City of Milpitas
Storm Drain Master Plan (Draft)
2021

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List of Abbreviations

Ac	Acres
BMP	Best Management Practices
CIP	Capital Improvement Program
CFS	Cubic Feet per Second
CMP	Corrugated Metal Pipe
CTP	Cooperating Technical Partnership
D/S	Downstream
EPA	Environmental Protection Agency
GIS	Geographic Information System
GPM	Gallons Per Minute
HDPE	High-Density Polyethylene
IDF	Intensity-Duration-Frequency
KW	Kinematic Wave
LiDAR	Light Detection and Ranging
MAP	Mean Annual Precipitation
MSL	Mean Sea Level
NAVD	National Adjusted Vertical Datum of 1988
NGVD	National Geodetic Vertical Datum of 1929
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resource Conservation Service
RWQCB	San Francisco Regional Water Quality Control Board
SDMP	Storm Drain Master Plan
TASP	Transit Area Specific Plan
UH	Unit Hydrograph
U/S	Upstream

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Executive Summary

Milpitas completed its first comprehensive storm drainage master plan in 2001, which was updated in 2013. This effort represents a significant re-envisioning of that updated document and has been undertaken to help guide the City of Milpitas (City) to implement a prioritized capital improvement program. This document represents a new and complete Storm Drain Master Plan (SDMP).

Milpitas has been incorporated for more than sixty years. It is beginning to experience the effects of aging storm drainage infrastructure, the need to maintain and replace expensive equipment and facilities, significant planned and completed redevelopment, and changing regulatory requirements. This SDMP identifies the capital improvements needed to maintain recommended levels of protection against local flooding from stormwater runoff, the need for a revenue stream that will allow the necessary capital improvements made, and the need to keep the storm drain system in working order into the future.

Storm Drainage and Flooding in Milpitas

Flooding within Milpitas is caused by two basic interrelated factors: 1) major creeks and channels that overflow due to limited capacity with flood flow, and 2) inadequate local drainage facilities. Since the operation and maintenance of major creeks and channels are, for the most part, outside the City's control, the focus of this document, therefore, is on local storm drainage collection and pumping facilities owned and operated by the City of Milpitas. Of the major creeks and channels, only Wrigley and Ford Creeks are owned and operated by the City and considered in detail herein.

Urbanization tends to increase the rate of runoff generated from local precipitation. Once primarily agricultural with an economy dominated by fruit and vegetable growers, Milpitas has evolved into a more fully urban community. (Urbanization is generally confined between Coyote Creek to the west and the Calaveras Foothills to the east.) Storm runoff in Milpitas is collected in a system of underground pipes and a network of street gutters. Local runoff flows into creeks and channels that run through the city, ultimately discharging to the San Francisco Bay. Drainage in Milpitas generally is from the southeast to the northwest. Storm drain systems close to the bay also tend to rely heavily upon pumping facilities to move water. Milpitas owns and operates 13 stormwater pumping stations.

Regional Storm Water Coordination

The Santa Clara Valley Water District (Valley Water) is Milpitas' primary partner in managing local stormwater issues. Valley Water's stated mission is to "provide Silicon Valley safe, clean water for a healthy life, environment, and economy." More specifically, Valley Water manages most of Milpitas's major drainage-ways, including Arroyo de los Coches, Berryessa Creek, Calera Creek, Coyote Creek, Lower Penitencia Creek, East Penitencia Creek, Piedmont Creek, and Tularcitos Creek.

Coordination with Valley Water is integral to the Storm Drain Master Plan's success since all the storm drainage systems within the city eventually discharge into a Valley Water-managed facility. Valley Water is keenly interested in any storm drain project that might impact one of their receiving creeks. In turn, Milpitas has a vested interest in how Valley Water manages its legislated flood protection responsibility. This master plan focuses on storm drainage and flood management, which are only two factors in the overall management of stormwater within Milpitas. The Storm Drain Master Plan must address infrastructure needs considering these factors: new capital assets (Capital Improvement Program); finances (utility asset management); operations and maintenance; and regulatory compliance (San Francisco Regional Water Quality Control Board's [RWQCB] Municipal Regional Storm Water Permit, National Pollutant Discharge Elimination System [NPDES] permit).

Basis of System Evaluation

Criteria used to design storm drain systems and evaluate their performance must be defensible yet simple to understand and apply. Ideally, the same criteria used to analyze system performance will also continue for future infrastructure design. Storm drain design criteria set forth by the City of Milpitas, in its July 15, 2010 standards and the Santa Clara County Drainage Manual (2007), is used in this master plan, with some additional provisions as discussed throughout the document.

The ICM model used to evaluate Milpitas' storm drainage system herein was first created as part of FEMA's Cooperating Technical Partnership (CTP) program to study flood hazards within Upper Penitencia Creek, Lower Penitencia Creek, and Berryessa Creek watersheds in San Jose and Milpitas. The basis for analyses described herein is consistent with broader floodplain studies in the area that have already been vetted with Valley Water and FEMA.

Schaaf & Wheeler used data provided by the City and Valley Water and data gathered in the field to construct an integrated hydrologic and hydraulic InfoWorks ICM (Integrated Catchment Modeling) model representing storm drain systems, creeks, and ground surface throughout Milpitas. This model uses a design storm event and land-use-based runoff coefficients to generate runoff from the surface areas tributary to each collection system. The hydraulic capacity of each drainage system component is calculated and resulting overflows to the two-dimensional surface are reviewed to confirm if drainage system performance criteria are being met. If the existing storm drainage system does not meet specific criteria, the model is then used to establish the capital improvement(s) needed so that those criteria are being met upon the completion of a prioritized capital improvement program.

Estimated Capital Costs and Annual Revenue Requirements

A prioritized capital improvement program (CIP) is established based on the analytical evaluation of Milpitas' existing storm drainage system using the integrated ICM hydrologic and hydraulic model, a.

Figure ES–1 shows the locations of city-wide high-priority capital improvement projects. Table ES-1 includes an estimate of the present worth of capital expenditures required to complete those projects shown in Figure ES–1 and provides capital costs for other low priority capital improvements needed to meet established storm drain performance criteria. Table ES-2 provides the estimated annual revenue stream needed to complete the CIP over 20 years assuming a six percent interest rate. This revenue stream includes high priority capital project, long-term equipment replacement, and annual operations and maintenance. Low priority projects that are optional or potential ancillaries to other site development or public projects are not included.

Table ES-1: Capital Improvement Program Costs

Category	Cost
High Priority Storm Drain Projects	$15,820,000
High Priority Pump Station Projects	$25,500,000
Low Priority Storm Drain Projects	$12,430,000
Low Priority Pump Station Projects	$2,150,000
Extension CIP	$510,000
Wrigley-Ford Creek Maintenance	$800,000
Total Budget	$57,210,000

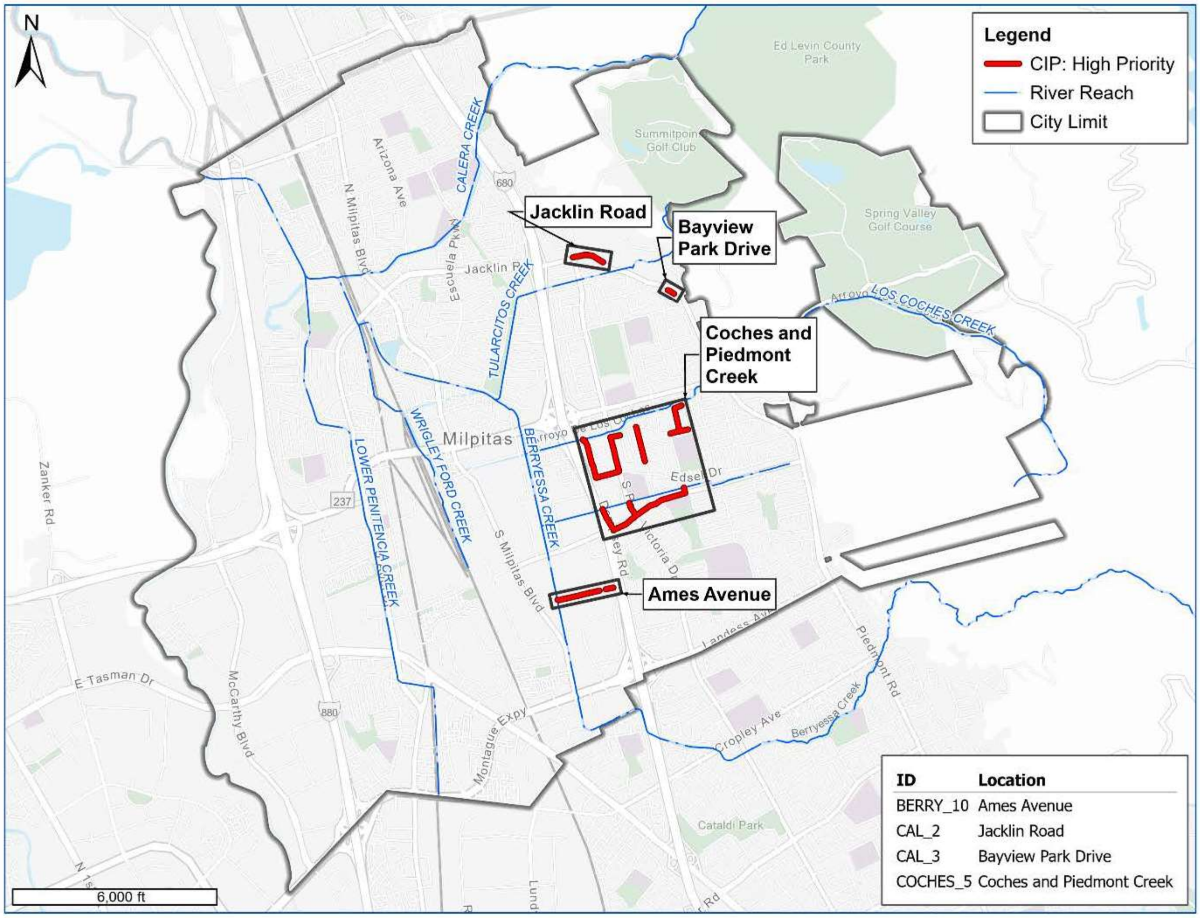


Figure ES-1: High Priority CIP

Table ES-2: Summary of Storm Drainage Budget Requirements

Category	Present Worth	Annualized Cost
High Priority Capital Improvements and Extension CIP	$40,000,000	$3,500,000
Long-Term Equipment Replacement	$13,000,000	$1,500,000
Annual Operations and Maintenance		$2,500,000
Total Budget	$53,000,000	$7,500,000

Work Products

The updated master plan intends to function at several levels. City planners and engineers responsible for capital improvements should find that this document contains sufficient background information and data to serve as a basis for CIP implementation and/or modification. For those City staff and other parties interested in a more in-depth examination of storm drain facilities within Milpitas, the companion InfoWorks ICM model is available.

Comparison to Previous Master Plan

This updated master plan and corresponding CIP differs from previous master plans due to the use of the ICM model, which integrates updated rainfall and a different hydrologic methodology as described in Chapter 2. Additionally, the model accounts for surface storage within streets and other open spaces and the precise timing of coincident creek discharges, which was not directly accounted for in previous master plans. These updates generally result in less flooding at the desired level of service and fewer CIP projects to meet the City's storm drainage criteria. Figure ES-2 depicts the CIP from the 2013 master plan for reference.

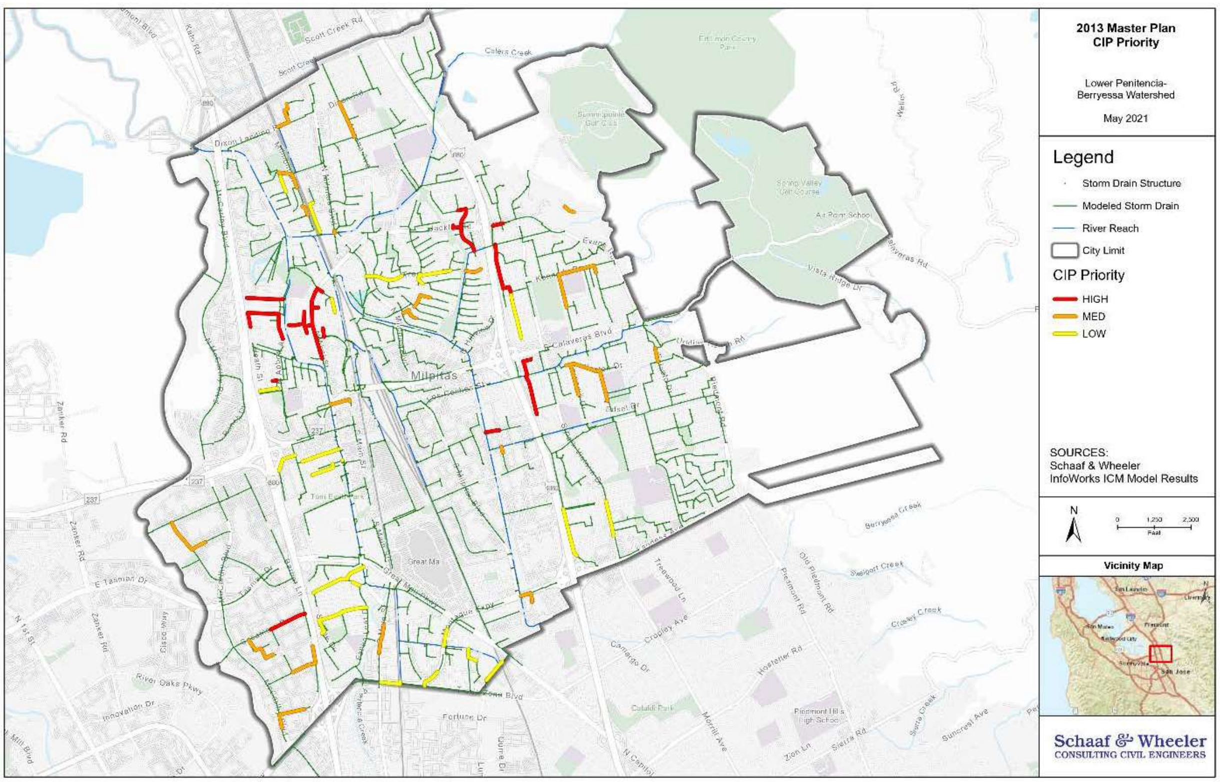


Figure ES-2: 2013 Master Plan CIP Priority

Chapter 1: Introduction

Milpitas completed a comprehensive storm drain master plan in 2001, which was last updated in 2013. This effort represents a substantially new evaluation of Milpitas' storm drain systems and floodplains using tools developed in conjunction with Santa Clara Valley Water District (Valley Water) as part of the Cooperating Technical Partner (CTP) program sponsored by the Federal Emergency Management Agency (FEMA). Under that program, the interaction of storm drains and floodplains within the combined Berryessa Creek and Lower Penitencia Creek watersheds was studied to complete updated flood hazard mapping.

This storm drain master plan update takes advantage of the work completed with Valley Water but adds further detail to comprehensively evaluate storm drain system performance in addition to flood hazard mapping. This master plan focuses on storm drainage infrastructure, while acknowledging that storm drain performance is fully intertwined with floodplain behavior during extreme stormwater runoff events.

This document is a guide for the City of Milpitas (City) to implement a prioritized capital improvement program (CIP) and secure sufficient funds for annual operation and maintenance, and long-term system replacement. This document represents an updated and complete storm drain master plan. Key objectives of this SDMP update include:

- Updating the geographical information systems (GIS) to include pipelines 12 inches and greater in diameter throughout the entire city to reflect all storm drain projects and operational improvements completed by the end of 2020, as well as any changed land uses.
- Utilizing the ICM program to create an integrated hydrologic and hydraulic model that accounts for surface storage and routing and the coincident timing of water surface elevations within the major streams and drainage ways.
- Preparing an updated Capital Improvement Program that remediates identified system deficiencies and provides for underserved areas within the Specific Plan Areas of Milpitas Metro and Main Street Gateway (previously TASP and Midtown).
- Updating projected capital improvement, operations, maintenance, and replacement schedules and costs.

Authorization

Schaaf & Wheeler Consulting Civil Engineers, Inc. prepared this updated Stormwater Master Plan for the City of Milpitas in accordance with the provisions of an agreement executed by the City in November 2019.

Study Area

Milpitas is located near San Francisco Bay in what is colloquially referred to as Silicon Valley. Downtown San José is eight miles to the south; San Francisco is about 45 miles to the northwest. The boundary that separates Santa Clara County from Alameda County also forms the northern border between Milpitas and neighboring Fremont. Incorporated Milpitas encompasses 13.5 square miles, all within the 315 square mile Coyote Creek watershed. Placing Milpitas within its regional context (Figure 1-1) demonstrates that events occurring well outside of the city proper can potentially impact flood risks within Milpitas. However, as stated previously, this master plan focuses on the impacts of events occurring within the city itself.

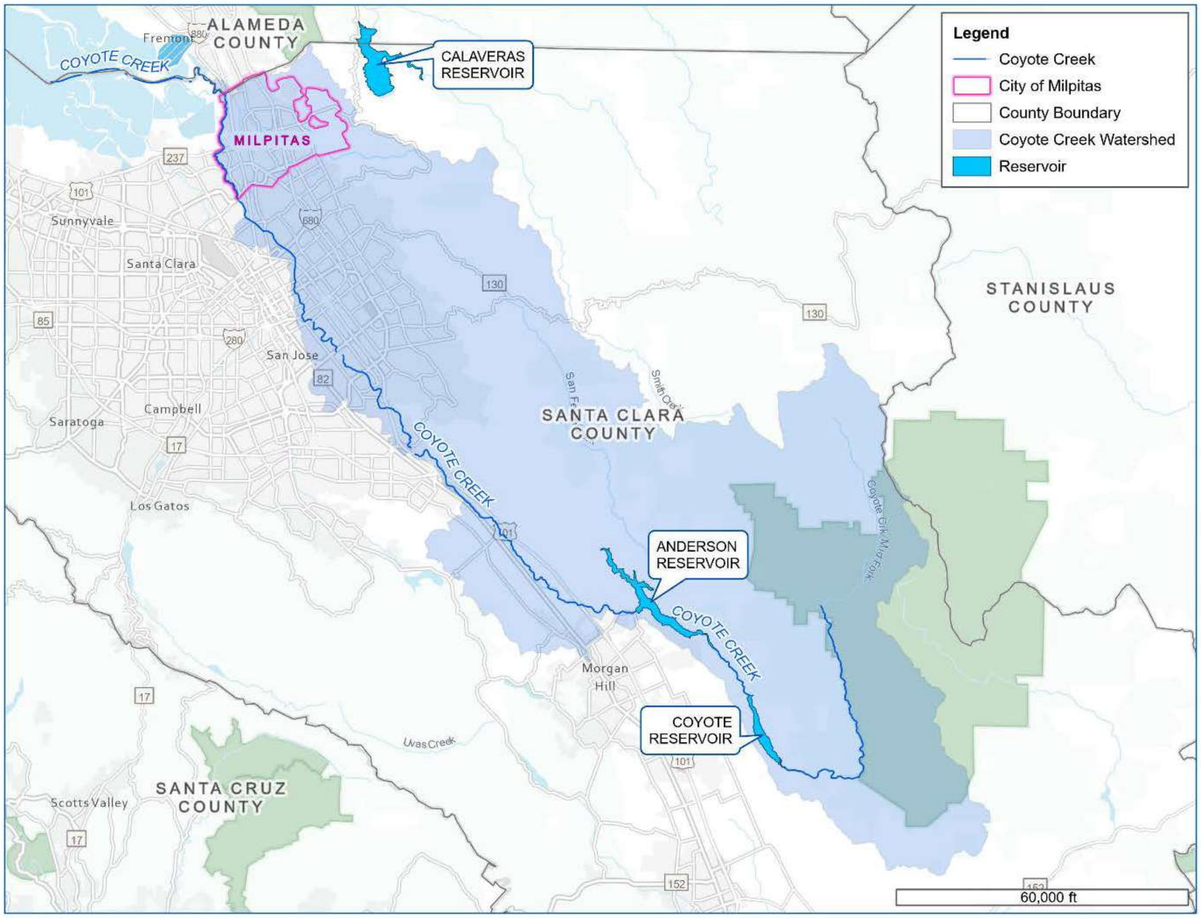


Figure 1-1: City of Milpitas within the Coyote Creek Watershed

Climate

Milpitas has a mild Mediterranean climate with average temperatures ranging from 46°F in the winter to 71°F in the summer. From May to October, there is virtually no chance of precipitation within the area, but winters can be cool and moist. Rainfall is the only significant cause of stormwater runoff (significant snowfall is extremely rare), averaging 14 inches per year near the bay, up to 18 inches annually near the eastern ridgeline.

Most precipitation events in the Milpitas area are either orographic when moist air is lifted over the hills, then cools and condenses, or cyclonic, where rain is with air masses' movement from higher barometric pressure regions to lower pressure. Cyclonic events can also be caused by frontal activity. Warm fronts are generally associated with broad bands of relatively low-intensity rainfall, while higher rainfall intensifies typify cold fronts. Convective precipitation (e.g., thunderstorms) caused by air heating at the ground often leads to too intense localized storms, but it is not common in this area.

Physiography

The city lies at the Diablo Range base, extending from its foothills on an alluvial plain of the Santa Clara Valley toward San Francisco Bay. Almost half of the city is east of Interstate 680, where elevations vary from about 40 feet mean sea level (MSL) at Evans Road to nearly 800 feet at Monument Peak just west of Calaveras Reservoir. Once on the valley floor, the land falls away from the hill's base toward the west and approaches sea level along the bay.

Soil deposits on the valley floor are characteristic of alluvial fan development. Calera, Tularcitos, Los Coches, and Berryessa Creeks deposited older fans of coarse sand and gravel at the foothill's base.

Throughout the city center, younger clays deposited between the creeks are interspersed with smaller amounts of old San Francisco Bay mud. At the western limits of Milpitas, Coyote Creek deposits are found along the edge of alluvial fan deposits from Lower Penitencia Creek. Most of the soil within Milpitas is either clay or clayey loam with very low infiltration rates when wetted and has a high runoff potential. At the western city limits near Coyote Creek, some soil is loamier with better infiltration characteristics and a moderate to high runoff potential.

Land Development and Drainage Characteristics

Urbanization tends to increase the rate of runoff generated from local precipitation. Once primarily agricultural with an economy dominated by fruit and vegetable growers, Milpitas has evolved into a more fully urban community. Urbanization is generally confined between Coyote Creek to the west and the Calaveras Foothills to the east. Although some selected hillside development is allowed in the General Plan, the hillside area (which comprises almost one-half of the city) is generally zoned for permanent open space, including the Ed Levin Regional Park. The western one half of the city has developed as a mix of residential, commercial, and industrial development, with parks, schools, and greenbelts woven into the urban fabric. Future growth in Milpitas, particularly non-hillside residential will tend to be infill development which will become denser as property values escalate.

Recent land-use changes and growth are concentrated within the Main Street Gateway and Milpitas Metro areas. Therefore, storm drain systems serving these tributary areas are the most potentially impacted by new development.

A system of underground pipes and a network of street gutters collect storm runoff in Milpitas. Local runoff flows into creeks and channels that run through the city, ultimately discharging to San Francisco Bay. Drainage in Milpitas generally is from the southeast to the northwest. Storm drain systems closer to the Bay also tend to rely heavily upon pumping facilities to move water.

Concurrent City Planning

The City of Milpitas has concurrent planning efforts focused within the Main Street Gateway and Milpitas Metro Specific Plan areas. These specific plans focus on the new BART station and downtown areas where infill development has occurred or is planned. Generally, impervious surface does not increase with infill development, so the impacts of the specific plan areas would be based on realigned roads or identifying currently underserved areas where parcels drain by gravity to the street frontage. Improvements associated with these underserved areas are described herein. At the time of this study, plans for new or re-routed city roadways had not been developed.

The City has developed a green stormwater infrastructure (GSI) plan as required under the NPDES Municipal Regional Permit Section C.3.j. The plan relies on information produced in the Santa Clara Basin Stormwater Resource Plan. Opportunities to combine green infrastructure with capacity projects are considered in the stormwater improvements identified in this study.

Work Products

As discussed in subsequent sections, the following information is available via GIS:

1. ***Inventory of Drainage Facilities.*** Information pertaining to each system component may be accessed graphically through GIS or numerically using the ICM model.
2. ***Tributary Drainage Areas.*** Land areas used to generate local runoff are available in GIS and tabular format with tributary areas, runoff coefficients, and concentration times.
3. ***Ground Surface.*** Ground surface information based on Countywide 2006 LiDAR, updated with available modified ground surface data from developments constructed between 2006 to 2018, is available in raster format.

Chapter 2: Methodologies

Criteria used to design storm drain systems and evaluate their performance must be defensible yet simple to understand and apply. Ideally, future infrastructure design will use the same criteria used to analyze system performance. As discussed in this chapter and the next, storm drain design criteria set forth by Milpitas in its July 15, 2010 standards and the Santa Clara County Drainage Manual (2007) are used in this master plan; some additional provisions as discussed herein.

An integrated hydrologic and hydraulic InfoWorks ICM model representing storm drain systems, creeks, and ground surface throughout Milpitas was constructed using data from the City and Valley Water and gathered in the field. This model uses a design storm event and land-use-based runoff coefficients to generate runoff from the surface areas tributary to each collection system. The hydraulic capacity of each drainage system component is calculated and resulting overflows to the two-dimensional surface are reviewed to confirm whether drainage system performance criteria are met. If the existing storm drainage system does not meet specific criteria, the model is then used to establish the capital improvement(s) needed so that those criteria are completed based on the capital improvement priority system described in Chapter 3.

Data Sources

The comprehensive master plan model was built upon an integrated hydrologic and hydraulic ICM storm drain system model previously created by Schaaf & Wheeler and Wood Rodgers under a Cooperating Technical Partnership (CTP) between Valley Water and FEMA. The CTP model built is based on as-built plans, field surveys, LiDAR and aerial surveys, photos, improvement plans, other data documents, and field investigations. The model has been calibrated to the design storm and validated with stream gauge data of historical events.

For the comprehensive master plan, Schaaf & Wheeler updated the CTP model with data provided by the City of Milpitas in the form of GIS shapefiles of the storm drain network received 04/02/2020. The purpose was to incorporate changes in the storm drain network and include pipes 12 inches in diameter or greater. The CTP model only had pipes 18 inches in diameter or greater. To fill in any new, missing, or conflicting information, Schaaf & Wheeler consulted record drawings for street improvements or tracts as needed. All elevations have been converted to the National Adjusted Vertical Datum of 1988 (NAVD) to match the most currently available LiDAR-based citywide topography.

The most common data transformation involves the conversion of the National Geodetic Vertical Datum of 1929 (NGVD):

NGVD +2.78 feet = NAVD (88).

Sub catchment parameters of the model are based on the current land use data provided by the city in form of GIS shapefile called "ZonedParcels2012". This is most current land use information available to characterize existing land surfaces. Aerial maps have been used to assign land use for areas not defined by the city parcel data and for recent development since 2012. Future zoning information could be used to analyze land use changes, but future land uses almost uniformly have lower percentages of impervious surface. This master plan does not rely on potential reductions in runoff that may not come to fruition.

Information regarding pump station operation has been obtained from record drawings and information from the 2013 master plan update, corroborated by conversations with City operations and maintenance staff in 2020.

Modeling Software

INFOWORKS ICM

InfoWorks ICM by Innovyze is a GIS-based, integrated modeling platform that incorporates both urban and river catchments. The integration of 1D and 2D hydrodynamic simulation techniques allows the modeling of both the above- and below-ground elements of catchments to represent all flow paths accurately.

Hydrology

Design Storm

Flood frequency analyses are used to design facilities that control storm runoff since it is impossible to anticipate every conceivable storm's effect. A common practice that both the Milpitas and Santa Clara County standards follow is constructing a design storm. A rainfall pattern is used in hydrologic models to estimate surface runoff – and compare the surface runoff to the capacity of drainage systems designed to convey this runoff to major facilities outside of the City's jurisdiction.

Precipitation-runoff frequency analyses are based on concepts of probability and statistics. Engineers generally assume that a rainfall event's frequency (probability) coincides with direct stormwater runoff frequency. However, the runoff generation depends on several factors (particularly antecedent moisture conditions in the drainage basin) not necessarily dependent upon the precipitation event.

The 10-year storm recurrence interval is used as the design storm to evaluate the flood control systems for this master plan. It is worth noting that over the typical 30-year life of a home mortgage, the chance of experiencing at least one 10-year event is about 96 percent.

The 100-year storm recurrence interval is used as the design storm to evaluate if pump stations have sufficient capacity. A 100-year design storm is assumed to evaluate pump stations as critical facilities where gravity conveyance is not possible, and flooding is likely to remain for extended periods.

Design Storm Duration

A 24-hour storm duration determines the governing design storm event and the conservative operational conditions of the City's drainage facilities. This storm duration is for the following reasons:

1. The 24-hour duration is a standard for many local, state, and federal agencies.

2. The 24-hour duration is short enough to be consistent with the watershed size and long enough to create volume-induced flood problems in the watershed. The 24-hour storm duration generally results in the most extensive floodplain relative to other storm durations within the local historical record.

Design Storm Rainfall Depths

Design rainfall depths are calculated using the TDS Regional Equation provided by Valley Water:[1]

[1] SCVWD 2013: Precipitation Gage Data and Depth-Duration-Frequency Analysis. Revised from Saah et al. 2004

$$P_{f,d} = A_{f,d} + B_{f,d}(MAP) \qquad \text{Equation 1}$$

Where:

$P_{f,d}$ Precipitation depth for a given storm frequency, f and duration, d, in inches

$A_{f,d}$ & $B_{f,d}$ Regression constants and coefficients

MAP Mean annual precipitation, in inches

Af,d and Bf,d values used are displayed below in Table 2-1 and Table 2-2, which are provided by Valley Water as well.

Table 2-1: $A_{f,d}$ and $B_{f,d}$ Values for Design Rainfall Depth Equation

Frequency	Recurrence Interval (Yr)	$A_{f,d}$	$B_{f,d}$
		24 Hr	24 Hr
10%	10	0.0028	0.1653

The resultant design rainfall depths from Equation 1 are displayed in Table 2-2.

Table 2-2: 24-hour Duration Design Storm Depth per MAP

MAP (in)	24-hour Design Rainfall Depth (in)
	10-Year
12	1.99
13	2.15
14	2.32
15	2.48
16	2.65
17	2.81
18	2.98
19	3.14
20	3.31
21	3.47

The resultant design rainfall depths from Equation 1 are displayed in Table 2-3.

Table 2-3: 24-hour Duration Design Storm Depth per MAP

MAP (in)	24-hour Design Rainfall Depth (in)							
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	200-Year	500-Year
12	1.26	1.72	1.99	2.30	2.51	2.71	2.91	3.15
13	1.37	1.86	2.15	2.49	2.72	2.93	3.14	3.40
14	1.48	2.01	2.32	2.67	2.92	3.15	3.37	3.65
15	1.59	2.15	2.48	2.86	3.12	3.37	3.60	3.89
16	1.70	2.30	2.65	3.05	3.32	3.58	3.83	4.14
17	1.81	2.44	2.81	3.24	3.53	3.80	4.06	4.39
18	1.92	2.59	2.98	3.42	3.73	4.02	4.29	4.63
19	2.03	2.74	3.14	3.61	3.93	4.23	4.52	4.88
20	2.14	2.88	3.31	3.80	4.14	4.45	4.75	5.13
21	2.25	3.03	3.47	3.99	4.34	4.67	4.98	5.38

Design Storm Temporal Distribution

The 24-hour design storm temporal distribution obtained from Valley Water is displayed as Figure 2-1 below. The temporal rainfall distribution is for a 24-hour design storm with 15-minute intervals.

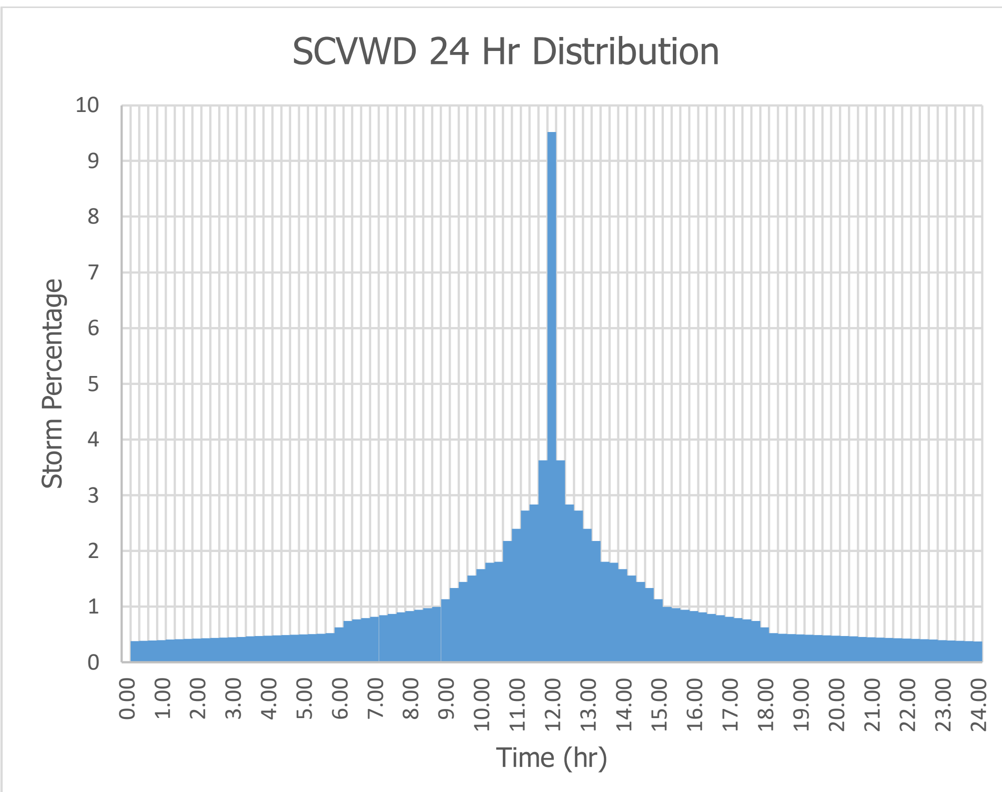


Figure 2-1: Valley Water 24hr Storm Distribution

Historical Storm

Schaaf & Wheeler used the December 11, 2014, a historical event, to validate the Lower Penitencia-Berryessa and Upper Penitencia watershed Model as part of the CTP program. This storm produced 1.2 inches of rain in 3.0 hours, 2.0 inches of rain fell in 12.0 hours, and 3.7 inches of rain fell in 24 hours period in San Jose. The estimated storm recurrence interval based on these depths varies from a 10-year 3-hour, a 5-year 12-hour, and a 100-year 24-hour interval, respectively, for a MAP of 18 inches. This 18-inch MAP is the average for Lower Penitencia-Berryessa and Upper Penitencia watersheds. The December 2014 storm event was one of the most intense urban rainfall events in recent memory, surpassing the wet year of 2016-2017.

It should be noted that based on the response time of the Lower Penitencia-Berryessa and Upper Penitencia watershed (approximately 1 hour), the durations between 3 hours and 12 hours are the most relevant for this study.

In addition, the February 16-20, 2017 event is used to validate the model. Approximately 1.2 inches of rain fell over 3 hours, and 2.7 inches of rain fell in 24 hours. The estimated storm recurrence interval based on these depths varies from a 2-year, 3-hour storm to a 2-year, 24-hour storm.

Rainfall/Runoff Transformation Method

Described below is the methodology for the transformation of the precipitation into the stormwater runoff. The general steps to transform rainfall into runoff are:

1. Apply a loss method to convert rainfall distributions into excess rainfall. The method is done by accounting for the portion of rainfall from the sky lost to surface depressions, evaporation, and soil infiltration. The amount of precipitation that is lost will not result in direct runoff. Losses also vary over time during a storm. For example, as wetted soil becomes more saturated, losses decrease, and more rainfall becomes surface runoff. Losses are a function of land use and soil conditions.

2. Transform the excess rainfall into surface runoff using the hydrograph methods subsequently described.

3. Route surface runoff hydrographs through the storm drain and creek systems. Where stormwater flows exceed a storm drain or creek's hydraulic capacity, some portion of the runoff hydrograph will be carried over the ground surface. The timing and depth of this overland flow produce flood hazard mapping.

Hydrograph Method

The transformation of rainfall into runoff can be calculated in a model using various methods. In the detailed regional calibration efforts for smaller urban watersheds that were conducted with Valley Water, Alameda County Public Works Agency (ACPWA), and the City of San Jose, Valley Water it concluded that hydrographs using the Kinematic Wave (KW) method best match recorded flows for the smaller urban watershed (less than 100 acres). For larger, rural, and hilly watersheds, hydrographs using the Snyder Unit Hydrograph (Snyder UH) match recorded flows well. Still, hydrographs using KW methods diverged from those developed using the Snyder UH and the recorded gauge data.

Therefore, the KW method is used for smaller urban watersheds less than 50 acres in size only, and the Snyder UH method is used for all other catchments within the study area.

The combination of the small urban and large rural or open space watersheds in the Lower Penitencia-Berryessa and Upper Penitencia Watershed drainage systems warrant both KW and Snyder's use of UH methods as described above.

Loss Method

The Horton Loss Rate Method reflects the effects of infiltration in the model because of its initial loss decay and recovery features. The method is also well documented by the Environmental Protection Agency (EPA) and has been used successfully throughout Alameda County, Redwood City, and other Bay Area agencies. Table 2-4 lists the reasonable range of parameters recommended by the EPA.

Infiltration rate ranges from Alameda County (see Table 2-5) are also considered and used in the calibration because of the proximity and geologic similarity to the Lower Penitencia-Berryessa and Upper Penitencia watersheds. These parameters have been calibrated for Alameda County and reflect local soil characteristics in Milpitas.

Horton Loss Equation:[2] $f_p = f_c + (f_0 - f_c)\ e\text{^}{-\beta t}$

where:

f_p = Infiltration Capacity

f_c = Constant Rate

f_0 = Maximum Infiltration Rate

β = Decay constant

Table 2-4: Horton Loss Equation Parameters[3]

Hydrologic Soil Group	Max. Infiltration Rate Range (in/hr)	Min. Infiltration Rate Range (in/hr)	Decay Const. Range (1/hr)	Drying Time Range (days)
A	5-10	>=0.45	2-7	2-14
B	4-8	0.30-0.15	2-7	2-14
C	3-6	0.15-0.05	2-7	2-14
D	1-2	0.00-0.05	2-7	2-14

[2] Handbook of Hydrology, David R. Maidment, 1993

[3] Storm Water Management Model User's Manual, Version 5.0, EPA, Revised July 2010

Table 2-5: Alameda County Loss Parameters [4]

Hydrologic Soil Group	Min. Infiltration Rate Range (in/hr)
A	>=0.45
B	0.35-0.40
C	0.14-0.25
D	0.05-0.09

The Horton Loss Rate Method can be used for both single-event design storms and continuous simulations with multiple intermittent storms because the initial loss capacity can be recovered. Antecedent soil moisture conditions and the soil storage saturation level before a storm were analyzed before determining the continuous storm simulation's appropriate initial loss rate.

Imperviousness

The percentages of directly connecting impervious surface, non-directly touching impervious surface, and porous surface for each land-use shown in Table 2-6 are determined by sampling five areas of the same land use and identifying the average percentage for each surface type. Directly connected impervious surfaces drain the city storm drain system with limited surface attenuation, such as driveways, street pavements, and sidewalks. Non-directly is connecting impervious areas within a sub-basin experience more peak flow attenuation by flowing across pervious surfaces before entering the storm drain. These are mostly roofs that are collected by gutters and discharge to previous lawns.

Table 2-6: Land Use Percent Impervious

LAND USE TYPE	Impervious (%)		Pervious (%)
	Directly Connecting	Non-Directly Connecting	Area
Hillside Very Low Density (HVL): up to 1 unit/10gross acres	2	2	96
Hillside Low Density (HDL): up to 1 unit/gross acre	5	6	89
Hillside Medium Density (HMD): up to 3 units/gross acre	9	37	54
Single Family Low Density (SFL): 3-5 units/ gross acre	17	59	24
Single Family Medium Density (SMD): 6-15 units/gross acre	20	56	24
Multi-Family Residential Medium Density (MFM): 7-11 units/gross acre	5	58	37
Multi-Family Residential High Density (MFH): 12-20 units/gross acre	33	37	30

[4] Alameda County Hydrology and Hydraulics Manual, ACPWA, 2003

LAND USE TYPE	Impervious (%)		Pervious (%)
	Directly Connecting	Non-Directly Connecting	Area
Multi-Family Residential, Very High Density (VHD): 31-40 units/gross acre	52	34	14
Urban Residential (URR): 41-75 units/gross acre	32	48	20
Mobile Home Park (MHP)	22	72	6
Residential Retail High Density Mixed Use (RRMU)	24	64	12
Boulevard Very High-Density Mixed Use (BVMU)	40	43	17
General Commercial (GNC)	65	24	11
Town Center (TWC)	56	36	8
Industrial Park (INP)	46	23	31
Public Facilities (PF)	32	30	38
Parks and Open Space (POS)	7	3	90

Hydrograph Method Parameters

Kinematic Wave

Based on calibration efforts conducted by the ACPWA that were reviewed with the City of San Jose, the KW method was selected for this project as the hydrologic transformation method for small urban sub-basins up to 50 acres in size.

The use of the KW method requires developing watershed parameters that result in consistent, reproducible results; because of the need to simplify the models enough to meet the software, hardware, and data management constraints, applying the KW method to watersheds that vary in size from about 3 acres to 50 acres. Therefore, some modification of KW method parameters is necessary, noting parameters have been carefully vetted during the calibration process to confirm their validity.

For example, the KW method requires a representative watershed "plane" width (Figure 2-6), overland flow length, and slope. In small watersheds, these planes would represent a row of residential lots draining to the street. In larger watersheds, the planes would represent a group of lots and streets draining to a central conveyance. In most cases of larger watersheds, the plane width was assumed to be twice the central conveyance's length, thus representing two planes draining to the central conveyance from each side.

Since this approach (two planes draining to each side of the central conveyance) does not explicitly account for the routing of the runoff in the central conveyance, the resistance parameters used have been carefully calibrated to account for this. They are expressed as overland flow roughness factor, N, developed from the *HEC-1 User Manual*, 1990, Table 12.1. (Table 2-7). Watersheds are broken down into small sub-catchments with a single plane as much as possible to reduce the impact of this simplification.

The KW method parameters include shed area, flow length, watercourse slope, and percent imperviousness. In contrast, the Horton Loss Rate parameters include maximum infiltration losses (initial loss rate) and minimum infiltration capacity (uniform loss rates).

Table 2-7: Kinematic Wave Method Resistance

Surface	N value	Source
Asphalt/Concrete	0.05-0.15	Harley (1975)
Bare Packed Soil Free of Stone	0.10	Hathaway (1945)
Fallov - No Residue	0.008-0.012	Engman (1986)
Convential Tillage - No Residue	0.06-0.12	Engman (1986)
Convential Tillage - With Residue	0.16-0.22	Engman (1986)
Chisel Plow - No Residue	0.06-0.12	Engman (1986)
Chisel Plow - With Residue	0.10-0.16	Engman (1986)
Fall Discing - With Residue	0.30-0.50	Engman (1986)
No Till - No Residue	0.04-0.10	Engman (1986)
No Till (20-40 percent residue cover)	0.07-0.17	Engman (1986)
No Till (60-100 percent residue cover)	0.17-0.47	Engman (1986)
Sparse Rangeland with Debris: 0 Percent Cover	0.09-0.34	Engman (1986)
Sparse Rangeland with Debris - 20 Percent Cover	0.05-0.25	Engman (1986)
Sparse Vegetation	0.053-0.13	Woolhiser (1975)
Short Grass Prairie	0.10-0.20	Woolhiser (1975)
Poor Grass Cover on Moderately Rough Bare Surface	0.30	Hathaway (1945)
Light Turf	0.20	Harley (1975)
Average Grass Cover	0.40	Hathaway (1945)
Dense Grass	0.17-0.30	Palmer (1946)
Bermuda Grass	0.30-0.48	Palmer (1946)
Dense Shrubbery and Forest Litter	0.40	Harley (1975)

Snyder Unit Hydrograph

Based on calibration efforts conducted by the ACPWA that were reviewed with the City of San Jose, the Snyder UH method is selected as the hydrologic transformation method for large sub-basins greater than 50 acres in size. The Snyder UH method is especially appropriate in open space because of the proper application as described previously.

