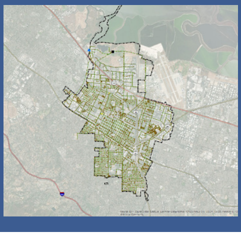
City of
Mountain View



Sewer Master Plan

Prepared by HydroScience Engineers
for the City of Mountain View | October 2022

HydroScience

City of Mountain View

Sewer Master Plan

Prepared by HydroScience Engineers, Inc.

October 2022

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LIST OF ACRONYMS AND ABBREVIATIONS

ADWF	average dry weather flow
APN	assessor's parcel number
ASTM	American Society for Testing and Materials
BRIC	Building Resilient Infrastructure and Communities
BSF	base sanitary flow
Cal OES	California Governor's Office of Emergency Services
CEQA	California Environmental Quality Act
CIP	cast iron pipe, capital improvement program/plan
City	City of Mountain View
CLEEN	California Lending for Energy and Environmental Needs
CMP	corrugated metal pipe
d	wastewater depth
D	pipe diameter
DIP	ductile iron pipe
DU	dwelling unit
DWF	dry weather flow
ESDC	engineering services during construction
ft	feet, foot
ft/s	feet per second
FY	fiscal year
GIS	Geographic Information System
GP	General Plan
gpd	gallon(s) per day
gpd/acre	gallon(s) per day per acre
gpd/DU	gallon(s) per day per dwelling unit
gpm	gallon(s) per minute
GSAP	Grade Separation and Access Project
GWI	groundwater infiltration
H_2S	hydrogen sulfide
HDPE	high density polyethylene
HDR	high-density residential
HGL	hydraulic grade line
hp	horsepower
HydroScience	HydroScience Engineers, Inc.
HVAC	heating, ventilating, and air conditioning
in	inch(es)
InfoSWMM	InfoSWMM by Innovyze®
InfoWorks	InfoWorks ICM by Innovyze®

LIST OF ACRONYMS AND ABBREVIATIONS

ISRF	Infrastructure State Revolving Fund Program
IT	Information Technology
LDR	low-density residential
LMDR	low medium-density residential
MCC	motor control center
MGD	million gallons per day
Moffett Field	Moffett Federal Airfield
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NPS	nonpoint source
O&M	operations and maintenance
OSHA	Occupational Safety and Health Administration
PF	public facilities
PG&E	Pacific Gas and Electric
PVC	polyvinyl chloride
PDWF	peak dry weather flow
PWWF	peak wet weather flow
RCP	reinforced concrete pipe
RDI/I	rainfall-dependent inflow and infiltration
RTU	remote telemetry unit
RWQCP	Regional Water Quality Control Plant
SDR	special drawing right
SPS	Shoreline Pump Station
SSO	sanitary sewer overflow
STL	steel
SWRCB	State Water Resources Control Board
TDH	total dynamic head
TM	technical memoranda
UF	unit flow
UPS	uninterruptible power supply
UWMP	Urban Water Management Plan
v	volt
VCP	vitrified clay pipe
VFD	variable frequency drive
WWF	wet weather flow
WWTP	wastewater treatment plant

SECTION 1 – INTRODUCTION AND PURPOSE

HydroScience Engineers (HydroScience) was retained by the City of Mountain View (City), to prepare the City's Sewer Master Plan (Master Plan), which includes development of a Capital Improvement Plan (CIP) based on the current and future (2030) planning horizons. This section outlines the background and Master Plan objectives.

1.1 Background

The City is located in Santa Clara County at the south end of the San Francisco Bay (see **Figure 1-1**), roughly 35 miles south of San Francisco and 10 miles north of San Jose. It is bounded by the City of Sunnyvale to the east, the City of Los Altos to the south, the City of Palo Alto to the west, and the San Francisco Bay to the north. Today, the City is an urban, industrial, and residential community serving a population of 82,376 according to the 2020 U.S. Census.

The City's last Sewer Master Plan (SMP) was prepared in 2010 and served as an update to the 1990 SMP. This Master Plan was completed in conjunction with the City's 2022 Water Master Plan, which is incorporated by reference in this report.

1.2 Objectives

The objectives of this Master Plan project are to:

- Conduct a comprehensive review and update of the City's wastewater collection system hydraulic model,
- Evaluate the hydraulic performance of the City's collection system under current conditions and under projected future conditions,
- Identify any hydraulic capacity deficiencies, and
- Develop a CIP and time schedule to address any hydraulic deficiencies and support system reliability based on a prioritized list of criteria.

These objectives were developed in collaboration with City staff.

1.3 Previous Studies/Existing Documents

The following is a brief description of previous studies and documents reviewed and used in the preparation of this Master Plan.

- **City of Mountain View Sewer Master Plan (August 2010):** The 2010 SMP served as a comprehensive update to the 1990 SMP. The 2010 Update addressed changes to the hydraulic model, revised growth assumptions, design criteria, and recommendations for hydraulic improvements. Additionally, the 2010 Update provided infrastructure condition replacement recommendations.
- **City of Mountain View Dry Weather Sanitary Sewer Flow Monitoring Study (November 2014):** This program was completed during the month of October 2014 for the purpose of measuring wastewater flows in the City's collection system during dry weather. The resulting data was used for the calibration of dry weather flows in the hydraulic model for this Master Plan. This report is included as **Appendix A**.
- **Los Altos Trunk Sewer Capacity Analysis (June 2020):** To accurately evaluate the Los Altos Trunk Main, flows from the City of Los Altos need to be included. Flows from this analysis were also utilized for this Master Plan.
- **City of Los Altos Sanitary Sewer Master Plan Update (February 2013):** The City of Los Altos' existing SMP includes details on the development of the Los Altos Trunk Main hydraulic model and was used for reference.
- **Alternative Trunk Sewer Alignment and Constructability Study (2017 Alternative Alignment Study) (March 2017):** The City conducted an alignment and constructability study to evaluate the option of building a pipeline to divert a large portion of the City's flows directly to the San Antonio Metering Station by gravity, bypassing the Shoreline Pump Station (Shoreline PS, SPS). The results of this study were used to facilitate the bypass interceptor analysis completed as part of this Master Plan (**SECTION 10**). This report is included as **Appendix B**.

The following technical memoranda (TMs) were completed in conjunction with this Master Plan. Where appropriate, details and summaries of findings from these TMs have been included in the body of the Master Plan Report.

- **TM #2 Water and Sewer System Modeling Software Evaluation:** This TM presented an evaluation of available software programs from the two leading software vendors, Innovyze and Bentley, for both water and sewer hydraulic modeling. The software evaluation included multiple criteria including: purchase and subscription pricing; available features and applications; software stability; software accuracy; platform compatibility needs/interface (i.e. ArcGIS, AutoCAD, SCADA); ease of use; technical support; and vendor reputation. It was recommended that the City transition to InfoWorks ICM for the sewer model, which is a fully dynamic modeling software that has the capability to model complex flow conditions including flow splits, of which there are several in the City's system. This allows for better calibration and ultimately allows the City to model the collection system more accurately.

- **TM #3 Water and Sewer Basic Assumptions and Criteria:** This TM presented the basic assumptions and performance criteria for hydraulic model evaluation for both water and sewer systems. For the sewer system, design storm options were presented for evaluating the collection system. The options were intended to provide a tiered approach starting with the most conservative 10-year, 24-hour storm with the peak hour coinciding with the diurnal peak. The prior analysis utilized a 10-year, 4-hour storm. Criteria for identifying capacity deficiencies were also presented in this TM.
- **TM #4 Water and Sewer Hydraulic Model Data:** This TM presented the data and process used to update and build the water distribution system and sewer collection system hydraulic models. For the sewer collection system hydraulic model, the TM summarized the hydraulic model data sources and the process implemented to rebuild and update the model. A brief discussion on the model flow development was also included.

In addition to the TMs completed, a wet weather flow monitoring study was completed, as described below:

- **City of Mountain View 2021 Sewer Flow Monitoring and Inflow/Infiltration Study:** This flow monitoring was completed from February to April 2021 to measure wastewater flows in the City's collection system during wet weather. The resulting data was used for the calibration of rainfall-dependent inflow and infiltration in the hydraulic model for this Master Plan. This report is included as **Appendix C**.

1.4 Report Organization

This Master Plan consists of 12 sections followed by appendices that provide supporting documentation for the analyses present in the body of the report. The sections are as follows:

- **SECTION 1 – Introduction and Purpose:** This section presents the background leading to the development of this Master Plan, a description of previous studies, and objectives and organization of the Master Plan.
- **SECTION 2 – Service Area and Wastewater Services:** This section describes the City's wastewater service area and the related wastewater services.
- **SECTION 3 – System Description:** This section describes the existing wastewater collection system and the overall system operation within the service area.
- **SECTION 4 – Condition Assessment:** This section summarizes the results of the onsite condition assessment of the Shoreline Pump Station (SPS) performed in July 2020 as well as the closed-circuit television (CCTV) inspection of the SPS pump influent channels.
- **SECTION 5 – Land Use:** This section outlines the existing land use categories provided by the City and details the methodology for using these land use categories for the development and calibration of the hydraulic model.
- **SECTION 6 – Hydraulic Model Development:** This section documents the process and assumptions associated with building the hydraulic model infrastructure and supplementing data gaps.
- **SECTION 7 – Flow Monitoring Programs:** This section presents the results of the dry weather (2014) and wet weather flow monitoring (2021) and the initial analysis of that data.

- **SECTION 8 – Wastewater Flow Analysis:** This section presents the development and calibration of wastewater flows used in the hydraulic model.
- **SECTION 9 – Capacity Analysis:** This section details the scenarios developed as part of the hydraulic model development and reviews the results and notable deficiencies identified under each scenario.
- **SECTION 10 – Bypass Interceptor Analysis:** This section presents the hydraulic analysis, recommendation, and associated cost estimate of a bypass interceptor to intercept the three major trunk lines in the collection system to flow by gravity to the San Antonio Metering Station, bypassing SPS.
- **SECTION 11 – Capital Improvement Program:** This section presents the recommended capital improvement projects, costs, and timeline for implementation.
- **SECTION 12 – References:** This section provides a list of the references used in the development of the Master Plan.

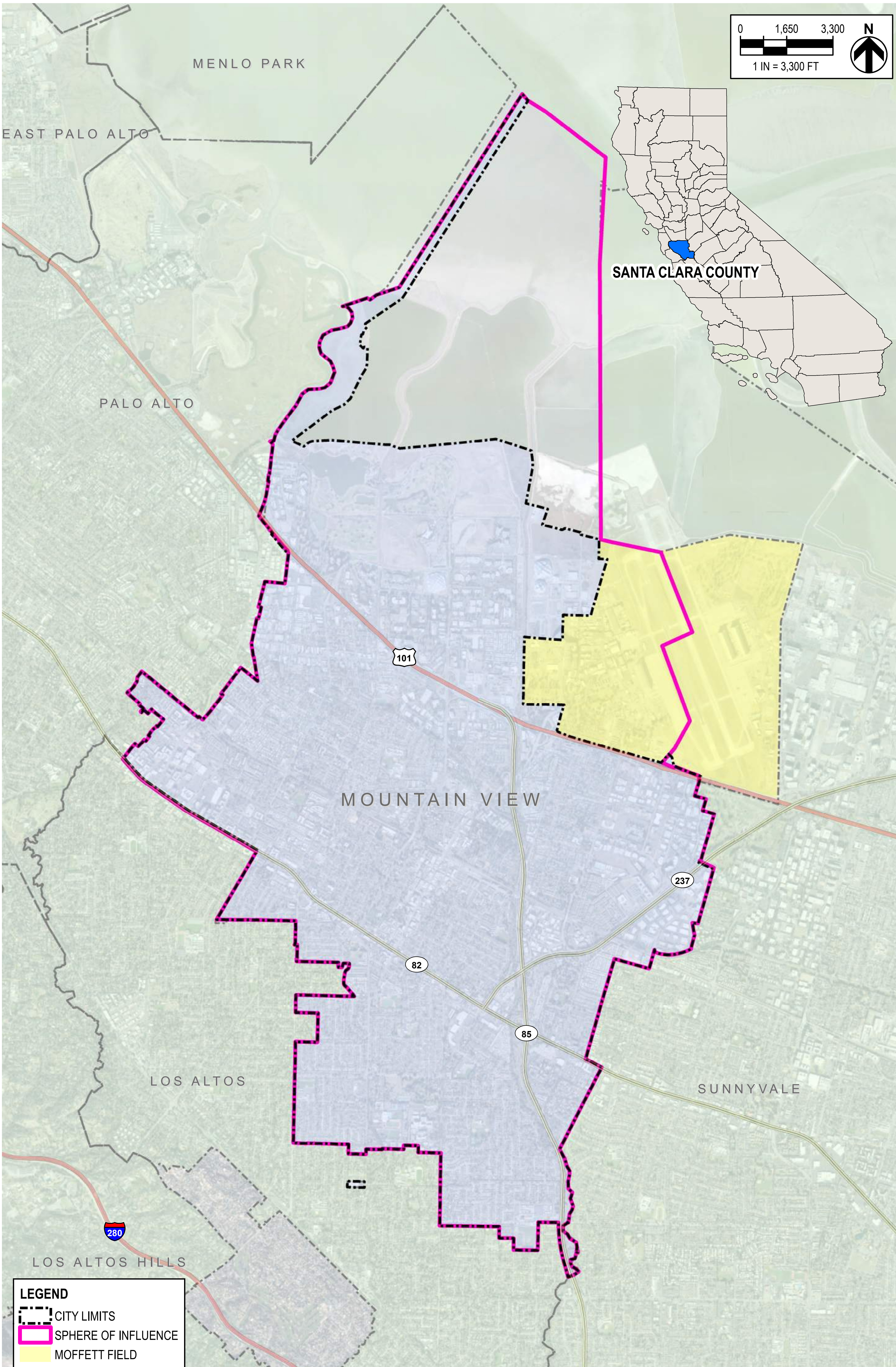

FIGURE 1-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
PROJECT LOCATION MAP

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SECTION 2 – SERVICE AREA AND WASTEWATER SERVICES

This section presents the City's service area as well as an overview of the wastewater services provided. The City's collection system will be detailed further in subsequent sections.

2.1 Service Area

The City provides wastewater collection services to all residents within the City limits as well as some in neighboring areas where topography supports gravity collection efficiency. Several users in the City of Los Altos along the southern border of the City discharge into the City's collection system as well as a group of homes in Palo Alto along the western edge of the City. For this Master Plan study, the contributing parcels in each of these areas were verified with each of the respective cities.

Moffett Federal Airfield (Moffett Field), located on the northeastern border of the City, falls within the sphere of influence of both the cities of Mountain View and Sunnyvale. It is not served by the potable water distribution system of either city, but it does contribute wastewater flow to both cities' collection systems.

Conversely, there are a number of City parcels that discharge flow into the City of Los Altos Trunk Main including a collection of parcels in the western corner of the City as well as some parcels in the northwestern part of the City.

Figure 2-1 displays all parcels contributing flow to the City's collection system as well as the City parcels discharging to the Los Altos Trunk Main. The approximate area within Moffett Field flowing to the City's collection system is also outlined

2.2 Wastewater Collection, Treatment, and Disposal

Elevations in the City gently slope down from south to north from approximately 210 ft to 3 ft. In general, the City's collection system carries flow from the south to the north through a collection of gravity collectors and interceptors funneled into three main trunk lines: the West Trunk, Central Trunk, and East Trunk. Flow from these three trunks is fed to the Shoreline Pump Station (SPS) at the northern low point of the City. There is a group of approximately ten residences that is served by a small lift station on Pastel Ln – Pastel Lift Station (Pastel LS) – in the southern half of the City due to the local topography in the area.

The site of the current-day SPS used to serve as the City's independent wastewater treatment plant (WWTP). In 1968, the cities of Mountain View, Los Altos, and Palo Alto came together to share in the cost of building a regional secondary treatment plant in anticipation of increased state regulation on the disinfection of wastewater effluent. The Palo Alto Regional Water Quality Control Plant (RWQCP) was constructed in 1972 at which point the cities of Mountain View and Los Altos retired their respective treatment plants.

The existing influent pump station of the City's wastewater treatment plant was then repurposed to pump flow to the RWQCP. Today, SPS flows are pumped through a 42-inch (in) pipeline (Interceptor Trunk) which typically flows by gravity to the San Antonio Metering Station on San Antonio Rd near the intersection of Casey Ave along the western border of the City. There, the City's flows are recorded before entering the 72-in joint interceptor with City of Los Altos flows flowing to the RWQCP.

The parcels in the western corner of the City that flow to the Los Altos Trunk Main pass through a flow meter at the intersection of Alma St and San Antonio Ave (Alma Recorder). The Los Altos Trunk Main carries wastewater flow from the City of Los Altos and the Town of Los Altos Hills to the San Antonio Metering Station where the flow is recorded and also enters the 72-in joint interceptor flowing to the RWQCP. The Alma Recorder is scheduled to be replaced and the configuration of the pipelines in the area reconstructed in 2023, pending easement negotiation with the City of Palo Alto, to provide improved hydraulics for measuring flows at this location.

Figure 2-2 presents an overview of the existing wastewater transmission and treatment facilities, and **Table 2-1** presents a summary of the total existing flows in each trunk according to the calibrated hydraulic model (detailed herein).

Table 2-1: Collection System Flow Summary

Trunk	ADWF (MGD)	ADWF (% of Total)	PWWF (MGD)	PWWF (% of Total)
West	2.85	45%	7.80	43%
Central	1.22	20%	3.48	19%
East	1.59	25%	5.23	29%
Los Altos	0.65	10%	1.75	9%
Total	**6.31**	**100%**	**18.25**	**100%**

The RWQCP is a tertiary wastewater treatment facility that currently serves East Palo Alto Sanitary District and the cities of Palo Alto, Mountain View, Los Altos, the Town of Los Altos Hills, and Stanford University. Treated wastewater is discharged to the Palo Alto Baylands or Renzel Marsh which flows to Matadero Creek and ultimately to the San Francisco Bay. Some treated wastewater is returned to participating cities – the cities of Palo Alto, East Palo Alto, Mountain View, Los Altos, the Town of Los Altos Hills, and Stanford University – as recycled water for irrigation and other non-potable uses.

According to the *Basic Agreement Between the City of Palo Alto, the City of Mountain View and the City of Los Altos for Acquisition, Construction and Maintenance of a Joint Sewer System* (Basic Agreement), each of the participating agencies is permitted to discharge an allotted quantity of wastewater to the RWQCP as presented in **Table 2-2**. As determined by the Basic Agreement, the contributing flow of each agency is calculated as average dry weather flow (ADWF) times a factor of 1.05.

Table 2-2: Discharge Capacity Rights

Agency	Allotted Flow Capacity (MGD)
City of Palo Alto	21.1
City of Mountain View	15.1
City of Los Altos	3.8
Total	**40.0**

Notes:
1. Sewage received by the party of the agreement from outside their territorial limits will be regarded as part of the party's capacity allocation. Thus, the capacity listed for the City of Palo Alto includes East Palo Alto Sanitary District and Stanford University and the capacity listed for the City of Los Altos includes the Town of Los Altos Hills.

The Basic Agreement also stipulates that each city is required to perform an engineering study to redefine future needs when flow from its respective area reaches 80% of its allotted flow capacity. According to the hydraulic model developed for this Master Plan, the City will not reach 80% of its allotted flow capacity of 15.1 MGD by 2030. Additionally, the 2022 Water Master Plan , consistent with the City's 2020 UWMP, projects a 2030 water use of 10.8 MGD. Conservatively assuming a return-to-sewer ratio of 80%, this translates to approximately 8.6 MGD of wastewater flow which is less than 60% of the City's allotted flow capacity at the RWQCP per the Basic Agreement.

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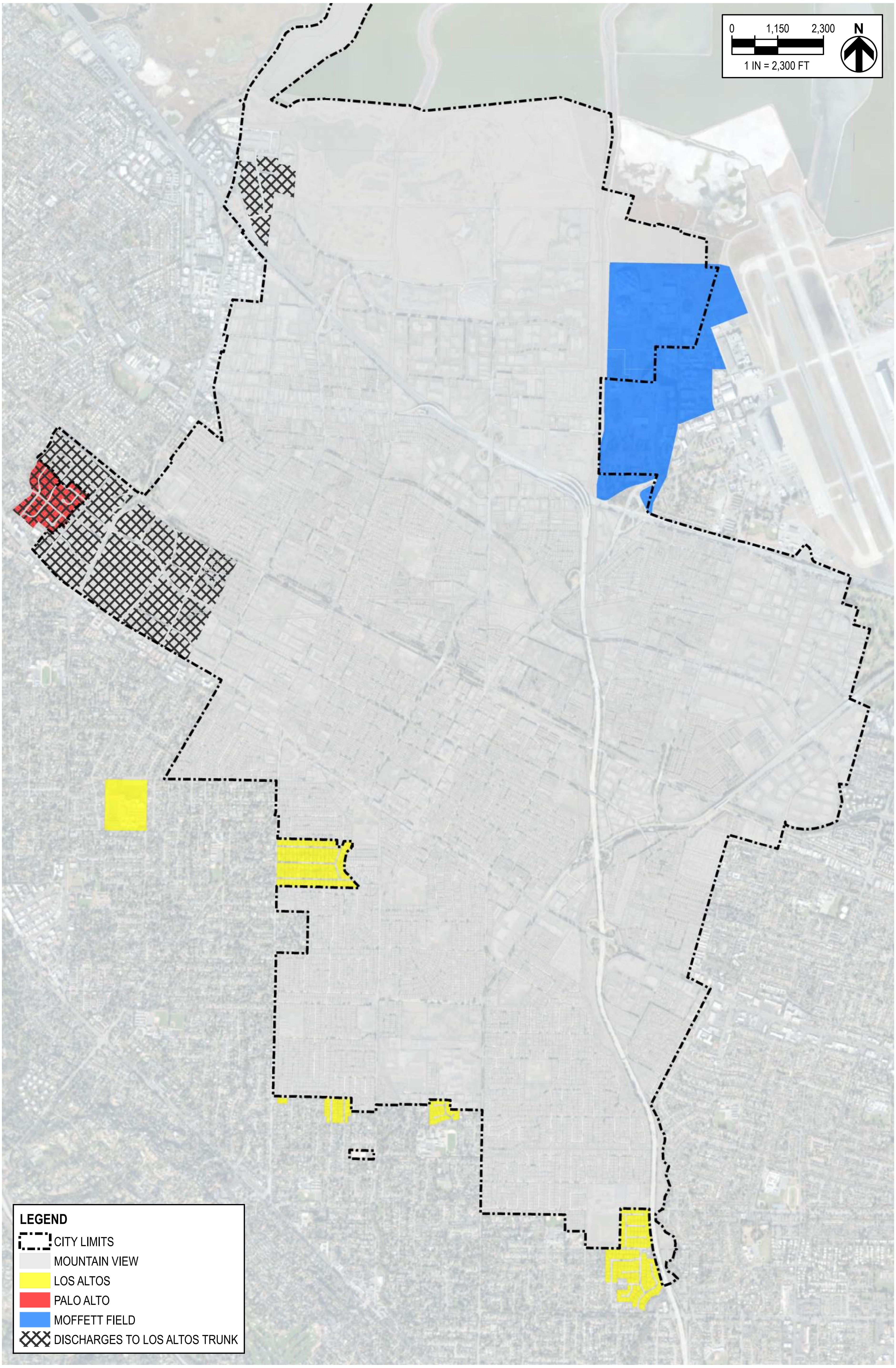

FIGURE 2-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
WASTEWATER SERVICE AREA

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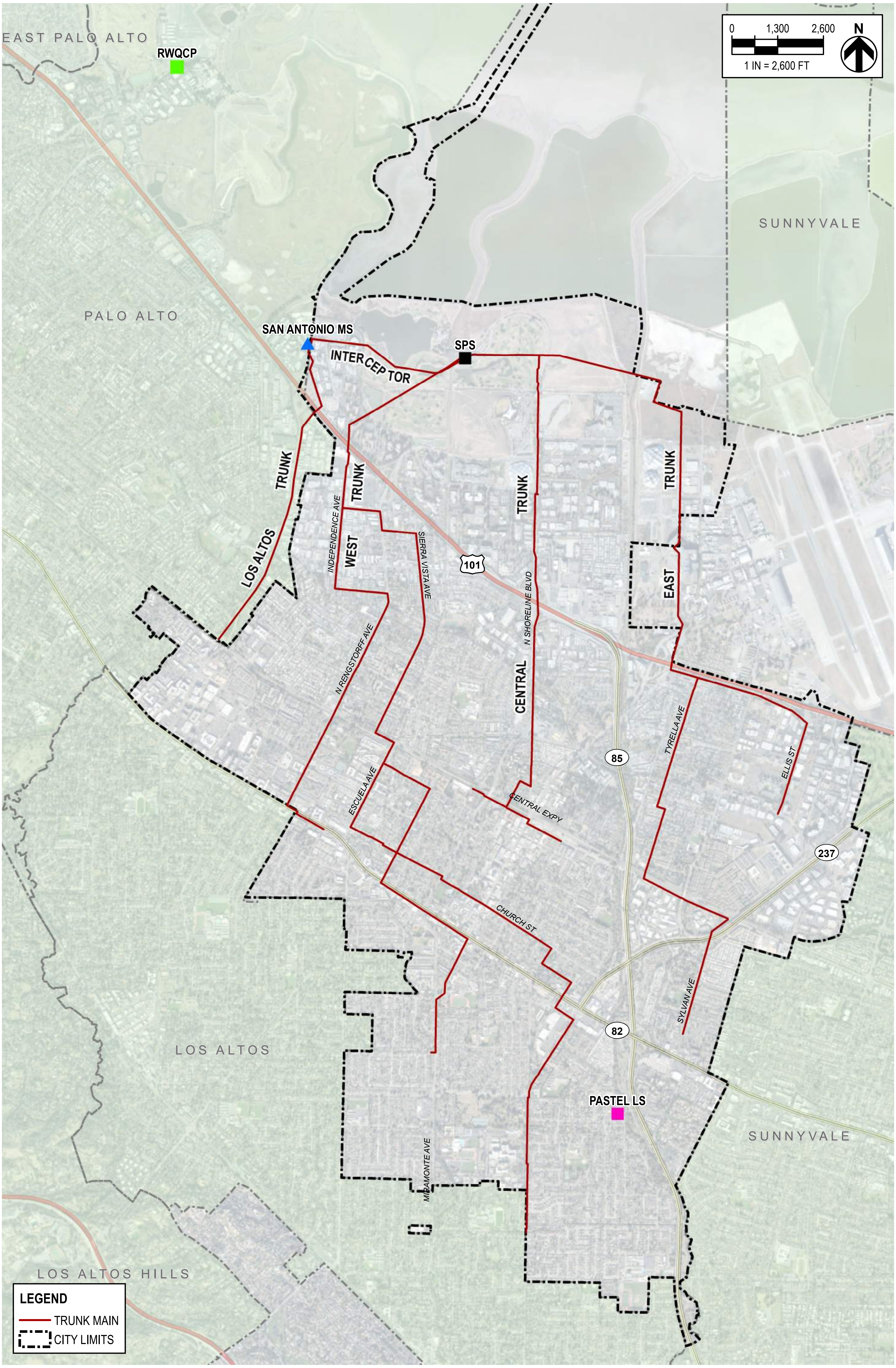

FIGURE 2-2
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
TRANSMISSION AND TREATMENT FACILITIES

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SECTION 3 – SYSTEM DESCRIPTION

This section provides a detailed description of the City's existing wastewater collection and conveyance system, an overview of which is presented on **Figure 3-1**.

3.1 Pipelines and Manholes

According to the City's GIS geodatabase, the wastewater collection system consists of approximately 157 miles of gravity sewers ranging from 4-in to 48-in in diameter. This does not include sewer mains owned and maintained privately within the City. Approximately 81% of the system is vitrified clay pipe (VCP) and approximately 93% of all pipe materials are 6-in to 15-in in diameter. As pipelines are replaced, it is likely that the percentages of high-density polyethylene (HDPE) and polyvinyl chloride (PVC) pipes in the system will increase as these are the materials primarily used for replacement today. **Table 3-1** provides a detailed breakdown of the collection system pipeline diameter and material based on the City's GIS geodatabase as well as updates incorporated from as-built drawings for CIP projects implemented since 2010. **Figure 3-2** and **Figure 3-3** display the pipeline diameters and materials, respectively, throughout the City.

As part of the City's preventative maintenance program, there are areas of the system that receive monthly, quarterly, or semi-annual cleaning while the remainder of the system is cleaned on an annual basis. Included in problem areas that receive monthly cleaning are three single siphons and one double inverted siphon that divert flow under Permanente Creek. Each of these four siphons are identified in **Figure 3-1**.

Based on the City's GIS geodatabase, the collection system includes approximately 3,500 manholes and cleanouts.

3.1.1 Flow Splits

A total of 165 flow splits were identified throughout the City's collection system. Flow splits are manholes that allow outflow via two or more pipes. There are four types of flow splits within the City's collection system, each identified in **Figure 3-4** and described as follows:

- **High point flow split** – high points from which wastewater flows in opposing directions (see **Figure 3-5**);
- **Even flow split** – manholes where incoming flow is split proportionally in two (or more) directions and outflow pipes have matching inverts (see **Figure 3-6**);
- **Overflow flow split** – manhole flow splits with overflow. At an overflow split during normal flow conditions, the flow takes one path and during high flow conditions, if the manhole begins to surcharge and flow reaches a certain depth, there is an "overflow" pipeline that is activated and will carry flow (see **Figure 3-7**); and
- **Flow diversion structure** – alters an existing even flow split to mimic an overflow flow split (see **Figure 3-8**). As can be seen in the figure, during normal dry weather operating conditions, all inflow is directed to one outflow pipe by the diversion structure. During higher flow conditions as the manhole begins to surcharge, the diversion structure is overtopped and some flow is diverted to an overflow pipe.

Table 3-1: Wastewater Collection System Pipelines by Material and Diameter

Diam (in)	Length of Pipe (ft)											
	CIP	CIPP	CMP	DIP	HDPE	PVC	RCP	STL	VCP	Unknown	Total	Total as %
4-in	-	-	-	-	-	58	-	-	220	441	**719**	<1%
6-in	113	-	-	-	-	2,586	104	70	129,026	570	**132,469**	16%
8-in	888	-	-	941	63,222	34,256	-	-	370,069	1,375	**470,751**	57%
10-in	335	-	-	281	3,822	2,985	-	-	65,600	22	**73,045**	9%
12-in	-	-	-	-	5,194	3,729	-	-	38,981	-	**47,904**	6%
14-in	-	-	-	-	2,345	-	-	-	-	-	**2,345**	<1%
15-in	242	-	-	-	1,611	593	6	-	35,603	16	**38,071**	5%
16-in	304	-	-	-	1,222	-	-	-	-	-	**1,526**	<1%
18-in	319	-	1,338	-	-	14	-	-	6,962	24	**8,657**	1%
21-in	-	353	-	-	-	-	-	-	12,715	-	**13,068**	2%
24-in	-	-	-	-	-	-	-	-	2,278	15	**2,293**	<1%
27-in	-	-	-	-	-	-	4,402	-	3,883	-	**8,285**	1%
30-in	-	-	-	-	-	-	3,605	-	3,333	-	**6,938**	<1%
33-in	-	-	-	-	-	-	5,811	-	-	-	**5,811**	<1%
36-in	-	573	-	-	-	455	3,019	-	-	-	**4,047**	<1%
39-in	-	1,417	-	-	-	-	323	-	-	-	**1,740**	<1%
42-in	-	1,886	-	-	-	-	5,349	63	-	-	**7,298**	<1%
48-in	-	-	-	-	-	-	59	-	-	28	**87**	<1%
Unknown	-	-	-	-	-	-	-	-	-	3,432	**3,432**	<1%
Total	**2,201**	**4,229**	**1,338**	**1,222**	**77,416**	**44,676**	**22,678**	**133**	**668,670**	**5,923**	**828,486**	-
Total as %	<1%	<1%	<1%	<1%	9%	5%	3%	<1%	81%	<1%	-	-

Notes:

1. CIP = Cast Iron Pipe
2. CIPP = Cured-in-Place Pipe
3. CMP = Corrugated Metal Pipe
4. DIP = Ductile Iron Pipe
5. HDPE = High-Density Polyethylene
6. PVC = Polyvinyl Chloride
7. RCP = Reinforced Concrete Pipe
8. STL = Steel
9. VCP = Vitrified Clay Pipe

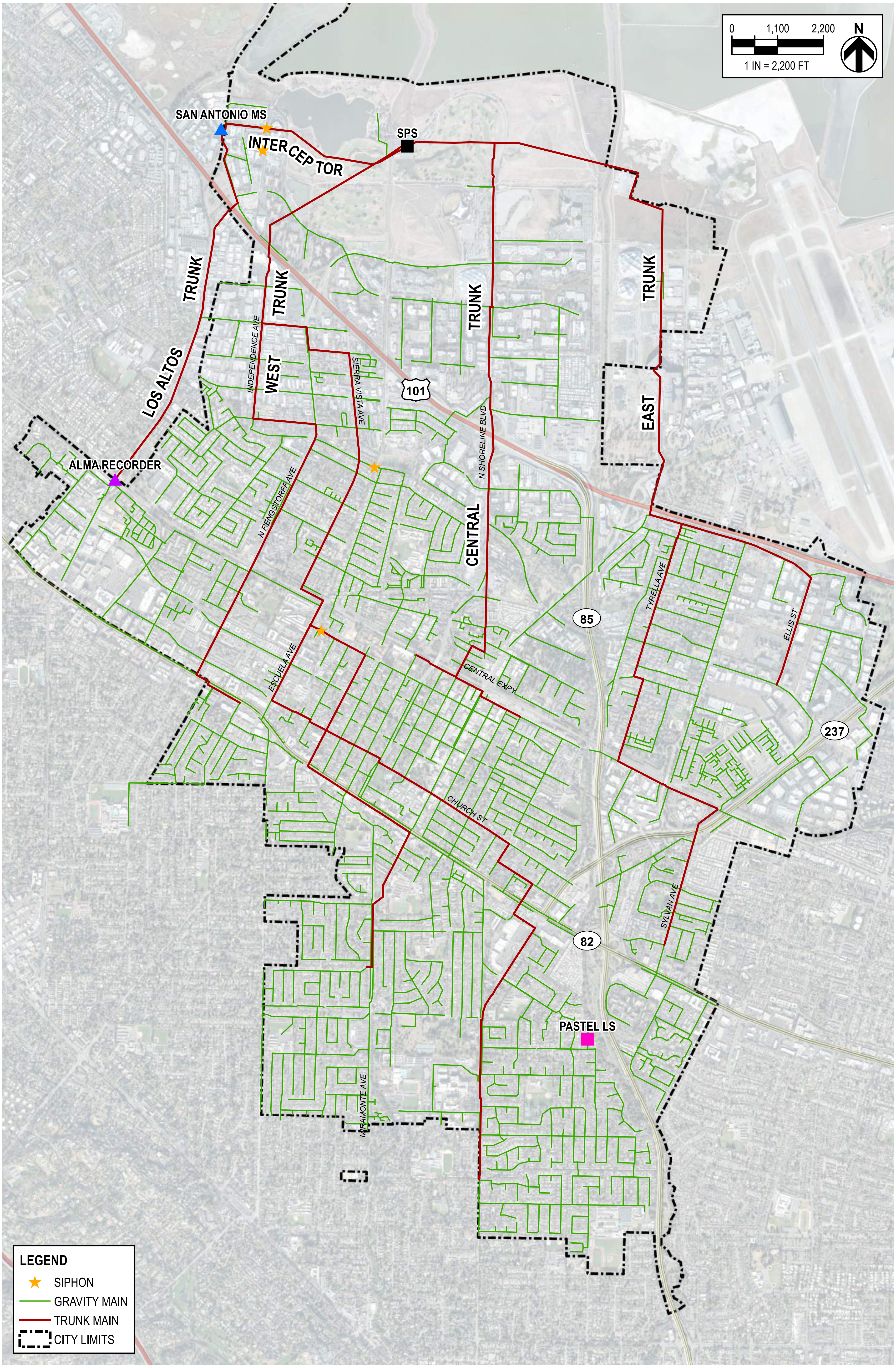

FIGURE 3-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
COLLECTION SYSTEM FACILITIES

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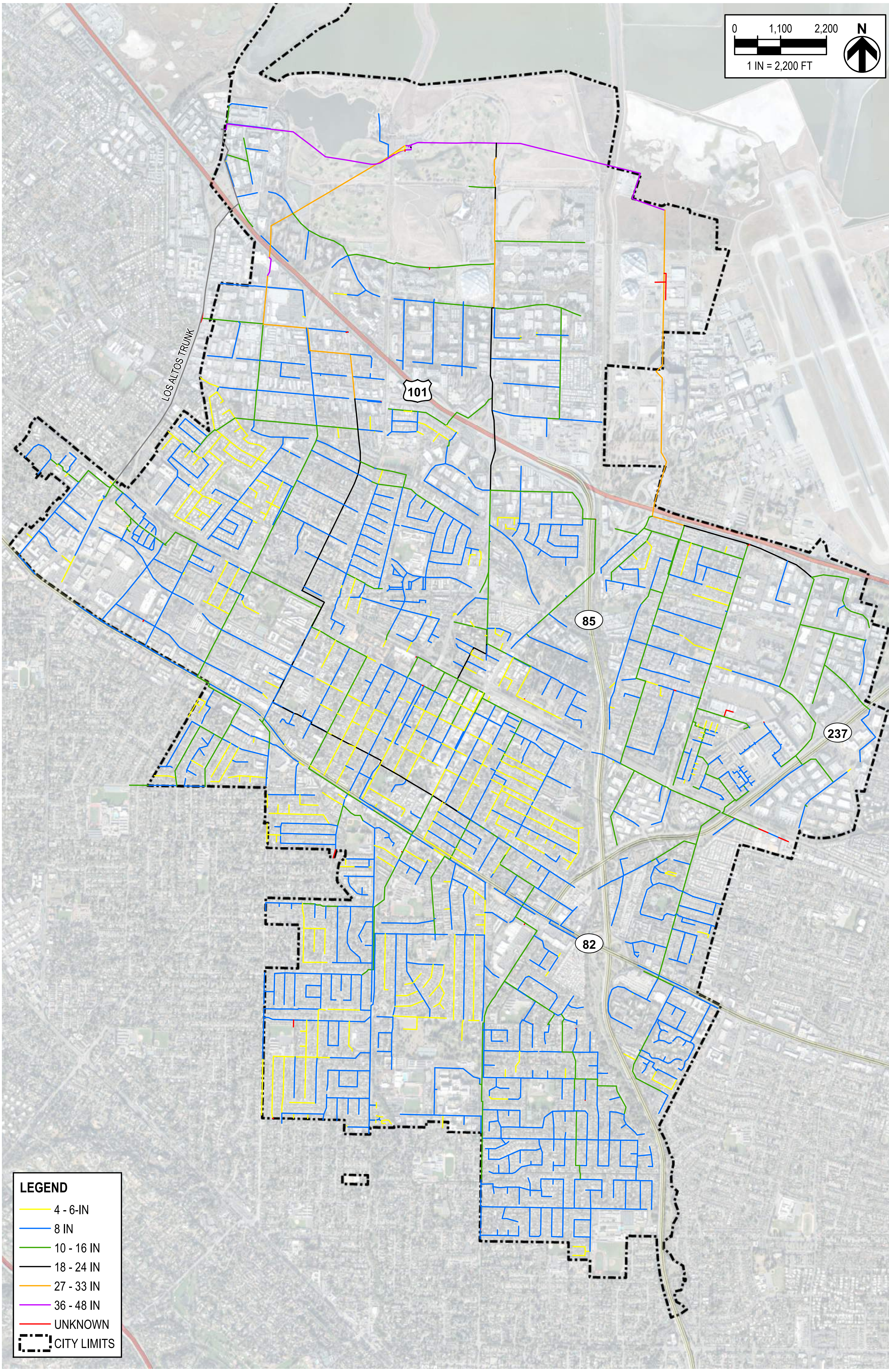

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FIGURE 3-2
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
COLLECTION SYSTEM PIPELINE DIAMETERS

