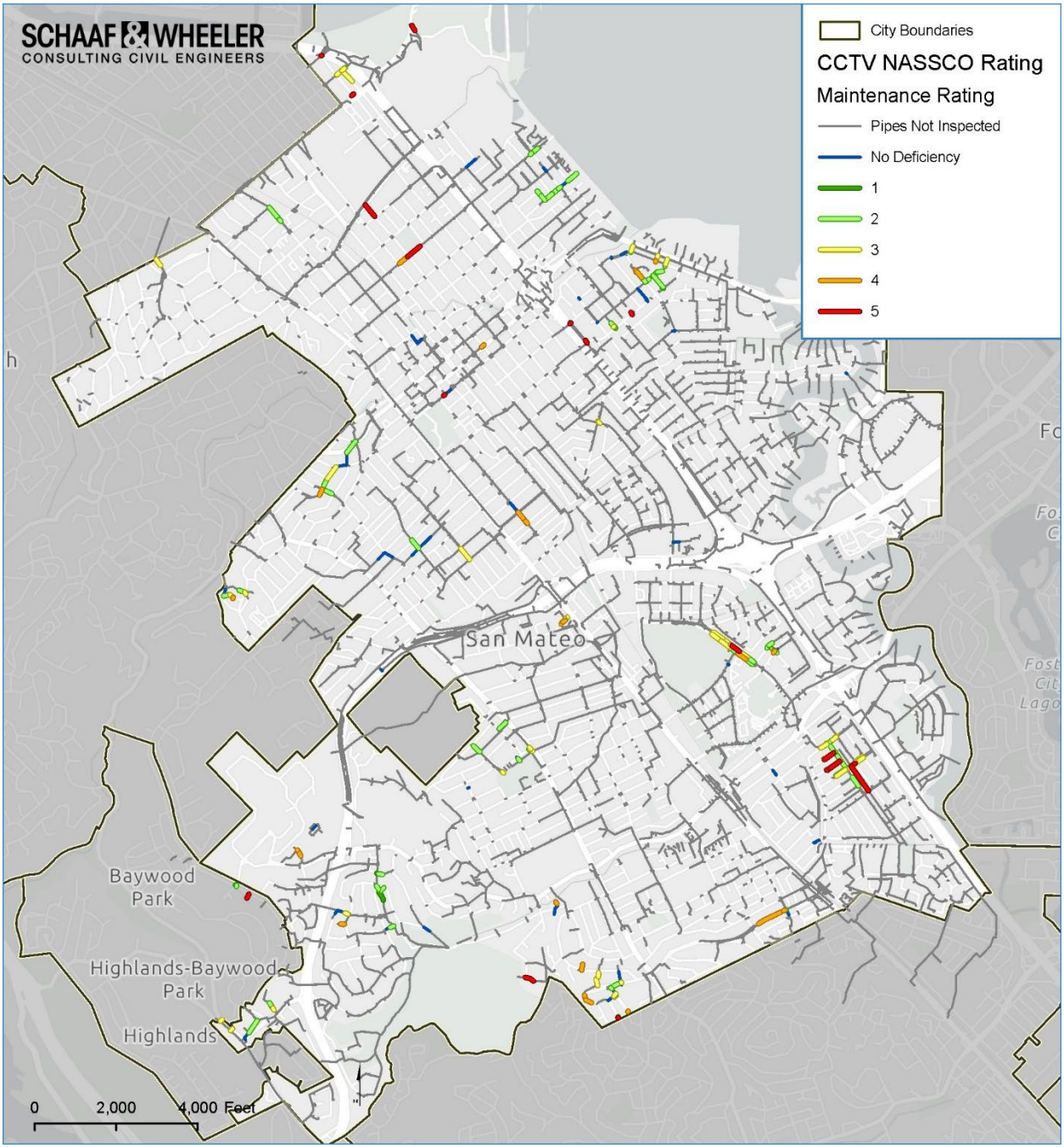


Figure 6-13: Operational/Maintenance Ratings for CCTV Reaches

Figure 6-14 shows images of both maintenance and structural ratings 1 through 3, and Figure 6-15 depicts ratings 4 and 5.

Score	Maintenance	Structural
1	 Some fine sediment accumulation or light infiltration	 Minor defects and aging not likely to cause failure
2	 Local sediment and debris accumulation not likely to impact capacity	 Localized failures (spalling and cracking) that could propagate, light corrosion
3	 Distributed sediment/debris accumulation beginning to impact capacity	 Joint failures; Corrosion and deformation without failure or significant capacity loss

Figure 6-14: Pipes with Maintenance and Structural Ratings 1–3 from CCTV Inspections

Score	Maintenance	Structural
4	 Significant loss of capacity due to sediment/debris accumulation	 Significant corrosion or deformation; large holes
5	 Near complete loss of capacity due to blockage	 Pipe failure, crushing, extreme corrosion

Figure 6-15: Pipes with Maintenance and Structural Ratings 4–5 from CCTV Inspections

Summaries of ratings and recommendations are shown in Table 6-3 through Table 6-5 for the inspected reaches. Overall, approximately 28% of the inspected system is in poor structural condition, rating between 4 and 5. Structural ratings of 4 and 5 generally require immediate attention and, in most cases, replacement. Structural ratings of 3 and 4 require repair or replacement but have not suffered an extreme loss of capacity and are at reduced risk of imminent failure. Similarly, maintenance ratings of 4 and 5 are likely indicate an extreme loss of conveyance capacity due to blockage that should be removed as soon as possible, while a rating of 3 and 4 indicates sufficient loss of capacity that affects the performance of the system without complete blockage.

Table 6-3: Inspected System Element Rating Summary

Rating	Structural		Maintenance	
	Length (ft)	Share of Total Inspected Length	Length (ft)	Share of Total Inspected Length
<2	17,255	60%	7,432	26%
2	1,271	4%	8,586	30%
3	2,407	8%	6,260	22%
4	3,038	11%	3,596	13%
5	4,767	17%	2,865	10%

Table 6-4: Inspected Drainage System Element Material Classifications

Material Class	Length (ft)	% of Length w/Structural Score of 5	Average Structural Score	% of Length w/Maintenance Score of 5	Average Maintenance Score
Asbestos Cement	284	0%	0	42%	4.42
Corrugated Metal	8,650	38%	2.65	6%	2.29
Polyethylene	5,279	2.9%	0.94	1.6%	1.14
PVC	1,232	3.9%	0.25	60%	3.34
Reinforced Concrete	14,438	7.2%	1.25	13.9%	2.53
Vitrified Clay	25	41.7%	3.98	0%	0

Table 6-5: Summary of Recommended Actions for Inspected Reaches

Recommended Action	Primary Deficiency Type	Quantity	Unit
Clean	Maintenance	740	LF
Clean and Re-televise	Maintenance	700	LF
Remove Deposit at Access	Maintenance	1	EA
Spot Repair	Maintenance	29	EA
Repair (CIPP)	Structural	1,762	LF
Repair (Open Trench)	Structural	40	EA
Full Replacement	Structural	7,745	LF

Portions of the system were not fully inspected due to the extent of blockage—the camera had to be withdrawn after partial inspection. It is possible that structural issues exist in the inspected reaches that were not identified due to blockage or heavy inundation during the inspections. Approximately 700 linear feet (about 2.5% of inspected pipe) of the inspected system was identified as in need of cleaning and re-televising to complete inspection of the pipe and provide a more reliable and comprehensive structural rating. Repair and replacement recommendations are applied to 33% of the inspected system. Some replacement recommendations apply to pipes based on material type, rather than condition. These would be considered low priority replacements to increase resilience.

6.2.3.1 Outfalls

Pipe system outfall condition has been documented to the greatest extent possible while preparing this SDMP (Appendix D). Some outfalls are not accessible due to access constraints and property ownership. Others were difficult or impossible to assess due to water levels in receiving bodies, for example, in the backwater of Marina Lagoon. However, those that were accessible during surveys and field investigations were evaluated, and their condition, size, material, and presence of backflow prevention (e.g. flap gates) were recorded.

6.2.4 Major Channels

The condition of open systems is equally important in evaluating the function of the City's stormwater system.

6.2.4.1 San Mateo Creek

San Mateo Creek handles runoff from a total direct drainage area of 4.2 square miles. However, the Crystal Spring Dam system, operated by the San Francisco Public Utilities Commission (SFPUC), also discharges to San Mateo Creek. A system of three dams and reservoirs (San Andreas, Lower Crystal Springs, and Upper Crystal Springs) stores and releases runoff from about 29 square miles. Dam locations are shown in Figure 6-16.

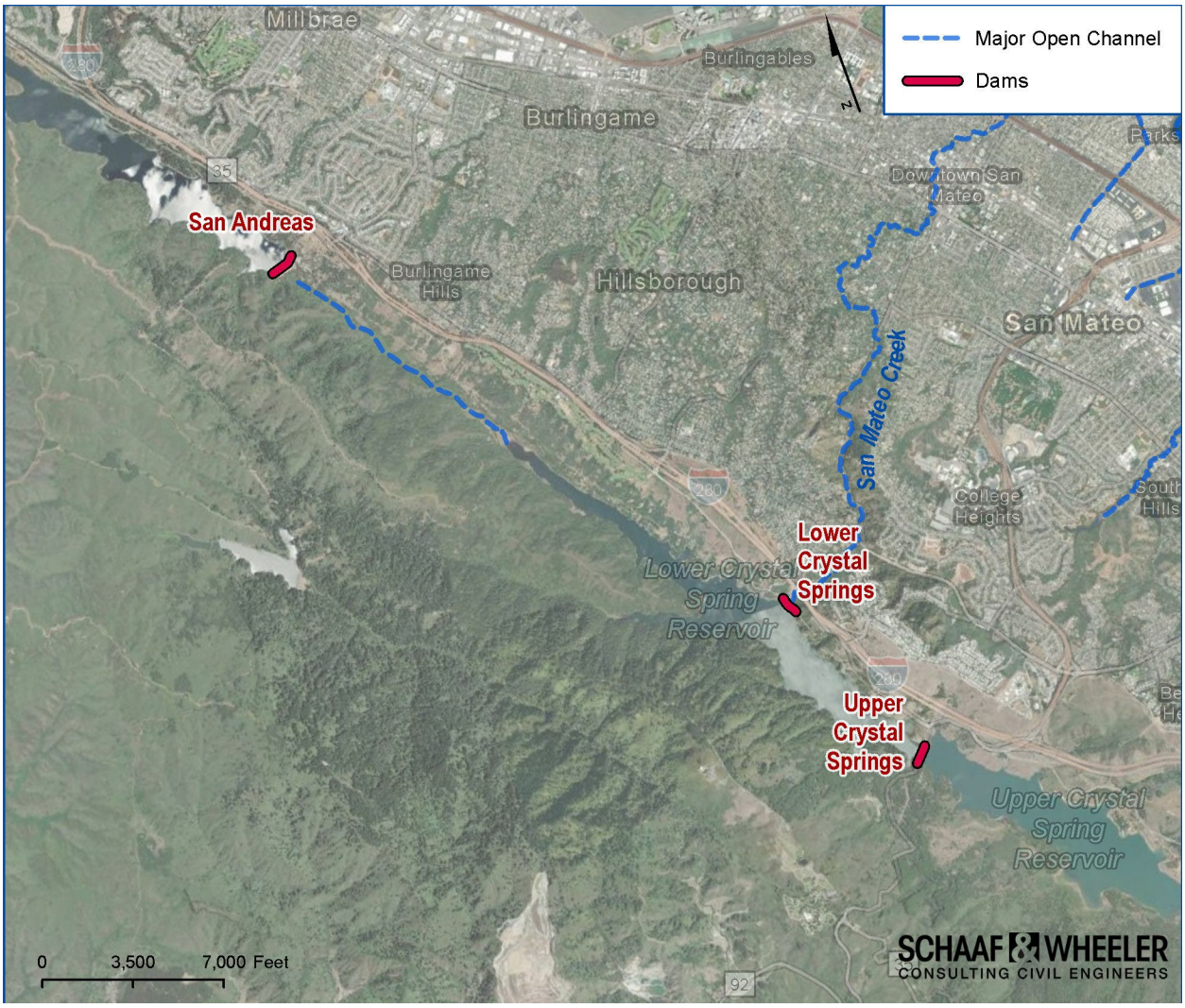


Figure 6-16: Dam Locations in the Crystal Springs Reservoir System

Lower Crystal Springs reservoir is impounded by an approximately 154-foot high concrete dam. Water is primarily released through a main outlet tunnel discharging through a 44-inch pipe line that sends stored water to San Francisco and other communities. The primary purpose of the facility is water supply storage; however, a roughly 200-foot wide spillway allows overflow in large storm events that discharges into San Mateo Creek upstream of City of San Mateo limits.



Figure 6-17: Lower Crystal Springs Dam Spillway During Construction of Improvements Completed in 2016

HEC-HMS model inputs have been modified from prior modeling based on the parameters of the dam modifications and 2017 County LiDAR. Based on current SFPUC operations, the modeled starting level is near the upper bound of winter operating conditions for conservatism. SFPUC monitors weather systems and releases some water accordingly to provide attenuation, with notifications to property owners downstream in advance of releases.

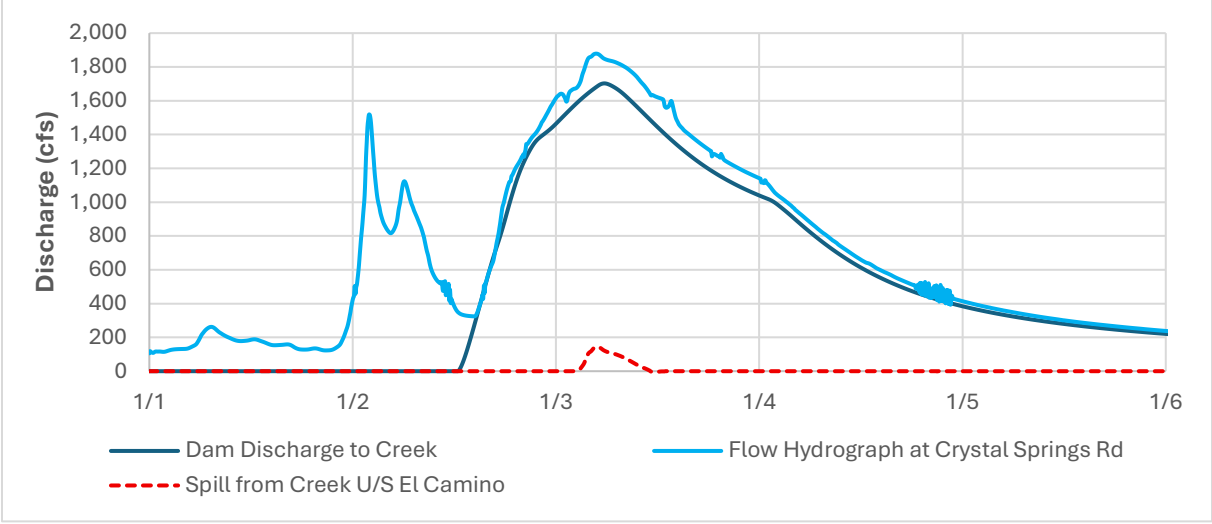


Figure 6-18: 100-year Storm San Mateo Creek Hydrographs

Culverts at Crystal Springs Rd and El Camino Real are not sized sufficiently to pass 100-year spills from the Crystal Springs Dam system, based on historical operations and previously modeled spillway discharge characteristics. The HEC-RAS model includes detailed lateral structures that spill discharge beyond the culvert and creek capacity onto an adjacent 2D surface, routing spill towards Highway 101 and Poplar Pump Station.

Existing condition maximum depth results are shown for 10- and 100-year models in Figure 6-18 and Figure 6-19.

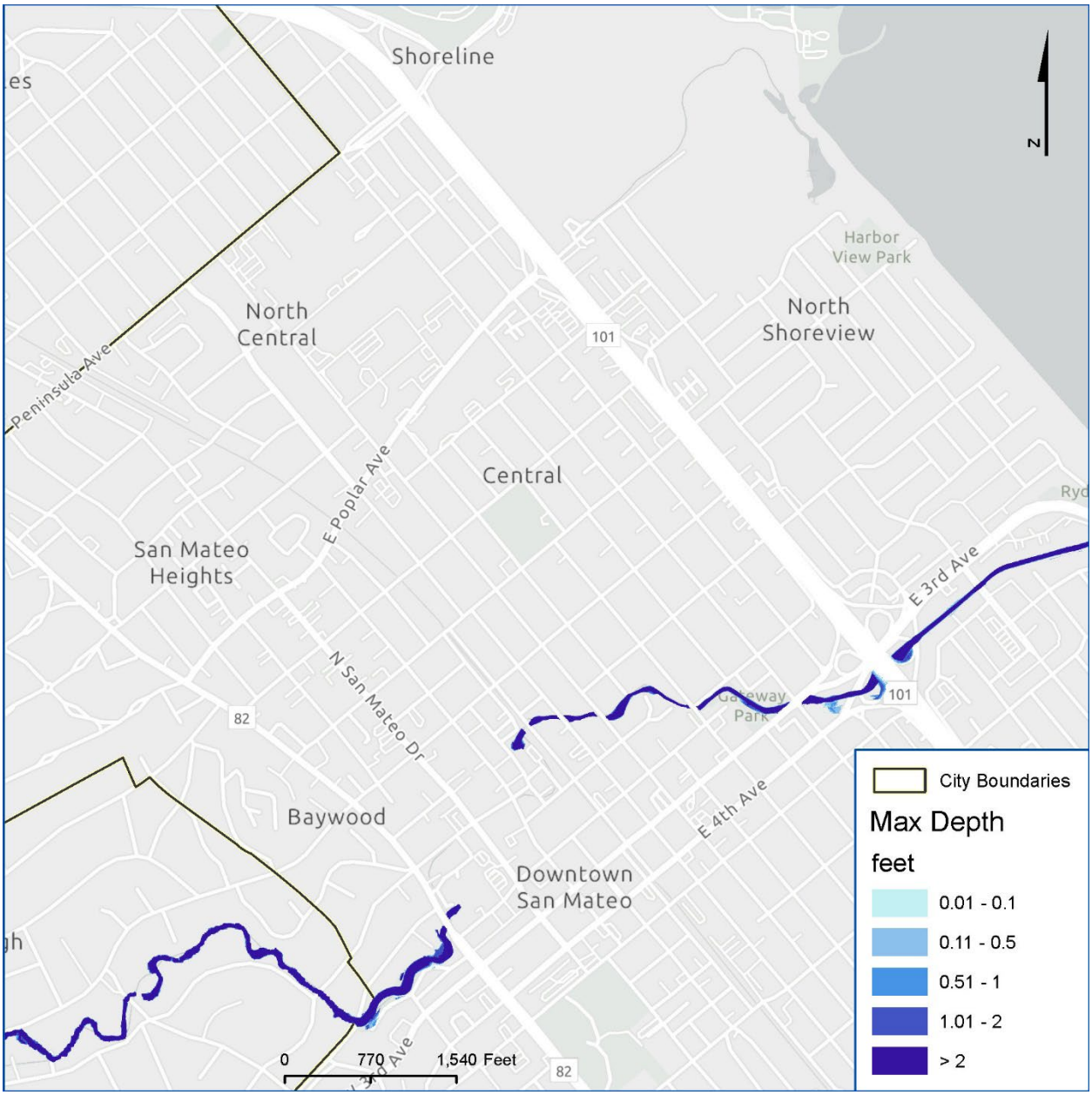


Figure 6-19: San Mateo Creek HEC-RAS Model 10-year Maximum Depth Map

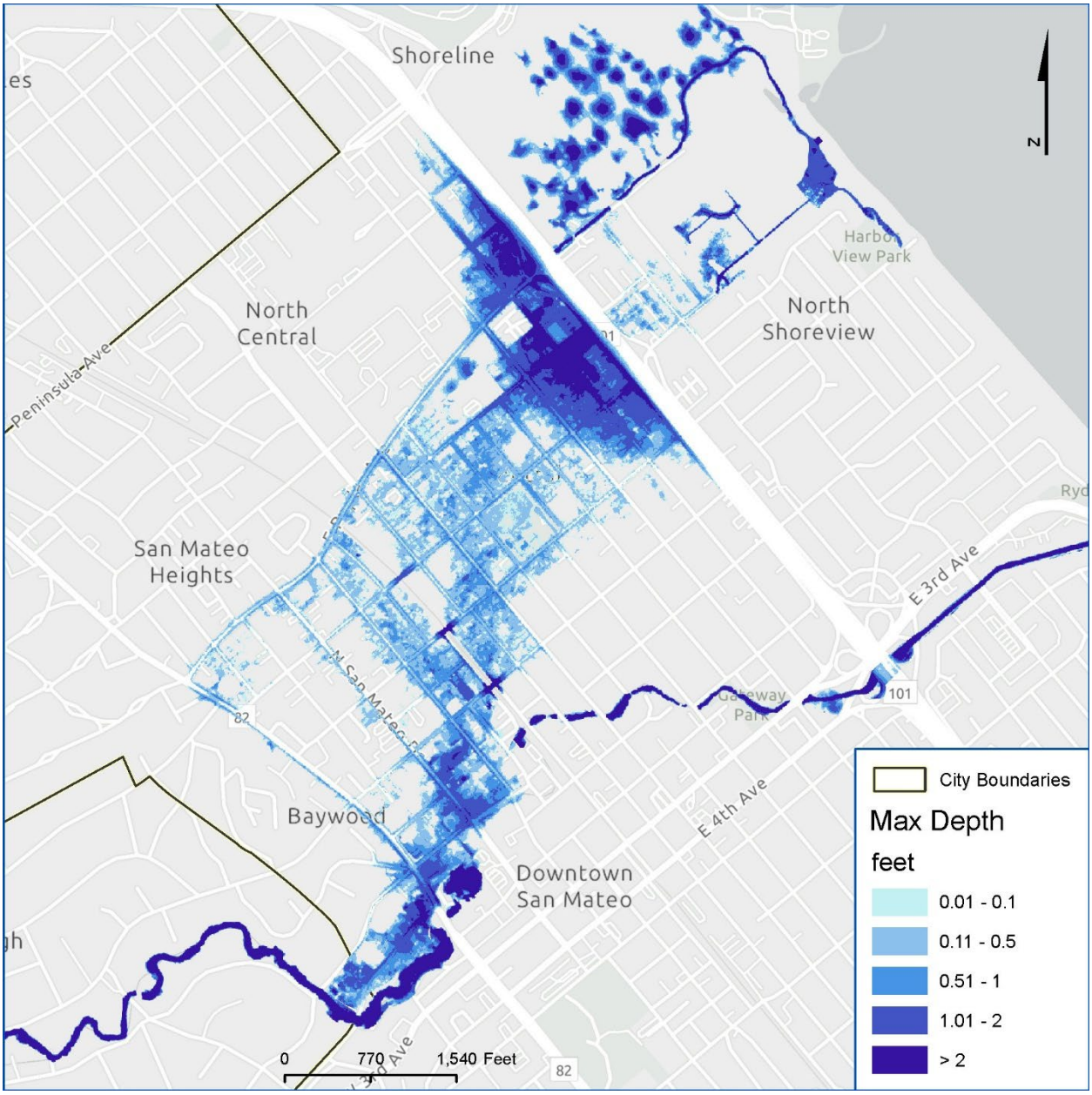


Figure 6-20: San Mateo Creek HEC-RAS Model 100-year Storm Maximum Depth Map

Spill from the channel occurs at the Crystal Spring Rd Crossing and just upstream of the long culvert near El Camino Real. Those culverts are not sized sufficiently to handle spill from Crystal Springs Dam upstream. SFPUC should be consulted to confirm that current operations and outlet parameters still produce a similar flood depth map.

Near the downstream end of San Mateo Creek, a small reach of the left bank, between South Norfolk St and J Hart Clinton Dr lacks a floodwall or levee. The left bank of this reach is classified as high ground, and recent modeling efforts revealed that the left bank is overtopped by a 100-year tide. A photo of this segment during a December 2025 king tide event is provided in Figure 6-20.



Figure 6-21: San Mateo Creek near J Hart Clinton Dr During the December 2025 King Tides (Tide Level is About 9 ft NAVD)

6.2.4.2 16th and 19th Ave Channels

The 16th Ave and 19th Ave Channels collect runoff from 2.4 and 3.1 square mile drainage areas, respectively. 10- and 100-year model results are shown in Figure 6-21 and Figure 6-22.

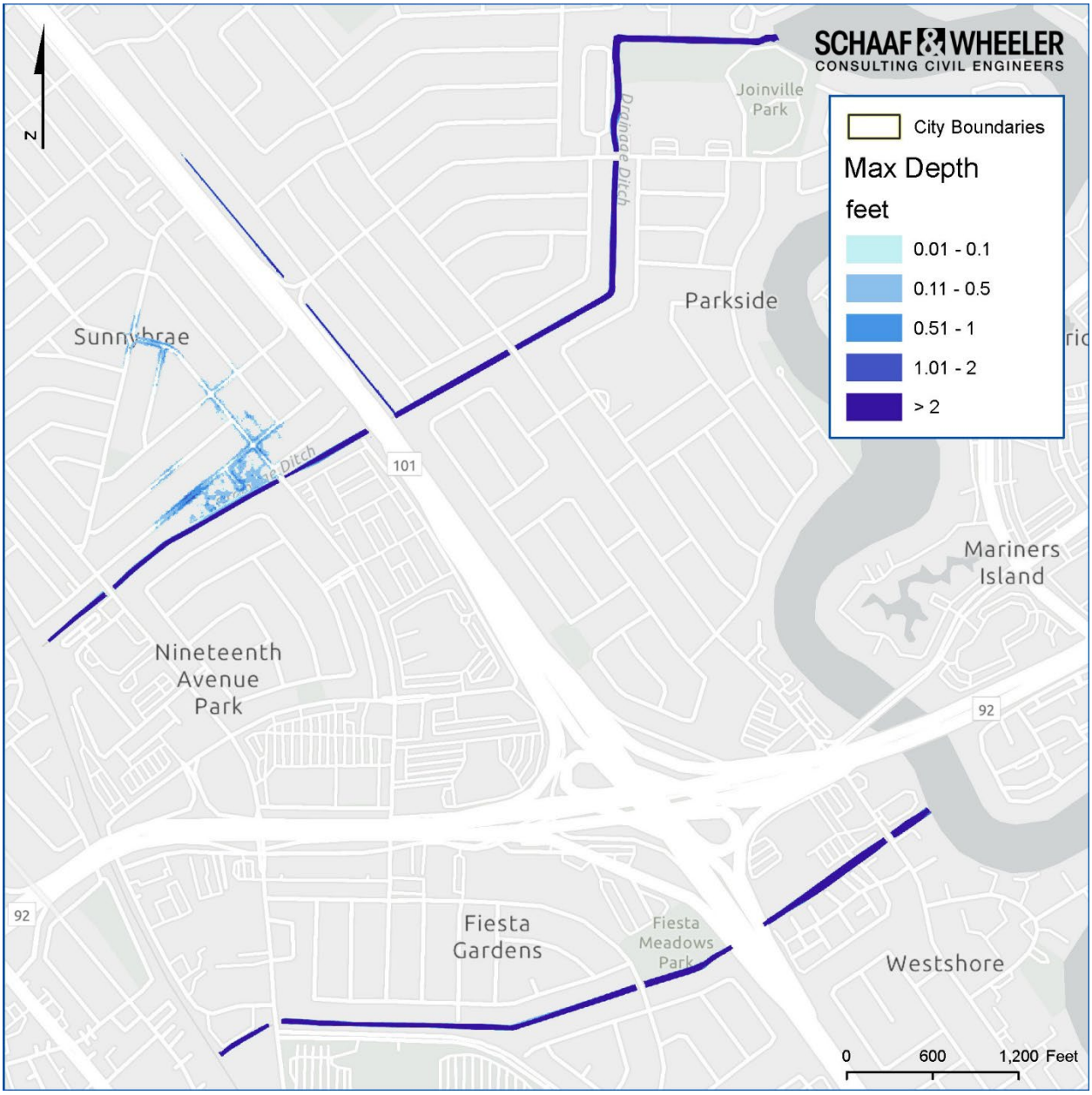


Figure 6-22: 16th and 19th Channel HEC-RAS Model 10-year Maximum Depth Result

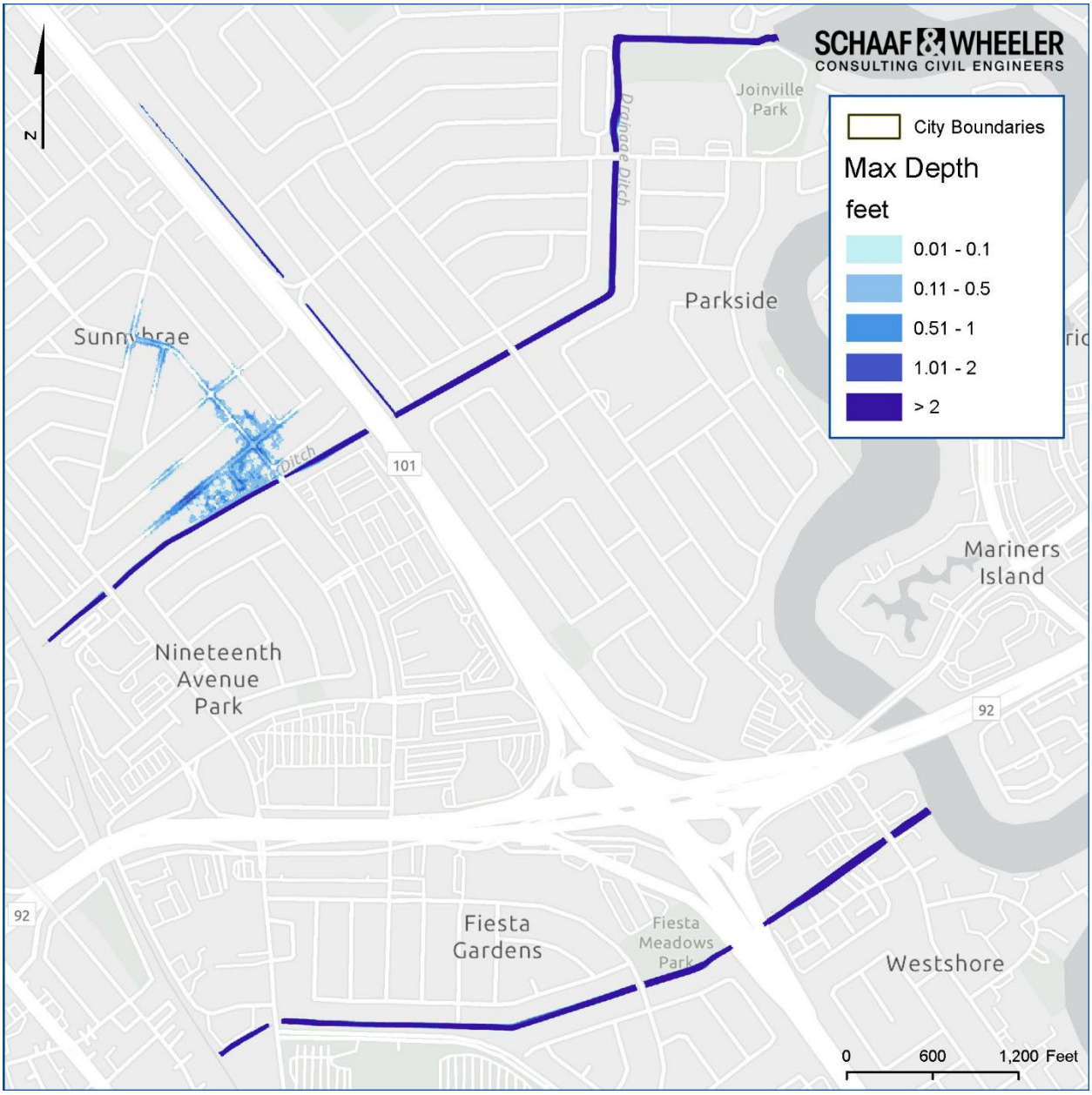


Figure 6-23: 16th and 19th Channel HEC-RAS Model 100-year Maximum Depth Result

The primary issue that appears in these channels is the size of the 16th Ave Channel culvert crossing at Highway 101. The maximum water surface elevation profile in that channel spills the left bank upstream of Grant St and backs up into the connected pipe systems up to Sunnybrae Blvd, where inlet grate and street gutter elevations are lower than the channel's water surface. This is true of both the 10- and 100-year storm events. Flooding was observed in this area during the New Year's Eve 2022 event as well.

A model of the New Year's Eve 2022 event shows that the volume and extent of flooding during that storm exceeded the modeled 100-year design storm flooding. That flooding was primarily the result of discharge exceeding the capacity of the existing 5'×10' Highway 101 crossing for an extended period of time compared with the shorter-lived peak of a design storm event model.

The modeled New Year's Eve 2022 storm flooding is shown in Figure 6-23. While peak discharge in the channel during the storm did not reach a 100-year peak, the volume spilled from the channel was greater due to the undersized culvert at Highway 101.