Snyder UH Equation:[5] $Q_p = 640 * C_p * A / t_L$

Where:

Q_p = Peak discharge of the unit hydrograph (cfs)

[5] Handbook of Hydrology, David R. Maidment, 1993

C_p = storage coefficient / peaking factor

A = Drainage Area (sq mi)

t_L = basin lag time (hr)

The selected Snyder UH method is the ACPWA revised version of the standard Snyder UH documented in HEC-1 and HEC-HMS. ACPWA revised the method based on historical calibration efforts performed in Alameda County, which has similar meteorological characteristics as Santa Clara County.

The two major input parameters of the method are Basin Lag Time and Basin Peaking Factor. The Basin Lag Time (see Equation 13) is based on the calculated Basin Roughness (see Equation 12) using the main watercourse Manning's *n* within a sub-basin.

TABLE 6 BASIN ROUGHNESS FACTORS FOR RURAL WATERSHEDS

Basin Type	Basin Roughness Factor (N)
1. Rural watersheds with generally clear stream bed and minimal vegetation growth in the drainage reaches.	0.05
2. Rural watersheds with moderate to high levels of vegetation growth, or rock and boulder deposits within the main drainage reaches.	0.07
3. Rural watersheds with dense vegetation or high levels of boulder deposits within the main drainage reaches.	0.08

(District 2003)

EQUATION 12 BASIN ROUGHNESS FACTOR FOR URBAN WATERSHEDS

$$N = 0.3318n^{0.6328} \quad (12)$$

where:

N = *basin roughness factor*

n = *Manning's roughness coefficient (from Table 7)*

(District 2003)

Figure 2-2: Basin Roughness (N) (ACPWA H&H Manual)

TABLE 7 MANNING'S ROUGHNESS COEFFICIENT	
Type of Facility	n
Reinforced Concrete Pipe	
Conduit > 36" diameter	0.012
Conduit ≤ 36" diameter	0.014
Corrugated Metal Pipe	
Annular	0.021
Helical	0.018
Concrete-Lined Channels	
Smooth-troweled	0.015
District Simulated Stone	0.017
Reinforced Concrete Box	
Cast-in-Place	0.015
Pre-Cast	0.014
Earth Channels	
Smooth Geometric	0.030 – 0.035
Irregular or Natural	0.045 – 0.050

(Chow 1959, District 1989)

EQUATION 13 BASIN LAG TIME

$$t_L = K \cdot N\left(\frac{L \cdot L_c}{\sqrt{S}}\right)^{0.38} \qquad (13)$$

where:

t_L = *lag time (hr)*
K = *distance factor*
for $L > 1.7$ *mi,* $K = 24$
for $L \leq 1.7$ *mi,* $K = 15.22 + 2.15L + 8.7/L$
if calculated value of K *is greater than 30, use* K=30
N = *basin roughness factor (from* ***Table 6****, rural watershed or* ***Equation 12****, urban watershed)*
L = *length of longest watercourse (mi) (see* ***Figure 4****)*
L_c = *length along longest watercourse measured from the point of concentration to a point opposite the watershed centroid (see* ***Figure 4****)*
S = *average stream slope (ft / mi from* ***Equation 11****)*

(District 1994)

Figure 2-3: Manning's n and Basin Lag Time (ACPWA H&H Manual)

The peaking factor is a function of overland basin storage. Large areas with flat slopes are associated with relatively high amounts of overland basin storage. Conversely, water that falls on steeply sloped areas will run off quickly, with little overland basin storage. The lower the basin storage, the higher the corresponding peaking factor.

EQUATION 14 BASIN PEAKING FACTOR

$$C_p = 0.6e^{0.06(S_o/A)} \qquad (14)$$

where:

C_p = *basin peaking factor* ($C_p \leq 0.85$)
S_o = *average watershed slope (percent from* ***Attachment 9****)*
For $S_o \leq 5\%$, $C_p = 0.6$
A = *drainage area (mi²; if* $A < 5mi^2$, *use* $A = 5mi^2$)

(District 1994)

Figure 2-4: Basin Peaking Factor (ACPWA H&H Manual)

Watershed Characteristics

To model the stormdrain system and include all pipes 12 inches in diameter and larger, relatively small sub-watersheds have been developed for representation in the models.

The area that contributes runoff to a drainage line within these watersheds is referred to as a "Drainage Area". The smaller sub-watersheds within the Drainage Areas are noted as "Sub-Basins".

Therefore, the hierarchy terminology for watersheds includes:

1. Watersheds – delineating the basin for each major creek system
2. Drainage Areas – delineating the basin for each named drainage line
3. Sub-Basins – delineating the smallest sub-basin for each drainage line

Watershed Size

Sub-Basins are delineated based on the 2006 topographic LIDAR mapping furnished by Valley Water and the County of Santa Clara, City of San Jose and Milpitas collector system locations, and overland release flow paths. The Sub-Basin size ranges from 0.007 acres in densely developed urban areas to approximately 2,200 acres in undeveloped hilly areas.

The Sub-Basin size ranges from 0.007 acres in densely developed urban areas to approximately 2,200 acres in undeveloped hilly areas.

Watershed Delineation

Sub-Basins are also delineated based on stream channels and storm drain networks. In the Sub-Basins where the overland flow path and the storm drain network flow path conflicted, the storm drain flow path usually governs, as it usually conveys the most flow.

ESRI ArcMap Hydrology tools condition the input terrain to include underground pipes and channels. This approach ensures that the hydraulically connected pipe systems not modeled (lateral pipes smaller than 12 inches) can be routed to the modeled pipes hydrologically. The conditioned terrain creates sub-basins automatically. Sub-Basins are then further divided to model pipes 12 inches and greater. This approach provides consistent, reproducible watershed delineation results.

Basin Width

For sub-basins using the KW method, a representative watershed "plane" width needs to be determined. Each sub-basin is classified as either a 2 plane KW or 1 plane KW, shown in Figure 2-6, based on the distance (d), which is the shortest distance between the sub-basin centroid to the longest flow path. If d is relatively small, then the sub-basin is classified as a 2 plane KW, assuming it is the longest flow path runs relatively down the center of the sub-basin; thus, there is 1 KW plane draining from each side of the longest flow path. If d is relatively large, the sub-basin is classified as a 1 plane KW, assuming it is the longest flow path is relatively located along the sub-basin side; thus, there is only 1 KW plane draining into the longest flow path from only one side.

An algorithm can classify the sub-basins automatically. The following calculations are performed using the centroid flow path (L_c), which is the length from sub-basin centroid to the sub-basin drainage node along the longest flow path.

Assume 2 plane KW – width (W_2),

$$W_2 = 4L_c$$

Assume 1 plane KW – width (W_1),

$$W_1 = 2L_c$$

The minimum of |d-0.33W_1| and |d-0.33W_2| controls whether 1 plane or 2 plane KW, and the appropriate width calculated will be used as the representative watershed "plane".

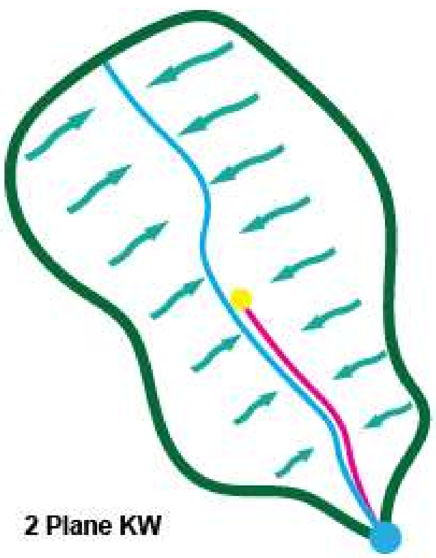


Figure 2-5: KW Planes

Watershed Soils

The National Resource Conservation Service (NRCS) soil survey and map information identifies soils in hydrologic soil groups based upon their infiltration properties.

Hydrologic soil groups "A," "B," "C," "C/D," and "D" are found present within the Berryessa/Penitencia drainage system. Group "A" has a higher infiltration rate than Group "D". "C/D" soils indicate a duality of hydrologic soil conditions. When the groundwater table is seasonably high (less than 24" from the surface), the soil has a "D" type response. When the water table is well-drained (greater than 24" from the surface), the soil has a "C" type response. The open space areas in the eastern portion of the Study area have more Groups "C" and "D". The urban areas along Upper Penitencia, Lower Penitencia, and Berryessa Creeks are mostly undetermined soil groups and are calibrated and validated under Valley Water's previous analysis. Figure 2-6 presents the NRCS Soils Map.

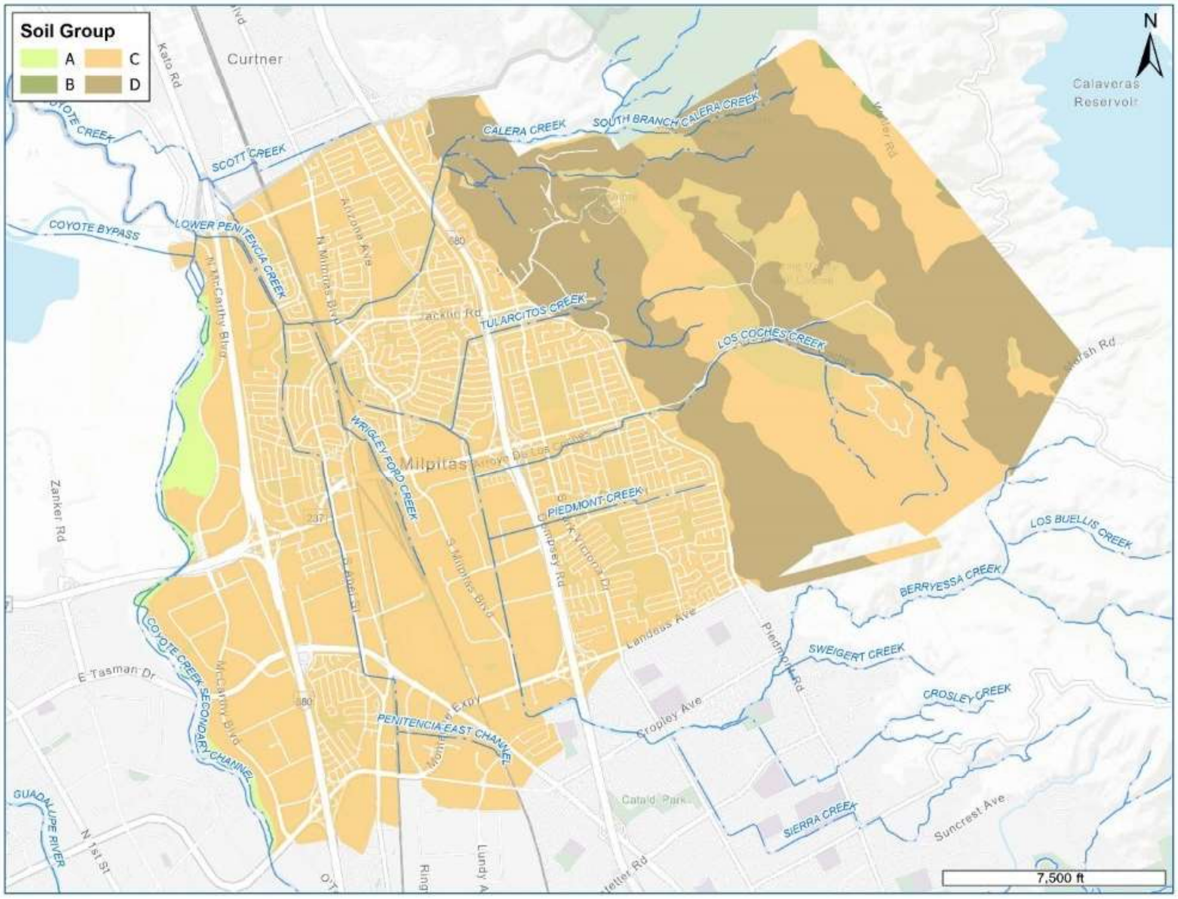


Figure 2-6: NRCS Soil Classification Map for the Berryessa/Penitencia Watershed

Each soil type consists of a combination of seven soil groups, A, A/B, B, B/C, C, C/D, D, and the NRCS assigns the dominant hydrologic soil group in the soil survey publication. In the event of an even split of percentages between two soil groups, the soil group with a lower infiltration rate is assigned.

Loss Rates

The NRCS Soil Group Boundaries are used to calculate each hydrologic soil group's quantity within each watershed. This information is input directly to the model in conjunction with each hydrologic soil group's infiltration properties (defined in Table 2-5).

Watershed Land Use

The City of Milpitas provided a land-use GIS reflecting the level of development within the city boundary. Each land use category was assigned a value of relative imperviousness based upon Table 2-6. To develop sub-basin-specific hydrology parameters for the KW and Snyder UH methods used a combination of percent imperviousness and underlying soil infiltration regime.

Figure 2-7 shows the land use. Areas with no defined land use in the City's GIS are assigned sub-basin-specific hydrology parameters based on the Esri World Imagery Map.

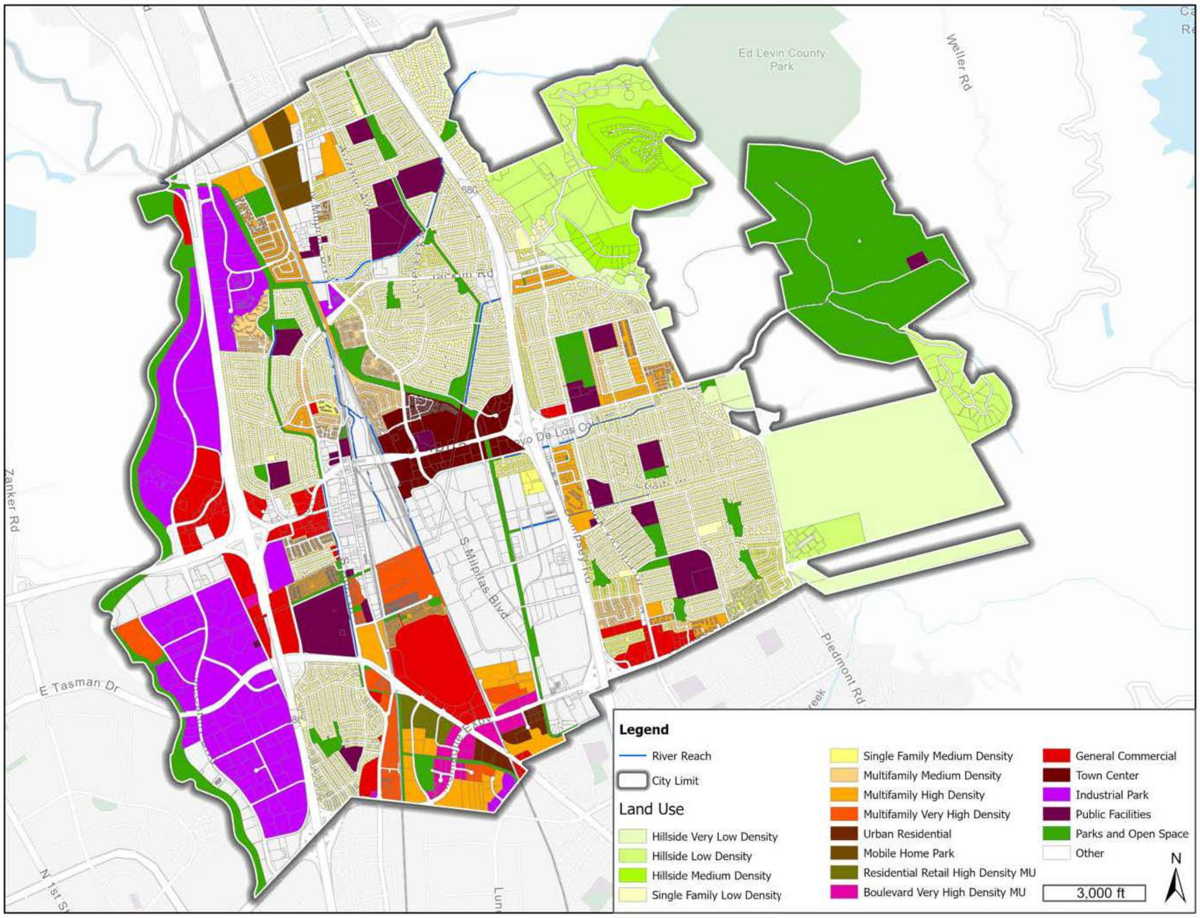


Figure 2-7: Milpitas Land Use

Hydraulics

A detailed representation of both the 'conduit and street' systems and the open channel systems is required in the model to evaluate the city's level of service storm drain system goals. This representation accounts for both conveyance and storage in the streets, conduits, and open channels. The InfoWorks ICM modeling software preforms the hydraulic analysis.

The Valley Water CTP model includes both the Upper Penitencia and Lower Penitencia/Berryessa watersheds, tributary creeks, and storm drain networks in San Jose and Milpitas. That CTP model is truncated at the Milpitas/San Jose boundary to focus on the City of Milpitas storm drain system. Channel, pipe, and 2D surface flow hydrographs are boundary inputs to the Milpitas SDMP model at the border between San Jose and Milpitas.

Conduit and Street Systems

The conduit and street systems were modeled using parameters as discussed in the following sections.

Conduit and Manhole Invert Elevations

The City of Milpitas GIS file of the storm drain network provided inverts of conduits and manholes. If inverts are missing from these two data sources, as-builts have been referenced to fill in any data gaps.

If data is still not found from these two data sources, an appropriate assumption is made by referencing upstream and downstream inverts and storm drain cover.

Conduit Manning's n Roughness

Manning's n-values for conduit and street systems are estimated based on values specified in the reference shown in Table 2-8.

Table 2-8: Manning's n

Conduit Material	Manning's n
Reinforced Concrete Pipe	
Conduit > 36" diameter	0.012, 0.012*
Conduit < 36" diameter	0.012, 0.014*
Corrugated Metal Pipe	
Annular	0.021
Helical	0.018
Concrete-Lined Channels	
Smooth-Troweled	0.015
ACPWA Simulated Stone	0.017
Reinforced Concrete Box	
Cast-In-Place	0.015
Pre-Cast	0.014

**Bottom third of the conduit is assigned the first manning's n listed, while the top two-thirds are assigned the second manning's n listed to account for friction loss.*

All conduits of unknown material were assumed to be reinforced concrete pipe. For conduits with materials not listed in the table above, Manning's n values were referenced from other sources.[6] These materials and their associated Manning's n values are listed in Table 2-9.

Table 2-9: Conduit Manning's n Not Listed in Table 2-8

Material	Manning's n
Asbestos Cement (ACP)	0.011
Polyvinyl Chloride (PVC)	0.009
Cast Iron Pipe (CIP)	0.012
Ductile Iron Pipe (DIP)	0.012

All conduit material descriptions by pipe segment were obtained from the City of Milpitas.

[6] Engineering ToolBox, (2004). Manning's Roughness Coefficients. [online] Available at: https://www.engineeringtoolbox.com/mannings-roughness-d_799.html [2020]

Conduit Manhole Losses

Manhole losses are calculated in InfoWorks ICM using the Normal Headloss Method. The method does not account for bend, drop, contraction, and expansion losses but does account for simple junction losses. However, storm drains are generally built on straight alignments of the same diameter from manhole to manhole; without significant bends, drops, contractions, or expansions. Also, particularly for the larger pipe diameters, manholes simply provide periodic access to the top of continuous pipelines.

Conduit Boundary Conditions

The storm drain network's downstream boundary conditions are dynamically linked to 1D open channels in the InfoWorks ICM model. Creek levees are assumed to hold their elevations if overtopped in a modeling scenario. Levees do not fail in the SDMP analyses as they might for a Federal Emergency Management Association (FEMA) flood hazard analysis.

Conduit Assumptions

All conduits are assumed to have their full conveyance available. That is, all conduits are modeled with no silt or debris.

Pump Stations

The Milpitas storm drain system relies heavily on pump stations to move runoff from pipe networks to creeks that flow to San Francisco Bay. There are 13 pump stations owned and operated by the City of Milpitas within the study area, as shown in Figure 2-9. These pump stations are generally located at the discharge points of the City of Milpitas's storm drain system to Lower Penitencia Creek, Berryessa Creek, or Coyote Creek. Four (4) of the 13 stations discharge directly into Coyote Creek. Wrigley-Ford Creek pump station discharges the tributary Wrigley, Ford, and Wrigley-Ford Creeks (owned and operated by the City of Milpitas) to Berryessa Creek.

The pump stations are input into the InfoWorks ICM model based on a combination of data obtained from record drawings and the previous City of Milpitas SDMP (2013). Conversations with City operations and maintenance staff confirmed there have been no changes to pump station capacities since 2013. As-built documents determine the pump station storage volumes, including the larger storage areas at California Circle and Hidden Lake. Where system drawings are not available, storage curves from previous work on the 2013 SDMP and ancillary studies were used. The City of Milpitas provided set levels for the pump stations on- and off-levels. Pumps are modeled as having a flat rate of discharge based on the peak operating levels. A sensitivity analysis validates this simplification, as discussed below.

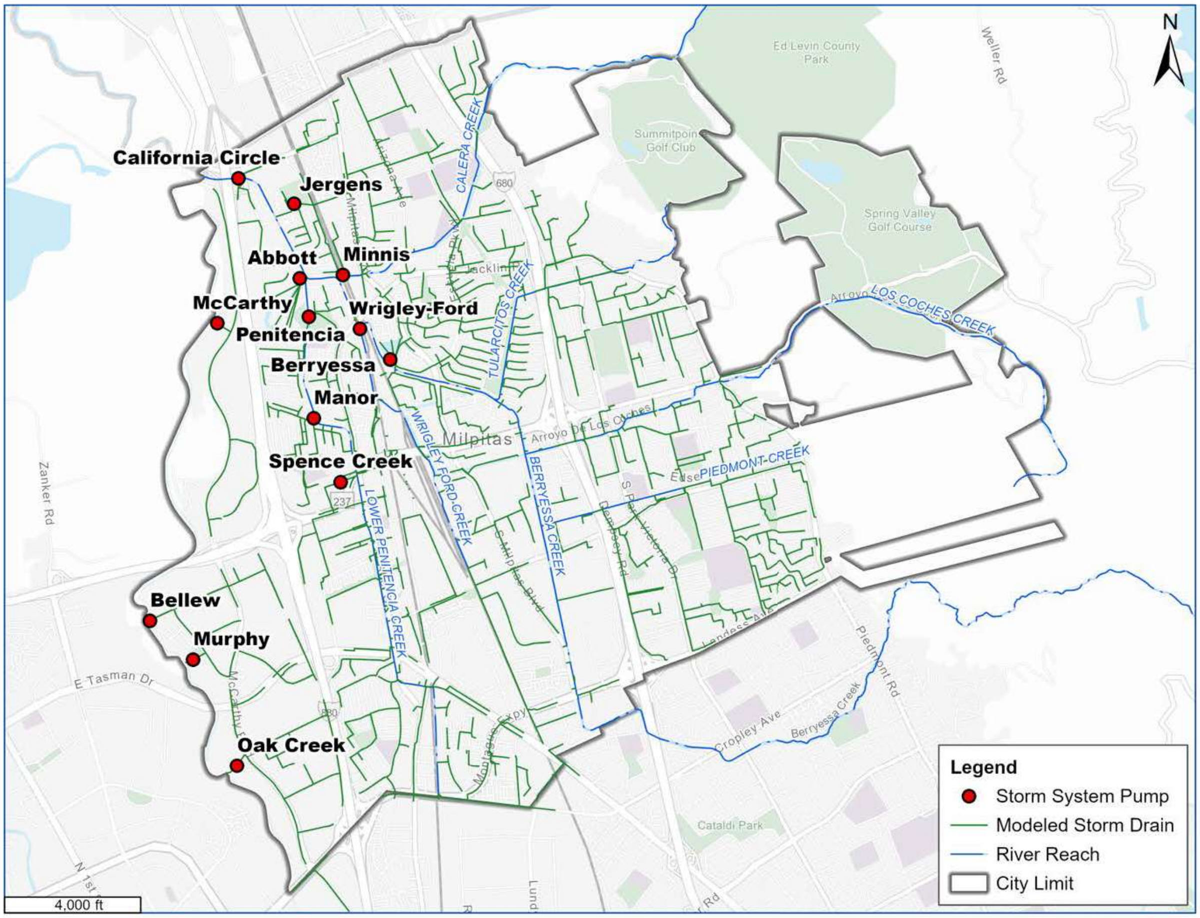


Figure 2-8: Pump Station Locations

Pumping Rate Sensitivity Analysis

To preserve the data associated with pump operation and simplify the ICM model, the pumps are not de-rated for minor losses; nor are pump curves inserted into the model. Pumps are modeled with a constant discharge set the maximum pump flow based on their pump curves. A sensitivity analysis has been performed to verify the validity of this assumption. The pipe or creek depth upstream and downstream of the pump station are compared using three pump capacity scenarios to test the pump capacity's sensitivity during the 100-yr storm, which exceeds the 10-year SDMP storm recurrence.

Downstream pipe depths are not sensitive to the pumping capacity, except at the Bellew, McCarthy, and Murphy Pump Stations that discharge to Coyote Creek, as summarized in Table 2-10. Since Coyote Creek is not specifically modeled and the pump discharges are in fact decoupled from the stage in the creek, these pump stations discharge under a free outfall condition, which artificially increases the sensitivity of the downstream water depth to pump capacity.

The sensitivity analysis indicates that the pumping capacity does not change the simulation outcome (maximum depth in the modeled downstream reaches) when the pumping capacity varies between 70% and 110% of its rated capacity. This constant-rate pump capacity methodology is deemed adequate for CIP formulation since there is no change in proposed CIPs based on a variable pumping rate tied to the creek stage at the pump station outfalls.

Table 2-10: Pump Station Sensitivity Analysis

Max Water Depth (ft) during the 100-yr Event							
Pump Station	Base Scenario		70% Pump Rated Capacity		110% Pump Rated Capacity		Receiving Creek
	U/S	D/S	U/S	D/S	U/S	D/S	
California	1.9	10.9	3.5	10.8	1.9	10.9	Lower Penitencia
Jurgens	4.8	13.3	4.8	13.2	4.8	13.3	Lower Penitencia
McCarthy	10.2	4.8	11.0	3.7	11.0	4.5	Coyote Creek
Minnis	6.3	10.3	13.1	10.2	6.6	10.2	Calera Creek
Abbott	7.5	11.4	7.7	11.4	7.5	11.4	Lower Penitencia
Penitencia	2.2	10.7	2.2	10.6	2.2	10.7	Lower Penitencia
Wrigley-Ford	5.4	10.8	6.3	10.7	5.5	10.8	Berryessa Creek
Berryessa	6.0	9.8	6.3	9.8	6.1	9.8	Berryessa Creek
Manor	3.2	9.2	5.2	9.1	3.4	9.1	Lower Penitencia
Spence Creek	7.4	12.9	7.3	12.8	7.4	12.9	Lower Penitencia
Bellew	22.8	1.2	23.2	1.0	22.8	1.2	Coyote Creek
Murphy	11.4	4.0	12.3	3.8	12.0	5.2	Coyote Creek
Oak Creek	6.2	5.0	7.0	4.8	6.9	4.6	Coyote Creek

Open Channel Systems

The open channel systems are modeled as discussed in the following subsections. Most of the open channels in the Lower Penitencia-Berryessa and Upper Penitencia Watershed in the CTP model are shown, including Upper Penitencia Creek, Sierra Creek, Sweigert Creek, Berryessa Creek, Los Coches Creek, Tularcitos Creek, Calera Creek, Wrigley Creek, Ford Creek, Wrigley-Ford Creek, East Penitencia Creek, and Lower Penitencia Creek. Crosley Creek is not modeled explicitly because its banks are not adjacent to any man-made structures. Crosley Creek was accounted for in hydrologic routing. In the truncated SDMP model for this report, Upper Penitencia, Berryessa upstream of Highway 680, and Sierra Creeks are removed from the model and input as hydrographs.

Open channel cross-sectional geometry is from a variety of sources, including surveys, as-builts, permitted design plans, previous HEC-RAS models, and the 2006 LiDAR dataset. Each source is described in the following sections, while Figure 2-9 spatially displays each creek's cross-section source (channels designated as "Survey" displays the year of the survey). This assumes projects funded by Valley Water at the time of this master plan will be completed but does not include future planned projects.

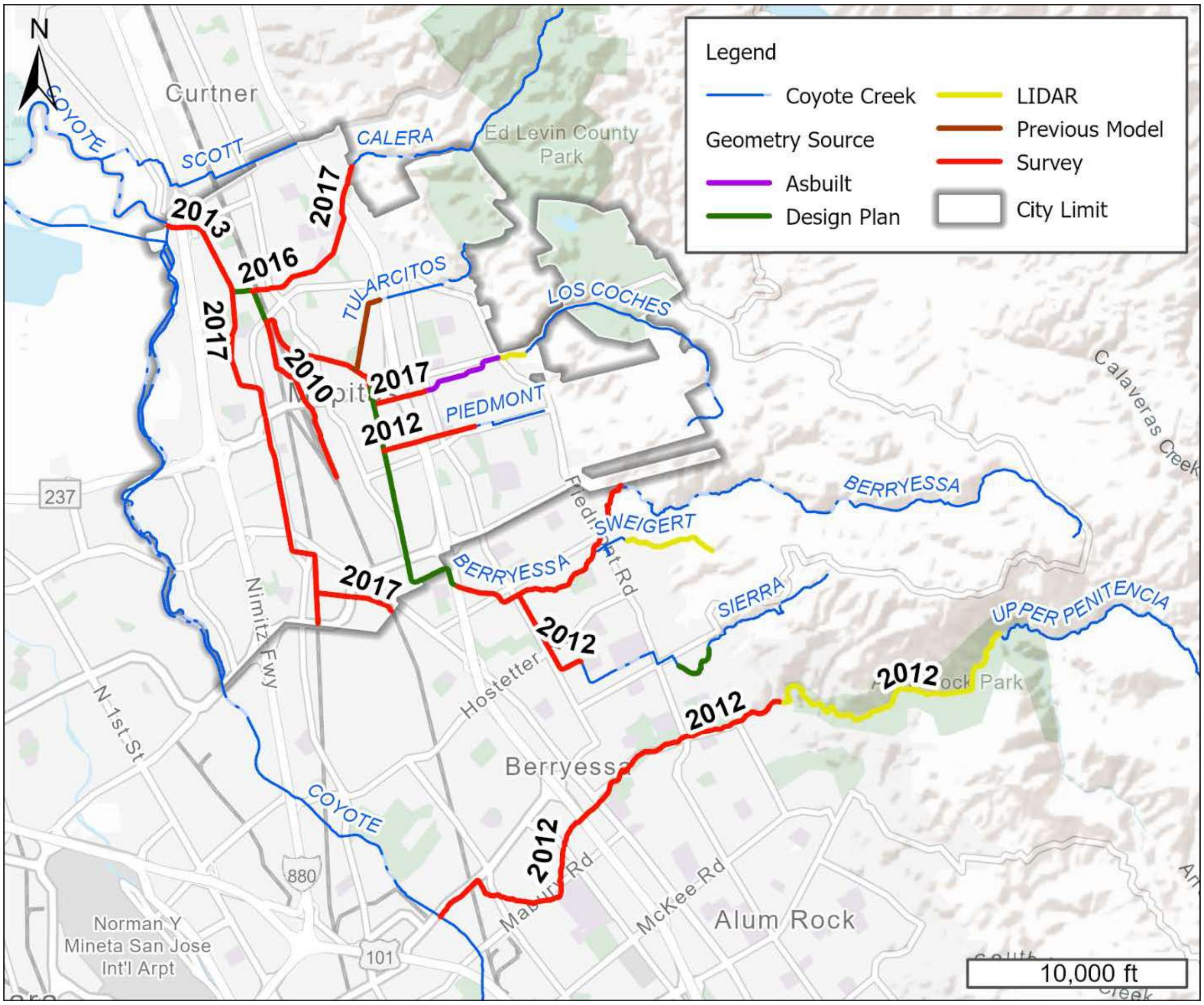


Figure 2-9: Map of Channel Cross Sectional Geometry Source

Levees and Floodwalls

All levees and floodwalls included in the model are assumed to not fail and therefore maintain flow within the channel to the top of bank, levee, or floodwall elevation. This produces the most conservative water surface elevation in the channel for storm drain modeling.

Storage Areas

Some storage areas in the model define pump station storage. California Circle Pump Station, Wrigley-Ford Pump Station, Berryessa Pump Station, Penitencia Pump Station, Jurgens Pump Station, and Abbott Pump Station are the pump stations with six storage areas defined. Storage areas are represented by stage-storage curves, which have been obtained from record drawings or previous studies.[7]Another storage area in the model associated with a pump station is Dixon Landing Park, which is used to store surcharges upstream of Jurgens Pump Station.

[7] "City of Milpitas Storm Drain Master Plan", Schaaf & Wheeler, July 2013

Table 2-11: Pump Station Storage Areas

Pump Station	Storage Area
California Circle Pump Station	Dixon Landing Lagoon
Berryessa Pump Station	Hidden Lake Lagoon
Penitencia Pump Station	Hall Park Lagoon
Abbott Pump Station	City of Milpitas Drainage Lagoon
Wrigley-Ford Pump Station	Storage area located where Ford Creek joins Wrigley-Ford Creek
Spence Creek Pump Station	Spence Creek Pump Station Storage Area

Channel Bankline

Channel bank lines connect channel overbanks to 2D floodplains hydraulically. Theses lines' ground elevations are sampled from a GIS terrain to ensure smooth transitions from channel overbanks to the 2D floodplains.

Channel bank lines represent floodwalls along Lower Penitencia Creek, Calera Creek, and Berryessa Creek. These elevated tops of wall floodwall elevations have been converted from the as-builts georeferenced to ortho-imagery. These lines, when used as floodwalls, contain runoff within channels until and unless the floodwalls are overtopped.

A map of flood walls and levees is displayed as Figure 2-10.

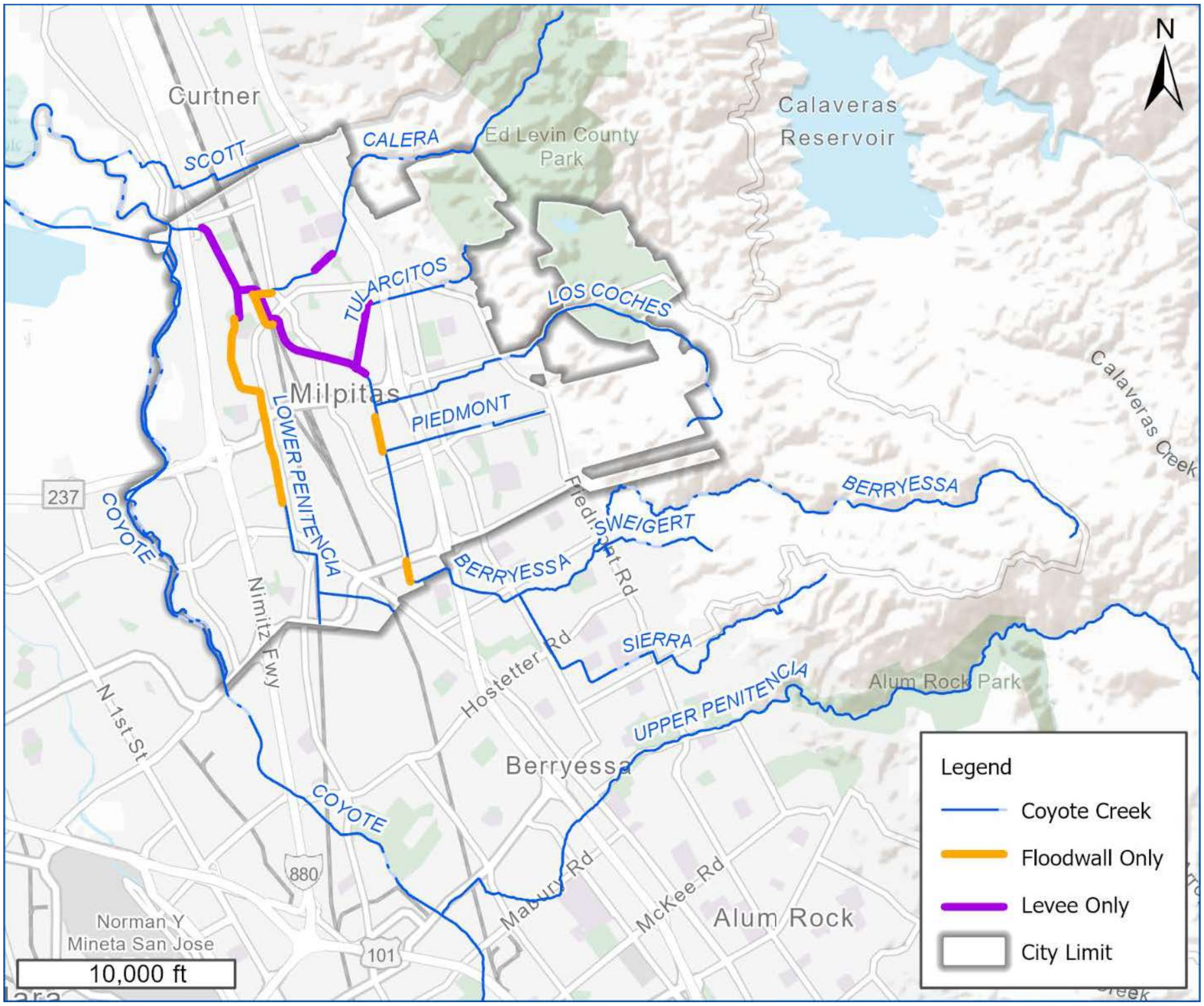


Figure 2-10: Flood Wall and Levee Map of the Berryessa/Penitencia Watershed

Two-Dimensional Surface

A two-dimensional (2D) surface is modeled and linked with pipe nodes to reflect flow exchanges and the hydraulic performance between different drainage facilities when pipe capacities exceed and flow spills to the 2D surface. The features that affect the predicted flooding extents with the spill to the 2D surface resolve the modeled 2D surface and surface roughness.

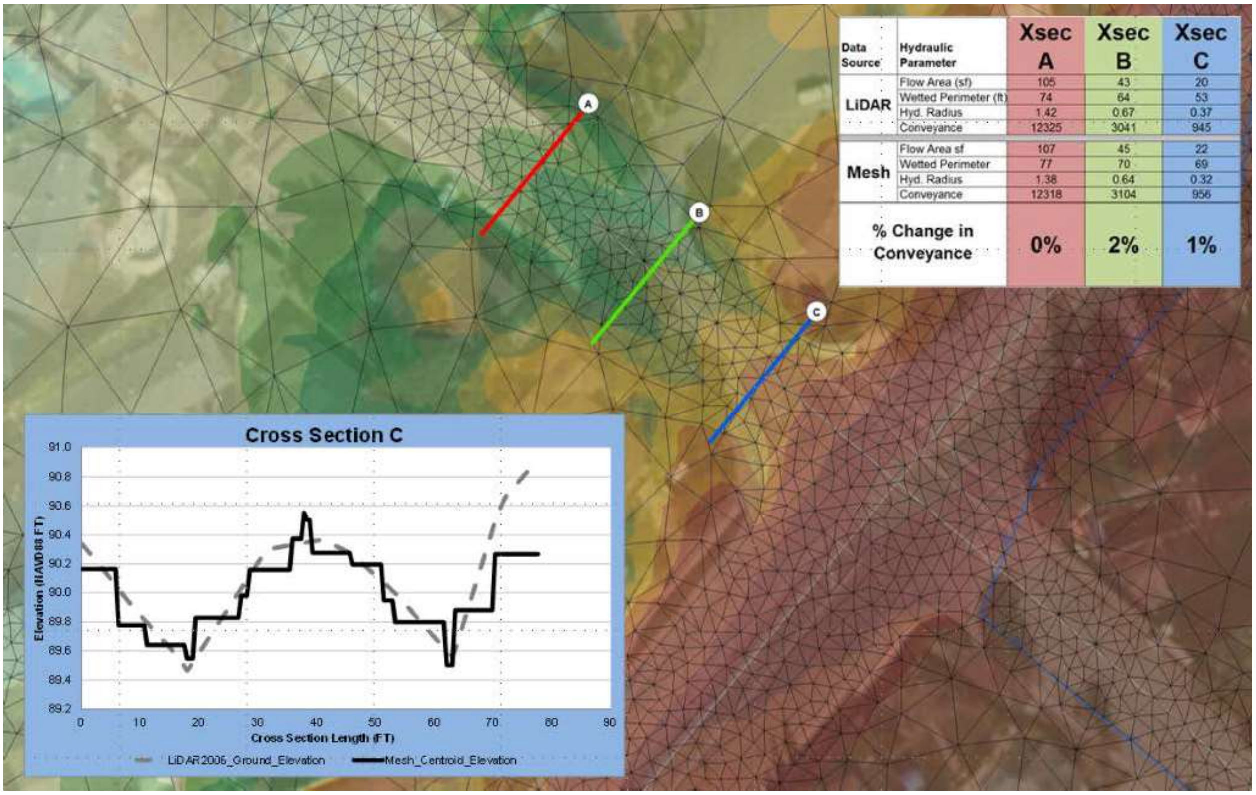


Figure 2-11: Cross Section Comparison between the Modeled vs. Terrain

Two-dimensional surface resolutions are classified into two groups. A high-resolution group built to reflect appropriate street conveyance. The model elements (triangles) representing the terrain resolution have sizes ranging from 25 to 500 square feet. The high-resolution group has sufficient accuracy in representing street conveyance, as indicated in Figure 2-11. The percent of cross-section conveyance difference between the modeled and original terrain is less than 5 percent.

A lower resolution group is built for other areas with no well-defined drainage conveyance geometry, such as open space, residential, and commercial areas. The model elements (triangles) that represent a lower resolution terrain have sizes ranging from 1,000 to 5,000 square feet.

Valley Water provided the LiDAR topographic terrain used for the 2D surface modeling and has a cell size/resolution of a 3-foot by 3-foot grid. In addition, known projects completed between the 2006 LiDAR and 2018 are inserted into the 2D surface as fill based on development design plans.

Two-dimensional Surface Manning’s n Roughness

Surface roughness over the 2D grid is estimated with Manning’s n values and is typically higher than the channel roughness because of the shallow flow characteristics. Aerial photography and land use classifications made estimates of Manning’s n-values for the 2D surface by considering the density of buildings on the overbank and the base “n” values of the open areas of the buildings. Although the density of buildings and the presence of fences and other obstructions vary considerably across a given land use polygon, it is assumed that a general urban roughness value reasonably represents the effects of structures within the floodplain.

Surface land uses are categorized into four different Manning’s n values displayed in Table 2-12:

Table 2-12: Floodplain Manning's n Values

Floodplain Description	N value
Street	0.025
Open Space, Institution, Public Facilities	0.03
Commercial	0.12
Residential and Industrial	0.13

Based on the above methodology, the model is sufficient to evaluate storm drain system performance for a 10-year design storm. The resulting flooding onto the two-dimensional surface identifies deficiencies that need correction with a Capital Improvement Program (CIP).

Chapter 3: Capital Improvement Program

This chapter describes deficiencies in major storm drain facilities and outfalls, historical problem areas, pumping and storage facilities, and other known flood hazards. Detailed descriptions of necessary capital improvement projects and their prioritization are provided in this chapter.

Existing Conditions Flooding

The master plan evaluates the existing storm drain system performance for the 10-year design storm. Figure 3-1 shows the flooding that is modeled to occur with the 10-year design storm. In this flood study, the master plan proposes CIP projects that can eliminate or ameliorate the identified inundation.

Existing conditions include the completion of City storm drain projects since previous master plans and Valley Water projects such as the Berryessa Creek and Lower Penitencia Creek Flood Protection Projects. A further reduction in existing conditions flooding relative to the prior master plan efforts is achieved by using a two-dimensional flow surface as described in Chapter 2. Previously, a more traditional method was used to evaluate existing conditions flooding and the need for capacity improvements. In more traditional methods, hydraulic gradients are compared to curb elevations and street capacities available to carry the storm drain overflows were made in the steady stage regime.

Now transient flood hydrographs are computed at each modeled node in the system. Available storage within the drainage system and on the ground surface are both considered in tandem. In general, there is less overflow, and the overflow that occurs results in less surface inundation. Therefore, the identified need for remedial capital improvement projects is lessened when compared to earlier studies.