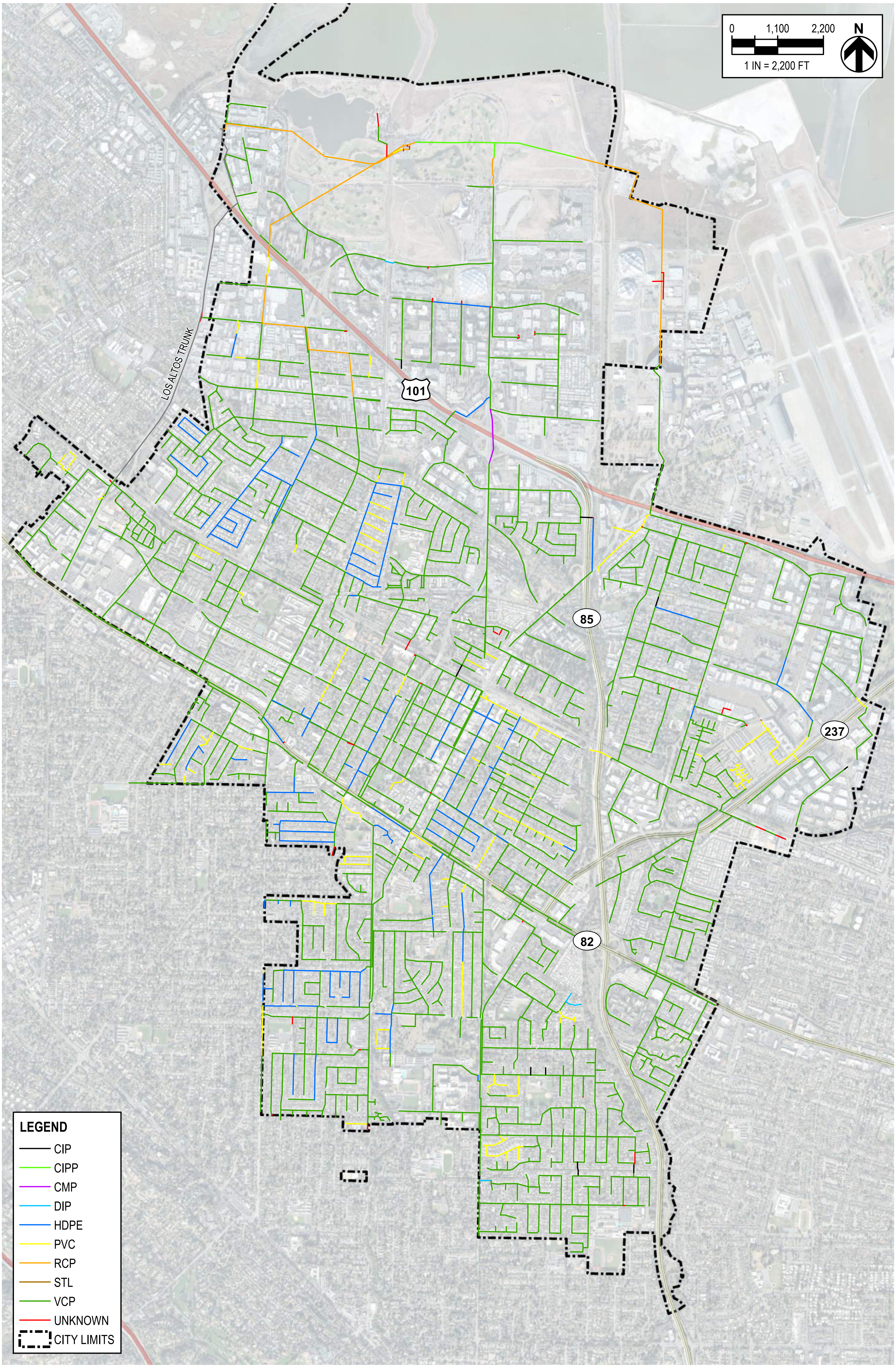

FIGURE 3-3
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
COLLECTION SYSTEM PIPELINE MATERIALS

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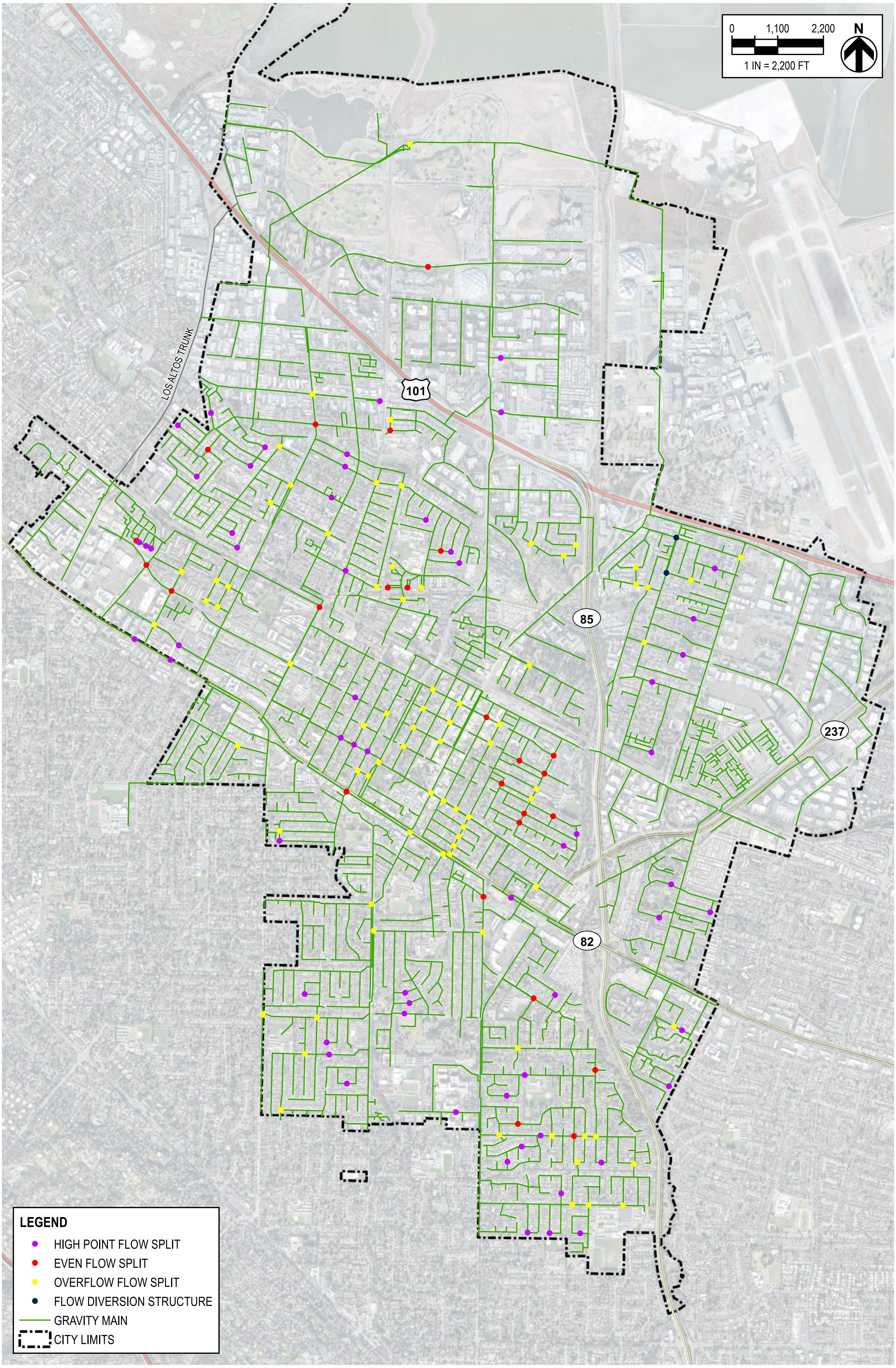

FIGURE 3-4
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
FLOW SPLITS

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Figure 3-5: Example High Point Flow Split

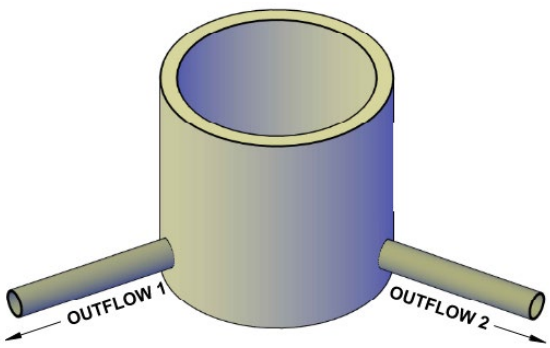


Figure 3-6: Example Even Flow Split

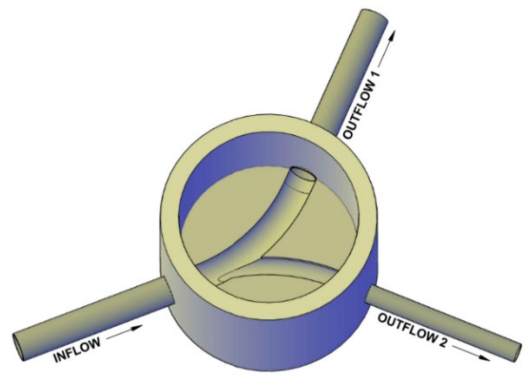


Figure 3-7: Example Overflow Flow Split

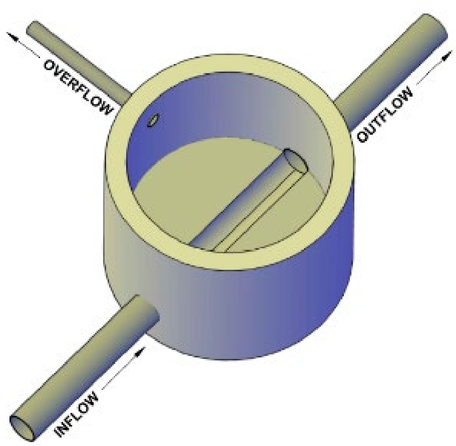
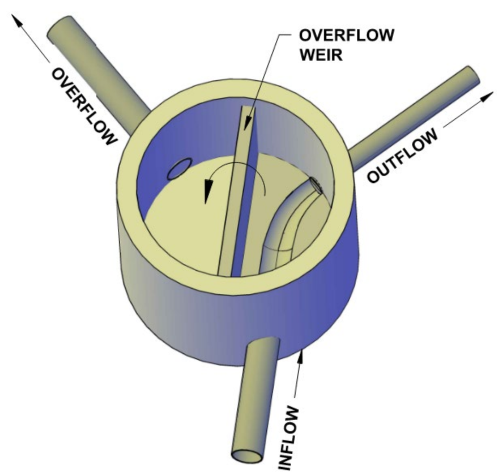


Figure 3-8: Example Flow Diversion Structure



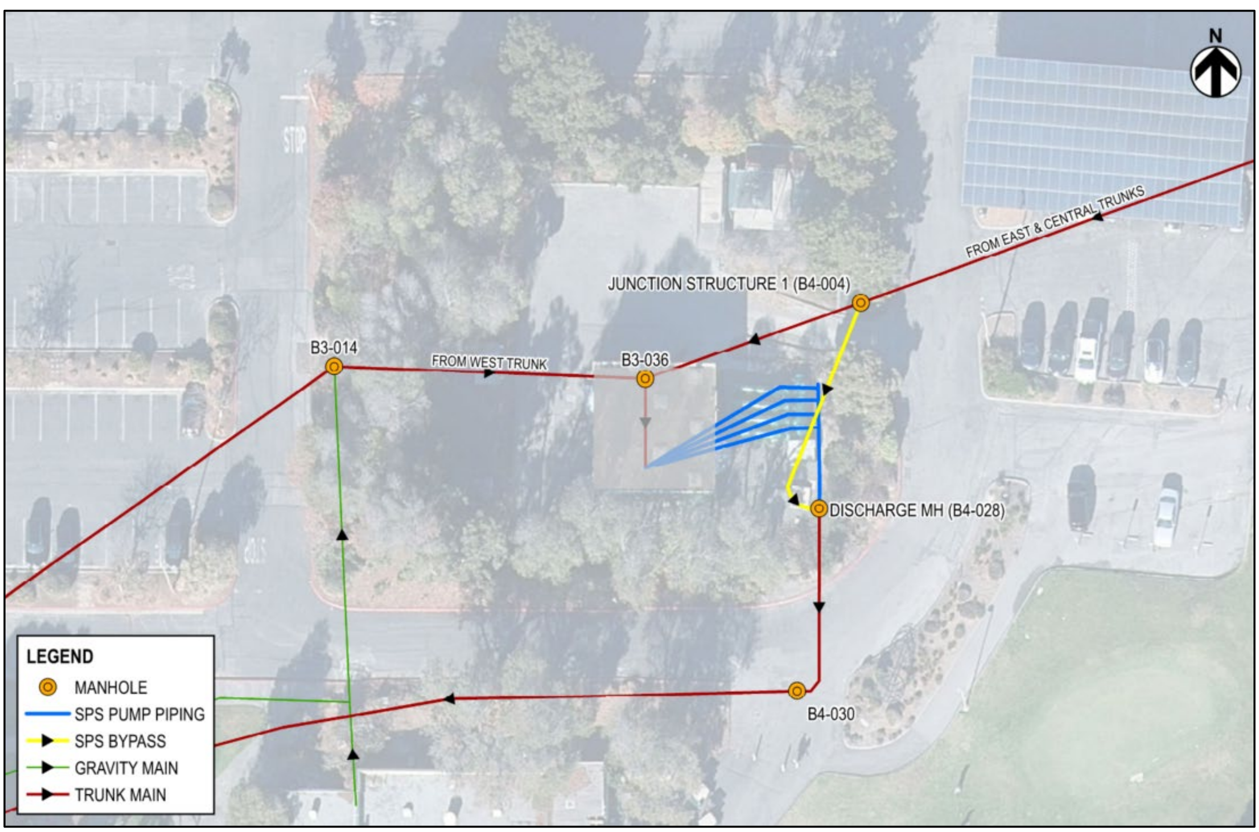
3.2 Pump Stations

The City owns and operates two wastewater pump stations, the SPS and the Pastel LS, which are shown in **Figure 3-1**. The Pastel LS is a small pump station located on Pastel Lane just west of the Stevens Creek Trail in the southeastern portion of the City that serves ten residential units. It is designed to lift wastewater collected from lower elevations and transmit it to the collector main on Sleeper Ave. From there, it flows by gravity through the collection system and ultimately to the SPS.

SPS is situated in the low elevation point of the City to which almost all wastewater flows and is pumped to the San Antonio Metering Station and ultimately to the RWQCP. It is located in the parking lot of Shoreline Golf Links on N Shoreline Blvd at the former site of the City's independent WWTP. The station receives flow from the East, Central, and West Trunks and provides approximately 30 ft of static lift. From the station, the wastewater flows through approximately one mile of 42-in pipe to the San Antonio Metering Station. The pump station was built in 1969 with renovations completed over the years as necessary. The most recent renovations were completed during 2019-2020.

Wastewater from the East and Central Trunks approaches SPS through Junction Structure 1 (B4-004) located along the entry driveway to the pump station. It then joins flow from the West Trunk at manhole B3-036 where all flow enters the pump station. **Figure 3-9** provides an overview of the pipe configurations surrounding SPS.

Figure 3-9: SPS Overview



From manhole B3-036, flow is split into three channels which flow to the pump intake piping located at the end of the channels. These influent channels and the pump intake piping area serve as the station's wet well, acting as an influent reservoir prior to pumping.

From the wet well, flow is pumped through one of three active vertical split case pumps – a fourth natural gas-driven pump is available on standby. The three active pumps (Pumps 1, 3, and 4), which alternate daily, are Worthington Model 16MNZ-1 dry pit low shaft centrifugal pumps with rated capacities of 9,000 gallons per minute (gpm) at 33 ft total dynamic head (TDH) and are driven by 100-horsepower (hp) 460-volt (v) variable frequency drives (VFDs). Two pumps running at full capacity are sufficient to convey peak wet weather flow (PWWF). The natural gas-driven standby pump (Pump 2), operated once a week for maintenance cycling, is a Worthington Model 20MNZ-1 and has a rated capacity of 12,000 gpm at 33 ft TDH.

Influent is pumped through the pump station, providing approximately 30 ft of static head. Effluent reaches the discharge manhole – indicated on **Figure 3-9** – and continues by gravity to the San Antonio Metering Station and ultimately to the RWQCP.

The pump station controls rely heavily on the use of VFDs to maintain wet well levels due to the wet well's limited storage volume. Upon rising wet well levels, the first pump will start at minimum speed. If the wet well level falls, this pump will maintain minimum speed until the first pump stop level is reached. If the level continues rising, the VFD will increase the speed of the first pump until the level begins dropping. If the level continues to rise with the first pump at full speed, the second pump will start and repeat the same process as described for the first pump. If wet well levels continue to rise with the first and second pumps running concurrently, the third pump will start, also following the same process. The pump start/stop wet well levels are listed in **Table 3-2**.

Table 3-2: SPS Set Points

Pump	Start Level (ft)	Stop Level (ft)
First	2.75	1.90
Second	3.50	2.50
Third	4.50	3.25

SPS is also equipped with an emergency gravity bypass pipeline constructed in 1985 by repurposing a portion of the abandoned WWTP influent piping to connect the manholes immediately upstream and immediately downstream of the pump station. This operates similarly to an overflow flow split (see **Figure 3-7**) in the case of an emergency at the pump station to prevent a sanitary sewer overflow (SSO). If the incoming pipeline from the east begins to surcharge, the overflow bypass is activated, carrying flow by gravity from Junction Structure 1 to the discharge manhole where it then continues by gravity to the San Antonio Metering Station as usual.

As part of this Master Plan, a condition assessment was performed of the SPS building, equipment, and surrounding area in July 2021 to assess the facility's current operational form and recommend any necessary improvements to address identified physical deficiencies. An assessment was also conducted of the pump suction lines in March 2022. Details of this condition assessment and identified deficiencies are presented in **Section 4.2.1**.

When the City's WWTP was repurposed, it was most economical for the City to utilize the existing trunk mains that flowed to the WWTP. Recently, the City has had to perform significant upgrades on SPS including emergency repairs. The City has performed a series of studies to determine the feasibility and cost effectiveness of building a large interceptor trunk (bypass interceptor) to intercept a majority of the City's flow before it reaches SPS and redirect it to flow directly to the San Antonio Metering Station by gravity. This would allow the City to significantly downsize the pumping facility at SPS and realize significant annual energy and maintenance cost savings. Prior studies evaluated various alignments to reroute flows by gravity to the RWQCP. Included in **SECTION 10** is a presentation of the detailed hydraulic analysis of the preferred alignment for this bypass interceptor.

SECTION 4 – CONDITION ASSESSMENT

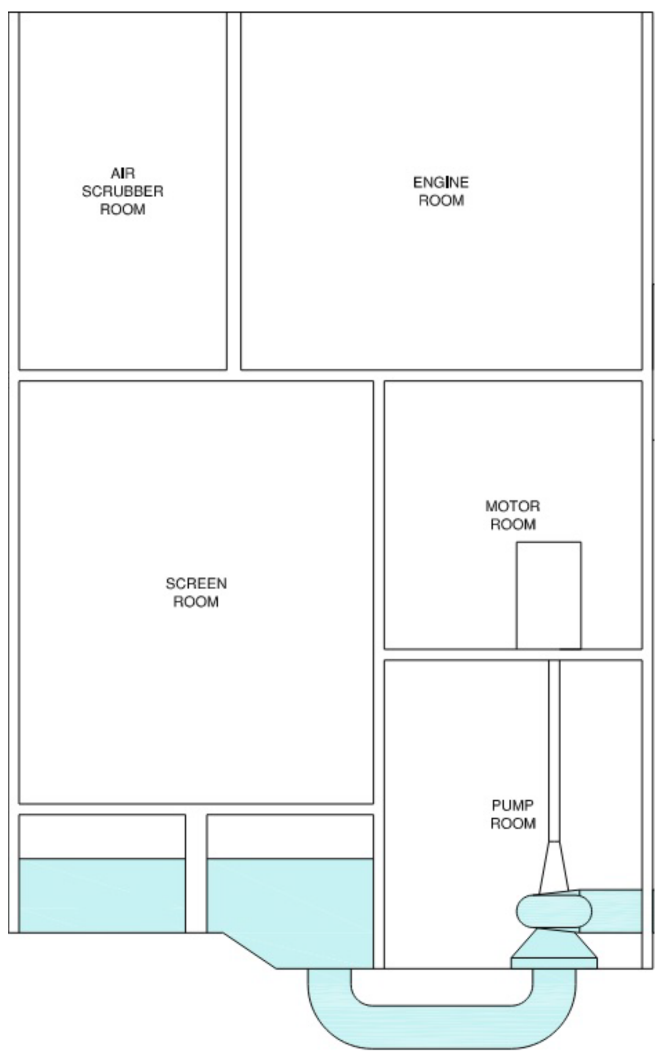
HydroScience staff conducted a condition assessment of the SPS as part of the Master Plan effort. Site visits were attended by both HydroScience and City Sewer Operations staff. Staff visited facilities on July 20, 2021 to visually document facility conditions. HydroScience also attended and reviewed CCTV footage collected on March 14, 2022 of the underground pump suction piping. Provided is a discussion of the notable deficiencies identified by HydroScience and Operations staff. The recommendations made in this section are based on the condition of the SPS as it is now. Facility inspection forms are included as **Appendix D**.

4.1 Site Visit

The facilities site visit portion of the condition assessment was conducted on July 20, 2021. During the site visit, HydroScience also conducted Operations staff interviews to capture any notable issues and/or challenges experienced in the operation and maintenance of the facility.

The building that houses SPS is the original cast-in-place concrete structure that has been rehabilitated as needed over the years. It is a five-room, three-story, above and below ground building, a profile of which is shown in **Figure 4-1**. The east side of the building has three stories – engine room, motor room, and pump room – and the west side has two stories – odor control/air scrubber room and screen room.

Figure 4-1: SPS Building Profile



SPS pumps an average of 6 MGD during dry weather. During low flows, two of the influent channels flow by gravity to the pump influent piping and during higher flows, the third channel is activated. Each channel includes a sewage grinder, each of which was replaced about five years ago and are maintained every six months.

The natural gas-driven standby pump was rebuilt in 2017 and the backup generator was installed in the early 2000s.

Junction Structure 1 (see **Figure 3-9**) includes a manually-operated gate to block flow from entering the bypass pipeline. During emergency repairs to the header pipes in 2015, City staff tried to operate this gate, it dropped part-way down due to corrosion in the manhole from hydrogen sulfide (H_2S) gas and has not been operated since for fear of the gate dropping all the way and requiring an emergency repair. It is noted that there are very few circumstances where this gate is necessary except to repair the bypass piping itself. The City is actively looking into removing this gate as there are few, if any, applications for which it is needed.

It was communicated by City staff that manhole B4-030 (see **Figure 3-9**) has experienced instances of small-volume sanitary sewer overflows (SSOs) when the pump station is being taken out of bypass mode. In this instance, the pumps are likely flowing at full capacity to draw down the gravity system that is surcharged. During the transition from bypass to pump operation, a small SSO occurs just downstream of the pump station. To mitigate this, the City installed a flanged riser on manhole B4-030 during the March 2022 CIPP repair of the Interceptor Trunk to seal it and prevent wastewater from overflowing.

The site of SPS is equipped with dewatering sump pumps because of the high groundwater table and the tendency for rainwater to collect at this low point. It was communicated by City staff during the site visit that the dewatering sump pumps and associated discharge piping are undersized and during recent heavy rainfall, high level alarms have been experienced due to stormwater runoff into the SPS site. A detailed analysis of the required pumping capacity at the SPS site was completed and is presented in the report titled *Shoreline Sewer Pump Station Assessment Report Project No. 14-32*, published April 28, 2017. According to this study, the capacity of the existing system is estimated at 80 to 100 gpm and the required capacity for a 100-year storm is approximately 2,208 gpm. Given that the existing sump pump and discharge piping are undersized, an upgrade of the existing sump pumps and piping would be required should the City choose to not implement the bypass interceptor (see **SECTION 10**) and maintain the existing SPS; these dewatering system upgrades could cost $1M or more. Alternatively, if the bypass interceptor is implemented and the existing SPS was replaced with a smaller pump station, upgrades to the sump pumps and piping could potentially be avoided if the existing SPS was demolished and the site regraded. Because there are no record drawings available for the existing sump pumps and the required action depends on the implementation of the bypass interceptor, it is recommended that the City perform an in-depth storm water drainage study to determine the best course of action.

SPS recently underwent a significant set of upgrades in 2019-2020 as part of SPS Improvement Project 17-48 which included replacing all the header piping from the pumps to the discharge manhole, including new wall penetrations, upgrading the ventilation fans, and upgrading the hydraulic unit for the influent channel bypass gate. Additionally, in March 2022, approximately 1,500 ft of pipeline downstream of the discharge manhole was lined using cured in-place pipe (CIPP) due to corrosion and collapsing pipe observed during previous CCTV inspection.

Recommendations: As detailed further in **SECTION 10**, the City is considering a new bypass interceptor through the City to intercept flow from the East, Central, and/or West Trunks. This would result in a significant decrease in the flows pumped by SPS and downsizing of the entire facility. It is noted that the recommendations here may be altered by the timeline of implementation of the bypass interceptor.

Based on the observations made at the site visit and conversations with City Sewer Operations staff, the following highest priority improvements are recommended by HydroScience for immediate action include:

- Install perimeter fencing and security cameras to improve site security.

- Perform an Arc Flash study and install warning labels on the motor control center (MCC) for operator safety.
- Reconstruct foundation and concrete slab for Pacific Gas and Electric (PG&E) electrical service connection to address differential settling of equipment. Demolish and reconstruct the existing pad per PG&E requirements. Reinstall existing equipment in kind. Reconnect any underground conduit connections and replace damaged materials as needed. Brace/anchor the existing utility service panel to the concrete structure once repairs are complete.

Other high priority improvements recommended to be addressed in the next five years include:

- Install fall protection on the pump room hatches to match the existing Occupational Safety and Health Administration (OSHA) compliant fall protection netting.
- Perform a Storm Water Drainage Study.
- Relocate air handler for the heating, ventilating, and air conditioning (HVAC) system located just above the stairs to prevent operator injury during maintenance.
- Replace and rehabilitate existing utility service panels and associated raceways.
- Replace Remote Telemetry Unit (RTU) 24, radio, and associated raceways and conductors.
- Replace existing control panel uninterruptible power supply (UPS).
- Relocate existing light switches at the rear entrance of the electrical building that are located behind the HVAC ducting to the other side of the doorway for operator safety.
- Relocate hydraulic power unit equipment for influent channel isolation slide gate to a separate location in the facility due to the elevated levels of H_2S gas in the screen room which could cause a failure of the controller for the primary point of facility isolation.
- Install backflow prevention redundancy on the facility's water utilities.

Medium priority improvements recommended for the pump station facility for consideration in the next five to ten years include:

- Remove or replace existing bypass/isolation slide gate in Junction Structure 1.
- Perform a thorough condition assessment of Junction Structure 1 and rehabilitate the concrete surface as necessary.
- Perform a thorough condition assessment of the influent flow channels and address H_2S gas corrosion. Repair and/or recoat with a protective coating system as necessary.
- Replace existing compressed air bubbler sewage level device in influent flow channels with an ultrasonic or submersible transducer level monitoring system and upgrade controls.
- Replace natural gas-driven standby pump with matching make/ model as three active pumps.
- Replace existing manual overhead equipment crane operator with an electric operator for pump extractions and maintenance/servicing.
- Slurry seal the site asphalt concrete pavement and address any low point areas and/or depressions.

Additional recommendations that are low priority and are primarily replacement of equipment at the end of useful life should be considered in the next ten to twenty years include:

- Replace existing pumps and motors with modern design close coupled submersible pumps equipped with semi-open impellers designed for modern sewage applications. Install pumps on a rail system and reconfigure the existing suction and discharge force main piping to include isolation valves.
- The hatch between the motor room and the maintenance/electrical room has an OSHA-compliant netting safety system which could be unreliable in the future; replace the netting as necessary to avoid any potential netting failures.
- Evaluate means of dampening the pump manifold discharge at the existing discharge manhole. The concrete vault is subject to extremely turbulent pressurized flow and exposing high levels of H_2S gas. Reconstruct with a smooth flow transition to the discharge manhole.
- Rehabilitate pump station interior architectural features such as wall coatings, floor anti-slip epoxy coatings, lighting, doors, restroom features, lockers, showers, etc.
- Replace existing microturbines which are nearing the end of their useful life.
- Repair or replace any non-functional instrumentation and equipment. If any equipment is no longer used, remove it from the pump station.
- Replace all existing MCCs and necessary raceways and conductors.

4.2 CCTV Review

CCTV review included the following facilities, detailed subsequently:

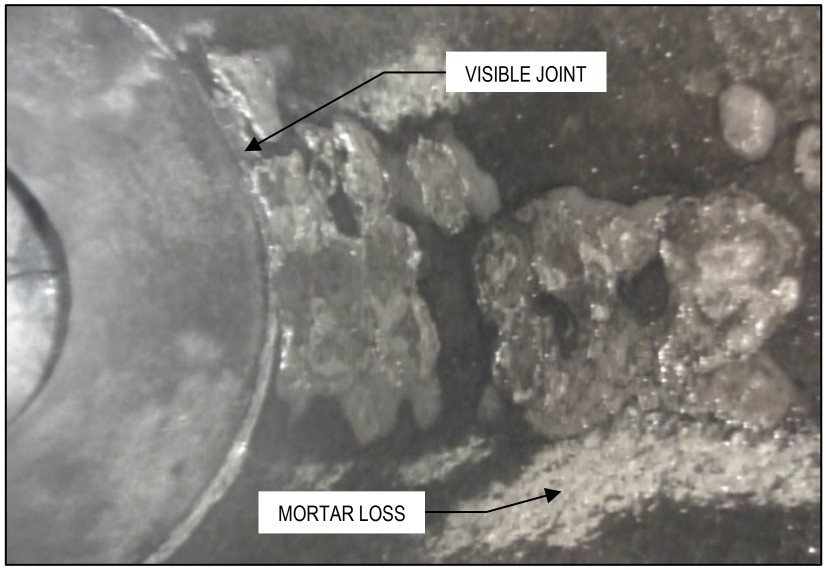
1. SPS pump intake piping downstream of the influent channels within the pump station, and
2. Collection system piping in the vicinity of SPS.

4.2.1 SPS Pump Intake Piping

The pump intake piping located at the end of the SPS influent channels is underground and has not been inspected since construction of the facility in 1960. During the March 2022 CIPP repair of the Interceptor Trunk downstream of the pump station, bypass pumping was installed around SPS which provided an opportunity to inspect the condition of the underground pump intake piping. The facility required additional dewatering during this inspection as the pump station's main sluice gate was unable to close completely due to silt buildup. On March 14 and March 15, 2022, CCTV footage was gathered with City and HydroScience Staff onsite.

According to record drawings dated Nov. 1960, these pipes were originally constructed as ¼-in steel with ¾-in mortar. Upon review of the CCTV footage, the pipes appear to maintain their round shape but have a very rough surface and evidence of mortar loss. The joint welds are visible (see **Figure 4-2**) which is not expected for a mortar lining; it appears that the mortar lining has come off quite a bit from the piping for all four pumps, slightly worse for channels one and two. Additional photo excerpts from the CCTV for each channel are included in **Appendix D**. Channel two also contained small amounts of grease buildup on the crown of the pipe which may be because this is the natural gas-driven backup pump that is utilized less often than the three active pumps.

Figure 4-2: CCTV Sample – Channel 1



The piping for all four channels does not appear to be at immediate risk of failure since it is not typically exposed to high levels of oxygen, resulting in slower rates of corrosion. HydroScience recommends that the City line these pipelines within the five-to-ten-year timeframe. Should the bypass interceptor option (see **SECTION 10**) be implemented, the SPS would be replaced with a smaller pump station and the pump intake piping repair would not be necessary.

4.2.2 SPS Vicinity Piping

During the March 2022 CIPP repair of the Interceptor Trunk, CCTV was performed on four additional pipe reaches in the vicinity of SPS – segments 1 through 4 as identified on **Figure 4-3** – on April 28-29, 2022. Details of the CCTV review and identified defects for each segment are detailed below:

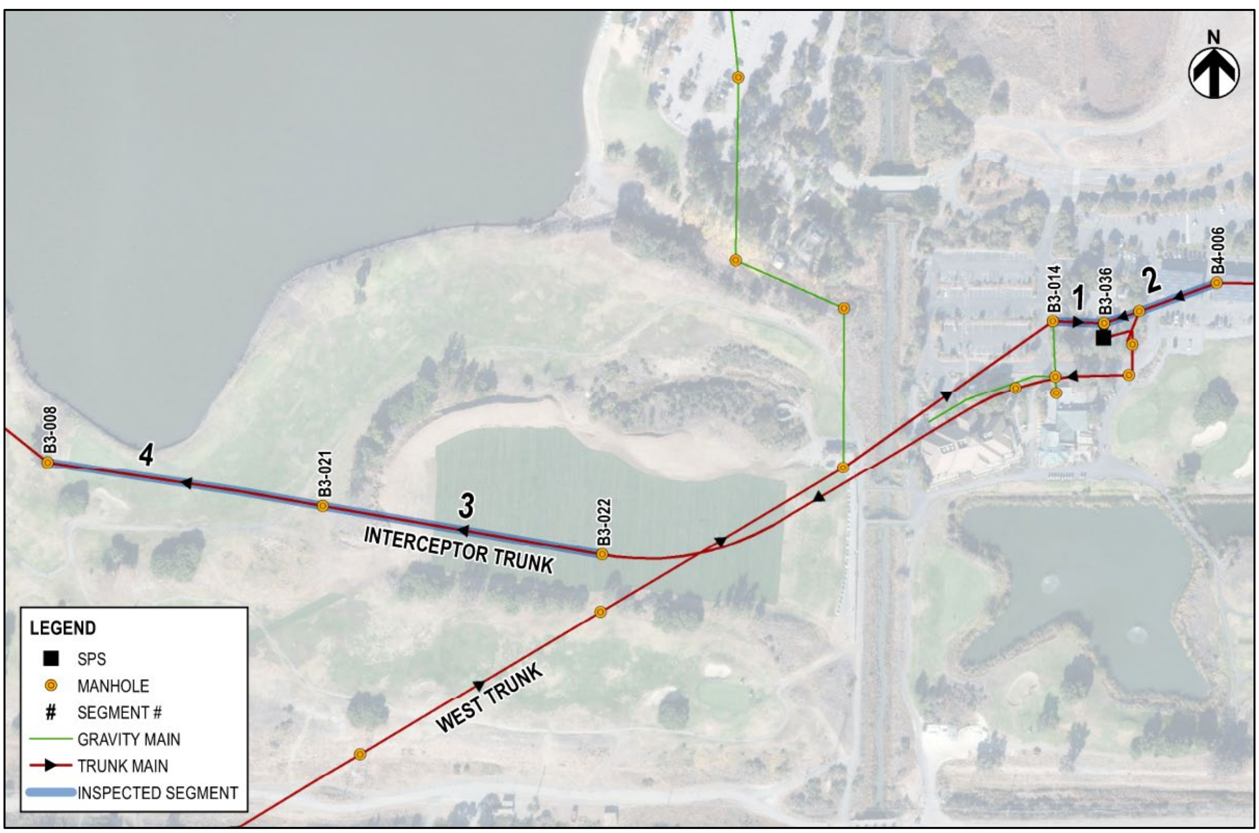
1. **B3-014 to B3-036:** This segment is a 39-in RCP portion of the West Trunk flowing eastward towards SPS. During the process of bypassing the SPS for the 2022 CIPP repair of the Interceptor Trunk, the contractor found debris approximately 6 to 8 ft downstream of MH B3-014. This debris was not visible in the CCTV footage as the pipe was over 50% full, limiting visibility. Additionally, the camera was stopped at the beginning and then footage was restarted approximately 6 ft downstream of MH B3-014; therefore, the terrain of the upstream end of the segment was not fully documented.

   The water level starts above 50%, drops down to approximately 50%, and then rises up to about 75% at the end of the segment; the higher water level at the upstream end is an indicator of debris at the upstream end, supporting the contractor's initial observation.

   Additional observations of the pipe include visible aggregate and reinforcement.

2. **B4-006 to B3-036:** This segment is a 39-in RCP portion of the joint interceptor that captures flow from both the East and Central Trunks flowing westward towards SPS and includes Junction Structure 1 (see Figure 3-9). Observations from the CCTV footage of this pipe include infiltration at three joints (runner at 26 ft and two drippers at 13.6 ft and 190.8 ft downstream), visible aggregate for most of the length of the pipe, and aggregate projecting at a joint 135 ft downstream. Additionally, the water level increases from 60% in the beginning to 75% at approximately 80 ft downstream. Without further investigation, the cause of this mild increase is unknown.
3. **B3-022 to B3-021:** This segment is the first segment downstream of the 2022 CIPP repair of the Interceptor Trunk. It is a 42-in RCP portion of the Interceptor Trunk flowing westward from SPS towards the San Antonio Metering Station. Observations from the CCTV footage of this pipe include visible and missing aggregate for the entire length of the pipe, fine roots at a joint about mid-way through the segment, and infiltration at two joints (gusher at 277 ft and dripper at 399 ft downstream).
4. **B3-021 to B3-008:** This segment is immediately downstream of segment 3 and is also a 42-in RCP portion of the Interceptor Trunk flowing westward from SPS towards the San Antonio Metering Station. Similar to segment 3, observations from the CCTV footage of this pipe include visible and missing aggregate for the entire length of the pipe, medium to large roots in two locations (from joint at 0.5 ft and from barrel at 6 ft downstream), and infiltration at two joints (runner at 100 ft and dripper at 405 ft downstream).

Figure 4-3: SPS Vicinity Piping Segments Inspected



To address the infiltration and roots, spot repairs could be sufficient. However, due to the recent pipe collapse in the Interceptor Trunk (addressed by the March 2022 CIPP repair of the Interceptor Trunk) and the visible aggregate in these RCP pipes indicating corrosion, CIPP is recommended for all four segments within the five-to-ten-year timeframe to provide long-term structural support.

As detailed in **SECTION 9**, segments 1 and 2 do not display any capacity deficiencies in any hydraulic modeling scenarios. Segments 3 and 4 do register a surcharge state of “2” in the 2030 scenario with the existing 42-in pipe; however, there is over 7 ft of freeboard and thus, the risk of SSO is minimal. With CIPP, the surcharging in segments 3 and 4 is slightly increased, but there remains over 7 ft of freeboard. CIPP is recommended as these pipes are also very deep.

In addition to the CIPP of these segments, it is recommended that the City CCTV the entire length of the Interceptor Trunk downstream of segment 4 to the San Antonio Metering Station. Due to the history of pipe collapse in the Interceptor Trunk and the fact that the entire pipeline is RCP which is susceptible to corrosion, CIPP repairs may be necessary for the remainder of the trunk as well.

Should the City decide to implement the bypass interceptor (see **SECTION 10**), these repairs would not be necessary as these segments would be replaced with smaller pipes installed within the existing pipes.

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SECTION 5 – LAND USE

For planning purposes, base sanitary flow (BSF) values – the flow directly contributed by the customer – are developed based on land use as it is assumed that the same, or similar, land use types will have similar wastewater discharge quantities and patterns. Thus, similar land use types can be consolidated for the Master Plan level analysis. This section discusses the land use data used for planning level sewer modeling.

5.1 Existing Land Use

Each parcel was first identified by its 2021 land use which accounted for areas that were not yet fully developed per the projected 2030 General Plan (GP) land uses. The City Planning Division provided two GIS shapefiles: one that included the 2021 land uses and another that presented the City's parcels. These two files were merged in GIS based on the parcel APNs listed in each shapefile, resulting in a land use assigned to each City parcel. The City's 2021 land use categories are listed in **Table 5-1** and presented in **Figure 5-1**.

For modeling purposes, land use types expected to generate similar wastewater flows were grouped into consolidated land use categories where appropriate to facilitate the unit flow (UF) analysis detailed in **SECTION 7**. These consolidated land use categories are presented in **Table 5-1** and **Figure 5-2**. Land uses that have little to no wastewater contribution (parks, open space, etc.) were excluded from further analysis. It is noted that this Master Plan was completed in conjunction with the Water Master Plan and thus, the land use categories presented here directly correspond to those presented and utilized in the Water Master Plan.

In addition to all parcels within City limits, there are a number of parcels within the cities of Palo Alto and Los Altos which contribute wastewater to the City's collection system at various locations along the south and southwestern edges of the City (see **Figure 2-1**). Each of the cities' respective SMPs were reviewed to confirm the land uses of each parcel; all contributing parcels from outside City limits were identified as and assigned the land use code LDR (low-density residential) except Los Altos High School which was assigned PF (public facility).

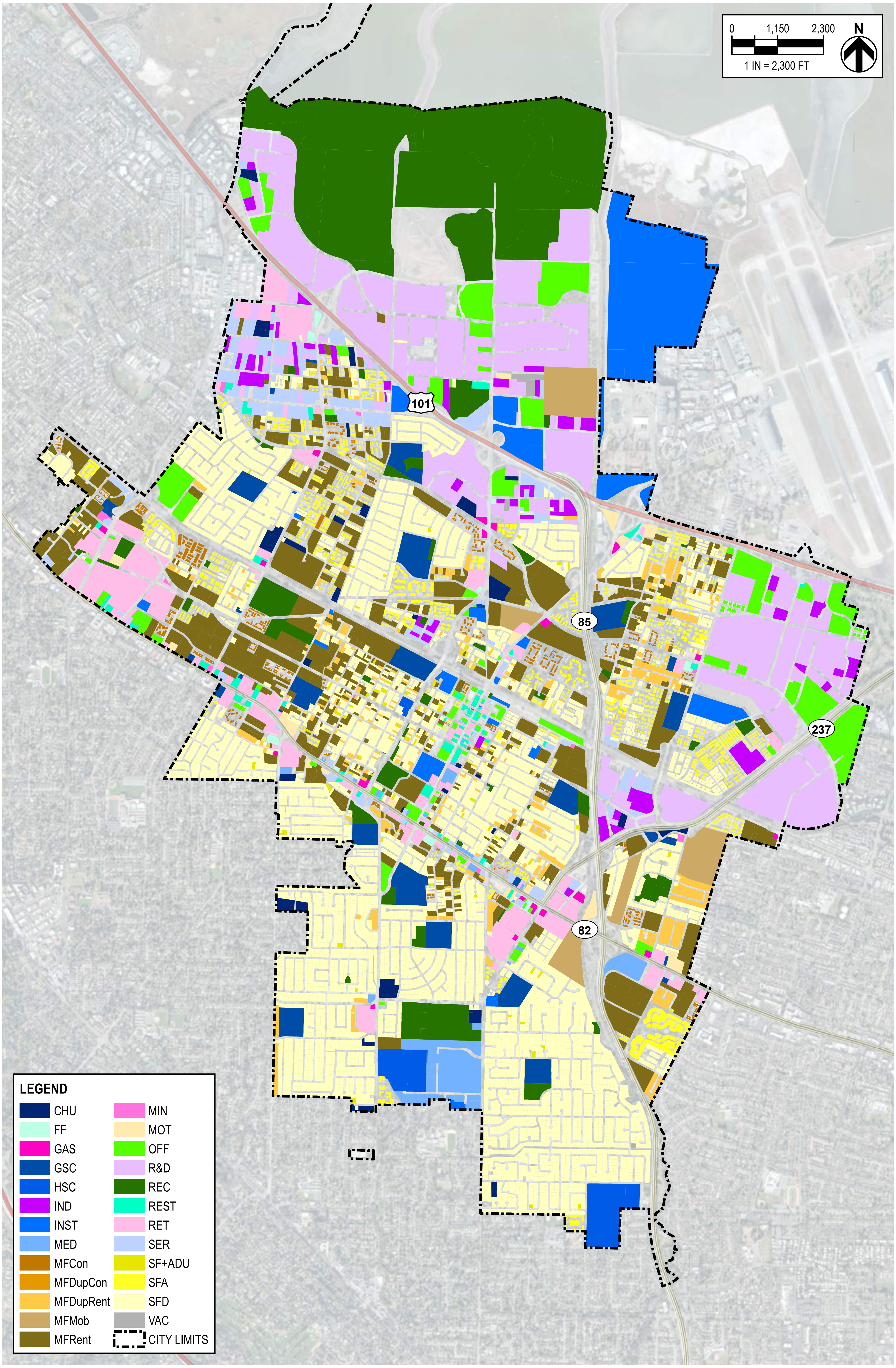
Moffett Field consists of NASA and the Google Bayview campus. Because this is a unique land use, this area was not assigned a land use category; it was modeled as a large discharger, detailed further in **Section 8.2.2**.

5.2 2030 Land Use

For this Master Plan, the future planning horizon is 2030. To determine land use changes for this planning horizon, 2030 land use projections consider near-term planned redevelopment as well as full utilization of any underutilized parcels determined in collaboration with City planning staff. **Figure 5-3** highlights the areas where land use type or utilization is expected to change by 2030, as well as areas of planned redevelopment.