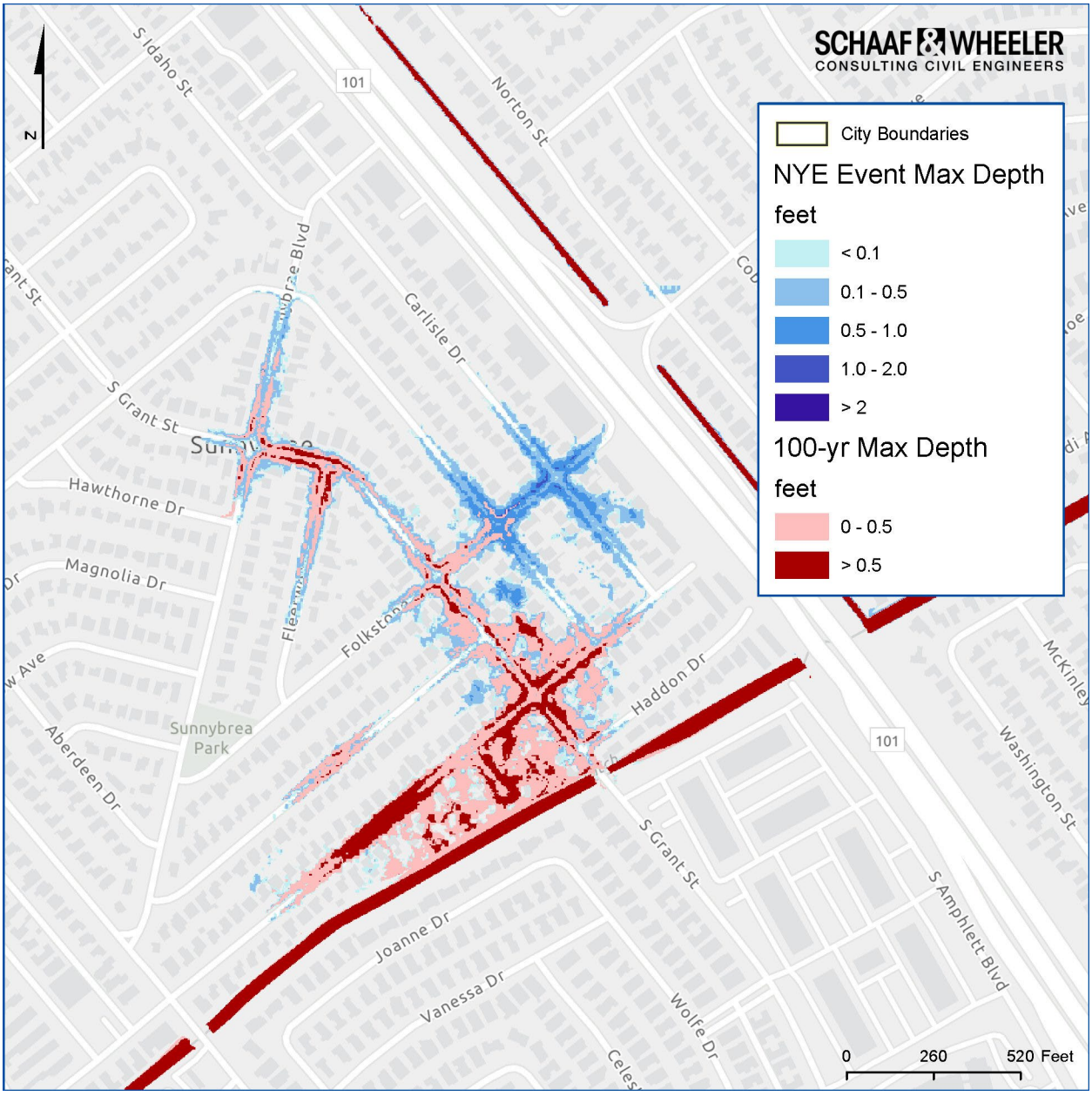


Figure 6-24: 16th and 19th Channel HEC-RAS Model New Year's Eve 2022 Storm Maximum Depth Result, Compared with 100-year Maximum Depths

6.2.4.3 Laurel Creek

Laurel Creek collects runoff from 4.24 square miles and discharges into the southern end of Marina Lagoon. The upstream drainage areas are impounded by two dams (shown in Figure 6-25)—one of which discharges to a small tributary, East Laurel Creek—and through a four-foot diameter culvert into the main channel north of Laurelwood Dr. Two models have been developed, one to evaluate the main Laurel Creek channel capacity, and another to examine East Laurel Creek hydraulics. Results from the models are shown in Figure 6-24 and Figure 6-25.

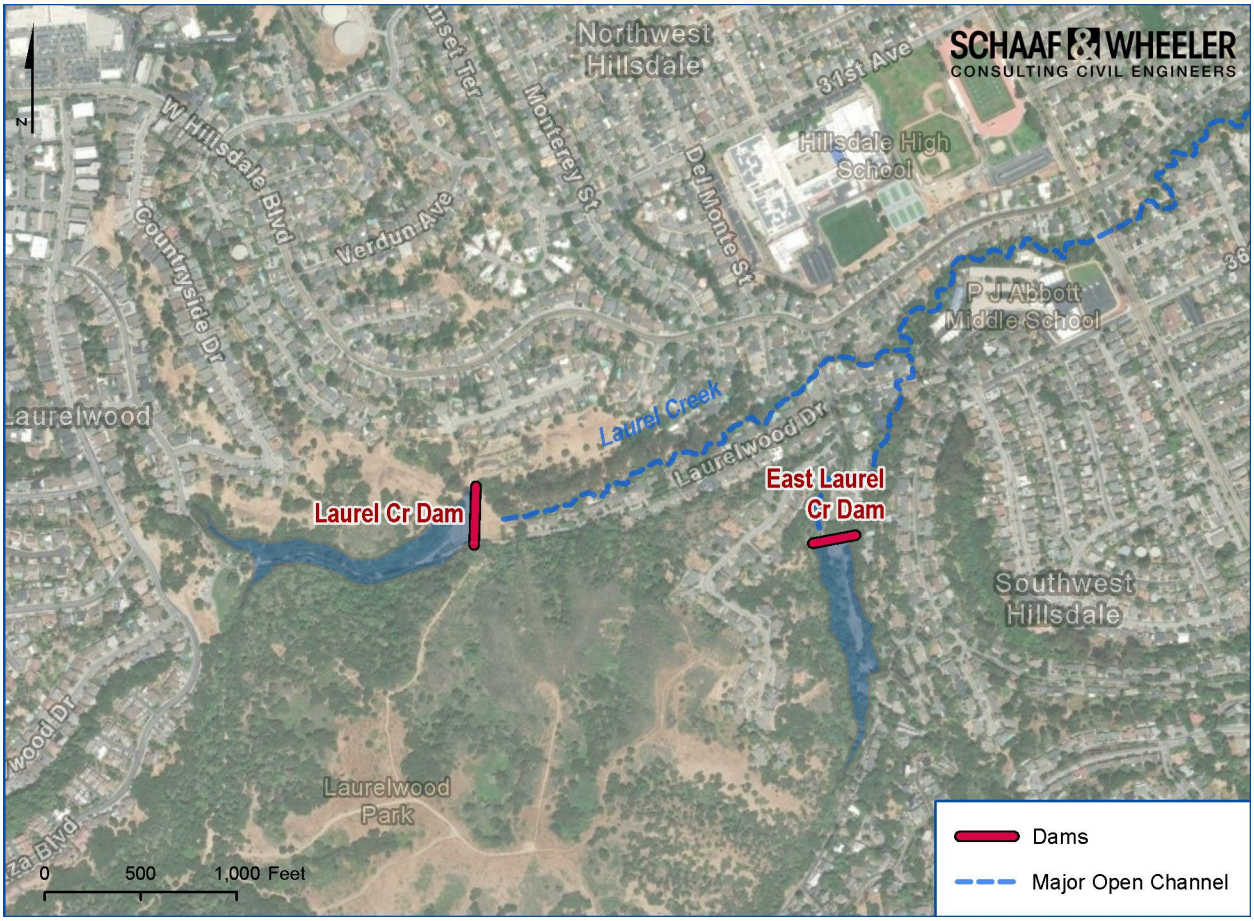


Figure 6-25: Laurel Creek and East Laurel Creek Dam Locations

The main channel appears to have sufficient capacity to convey 10- and 100-year runoff reaching the creek from the outlet of west Laurel Creek Dam to El Camino Real. The 100-year storm begins to spill at the upstream end of the El Camino Real culvert, flooding the railroad underpass to the west. The pump station is included in the models and discharges spilled waters back into the channel downstream of the spill. During 10- and 100-year storms, the model shows that spill occurs at Curtiss St crossing and the George Hall Elementary School culverts. Storm drain systems and Casanova Pump Station are also included in the model to evaluate their effectiveness at reducing flooding from creek spills.

East Laurel Creek primarily handles discharge from East Laurel Creek dam, which enters the creek through a 30-inch discharge pipe or over the dam spillway. There is a very small local

drainage area to the channel, but it does not shed significant runoff to the channel compared with dam outlet discharges from a 0.96 square mile drainage area.

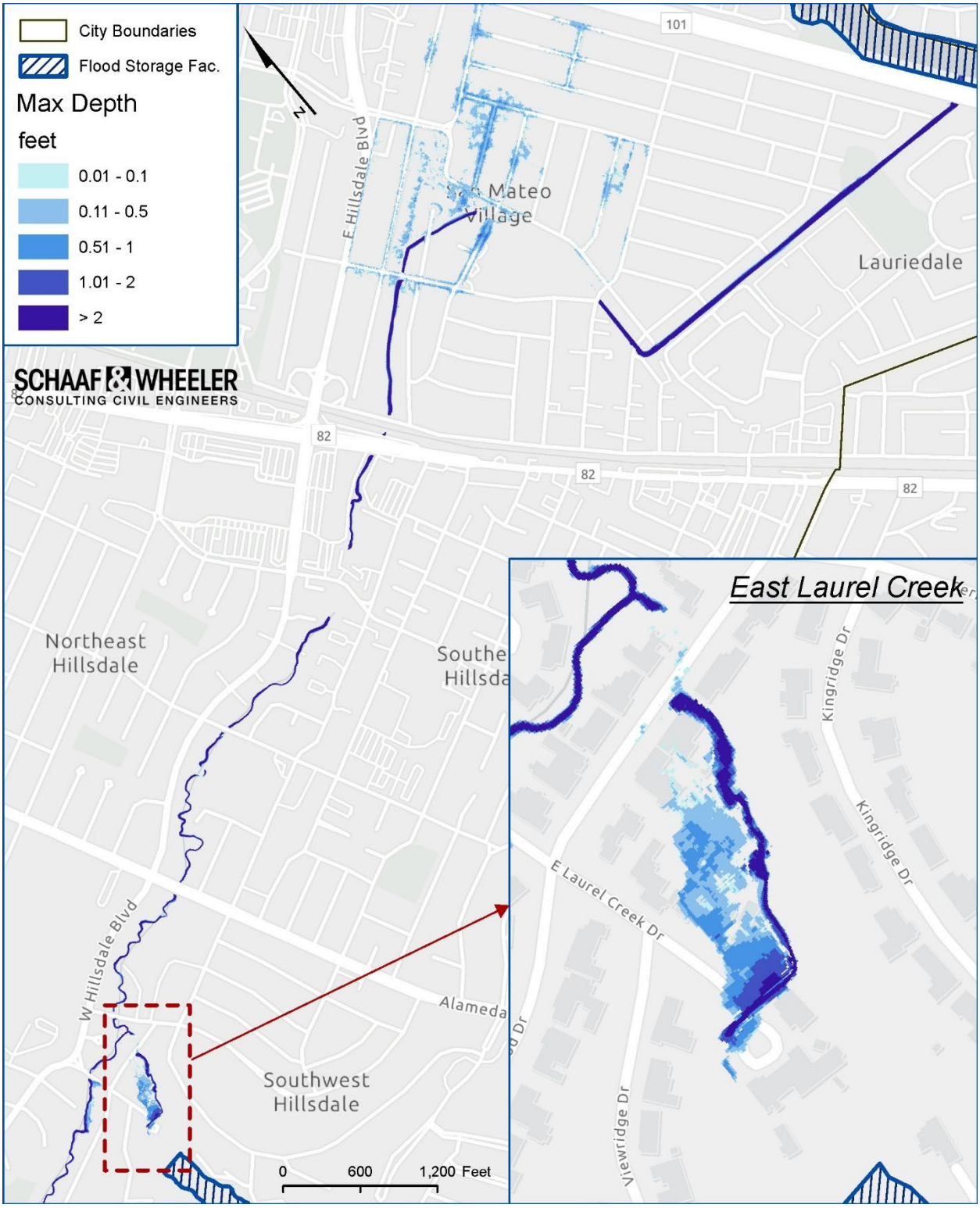


Figure 6-26: Laurel and East Laurel Creek HEC-RAS Model 10-year Maximum Depth Result

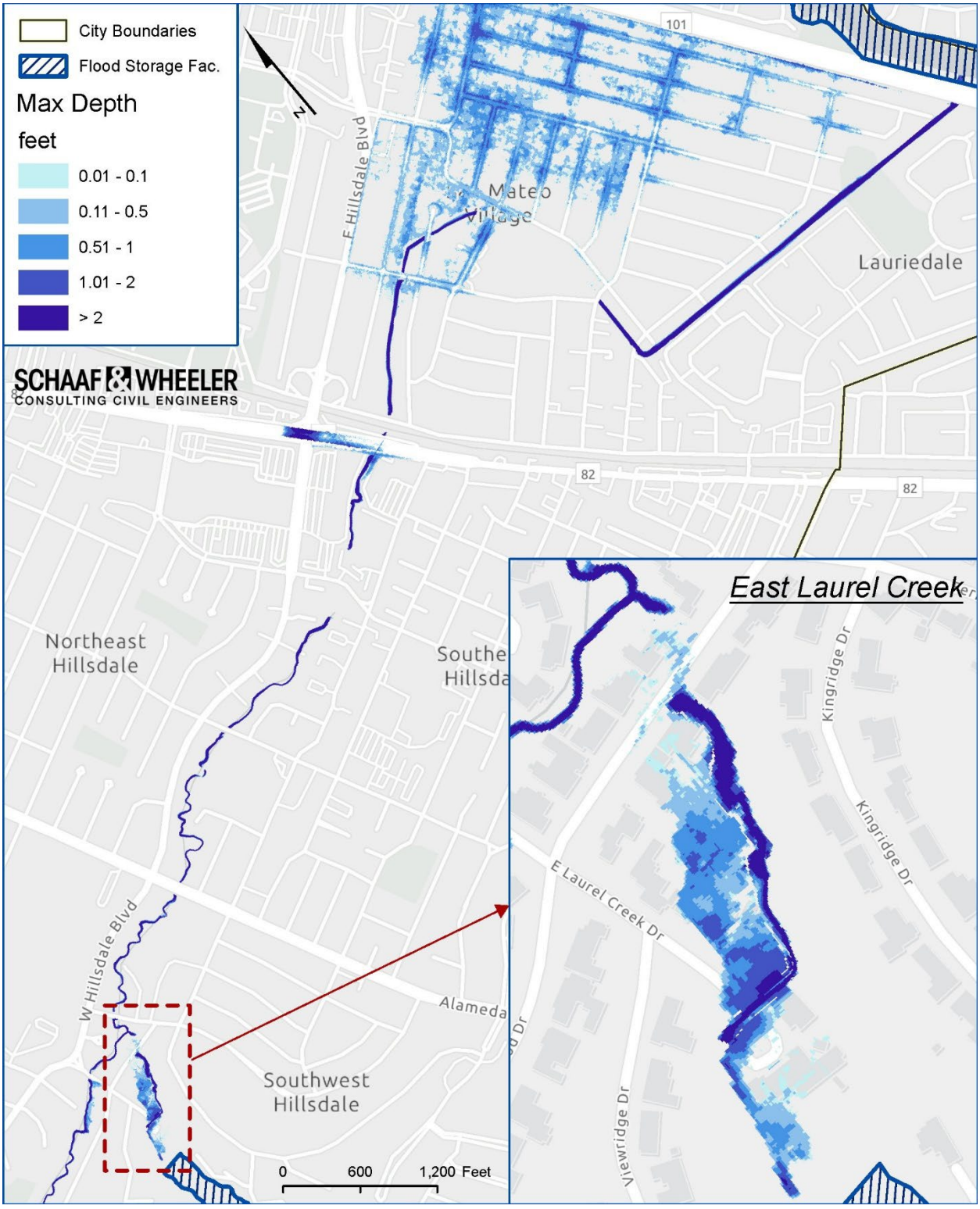


Figure 6-27: Laurel and East Laurel Creek HEC-RAS Model 100-year Maximum Depth Result

The model of East Laurel Creek and the 48-inch culvert that crosses Laurelwood Dr shows that the existing channel does not provide sufficient capacity to convey a 10-year peak discharge

from the dam upstream. However, it does not show significant surcharge onto Laurelwood Dr. On a number of occasions, residents have informed City staff that the culvert surcharges. Modeling indicates that the pipe capacity is sufficient to handle a 10-year peak discharge from the upstream dam's outlet works and spillway without issue. Other factors must therefore be considered as potential causes of recurring issues. The upstream end of the culvert consists of a large, metal debris rack, then an extremely sharp bend to the right at the headwall (as seen in Figure 6-26).



Figure 6-28: Existing 48-inch Diameter Culvert Inlet on East Laurel Creek at Laurelwood Dr.

Vegetation Management

Baseline models assume that the vegetation and sediment in the channel are managed to maintain conveyance. However, City staff have noted a particular reach of the Creek between 39th Ave and 40th Ave where vegetation is dense and has the potential to impact conveyance capacity (shown in Figure 6-27).



Figure 6-29: Vegetation Conditions at Laurel Creek Between 39th Ave and 40th Ave

To evaluate the impact of dense vegetation on Laurel Creek's capacity, the Manning's n roughness value has been raised in the model from 0.03 to 0.075 for near 39th Ave and to

0.1 for near 40th Ave. These roughness values better represent dense wetland vegetation. During a 100-year event, the George Hall Culvert pressurizes, and spill from the creek's north bank flows towards the area drained by the Casanova Pump Station. This is also the point most impacted by dense vegetation downstream. Spill rates and volumes nearly double at the upstream end of the culvert (Figure 6-28).

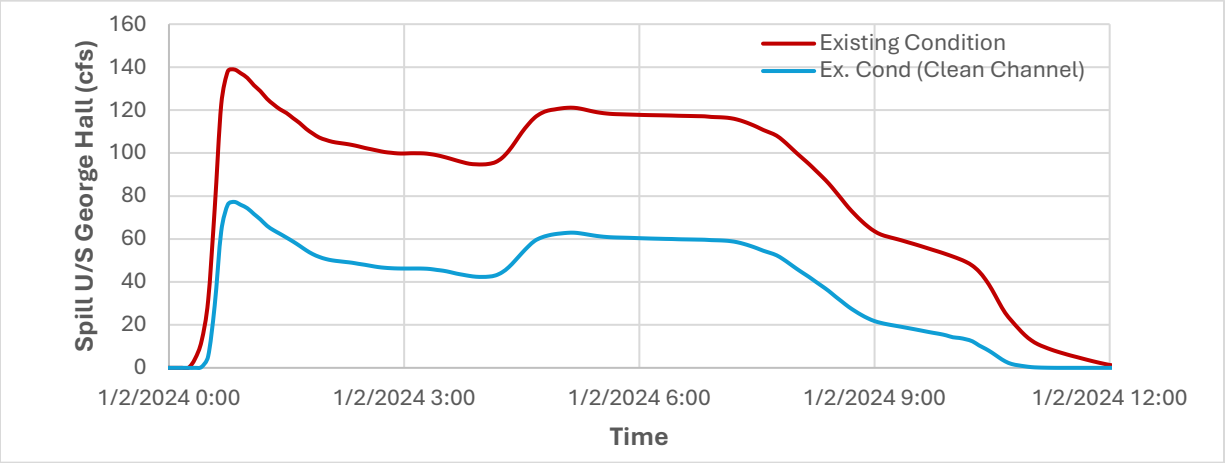


Figure 6-30: 100-year Spill Hydrographs Upstream of George Hall Elementary

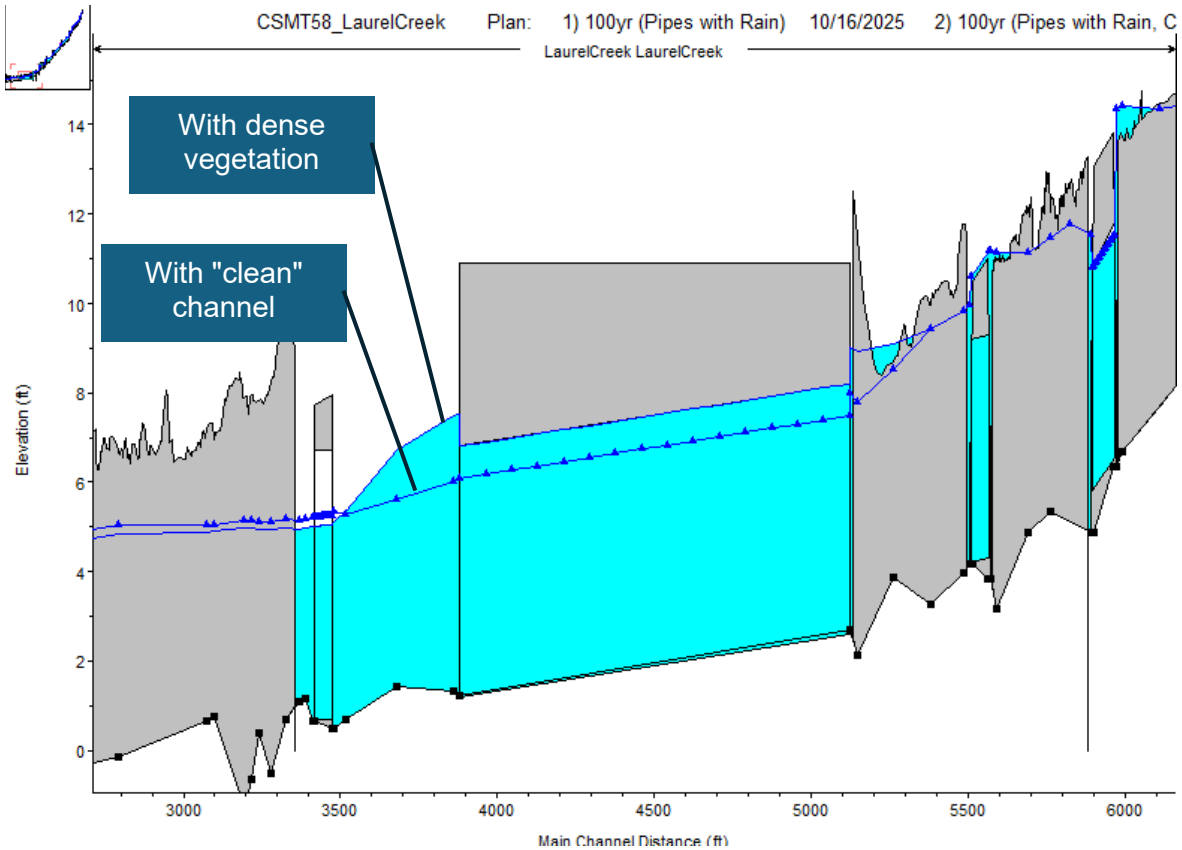


Figure 6-31: Laurel Creek 100-year Water Surface Profiles from HEC-RAS

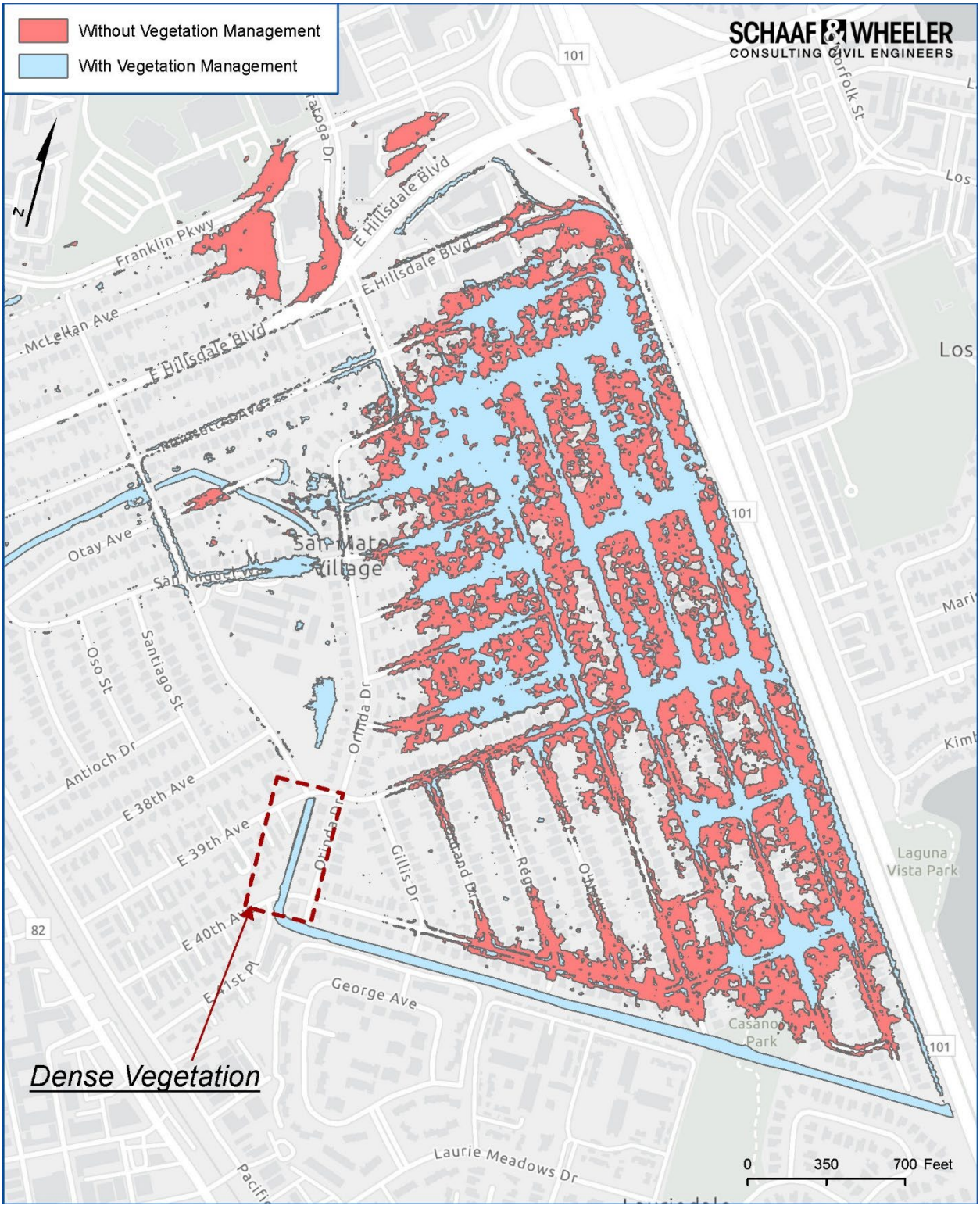


Figure 6-32: Laurel Creek Floodplain Vegetation Impacts—Areas with Greater Than Six Inches of Flooding During a 100-year Design Storm)

6.2.5 Flood Control Facilities

Marina Lagoon is the largest flood control facility in the City of San Mateo, capturing drainage from approximately 11.3 square miles of area in the City and neighboring jurisdictions, or about 75% of the SDMP drainage area. The water level in the lagoon is managed by the O'Neill Slough Tide Gate at the southern end and an existing pump station at the northern end, discharging to San Francisco Bay beneath J Hart Clinton Dr.

The lagoon is a mapped FEMA floodplain, with a base flood elevation (BFE) of 2.5 feet NAVD. However, the lagoon also functions as a recreational amenity for the City, with higher pump set levels during the summer months. While the lagoon has a total storage volume of approximately 946 acre-feet, its active flood control volume is characterized as the 490 acre-feet between elevation -0.4 feet NAVD and the BFE.

The City is responsible for maintaining the lagoon's flood storage capacity and must monitor sediment accumulation. A dredging assessment was completed in 2018 by Moffatt & Nichol that characterized sediment grain sizes in the lagoon and compared bathymetric characteristics at the time of the study with the 1965 construction drawings.

The study also estimated that the sedimentation rate in the lagoon is approximately 5,410 cubic yards per year, with the greatest amount of accretion occurring in the southern part of the lagoon where flow enters the facility, velocities decrease, and suspended sediments are able to settle. The report observed differences from a 1996 survey for elevation 95 ft CSM and 97 ft CSM, representing winter and summer operating levels. For this study, updated survey is available to compare to the 1996 bathymetry. Available storage volume in the lagoon and changes thereto are summarized in Table 6-6.

Table 6-6: Marina Lagoon Storage Volume

	Volume at 95 ft CSM (acre-ft)	Volume at 97 ft CSM (acre-ft)	Flood Storage (acre-ft)
1996	638	1,022	384
2024	505	870	365
Change	**-133**	**-152**	**-19**

This would indicate that the majority of storage volume has been lost beneath the winter flood management operating levels to within the dead storage pool: below elevation 95 ft CSM. Only about 12.5% of sediment accumulated over 28 years has impacted active flood storage in the facility. This would amount to a reduction of about 675 cubic yards per year, or 0.5 acre-feet per year of the 365 acre-feet of available active flood storage volume. Assuming that the accretion rate and distribution remains constant, this would result in a loss of about 0.1% of active flood storage over a 10-year period. However, it is not likely that the accretion rate within the active flood storage range remains constant. As the bottom fills in, the rate of accretion at higher elevations may accelerate.

Additionally, since it has been observed that the southern portion of the lagoon has been impacted disproportionately by sediment accumulation, dredging work should be heavily considered to ensure that the capacity of Laurel Creek is maintained and flood storage volume is maintained to manage runoff from large storm events.

7. Capital Improvement Program

7.1 Overview

Chapter 6 discusses the City's storm drainage system condition, capacity, and known and modeled deficiencies. It further lays out a strategy for addressing various issues and improving overall system performance and level of service to the community.

This chapter provides an analysis of the cost of proposed projects, system improvements, and maintenance.

7.2 Cost Basis

Costs have been estimated based on a variety of available information, including the following:

- Cost estimation guides (e.g. RSMeans)
- Inflation indices, published by the Engineering News-Record (ENR)
- Actual cost and bid data from recent projects
- Engineering judgment

The ENR Construction Cost Index (CCI) for San Francisco as of January 2025 is approximately 15,500, compared with a 20-city average of 13,170. Schaaf & Wheeler performed a detailed unit cost analysis based on RSMeans data for storm drain pipe and structures. This information has been used with adjustment based on the ENR CCI to 2025 (Table 7-1). In some cases, elliptical and box sections may be required. Costs estimated for those sections are presented in Table 7-2. Available recent project bid data and Caltrans unit cost analysis has been used to verify the accuracy of the adjusted unit cost values.

Table 7-1: Storm Drain Conveyance Conduit Unit Costs in 2025 Dollars (Per Linear Foot)

Item	New Conduit Unit Cost	Removal/Disposal Unit Cost
12" Pipe	$210	$30
15" Pipe	$240	$35
18" Pipe	$260	$40
21" Pipe	$290	$45
24" Pipe	$320	$50
27" Pipe	$370	$55
30" Pipe	$410	$60
36" Pipe	$490	$70
42" Pipe	$570	$75
48" Pipe	$670	$80
54" Pipe	$780	$85
60" Pipe	$880	$95
66" Pipe	$1,010	$100

Item	New Conduit Unit Cost	Removal/Disposal Unit Cost
72" Pipe	$1,190	$120
84" Pipe	$1,670	$150
96" Pipe	$2,010	$200
120" Pipe	$2,510	$225
132" Pipe	$2,760	$250
144" Pipe	$3,010	$300
66" Pipe	$1,010	$100
72" Pipe	$1,190	$120
84" Pipe	$1,670	$150
96" Pipe	$2,010	$200
120" Pipe	$2,510	$225
132" Pipe	$2,760	$250
144" Pipe	$3,010	$300

Table 7-2: Storm Drain Elliptical and Box Section Unit Costs in 2025 Dollars (Per Linear Foot)

Item	New Conveyance Unit Cost
14"×23" Elliptical	$390
19"×30" Elliptical	$480
24"×38" Elliptical	$615
12"×24" Box	$1,140
24"×36" Box	$1,870
24"×48" Box	$2,020
24"×60" Box	$2,410
30"×60" Box	$2,770
36"×60" Box	$3,130
36"×72" Box	$3,640
48"×72" Box	$4,940
48"×96" Box	$5,250
60"×96" Box	$6,200
72"×96" Box	$7,150
96"×96" Box	$9,050
96"×120" Box	$10,300

Unit costs have also been estimated for storm drain structures, including the connection of new and existing pipe. These are summarized in Table 7-3. Repair and maintenance costs are calculated on a different basis than pipe replacement and new pipe costs. These are summarized in Table 7-4.

Table 7-3: Storm Drain Structure Unit Costs in 2025 Dollars

Connecting Conduit Width	Unit Cost
30"	$17,500
36"	$17,700
42"	$18,300
48"	$18,600
54"	$20,200
60"	$20,600
66"	$22,400
72"	$22,900
84"	$26,100
96"	$28,700
120"	$33,900
132"	$36,500
144"	$39,000

Table 7-4: Storm Drain Repair Costs

Item	Spot Repair (Each)	CIPP (Cost per LF)
8" Pipe	$9,000	$60
10" Pipe	$10,000	$65
12" Pipe	$11,000	$70
15" Pipe	$12,000	$85
18" Pipe	$13,000	$100
21" Pipe	$14,000	$115
24" Pipe	$15,000	$130
30" Pipe	$17,000	$160
36" Pipe	$19,000	$190
42" Pipe	$21,000	$220
48" Pipe	$23,000	$250
54" Pipe	$25,000	$280
60" Pipe	$27,000	$310

Conveyance capacity and repair project costs include 30% contingency, 15% mobilization and traffic control, and 20% design/engineering and administration cost. Environmental permitting is estimated as 20%–25% of construction cost, where it is anticipated to be a probable requirement beyond a CEQA negative declaration. Other special costs, including special construction methods, property or easement acquisition, or utility relocation are considered on a case-by-case basis, where such costs are anticipated to be predictably significant.