Improvement Projects

Recommended CIP projects are identified graphically, and general project routes are given. The following color code is used throughout this chapter to highlight system performance and general CIP prioritization, as described by Table 3-1:

Green	Satisfactory Performance / No Improvement Necessary
Red	High Priority Project
Orange	Low Priority Project

CIP priorities are assigned based on inundation impact in the existing condition. A high-priority CIP project addresses extensive flooding on residential or commercial properties that spreads across several streets and can widely disrupt traffic, residential and commercial activities. A low-priority CIP project manages flooding only in streets, minor flooding on properties, or flooding that can only be partially alleviated. No numeric threshold was used in categorizing CIP priorities. The tables of statistics associated with each collection system group give a general indication of the capital expenditure level necessary to correct storm drain deficiencies. Sometimes installing additional pipe lengths is required to complete corrective action.

City-owned storm drainpipes 12 inches and larger in diameter have been evaluated. Pipes that act as laterals are not in the mainline analysis but are assumed to be part of the system that delivers flow into the main drainage lines.

Alternative Improvement Projects

To increase storm drain system capacity, two essential types of projects are available: installing a new relief sewer parallel to the system lacking capacity; or replacing the overloaded pipe with a larger diameter pipe in the same alignment. The two alternatives can be made equivalent to one another using the following formula, assuming that pipe material and length are equal:

$$D_R = \left(D_e^{2.63} + D_p^{2.63} \right)^{0.38}$$

where D_R = diameter of replacement pipe;

D_e = diameter of overloaded pipe; and

D_p = diameter of parallel relief drain.

The City's selection of a capacity improvement strategy will vary from project to project and be governed by construction constraints, including available rights-of-way and existing utilities. It is most likely that the storm drain CIP for Milpitas will utilize parallel relief drains unless right-of-way and utility constraints appear to favor the pipe's actual replacement, which is more costly.

Installing new parallel drains should be more cost-effective than replacing pipes in most cases since the required pipe size is smaller, and the existing pipe does not need removal. Given the 50 percent contingency applied to unit cost estimates, there is no differentiation between the cost of pipe replacement and parallel drain installation in the CIP. (That is, the cost of existing pipe removal is included in the large contingency.)

Therefore, the default project for in-street improvements is a parallel relief drain, while the default project for improvements within existing off-street easements is pipe replacement. The CIP assumes storm drain size is not allowed to decrease in the downstream direction. Thus, an additional downstream pipe may be listed in the CIP, although there is no indication of substandard storm drain performance based on hydraulic grade calculations.

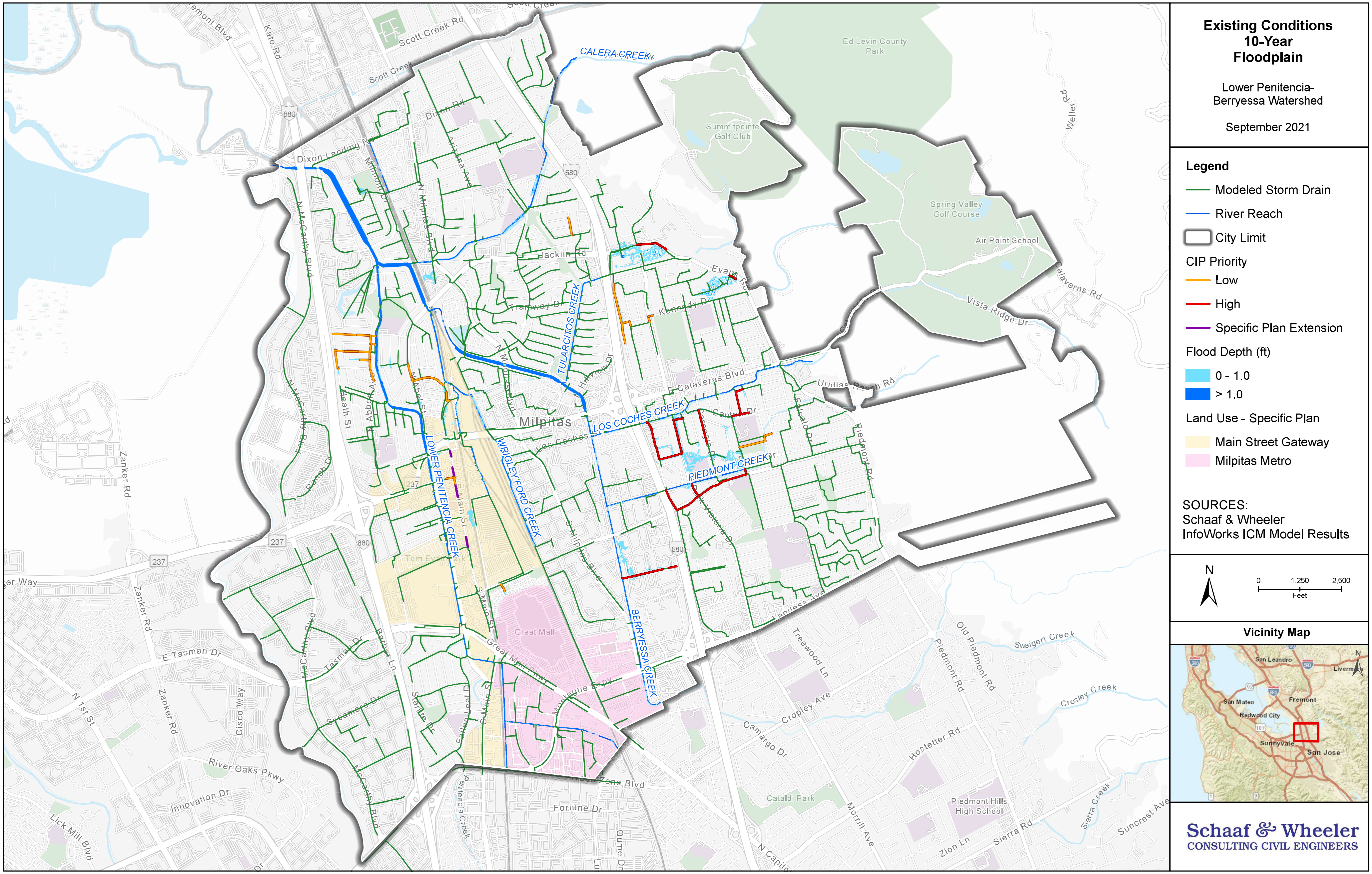

Existing Conditions
10-Year
Floodplain
Lower Penitencia-
Berryessa Watershed
September 2021
Legend
Modeled Storm Drain
River Reach
City Limit
CIP Priority
Low
High
Specific Plan Extension
Flood Depth (ft)
0 - 1.0
> 1.0
Land Use - Specific Plan
Main Street Gateway
Milpitas Metro
SOURCES:
Schaaf & Wheeler
InfoWorks ICM Model Results
N
0
1,250
2,500
Feet
Vicinity Map
Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS
Milpitas
CALERA CREEK
TULARCITOS CREEK
LOS COCHES CREEK
PIEDMONT CREEK
BERRYESSA CREEK
WRIGLEY FORD CREEK
LOWER PENITENCIA CREEK
Great Mall
Ed Levin County Park
Summitpointe Golf Club
Spring Valley Golf Course
Air Point School
Cataldi Park
Piedmont Hills High School

Figure 3-1

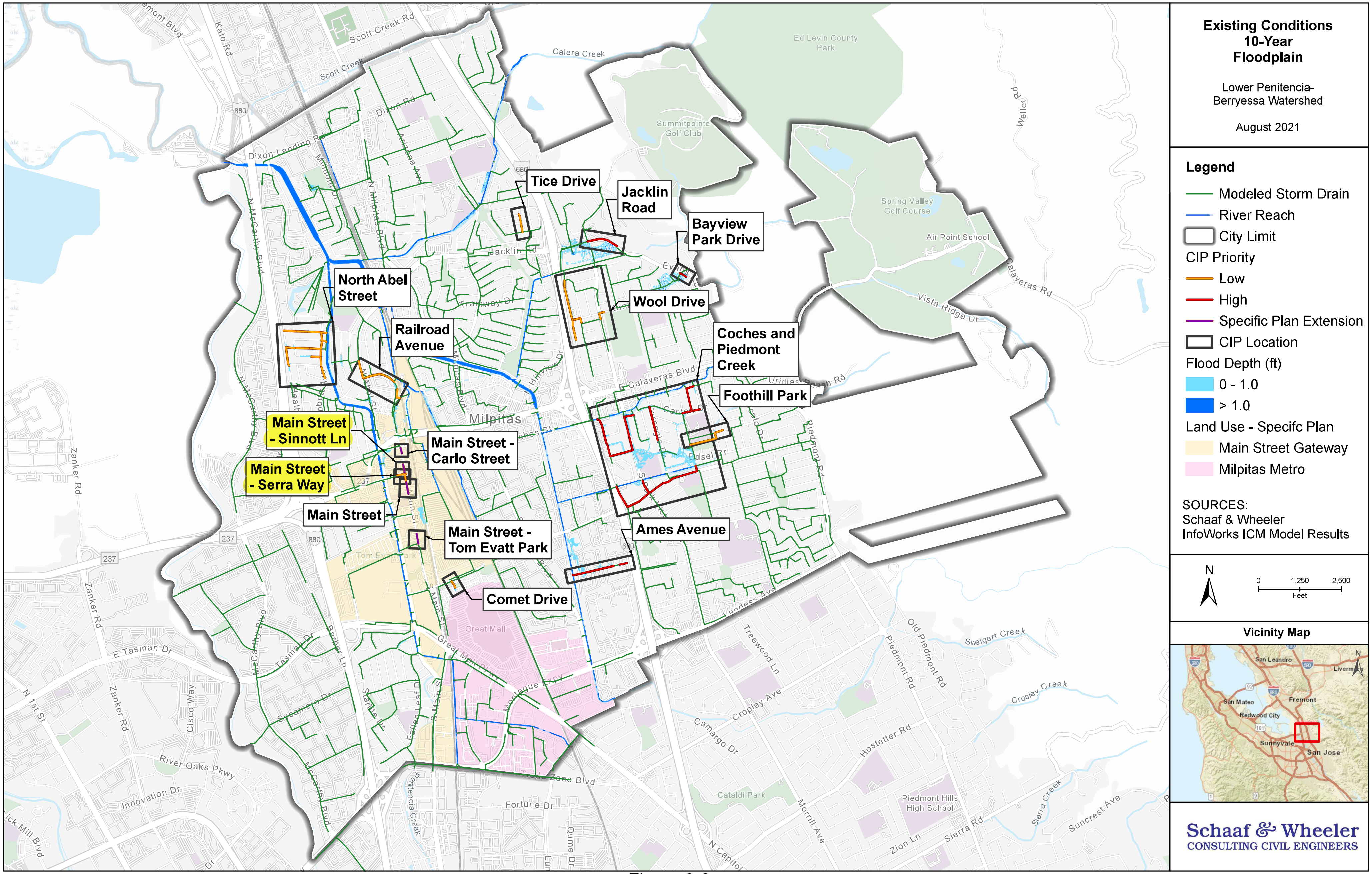
Capital Improvement Program

CIP projects are identified in Figure 3-2 and Table 3-1, which will correct inadequate storm drain capacity caused primarily by undersized pipes during a 10-year design storm. The inundation areas that each of the projects specifically address is shown in Figures 3-4 to 3-10. Detailed figures for each CIP project are included in Appendix A.

Table 3-2 provides high priority CIP details including both the parallel and replacement options. Table 3-3 serves the same function for the low priority CIP. The high priority CIP is 20 years. Low priority projects would not commence until the high priority CIP is completed in its entirety, unless there is associated work wherein a low priority project can be ancillary or conditioned. Each CIP project, whether high or low priority, has been established so that there would be no adverse hydraulic impacts associated with its completion. Therefore, there is no required order to the high or low priority projects. The City may complete capital projects in any order and spread projects throughout the twenty-year CIP.

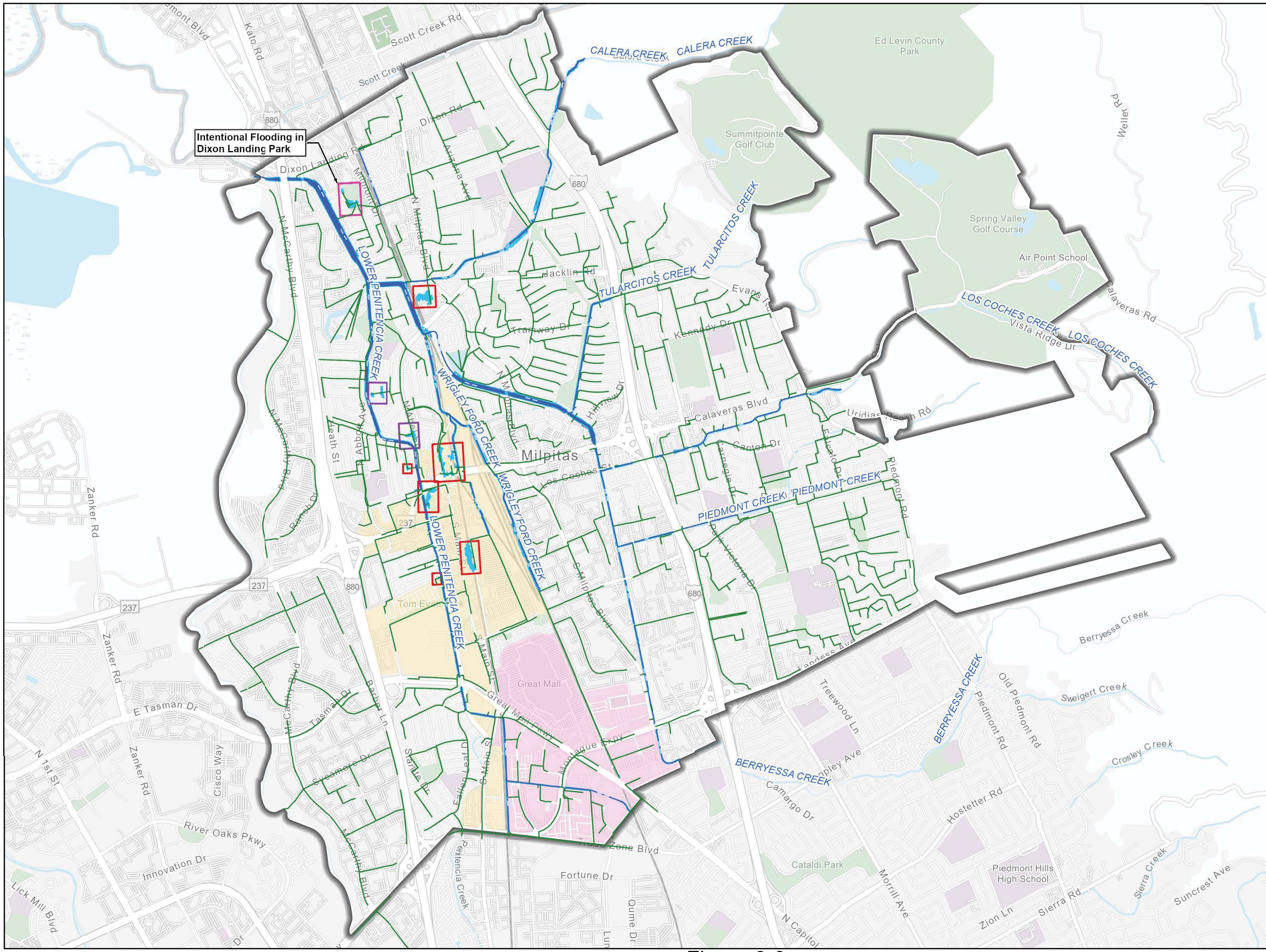
However, the fourteen (14) identified CIP projects do not resolve surcharges from high creek levels downstream of the storm drain network, nor do they address inlets or manholes at isolated low points with low ground cover. The inundated areas are areas not resolved by the fourteen CIP projects, shown in Figure 3-3. These areas would only be resolved by major creek projects (outside of the scope of this Master Plan) or by the installation of small, localized pump stations, all found to be cost-prohibitive. The inundations are located mainly on the streets and empty lots. There is minor flooding on residential and commercial properties.

Figure 3-3 shows that Dixon Landing Park has been intentionally flooded by design, to avoid a larger pump station when Jurgens Pump Station was constructed in 1983. However, this situation has proved to be untenable. A high priority CIP for Jurgens Pump Station is identified to provide the necessary pumping capacity and remedy this problem.



Existing Conditions
10-Year
Floodplain
Lower Penitencia-Berryessa Watershed
August 2021
Legend
Modeled Storm Drain
River Reach
City Limit
CIP Priority
Low
High
Specific Plan Extension
CIP Location
Flood Depth (ft)
0 - 1.0
> 1.0
Land Use - Specifc Plan
Main Street Gateway
Milpitas Metro
SOURCES:
Schaaf & Wheeler
InfoWorks ICM Model Results
N
0
1,250
2,500
Feet
Vicinity Map
Tice Drive
Jacklin Road
Bayview Park Drive
North Abel Street
Wool Drive
Railroad Avenue
Coches and Piedmont Creek
Foothill Park
Main Street - Sinnott Ln
Main Street - Carlo Street
Main Street - Serra Way
Main Street
Ames Avenue
Main Street - Tom Evatt Park
Comet Drive
Milpitas
Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Figure 3-2



Post-CIP
10-Year Floodpain

Lower Penitencia-
Berryessa Watershed

April 2021

Legend

- Modeled Storm Drain
- River reach
- City Limit

Land Use

- Midtown Specific Plan
- Metro Area Specific Plan

Remaining Inundation

- High Creek Level
- Intentional Flooding
- Low Point

Flood Depth (ft)

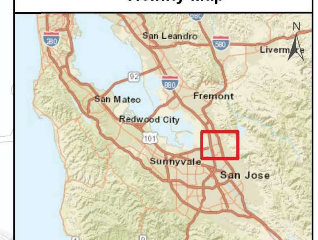
- 0 - 1.0
- > 1.0

SOURCES:
Schaaf & Wheeler
InfoWorks ICM Model Results

N

0 1,250 2,500
Feet

Vicinity Map



Schaaf & Wheeler
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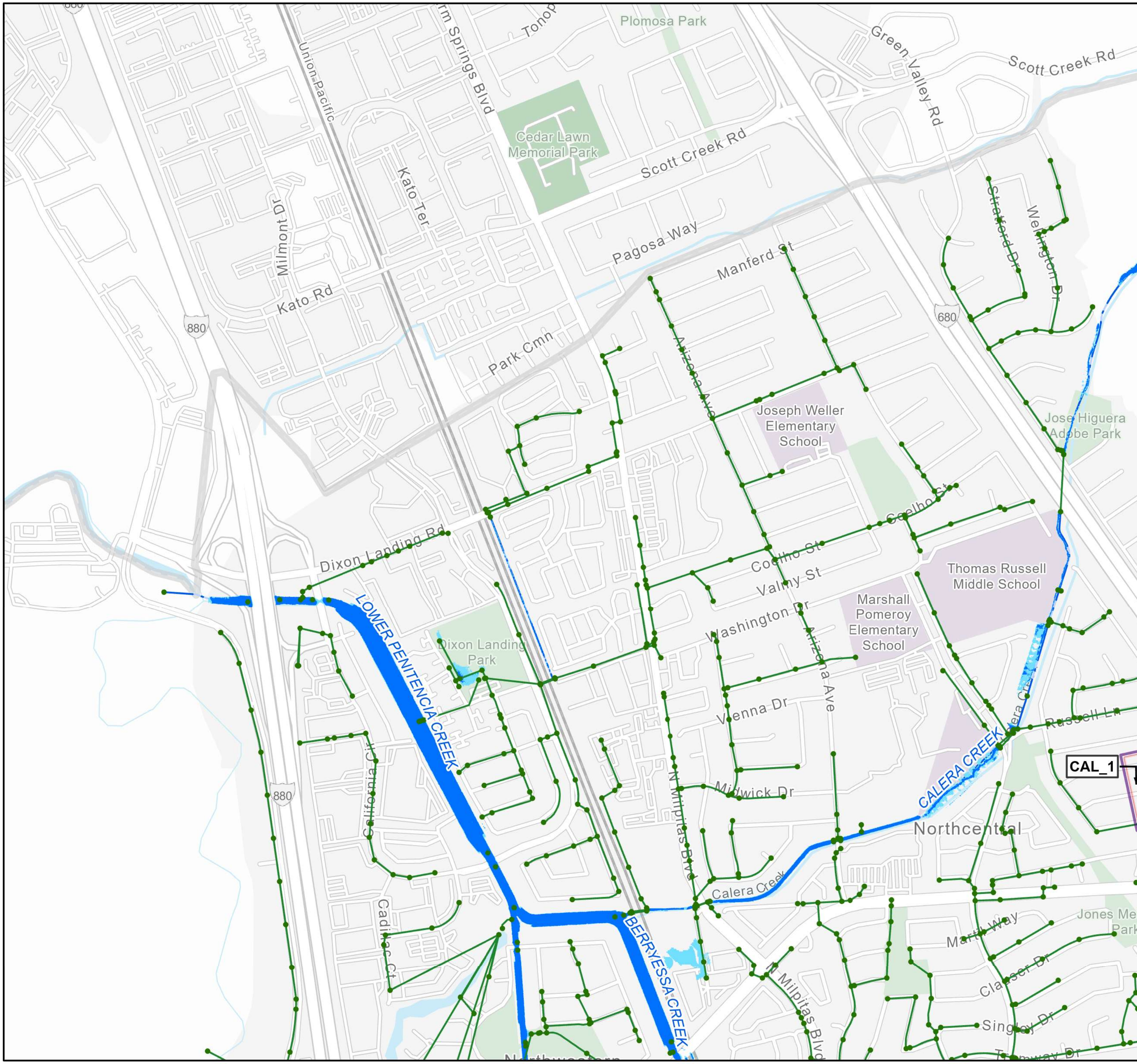
Figure 3-3

Milpitas
Storm Drainage CIP

A1

SD Improvements

Page 1 of 6



Legend

- Storm Drain Manhole
- Modeled Storm Drain
- River Reach
- Inundation Addressed

CIP Priority

- Low
- High

Flood Depth (ft)

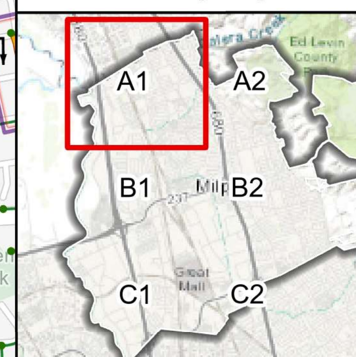
- 0 - 1.0
- > 1.0

*Pipe annotation: "x to y" where x is existing size and y indicates proposed

N

1 inch : 1,200 feet

Vicinity Map



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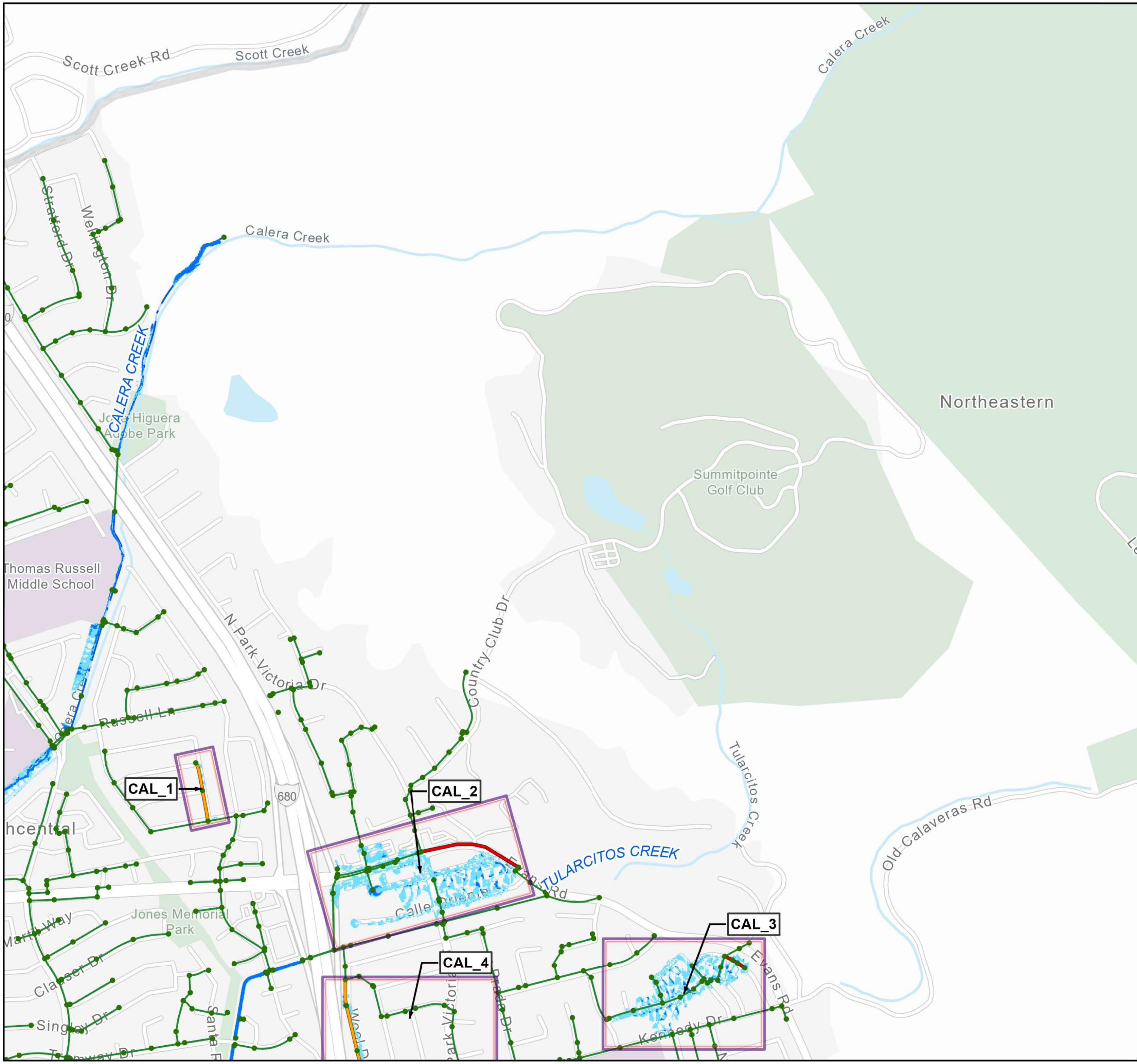


Milpitas
Storm Drainage CIP

A2

SD Improvements

Legend

- Storm Drain Manhole
- Modeled Storm Drain
- River Reach
- Inundation Addressed

CIP Priority

- Low
- High

Flood Depth (ft)

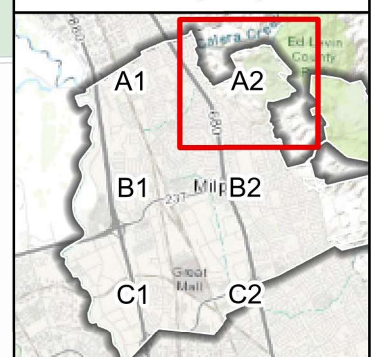
- 0 - 1.0
- > 1.0

*Pipe annotation: "x to y" where x is existing size and y indicates proposed

N

1 inch : 1,200 feet

Vicinity Map

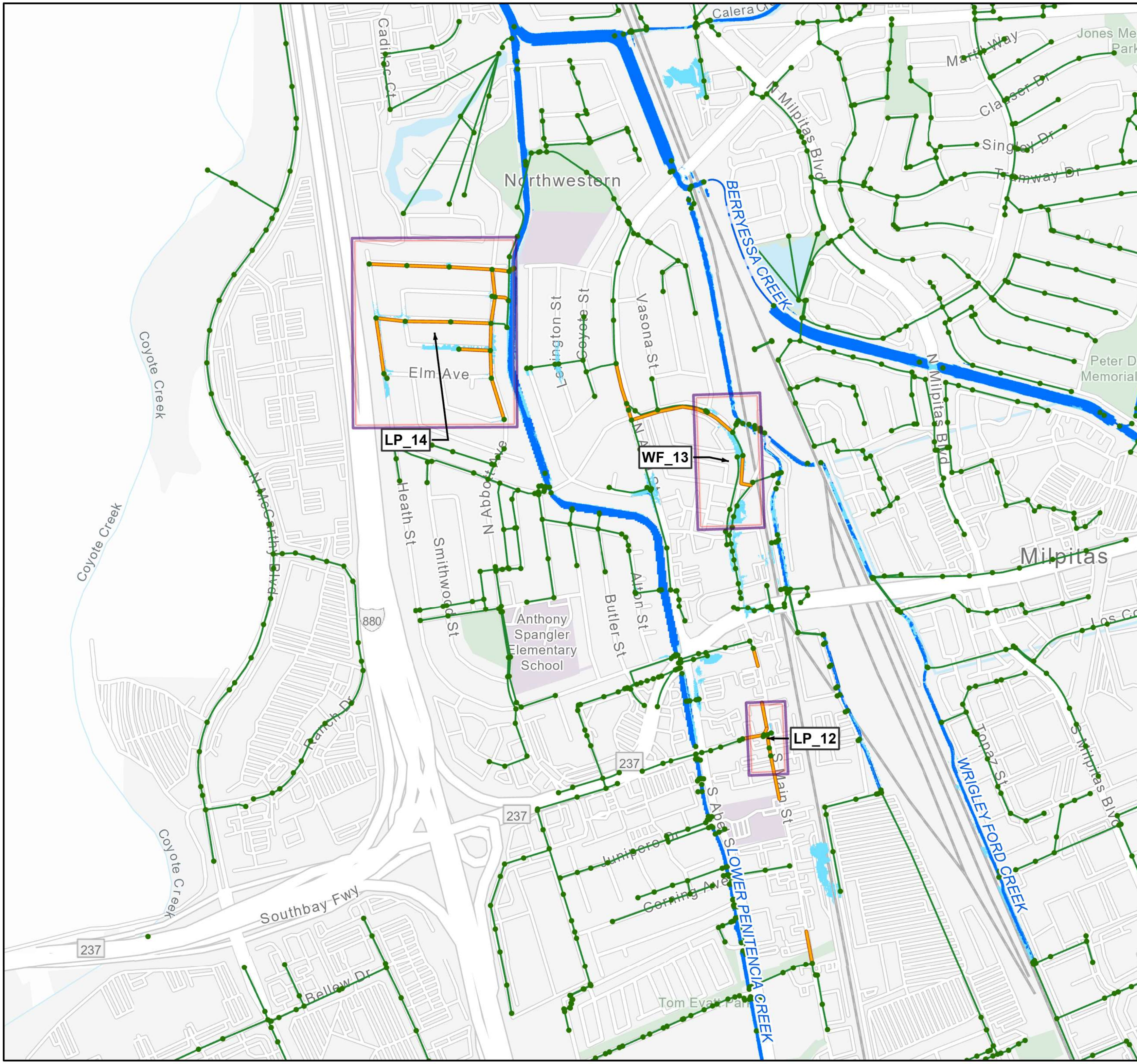


Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Milpitas Storm Drainage CIP

B1

SD Improvements



Legend

- Storm Drain Manhole
- Modeled Storm Drain
- River Reach
- Inundation Addressed

CIP Priority

- Low
- High

Flood Depth (ft)

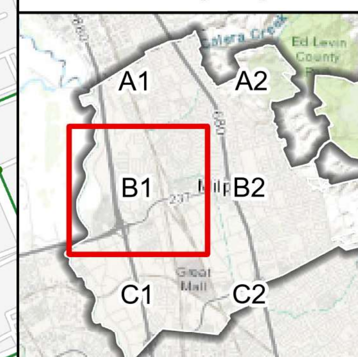
- 0 - 1.0
- > 1.0

*Pipe annotation: "x to y" where x is existing size and y indicates proposed

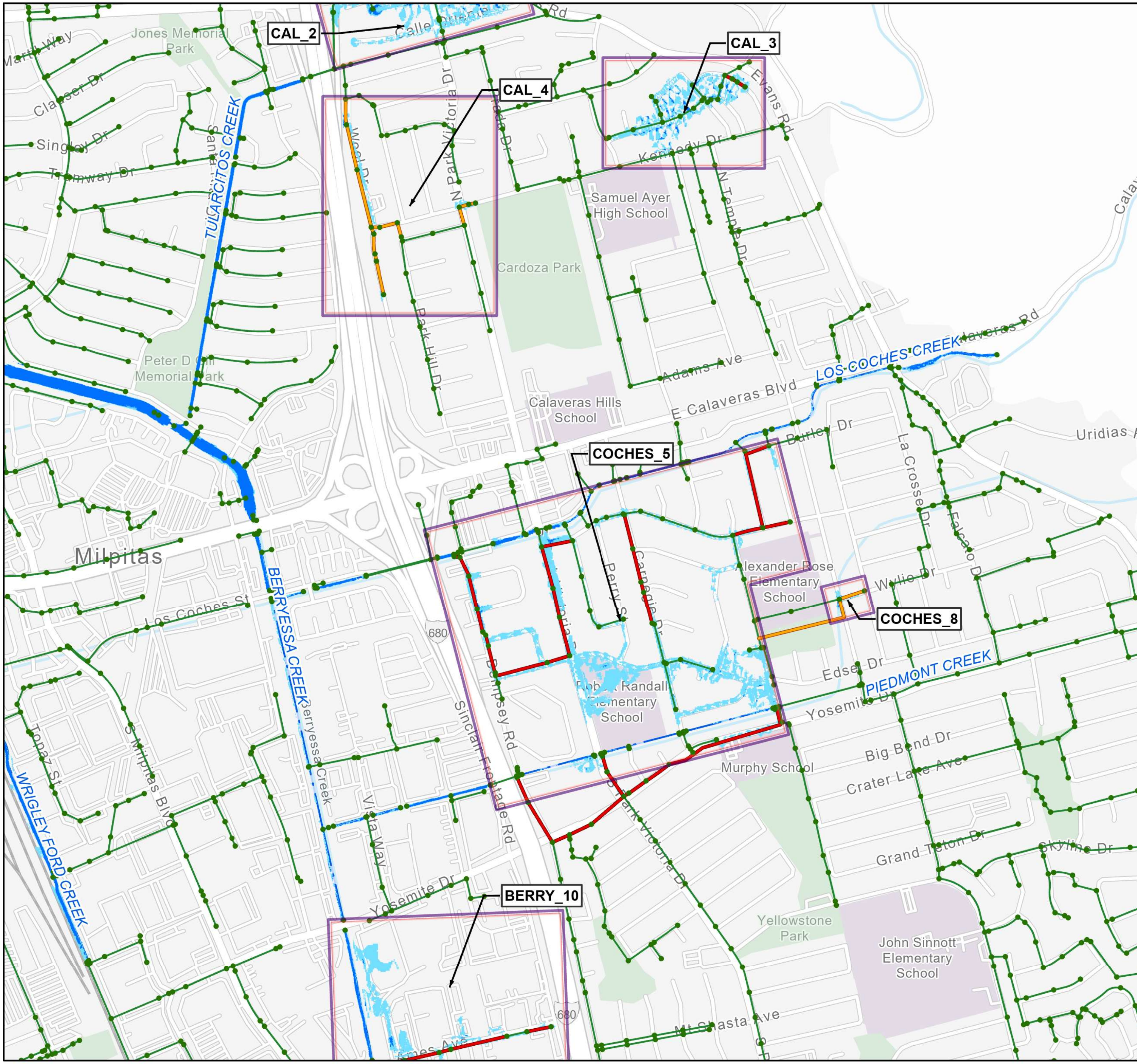
N

1 inch : 1,200 feet

Vicinity Map



Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS



Milpitas
Storm Drainage CIP

B2

SD Improvements

Page 4 of 6

Legend

- Storm Drain Manhole
- Modeled Storm Drain
- River Reach
- Inundation Addressed

CIP Priority

- Low
- High

Flood Depth (ft)

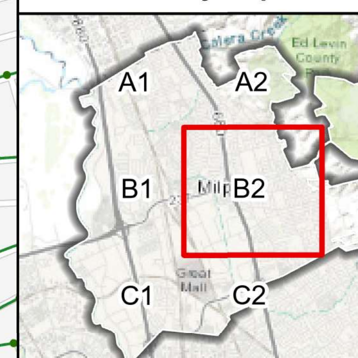
- 0 - 1.0
- > 1.0

*Pipe annotation: "x to y" where x is existing size and y indicates proposed

N

1 inch : 1,200 feet

Vicinity Map



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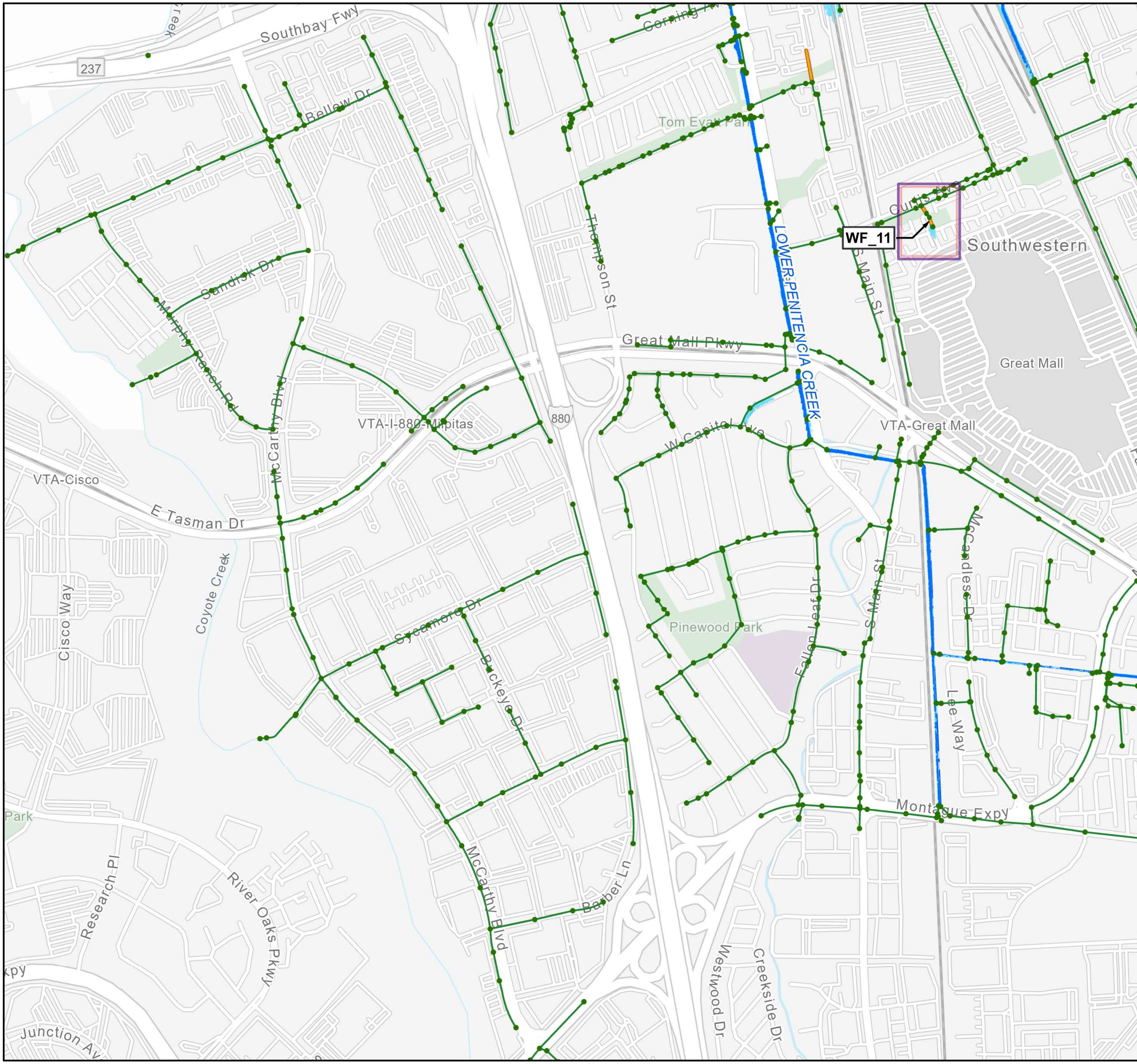


Milpitas Storm Drainage CIP

C1

SD Improvements

Page 5 of 6

Legend

- Storm Drain Manhole
- Modeled Storm Drain
- River Reach
- Inundation Addressed

CIP Priority

- Low
- High

Flood Depth (ft)

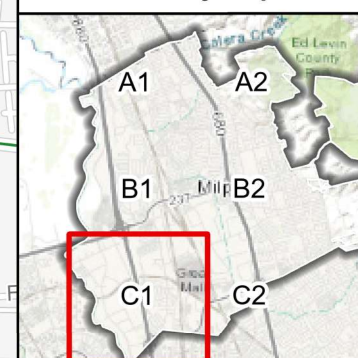
- 0 - 1.0
- > 1.0

*Pipe annotation: "x to y" where x is existing size and y indicates proposed

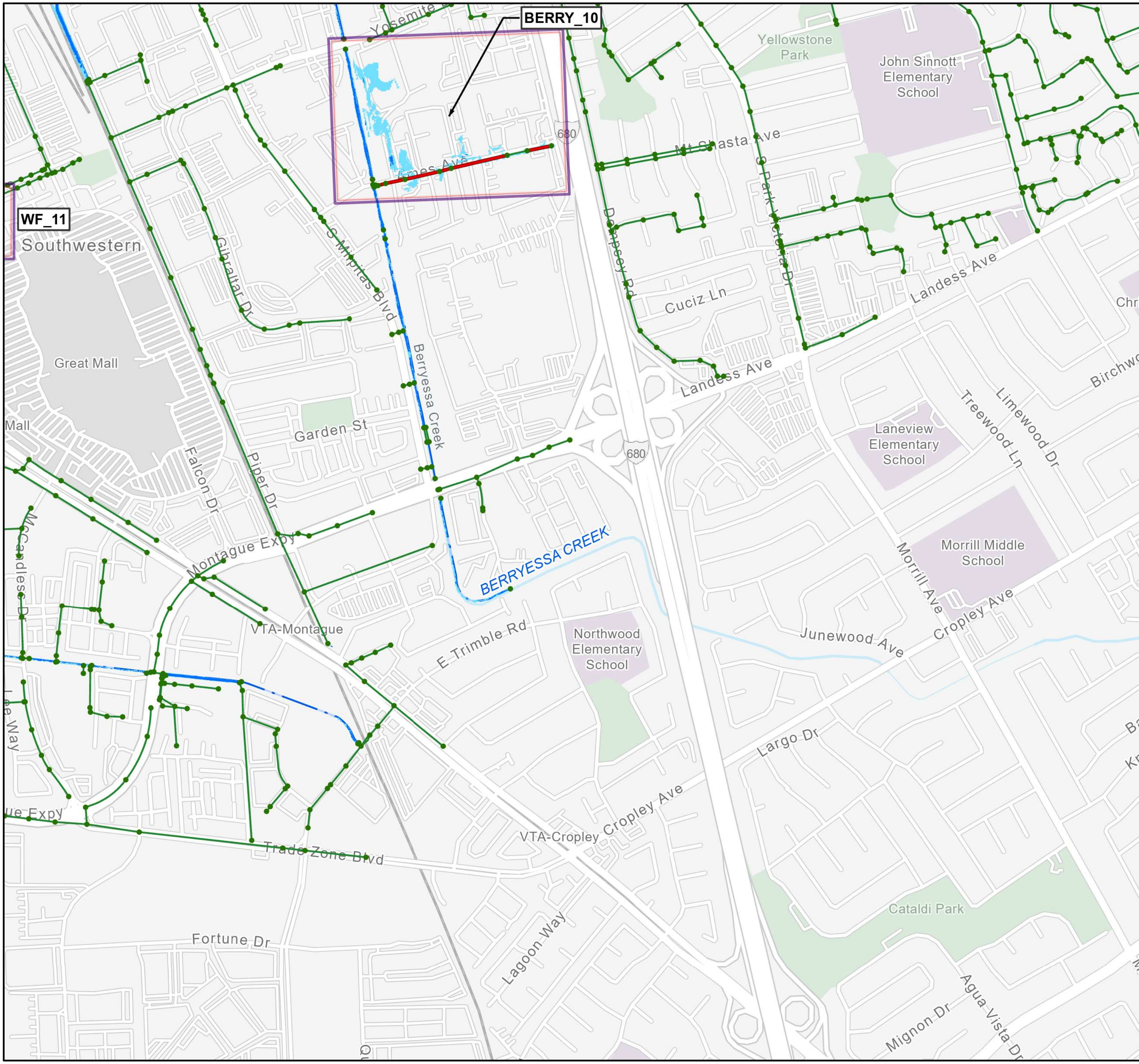
N

1 inch : 1,200 feet

Vicinity Map



Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS



Milpitas Storm Drainage CIP

C2

SD Improvements

Page 6 of 6

Legend

- Storm Drain Manhole
- Modeled Storm Drain
- River Reach
- Inundation Addressed

CIP Priority

- Low
- High

Flood Depth (ft)

- 0 - 1.0
- > 1.0

*Pipe annotation: "x to y" where x is existing size and y indicates proposed

N

1 inch : 1,200 feet

Vicinity Map

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Table 3-1: Capital Improvement Projects

ID	Project	Priority	Replacement Option	Parallel Option
CAL_2	Jacklin Road	High	Install approximately 950 LF of 30-inch RCP along Jacklin Rd between Calle Oriente and N Park Victoria Dr.	Use replacement option.
CAL_3	Bayview Park Drive	High	Replace approximately 210 LF of existing 12-inch RCP on Bayview Park Dr with 24-inch RCP.	Install approximately 210 LF of 24-inch RCP on N Bayview Dr.
COCHES_5	Coches and Piedmont Creek	High	Replace approximately 490 LF of existing 12-inch RCP on Edsel Dr between Dempsey Rd and Shirley Dr with 36-inch RCP. Replace approximately 640 LF of existing 18-inch RCP and approximately 710 LF of existing 21-inch RCP with 36-inch RCP on Dempsey Road between Edsel Dr to the outfall.	Install approximately 1840 LF of 36-inch RCP on Edsel Dr between Dempsey Rd and Shirley Dr and on Dempsey Road between Edsel Dr to the outfall.
			Install approximately 350 LF of 18-inch RCP from Rodrigues Ave to S Park Victoria Dr. Install approximately 1390 LF of 36-inch RCP from S Park Victoria Dr to Edsel Dr., directing flow away from Los Coches Creek.	Use replacement option of installing approximately 350 LF of 18-inch RCP from Rodrigues Ave to S Park Victoria Dr. Install approximately 1390 LF of 36-inch RCP from S Park Victoria Dr to Edsel Dr., directing flow away from Los Coches Creek.
			Replace approximately 710 LF of existing 27-inch RCP and 490 LF of existing 30-inch RCP with 36-inch RCP on Carnegie Dr between Mercury Ct and Canton Dr.	Replace approximately 710 LF of existing 27-inch RCP and 490 LF of existing 30-inch RCP with 36-inch RCP on Carnegie Dr between Mercury Ct and Canton Dr.
			Connect existing at Burley Drive outfall to new improvements where Lawton Dr and Canton Dr intersects with approximately 920 LF of new 24-inch RCP to direct flow away from Los Coches Creek.	Use replacement option for connect existing at Burley Drive outfall to new improvements where Lawton Dr and Canton Dr intersects with approximately 920 LF of new 24-inch RCP to direct flow away from Los Coches Creek.
			Replace approximately 260 LF of existing 12-inch RCP with 18-inch RCP on Canton Dr between Beacon Dr and Lawton Dr. Replace approximately 280 LF of existing 12-inch RCP with 24-inch RCP on Canton Dr between Lawton Dr and Roswell Dr.	Install 15-inch RCP on Canton Dr between Beacon Dr and Lawton Dr. Install 24-inch RCP on Canton Dr between Lawton Dr and Roswell Dr.
			Install approximately 1260 LF of 54-inch RCP on Yosemite Dr between Roswell Dr and Zion Ct. Replace approximately 250 LF of existing 12-inch RCP with 54-inch RCP on Yosemite Dr between Zion Ct and S Park Victoria Dr. Install approximately 390 LF of 54-inch RCP on Yosemite Dr from S Park Victoria Dr to existing main between S Park Victoria Dr and Dempsey Rd. Replace approximately 220 LF of existing 18-inch RCP and approximately 170 LF of existing 33-inch RCP on Yosemite Dr between S Park Victoria Dr to Dempsey Rd with 60-inch RCP. Replace approximately 690 LF of existing 42-inch RCP on Dempsey Rd	Use replacement option.