Table 5-1: Consolidated Land Use Categories

2021 Land Use Codes		Consolidated Land Use Categories	
Code	**Land Use**	**Code**	**Land Use**
VAC	Vacant	-	-
RET	Retail, including Banks and Personal Services	COM	Commercial
MIN	Mini-Mart		
GAS	Gas Station		
MFRent	Multi-Family 3+ Rental	HDR	High Density Residential
R&D	Research and Development	IND	Industrial
IND	Industrial		
SFD	Single Family Detached	LDR	Low Density Residential
SFA	Single Family Attached	LMDR	Low Medium Density Residential
SF+ADU	Single Family + ADU		
MFMob	Multi-Family Mobile Homes	MBL	Mobile Homes
MFCon	Multi-Family 3+ Condo	MDR	Medium Density Residential
MFDupRent	Multi-Family Duplex Rental		
MFDupCon	Multi-Family Duplex Condo		
MOT	Motel	MOT	Motel/Hotel
REST	Restaurant	MU	Mixed Use
Mixed Use	Mixed Use		
FF	Fast Food		
OFF	Office	OFF	Office
REC	Recreation	OS	Open Space (No Flow)
SER	Services – Vehicle, Construction, Business and similar	PF	Public Facility
MED	Medical		
INST	Institution		
HSC	High School		
GSC	Grade School		
CHU	Church		



HydroScience

FIGURE 5-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2021 LAND USE

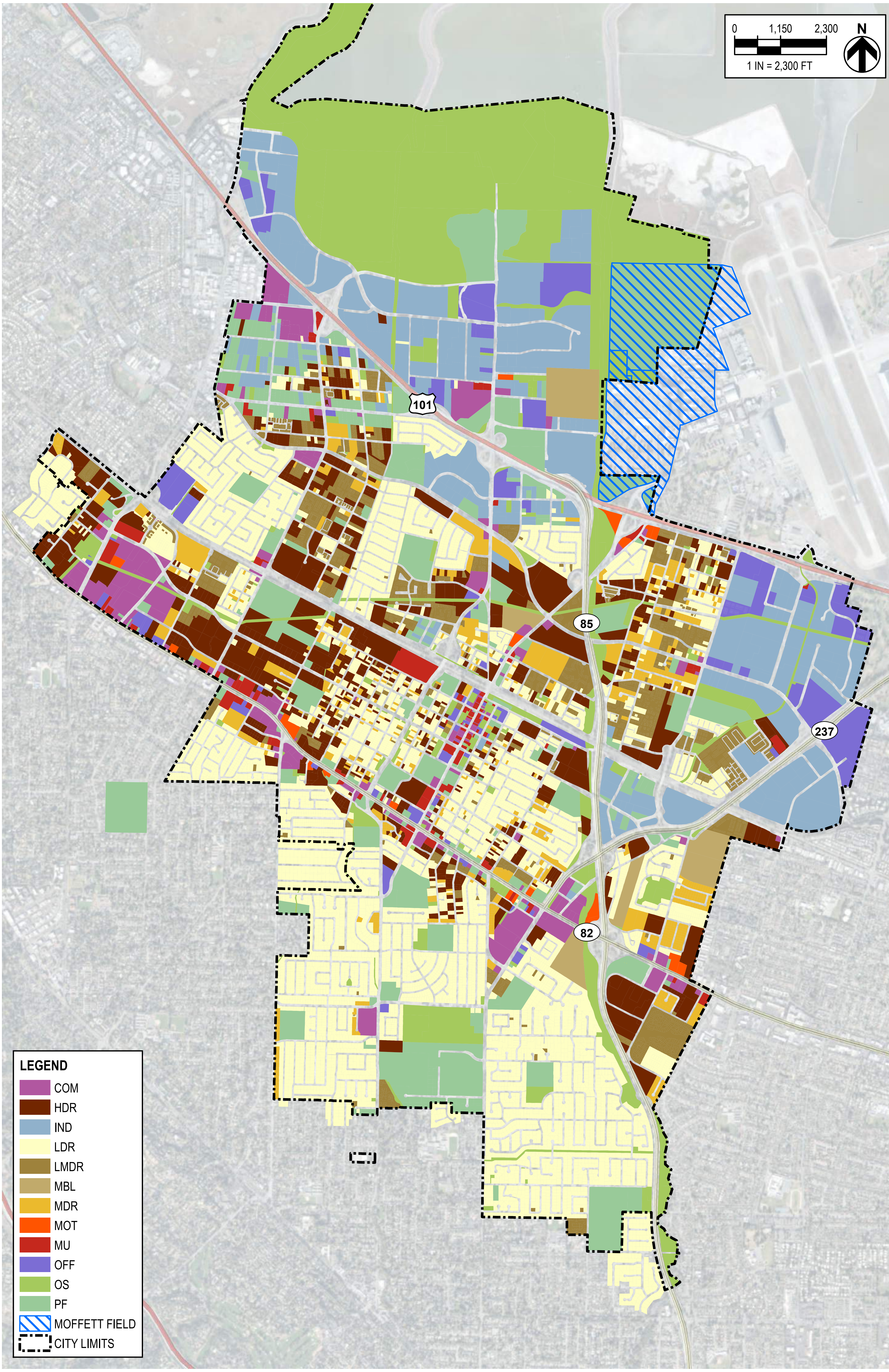


FIGURE 5-2
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
CONSOLIDATED LAND USE CATEGORIES

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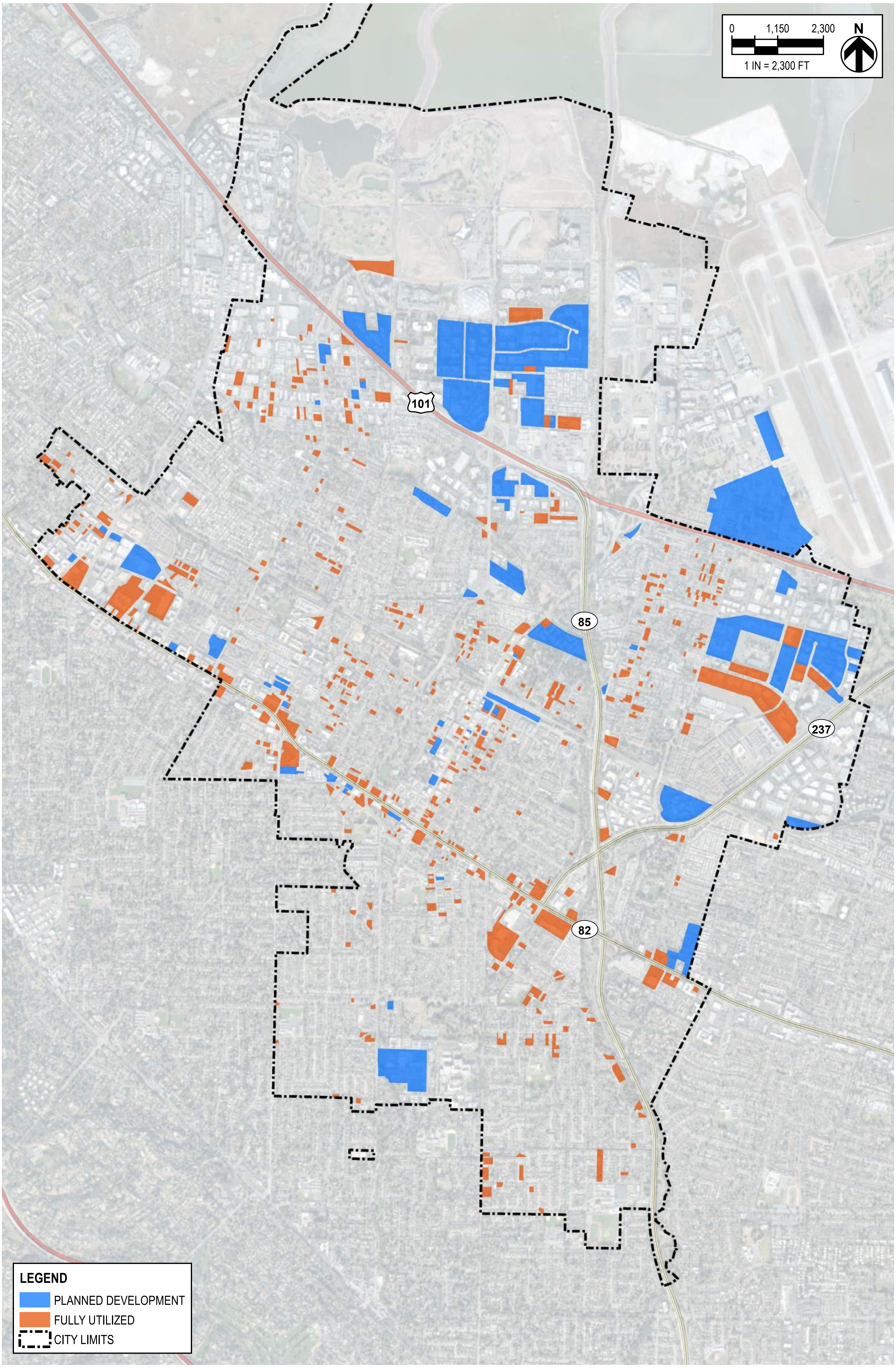

FIGURE 5-3
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
MODIFIED 2030 LAND USE

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SECTION 6 – HYDRAULIC MODEL DEVELOPMENT

The City built the existing wastewater collection system hydraulic model using InfoSWMM by Innovyze® (InfoSWMM) as part of the 2010 SMP. For the hydraulic model update as part of this Master Plan, HydroScience utilized InfoWorks ICM by Innovyze® (InfoWorks). InfoWorks is a fully dynamic software that uses the full Saint-Venant equation to solve complicated hydraulics, which is particularly important for accurately representing complicated systems with flow splits, of which the City's collection system includes 165 (see **Section 3.1.1**).

This section describes the process of reviewing the City's existing InfoSWMM wastewater collection system hydraulic model and the process of updating the model including the translation into the InfoWorks software.

The two primary sources for the InfoWorks hydraulic model development were the 2010 InfoSWMM hydraulic model (2010 model) and the City's GIS geodatabase. The 2010 model was reviewed and compared against the City's GIS geodatabase as well as the City's sewer collection system 701 maps. During this review, a number of discrepancies were discovered in pipe flow direction and the attributed slope as represented in the 2010 model. In particular, many of the flow splits (see **Figure 3-4**) in the system were not accurately represented. These discrepancies were compiled and sent to City staff for verification.

The City's GIS geodatabase does not contain pipe invert elevations in each pipe segment; rather, a single invert elevation is contained in the manhole attribute table. This results in the same invert elevation for both the incoming and outgoing pipes. While this is generally acceptable for manholes with a single inflow and outflow pipe, in cases where there are several pipes entering/exiting a single manhole at varying elevations, assigning the same invert to all pipes results in a misrepresentation of flow and direction at that manhole. This discrepancy particularly affects overflow flow splits (**Figure 3-7**).

Upon reviewing the 2008 manhole survey data, it was determined that it provided the manhole rim and base elevations only – it did not provide inverts for multiple pipes entering/exiting manholes at different elevations. Thus, HydroScience conducted a comprehensive review of the GIS and 2010 model in order to identify potential overflows/flow splits in the system. Actions that can be taken to improve the City's GIS geodatabase include data mining for missing information and ensuring the translation of all future survey data into the geodatabase as it occurs.

The 2010 model features were exported as a set of GIS shapefiles. Each of the manholes and pipelines were spatially matched and joined with its corresponding feature in the City's GIS respective feature class. A quality check was performed to ensure the features were all matched correctly, and adjustments were made where necessary. The result was a new set of manhole and pipe shapefiles containing the data from both the City's GIS geodatabase and the 2010 model.

A new hydraulic model was then built in InfoWorks using the new shapefiles. The resulting model network is presented on **Figure 6-2**. It is noted that all pipelines from the City's GIS geodatabase were included in the hydraulic model, including those identified as private, in order to accurately represent the flow pathways through the system; however, private pipelines were not included in the analysis portion of this Master Plan.

The following sections describe the model building process for each facet of infrastructure.

6.1 Manholes

The new manholes shapefile, containing the data from both the City's GIS geodatabase and the 2010 model, was imported into InfoWorks as a set of nodes. The final existing scenario model network includes a total of 3,889 nodes. The most crucial piece of data for the nodes is the ground surface elevation because the distance between the pipe invert and the ground surface dictates the relative risk of an SSO.

As part of this Master Plan effort, additional manhole surveys were completed in July 2021. During the network development and review process, approximately 150 manholes were identified for survey to reconcile data discrepancies and/or missing data. Many of the survey locations were chosen to confirm flow split inverts which were generally represented in the 2010 model with a single inflow/outflow invert. The objective was to understand how flow is directed at each flow split. All identified *even* flow splits (**Figure 3-6**) and *overflow* flow splits (**Figure 3-7**) were surveyed. High point flow splits are not as consequential and therefore were not included.

Figure 6-1 identifies the manholes surveyed in 2021. The following data was collected for each surveyed manhole:

- Ground surface elevation;
- Invert elevations of all pipes flowing in and out;
- Direction of flow of all pipes flowing in and out; and
- Diameter of all pipes flowing in and out.

This data allowed each flow split to be classified as *even* or *overflow* as detailed in **Section 3.1.1**.

To fill in any remaining data gaps in ground surface elevation, April 2020 LiDAR data was obtained from Santa Clara County and ground surface elevations were extracted for each manhole as needed.

In summary, the ground surface (manhole rim) elevations were populated from the following sources in the following order of priority:

1. 2021 manhole survey;
2. GIS feature class "ssManhole" data;
3. 2008 manhole survey; or
4. Santa Clara County LiDAR data.

Ground surface elevation data sources for each manhole were tracked in the hydraulic model using data flags (see **Section 6.5**).

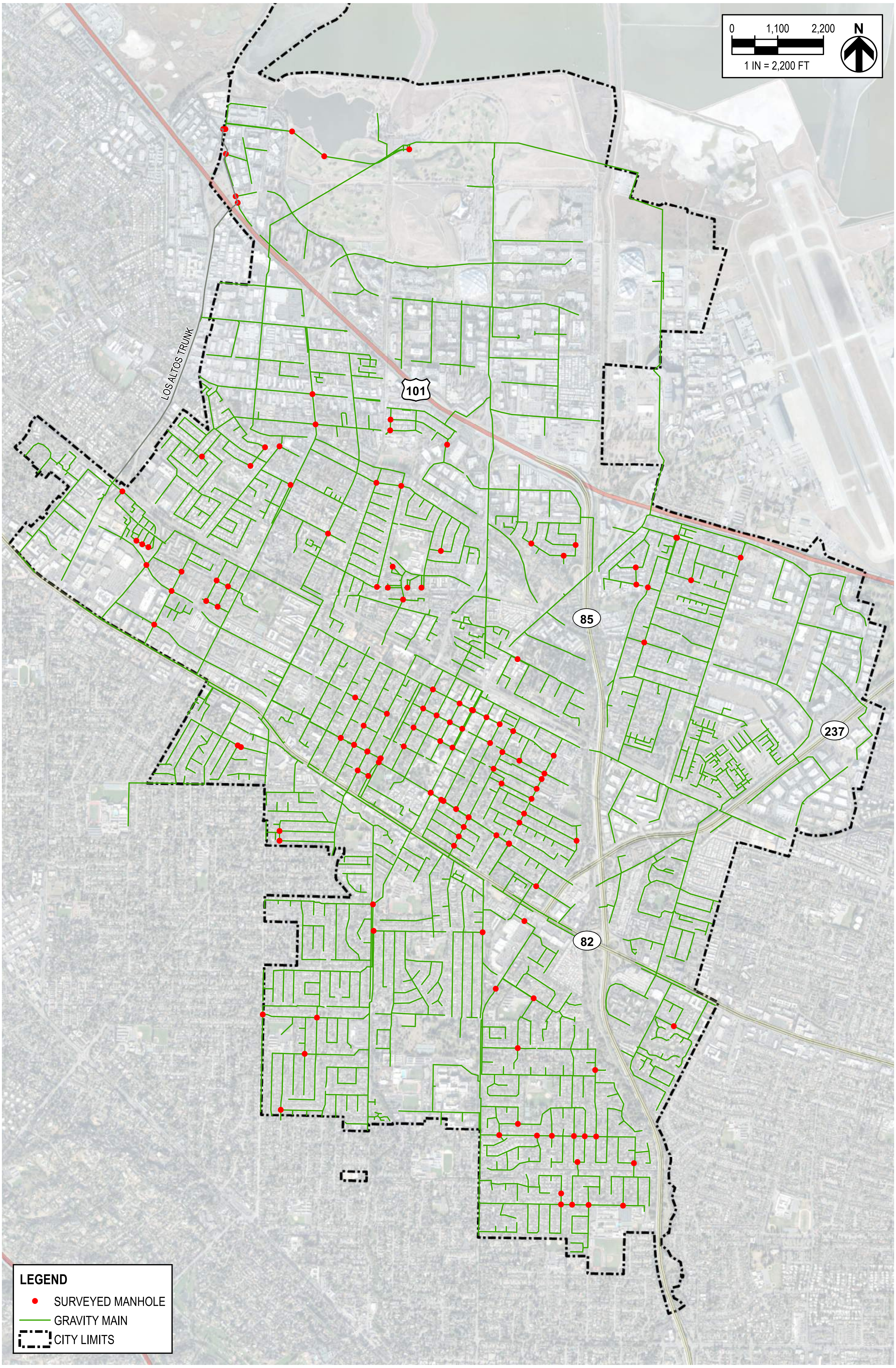


FIGURE 6-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2021 MANHOLE SURVEY LOCATIONS

HydroScience

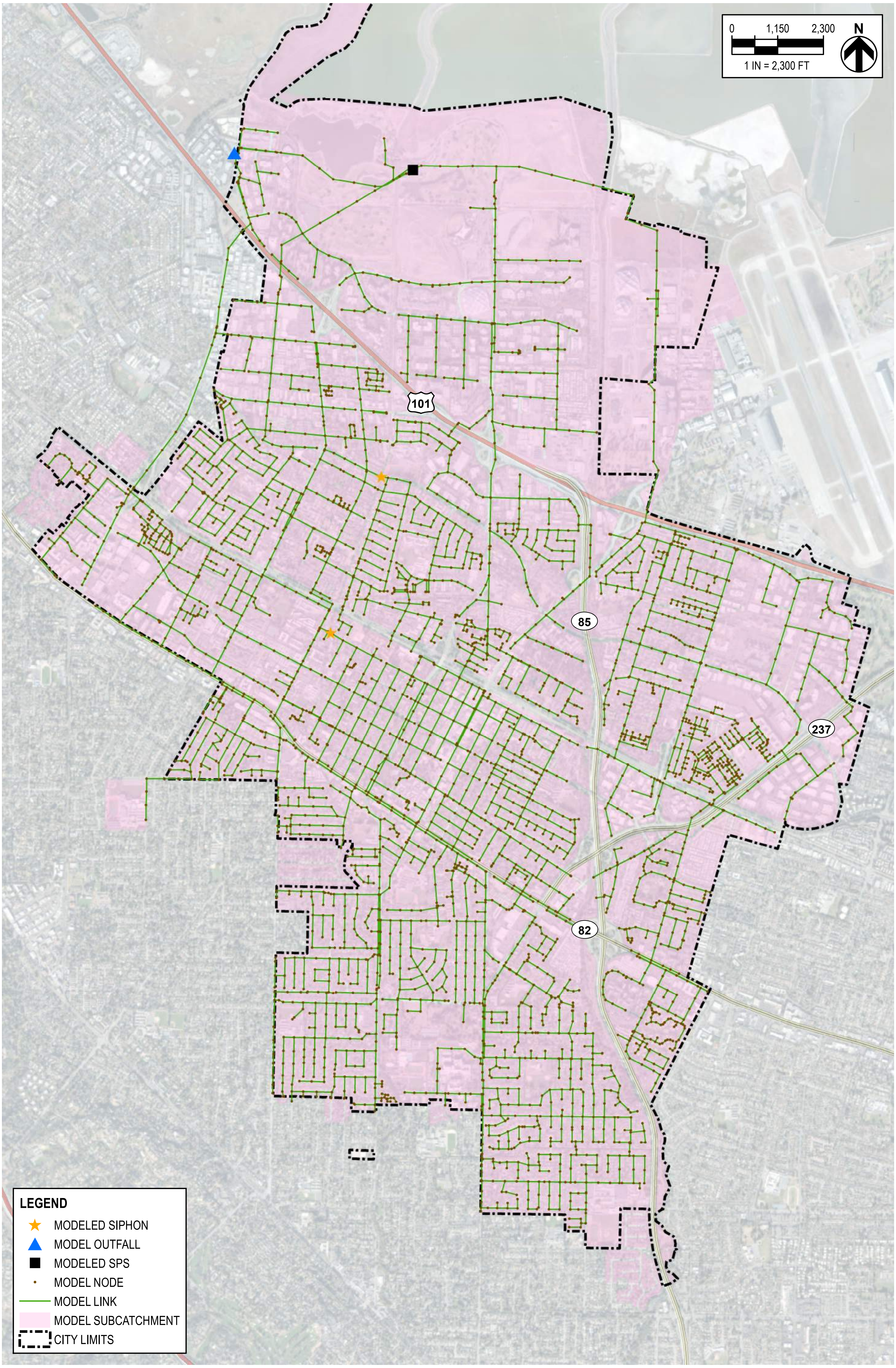


FIGURE 6-2
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
MODEL NETWORK

HydroScience

6.2 Pipes

After importing nodes, the new pipes shapefile was imported into InfoWorks as a set of links. In InfoWorks, all pipes are represented as links and within each link, the distinction is made whether the link is gravity pipe or pressurized pipe. Imported links are connected to the nodes located at each end, identified spatially upon import. Where a node is not available for a link to connect to, a node is generated by InfoWorks. This can be the location of a cleanout or other network structure that is not included in the manhole shapefile, or this can be a result of incomplete digitization of the shapefiles. These nodes are automatically assigned Asset IDs beginning with "XXXX" by InfoWorks and were assigned ground surface elevations based on LiDAR data, as detailed in **Section 6.1**.

Four siphons were identified in the collection system based on the City's maintenance records. Two of these siphons are located in small diameter terminal pipes, located on Casey Ave and Coast Ave, and current record drawings were not available for them; additional information was not pursued as these are not critical facilities. A double siphon is located where Permanente Creek crosses W Middlefield Rd; this was included in the hydraulic model per record drawing PIN #5028. The last siphon is located where Permanente Creek crosses Villa St; record drawing PIN #1311 confirms the existence of the siphon, but it is an incomplete plan and thus, does not include all details. In the absence of invert elevation data, the bottom elevation of this siphon is estimated to be the same as the double siphon to the north that also passes under Permanente Creek.

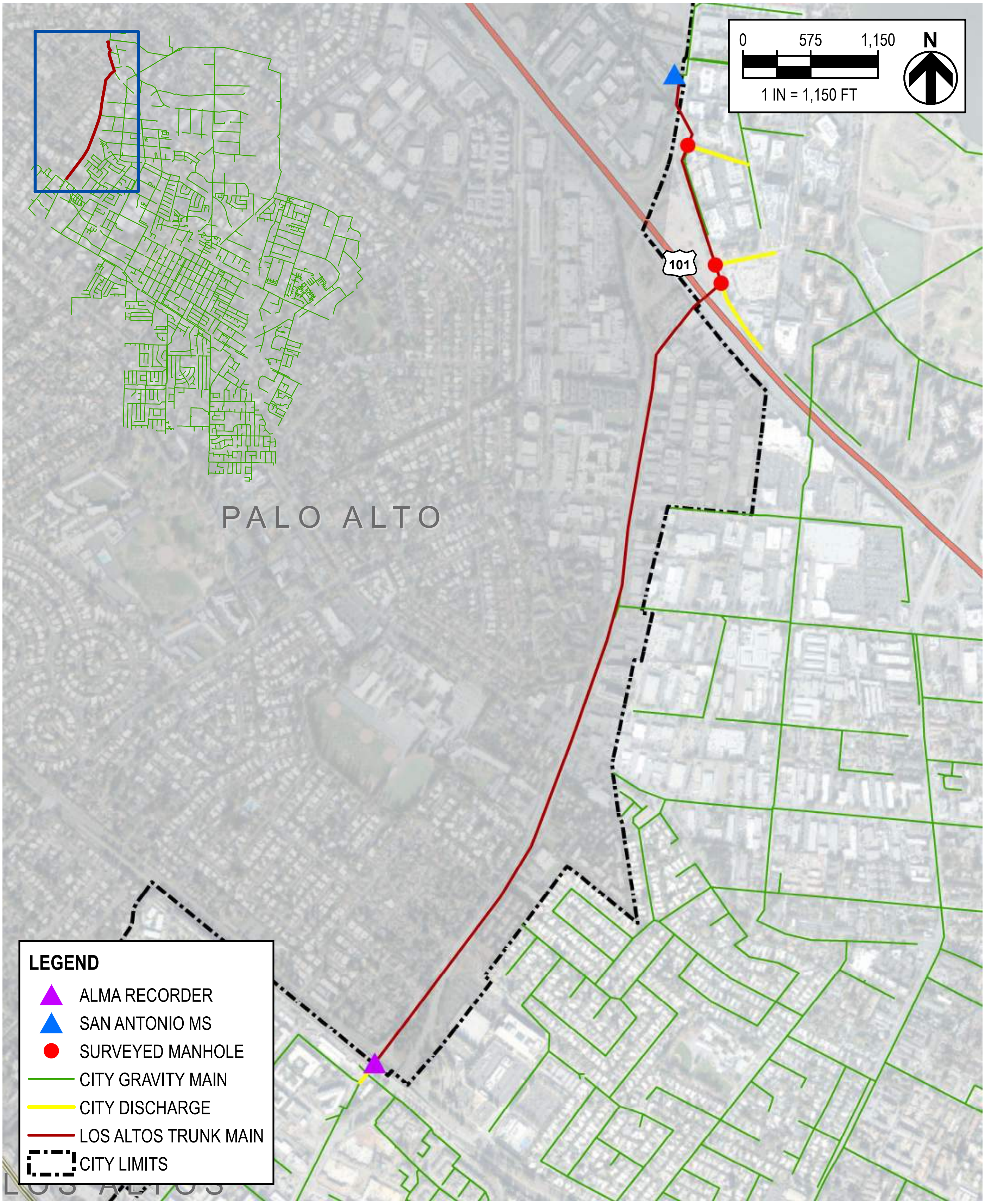
6.3 Pump Stations

The Pastel LS was not included in the hydraulic model due to its small tributary area and size. For modeling purposes, each of the residential units in the tributary area was routed to enter the collection system directly downstream of the lift station.

The three active motor-driven pumps operating at SPS were included in the model using the factory pump curve which is included in **Appendix E**. The fourth natural gas-driven standby pump was not included in the model as this is utilized only in emergency situations such as a power outage. The pumps were modeled in parallel with VFD controls using standard operating parameters. The set points for each pump presented in **Table 3-2** were included as absolute elevations relative to the bottom of the wet well.

6.4 Los Altos Trunk Main

Included in the scope of this Master Plan is the addition of the Los Altos Trunk Main to the City's hydraulic model to provide a more accurate hydraulic assessment of the entire collection system. The Los Altos Trunk Main conveys wastewater from the City of Los Altos, which is located to the southwest of the City, including contribution from the Town of Los Altos Hills. It runs along the western border of the City through the City of Palo Alto to the San Antonio Metering Station where the flow is recorded and combines with the City's wastewater flow pumped from SPS. From there, it enters the 72-in joint interceptor flowing to the RWQCP. **Figure 6-3** presents an overview of the Los Altos Trunk Main including each point where flow from the City discharges into it.



HydroScience

FIGURE 6-3
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
LOS ALTOS TRUNK MAIN

To improve data accuracy, each of the manholes at which City wastewater discharges to the Los Altos Trunk Main was included in the 2021 manhole survey completed as part of this Master Plan, excluding the connection at the southern end of the trunk at the Alma Recorder. Record drawings and previous survey data were available for the manholes surrounding the Alma Recorder.

The hydraulic model of the Los Altos Trunk Main was updated as part of the *2020 Los Altos Trunk Sewer Hydraulic Analysis*. The model was available for integration into the City's hydraulic model. To combine the two hydraulic models, all duplicated manholes and pipelines were removed. The City model ID was retained and the Los Altos Trunk Main ID of each duplicate manhole and corresponding pipeline was removed.

6.5 Network Review

With the initial import of the hydraulic model complete, the next step was to review the data, reconcile any discrepancies, and fill in any missing data. The review was conducted by evaluating each fatal error identified by InfoWorks during its network validation process as well as reviewing the network in *Profile View* (see **Figure 6-5**).

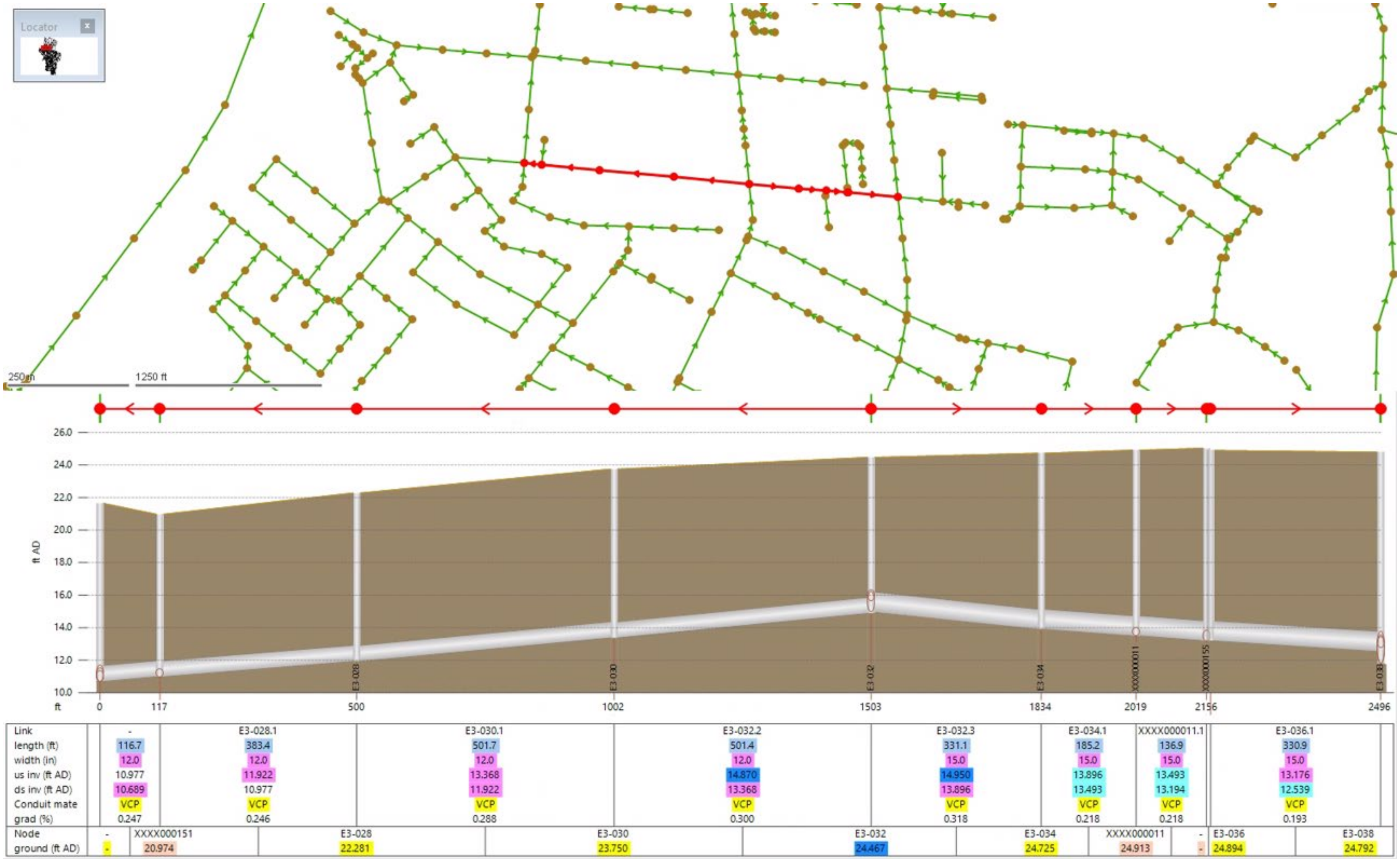
Each update to the model data was marked with a data flag and different flags were created to represent the varying data sources and reasons for the updates. **Figure 6-4** shows the various data flags that were developed and used during the model network review process.

Figure 6-4: Data Flags

Name	Display Colour	Obsolete	Description
#A		☐	Asset Data
#D		☐	System Default
#G		☐	Data From GeoPlan
#I		☐	Model Import
#S		☐	System Calculated
#V		☐	CSV Import
BYP		☐	Bypass interceptor analysis
CCTV		☐	Estimated from CCTV
CIP		☐	Near future CIP project
CITY		☐	Verification from City Staff
DWG		☐	Data from record drawing
GIS		☐	Data from City's GIS geodatabase
HCIP		☐	HydroScience-recommended CIP project
INT		☐	Interpolated
LID		☐	Elevation from LiDAR data
MIN		☐	Minimum slope (0.004)
PREV		☐	Data from previous model
SUR		☐	Data from 2021 manhole survey
SUR8		☐	Data from 2008 manhole survey

Notes:
1. Flags with descriptions highlighted gray are system defaults.

Figure 6-5: InfoWorks *Profile View*



Link	-	E3-028.1	E3-030.1	E3-032.2	E3-032.3	E3-034.1	XXXX000011.1	E3-036.1
length (ft)	116.7	383.4	501.7	501.4	331.1	185.2	136.9	330.9
width (in)	12.0	12.0	12.0	12.0	15.0	15.0	15.0	15.0
us inv (ft AD)	10.977	11.922	13.368	14.870	14.950	13.896	13.493	13.176
ds inv (ft AD)	10.689	10.977	11.922	13.368	13.896	13.493	13.194	12.539
Conduit mate	VCP	VCP	VCP	VCP	VCP	VCP	VCP	VCP
grad (%)	0.247	0.246	0.288	0.300	0.318	0.218	0.218	0.193

Node	-	XXXX000151	E3-028	E3-030	E3-032	E3-034	XXXX000011	-	E3-036	E3-038
ground (ft AD)	-	20.974	22.281	23.750	24.467	24.725	24.913	-	24.894	24.792

Any “disconnected” nodes were reviewed and reconciled. Next, the InfoWorks *Inference* tool was used to interpolate missing pipe invert elevations with known invert data upstream and downstream. In the most upstream parts of the system, where there was no upstream invert to interpolate between, inverts were extrapolated based on the minimum slope identified in the City’s Standard Design Criteria for Sanitary Sewers (Appendix C of the 2018 SSMP); this states that the minimum slope for sanitary sewer pipes will be 0.004 ft/1 ft (0.4%).

The *Inference* tool was also used to interpolate unknown pipe diameters where the upstream and downstream diameters were known and matching. Locations where the upstream and downstream diameters differed were reviewed and updated based on available record drawings. Where the diameter could not be identified, the smaller of the two adjacent pipe diameters was assumed. In the most upstream parts of the system where pipe diameters were unknown, a minimum pipe size of 8-inches was assumed per the City’s Standard Design Criteria for Sanitary Sewers. A table of the pipelines with interpolated diameters in the model is included in **Appendix E**.

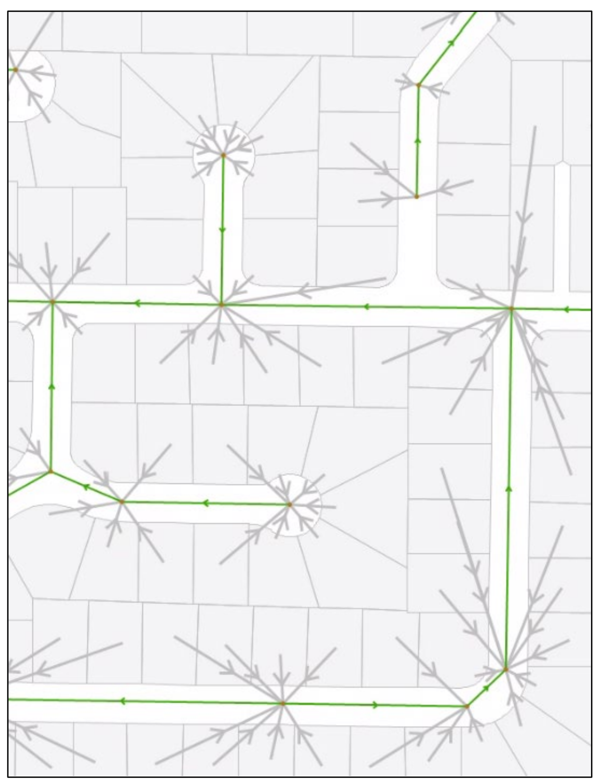
All locations where the model calculated an inverse slope and at manholes where the invert of an outgoing pipe was higher than the invert of the incoming pipe (i.e., a jump) were individually reviewed. Many of the locations where an inverse slope was calculated by the model were misrepresented flow splits that were not captured and represented in the original 2008 survey with separate invert elevations. These locations were updated based on record drawings, where available; where there were still discrepancies, these manholes were included in the 2021 manhole survey (**Section 6.1**).

6.6 Parcel Load Allocation

In InfoWorks, the wastewater flows contributed by each parcel are stored in subcatchments. For the Master Plan, all parcels within the City’s service area – detailed in **Section 2.1** and displayed in **Figure 2-1** – was imported into the hydraulic model as a subcatchment.

Each parcel was then assigned to the corresponding manhole that it flows into. Subcatchments were initially assigned the nearest manhole. A quality control check was performed alongside the City’s “ssLateralLine” GIS geodatabase feature class to ensure each parcel’s flow was matching its actual flow path in the collection system; adjustments were made as necessary. **Figure 6-6** is a sample of the final load allocation from the subcatchment centroid to the corresponding receiving manhole.

Figure 6-6: Sample Load Allocation



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SECTION 7 – FLOW MONITORING PROGRAMS

Two sets of flow monitoring data were utilized in the calibration of the hydraulic model for this Master Plan effort. The first data set was from a flow monitoring program conducted in October 2014 and represents dry weather flow monitoring. The second was performed as part of this Master Plan between February and March 2021 and represents wet weather flow monitoring. This section details the model scenarios developed for calibration and presents the flow monitoring programs.

7.1 Calibration Scenario Development

Due to the Stay-at-Home orders implemented in response to the COVID-19 pandemic, sewer flows for 2020 and early 2021 were atypical of historic results. As businesses were forced to shut down and many people transitioned to working from home or lost their jobs, both water use and sewer flows shifted from commercial areas to residential areas.

As a result, sewer flow conditions captured in the 2021 Flow Monitoring Study are not likely representative of typical dry weather wastewater flow conditions. To reconcile this, dry weather flow (DWF) was calibrated using data from the 2014 Flow Monitoring Report. Then, once calibrated to DWF, data from the 2021 Flow Monitoring Report was utilized to calibrate the collection system to WWF. Although the 2021 flows may not represent typical DWF, the difference between DWF and WWF provides important information on how the system responds to rainfall – i.e., the extent of rainfall dependent infiltration and inflow (RDI/I).

Since the development of the 2010 model, the City has implemented CIP projects that were incorporated into the updated model for this Master Plan. Record drawings were provided by City staff for each project. Most of these projects consisted of upsizing gravity mains. One project, however, had a more significant effect on downstream flows. Just south of Hwy 101, the collection system previously crossed underneath Stevens Creek toward State Route (SR) 85. In 2019, as part of the City's Leong Drive Water and Sewer Main Replacements Project, this crossing was abandoned and wastewater that once flowed to the Central Trunk was diverted to the East Trunk. Because this project was constructed after the 2014 Flow Monitoring Study, it was necessary to develop two different scenarios for the calibration of the model for DWF using data from the 2014 Flow Monitoring Study and WWF using data from the 2021 Flow Monitoring Study. **Figure 7-1** identifies the CIP projects implemented between 2010 and 2014, between 2014 and 2021, and planned or completed after 2021.

The scenarios developed for the calibration of the City's collection system are presented below:

- **2014 DWF Calibration (Base):** This scenario includes all infrastructure updates that were completed between 2010 – the most recent update of the hydraulic model – and 2014 – the date of the DWF flow monitoring and assumes the same land uses as those for 2021 (see **Section 5.1**). This scenario includes adjustments at critical flow splits as detailed in **Section 8.2.4** and was used for the calibration of DWF including GWI, BSF, and diurnal patterns.
- **2021 WWF Calibration:** This scenario builds upon the 2014 DWF Calibration scenario and includes infrastructure updates that were completed between 2014 and 2021 – the date of the WWF flow monitoring. The adjustments made in the 2014 DWF Calibration scenario were returned to design conditions. This scenario was used for the calibration of WWF.

7.2 Dry Weather Flow Monitoring

The City conducted flow monitoring during the 2014 dry weather season. Eighteen Teledyne Isco 2150 flow meters were installed throughout the City between October 1, 2014 and October 19, 2014. This type of meter is a velocity flow meter installed in collection system manholes to measure both depth and velocity at either 5- or 15-minute intervals for the duration of the flow monitoring period. Depth, velocity, and flow (calculated using the Continuity Equation) were presented in the 2014 Flow Monitoring Report in the form of daily averages, and the full dataset was provided separately as MS Excel files in the form of 15-minute averages. The 2014 Flow Monitoring Report is included as **Appendix A**.

This study collected only dry weather flow and thus did not have accompanying rain gauges. The locations of each of the 18 flow meters are identified in **Figure 7-2**. It is noted that in the 2014 Flow Monitoring Report most of the flow meters are referred to by the manhole ID they were placed in while some are referred to as "MV-#." **Figure 7-2** identifies each flow meter with the Report ID in parentheses for reference. For the duration of this Master Plan, all flow meters are referred to by the respective manhole ID.

HydroScience reviewed the location of each individual flow meter by comparing the City's GIS geodatabase, Google Maps imagery, and the photos included in the report taken at each installation site. One discrepancy was found: flow meter F1-108 was installed in the neighboring manhole F1-110. There are no additional tributary areas between these two manholes, so this did not affect the flow analysis. For the duration of this Master Plan, this flow meter is referred to as F1-110, the manhole ID it was installed in.

7.2.1 2014 Subbasin Delineation

The flow meters served as the basis for the delineation of tributary subbasins in the collection system for use during the model calibration process. Generally, for subbasins located at the most upstream ends of the collection system, all flow from the subbasin is captured by a single flow meter which in turn flows to downstream subbasins. Where there are flow splits, flow can be directed to more than one downstream subbasin, so it is important to understand the flow behavior at each flow split to properly delineate subbasin boundaries.

The identification of all flow splits (see **Section 3.1.1**) was imperative to track flow through the system. Each flow split was reviewed and identified as either a "non-critical" or "critical" flow split, as follows:

- **Non-Critical Flow Split:** These flows splits and associated flows are fully contained within a single subbasin. The flow is split then returns to the same subbasin or flow path before it reaches the next downstream flow meter.
- **Critical Flow Split:** These are flow splits that divert flow to two different downstream subbasins (or flow meters). All critical flow splits were evaluated using data from the 2021 manhole survey detailed in **Section 6.1** or record drawings, where available.

It is noted that the distinction between critical and non-critical flow splits is dependent on the subbasins created by the flow meter locations. Because of this, the critical/non-critical distinction may differ for each flow monitoring program. For the 2014 flow monitoring program, four of the 165 total flow splits in the City's collection system are considered critical.