Pump station costs include 40% contingency for all projects, given their greater degree of complexity. Some pump station projects are limited to condition assessment and have no further markup. Those that require construction also include a 35% markup on construction costs to cover design/engineering, administration, and permitting.

Costs for recommended maintenance assume that the City's staff are capable of recommended debris removal and cleaning, and that pipes will be re-inspected once cleaned to determine repair needs. Costs of maintenance projects are defined by a cost per linear foot determined by the pipe's NASSCO rating, assuming that a higher rating means that more effort is required to clean the pipes and dispose of removed materials (Table 7-5). O&M project costs identified in this document also include a fixed $1,000 component to account for crew time and equipment mobilization and a 20% contingency to account for unexpected conditions where CCTV was not able to capture the condition of the entire pipe.

All costs presented in this chapter include contingency and other cost factors beyond construction subtotals.

Table 7-5: Storm Drain Maintenance Project Cost

O&M NASSCO Rating	Maintenance Cost (per LF)
5	$100
3 to 4.9	$80
<3	$50

7.3 Project Prioritization

Projects have been assigned one of four priorities: Low, Medium, High, or Very High. Factors considered when assigning priorities to gravity pipe system capacity projects are summarized in Table 7-6. A modified set of scoring criteria were used to rank creek capacity improvements, as shown in Table 7-7.

Condition-related project priorities are defined by NASSCO ratings applied to each pipe, the extent of the deficiency, and overlap with known, documented drainage issues. In general, projects have been defined to bundle pipes adjacent to one another or in close proximity. In some cases, individual pipes under the same project are assigned different ratings, and in those cases, a maximum and an average score weighted by pipe length was estimated to support prioritization. The Bettina-Kingridge project was elevated to Very High priority, as its condition is the likely cause of documented drainage issues.

In theory, there is no maximum score achievable for gravity system improvements, creek improvements, and pump station improvements because certain scoring elements are based on an uncapped quantity (e.g. number of projects upstream, number of properties with reduced flood risk, etc). However, the maximum score awarded to any project in each category is listed in the tables.

Once all recommended projects were evaluated, scores were normalized to a scale of 0 to 100 based on the number of structures impacted by a 10-year storm event in each category (creeks, pump stations, and gravity systems) and life safety risk factors. This forms a unified and ranked CIP that includes all recommended stormwater capital improvements. Scores vary between 0 and 88, with the highest scoring projects providing capacity for a number of upstream projects, improving facilities that have major impacts.

Table 7-6: Project Scoring and Prioritization Factors for Gravity Pipe Systems

Factor	Description	Categories/Method	Points	Max Score Awarded
10-year Flooding Depth	Tiered based on modeled depths	0.1-0.5 ft 0.5-1.0 ft > 1.0 ft	5 10 20	20
2-year Capacity	Model shows ponding >0.1 ft during a 2-year storm event	No Ponding Ponding > 0.1 ft	0 5	5
Properties with Reduced Flooding	Awards points for removing flooding from properties	# of parcels	1 per parcel	50
Downstream Project Need	Deprioritizes projects requiring downstream capacity improvements	D/S project required No D/S project required	0 10	10
Alleviates Recurring Issues	Projects aimed at solving known or recurring issues	No alleviation Indirect Direct	0 10 20	20
High Existing O&M Burden	Increases priority of projects that offset frequent, costly maintenance	No Benefit Offsets O&M Burden	0 10	10
Overlap with Condition Deficiency	Prioritizes projects if they overlap with condition deficiencies	NASSCO Rating (1-5)	5x NASSCO Rating	25
Proximity to Other Condition Projects	Prioritizes a project if it can be combined with adjacent repair needs	NASSCO Rating of Adjacent Projects (1-5)	2x NASSCO Rating	10
Overlap with Other City CIP	Overlap with pavement or Sanitary Sewer Improvements	0-10% Overlap 10-50% Overlap 50-100% Overlap	0 5 10	10
Bubble-ups	Increases priority of projects that eliminate bubble-ups	# of bubble-ups connected to mains	2 per bubble-up	10
Upstream Projects	Increases priority of projects with other CIPs upstream	# of Projects	5 per project	30
Equity Priority Area	Awards points to projects serving Community Development Block Grant priority neighborhoods	Not within priority area Within priority area	0 5	5
Multi-jurisdictional	Increases priority of projects not requiring coordination with others (e.g. Caltrans, railroad)	Coordination req'd Not req'd	0 10	10
Project Cost	Modifies project ordering based on project cost relative to maximum CIP cost	Cost > $1,000,000 $250,000-1,000,000 $100,000-250,000 < $100,000	0 2 5 10	10
Other Factors	Other factors on a per-project basis (e.g. safety issues)	N/A	Maximum 10 pts	10

Table 7-7: Project Scoring and Prioritization Factors for Creek Improvements

Factor	Description	Categories/Method	Points	Max Score Awarded
10-year Flooding	Model shows 10-year flooding due to creek capacity limitation	0.1-0.5 ft 0.5-1.0 ft > 1.0 ft	20 40 80	80
Structure Risk (10-year)	Number of Structures Impacted by 10-year Storm	# of parcels	1 per structure	65
100-year Flooding	Model shows 100-year flooding due to creek capacity limitation	0.1-0.5 ft 0.5-1.0 ft > 1.0 ft	10 20 40	80
Structure Risk (100-year)	Number of Structures Impacted by 100-year Storm	# of parcels	1 per structure	145
Upstream Projects	Increases priority of projects with other CIPs upstream	# of Projects	10 per project	130
Equity Priority Area	Awards points to projects serving Community Development Block Grant priority neighborhoods	Not within priority area Within priority area	0 5	20
Multi-jurisdictional	Increases priority of projects not requiring coordination with others (e.g. Caltrans, railroad)	Coordination req'd Not req'd	0 10	10
Project Cost	Modifies project ordering based on project cost relative to maximum CIP cost	Cost > $1,000,000 $250,000-1,000,000 $100,000-250,000 < $100,000	0 20 50 100	50
SLR Susceptibility	Project is exposed to tidal levels and flooding depths and extents are susceptible to SLR	Not susceptible Susceptible	0 20	20
Other Factors	Other factors on a per-project basis (e.g. safety issues)	N/A	Maximum 100 pts	100

7.4 System Capacity Improvements

The development of capital improvement projects focused first and foremost on addressing the most severe and frequently recurring flooding issues identified by City staff and by the models. These are locations where recurring flooding is a known, documented issue, even for small storms, and where the model shows that high intensity rainfall during a 10-year event is likely to cause system surcharge and introduce risk factors during an event (e.g., property damage, extensive and drawn-out disruption to traffic and emergency services, and potential threats to life safety).

The full suite of MIKE+ and HEC-RAS models have also been used to evaluate 100-year flood risk for the entire stormwater system, with a particular focus on creeks and terminal pump stations (Coyote, Poplar, and Marina Lagoon). These facilities generally have greater potential to cause widespread damage when they lack the desired capacity or otherwise experience failures.

Priority is also given to condition deficiencies based on their level of severity. The models are built to evaluate pipe capacity assuming that they are operating in a free and clear condition; that is, pipes are free of blockage or failures that can cause a loss of conveyance capacity, which can lead to more severe flooding or, in extreme cases, sink holes and failure of other infrastructure.

7.4.1 Capacity Requirements

The existing conditions model supports the development of a set of capital improvement projects across the City. These projects aim to address capacity deficiencies by reducing the 2D flooding depth results to a maximum of six inches above the ground surface wherever possible. In some cases, flooding in excess of that desired level of service is caused by systems not owned by the City or remains at isolated low points where private systems likely exist but are not modeled.

Recommended system capacity improvement projects primarily consist of pipe system capacity upgrades. The hydraulic models have been used to evaluate the capacity of the City's pump stations as well and revealed that additional capacity is needed at the 4000 Pacific Pump Station. That requirement is not discussed in detail in this section and is instead acknowledged in the pump station recommendations presented in Chapter 5.

7.4.1.1 Gravity System Capacity Projects

Capacity CIPs is shown in Figure 7-1 through Figure 7-5. Project lengths and costs are summarized in Table 7-8. More detailed maps are provided in Appendix K.

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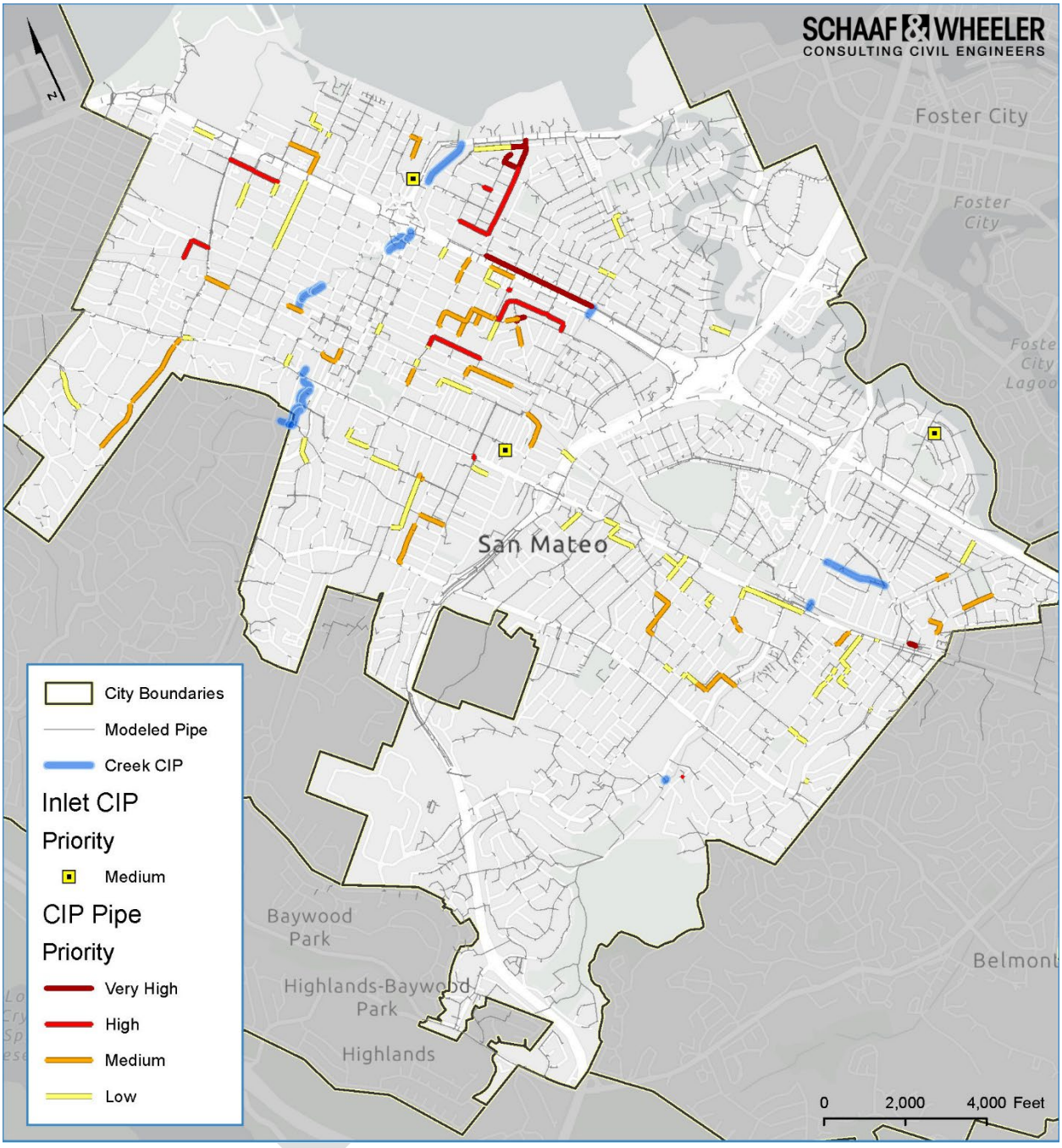


Figure 7-1: Prioritized CIP Project Map (Baseline)

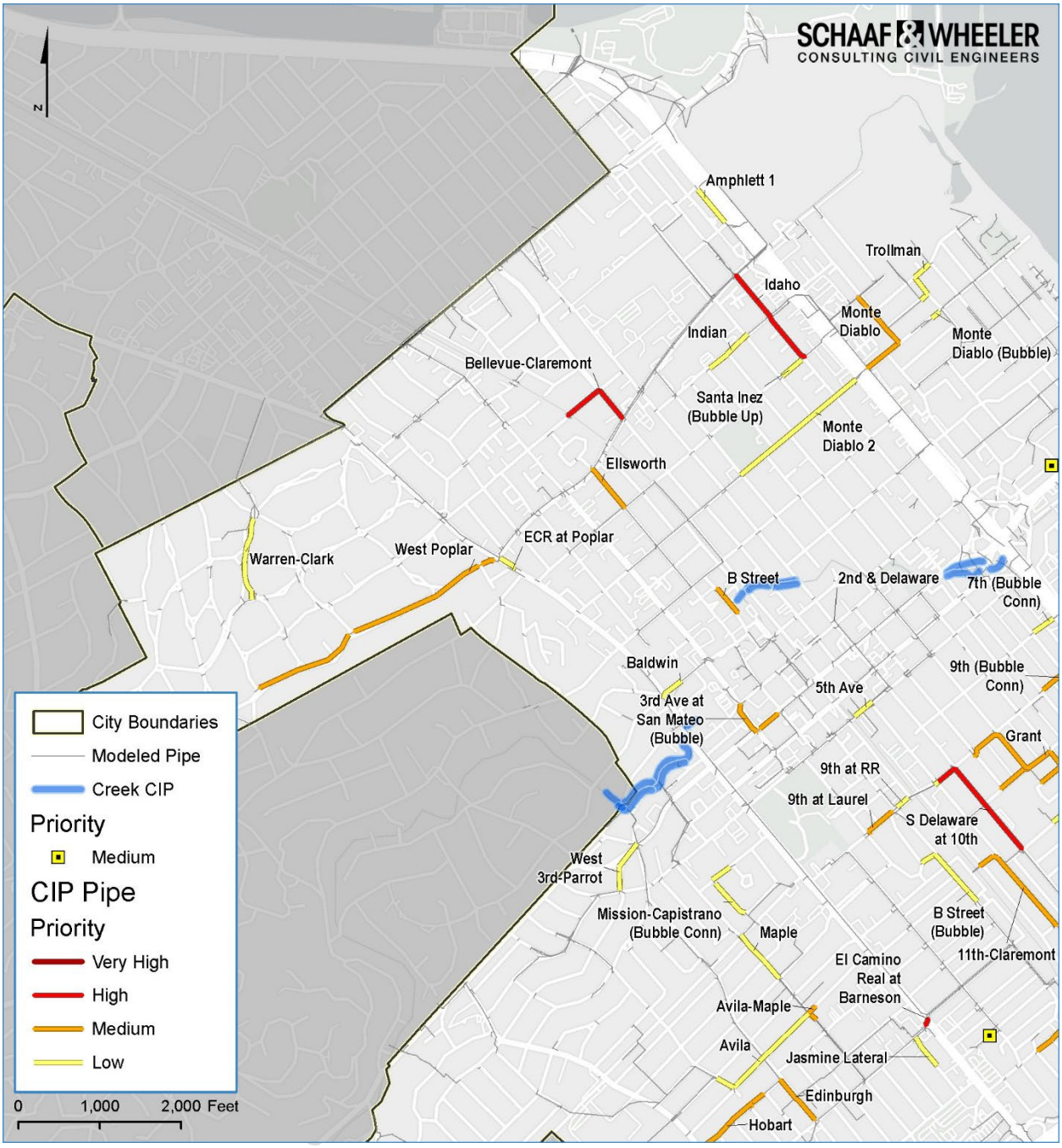


Figure 7-2: Detailed Prioritized CIP Project Map (Northwest Quadrant)

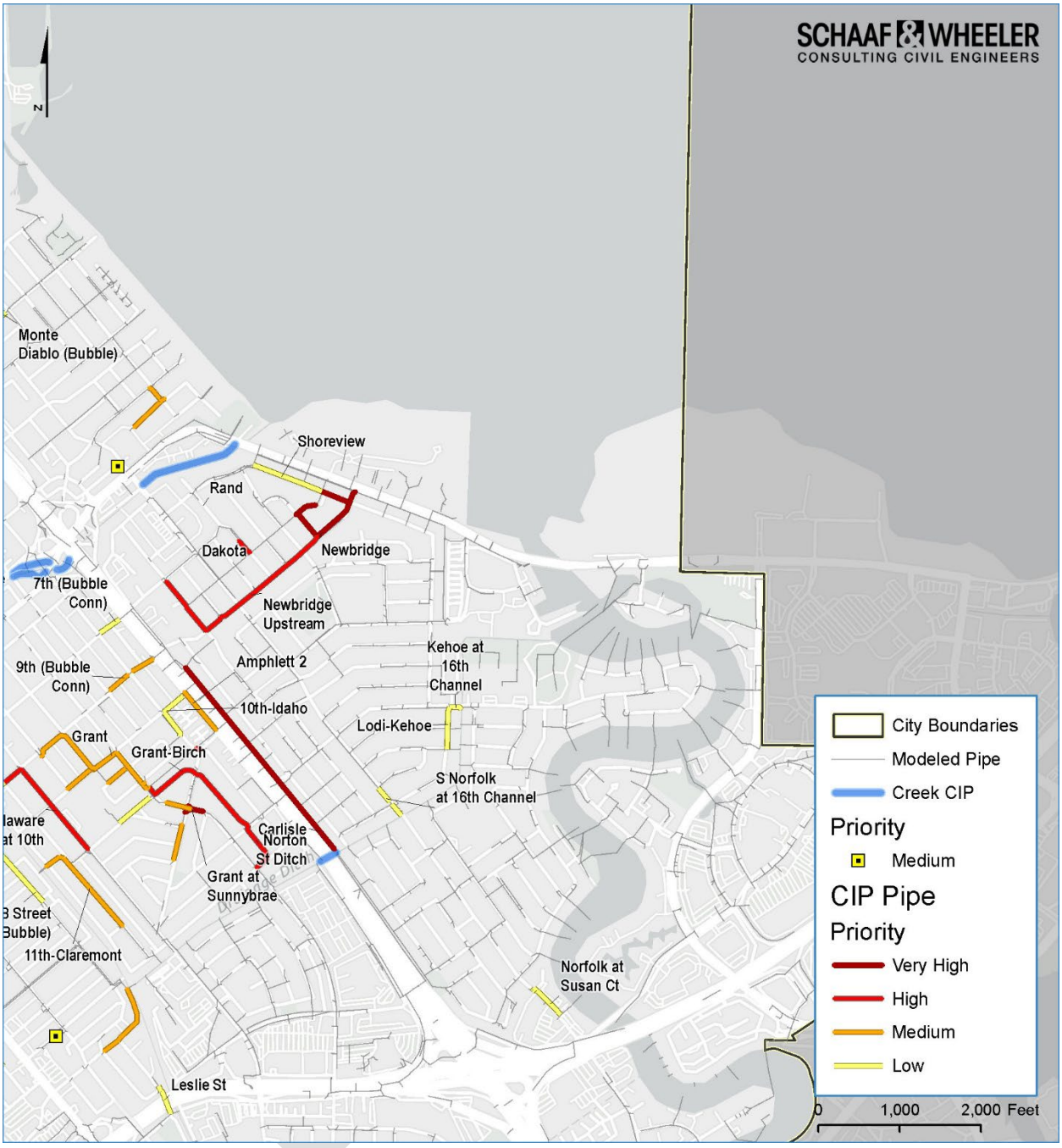


Figure 7-3: Detailed Prioritized CIP Project Map (Northeast Quadrant)

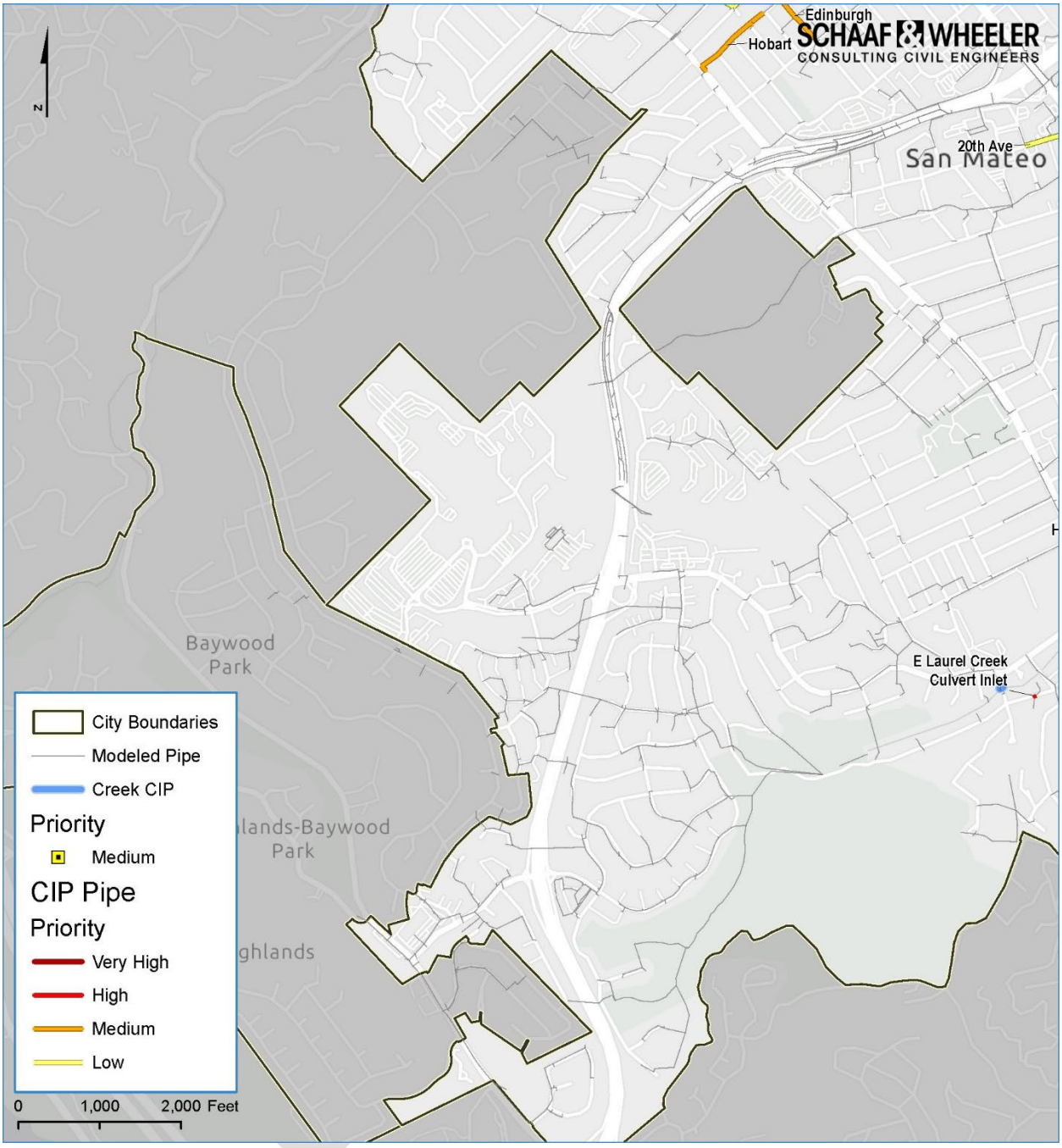


Figure 7-4: Detailed Prioritized CIP Project Map (Southwest Quadrant)

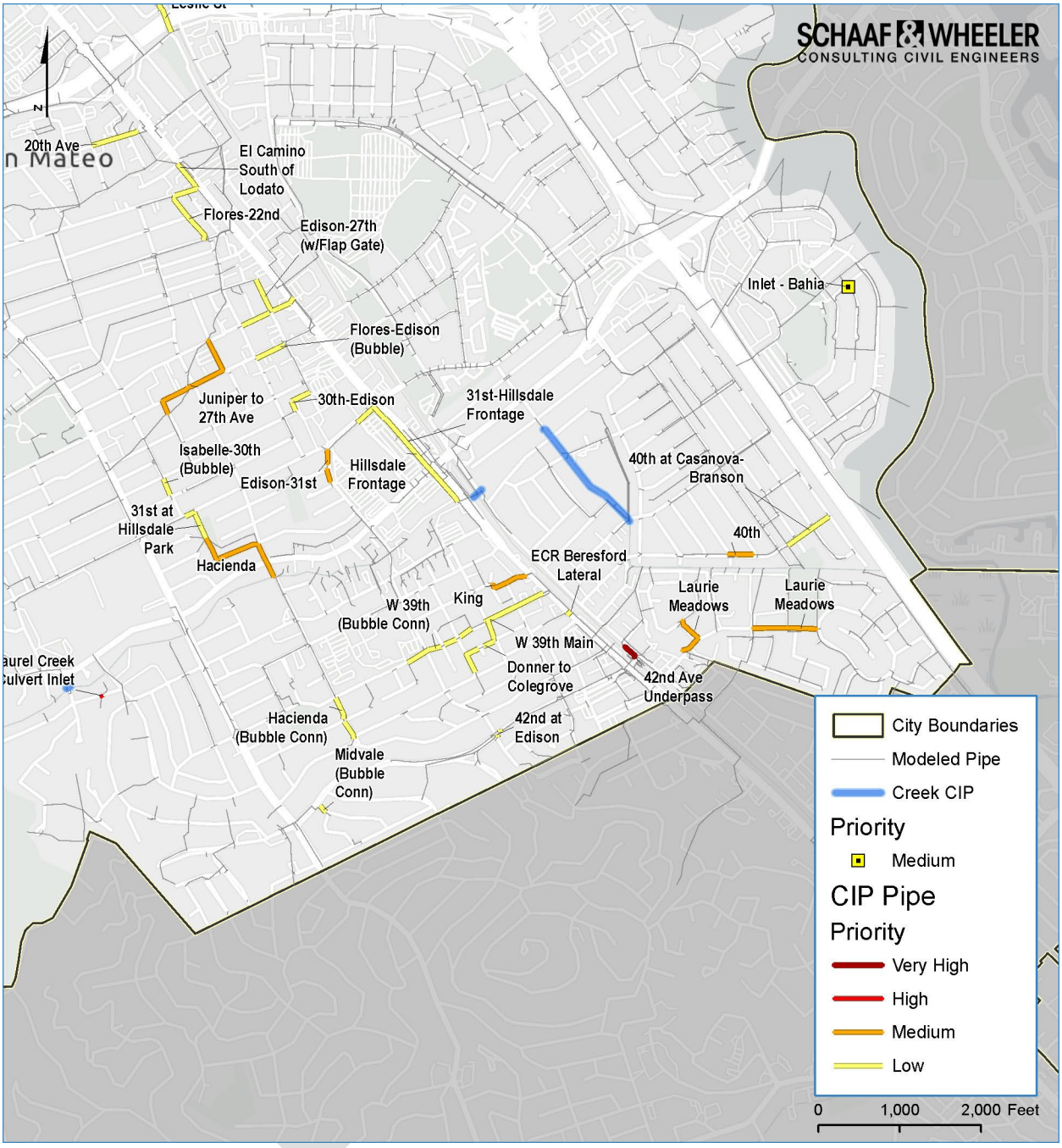


Figure 7-5: Detailed Prioritized CIP Project Map (Southeast Quadrant)

Table 7-8: Storm Drain Capacity Project Cost Estimate Summary

Priority	Project	Length (LF)	Approx. Cost
Very High	Grant St at Sunnybrae Blvd	301	$430,000
Very High	Newbridge Ave	1,772	$2,230,000
Very High	42nd Ave Underpass	195	$230,000
Very High	Norton St Ditch	2,893	$4,800,000
High	Carlisle	1,660	$1,960,000
High	E Laurel Creek Culvert Inlet	10	$550,000
High	Newbridge Ave Upstream	2,196	$2,320,000
High	Bellevue-Claremont	950	$660,000
High	El Camino Real at Barneson	90	$610,000
High	Grant-Birch	746	$800,000
High	Dakota Ave	211	$220,000
High	Idaho	1,327	$940,000
High	S Delaware at 10th	1,551	$1,420,000
Medium	Ellsworth	615	$1,010,000
Medium	Juniper to 27th Ave	1,436	$1,440,000
Medium	Grant St	2,026	$1,170,000
Medium	Amphlett 2	618	$360,000
Medium	11th-Claremont	1,366	$850,000
Medium	40th	310	$220,000
Medium	9th at Laurel	409	$240,000
Medium	West Poplar	777	$400,000
Medium	Edison-31st	330	$190,000
Medium	Quebec-Dolan	747	$390,000
Medium	Edinburgh	649	$760,000
Medium	King	476	$490,000
Medium	17th Ave	977	$690,000
Medium	3rd Ave at San Mateo (Bubble)	695	$360,000
Medium	Hobart	1,115	$910,000
Medium	Hacienda	1,286	$1,210,000
Medium	Avila-Maple	235	$220,000
Medium	B St	395	$240,000
Medium	Laurie Meadows	1,350	$1,240,000
Medium	Monte Diablo	1,251	$1,470,000
Medium	Sunnybrae Blvd	458	$250,000
Medium	B St (Bubble)	772	$570,000
Low	Hillsdale Frontage	1,552	$1,950,000
Low	Jasmine Lateral	440	$340,000
Low	Shoreview	906	$640,000
Low	Birch Bubble	493	$220,000
Low	31st-Hillsdale Frontage	279	$250,000
Low	Maple	715	$520,000

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Priority	Project	Length (LF)	Approx. Cost
Low	Avila	1,530	$1,330,000
Low	9th at RR	236	$230,000
Low	W 39th (Bubble Conn)	798	$480,000
Low	Monte Diablo 2	1,844	$1,360,000
Low	Monte Diablo (Bubble)	124	$80,000
Low	9th (Bubble Conn)	1,571	$850,000
Low	40th at Casanova-Branson	618	$500,000
Low	Mission-Capistrano (Bubble Conn)	791	$400,000
Low	W 39th Main	803	$920,000
Low	Santa Inez (Bubble Up)	309	$150,000
Low	7th (Bubble Conn)	292	$150,000
Low	Isabelle-30th (Bubble)	218	$130,000
Low	42nd at Edison	63	$90,000
Low	Leslie St	381	$270,000
Low	Warren-Clark	1,033	$820,000
Low	Edison-27th (w/Flap Gate)	754	$440,000
Low	31st at Hillsdale Park	474	$370,000
Low	Midvale (Bubble Conn)	103	$70,000
Low	Hacienda (Bubble Conn)	535	$300,000
Low	Trollman	617	$330,000
Low	Norfolk at Susan Ct	504	$380,000
Low	Donner to Colegrove	822	$490,000
Low	S Norfolk at 16th Channel	411	$770,000
Low	West 3rd-Parrot	657	$500,000
Low	ECR at Poplar	251	$230,000
Low	Flores-22nd	981	$1,260,000
Low	Flores-Edison (Bubble)	760	$400,000
Low	20th Ave	594	$230,000
Low	Baldwin	321	$210,000
Low	Kehoe at 16th Channel	158	$170,000
Low	Amphlett 1	527	$270,000
Low	Indian	629	$390,000
Low	5th Ave	284	$250,000
Low	30th-Edison	383	$420,000
Low	Lodi-Kehoe	525	$550,000
Low	El Camino South of Lodato	395	$470,000
Low	10th-Idaho	670	$470,000
Low	ECR Beresford Lateral	69	$120,000
	Total:	**58,526**	**$52,730,000**

Based on documented, recurring, localized issues, a handful of locations were identified where additional inlet capacity would be beneficial, even without significant pipe capacity improvements. These were not scored but were all considered to be medium priority projects.

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These are not projects that reduce flooding with relatively extreme depths, durations, or extents. The medium priority level balances the localized nature of flooding, the near-term benefit, and the relatively low cost of implementation.

The majority of projects identified here remain within City ROW or along existing storm drain system alignments where easements already exist. In a few cases, the City may require additional easement to construct projects or may need to consider alternative alignments to remain within ROW or existing easements.

Certain projects include cost modifiers to account for special factors like coordination with neighboring Cities and Counties, state agencies, or federal permitting.

7.4.1.2 Creek Capacity Projects

Creek projects are also shown that primarily address 100-year capacity deficiencies. For 16th Ave Channel, the existing Highway 101 culvert is undersized and will either require a replacement or additional culverts to flank the existing 5'×10' box culvert. For Laurel Creek, the two highest priorities are to perform vegetation management between 39th and 40th Ave to restore existing channel section capacity and to construct the Curtiss Ave Bypass capital improvement project. Finally, recommended projects along San Mateo Creek include enlargement of the Crystal Springs Rd culvert and levee improvements to address capacity deficiency in the downstream tidal reach and in the event of Crystal Springs Dam overflow.

These projects have been prioritized based on a modified set of criteria compared with pipe system capacity CIPs. Costs of those projects are provided in Table 7-9. The locations of the projects are shown in Figure 7- and Figure 7-.