ID	Project	Priority	Replacement Option	Parallel Option
			between Yosemite Dr and the outfall with 60-inch RCP. Remove or abandon approximately 274 LF of existing 39-inch RCP, approximately 170 LF of existing 42 – inch RCP, and outfall along S Park Victoria Dr. Install approximately 425 LF of 30-inch RCP to direct flow away from Piedmont Creek from the 3 inlets along S Park Victoria Park Dr. currently draining to Piedmont Creek in approximately 25 LF of 15-inch RCP.	
BERRY_10	Ames Avenue	High	On Ames Ave replace approximately 230 LF of existing 18-inch RCP with 24-inch RCP. Replace approximately 640 LF of existing 24-inch RCP with 30-inch RCP. Replace approximately 510 LF of existing 24-inch RCP with 42-inch RCP. Replace approximately 80 LF of 30-inch RCP and approximately 50 LF of existing 30-inch CMP with 42-inch RCP.	Install approximately 1050 LF of 24-inch RCP and approximately 430 LF of 42-inch RCP on Ames Ave.
LP_14	North Abel Street	Low	Replace approximately 160 LF of existing 12-inch RCP on Maple Ave with 24-inch RCP.	Parallel option: Install approximately 160 LF of 24-inch RCP on Maple Ave.
			Along Redwood Ave, replace approximately 740 LF of existing 15-inch RCP and 400 LF of existing 21-inch RCP with 30-inch RCP, and 160 LF of existing 24-inch RCP with 36-inch RCP.	Install approximately 1,300 LF of 24-inch RCP in Redwood Ave from Heath St to the existing Abbott Ave SD.
			On Abbott Ave, replace approximately 400 LF of existing 15-inch RCP with 24-inch RCP between Walnut Dr and Elm Ave; approximately 255 LF of existing 21-inch RCP from Elm Ave to Willow Ave and approximately 250 LF of existing 24- inch RCP with 42-inch RCP from Willow Ave to Chestnut Ave. Install approximately 520 LF of 42-inch RCP in Abbott Ave from Chestnut to Redwood Ave.	Install approximately 1,425 LF of 36-inch RCP in Abbott Ave from Walnut Dr to Redwood Ave.
			Along Chestnut Ave, replace approximately 270 LF of existing 18- inch RCP, 420 LF of existing 21-inch RCP, and 370 LF of existing 24-inch RCP with 42-inch RCP.	Install approximately 1,060 LF of 36-inch RCP in Chestnut Ave from Heath St to Abbott Ave.
			Replace approximately 50 LF of 12-inch RCP and 520 LF of 15-inch RCP with 42-inch RCP in Heath St from Elm Ave to Chestnut Ave.	Install approximately 520 LF of 36-inch RCP in Heath St from Elm Ave to Chestnut Ave.
CAL_4	Wool Drive	Low	Replace approximately 315 LF of existing 15-inch RCP and approximately 350 LF of existing 18-inch RCP on Moretti Ln to Kennedy Dr with 24-inch RCP. Replace approximately 70 LF of existing 18-inch RCP and approximately 540 LF of existing 27-inch RCP with 30-inch RCP on Moretti Ln and Wool Dr adjacent to	Install approximately 570 LF 24-inch RCP on Moretti Ln to Kennedy Dr. Install approximately 1270 LF 30-inch RCP on Wool Dr from Kennedy Dr.

ID	Project	Priority	Replacement Option	Parallel Option
			Kennedy Dr. Replace approximately 660 LF of existing 27-inch RCP on Wool Dr with 36-inch RCP between Kennedy Dr and Traughber St.	
			Replace approximately 150 LF of existing 12-inch RCP on Kennedy Dr and approximately 170 LF of existing 15-inch RCP on N Park Victoria Dr between Kennedy Dr and Park View Dr with 18-inch RCP.	Install approximately 150 LF of 18-inch RCP on Kennedy Dr and approximately 170 LF of 18-inch RCP on N Park Victoria Dr between Kennedy Dr and Park View Dr.
			Replace approximately 920 LF of existing 27-inch RCP extending from Park View Dr to Kennedy Dr to where it intersects with Wool Dr with 30 RCP.	Install approximately 920 LF of 18-inch RCP extending from Park View Dr to Kennedy Dr to where it intersects with Wool Dr.
COCHES_8	Foothill Park	Low	Replace approximately 250 LF of existing 12-inch RCP with 24-inch RCP on Wylie Dr between Bixby Dr and Temple Dr. Install approximately 150 LF of 30-inch RCP along S Temple Dr. from Wylie Dr. and then approximately 800 LF of 30-inch RCP between S Temple Dr. and Roswell Dr.	Use replacement option.
CAL_1	Tice Drive	Low	Replace approximately 260 LF of existing 12-inch RCP and approximately 290 LF of existing 15-inch RCP on Tice Dr. between Rivera St and Horcajo St with 18-inch RCP.	Install approximately 610 LF of 18-inch RCP on Tice Dr between Rivera St and Horcajo St.
WF_11	Comet Drive	Low	Replace approximately 220 LF of existing 12-inch RCP with 24-inch RCP on Comet Dr between Metro Walk Dr and Curtis Ave.	Install approximately 220 LF of 24-inch RCP on Comet Dr between Metro Walk Dr and Curtis Ave.
WF_13	Railroad Avenue	Low	Install approximately 350 LF of 36-inch RCP from Railroad Ave to Marylinn Dr. Install approximately 1020 LF of 42-inch x 28-inch rectangular RCP along Marylinn Dr to N Abel St. Replace the approximately 150 LF of existing 24-inch RCP with 42-inch x 28-inch rectangular RCP.	Use replacement option.
LP_12	Main Street – Serra Way	Low	Replace approximately 190 LF of existing 12-inch RCP on S Main St with 15-inch RCP. Replace approximately 270 LF of existing 12-inch RCP from S Main St to Serra Way with 18-inch RCP.	Install approximately 190 LF of 15-inch RCP on S Main St. Install approximately 270 LF of 18-inch RCP from S Main St to Serra Way.

Table 3-2: High Priority Capital Improvement Program

ID	Name	Location	From	To	Replacement Option Size (in)	Replacement Option Lineal Feet	Parallel Option Size (in)	Parallel Option Lineal Feet
CAL_2	Jacklin Road	Jacklin Rd	Calle Oriente	N Park Victoria	30	950	Use replacement option	
CAL_3	Bayview Park Dr	Bayview Park Dr			24	210	24	210
COCHES_5	Coches and Piedmont Creek	Edsel Dr	Dempsey Rd	Shirley Dr	36	490	36	490
		Dempsey Rd	Edsel Dr	Outfall	36	1,350	36	1,350
		Canton Dr	Rodrigues Ave	S Park Victoria	18	350	18	350
		S Park Victoria	Canton Dr	Edsel Dr	36	1,390	36	1,390
		Carnegie Dr	Mercury Ct	Canton Dr	36	710	36	710
		Lawton Dr	Burley Dr	Canton Dr	24	920	24	920
		Canton Dr	Beacon Dr	Lawton Dr	18	260	15	260
		Canton Dr	Lawton Dr	Roswell Dr	24	280	24	280
		Yosemite Dr	Roswell Dr	Zion Ct	54	1,260	Use replacement option	
		Yosemite Dr	Zion Ct	S Park Victoria	54	250		
		Yosemite Dr	S Park Victoria	(E) Storm Drain	54	390		
		Yosemite Dr	S Park Victoria	Dempsey Rd	60	390		
		Dempsey Rd	Yosemite Dr	Outfall	60	690		
		S Park Victoria	S Park Victoria	Piedmont Creek	30	425		
BERRY_10	Ames Avenue	Ames Ave	Sinclair Frontage		24	230	24	230
		Ames Ave			30	640	24	610
		Ames Ave			42	510	24/42	210/300
		Ames Ave		Berryessa Creek	42	130	42	130

Table 3-3: Low Priority Capital Improvement Program

ID	Name	Location	From	To	Replacement Option Size (in)	Replacement Option Lineal Feet	Parallel Option Size (in)	Parallel Option Lineal Feet
LP_14	North Abel Street	Maple Ave	Abbott Ave	Lower Pen Ck	24	160	24	160
		Redwood Ave	Heath Street		30	1,140	24	1,300
				Abbot Ave SD	36	160		
		Abbott Ave	Walnut Dr	Elm Ave	24	400	24	1,425
		Abbott Ave	Elm Ave	Willow Ave	42	255		
		Abbott Ave	Willow Ave	Chestnut Ave	42	250		
		Abbott Ave	Chestnut Ave	Redwood Ave	42	520		
		Chestnut Ave	Heath St	Abbott Ave	42	1,060	36	1,060
		Heath St	Elm Ave	Chestnut Ave	42	570	36	520
CAL_4	Wool Drive	Moretti Ln		Kennedy Dr	24	665	24	570
		Wool Dr	Kennedy Dr		30	610	30	1,270
		Wool Dr		Traughber St	36	660		
		Kennedy Dr	Cardoza Park	N Park Victoria	18	150	18	150
		N Park Victoria	Park View Dr	Kennedy Dr	18	170	18	170
		Kennedy Dr	Park View Dr	Wool Dr	30	920	18	920
COCHES_8	Foothill Park	Wylie Dr	Bixby Dr	Temple Dr	24	250	Use replacement option	
		S Temple Dr	Wylie Dr	SD Easement	30	150		
		SD Easement	S Temple Dr	Roswell Dr	30	800		
CAL_1	Tice Drive	Tice Dr	Rivera St	Horcajo St	18	550	18	610
WF_11	Comet Drive	Comet Dr	Metro Walk Dr	Curtis Ave	24	220	24	220
WF_13	Railroad Avenue	Railroad Ave		Marylinn Dr	36	350	Use replacement option	
		Marylinn Dr	Railroad Ave	N Abel St	42x28	1,170		
LP_12	Main Street – Serra Way	S Main St		Serra Wy	15	190	15	190
		Serra Way	S Main St	Serra Wy	18	270	18	270

Underserved Areas

Potential future CIP projects have been identified within areas currently underserved by storm drainage infrastructure to address potential future development needs within those areas and the connection of new storm drainage infrastructure to the City's system. Main Street Gateway Specific Plan and Milpitas Metro Specific Plan (formerly Transit Area Specific Plan) are recognized as underserved areas (see Figure 3-3) within the city. The Milpitas Metro Specific Plan area is now almost entirely built out, and there are no longer identified areas that do not have adequate storm water runoff collection systems.

However, some vacant and underutilized parcels hold potential for development or redevelopment. Four (4) potential CIP projects are proposed to address those that do not have a storm drain line serving them already in the Main Street Gateway Specific Plan. These projects are identified in Table 3-4, and detailed maps are included in Appendix A. The identified projects have no priority. While they are not necessary to correct deficiencies within the existing storm drain system, they will need to be built when the developments they service occur.

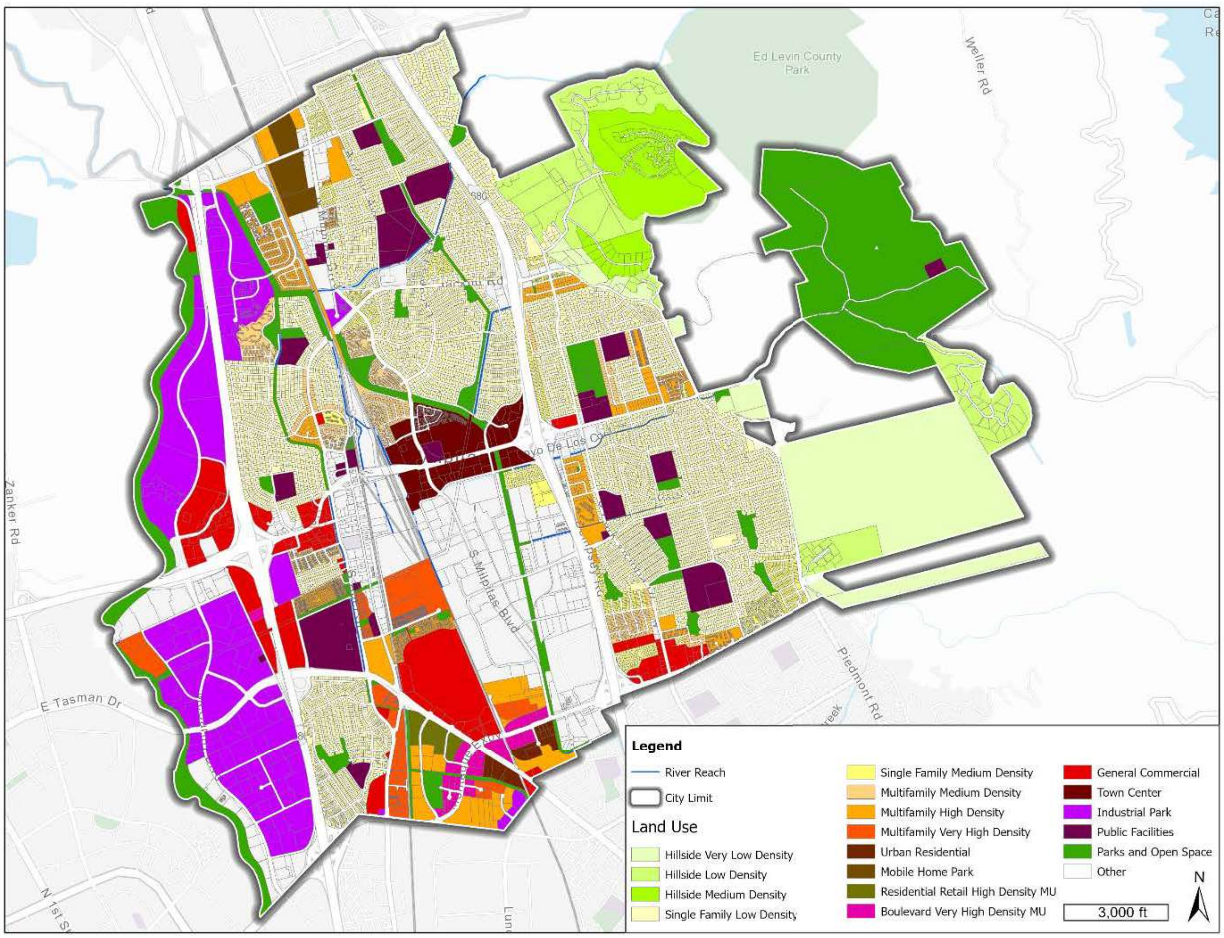


Figure 3-10: Milpitas Future Plan Land Use Map

Table 3-4: Capital Improvement Projects for Underserved Areas

ID	Project	Priority	Replacement Option	Parallel Option
LP_15	Main Street – Tom Evatt Park	---	Install approximately 170 LF of 18-inch RCP on S Main St to the existing 24-inch RCP on Carlo St.	Use replacement method.
LP_16	Main Street	---	Install approximately 450 LF of 18-inch RCP on S Main St to the improvement project of the existing system on Serra Way.	Use replacement method.
LP_17	Main Street – Sinnott Lane	---	Install approximately 70 LF of 18-inch RCP on S Main St from near Sinnott Ln to the improvement project of the existing system.	Use replacement method.
LP_18	Carlo Street	---	Install approximately 300 LF of 18-inch RCP on S Main Street to the existing system.	Use replacement method.

Multi-benefit and Green Street Projects

The City has developed a green stormwater infrastructure (GSI) plan as required under the NPDES Municipal Regional Permit Section C.3.j. The plan relies on information produced in the Santa Clara Basin Stormwater Resource Plan. There are several green streets and public parcel projects identified in the plan that are depicted in Figure 3-11. It may be beneficial to construct these green stormwater infrastructure projects simultaneously as the CIP identified herein. These opportunities are identified where GSI projects and storm drain CIP projects overlap or are directly adjacent to one another. These opportunities are listed in Table 3-5. For those CIP projects which do not overlap with green stormwater infrastructure projects identified, each should be reviewed for GSI potential even if it is not included in the GSI Plan.

Table 3-5: Capital Improvement Projects to Include Green Stormwater Infrastructure

CIP ID	Project	GSI Location	GSI Type
BERRY_10	Ames Avenue	Street	Green Street
COCHES_5	Carnegie Drive	Street	Green Street
COCHES_5	South Park Victoria	Street	Green Street
COCHES_5	Yosemite Drive	Street	Green Street and detention
COCHES_8	Foothill Park	Park/Open Space	Green Street and detention
WF_11	Comet Drive	Park/Open Space	Green Street and detention

For Green Street projects, the street greening could be included as part of the design of the storm drain system capacity upgrade when on the same stretch of street. Detention projects would require more detailed concept design to accomplish both flood control and green infrastructure goals. Flooding identified near Foothill Park could potentially be remedied with an integrated project which greens the park and provides detention storage. Similarly, the Yosemite Drive and South Park Victoria projects could potentially be remedied by storage facilities in Merryhill and Robert Randall schools, respectively. Storage facilities are discussed in more detail in subsequent sections.

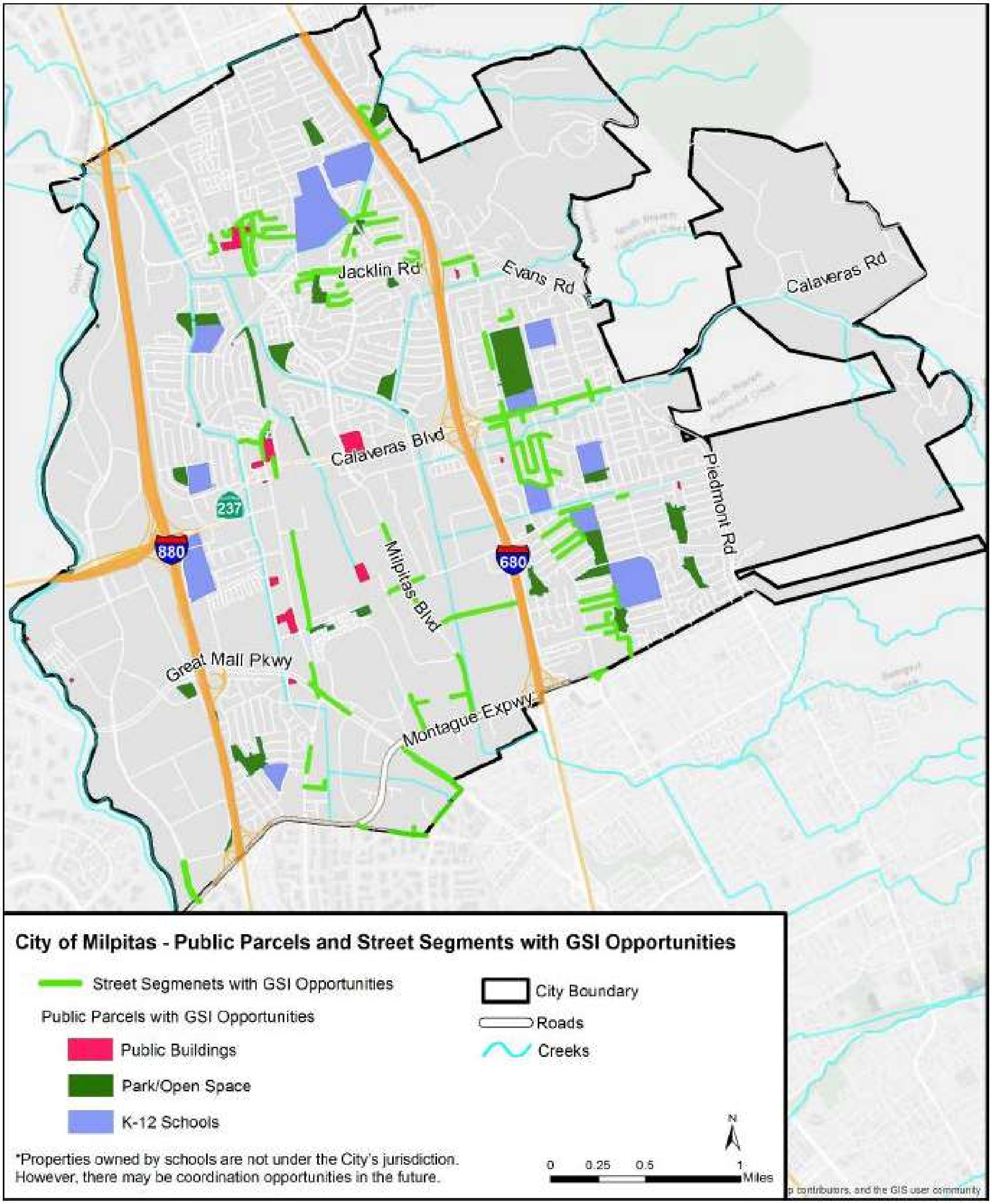


Figure 3-11: Green Stormwater Opportunity Map from Santa Clara Basin Stormwater Resource Plan

Storm Trouble Spots

City staff have identified 15 inlet locations that flood during storms. These are shown in Figure 3-12. To address these storm trouble spots, inlet replacements are proposed for these locations except for the one at Evans at Bayview Park. That storm trouble spot will be addressed by the Bayview Park Drive CIP project.

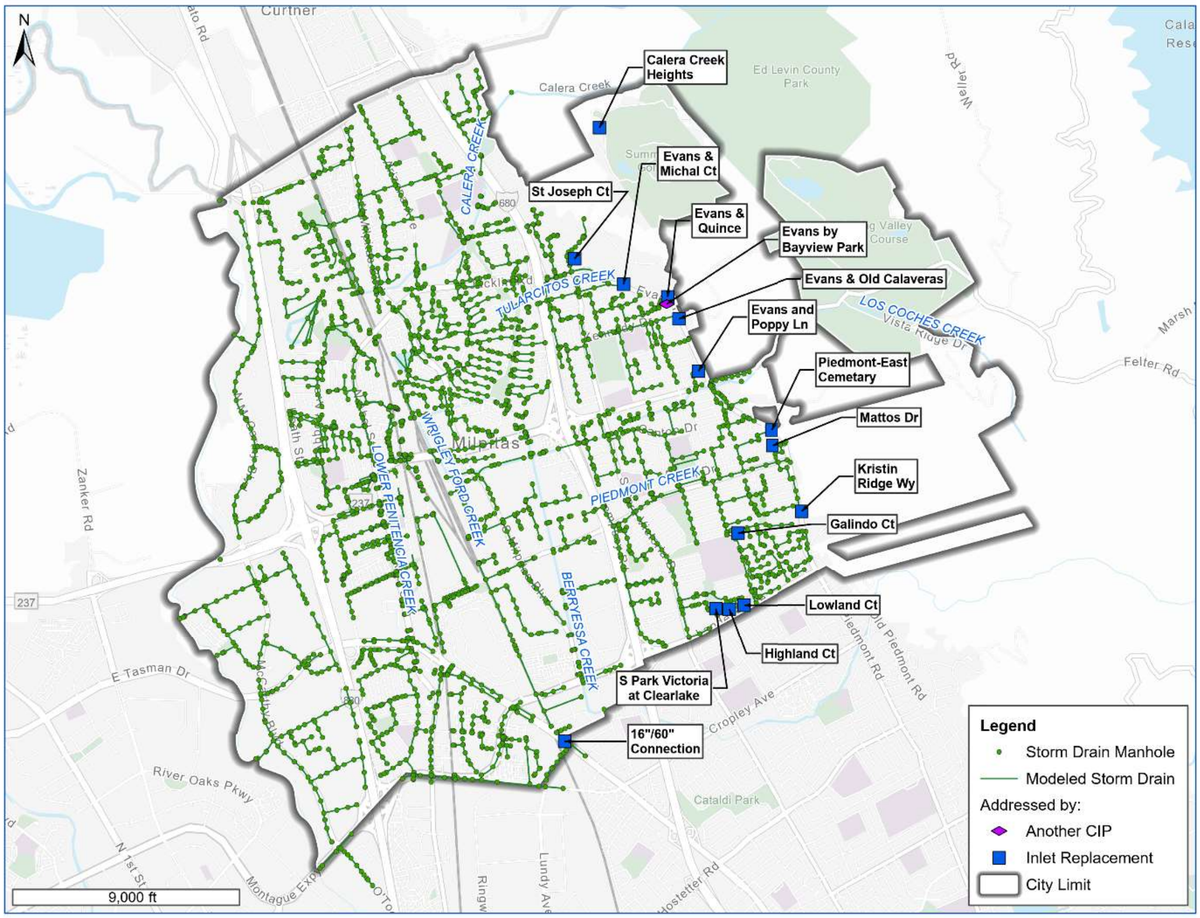


Figure 3-12: Storm Trouble Spots

Storage Facilities

Two basic categories of stormwater storage are commonly used: detention and retention. Some facilities blur the distinction between the two but, in general:

1. Detention refers to the temporary storage of incoming runoff that exceeds the permissible release. After the storm event, the facility empties and returns to its natural function – such as a parking lot or park.

2. Retention facilities, on the other hand, hold on to the excess runoff for an indefinite period. Natural ponds and lakes exemplify retention facilities where water levels change only through evaporation, infiltration, and additional storm runoff.

With the tight clay that underlies much of Milpitas, true retention facilities are not advantageous. However, several storage facilities in the city do serve a dual role for both stormwater detention and retention. For instance, pumps are used to move to attenuate flood waves through the facility, but a permanent pool of water remains behind for aesthetic (or perhaps recreational) purposes.

Properly designed, constructed, and maintained stormwater storage facilities could reduce peak flows, thereby better utilizing the capacity of downstream conveyance facilities. Such facilities can also potentially mitigate the need for system upgrades. The efficacy of any detention facility and ancillary improvements in the quality of storm runoff to receiving waters needs evaluation on a case-by-case basis. However, some general design criteria need to be applied to every basin:

1. Basins should be sized so that their output does not exceed the downstream facilities' design capacity.
2. There must be an overflow section capable of safely discharging the 100-year peak inflow (should outlet works become clogged) without causing property damage.
3. At least one foot of freeboard over the maximum 100-year water surface elevation should be provided for excavated basins. Three feet of freeboard (minimum) must be provided where berms or levees create basins.
4. Infiltration capacity shall not be considered when designing basins, unless percolation rates are determined by on-site soil testing certified by a Civil or Geotechnical Engineer.
5. Debris and sediment loading must be considered in design (see below).
6. Ponds and basins need to be designed with shallow side slopes (5:1 minimum) so that people and animals may extricate themselves from the water should the need arise. A safety shelf may also be considered. Facilities that pose an inordinate risk to the public should be fenced off. Openings larger than six inches in diameter must be screened to protect children and animals.
7. A mechanism for draining the basin should be provided. If the basin also serves as a pumping forebay, the pumping facilities must fully dewater the basin.
8. Facilities designed for the permanent (or semi-permanent) retention of water should be deep enough to avoid eutrophication (accumulation of excess nutrients that stimulates plant growth) and breeding insects. Pond surface areas should be at least one-half acre, with a minimum depth of 10 feet over at least a quarter of the area. The average depth over the rest of the pond needs to be at least five feet. Basin outlets should be positioned opposite the inlet to promote circulation. Stocking permanent ponds with fish also promote good water quality.
9. Underdrain systems to minimize wetness should be considered for detention facilities not intended as permanent water features. This helps prevent the facility from encouraging insect populations and provides for a quicker return to its dry weather function.
10. Basin bottoms and sides should be stabilized with vegetation to withstand periodic flooding and prevent erosion. Basin outlets need to be provided with erosion protection, such as riprap.

Debris Loading

Detention and retention basins will eventually fill up with sediment and other debris, reducing their storage capacity to where they will not operate as designed. Therefore, some considerations of debris loading must be made for each basin. Depending upon the desired frequency of maintenance, some allowance for "dead" storage should be made to handle sediment and debris. Based on work by Schaaf & Wheeler for the Santa Clara Valley Water District, the following empirical relationships (debris load per unit drainage area) are used to evaluate debris loading:

Highly urban areas	0.1 acre-feet/mi^2/year
Hillside open space	0.4 acre-feet/mi^2/year

Chapter 4: Pump Stations

Each of Milpitas' thirteen (13) stormwater pumping stations is evaluated based on the set of criteria described herein. Detailed pump station assessment evaluation criteria are presented in Appendix B. This chapter describes how well each of the City's pumping facilities performs against the established performance criteria, identifies those stations with deficiencies, prioritizes the correction of said deficiencies, and establishes the requisite master plan improvements to remedy those deficiencies.

Pump Station Performance Criteria

Stormwater pump stations owned and operated by Milpitas must meet, at a minimum, the criteria established herein. If a pump station is substantially improved or rehabilitated, the performance and design guidelines provided in the Appendix should be followed.

Capacity

Every pump station should be capable of discharging the 100-year runoff from its tributary area. One way to accomplish this is a combination of pumping capacity and retention storage. Pump stations with lesser capacity (e.g. a 10-year capacity) may only be considered if there is a fail-safe way to overflow excess flows without causing property damage. Nearly all the pumping facilities within the city meet these criteria. Table 4-1 compares current pump station capacities to the potential 100-year inflow.

Number of Pumps

For redundancy, in every pump station, installing at least two identical pumps is necessary. It is unnecessary to include standby pumps because providing excess capacity is expensive and not justified by the relatively small risk of having a major storm event coincide with mechanical failure. (Schedule pump maintenance for the summer months.)

No pumping station in Milpitas is equipped with fewer than two identical pumps. Most stations have three main pumping units, and the Jurgens Pump Station has four. Each of the stations (except California Circle, Abbott, and Minnis) has a smaller electric dewatering pump to drain the wet well when water falls below the minimum allowable pumping level for the large stormwater pumps. Permanent retention ponds are maintained at the California Circle, Berryessa and Abbott stations, eliminating a small dewatering pump's utility. In contrast, the Minnis and Berryessa stations utilize submersible pumps capable of nearly completely dewatering their respective wet wells.

Standby Power

An emergency engine-generator capable of starting the largest motor while simultaneously running all other motors and auxiliary loads should be installed at each stormwater pump station where the primary pump drivers are electric motors. Pump stations without standby power or engine-driven pumps are at risk of becoming inoperable during an electrical power outage.

The lack of adequate automatic standby power is a potentially significant deficiency. When mapping special flood hazards, FEMA will only consider pumping capacity for those pumps with engine drive units or motor drivers that can be started and operated at the station with an automatic standby power generator installed. Portable generators and manual power transfer capabilities are not sufficient.

Pump Station Evaluations

Table 4-1 provides a summary of pump station capacities and emergency readiness throughout Milpitas. Further detailed evaluation for each station follows the identified deficiencies and recommended improvements. Figure 4-1 shows pump station locations within the city. When evaluating pump station capacity, available storage is in consideration. Table 4-2 provides a summary of pump stations with recommended CIP.

Table 4-1: Pumping Station Summary

ID	Facility	Year Built	Approximate Capacity	Primary Drivers	Standby Power	Description
1	California Circle Pump Station	1983	100-year	Engines	n/a	Page 4-4
2	Jurgens Pump Station	1989	10-year	Engines	n/a	Page 4-6
3	McCarthy Pump Station	1994	100-year	Engines	n/a	Page 4-8
4	Abbott Pump Station	1983	100-year	Motors	NO	Page 4-9
5	Minnis Pump Station	1978	100-year	Motors	NO	Page 4-11
6	Penitencia Pump Station	1960	100-year	Engines	n/a	Page 4-12
7	Wrigley-Ford Pump Station	1993	100-year	Engines	n/a	Page 4-15
8	Berryessa Pump Station	1977	100-year	Engines	n/a	Page 4-16
9	Manor Pump Station	1993	100-year	Motors	n/a	Page 4-18
10	Spence Creek Pump Station	1988	100-year	Motors	NO	Page 4-19
11	Bellew Pump Station	1985	100-year	Motors/ Engine	YES	Page 4-22
12	Murphy Pump Station	1983	100-year	Engines	n/a	Page 4-23
13	Oak Creek Pump Station	1979	100-year	Engines	n/a	Page 4-24

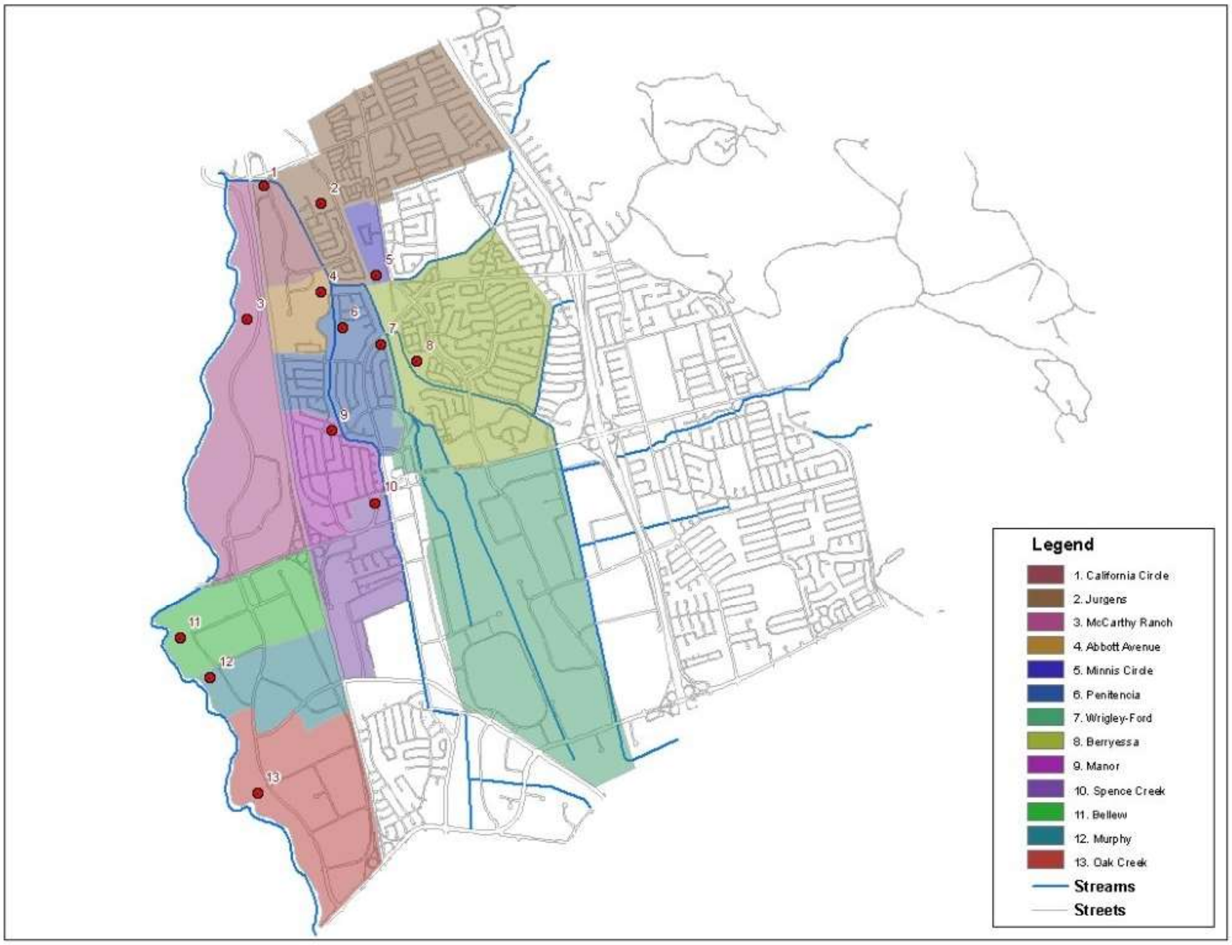


Figure 4-1: Storm Water Pump Stations in Milpitas

Table 4-2: Pump Station CIP Summary

Pump Station	Priority	CIP Details
Jurgens	High	Pump station replacement to provide increased level of design protection and eliminate the periodic flooding of Dixon Landing Park
Abbott	Low	Provide complete station replacement, including raising floor above base flood elevation
Penitencia	High	Complete pump station replacement
Spence Creek	Low	Provide permanent standby power
Murphy	High	Control system rehabilitation
Oak Creek	High	Control system rehabilitation

California Circle Pump Station

Facility ID	**SD-1**
Location	California Circle at Dixon Landing Road
Discharges to	Lower Penitencia Creek at STA 15+00
WSEL at Discharge Location	11.4 feet NAVD
Pipe Discharge Elevation	Invert 13.8 feet NAVD
Storage	2.5-acre wet pond
Design Lagoon Elevation	9.9 feet NAVD
Top of Lagoon Bank	14.0 feet NAVD
Tributary Area	263 acres
Station Capacity	117 cfs

This facility drains a retention pond located at the intersection of Dixon Landing Road and Interstate 880. The lagoon is designed as a wet pond with standing water; the normal minimum water surface elevation is 4.5 NAVD. Stormwater is pumped through three 28-inch diameter (SDR 26) high-density polyethylene (HDPE) pipes to Lower Penitencia Creek, near the top of the levee. This facility was originally designed to drain an industrial area of 150 acres. However, a detailed accounting of tributary area based on Interstate 880 / Highway 237 freeway interchange plans indicates that 263 acres are potentially tributary to the lagoon, as tabulated below.

Table 4-3: Areas Tributary to California Circle Lagoon

Location	Land Use	Tributary Area (acres)
Abbott Avenue	Residential	53
California Circle	Industrial	83
Route 880/237	Freeway	127
Total		263

Of these 263 acres, 210 acres (about 80 percent) are directly tributary to the lagoon and pump station. Runoff from the Abbott Avenue area can be discharged into Hall Park Lagoon and thence to Penitencia Creek through a storm drain outfall. Still, runoff above its capacity flows into the ditch running between Glenmoor Circle and Redwood Avenue and then into the freeway channel. The Abbott Lagoon drains the area between the outfall to Hall Park Lagoon on the south and the California Circle storm drain system on the north. This facility is adequate, so overflows are not anticipated from these potentially tributary areas, and they are not included in Table 4-3.

Since the last master plan update, engine components including starters and radiators have been replaced and the controls have been refurbished.

Equipment Schedule

Pumps	(3) Aurora 24P axial flow rated 17,000 gpm at 14 feet TDH (86hp)
Prime Power	(3) Caterpillar 3208 diesel engines rated at 175hp (2,400 rpm)
Standby Power	Not required
Fuel Storage	2,000 gallons for 96 hours run time at peak load with 3 pumps
Finish Floor Elevation	14.3 feet NAVD
Base Flood Elevation	12.5 feet NAVD

Resolution of Previously Identified Deficiencies

1. The finished floor elevation is six inches below the base flood elevation as shown on the currently effective Flood Insurance Rate Map (FIRM). Flood hazards depicted by the FIRM are based on spills from Lower Penitencia Creek becoming trapped behind the downstream levee adjacent to the pump station. However, unpublished flood hazard mapping based on the FEMA CTP work indicates this is not an issue. For the Storm Drain CIP, the CTP work is considered the best available information.

2. The discharge pipe invert at elevation 13.8 feet NAVD is two feet above the 100-year water surface elevation in Lower Penitencia Creek based on effective FEMA data. However, if the creek were to rise above the published elevation, creek water could potentially flow into the pond back through the discharge pipes when the pumps are off. Eventually, the volume of water that flows back into the lagoon will cause the pumps to start again, thereby eliminating the problem. When fewer than three pumps are operating, some water will be re-circulated through the system (which is inefficient). Still, since this situation is beyond the design condition, this deficiency does not require remedial action.

Capital improvements are not proposed for California Circle Pump Station, noting that the control systems were upgraded since the last master plan update in 2013.

California Circle Lagoon Operation

Surcharging storm drains within the California Circle area control the maximum allowable water surface in the lagoon. Due to the grade up to Dixon Landing Road, California Circle does not naturally release to the lagoon, so excess water on the street does not drain. Maximum design water surface elevations in the lagoon for the above-listed pumping levels and the lowest adjacent street grade, located on California Circle opposite Lower Penitencia Creek from Terra Mesa Way, are indicated in Table 4-4.

Table 4-4: California Circle Lagoon Operation

	10-year
Lowest Adjacent Street Grade (feet NAVD)	12.28
Maximum Lagoon Stage (feet NAVD)	6.51

Jurgens Pump Station

Facility ID	**SD-2**
Location	345 Jurgens Drive
Discharges to	Lower Penitencia Creek at STA 26+50
Design WSEL at Discharge Location	12.3 feet NAVD 88
Storage	1 ac-ft in City Park
Tributary Area	433 acres (residential)
Station Capacity	150 cfs
10-year Inflow	144.0 cfs
100-year Inflow	145.5 cfs
Deficit	Not applicable due to storage in Dixon Landing Park However, City desires to eliminate the need to store water in the park.