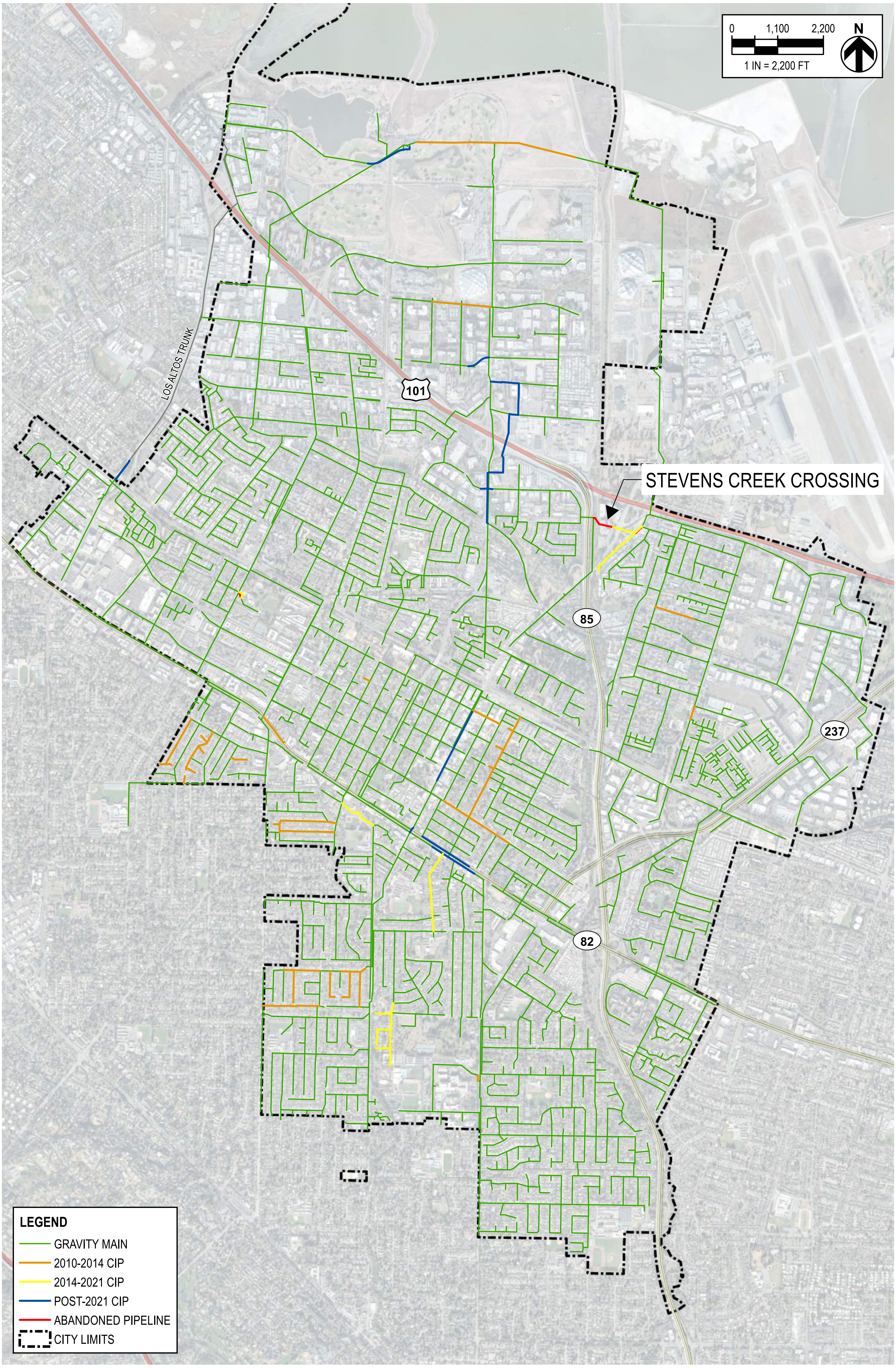


FIGURE 7-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
INFRASTRUCTURE UPDATES

HydroScience

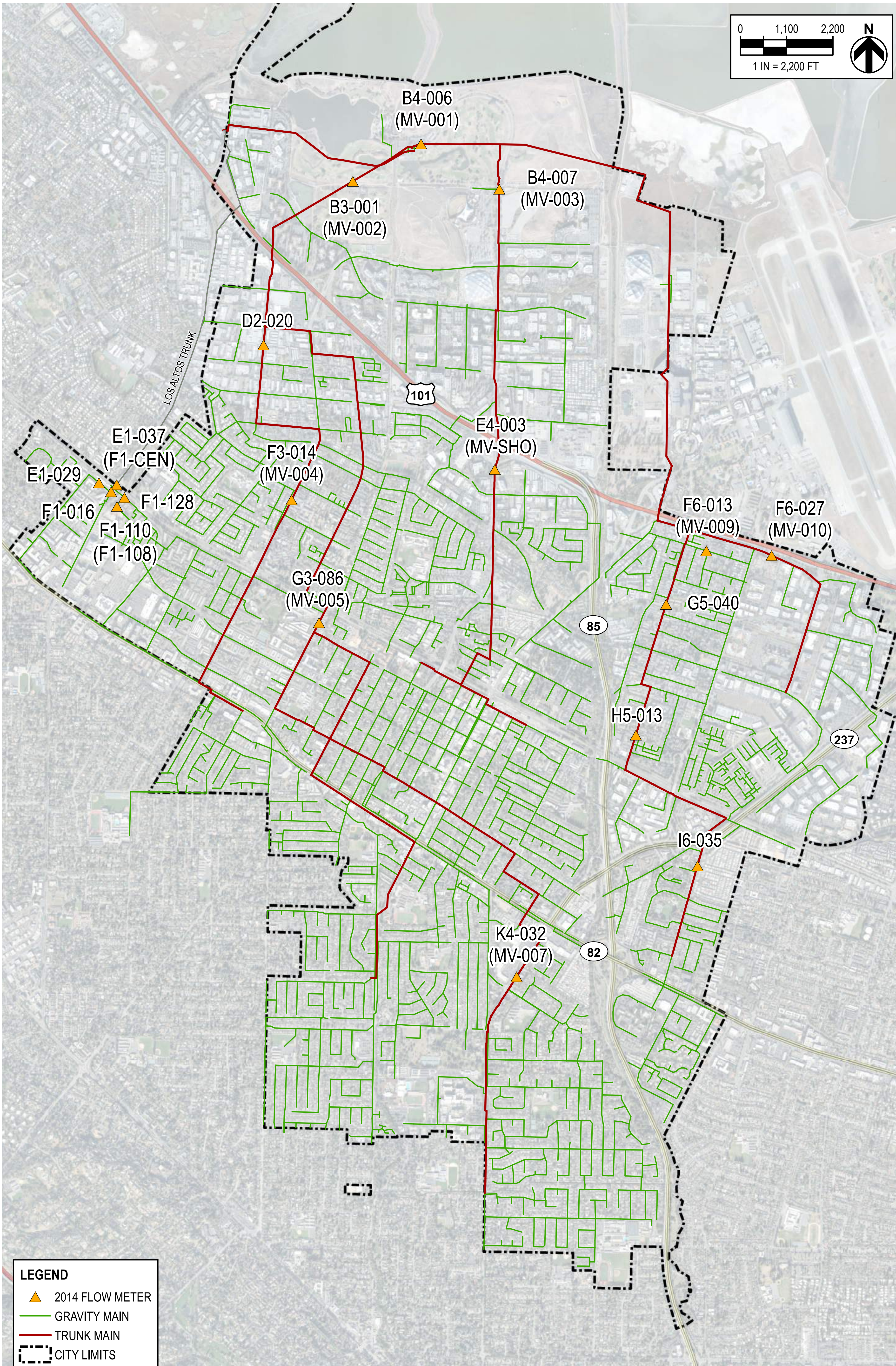
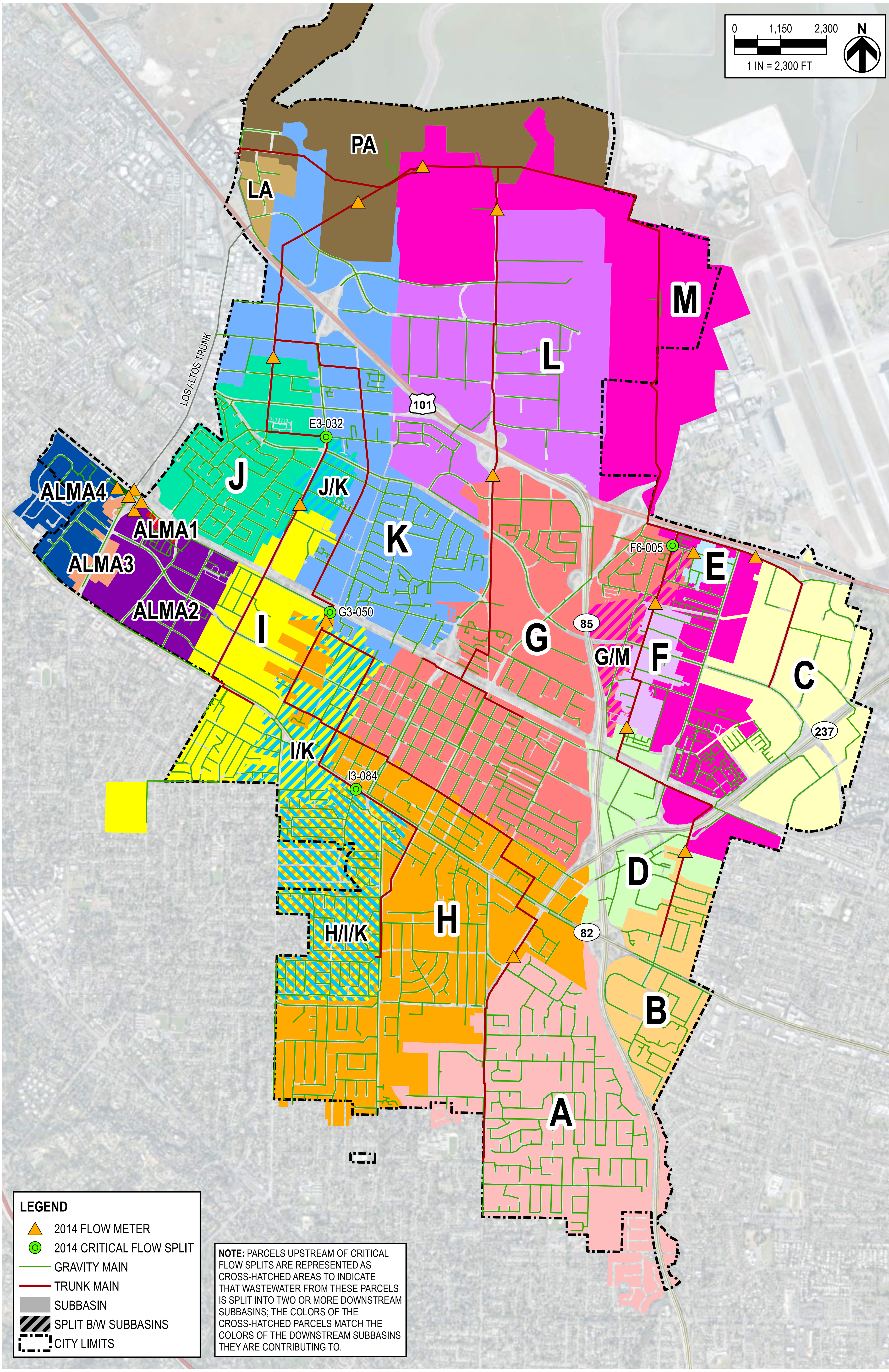


FIGURE 7-2
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2014 FLOW METER LOCATIONS

HydroScience



HydroScience

FIGURE 7-3
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2014 SUBBASIN DELINEATION

The results of the subbasin delineation, including identification of each critical flow split, are displayed in **Figure 7-3**. Parcels upstream of critical flow splits are represented as cross-hatched areas to indicate that wastewater from these parcels is split into two or more downstream subbasins; the colors of the cross-hatched parcels match the colors of the downstream subbasins they are contributing to. **Figure 7-4** is a schematic representation of the subbasin delineation showing the flow through the subbasins and flow meters in the collection system. It is noted that there are two small areas of the system that are not captured by any flow meters. The first is the area northwest of SPS that joins the flow from SPS at the intersection of San Antonio Rd and Casey Ave. This area is labeled as "PA" because it flows directly to the City of Palo Alto without measurement by any flow meters within the City. The second area is located to the west of the West Trunk. Flow from this area discharges directly to the Los Altos Trunk Main at multiple points along Bayshore Pkwy. This area is labeled as "LA" because it discharges to the Los Altos Trunk Main. Additionally, **Table 7-1** presents a summary of the areas by land use within each subbasin.

Table 7-1: Total Acreage of Each Land Use by Subbasin (2014)

Subbasin	COM	HDR	IND	LDR	LMDR	MBL	MDR	MOT	MU	OFF	OS	PF	Total
A				389	9					1	57	121	**577**
B	10	57		31	31		43	4	2	2	< 1	11	**191**
C		5	240		10				3	85	18		**360**
D	1	14	30	29	3	27	2	4			19	17	**146**
E		6		9	2		< 1				1		**17**
F		4		10	30		23				< 1	1	**70**
G	14	123	37	206	55	10	65	8	26	23	57	59	**682**
G/M		16	1	4	9		16				12	12	**70**
H	53	71	1	288	19	17	38	3	10	12	54	95	**662**
H/I/K	2	7		198	2		7		< 1		8	15	**239**
I	12	82		61	11		20		11	5	4	66	**273**
I/K	11	42		45	5		9	4	2	< 1	4	15	**137**
J	7	15	13	122	16		1		3	21	5	37	**241**
J/K		15		3	17		6				< 1	2	**42**
K	39	83	125	102	70		34		2	4	67	82	**608**
L	18		299	22		38		1	2	64	117	79	**641**
M	3	24	44	35	52	13	8		2	13	314	193	**700**
ALMA1	< 1	< 1		1	2								**4**
ALMA2	53	31		13	18		133	1	7	7	6	3	**273**
ALMA3	6	2		< 1				2	9	< 1	2	3	**24**
ALMA4	4	39		44	4		7		< 1		8	2	**109**
ALMA												2	**2**
LA			23							12		3	**39**
PA			19								567		**586**
Total	**234**	**635**	**832**	**1,613**	**367**	**105**	**412**	**26**	**79**	**250**	**1,321**	**818**	**6,693**

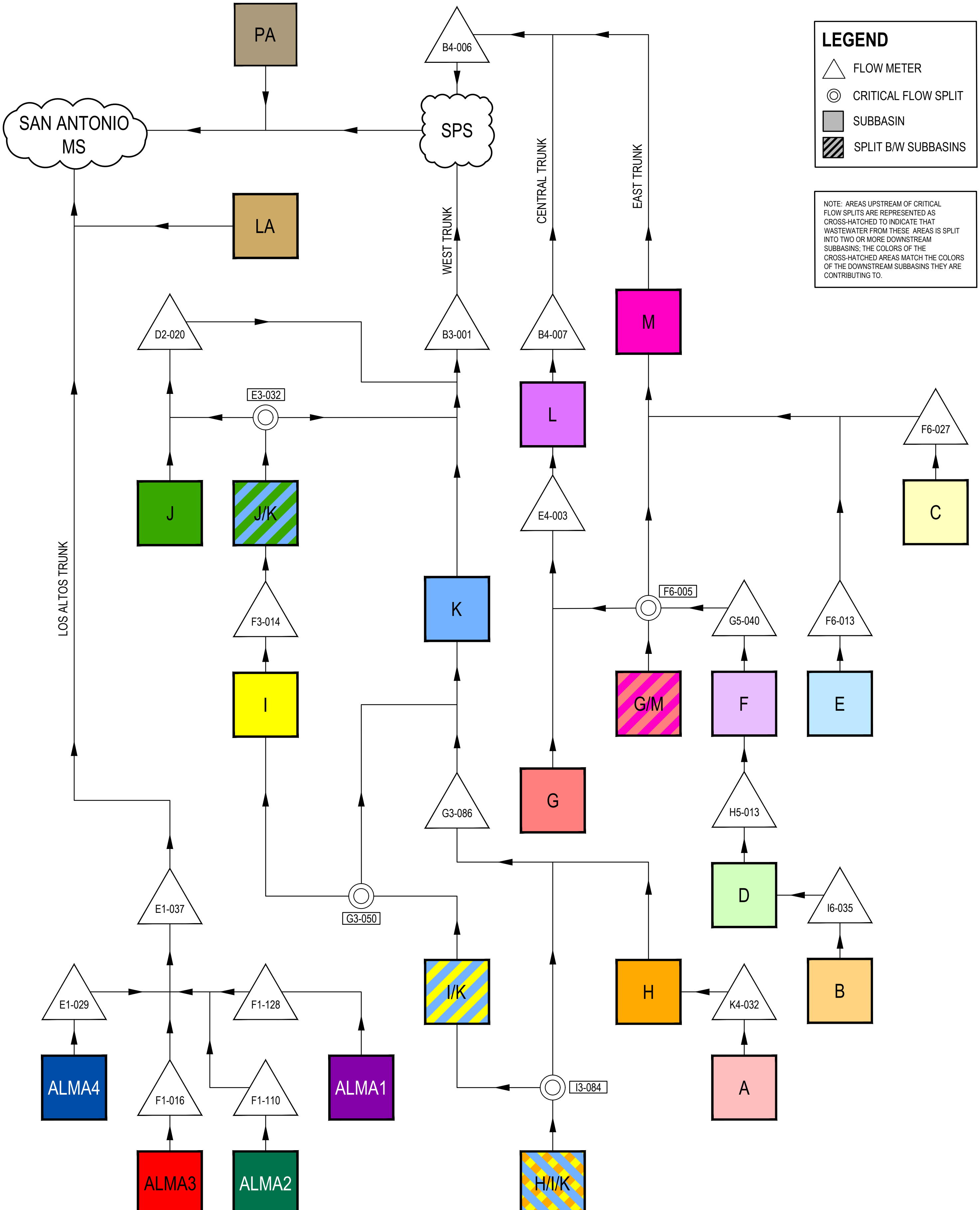


FIGURE 7-4
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2014 SEWER SUBBASIN SCHEMATIC FLOW DIAGRAM

HydroScience

7.3 Wet Weather Flow Monitoring

The City conducted wet weather flow monitoring during the 2020-2021 winter season in support of this Master Plan effort. Sixteen HACH FL902 area-velocity flow meters were installed throughout the City between February 17, 2021 and April 18, 2021. Similar to the meters used in the 2014 Flow Monitoring study, these flow meters measure both depth and velocity at either 5- or 15-minute intervals for the duration of the flow monitoring period. Depth, velocity, and flow (calculated using the Continuity Equation) were presented in the 2021 Flow Monitoring Report in the form of daily averages, and the full dataset was provided separately as MS Excel files in the form of 15-minute averages. The 2021 Flow Monitoring Report is included as **Appendix C**.

The flow meter locations were reviewed and selected by HydroScience in coordination with City staff. For consistency between the dry weather (2014) and wet weather (2021) data sets, many of the flow meter locations were duplicated with a few exceptions.

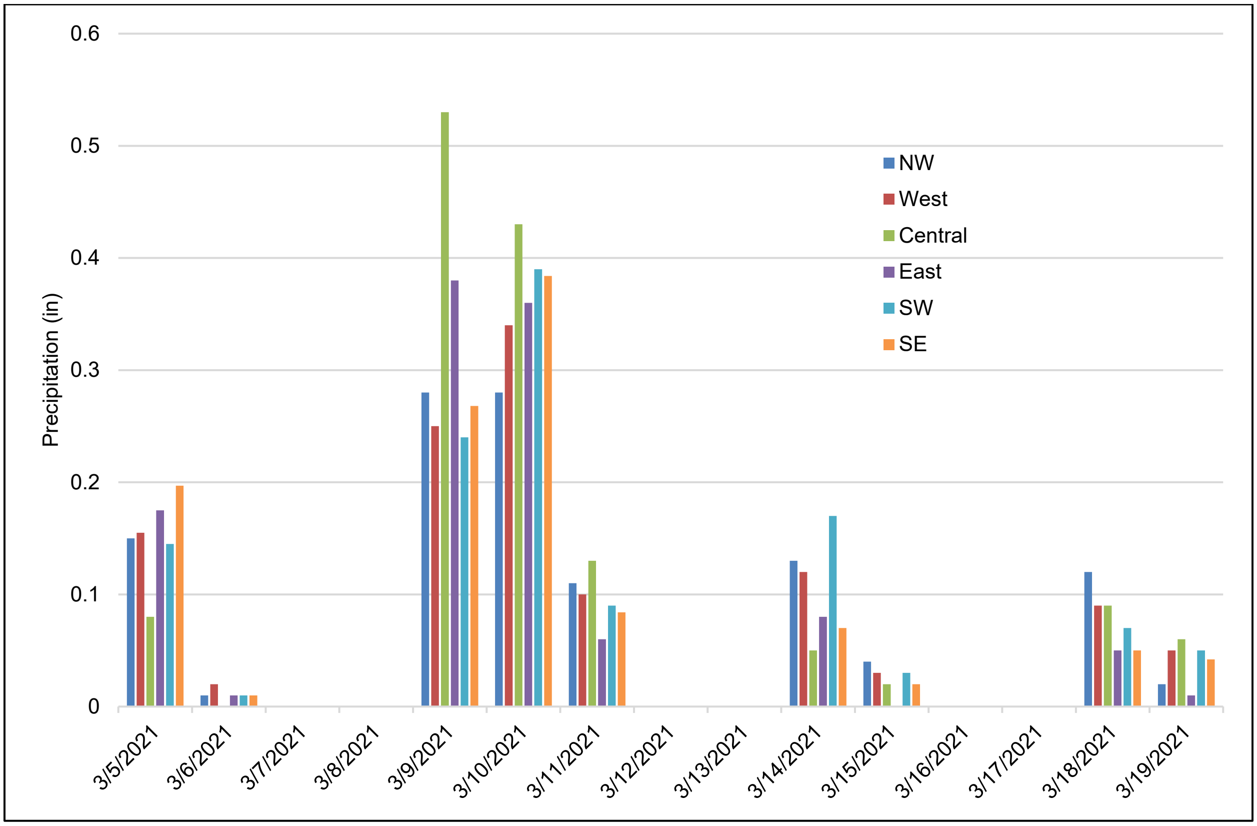
Similar to the 2014 Flow Monitoring Report, some of the flow meters in the 2021 Report are identified with the convention of “MV-#.” HydroScience verified the location of installation of each individual flow meter and for the duration of this Master Plan, all flow meters are referred to by the respective manhole ID.

The following list summarizes the differences between the flow meter locations for the 2014 and 2021 flow monitoring studies as well as the discrepancies found in the manhole installation locations:

- **E1-035.** In the area of the Alma Recorder, there were five flow meters installed for the 2014 Flow Monitoring study. For the 2021 study, the four upstream flow meters were omitted allowing the single flow meter to capture all flow entering the Los Altos Trunk. Upon review of the installed location, it was determined that this flow meter was placed in the manhole immediately downstream of the intended manhole in E1-035. There are no additional tributary areas between these two manholes, so this did not affect the flow analysis.
- **F4-070.** The flow meter that was placed in manhole E4-003 in 2014 was relocated upstream to manhole F4-070 to avoid the Caltrans permitting process.
- **H6-028.** An additional flow meter was placed in manhole H6-028 to isolate an area of the collection system that was identified as having high rates of inflow and infiltration. This flow meter is listed as H6-036 in the 2021 Flow Monitoring Report; however, it was determined that it was placed in the manhole just downstream – H6-028. There are no additional tributary areas between these two manholes, so this did not affect the flow analysis.
- **E2-036.** A flow meter was placed in manhole E2-036 to isolate an area of the collection system where the City has observed abnormally high flows during wet weather events.

As part of the 2021 Flow Monitoring study, six rain gauges were also installed throughout the City for the duration of the flow monitoring. Depth of precipitation was presented in the 2021 Flow Monitoring Report in the form hourly averages to the nearest hundredth of an inch, and the raw data was recorded in 15-minute increments to the nearest hundredth of an inch. Average daily rainfall for each of the six rain gauges for the month of March is shown in **Figure 7-5**. A small amount of rainfall was recorded on February 20th but was not considered significant. There was no rainfall recorded after March 19th.

Figure 7-5: Daily Rainfall During Monitoring Period



The maximum rainfall for each of the six rain gauges is presented alongside the National Oceanic and Atmospheric Administration (NOAA) precipitation frequency estimates for Mountain View, CA (**Table 7-2**). The largest storm captured during the 2021 flow monitoring period took place March 10, 2021. This rainfall event was selected for use in the model calibration for wet weather. It is more ideal to use at least a 1- to 2-year storm event to calibrate estimated RDI/I responses. However, there were some observed RDI/I responses for the March 10, 2021 storm, and this was the best data available at the time of this study.

Table 7-2: NOAA Rainfall Frequency vs. Rain Gauge Data

	NOAA Frequency Estimates				Rain Gauge Data					
Duration	1-yr	2-yr	5-yr	10-yr	NW	W	C	E	SW	SE
1-hr	0.33	0.40	0.50	0.59	0.11	0.19	0.19	0.14	0.20	0.17
2-hr	0.48	0.58	0.73	0.86	0.14	0.20	0.20	0.15	0.21	0.18
6-hr	0.85	1.04	1.31	1.54	0.21	0.26	0.33	0.24	0.27	0.25
12-hr	1.12	1.38	1.74	2.05	0.27	0.33	0.42	0.36	0.39	0.38
24-hr	1.36	1.68	2.12	2.50	0.32	0.37	0.50	0.36	0.42	0.41

The locations of all flow meters and rain gauges utilized as part of the 2021 wet weather flow monitoring are shown in **Figure 7-6**. For comparison, the flow meters from the 2014 flow monitoring program are also shown on this figure. Also for reference, the ID listed in the 2021 Flow Monitoring Report is provided in parentheses.

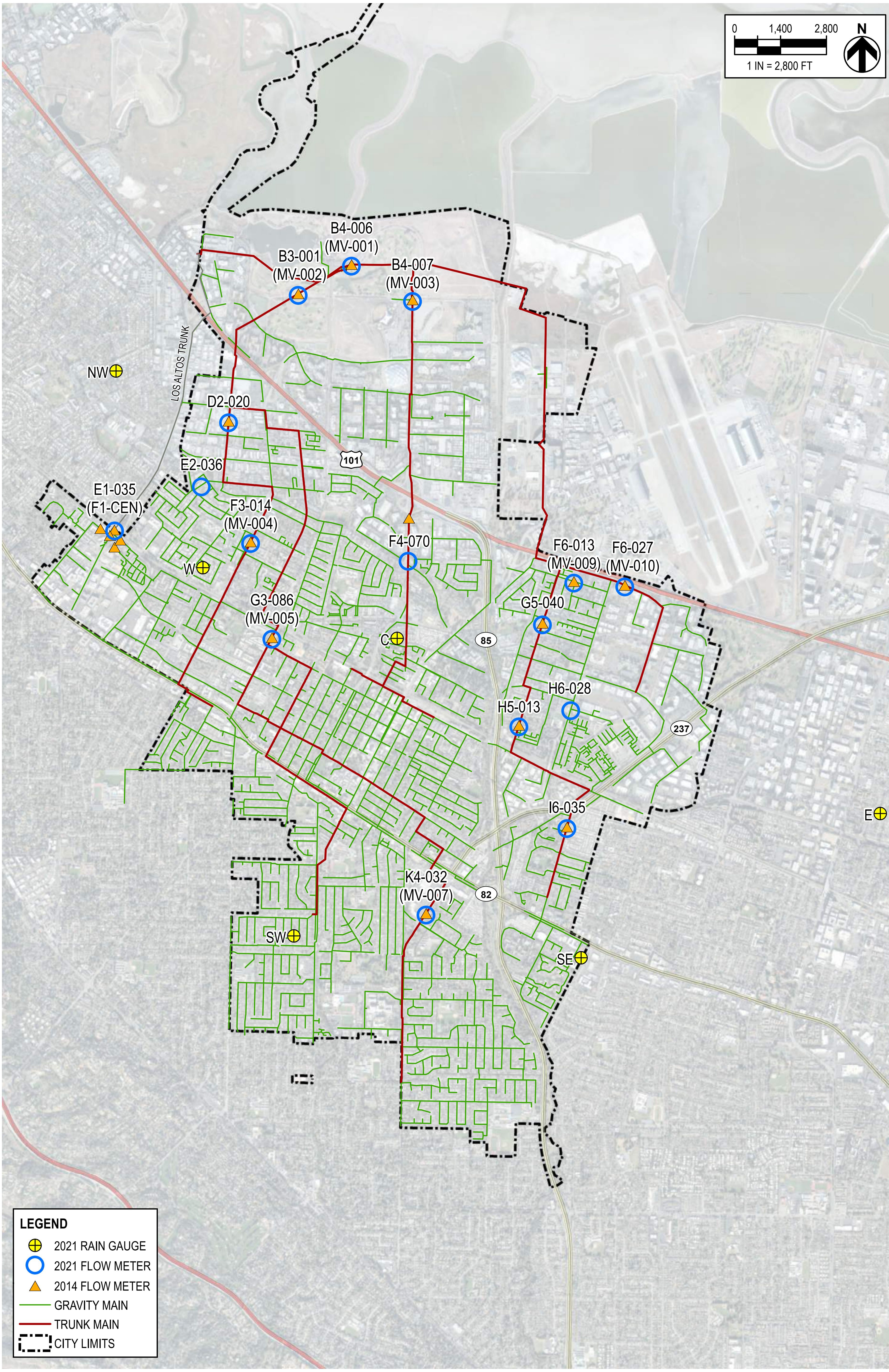
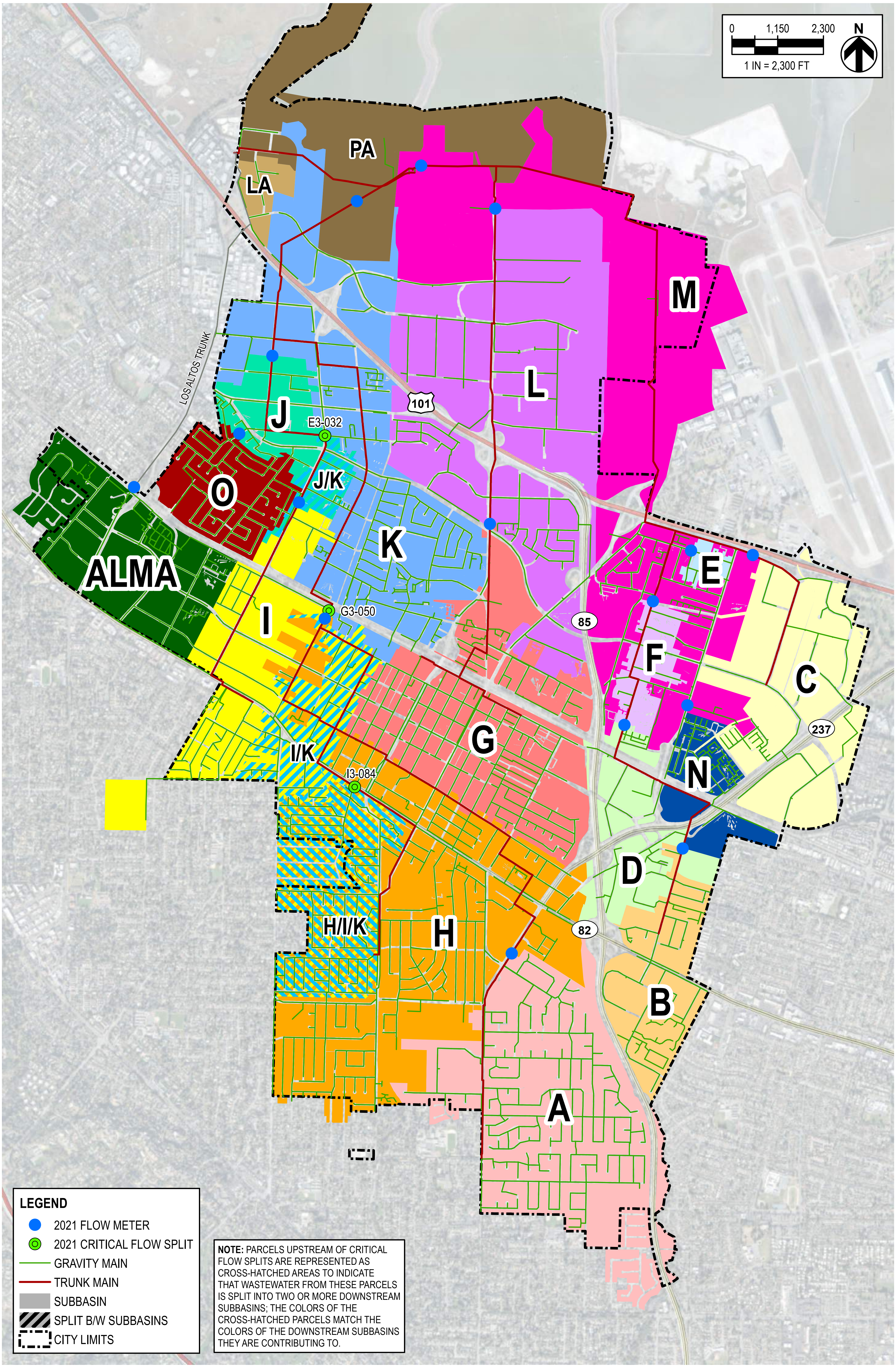


FIGURE 7-6
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2021 FLOW METER AND RAIN GAUGE LOCATIONS

HydroScience



HydroScience

FIGURE 7-7
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2021 SUBBASIN DELINEATION

7.3.1 2021 Subbasin Delineation

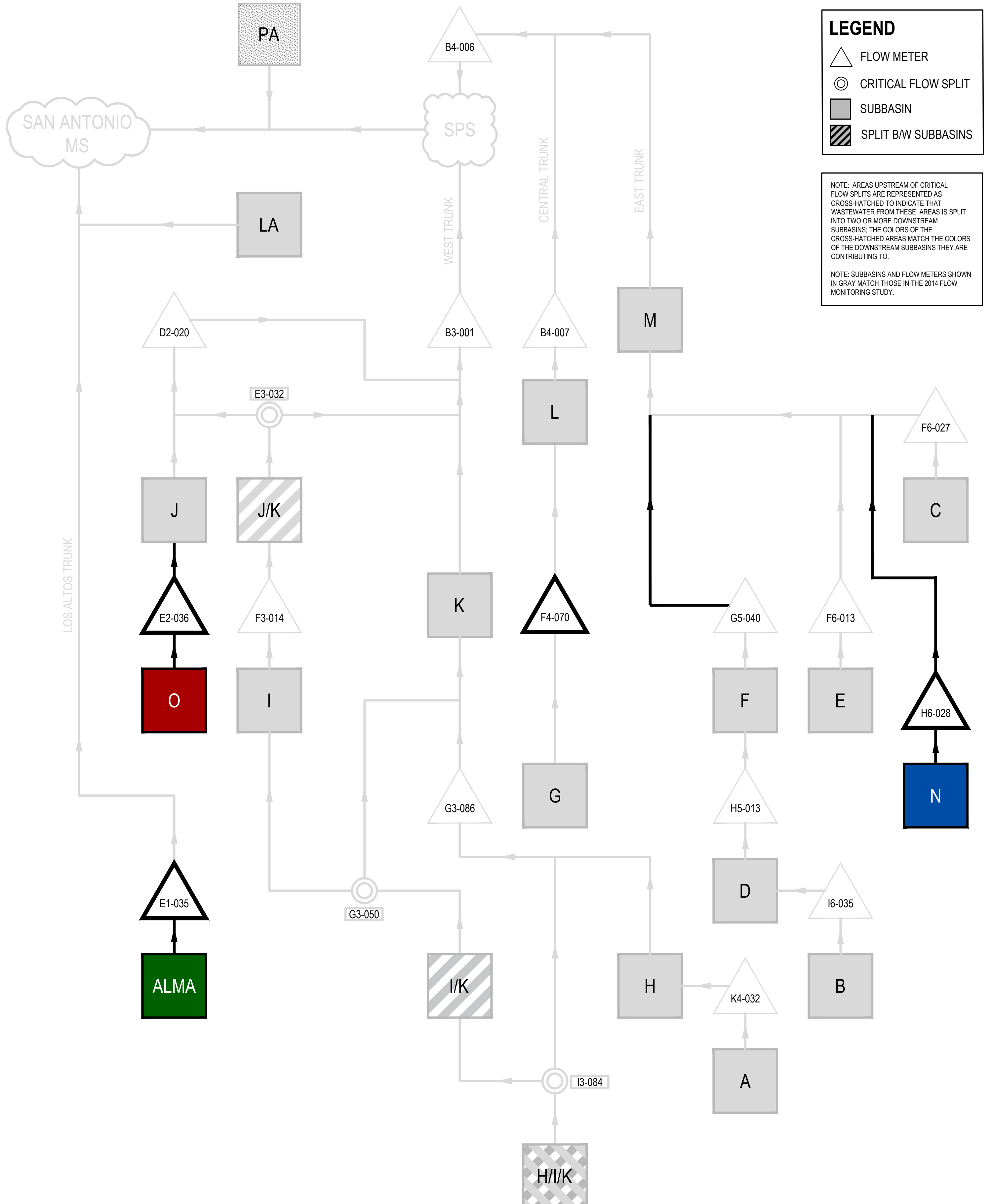
The tributary areas of each of the 2021 flow meters were delineated using the same methods described in **Section 7.2.1**. Due to infrastructure upgrades that took place between 2014 and 2021, specifically at the Stevens Creek crossing (see **Figure 7-1**), one of the critical flow splits identified for the 2014 flow monitoring program is now non-critical. The other two previously-identified critical flow splits remain for the 2021 flow monitoring program.

The results of the 2021 subbasin delineation are presented in **Figure 7-7**. **Figure 7-8** is a schematic representation of the subbasin delineation showing the flow through the subbasins and flow meters in the collection system.

Note that the subbasins for the 2014 and 2021 differ slightly due to the changes listed in **Section 7.1** as well as the infrastructure updates made between 2014 and 2021. **Figure 7-8** identifies each of the differences between the 2014 and 2021 schematic flow diagrams. A summary of the various contributing consolidated land uses within each subbasin is shown in **Table 7-3**.

Table 7-3: Total Acreage of Each Land Use by Subbasin (2021)

Subbasin	COM	HDR	IND	LDR	LMDR	MBL	MDR	MOT	MU	OFF	OS	PF	Total
A				389	9					1	57	121	**577**
ALMA	63	78		59	25		140	3	11	8	16	9	**411**
B	10	57		31	31		43	4	2	2	< 1	11	**191**
C		5	240		10				3	85	18		**360**
D	1	14	30	29	3	27	2	4			19	17	**146**
E		6		9	2		< 1				1		**17**
F		4		10	30		23				< 1	1	**70**
G	8	76	4	160	29	10	46		24	14	33	47	**451**
H	53	71	1	288	19	17	38	3	10	12	54	95	**662**
H/I/K	2	7		198	2		7		< 1		8	15	**239**
I	12	82		61	11		20		11	5	4	66	**273**
I/K	11	42		45	5		9	4	2	< 1	4	15	**137**
J	7	11	13	18	16		< 1		3		4	27	**100**
J/K		15		3	17		6				< 1	2	**42**
K	39	83	125	102	70		34		2	4	67	82	**608**
L	23	34	332	52	20	38	14	3	2	73	129	90	**811**
LA			23							12		3	**39**
M	4	43	27	35	38		30	7	3	13	333	205	**737**
N	< 1	8	17	20	30	13					4	< 1	**94**
O		4		104	< 1		1			21	1	10	**141**
PA			19								567		**586**
Total	**234**	**640**	**832**	**1,613**	**367**	**105**	**412**	**26**	**74**	**250**	**1,321**	**818**	**6,693**



NOTE: AREAS UPSTREAM OF CRITICAL FLOW SPLITS ARE REPRESENTED AS CROSS-HATCHED TO INDICATE THAT WASTEWATER FROM THESE AREAS IS SPLIT INTO TWO OR MORE DOWNSTREAM SUBBASINS; THE COLORS OF THE CROSS-HATCHED AREAS MATCH THE COLORS OF THE DOWNSTREAM SUBBASINS THEY ARE CONTRIBUTING TO.

NOTE: SUBBASINS AND FLOW METERS SHOWN IN GRAY MATCH THOSE IN THE 2014 FLOW MONITORING STUDY.

FIGURE 7-8
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
2021 SEWER SUBBASIN SCHEMATIC FLOW DIAGRAM

HydroScience

SECTION 8 – WASTEWATER FLOW ANALYSIS

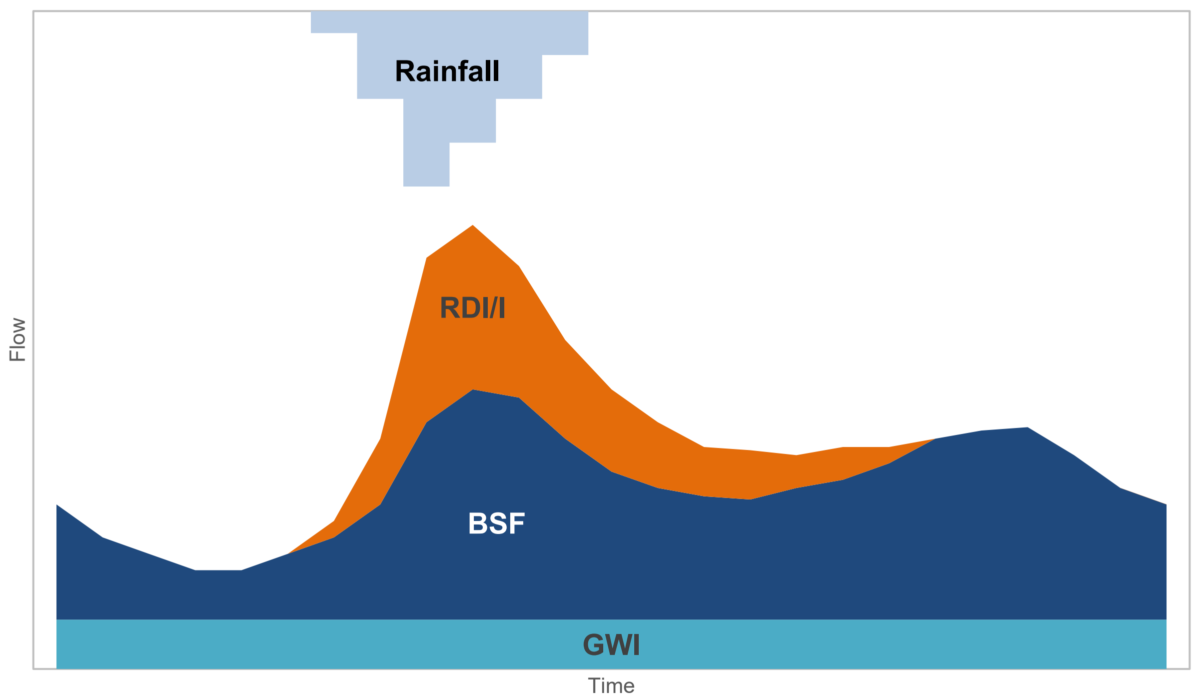
The hydraulic model was calibrated using the flow monitoring data presented in **SECTION 7**. This section summarizes typical wastewater flow components, describes the basis for developing design wastewater flows, and presents the results of the model calibration.

8.1 Typical Wastewater Flow Components

Wastewater flows consist of three primary flow components: GWI, BSF, and RDI/I as illustrated in **Figure 8-1**.

- **Groundwater Infiltration (GWI)** – For modeling purposes, this document defines GWI as any and all sources of constant flow. GWI is comprised both of groundwater infiltration and any other constant flow that is generally detected during low flow periods. Constant flows can be associated with leaking faucets, running toilets, and/or manufacturing or industrial processes, etc. This can also capture any meter reading variance at extremely low flows (see **Section 8.2.1**).
- **Base Sanitary Flow (BSF)** – This represents the direct diurnal flow contributions from customers. This is represented by a combination of unit flows and diurnal patterns, which is further detailed in **Section 8.2.2**.
- **Rainfall-Dependent Inflow and Infiltration (RDI/I)** – This is the volume of rainfall that enters the wastewater collection system through various avenues (holes in manhole covers, elevated water table, etc.). This is discussed in **Section 8.3.2**.

Figure 8-1: Typical Wastewater Flow Components



To understand hydraulic capacity of a wastewater collection system, both dry weather flow (DWF) and wet weather flow (WWF) conditions must be evaluated. DWF includes both GWI and BSF while WWF includes GWI, BSF, and RDI/I. The flow monitoring and rainfall data collected by the City was evaluated for each of these components, and this process is described in the following sections.