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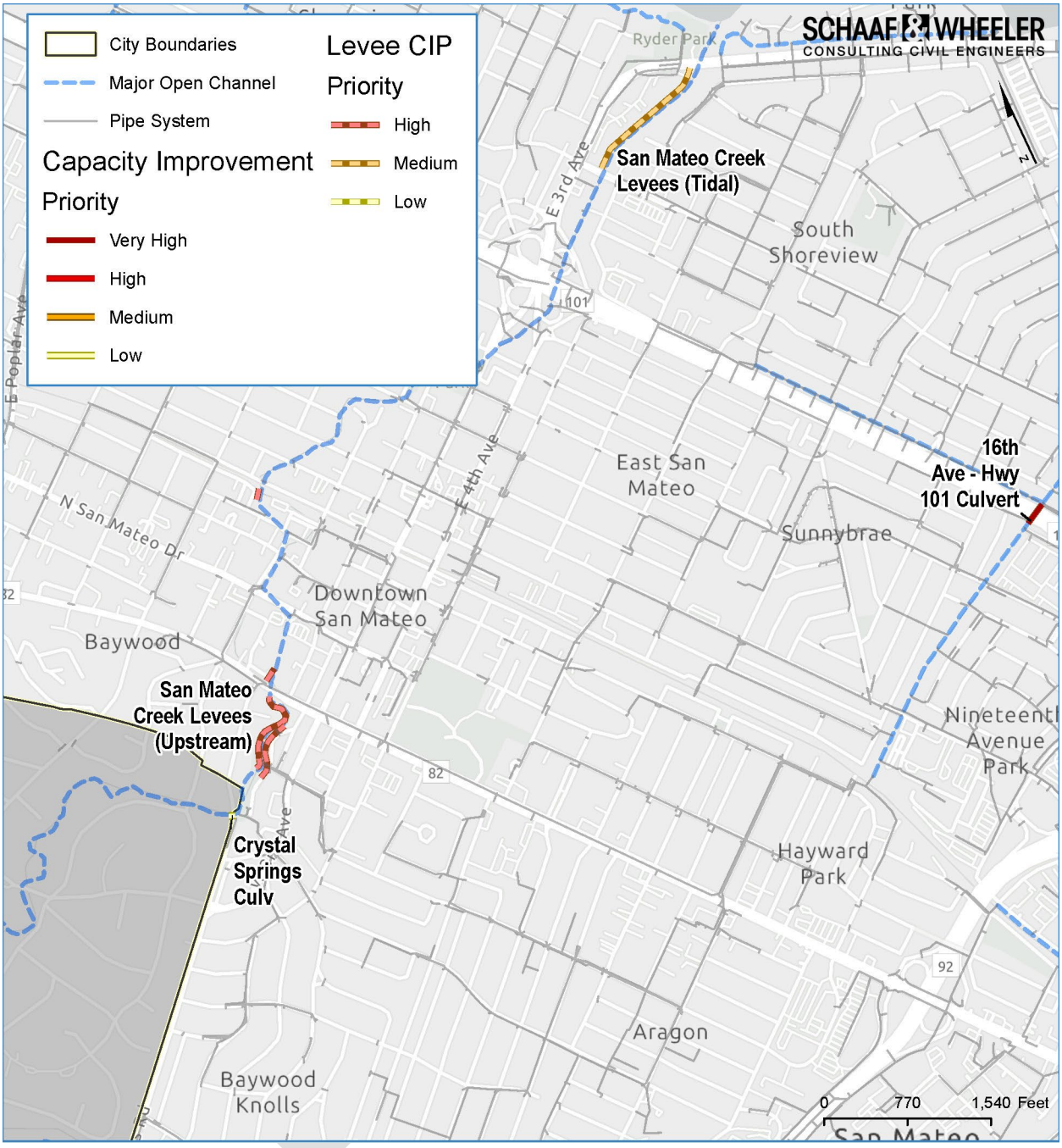


Figure 7-6: Creek CIP Project Map (San Mateo Creek and 16th Ave Channel)

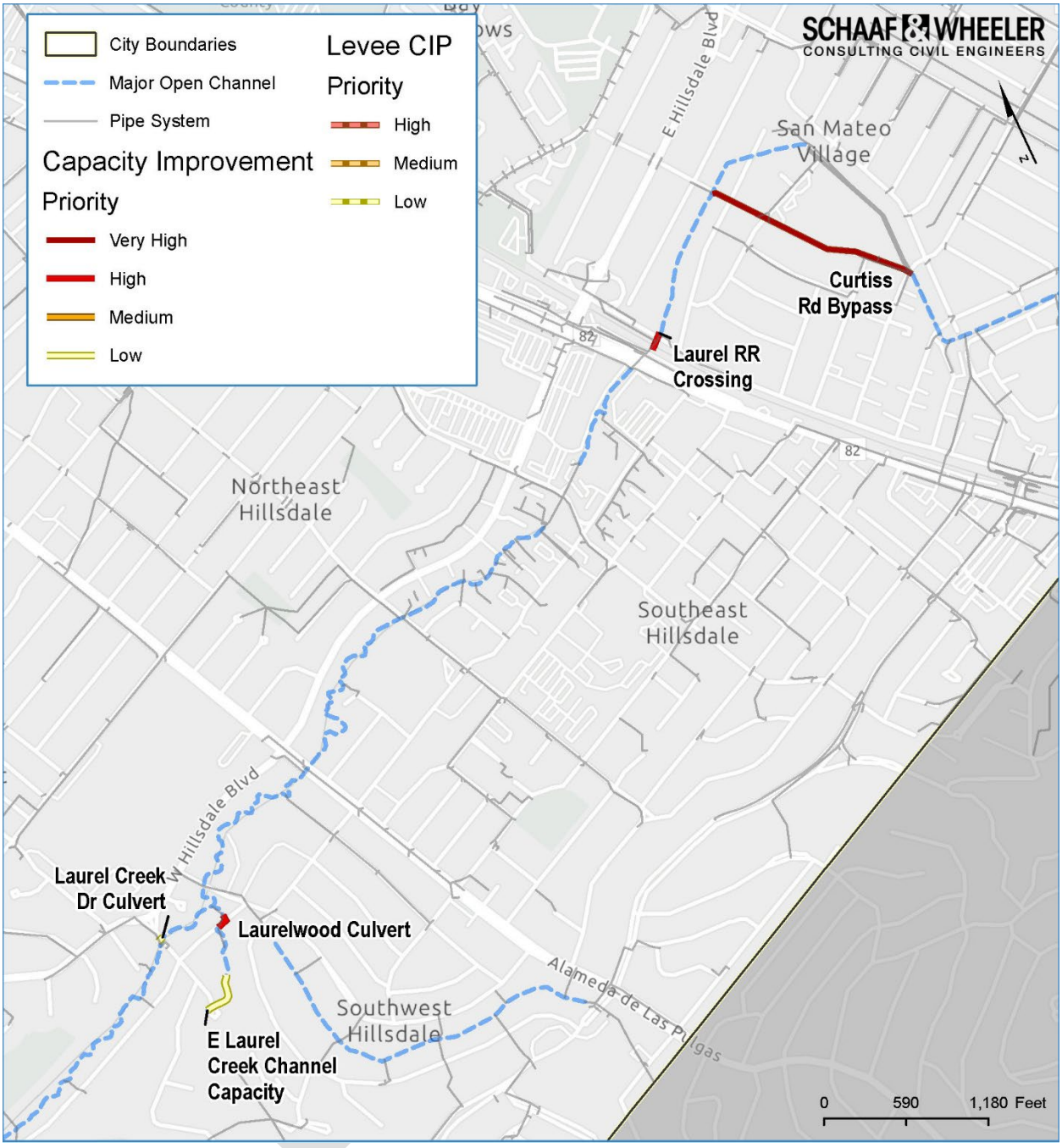


Figure 7-7: Creek CIP Project Map (Laurel Creek)

Table 7-9: Recommended Creek Capacity Projects Summary

Priority	Project	Length (LF)	Approx. Cost
Very High	Laurel Cr - Curtiss Bypass	1,560	$4,650,000
Very High	16th Ave Channel - Highway 101	220	$23,400,000
High	Laurel Cr - RR Xing	140	$4,830,000
High	San Mateo Cr - Levees, Upstream	1,700	$1,250,000
High	E Laurel Cr - Laurelwood Culvert	175	$570,000
Medium	San Mateo Cr - Levees, Tidal	1,260	$710,000
Low	E Laurel Cr - Channel Cap. Improvement	350	$1,160,000
Low	San Mateo Cr - Crystal Springs Rd Culvert	60	$940,000
Low	Laurel Cr - Laurel Creek Rd Culvert	50	$240,000
		Total:	**$38,270,000**

7.4.1.3 CIP Model Results

An overview of 2D model results with the entire set of capital improvement projects completed compared with existing conditions is detailed in Figure 7-8. Areas where existing flooding is mitigated are shown in red, while remaining flooding depths are shown in blue. Additional, more detailed maps of the model results are provided in Appendix I.

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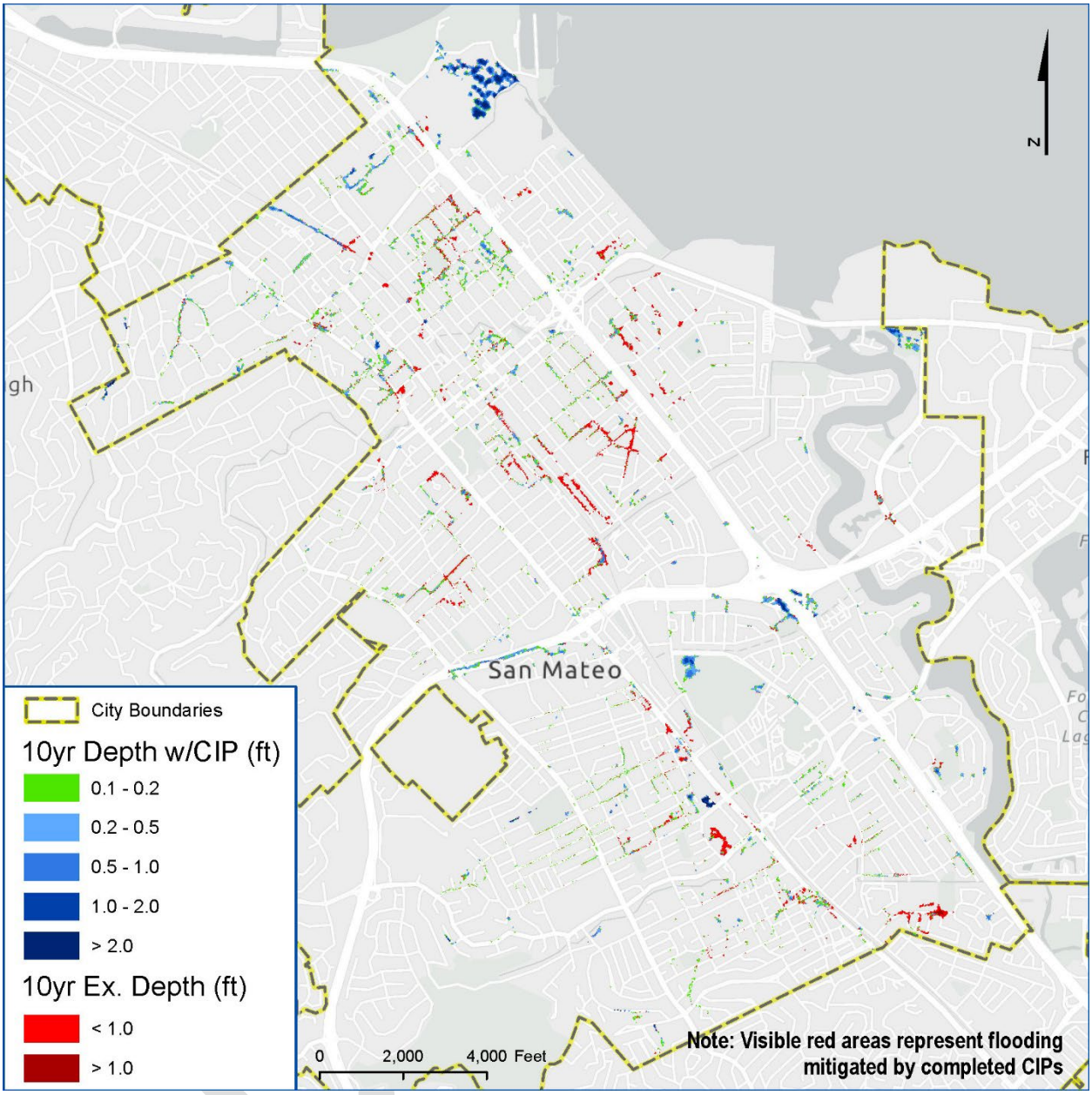


Figure 7-8: 10-year Model Maximum Depth with CIPs Compared with Existing Condition Maximum Depth

7.4.2 Condition-Related Projects

In general, structural deficiencies identified during CCTV inspections are considered a priority in the development of repair and replacement (R&R) and O&M projects. Projects are identified and prioritized for R&R projects and for O&M projects. R&R projects are generally considered capital projects while operations and maintenance projects are simply a prioritized guide to aid the City in directing system cleaning and minor repair work.

Condition-related projects are summarized in Table 7-10 and Table 7-11. Maps of the projects are provided in Figure 7-9 through Figure 7-12.

Table 7-10: Repair and Replacement (R&R) Project Summary

Priority	Description	Spot Repairs	Replace Length (LF)	CIPP Lining (LF)	Estimated Cost
Very High	Bettina-Kingridge	0	797	0	$647,000
High	Saratoga	3	522	0	$433,000
High	Monte Diablo	0	654	0	$396,000
High	San Miguel	1	385	0	$224,000
High	Cupertino	1	482	279	$563,000
High	Ocean View	0	549	0	$512,000
High	Pasadena Repair	2	0	0	$41,000
High	Oak Valley	0	105	337	$131,000
High	Fallon	0	408	0	$271,000
High	Palm	2	0	0	$47,000
High	Fairfax	1	0	555	$94,000
High	Almaden	0	303	0	$196,000
High	42nd-Kingridge	0	276	0	$182,000
High	David St	4	59	0	$152,000
High	Viewridge	0	0	230	$95,000
High	Glendora	1	156	0	$173,000
High	Parrot-CSM	1	173	0	$163,000
High	Santa Clara	1	305	0	$252,000
Medium	Parrot	2	144	0	$136,000
Medium	Polhemus	0	139	0	$134,000
Medium	Parkside	0	129	0	$148,000
Medium	Edinburgh	1	0	0	$32,000
Medium	Mason at Bedford Cr	0	20	0	$76,000
Medium	Newlands-42nd	0	282	0	$186,000
Low	Countryside	0	182	0	$112,000
Low	Peninsula	0	69	0	$66,000
Low	Quebec	0	120	0	$84,000
Low	40th	1	0	0	$24,000
Low	42nd Ave	1	39	0	$73,000
Low	Patricia	0	12	0	$36,000
Low	Shoreview	1	0	0	$26,000
Low	Woodberry	1	0	0	$26,000
Low	Alderwood	0	63	0	$71,000

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Table 7-11: Recommended Operations & Maintenance Actions for CCTV Reaches

Priority	Name	Length (ft)	Est. Cost
High	5th at Ellsworth	41	$5,900
High	Amphlett	26	$4,100
High	Bayshore	113	$14,600
High	Ada	68	$9,200
Medium	San Miguel	24	$3,400
Medium	Overland	96	$10,200
Medium	Almaden	314	$31,100
Medium	20th at ECR	288	$28,600
Low	Santa Clara	145	$9,800
Low	Bayswater	463	$28,700
Low	Shoreview	136	$9,200

R&R projects are not shown where capacity projects have already been identified. However, O&M projects are identified even where overlap occurs. Cleaning should still be performed on portions of the system slated for replacement as part of a capacity project. This is intended to help maintain system capacity while capital projects are implemented.

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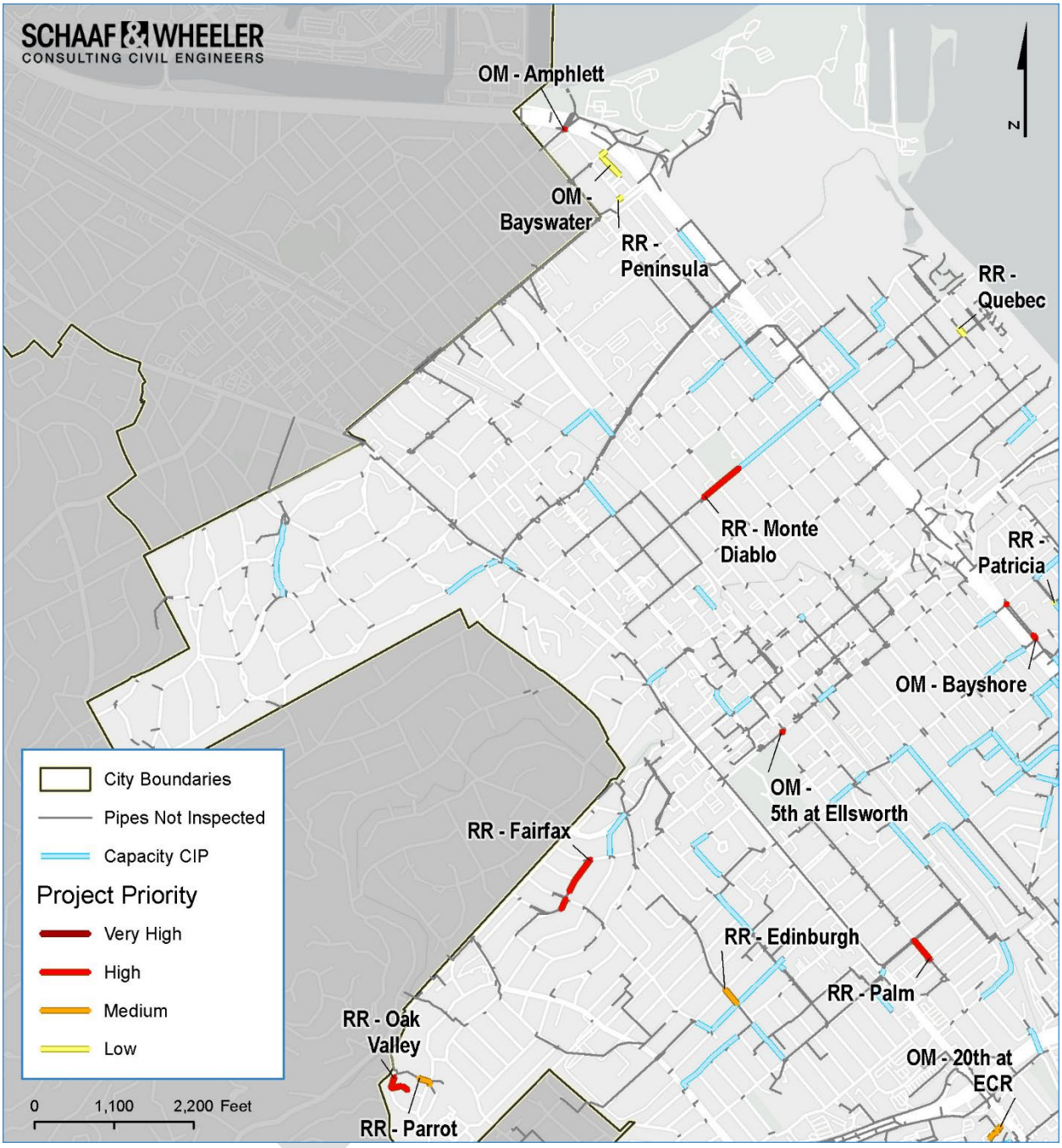


Figure 7-9: Prioritized R&R and Maintenance Project Overview Map (Northwest Quadrant)

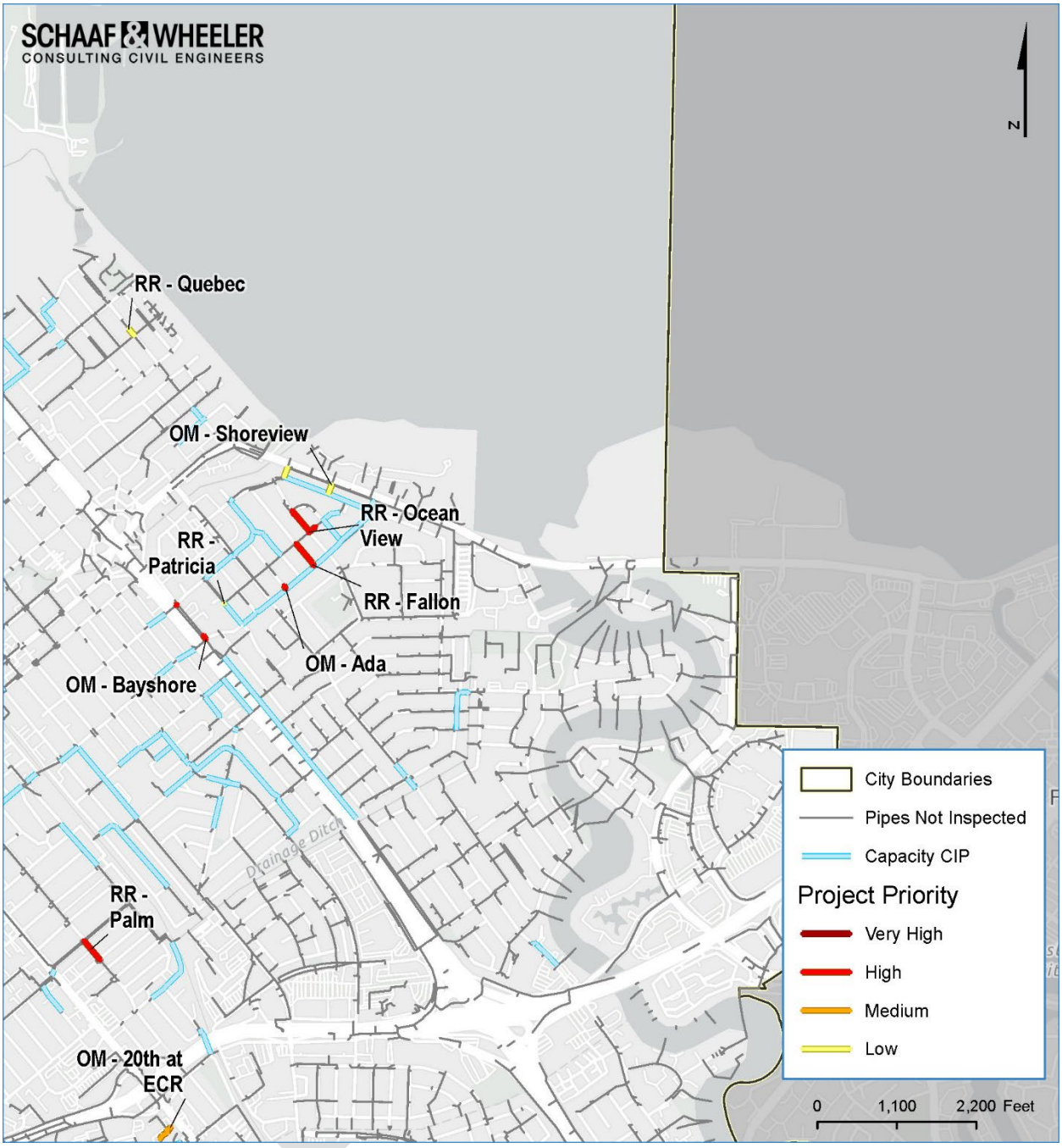


Figure 7-10: Prioritized R&R and Maintenance Project Overview Map (Northeast Quadrant)

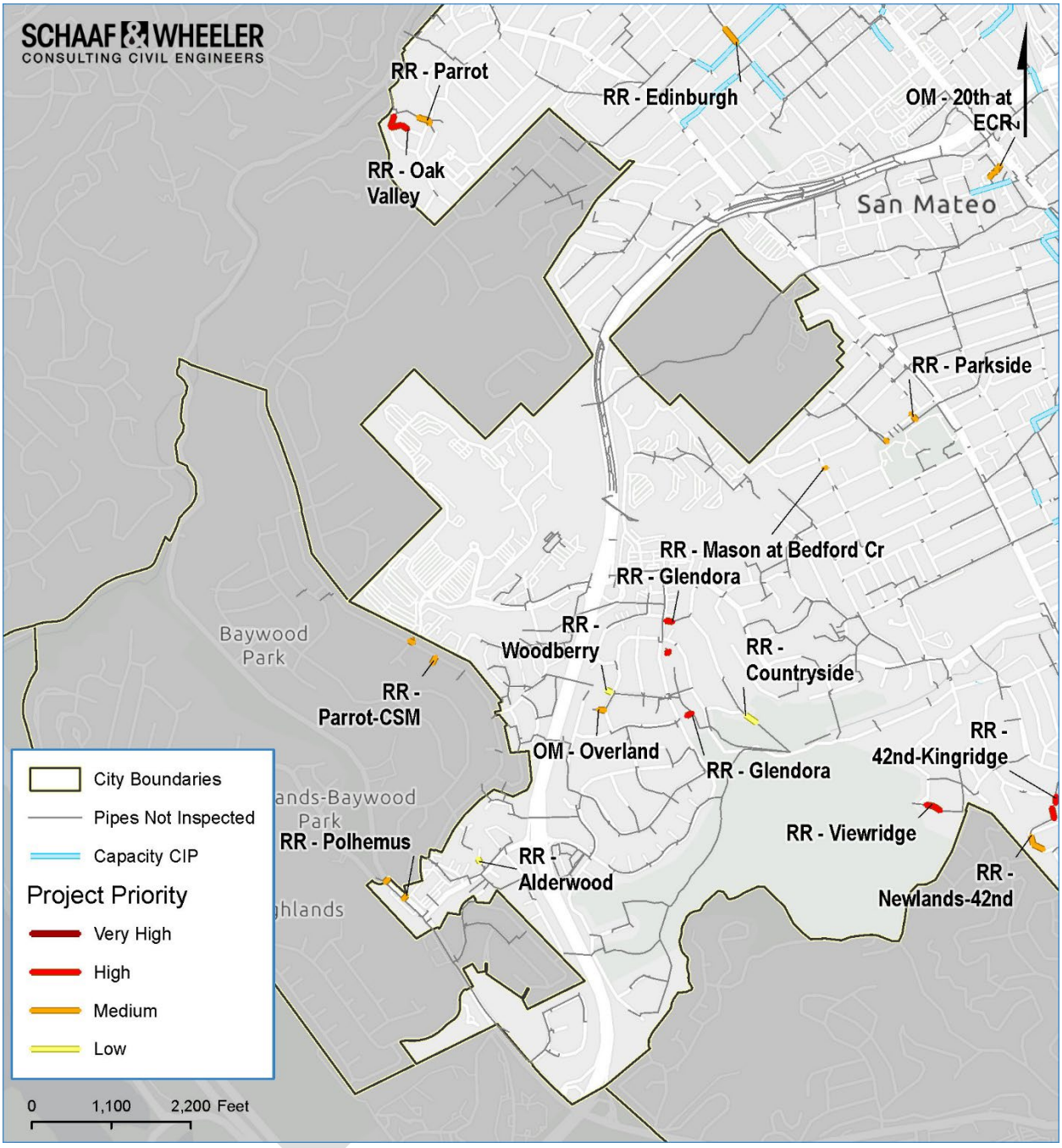


Figure 7-11: Prioritized R&R and Maintenance Project Overview Map (Northeast Quadrant)

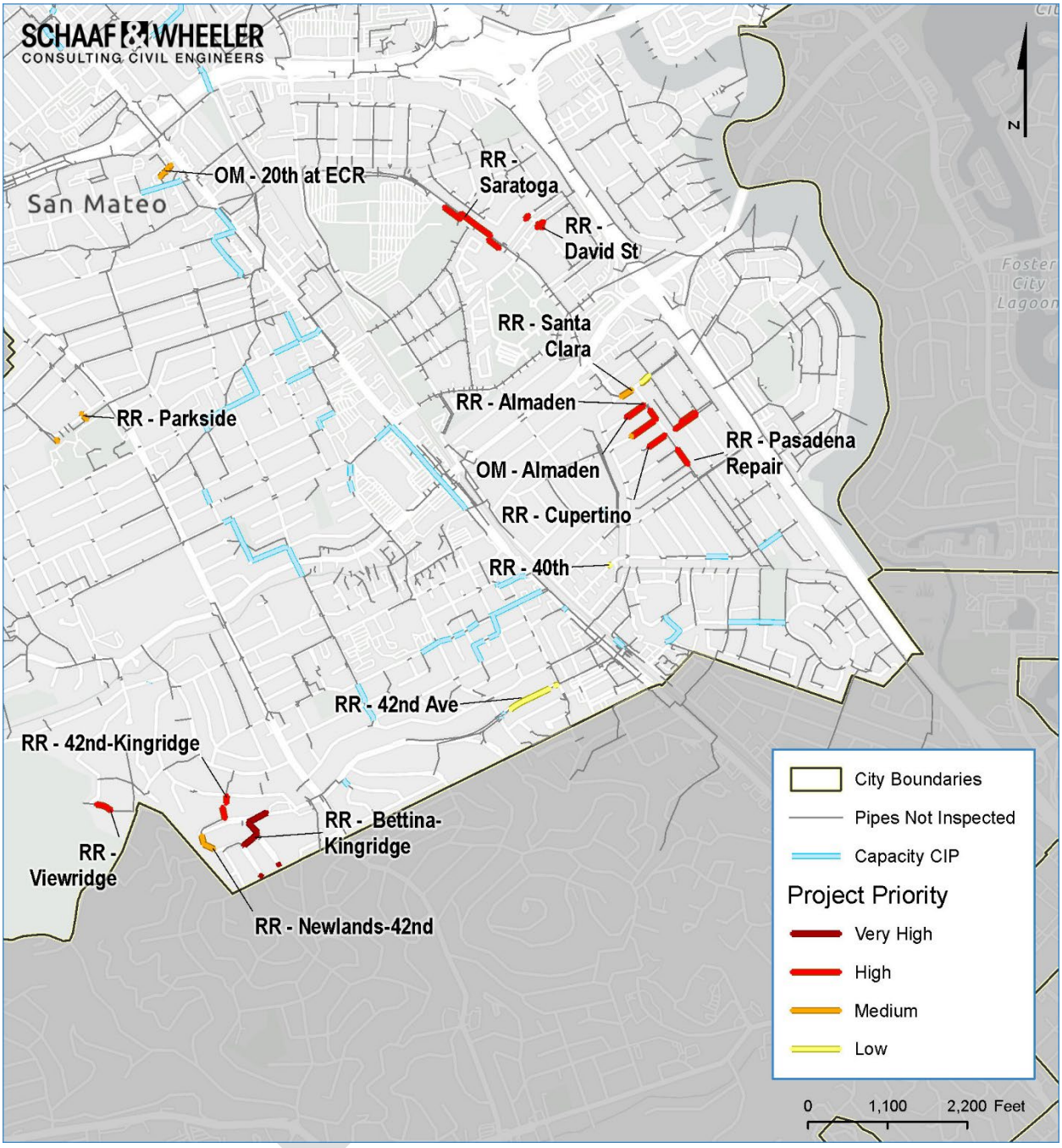


Figure 7-12: Prioritized R&R and Maintenance Project Overview Map (Northeast Quadrant)

Regardless of designation, each project's proximity to capacity projects and other repairs and cleaning tasks should be considered while City staff develop a strategy for completing the projects. Efficiency may be found in bundling capacity, repair, and maintenance tasks into the same contracts where projects are adjacent to one another. For example, the Newbridge Ave Very High priority capacity project is immediately adjacent to an R&R project to address structural deficiencies in the system immediately to the west.

7.4.3 Solutions to Known Drainage Issues

Problem areas identified by City residents and staff are identified in Chapter 2.5. These problem areas have been considered as a supplement to modeling in the development and prioritization of drainage projects. However, some require solutions that extend beyond the limits of closed conduit drainage systems examined by this document. A summary of identified causes of drainage issues and recommended solutions are summarized in Table 7-12. This table is provided in no particular order and is not intended to imply any level of priority.