Located in Dixon Landing Park, this facility drains mixed residential areas between Penitencia Creek and Interstate 680 at Milpitas's northern end. The system was designed to function in tandem with detention storage in the park itself since the pump station is undersized even for a ten-year event. During the February 3, 1998 storm, Jurgens Pump Station was overwhelmed by storm runoff (albeit some from Berryessa Creek overflows) to the point at which engine batteries and other control equipment were inundated, thus shutting down the station. A subsequent investigation of local rainfall during the storm, however, indicated that even if Berryessa Creek had not spilled through a gap in its levee near the railroad, local runoff rates that exceeded pumping capacity would still have overwhelmed the station and caused its failure since control equipment was located less than one foot above the finished floor elevation.

A failed attempt was made to "flood-proof" the pump station by sealing floor openings and raising essential control equipment above the floor so that the equipment does not shut off during a flooding event. Based on the CTP model and assuming the pumps do not shut off during a storm, water will pond to the following elevation with the current pumps in operation.

$$WSEL_{100} = 12.0 \text{ feet NAVD (2 feet above finished floor)}$$

Maximum ten-percent flood limits are shown in Figure 4-2. Based on available topography and aerial photographs, the one-percent flooding does not inundate private property. Periodic inundation is limited to facilities within Dixon Landing Park, including the snack bar and restrooms.

Capital Improvement Program

To eliminate the temporary storage of excess runoff within Dixon Landing Park, a new station with a capacity of at least 243 cfs (110,000 gpm) is required. It is not feasible to retrofit the existing pumping facility to nearly double its capacity. Such a project would entail building an upsized replacement pump station while the existing station continues to operate in parallel, replacing the existing 72-inch diameter discharge pipe to Lower Penitencia Creek with at least a 96-inch diameter discharge pipe, and once the replacement pump station is fully tested and operational, demolishing the old pump station. This method of construction reduces risk to the City and eliminates the need for very costly bypass pumping capacity during construction. Park reconfiguration of the park will be necessary.

An order of magnitude estimate of construction cost is $10 to $15 million. This is part of the Capital Improvement Plan as a ***High Priority*** project with an estimated cost of $15 million in 2021 dollars.

Figure 4-2: 10-year Ponding Adjacent to Jurgens Pump Station

Equipment Schedule

Pumps	(4) Johnston 24PO axial flow rated 16,000 gpm at 10 feet TDH (700rpm, 60hp) (1) 3,000 gpm 25 hp electric jockey
Prime Power	(4) Caterpillar 3208 diesel engines rated at 150 hp (2,400 rpm) (4) Randolph right angle gear drives (7:2) rated at 110 hp
Standby Power	Not required
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	2,500 gallons; 125 hours at peak load (4 pumps)
Finish Floor Elevation	10.0 feet NAVD
Effective BFE	12.0 feet NAVD

Station Operation

In response to the February 1998 station shutdown, the City tried flood-proofed the equipment by sealing access openings on the floor and relocating the controls, so the station can continue to operate even with a base flood elevation two feet above the finished floor. Unfortunately, this has not prevented water from the wet well from entering the equipment room when the park floods. It is imperative that the engines do not shut down during a storm, so if the high priority pump station replacement project is delayed and the City believes that could happen, the wet well should be sealed. Since the last master plan update, improvements have been made to building ventilation, so the engines no longer overheat.

McCarthy Pump Station

Facility ID	**SD-3**
Location	1005 N McCarthy Boulevard
Discharges to	Coyote Creek at STA 145+00
Design WSEL at Discharge Location	18.6 feet (NAVD '88)
Storage	Wet Well
Tributary Area	185 acres (mixed use)
Station Capacity	400 cfs
10-year Inflow	259.3 cfs

Located in the McCarthy Ranch Development, this facility drains mixed-use areas between Coyote Creek and Interstate 880, north of State Highway 237. This station has excess capacity and the luxury of leaving one pump as standby. The facility is 25 years old, but every indication is that the pumping plant is well-maintained and operating as intended.

Equipment Schedule

Pumps	(3) Cascade 48AM axial flow (500 rpm, 560 hp, 60,000 gpm at 28 feet TDH)
	(1) Cascade 12MF 3,400 gpm 30 hp electric jockey
Prime Power	(3) Caterpillar 3412 diesel engines rated at 750 hp (2,100 rpm)
Standby Power	Not required
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	2,000 gallons; 22 hours at peak load (3 pumps)
Finish Floor Elevation	18.5 feet NAVD
Effective BFE	Shaded Zone X (area of moderate flood hazard)

Pump Station Operation

Capital improvements for increased capacity are not necessary for the McCarthy Pump Station. However, to enhance operational efficiencies and minimize pump cycling, it is recommended to have the pump starts rotate so that each motor will start no more than five times per hour.

During the February 2017 flood event on Coyote Creek, during which the water surface in the creek was within a few feet from the top of the levee at this pump station, backwater in the creek caused water to spill out of the pump station discharge surge chamber. Each of the pump discharge pipes has an individual 48-inch diameter flap gate. Access to the surge chamber is through a bolt-down steel plated cover designed for pressure conditions. Further investigation into the condition and operational efficacy of the bolt-down cover and each individual pump discharge flap gate within the surge chamber is recommended.

Abbott Pump Station

Facility ID	**SD-4**
Location	1225 N Abbott Avenue
Discharges to	Lower Penitencia Creek at STA 46+50
Design WSEL at Discharge Location	10.9 feet NAVD
Outfall Invert Elevation	18.3 feet NAVD
Storage	27 ac-feet in lagoon
Tributary Area	53 acres (park and industrial)
Station Capacity	24 cfs
Required Capacity	17 cfs
Excess Capacity	17 cfs
10-year Lagoon Level	9.5 feet NAVD
100-year Lagoon Level	10.3 feet NAVD

Located on Abbott Avenue, the facility serves as a recreational and aesthetic feature inside an industrial park. If the pump station is functioning properly, there is no problem with flooding in the area. Pump motors have recently been rewound, and the station is in good working order.

However, the prime drivers are electric motors without any provision for standby power. If the pump station's power supply were to fail during a 24-hour storm, the lagoon could exceed the maximum ponding level. Ponding levels above 12.0 feet NAVD will begin to flood the adjacent property, so making standby power provisions will reduce the risk of flooding in extreme events.

Since the last master plan update, the pump motors have been rebuilt.

Equipment Schedule

Pumps	(2) Aurora axial flow pumps rated 5,350 gpm at 16 feet TDH
Prime Power	(2) Westinghouse 30 hp vertical electric motors (480V, 3 phase)
Standby Power	None
Fuel Storage	n/a
Finished Floor Elevation	13.7 feet NAVD
Effective BFE	10.3 feet NAVD

Deficiencies

1. The pump station is not provided with standby power in the form of an emergency engine-generator set; so, if the power were to fail during an intense storm, adjacent properties could be flooded depending upon prior lagoon levels and duration of the power outage.

2. Abbott Pump Station discharges to Penitencia Creek via twin 18-inch diameter high-density polyethylene outfalls through the western levee without flap gates. However, the discharge outfalls are almost 1.5 feet above the design water surface in Penitencia Creek. Should water levels ever exceed the design freeboard, the situation would exceed the design condition because any water that runs back through the pump discharge pipes into the lagoon would eventually cause the pumps to start. Hence, this "deficiency" does not require remedial action.

Capital Improvement Recommendation

Providing emergency standby power is a ***Low Priority*** project associated with the Abbott station. (Note that engine-generator sizing is approximate only and requires a full load analysis.). The engine-generator should be in a building like the pump house to preserve this station's aesthetic feel. Estimated capital costs are summarized in Table 4-5.

Table 4-5: Medium Priority CIP for Abbot Pump Station

Capital Improvement	Estimated Cost
125 kW standby engine-generator	$225,000
Automatic transfer switch	$75,000
Electrical modifications	$100,000
Building for standby power equipment and site improvements	$200,000
Construction Subtotal	$600,000
Engineering, CM, and Administration (20%)	$120,000
Contingencies (50% nominal)	$380,000
	$1,100,000

Supplemental Recommendation

A style of pump with fewer maintenance requirements might be more appropriate at this pump station. In 2005 a pump specialist recommended the replacement of the existing line shaft pumps with axial flow submersible pumps because, in his opinion, they should require less maintenance and experience less corrosion. The pump specialist's recommendation is retained in this Storm Drain Master Plan as information only.

Repair and the replacement of parts for the two existing pumps would cost about $60,000. If the impellers are not available "off the shelf", the disassembled pump(s) would likely take up shop space while awaiting delivery of that part. This would add shop rental costs to the costs already enumerated. The cost to replace the existing Aurora Verti-Line 14P pumps with the same pump type would be about $300,000. Replacement of the existing pumps with axial flow submersible pumps requiring less maintenance is about $250,000.

Minnis Pump Station

Facility ID	**SD-5**
Location	1125 N Milpitas Boulevard
Discharges to	Calera Creek at STA 1+50
Design WSEL at Discharge Location	9.6 feet NAVD 88
Storage	None
Tributary Area	30 acres (commercial and industrial)
Station Capacity	33 cfs
10-year Inflow	8.5 cfs
100-year Inflow	20 .4cfs
Excess Inflow	12.6 cfs

Located off North Milpitas Boulevard, the Minnis Pump Station drains a low-lying area adjacent to Minnis Circle that cannot drain by gravity into Calera Creek. The station is located within a mapped 100-year special flood hazard area (Zone AH Elevation 16 feet NAVD). A projected capacity deficit exists for the 100-year inflow. However, even if this capacity deficit were to be corrected, the area would still be subject to 100-year flooding from Calera Creek until Valley Water solves Calera Creek's capacity issues. Therefore, improving pump station capacity has been downgraded from medium priority to low priority. However, when the Minnis station gets scheduled for long-term replacement (Chapter 9), pumping capacity should be increased to 100-year as described below.

The station is equipped with recently replaced submersible electric pumps and motors, and a set of plugs and automatic transfer switch for standby power if a portable generator can be made available.

The pump station is a duplex Flygt-style station with submersible pumps and motors mounted on a rail with a 14-inch quick disconnect discharge elbow. The pumps are housed in an 11-foot square underground structure. Personnel do not enter this structure but rather pull the rail systems pumps to the surface for lubrication and repair. Electrical meters and controls are enclosed in weatherproof housings and mounted on a pedestal above the pump access slab.

Since the last master plan update, the pumps and motors have been replaced and a portable standby power with ATS has been added.

Equipment Schedule

Pumps	(2) Flygt CP 3300 submersible electric rated 4,500 gpm at 45 feet
Standby Power	Trailer mounted portable with automatic start and transfer switch
Control Power	120 VAC (no backup)
Fuel Storage	n/a
Finished Slab	16.7 feet NAVD
Effective BFE	15.9 feet NAVD (Zone AH)

Deficiencies

Pump station capacity is not sufficient for the influent 100-year design flow, and without pumping, this water becomes trapped by the Calera Creek floodwall. Given that there is sufficient reaction time to bring portable standby power to the site and the pump station has more than 100-year discharge capacity, no capital improvement project is identified for Minnis Pump Station.

Penitencia Pump Station

Facility ID	**SD-6**
Location	La Honda Drive
Discharges to	Lower Penitencia Creek at STA 57+50
Design WSEL at Discharge Location	10.0 feet NAVD
Storage	Hall Memorial Park Lagoon
Tributary Area	215 acres (residential)
Station Capacity	65 cfs
10-year Inflow	64.4 cfs
100-year Inflow	80.4 cfs
10-year Lagoon Level	8.3 feet NAVD
100-year Lagoon Level	9.0 feet NAVD
Top of Lagoon Bank	14 feet NAVD

This ancient pump station sits across Penitencia Creek from the Hall Park Lagoon. A 60-inch gravity bypass pipe allows storm runoff to drain when creek levels are low. Another 60-inch pipe crosses beneath the creek and ties the lagoon to the pump station wet well. This pipe enters the lagoon in a bubble-up box equipped with a combination flap gate and slide gate. With the slide gate open, water levels in the lagoon and wet well equalize, so the system behaves as a single detention pond. In combination with available lagoon storage, the pumping station has sufficient capacity. A discharge standpipe, located above the creek floodwall elevation, provides backflow protection from Penitencia Creek. While the facility remains operational, it needs to be completely replaced soon.

Using the Jarad Global Positioning System and a rod to measure water depths, Schaaf & Wheeler conducted surveys of the lagoon between July 20 and July 25, 2000. The references used were the North American Vertical Datum of 1988 (NAVD88) and the North American Horizontal Datum of l983 (NAD83).

Since the last master plan update, the engines have been rebuilt.

Storage Capacity

Based on the survey, Hall Park Lagoon can store about 25 acre-feet before spilling north onto Abbott Avenue. Its summer water surface elevation is 6.4 feet, and the average depth of bottom sediment is about 1.5 feet. The lake overflows when its water surface elevation reaches about 13.5 feet.

Lagoon Odors

During the fall, when the City draws down the lake in preparation for winter storms, some neighbors have complained of odors. Adding oxygen can minimize odors caused by the activity of microbes in the sediment and water. Aerators were not operating at Hall Park during Schaaf & Wheeler's survey. Operating the aerators could reduce odors if the one-foot reduction in water surface during the winter is a problem because the lagoon becomes very shallow (about a foot deep). A microbiologist could help identify and implement further biological and chemical solutions.

Storm Drain Backup

All of the storm drain outfalls into the lagoon are above the summer water surface elevation of 6.4 feet, so lagoon water is not likely to back up into neighboring storm drains during the summer months. Design lagoon levels are based upon the 2000 survey of Hall Park Lagoon and the pumping equipment data and operating levels contained herein. Figure 4-3 shows the storage-elevation curve for the lagoon.

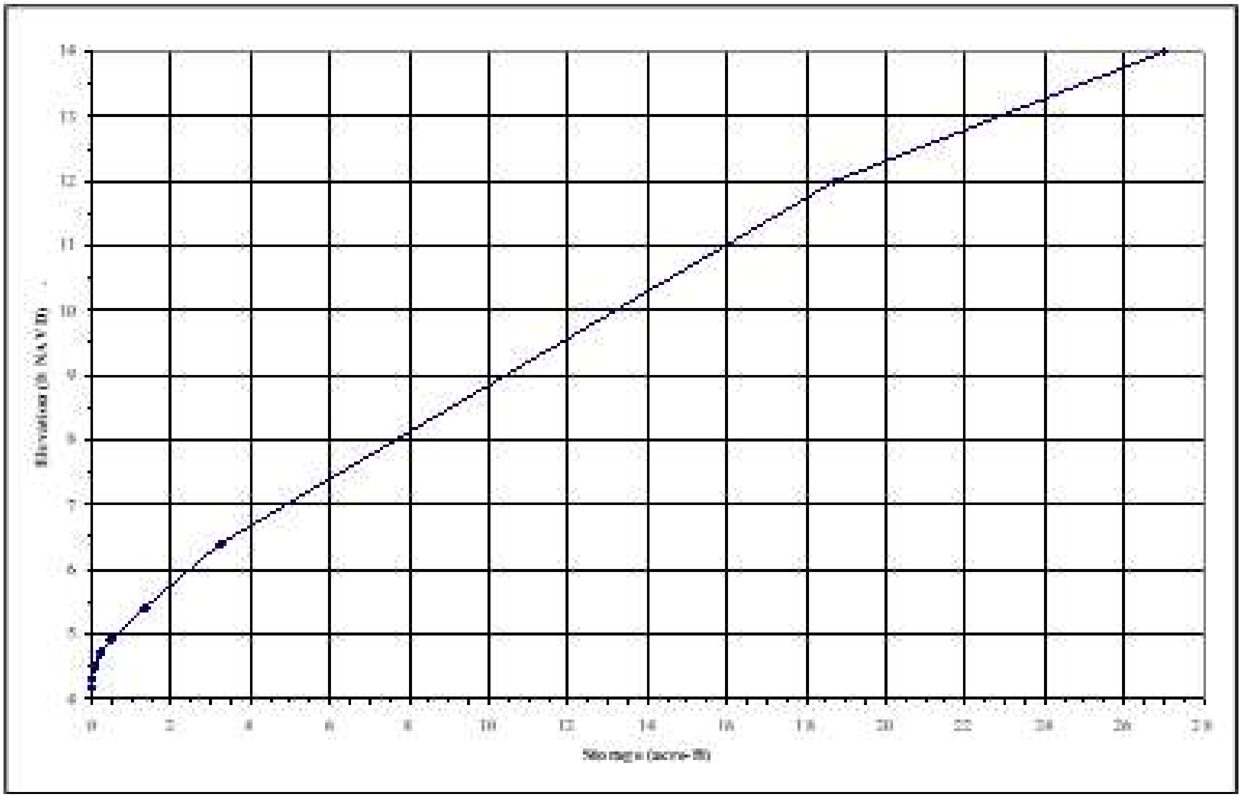


Figure 4-3: Storage Elevation Curve for Hall Memorial Park Lagoon

Pump Station Equipment Schedule

Pumps	(3) Fairbanks Morse 6310 axial flow (700 rpm, 40 hp, 9,750 gpm at 12 feet TDH)
	(1) Fairbanks Morse 6360 (840 gpm 7.5 hp electric jockey)
Prime Power	(3) Fiat 8041I05 diesel engines rated at 60 hp
Standby Power	Not required
Finish Floor Elevation	14.3 feet NAVD
Effective BFE	15.4 feet NAVD

Capital Improvement Recommendation

Given its age and the equipment condition, a complete station replacement for the Penitencia Pump Station is a ***High Priority***, including raising the floor above the base flood elevation. Based on a survey of available storage volume, the resulting 100-year water surface elevation of 10.1 feet is less than the spill elevation. It does not affect storm drain performance or recommended improvements, so the assumed pump station capacity and operation do not necessarily need to be modified. Detailed design will need to account for proper submergence for pump operation and maintain sump dimensions recommended by the Hydraulic Institute and pump manufacturers. The new axial flow pumps will likely be electric motor driven with a standby diesel engine-generator set that may require Tier 4 emissions control equipment. Estimated capital costs are provided in Table 4-6.

Table 4-6: High Priority CIP for Penitencia Pump Station

Capital Improvement	Estimated Cost
Demolish existing structure and equipment	$500,000
Wetwell and sump modifications, trash rack	$1,000,000
Furnish and install (3) axial flow pumps	$700,000
Furnish and install new motors and electrical panels	$1,250,000
New pump station building	$700,000
Automatic standby power generator	$750,000
Site grading and pump discharge outfall construction	$600,000
Construction Subtotal	$5,500,000
Engineering, CM, and Administration (20%)	$1,000,000
Contingencies (50% nominal)	$3,500,000
	$10,000,000

Wrigley-Ford Pump Station

Facility ID	**SD-7**
Location	Levee access from Marylinn Drive
Discharges to	Berryessa Creek at STA 24+00
Design WSEL at Discharge Location	10.1 feet NAVD
Storage	Forebay and channel storage
Tributary Area	760 acres (commercial and industrial)
Station Capacity	432 cfs
10-year Inflow	297.6 cfs
100-year Inflow	408.2 cfs
Excess Capacity	23.8 cfs

The downstream reach of Wrigley-Ford Creek was created when Valley Water realigned the original Berryessa Creek channel in 1974. To prevent Berryessa Creek flows from backing up into the old channel, a flood-gate structure with three 60-inch discharge pipes was built in 1976. At the time, Wrigley-Ford Creek’s high flows would combine with high Berryessa stages and flood residential properties adjacent to the old channel. High water surface elevations in Wrigley-Ford Creek also made local drainage to that creek problematic.

In 1991 Valley Water built the Wrigley-Ford Pump Station to pump tributary creek flows into Berryessa Creek, eliminating the local flooding and gravity drainage problems. This pump station is outfitted with a weir and low flow gravity bypass system so that the pumps only operate when hydrologic conditions warrant. Recirculation piping was also constructed, enabling the pump station to be tested before each storm season using a limited amount of water that is generally available year-round. A resistive load bank furnished for the standby diesel engine-generator set so that the EG-set may be exercised and tested against load during the summer months.

Equipment Schedule

Pumps	(3) Couch EC54 axial flow (240 rpm, 130 hp, 65,000 gpm at 5.8 feet TDH)
	(1) Flygt 3102X-441 submersible (500 gpm 5 hp electric jockey)
Prime Power	(3) US Motors Model RE 150hp, 1200 rpm horizontal electric motors
	(3) Amarillo Gear Co. 5:1 right angle propeller pump drive
Standby Power	400 kV Caterpillar 3406TA diesel engine-generator set (600 hp)
Fuel Storage	500 gallons; 24 hours with 3 pumps, 52 hours with 1 pump
Control Power	120 VAC backed up by 24 VDC batteries with charger
Finish Floor Elevation	20.7 feet NAVD
Effective BFE	16.4 feet NAVD

Pump Station Operation

Capital improvements are not necessary for the Wrigley-Ford Pump Station. Originally set pump operating levels may still be used, as they will ensure that the pumps do not start more than twice per hour as recommended by the motor manufacturer. The pumps regularly rotate, allowing all three pumps to alternate for lesser storm events, and using forebay and channel storage prevent cycling.

Berryessa Pump Station

Facility ID	**SD-8**
Location	Folsom Circle
Discharges to	Berryessa Creek at STA 48+75
Design WSEL at Discharge Location	10.3 feet (NAVD 88)
Storage	52 acre-feet based on 2000 survey of Hidden Lake
Tributary Area	550 acres (res. and commercial)
Station Capacity	150 cfs
10-year Inflow	187 cfs
100-year Inflow	329.1 cfs
Normal Lake Level	9.0 feet NAVD
10-year Lake Level	9.0 feet NAVD
100-year Lake Level	13.3 feet NAVD (not including Calera Creek overflows)
Allowable Lake Level	12.0 feet NAVD
Lake Spill Elevation	13.5 feet NAVD

Hidden Lake was originally constructed as a storm drainage detention facility to act as a forebay for the Berryessa Pump Station, serving residential and commercial areas on both sides of Berryessa Creek. A 60-inch diameter storm drain crosses the creek and drains the Beresford Meadows area and the Town Center. The current operating practice is to use this lake as an aesthetic amenity throughout the year. Residents have complained of objectionable odors and sights whenever the City has lowered the normal water level for winter pumping in the past.

Using the Jarad Global Positioning System and a rod to measure water depths, Schaaf & Wheeler conducted surveys of Hidden Lake between July 20 and July 25, 2000. The references used were the North American Vertical Datum of 1988 (NAVD88) and the North American Horizontal Datum of 1983 (NAD83). This lake can store about 52 acre-feet before spilling north onto Erie Circle (Figure 4-4). Its summer water surface elevation is 8.8 feet, and the average depth of bottom sediment is about 0.75 feet. The lake overflows when its water surface reaches about 13.5 feet in elevation. Local street grades are about 14 feet in elevation. Some flooding of adjacent properties can be expected in a 100-year runoff event once the lagoon elevation reaches about 12 feet.

Berryessa Pump Station was rehabilitated in 2006, including installing replacement equipment and the elevations of all controls to the flood-proofed elevation of 16.78 feet NAVD. Electrical equipment has been replaced since 2013. Although the building itself is not flood-proofed, equipment essential to pump function that would fail if submerged is raised above the regulatory flood elevation. The electric motor, air intake stationary louver, main distribution panel, metering panel, jockey pump starter, and backup diesel engine have all been raised above the minimum flood-proofing elevation. In addition, conduits are run from the ceiling. With these essential elements above water, the pumps can operate if the building itself is flooded. Electronic controls have also recently been replaced. Occasional problems with odors during low lake levels have been resolved using aerators.

Since the last master plan update, the electronics have been replaced.

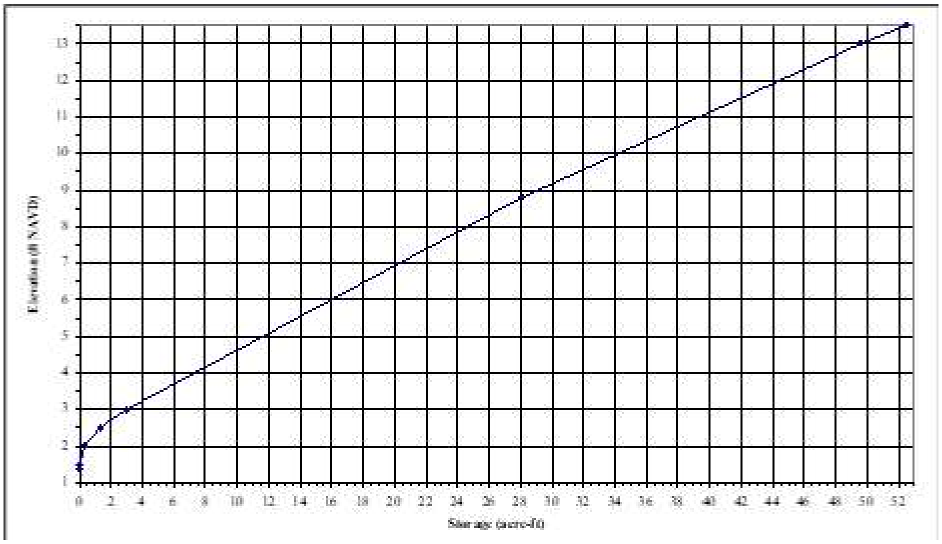


Figure 4-4: Storage Elevation Curve for Hidden Lake

Equipment Schedule

Pumps	(3) Berkeley 30M26 580 rpm 140 hp axial flow rated 22,500 gpm at 14 feet TDH
	(1) Berkeley 10K3M 7.5 hp 650 gpm jockey
Prime Power	(3) Waukesha-Scania\F67D3U 150 hp diesel engines
	(1) GE 240V, 3φ electric motor (jockey)
Standby Power	Not required
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	1,000 gallons; ~48 hours run time at peak load
Flood-proofed Elevation	16.8 feet NAVD
Effective BFE	17.0 feet NAVD

Deficiencies

There are no identified pump station deficiencies and no recommended capital improvements.

Manor Pump Station

Facility ID	**SD-9**
Location	Marylinn Ave. and Barker St.
Discharges to	Lower Penitencia Creek at STA 90+00
Design WSEL at Discharge Location	9.5 feet NAVD
Storage	Wet Well Only
Tributary Area	146 acres (residential and commercial)
Station Capacity	95 cfs
10-year Inflow	95.7 cfs
100-year Inflow	113.3 cfs

Residential and commercial areas drain to the Manor Pump Station, which activates when the adjacent 21-inch diameter bypass can no longer drain local runoff into Penitencia Creek, either because it becomes overloaded, or the creek stage is high.

Since the last master plan update, the pumps and motors have been rebuilt.

Equipment Schedule

Pumps	(3) Flygt 7060-885, 880 rpm, 85 hp submersible axial flow (14,000 gpm at 12′)
	(1) Flygt CP-3102 submersible centrifugal jockey pump (5 hp) at 600 gpm
Standby Power	600A automatic transfer switch for on-site engine-generator
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	n/a
Electrical Pad Elevation	18.2 feet NAVD
Effective BFE	16.0 feet NAVD

A third axial flow pump has been added to the pump station since the completion of the 2001 master plan, so the station now has adequate capacity for the design 10-year inflow. The pumps and motors were recently rebuilt.

Deficiencies

There are no identified pump station deficiencies and no recommended capital improvements.

Spence Creek Pump Station

Facility ID	**SD-10**
Location	11 Butler Street
Discharges to	Lower Penitencia Creek at STA 110+00
Design WSEL at Discharge Location	9.1 feet NAVD
Storage	Wetwell Only
Tributary Area	109 acres (residential and commercial)
Station Capacity	94 cfs
10-year Inflow	43.5 cfs
100-year Inflow	68.0 cfs
Excess Capacity	8cfs

***Note pump station inoperable at time of study**

Residential and commercial areas drain to Spence Creek until Penitencia Creek backwater forces runoff over a weir into the Spence Creek Pump Station. This facility discharges water to Lower Penitencia Creek through 600 feet of 42" diameter RCP force main. At the time of the master plan update, this station was inoperable due to pump equipment issues, electrical issues, and pump control issues.

This pump station is needed based on the 10-year storm CTP model. Figure 4-5 shows that without the pump, residential areas will become inundated.

Figure 4-5: 10-year Inundation With and Without Spence Creek Pump Station

Equipment Schedule

Pumps	(2) Flygt 7080-885, 880 rpm, 215 hp submersible axial flow (21,000 gpm at 26′) (1) Flygt CP-30856 submersible centrifugal jockey pump (3 hp) at 300 gpm
Standby Power	800A Kirk-Key Interlock (manual transfer switch) for portable engine-generator
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	n/a
Electrical Pad Elevation	18.2 feet NAVD
Effective BFE	18.0 feet NAVD

Deficiency

The station is inoperable, and while a plug and manual transfer switch are provided for a portable engine generator-set, there is no guarantee that either the EG-set or personnel to plug it in and turn it on will be available when power fails. Without any associated flood storage, adjacent areas will begin to flood just as soon as the power is gone. (This can occur with relatively minor storms if Penitencia Creek levels preclude gravity drainage.) The station needs to be retrofit with a permanent skid-mounted 400kW engine generator-set equipped with an automatic transfer switch to provide emergency power whenever the PG&E power supply fails, and there is a call for one of the pumps. Electrical work is required to make the station operational, and the current bubbler level sensor needs replacement. ***[Low Priority]***

Capital Improvement Recommendation

Permanent standby power needs to be furnished at the site. Estimated capital costs are:

Table 4-7: Low Priority CIP for Spence Creek Pump Station

Capital Improvement	Estimated Cost
800A automatic transfer switch	$60,000
Motor Control Center modifications	$80,000
Miscellaneous electrical work	$40,000
400kW EG-Set in acoustic enclosure	$240,000
Engineering and Administration (20%)	$80,000
Contingency (50%)	$250,000
CIP Cost	**$750,000**

Bellew Pump Station

Facility ID	**SD-11**
Location	481 Murphy Ranch Road
Discharges to	Coyote Creek at STA 616+00
Design WSEL at Discharge Location	32.7 feet NAVD
Storage	Wet well only
Tributary Area	270 acres (industrial)
Station Capacity	375 cfs
10-year Inflow	45.6 cfs
100-year Inflow	49.7 cfs
Excess Capacity	325.3 cfs

Located at the end of Bellew Drive within the Milpitas Business Park Development, this facility drains the industrial area located between Coyote Creek and Interstate 680; from State Highway 237 to the Hetch-Hetchy aqueduct. This station has excess capacity to discharge the 100-year inflow.

Since the last master plan update, the underground fuel tank has been replaced with a belly tank under the emergency generator.

Equipment Schedule

Pumps	(3) Cascade 42MF axial flow (460 rpm, 600 hp, 56,000 gpm at 29 feet TDH)
	(1) Cascade 10MF 3,100 gpm 40 hp electric jockey
Prime Power	(2) Baldor 1,800 rpm 600 hp electric motors with variable frequency drive
	(1) Caterpillar 3412 diesel engine rated at 750 hp (2,100 rpm)
Standby Power	650 kW diesel generator to run electric motors
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	2,500 gallons; 72 hours at peak load (direct drive engine) 1,450 gallons for diesel generator
Finish Floor Elevation	25.2 feet NAVD
Effective BFE	Shaded Zone X (area of moderate flood hazard)

Identified Deficiencies

Capital improvements are not necessary to maintain adequate pumping capacity at the Bellew Pump Station. However, during the Coyote Creek flood event of 2017, which produced water surface elevations in the creek that were a few feet from spilling over the levee at the Bellew Pump Station outlet, the outlet itself was submerged and water was flowing back through the pump discharge into the station wet well. City crews tried to improve the backflow prevention but could not; an engineered solution is needed to prevent creek backflow when the outlet is submerged and prevent excessive pump cycling due to high stage at Coyote Creek.

Murphy Pump Station

Facility ID	**SD-12**
Location	801 Murphy Ranch Road
Discharges to	Coyote Creek at STA 636+00
Design WSEL at Discharge Location	34.0 feet NAVD
Storage	Wet well only
Tributary Area	130 acres (industrial)
Station Capacity	200 cfs
10-year Inflow	118.8 cfs
100-year Inflow	197.5 cfs
Excess Capacity	2.5 cfs

Located just south of the Hetch-Hetchy aqueduct in the Milpitas Business Park Development, this facility drains the industrial area located between Coyote Creek and Interstate 680; from Hetch-Hetchy to Tasman Drive. This station has excess capacity to discharge the 100-year inflow. A control system upgrade is planned soon.

Equipment Schedule

Pumps	(3) Cascade 30MF axial flow (525 rpm, 250 hp, 30,000 gpm at 27 feet TDH)
	(1) Cascade 8MF 2,900 gpm 25 hp electric jockey
Prime Power	(3) Cumins NT655P diesel engines rated at 335 hp (2,600 rpm)
Standby Power	Not required
Control Power	120 VAC backed up by 24 VDC batteries with charger
Fuel Storage	2,000 gallons; 120 hours at peak load (3 pumps)
Finish Floor Elevation	27.7 feet NAVD
Effective BFE	Shaded Zone X (area of moderate flood hazard)

Identified Deficiencies

Capital improvements are not necessary to maintain adequate pumping capacity at the Murphy Pump Station. However, during the Coyote Creek flood event of 2017, which produced water surface elevations in the creek that were a few feet from spilling over the levee at the Murphy Pump Station outlet, the outlet itself was submerged and water was flowing back through the pump discharge into the station wet well. City crews tried to improve the backflow prevention but could not; an engineered solution is needed to prevent creek backflow when the outlet is submerged and prevent excessive pump cycling due to high stage at Coyote Creek.

Control systems have exceeded their life expectancy and with performance issues becoming increasingly problematic, control system rehabilitation is a ***high priority project*** expected to cost $250,000.

Oak Creek Pump Station

Facility ID	**SD-13**
Location	1521 McCarthy Boulevard
Discharges to	Coyote Creek at STA 678+00
Design WSEL at Discharge Location	38.3 feet NAVD
Storage	Wet Well and Pipe
Tributary Area	280 acres (industrial)
Station Capacity	320 cfs
10-year Inflow	102.5 cfs
100-year Inflow	216.4 cfs
Excess Capacity	103.6 cfs

Oak Creek Pump Station drains an industrial area at the southwestern corner of Milpitas, between Coyote Creek and Interstate 680 Tasman Drive to Montague Expressway. Because the direct-drive engines appear to be slightly overloaded when the Coyote Creek stage is high, they tend to run warm. A control system upgrade is planned soon.

Equipment Schedule

Pump	(3) Aurora 36P axial flow (590 rpm, 600hp, 48,000 gpm at 28.5 feet TDH) (1) Aurora 10LM 2,900 gpm 25 hp electric jockey
Prime Power	(3) Caterpillar 3408 diesel engines rated at 480 hp (2,100 rpm)
Standby Power	Not required
Fuel Storage	2,000 gallons; 80 hours at peak load (3 pumps)
Finish Floor Elevation	33.7 feet NAVD
Effective BFE	Shaded Zone X (area of moderate flood hazard)

Identified Deficiencies

Capital improvements are not necessary to maintain adequate pumping capacity at the Oak Creek Pump Station. However, during the Coyote Creek flood event of 2017, which produced water surface elevations in the creek that were a few feet from spilling over the levee at the Oak Creek Pump Station outlet, the outlet itself was submerged and water was flowing back through the pump discharge into the station wet well. City crews tried to improve the backflow prevention but could not; an engineered solution is needed to prevent creek backflow when the outlet is submerged and prevent excessive pump cycling due to high stage at Coyote Creek.

Control systems have exceeded their life expectancy and with performance issues becoming increasingly problematic, control system rehabilitation is a ***high priority project*** expected to cost $250,000.

Chapter 5: Operations, Maintenance, and Replacement

The intent of this Master Plan is not as a treatise on storm drain system operations and maintenance requirements or techniques (City operations and maintenance staff are the foremost authorities on this subject.) Rather, some foresight provided into anticipated ongoing maintenance schedules, including periodic replacement of major storm drain system components.

Milpitas is over 60 years old, and some of its older storm drainage infrastructures, particularly pumping equipment, are reaching the end of its useful life. Over the next several decades, major equipment replacements will be needed, and the City needs to set aside sufficient funds for annual facility maintenance and a systematic long-term replacement program, as outlined in Chapter 6.

General Maintenance Regimen

Table 5-1 presents very general criteria that may be useful in establishing a routine maintenance regimen. Again, city staff will have the best feel for the necessary frequency and extent of ongoing maintenance on a system-by-system basis. Also, maintenance needs will fluctuate depending upon seasonal and annual factors, particularly the amount of precipitation, and to a lesser extent, the general climate.

It is vitally important that all collection, storage, and pumping systems be in working order prior to the start of Milpitas's wet season near the end of October. Realizing the limited number of maintenance staff and the finite number of hours in a year, it is a given that certain items will have higher priorities than others.

Table 5-1: Storm System Maintenance Guidelines

Category	Schedule
Inlet Inspection	annually (summer-fall)
Inlet Cleaning	as required (ongoing)
Storm Drainpipe Cleaning	continuous if possible (ongoing)
Channel Cleaning	annually (fall)
Detention Basin Dredging	every ten years
Wet Well Cleaning	annually (fall)
Direct Observation/Inspection	monthly (year-round)
Pump Exercising	monthly (year-round)
Engine Exercising	monthly at full load (year-round)
Equipment Lubrication	per manufacturers' recommendations
Preventative Maintenance per Equipment O&M	annually (spring)
Clean and Polish Diesel Fuel/Remove Water	annually (fall)
Underground Storage Tank Inspection	weekly
Aboveground Storage Tank Inspection	monthly
Motor / Engine Control Testing	annually (fall)

Collection System Maintenance

The storm drain and channel system cannot function if one of its components is plugged. Even though hydraulic analyses say criteria are met, blocked inlets, pipes, or channels will cause flooding, potentially with serious consequences; lagoons and pumping forebays need to be monitored and periodically dredged to preserve design capacities. Even the most rigorous maintenance programs cannot prevent all problems during a storm event; still, problems must not accumulate.

It is also important to maintain the more natural drainage features such as open channels and lagoons as drainage features, so they do not become jurisdictional and require extensive regulatory permits to perform what should be routine maintenance.

Based on system history, the most significant problems occur at the base of the foothills, where sediment- and debris-laden runoff are easily carried within the steeper pipes and streets. This sediment and debris, some of which originates outside of the city limits in unincorporated Santa Clara County, are deposited as the topography flattens out to the west.

Adding debris basins and modifying inlets along Evans Road and Piedmont Road could help with the maintenance effort. Depending on the desired frequency for maintenance, storage in debris basins made to handle sediment and debris as described in Chapter 3. Retrofitting certain storm drain inlets to mimic the existing inlet for Piedmont Creek on Piedmont Road, as shown in Figure 5-1, would also help ease downstream maintenance.

Figure 5-1: Trash and Debris Protection at Piedmont Creek Inlet

Another area of concern is where so-called "self-cleansing" velocities of two feet per second are not maintained even with significant runoff. This circumstance may occur in larger diameter pipelines, particularly in the terminal drainage areas west of Interstate 880. Collection systems in terminal drainage areas have been designed to handle the 100-year discharge where pipes are continuously submerged in water due to backwater from pump stations.

Open Channel Maintenance

Open channels are important to maintain to allow for proper stormwater drainage from the city storm drain system. Debris and overgrown vegetation in open channels owned by the City should be removed to keep creek levels low so that stormwater in the collection system can drain out and not surcharge. There are surcharges causing flooding in the storm drain network in the existing system due to high creek levels in Wrigley-Ford Creek. Figure 5-2 shows that decreasing the Manning's roughness coefficient of Wrigley-Ford Creek to reflect the effects of maintained creek results in reduced surcharges in the collection system. Figures 5-3 and 5-4 show the creek WSEL profile with and without maintenance allowed by the Wrigley-Ford Creek maintenance permit for 10-year and 100-year storm.