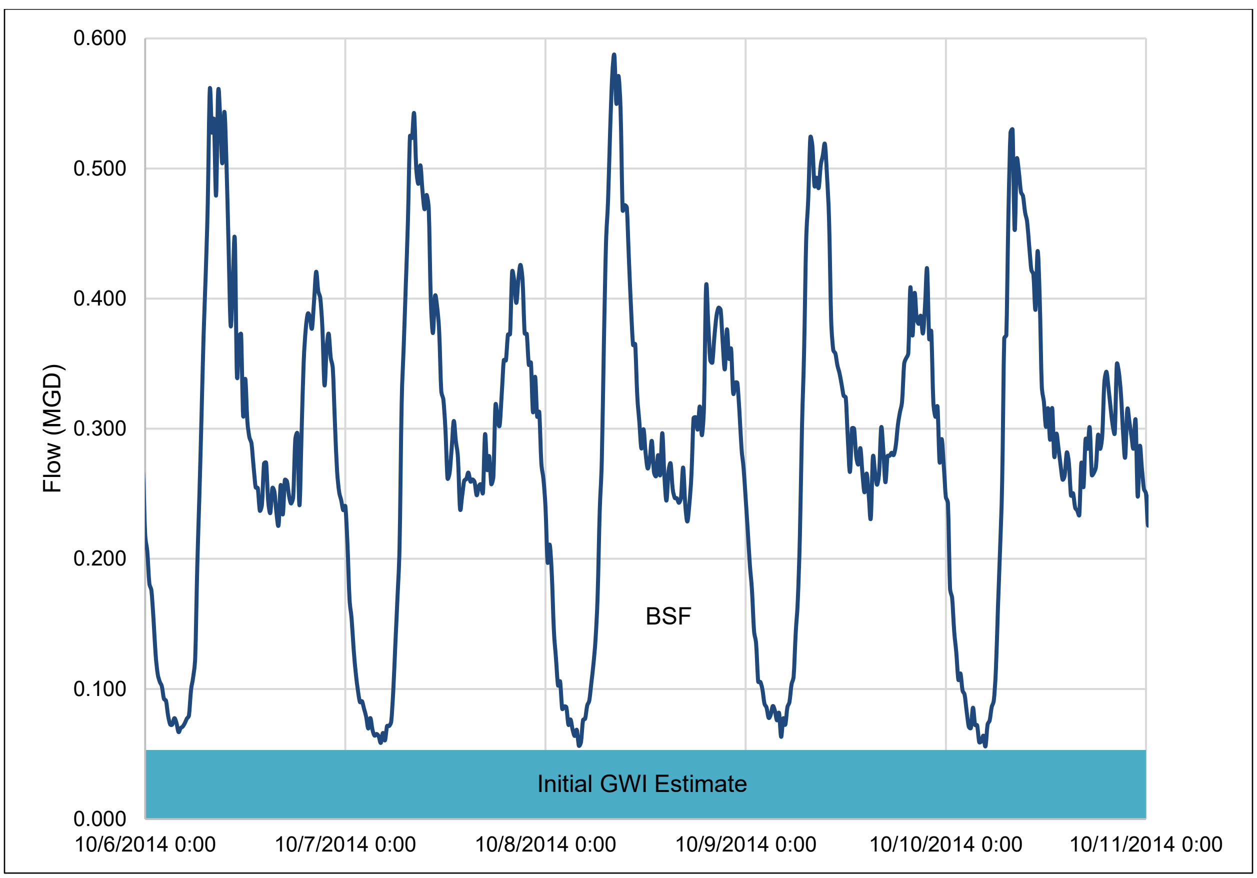
8.2 Dry Weather Flow (DWF)

The calibration for DWF includes GWI on a subbasin-by-subbasin basis, BSF calculated using unique unit flow (UF) factors for each land use, and diurnal patterns for groups of similar land use types. The process for developing each of these components and the hydraulic model DWF calibration results are discussed below.

8.2.1 Groundwater Infiltration (GWI)

To isolate BSF for the calculation of UF factors, an estimate of GWI was developed. For each 2014 flow meter, the daily minimum flow was calculated to serve as the initial basis for the GWI estimates. **Figure 8-2** presents an example of the DWF for flow meter I6-035 and the corresponding initial GWI estimate. Once GWI estimates were developed for each flow meter, GWI from any upstream flow meter(s) was subtracted to isolate GWI for each subbasin.

Figure 8-2: Flow Meter I6-035 DWF



Using these initial values, a unique unit GWI (gpd/acre) was calculated for each subbasin by dividing the total estimated GWI by the total contributing subbasin acreage. This unit GWI was then applied to each parcel by multiplying the parcel area by the unit GWI for the respective subbasin. These initial GWI estimates served as a starting point for model calibration and were further refined during the calibration using an iterative process described in **Section 8.2.4**.

Because there is no detailed flow data downstream of the PA and LA areas, modeled GWI for these parcels was interpolated based on average values in the surrounding subbasins.

8.2.2 Base Sanitary Flow (BSF)

Total BSF for each subbasin was calculated by subtracting estimated GWI from the total average DWF which was then used to calibrate UF factors. This process was coordinated with the water unit demand analysis conducted as part of the City's 2022 Water Master Plan.

Customer water meter billing data for fiscal year (FY) 2015/16 to 2019/20 was processed by matching each account to its corresponding parcel and land use. A unique average water demand factor was developed for each of the 11 consolidated land use categories (**Table 5-1**). For the low-density residential land uses, factors were developed on a per-parcel basis assuming that one dwelling unit (DU) occupies each parcel, resulting in units of gpd per dwelling unit (gpd/DU). For all other land uses, factors were developed in units of gpd per acre (gpd/acre). Parcels observed to be vacant based on aerial imagery were assigned no flow and irrigation accounts were removed as they do not contribute to wastewater flows.

Initial wastewater UF factors were estimated at 75% of the initial water demand factors and applied to the hydraulic model. The UF factor development process was initiated in the most terminal/upstream subbasins with the fewest land use types, and UF factors were iteratively adjusted, in conjunction with GWI, until the modeled flows at the location of each 2014 flow meter matched the observed flows at that location reasonably well.

Once finalized, the UF factors were again compared to the final water unit demand factors developed as part of the Water Master Plan as a final quality check. **Table 8-1** presents the final calibrated UF factors and the total system flow contributed by each land use category in the 2014 DWF Calibration scenario.

Large dischargers – customers that discharge a disproportionately large volume of wastewater for their parcel size and land use type – were identified and removed from the UF factor analysis to avoid skewing UF factors. For the 2014 DWF Calibration, large dischargers were identified using the customer water meter billing data for FY 2015/16 as this was the best available data to represent the 2014 flow monitoring period. For that year, the top water users in terms of average water use in gpd were flagged for review. Irrigation accounts were removed, and wastewater flows were estimated at 75% of average water use. Each flagged account was ultimately classified as a large discharger if its estimated wastewater flow based on the meter billing data was at least 1.5 times the estimated wastewater flow calculated using the parcel's land use area based UF factor.

Wastewater flows for large dischargers were manually added to the model as point sources to ensure they and their impact to the collection system were accurately represented.

As mentioned in **Section 5.1**, Moffett Field is a unique land use that was also included in the model as a point source. Wastewater flow for Moffett Field is metered by the City on Parsons Ave. For the 2014 DWF Calibration scenario, Moffett Field flows were assigned based on the average annual measured flows from September 2014 to August 2015. This data represents flow prior to construction of the Google Bayview campus and is assumed to represent typical NASA sewer discharges.

Table 8-1: Final Calibrated UF Factors

Land Use Code	UF Factor	Unit	Quantity	Total Flow (MGD)	Diurnal Pattern
MU	3,800	gpd/acre	79	0.30	Mixed-Use
COM	1,400	gpd/acre	234	0.33	Non-Residential
HDR	1,700	gpd/acre	627	1.07	High-Density Residential
IND	550	gpd/acre	832	0.46	Non-Residential
LDR	120	gpd/DU[1]	9,599	1.15	Residential
LMDR	180	gpd/DU[1]	5,338	0.96	Residential
MDR	1,100	gpd/acre	412	0.45	High-Density Residential
MBL	800	gpd/acre	93	0.07	Residential
OFF	1,000	gpd/acre	250	0.25	Non-Residential
PF	700	gpd/acre	831	0.48	Non-Residential
MOT	2,800	gpd/acre	26	0.07	Residential
2014 Large Dischargers	-	-	-	0.23	Various[1]
Total	**-**	**-**	**-**	**5.82**	**-**

Notes:
1. Large dischargers were assigned a diurnal pattern based on their original land uses.

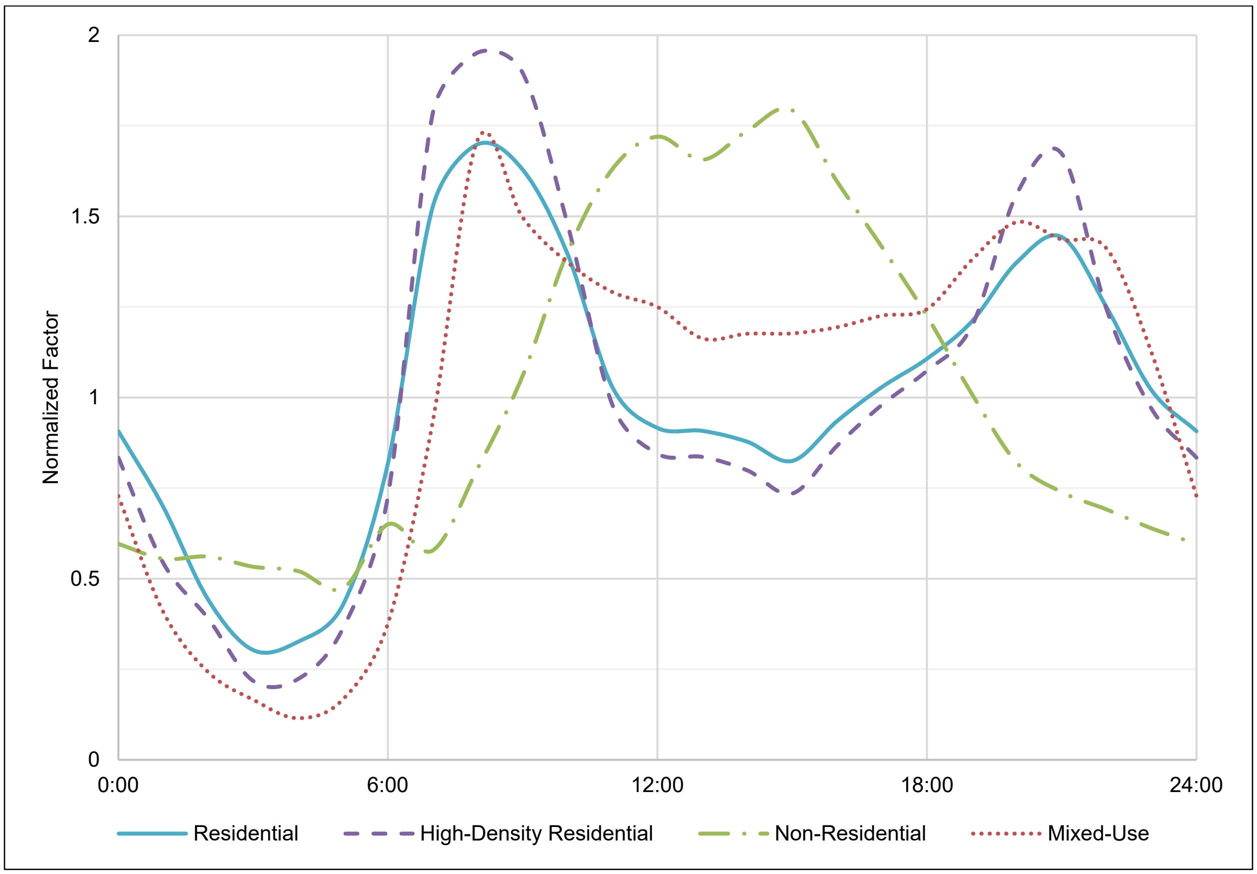
Flows utilized for the Los Altos Trunk Sewer Capacity Analysis were used to account for the wastewater flow entering the Los Altos Trunk Main from the City of Los Altos. Per Table 1 of the Los Altos Trunk Sewer Capacity Analysis, the City of Los Altos contributes 13.53 MGD and the Town of Los Altos Hills contributes 2.60 MGD during peak wet weather flow (PWWF) conditions. These two flow contributions were included in the hydraulic model as point sources entering the Los Altos Trunk Main at manhole E1-035 (Z1S-127 per Los Altos ID). The hydraulic modeling prepared for the Los Altos Trunk Sewer Capacity Analysis was a steady state model running at peak conditions and these flows were modeled as constant flows to be conservative.

8.2.3 Diurnal Patterns

To account for hourly flow fluctuations, a set of diurnal patterns was developed for different types of land use categories. Using the subbasins delineated for the 2014 flow monitoring program, subbasins were identified that were predominantly a single land use category and the flows from those subbasins were normalized to create a 24-hour diurnal pattern for that land use. For example, subbasin E, captured by flow meter F6-013, consists of all residential land uses (LDR, LMDR, MDR, and HDR); this subbasin was used to develop the residential diurnal patterns.

Diurnal patterns were adjusted iteratively to match observed flows during the calibration process. **Figure 8-3** displays the calibrated set of diurnal patterns developed.

Figure 8-3: Diurnal Patterns



8.2.4 Dry Weather Calibration Results

Calibration is an iterative process. Adjustments are made to all variables (i.e. GWI, UF Factors, and diurnal patterns) while comparing modeled flow with observed flow at each flow meter (see example **Figure 8-4**). One change to a subbasin affects all downstream subbasins. Thus, the general methodology is to begin calibration with the terminal upstream subbasins with the fewest land use types and unknowns, followed by calibration of downstream subbasins.

Total modeled flows were also compared to the historical daily average flows recorded at the San Antonio Metering Station and the Alma Recorder. While hourly flows are not available at those metering stations, the data does provide an additional check point to validate the total average modeled flows.

Where there were consistent discrepancies between modeled and observed flows, upstream critical flow splits (see **Figure 7-3**) were reviewed using available record drawings, survey data, and available CCTV footage. The following adjustments were made for the 2014 DWF Calibration scenario.

- F6-005 is a critical flow split at the intersection of Evandale Ave and Tyrella Ave. There is a diversion structure located at the west outflow of manhole F6-005 that was measured as part of the 2021 manhole survey. During low flows, wastewater is diverted to the east as part of subbasin M. When the diversion structure is overtopped, wastewater flows to the west to subbasin G. The elevation of the diversion structure was adjusted slightly during dry weather model calibration to aid in the calibration of downstream meter flows. This could represent debris build up at or downstream of the diversion.
- E3-032 is a critical flow split in the intersection of Rengstorff Ave and Rock St. Wastewater enters this manhole from the south, and during dry weather, it exits to the east and west. During higher flows, the north exit acts as an overflow outflow. Flow exiting to the east and north is captured as part of subbasin K; flow exiting to the west is captured as part subbasin J which ultimately flows into subbasin K. During dry weather calibration, modeled flows in subbasin J were significantly higher than observed flows, while modeled flows for subbasin K were reasonably accurate. To account for this, the invert of the west exit was adjusted to represent a possible downstream blockage and aid in the calibration of subbasin J flows. When compared to 2021 flow meter flows, subbasin J flows in 2014 appeared significantly lower than expected, supporting the theory that there may have been a blockage or other flow obstruction in the manhole during the course of the 2014 Flow Monitoring Study.
- G3-050 is a critical flow split located in the intersection of Crisanto Ave and Escuela Ave. Flow enters this manhole from the southwest and exits to the east, joining the parallel 21-in pipeline in Escuela Ave as part of subbasin K or continues to the northeast to Crisanto Ave as part of subbasin I which then continues downstream to subbasins J and/or K. This manhole was inaccessible during the 2021 manhole survey because it has been paved over. The inverts of this manhole were estimated based on CCTV data and it is noted that there are high flows in the pipeline. During dry weather calibration, the outflow inverts of this manhole were adjusted slightly to aid in the calibration of downstream subbasins.
- Observed flow meter data from the 2014 Flow Monitoring Study for subbasins D and F were determined to be inconsistent with 2021 Flow Monitoring data and the hydraulic model. No obvious source was identified for the issue, so these subbasins were not used in calibration of DWF.

Table 8-2 presents a comparison between modeled and observed DWF. For the overall system, modeled flows are within 4% of the observed flows on an average dry day. A calibration within 5% is considered a good representation of actual conditions in the collection system.

It is noted that calibration within 5% in areas of lower flow is more difficult because (1) flow meters are typically operating at the lower end of their operating range, which can result in measurement noise and (2) at lower flows, a small deviation from observed flows results in a larger percent difference. For this reason, a few of the smaller subbasins have differences in modeled versus observed average flows greater than 10% (i.e. Subbasins E and ALMA 1); these represent a very small portion of total system flow. All downstream subbasins capturing larger flows – the most critical parts of the system – have modeled flows that match observed flows within 10%; these are values that are most indicative of the calibration quality.

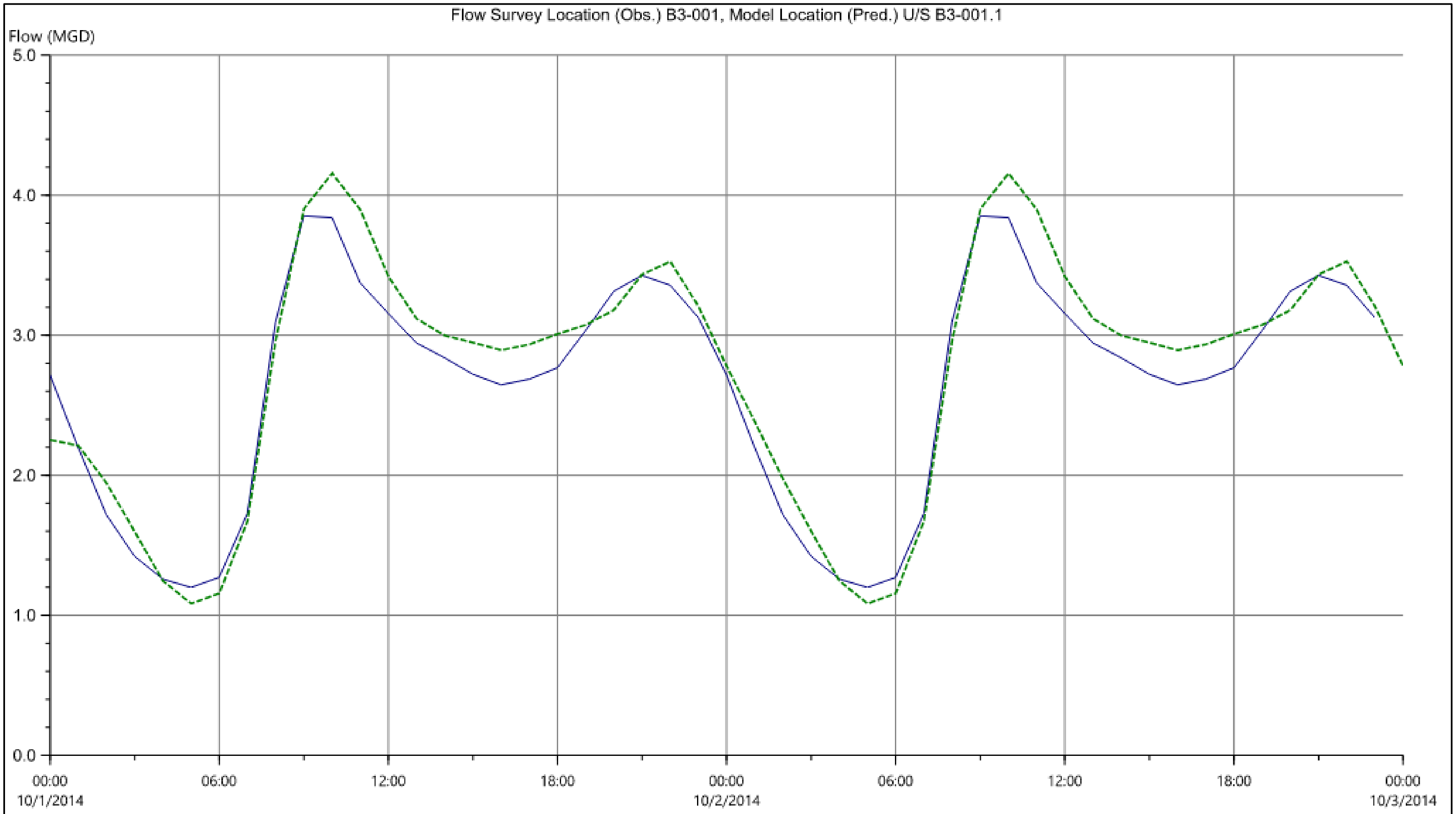
Figure 8-4 presents a sample of the final modeled versus observed flows for dry weather. All dry weather calibration charts are provided in **Appendix F**.

Table 8-2: Calibrated Model vs. Observed Dry Weather Flow

Flow Meter	Subbasin	Model Avg Flow (MGD)	Observed Avg Flow (MGD)
K4-032	A	0.342	0.332
I6-035	B	0.252	0.262
F6-027	C	0.312	0.294
H5-013	D	0.389	0.524
F6-013	E	0.030	0.024
G5-040	F	0.506	0.377
E4-003	G	1.079	1.096
G3-086	H	1.101	1.137
F3-014	I	0.384	0.380
D2-020	J	0.377	0.353
B3-001	K	2.759	2.654
B4-007	L	1.474	1.352
B4-006	M	2.764	3.186
F1-128	ALMA1	0.018	0.013
F1-110	ALMA2	0.393	0.270
F6-016	ALMA3	0.062	0.055
E1-029	ALMA4	0.133	0.168
E1-037	ALMA	0.606	0.512
	Total System Flow[1]	**6.129**	**6.352**

Notes:
1. Total system flow is represented by flow meters B3-001, B4-006, and E1-037.

Figure 8-4: Sample Dry Weather Model Calibration Chart



	Flow		
	Min (MGD)	Max (MGD)	Volume (US Mgal)
Observed	1.198	3.853	5.186
Base (2014) Calibration>DWF	1.084	4.158	5.528

8.3 Wet Weather Flow (WWF)

The WWF calibration consists of the RDI/I component. Data from the 2021 Flow Monitoring program was utilized to calibrate WWF including RDI/I on a subbasin-by-subbasin basis for the rainfall event that took place on March 10, 2021. The process for developing RDI/I and the hydraulic model WWF calibration results are discussed below.

8.3.1 2021 Dry Weather Flow Development

While the dry weather flows from the 2021 flow monitoring program are not representative of typical wastewater flows due to the COVID-19 pandemic, dry weather flows are necessary to conduct the RDI/I calibration. To solve this, a set of representative dry weather flows was created on a subbasin-by-subbasin basis to quickly model the dry weather flows captured during the 2021 Flow Monitoring program to serve as the basis for applying RDI/I for the calibration of WWF.

An average dry weather flow was calculated for the fully dry week prior to the March rainfall (February 22 through February 26, 2021). The average flow during this dry week for each subbasin was applied evenly across the entire tributary area of the subbasin, and a unique diurnal pattern was developed for each subbasin. Each of the subbasin flows were calibrated to the representative dry weather flows prior to proceeding with WWF calibration.

8.3.2 Rainfall-Dependent Inflow and Infiltration (RDI/I)

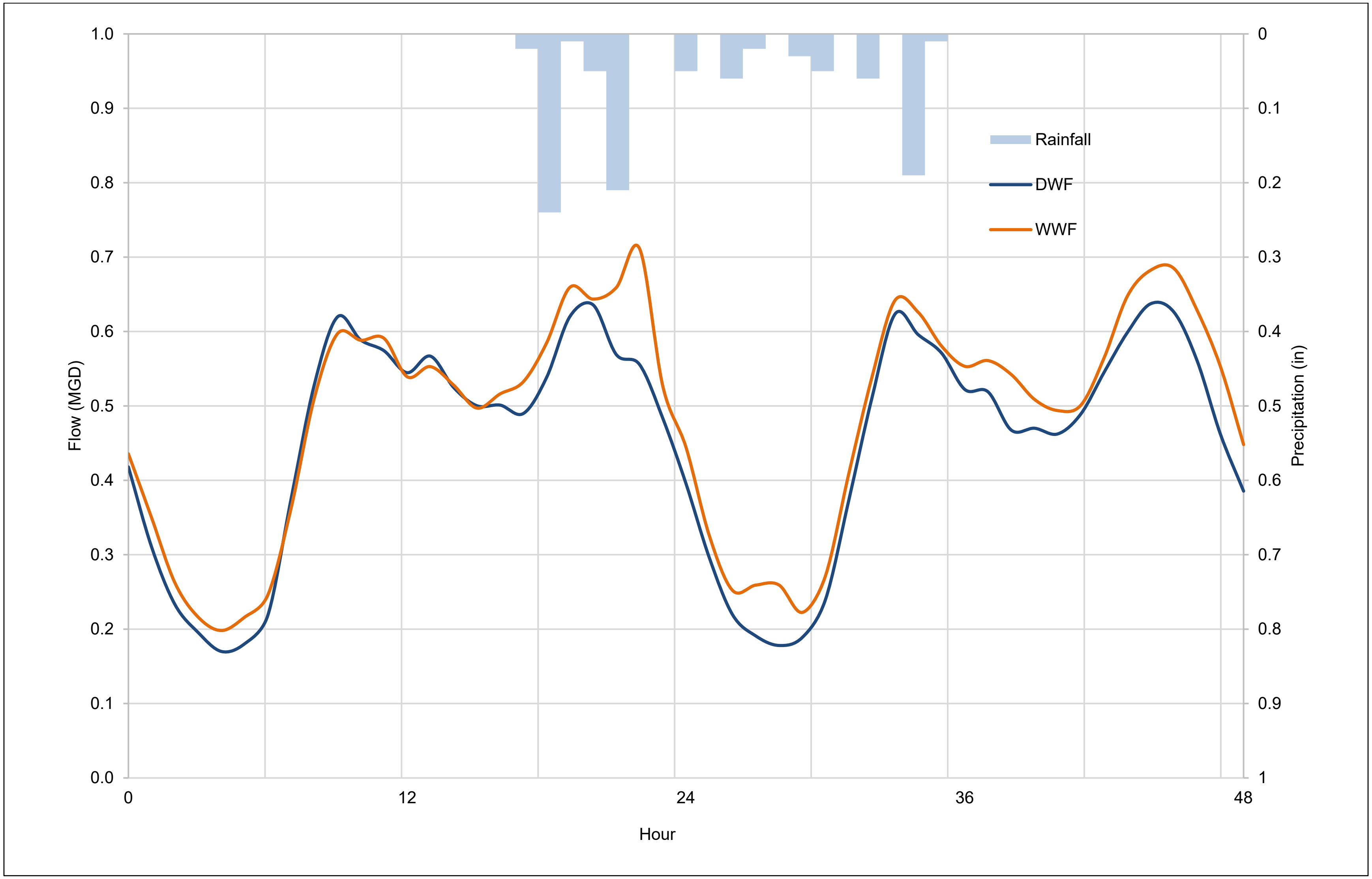
RDI/I is represented by an R-factor: a number used to represent the percentage of rainfall by volume that enters the wastewater collection system, or the ratio of RDI/I to total volume of rainfall. R-factors are a moderately simplified way of estimating a collection system's WWF response and this method is industry standard. There is no separate estimation of antecedent moisture conditions in the soil. It is typically best to use at least a 2-year storm to estimate R-factors, as it will more likely represent conditions similar to a large design storm. Due to the ongoing drought conditions in California, the largest storm captured during the 2021 Flow Monitoring period was a 1-year storm on March 10, 2021 (see **Table 7-2**) which is used in this wet weather calibration.

The volume of RDI/I was initially calculated by subtracting the volume of 2021 DWF in the system from the volume of total WWF. **Figure 8-5** presents a sample of the WWF for the March 10, 2021 storm at flow meter G5-040 (subbasin F) overlaid on the DWF from the week prior to the rainfall.

The total volume of rainfall that fell on each subbasin was calculated by multiplying the depth of precipitation measured by the rain gauges by the total tributary area in each respective subbasin. Areas that do not contribute wastewater flow (i.e. open space) are excluded as they have no route of contribution to RDI/I. To calculate initial rainfall volumes, each parcel was assigned to the nearest of six rain gauge as displayed in **Figure 8-6**.

Initial R-factor estimates were then calculated for each subbasin by dividing the volume of RDI/I at the respective flow meter by the total volume of rainfall for the entire subbasin, resulting in an estimated percentage of rainfall that enters the collection system during rainfall events. These initial R-factors served as the basis for calibrating the collection system's response to rainfall.

Figure 8-5: Flow Meter G5-040 WWF



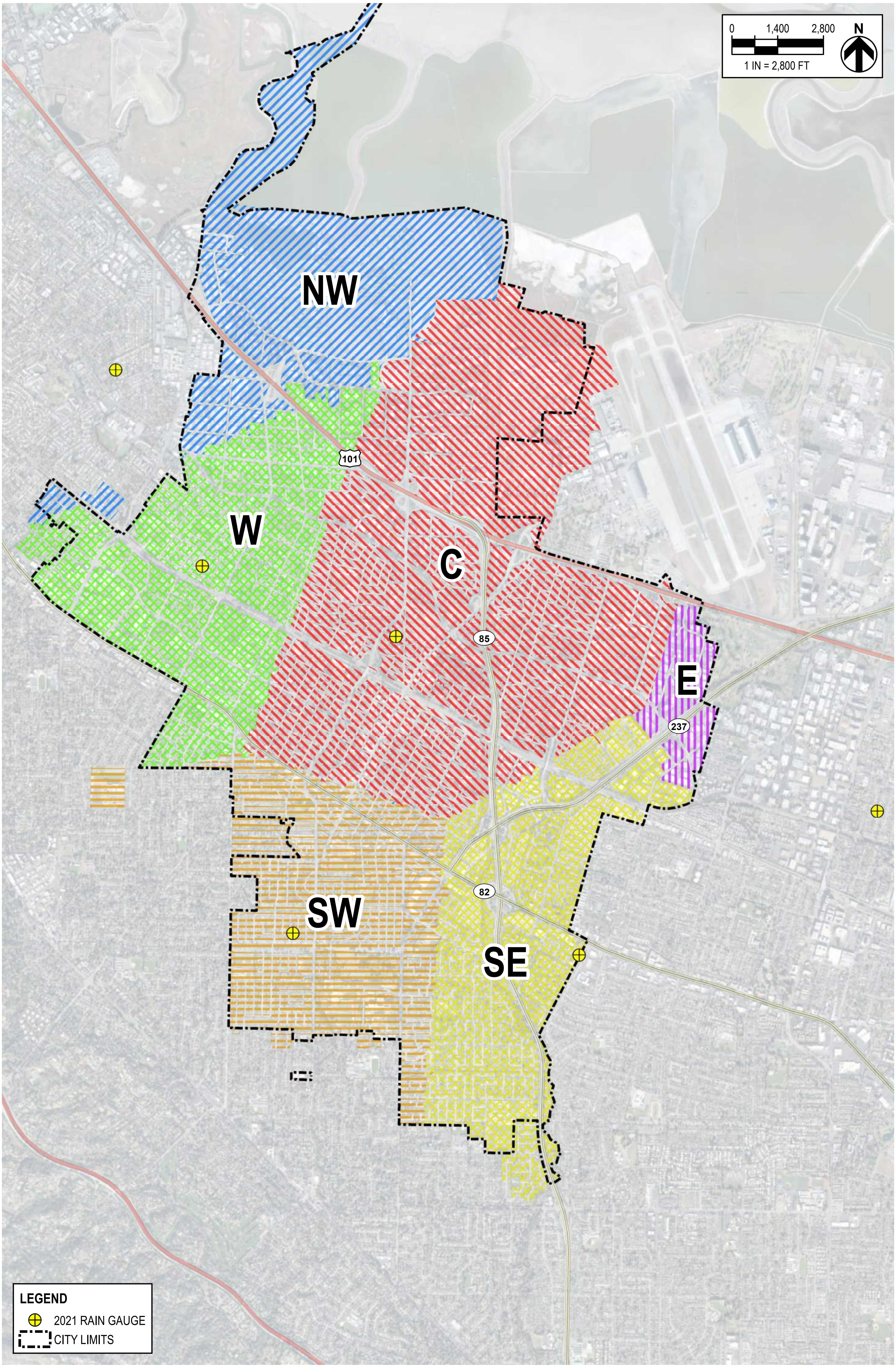


FIGURE 8-6
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
RAIN GAUGE ASSIGNMENTS

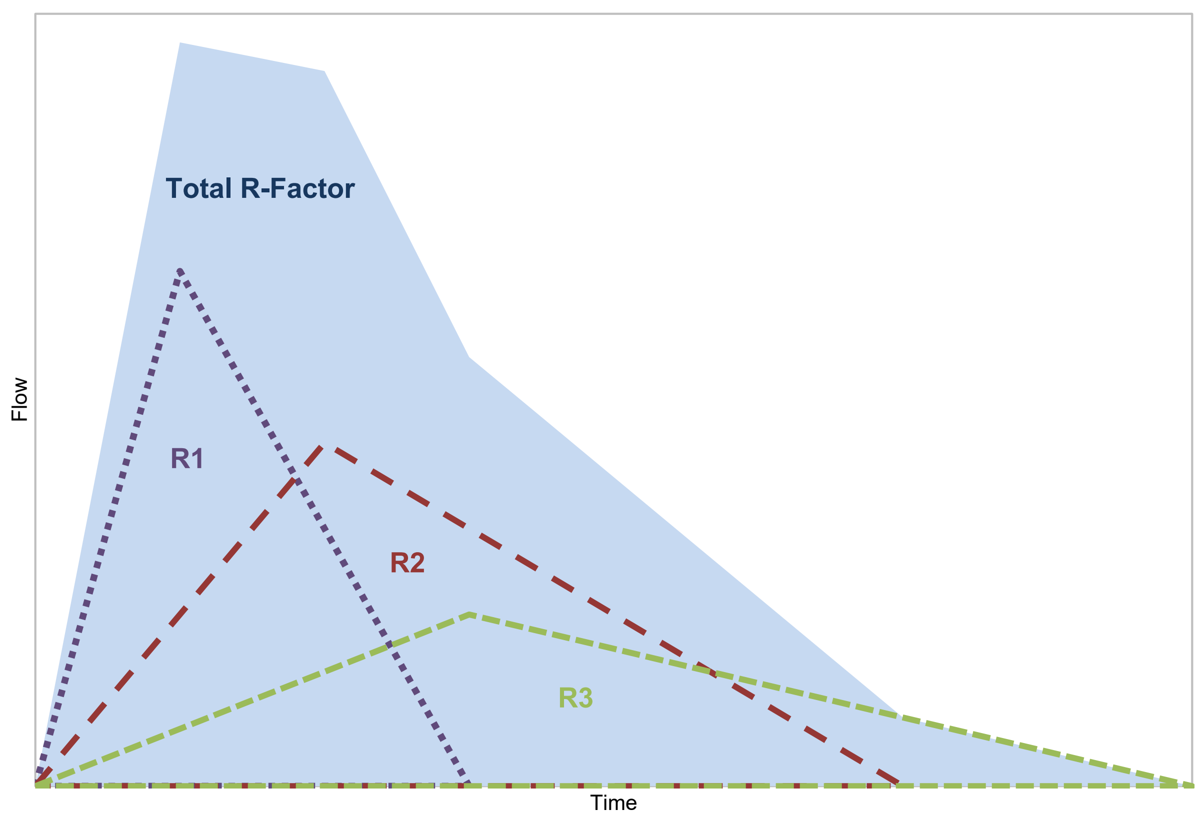
HydroScience

For modeling purposes, R-factors are further broken down using the Tri-Triangular Method. The Tri-Triangular Method, which is the industry standard for representing wet weather flows, involves summing three separate hydrographs to derive a single unit hydrograph. The total R-factor is divided into three separate values, represented as triangles (see **Figure 8-7**), based on the speed at which rainfall enters the sewer system, as described below:

- **Rapid (R1)** – Stormwater inflow that enters the sewer system most rapidly, typically through entry points such as holes in manhole covers. The peak of this triangle occurs one to three hours after the start of the rainfall.
- **Intermediate (R2)** – A combination of rapid stormwater inflow and slow rainfall-dependent infiltration.
- **Slow (R3)** – Long term delayed rainfall-dependent infiltration. The peak of this triangle occurs the longest time after the start of the rainfall, and the effects of this infiltration can continue long after the end of the rainfall.

Total R-factors are simply the sum of the individual R-factors (R1 + R2 + R3).

Figure 8-7: Tri-Triangular R-Factor Method



8.3.3 Wet Weather Calibration Results

The calculated initial R-factors were split into R1, R2, and R3 values at an arbitrary percentage of 50%, 40%, and 10% of the calculated R-factor, respectively, for each subbasin. R-factors were iteratively adjusted for each sub-basin by comparing modeled flows to observed flows at each flow meter during the day of, and the two days following, the March 10, 2021 storm. During this part of the calibration process, emphasis was placed on capturing the peak flows as accurately as possible to avoid underestimating capacity deficiencies in the analysis scenarios.

After completing all R1, R2, and R3 adjustments, the wet weather modeled flows were determined to align as close to the observed flows as reasonably possible. Due to the small size of the rainfall event, many of the flow meters displayed a small response to the rainfall while some displayed no detectable response. A lack of detectable response could be attributed to the small size of the storm as well as some attenuation of the flows in the downstream portions of the system, but is not interpreted to mean that the subbasin is impermeable to RDI/I. In areas where there was no observed response, an average of the surrounding subbasins was calculated and assigned to the parcels in those subbasins. This same methodology was applied to the portions of the system that are not captured by a flow meter – namely PA and LA.

A sample wet weather calibration chart is displayed in **Figure 8-8** and wet weather calibration charts for all flow meters are provided in **Appendix G**.

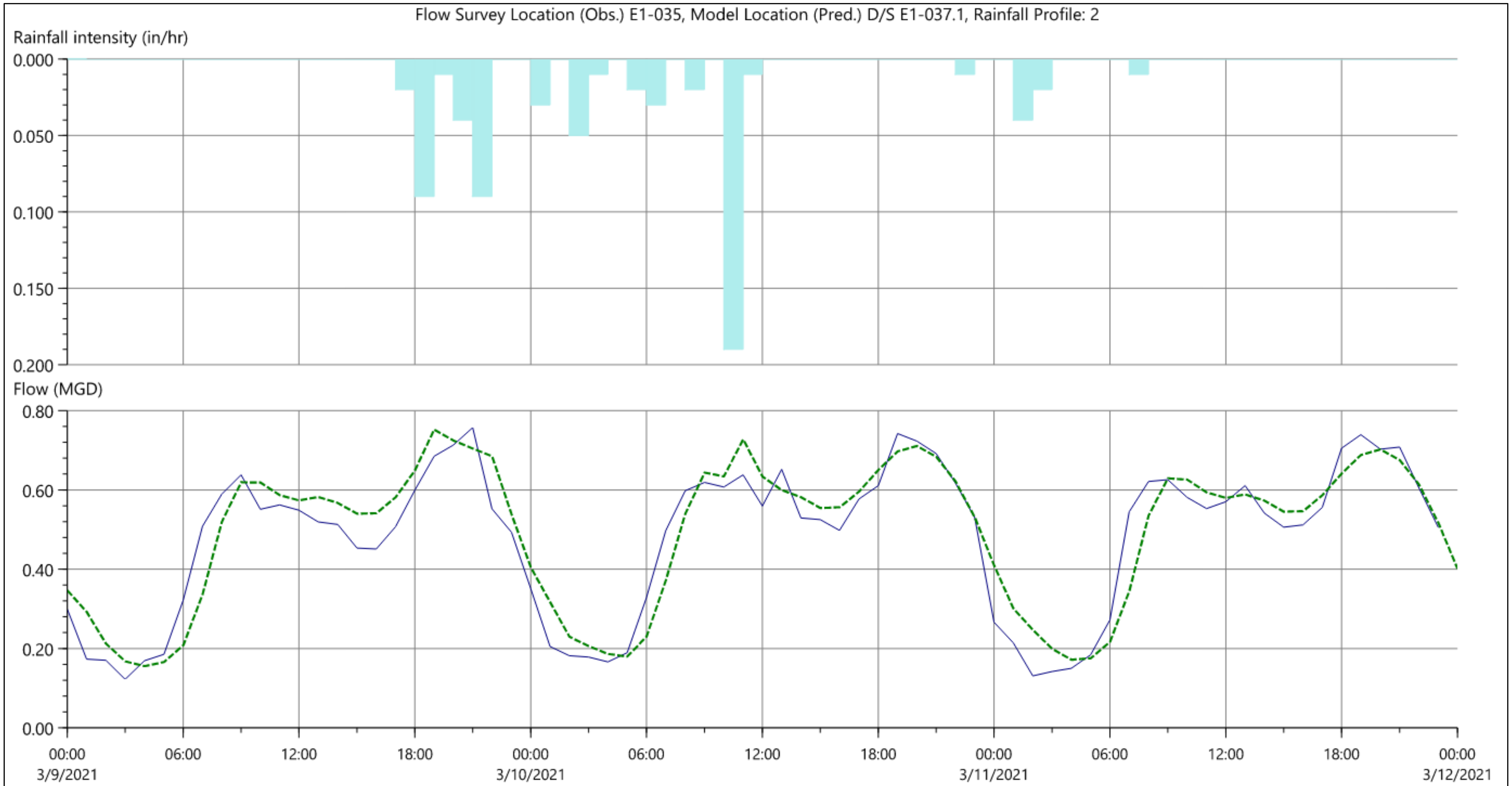
For wet weather calibration of the system, average modeled flows for the day of the March 10, 2021 storm and two days following the storm are 3% higher than observed flows and modeled PWWF is within 5% of observed peak flows. A calibration within 5% is considered a good representation of actual conditions in the collection system.

Though pipeline improvements can decrease system RDI/I rates, generally, the City's RDI/I rates are already very low and thus, it was assumed that the R-factors in the City's collection system have not decreased since the development of the 2010 model. The calibrated R-factors were scaled up to maintain the values used in the 2010 model and SMP.

The final calibrated and adjusted R-factors for each subbasin are presented in **Table G-1** included in **Appendix G**.

In summary, total adjusted R-factors for each subbasin range from 1.3% up to 3.7%, which represents the percentage of rainfall entering the collection system. A low R-factor is typically less than 3% and 10% is considered high. The weighted average for the entire system is 1.4%; this is considered relatively low RDI/I with limited rainfall entering the collection system.

Figure 8-8: Sample Wet Weather Calibration Chart



	Rainfall			Flow		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (MGD)	Max (MGD)	Volume (US Mgal)
Rain	0.690	0.190	0.009			
Observed				0.122	0.757	1.418
...WWF 3/10/2021 Storm				0.156	0.753	1.479

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SECTION 9 – CAPACITY ANALYSIS

With the hydraulic model calibrated to DWF and WWF, the sewer collection system was evaluated under design storm for existing (2021) and 2030 conditions. This section describes the model analysis scenario development, capacity analysis, and resulting identified deficiencies.

9.1 Analysis Scenario Development

The calibrated values for GWI, BSF, and RDI/I were applied to the following scenarios developed to analyze the City's collection system:

- **Existing:** This scenario utilizes the 2014 DWF Calibration scenario GWI, UF factors, and diurnal patterns combined with the 2021 land uses. It also includes the calibrated R-factors and infrastructure updates from the 2021 WWF Calibration scenario. The major updates made between the 2014 DWF Calibration and Existing analysis scenarios are:
  - All adjustments made during the dry weather calibration, described in **Section 8.2.4**, were returned to design conditions assuming any flow obstructions have been cleaned and all flow splits are operating as designed.
  - A new set of 27 existing large dischargers was identified using the water meter billing data from FY 2018/19 through FY 2019/20. An average was taken over two years to account for potential nuances in a single year and to identify trends over the recent years. This is more indicative of the customers that will likely continue to be large dischargers in future years. The large dischargers identified for the Existing scenario are presented on **Figure 9-1**.
  - Moffett Field flows metered in 2014 and 2015 were averaged and assumed to represent flows from NASA for the Existing scenario – this was the most reliable data available due to meter read errors in other years. Additional flows from the Google Bayview campus were estimated based on the area and land use type.
- **2030:** This scenario maintains the same GWI and diurnal patterns as the Existing scenario. The updates made between the Existing and 2030 scenarios were as follows:
  - Land uses were updated based on the 2030 land use assumptions presented in **Section 5.2**.
  - Large dischargers identified for the Existing scenario were maintained except where the parcels were identified for redevelopment as part of the 2030 land use assumptions. For the 2030 scenario, 24 of 27 large dischargers remain.
  - UF factors were updated for future conditions based on the water use projections presented in the 2022 Water Master Plan, which are also coordinated with the City's 2020 Urban Water Management Plan (UWMP). Specifically, residential land use factors are expected to increase due to increases in population and housing density in the future. **Table 9-1** presents the existing and future land use factors to highlight those that were updated for the future scenario.