Table 7-12: Probable Causes and Recommendations to Alleviate Recurring Flooding Issues

Issue	Location	Cause or Source	Recommendation
1	Rollins Rd	Capacity deficiency/Tidal	A 36-inch intertie has been contemplated previously. This would require coordination with Burlingame
2	Indian Ave at N Humboldt St	Capacity deficiency (10-year)	Complete Idaho and Indian projects
3	E Bellevue Ave & RR	RR ditch/bubble-up systems discharge to a low point at Bellevue	Complete Bellevue—Claremont project, connecting railroad low point/bubble-up to Poplar Ave main
4	W Poplar (Hurlingham to Engle)	Bubble-up system	Complete West Poplar project to connect bubble-ups to system at El Camino Real
5	228 Norfolk Alleyway	Lack of Inlet at Sag Point	Implement Ellsworth project, decreasing impoundment in railroad underpasses during storms
6	Santa Inez, Monte Diablo, and Tilton RR crossings	Capacity deficiency	Complete the B St project
7	N B St	Capacity deficiency (Observed during NYE 2022 Event)	City is currently developing a project to connect the bubble-up to nearby pipe systems with an outfall to San Mateo Creek
8	2nd & Delaware	Documented issue (Bubble-up system)	Implement the following projects: • Newbridge Ave • Newbridge Ave Upstream • Dakota Ave • Shoreview Newbridge Ave must be implemented first because it creates capacity for upstream projects and addresses a severe CMP condition-deficiency identified by CCTV
9	Mildly sloped pipe systems around Patricia	Capacity deficiency (10-year storm)	Based on O&M staff comments, inlets may be undersized. System on the north side of the street is a bubble-up system. Implement 3rd Ave at San Mateo project to eliminate bubble-up
10	3rd Ave between ECR and San Mateo Dr	Documented issue (Bubble-up system)	Implement 3rd Ave Bypass project to mitigate culvert capacity deficiency identified by models during storm events causing >10-year peak flows

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Issue	Location	Cause or Source	Recommendation
11	Flooding from San Mateo Creek at Arroyo Court	Culvert capacity deficiency	Implement 9th Ave at Laurel project, connecting Laurel Ave bubble-ups to 9th Ave system at B St
12	9th Ave at Laurel Ave (Bubble-up System)	Bubble-up system	Implement S Delaware St at 10th Ave project
13	Rosewood Dr	Bubble-up system	System in this area relies on bubble-ups and a converted 20" DIP water main. Implement the following projects: • Norton St Ditch • Carlisle • Grant-Birch • Grant St These projects create sufficient downstream capacity and eliminate problematic bubble-up systems This system is also impacted by the undersized 16th Ave Channel culvert at Highway 101
14	9th Ave & S Claremont	Capacity deficiency on Delaware to 10th	Immediate capacity deficiencies at the intersection can be mitigated with the following projects: • Grant St at Sunnybrae Blvd • Grant-Birch • Carlisle The Carlisle project is primarily intended to mitigate other issues, but would divert runoff from the capacity deficient line on Grant St at Sunnybrae Blvd This system is also impacted by the undersized 16th Ave Channel culvert at Highway 101
15	Bubble up systems at 10th Ave and Grant St	Bubble-up system	Implement S Norfolk at 16th Ave Channel project to connect bubble-up to nearby system
16	Sunnybrae & Grant St	Capacity deficiency (<10-yr storm)	Implement the recommended improvements to Marina Lagoon pump station
17	16th Ave Channel at Hwy 101	Undersized 5'×10' box culvert	17th Ave is served by bubble-up systems and a ditch adjacent to the railroad. Implement the 17th Ave project to eliminate problematic bubble-ups
18	Lodi St near 16th Ave Channel	Capacity deficiency (100-yr storm)	Add an inlet at this location connecting to the pipe on 16th Ave

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Issue	Location	Cause or Source	Recommendation
19	Marina Lagoon Pump Station	Aging pump station	Maintenance issues with bubble-up systems can be solved by implementing the Hobart project. Downstream projects are not strictly required to eliminate the bubble-ups. However, downstream capacity issues can be addressed by implementing the Avila Medium Priority project, diverting flow from the system on Edinburgh St
20	17th near Railroad	Bubble-up systems (documented issue)	Implement the Flores-Edison project
21	Obstructed gutter on 16th Ave	Potential causes include settlement or roots lifting gutter	Implement Edison-31st project, connecting bubble-ups to system on 31st Ave
22	Hobart (Alameda to Quince)	Bubble-up systems, downstream capacity deficiency	Implement Hillsdale Frontage project
23	Flores St at 27th & 28th (Bubble-up System)	Bubble-up system	Implement Hillsdale pump station short- and long-term improvements (install permanent emergency generator to address lack of response time)
24	Arrowood & Edison	Bubble-up systems	Install inlets at the intersection where problems have been observed
25	Hillsdale shopping center frontage	Large storm events, flat system	Implement King Ln project, connecting King Ln bubble-up to system on 38th Ave near El Camino. Nearby W 39th Main, W 39th (bubble-up connection), and Donner to Colegrove projects may also prevent overland flow from reaching King Lane
26	Hillsdale Underpass flooding	Pump station	Implement '40th' project
27	Llano St & Bahia St	Lack of inlets at intersection (documented issue)	Culvert hydraulics are problematic during frequent events, and the culvert is undersized for a 100-year storm event. Trash rack and culvert hydraulics improvements can mitigate more frequent issues, but the E Laurel Cr–Laurelwood Culvert improvement project is necessary to meet creek capacity standards
28	King Lane flooding at King Ct	Bubble-up system	Implement 42nd Ave Underpass project, install permanent emergency generator to address lack of response time and provide 20% additional pump station capacity
29	O'Neill Dr near Casanova Pump Station	Capacity deficiency	Implement Laurie Meadows project, installing equalizer pipes to spread drainage from Belmont across multiple existing outfalls

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Issue	Location	Cause or Source	Recommendation
30	E Laurel Creek Culvert at Laurelwood Dr	Inlet hydraulics, blinding	Maintenance staff note debris buildup at inlets. CCTV shows relatively severe maintenance issues downstream of Camden between Edison and Beresford requiring cleaning. Implement 42nd at Edison low priority project downstream at Edison
31	42nd Ave underpass	Pipe capacity deficiency, pump station?	CCTV structural rating of 4–5 around Kingridge and 42nd Ave. Complete Bettina-Kingridge R&R project
32	Laurie Meadows Dr flooding	Capacity deficiency	Complete 16th Ave Channel–Highway 101 Culvert project, adding two 5'×8' boxes around the existing crossing or providing equivalent conveyance capacity
33	42nd Ave near Camden	Lack of inlet capacity, maintenance needs	Construct inlet and lateral connecting to nearby storm drain system at 3rd Ave
34	42nd & Kingridge	CCTV shows maintenance/replacement need	Construct B St Bubble project—a new lateral that connects the inadequate bubble-up system to the main on B St to the southeast

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7.5 Pump Station Improvements

Seven of the City's ten existing stormwater pump stations were included in the hydraulic model. To be conservative, it is assumed by the model that gated gravity outfalls to the San Francisco Bay at various stations remain fully closed during high tide conditions and interior drainage is handled solely by the pump stations at those locations. Pump station capacity has been evaluated generally for a 10-year storm, except for Marina Lagoon Pump Station, which is evaluated on a 100-year standard. Modeling of existing and improved conditions indicates that the pump stations largely provide sufficient capacity to protect against 10-year flooding.

The current condition of these stations is evaluated in detail in Chapter 5. Inflow rates to the pump stations are based on the results of the SDMP hydraulic modeling. This helps to consider the overall effect of attenuation in a complex system of channels, pipe systems, and storage basins on the operation of the pump stations.

The Pump Station Assessment yields a set of capital improvements required for each station for both short- and long-term planning. Replacement priorities and costs based on the Pump Station Assessment, including contingency and design costs, are summarized in Table 7-13.

Table 7-13: Pump Station Assessment Cost Summary

Priority	Project	Approx. Age	Estimated Cost
Very High	Marina Lagoon Short-Term	43	$7,250,000
	Marina Lagoon Long-Term	43	$100,000,000
	4000 Pacific Long-Term*	38	$3,680,000
	4000 Pacific Short-Term	38	$960,000
High	Hillsdale Short-Term	62	$960,000
	Hillsdale Long-Term	62	$2,260,000
	3rd & Detroit Short-Term	43	$1,150,000
	31st Ave Short-Term	5	$120,000
	Coyote Short-Term	6	$700,000
Medium	Poplar Short-Term	6	$300,000
	Fathom Dr Short-Term	10	$740,000
	3rd & Detroit Long-Term	43	$7,800,000
	Casanova Short-Term	22	$180,000
	16th Ave Long-Term	43	$860,000
	16th Ave Short-Term	43	$530,000
Low	Casanova Long-Term	22	$105,000
	Fathom Long-Term	10	$70,000
TOTAL:			**$127,665,000**

*Overlap with capacity project; 4000 Pacific PS also requires gravity improvements immediately upstream (42nd Ave Underpass Project)

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7.6 Climate Change Impact

Additional model scenarios have been run to evaluate the impact of SLR and increased precipitation on an improved system. SLR impacts how storm drainage systems in the City function because precipitation is more likely to occur coincident with higher tidal boundaries. Interior systems that drain to pump stations will primarily be impacted by changes in rainfall intensity, while pump stations exposed to higher tides may operate more frequently and for longer periods of time, causing additional wear on mechanical parts.

7.6.1 Storm Drain System Impacts

The MIKE+ storm drain model has been designed to evaluate two climate chance scenarios:

- 2050 Scenario: 10-year storm depth increased uniformly by 10%, with 100-year tide elevated by 0.8 ft
- 2100 Scenario: 10-year storm depth increased uniformly by 30%, with 100-year tide elevated by 3.1 ft

The existing conditions model has been run for 2050 climate change, while the CIP model has been run for both 2050 and 2100 climate change, assuming that recommended projects are completed.

2D model results are compared with existing conditions in Figure 7-13 and Figure 7-14, highlighting areas most impacted by near-term climate change. 2D model results are compared with recommended CIPs complete in Figure 7-15 and Figure 7-16. These model scenarios highlight where enlargement of recommended projects might be necessary to ensure projects are as resilient as possible, and where additional projects beyond those recommended in the near-term might become necessary.

The majority of recommended capital improvement projects appear resilient beyond 2050. Areas most impacted by climate change are low-lying and include Sunnybrae Blvd and Grant St, areas on the west side of Highway 101 near San Mateo Creek, 42nd Ave underpass, Indian Ave near Highway 101, and Patricia Ave from Dakota Ave to Newbridge Ave.

Larger format maps are provided in Appendix I.

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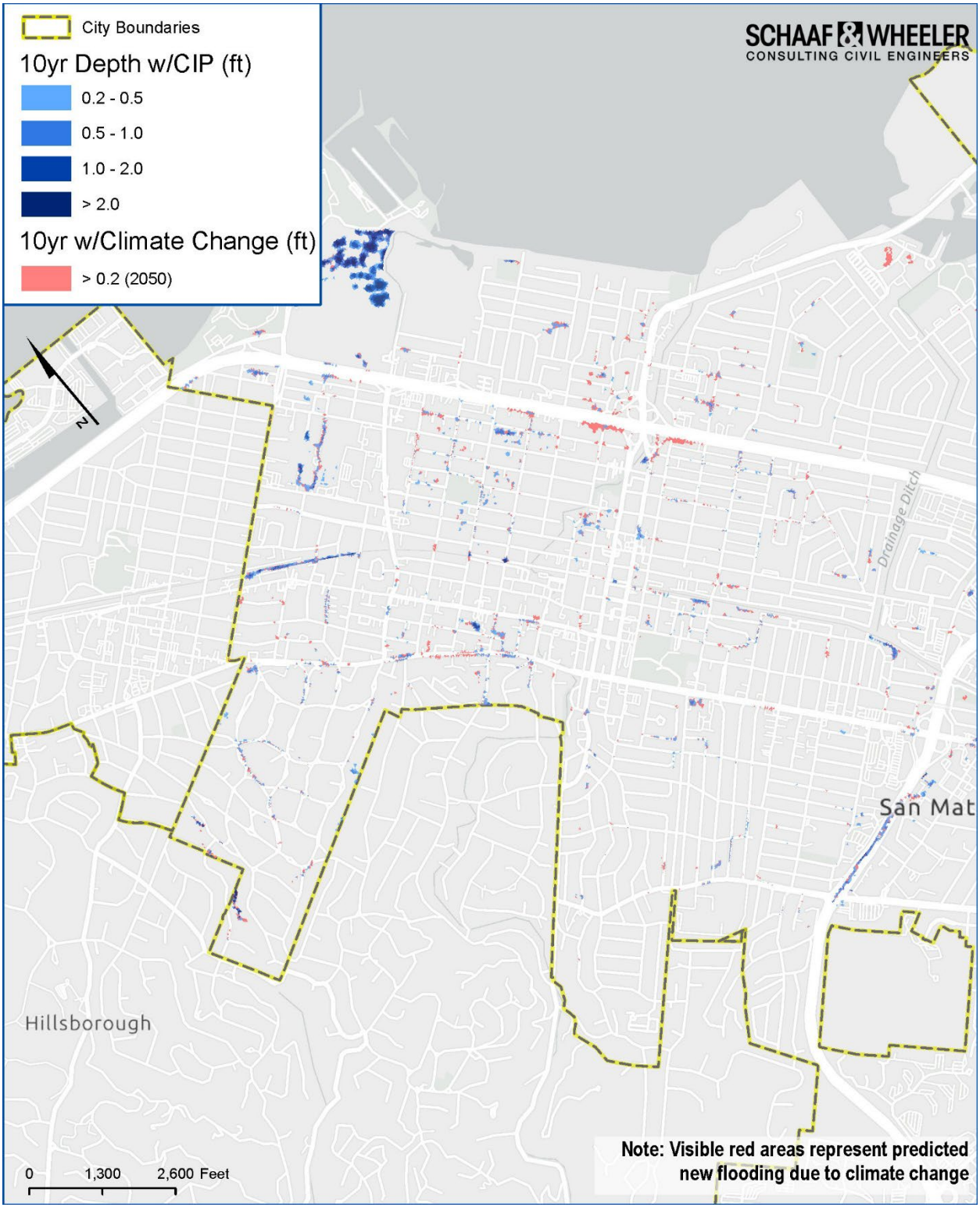


Figure 7-13: 10-year Storm MIKE+ Existing Storm Drain System Model Depth Result Overlain on 2050 Climate Change Scenario Results (Northern Areas)

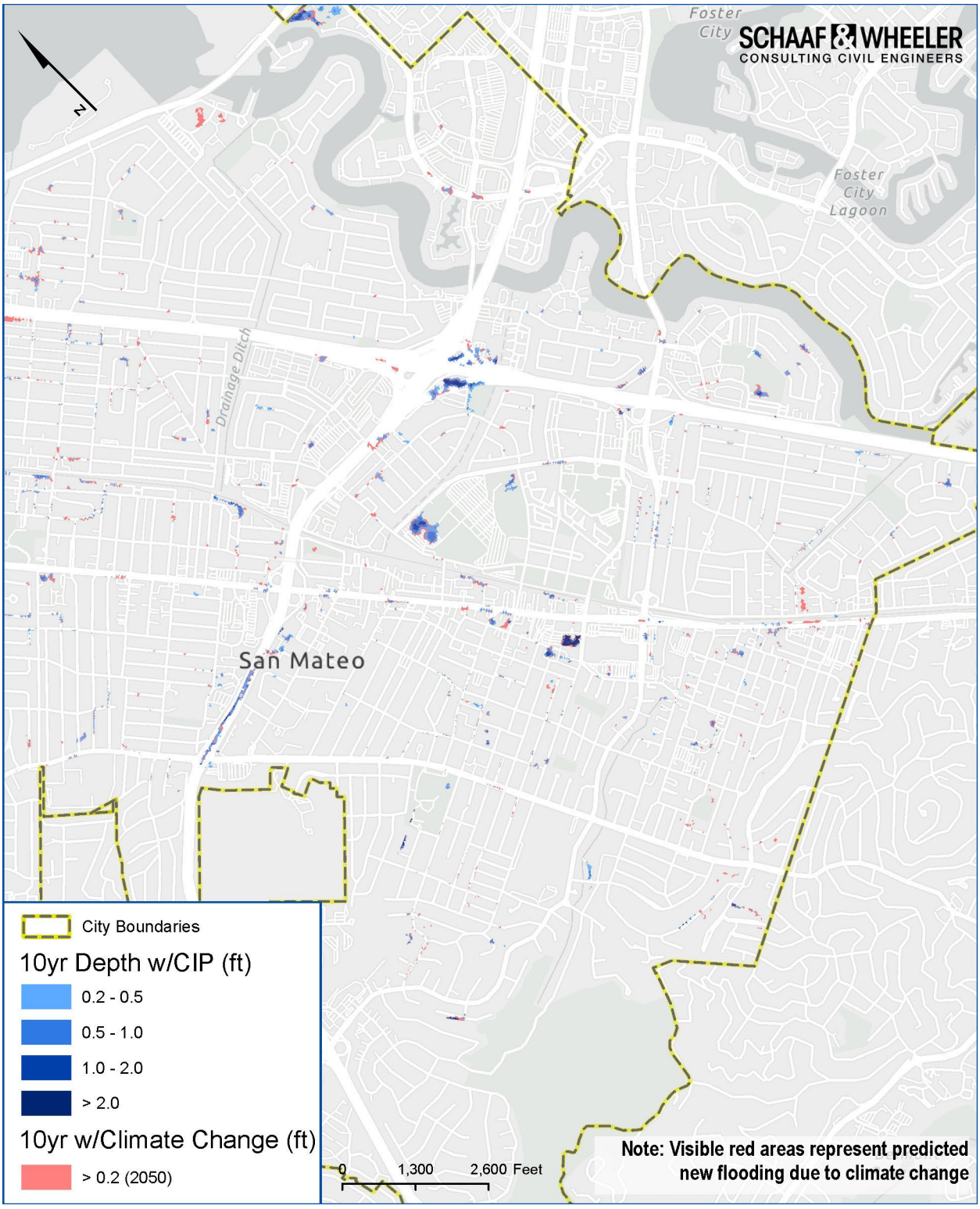


Figure 7-14: 10-year Storm MIKE+ Existing Storm Drain System Model Depth Result Overlain on 2050 Climate Change Scenario Results (Northern Areas)

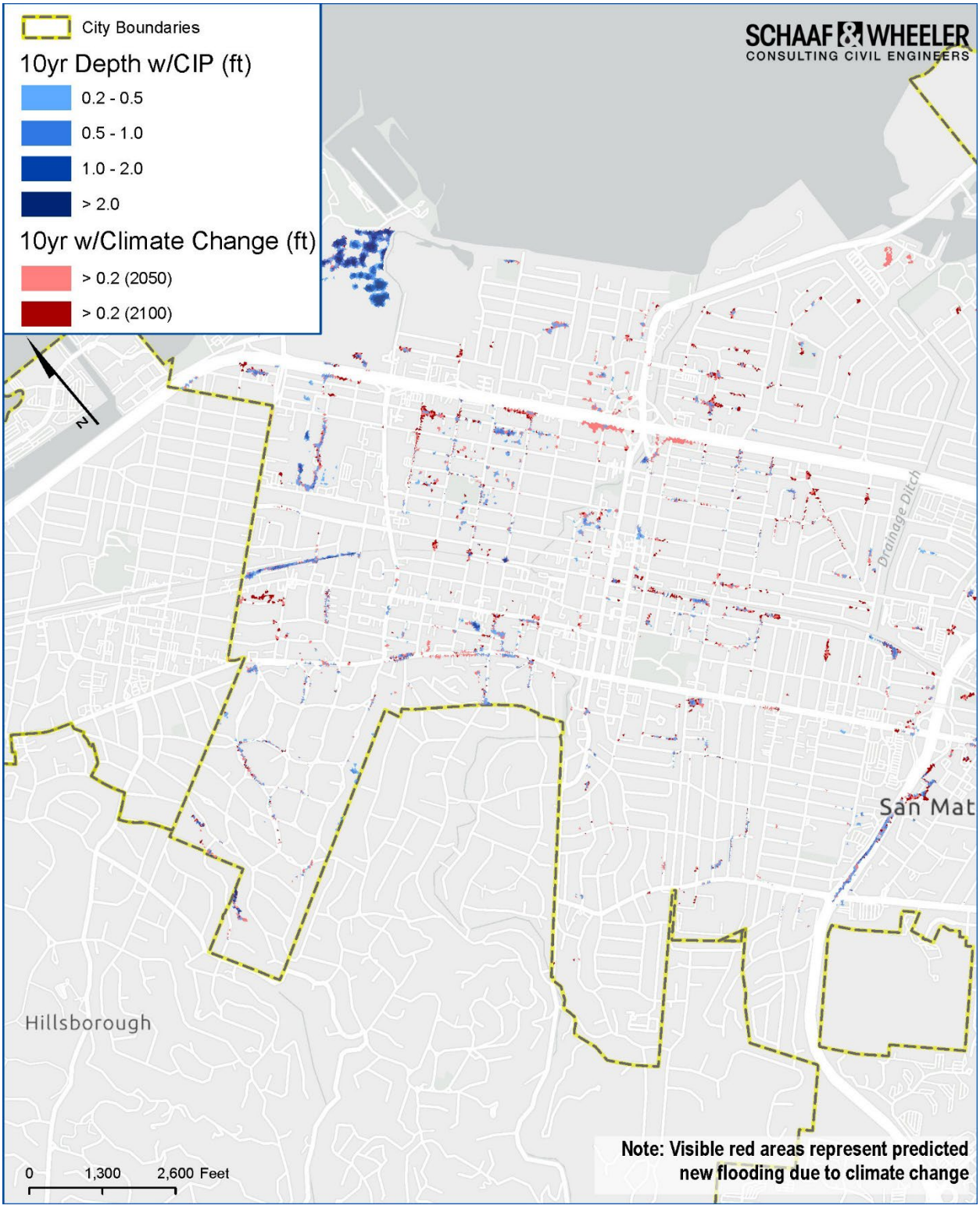


Figure 7-15: 10-year Storm MIKE+ CIP Model Depth Result Overlain on 2050 and 2100 Climate Change Scenario Results (Northern Areas)

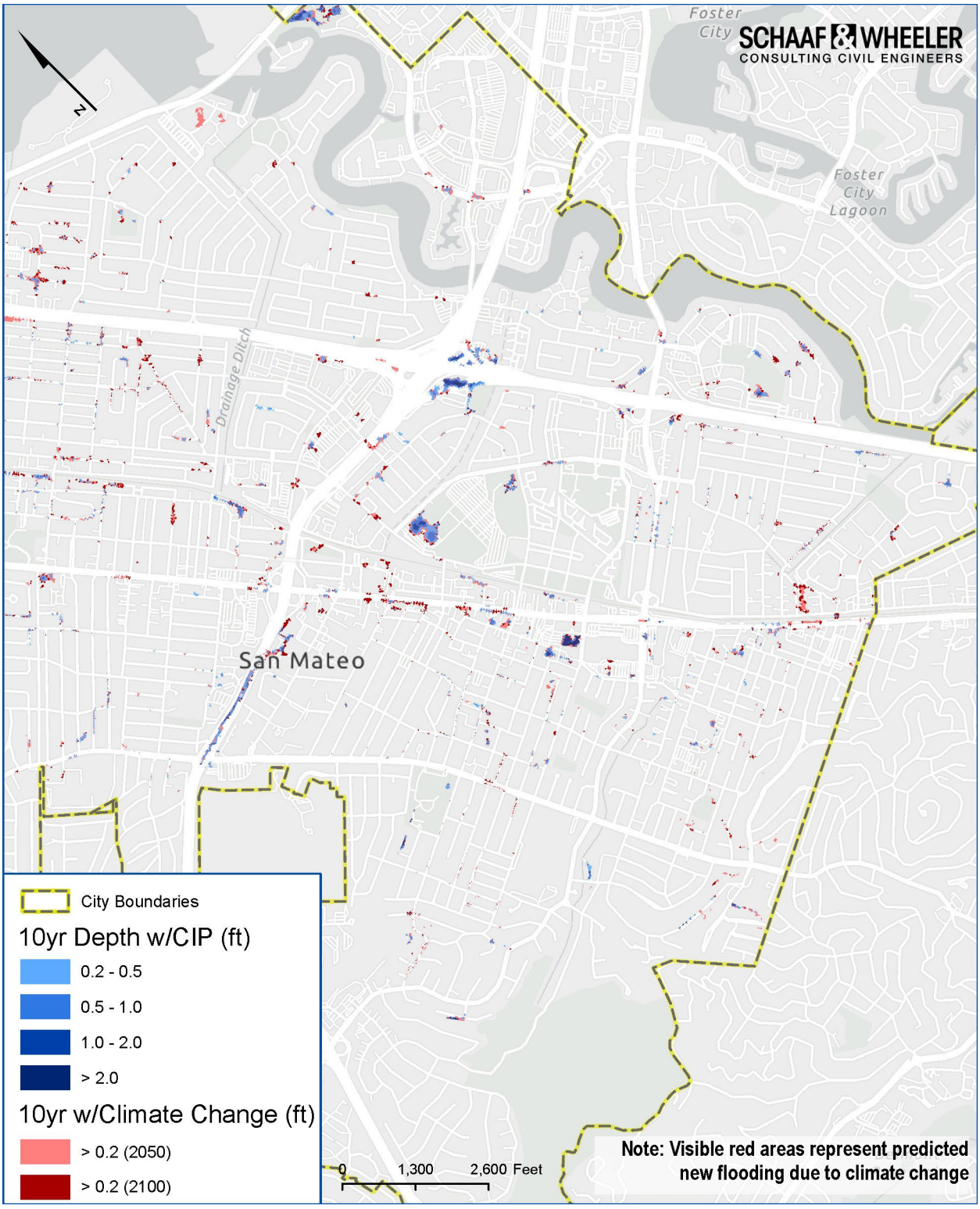


Figure 7-16: 10-year Storm MIKE+ CIP Model Depth Result Overlain on 2050 and 2100 Climate Change Scenario Results (Southern Areas)

7.6.2 Creek System Impacts

7.6.2.1 San Mateo Creek

Impacts on San Mateo Creek come from the following three sources:

- Increases in extreme tide levels due to SLR
- Discharge from Lower Crystal Springs Dam
- Runoff from unimpounded drainage areas to San Mateo Creek (downstream of the Dam)

The 10-year storm model has been evaluated with an additional 10% precipitation depth and 0.8 feet of SLR occurring by 2050, and for an additional 30% precipitation depth and 3.1 feet of SLR occurring by 2100. The Creek is generally able to handle additional runoff during a 10-year storm event. However, the elevated tide causes far greater spill over the banks where floodwalls and levees do not exist (Figure 7-17).

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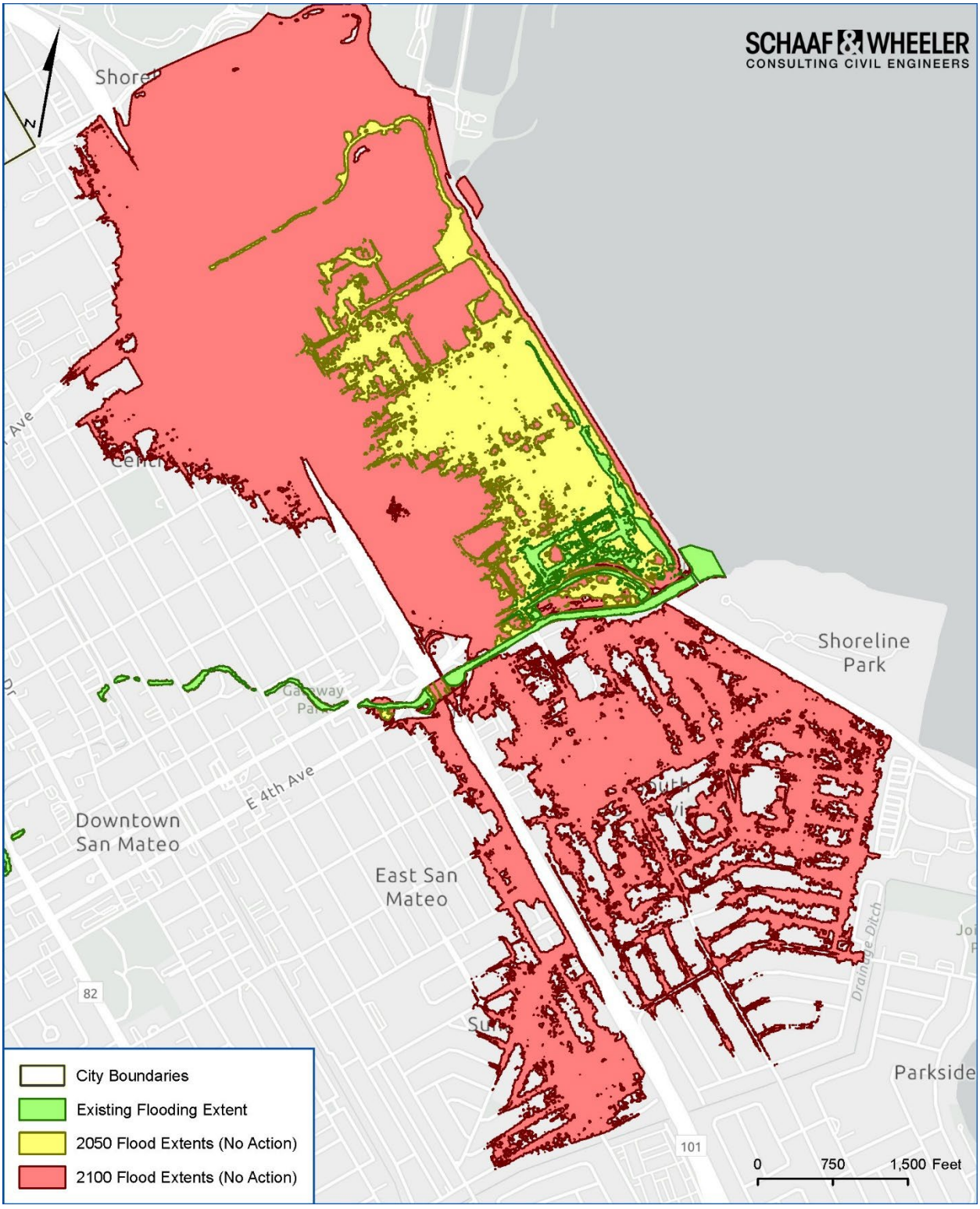


Figure 7-17: San Mateo Creek 10-year Storm with 100-year Tide Climate Change HEC-RAS Model Result, without CIP Complete.

Spill is primarily concentrated just upstream of J Hart Clinton Dr and impacts the North Shoreview neighborhood for the 2050 scenario. By 2100, the 10-year storm model with 3.1 feet of SLR shows that tidal impacts will reach further upstream, affecting the reach between 3rd Ave at Humboldt and the Highway 101 Northbound off-ramp. Existing floodwalls downstream will also become freeboard deficient. Two options will likely be available to mitigate for the effects of SLR by 2100, including the following projects:

1. Add floodwalls upstream of the Highway 101 Northbound off-ramp, and elevate existing floodwalls to provide more freeboard
2. Construct a new pump station and tide gate structure at the downstream end of San Mateo Creek, with a capacity of approximately 500–600 cfs to maintain existing BFEs in the creek during high tide conditions

The Downstream San Mateo Creek Levee project is recommended in order to mitigate tidal flooding. The City may wish to elevate the priority of this project based on the vulnerability to short-term climate change.

The model has been modified to evaluate the potential effectiveness of tide gates at J Hart Clinton Dr to mitigate increases in overtopping of the left bank, with no other upstream projects complete. Without other interventions, this is not an effective strategy to completely mitigate near-term climate change impact. During a 10-year design storm event with a 100-year coincident tide, the J Hart Clinton tide gate model shows some reduction in flooding extents for the 2050 climate change scenario (Figure 7-18) and more significant reduction for a 2100 climate change scenario (Figure 7-19).

The models show that operating a tide gate on San Mateo Creek is an effective means of reducing flooding, as the volume of interior runoff discharge spilled from the creek is much lower than the volume of tidal waters spilling the Creek's banks. However, significant flooding still occurs upstream in the absence of other measures (e.g. floodwalls or a pump station).