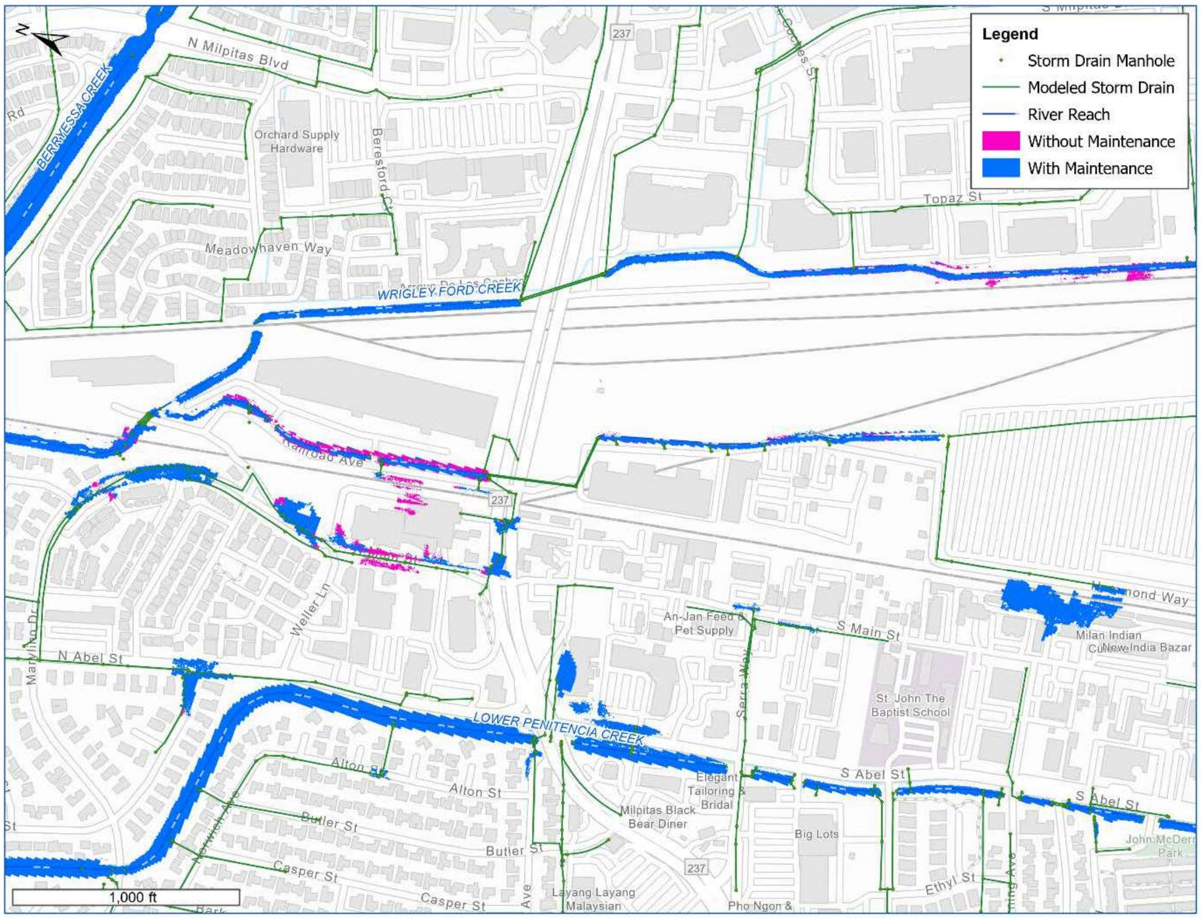
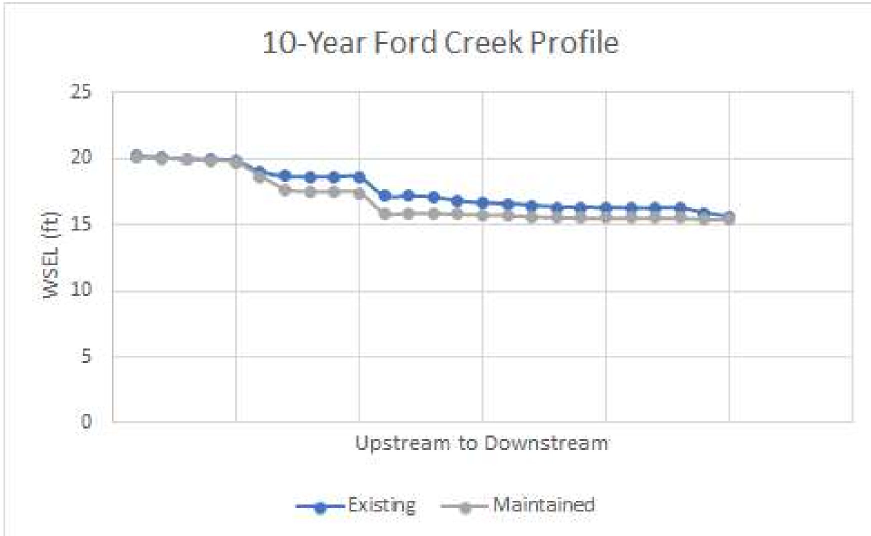
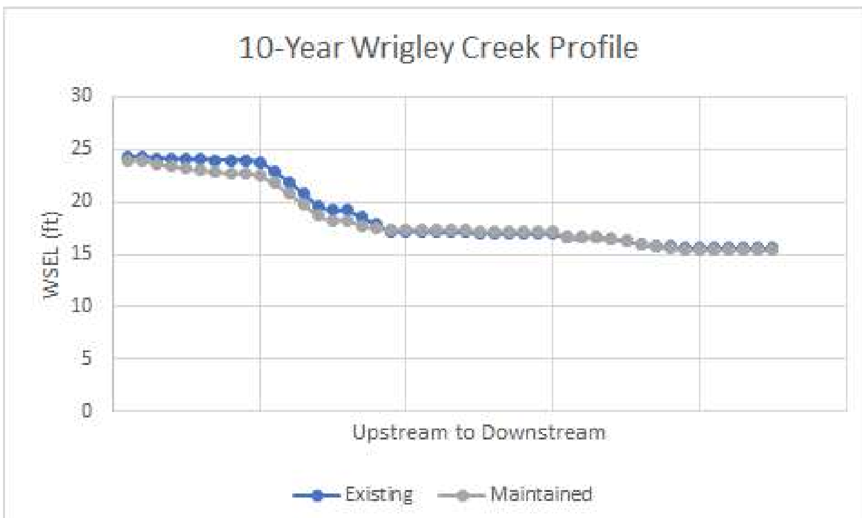


Figure 5-2: Wrigley-Ford Creek Flooding with and Without Maintenance for 10-Year Storm





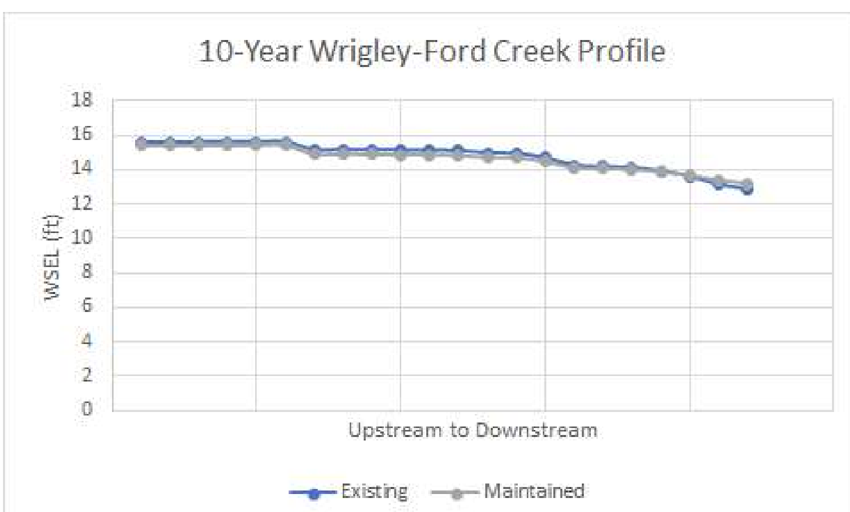
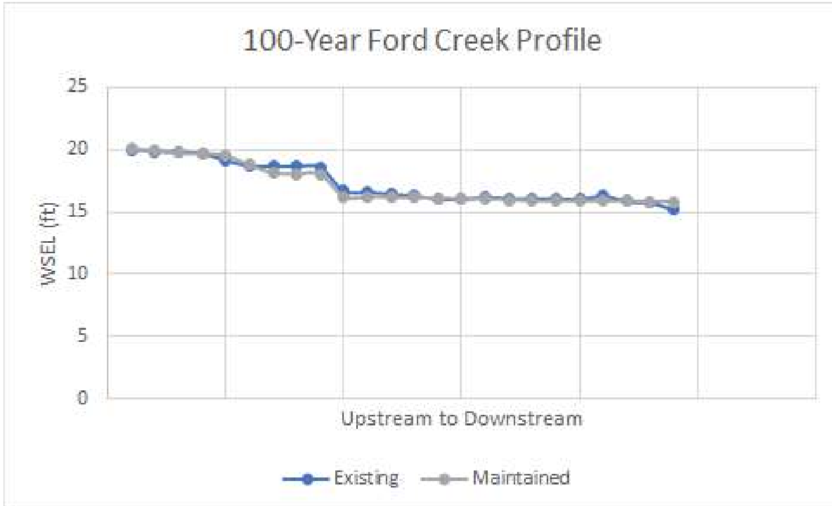
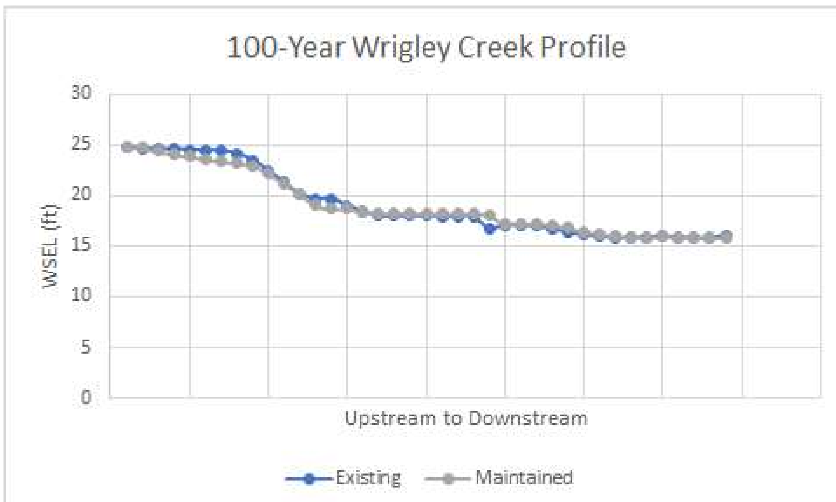


Figure 5-3: 10-Year Storm Profile with and without Maintenance on Wrigley, Ford, and Wrigley-Ford Creek





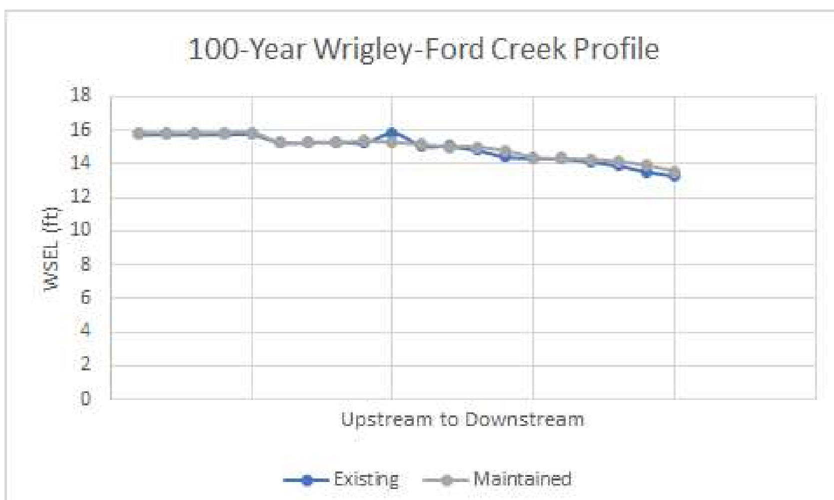


Figure 5-4: 100-Year Storm Profile with and without Maintenance on Wrigley, Ford, and Wrigley-Ford Creek

Pumping Facility Maintenance

Pumping stations are critical to maintain since mechanical or electrical failure can jeopardize system operation. Each pump station should have a bound copy of its site-specific operations and maintenance manual on-site, and all personnel need to be familiar with the contents of these manuals.

Proper equipment lubrication and maintenance following manufacturers' recommendations (which must be included in the operations and maintenance manual) is essential to efficient operation and longevity, particularly when one considers how infrequently pump operation may occur. For this reason, any pump station control system that does not automatically alternate lead and lag pump status so that each pump within a station operates roughly the same number of hours every year should be retrofit to do so.

Appendix B outlines pump station design, maintenance, and operation features that can help further the maintenance effort. All engine drive units installed run on diesel fuel. Table 5-2 summarizes the recommended frequency.

Table 5-2: Typical Maintenance Frequency for Engines and EG Sets

Maintenance Task	Operating Time	Calendar Time
Inspect fuel, oil level, coolant	8 hr	1 m
Inspect air cleaner, battery	50 hr	1 yr
Clean governor linkage, breather, air cleaner	100 hr	1 yr
Clean fuel filter, replace oil filter, change crankcase oil, check switchgear	200 hr	1 yr
Clean commutator, collector rings, relays, cooling system; inspect brushes, valve clearances, starting and stopping systems, water pump	500 hr	1 yr
Check injectors, grind valves (if required), remove carbon, clean oil passages, replace secondary fuel filter, clean generator, and grease bearings	1000 hr	2 yr

Municipal Regional Stormwater Permit Requirements

Milpitas participates in the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP) as a co-permittee under the California Regional Water Quality Control Board San Francisco Bay Region (Water Board) Municipal Regional Stormwater NPDES Permit (Order No. R2-2015-0049). Also referred to as the "MS4 Permit" or "MRP", it became effective November 19, 2015. Requirements outlined in the City's MS4 Permit are subject to change. As such, this storm drain master plan does not intend to document specific NPDES requirements or their implementation; but rather provide a brief background regarding the requirements likely to affect system-wide operation and maintenance. An allowance is made in Chapter 6 for typical annual costs to satisfy system-wide permit requirements. A permit update (MRP3.0) is in the draft form currently and anticipated to be adopted in 2021.

Regulatory Background

The Water Board has found that stormwater runoff from urban and developing areas within the San Francisco Bay region contains significant sources of pollutants that contribute to water quality impairment in the waters of the region. In Milpitas, these could include creeks, streams, and San Francisco Bay. In conformance with the Clean Water Act, the Water Board has established total maximum daily loading limits (TMDLs) for various pollutants to gradually eliminate the water bodies' impairment and attain water quality standards.

As a co-permittee, Milpitas is required to effectively prohibit the discharge of anything other than stormwater into storm drain systems and watercourses. It is specifically prohibited from discharging rubbish, refuse, sediment, or other solid wastes into surface waters or anywhere such trash will eventually transport to surface waters, including floodplain areas.

Routine Practices

Implement best management practices (BMPs) to control and reduce polluted stormwater and non-stormwater discharges to storm drains and watercourses during operation, inspection, and routine repair and maintenance activities of municipal facilities and infrastructure, including storm drain infrastructure. These practices apply to:

- Road repair and maintenance
- Sidewalk and other hardscape repairs, maintenance, and cleaning
- Structural maintenance (e.g., bridge repair) and graffiti removal
- Stormwater pump station operation and maintenance
- Corporation yard activities
- Construction sites
- Pesticide toxicity control

Milpitas must implement an industrial and commercial site control program at all sites that could reasonably be considered to cause storm water runoff pollution. Routine inspections and enforcement to abate actual or potential pollution sources need to be consistent with an Enforcement Response Plan prepared to confirm the implementation of appropriate and effective pollutant controls by industrial and commercial site operators. In addition, Milpitas is responsible detecting and eliminating illicit discharges by any party within its jurisdiction. An illicit discharge program shall be developed and implemented to include active surveillance, a centralized point of contact for complaints, a tracking system, and reporting. Public outreach and water quality monitoring, which can be collaborative with other co-permittees such as the Santa Clara Valley Water District, also permit requirements.

New Development and Redevelopment

Milpitas administers the implementation of new development and redevelopment projects to comply with the Municipal Regional Stormwater Permit requirements. For regulated projects (which is a function of size, land use, and location), this includes project review and permitting in the areas of site design, onsite stormwater treatment, hydro-modification management, landscaping, trash enclosures, plumbing, swimming pool water disposal, and fire test water disposal. The MS4 Permit does allow the City to consider the construction of regional stormwater treatment facilities in lieu of treatment on individual building sites. Such regional stormwater treatment facilities are not factored into capital planning for the stormwater system described in this master plan document.

Green Infrastructure

The City of Milpitas Green Infrastructure Plan (GI Plan) aims to gradually transform the urban landscape and storm drainage systems from “gray” to “green”. It involves shifting from having stormwater runoff flow directly off impervious surfaces into the storm drainage system to having runoff flow into a local, sustainable system such as draining into vegetated areas for infiltration and evaporation, collecting runoff for non-potable uses, using permeable pavements, and treating runoff with biotreatment. This green infrastructure will help limit the transport of pollutants in stormwater by reducing runoff. Coordinating the proposed CIP projects with street greening can lower the marginal cost of stormwater management.

Trash Capture

Trash originating from the city's Municipal Separate Sewer System (MS4) must be captured before it enters the waterways by the year 2022 (the draft permit may revise the deadline to 2023). The City is accomplishing this through full trash capture systems and devices located at individual storm drain inlets. One example is the screening device at Wrigley-Ford Pump Station. It is outside the scope of the master plan to identify and model these devices; however, the design and operation of these devices may result in impediments to flow and the potential to cause localized flooding. This needs consideration during the locating and design of these devices, and ongoing operations and maintenance.

System Replacement

With predominantly reinforced concrete pipe, collection system materials can be expected to last indefinitely, so a major replacement schedule for the pipe is not presented. System breaks, joint misalignment, and other problems do occur, of course, so periodic collection system rehabilitation has been included with the estimated annual maintenance cost.

On the other hand, pumping facilities rely heavily on mechanical and electrical equipment that will wear out, particularly since the stations are not operated constantly. On average, pumping equipment can be expected to last anywhere from 20 to 30 years with proper maintenance. Structural facilities should last much longer – at least 50 years – although metal, wood, and even concrete surfaces all require regular care.

Table 5-3 lists Milpitas' pumping facilities, their approximate age, and possible dates for mechanical and electrical equipment replacement to be completed within 5-year intervals based on input from City staff. Major rehabilitation might include complete pump station replacement, depending upon the circumstances. City maintenance crews need to monitor the condition of these facilities and prepare for system replacement several years in advance.

More detailed pump station assessments are provided in Chapter 6. Thorough individualized pump station assessments should be made prior to undertaking major equipment replacement or station rehabilitation.

Table 5-3: Pumping Facility Replacement

ID	Station Name	Originally Built	Age (years)	Recent Equipment Replacement	Proposed Schedule for	
					Equipment Replacement	Major Rehabilitation
1	California Cr	1983	38		2030	2070
2	Jurgens	1989	32	Ventilation	High Priority CIP	High Priority CIP
3	McCarthy	1994	27		2045	2075
4	Abbott	1983	38	Equip rehab/motors	2040	2070
5	Minnis[1]	1978	43	Pumps/ATS	2050	2050
6	Penitencia[1]	1960	61	Engines rebuilt	High Priority CIP	High Priority CIP
7	Wrigley-Ford	1993	28		2035	2070
8	Berryessa[2]	1977	44	Electronics	2045	2045
9	Manor	1993	28	Pumps and motors	2040	2070
10	Spence Ck	1988	33		2030	2065
11	Bellew[3]	1985	36		2060	2060
12	Murphy	1983	38		2035	2070
13	Oak Creek	1979	42		2025	2055

[1]Scheduled as High-priority CIP
[2]All pumping, electrical, and control equipment replaced and flood-proofed in 2006
[3]Two engines replaced with electric motors, variable frequency drives, and controls in 2012

Chapter 6: Storm Drainage Funding Requirements

This chapter summarizes budget requirements to fund Capital Improvement Program projects described in Chapter 3, and facility maintenance and replacement as outlined in Chapter 4 and 5. Table 6-1 summarizes these projects. Table 6-2 provides summary costs for the high priority CIP and Table 6-3 lists low priority capital project summary costs. Detailed cost breakdowns may be found in Appendix C.

Table 6-1: Comprehensive Master Plan CIP Projects

CIP Project	Priority
Abbott Pump Station	Low
BERRY_10	High
CAL_1	Low
CAL_2	High
CAL_3	High
CAL_4	Low
COCHES_5	High
COCHES_8	Low
Inlet Replacements	High
Jurgens Pump Station	High
LP_12	Low
LP_14	Low
LP_15	Extension
LP_16	Extension
LP_17	Extension
LP_18	Extension
Murphy Pump Station	High
Oak Creek Pump Station	High
Penitencia Pump Station	High
Spence Creek Pump Station	Low
WF_11	Low
WF_13	Low
Wrigley-Ford Creek Maintenance	Low

Table 6-2: High Priority CIP Project Cost Summary

Improvement ID	Improvement Street	Construction	Engineering/ Inspection	CIP Total
BERRY_10	Ames Avenue	$1,390,000	$280,000	$1,670,000
CAL_2	Jacklin Road	$800,000	$160,000	$960,000
CAL_3	Bayview Park Drive	$180,000	$40,000	$220,000
COCHES_5	S Park Victoria Drive	$9,640,000	$1,930,000	$11,570,000
INLETS	Inlet Replacements	$1,170,000	$230,000	$1,400,000
JURGENS	Jurgens Pump Station	$12,500,000	$2,500,000	$15,000,000
MURPHY	Murphy Pump Station	$250,000		$250,000
PENITENCIA	Penitencia Pump Station	$8,000,000	$2,000,000	$10,000,000
OAK	Oak Creek Pump Station	$250,000		$250,000
		$34,180,000	$7,140,000	**$41,320,000**

Table 6-3: Low Priority CIP Project Cost Summary

Improvement ID	Improvement Street	Construction	Engineering/ Inspection	CIP Total
ABBOTT	Abbott Pump Station	$1,200,000	$200,000	$1,400,000
CAL_1	Tice Drive	$280,000	$60,000	$340,000
CAL_4	Wool Drive	$1,930,000	$390,000	$2,320,000
COCHES_8	Foothill Park	$860,000	$170,000	$1,030,000
LP_12	Main St – Serra Way	$240,000	$50,000	$290,000
LP_14	North Abel Street	$4,250,000	$850,000	$5,100,000
SPENCE	Spence Creek PS	$620,000	$130,000	$750,000
WF_11	Comet Drive	$160,000	$30,000	$190,000
WF_13	Railroad Avenue	$1,970,000	$390,000	$2,360,000
WFC	Wrigley-Ford Creek Maint	$600,000	$200,000	$800,000
		$12,110,000	$2,470,000	**$14,580,000**

Table 6-4: Storm Drain Extension CIP Project Cost Summary

Improvement ID	Improvement Street	Construction	Engineering/ Inspection	CIP Total
LP_15	Main St/Tom Evatt Pk	$150,000	$30,000	$180,000
LP_16	Main St	$130,000	$30,000	$160,000
LP_17	Main St/Sinnott Ln	$50,000	$10,000	$60,000
LP_18	Main St/Carlo St	$90,000	$20,000	$110,000
		$420,000	$90,000	**$510,000**

Table 6-5 summarizes estimated annual costs for implementing the proposed priority Capital Improvement Program, the next series of long-term replacement projects (through 2050), and annual system maintenance. All cost estimates are in 2021 dollars (ENR Index = 13,500). Annual equal payment capital recovery costs assume 20-year financing with a six percent interest rate. The cost of money associated with actual project timing is assumed to be included with CIP contingencies. To set aside sufficient funds for high priority work, amortized annual costs for low priority projects are not calculated since these optional projects would likely be built only with outside funding in conjunction with other work or undertaken after the 20-year CIP.

Table 6-5: Storm Drainage Funding Requirements

Category	Present Worth	Annualized Cost
High Priority CIP Implementation[1]	$40,000,000	$3,500,000
Long-Term Equipment Replacement[2]	$13,000,000	$1,500,000
Annual Operations and Maintenance[3]	---	$2,500,000
Total Budget	$53,000,000	$7,500,000

[1]See Table 6-2 and Table 6-4 (summary present worth is rounded)
[2]See Table 6-11; includes next series of scheduled replacements only, for consistency with a 20-year CIP
[3]See Table 6-9

Spread over Milpitas' 6,048 acres of developed or developable land, the average annual cost per acre is $1,240 to fund high priority Master Plan improvements, replace equipment over the life of the CIP, and maintain storm drainage facilities.

Cost Basis of Capital Improvement Program

Chapter 3 discusses evaluation criteria used to prioritize improvements. Based on hydrologic and hydraulic analyses of stormwater collection and pumping facilities, master plan improvements will bring systems into compliance with performance criteria. This is a master plan level effort. Hence, many of the practical constraints that will govern the detailed design and construction of actual infrastructure improvements are unknown at this time, such as:

- Utility interference and relocation;
- Right-of-way and/or easement availability;
- Traffic control requirements;
- Geotechnical and hazardous waste conditions;
- Archaeological discoveries and environmental impacts; and/or
- Regulatory and permitting requirements.

Since these impacts cannot be estimated with any certainty, this master plan's approach is to estimate capital improvement costs based on current construction market conditions and apply 10% for mobilization and demobilization, 5% for traffic control, and 40% on contingency. A 40% contingency has been included to account for 15% design and 25% construction contingency. Table 6-6 and 6-7 provide unit cost information for storm drain collection systems. Table 6-8 summarizes the calculation of estimated CIP cost by Master Plan improvement priority. Costs are based on bids and other data from past storm drain projects adjusted to the current ENR index (13,500).

Table 6-6: Storm Drainpipe Collection Costs per Lineal Foot

Diameter	18"	24"	30"	36"	42"	48"	54"	60"	72"	84"	94"
Pipe	283	350	447	516	614	692	789	898	1,171	1,590	1,920
10% Mob/Demob	28	35	45	52	61	69	79	90	117	159	192
5% Traffic Control	14	18	22	26	31	35	39	45	59	80	96
40% Contingency	113	140	179	206	246	277	316	359	469	636	768
Total Unit Cost	**439**	**543**	**693**	**800**	**952**	**1,073**	**1,223**	**1,392**	**1,816**	**2,465**	**2,976**

Table 6-7: Storm Drain Manhole Costs per Unit

Diameter	18"	24"	30"	36"	42"	48"	54"	60"	72"	84"	94”
Manhole	13,369	13,655	13,941	14,228	14,514	14,801	16,080	16,406	18,268	20,915	23,005
10% Mob/Demob	1,337	1,366	1,394	1,423	1,451	1,480	1,608	1,641	1,827	2,091	2,301
5% Traffic Control	668	683	697	711	726	740	804	820	913	1,046	1,150
40% Contingency	5,347	5,462	5,577	5,691	5,806	5,920	6,432	6,562	7,307	8,366	9,202
Total Unit Cost	**20,722**	**21,165**	**21,609**	**22,053**	**22,497**	**22,941**	**24,923**	**25,429**	**28,315**	**32,418**	**35,658**

Table 6-8: Storm Drain Capital Improvements Costs

Category	Cost
High Priority Storm Drain Projects	$15,820,000
High Priority Pump Station Projects	$25,500,000
Low Priority Storm Drain Projects	$12,430,000
Low Priority Pump Station Projects	$2,150,000
Extension CIP	$510,000
Wrigley-Ford Creek Maintenance	$800,000
Total Budget	$57,210,000

Including the extension of storm drains into underserved areas, the high priority CIP totals $40 million. By comparison the recommended high priority CIP established in the 2013 storm drain master plan update totaled $23 million when adjusted to 2021 dollars. Although funding requirements for storm drain projects are reduced as discussed in Chapter 3, two major pump stations are now slated for high-priority capital improvement for complete replacement. The passage of time has moved Penitencia Pump Station from the long-term replacement schedule into an immediately necessary project that involves building a completely new facility. Jurgens Pump Station was on the long-term replacement schedule, but the City is no longer satisfied with its original design premise, which intentionally used Dixon Landing Park as a forebay and storage basin. These two projects are anticipated to add $25 million to the high priority CIP.

Annual Maintenance Costs

Existing storm drainage infrastructure and new improvements to be constructed from the CIP must be operated and maintained as described in Chapter 5. Based on these regimens and input from City staff, the annual funding levels summarized by Table 6-9 are recommended for facility operation, preventative maintenance, programmed replacement, and mandated non-point source control programs. Some allowance should also be made for increased power and fuel costs for pumping.

Table 6-9: Storm Drain Annual Operation and Maintenance Costs
(All costs in 2021 dollars; ENR = 13,500)

Category	Cost
Annual Operations	$600,000
Preventative Maintenance	$500,000
Wrigley-Ford Creek Maintenance	$800,000
NPDES Permit Compliance	$300,000
Programmed Replacement	$300,000
Total Annual Costs	**$2,500,000**

Cost of Major Facility Replacement

Replacing major mechanical equipment for pumping stations is outside of the annual allowance made for programmed replacement. Detailed cost estimates to replace equipment at the Abbott Pump Station and Oak Creek Pump Station have been prepared. Estimated costs in 2020 dollars for other pump station replacement projects are based on the unit costs indicated in Table 6-10. Equal payment series capital-recovery fund amounts for equipment replacement and major rehabilitation are given in Table 6-11, based on an interest rate of six percent, and beginning to accumulate the annual fund in 2020.

Table 6-10: Storm Pumping and Storage Unit Costs

Category	Unit Cost
Axial Flow Pump and Driver	$3,500 per cfs of capacity
Direct Drive Engine	$950 per horsepower
Engine-Generator Set	$700 per kilowatt
Pump Building	$400 per square foot
Storage Excavation	$40 per cubic yard

Table 6-11: Pumping Facility Replacement

ID	Facility	Next Scheduled Replacement			Second Scheduled Replacement		
		Year	Cost	Annual Fund	Year	Cost	Annual Fund
1	California Circle	2030	$1,000,000	$136,000	2070	$2,000,000	$129,000
2	Jurgens	CIP	$0	$0	2075	$3,000,000	$185,000
3	McCarthy	2040	$3,000,000	$262,000	2080	$5,000,000	$313,000
4	Abbott	CIP	$0	$0	2050	$1,000,000	$73,000
5	Minnis	2050	$1,000,000	$73,000	2080	$1,000,000	$63,000
6	Penitencia	CIP	$0	$0	2070	$2,500,000	$161,000
7	Wrigley-Ford	2035	$2,000,000	$206,000	2065	$3,000,000	$226,000
8	Berryessa	2045	$3,000,000	$234,000	2060	$1,000,000	$67,000
9	Manor	2040	$1,000,000	$87,000	2070	$1,000,000	$65,000
10	Spence Creek	CIP	$0	$0	2065	$1,000,000	$65,000
11	Bellew	(2012)	$0	$0	2055	$3,000,000	$208,000
12	Murphy	CIP	$0	$0	2070	$3,000,000	$193,000
13	Oak Creek	2025	$2,000,000	$475,000	2055	$3,000,000	$208,000
	Total		**$13,000,000**	$1,500,000		**$30,000,000**	$2,000,000

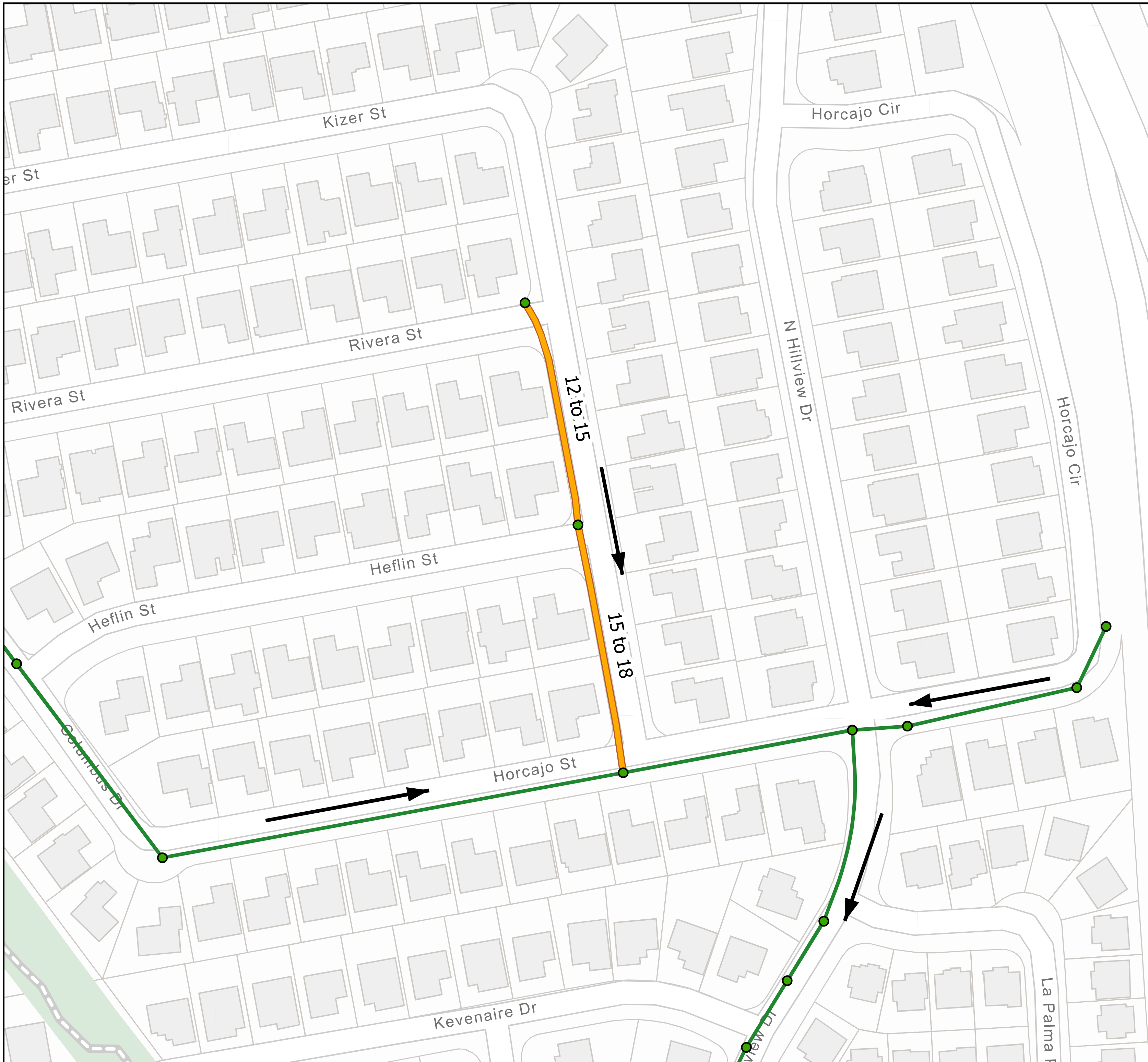
Note: CIP costs may be found in Table 6-2 (high priority) and Table 6-3 (low priority)

Appendix A Storm Drain Inundation Maps

Milpitas Storm Drainage CIP

CAL_1
Tice Drive
SD Improvements

Page 1 of 15



Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

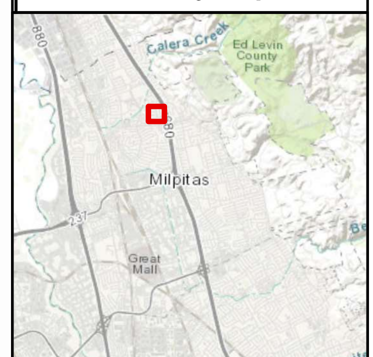
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 150 feet

Vicinity Map



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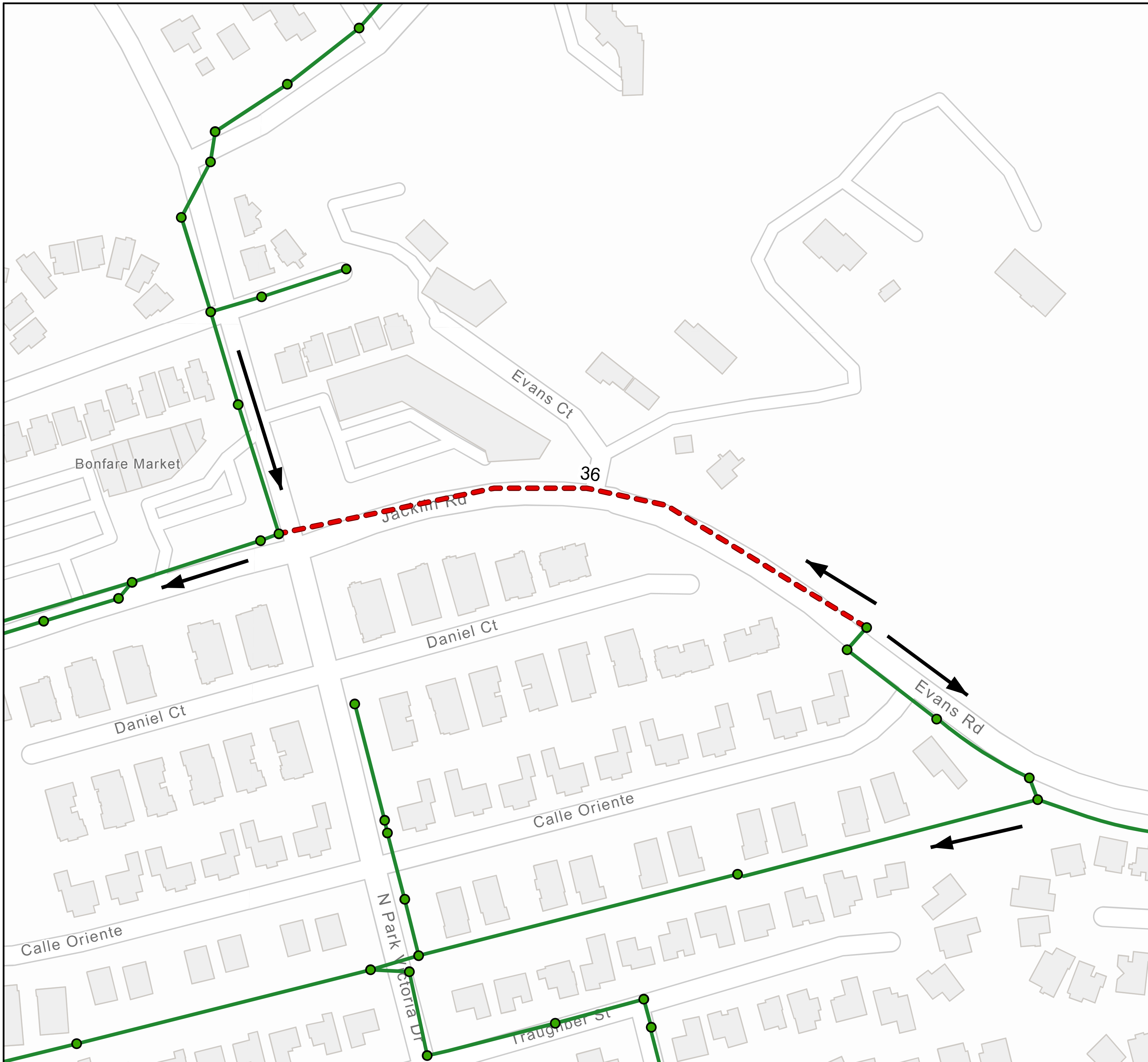


Milpitas
Storm Drainage CIP

CAL_2
Jacklin Road
SD Improvements

Page 2 of 15

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

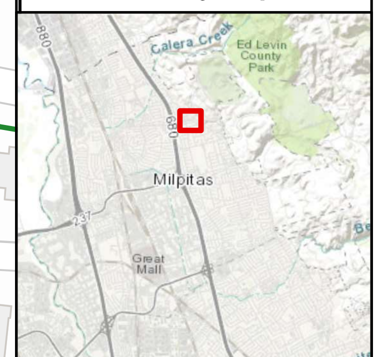
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 200 feet

Vicinity Map



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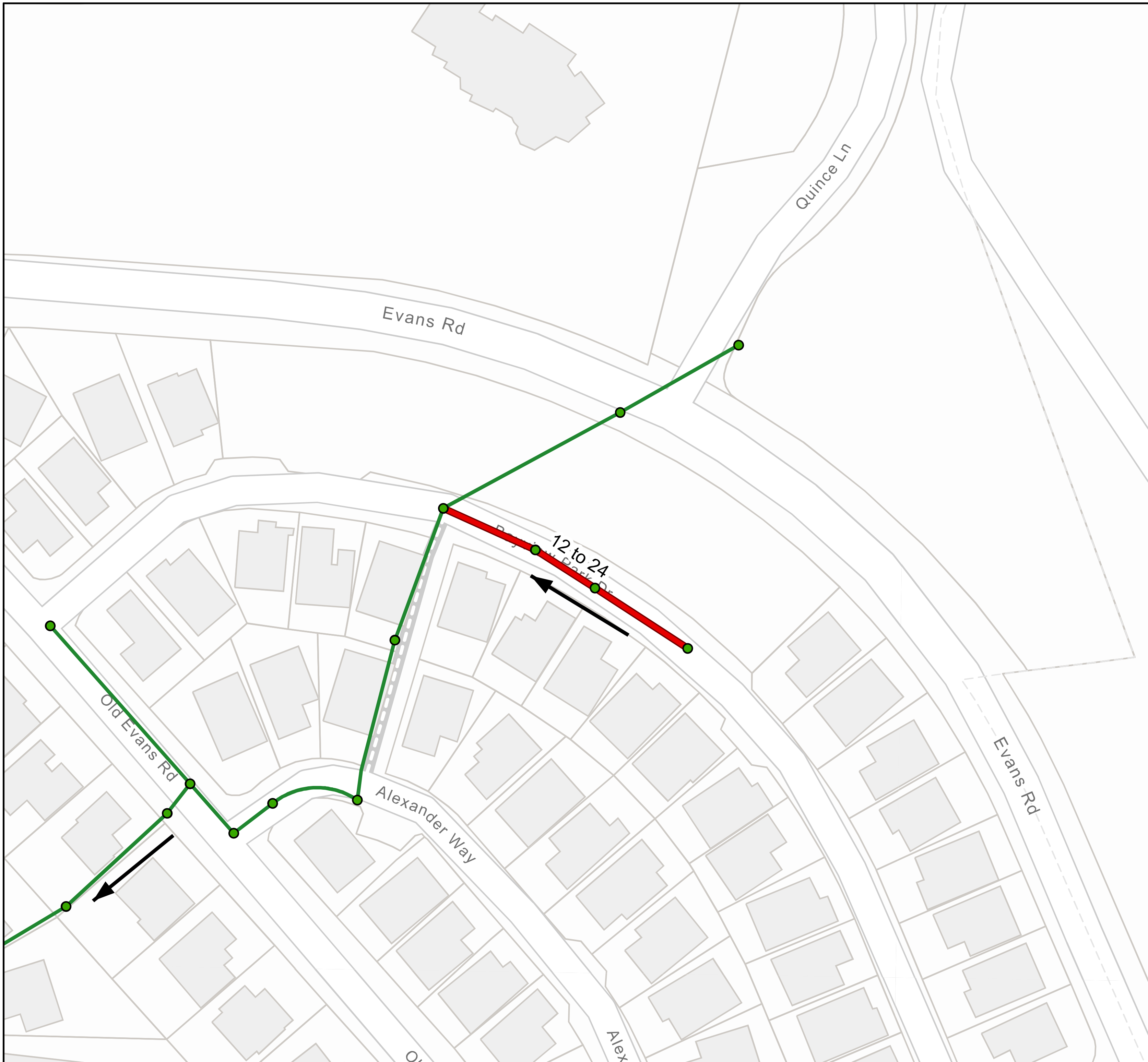


Milpitas
Storm Drainage CIP

CAL_3
Bayview Park Drive
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

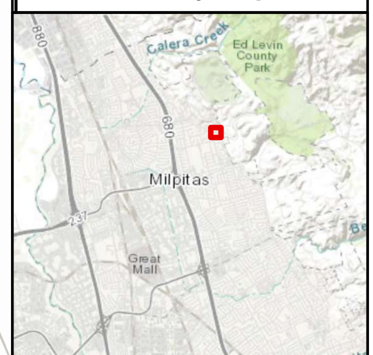
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 100 feet

Vicinity Map



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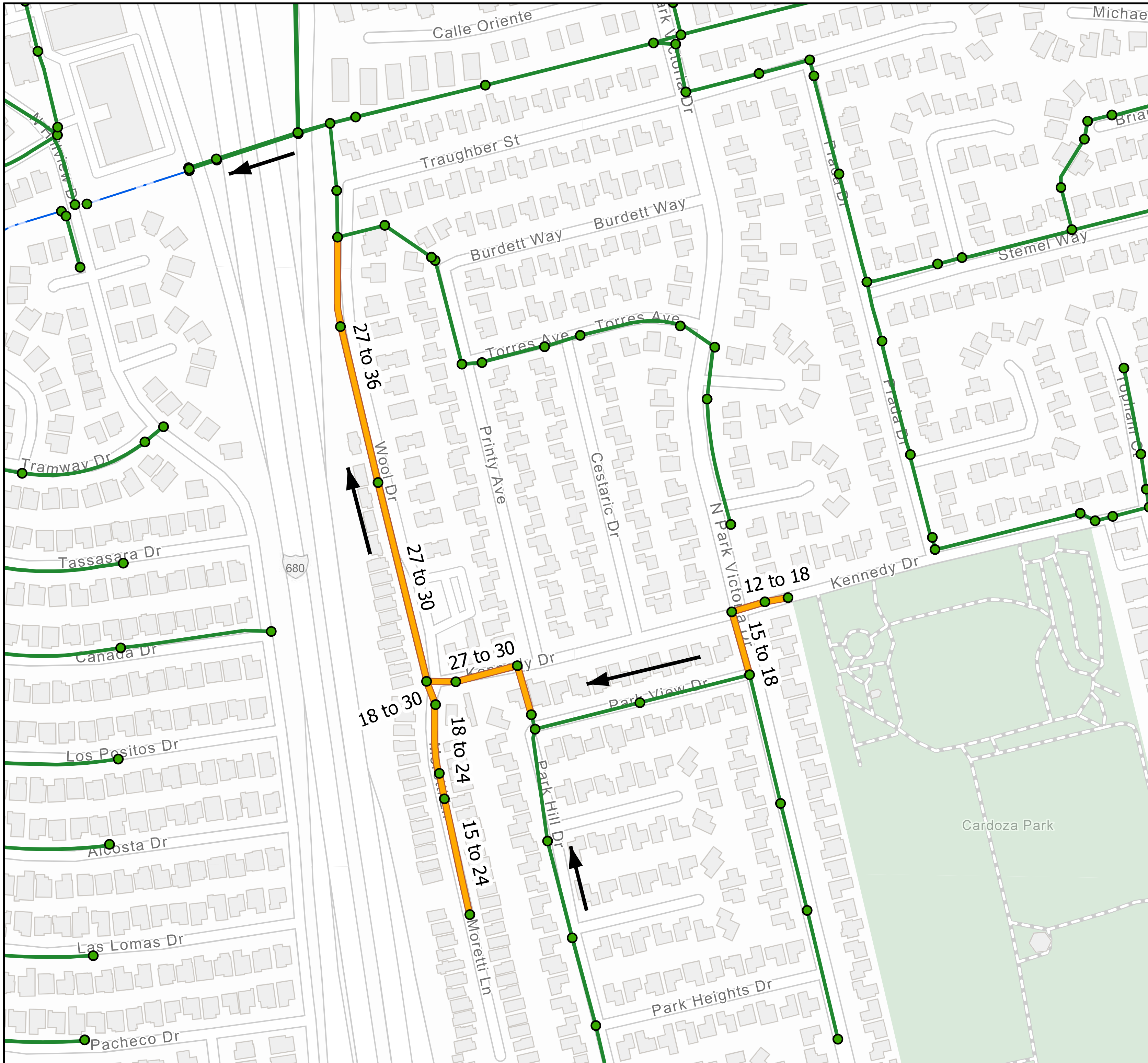
Milpitas Storm Drainage CIP

CAL_4
Wool Drive
SD Improvements



Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

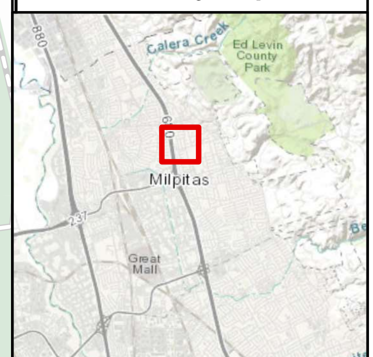
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 350 feet