- **Existing_w/CIP:** This scenario represents flow conditions from the Existing scenario with system infrastructure updated to reflect currently planned upcoming CIP projects for which plans were provided by the City and locations are identified in **Figure 7-1**.
- **2030_w/CIP:** This scenario represents flow conditions from the 2030 scenario with system infrastructure updated to reflect currently planned upcoming CIP projects for which plans were provided by the City and locations are identified in **Figure 7-1**.

Table 9-1: Future UF Factor Updates

Land Use Code	Existing UF Factor	Future UF Factor	Unit
MU	3,800	3,800	gpd/acre
COM	1,400	1,400	gpd/acre
HDR	1,700	1,830	gpd/acre
IND	550	550	gpd/acre
LDR	120	130	gpd/DU[1]
LMDR	180	185	gpd/DU[1]
MDR	1,100	1,165	gpd/acre
MBL	800	800	gpd/acre
OFF	1,000	1,000	gpd/acre
PF	700	700	gpd/acre
MOT	2,800	2,800	gpd/acre

Notes:
1. Assume one dwelling unit per parcel.

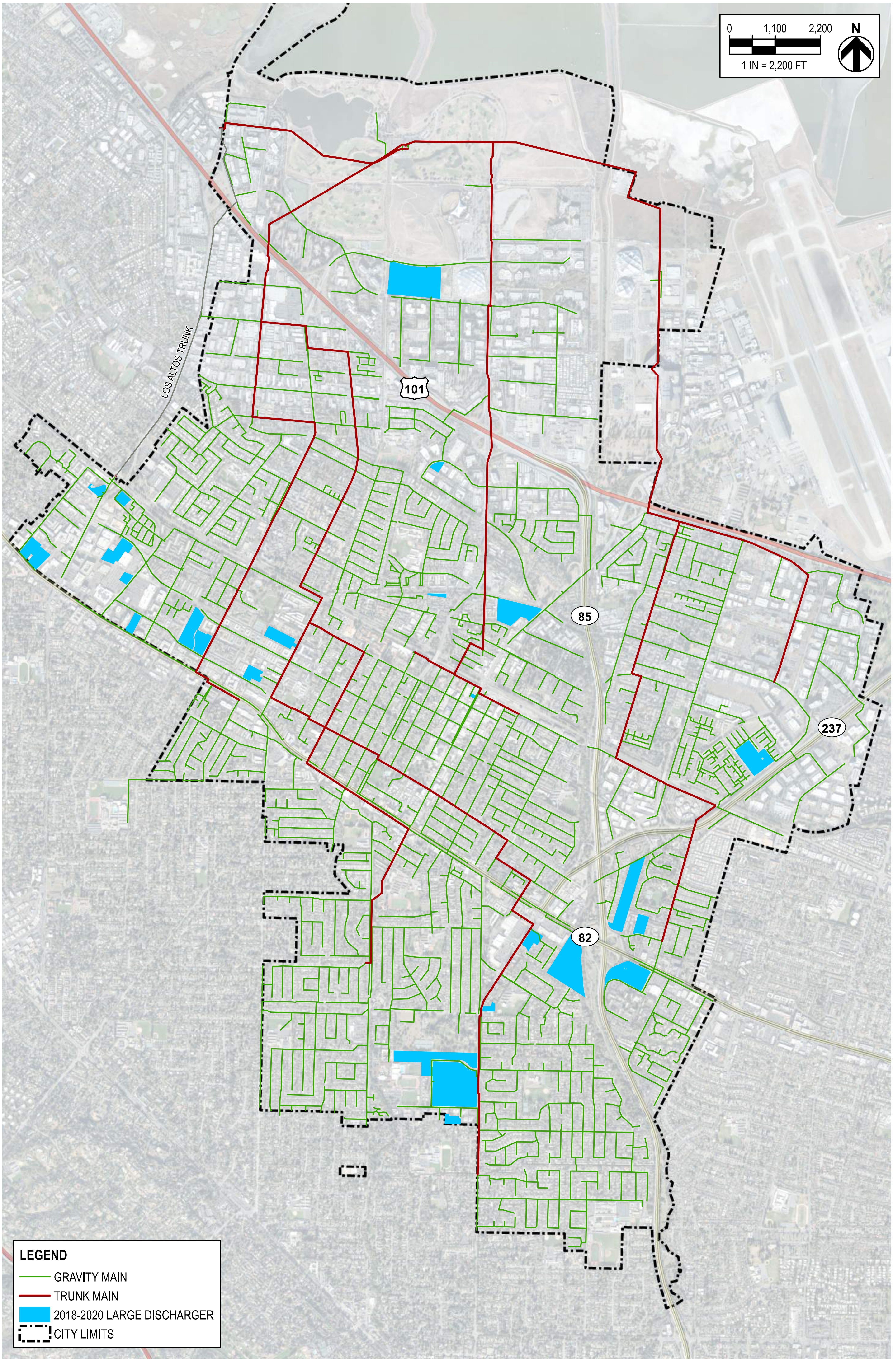


FIGURE 9-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
LARGE DISCHARGERS

HydroScience

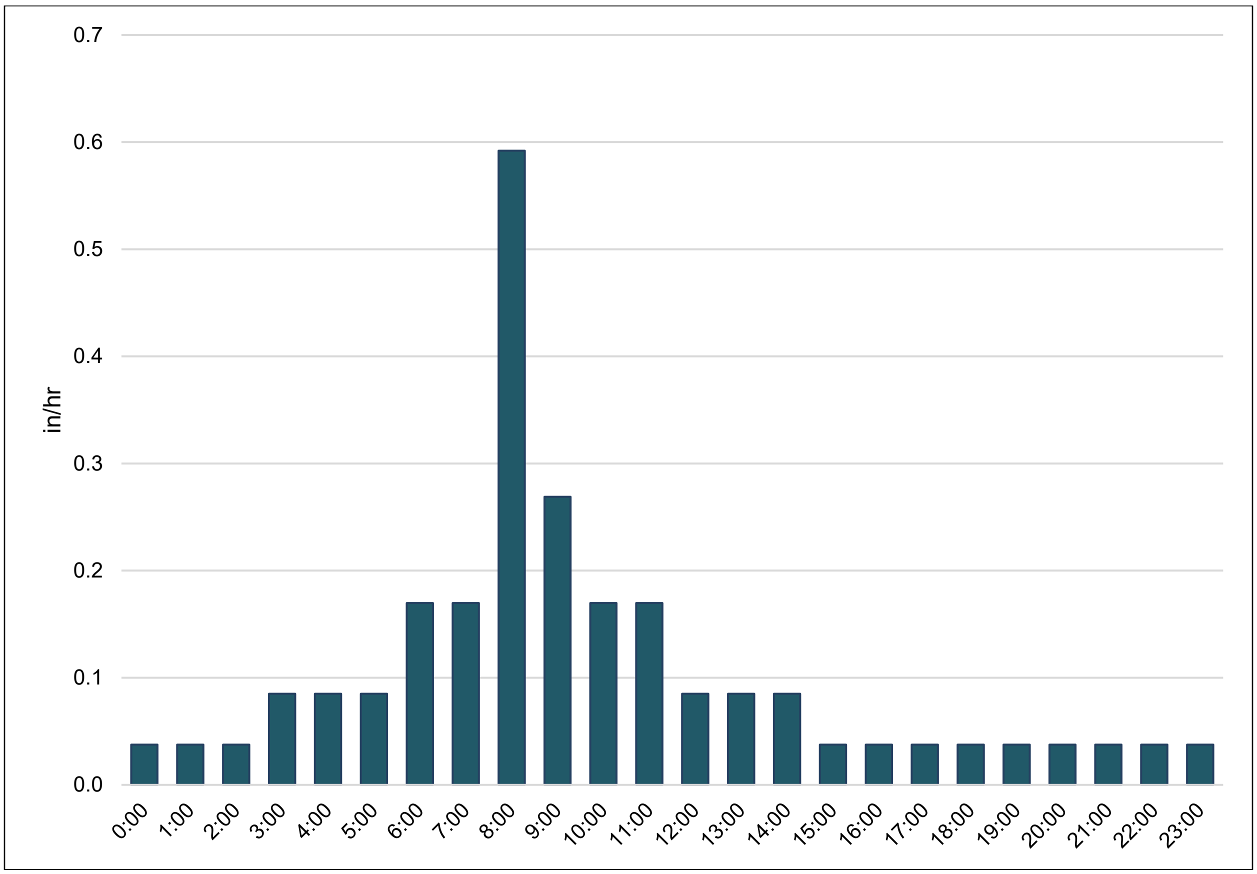
9.2 Design Storm

To estimate design WWF, a design storm is applied to the model and WWF is generated based on the calibrated R-factors applied to each subbasin.

A range of design storms was considered for the design condition of this Master Plan. Previous modeling used a 10-year, 4-hour constant distribution storm. Often a 10-year, 24-hour storm is used with a peak hour coinciding with the peak of the diurnal curve. The choice of design storm is a balance between the risk of capacity deficiency and the cost of building infrastructure that is designed to handle high flows that are only experienced occasionally.

For this analysis, the typical 10-year 24-hour storm, presented in **Figure 9-2**, with a peak hour coinciding with the peak of the typical diurnal pattern was deemed appropriate.

Figure 9-2: 10-Year 24-Hour Design Storm



Source: NOAA Atlas 14 Volume 6 Version 2 precipitation frequency estimates in inches for Mountain View, California (https://hdsc.nws.noaa.gov/hdsc/pfds).

9.3 Deficiency Criteria

The main criteria used to evaluate the capacity of the existing modeled pipes under design flow conditions is the ratio of wastewater depth (d) to pipe diameter (D), or d/D. In InfoWorks, if d/D is greater than one, this means the pipe is flowing full under pressure and the hydraulic grade line (HGL) is higher than the crown of the pipe. InfoWorks reports a “surcharge state” for each pipe; **Table 9-2** summarizes the definition of each surcharge state.

Table 9-2: InfoWorks Surcharge States

Surcharge State	Definition
< 1 [1]	Depth of flow is less than the diameter of the pipe.
= 1 [2]	Pipe is surcharged due to backwater from a downstream deficiency.
= 2 [2]	Pipe is hydraulically under capacity and needs to be upsized.

Notes:
1. Surcharge state = d/D
2. A surcharge state of 1 or 2 is not a representation of d/D, but rather an indicator of the defined condition, as shown.

In the model, the HGL reaching the ground surface indicates a potential SSO. Even for a pipe flowing under capacity or surcharged, if the freeboard – the distance between the ground surface and the HGL – is 5 ft or greater, the risk of SSO is minimal. Each pipe with a surcharge state of “2” was reviewed and was identified as deficient if the freeboard was also less than 5 ft.

9.4 Design Criteria

Once deficiencies were identified, HydroScience developed recommendations to address each deficiency. The design criteria listed here are the parameters used to size improvements to address identified deficiencies. **Table 9-3** summarizes the City’s Standard Design Criteria for Sanitary Sewers, found in Appendix C of the City’s 2018 SSMP. All recommended improvements were designed to meet these standards using the calibrated hydraulic model.

Table 9-3: City Sewer Design Criteria Summary

Parameter	Criteria
Material	VCP, extra strength (ASTM C700) with elastomeric joints (ASTM C425); or PVC pipe, SDR 26 (ASTM D3034 or F679) C900 PVC may be used in place of SDR 26 PVC.
Size	Minimum diameter = 8 in.
Depth	Minimum depth from finished grade to sewer invert = 5 ft. Maximum depth from finished grade to sewer invert = 22 ft.
Slope	Minimum slope = 0.004 ft/ft (0.4%). Design slope shall provide a velocity of 2 ft/s when the sewer is flowing half full (d/D = 0.5) where d/D refers to the depth-to-diameter ratio. Wherever possible, the design slope will provide a velocity of 2 fps during peak daily dry weather flow. The maximum slope will be limited to a velocity of 10 fps during any flow condition.
Capacity	Maximum depth of flow will be: Sewers ≤ 12-inches in diameter, d/D = 0.5 Sewers > 12-inches in diameter, d/D = 0.75

9.5 Model Outfall

The San Antonio Metering Station was modeled as the collection system outfall. To perform a boundary condition sensitivity analysis, each modeled scenario was run under the following three conditions:

1. Free outfall;
2. Pipeline half full at San Antonio Metering Station; and
3. Pipeline full at San Antonio Metering Station.

The free outfall and half full pipeline boundary condition scenarios provide approximately the same results; the full pipeline boundary condition results in some backwatering effects into the Interceptor Trunk pipeline. These backwater effects do not extend upstream into the collection system further than the Interceptor Trunk, and backwater effects do not indicate additional capacity deficiencies.

The free outfall boundary condition was used for the identification of hydraulic deficiencies because this allows for identification of hydraulic deficiencies due solely to pipe size and slope. For increased accuracy of any potential backwater effects at the San Antonio Metering Station, it is recommended that the City obtain information on the HGL at the San Antonio Metering Station and update the boundary conditions of the hydraulic model.

9.6 Hydraulic Capacity Deficiencies

The capacity analysis was completed for all scenarios in order to prioritize the recommendations; capacity deficiencies identified in the Existing scenarios are prioritized higher than those triggered by 2030 flows.

All recommendations to address capacity deficiencies were sized to accommodate the 2030_w/CIP scenario.

9.6.1 Existing Scenario

Capacity deficiencies were identified for the Existing_w/CIP scenario according to the deficiency criteria listed in **Section 9.3**. **Figure 9-3** identifies all segments in the system that register a surcharge state of “1” or “2,” and each surcharge state of “2” is addressed below.

A. **Martens Ave and Alexander Ct:** In this area, the configuration of the pipelines in the previous hydraulic model did not agree with the configuration presented in the City’s GIS geodatabase. City staff confirmed that the configuration in the GIS geodatabase was the most current. The resulting configuration is two 8-in segments between 12-in pipes. The 8-in pipes are under capacity and the backwater effects upstream result in less than 4 ft of freeboard at manholes K5-009 and K5-011 in Martens Ave. Upsizing the 8-in segments would resolve this deficiency.
B. **Miramonte Ave between Sladky Ave and Hans Ave:** This area contains 1,400 ft of 8-in pipe that is under capacity. The downstream end of the surcharged segments is shallow and results in less than 5 ft of freeboard. These pipes should be upsized to address this deficiency.

C. **Castro St between Harpster Dr and Sonia Way:** There is a single 8-in pipe between 10-in pipelines in this area that registers a surcharge state of "2." There is over 8 ft of freeboard, so this does not meet the criteria for a hydraulic deficiency, though it is not standard practice to have a larger diameter pipe flowing into a smaller diameter pipe.

D. **Mountain View Ave between Park Dr and El Camino Real:** There are two 8-in pipelines between 10-in pipelines in this area, one of which registers a surcharge state of "2" due to its shallower slope. There is over 6 ft of freeboard, so this does not meet the criteria for a hydraulic deficiency, though it is not standard practice to have a larger diameter pipe flowing into a smaller diameter pipe.

E. **Sondgroth Way/Showers Dr:** This area, beginning on Sondgroth Way flowing west, turning north on San Antonio Cir, and crossing San Antonio Rd, registers many segments as under capacity with a surcharge state of "2." Though the pipes in this area are very deep, the backwater effects result in less than 5 ft of freeboard at manholes F2-086 and F2-013 upstream within the property of the residential complex situated between Showers Dr and Ortega Ave. The segments under capacity should be upsized to address this deficiency.

F. **Independence Ave between Old Middlefield Way and Leghorn St:** This area contains two 12-in segments that are surcharged just above the crown of the pipe. Every manhole along this stretch maintains greater than 8 ft of freeboard and thus, this area does not meet the criteria for a deficiency and no action is necessary.

G. **Space Park Way between N Shoreline Blvd and Armand Ave:** This area contains two 8-in segments that are surcharged just above the crown of the pipe. Every manhole along this stretch maintains greater than 5 ft of freeboard and thus, this area does not meet the criteria for a deficiency and no action is necessary.

   It is recommended, however, to verify the inverts of manhole D4-021 in the intersection of N Shoreline Blvd and Space Park Way; the 8-in surcharged pipeline in Space Park Way connected to this manhole appears that it may be misrepresented as the 8-in pipeline connects to the 18-in pipeline in N Shoreline Blvd with matching inverts which, as mentioned previously, is not standard practice and is typically not how pipelines are designed. It is noted that this manhole is part of an upcoming City CIP project to reroute Plymouth St on the west side of N Shoreline Blvd to meet Space Park Way. During this project, the inverts of manhole D4-021 should be verified and updated in the hydraulic model.

There is a single segment along the Los Altos Trunk that is surcharged just above the crown of the pipe. The scope of this Master Plan does not include a capacity analysis of the Los Altos Trunk and thus, this is not addressed. Additionally, the inverts of this pipeline are interpolated between two surveyed manholes at the most upstream and most downstream portions of the trunk and thus are likely not a fully accurate representation of actual pipe slopes.

All other pipelines registering a surcharge state of "1," as displayed in **Figure 9-3**, are locations where a smaller diameter pipe flows into a larger diameter pipe with matching inverts at the connecting manhole. This configuration is not standard practice and pipes of differing diameter are not typically designed with matching inverts, but rather with matching crowns. It is likely that these connections are misrepresented in the model. It is recommended that the City verify the incoming and outgoing inverts at each of these locations and update the model to improve its accuracy and reliability.

Hydraulic model output reports for the analyzed scenario are included in **Appendix H**.

9.6.2 2030 Scenario

There are four additional locations that registered a surcharge state of “2” in the 2030_w/CIP scenario that did not in the Existing_w/CIP scenario. Each of these locations are identified in **Figure 9-4** and discussed below:

- **H. Parsons Ave:** This area contains a single 8-in pipeline that is surcharged just above the crown of the pipe. The location of the surcharging maintains over 9 ft of freeboard and thus, this does not meet the criteria for a deficiency and no action is required. Additionally, this is the location that wastewater from Moffett Field discharges into the City-owned collection system. The City’s GIS geodatabase has limited information on these few pipelines so in the hydraulic model, the inverts are interpolated using the minimum design criteria slope of 0.4% and the diameters are assumed to be 8-in per the minimum design criteria pipe size. The City should verify the pipe diameters and inverts of these pipelines to increase the accuracy and reliability of the hydraulic model.
- **I. Dierick Dr between Wasatch Dr and Belshaw Dr:** This area contains a single pipe segment that is surcharged just above the crown of the pipe due to a very shallow slope. There is greater than 15 ft of freeboard and thus, this does not meet the criteria for a deficiency and no action is required.
- **J. Grant Rd between Martens Ave and Bentley Sq:** This area contains a single 15-in segment that is surcharged just above the crown of the pipe due to a very shallow slope. There is greater than 8 ft of freeboard and thus, this does not meet the criteria for a deficiency and no action is required.
- **K. Interceptor Trunk:** In the 2030 scenario, 3,835 ft of the 42-in pipeline carrying effluent discharge from SPS is surcharged just above the crown of the pipe beginning at the SPS discharge manhole and continuing through Shoreline Golf Links until the pipeline intersects with Casey Ave. The pipeline in this area is very deep, maintaining over 12 ft of freeboard throughout. Thus, this does not meet the criteria for a deficiency and no action is required. An option to reduce flow in this pipeline is evaluated in **SECTION 10**, which consists of a gravity bypass of the SPS. If implemented, this would significantly reduce flow in this pipeline and address the surcharging.

Hydraulic model output reports for the analyzed scenario are included in **Appendix H**.

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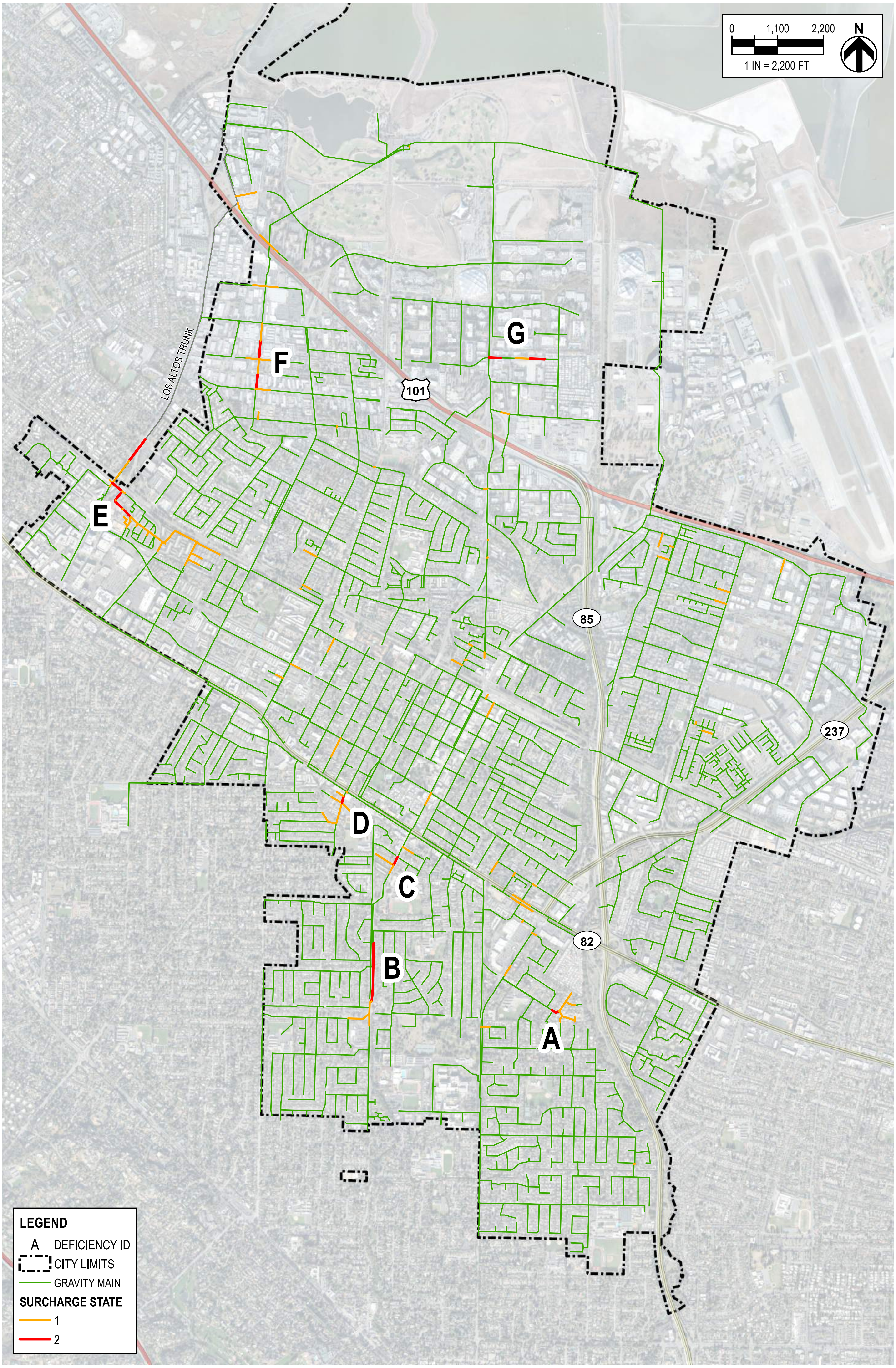

FIGURE 9-3
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
MODEL RESULTS - EXISTING SCENARIO

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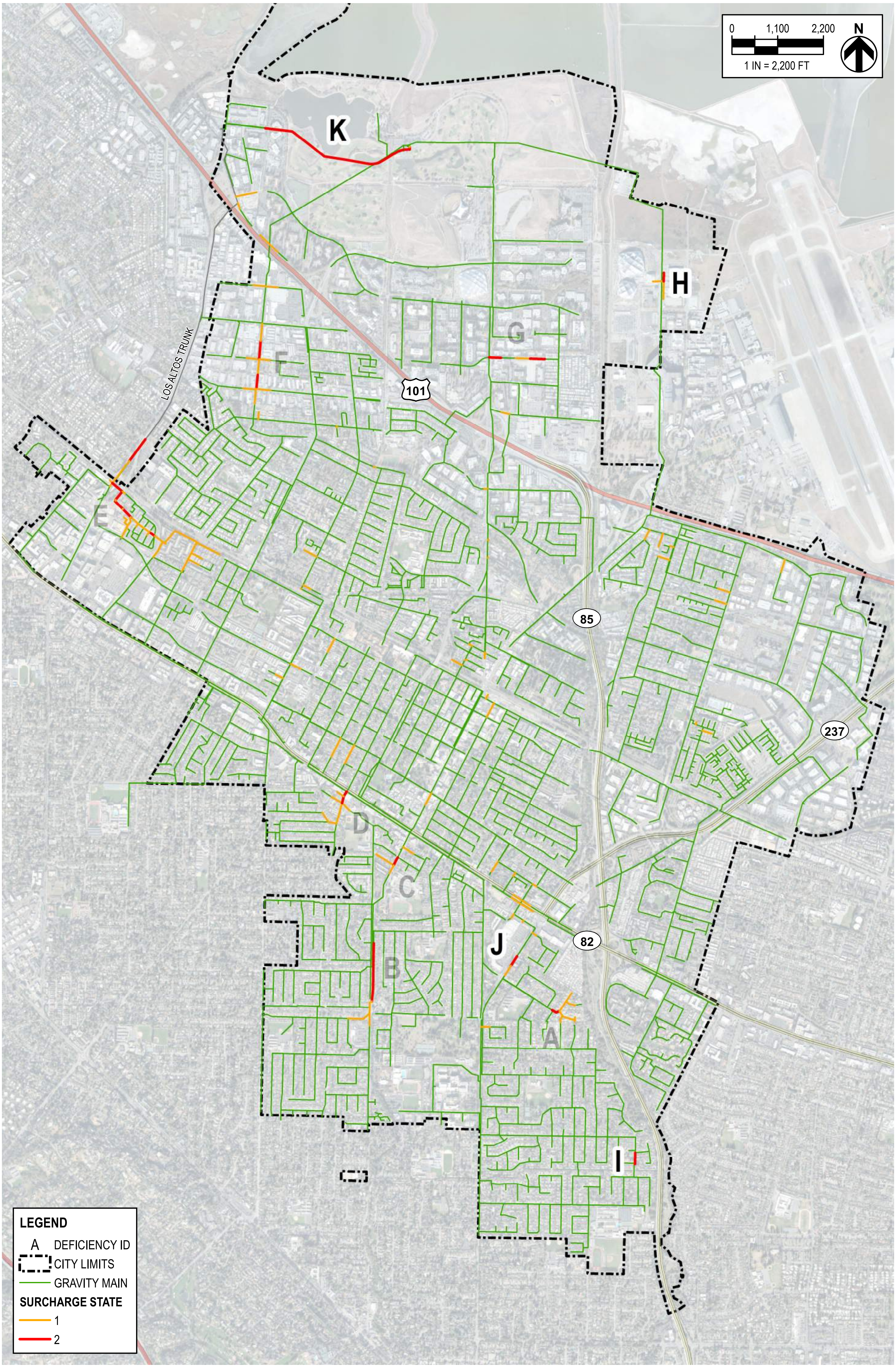

FIGURE 9-4
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
MODEL RESULTS - 2030 SCENARIO

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SECTION 10 – BYPASS INTERCEPTOR ANALYSIS

Since the City's independent WWTP was abandoned in 1972 and the influent pumps were repurposed to operate the current-day SPS, the City has spent significant capital on improvement, operation, and maintenance of the SPS facilities. In the City's 1991 and 2010 SMPs, analyses were completed to develop cost estimates and determine the feasibility of an alternative route for wastewater to bypass SPS by gravity.

Today, the SPS is in serviceable condition and had a set of significant upgrades completed in 2020, but many of the key components are either at or nearing the end of their useful life (see **SECTION 4**). Additionally, in 2017, the City completed an Alternative Alignment Study (2017 Alternative Alignment Study) to expand on the previous analyses completed for bypassing the SPS. For this Master Plan, using the preferred alternative alignment from the 2017 Alternative Alignment Study, HydroScience expanded on the specifics of the recommendations for a new bypass interceptor and developed an updated cost estimate including pipeline construction, new pump station construction, and energy costs. This section presents the hydraulic analysis, results, and accompanying recommendation for the construction of a new bypass interceptor.

10.1 Alignment Options

The 2017 Alternative Alignment Study analyzed multiple options for various alignments through the City's existing collection system. After a full analysis of each of the options, including consideration of hydraulics, soil conditions, seismic considerations, traffic impacts, potential utility conflicts, right-of-way permitting concerns, construction methods, environmental impacts, costs, and more, the alignment ultimately recommended in that study – Alignment 2 – is the alignment that was analyzed in this Master Plan.

The bypass interceptor alignment analyzed herein is presented as three options according to which of the City's trunk lines it is collecting flow from:

- Option 1 – West Trunk Only
- Option 2 – West and Central Trunks
- Option 3 – West, Central, and East Trunks

This analysis is intended to evaluate the bypass interceptor hydraulics in more detail and provide updated costs related to the implementation options. This analysis does not include a review of the other considerations such as soil conditions, seismic considerations, and environmental impacts that were already identified in the 2017 Alternative Alignment Study which is included as **Appendix B**.

Figure 10-1 presents an overview of the three options, each detailed subsequently. Note that in this figure, the alignment options are cumulative, i.e. Option 2 includes Option 1, and Option 3 includes Options 2 and 1. For the purposes of this analysis, manholes on the bypass interceptor were assigned arbitrary identifiers "A-#" beginning with A-1 at the most upstream end of the Option 3 alignment.

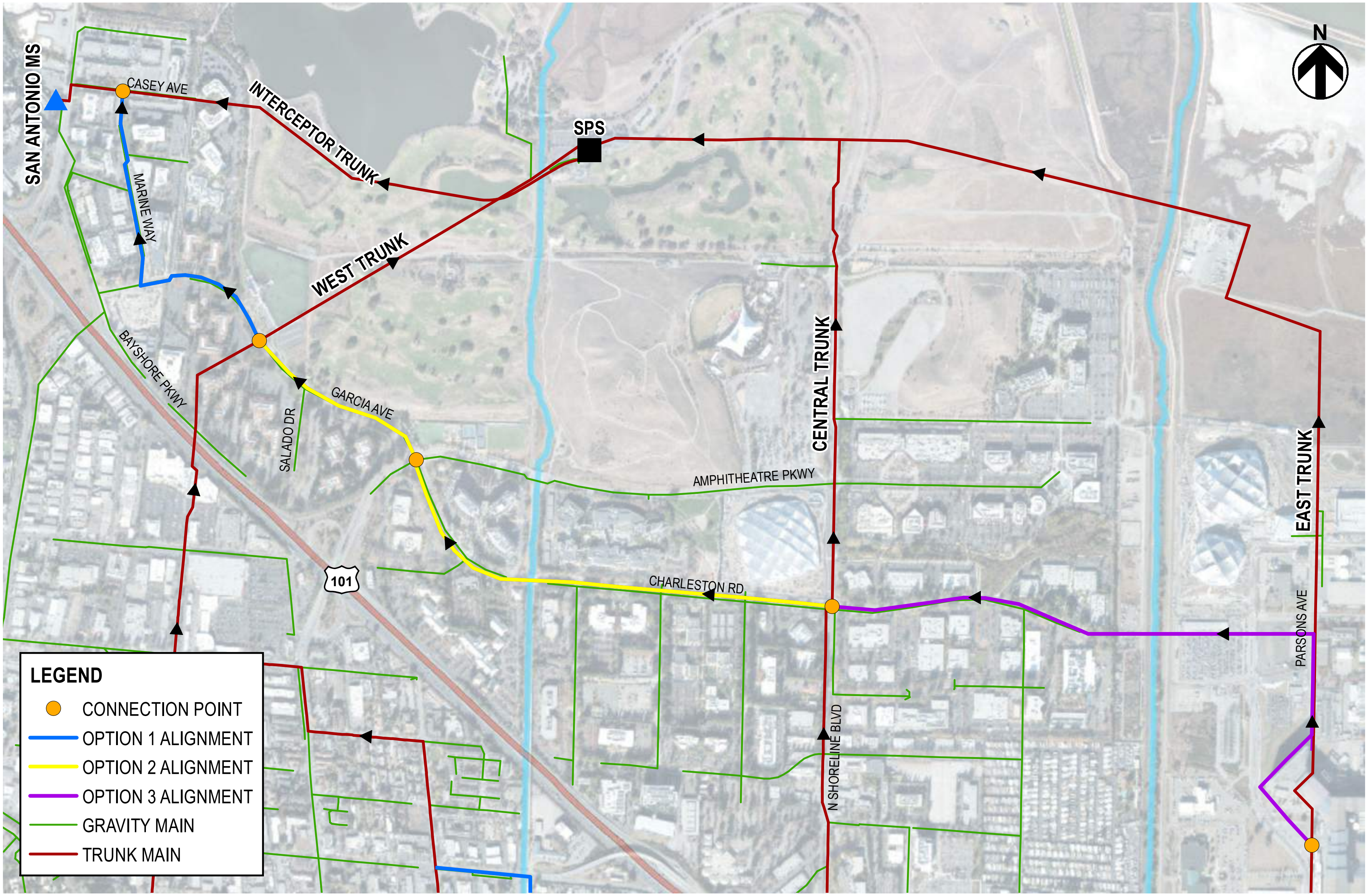

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FIGURE 10-1
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
BYPASS INTERCEPTOR OVERVIEW

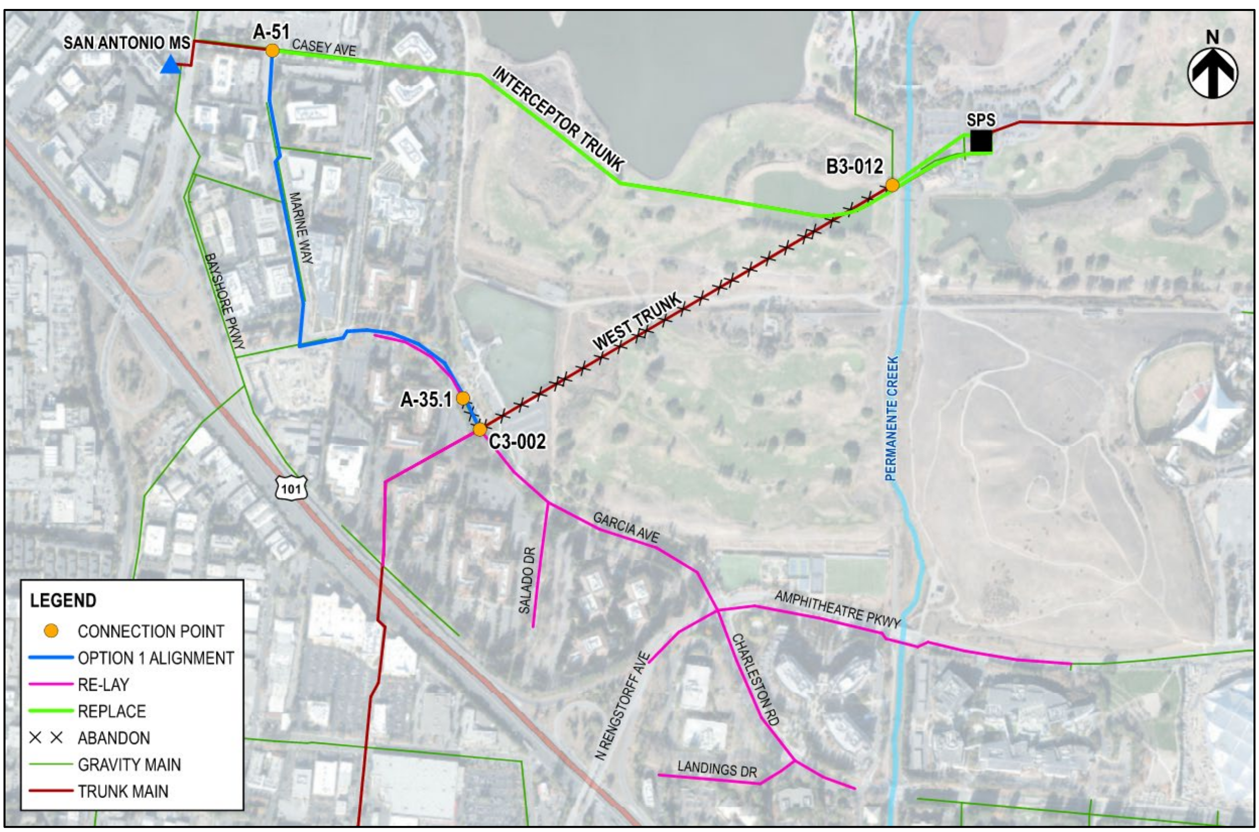
10.1.1 Option 1 – West Trunk Only

Option 1 (see **Figure 10-2**) captures only the flow from the West Trunk and diverts it west to the San Antonio Metering Station by gravity. In this option, the bypass interceptor connects to the existing system at manhole C3-002 and includes 2,960 ft of 42-in pipe. It then connects to the existing 42-in pipeline in Casey Ave at new manhole A-51 where it continues to the metering station.

In order to maintain enough elevation to flow by gravity to the San Antonio Metering Station, the bypass interceptor must intercept the flow from this manhole at a higher invert elevation than the existing collection system operates at. This, in turn, requires that all pipelines connecting to this manhole be re-laid at a higher elevation and/or shallower slope. The pipeline in Garcia Ave to the northwest of manhole C3-002 currently flows into C3-002 from the northwest. In order to avoid this flow entering C3-002 and exiting at a 180 degree angle, a new manhole (A-35.1) should be built on the existing pipeline with approximately 10 ft of 8-in pipe built to connect manhole A-35.1 to the new interceptor pipeline. The remaining 200 ft of the existing 8-in between A-35.1 and C3-002 should then be abandoned.

The portion of the West Trunk north of Hwy 101 should be re-laid at a shallower slope and should additionally be up sized to 36-in pipe. The existing 30-in segment crossing under Hwy 101 is not required to be upsized.

Figure 10-2: Bypass Interceptor Option 1



In the existing system, there are no flow contributions to the West Trunk downstream of manhole C3-002 until manhole B3-012. This allows 2,650 ft of the existing West Trunk to be abandoned and requires that 580 ft of the West Truck be reduced to an 8-in pipe to be installed within the existing West Trunk leading to SPS. Additionally, the portion of the Interceptor Trunk carrying flow from SPS to the new manhole A-51 must be replaced with a smaller pipe due to the existing shallow slopes in this pipeline. Kept as is, the reduced flows would not provide the minimum velocities necessary to flow by gravity, so the reduced diameter pipeline is sized to operate as a force main. This option requires a 30-in pipe. This 30-in pipe can be installed within the existing Interceptor Trunk, but because this force main flows into a larger diameter pipe, the 30-in pipe should be installed at the top of the existing pipe to avoid a small diameter pipe matching inverts with a large diameter pipe and potential backwater effects into the smaller pipe.

Figure 10-2 presents the alignment of the interceptor and identifies all existing pipelines that need to be re-laid, abandoned, and replaced as part of Option 1. It is noted that the segments that need to be re-laid, specifically the portions in Amphitheatre Pkwy and Charleston Rd upstream of the intersection with Amphitheatre Pkwy, have limited soil cover and would likely not be able to maintain minimum slopes with sufficient cover for this option.

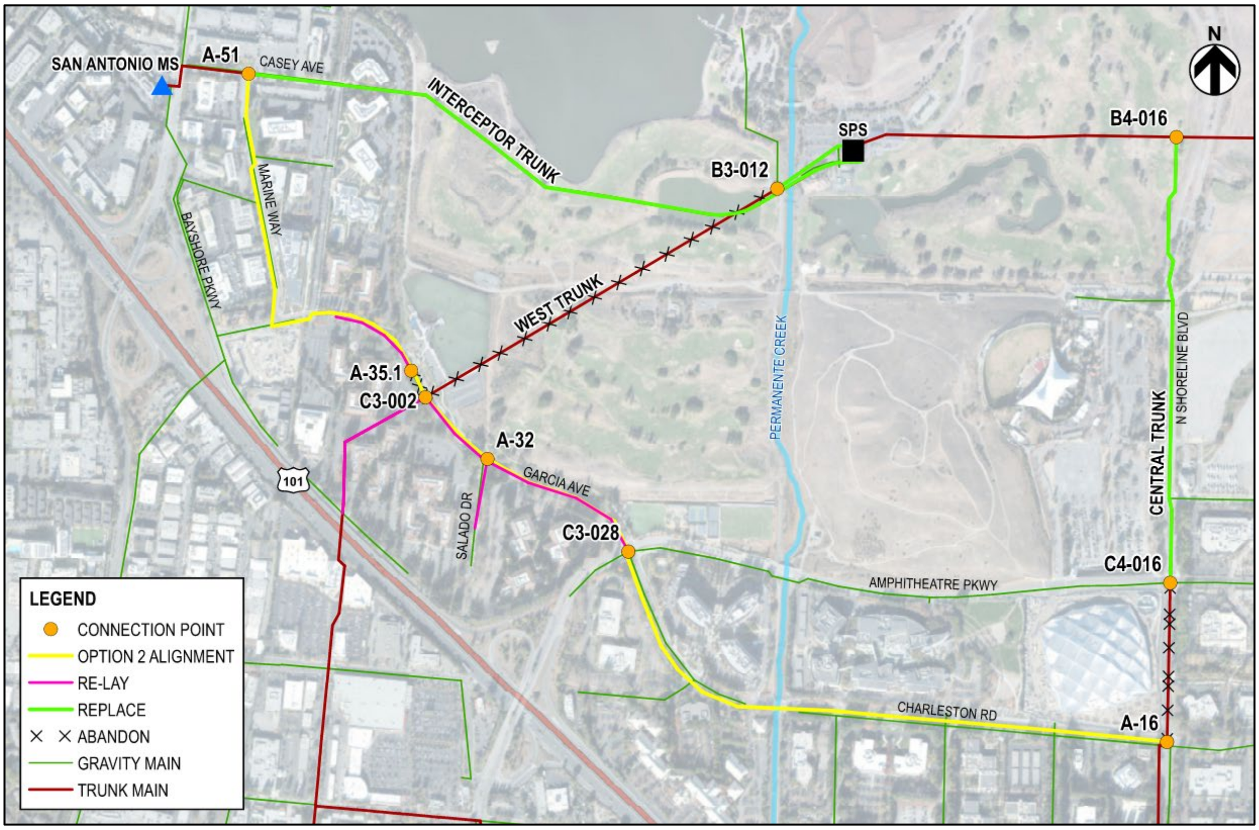
10.1.2 Option 2 – West and Central Trunks

Option 2 (see **Figure 10-3**) collects flow from both the Central and West Trunks and diverts it west to the San Antonio Metering Station by gravity. It requires the construction of a new manhole (A-16) on the existing 30-in pipeline flowing north in the intersection of Charleston Rd and N Shoreline Blvd.

Similar to Option 1, the interceptor connects to the existing collection system at manhole C3-002, includes 2,960 ft of 42-in and 5,820 ft of 30-in pipe, and requires existing pipelines to be re-laid at a higher elevation and/or shallower slope. However, this option also connects to the existing collection system further upstream at manhole C3-028 in the intersection of Charleston Ave and Rengstorff Ave. Because of this additional connection, there is no need for pipelines upstream of manhole C3-028 to be re-laid, decreasing the length of existing pipe to be re-laid compared to Option 1. For this option, there is sufficient soil cover to allow the identified existing segments to be re-laid while still maintaining minimum slopes. As in Option 1, the existing 8-in pipeline in Garcia Drive northwest of manhole C3-002 should be rerouted to the bypass interceptor pipeline at manhole A-35.1. Additionally, if the pipeline in Salado Dr is reconnected to the new bypass interceptor pipeline at new manhole A-32, then only the downstream segment in Salado Dr needs to be re-laid. As in Option 1, the portion of the West Trunk north of Hwy 101 should be re-laid at a shallower slope and upsized to 36-in pipe. The existing 30-in segment crossing under Hwy 101 is not required to be upsized.