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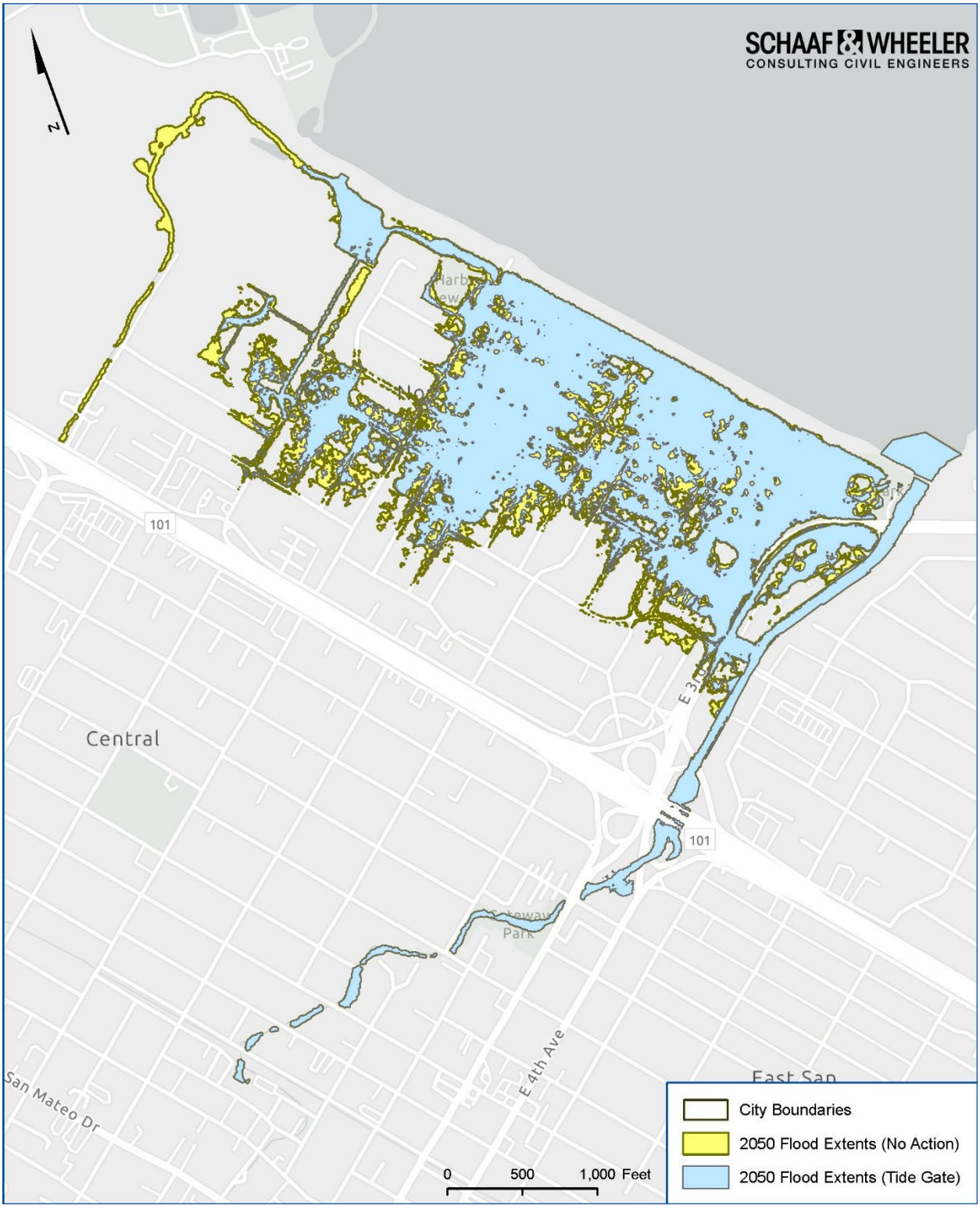


Figure 7-18: San Mateo Creek 10-year Storm with 100-year Tide Climate Change HEC-RAS Model Results – Effect of Operating a Tide Gate at J Hart Clinton Dr (2050 Scenario)

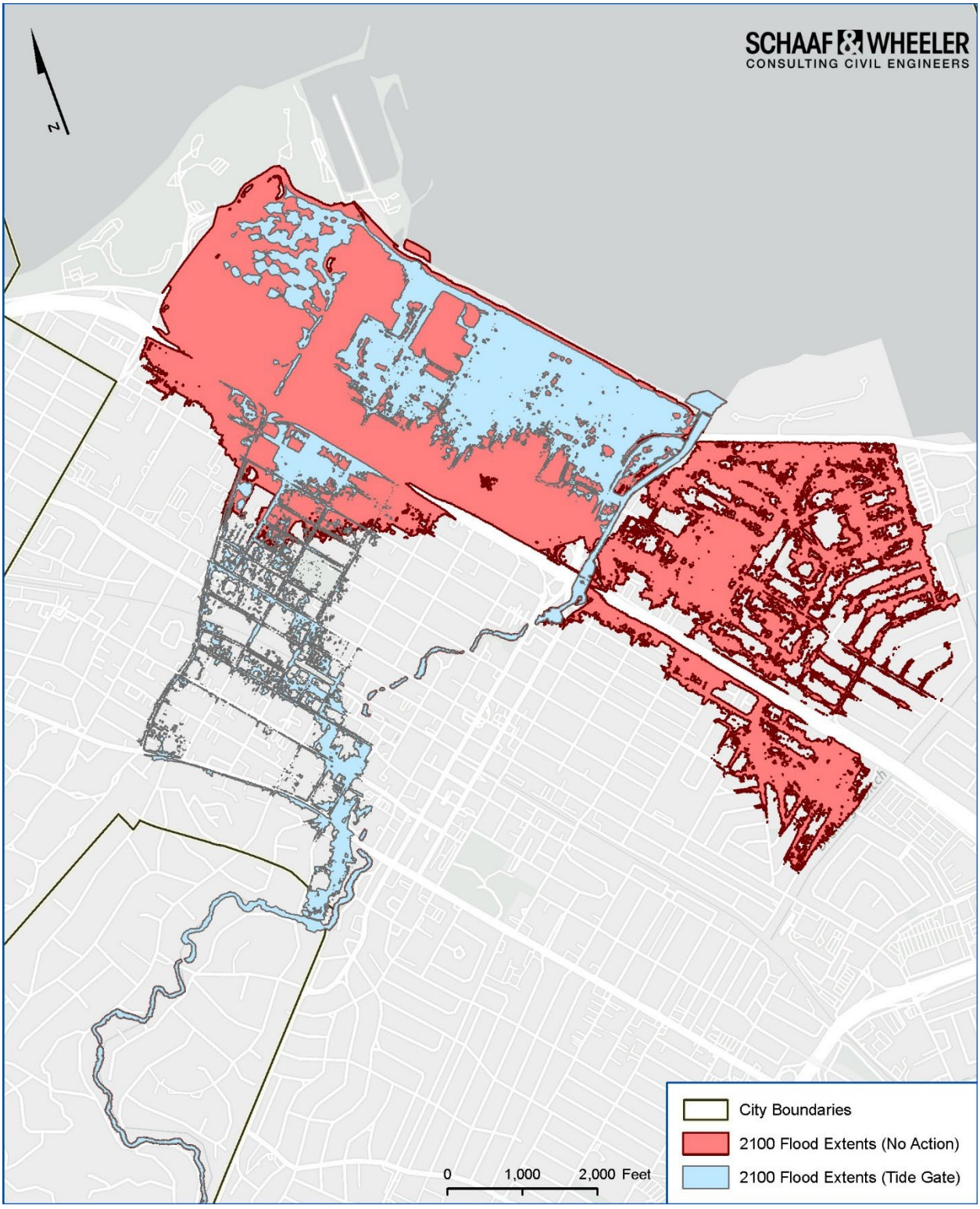


Figure 7-19: San Mateo Creek 10-year Storm with 100-year Tide Climate Change HEC-RAS Model Results – Effect of Operating a Tide Gate at J Hart Clinton Dr (2100 Scenario)

For a 100-year design storm, it is difficult to fully consider the impacts of additional spill from Lower Crystal Springs Reservoir without making assumptions. SFPUC is the dam and reservoir operator and ultimately will decide how to change operations. Close coordination with SFPUC will be necessary to provide flood protection in the future.

Assuming that operational parameters at the Crystal Springs Dam do not change to prevent additional spill during a major design storm event, the discharge rate near El Camino Real could increase by about 17% in response to an 8% increase in 100-year event precipitation by 2050 and a 44% increase in peak discharge in response to the 25% increase in precipitation depth by 2100 (Figure 7-20).

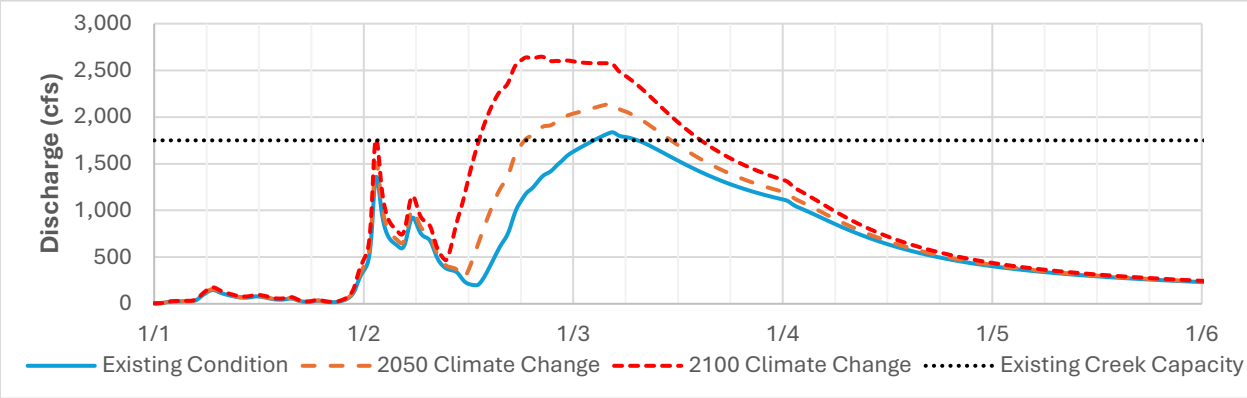


Figure 7-20: San Mateo Creek HEC-HMS Model Discharge near El Camino Real

Managing flood storage availability by releasing from the Dam at 300 cfs (the approximate present day channel capacity without flooding any downstream property) at the start of the storm prevents flooding during the existing 100-year design storm event and reduces 2050 discharge considerably. However, it will likely not be sufficient to prevent significant flooding in the 2100 model scenario (Figure 7-21).

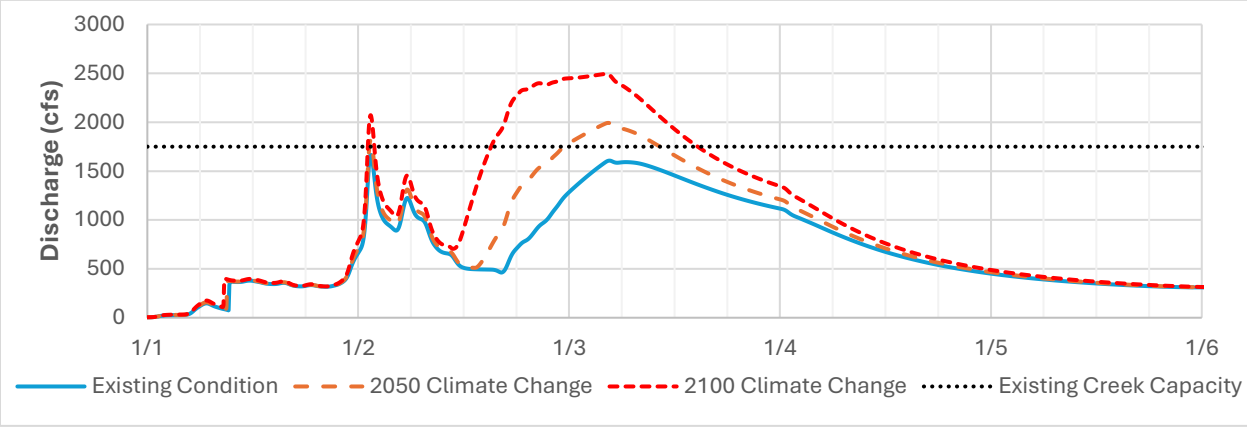


Figure 7-21: San Mateo Creek HEC-HMS Model Discharge near El Camino Real, with 300 cfs Release from Crystal Springs Dam

The model has been used to identify a potential area of impact due to increased extreme precipitation event depths, assuming no action is taken to mitigate additional spill from Crystal Springs Dam and no flood protection measures are installed downstream along San Mateo Creek.

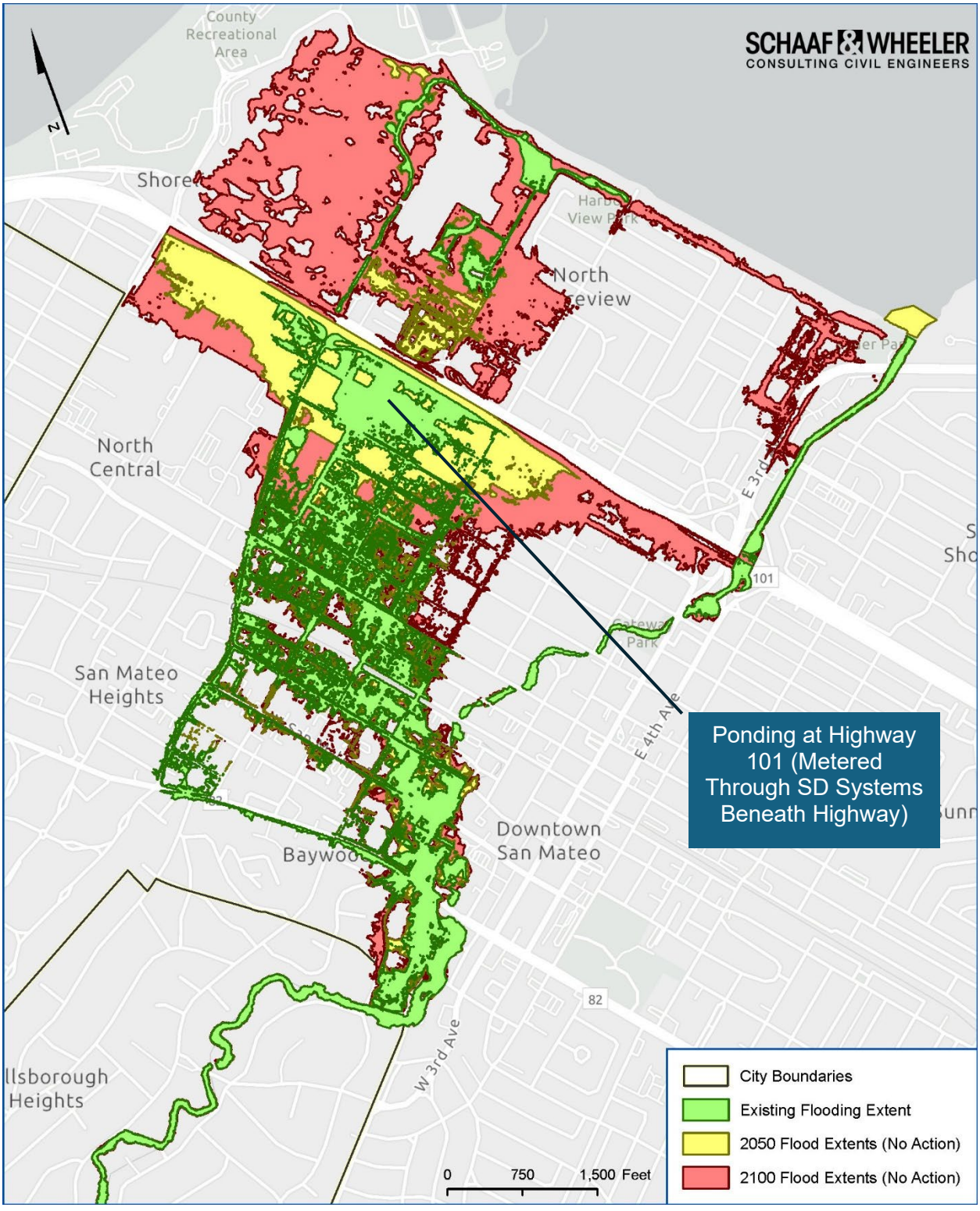


Figure 7-22: San Mateo Creek HEC-RAS Model Inundation Areas (>0.1 ft Depth) for 100-year Storm and 10-year Tide Existing and Climate Change Scenarios, without SLR

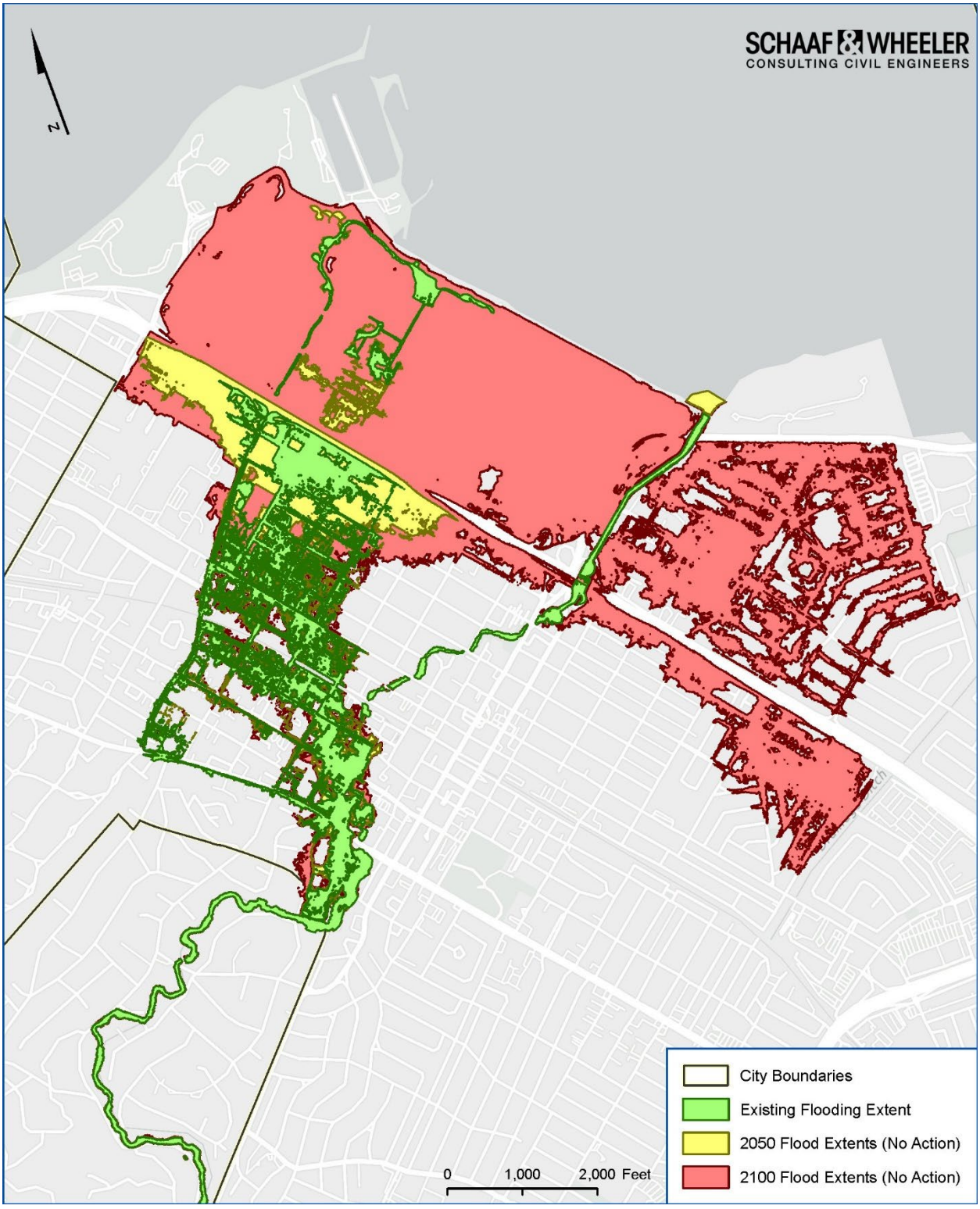


Figure 7-23: San Mateo Creek HEC-RAS Model Inundation Areas (>0.1 ft Depth) for 100-year Storm and 10-year Tide Existing and Climate Change Scenarios, with SLR

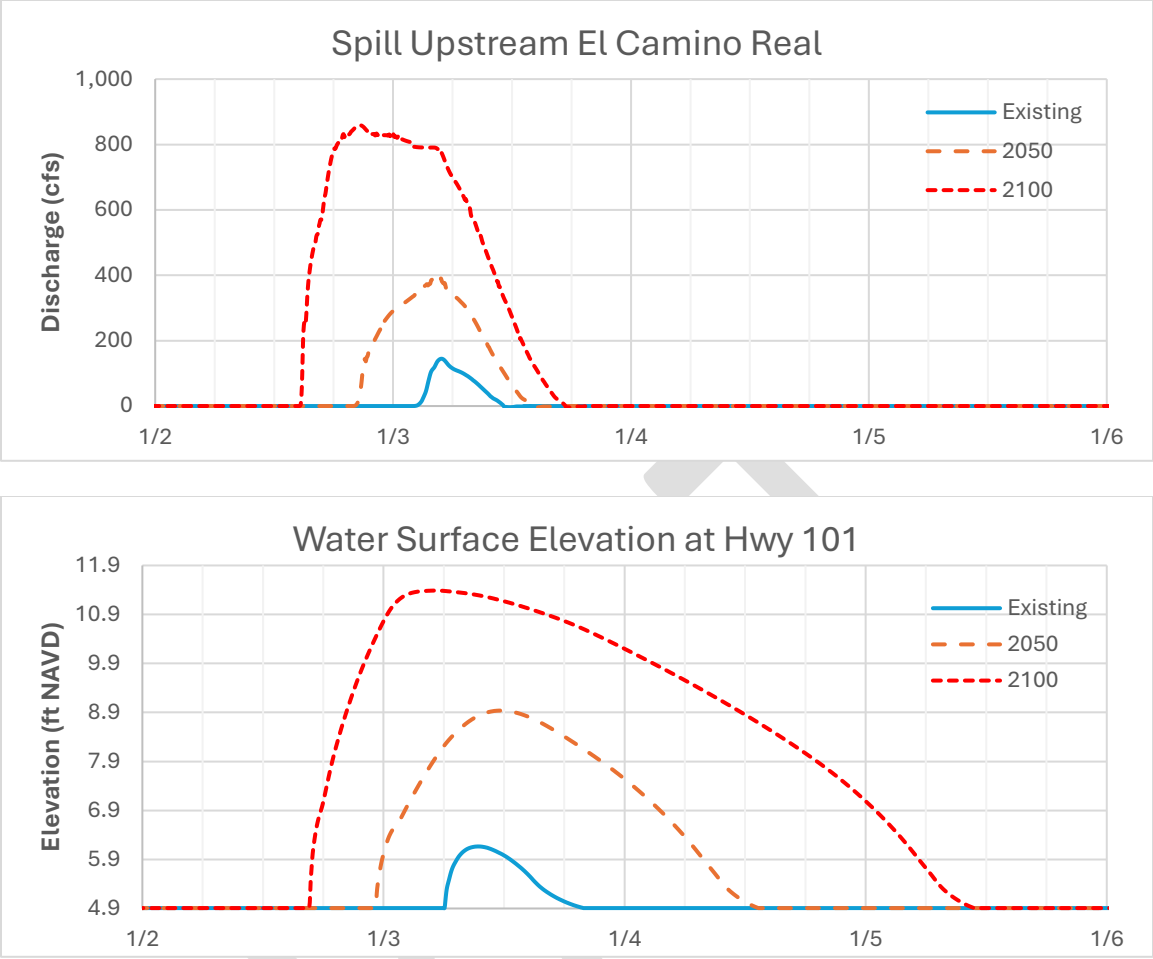


Figure 7-24: San Mateo Creek HEC-RAS Model Overbank Spill and Ponding Water Surface Elevation for 100-year Storm—Existing and Climate Change Scenarios

These model results remain speculative; their purpose is to evaluate the maximum potential for increased flood risk with climate change based on best available science, assuming there are no changes to current publicly-documented operational paradigms (e.g., flood storage availability, spillway crest, outlet capacity, etc.). Open dialog with SFPUC about flood risk downstream of the Dam that considers climate change is necessary to ensure that the City is prepared for future discharge in San Mateo Creek. Reservoir outlet operation will influence the magnitude and cost of flood protection projects required along San Mateo Creek banks where the primary source of flood waters is fluvial.

If no operational changes are made at Crystal Springs Dam by 2050, a hypothetical model has been run to estimate the height and extent of flood protection measures along the Creek in San Mateo. The following two options were explored:

1. Floodwalls with sufficient height to contain additional spill from the reservoir
2. Floodwalls to prevent flooding for existing 100-year discharge in the Creek, supplemented with a 3rd Ave bypass system

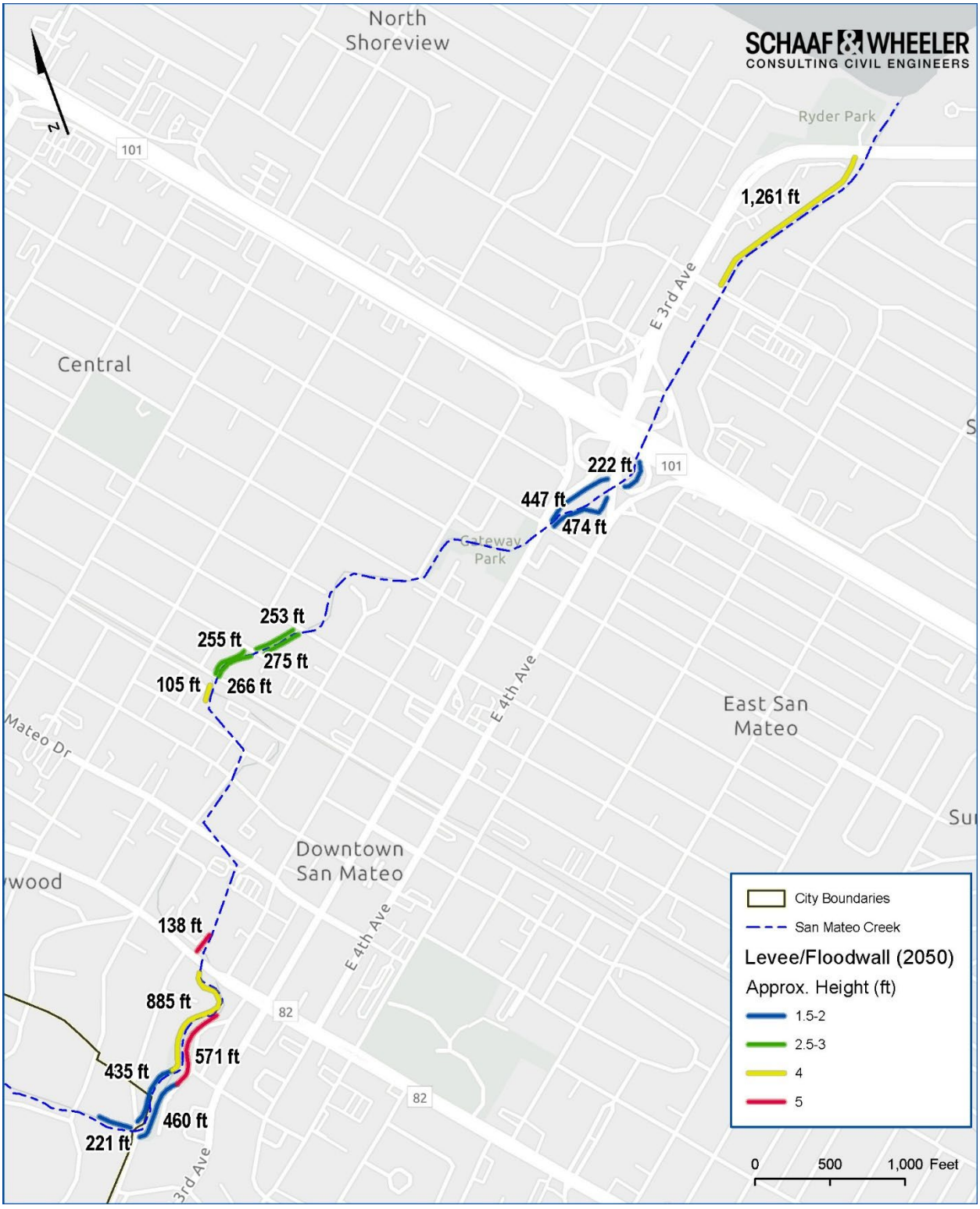


Figure 7-25: San Mateo Creek Flood Wall Characteristics to Prevent Overtopping in 2050, with No Change to Crystal Springs Dam Model Operating Parameters

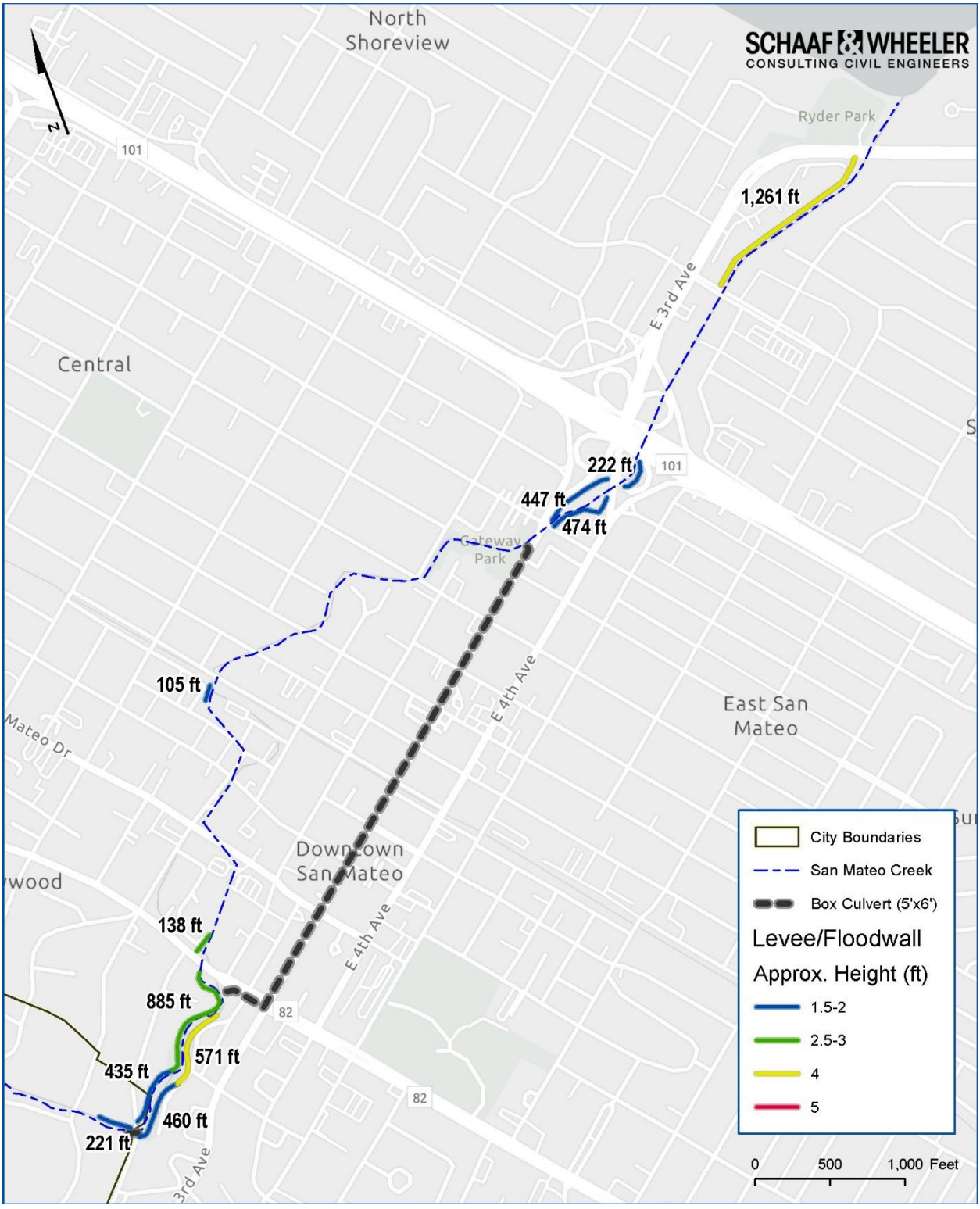


Figure 7-26: San Mateo Creek Flood Wall Characteristics to Prevent Overtopping in 2050, with No Change to Crystal Springs Dam Model Operating Parameters

By 2100, the magnitude of levee and bridge/culvert capacity improvements required to pass an additional 500 cfs may become cost prohibitive. Nearly every bridge and culvert would require additional capacity, or the size of the 3rd Ave Bypass culvert supplementing existing capacity would need to double.

The model has been used to also evaluate the implications of altering operations at Lower Crystal Springs Dam. A model scenario was added that assumes the existing initial storage capacity remains fixed, but low-level outlet release increases from a constant 300 cfs to 600 cfs. The capacity to release at this rate exists today; however, doing so would require improvements to channel capacity or other means of protecting property from flooding between the dam and the City limit that may not be practical. Releases to the Creek are currently limited to approximately 250–300 cfs to mitigate downstream property damage.

Sensitivity to initial storage levels has also been tested, lowering the starting water surface elevation from 287.8 ft NAVD to 284.8 ft NAVD and beginning 300 cfs release earlier in the storm. While this achieves the highest degree of downstream flood mitigation for the 2100 climate change scenario, the reservoir importantly remains a water supply facility, not a flood control facility. Lower operating levels during the winter may not be feasible, particularly if water supply needs increase in the future as well. Fundamentally, this scenario represents an increase in flood storage capacity that could be created by elevating one or more upstream dam/spillway crests.

HEC-HMS discharge hydrographs are shown for 2100 climate change model scenarios compared with existing conditions in Figure 7-27. Peak discharge still increases significantly over existing and 2050 models. However, this represents a significant decrease in spill volume from Lower Crystal Springs Dam.

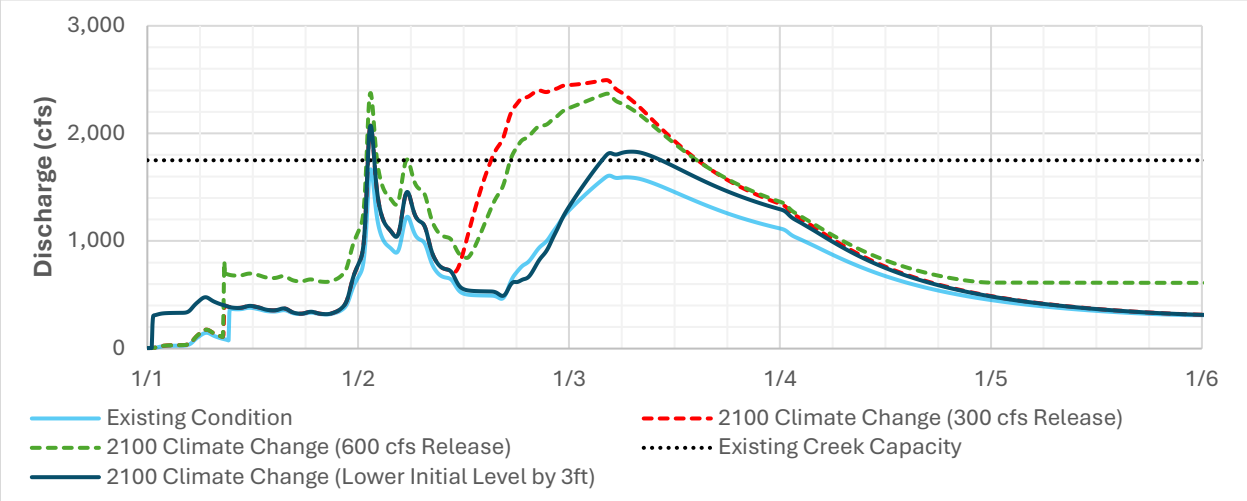


Figure 7-27: San Mateo Creek HEC-HMS Model Discharge near El Camino Real, with 600 cfs Release from Crystal Springs Dam Compared to 300 cfs Release

Preparing for long-term climate change impacts to San Mateo Creek will require close coordination with SFPUC and other regional agencies to improve existing hydrologic and hydraulic models and develop an adaptation strategy. It is not likely that construction of levees or increased conveyance capacity to accommodate a 65%–70% increase in discharge is a feasible, cost-effective strategy, given constraints around the Creek.