Vicinity Map



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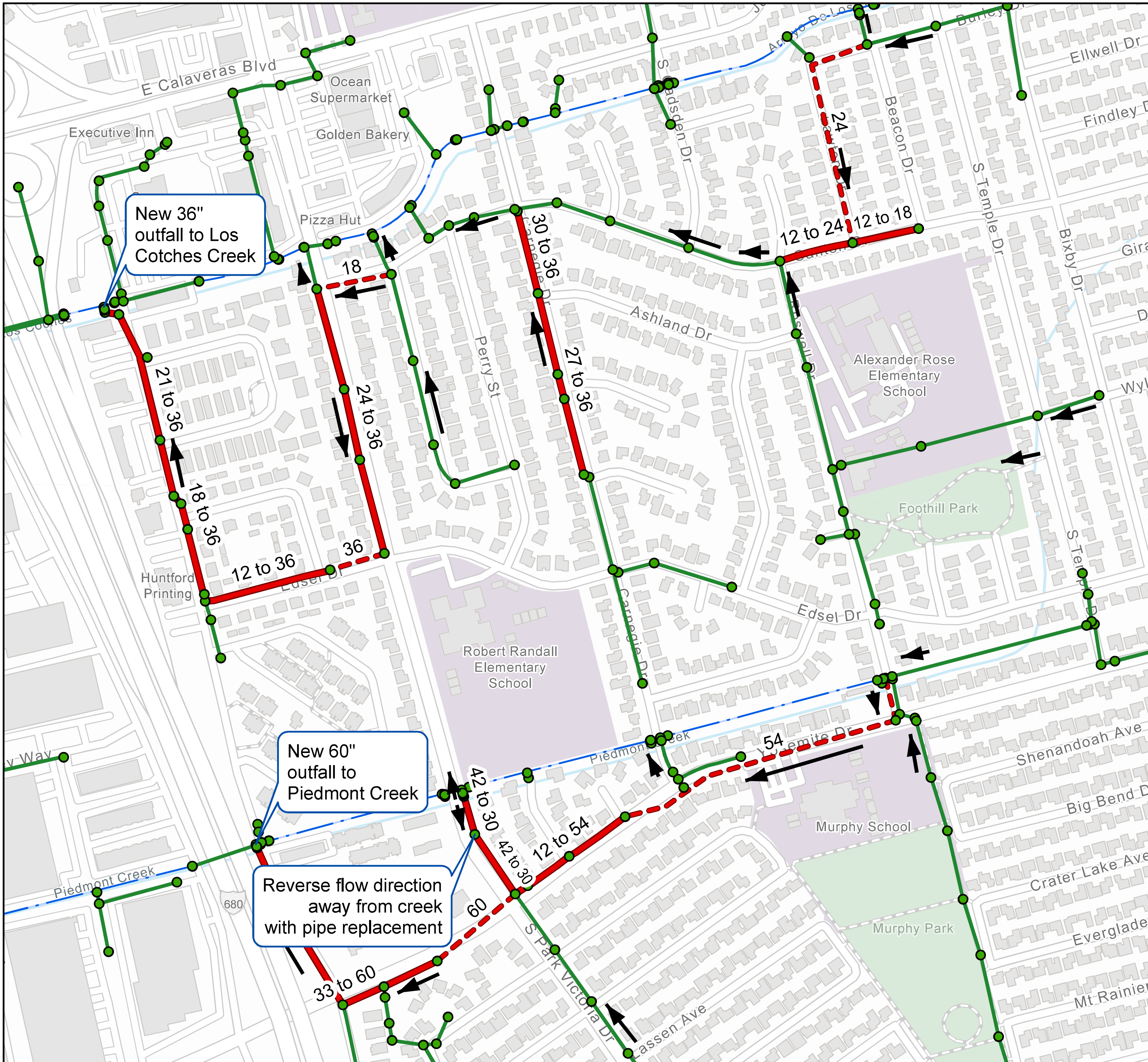


Milpitas
Storm Drainage CIP

COCHES_5
Coches and Piedmont Creek
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

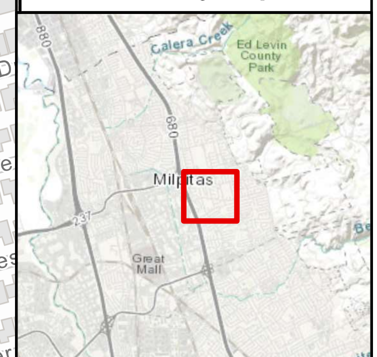
CIP Priority
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 500 feet

Vicinity Map



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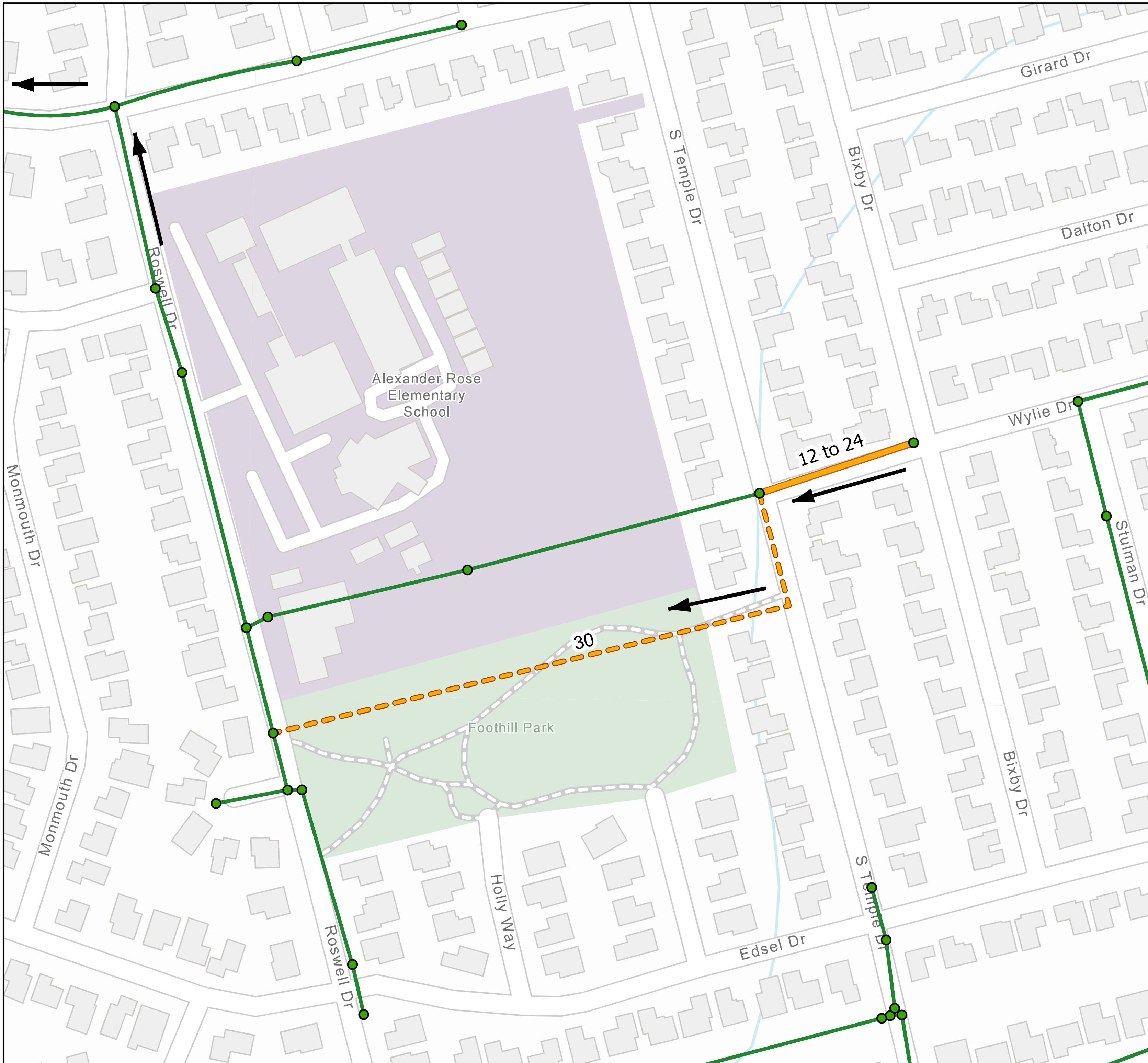
Milpitas
Storm Drainage CIP

COCHES_8
Foothill Park
SD Improvements

Page 6 of 15



Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 200 feet

Vicinity Map

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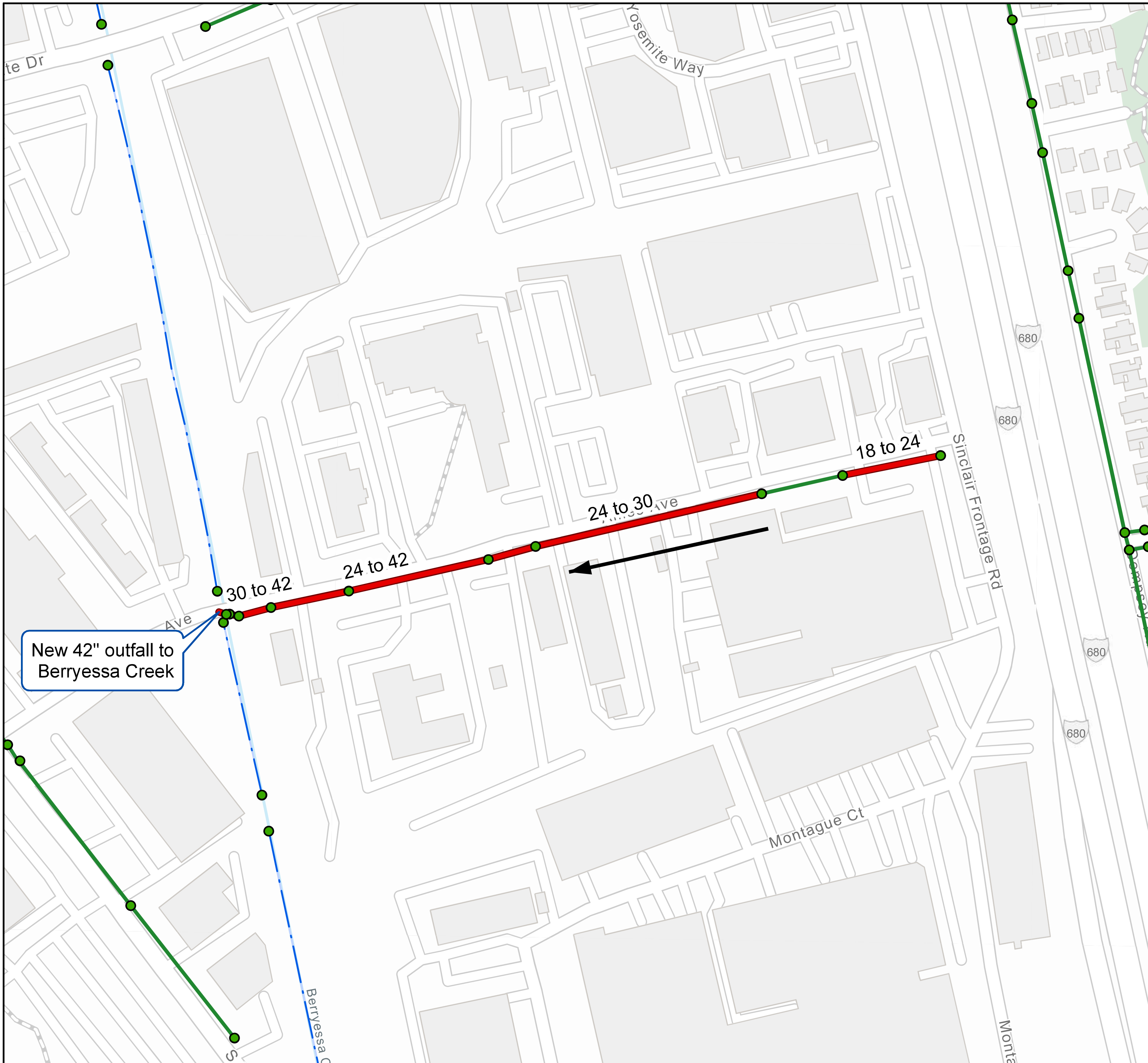
Milpitas
Storm Drainage CIP

BERRY_10
Ames Avenue
SD Improvements



Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

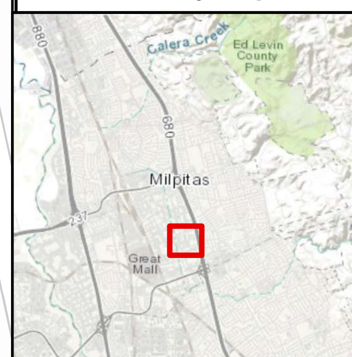
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 300 feet

Vicinity Map



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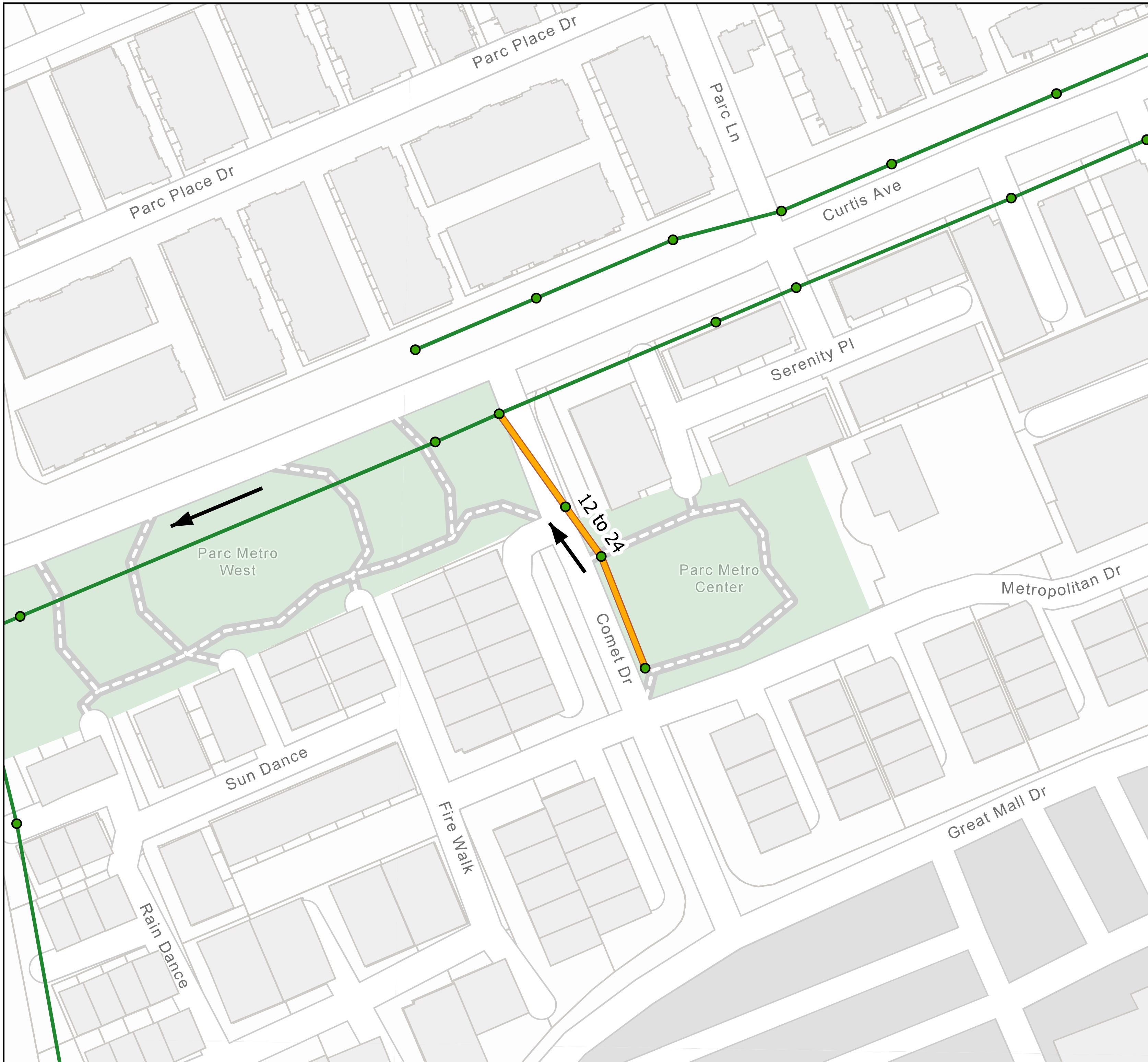
Milpitas Storm Drainage CIP

WF_11
Comet Drive
SD Improvements



Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

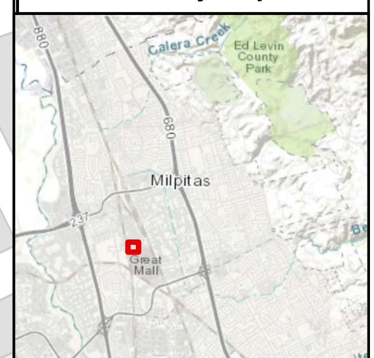
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 100 feet

Vicinity Map



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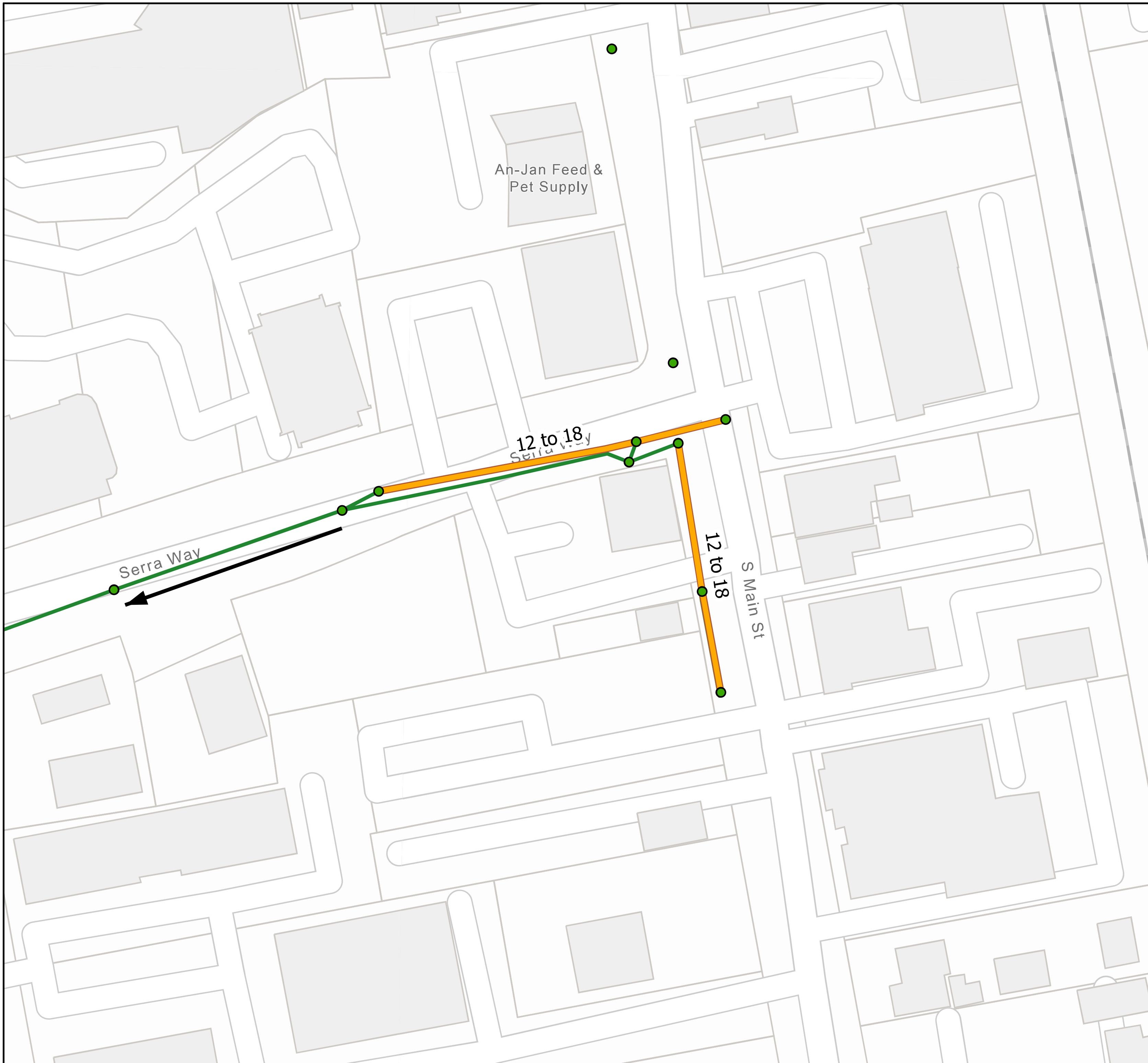


Milpitas
Storm Drainage CIP

LP_12
Main Street - Serra Way
SD Improvements

Page 9 of 15

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

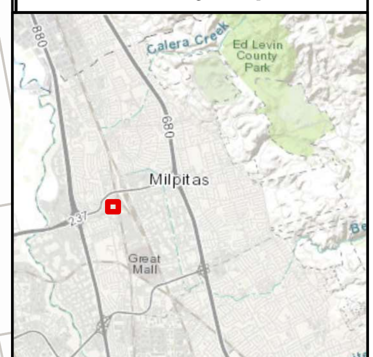
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 100 feet

Vicinity Map



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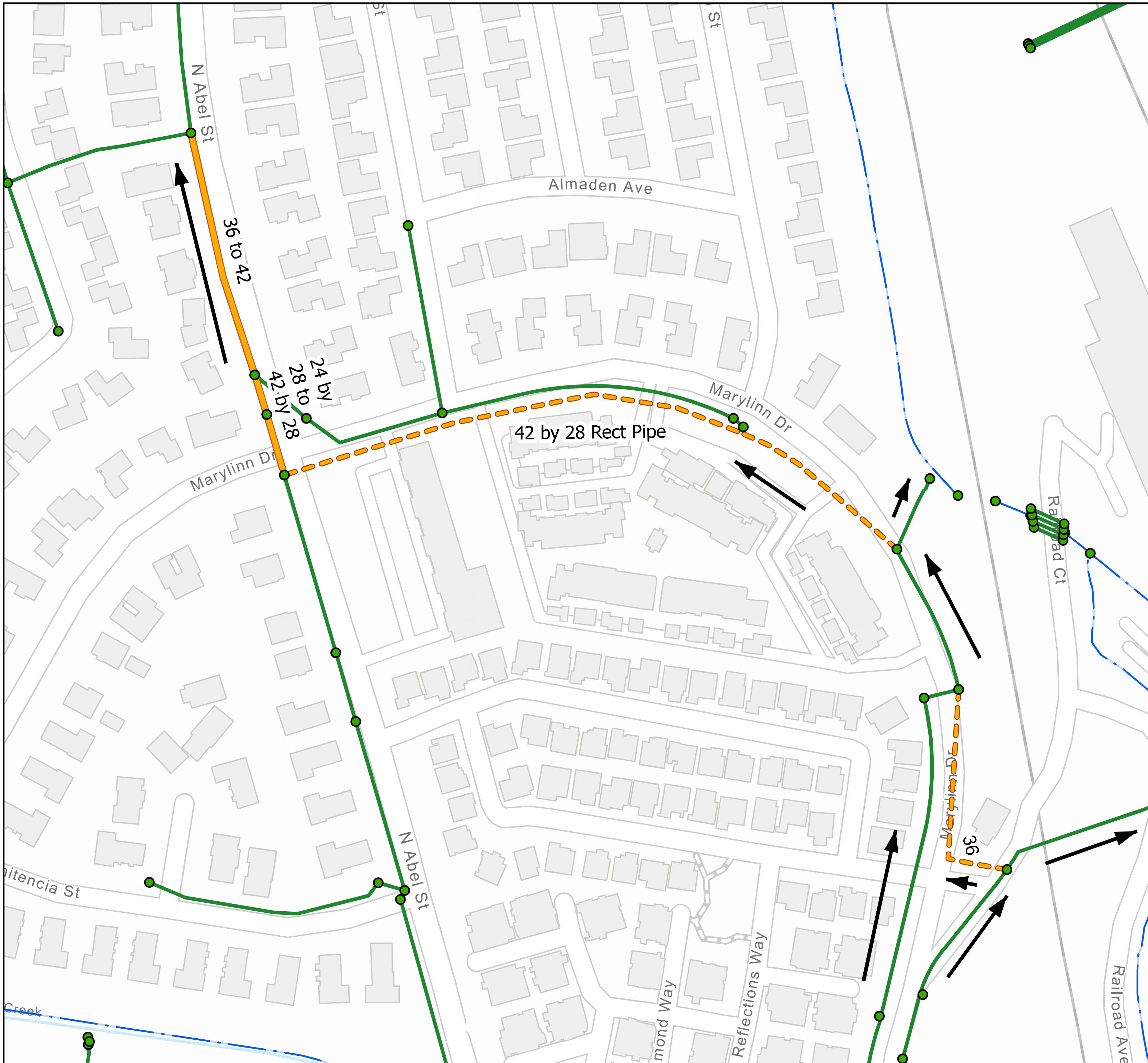

Milpitas
Storm Drainage CIP

WF_13
Railroad Avenue
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 200 feet

Vicinity Map

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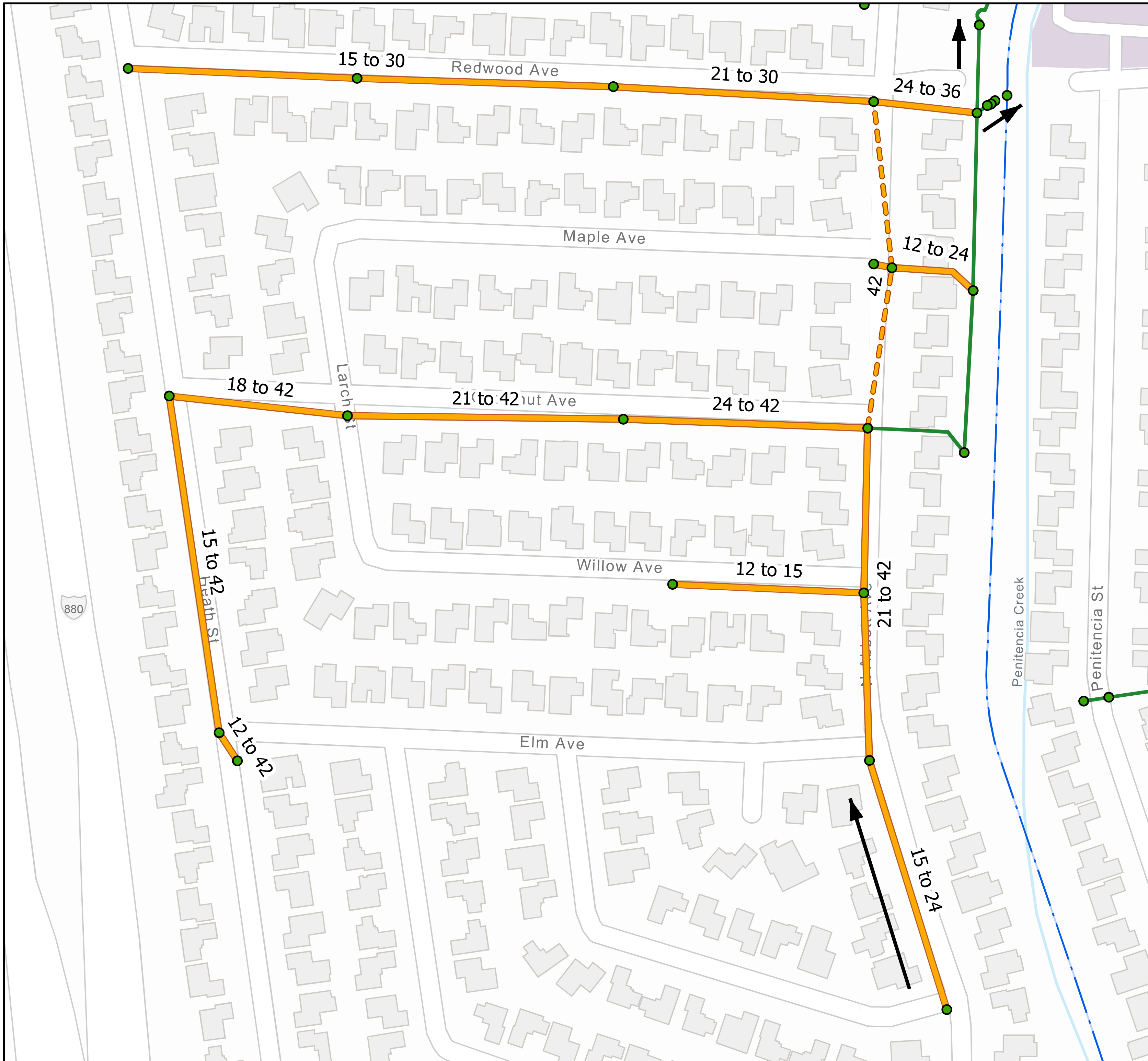


Milpitas
Storm Drainage CIP

LP_14
North Abel Street
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

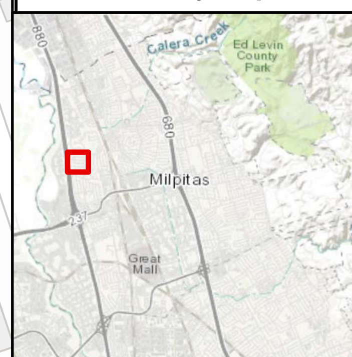
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 200 feet

Vicinity Map



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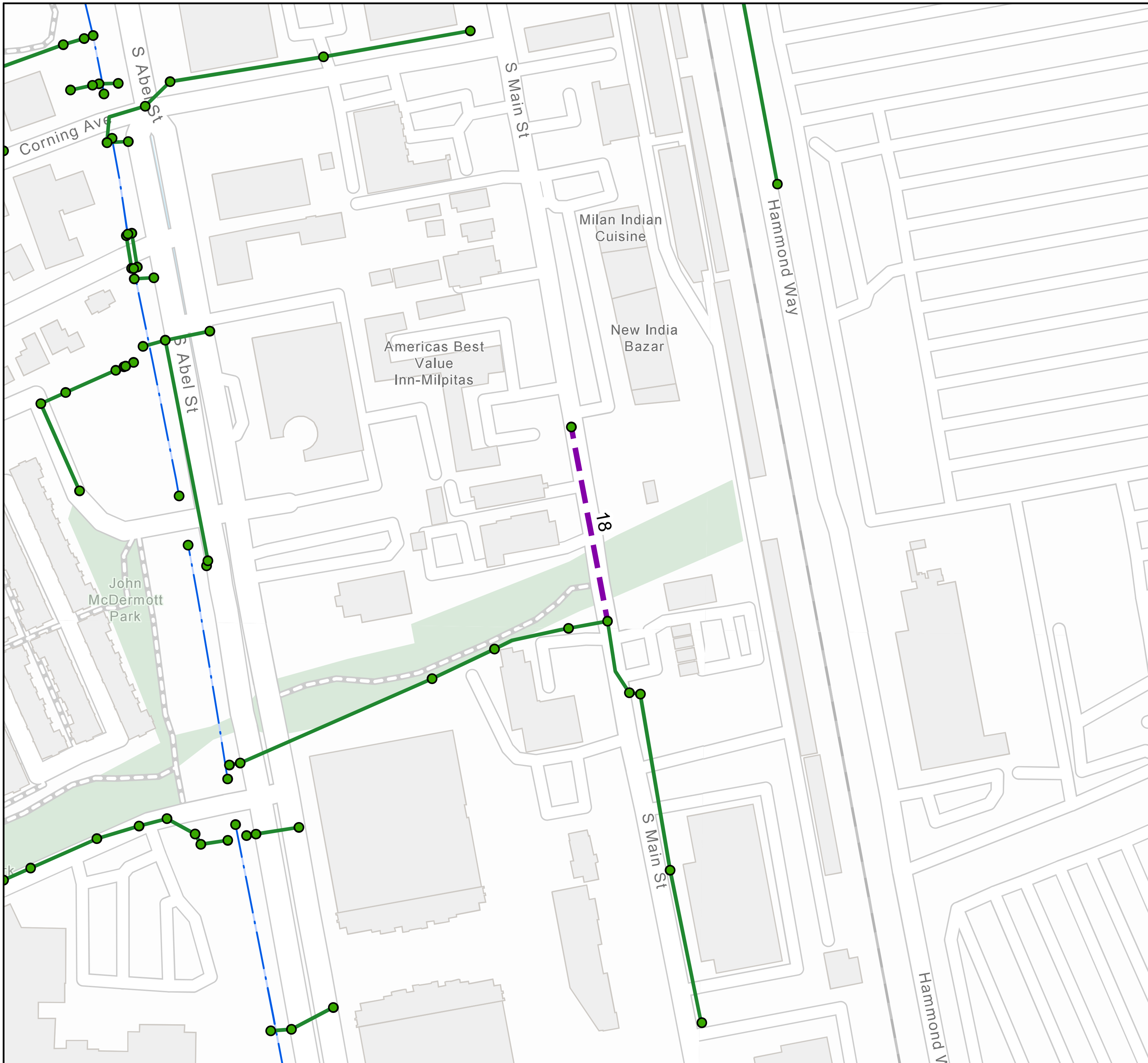


Milpitas
Storm Drainage CIP

Under-served Area
LP_15
Main Street - Tom Evatt Park
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

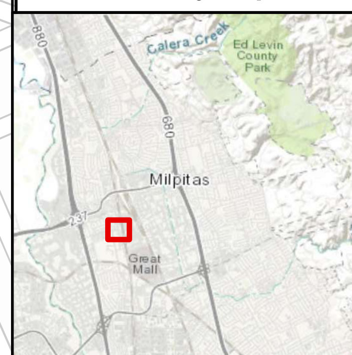
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 200 feet

Vicinity Map



Schaaf & Wheeler
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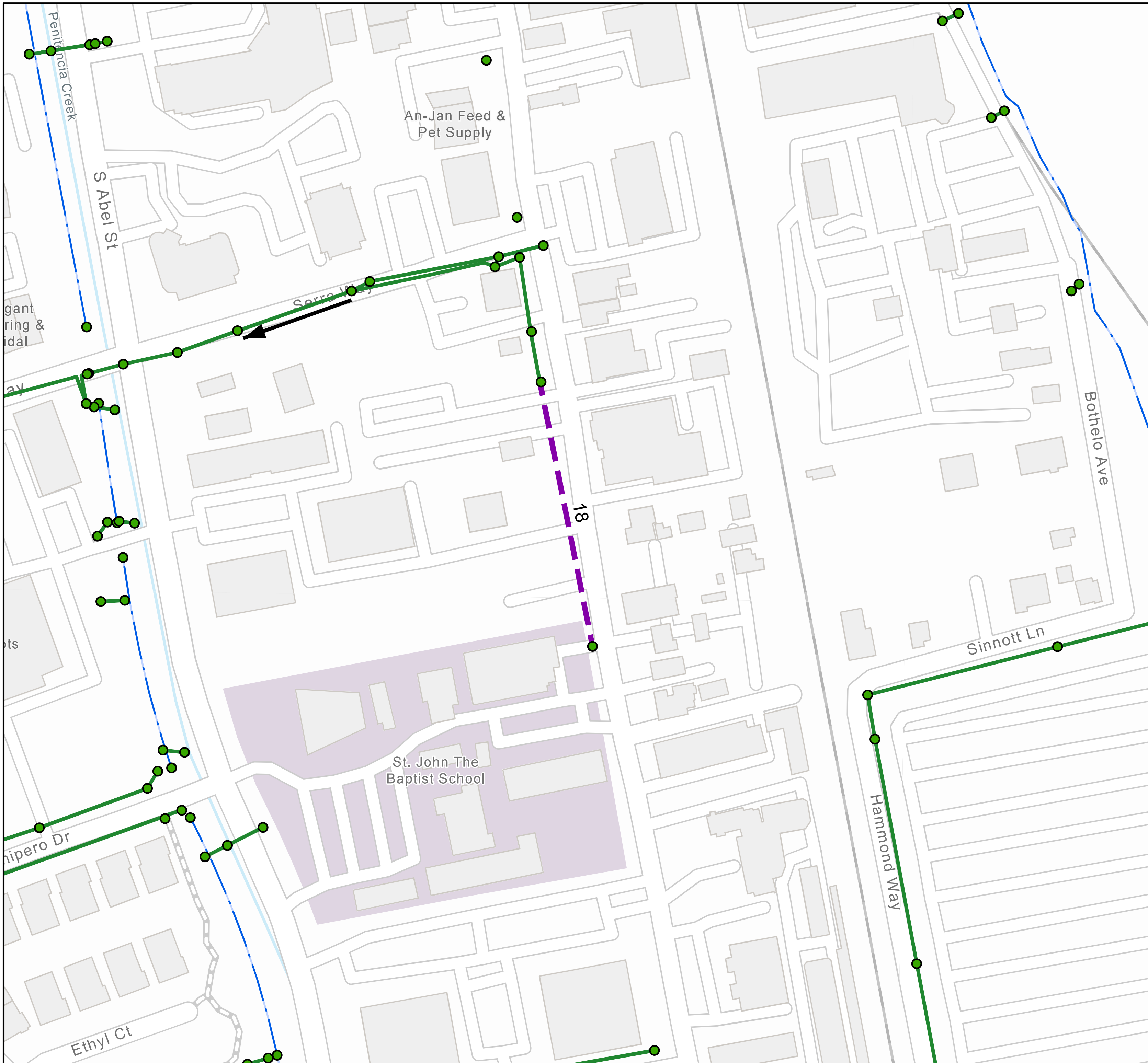


Milpitas
Storm Drainage CIP
Under-served Area
LP_16
Main Street
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

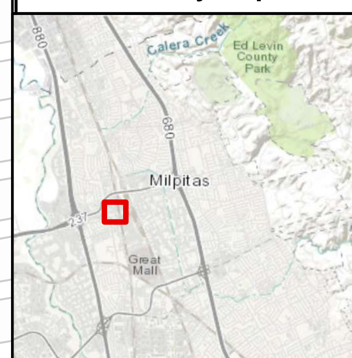
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

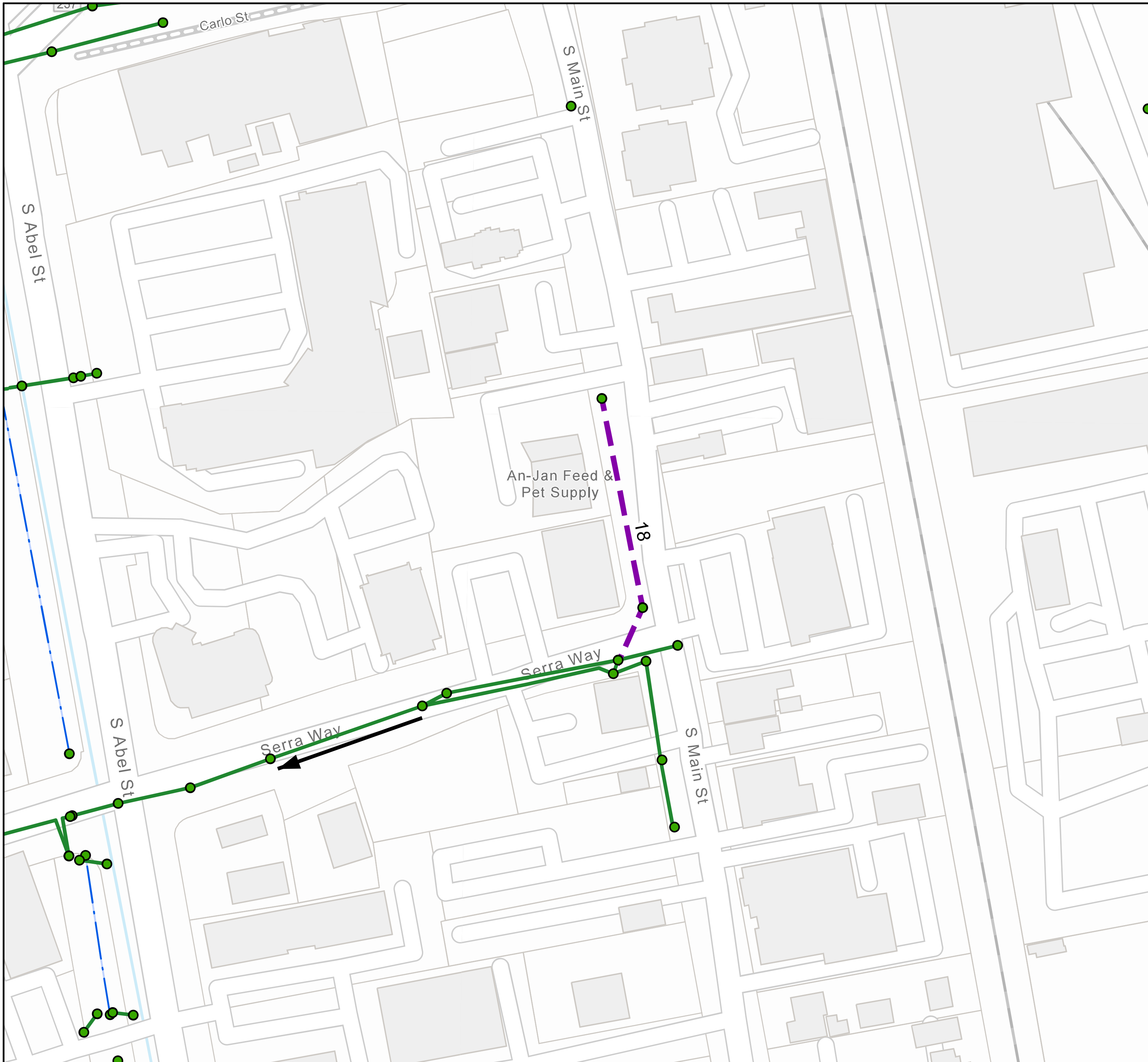
1 inch : 200 feet

Vicinity Map



Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Milpitas
Storm Drainage CIP
Under-served Area
LP_17
Main Street - Sinnott Ln
SD Improvements



Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

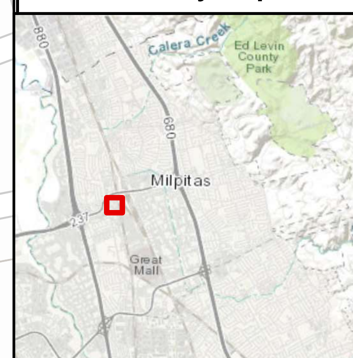
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 150 feet

Vicinity Map



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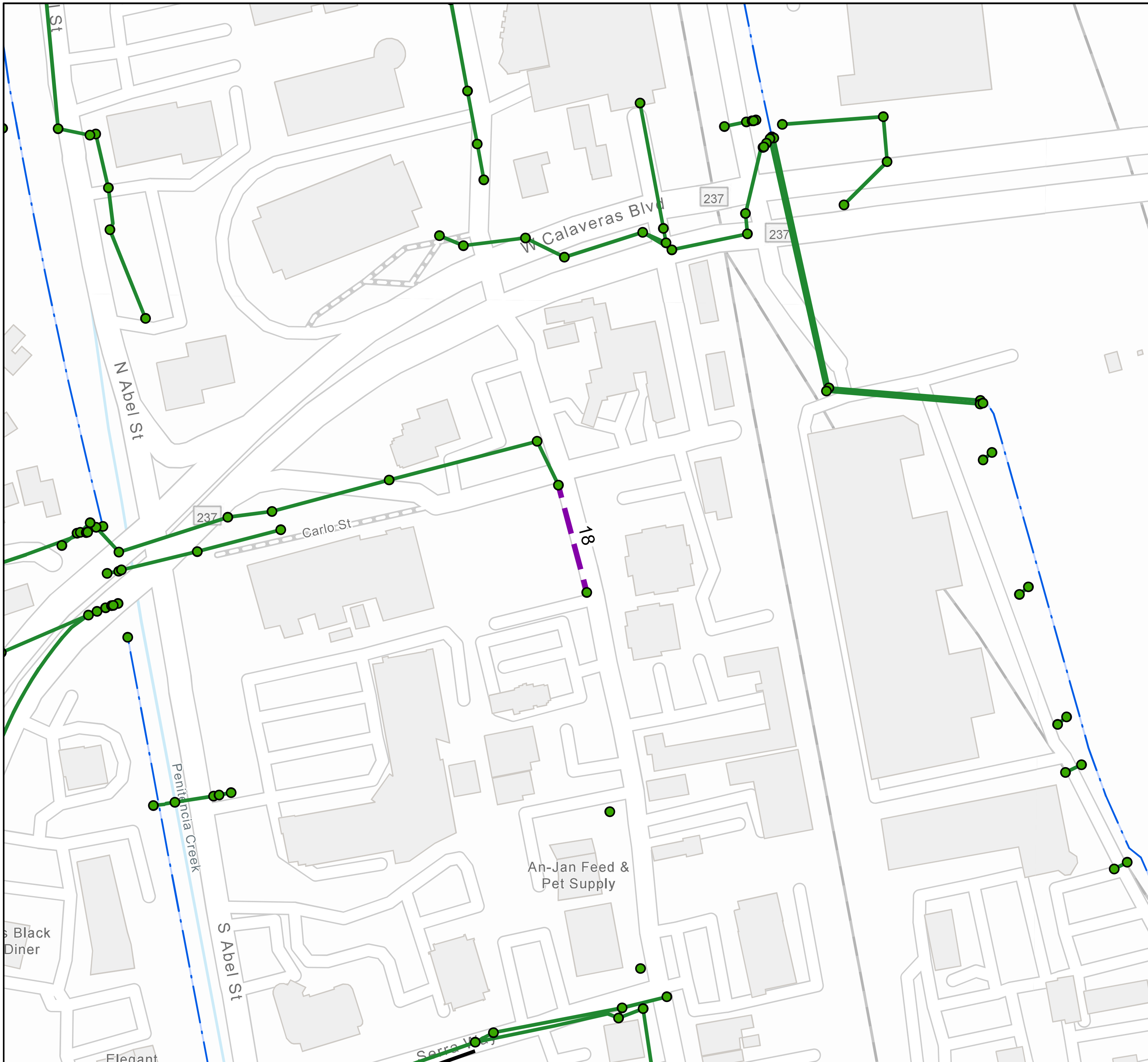


Milpitas
Storm Drainage CIP

Under-served Area
LP_18
Main Street - Carlo Street
SD Improvements

Legend

- Storm Drain Structure
- Modeled Storm Drain
- River reach

CIP Priority

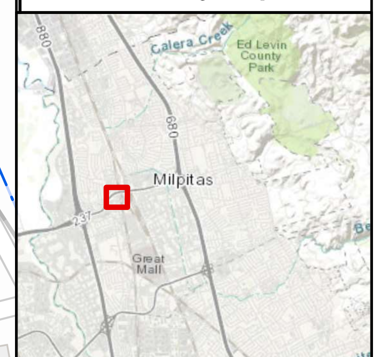
- New High Priority SD
- New Low Priority SD
- Specific Plan SD Extension
- Low Priority SD Replacement
- High Priority SD Replacement

*Pipe annotation: "a to b" where 'a' is existing size and 'b' indicates proposed

N

1 inch : 200 feet

Vicinity Map



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Appendix B Pump Station Recommendations

General recommendations for pump station design and operation, including upgrading and rehabilitating existing stations are contained in this appendix.