As with Option 1, Option 2 allows 2,650 ft of the existing West Trunk to be abandoned and requires that 580 ft of the West Truck be reduced to an 8-in pipe to be installed within the existing 33-in and 39-in pipe leading to SPS. Additionally, interception of the Central Trunk allows 1,030 ft of the existing Central Trunk between manholes A-16 and C4-016 to be abandoned as there are no flow contributions along this segment.

Figure 10-3: Bypass Interceptor Option 2



Due to the decreased flows, 2,930 ft of the existing Central Trunk – ranging from 27-in to 33-in in diameter between manholes C4-016 and B4-016 – have velocities under 2 ft/s. Velocities under 2 ft/s can lead to debris build up and increased levels of H_2S gas which contributes to higher corrosion rates, specifically for concrete pipes; 740 ft of this pipeline is RCP which is prone to corrosion. It is recommended that 12-in pipe be installed within this pipeline to increase minimum dry weather velocities. Though 12-in pipe at this slope does not result in all segments meeting the minimum velocity of 2 ft/s during dry weather, it does result in some segments reaching 2 ft/s where they previously did not, and the remaining segments are closer to 2 ft/s. However, CCTV inspection of this pipeline is recommended to determine the current condition of the pipes; if they appear to be in good condition and at low risk for corrosion, then smaller pipe installation might be avoidable with regular cleaning and maintenance.

As in Option 1, the portion of the Interceptor Trunk carrying flow from SPS to manhole A-51 must be replaced with a smaller pipe to operate as a force main due to the reduced flows. This smaller pipe can be installed within the existing trunk and should be installed at the top of the pipe to prevent the small diameter pipe from matching inverts with the existing 42-in pipe downstream. This option requires a 24-in force main. **Figure 10-3** presents the alignment of the interceptor and identifies all existing pipelines that would need to be re-laid, abandoned, and replaced as part of Option 2.

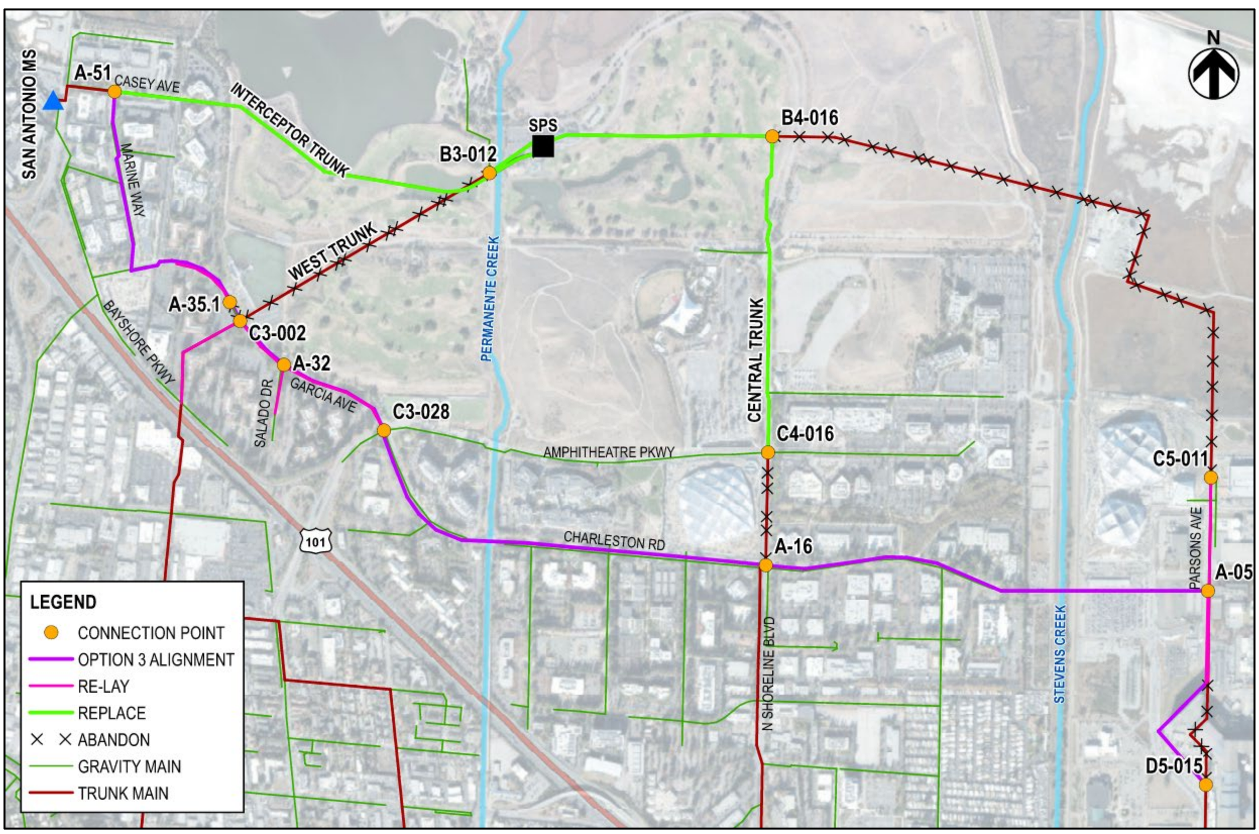
10.1.3 Option 3 – West, Central, and East Trunks

Option 3 (see **Figure 10-4**) captures flow from the East, Central, and West Trunks and diverts it west to the San Antonio Metering Station by gravity. It connects to the East Trunk at manhole D5-015 just north of RT Jones Rd near the NASA Wind Tunnel and includes 2,960 ft of 48-in, 5,820 ft of 36-in, and 6,240 ft of 30-in pipe. It is noted that for this option, the portion of the bypass interceptor between manholes C3-002 and A-51 requires 48-in diameter pipe, which then transitions down to 42-in pipe when it connects to the existing system at A-51. It is typically not recommended to have larger diameter pipes flowing into smaller diameter pipes; however, there are no modeled capacity deficiencies (as defined in **Section 9.3**) in the existing 42-in pipelines.

Beginning at manhole D5-015, the interceptor is routed around the NASA Wind Tunnel, allowing the existing pipe segments running underneath the wind tunnel to be abandoned. A portion of the existing pipeline just north of the Wind Tunnel is then re-laid at a shallower slope to maintain the elevation required to continue by gravity to the San Antonio Metering Station. Before reaching Allen Rd, the interceptor turns west and requires a 24-in double barrel inverted siphon crossing under Stevens Creek. It then follows Charleston Rd to N Shoreline Blvd where it continues along the same path with the same requirements as Option 2.

For this option, the portion of the existing West Trunk to be re-laid north of Hwy 101 should be further upsized to 42-in pipe due to the shallow slope and increased flows in the bypass interceptor pipeline downstream. The existing 30-in segment crossing under Hwy 101 is not required to be upsized.

Figure 10-4: Bypass Interceptor Option 3



As with Option 2, Option 3 includes the abandonment of 2,650 ft of the existing West Trunk, 580 ft of the West Truck reduced to an 8-in pipe to be installed within the existing 33-in and 39-in pipeline of the West Trunk, abandonment of 1,030 ft of the existing Central Trunk, and 2,930 ft of the existing Central Trunk reduced to a 12-in pipe to be installed within the existing Central Trunk. Additionally, the only flow contribution to the East Trunk downstream of new manhole A-05 is Moffett Field. The East Trunk between manholes A-05 and B4-016 has very shallow slope which, with the reduced flows in this pipeline, would likely cause maintenance and odor issues. As an alternative, the Moffett Field flows should be rerouted from their current discharge point (C5-011) to discharge to the bypass interceptor at manhole A-05. Due to the lack of available information about Moffett Field's internal collection system, it is unclear whether this flow can be rerouted by gravity, or a small lift station is required. With the rerouting of Moffett Field flows, the remainder of the East Trunk between manholes C5-011 and B4-016 can be abandoned.

With the decreased flows in the existing pipeline between manhole B4-016 and SPS, 12-in pipe should be installed within this trunk. Most of these segments reach velocities of 2 ft/s with the 12-in diameter, but increased frequency of cleaning of these pipelines may be required to prevent debris buildup and/or odor issues. Additionally, the portion of the Interceptor Trunk carrying flow from SPS to manhole A-51 requires replacement with a 4-in force main installed within the existing Interceptor Trunk.

Figure 10-4 presents the alignment of the interceptor and identifies the existing pipeline that would need to be re-laid, abandoned, and replaced as part of Option 3.

10.2 Hydraulic Modeling

The bypass interceptor was analyzed for the potential to downsize SPS by intercepting the West, Central, and/or East Trunks and diverting their flow directly to the San Antonio Metering Station by gravity. To do this, three analysis scenarios were created to model each of the bypass interceptor options described above. The 2030_w/CIP scenario (see **Section 9.1**) was used as the basis for this analysis, and pipeline sizing for each option was determined based on the design criteria summarized in **Section 9.4**.

Table 10-1 presents a summary of the flow collected by the bypass interceptor under ADWF, peak DWF (PDWF), and PWWF conditions as well as the remaining flow entering SPS for all three options.

Table 10-1: Bypass Interceptor Flow Summary

Option	Trunk Connection Points	Flow Collected (MGD)[1]			Remaining SPS Flow (MGD)		
		ADWF	PDWF	PWWF	ADWF	PDWF	PWWF
-	Current Operation	-	-	-	7.4	11.4	19.1
1	West only	3.2	4.8	8.5	4.2	6.5	11.2
2	West and Central	4.6	7.2	12.0	2.8	4.2	7.3
3	West, Central, and East	7.3	11.2	18.4	0.1	0.2	0.3

Notes:
1. Flow recorded in the most downstream pipeline of the alternative alignment before it reconnects to the existing collection system at new manhole A-51 – see **Figure 10-2** for reference.

10.2.1 SPS Emergency Gravity Bypass

As shown on **Figure 3-9**, SPS is equipped with an emergency bypass that operates by gravity – similar to an overflow flow split (see **Figure 3-7**) – to allow wastewater to continue flowing eastward in the event of an emergency or necessary shutdown of the pump station. To operate the emergency bypass, the upstream pipelines begin to surcharge until the HGL reaches the elevation of the bypass pipe.

To analyze this emergency condition, the model was run under both the Existing and 2030_w/CIP scenarios with the pumps at SPS turned off. This analysis was completed under dry weather conditions as it is assumed that SPS maintenance would occur during the dry season to minimize risk. The required increase in the HGL upstream of SPS results in backwater surcharging in the upstream collection system including all three trunk mains (West, Central, and East). **Figure 10-5** and **Figure 10-6** present the modeling results for the Existing and 2030_w/CIP scenarios, respectively.

The severity of the surcharging was investigated by reviewing the available freeboard in the portions of the system experiencing backwater effects, which is also presented on **Figure 10-5** and **Figure 10-6**. The manhole with the least remaining freeboard (HGL closest to the ground surface) is manhole B4-017, located adjacent to the Shoreline Amphitheatre, with 2.0 ft of freeboard in the Existing scenario, and 1.8 ft of freeboard in the 2030_w/CIP scenario. Though the hydraulic model does not indicate an SSO during SPS bypass operation, freeboard this low indicates a high risk of SSO and thus, it is recommended that this location be monitored during SPS bypass operation.

Historically, the Moffett Field flow meter has experienced meter read errors when the City has operated the emergency gravity bypass for SPS maintenance. Additionally, the operation of the gravity bypass under both existing and future flow conditions is an additional consideration in the overall reliability of the collection system in emergencies that should be weighed into the City's decision to implement the bypass interceptor pipeline.

Given that the invert of the manhole where Moffett Field wastewater discharges to the City collection system (-2.6 ft) is lower than the invert of the bypass overflow pipeline (-0.7 ft), the flow must back up past the Moffett discharge manhole in order to operate the gravity bypass. Assuming the Moffett Field flow meter is a depth-velocity meter, this is likely the reason its accuracy has been affected during operation of the SPS emergency gravity bypass.

Implementation of any option of the bypass interceptor presented herein would be accompanied by a downsizing of the existing SPS which would be designed with backup power and pumping redundancy, and an alternate bypass. This would eliminate the need for the existing gravity bypass and extensive backwater surcharging currently required.

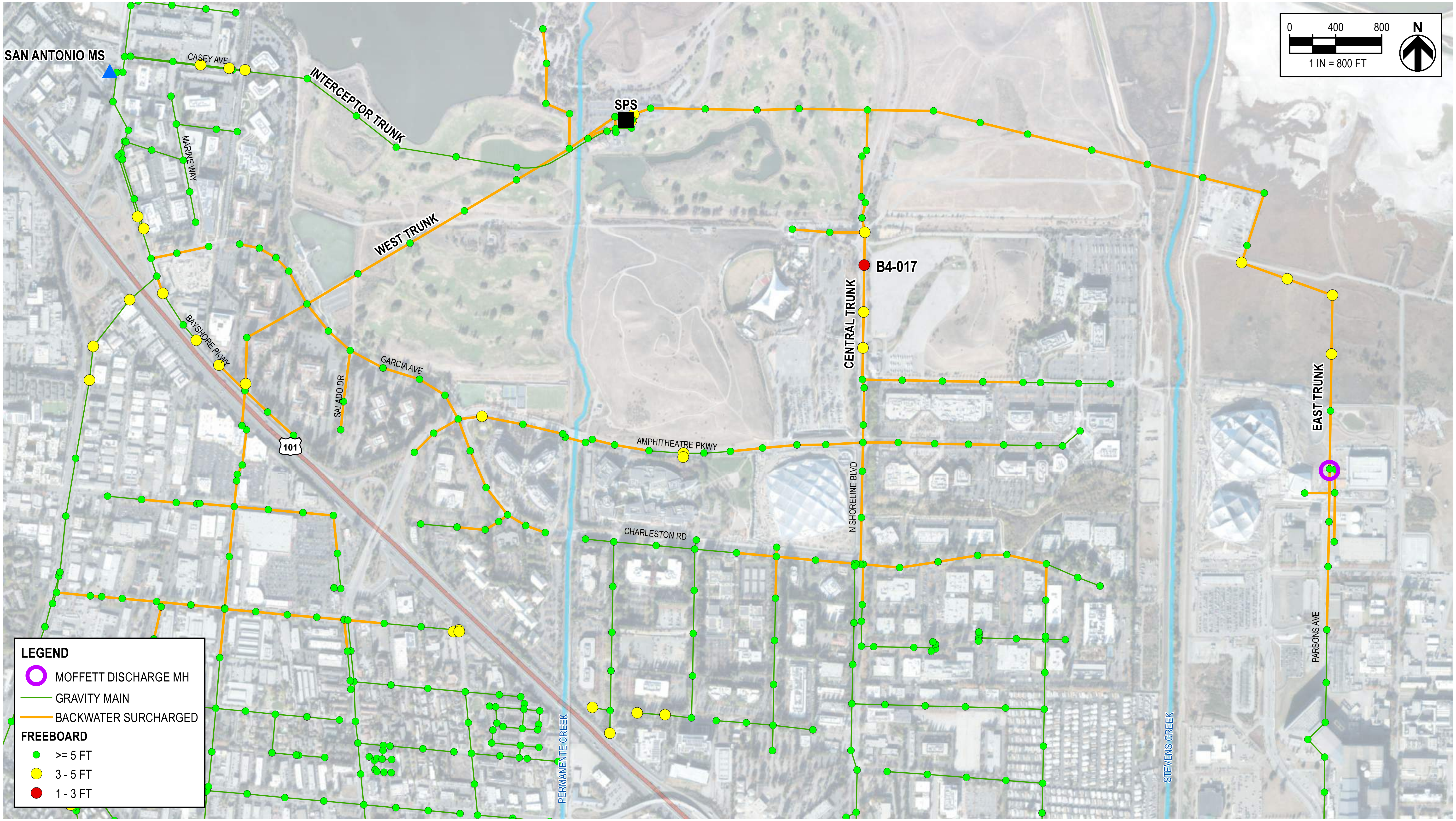


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FIGURE 10-5
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
MODEL RESULTS - SPS EMERGENCY GRAVITY BYPASS - EXISTING DWF

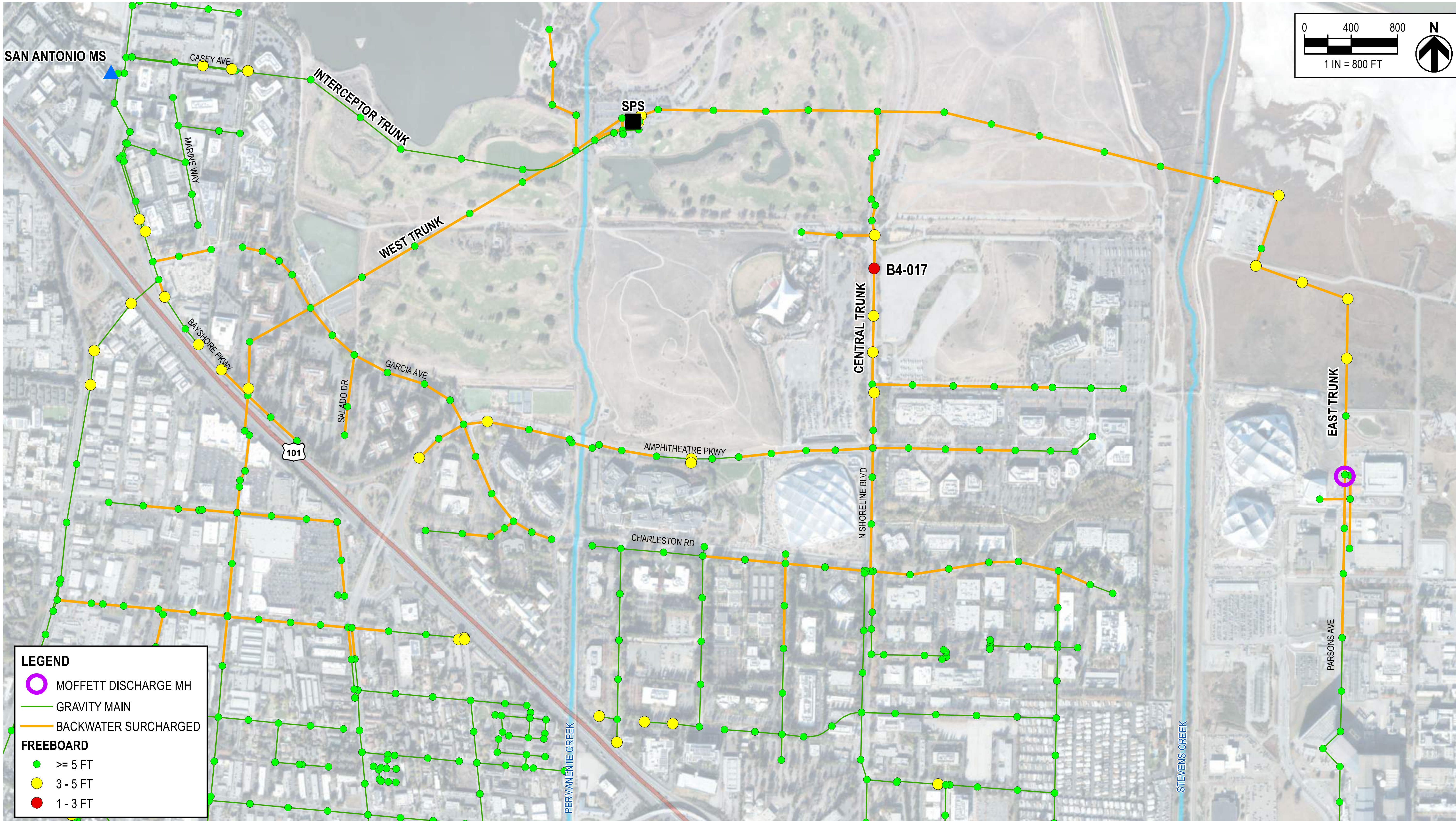

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FIGURE 10-6
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
MODEL RESULTS - SPS EMERGENCY GRAVITY BYPASS - 2030_W/CIP DWF

10.3 Cost Estimates

In order to determine the most cost-effective option, cost estimates were developed for each of the options using the results of the hydraulic modeling. Cost considerations include:

- Construction of the bypass interceptor pipeline itself;
- Alterations to existing pipelines including re-laying at higher elevations and/or shallower slopes, slip lining, and abandoning;
- Replacement of the existing SPS with a smaller pump station; and
- Estimated annual energy costs to operate the smaller pump station.

The basis of cost estimate development for each of these items is presented in the following sections, and a breakdown of units costs and detailed calculations are included in **Appendix I**.

10.3.1 Pipeline Construction Costs

Table 10-2 includes a cost estimate for the pipelines included in each bypass interceptor option, as described in the sections above. This includes the construction of the bypass interceptor itself; all existing pipe that would need to be re-laid to accommodate the higher invert elevations of the bypass interceptor compared to existing conditions; smaller pipe that would need to be installed within the existing trunks due to decreased flows, including the Interceptor Trunk; and the abandonment of existing pipelines that are left without flow. It is noted that the cost estimates for smaller pipes installed within larger existing pipes include an additional allotment for annular space controlled low-density fill.

Table 10-2: Pipeline Cost Estimates

	Option 1	Option 2	Option 3
Total Cost	$24,110,000	$34,150,000	$54,432,000

10.3.2 Pump Station Costs

Flows entering SPS are drastically reduced for all three bypass interceptor options under both dry weather and wet weather conditions, as displayed in **Table 10-1**. Modifications to the existing pump station are likely infeasible due to the unique nature of the current-day SPS. Cost estimates for replacing the existing SPS with a smaller and more standard pump station are provided for each option in **Table 10-3**. These cost estimates were developed based on the remaining PWWF entering the pump station (see **Table 10-1**) and the pump station construction cost curve from “Pumping Station Design: Revised 3rd Edition,” Jones, PE, DEE, Garr M., et al., 2011 which is included in **Appendix I**.

The new pump station should be constructed at a higher elevation which would likely eliminate the need to upsize the stormwater dewatering pump station at the existing SPS site.

Table 10-3: Pump Station Cost Estimates

Parameter	Option 1	Option 2	Option 3
PWWF Capacity (MGD)	11.2	7.3	0.3
Construction of New Pump Station	$11,909,000	$9,531,000	$603,000
Demolition of Existing Pump Station[1]	$500,000	$500,000	$500,000

Notes:
1. Rough estimate for station demolition, haul away, and back fill of depression.

10.3.3 Energy Costs

With lower flows entering the pump station, operation of a smaller pump station in place of the current SPS additionally decreases operational energy costs. Energy costs were calculated for this analysis based on the remaining ADWF presented in **Table 10-1**, an estimated total head during ADWF, and a specific gravity of 1.3 for each option using the following equation:

$$kW = (\text{Total head (ft)} * Q\ (\text{gpm}) * SG * 0.7457)/3956$$

Only the energy used to pump wastewater flow was included in the calculation as that is assumed to be the majority of the energy usage.

Energy costs fluctuate throughout the day based on time of use; for this analysis, an average energy unit cost of $0.14/kWh was used based on the average commercial electricity rate in Mountain View according to electricitylocal.com to determine a rough energy cost estimate. Over time the energy costs add up, therefore the 20-year cost is also included in **Table 10-4**.

Table 10-4: Energy Cost Estimates

Parameter	Existing	Option 1	Option 2	Option 3
Estimated Annual Energy Use (kWh)	280,000	88,000	59,000	2,300
Annual Energy Cost	$39,500	$12,500	$8,500	$500
20-Year Energy Cost[1]	$1,061,000	$333,000	$223,000	$10,000

Notes:
1. Estimated 20-year energy cost calculated assuming an annual inflation rate of 3%.

10.3.4 Cost Summary

Table 10-5 presents a summary of the total costs of each bypass interceptor option as detailed above.

Table 10-5: Total Cost Estimate

Item	Option 1	Option 2	Option 3
Pipeline Costs	$24,110,000	$34,150,000	$54,432,000
Pump Station Costs	$11,909,000	$9,531,000	$603,000
Abandon SPS Cost	$500,000	$500,000	$500,000
20-Year Energy Costs	$333,000	$223,000	$10,000
Total Cost	**$36,852,000**	**$44,404,000**	**$55,545,000**
Flow Collected (MGD)	**8.5**	**12.0**	**18.4**
Unit Cost ($M/MGD)	**$4.5M**	**$3.7M**	**$3.0M**

Cost Savings: Should the City build the bypass interceptor and replace the existing SPS with a smaller pump station, there are a number of SPS facility upgrades that would no longer be necessary as well as a notable annual energy savings to operate a smaller pump station than the existing SPS. Energy costs to operate a smaller pump station relative to the energy costs of operating the existing SPS are estimated at approximately 31% for Option 1, 21% for Option 2, and <1% for Option 3.

Table 10-5 also provides the unit cost ($M per MGD collected) for each proposed option. Option 3, while the most expensive, also collects the most wastewater flow via gravity, resulting in a lower unit cost.

It is also noted that all pipeline that is replaced or re-laid as part of the bypass interceptor implementation would address any immediate and future rehabilitation or replacement needs for those pipelines.

10.4 Non-Economic Screening

The SPS was created as an interim solution at the closing of the City's WWTP and was not intended as the permanent solution for the City. Over the years, the City has considered options to redirect flow from the SPS to the metering station via a gravity bypass interceptor. When weighing the benefits for constructing a bypass interceptor versus maintaining the existing SPS, the following factors are considered:

- **Reliability** – As it is currently operating, the SPS is a highly critical facility. With more of the City's wastewater flowing by gravity, there is increased reliability especially in the case of an emergency power outage with decreased reliance on a large pump station.
- **Risk Mitigation** – The consequence of failure can be severe. A backup at the existing SPS presents a risk of large SSO due to the volume of flows entering the facility and SPS is located in a sensitive habitat.

The non-economic screening analysis utilizes selected project criteria, or decision factors, which are intended to define each selected project using a weighted scoring system. **Table 10-6** describes the decision factors selected for the non-economic analysis as they pertain to the alternatives analyzed.

Table 10-6: Decision Factors

Decision Factor	Description
Enhances Maintenance Worker Safety	The expected degree of enhanced maintenance worker safety conditions due to the implementation of lift station improvement.
Improves System Reliability	The implementation of the project alternative and its effectiveness to bolster the collection system's automatic operation in the event of a power outage or malfunction.
Reduced Risk of SSO	The effectiveness of the improvement to mitigate potential for SSOs.
Implementation Time/ Constructability	The likelihood that the alternative could be successfully implemented in a relatively short period. Considers unknowns and construction complexities that could unexpectedly delay completion or create prolonged impacts to the community.
Reduced Maintenance Frequency	The effectiveness of the improvement to minimize maintenance frequency and support automatic functionality.

The decision factors are used for ranking the alternatives.

10.4.1 Comparative Rating Methodology

The evaluation employs the use of a weighted matrix that considers the relative importance (weight) of each decision factor. This analysis presents a comparison of each of the projects by assigning a relative rating for each on a scale of 1 to 5, with 5 being the most desirable or favorable. First, factor importance must be evaluated to develop the weighted matrix. This analysis employs the Pairwise Comparison Method to develop the weight of each decision factor. In this analysis, each decision factor identified in **Table 10-6** is evaluated head-to-head with the other decision factors and scored based on relative importance. **Table 10-7** describes the criteria for scoring each of the decision factors. The scores are totaled for the leading factor (Factor A) and normalized such that the highest score is equal to 5. The resulting normalized totals represent the weighted factors that will be used for the project prioritization.

Table 10-7: Comparative Rating Criteria

If Factor A is:	Factor A	Factor B
Much more important than Factor B	5	1
More important than Factor B	4	2
Equal in importance to Factor B	3	3
Less important than Factor B	2	4
Much less important than Factor B	1	5

Table 10-8 presents the pairwise comparison of each decision factor included in the alternative screening and prioritization and the resulting weights for each factor.

Table 10-8: Pairwise Comparison of Decision Factors

	Factor vs Factor	Factor B: Enhances Maintenance Worker Safety	Factor B: Improves System Reliability	Factor B: Reduces Risk of SSO	Factor B: Implementation Time/ Constructability	Factor B: Reduces Maintenance Frequency	**Total**	**Normalized Weight**
Factor A	Enhances Maintenance Worker Safety		5	5	5	5	20	**5**
	Improves System Reliability	1		5	5	5	16	**4**
	Reduces Risk of SSO	1	1		4	4	10	**3**
	Implementation Time/ Constructability	1	1	2		3	7	**2**
	Reduces Maintenance Frequency	1	1	2	3		7	**2**

Each alternative was then evaluated and scored based on each individual decision factor and ranked based on the total scores. This pairwise comparison identifies the highest priority factors, which will in turn determine the criticality of the selected projects. Based on the results of this comparison, the two factors with the highest priority are:

- Enhances Maintenance Worker Safety
- Improves System Reliability

The lowest priority factors, both with a decision factor score of 2, are:

- Implementation Time/Constructability
- Reduces Maintenance Frequency

The normalized scores then feed into the matrix analysis to rank how well each project addresses or achieves each factor. Each project is evaluated for each of the decision factors and rated on a scale of 1 to 5 where:

- 1 is least favorable;
- 5 is most favorable; and
- 3 is neutral.

These ratings are then multiplied by the normalized pairwise score to produce a weighted rating. The weighted ratings are summed to produce an overall score for the project. The following section discusses project ratings.

10.5 Analysis Summary

Each bypass interceptor option presented herein along with the option of maintaining the existing SPS is scored according to the decision factors and scoring system presented in **Section 10.4**. The scoring is designed to identify the alternative that best meets the City's needs prior to considering cost. The weighted decision matrix is presented as **Table 10-9**.

Table 10-9: Weighted Decision Matrix

Factor	Weight	Existing SPS		Option 1		Option 2		Option 3	
		Rating	WR	Rating	WR	Rating	WR	Rating	WR
Enhances Maintenance Worker Safety	5	2	**10**	3	**15**	4	**20**	5	**25**
Improves System Reliability	4	1	**4**	2	**8**	3	**12**	5	**20**
Reduces Risk of SSO	3	1	**3**	2	**6**	3	**9**	4	**12**
Implementation Time/ Constructability	2	5	**10**	4	**8**	3	**6**	2	**4**
Reduces Maintenance Frequency	2	1	**2**	3	**6**	3	**6**	5	**10**
Total Weighted Rating		**29**		**43**		**53**		**71**	

According to these ratings and weightings, the highest-ranking option is Option 3. City Staff communicated that the most significant concern with SPS is reliability. Though a smaller pump station is still required with the implementation of the bypass interceptor, the flows are drastically decreased leaving all but 0.3 MGD (ADWF) of the City's wastewater to flow through the City's collection system to the RWQCP by gravity, which ultimately significantly increases the reliability of the system especially in emergencies. The pump station required under Option 3 could be a small submersible packaged pump station similar to the size of a manhole. There are also significant energy savings that would be realized with Option 2. The main draw back for the implementation of a bypass interceptor is the initial construction cost as detailed in the previous section.

Should the City decide to move forward with implementation of the bypass interceptor, construction will likely be completed in phases with the lower portions of the pipeline constructed first. As proposed, the interceptor pipeline has little margin for adjustment in the invert elevations, particularly in the lower portion of the pipeline downstream of the interception of the West Trunk flows at manhole C3-002. If pursued, it is recommended that the City complete the design of the entire pipeline prior to beginning the first phases of construction to account for any potential utility or other conflicts that may be identified in the upper portions of the pipeline.

Additionally, during the predesign phase, the HGL at the San Antonio Metering Station should be analyzed in greater detail to determine the effects on the bypass interceptor, as described specifically in Option 3. For general collection system capacity analysis purposes, a boundary condition at the system outfall will likely have little to no effect on the collection system as a whole, but it may affect the required diameter of the pipe segments in the immediate area of the metering station.

SECTION 11 – CAPITAL IMPROVEMENT PROGRAM

This section presents the projects proposed to continue to ensure the reliable and safe collection of wastewater for City residents and the estimated costs and schedule for implementation of the CIP. Projects are designed to address pipeline capacity deficiencies, minimize the risk of SSO, and address any other physical deficiencies identified at City wastewater facilities. In general, the purpose of a sewer system CIP is to:

- Maintain and enhance the City's sewer infrastructure to benefit the community and contribute to the overall quality of life in the City;
- Address health and safety concerns and to comply with regulatory requirements; and
- Develop and implement projects to ensure continued and reliable sewer collection services to meet the City's needs.

11.1 Existing Capital Improvement Plan Projects

The five-year CIP is adopted biennially, with a full plan developed in odd-numbered years and a focus only on the upcoming FY in even-numbered years. The City's program is intended to provide safe and reliable system for collecting and transporting wastewater to the RWQCP. Provided below is a brief summary of the active improvements as well as those currently planned in the service area that will be funded through the Wastewater Enterprise Fund.

Non-Discretionary Projects (FY 22/23-26/27): Non-discretionary projects are primarily annual and periodic infrastructure maintenance projects to preserve the City's significant investment in its infrastructure and facilities. They also include projects required for regulatory compliance and include small inflationary adjustments over time. Provided is the project name, number, brief description, and dedicated annual funding – summarized in **Table 11-1**.

- **Wastewater System Improvements (Project 23-07):** Unscheduled improvements/repairs to the City's wastewater collection and pumping system.
- **Annual Storm/Sanitary Sewer Main Replacement (Project 23-09):** Repair and replace storm and sanitary sewer pipes, manholes and systems identified by the City's annual line televising program.
- **Developer Reimbursements (Project 23-22):** Construction of street and utility improvements concurrent with private development. Adjacent properties benefiting from street and utility improvements will be required to reimburse the City for the improvements.
- **Information Technology (IT) Projects:** These are five separate IT projects designed to support City facilities and staff. These projects are funded by a variety of sources; only funding sourced from the Wastewater Fund is summarized herein.
    - Land Management System and Paperless Permitting System (Project 11-18),
    - WiFi Systems at City Facilities (Project 13-18),
    - Permanent Audio/Video Equipment in Conference Rooms (Project 17-18),
    - Citywide Website Software Update/Content Migration (Project 21-32), and
    - IT Infrastructure and Telecommuting Support (Project 21-33).

Table 11-1: Summary of Non-Discretionary Projects Sourced from Wastewater Fund

Project No.	FY 22/23	FY 23/24	FY 24/25	FY 25/26	FY 26/27[1]	Total
23-07	$174,000	$178,000	$181,000	$185,000	$189,000	**$907,000**
23-09	$1,750,000	$1,785,000	$1,821,000	$1,857,000	$1,894,000	**$9,107,000**
23-22	$33,000	$34,000	$34,000	$35,000	$36,000	**$172,000**
IT Projects	$30,000	$30,000	$30,000	$30,000	$30,000	**$150,000**
Total	**$1,987,000**	**$2,027,000**	**$2,066,000**	**$2,107,000**	**$2,149,000**	**$10,336,000**

Notes:
1. Assuming 2% increase in budget annually except for IT projects which are typically amended annually.

Discretionary Projects (FY 22/23-25/26): Discretionary projects are those that do not fit the non-discretionary description and require approval of City Council. The sources of potential discretionary projects include City plans and studies (e.g., Precise Plans, Transportation Plans, Sea-Level Rise Study, Utility Master Plans, Parks and Open Space Plans, etc.), City Council goals and priorities, project submittals from all City departments, and unscheduled projects in the current CIP. The following projects are currently slated for funding in FYs shown in **Table 11-2**.

- **MOC – Confined Space/Trench – Design and Construction (Project 22-33):** Design project would include building an “in-ground” confined space and trench rescue training prop. Cal-OSHA requires fire departments to perform annual confined space and trench rescue training. This prop will provide a suitable location for those mandatory drills. The prop will also be used by Public Works personnel for the same purpose. Total funding for construction planned in FY 21/22 is $250,000, of which $62,000 is sourced from the Wastewater Fund. Total funding for this project in FY 23/24 is $710,000, of which $177,000 is sourced from the Wastewater Fund.
- **Electrical Arc Flash Assessment (Project 22-39):** This project is to conduct arc flash evaluations at pumps and wells to meet State Safety Regulations (OSHA standard §2940.11. Protection from Flames and Electric Arcs). Total funding for this project is $120,000 of which half ($60,000) was sourced from the Wastewater Fund in FY 21/22.
- **Citywide Trash Capture, Phase II (Project 22-40):** These projects propose to install trash capture devices on the City's storm drain system to work towards the required trash load reduction (100% by 2022) of the Municipal Regional Stormwater National Pollutant Discharge Elimination System (NPDES). Total funding for this project is $1,130,000 which was sourced from the Wastewater Fund in FY 21/22.
- **Downtown Utility Improvements – Design and Construction (Project 22-41):** This project is to design and construct the relocation/abandonment of the 16-in water transmission main outside the Moffett Blvd/Castro St/Central Expy intersection and replacement with an 18-in bypass outside the Transit Center Grade Separation and Access Project (GSAP). This project will also upsize the 900 ft of existing sanitary sewer main downstream of the Transit Center GSAP and relocate the water main and sanitary sewer main impacted by the W Evelyn Ave ramp portion of the Transit Center GSAP. Total funding for this project is $8,210,000, of which $2,000,000 is sourced from the Wastewater Fund in FY 22/23 and $300,000 was sourced from the Wastewater Fund in FY 21/22.

- **Middlefield Rd and Moffett Blvd Sewer Replacement – Design (Project 22-42):** As part of the sewer system plan to eliminate the sewer crossing of Stevens Creek and SR 85, wastewater flow is proposed to be reversed to flow south on Moffett Blvd and then connect to Middlefield Rd. Total funding for design of this project is $750,000 which was sourced from the Wastewater Fund in FY 21/22.
- **Storm Drain System Improvements (Project 23-xx):** Improve the existing City storm drain system based on findings from the City's 2017 Storm Drain Master Plan and the resulting CIP recommendations. Total funding for this project is $1,410,000 which is sourced from the Wastewater Fund in FY 22/23.
- **Utility Rate Study (Project 23-46):** The City plans to conduct a comprehensive cost of service analysis for the City's water, recycled water and sewer utilities to evaluate the City's rate structures and provide options for funding future expenditures. The last water and sewer rate study was completed in 2013. Total funding for this project is $200,000 of which $100,000 is sourced from the Wastewater Fund in FY 22/23.
- **Cross Culvert Removal and Storm Drain Extension (Project 24-xx, 25-xx and 26-xx):** This project proposes to remove cross culverts at one intersection per year. Project scope includes removal of cross culverts and construction of new curb ramps, curbs, gutters, roadway pavement and storm drainage. Total funding for this project is $2,040,000 of which $950,000 is sourced from the Wastewater Fund for FY 23/24-25/26.
- **Middlefield Rd and Moffett Blvd Sewer Replacement – Construction (Project 24-xx):** As part of the sewer system plan to eliminate the sewer crossing of Stevens Creek and SR 85, the sewage flow is proposed to be reversed to flow south on Moffett Blvd and then connecting to Middlefield Rd. Total funding for construction of this project is $8,350,000 which is sourced from the Wastewater Fund in FY 23/24.
- **San Antonio Rd Sewer Improvements – Construction Phase I (Project 24-xx):** Construction phase to replace sewer on Showers Dr to California St (CIP 17-50 Design). Total funding for this project is $810,000 which is sourced from the Wastewater Fund in FY 23/24.

Table 11-2: Summary of Discretionary Projects Sourced from Wastewater Fund

Project No.	FY 22/23	FY 23/24	FY 24/25	FY 25/26	Total
22-33	-	$177,000	-	-	**$177,000**
22-41	$2,000,000	-	-	-	**$2,000,000**
22-42	$750,000	-	-	-	**$750,000**
23-xx	$1,410,000	-	-	-	**$1,410,000**
23-46	$100,000	-	-	-	**$100,000**
24-xx, 25-xx, & 26-xx	-	$300,000	$300,000	$350,000	**$950,000**
24-xx (Middlefield/Moffett)	-	$8,350,000	-	-	**$8,350,000**
24-xx (San Antonio - Const)	-	$810,000	-	-	**$810,000**
Total	**$4,260,000**	**$9,637,000**	**$300,000**	**$350,000**	**$14,547,000**

Notes:
1. None of these projects have costs for FY 26/27.

Summary: Some projects funded in prior FYs have been amended to receive additional funds in FY 22/23. A summary of non-discretionary, discretionary, and amendments to existing capital improvement projects sourced from the Wastewater Fund through FY 26/27 is provided in **Table 11-3**.

Table 11-3: Summary of All Current Projects Sourced from Wastewater Fund

Project Type	FY 22/23	FY 23/24	FY 24/25	FY 25/26	FY 26/27	Total
Non-Discretionary	$1,987,000	$2,027,000	$2,066,000	$2,107,000	$2,149,000	**$10,336,000**
Discretionary	$4,260,000	$9,637,000	$300,000	$350,000	-	**$14,547,000**
Amendments[1]	$806,000	-	-	-	-	**$806,000**
Total	**$7,053,000**	**$11,664,000**	**$2,366,000**	**$2,457,000**	**$2,149,000**	**$25,689,000**

Notes:
1. Total amendments impacting the Wastewater Fund in FY 22/23 (Source: City of Mountain View California Operating Budget Fiscal Year 2022-23, June 14, 2022).

11.2 Cost Basis

The purpose of approximating the cost of construction is to appropriate a conservative level of funding for each identified project included within the proposed scope of the City's upcoming CIP. **Appendix J** contains details of the preliminary cost estimates for all projects analyzed as part of this Master Plan. This section describes the basis for development of the unit costs and associated costs for project specific details including pipelines, mobilization, traffic control, and pavement resurfacing.

11.2.1 Construction Unit Costs

Unit costs for pipelines, mechanical equipment, and appurtenances were estimated based on recent project bid tabulation for similar Capital Improvement Projects in the Bay Area as well as HydroScience experience estimating the cost of similar local projects. Loaded pipeline unit costs per foot of pipeline installed by open cut construction and CIPP were provided by the City. Loaded pipeline unit costs using pipe bursting methods were developed using relevant pipeline replacement project cost of construction bid averages divided by the total foot-inches (the length of pipe multiplied by the diameter in inches) of pipeline included within the scope of each specific project. Other costs include traffic control, manhole modification, lateral reinstatement, etc.

For conceptual planning and cost comparisons of pipeline projects, the parameters and associated unit costs presented in **Table 11-4** were globally applied to projects.

Table 11-4: Cost Estimating Parameters

Cost Parameter	Unit Cost	Unit
Mobilization/Demobilization	5%	Construction subtotal
PVC Pipe Replacement (Pipe Bursting) – 10-in[1]	$280 per foot	Pipeline length
PVC Pipe Replacement (Pipe Bursting) – 12-in[1]	$336 per foot	Pipeline length
PVC Pipe Construction (Open Cut) – 12-in[1]	$440 per foot	Pipeline length
PVC Pipe Construction (Open Cut) – 15-in[1]	$610 per foot	Pipeline length
CIPP – 39-in	$475 per foot	Pipeline length
CIPP – 42-in	$510 per foot	Pipeline length
Modify Existing Manhole	$3,000 per manhole	Number of manholes
Lateral Reinstatement	$1,500	Number of laterals
Pavement Replacement (Pipe Bursting)[2,3]	$1,000 per pit	Number of pits
Pavement Replacement (Open Cut)[3]	$50 per foot	Pipeline length
Sewer Bypassing[4]	$25 per foot	Pipeline length
Traffic Control During Construction	$10,000	Lump sum
Pre-Construction Cleaning/CCTV[5]	$4 per foot	Pipeline length
Post-Construction CCTV	$3 per foot	Pipeline length

Notes:
1. Loaded cost including costs associated with earthwork, demolition, sheeting, shoring & bracing, and testing & commissioning.
2. Assumes 20 ft of pavement replacement at each manhole.
3. Pavement repair assumes minimum four-foot-wide trench.
4. Sewer bypass costs are determined by expected flow capacity for sizing of temporary equipment and materials.
5. Applies only to pipe bursting to confirm there are no sags preventing the feasibility of pipe bursting.