7.6.2.2 16th and 19th Ave Channels

16th and 19th Ave Channels are less susceptible to large fluctuations in discharge upstream due to loss of peak attenuation at existing dams. Assuming that Marina Lagoon's BFE is maintained by performing any required improvements to mitigate for climate change impacts, the HEC-RAS models for 16th and 19th Ave Channels have been updated with HEC-HMS climate change scenario hydrograph inputs for 2050 and 2100 design rainfall adjustments.

Model results for the 16th Ave Channel with the Highway 101 culvert improvement and for 19th Ave Channel are shown in Figure 7-28 and Figure 7-29.

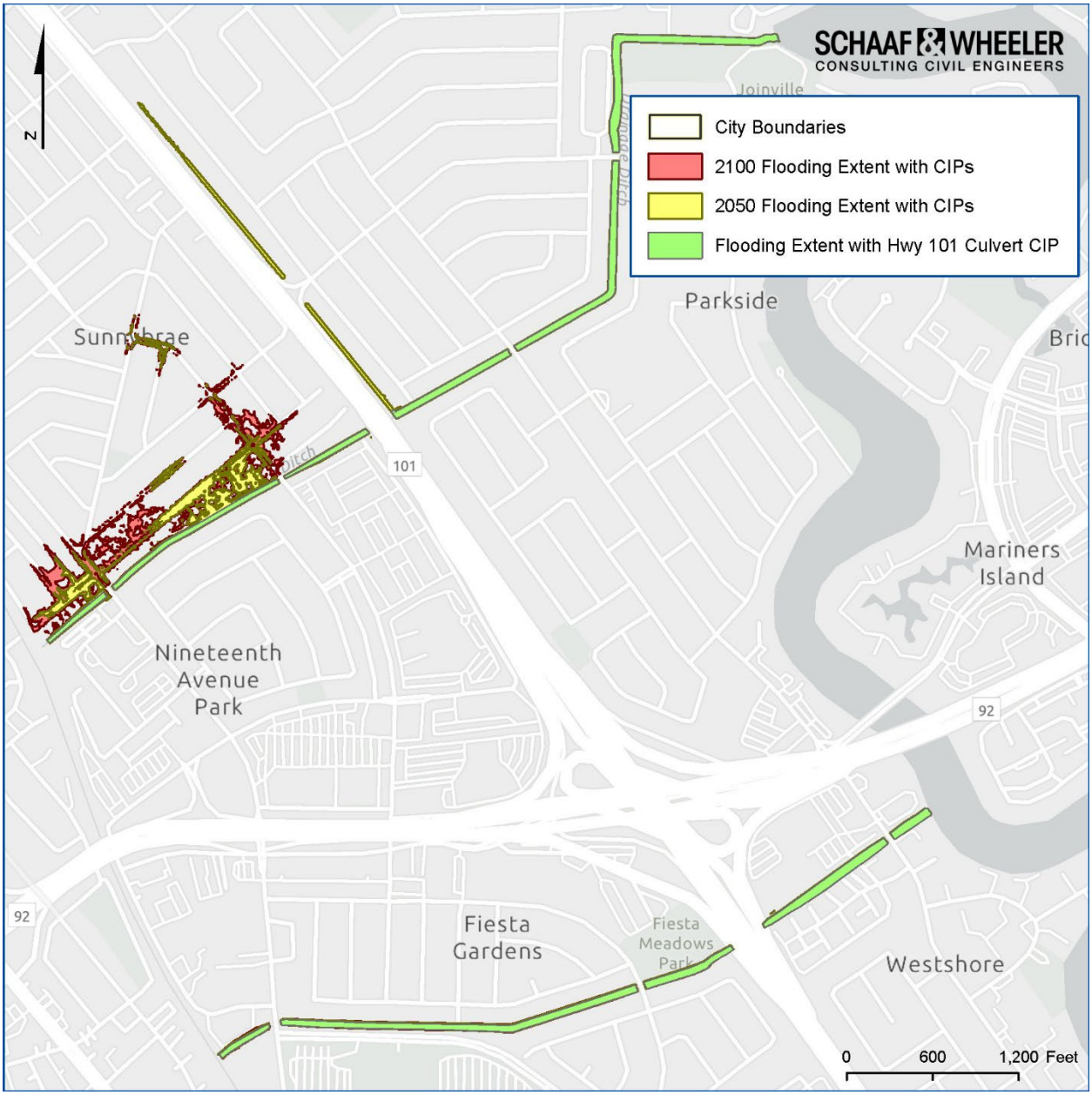


Figure 7-28: 16th and 19th Ave Channel HEC-RAS Model Inundation Areas (>0.1 ft Depth) for 100-year Storm—Existing and Climate Change Scenarios

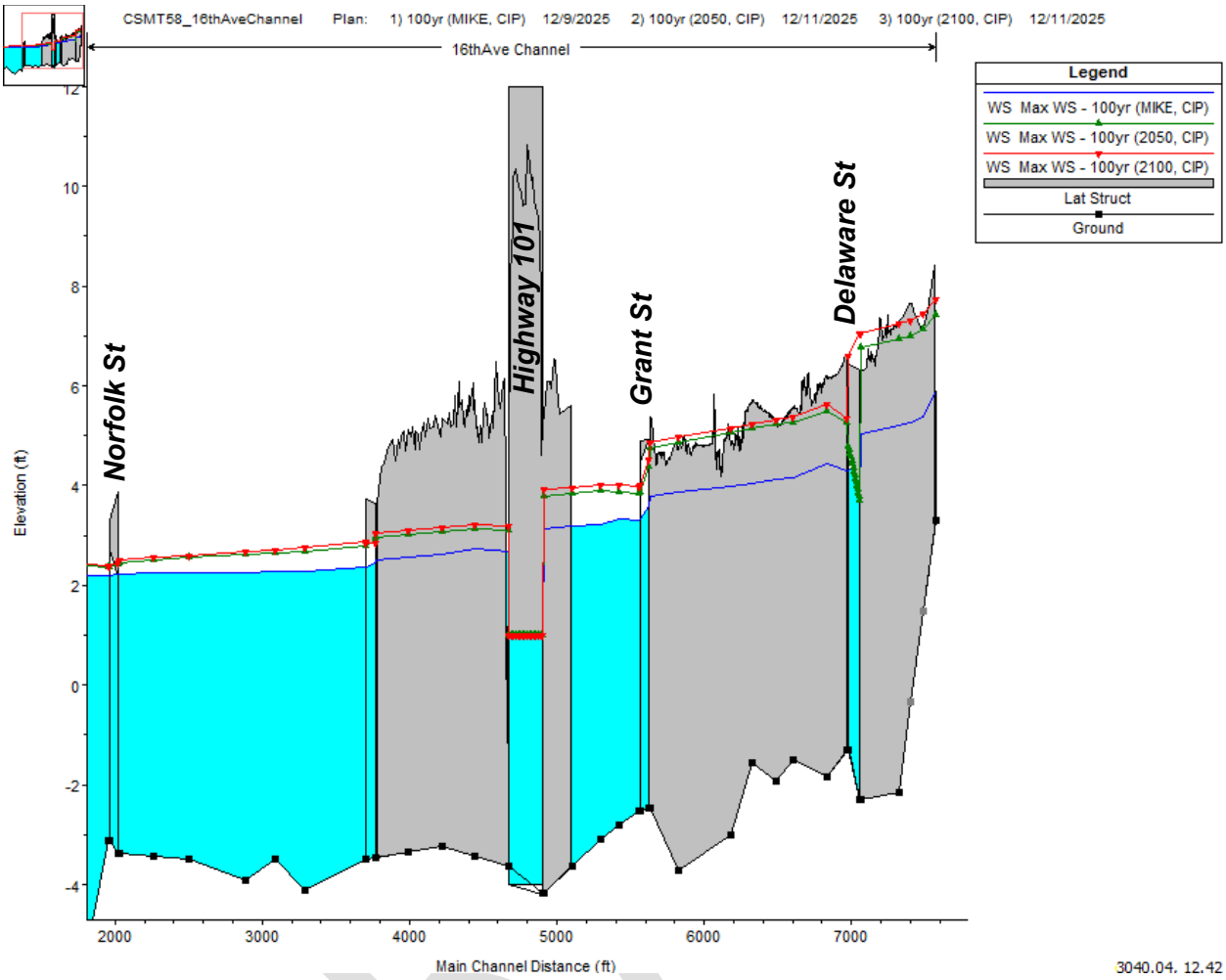


Figure 7-29: 16th Ave Channel Maximum Water Surface Profile for Existing 100-year Storm with CIP and Two Climate Change Models

While the proposed Highway 101 crossing performs well for the 2050 and 2100 events, the upstream crossings at Grant St and Delaware St become insufficient by 2050.

Four potential options for mitigating climate change impacts exist as follows:

1. Increase the size of the Highway 101 capital improvement project to minimize future tailwater on upstream crossings
2. Increase the size of the crossings at Grant St and Delaware St
3. Construct floodwalls where necessary along about 520 linear feet of channel upstream of Delaware St and 1,350 linear feet of channel between Delaware St and Grant St
4. Construct a large bypass culvert along 16th Ave taking flow from upstream of Delaware St to downstream of Grant St

Some combination of enlargement, floodwalls, and bypass capacity may be feasible as well. Depending upon the preferred strategy, existing outfalls may require flap gates and pipe system interconnects to prevent backwater flooding. The HEC-RAS model may be a useful tool in determining which projects may become necessary with future planning efforts.

The Grant St crossing is already a clear span bridge over the channel, and with adjacent property ownership constraints, upsizing the crossing may be challenging. In contrast, the Delaware St crossing is a box culvert, and enlarging it to create additional capacity may be more feasible.

For 19th Ave Channel, the hydraulic grade line (HGL) profile results for climate change scenarios are shown in Figure 7-30.

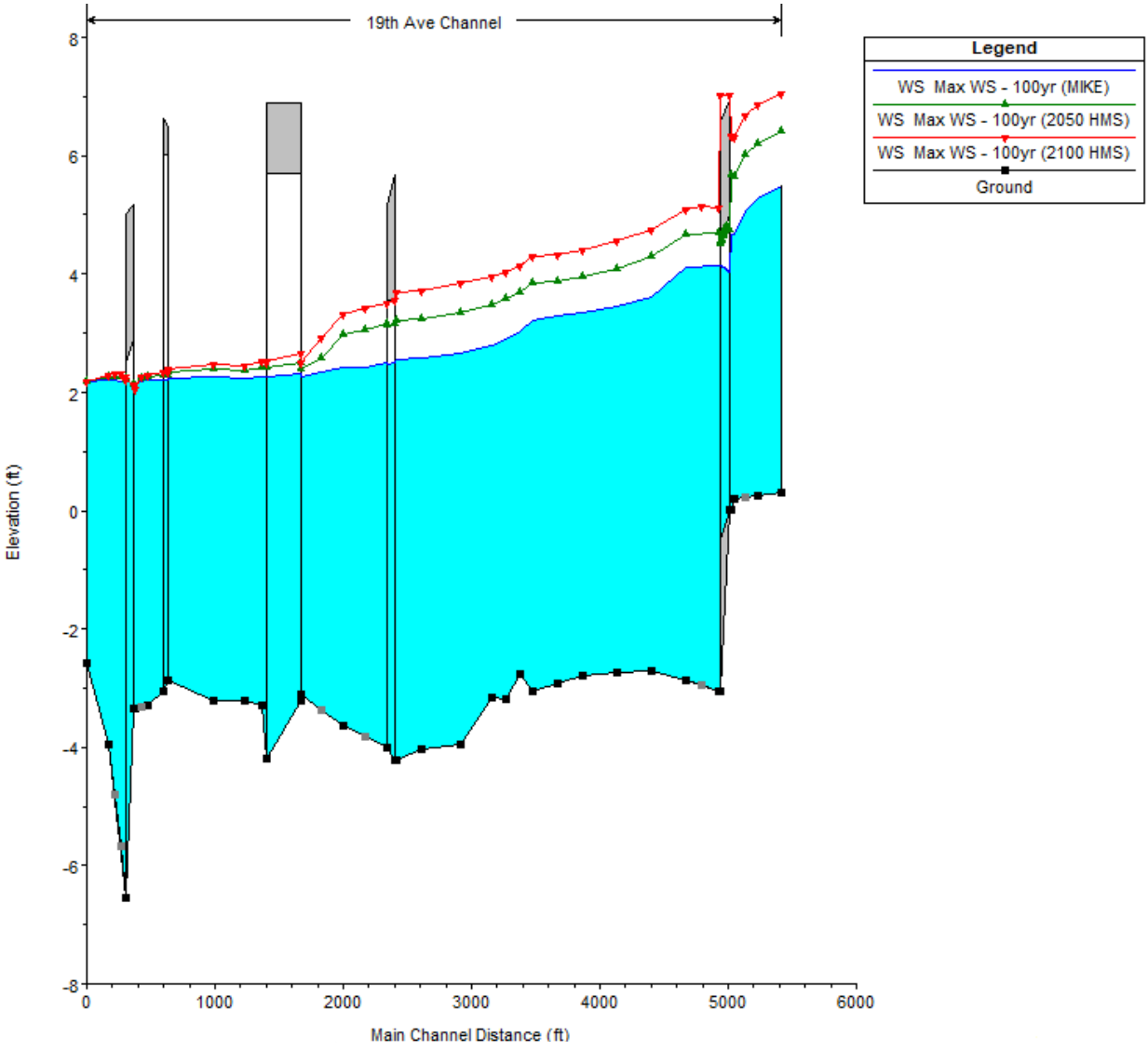


Figure 7-30: 19th Ave Channel Maximum Water Surface Profile for Existing 100-year Storm and Two Climate Change Models

The upstream-most crossing pressurizes with 2050 climate change estimates. A 2100 climate change scenario causes a total loss of freeboard in the creek upstream of the S Delaware St crossing. Small variations in vegetation management and roughness could then cause overtopping and flooding on the north side of 19th Ave Channel. While 19th Ave Channel is far less likely to be the direct cause of flooding with climate change, increased tailwater on connected stormwater systems is likely to require mitigation. Streets in the Fiesta Gardens

neighborhood north of the channel have gutter flowlines below the maximum HGL in the creek for a 100-year storm for the 2100 climate change scenario.

7.6.2.3 Laurel Creek

Flooding sources along Laurel Creek are similar to those for San Mateo Creek, except the Creek is not as heavily exposed to tidal waters. Instead, Laurel Creek is influenced by conditions in Marina Lagoon. The two dams also impound significantly less runoff than the Crystal Springs system.

The HEC-HMS models have been utilized to evaluate the effectiveness of the two existing Laurel Creek dams with climate change.

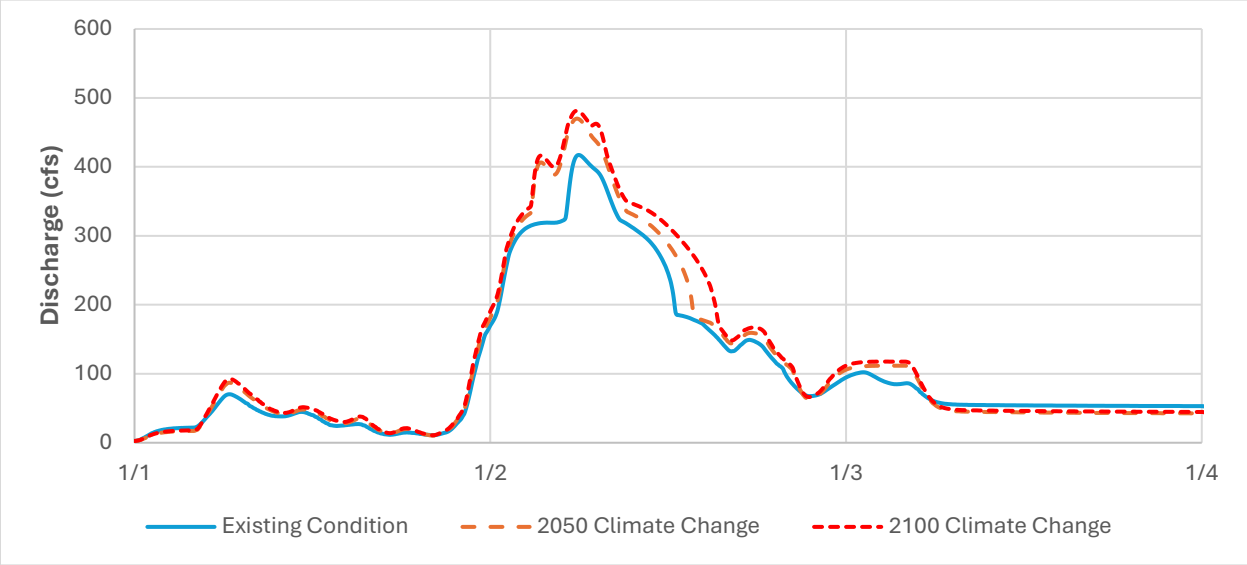


Figure 7-31: Combined HEC-HMS Discharge from Both Laurel Creek Dams

Increases in spill from the dams and increased runoff from downstream drainage areas accumulate and have the potential to more heavily impact the neighborhoods draining to Casanova Pump Station, where the Creek banks are already susceptible to overtopping during a 100-year storm. The HEC-RAS model has been updated, with a completed Curtiss Rd project and increased inflow boundaries to evaluate the change in flood risk in the vicinity of Highway 101 (Figure 7-32).

The model results primarily highlight the risk of overflow at the upstream end of the El Camino Real culvert. That spill accumulates in the nearby underpass; however, it quickly overwhelms the El Camino Real pump station, fills the underpass with more than 10 feet of water, and discharges along Hillsdale Blvd to the north. Discharge to Hillsdale Blvd accumulates in the neighborhood, where it is metered back into Laurel Creek via Casanova Pump Station.



Figure 7-32: Laurel Creek HEC-RAS Model Inundation Areas (> 0.1 ft Depth) for 100-year Storm Existing and Climate Change Scenarios

7.6.2.4 Marina Lagoon

The HEC-HMS climate change model provides a useful tool to rapidly evaluate the potential impact of climate change on Marina Lagoon, without running the entire City storm drain model. Two potential adaptation measures have been considered for Marina Lagoon to mitigate climate change impacts:

1. Operate the station within its existing footprint, with set levels lowered during winter to create additional flood storage for short-term mitigation, and add one pump by 2100
2. With the recommended pump station rehabilitation, provide two new pumps—adding at least one pump by 2050 and another by 2100

The first option would require set levels to drop by a full foot to achieve sufficient protection in 2050. This is not feasible without fully replacing existing engines and gear boxes to accommodate higher dynamic head conditions during storms.

The second strategy is more in line with the recommendations of the pump station condition assessment: constructing a new wet well that would accommodate operational flexibility with new pumps. Two sub-alternatives were evaluated in the model: new pumps with lower set levels (providing additional flood storage availability) and new pumps with set levels above the existing pumps.

HEC-HMS does not fully consider system hydraulics, storage in storm drain systems, streets, and channels connecting to the Lagoon. As a result, it overestimates peak water surface elevations compared with the MIKE+ model. However, it can still be used to evaluate additional pump capacity and changes in set levels to maintain the existing peak water surface elevation in the lagoon. The model was modified to include increased precipitation depths representative of estimated conditions in 2050 and 2100. With existing pump capacity and level settings, the results are shown in Figure 7-33.

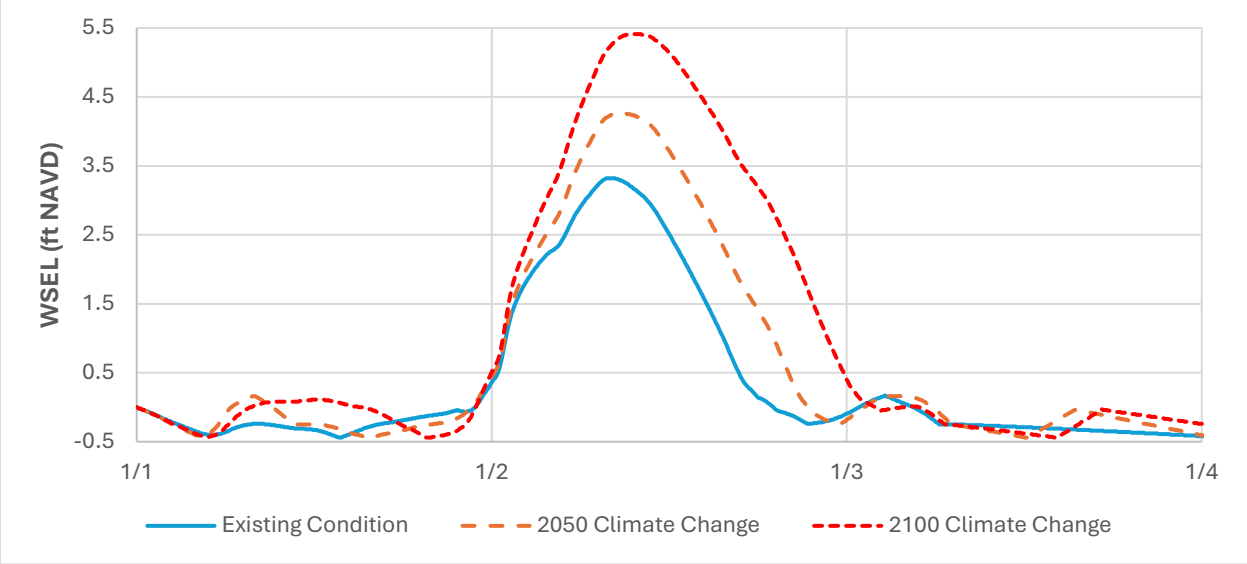


Figure 7-33: Marina Lagoon HEC-HMS 100-year Storm Water Level Results for Existing Conditions and with Increased Precipitation (Existing Pumps)

The HMS model has been used to evaluate the need for additional pump capacity to prevent flooding with greater 100-year storm influent. The pump station condition assessment has identified the need for greater operational flexibility than the current pumps offer, suggesting that an additional wet well should be constructed with new pumps that accommodate lower lagoon levels. This analysis builds on that recommendation to evaluate the necessity of those pumps for meeting flood prevention standards into the future.

The model has been modified, assuming that additional pumps are added with the same capacity as the existing pumps with the goal of maintaining or improving the existing station's level of flood protection.

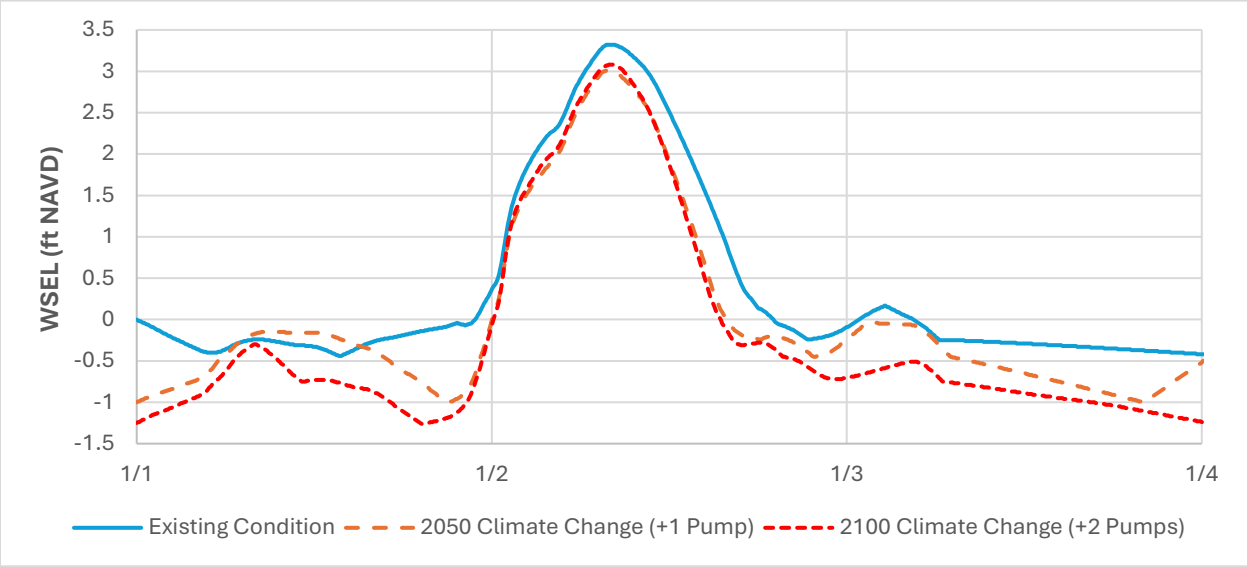


Figure 7-34: Marina Lagoon HEC-HMS 100-year Storm Water Level Results with Additional Pumps for Climate Change Scenarios

Even if new pumps are installed to provide capacity for future flood protection needs, SLR could eventually begin to impact the existing pumps. Given a 2100 intermediate SLR estimate of 3.1 feet, the 10-year tide level rises to 12.2 ft NAVD. This exceeds the elevation of the existing discharge lines and increases dynamic head conditions. At best, this would alter the design operating point on the pump curve, effectively decreasing station capacity, and at worst, it causes the existing motors to fail.

Based on the condition of the existing station (discussed in Chapter 5) and this evaluation of potential climate change impacts, Marina Lagoon pump station should be improved and rehabilitated in two phases. The first phase would be to construct a new wet well with a minimum of two new pumps, each matching the design capacity of the existing pumps and operating with a greater range of total dynamic head conditions. The second phase would be full replacement of all five existing pumps and motors to provide equal capacity to existing conditions and accommodate increased static head conditions.

7.6.3 Recommended Resilience Projects

Climate change modeling shows that many of the recommended projects are sized appropriately to provide resilience against changes in precipitation and sea levels. In some areas, recommended projects are not sufficient to mitigate flooding with increased precipitation or sea levels.

Additional improvements to creeks, gravity systems, and pump stations where climate change is most impactful are summarized in Table 7-14.

Table 7-14: Capacity Improvements for Climate Change Resilience

	Description	Climate Change Scenario	Cost Added to Baseline CIP Project	New Project or Baseline CIP Extension	Total Cost Impact
Gravity Systems	9th at RR	2100	$280,000	$140,000	**$420,000**
	Amphlett near 3rd Ave	2050	$2,760,000		**$2,760,000**
	Detroit PS Main from Newbridge	2100	$1,290,000	$880,000	**$2,170,000**
	Dore System Improvements	2050	$8,000,000		**$8,000,000**
	Pacific Ave Channel Overflow	2050	$1,560,000		**$1,560,000**
	RR Xing to 19th Ave Channel	2100	$790,000		**$790,000**
Creeks	16th Ave - Add Flap Gates	2100		$30,000	**$30,000**
	16th Ave - Hwy 101 Capacity	2100	$1,190,000		**$1,190,000**
	16th Ave - Delaware St Culvert	2100		$2,040,000	**$2,040,000**
	16th Ave - Grant St Culvert	2100		$1,070,000	**$1,070,000**
	19th Ave - Outfall Interconnect	2100		$1,970,000	**$1,970,000**
	Laurel Creek - Curtiss Bypass	2050	$1,640,000		**$1,640,000**
	Laurel Creek - Curtiss Rd Culvert	2100		$990,000	**$990,000**
	Laurel Creek - Edison St Culvert	2050		$9,630,000	**$9,630,000**
	Laurel Creek - El Camino Real	2050		$6,930,000	**$6,930,000**
	Laurel Creek - Fernwood Culvert	2050		$1,250,000	**$1,250,000**
	Laurel Creek - Otay Ave Culvert	2050		$610,000	**$610,000**
	Laurel Creek - RR Crossing	2050	$1,930,000		**$1,930,000**
	San Mateo Cr Levees (2050 CC)	2050	$1,150,000	$1,990,000	**$3,140,000**
	San Mateo Cr Tide Gate	2050	$280,000		**$280,000**
PS	4000 Pacific PS Additional Capacity for Climate Change	2050	$650,000		**$650,000**
				Total Climate Change Cost Impact:	**$49,050,000**

For recommended projects where climate change requires additional capacity beyond that determined in the baseline CIP model, the City may wish to budget for increased costs to accommodate climate change impacts. While capital cost may increase near-term to enlarge recommended baseline CIP projects, it is likely more cost-effective and less disruptive than constructing projects that do not provide resistance to climate change.

Additional improvements may be required upstream or downstream of the recommended baseline projects. In cases where a cost is identified for both a recommended project and a new project or baseline CIP extension in Table 7-14, the City may wish to expand the scope of

recommended projects to include adjacent improvements identified in the climate change capacity analysis.

For San Mateo Creek, projects that provide resilience to 2050 climate change are recommended. Beyond 2050, capacity requirements have been considered as if no changes occur to present day Crystal Springs system facilities and operations. Absent adaptation measures upstream by other parties, 2100 climate change adaptation measures on San Mateo Creek would require about $50–$200 million in capital improvements, including the following:

- Increasing the capacity of nearly every bridge and culvert
- Constructing roughly 2,500 linear feet of additional levee improvements beyond those required for 2050 climate change
- Constructing a pump station capable of discharging about 2,000 cfs

This magnitude of capital improvements should be considered alongside regional interagency coordination and further climate change impact study.

Most pump stations appear to provide adequate capacity to accommodate climate change, except the 4000 Pacific Pump Station, Detroit Pump Station, and Marina Lagoon Pump Station. The Marina Lagoon Pump Station Long-Term Improvement Project incidentally incorporates the additional capacity needed to handle increased inflows to the lagoon, so a separate climate change adaptation project is not recommended. Likewise, Detroit Pump Station's baseline improvements include updating the station with an emergency generator to provide reliable backup power to both pumps and ensure that adequate capacity is available. Therefore, additional improvements have been recommended only for the 4000 Pacific Pump Station to provide climate resilience.

7.7 Operations and Maintenance (O&M)

The storm drain and channel system cannot function if one of its components is plugged or blocked. Although hydraulic analyses show that criteria are met, blocked inlets, pipes, or channels will cause flooding.

CCTV and topside inspection identified portions of the pipe system affected by the accumulation of sediment and debris that may cause a loss of conveyance capacity. However, lagoons and pumping forebays also need to be monitored and periodically dredged to preserve design capacities.

A portion of the City's annual budget should be dedicated to restoring the capacity of closed conduit systems by completing O&M projects identified in this document and to acquiring the required permits to restore conveyance capacity for outlets and open channels.

Maintaining the more natural drainage features (i.e., open channels and lagoons) as conveyances is important. However, designation as wetlands and habitat complicates matters, as regulatory permits may be required to perform maintenance, and access is limited for significant reaches of the channel network. This is the case for many open channel drainage ways in San Mateo. Two particular locations have been identified where existing dense vegetation impacts channel capacity. Vegetation at the 16th Ave Channel does not appear to significantly increase flooding, though it could impact upstream pipe systems. On Laurel Creek, however, dense vegetation downstream of George Hall Elementary causes a considerable increase in water surface elevations and would increase the extent and depth of flooding in the

adjacent neighborhood in a 100-year storm event. The City should prioritize obtaining permits necessary to clear vegetation in that reach of the channel.

Open channel maintenance priorities include the following:

- Replace broken flap gates
- Manage vegetation, particularly at 16th Ave Channel and Laurel Creek between 29th Ave and 40th Ave)
- Manage sediment management in creeks, particularly where sedimentation may begin to impact outfalls
- Manage sediment in Marina Lagoon, particularly at the southern reaches where sediment influx from Laurel Creek and the O'Neill Slough Tide Gate most readily settles within the Lagoon's live flood storage operating range
- Keep debris racks clean and in a state of good repair

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Prioritized Improvement Program

This section provides a summary of estimated investment needs for the system to meet the target standards discussed in Chapters 4 and 5. More detailed project cost estimates (which include a line item summary of construction subtotals, mobilization, traffic control design/engineering, administration, contingency, permitting, and other special costs) are provided in Appendix L. A full summary of construction subtotals by pipe segment is provided in Appendix M. Detailed project sheets, including maps and descriptions of the required improvements are provided in Appendix N for Very High priority projects.