Pump Station Design Guidelines

These recommendations apply to the design of new or substantially renovated pumping facilities.

Capacity

Every pump station should be capable of discharging the 100-year runoff from its tributary area. A combination of pumping capacity and retention storage can accomplish this. Pump stations with lesser capacity (e.g., ten-year) should be considered only if there is a fail-safe way to overflow excess flows without causing property damage. Nearly all the pumping facilities within the city meet these criteria. Table 6-1 indicates whether individual pump stations have sufficient capacity.

Number of Pumps

For redundancy, at least two identical pumps must be installed in every stormwater pump station. It is not unnecessary to include standby pumps because providing excess capacity is expensive and not justified by the relatively small risk of having a major storm event coincide with mechanical failure. (Schedule pump maintenance for the summer months as well.) However, installing a larger number of smaller pumps is generally better than a lesser number of large pumps for the same capacity. When individual pumps comprise a smaller percentage of overall pump station capacity, having one pump out is less detrimental. In terms of redundancy and ease of maintenance, all pumping units within one particular station should be identical.

No pumping station in Milpitas is equipped with fewer than two identical pumps. Most stations have three main pumping units, and the Jurgens Pump Station has four. All stations (except California Circle, Abbott, and Minnis) have a smaller electric dewatering pump to drain the wet well when water falls below the minimum allowable pumping level for the large stormwater pumps. Permanent retention ponds are maintained at the California Circle and Abbott stations, while the Minnis station utilizes submersible pumps capable of dewatering the wet well.

Pump and Driver Types

Pump selection is on a station-by-station basis and needs coordination with City operations staff for consistency with other similar pump stations. Prime power for new pumping stations should be by an electric motor rather than a direct drive engine if at all possible, particularly for frequent operation. Electric motors are quieter, require less space and ventilation, and are not subject to tighter air quality restrictions in the future. Diesel engines drive most of the existing stormwater pumps in Milpitas. This pump driver style eliminates the need for standby power at most stations, and the City has generally experienced reliable operation.

While electric motors are recommended for new stations, the costs of adding power distribution and switching equipment, motor starters, and a standby engine-generator set to older stations generally preclude the replacement of engine drive units with electric motors. Hence, when old pumping units are upgraded or replaced, the type of replacement drive units should be consistent with existing equipment.

Pump Operation and Cycling

Lead and lag pumps should be automatically alternated on every start to minimize pump cycling and extend the operating life of the equipment. Sufficient operational wet well storage (Vol_p) must also be available to prevent excessive pump cycling for proposed operating levels:

$$Vol_p = t\left(\frac{Q_i}{Q_p}\right)\left(Q_p - Q_i\right)$$

where	Vol_p	=	active sump volume per pump (cubic feet)
	t	=	pump cycle time to fill and empty volume (minutes)
	Q_i	=	inflow into station (cubic feet per minute)
	Q_p	=	flow rate of pump (cubic feet per minute)

Differentiating the equation shows that the minimum pump cycle time occurs when flow into the pump station is exactly one-half the pumping rate. Required sump volume is determined by setting the maximum number of pumps starts per hour below the maximum criterion established by the pump, motor, or engine manufacturers. In the absence of specific data, the pump starts should be limited to six per hour. This criterion is based on general limits set by large electric motor manufacturers; diesel engine suppliers also recommend that engines should run at least five to ten minutes at full operating temperatures each time they're started.

Pumping equipment must be specified so that motor or engine nameplate ratings are not exceeded at any point on the pump characteristic curve. Pump performance under different hydraulic conditions should be analyzed to ensure that pumps operate within manufacturers' recommended limits. Pumps must discharge their rated flow against the 100-year design tailwater elevation at the station outfall.

This criterion is evaluated on a station-by-station basis. Most of the control systems used by the City can automatically rotate lead and lag pump sequences.

Forebay, Intake and Wet Well Design

If retention storage is necessary, designing the pump station forebay provides access to that storage over the pumps' operating range. The design of the forebay also plays a role in whether the retention will be wet or dry. Certain three-dimensional hydraulic phenomena often present in large pump intakes must be avoided to minimize the potential for submerged vortices, free-surface vortices, stagnations and flow separations, uneven or unsteady flow distribution, swirl of flow entering the pumps, or air entrainment.

These phenomena can lead to the degradation of pump performance, including decreased pump capacity, reduced pump efficiency, excessive wear, and increased vibration and noise. Although quantifying this complex hydraulic behavior is virtually impossible without scaled physical model studies, pumping intakes and sumps designed in conformance with one of the following standards should perform at their optimal level under varying operating conditions:

British Hydromechanics Research Association (BHRA)

Hydraulic Institute

Pump manufacturer design guidelines

Failure to properly design intake configurations can lead to negative performance, as described above. Since it is difficult to quantitatively analyze the three-dimensional flow phenomena associated with large pump intakes, any unusual intake, wet well, or sump designs that do not conform to an established standard are subject to physical model testing.

Most of Milpitas' pump stations are designed to house several pumps in the same wet well. Under these conditions, the primary objective in inlet design is still to provide an even, air-free flow distribution to each pump intake regardless of pump configuration or which pumps are operating. Given the difficulty of providing uniform flow distribution in multiple pump sumps, current design standards favor using a “unitized” wet well, whereby a number of single-pump sumps (pump cells) are placed side by side.

In Milpitas, only the Wrigley-Ford Pump Station utilizes this unitized sump approach and strictly meets current Hydraulic Institute standards for sump design. During individual pump station evaluations, inlet and sump dimensions are compared to recommended standards. Most station dimensions do not strictly match those standards. Correcting the deficiencies, however, can be extremely difficult and expensive. Since most pump stations operate for only a limited number of hours in any year, and there has been no demonstrated catastrophic loss of efficiency, the master plan does not recommend correcting sump design deficiencies. In order to minimize problems caused by deficient sump inlet design, proper pump submergence must be maintained. This will lower intake velocities and help reduce the risk of vortex formation and air entrainment.

Pumping equipment that demonstrates excessive wear, vibration, noise, and particularly cavitation may be indicative of more serious hydraulic problems associated with the sump and intake. In those instances, physical model studies and sump rehabilitation is warranted.

Submergence

Established operating prevent excessive cycling and provide for adequate pump submergence, defined as the minimum allowable height of the low water level above the pump suction inlet. Inadequate submergence can lead to the inducement of free-surface and submerged vortices, the entrainment of air, a reduction in pumping capacity, and premature pump failure. Limited pump submergence can also potentially lead to pump cavitation, which may cause severe damage. As a rule of thumb, the minimum water depth should be two bell diameters over the wet well floor for submergence of one and a half bell diameters (BHRA guidelines). Recent design guidelines published by the Hydraulic Institute (1998) suggest the following formula for establishing minimum pump submergence:

$$S = D\,(1 + 2.3\,F_D)$$

where S = submence (feet)

D = pump bell diameter (feet)

F_D = Froude number at pump inlet, given as:

$$F_D = \frac{V}{(g\,D)^{0.5}}$$

V = velocity at the pump suction inlet (fps)

g = gravitational acceleration

Discharge Piping

Pump discharge piping must protect upstream systems and properties from damage caused by backwater from high tides or tailwater. Discharge flap gates or check valves should be provided, or pipe discharge elevations need to be three feet above the design tailwater level and/or one foot above protective levees.

Pump Testing

Station design should provide for the testing of pumps with water under non-storm conditions. A means for recirculating water, or some other method, can be provided to enable a test of reasonable duration (15 minutes). This may also be important since engines need exercise at least once a month and should be done so under load (see Chapter 9).

Most pump stations in the city do not have the means to recirculate water for testing. (Wrigley-Ford Storm Water Pump Station is a notable exception.) Those stations with large forebays – California Circle, Abbott, Penitencia, and Berryessa – can simply discharge water over an extended period without necessarily needing an influent runoff. For other stations, unfortunately, providing additional piping for testing is difficult.

Standby Power

An emergency engine-generator, capable of starting the largest motor while running all other motors and auxiliary loads, should be installed at each stormwater pump station that does not utilize engines for prime pump drivers. Diesel is the preferred fuel, but natural gas engines may be considered as an alternative since they are reliable and burn cleanly. Natural gas engines, however, tend to be underpowered compared to diesel engines. There is also a risk that the fuel will not be available when needed.

Gasoline is not an acceptable fuel for stationary engines because it is a fire and explosion hazard, and the allowable storage period is very short. Diesel fuel is much less hazardous and can be stored for up to a year in double-walled tanks meeting requirements set forth by the Fire Department. All fuel piping must be double contained.

Discussions with City operations and maintenance personnel indicate that a majority of the city's pumping stations have been upgraded to meet current fuel storage requirements, with double-walled tanks (about one-half of which are above ground), double-contained fuel piping, and leak detection.

Engine-generators should be housed in a sound-attenuated weatherproof enclosure or inside buildings meeting appropriate codes for such use. Proper ventilation will be provided for engine aspiration and cooling. Some means for exercising the engine-generator set under load must also be furnished, either through pump testing with water as described above, load banks, or a combination of both. Generators must be present on-site and connected to the power supply with an automatic transfer switch to be considered as available in an emergency. The use of portable generators, or even permanently parked generators with manual transfer switches, is not recommended since crews may not be able to respond to high water alarms, physically reach the pump station with a generator, and manually restore power before property damage has occurred. This is the current condition at the Spence Creek and Manor stations.

Small lift or pumping stations that generally handle "nuisance" flows (if the pump station were to fail to operate significant property damage does not occur) would not necessarily require a standby power source.

Controls and SCADA

The pump starts and stops using a programmable logic controller (PLC) or programmable pump controller. Pump station controls are tied into the City's Supervisory Control and Data Acquisition (SCADA) telemetry system. City operations and maintenance staff shall coordinate pump station controls and level monitoring systems regarding function and standardization. They must also provide control with standby power to ensure that the station can function even during prolonged power outages. The preferred mechanism for providing standby power to control systems is rechargeable batteries.

Equipment Housing

All electrical equipment in or open to the wet well must be explosion-proof and placed a minimum of one foot above the base flood elevation (BFE). Submersible motors should also be explosion-proof. Control panels must not be located so they are subject to possible flooding. All equipment must be housed in NEMA-rated weatherproof enclosures or in buildings. Sufficient lighting (including back-up battery power) should be provided so that crews may work on equipment during the night. Also, access must be provided that will allow for the removal and reinstallation of all equipment.

Consider noise abatement, visual impacts, and odor control when locating a pump station and designing the equipment housing. This is particularly important where the installation of engine units are near residential areas.

Ventilation

Good ventilation is important to maintaining a dry, benign environment for mechanical and electrical equipment within a pump station. Proper ventilation helps reduce the deterioration of equipment due to condensation and provides better working conditions for City crews. Without adequate ventilation, enclosures below grade may be classified as confined spaces, requiring special permits and rescue equipment for anyone entering them. Explosive gases from illegally dumped flammable liquids may also accumulate in wet wells and ancillary spaces. Many deaths and illnesses have been attributed to poor ventilation at pump stations.

Wet wells can intentionally be designed as a confined space, particularly if there is no regular need for personnel to enter them. (The only equipment allowed in such a wet well includes explosion-proof measuring devices and submersible pumps.) However, proper ventilation should be provided for pump station buildings, particularly those housing engines or engine-generators.

All heating, ventilating, and cooling systems should be designed in conformance with city ordinances; uniform building, fire, mechanical, plumbing, and energy codes; the National Electric Code and NFPA; EPA regulations; Occupational Safety and Health Act (OSHA) requirements; and ASHRAE design standards.

Radiator exhaust ducts should be designed based on actual airflow requirements, but in general, they need to discharge at air velocities no greater than 800 to 1500 feet per minute. Intake louvers that bring air into the pump station should be designed with sufficient free area to maintain velocities of 250 to 400 feet per minute. This helps keep the rain out of the pump station when the engines are operating.

Low Flow Bypass

If conditions permit, a gravity outflow pipe that bypasses the pump station should be installed. During low tailwater conditions, a substantial savings in pumping costs can be realized. Pumping stations with bypass capability include Penitencia, Wrigley-Ford, Spence Creek, and Manor.

Pump Station Operation and Maintenance Guidelines

These general recommendations apply to all pumping facilities as appropriate.

Pumps

Large axial flow pumps with right angle gear drives, which are the predominant pump type in the systefm, actually require fairly little maintenance. Shafts and bearings need to be periodically balanced and/or replaced. The frequency of inspection (pumps will need to be pulled out of the building) will vary depending upon the "L-10" bearing life rating of the pump in question. Average bearing life is defined as the operating hours at which half of the group of bearings fails, and the rest continue to operate. AFBMA (the Anti-Friction Bearing Manufacturers Association) defines average life statistically as three to five times the L-10 life. (For example, the Wrigley-Ford pumps have 50,000-hour bearings.) Although grease is the most maintenance-free bearing lubricant, most of the pumps in Milpitas have drip fee oil systems, which ensure the lowest bearing operating temperature. Consequently, the oiling reservoir needs to be checked on a routine basis and topped off as necessary

Engines

Manufacturers' maintenance instructions should be followed to the letter, particularly when the engine is still under warranty. Maintenance schedules depend somewhat on whether the engine is used as the prime pump driver (as in most stations) or is on standby (such as for power generation).

A typical schedule of maintenance based on references provided by Cummins/Onan (Sanks, 1989) is provided as Table B-1, giving both operating hours and calendar time.

Diesel engines should be operated at full power for at least 15 to 30 minutes after reaching operating temperatures once a month to eliminate carbon deposits. Unfortunately, without significant stormwater inflow to the station, pump engines cannot be run under load for any significant length of time (the water quickly runs out). Wrigley-Ford Pump Station is equipped both with a resistive load bank to provide a working load for automatic exercise and a discharge pipe system to recirculate available water back into the forebay. The pumps can be tested and exercised without large amounts of inflow.

Other stations with permanent water storage (California Circle Lagoon, Abbott Lagoon, Hall Memorial Park lagoon, and Hidden Lake) could be run for 15 minutes every month, although lagoon levels will not necessarily return to normal quickly. Unless provisions for the recirculation of test water at the other pump stations are made, those engines cannot be exercised under load during the summer months. Providing for test water recirculation has not been included in the Capital Improvement Program because retrofitting existing stations to operate in this manner is difficult.

Diesel oil is safer to store than most fuels and is easy to obtain and transport, but diesel deteriorates in storage and must be turned over every six months to one year.

Table B-1: Typical Maintenance Frequency for Engines and EG-Sets

Maintenance Task	Operating Time	Calendar Time
Inspect fuel, oil level, coolant	8 hr	1 m
Inspect air cleaner, battery	50 hr	1 yr
Clean governor linkage, breather, air cleaner	100 hr	1 yr
Clean fuel filter, replace oil filter, change crankcase oil, check switchgear	200 hr	1 yr

Maintenance Task	Operating Time	Calendar Time
Clean commutator, collector rings, relays, cooling system; inspect brushes, valve clearances, starting and stopping systems, water pump	500 hr	1 yr
Check injectors, grind valves (if required), remove carbon, clean oil passages, replace secondary fuel filter, clean generator, grease bearings	1000 hr	----

Assumed Pump Operating Levels

Operating levels are assumed at individual pumping facilities for storm drain master plan modeling. It is noted that the City changes these levels depending upon station operating needs, including repair and maintenance. Actual operating levels and station operation manual guidelines supersede the levels provided herein. All levels are feet NAVD with distance above the wet well floor in parentheses.

California Circle Pump Station (SD-1)

Current pump operating levels are:

#3 ON	9.3 feet NAVD
#2 ON	7.5
#1 ON	5.8
PUMPS OFF	4.5

When the pump settings listed above are analyzed, the maximum one-percent lagoon level is 11.0 feet NAVD. Although this is less than the minimum elevation on California Circle, it is only 0.8 feet lower than the one-percent water surface elevation in Lower Penitencia Creek. To minimize pond fluctuations while beginning pumping in time to accommodate inflow during heavy runoff periods, the pump-on levels for the second and third pumps in the rotation could be lowered during the rainy season. Since a large volume of storage is available, the pump set points can be set closer together without excessive cycling, which is assumed for storm drain master plan modeling.

HI ALARM	11.0	(17.0')
#3 ON	7.5	(15.5')
#2 ON	6.5	(12.5')
#1 ON	5.5	(11.5')
#3 OFF	5.0	(11.0')
#2 OFF	4.7	(10.7')
#1 OFF	4.5	(10.5')
LOW ALARM	1.5	(7.5')

Jurgens Pump Station (SD-2)

To minimize pump cycling the following operating levels are assumed.

HI ALARM	9.5	(19.0')
#4 ON	8.0	(17.5')
#3 ON	7.5	(17.0')
#2 ON	7.0	(16.5')
#1 ON	5.0	(14.5')
#4 OFF	1.5	(11.0')
#3 OFF	1.0	(10.5')
#2 OFF	0.5	(10.0')
#1 OFF	0.0	(9.5')
LOW ALARM	-1.0	(8.5')

McCarthy Pump Station (SD-3)

To enhance operational efficiencies and minimize pump cycling, however, it is recommended that pump starts rotate and the following operating levels are assumed. Pumps will start no more than five times per hour.

HI ALARM	2.0	(19.0')	
#3 ON	1.5	(18.5')	
#2 ON	1.0	(18.0')	
#1 ON	0.5	(17.5')	
#3 OFF	-4.5	(12.5')	
#2 OFF	-5.0	(12.0')	
#1 OFF	-5.5	(11.5')	
LOW ALARM	-6.0	(11.0')	Drainage Pumps
JOCKEY ON	-15.5	(1.5')	
JOCKEY OFF	-17.0	(-0.5')	
LOW ALARM	-17.5	(-1.0')	Jockey Pump

Abbott Pump Station (SD-4)

These operating levels are sufficient to prevent pump cycling, while maintaining the aesthetic function of the lagoon. (Elevations are given as feet NAVD with distance above the wet well floor in parenthesis.

HI ALARM	11.0	(16.0')
#2 ON	9.5	(14.5')
#1 ON	9.0	(14.0')
#2 OFF	8.5	(13.5')
#1 OFF	8.0	(13.0')
LOW ALARM	6.0	(11.0')

Minnis Pump Station (SD-5)

The following operating levels allow for eight starts per hour in alternation, since wet well storage is very limited. Pump settings are provided in feet NAVD with the distance from the wet well floor in parentheses.

HI ALARM	5.0	(12.0')
#2 ON	4.5	(11.5')
#1 ON	4.0	(11.0')
#2 OFF	-3.5	(3.5')
#1 OFF	-4.0	(3.0')
LOW ALARM	-4.5	(2.5')

Penitencia Pump Station (SD-6)

Original pump operating levels are assumed per record plans.

HI ALARM	12.0	(12.2')
#3 ON	9.8	(10.0')
#2 ON	8.3	(8.5')
#1 ON	6.3	(6.5')
ALL OFF	5.3	(5.5')
SUMMER WSEL	6.4	(6.6')

Wrigley-Ford Pump Station (SD-7)

Minimum individual pump cycle times (based on pump rotation) and settings are given below.

HI ALARM	13.7	(17.0')	
#3 ON	13.2	(16.5')	2 starts per hour
#2 ON	12.7	(16.0')	2 starts per hour
#1 ON	12.2	(15.5')	1 start per hour
#3 OFF	12.0	(15.3')	
#2 OFF	11.7	(15.0')	
#1 OFF	11.2	(14.5')	
JOCKEY ON	-5.5	(1.0')	
JOCKEY OFF	-6.5	(-0.5')	

Berryessa Pump Station (SD-8)

City staff has provided current pump settings, which are indicated below as referenced from the pump station sump floor (elevation -6.0 feet NAVD). The resulting 100-year water surface elevation of 10.7 feet would not cause spill out of the lake or property damage in the absence of Calera Creek overflows, and with overflow from Calera Creek, the resulting flood level of 15 feet remains below the pumping equipment flood-proof elevation. (Levels have been referenced to feet NAVD with distance above the wet well floor given parenthetically.)

HI ALARM	10.0	(16.0')
#3 ON	9.5	(15.5')
#2 ON	9.0	(15.0')
#1 ON / #3 OFF	8.5	(14.5’)
#2 OFF	8.0	(14.0’)
#1 OFF	7.0	(13.0')
SUMMER WSEL	9.0	(15.0')

Manor Pump Station (SD-9)

The assumed design operating levels are sufficient to prevent pump cycling, while maintaining minimum pump submergence levels.

HI ALARM	7.7	(12.5')	
#3 (NEW) ON	6.7	(11.5')	
#2 ON	6.2	(11.0')	
#1 ON	5.7	(10.5')	
#3 (NEW) OFF	2.7	(7.5')	
#2 OFF	1.7	(6.5')	
#1 OFF	0.7	(5.5')	
LOW ALARM	-0.7	(4.5')	Stops Drainage Pumps
JOCKEY ON	-3.8	(1.0')	
JOCKEY OFF	-6.8	(-2.0')	
LOW ALARM	-7.3	(-2.5')	Stops Jockey Pump

Spence Creek Pump Station (SD-10)

The design operating levels are sufficient to prevent pump cycling, while maintaining minimum pump submergence levels. Spence creek itself also provides storage volume that helps limit pump starts. (All levels are referenced to feet NAVD with distance from wet well bottom indicated parenthetically.)

HI ALARM	14.2	(10.5')	
#3 ON	14.0	(10.3’)	
#2 ON	13.7	(10.0')	
#1 ON	11.7	(8.0')	
#3 OFF	11.2	(7.5’)	
#2 OFF	10.2	(6.5')	
#1 OFF	9.2	(5.5')	
LOW ALARM	8.2	(4.5')	Stops Drainage Pumps
JOCKEY ON	4.7	(1.0')	
JOCKEY OFF	1.7	(-2.0')	
LOW ALARM	1.2	(-2.5')	Stops Jockey Pump

Bellew Pump Station (SD-11)

HI ALARM	15.0	(19.0')	
#3 ON	14.0	(18.0')	
#2 ON	13.5	(17.5')	
#1 ON	13.0	(17.0')	
#3 OFF	8.0	(12.0')	
#2 OFF	7.5	(11.5')	
#1 OFF	7.0	(11.0')	
LOW ALARM	6.5	(10.5')	Drainage Pumps
JOCKEY ON	-2.5	(1.5')	
JOCKEY OFF	-4.1	(-0.1')	
LOW ALARM	-17.5	(-1.0')	Jockey

Murphy Pump Station (SD-12)

HI ALARM	19.2	(16.2')	
#3 ON	19.0	(16.0')	
#2 ON	18.5	(15.5')	
#1 ON	18.0	(15.0')	
#3 OFF	14.0	(11.0')	
#2 OFF	13.5	(10.5')	
#1 OFF	13.0	(10.0')	
LOW ALARM	11.5	(8.5')	Drainage Pumps
JOCKEY ON	4.0	(1.0')	
JOCKEY OFF	2.8	(-0.2')	
LOW ALARM	2.5	(-0.5')	Jockey Pump

Oak Creek Pump Station (SD-13)

HI ALARM	20.0	(14.0')	
#3 ON	19.5	(13.5')	
#2 ON	19.0	(13.0')	
#1 ON	18.5	(12.5')	
#3 OFF	17.0	(11.0')	
#2 OFF	16.5	(10.5')	
#1 OFF	16.0	(10.0')	
LOW ALARM	15.5	(9.5')	Drainage Pumps
JOCKEY ON	7.0	(1.0')	
JOCKEY OFF	5.8	(-0.2')	
LOW ALARM	5.5	(-0.5')	Jockey Pump

Appendix C Detailed CIP Cost Estimates

Appendix C

CIP Cost Summary						
Improvement Name	Improvement Street	Priority	Construction Total	Engineering/ Inspection	CIP Total	CIP Total
CAL_1	Tice Drive	Low	$ 281,991	$ 56,398	$ 338,390	$ 340,000
CAL_2	Jacklin Road	High	$ 800,907	$ 160,181	$ 961,089	$ 960,000
CAL_3	Bayview Park Drive	High	$ 178,623	$ 35,725	$ 214,347	$ 210,000
CAL_4	Wool Drive	Low	$ 1,925,345	$ 385,069	$ 2,310,413	$ 2,310,000
COCHES_5	S Park Victoria	High	$ 9,638,785	$ 1,927,757	$ 11,566,542	$ 11,570,000
COCHES_8	Foothill Park	Low	$ 863,366	$ 172,673	$ 1,036,039	$ 1,040,000
BERRY_10	Ames Avenue	High	$ 1,385,662	$ 277,132	$ 1,662,795	$ 1,660,000
WF_11	Comet Drive	Low	$ 163,431	$ 32,686	$ 196,117	$ 200,000
LP_12	Main Street-Serra Way	Low	$ 236,388	$ 47,278	$ 283,666	$ 280,000
WF_13	Railroad Avenue	Low	$ 1,968,155	$ 393,631	$ 2,361,786	$ 2,360,000
LP_14	North Abel	Low	$ 4,247,749	$ 849,550	$ 5,097,299	$ 5,100,000
LP_15	Main Street-Tom Evatt Park	Underserved	$ 152,277	$ 30,455	$ 182,732	$ 180,000
LP_16	Main Street	Underserved	$ 127,256	$ 25,451	$ 152,708	$ 150,000
LP_17	Main Street-Sinnott Ln	Underserved	$ 52,107	$ 10,421	$ 62,528	$ 60,000
LP_18	Main Street-Carlo Street	Underserved	$ 94,730	$ 18,946	$ 113,676	$ 110,000
INLET_19	Inlet Replacements	High			$ 1,400,000	$ 1,400,000

High	**$ 16,000,000**
Low	**$ 12,000,000**
Underserved	**$ 1,000,000**
Total	**$ 29,000,000**

Detailed CIP Cost Summary

Project	Length (ft)	Diameter (in)	Priority	Pipe Cost	MHs	MH Cost	Outlets	Outlet Cost	Pipe Length	Pipe Cost	MH Cost	Outlet Cost	Subtotal	Mob/Demob	Traffic Control	Contingency	Const Total	Eng/Inspect	CIP Total
CAL_1	261	18	Low	$ 73,915	1	$ 13,369	0	$ -											
CAL_1	287	18	Low	$ 81,278	1	$ 13,369	0	$ -	548	$ 155,192	$ 26,737	$ -	$ 181,930	$ 18,193	$ 9,096	$ 72,772	$ 281,991	$ 56,398	$ 338,390
CAL_2	946	36	High	$ 488,259	2	$ 28,456	0	$ -	946	$ 488,259	$ 28,456	$ -	$ 516,714	$ 51,671	$ 25,836	$ 206,686	$ 800,907	$ 160,181	$ 961,089
CAL_3	83	24	High	$ 29,079	1	$ 13,655	0	$ -											
CAL_3	53	24	High	$ 18,569	1	$ 13,655	0	$ -											
CAL_3	76	24	High	$ 26,627	1	$ 13,655	0	$ -	212	$ 74,275	$ 40,965	$ -	$ 115,241	$ 11,524	$ 5,762	$ 46,096	$ 178,623	$ 35,725	$ 214,347
CAL_4	314	24	Low	$ 110,011	1	$ 13,655	0	$ -											
CAL_4	68	24	Low	$ 23,824	1	$ 13,655	0	$ -											
CAL_4	184	24	Low	$ 64,465	1	$ 13,655	0	$ -											
CAL_4	66	30	Low	$ 29,494	1	$ 13,941	0	$ -											
CAL_4	543	30	Low	$ 242,652	2	$ 27,883	0	$ -											
CAL_4	426	36	Low	$ 219,778	1	$ 14,228	0	$ -											
CAL_4	238	36	Low	$ 122,787	1	$ 14,228	0	$ -											
CAL_4	62	18	Low	$ 17,558	0	$ -	0	$ -											
CAL_4	91	18	Low	$ 25,771	0	$ -	0	$ -											
CAL_4	173	18	Low	$ 48,993	0	$ -	0	$ -											
CAL_4	135	30	Low	$ 60,328	1	$ 13,941	0	$ -											
CAL_4	168	30	Low	$ 75,075	1	$ 13,941	0	$ -											
CAL_4	77	30	Low	$ 34,409	2	$ 27,883	0	$ -	2,545	$ 1,075,146	$ 167,011	$ -	$ 1,242,158	$ 124,216	$ 62,108	$ 496,863	$ 1,925,345	$ 385,069	$ 2,310,413
COCHES_5	289	18	High	$ 81,788	0	$ -	0	$ -											
COCHES_5	395	36	High	$ 203,527	2	$ 28,456	0	$ -											
COCHES_5	274	36	High	$ 141,566	1	$ 14,228	0	$ -											
COCHES_5	367	36	High	$ 189,546	1	$ 14,228	0	$ -											
COCHES_5	216	36	High	$ 111,282	1	$ 14,228	0	$ -											
COCHES_5	490	36	High	$ 252,797	1	$ 14,228	0	$ -											
COCHES_5	26	36	High	$ 13,414	1	$ 14,228	0	$ -											
COCHES_5	255	36	High	$ 131,557	1	$ 14,228	0	$ -											
COCHES_5	100	36	High	$ 51,591	1	$ 14,228	0	$ -											
COCHES_5	40	36	High	$ 20,636	1	$ 14,228	0	$ -											
COCHES_5	218	36	High	$ 112,469	1	$ 14,228	0	$ -											
COCHES_5	331	36	High	$ 170,767	1	$ 14,228	0	$ -											
COCHES_5	206	36	High	$ 106,278	1	$ 14,228	0	$ -											
COCHES_5	53	36	High	$ 27,343	1	$ 14,228	0	$ -											
COCHES_5	8	36	High	$ 4,334	0	$ -	0	$ -											
COCHES_5	12	36	High	$ 5,985	0	$ -	1	$ 45,000											
COCHES_5	296	36	High	$ 152,710	1	$ 14,228	0	$ -											
COCHES_5	96	36	High	$ 49,527	1	$ 14,228	0	$ -											
COCHES_5	315	36	High	$ 162,512	1	$ 14,228	0	$ -											
COCHES_5	321	36	High	$ 165,608	1	$ 14,228	0	$ -											
COCHES_5	12	36	High	$ 6,191	1	$ 14,228	0	$ -											
COCHES_5	157	36	High	$ 80,998	1	$ 14,228	0	$ -											
COCHES_5	255	18	High	$ 72,215	0	$ -	0	$ -											
COCHES_5	284	24	High	$ 99,501	0	$ -	0	$ -											
COCHES_5	916	24	High	$ 321,030	1	$ 13,655	0	$ -											
COCHES_5	165	54	High	$ 129,948	1	$ 16,080	0	$ -											
COCHES_5	1100	54	High	$ 868,054	2	$ 32,159	0	$ -											
COCHES_5	260	54	High	$ 205,139	1	$ 16,080	0	$ -											
COCHES_5	193	54	High	$ 152,276	1	$ 16,080	0	$ -											
COCHES_5	56	54	High	$ 44,184	1	$ 16,080	0	$ -											
COCHES_5	390	54	High	$ 307,867	1	$ 16,080	0	$ -											
COCHES_5	224	60	High	$ 201,098	1	$ 16,406	0	$ -											
COCHES_5	170	60	High	$ 152,619	1	$ 16,406	0	$ -											
COCHES_5	430	60	High	$ 386,037	1	$ 16,406	0	$ -											
COCHES_5	250	60	High	$ 224,440	1	$ 16,406	0	$ -											
COCHES_5	4	60	High	$ 3,501	1	$ 16,406	0	$ -											
COCHES_5	9	60	High	$ 8,170	0	$ -	1	$ 45,000											
COCHES_5	150	30	High	$ 66,986	2	$ 27,883	0	$ -											
COCHES_5	274	30	High	$ 122,399	0	$ -	0	$ -	9,607	$ 5,607,889	$ 520,682	$ 90,000.00	$ 6,218,571	$ 621,857	$ 310,929	$ 2,487,428	$ 9,638,785	$ 1,927,757	$ 11,566,542
COCHES_8	246	24	Low	$ 86,187	0	$ -	0	$ -											
COCHES_8	960	30	Low	$ 428,998	3	$ 41,824	0	$ -	1,206	$ 515,186	$ 41,824	$ -	$ 557,010	$ 55,701	$ 27,851	$ 222,804	$ 863,366	$ 172,673	$ 1,036,039
BERRY_10	227	24	High	$ 79,531	0	$ -	0	$ -											
BERRY_10	528	30	High	$ 235,949	3	$ 41,824	0	$ -											

Appendix C

Project	Length (ft)	Diameter (in)	Priority	Pipe Cost	MHs	MH Cost	Outlets	Outlet Cost	Pipe Length	Pipe Cost	MH Cost	Outlet Cost	Subtotal	Mob/Demob	Traffic Control	Contingency	Const Total	Eng/Inspect	CIP Total
BERRY_10	111	30	High	$ 49,603	1	$ 13,941	0	$ -											
BERRY_10	325	42	High	$ 199,664	1	$ 14,514	0	$ -											
BERRY_10	180	42	High	$ 110,583	1	$ 14,514	0	$ -											
BERRY_10	75	42	High	$ 46,076	1	$ 14,514	0	$ -											
BERRY_10	21	42	High	$ 13,147	0	$ -	0	$ -											
BERRY_10	25	42	High	$ 15,113	0	$ -	1	$ 45,000	1,492	$ 749,667	$ 99,309	$ 45,000.00	$ 893,976	$ 89,398	$ 44,699	$ 357,590	$ 1,385,662	$ 277,132	$ 1,662,795
WF_11	91	24	Low	$ 31,882	1	$ 13,655	0	$ -											
WF_11	46	24	Low	$ 16,116	1	$ 13,655	0	$ -											
WF_11	86	24	Low	$ 30,131	0	$ -	0	$ -	223	$ 78,129	$ 27,310	$ -	$ 105,439	$ 10,544	$ 5,272	$ 42,176	$ 163,431	$ 32,686	$ 196,117
LP_12	77	15	Low	$ 20,097	0	$ -	0	$ -											
LP_12	113	15	Low	$ 29,494	0	$ -	0	$ -											
LP_12	72	18	Low	$ 20,390	0	$ -	0	$ -											
LP_12	197	18	Low	$ 55,790	2	$ 26,737	0	$ -	459	$ 125,771	$ 26,737	$ -	$ 152,509	$ 15,251	$ 7,625	$ 61,003	$ 236,388	$ 47,278	$ 283,666
WF_13	1019	42x28	Low	$ 688,424	3	$ 40,965	0	$ -											
WF_13	84	42x28	Low	$ 57,036	1	$ 13,798	0	$ -											
WF_13	62	42x28	Low	$ 41,899	1	$ 13,941	0	$ -											
WF_13	380	42	Low	$ 233,454	0	$ -	0	$ -											
WF_13	349	36	Low	$ 180,259	0	$ -	0	$ -	1,895	$ 1,201,073	$ 68,705	$ -	$ 1,269,778	$ 126,978	$ 63,489	$ 507,911	$ 1,968,155	$ 393,631	$ 2,361,786
LP_14	347	30	Low	$ 155,065	1	$ 13,941	0	$ -											
LP_14	389	30	Low	$ 173,834	1	$ 13,941	0	$ -											
LP_14	395	30	Low	$ 176,515	1	$ 13,941	0	$ -											
LP_14	157	36	Low	$ 80,998	2	$ 28,456	0	$ -											
LP_14	395	24	Low	$ 138,390	1	$ 13,655	0	$ -											
LP_14	254	42	Low	$ 156,045	1	$ 14,514	0	$ -											
LP_14	250	42	Low	$ 153,588	1	$ 14,514	0	$ -											
LP_14	246	42	Low	$ 151,315	1	$ 14,514	0	$ -											
LP_14	254	42	Low	$ 155,922	1	$ 14,514	0	$ -											
LP_14	28	24	Low	$ 9,810	0	$ -	0	$ -											
LP_14	134	24	Low	$ 46,948	0	$ -	0	$ -											
LP_14	51	42	Low	$ 31,332	1	$ 14,514	0	$ -											
LP_14	516	42	Low	$ 317,006	2	$ 29,028	0	$ -											
LP_14	272	42	Low	$ 167,104	1	$ 14,514	0	$ -											
LP_14	418	42	Low	$ 256,799	1	$ 14,514	0	$ -											
LP_14	370	42	Low	$ 227,310	1	$ 14,514	0	$ -											
LP_14	161	42	Low	$ 98,911	1	$ 14,514	0	$ -	4,637	$ 2,496,891	$ 243,592	$ -	$ 2,740,483	$ 274,048	$ 137,024	$ 1,096,193	$ 4,247,749	$ 849,550	$ 5,097,299
LP_15	300	18	Underserved	$ 84,874	1	$ 13,369	0	$ -	300	$ 84,874	$ 13,369	$ -	$ 98,243	$ 9,824	$ 4,912	$ 39,297	$ 152,277	$ 30,455	$ 182,732
LP_16	406	18	Underserved	$ 115,007	1	$ 13,369	0	$ -											
LP_16	243	18	Underserved	$ 68,732	1	$ 13,369	0	$ -	243	$ 68,732	$ 13,369	$ -	$ 82,101	$ 8,210	$ 4,105	$ 32,840	$ 127,256	$ 25,451	$ 152,708
LP_17	72	18	Underserved	$ 20,249	1	$ 13,369	0	$ -	72	$ 20,249	$ 13,369	$ -	$ 33,617	$ 3,362	$ 1,681	$ 13,447	$ 52,107	$ 10,421	$ 62,528
LP_18	169	18	Underserved	$ 47,747	1	$ 13,369	0	$ -	169	$ 47,747	$ 13,369	$ -	$ 61,116	$ 6,112	$ 3,056	$ 24,446	$ 94,730	$ 18,946	$ 113,676
				Total					**$ 24,553**	**$ 12,789,081**	**$ 1,344,805**		**$ 14,268,885**	**$ 1,426,889**	**$ 713,444**	**$ 5,707,554**	**$ 22,116,772**	**$ 4,423,354**	**$ 26,540,127**

Appendix C

Rates		
Type	Diameter (in)	Rate ($/LF)
Pipe	8	$ 230
Pipe	10	$ 230
Pipe	12	$ 230
Pipe	15	$ 261
Pipe	18	$ 283
Pipe	21	$ 326
Pipe	24	$ 350
Pipe	27	$ 416
Pipe	30	$ 447
Pipe	33	$ 483
Pipe	36	$ 516
Pipe	42	$ 614
Pipe	48	$ 692
Pipe	54	$ 789
Pipe	60	$ 898
Pipe	66	$ 1,006
Pipe	72	$ 1,171
Pipe	84	$ 1,590
Pipe	96	$ 1,920
Pipe	42x28	$ 676
MH	8	$ 13,082
MH	10	$ 13,082
MH	12	$ 13,082
MH	15	$ 13,226
MH	18	$ 13,369
MH	21	$ 13,512
MH	24	$ 13,655
MH	27	$ 13,798
MH	30	$ 13,941
MH	33	$ 14,085
MH	36	$ 14,228
MH	42	$ 14,514
MH	48	$ 14,801
MH	54	$ 16,080
MH	60	$ 16,406
MH	66	$ 17,906
MH	72	$ 18,268
MH	84	$ 20,915
MH	96	$ 23,005
Outlets	1	$ 45,000
Inlets	1	$ 100,000

Appendix C

Pump Station Replacement Fund Summary															
		First Replacement							Second Replacement						
ID	Facility	Year	Cost (P)	years (n)	F/P	Cost (F)	A/F	Annual	Year	Cost (P)	years (n)	F/P	Cost (F)	A/F	Annual
1	California Circle	2030	$ 1,000,000	10	1.79	$ 1,791,000	0.0759	$ 135,937	2070	$ 2,000,000	50	18.42	$ 36,840,000	0.0035	$ 128,940
2	Jurgens					$ -		$ -	2075	$ 3,000,000	55	24.65	$ 73,950,000	0.0025	$ 184,875
3	McCarthy	2040	$ 3,000,000	20	3.21	$ 9,621,000	0.0272	$ 261,691	2080	$ 5,000,000	60	32.988	$ 164,940,000	0.0019	$ 313,386
4	Abbott					$ -	0.0182	$ -	2050	$ 1,000,000	30	5.744	$ 5,744,000	0.0127	$ 72,949
5	Minnis	2050	$ 1,000,000	30	5.74	$ 5,744,000	0.0127	$ 72,949	2080	$ 1,000,000	60	32.988	$ 32,988,000	0.0019	$ 62,677
6	Penitencia		$ -			$ -		$ -	2070	$ 2,500,000	50	18.42	$ 46,050,000	0.0035	$ 161,175
7	Wrigley-Ford	2035	$ 2,000,000	15	2.4	$ 4,794,000	0.043	$ 206,142	2065	$ 3,500,000	45	13.765	$ 48,177,500	0.0047	$ 226,434
8	Berryessa	2045	$ 3,000,000	25	4.29	$12,876,000	0.0182	$ 234,343	2060	$ 1,000,000	40	10.286	$ 10,286,000	0.0065	$ 66,859
9	Manor	2040	$ 1,000,000	20	3.21	$ 3,207,000	0.0272	$ 87,230	2070	$ 1,000,000	50	18.42	$ 18,420,000	0.0035	$ 64,470
10	Spence Creek					$ -	0.0759	$ -	2065	$ 1,000,000	45	13.765	$ 13,765,000	0.0047	$ 64,696
11	Bellew					$ -	0.0065	$ -	2055	$ 3,000,000	35	7.686	$ 23,058,000	0.009	$ 207,522
12	Murphy	2035				$ -	0.043	$ -	2070	$ 3,000,000	50	18.42	$ 55,260,000	0.0035	$ 193,410
13	Oak Creek	2025	$ 2,000,000	5	1.34	$ 2,676,000	0.1774	$ 474,722	2055	$ 3,000,000	35	7.686	$ 23,058,000	0.009	$ 207,522
			$ 13,000,000					**$ 1,473,000**		**$ 30,000,000**					**$ 1,955,000**