11.2.2 Soft Costs

Soft costs are additional project costs that are not considered to be contractor construction costs. These costs include engineering design, permitting, construction administration, and construction management. Typical soft costs include:

- Permitting for proximity to Caltrain and major roadways;
- Engineering design is expected to also address California Environmental Quality Act (CEQA) requirements to identify significant environmental impacts, if any;
- Administrative and construction management costs are those costs associated with the administration of the contract and management of the project; and,
- The construction contingency provides an allotment of funds designated for unexpected issues that can change the scope of the project.

Table 11-5 provides a summary of the parameters used for estimating the soft costs. Soft costs were applied to all projects.

Table 11-5: Soft Cost Estimating Parameters

Cost Parameter	Cost	Applied to:
Permitting[1]	7%	Construction Subtotal
Engineering Design and Consulting Services	17.5%	Construction Subtotal
Engineering Services during Construction (ESDC), Construction Management, & Inspection Services	20%	Construction Subtotal
Construction Cost and Market Contingency	30%	Construction Subtotal

Notes:
1. Applied to projects only if applicable.

11.3 Proposed CIP Projects

This section presents the wastewater collection system CIP projects proposed to address the City's infrastructure improvement needs within the next 20-year period. The proposed CIP takes into consideration the identified deficiencies in the system and presents projects to address those deficiencies along with projects to improve system reliability. Detailed cost estimates are provided in **Appendix J**.

11.3.1 High Priority Improvements (FY 22/23-26/27)

The following projects are the highest priority projects for implementation in the next five years. In addition to supporting the development of a SCADA Master Plan, these projects generally address surcharging in the sewer collection system and are prioritized to prevent SSOs. A summary of the costs for FY 22/23-26/27 improvements is provided in **Table 11-6**.

SCADA Master Plan: The intent of a SCADA Master Plan is to develop a programmatic approach to improving and maintaining the SCADA system. The SCADA Master Plan will document the existing state of the system, identify the requirements and gaps in the system, evaluate alternatives, and develop a CIP for implementation over a defined period of time, ideally in coordination with other facility improvements. It is recommended that the City develop this plan in the near term so that SCADA improvements can be coordinated with other recommended facility improvements.

The estimated cost to develop the SCADA Master Plan, including a 30% contingency, is $325,000. Funding for the project would be shared between Water and Wastewater Funds. It is estimated that the contribution from the Wastewater Fund would be $162,500.

Sondgroth Way/Showers Dr: There is a capacity deficiency in an existing 8- and 10-in pipeline that crosses through the Crossings HOA from Showers Dr along Sondgroth Way, through an easement to San Antonio Cir upstream of the Alma Recorder.

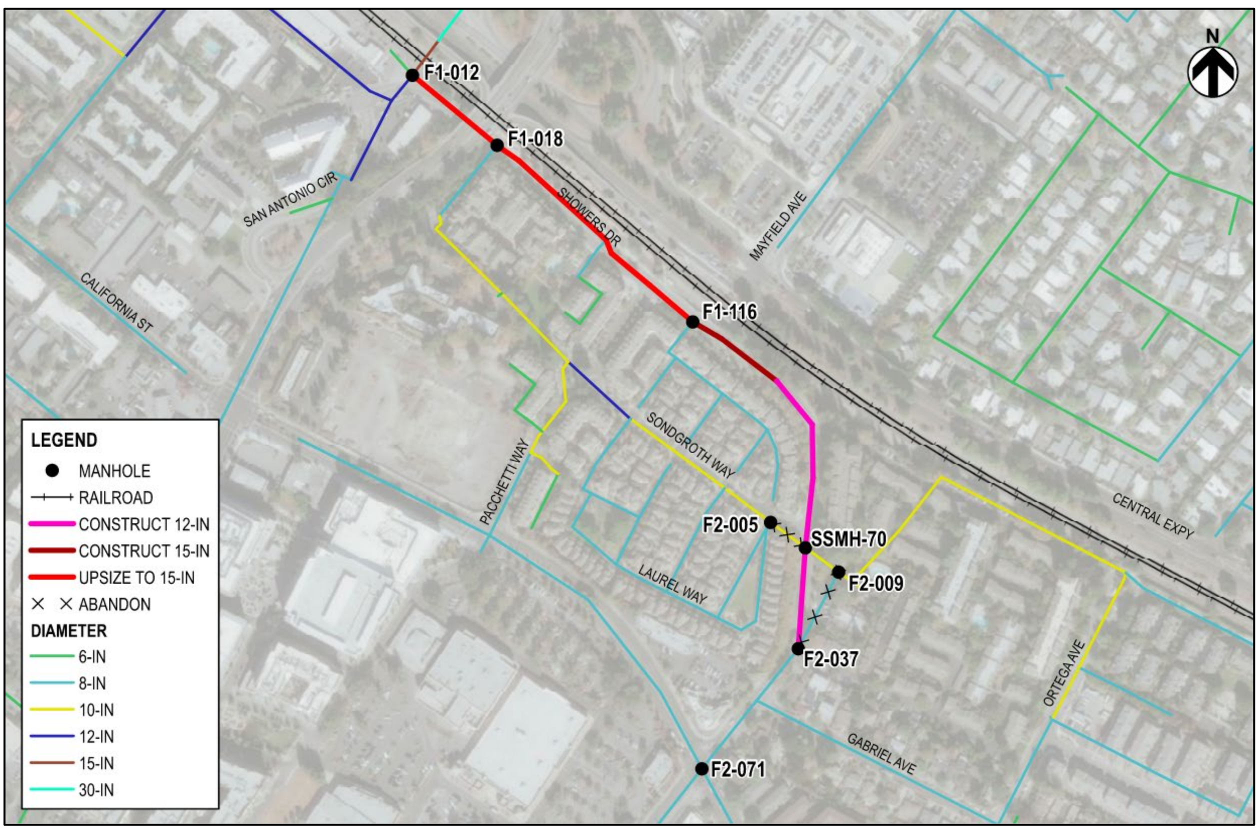
To address the deficiency, it is recommended to construct a new alignment along Showers Dr to avoid construction through easements, through the Crossings HOA, and along a walking path between developments to San Antonio Circle. This includes 780 ft of new 12-in and 295 ft of new

15-in sewer main along Showers Dr from manhole F2-037 to F1-116 with a new manhole (SSMH-70) on the existing 10-in at Showers Drive and Sondgroth Way constructed in the City ROW. All flow at this manhole would be diverted north to the new pipeline and the existing pipeline between SSMH-70 and F2-005 would be abandoned. The existing 8-in and 10-in pipe in Showers Drive and San Antonio Circle between manholes F1-116 to F1-012 would be upsized to 15-in as shown in **Figure 11-1**.

This project would also include a diversion structure in F2-071 to reroute flows away from Pachetti Way to relieve the existing surcharged 8- and 10-in pipelines. Flows would be routed into the new pipeline in Showers Drive from manhole F2-037 and the existing pipeline between F2-037 and F2-009 would be abandoned. Encroachment permits may be required due to the proximity to Central Expy and Caltrain.

The estimated cost for this project is $2,715,000, including permitting, engineering design, ESDC, construction management and inspection, and contingency.

Figure 11-1: Sondgroth Way/Showers Dr



Martens Ave and Alexander Ct: It is recommended that the two 8-in segments in Martens Ave, totaling 200 ft, be upsized to 12-in pipe as shown in **Figure 11-2**. Because of the diameter discrepancies noted in the field and in reference documents in this area, it is recommended that the City confirm the existing diameters of these pipelines before proceeding with design.

The estimated cost for this project is $176,000 including engineering design, ESDC, construction management and inspection, and contingency.

Figure 11-2: Martens Ave and Alexander Ct

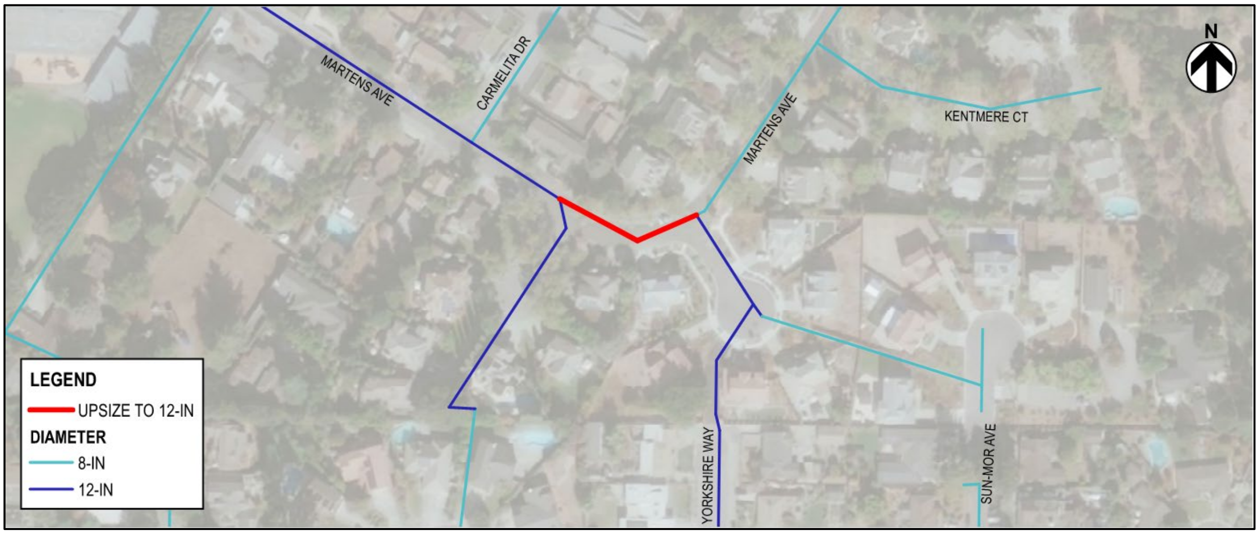


Table 11-6 presents a summary of all costs for the implementation of the high priority projects

Table 11-6: High Priority Improvements Cost Estimate (FY 22/23-26/27)

Description	Cost
SCADA Master Plan	$125,000
Sondgroth Way/Showers Dr	$1,556,000
Martens Ave and Alexander Ct	$105,000
Subtotal	**$1,786,000**
Permitting (7%)[1]	$109,000
Engineering Design and Consulting Services (17.5%)[2]	$290,000
ESDC, Construction Management, & Inspection Services (20%)[2]	$332,000
Construction Cost and Market Contingency (30%)	$537,000
High Priority Total	**$3,054,000**

Notes:
1. Only applied to Sondgroth Way/Showers Dr project.
2. Not applied to SCADA Master Plan project.

11.3.2 Medium Priority Improvements (FY 27/28-31/32)

The pipeline improvements in this phase are intended to address mild surcharging coupled with correcting design deficiencies and are not considered hydraulic deficiencies. This phase also includes the rehabilitation of the Shoreline PS. These improvements would be implemented in the six-to-ten-year timeframe. A summary of the costs for FY 27/28-31/32 improvements is provided in **Table 11-7**.

Castro St between Harpster Dr and Sonia Way: There is a single 8-in pipe segment (220 ft) along Castro St between 10-in pipelines from Sonia Way to Harpster Dr that registers a surcharge state of "2." There is over 8 ft of freeboard, so this does not trigger the criteria for a hydraulic deficiency. However, it is recommended that the segment be upsized to match the upstream and downstream 10-in pipelines as it is generally not preferred to have a larger diameter pipe flowing into a smaller diameter pipe. The condition of upstream and downstream pipelines should be evaluated to determine whether the pipeline would be considered for near-term rehabilitation or replacement.

The estimated cost for this project is $168,000, including engineering design, ESDC, construction management and inspection, and contingency.

Figure 11-3: Castro St between Harpster Dr and Sonia Way

Mountain View Ave between Park Dr and El Camino Real: There are two 8-in pipe segments totaling 300 ft between 10-in pipelines along Mountain View Ave between Park Dr and El Camino Real, one of which registers a surcharge state of “2” due to its shallower slope. There is over 6 ft of freeboard, so this does not trigger the criteria for a hydraulic deficiency. However, it is recommended that the segment be upsized to match the upstream and downstream 10-in pipelines as it is generally not preferred to have a larger diameter pipe flowing into a smaller diameter pipe. The condition of upstream and downstream pipelines should be evaluated to determine whether the pipeline would be considered for near-term rehabilitation or replacement. Due to the proximity to El Camino Real, an encroachment permit may be required.

The estimated cost for this project is $220,000, including permitting, engineering design, ESDC, construction management and inspection, and contingency.

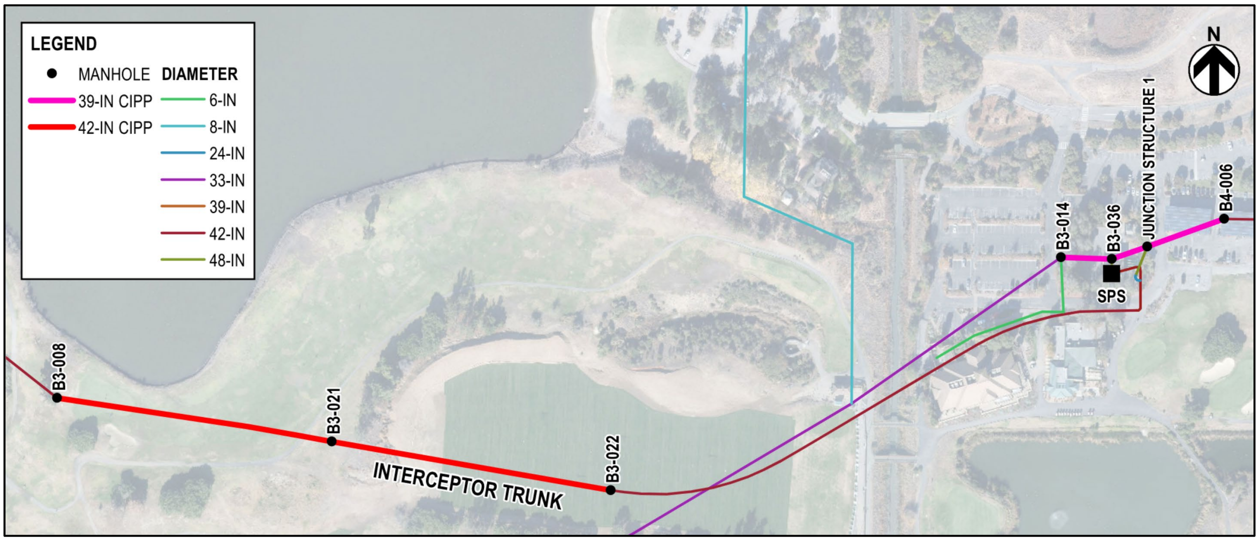
Figure 11-4: Mountain View Ave between Park Dr and El Camino Real

Trunk Main CIPP Repairs: This project includes the recommended CIPP repair of 325 ft of 39-in pipe just upstream of SPS and 1,065 ft of 42-in pipe downstream of SPS along the Interceptor Trunk as shown in **Figure 11-5**. If CCTV of the remainder of the Interceptor Trunk shows signs of corrosion through the entire length of the Trunk, it is recommended that the entire trunk be addressed as a whole.

If the City proceeds with implementation of any option of the bypass interceptor, the Interceptor Trunk will be replaced with a smaller pipe installed within the existing 42-in pipeline and the CIPP recommended here will not be necessary. Also, with implementation of any option of the bypass interceptor, the segment between manholes B3-014 and B3-036 will be replaced and the CIPP recommended here will not be necessary. The segment between manholes B4-006 and B3-036 will still need to be repaired by CIPP unless option 3 of the bypass interceptor is implemented.

The estimated cost for this project is $1,378,000, including permitting, engineering design, ESDC, construction management and inspection, and contingency.

Figure 11-5: Trunk Main CIPP Repairs



Shoreline PS Phase 1 Improvements: This project is a comprehensive rehabilitation of the Shoreline PS including mechanical, electrical, communication, and civil improvements. These improvements are slated for the five-to-ten-year timeframe and thus, if the bypass interceptor is implemented, and the existing Shoreline PS replaced with a smaller facility, within five years, the cost of these improvements could be avoided. It is noted that while the bypass interceptor pipeline is under construction, the existing Shoreline PS will need to remain in commission until flow is diverted and the new pump station is installed and thus, some of these improvements might still be necessary in the interim, depending on the timing of the bypass interceptor construction.

The highest priority improvements include reconstruction of the PG&E electrical service equipment, which will address differential settling of the existing equipment, and installation of perimeter fencing and cameras to improve site security.

Recommended mechanical improvements include demolition and replacement of the natural gas-driven standby pump with a new end-suction centrifugal pump & electric motor, with make/model similar to the three active pumps, replacement of the existing bypass/isolation slide gate in Junction Structure 1, and replacement of the existing manual overhead equipment crane operator with an electric operator for pump extractions and maintenance/servicing. The City would perform a thorough condition assessment of Junction Structure 1 and rehabilitate the concrete surface as necessary. It is also recommended that the influent flow channels located in the screening room be rehabilitated to address structural concrete repair and mechanical degradation due to H_2S corrosion. The protective coating system for the influent channels should be repaired or reapplied, as necessary.

Based on the CCTV review of the underground SPS intake piping, it is recommended that each of the four steel pump intake pipes, carrying flow from the influent channels to the pumps, be rehabilitated by CIPP lining. The existing pipes reduce from 39-in opening down to 20-in diameter pipes by reducer fittings. Due to this diameter transition and the curved shape of the pipes, the CIPP lining would require a custom design. The estimated cost for this custom CIPP lining is approximately $50,000 per intake pipe.

Safety elements include equipping pump room hatches with fall protection to match the existing OSHA-compliant fall protection netting. Existing HVAC should be reconfigured to relocate an existing air handler currently installed over the Maintenance Shop staircase to prevent operator injury during equipment maintenance. The light switches at the rear entrance of the electrical building that are located behind the HVAC ducting should be relocated to the other side of the doorway for operator safety.

This project includes the improvement of controls and communication equipment including control panel and RTU replacement; conduits, handholes, and equipment wiring; instrumentation; control system integration; and testing and commissioning. Replacement of the existing compressed air bubbler sewage level device in the influent flow channels is recommended with industry standard ultrasonic or submersible transducer level monitoring system. Related improvements to the existing controls would also be required.

This project also includes a Storm Water Drainage Study to determine the best course of action for addressing the undersized storm water dewatering sump pumps at the SPS site. Recommended site civil improvements also include asphalt pavement rehabilitation and reconstruction to address cracking, base failures, and regrading existing low point areas and/or depressions. Existing pavement was found to generally be in good condition and only slurry seal is necessary outside of depressed areas.

The total estimated cost for this project is $4,278,000, including engineering design, ESDC, construction management and inspection, and contingency. Some or all of this cost may be avoided with the implementation of any option of the bypass interceptor and replacement of SPS within the next five to ten years.

Table 11-7 presents a summary of all costs for the implementation of the medium priority projects.

Table 11-7: Medium Priority Improvements Cost Estimate (FY 27/28-31/32)

Description	Cost
Castro St between Harpster Dr and Sonia Way	$100,000
Mountain View Ave between Park Dr and El Camino Real	$126,000
Trunk Main CIPP Repairs	$790,000
Shoreline PS Phase 1 Improvements	$2,554,000
Subtotal	**$3,570,000**
Permitting (7%)[1]	$64,000
Engineering Design and Consulting Services (17.5%)	$625,000
ESDC, Construction Management, & Inspection Services (20%)	$714,000
Construction Cost and Market Contingency (30%)	$1,071,000
Medium Priority Total	**$6,044,000**

Notes:
1. Only applied to Mountain View Ave and Trunk Main CIPP projects.

11.3.3 Low Priority Improvements (FY 32/33-41/42)

This phase includes the replacement of mechanical equipment at the Shoreline PS upon reaching the end of its useful life. These improvements would be implemented in the 10- to 20-year timeframe. A summary of the costs for FY 32/33-41/42 is provided in **Table 11-8**.

Shoreline PS Phase 2 Improvements: This project includes the replacement of equipment at the end of its useful life including the replacement of pumps and motors. It is recommended that the existing end suction centrifugal pumps be replaced with modern design close-coupled submersible pumps equipped with semi-open impellers designed for modern sewage applications. The pumps would be installed on a rail system and the existing suction and discharge force main piping would be reconfigured to include isolation valves.

The existing discharge manhole (manhole B4-028, see **Figure 3-9**) currently operates as a break structure, transitioning pressurized sewage discharge to gravity flow. Due to the turbulence, the structure is subject to high levels of H_2S gas. The elevated H_2S gas will promote accelerated corrosion of the concrete structure if not protected from H_2S gas exposure or the turbulent flow is not mitigated. It is recommended that the City evaluate a means of dampening discharge from the pump manifold in order to establish a smooth transition to gravity flow in the discharge manhole.

Microturbines would be replaced along with any other instrumentation that has reached the end of its useful life. All MCCs would also be replaced along with necessary raceways and conductors.

Interior architectural improvements would modernize the SPS facility in accordance with the latest codes and safety requirements. These improvements would include coatings, anti-slip epoxy coatings, stairs and handrails, lighting, doors and hardware, windows, venting, drains, restroom facilities, lockers, showers, etc. These rehabilitations will address outdated safety features and bring the SPS current with any relevant occupational building codes and standards.

The total estimated cost for this project is $3,236,000, including engineering design, ESDC, construction management and inspection, and contingency. Some, or all, of this cost may be avoided with the implementation of any option of the bypass interceptor and replacement of SPS within the next ten to fifteen years.

Table 11-8: Low Priority Improvements Cost Estimate (FY 32/33-41/42)

Description	Cost
Shoreline PS Phase 2 Improvements	$1,932,000
Subtotal	**$1,932,000**
Engineering Design and Consulting Services (17.5%)	$338,000
ESDC, Construction Management, & Inspection Services (20%)	$386,000
Construction Cost and Market Contingency (30%)	$580,000
Low Priority Total	**$3,236,000**

11.3.4 Annual Sewer Main Rehabilitation and Replacement

The City has identified sewer main rehabilitation and replacement for the current five year timeframe (FY 22/23-26/27) and those projects are in active planning, design, and construction. Continued rehabilitation and replacement of sewer main is recommended from FY 27/28 forward. Rehabilitation and replacement of sewer main is driven by CCTV based condition assessment of the collection system. The City currently has a CCTV program in place to document the condition of the sewer collection system and prioritize improvements. The City's current CCTV inspection schedule operates on an eight-year cycle by region for full system CCTV assessment. Budget for ongoing rehabilitation and replacement is allocated through the City's existing non-discretionary Project 23-09. Future allocation for this project is estimated using an annual 2% increase.

11.3.5 Proposed CIP Projects Summary

The proposed CIP projects are summarized in **Table 11-9** and presented on **Figure 11-6**.

Table 11-9: Proposed CIP Projects

Project ID	Priority	Location	Length (ft)	Existing Diameter (in)	New Diameter (in)	Method	Total Cost
-	High	SCADA Master Plan (not shown on **Figure 11-6**)	-	-	-	-	$163,000
1	High	Sondgroth Way/Showers Dr	735	8	15	Replace	$2,715,000
			330	10	15	Replace	
			295	-	15	New	
			780	-	12	New	
2	High	Martens Ave and Alexander Ct	200	8	12	Replace	$176,000
3	Medium	Castro St between Harpster Dr and Sonia Way	220	8	10	Replace	$168,000
4	Medium	Mountain View Ave between Park Dr and El Camino Real	300	8	10	Replace	$220,000
5	Medium	Trunk Main CIPP Repairs	325	39		CIPP	$1,378,000
			1,065	42		CIPP	
6	Medium	Shoreline PS Phase 1 Improvements	-	-	-	-	$4,278,000
6	Low	Shoreline PS Phase 2 Improvements	-	-	-	-	$3,236,000

Notes:
1. Total cost includes permitting (where applicable), engineering design, ESDC, construction management & inspection, and contingency.

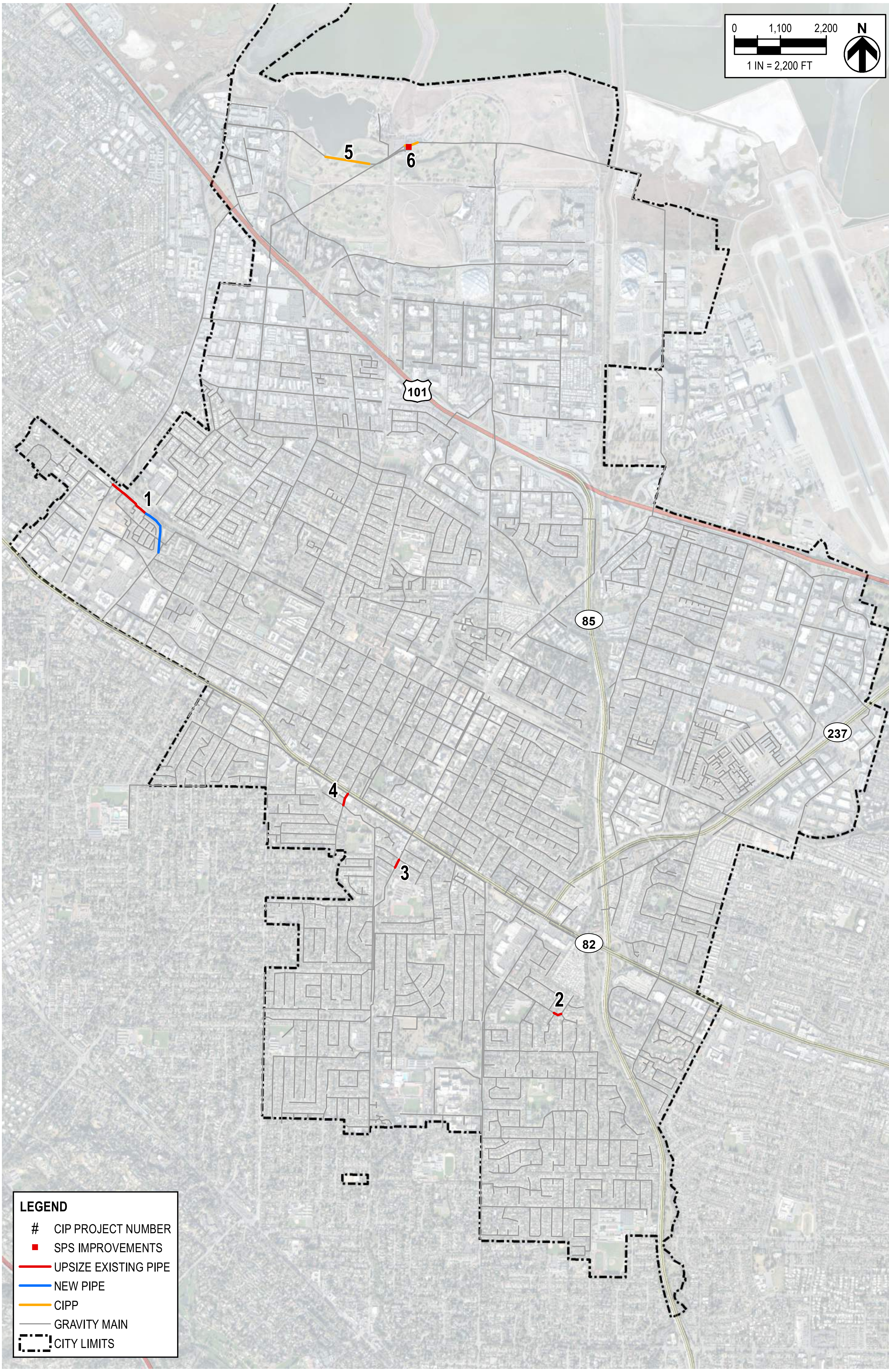


FIGURE 11-6
CITY OF MOUNTAIN VIEW
SEWER MASTER PLAN
PROPOSED CIP PROJECTS

HydroScience

11.4 Proposed Capital Improvement Plan

The Sewer CIP is primarily funded by the Wastewater Enterprise Fund. Provided in **Table 11-10** is a proposed budget and schedule for the recommended CIP projects over the next 20 years. The most critical projects are scheduled for earlier implementation. It is noted that the timing of projects can be adjusted based on operating conditions and available funding.

Table 11-10: Summary of All Current Projects Sourced from Wastewater Fund

Project Type	FY 22/23-26/27	FY 27/28-31/32	FY 32/33-36/37	FY 37/38-41/42
Discretionary	$14,547,000	-	-	-
Amendments	$806,000	-	-	-
Non-Discretionary[1]	$1,229,000	$1,325,000	$1,476,000	$1,620,000
Annual Replacement[2]	$9,107,000	$10,054,000	$11,102,000	$12,255,000
Proposed Improvements				
High Priority	$3,054,000	-	-	-
Medium Priority	-	$6,044,000	-	-
Low Priority	-	-	$3,236,000	-
Total Proposed Projects	**$28,743,000**	**$17,423,000**	**$15,814,000**	**$13,875,000**

Notes:
1. Non-discretionary project budget is based on existing City non-discretionary projects through FY 25/26. Budget for FY 26/27 through FY 41/42 is estimated assuming 2% increases annually consistent with adopted budget allocation.
2. Annual replacement budget is based on existing City Project 23-09 through FY 25/26. Budget from FY 26/27 through FY 41/42 is estimated assuming 2% increases annually consistent with adopted budget allocation.

11.5 Other Recommendations

Interceptor Trunk: It is recommended that the City perform CCTV inspection on the entire length of the Interceptor Trunk from manhole B3-008 (see **Figure 4-3**) to the San Antonio Metering Station to determine the condition of the pipeline. The recently inspected segments are included in the CIP project recommendations for CIPP based on the corrosion observed in CCTV and the recent pipe collapse in the Interceptor Trunk. The entire length of the pipe could be susceptible to collapse and the length of affected pipe should be determined before proceeding with CIPP of the recently inspected segments.

Space Park Way between N Shoreline Blvd and Armand Ave: It is recommended the City verify the inverts of manhole D4-021 in the intersection of N Shoreline Blvd and Space Park Way; the 8-in surcharged pipeline in Space Park Way connected to this manhole may be misrepresented, as the 8-in pipeline connects to the 18-in pipeline in N Shoreline Blvd with matching inverts which is not typically how pipelines are designed and constructed. It is noted that this manhole is part of an upcoming City CIP project to reroute Plymouth St on the west side of N Shoreline Blvd to meet Space Park Way. During this project, the inverts of manhole D4-021 should be verified and updated in the hydraulic model.

Parsons Ave: The City's GIS geodatabase has limited information on the upstream collection system within Moffett Field; as a result, in the hydraulic model, the inverts are interpolated using the minimum design criteria slope of 0.4% and the diameters are assumed to be 8-in per the minimum design criteria pipe size. If the City were able to verify the pipe diameters and inverts of these pipelines, that would increase the accuracy of the hydraulic model.

Observed High Flows: There are three areas of the collection system where the City has observed higher flows than expected during wet weather, including:

- Pipeline connecting Terra Bella Ave to Morgan St, continuing west on Spring St, and crossing under Hwy 101 (see additional detail below);
- Alvin St beginning at Thompson Ave, continuing west to Victory Ave, turning northeast on Victory Ave, crossing under W Middlefield Rd to Rock St; and
- Joaquin Rd between Plymouth St and Charleston Rd and continuing east on Charleston Rd to N Shoreline Blvd.

These areas were evaluated using the hydraulic model and no capacity deficiencies were identified. It is recommended that these areas be CCTV inspected to determine if there are any structural defects contributing to the observed high flow rates.

Inspection Priority: Based on the 2021 Flow Monitoring Report data, Subbasin O (see **Figure 7-7**) shows the highest rates of RDI/I. Specifically, this area shows high rates of R3 – long term delayed rainfall-dependent infiltration (see **Figure 8-7**). Following the March 10, 2021 storm – which was a small storm – flows took over a week to return to their typical values. It is recommended that this area be prioritized for CCTV inspection to determine the cause of the high rates of RDI/I and/or wet weather GWI.

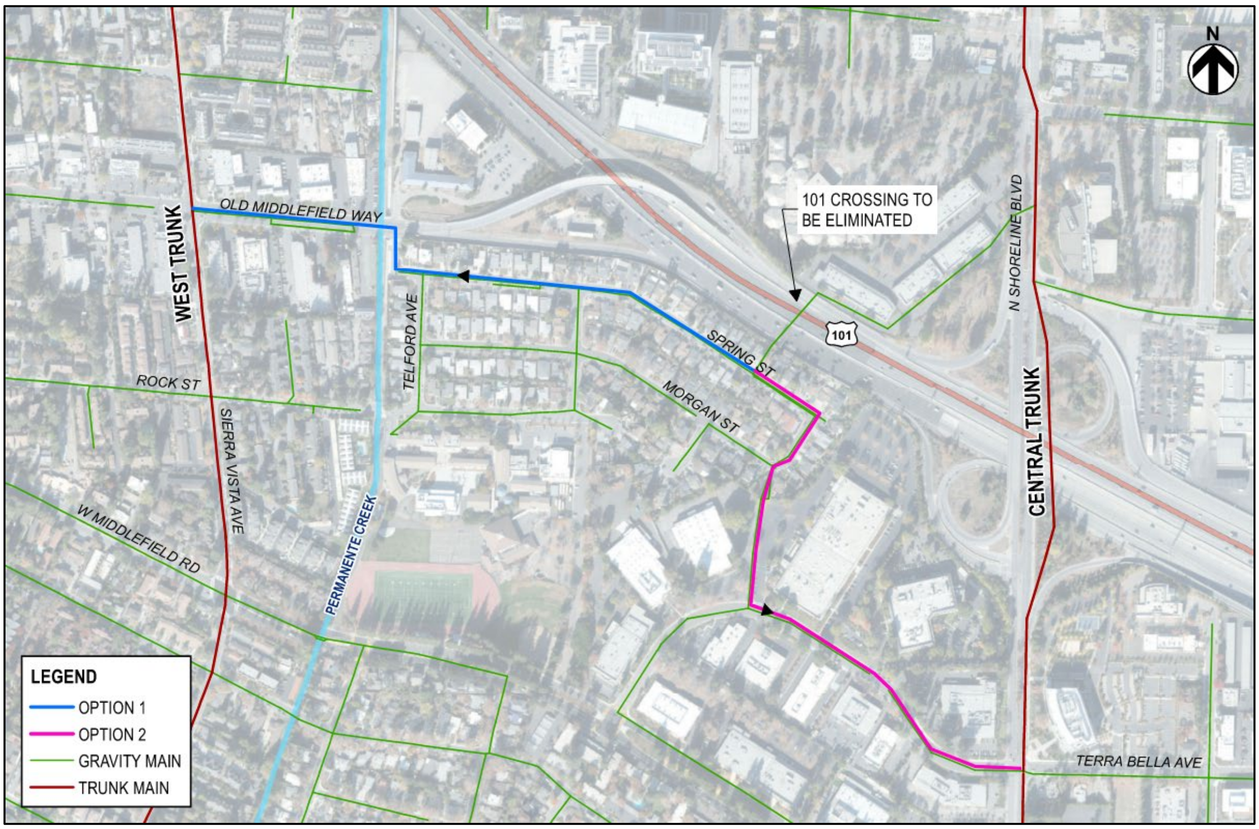
Hwy 101 Crossing near Spring St: The pipeline connecting Terra Bella Ave to Morgan St, continuing west on Spring St, and crossing under Hwy 101 has been observed by the City to have higher flows than expected and does not display any capacity deficiencies according to the hydraulic model. It was communicated by City staff that when CCTV was conducted in this segment, the camera was unable to complete the inspection because of an obstruction in the pipeline. Because this pipeline runs under Hwy 101, it is difficult to maintain due to permitting complications. For this reason, the City has expressed the desire to eliminate this Hwy 101 crossing.

Based on the pipeline inverts in the hydraulic model, there are two options to reroute the flow from this area to a different part of the system to eliminate this crossing, each identified on **Figure 11-7**:

1. **North to West Trunk:** This alignment option connects to the existing system in Spring St near Hwy 101. It is a new pipeline flowing west in Spring St (or replacement of the existing pipeline that currently flows east in Spring St) that turns onto Old Middlefield Way and connects to the existing 27-in West Trunk in Sierra Vista Ave. This option would require an inverted siphon to cross Permanente Creek in Old Middlefield Way. Based on the pipe inverts in the hydraulic model, this option has about 8 ft of elevation drop between the connection points in Spring St and Sierra Vista Ave resulting in a constant slope of 0.003 ft/ft; the City's design standard minimum slope is 0.004 ft/ft. A small lift station along this alignment could mitigate the lack of available slope.

2. **South to Central Trunk:** This alignment connects to the existing system in the same location as option 1, in Spring St. It is also a new pipeline that flows east in Spring St and south through the property south of Morgan St. It turns east in Terra Bella Ave and connects with the Central Trunk in N Shoreline Blvd. This option would require a lift station as the invert at the Central Trunk connection point is approximately 7 ft higher than the connection point in Spring St. Additionally, a gravity pipeline along this alignment would be flowing against the slope of the ground surface; a small force main would likely be a better option for this alignment.

Figure 11-7: Spring St Hwy 101 Crossing Elimination Options



11.6 Funding for CIP Projects

The City uses enterprise funds to account for City operations that are financed and operated like private business enterprises. The City's enterprise utility funds are fully funded by the rates charged to customers; there is no General Fund support to the utility funds. Use of this type of fund permits user charges, including sewer rates and connection fees, to finance or recover the cost of providing the City's services to customers on a continuing basis. The Wastewater Fund is used for operation, maintenance, and capital costs associated with the City's sanitary sewer system and the City's share of the operation and capital costs of the RWQCP. The Wastewater Fund accounts for activities associated with providing wastewater collection services including construction and maintenance of the sewer collection system. The sanitary sewer capital costs are funded with sanitary sewer service charges from ratepayers, and the relatively new funding sources of utility impact fees in North Bayshore and the sanitary sewer capacity charges on new development. A separate reserve is used to account for the capacity and development impact fees collected to fund capital projects. A general reserve is used for emergencies, contingencies, and rate stabilization. Provided is a description of various funds that are conditionally available for project funding:

- **Wastewater Revenue Fund –** Restricted to operation and maintenance of all facilities required to transport and process wastewater. This is a dedicated fund supported by sewer service charges.
- **CIP Reserve –** General Fund surpluses as approved by the City Council and a portion of lease revenues. There are no restrictions on the type or location of projects to be funded.
- **Construction and Conveyance Tax –** Revenues derived from construction and real property conveyance fees. Expenses are restricted to implementation of the CIP, including servicing bonds issued in connection with capital improvements; however, there are no restrictions on the type or location of projects.
- **Shoreline Regional Park Community Fund –** The State Legislature created the Shoreline Regional Park Community (Shoreline Community) Fund. Tax increments derived on the difference between the frozen base year value and the current FY assessed value and other revenues generated from the activities of the Shoreline Community are to be utilized to develop and support the Shoreline Community and surrounding North Bayshore Area. In addition to annual operations and maintenance expenses, the Shoreline Community is used for various types of capital projects, including utility (water, sewer, storm drain) improvements, to support the North Bayshore Area:
- **Development Fees and Charges**
  - Impact Fees: For water infrastructure, uses are restricted to projects/improvements identified in the Shoreline Community Development Impact Fee – Sewer nexus study.
  - Utility Capacity Charges: Used for new or upsized water and sewer utility mains to meet growing service demands Citywide.
  - Community and Public Benefit Funds: A developer may be required by Council under certain conditions to provide community or public benefits, such as area improvements or affordable housing, as a result of their development project. A developer may pay a fee in lieu of providing these community or public benefits which will then be used by the City to provide capital improvements in the general area of the development as approved by the City Council.
- **Equipment Maintenance and Replacement Fund –** The purpose of this fund is to account for centralized fleet maintenance costs and to charge a proportionate share to all funds utilizing maintenance services. In addition, this fund accounts for certain equipment replacement requirements of the City.

Other funding sources may include grant and loan programs for both sewer system reliability and improvements. Funding may be in the form of grants or loans. Notable programs that the City may qualify for include:

- **California Infrastructure and Economic Development Bank**
  - *Public Agency Revenue Bonds:* Bond financings for various State and local government agencies for various public or economic development projects.

- ◦ *Infrastructure State Revolving Fund (ISRF) Program:* The ISRF Program provides direct loan financing to public agencies and nonprofit corporations sponsored by public agencies, for a wide variety of infrastructure and economic development projects. ISRF financing is available in amounts ranging from $50,000 to $25 million with loan terms for the useful life of the project up to a maximum of 30 years. Eligible ISRF applicants include any subdivision of a local government, including cities, counties, special districts, assessment districts, joint powers authorities, and eligible nonprofit corporations.
- ◦ *California Lending for Energy and Environmental Needs (CLEEN) Center:* The CLEEN Center provides direct loan financing to public agencies to help meet the State's goals for greenhouse gas reduction, water conservation, and environmental preservation. Financing can be in amounts from $500,000 to $30 million.

- **California Governor's Office of Emergency Services (Cal OES)**
  - *Building Resilient Infrastructure and Communities (BRIC):* BRIC implements a sustained pre-disaster natural hazard mitigation program to reduce overall risk to the population and structures from future hazard events, while also reducing reliance on federal funding in future disasters. Eligible subapplicants with projects that mitigate risk to public infrastructure, include innovative partnerships, mitigate risk to one or more lifelines, incorporate nature-based solutions, or incentivize adoption and enforcement of modern building codes, are especially encouraged to apply.
- **State Water Resources Control Board (SWRCB)**
  - *Clean Water State Revolving Fund:* This program provides financing for eligible projects to restore and maintain water quality in the State. Eligible projects include planning/design and construction of wastewater and water recycling projects including: wastewater treatment, local sewers, sewer interceptors, water reclamation facilities, nonpoint source (NPS) projects identified in California's NPS plan, estuary projects, stormwater reduction, and treatment facilities.

The scope of the project will dictate funding qualification. As projects are defined, an inquiry can be submitted to the California Financing Coordinating Committee to determine program qualification. **Appendix K** includes a copy of the Common Inquiry Form. Additionally, the California State Library has created the *California Grants Portal*, a website (www.grants.ca.gov/) that provides a centralized location to find State grant opportunities. Grant seekers are now able to see all current grant and loan opportunities that are offered on a competitive or first-come basis and can search and filter their results.

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SECTION 12 – REFERENCES

Provided are a list of sources used in the development of the Master Plan. These documents are incorporated herein by reference.

City of Los Altos, 2013 Sanitary Sewer Master Plan Update, February 2013

City of Los Altos, Los Altos Trunk Sewer Capacity Analysis, June 2020

City of Mountain View, 2010 Sewer Master Plan, August 2010

City of Mountain View, 2014 Dry Weather Sanitary Sewer Flow Monitoring Study, November 2014

City of Mountain View, 2018 Sewer System Management Plan, June 2018

City of Mountain View, 2020 Urban Water Management Plan, June 8, 2021

City of Mountain View, 2021 Wet Weather Sewer Flow Monitoring and Inflow/Infiltration Study, November 2021

City of Mountain View, 2022 Water Master Plan, August 2022

City of Mountain View, Alternative Trunk Sewer Alignment and Constructability Study, March 2017

City of Mountain View, California Operating Budget Fiscal Year 2022-23, June 14, 2022

City of Mountain View, Capital Improvement Program Adopted FY 2021-22, Planned FY 2022-23 through 2025-26, Adopted June 22, 2021

City of Mountain View, Mountain View 2030 General Plan, Adopted July 10, 2012

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