Table 7-15: Prioritized Capital Improvement Project Cost Estimate Summary

Priority	Project	Type	Score	Length (LF)	Approx. Cost
Very High	Marina Lagoon Short-Term	Pump Stations	88	–	$7,250,000
	Marina Lagoon Long-Term	Pump Stations	82	–	$100,000,000
	4000 Pacific Long-Term	Pump Stations	70	–	$3,680,000
	4000 Pacific Short-Term	Pump Stations	66	–	$960,000
	Laurel Cr–Curtiss Bypass	Creeks	64	1,560	$4,650,000
	16th Ave Channel -Highway	Creeks	62	220	$23,400,000
	Grant St at Sunnybrae Blvd	Gravity System	58	301	$430,000
	Newbridge Ave	Gravity System	58	1,772	$2,250,000
	42nd Ave Underpass	Gravity System	56	195	$230,000
	Norton St Ditch	Gravity System	48	2,800	$4,800,000
	Bettina-Kingridge (RR)	R&R	50	800	$647,000
High	Saratoga (RR)	R&R	49	1,006	$432,000
	Hillsdale Short-Term	Pump Stations	48	–	$960,000
	Carlisle	Gravity System	47	1,660	$1,960,000
	E Laurel Creek Culvert Inlet	Gravity System	47	10	$550,000
	Laurel Cr–RR Xing	Creeks	46	140	$4,830,000
	Hillsdale Long-Term	Pump Stations	45	–	$2,260,000
	Monte Diablo (RR)	R&R	45	654	$396,000
	Newbridge Ave Upstream	Gravity System	44	2,196	$2,320,000
	San Mateo Cr–Levees,	Creeks	43	1,700	$1,150,000
	Bellevue-Claremont	Gravity System	42	950	$660,000
	San Miguel (RR)	R&R	42	771	$224,000
	Cupertino (RR)	R&R	41	984	$563,000
	El Camino Real at Barneson	Gravity System	40	90	$610,000
	Ocean View (RR)	R&R	40	549	$512,000
	Pasadena Repair (RR)	R&R	39	437	$41,000
	3rd & Detroit Short-Term	Pump Stations	38	–	$1,150,000
	Grant-Birch	Gravity System	38	746	$800,000
	Oak Valley (RR)	R&R	37	442	$131,000
	Fallon (RR)	R&R	37	408	$271,000
	31st Ave Short-Term	Pump Stations	36	–	$120,000
	Palm (RR)	R&R	36	358	$47,000
	Fairfax (RR)	R&R	36	720	$94,000
	Coyote Short-Term	Pump Stations	35	–	$700,000
	Dakota Ave	Gravity System	35	211	$220,000

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Priority	Project	Type	Score	Length (LF)	Approx. Cost
	E Laurel Cr–Laurelwood	Creeks	34	175	$570,000
	Almaden (RR)	R&R	34	303	$196,000
High	42nd-Kingridge (RR)	R&R	34	276	$182,000
	David St (RR)	R&R	33	315	$152,000
	Idaho	Gravity System	32	1,327	$940,000
	S Delaware at 10th	Gravity System	32	1,551	$1,420,000
	Viewridge (RR)	R&R	32	230	$95,000
	Glendora (RR)	R&R	32	261	$173,000
	Parrot-CSM (RR)	R&R	31	207	$163,000
	Santa Clara (RR)	R&R	31	484	$252,000
Medium	Poplar Short-Term	Pump Stations	30	–	$300,000
	Parrot (RR)	R&R	30	233	$136,000
	Polhemus (RR)	R&R	29	139	$134,000
	Parkside (RR)	R&R	28	129	$148,000
	Ellsworth	Gravity System	28	615	$1,010,000
	San Mateo Cr–Levees, Tidal	Creeks	28	1,260	$710,000
	Juniper to 27th Ave	Gravity System	27	1,436	$1,440,000
	Edinburgh (RR)	R&R	27	270	$32,000
	Fathom Dr Short-Term	Pump Stations	26	–	$740,000
	Mason at Bedford Cr (RR)	R&R	26	20	$76,000
	Grant St	Gravity System	25	2,026	$1,280,000
	Amphlett 2	Gravity System	25	618	$360,000
	11th-Claremont	Gravity System	24	1,366	$850,000
	40th	Gravity System	24	310	$220,000
	9th at Laurel	Gravity System	24	409	$240,000
	3rd & Detroit Long-Term	Pump Stations	23	–	$7,800,000
	West Poplar	Gravity System	23	777	$400,000
	Edison-31st	Gravity System	22	330	$190,000
	Quebec-Dolan	Gravity System	22	747	$390,000
	Newlands-42nd (RR)	R&R	22	282	$186,000
	Edinburgh	Gravity System	21	649	$760,000
	King	Gravity System	21	476	$490,000
	17th Ave	Gravity System	21	977	$690,000
	3rd Ave at San Mateo (Bubble)	Gravity System	21	695	$360,000
	Hobart	Gravity System	21	1,115	$910,000
	Hacienda	Gravity System	21	1,286	$1,210,000
	Avila-Maple	Gravity System	21	235	$220,000
	B St	Gravity System	21	395	$240,000
	Laurie Meadows	Gravity System	20	1,350	$1,240,000
	Monte Diablo	Gravity System	20	1,251	$1,470,000
	Casanova Short-Term	Pump Stations	20	–	$180,000
	Sunnybrae Blvd	Gravity System	20	458	$250,000
	16th Ave Long-Term	Pump Stations	20	–	$860,000
	16th Ave Short-Term	Pump Stations	20	–	$530,000
	B St (Bubble)	Gravity System	20	772	$570,000

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Priority	Project	Type	Score	Length (LF)	Approx. Cost
Low	Hillsdale Frontage	Gravity System	19	1,552	$1,950,000
	Countryside (RR)	R&R	18	182	$112,000
Low	Jasmine Lateral	Gravity System	18	440	$340,000
	Shoreview	Gravity System	18	906	$640,000
	Birch Bubble	Gravity System	18	493	$220,000
	31st-Hillsdale Frontage	Gravity System	18	279	$250,000
	Maple	Gravity System	18	715	$520,000
	Peninsula (RR)	R&R	17	69	$66,000
	Quebec (RR)	R&R	17	120	$84,000
	Avila	Gravity System	17	1,530	$1,330,000
	9th at RR	Gravity System	17	236	$230,000
	W 39th (Bubble Conn)	Gravity System	17	798	$480,000
	Monte Diablo 2	Gravity System	17	1,844	$1,360,000
	Monte Diablo (Bubble)	Gravity System	17	124	$80,000
	9th (Bubble Conn)	Gravity System	16	1,571	$850,000
	40th at Casanova-Branson	Gravity System	16	618	$500,000
	Mission-Capistrano (Bubble	Gravity System	16	791	$400,000
	W 39th Main	Gravity System	16	803	$920,000
	Santa Inez (Bubble Up)	Gravity System	16	309	$150,000
	7th (Bubble Conn)	Gravity System	16	292	$150,000
	Isabelle-30th (Bubble)	Gravity System	16	218	$130,000
	40th (RR)	R&R	16	28	$24,000
	42nd Ave (RR)	R&R	16	651	$73,000
	E Laurel Cr–Channel Cap.	Creeks	15	350	$1,160,000
	Patricia (RR)	R&R	15	12	$36,000
	42nd at Edison	Gravity System	15	63	$90,000
	San Mateo Cr–Crystal Springs	Creeks	15	60	$940,000
	Leslie St	Gravity System	14	381	$270,000
	Warren-Clark	Gravity System	14	1,033	$820,000
	Edison-27th (w/Flap Gate)	Gravity System	14	754	$440,000
	31st at Hillsdale Park	Gravity System	14	474	$370,000
	Midvale (Bubble Conn)	Gravity System	14	103	$70,000
	Hacienda (Bubble Conn)	Gravity System	14	535	$300,000
	Trollman	Gravity System	12	617	$330,000
	Norfolk at Susan Ct	Gravity System	12	504	$380,000
	Donner to Colegrove	Gravity System	12	822	$490,000
	S Norfolk at 16th Channel	Gravity System	12	411	$770,000
	West 3rd-Parrot	Gravity System	12	657	$500,000
	ECR at Poplar	Gravity System	12	251	$230,000
	Flores-22nd	Gravity System	11	981	$1,260,000
	Flores-Edison (Bubble)	Gravity System	11	760	$400,000
	20th Ave	Gravity System	11	594	$230,000
	Baldwin	Gravity System	11	321	$210,000
	Kehoe at 16th Channel	Gravity System	11	158	$170,000
	Amphlett 1	Gravity System	10	527	$270,000

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Priority	Project	Type	Score	Length (LF)	Approx. Cost
	Indian	Gravity System	10	629	$390,000
	5th Ave	Gravity System	10	284	$250,000
Low	30th-Edison	Gravity System	10	383	$420,000
	Laurel Cr–Laurel Creek Rd	Creeks	9	50	$760,000
	Lodi-Kehoe	Gravity System	8	525	$550,000
	El Camino South of Lodato	Gravity System	8	395	$470,000
	10th-Idaho	Gravity System	8	670	$470,000
	ECR Beresford Lateral	Gravity System	8	69	$120,000
	Casanova Long-Term	Pump Stations	5	–	$105,000
	Shoreview (RR)	R&R	3	136	$26,000
	Woodberry (RR)	R&R	2	83	$26,000
	Alderwood (RR)	R&R	1	63	$71,000
	Fathom Dr Long-Term	Pump Stations	0	–	$70,000
			Total:	**75,670**	**$224,486,000**

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7.8.1 Pipe Capacity Improvement Program Summary

Costs by priority for pipe system capacity improvements are summarized in Table 7-16.

Table 7-16: Baseline System Capacity Project Cost Estimate Summary

Priority	Description	Number of Projects	Estimated Cost
Very High	Projects that address urgent, heavily disruptive, repetitive, or very high-risk flooding issues	4	$7,710,000
High	Projects that address high impact flooding or require the completion of a higher priority downstream project	9	$9,480,000
Medium	Projects that address moderate magnitude flooding or require completion of a high priority project	22	$14,790,000
Low	Projects that address low magnitude flooding with relatively little risk of damage/disruption	44	$20,770,000
	Total:	**79**	**$52,750,000**

7.8.2 Pipe Condition Improvement Program Summary

Repair and replacement project costs are summarized separately in Table 7-17. These are generally characterized by in-kind replacements (i.e. no change in pipe capacity is required) that replace aging systems that were identified by CCTV inspection as in poor condition. Their priority is defined by the severity of identified defects or capacity losses due to collapse or failures.

Table 7-17: Repair and Replacement Project Cost Estimate Summary

Priority	Description	Number of Projects	Estimated Cost*
Very High	Projects replacing completely failed pipe or pipe segments at risk of imminent failure due to corrosion, cracking, or joint failure	1	$650,000
High	Projects replacing failing pipe with significant but not total capacity loss	17	$3,920,000
Medium	Projects replacing pipe in poor condition with some capacity loss, but not likely to experience sudden failure	6	$710,000
Low	Projects replacing pipe in fair condition but showing signs of aging. With continued deterioration, these systems may become susceptible to failure	9	$520,000
	Total:	**33**	**$5,800,000**

*Rounded to nearest $10,000 increment

Repair and replacement projects include approximately 6,300 linear feet of inspected CMP, with a cost of about $3,200,000 (an average unit cost of about $510 per linear foot). Given the age and condition of much of the metal pipe in the City, replacement and repair of metal pipe makes up the greatest share of the recommended condition project costs. Although CCTV was biased to reaches deemed to be in poor condition based on surface inspection, it is reasonable to assume that additional replacement could be required.

There is approximately 21,070 linear feet of CMP or other metal pipe with attributed material type in the GIS, making up approximately 9% of the total length of pipe with a material type

assigned. About 61% of the system has no attributed material type, so the actual length of CMP in the system is uncertain. About 40% of the inspected CMP was assigned a structural NASSCO score of 4 or 5.

Taking the attributed system elements and the CCTV inspection results as representative of the entire system, there could be as much as 23,000 linear feet of CMP in need of repair or replacement. At an average cost of $510 per linear foot, an additional $11,000,000 in repair and replacement work may be necessary, though there is a high degree of uncertainty in that estimate. Repairs in the rest of the system also can't be grouped into projects or prioritized without additional condition assessment. Additional targeted CCTV would be beneficial to identify and prioritize other locations in need of repair that were not identified by this SDMP.

7.8.3 Creek Capacity Improvement Program Summary

The cost of creek capacity improvements required to provide 100-year level-of-service are summarized in Table 7-18.

Table 7-18: Creek System Capacity Improvement Project Cost Estimate Summary

Priority	Description	Number of Projects	Estimated Cost*
Very High	Projects providing capacity for numerous upstream projects and mitigating extensive or frequent flooding with potential for high levels of property damage	2	$28,050,000
High	Projects providing capacity for some upstream projects and mitigating moderate levels of flooding with some property damage	3	$6,650,000
Medium	Primarily standalone projects preventing rare creek flooding with limited impact	1	$710,000
Low	Standalone projects preventing very rare creek flooding with little risk of property damage or threat to life safety	3	$2,860,000
	Total:	**9**	**$38,270,000**

*Rounded to the nearest $10,000 increment

7.8.4 Pump Station Improvement Program Summary

The investment needed to keep pump stations operating smoothly and reliably is broken down into two categories. For each station, the Assessment identified items in need of near-term replacement or refurbishment full rehabilitation requirements. The cost of recommended pump station improvements by priority level are summarized in Table 7-19.

Table 7-19: Estimated Cost of Near-Term Pump Station Projects and Full Rehabilitation

Priority	Estimated Cost of Near-Term PS Improvements	Estimated Cost of Long-Term PS Improvements
Very High	$8,210,000	$103,680,000
High	$2,930,000	$2,260,000
Medium	$1,750,000	$8,660,000
Low	--	$180,000
Total:	**$12,890,000**	**$114,780,000**

Pump Station rehabilitation costs are discussed in greater detail in Appendix H and have been prioritized based on station age and equipment condition.

7.8.5 Total CIP Cost

Total estimated CIP costs by priority are summarized in Table 7-20. This represents the total amount of long-term investment required (in 2025 dollars) to meet the storm drain system standards laid out in this document where it is reasonable to do so and to reduce flood risk to a practical degree. The magnitude of improvement required at Marina Lagoon Pump Station, arguably one of the most impactful flood control facilities in the City, skews investment needs. Financing the project will require special consideration, so the prioritized CIP is presented both with and without long-term Marina Lagoon Pump Station improvements.

Table 7-20: Total Proposed Capital Project Cost Summary

Priority	Number of Projects*	Estimated Cost (in Millions)	Est. Cost without Marina Lagoon PS (in Millions)
Very High	11	$148.3	$48.3
High	34	$25.3	$25.3
Medium	35	$26.6	$26.6
Low	58	$24.3	$24.3
Total:	**138**	**$224.5**	**$124.5**

A breakdown of capital requirements for the stormwater system by category is provided in Figure 7-35. This does not include other maintenance and permit compliance activities (e.g. street sweeping, trash capture projects, education and outreach, etc.). Very High priority improvement costs by category are apportioned as shown in Figure 7-36 (with Marina Lagoon Pump Station) and Figure 7-37 (without Marina Lagoon Pump Station).

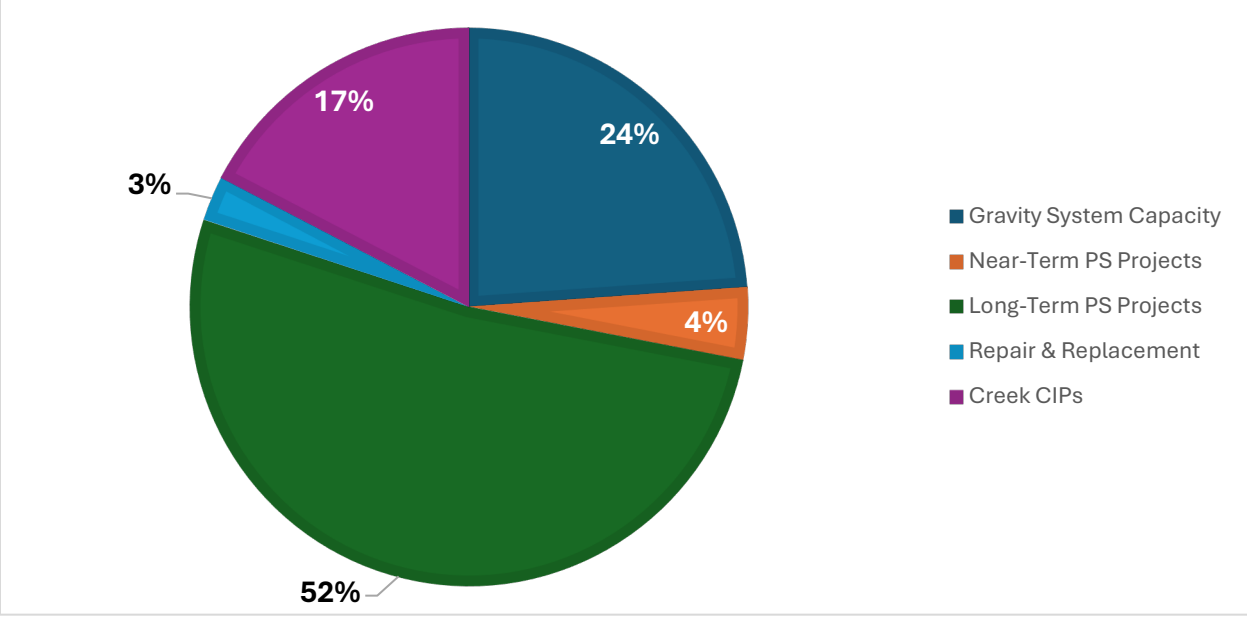


Figure 7-35: Stormwater Capital Spending Categories as a Proportion of Total (Full CIP)

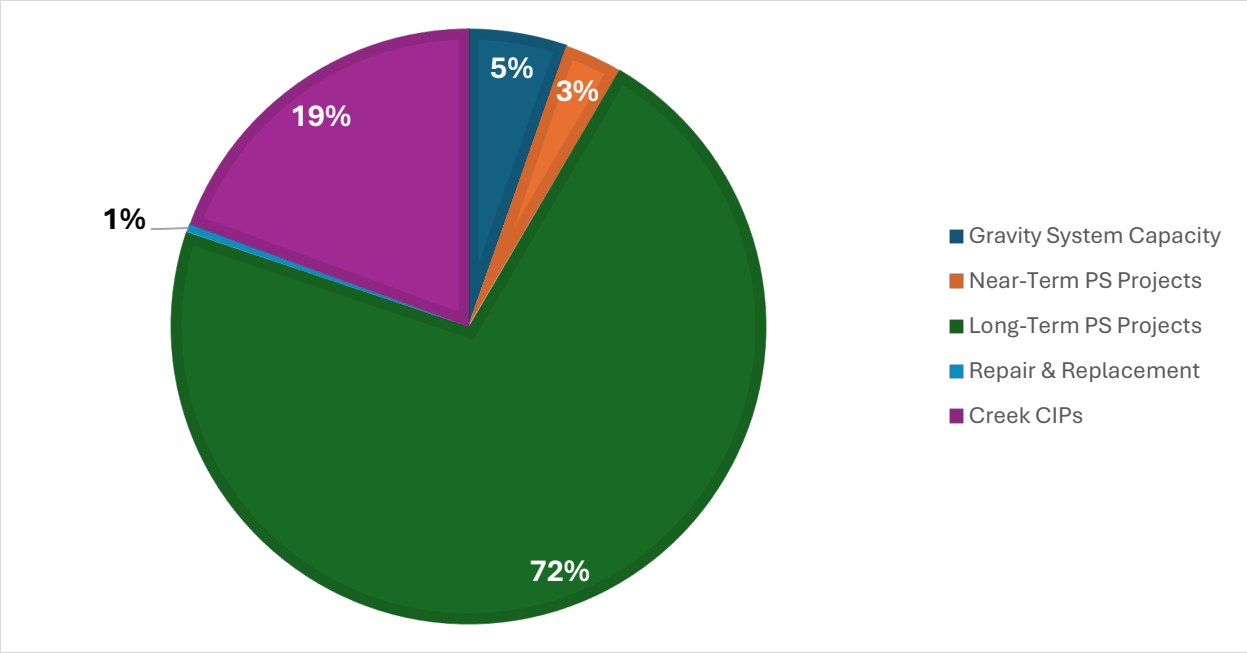


Figure 7-36: Stormwater Capital Spending Categories as a Proportion of Total (Very High Priority, Near-Term Recommended Plan)

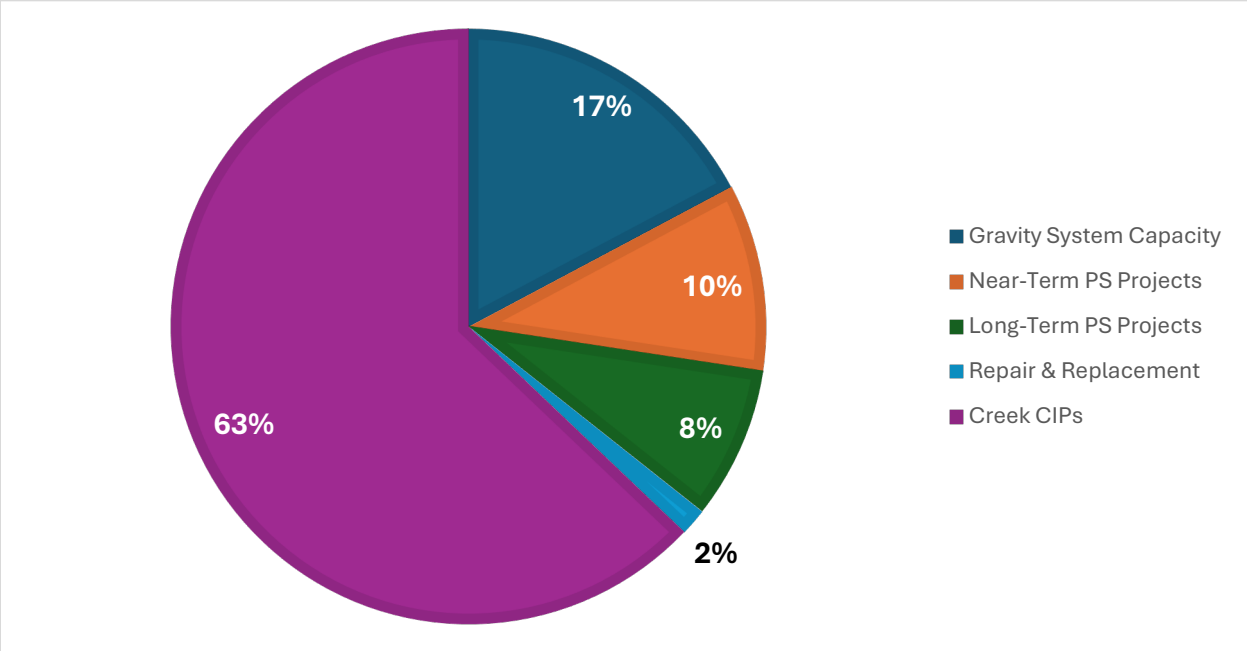


Figure 7-37: Stormwater Capital Spending Categories as a Proportion of Total (Very High Priority, Near-Term Recommended Plan, Less Marina Lagoon Pump Station)

7.8.6 System Maintenance

A detailed maintenance cost analysis has not been completed. However, CCTV footage shows that sediment and debris removal is needed throughout the system, and this document has identified essential open channel conveyances that must be regularly maintained. The City's stormwater budget should include a minimum of $100,000 annually dedicated to performing system maintenance, including sediment and debris removal and minor repairs.

Given the importance of open channel conveyance in the City's systems and the potential for flooding should those conveyances become blocked, regulatory permits allowing for regular sediment removal and vegetation management are recommended.

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7.9 Additional Considerations

7.9.1 Impacts of Climate Change on CIP Cost

Evaluating 2050 and 2100 climate change models provides the City an opportunity to construct more resilient projects today. Recommended resilience measures have been categorized into those targeted at mitigating 2050 climate change impacts and longer-term 2100 climate change impacts (Table 7-14).

Table 7-21: Overall Cost of Recommended Climate Change Adaptation Measures by Scenario

	Recommended by 2050		Recommended by 2100	
Project Type	**Cost Added to Baseline CIP Project**	**New Project or Baseline CIP Extension**	**Cost Added to Baseline CIP Project**	**New Project or Baseline CIP Extension**
Gravity Systems	–	$8,620,000	$810,000	$2,360,000
Creeks	$4,720,000	$20,690,000	$1,190,000	$6,100,000
Pump Stations	$650,000	--	--	--
Total:	**$4,020,000**	**$18,650,000**	**$2,000,000**	**$8,460,000**

Projects recommended to adapt to near-term (2050) climate change are sized to also accommodate long-term (2100) climate change. Projects required by 2100 are in addition to those needed by 2050.

The CIP is prioritized to focus on the most urgent needs first. While climate change resilience measures have an overall cost of over $38 million, a portion of that investment might be spent well before 2050, as High Priority or Very High Priority projects are completed. Of the costs added to baseline CIPs, $5,500,000 can be attributed to High Priority and Very High Priority projects.

7.9.2 Other Recommended Studies

This SDMP provides an overview of stormwater system condition, capacity, and infrastructure improvement needs. As such, a variety of additional, focused studies and investigations may be undertaken that can build a more comprehensive understanding of the condition and available capacity of existing systems, vulnerability to other factors such as SLR and subsidence, and regional efforts to prevent flooding. Additional studies that have been identified as beneficial efforts for the City to consider include the following:

- Additional condition assessment, primarily focused on low-lying areas with Highway 101 crossings
- SLR and subsidence vulnerability for coastal protection systems
- Comprehensive analysis of flood risk mitigation CIP cost for combined interior and coastal flooding sources, including SLR
- Regional planning effort focused on Crystal Springs Dam and San Mateo Creek

The condition assessment supporting this effort focused on the portions of the system that were most likely to need repair, replacement, or maintenance. As such, CCTV inspection did not reach every part of the system. The City may wish to identify additional reaches of the system where CCTV may be useful or necessary. This study has identified certain Highway 101 crossings that are in imminent need of additional capacity. However, smaller crossings were not initially part of the City's dataset. Many of these were instead located on as-built drawings in the

course of this planning effort and were added to the storm drain system GIS. The condition of a number of these crossings is not known to the City.

SLR vulnerability and subsidence are interconnected issues. The impact of SLR on the City's drainage systems has been considered by this SDMP, and some projects are recommended in coastally-influenced San Mateo Creek. However, SLR impact to coastal protection systems that protect the City from tidal flooding is not the focus of this SDMP and should be studied further.

A variety of subsidence studies and monitoring efforts exist in the Bay Area that can be referenced to inform the City of potential risks posed to low-lying areas. However, the most useful data comes from benchmark surveys within the City that evaluate actual subsidence rates and predicted subsidence magnitudes. That data should be used in conjunction with an up-to-date analysis of coastal flood protection systems to evaluate SLR vulnerability across the City's shorelines. While many recently completed flood protection projects provide a good degree of resilience against wave runup and SLR, projects built preceding current climate modeling and OPC adaptation guidelines (including portions of Coyote Point County Park, Bayfront Levee, San Mateo Creek floodwalls, and coastlines in the vicinity of Marina Lagoon Pump Station) should be evaluated in greater detail than that presented in this report.

San Mateo is part of OneShoreline, which has a number of efforts currently underway in San Mateo County to plan and build resilient shoreline protection. These will likely be an important part of further coastal protection planning efforts in the City.

Lastly, San Mateo Creek has a complex watershed. The creek conveys significantly attenuated discharge from a very large area outside of the City. Based on approximate modeling performed for this SDMP, the Crystal Springs Dam system and San Mateo Creek could be highly vulnerable to the impacts of climate change, and in particular, to increased extreme rainfall event intensities. The SFPUC-operated facility affects areas of the City and its neighbors when spill from the reservoir exceeds the capacity of the Creek. The City may be able to mitigate the impacts of near-term climate change without extensive interagency coordination.

Longer-term planning efforts with more refined models are needed to evaluate actual risk and capital improvement needs. This effort may include evaluation of additional reservoir management scenarios, forecast-informed reservoir operation (FIRO) planning, and future infrastructure improvements that balance flood mitigation with water supply needs throughout the SFPUC service area. SFPUC has indicated that they are working with other agencies to develop radar-based systems to monitor precipitation conditions and develop a more in-depth understanding of the drainage area's hydrology and operational needs.

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Funding

8.1 Overview

This chapter presents the funding strategies, with their implications, that are available to the City to fund capital improvement projects and maintenance program recommendations within this SDMP. This chapter presents a high-level overview of financial options.

Funding is required for equipment, infrastructure, problem-spot maintenance projects, and ongoing operations, maintenance, administration, and regulatory obligations associated with operation of the City's storm drain system. The City has a stormwater fee. Other means of funding projects are explored in this chapter that would enable the City to more efficiently implement the CIP.

Properly managing the implementation of the CIP, in addition to coastal protection projects not considered in this document, may also require establishing reserves and using debt financing.

8.2 Stormwater Fee

While California law does not enable municipalities to impose utility rates for stormwater management services, the storm drain system shares similarities to traditional utilities. Local agencies are able to pass fee initiatives.

In 2023, property owners in the City of San Mateo voted in favor of the Community Flood and Storm Protection Fee, which took effect in Fall 2024 and includes rate increases tied to the Consumer Price Index, with a 3% cap. Rate increases are required to be approved by Council annually and must not exceed projected program costs.

The fee varies by land use type and parcel area, with non-residential uses charged by impervious surface area and residential parcels charged purely by parcel size. Information about the City's stormwater fee can be found on the City of San Mateo Public Works Community Flood & Storm Protection Initiative website.

The stormwater protection fee is a critical source of funding for implementing the CIP presented in this document. In 2025, revenue from the fee amounted to $3.9 million. However, this is an annual fee, so supplementary funds to implement the CIP will be required, given the magnitude of estimated costs. Even over several years, loans, grants, or other funds will be required to efficiently and effectively implement projects identified in this plan.

8.3 Inter-Fund Loans

Revenues from the stormwater fee are managed in a dedicated Storm Enterprise Fund. The City's finance structure is comprised of a number of other funds, each with its own purpose. Loans may be taken from these other funds to plan and implement specific projects. These loans can be used to finance up-front costs for a project that exceed stormwater fee annual revenues.

The City also possesses a Sewer Enterprise Fund, used to maintain and implement capital projects for the wastewater collection system and treatment plant. The Sewer Enterprise Fund is likely the best candidate for a source of inter-fund loans, as the two systems are maintained within the same City department (Public Works). To complete projects, other jurisdictions have

utilized similar inter-fund loan agreements and policies between wastewater and stormwater funds.

8.4 Development Impact Fees

A development impact fee is a one-time fee imposed as a condition of approval on new development, infill, or redevelopment that creates new, unmitigated impermeable surface area. Development impact fees are authorized by Government Code 66000 et seq., created by the Mitigation Fee Act, and commonly referred to as “AB 1600” fees.

An impact fee may be function as either of the following:

3. A buy-in to existing infrastructure
4. The incremental costs of new facilities required to serve new development that will create additional impermeable surface areas

A combination of these two impact fees may also be used to repay existing customers for historical capital investments. However, these impact fees cannot be used to fund operating or maintenance costs, which must be met through the SDMP’s annual fees.

Should an impact fee be considered, it should include escalation tied to an index (e.g. the Consumer Price Index or ENR CCI), similar to the stormwater fee. Implementation of a development impact fee typically requires a nexus study and adoption by the Council prior to incorporation into the Comprehensive Fee Schedule.

8.5 Benefit-Assessment District

A benefit-assessment district assigns project costs in direct proportion to the benefits received. Benefit assessment districts are often formed for specific projects within a watershed. The only properties assessed are those that directly benefit from the projects and in direct proportion to that benefit.

The City has utilized assessment districts to fund past projects providing specific, measurable benefit to a specific subset of properties (e.g. South Bayfront Levee Improvements). The use of these assessment districts pre-date the existence of the City’s stormwater fee.

8.6 Grants

Grants can be an important source of funds for capital improvements, and particularly for large, multi-benefit, multi-year projects that exceed the annual stormwater fee revenue. Grants for stormwater and flood control generally target large-scale projects. However, opportunities may be identified to bundle smaller projects together for grant applications or fold smaller capital improvement projects into a larger project earmarked for potential grant funding.

Grant sources from FEMA may include the following:

- **FEMA Building Resilient Infrastructure and Communities (BRIC).** This grant program provides funding to projects that mitigate risks posed by natural hazards and protect life and property from future disasters. The program requires that applicants have projects identified in a local hazard mitigation plan
- **FEMA Flood Mitigation Assistance (FMA).** This grant program assists with both planning and implementation of flood mitigation projects, including small local flood reduction projects and plans to prevent flood damage. This program also requires that

applicants have projects identified in a local hazard mitigation plan. Applications by local governments must be submitted to CalOES because they are considered sub-applicants.

This SDMP clearly identifies projects that meet the eligibility requirements for BRIC and FMA grants. Both grant programs provide a 75% federal and 25% state and local match. FEMA strongly encourages integration of green infrastructure into applicant projects. Applications for both grants must include FEMA benefit-cost analysis (BCA)

At the state level, California administers grant funding through the Integrated Climate Adaptation and Resiliency Program (ICARP). This program offers funding through various grants to public agencies and other organizations to plan and implement projects that advance climate resilience and mitigate climate risks. Some of these grants are focused on regional efforts through inter-agency partnerships. San Mateo is already part of OneShoreline, an organization with the express purpose of working across jurisdictional boundaries to develop projects that mitigate climate change impacts. San Mateo also coordinates with SFPUC and neighboring jurisdictions on issues surrounding San Mateo Creek. The Bay shoreline and San Mateo Creek are both regional flooding sources with potential vulnerability to climate change. Projects in this SDMP may be eligible for current or future ICARP grant programs.

Many grant programs favor projects with climate resilience and green infrastructure elements. Working with other departments or programs within Public Works to incorporate those elements into a project can be beneficial in building a competitive grant application. For example, a large flood control project may be identified on a street that requires widening or sidewalk improvement as part of a transportation plan document and is a candidate for green infrastructure design.

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